CITY OF SANTA BARBARA

SUBSURFACE DESALINATION INTAKE FEASIBILITY STUDY

TECHNICAL MEMORANDUM NO. 3
BASIS OF DESIGN AND INITIAL SCREENING

January 2016

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City of Santa Barbara

Subsurface Desalination Intake and Potable Reuse Feasibility Studies

TECHNICAL MEMORANDUM
NO. 3
Basis of Design and Initial Screening: Subsurface Desalination Intake

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3.0 BASIS OF DESIGN AND INITIAL SCREENING ANALYSIS

3.1 Introduction

This section provides a basis of design (BOD), conceptual design, and the initial screening analysis that evaluates the technical feasibility of each of the subsurface intake (SSI) alternatives considered in this study. Establishing the BOD and conceptual design for the various SSI alternatives will help to identify the SSI alternatives that are determined technically feasible through initial screening using the criteria that were defined in the Work Plan (attached as Appendix A).

3.2 Basis of Design

A BOD is necessary to establish conceptual designs for each subsurface intake alternative evaluated as part of this study. The BOD is determined by the following technical criteria:

- Raw water production capacity
- Available project sites
- Subsurface intake technology alternatives
- Subsurface properties
- Coastal and sediment transport hazards
- Water quality and treatment needs
- Project life
- Reliability considerations

The Work Plan presented in Appendix A provides a summary of the methodology used to establish the BOD presented in this section.

3.2.1 Capacity

As described in the Work Plan, the target yield for each alternative is based on the City's permitted capacity for its existing screened, open ocean intake, which is the amount of seawater necessary to produce 10,000 acre-feet per year (AFY) of desalinated water. To achieve this amount of product water, each subsurface intake will need to produce 15,898 gallons per minute (gpm) of seawater. This intake flow rate is based upon a 45 percent RO recovery rate plus the volume of raw water required for backwashing pretreatment filters[1].

[1] Because it is unknown if a subsurface intake can produce the quality of water required to completely eliminate pretreatment, and the City's desalination plant is existing and uses pretreatment filters that require backwash, the volume of intake water required for filter backwash is included in the calculated subsurface intake capacity. Consistent with the existing facility operation, the BOD intake capacity does not recycle backwash water to reduce required intake flow.

3.2.2 Project Site Alternatives

As presented in the Work Plan (Appendix A), project site alternatives for a subsurface intake (SSI) were selected based upon (a) their proximity to the City's desalination plant, (b) proximity to the existing intake pipeline, (c) the City's existing easement for a railroad crossing, and (d) the availability of prior geotechnical data. The following locations were determined to meet these criteria:

1. East Beach
2. West Beach
3. Leadbetter Beach
4. 401 E. Yanonali Street (i.e., City Corporation Yard, APN #017-540-006)
5. 103 S. Calle Cesar Chavez (APN #017-113-020)

This study will focus on the onshore and offshore areas that are located on City-owned property. For offshore areas, only the submerged tideland areas that fall within the sovereign lands legislatively granted to the City, pursuant to Chapter 78, Statutes of 1925, as amended (Grant) will be considered. The seaward limit of this Grant is the U.S. pierhead line, established by the Secretary of the Navy and located one-half (1/2) mile offshore.

Figure 3.1 presents the City-owned parcels where SSI facilities may be located. Areas shown with cross hatching are locations where existing uses or environmental considerations preclude siting of SSI facilities (e.g., Stearns Wharf vicinity, tidal pools where stream discharges occur, etc.).

3.2.3 Subsurface Intake Alternatives

The subsurface intake facilities evaluated in this study vary by capacity, design, construction impact, and construction method. The Work Plan (Appendix A) identifies the following six types of SSI alternative considered in this study:

1. Vertical wells
2. Lateral beach wells (Onshore infiltration galleries)
3. Radial collector wells (i.e., Ranney wells)
4. Slant wells
5. Subsurface infiltration galleries (SIG) – offshore
6. Horizontal directionally drilled (HDD) wells (e.g., Neodren)

Figure 3.1 - Available Sites for SSI Facilities

carollo

These SSI's were included in this study based upon the state of intake technology, operating desalination projects, and recent studies.[2,3,4,5,6] The following subsections describe the design, construction, and operational considerations for each of these SSI alternatives. Relevant global experience is also presented for each alternative.

Vertical Wells

Vertical wells (Figure 3.2) are the most common type of SSI for desalination facilities. These facilities are located as close as feasible to the shoreline to maximize the proportion of seawater and minimize the proportion of freshwater pumped by each well. They consist of a submersible or vertical turbine pump installed inside a well casing. The well casing is typically a non-metallic (e.g., fiberglass or PVC) pipe that lines the well borehole to protect the native soils/sediments from collapsing into the well. The diameter of the casing has to be adequately sized to house the well intake pump and to provide ample room for pump service.

Where consolidated, porous rock is present in the producing (e.g., intake) zone of a vertical well, it is possible that no well screen may be required (a.k.a., an "open hole well"). However, where unconsolidated soils are present (e.g., sand), the well screen is used in the intake portion of the well and is a sieve-like structure with slotted or perforated openings. The screen is located at depth corresponding to the water bearing zone of the aquifer. Screen depth, size of openings, diameter, and length are key well performance design criteria. These well parameters are selected to maximize the well's operational capacity, control well entrance velocity, and to avoid excessive entrance of sand and other particulates, which have a negative impact on water quality and the useful life of the well and pump.

The performance of the well screen is enhanced by a gravel (filter) pack, which consists of clean, uniform, well-rounded gravel and sand placed between the borehole wall and the well screen to pre-filter the groundwater entering the well. Typically, the gravel pack is placed opposite the full length of the screen and extends at least five feet above the well screen.

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[2] Mackey. E.D., et al. 2011. Assessing Seawater Intake Systems for Desalination Plants. Water Research Foundation. Denver, CO.
[3] Kennedy/Jenks Consultants. 2011. scwd[2] Seawater Desalination Intake Technical Feasibility Study. Prepared for scwd[2] Desalination Program. September 2011.
[4] SWRCB. 2012. Mitigation and Fees for the Intake of Seawater by Desalination and Power Plants, Final Report. March 12, 2012.
[5] Missmer. 2013. Subsurface Intakes for Seawater Reverse Osmosis Facilities: Capacity Limitation, Water Quality Improvement, and Economics. Desalination. Elsevier. 322 (2013) 37-51.
[6] ISTAP Phase 1, 2014. Final Report: Technical Feasibility of Subsurface Intake Designs for the Proposed Poseidon Water Desalination Facility at Huntington Beach, California. Published under the Auspices of the California Coastal Commission and Poseidon Resources (Surfside) LLC. October 9, 2014

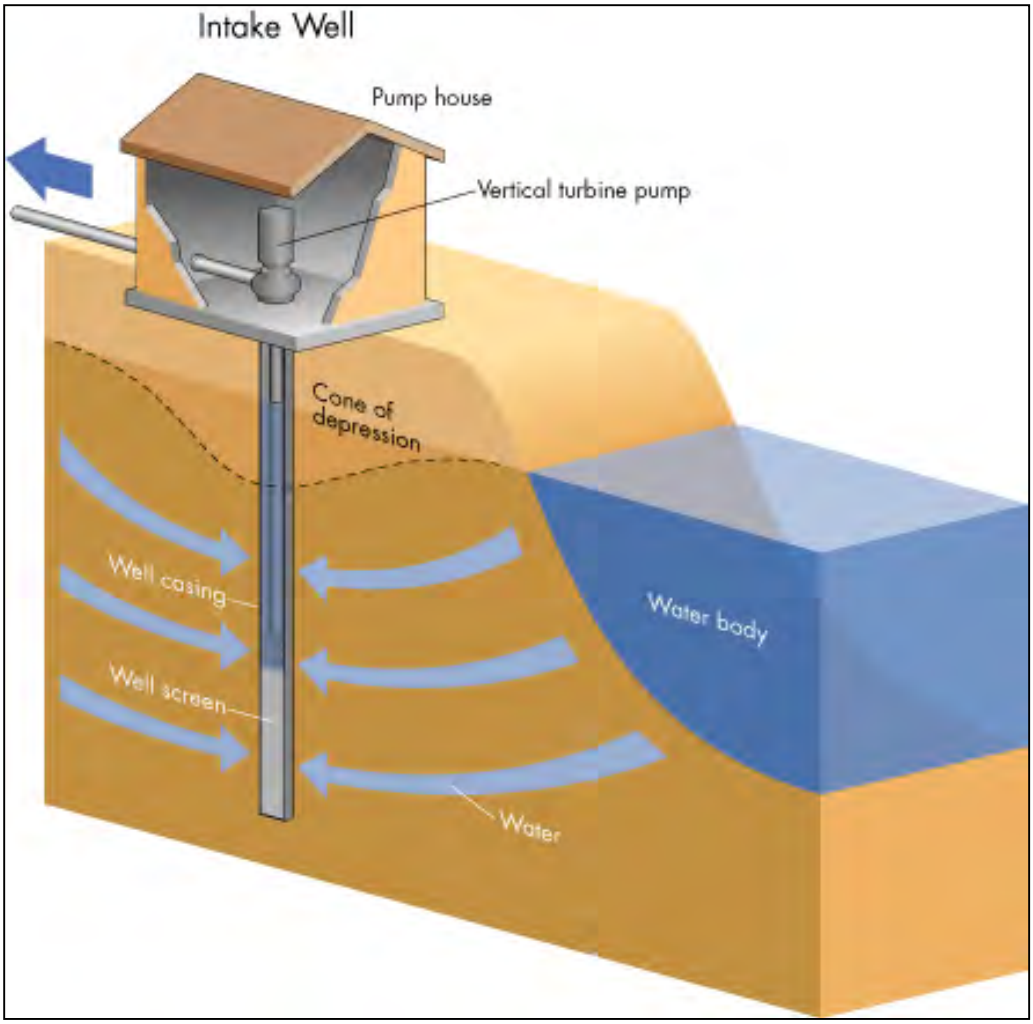

Figure 3.2 Vertical Well

In accordance with California regulations for potable water production wells, a well seal is installed above the filter pack to prevent soil and surface contaminants from entering the well screen area. The well seal is a cylindrical layer of cement, bentonite, or clay placed in the annulus of the well between the well casing and the borehole. Typically, the well seal extends at least 2 feet above the top of the gravel pack. The above ground portion of the well is finished with a concrete surface seal. The surface and well seals protect the well from surface runoff contamination and supports the casing.

Technical analysis of subsurface hydrologic properties (e.g., attained through drilling an exploratory well) is used to determine the depth and productivity of permeable sediments that may be tapped by each well, and how well connected these layers are to the ocean. Similarly, analysis of the relative contribution of seawater to freshwater is of critical importance to assess potential creation of seawater intrusion into the near coastal areas of existing aquifers, or possible draining of coastal habitat areas.

The most common challenges with vertical wells as SSI facilities include the following:

- Corrosion of the casing (i.e., when steel or stainless steel casings are used),
- Improper or defective construction techniques,
- Formation of mineral crusts or bacterial slimes in the screened section of the well that cause clogging and loss of production, and
- Loss of wellhead facilities caused by coastal erosion.
- Visual impact of having the pump house structure near the shore at an elevation that is above the flood zone.

Each vertical well requires a service road, distribution pipelines to convey the water to the desalination facilities, and a power supply.

Experience at Other Locations

Vertical wells are used at the 0.6 million gallons per day (mgd) Sand City desalination plant in Monterey County. However, because of loss of capacity of these wells over time and changes in source water quality, the plant currently operates at a reduced capacity of 0.3 to 0.4 mgd.

Marina Coast Water District (also in Monterey County and on California's central coast) operated a vertical well with a capacity of 400 gpm in the 1990's. No operational record is available since the facility has not been operated since it was commissioned and the facility has remained idle for nearly 20 years. However, coastal erosion currently threatens to daylight this vertical well and the well vault that was once buried can be plainly seen from a public beach access parking lot located off of Reservation Road.

Another example of a desalination plant with vertical beach wells is the 1.2 mgd Morro Bay desalination facility, also located on California's central coast. The plant source water is supplied by five beach wells, each with a production capacity of 0.3 to 0.5 mgd. The Morro Bay facility was originally designed without pretreatment filters, which resulted in plugging of the RO cartridge filters within 30 minutes of starting operations during an attempt to run the plant in 1996. The wells have high iron concentrations (i.e., 5 to 17 milligrams per liter [mg/L], greater than normal seawater) which required the installation of a pretreatment filter system to remove the iron before the RO process.

The Morro Bay desalination plant experienced another challenge associated with the use of a vertical well SSI system. The beach well intake water was contaminated by MTBE from a leaking underground gasoline tank. Similar problems were observed at Santa Catalina Island that uses a beach well intake for its 0.132 mgd seawater desalination plant.

The 21 mgd Sur desalination plant in Oman is the world's largest operating desalination plant using vertical wells. The wells tap into high-yield limestone aquifers[7]. The well field includes 28 (i.e., 25 active and 3 backup) beach wells, each capable of producing 1.6 to 2.3 mgd. Each well is between 260 and 330 feet deep and is equipped with a stainless steel submersible pump. The well field extends along the shoreline for 2.5 miles. Individual wells are completed with intake structures over 7 feet high that are visible from on and offshore. As typical for most existing well installations internationally, the seawater intake piping is located aboveground and supported on concrete piers. The intake wells are located within the boundaries of the desalination plant site and the seashore in this location is inaccessible for recreational or other uses. The intake area is fenced off by barbed wire to prevent damage from vandalism or acts of terrorism. The transmissivity of the Sur intake well field is one of the highest in the world, due to the very porous and unique characteristics of the limestone bedrock in the area.

Despite high permeability of the coastal aquifer of the Sur plant, the use of vertical intake wells for the expansion of the plant was found infeasible due to the high cost of this type of intake and unacceptable impacts to the adjacent aquifer. Currently, this plant is undergoing expansion, employing an open ocean intake instead of vertical wells.

The longest operating desalination facility with vertical wells is the 14.3 mgd Pembroke plant in Malta (located in the Mediterranean Sea, south of Italy) that has been operational since 1991. Another desalination plant with size comparable to Santa Barbara is the 6.3 mgd Ghar Lapsi desalination plant in Malta. Source water for this facility is supplied by 15 vertical beach wells, each with a unit capacity of 1.0 mgd. Similar to the Sur plant in Oman, these wells produce water from highly permeable coastal limestone aquifers.

Because of their need to be periodically serviced, intake wells are typically completed with wellheads extending above ground and often within the boundaries of the plant to minimize vandalism, terrorism, and theft.

Reliable Water Quality

Vertical wells installed on the beach would likely produce a source water lower in salinity than raw seawater. Based on experience from other near coastal wells, and depending on the aquifer characteristics, a 10 to 50% blend of seawater and groundwater (from inland aquifers) is likely. The extraction of groundwater from near-shore and inland areas has caused seawater intrusion into the near coastal fresh water aquifers in other locations. For example, such seawater intrusion has been observed as a result of over-pumping of near-shore aquifers used for agricultural irrigation in Monterey County, California.

[7] David, B., Pinot, J.-P., Morrillon, M. (2009).Beach wells for large scale reverse osmosis plants: The Sur case study. Proceedings of the International Desalination Association World Congress on Desalination and Water Reuse, Atlantis, The Palm, Dubai, UAE, November 7-12, 2009, IDAW/DB09-106.

Reliable Capacity

Yield at many vertical well sites diminishes over time as a result of physical or biological clogging. Vertical wells require periodic redevelopment in order to maintain their production capacity. Chemical treatment of the well using carbonic acid or sulfuric acid may be required to restore production, however, this maintenance and redevelopment can be conducted as a straightforward operational maintenance effort, and would be similar to that commonly conducted on most municipal wells, requiring a crew of approximately five people over one to two weeks for each well. The frequency of redevelopment will vary for each location.

Maintenance Requirements

The productivity and operation of vertical wells, pumps, and electrical controls located in a coastal environment must be monitored frequently due to the highly corrosive seawater environment in which the wells are located. Vertical wells are often exposed to biological fouling, which unless removed, could cause significant reduction of well production. Likewise, wells located along the beach can be exposed to inundation and erosion during storm events and the corrosive beach environment increases the amount of required maintenance. Well pumps typically require rebuilding or replacement every five years because of corrosion.

Construction Related Impacts

Construction of each well and access road will require an area of approximately 100 feet by 100 feet. The area must be fenced off during construction and a smaller footprint can be established during ongoing operations. Access to the well construction area on the beach by the public will be prohibited.

Construction Schedule

Site preparations (access road, pad, fencing) and drilling, development, and testing of a single vertical well typically requires approximately two months, while the construction of all service facilities associated with the intake system (electrical supply, access roads, pump station and building, instrumentation for well monitoring and control, etc.) may take an additional 6 months.

Lateral Beach Wells (Onshore Infiltration Galleries)

Lateral beach wells (Onshore Infiltration Galleries; Figure 3.3) are similar in configuration and mode of operation to those installed offshore (refer to subsection below for SIG) except that they are generally limited to the width and length of the available beach. The natural beach sand is excavated to a depth of 15 to 30 feet (typically at least 4 feet below the static water level) in order to intercept the shallow upper unconfined coastal aquifer as it enters the ocean. Perforated PVC collection pipes are installed at the bottom of the pit and covered with several layers of gravel and sand of predetermined size and gradation, similar

to the filtration layers of an offshore SIG. The onshore infiltration gallery is connected via a pipe to a large diameter sump and pump station located shoreward. The pump station houses vertical line shaft or submersible pumps.

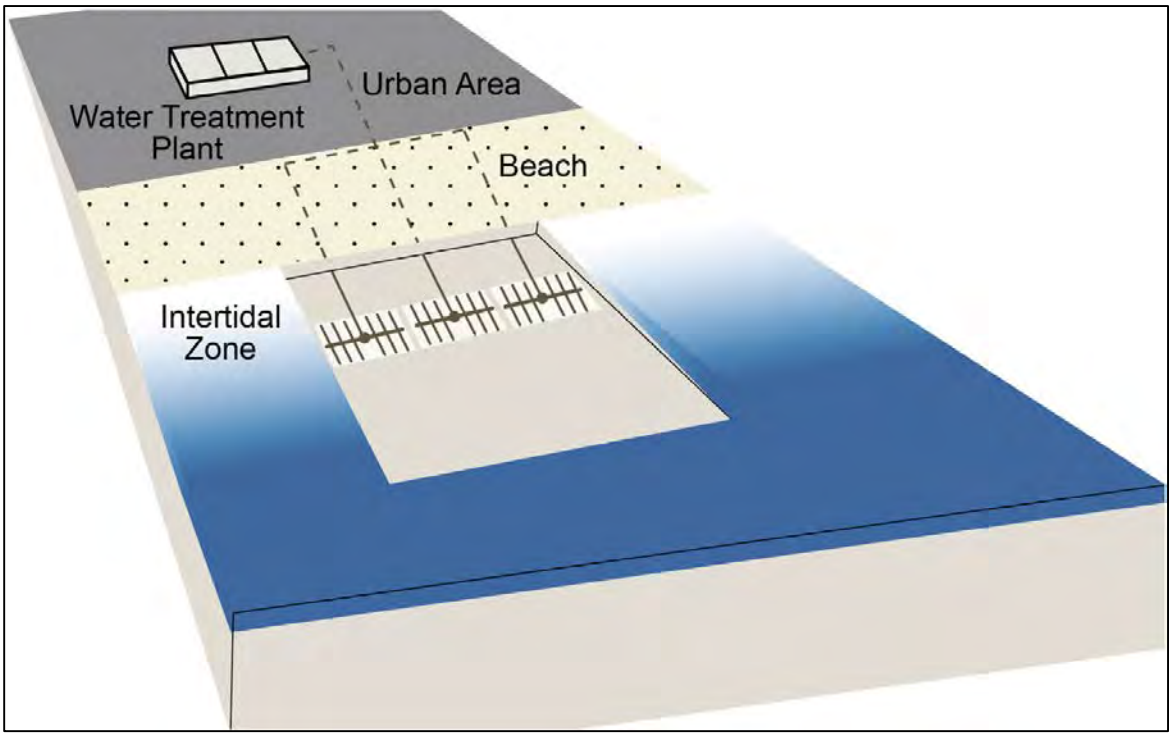

Figure 3.3 Onshore Infiltration Gallery at Alicante 1 Plant (Spain)

The main difference between the onshore and offshore infiltration galleries is that the onshore infiltration galleries receive water only from the four sides and bottom of the filtration cells while the offshore infiltration galleries also collect water from their top surface (the ocean water column above the infiltration cells). As a result, onshore galleries usually require 30 to 35 percent more filtration surface area to collect the same volume of water. In addition, the onshore infiltration galleries may also collect water from the shallow surface coastal aquifers, resulting in a mixture of fresh and salt water, while the offshore SIGs receive water from primarily the ocean. Thus, the onshore infiltration gallery may collect water of lower salinity than the offshore SIGs. Onshore infiltration galleries may also capture contamination that is migrating in the shallow groundwater from inland sources.

Because onshore infiltration galleries typically require significantly more excavation surface and are more disruptive to the beach environment than offshore galleries, they have only found application for very small projects. Such galleries are typically only used if the offshore conditions are not conducive for the construction of infiltration galleries due to the low transmissivity of the ocean bottom sediments (e.g., rocky ocean bottom in the surf zone) or if the surf zone is naturally exposed to intense beach erosion and sand transport, which would destroy the infiltration gallery. Under such circumstances, installing infiltration

gallery on the sandy beach will be more advantageous due to the lower excavation costs and potentially higher transmissivity of the subsurface substrate.

Experience at Other Locations

At this time, there are no onshore infiltration gallery seawater intake installations operating anywhere in the world matching the size and complexity of the intake required for this study. There are also no desalination plants using onshore infiltration galleries in California or elsewhere in the United States. However, an example of an onshore infiltration gallery for the collection of fresh river water is installed in the Town of Bethlehem, New York. The infiltration gallery was installed along the western banks of the Hudson River. The gallery was originally intended to produce 6 mgd in 1996, although capacity was significantly reduced, and the average capacity over the first year of operation was only 2.3 mgd.

The gallery ultimately produced significantly less capacity than originally intended due to a combination of factors, but iron fouling and siltation within the Hudson River are noted as the two primary reasons. Less than three weeks after initial operation, the Hudson River flooded and 1 to 6 inches of silt and mud were deposited on the river bottom, significantly reducing the capacity of the gallery. Subsequently, dredging was conducted and increased capacity to 2.3 mgd. Back-flushing the system was attempted to re-mobilize the silt and mud, but was unsuccessful. The infiltration gallery currently produces approximately 0.8 to 1.0 mgd.

In 1997, a study was conducted that concluded siltation was the primary mechanism responsible for reducing yield to the gallery. Results showed the maximum yield of the system with siltation was between 1.4 and 2.2 mgd and up to 3.6 to 4.3 mgd without siltation. Dredging of the river was again conducted two years after initial operation; however, the results were short lived. Nine days following the dredging activity, silt deposition increased again. It was concluded that frequent dredging was required to increase system capacity, which was infeasible. Ultimately another intake was recommended.

Additional factors which contributed to the reduced yield of the infiltration gallery included:

- Iron fouling as a result of high iron concentrations. Initial iron concentrations were as high as 12-13 mg/L but stabilized at 3-5 mg/L. A video inspection of the infiltration gallery in 2002 revealed significant iron deposition on the intake pipe and throughout the intake structure;
- Limited aquifer thickness along the riverbank (28 feet);
- Presence of a wooden bulkhead which likely limits lateral flow and recharge of the gallery from the Hudson River; and
- Installation of a landward clay barrier within the trench to limit shallow groundwater recharge, which contains elevated iron and manganese.

The Town of Bethlehem filed litigation with the design engineer due to the reduced capacity from the infiltration gallery. A settlement was reached in 2001 that ultimately involved the installation of several angled wells beneath the Hudson River to obtain additional capacity.

An onshore infiltration gallery with lateral collection wells serves the Alicante 1 seawater RO plant in Spain and has a total installed capacity of 3.2 mgd (Figure 3.3). The system consists of a 0.61 mile long tunnel at a depth of 46.2 feet underneath the surface of the beach. The tubular tunnel has a diameter of 10.4 feet and includes 104 pipe laterals constructed perpendicular to the tunnel access and extending in the direction of the ocean. The wells are constructed in a highly porous limestone formation mixed with some rock and carbonate sand. These wells tap a shallow unconfined aquifer and are located between 200 and 363 feet from the shore and are recharged by vertical water movement from the aquifer lying above the tunnel.

Study of the source water quality collected from this intake compared to vertical wells collecting flow from the same aquifer indicate that the onshore infiltration gallery has generated lower water quality with much higher turbidity (3.58 NTU vs. 0.5 NTU) and higher silt density index (SDI). Figure 3.4 shows the difference between the SDI test of the seawater collected from the vertical wells (A) and the onshore infiltration gallery (B), which clearly indicates the significantly lower quality of the seawater collected by the onshore infiltration gallery. In this example, water produced by the onshore infiltration gallery requires further pretreatment to be used as source water for reverse osmosis desalination.

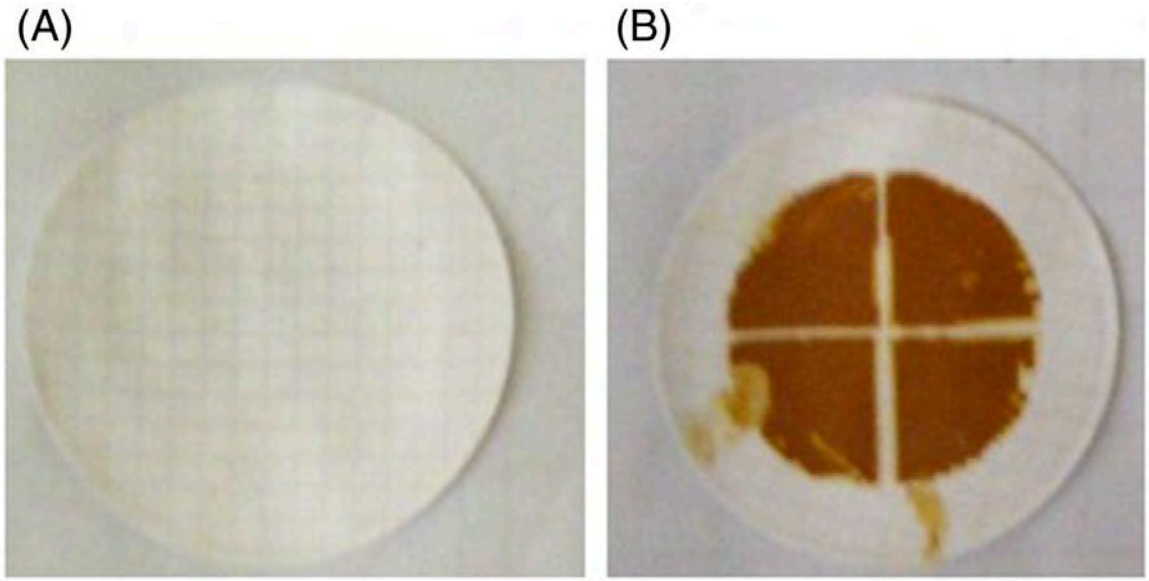

Figure 3.4 SDI Test for Vertical Wells (A) and Onshore Infiltration Gallery (B)[8]

[8] Allan J., R. Cheng, T. Tseng, K. Wattier, Update for the Pilot and Demonstration Scale Research Evaluation of Under-Ocean Floor Seawater Intake and Discharge, Presentation at 2009 Annual Conference & Exposition of AWWA, June 16, 2009.

Reliable Water Quality

As described above, previous implementation studies of onshore infiltration galleries showed the onshore infiltration galleries produced poor water quality with a significantly higher turbidity than typically expected from wells. Therefore, pretreatment is assumed to be required. Because of the orientation of the gallery and its physical position in relation to the ocean and shallow aquifer systems, it is likely that produced water would be a mixture of ocean water and shallow groundwater.

Reliable Capacity

As demonstrated by the use of this intake type to supply fresh water, the production capacity of onshore infiltration galleries may diminish with time. Iron fouling and siltation can significantly impede capacity of onshore infiltration galleries. The long-term reliability of an onshore infiltration gallery is unknown because this type of intake is not commonly used for collecting source seawater for desalination.

Maintenance Requirements

Maintenance of the onshore infiltration galleries may require backflushing, although chemical treatment may be necessary if iron bacteria or inorganic deposits create encrustation and reduce the gallery's production capacity.[9] Use of such intakes for small facilities in the Middle East has shown the need for periodic disinfection of the intake collection piping of the gallery due to heavy growth of bacteria in the pipe. There is a concern for integrity of the system in locations where beach erosion occurs either annually or during significant storm events. Lastly, the corrosive beach environment increases the amount of required maintenance on the intake gallery's pump station and other facilities.

Construction Related Impacts

The construction of the onshore infiltration gallery will be less complex compared to an offshore SIG because it would not involve marine construction work. However, construction of the onshore infiltration gallery, access road, sump, and pump station will significantly impact public access to the beach during the construction period. There may also be impacts to local shorebird populations during construction and periodic maintenance activities.

[9] CDM, Support Technical Memorandum, NYDEC Specific Comment 74, Chapter 18 – Alternatives of Infiltration Gallery, September 2, 2011

Construction Schedule

Due to the concerns associated with protecting the coastal near-shore environment and the larger surface area of the system, the overall construction time is expected to be comparable for onshore and offshore SIG of similar size and complexity. Approximately two years is estimated to be required to construct the onshore infiltration gallery and pump station.

Radial Collector Wells (Ranney Wells)

This type of well consists of a concrete caisson that extends below the ground surface with horizontal collector screens (laterals) that project out in a radial pattern into the surrounding aquifer (refer to Figure 3.5 A and B).

Since the well screens in the collector wells are placed horizontally within the most productive zone of an aquifer, a higher rate of source water collection is possible than with typical vertical wells. This allows the same intake water quantity to be collected with fewer wells. Individual collector wells are typically designed to collect between 1 mgd and 5 mgd of source water per well, depending on the thickness and permeability of the aquifer.

The caisson of the radial collector well is typically constructed of reinforced concrete that is from 9 to 20 feet in diameter with a wall thickness of approximately 1.5 feet. The caisson depth varies according to site-specific geologic conditions, ranging from approximately 30 to 100 feet.

The number, length, and location of the horizontal laterals are determined based on a detailed hydrogeological investigation. Typically, the diameter of the laterals ranges from 8 to 12 inches and their length extends up to 150 feet. The size of the lateral well screen slots are selected to exclude the underground native soils. If necessary, an artificial gravel-pack filter can be installed around the screens to suit finer-grained deposits. Usually, one well has 4 to 10 laterals preferentially oriented towards the source water body (e.g., ocean, river).

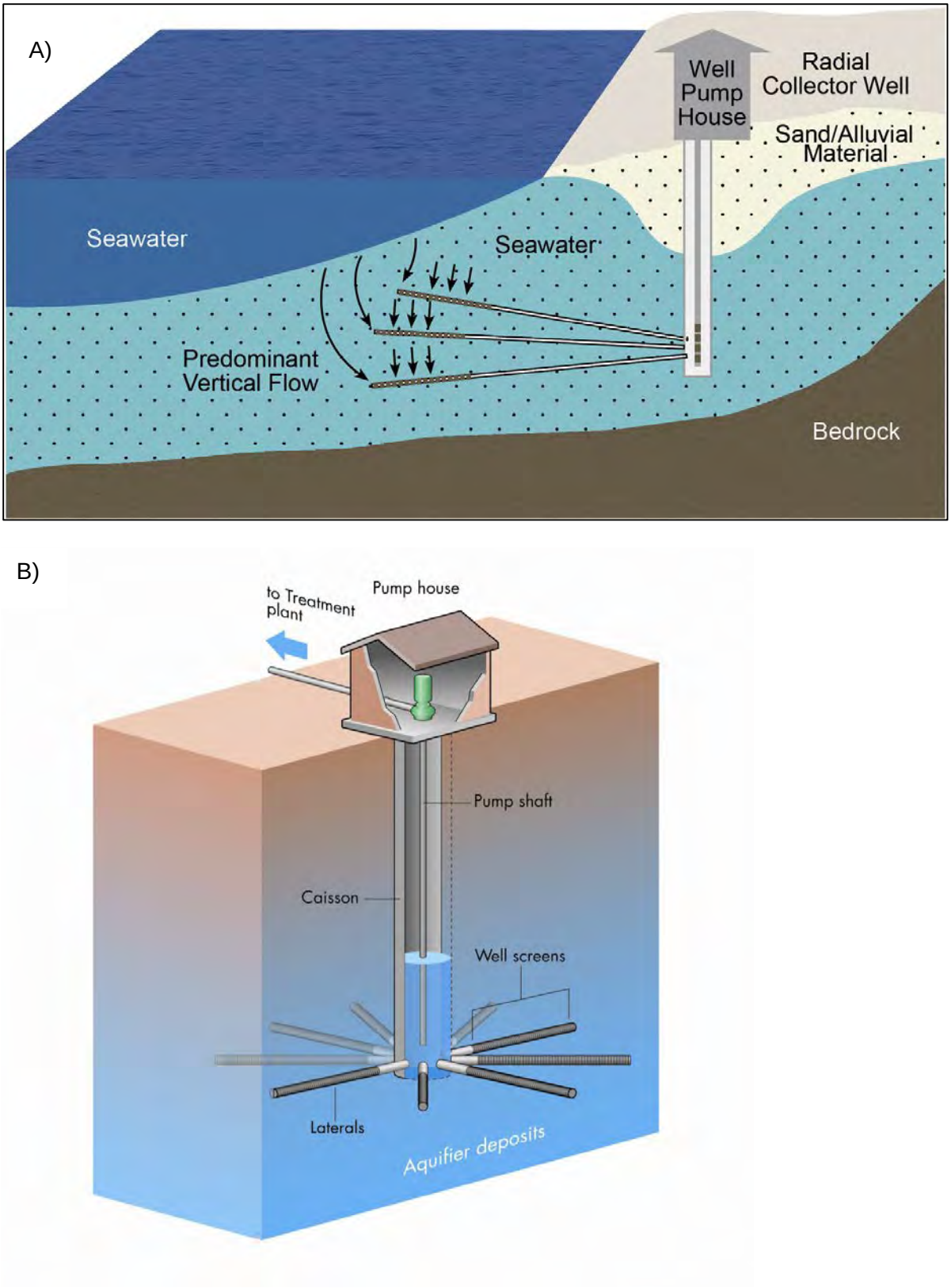

Figure 3.5 Radial Collector Well (Voutchkov and Kennedy Jenks, 2013)

Radial collector wells typically have an intake pump station with multiple pumps installed above the well caisson. The collector pump station can be designed with submersible pumps (although typically designed with vertical turbine pumps due to energy efficiencies) to minimize noise. Protecting the radial collector well from beach erosion and reducing impacts on the beach often requires the collector be located back away from the shoreline. This, however, results in the horizontal laterals not extending beneath the ocean surface and therefore the produced water quality would likely be a mixture of seawater and groundwater, similar to the beach infiltration gallery or vertical well. To minimize capture of groundwater, it is possible to design a collector well that it is buried below grade very near the shoreline and which includes a pipe that conveys the seawater landward by gravity to a "clear well" (i.e., similar to a deep sump) that can be operated as the pump station. This design would produce water with a higher percentage of seawater than a collector well sited farther inland.

Experience at Other Locations

Radial collector wells are not as commonly used as vertical wells for seawater intakes. In fact, the only radial collector well installation used as a seawater intake for desalination is located in Salina Cruz, Mexico and consists of three wells that are designed to deliver 3.8 mgd each. The Salina Cruz collector wells have encountered significant problems due to beach erosion (see Figures 3.6 and 3.7) and have also created significant environmental impacts. Figure 3.7 shows the same wells depicted on Figure 3.6, after four years of operation. Beach erosion affected the productivity of the wells, which has decreased by more than 20%. Additionally, the source water quality has degraded over time, the system caused seawater intrusion, and has caused the complete drainage and destruction of an adjacent coastal wetlands habitat.

Figure 3.6 Effects of Beach Erosion after One Year of Operation

Figure 3.7 Effects of Beach Erosion after Four Years of Operation

Reliable Water Quality

Experience at the Salina Cruz seawater desalination facility, installed in 2001, shows highly variable source water quality. The source seawater from these wells contains high levels of iron and manganese and has to be treated using greensand filters prior to employing seawater desalination technologies. Another potential water quality challenge with wells occurs in source water containing hydrogen sulfide; this compound will likely be oxidized to elemental sulfur, which could cause RO membrane fouling.[10]

Reliable Capacity

Experience with the Salina Cruz facility shows limited reliability of collector wells located near the shoreline due to erosion around the caisson as well as the previously mentioned variable source water quality.

Maintenance Requirements

Because of the large capacity of a radial collector well system (5 mgd per well or greater) these wells typically contain heavy pumps, which will have to be accessed with a crane for

[10] Missimer, T. M., R. G. Maliva, M. Thompson, W. S. Manahan, K. P. Goodboy (2010) Reduction of Seawater Reverse Osmosis Treatment Costs by Improvement of Raw Water Quality: Innovative Intake Designs, Desalination and Water Reuse, vol. 20/3, pp. 12-22.

periodic maintenance. Maintenance will require periodic (monthly or biweekly) access of service vehicles to the beach. The frequent operation of service vehicles on beach front areas can detract from the recreational value of the beach and will have visual and aesthetic impacts. Collector wells also require occasional redevelopment of the laterals, requiring access into the caisson with divers and jetting equipment. Lastly, the corrosive beach environment increases the amount of required maintenance.

Construction Related Impacts

Construction of radial collector wells will require a larger working area (200 x 100 feet) than typical vertical wells due to the size of the required equipment. An access road and pump station with power will also be required. The construction process will involve the use of a large crane and clamshell for advancing the caisson, which is constructed at the site with reinforced concrete.

The visual and aesthetic impacts of collector well intakes will be dependent upon the location of the wellhead and the style of well completion used. If the collector well intake must be constructed above-grade, the pumps, electrical controls, motors, and auxiliary equipment typically are housed in a building constructed above the wet well of the caisson and/or above known or anticipated high water (e.g., tidal or flood) elevations. Because collector wells typically yield 5 to 7 times more water than conventional vertical wells, fewer collector wells are required to achieve the same target yield.

The above-grade pump house facility can be designed in virtually any architectural style; however, this facility and its access provisions would change the visual landscape of the area in the vicinity of the intake. If built above ground, the individual wells may need to be fenced-off or otherwise protected from unauthorized access. Figure 3.8 presents a rendering of what a typical radial collector well subsurface intake system would look like on a beach.

If intake wells are located in visually sensitive areas (e.g., public beaches), the installation of above-grade wells may degrade the recreational and tourism uses and value of the area (i.e., seashore), and will change the area's appearance and character. A potential solution is to construct the intake wells below-grade, at grade or near-grade to minimize impacts. The electrical controls and auxiliary equipment of the well intake system could be installed within a watertight structure or located in a building outside of the beach area. In these cases, there may be little or no aesthetic impacts. However, the costs associated with this type of collector well intake configuration would increase measurably.

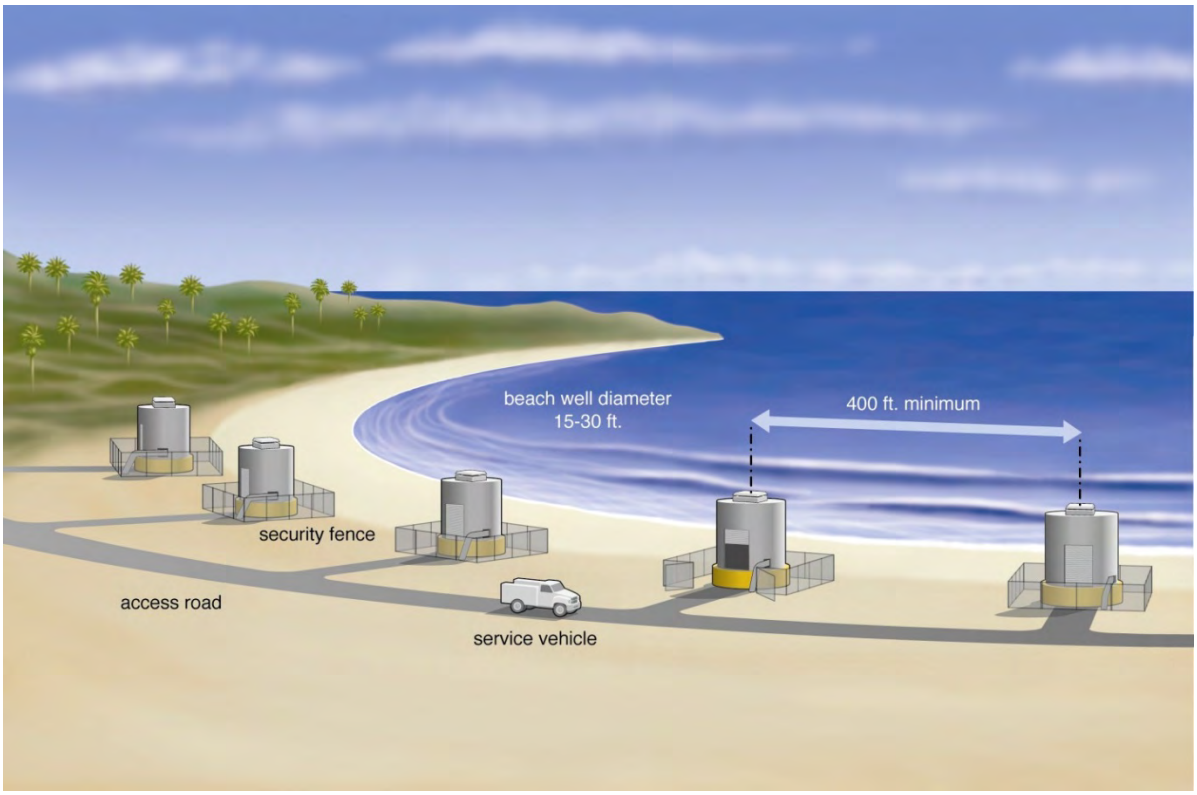

Figure 3.8 Radial Collector Well Visual and Aesthetic Impacts

Construction Schedule

Based on past experience, it takes approximately 1.5 – 2 years to construct a collector well and associated pump station.

Slant Wells

Slant wells are subsurface intake wells drilled at an angle and extending under the ocean floor to maximize the collection of seawater and the beneficial effect of the natural filtration of the collected water through the ocean floor sediments (refer to Figure 3.9). Well construction is similar to a vertical well except that it is done at an angle. Specialized drilling equipment must be employed for each slant well project.

Slant wells are a patented subsurface intake technology, which was tested for the first time in the US by the Orange County Municipal Water District at their pilot desalination facility located in Dana Point, California.[11] This well was drilled at an angle of approximately 23 degrees and extended approximately 200 feet under the ocean floor at a depth of 100 to

[11] U.S. Department of the Interior Bureau of Reclamation, 2009, Desalination and Water Purification Research and Development Program Report No. 152, Results of Drilling, Construction, Development, and Testing of Dana Point Ocean Desalination Project Test Slant Well, January 2009, Prepared by Geoscience, Inc.

200 feet below the ocean bottom. The same technology is currently being pilot-tested by California American Water Company (CalAm) in Monterey, California.

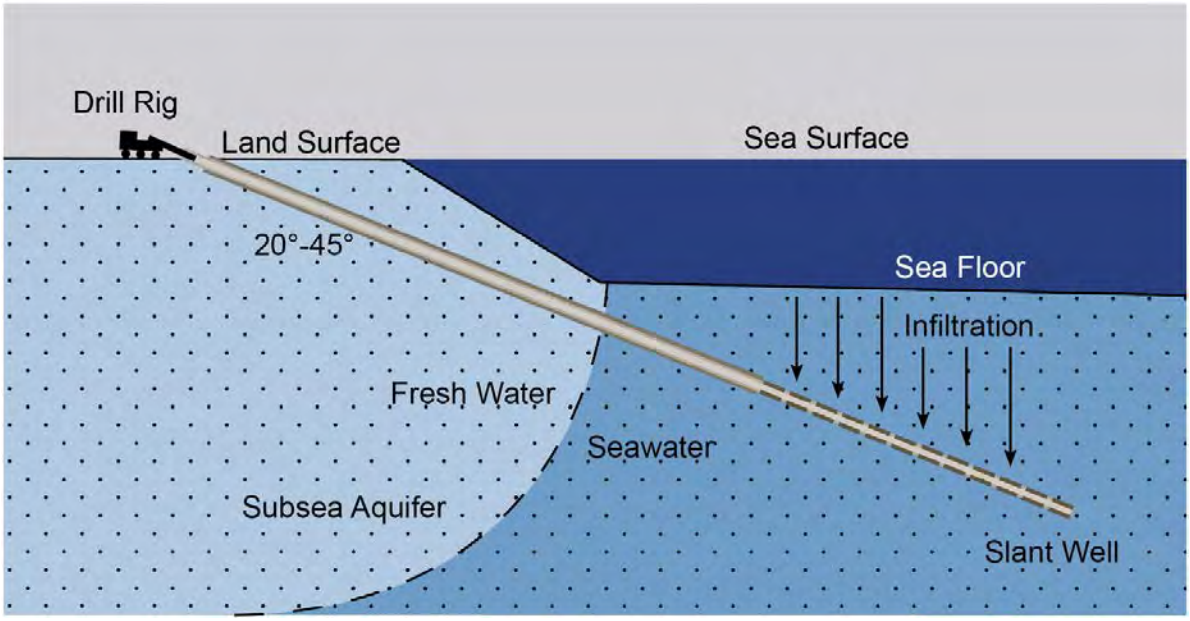

Figure 3.9 Slant Well Intake Schematic (From Missimer et al., 2013)

Experience at Other Locations

At present, no full-scale desalination plants exist that employ slant wells for source seawater collection. As discussed previously, the slant wells have been tested for over six years at the Dana Point test site and are currently being evaluated at a test site near Monterey, CA. The overall experience with Dana Point was positive, but results indicated the system did not collect only seawater but instead a mix of seawater and fresh water from the alluvial aquifer in the vicinity of the intake location.

The salinity of the well water has been observed to increase over time and to stabilize to a level of approximately 17,000 mg/L (50% seawater). However, as salinity increased slowly over time, the content of iron and manganese in the collected seawater increased and reached levels of over 14 mg/L, which would require complex pretreatment to implement at full scale. In addition, source water was found to have very low dissolved oxygen concentrations, which will require re-aeration prior to conveyance to the outfall of the wastewater plant.

At the Monterey slant well site, drilling and testing is continuing; recent water quality data shows that the slant well is producing 90% seawater at this time.

The Independent Science Advisory Panel (ISTAP) that was assembled by the California Coastal Commission tasked with the review of alternative subsurface intake technologies for the Huntington Beach seawater desalination plant concluded that slant well technology

is not yet proven and "the long-term performance of the technology has yet to be confirmed."[12]

Reliable Water Quality

Based on the experience from the Dana Point and Monterey slant well test sites, the collected source water is expected to have TDS concentrations lower than seawater (35,000 mg/L), in a range of 17,000 mg/L to 31,500 mg/L. As with many of the subsurface intakes tested or operated along the California coastline, it is possible that the collected source water will have a high iron and manganese content (2 to 14 mg/L, refer to Figure 3.10)[13], which will require the use of pretreatment that is more complex than currently used at the Santa Barbara desalination plant.

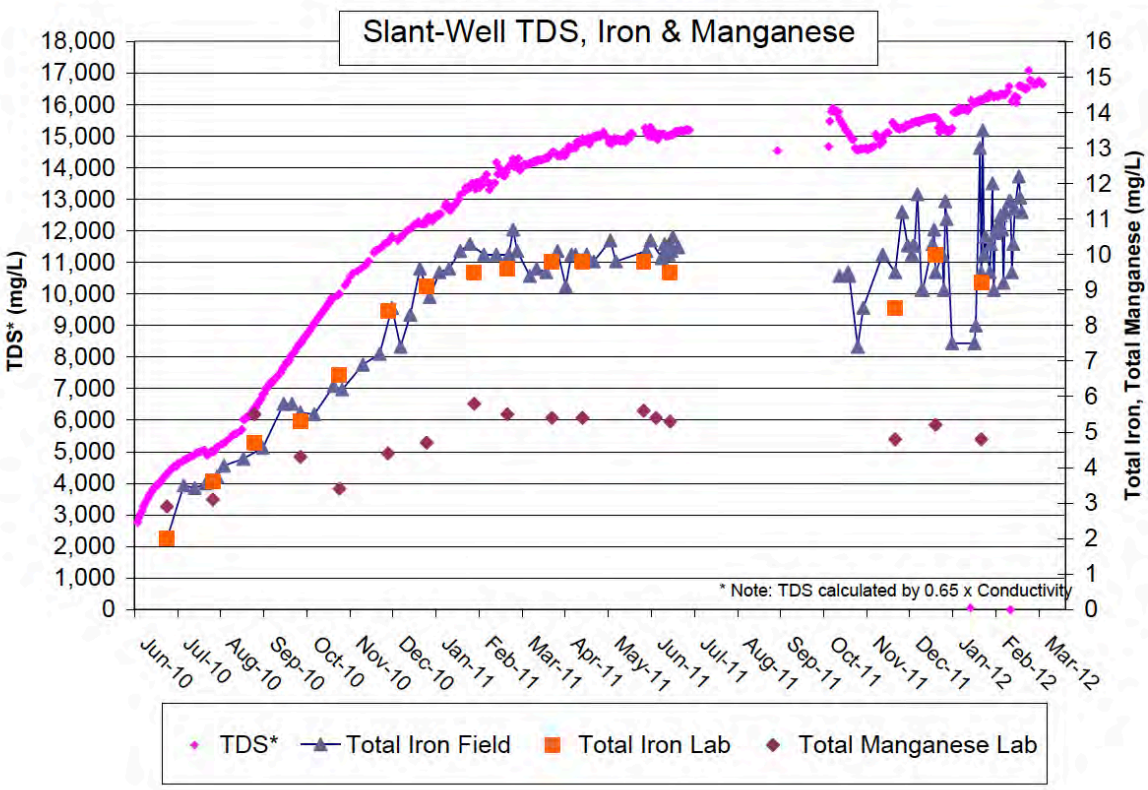

Figure 3.10 Source Water Quality Produced by the Dana Point Slant Well

Conservatively designed low-rate greensand filters or an ultrafiltration (UF) membrane system with pre-oxidation upstream of the existing conventional pretreatment system will need to be installed to allow process control, avoid irreversible RO membrane fouling, and

[12] ISTAP Phase 1, 2014. Final Report: Technical Feasibility of Subsurface Intake Designs for the Proposed Poseidon Water Desalination Facility at Huntington Beach, California. Published under the Auspices of the California Coastal Commission and Poseidon Resources (Surfside) LLC. October 9, 2014

[13] Wetterau, G. R. Bell, G. Filteau, L. Voelz (2013) Iron and Manganese Removal for Seawater Pretreatment, Presentation at AWWA Annual Conference & Exposition, June 2013.

ensure reliable production of potable water from the City's desalination plant.

Long-term testing at the Dana Point slant well also indicated that the source water has very low oxygen content (usually less than 1.0 mg/L) and its discharge to the ocean could trigger apoxia in the vicinity of the discharge. Desalination plant treatment processes do not add appreciable amount of dissolved oxygen (DO) to the intake water. Low DO concentration of the product water will require either product water re-aeration or will result in a significant use of chlorine. The brine would also need to be supplemented with DO prior to discharge to maintain compliance with the City's outfall permit conditions.

Reliable Capacity

Operation of slant wells at the Dana Point and Monterey test sites in California demonstrated that this intake technology was capable of delivering 2,000 to 2,200 gpm of water. The slant wells at the Dana Point test facility have operated for a period of three years and have shown steady performance. However, the most recent reports associated with this testing indicate that the well efficiency has been reduced from 95% in 2006 down to 52% in 2012.[14] It should be noted that there is no proven track record for full-scale installation.

Maintenance Requirements

Maintenance requirements for the slant wells are expected to be similar to these for the vertical intake wells. Since no long-term full-scale operational experience exists, and this technology is novel, it is difficult to anticipate all maintenance requirements for these wells at this time.

Construction Related Impacts

Experience at the Dana Point and Monterey test well facilities shows that slant well construction site may occupy an area of 0.55 acres.[15] Construction of the slant well is similar in nature to the construction of vertical wells; however, the overall construction of slant wells is significantly more complex, due to the techniques required. Although the drilling equipment is the same used for vertical wells, the drilling rate is significantly (approximately two times) slower. Furthermore, the slant well will have a larger radius of influence and will occupy approximately two times the space needed for construction.

[14] Geoscience, Inc. 2012, Aquifer Pumping Test Analysis and Evaluation of Specific Capacity and Well Efficiency Relationships SL-1 Test Slant Well Doheny Beach, Dana Point, California Prepared for: Municipal Water District of Orange County September 7, 2012.
[15] Geoscience, Inc. 2005. Phase 2 Research and Development Subsurface Intake System Feasibility Investigation: Test Slant Well Construction and Testing Plan. Dana Point Ocean Desalination Project. October 14, 2005.

Construction Schedule

The required construction time for the slant wells will be considerably more than the construction of vertical intake wells because of their complexity and the custom drilling equipment that is required. Based on experience at Dana Point, the construction and commissioning of a single well will take approximately one year; construction of the entire slant well program for full scale production will take approximately two additional years.

Subsurface Infiltration Galleries (SIG)

Subsurface infiltration galleries (SIGs) consist of an engineered slow sand media filtration bed located at the bottom of the source surface water body (i.e., ocean, lake, or river). The bottom of this engineered filtration bed contains a number of equidistant horizontal perforated pipes which convey filtered source water collected from the bed to the wet well of an intake pump station located on shore (see Figure 3.11).

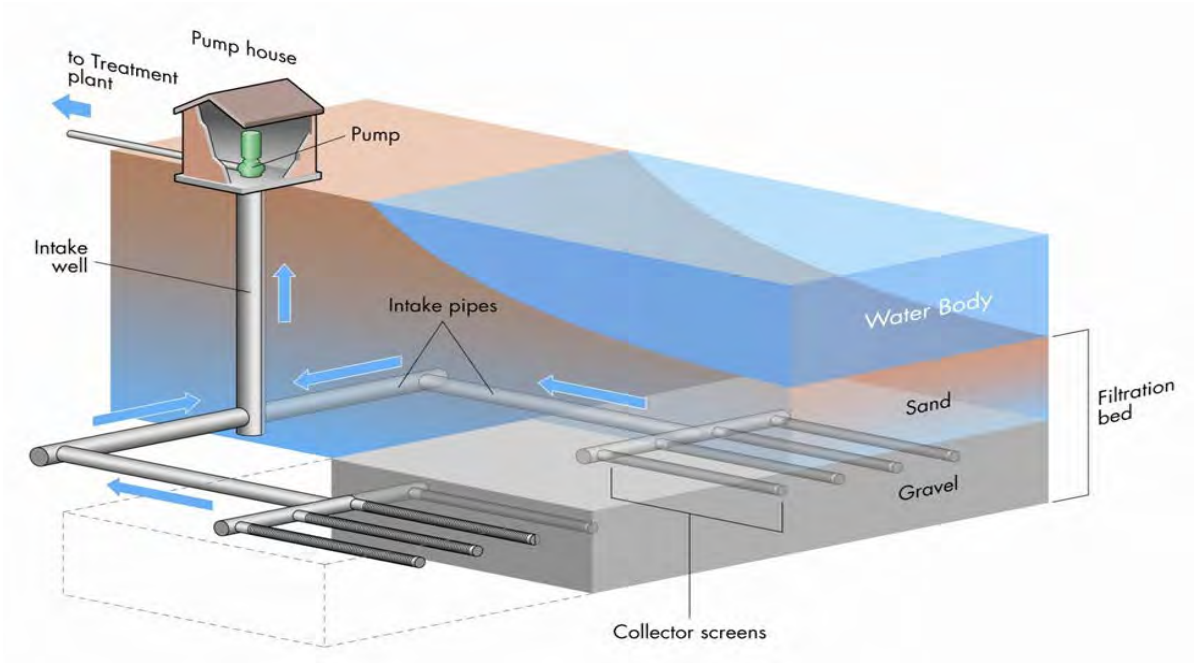

Figure 3.11 Subsurface Infiltration Gallery

Infiltration galleries are typically implemented when conventional horizontal or vertical intake wells cannot be used due to unfavorable hydrogeological conditions. For example, they are suitable for intakes where the permeability of the underground soil formation is relatively low, or in the case of river or seashore filtration, where the thickness of the beach or the onshore sediments is insufficient to install conventional intake wells.

Filtration beds associated with SIGs are sized and configured using the same design criteria as slow sand filters. The design surface-loading rate of the filter media is typically 0.05 to 0.10 gpm/ft^2. Such well systems could be installed in areas with good natural underwater-current driven sediment transport, because they rely mainly on wave and

current action to remove the solids retained and accumulated on the surface of the infiltration bed during the filtration process. One potential challenge of SIGs is the biofouling of the filtration media, which could reduce their production capacity over time. The depth of the engineered media installed in the filtration cells is usually between 10 and 15 feet. For the purposes of this study, a media depth of 15 feet is assumed.

The natural seabed filtration process removes practically all coarse solids and particulates of a size of 50 µm or larger from the seawater and precludes marine organisms in all phases of development (adults, juveniles and larvae) from entering the desalination plant. It is unknown if these larval stage marine organisms are entrained in the offshore media (i.e., no core samples have been made available for analysis). This system is also an effective barrier against the heavy solid loads generated during algal blooms and oil spills.

Experience at Other Locations

The largest seawater desalination plant with a subsurface infiltration gallery (SIG) system in operation is the 13.2 mgd Fukuoka District desalination facility in Japan. This plant has been in operation since 2006; the SIG is 1,035 feet long and 212 feet wide (with total filtration area of 5 acres), and it is designed to collect source seawater at a capacity of 34 mgd. The design infiltration velocity is 0.10 gpm/ft^2.

The filtration media used in the SIG is configured in three distinctive layers (refer to Figure 3.12): bottom – 7.5 foot layer of graded gravel pack with a gravel size between 20 and 40 mm, which surrounds the horizontal well collectors; middle – 1.0 foot interim layer of finer graded gravel size between 2.5 and 13 mm; and top – 5.0 foot layer of natural sand excavated from the ocean bottom. The top of the filtration media is submerged at 38 feet below the ocean surface.

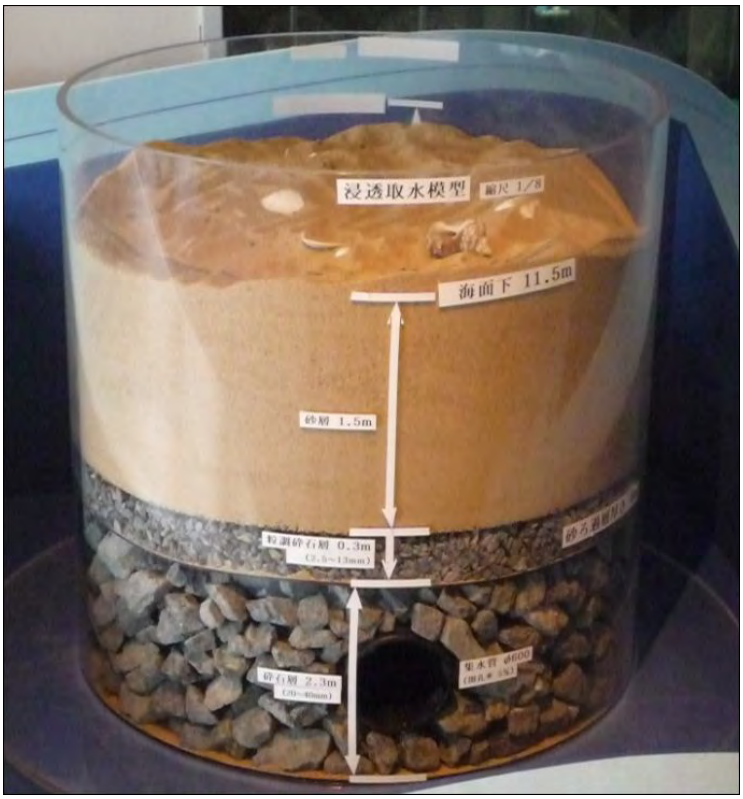

Figure 3.12 Seabed Filtration Media Configuration of the Fukuoka Desalination Plant

The SIG collectors are 200 feet long, 24 inch diameter polyethylene pipe screens (refer to Figure 3.13). The collector pipes are designed for an inflow velocity of 0.1 fps. The screens collect the source water flow into a central pipe with a diameter of 62 inches and length of 3,860 feet, which conveys this water into a two-tank water collection well for pumping to the desalination plant. The collected water is pretreated with UF membrane filtration prior to desalination in the seawater RO membrane system.

Figure 3.13 Segment of 600-mm Intake Collector Screen

Based on a site visit completed in 2014 by Water Globe Consulting, the Fukuoka desalination plant's SIG has experienced gradual reduction of the infiltration capacity due to biological fouling, resulting in reduction of plant productivity by 25%. SIG productivity reduction did not occur in the first ten years of plant operation, but began to take place gradually afterwards.

Reliable Water Quality

Based on the experience with other desalination projects employing SIGs (e.g., Fukuoka, Japan and Long Beach Pilot Infiltration Gallery Project), the water produced by the SIG will likely require additional pretreatment. The Fukuoka desalination plant employs ultrafiltration pretreatment and cartridge filtration prior to SWRO desalination despite the fact that this water is already filtered by the infiltration gallery.[16]

Water quality results from the Long Beach infiltration gallery test show that the water quality collected by the infiltration gallery is not adequate to be directly used for RO desalination because of the high turbidity (2.58 NTU vs. RO feed water quality requirement of 0.1 NTU or less) and high silt content (Silt Density Index of 6.67 vs. RO feed water quality requirement of 4 or less).[17] The source water produced by the infiltration gallery in Long Beach has caused biological fouling on the cartridge filters located downstream of the SIG

[16] http://www.niph.go.jp/soshiki/suido/pdf/h21JPUS/abstract/r9-2.pdf
[17] Allan J., R. Cheng, T. Tseng, K. Wattier, Update for the Pilot and Demonstration Scale Research Evaluation of Under-Ocean Floor Seawater Intake and Discharge, Presentation at 2009 Annual Conference & Exposition of AWWA, June 16, 2009.

such that these filters had to be replaced weekly (see Figure 3.14). For comparison, the normal replacement schedule of cartridge filters at desalination plants with open intakes is 8 to 12 weeks.

Figure 3.14 Cartridge Filter at Long Beach SIG after One Week of Operation

Reliable Capacity

Because of the limited experience with the long-term operation of infiltration galleries for collection of seawater for desalination, the long-term reliability of this type of intake is uncertain. Experience at the Long Beach infiltration test facility has shown that the volume and water quality produced by this type of intake vary significantly seasonally and have changed over time.

Maintenance Requirements

As part of routine maintenance for a SIG, filter beds should be dredged every one to three years in order to remove the sediment and entrained marine life that may accumulate in the filter media and reduce capacity (and increase head loss) over time. If this material is not removed, intake flow would decrease and the desalination plant would have to reduce its production. The dredged material must be disposed away from the intake filter beds in order to prevent the removed solids from returning to the SIG. Dredging and disposal

activity could increase stress on the marine ecosystem in the area, and would render the area unavailable for recreational activities during maintenance activities.

Construction Related Impacts

Based on the design of the SIG intake considered for the Huntington Beach desalination project[18], a system sized to provide approximately 23 mgd of seawater would impact approximately seven acres of seafloor, requiring excavation to an engineered media depth of 14 feet. Local sediment transport conditions should be considered when determining the overall excavation depth. A 23 mgd SIG system modeled after the Huntington Beach design concept would likely consist of eight (8) intake filtration bed cells and eight (8) 10-inch diameter connector pipelines spaced at 330-foot intervals. The total length of the intake filtration bed gallery would be 2,600 feet (0.5 miles) in length. The actual excavation area is likely to be 20 to 30% larger than the footprint of the infiltration gallery cells because of the additional construction laydown area, which will be needed to load the trucks transporting the excavated soils (refer to Figure 3.15).

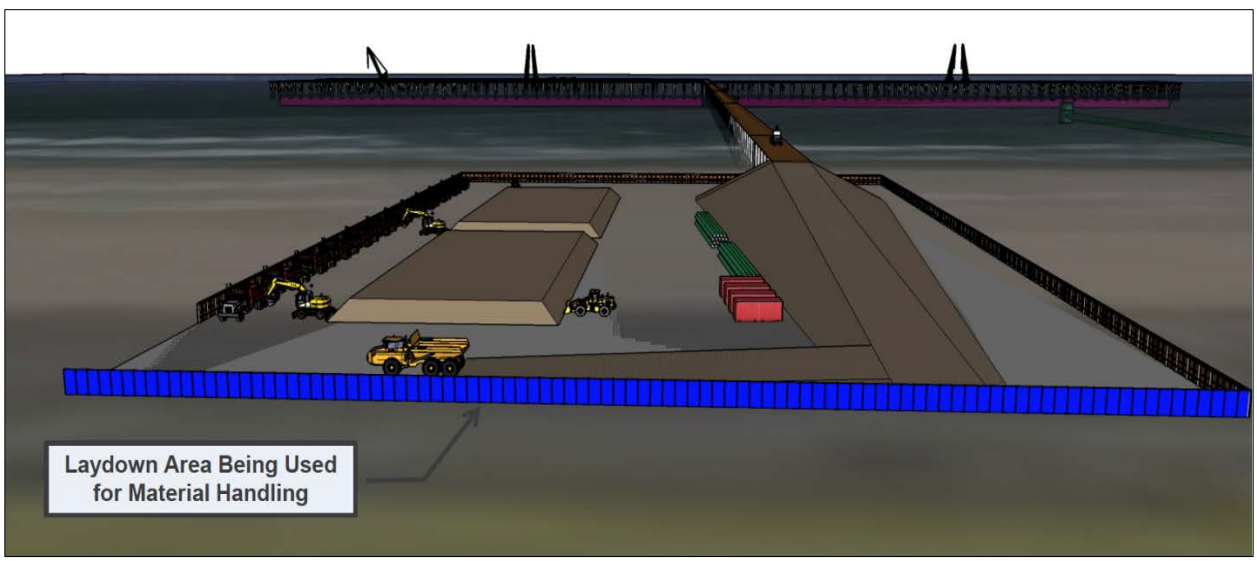

Figure 3.15 Construction Area for the SIG

Because of the need to excavate within the ocean under variable conditions, the Independent Science Advisory Panel (ISTAP) convened by the California Coastal Commission to evaluate SSI alternatives for Huntington Beach determined that construction of a SIG would require construction of a permanent pier that could be used for construction vehicles and subsequently maintenance of the facilities.[19] Such a pier is presented in Figure 3.15. Barges with the ability to create stable platforms (i.e., barges on stilts) were

[18] ISTAP Phase 2, 2015. Final Report: Feasibility of Subsurface Intake Designs for the Proposed Poseidon Water Desalination Facility at Huntington Beach, California. Published under the Auspices of the California Coastal Commission and Poseidon Resources (Surfside) LLC. August 17, 2015

[19] ISTAP. 2015. Phase 2 Report: Feasibility of Subsurface Intake Designs for the Proposed Poseidon Water Desalination Facility at Huntington Beach, California. August 17, 2015.

also considered, however these barges didn't provide the access required for long-term maintenance and dredging of the fill material.

The construction of a SIG intake will be more complex than other intake alternatives because of the significant amount of construction on and offshore, along a large portion of the beach. The construction of this type of intake would have many significant environmental impacts, including:

- Irreversible destruction/loss of benthic marine and coastal habitat within the footprint of the SIG, which must be removed in order to install the infiltration galley cells.
- Excavation and construction of onshore intake pump wet-wells and trenches for collector piping along a strip of the shoreline would limit public access to the beach, which would result in a significant impact on the beneficial use of the shoreline by the public and will cause measurable loss of local tax revenue and income to visitor serving businesses in the city.
- The need to dewater and dispose large amounts of ocean bottom sediments to a sanitary landfill or ocean disposal site. Landfills may not be capable of accepting such a large volume of solids waste over the relatively short excavation period and ocean disposal may have regulatory restrictions.
- There are an extremely large number of construction vehicles and dump trucks required. If native bottom sediments of the excavated ocean floor are unsuitable, they will need to be replaced with engineered media (e.g., sand) that will need to be delivered on site, the amount of construction truck trips and associated traffic congestion will double. The total amount of truck traffic associated with the construction of the infiltration gallery can be multiple times higher than the truck traffic associated with the construction of a desalination plant.
- Because of the order-of-magnitude increase in traffic load due to the construction of the SIG, the total direct greenhouse gas (GHG) emissions associated with project construction will greater than other SSI alternatives. [18]

Construction Schedule

The construction of the SIG and pump station could be completed within the timeframe of the construction of other alternative subsurface intakes (i.e., approximately two years). However, this would only be possible with multiple crews completing simultaneous excavation and construction along the entire length of the intake footprint. Because of the significantly higher construction intensity and labor needs, traffic and other construction impacts associated with the construction of the SIG will exceed any other type subsurface intake considered.

Horizontal Directionally Drilled (HDD) Wells

The HDD well systems are shallow directionally drilled wells that consist of blank well casings with one or more horizontal perforated screens bored at an angle (typically inclined at 15 to 20 degrees) and extending from the surface entry point underground past the mean-tide line. This type of well has found application mainly in seawater desalination installations. One of the most widely used HDD well intakes today is the Neodren™ well intake system. A general schematic of Neodren™ horizontal directionally drilled (HDD) well intake is shown on Figure 3.16.[20]

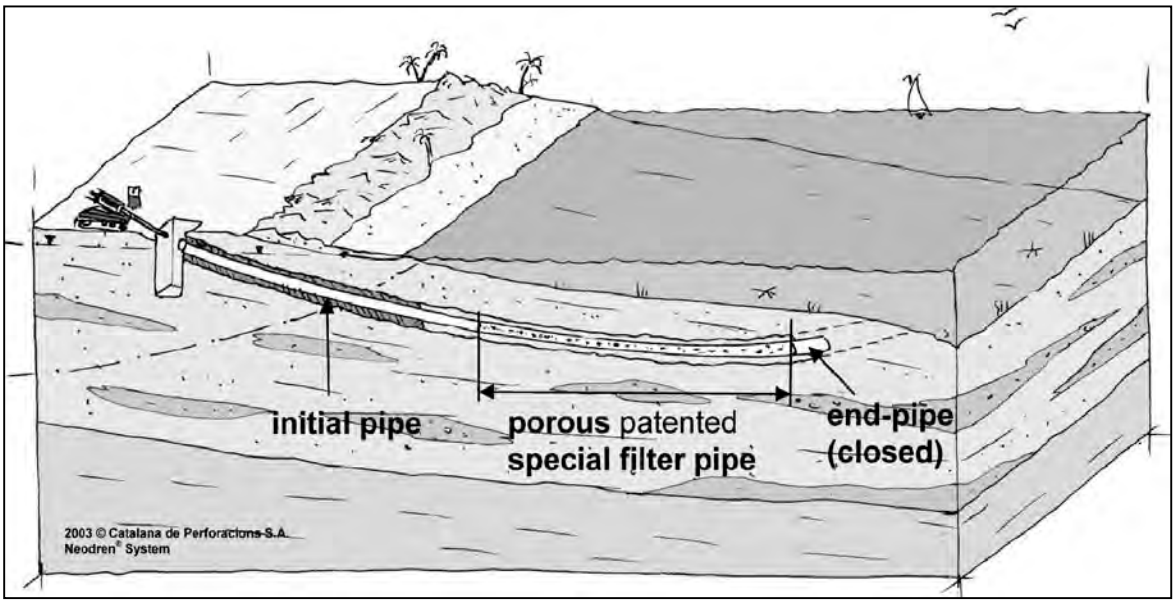

Figure 3.16 Neodren™ HDD Intake System

The source water is collected via a number of perforated high density polyethylene (HDPE) pipes in a cluster configuration. Similar to the SIG, ocean water is filtered through the ocean bottom sediments before it reaches the desalination plant. It is unknown if larval stage marine organisms are entrained in the ocean bottom sediments. The typical HDD pipe size diameter is 18 inches; however, diameters as large as 28-inches could be feasibly installed. Typically, the individual HDD collector pipes deliver the source water into a common wet well where it is pumped to the desalination plant for treatment. Individual HDD collector well pipes of this type yield between 1.1 to 3.4 mgd.

An array of HDD collector pipes are usually installed at a depth between 16 and 33 feet below the ocean bottom in separate boreholes by drilling under the ocean seabed at distances of up to 2,000 feet to a location that can yield seawater and avoid capturing groundwater from the fresh near-shore aquifer. The drilling technology employed for the installation of the Neodren™ system is well proven and has found application for a number

[20] Peters, T. and D. Pinto (2008) Seawater Intake and Pretreatment/Brine Discharge – Environmental Issues, Desalination, 221, pp. 576-584.

of other applications such as laying fiber-optic cables and oil and gas pipelines. The capacity of the collected seawater depends on the number and the diameter of the horizontally drilled perforated pipes; the length of the perforated portion of the pipes; and the transmissivity and depth of the seabed in which the collector pipes are drilled.

Experience at Other Locations

The Neodren™ HDD intake technology is patented by the Spanish company Catalana de Perforacions. This technology has been used for over ten years in several small and medium-size seawater desalination plants in Spain, but does not have any applications in the United States.

One of the largest seawater desalination plants using HDD wells in operation is the New Cartagena Canal (San Pedro de Pinatar) plant. It is located in Almeria, Spain and has a capacity of 17 mgd. The intake system consists of 20 HDD wells arranged in a fan shape as depicted in Figure 3.17.

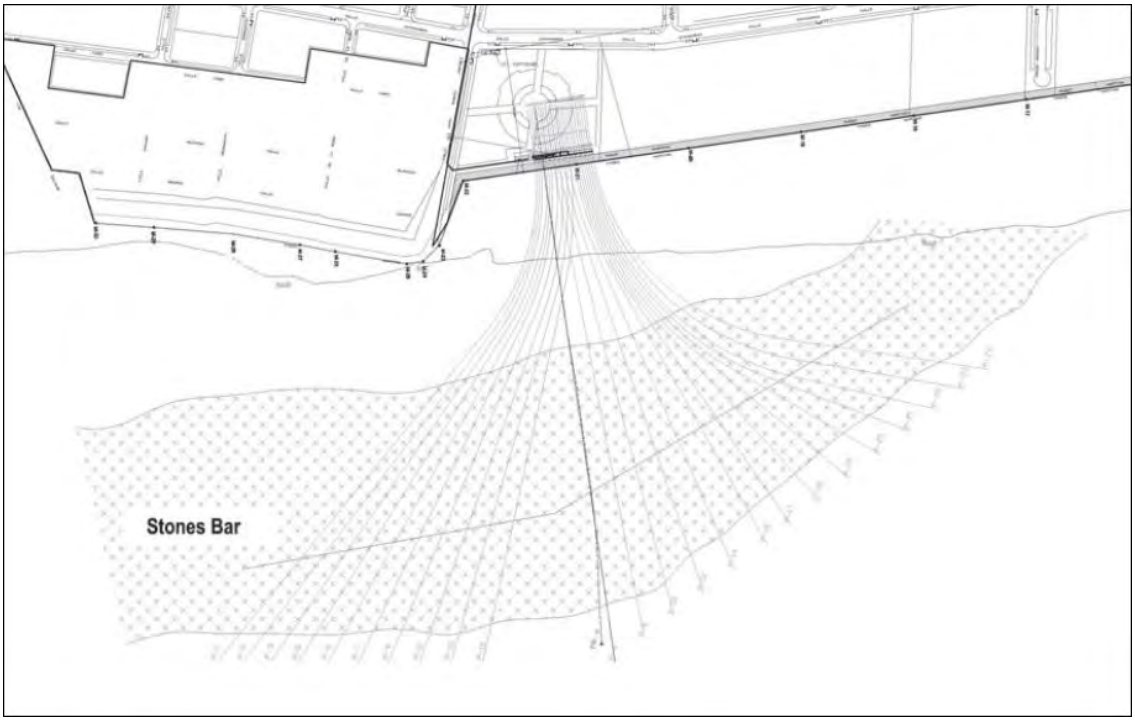

Figure 3.17 HDD Intake of San Pedro de Pinatar SWRO Plant

The individual intake wells are between 1,650 and 1,980 feet long and are 14-inches diameter. Each well produces between 2.3 and 3.1 mgd and the desalination plant operates at 45% recovery. The water is collected in a large wet well (located under a parking lot to reduce visual impacts) located underground and pumped to the plant using submersible pumps.

Experience with the use of HDD wells at this plant, indicates that the plant intake has encountered significant “technical issues and limitations” causing the plant’s owner to

switch to an open water intake system for the plant's Phase-2 expansion.[21] Four of the wells lost over 40% of their production capacity within the first nine months of plant operation; furthermore, the capacity of the other wells has continued to diminish over time. Such productivity reduction triggered the need to install additional intake wells and ultimately to build open intake for the second phase of the plant expansion.

Reliable Water Quality

Available data[22] indicates that while the HDD system can successfully reduce source seawater turbidity and total organic carbon (TOC), this reduction is not typically adequate to eliminate the need for pretreatment. The source water would still require a filtration system prior to membrane separation. For the site specific conditions of a project, the actual source water quality which is difficult to predict and it is unknown whether an existing desalination plant pretreatment system is adequate without further expansion. If this intake technology is determined to be feasible, pilot testing would be necessary to determine the need for additional pretreatment.

Reliable Capacity

When HDD wells were introduced on the market in late 1998, they initially received acceptance in Spain and have been considered a viable intake alternative for a number of other countries. However, after five years of operational experience, many of the plant intakes have faced production reliability challenges (loss of productivity due to blockage of the perforated piping). As a result, HDD wells have not been used for full-scale desalination projects worldwide since 2010.

An HDD intake system that would be capable of collecting approximately 23 mgd of source water needed for the City's desalination plant would consist of multiple intake wells (potentially as many as 27), which will collect water into a common wet well located inland from the beach.

Maintenance Requirements

Based on experience with other forms of infiltration galleries, HDD intake maintenance may be challenging due to their historical tendency to clog. Furthermore, experience shows that once the intake collectors become plugged and the productivity of the individual collectors' decreases, it is impossible to recover the original full capacity.[23] In addition, maintenance of

[21] California Coastal Commission CDP application E-06-013 November 15, 2007, hearing transcript pages 170-171.
[22] Peters, T. and D. Pinto (2008) Seawater Intake and Pretreatment/Brine Discharge – Environmental Issues, Desalination, 221, pp. 576-584.

[23] Rachman, R., S. Li, T. Missimer (2014) SWRO water quality improvement using subsurface intakes in Oman, Turks and Caicos Island, and Saudi Arabia, Desalination 31, pp. 88-100.

HDD intake pumps will be required, similar to collector well, SIG, beach infiltration gallery, and open ocean intake systems.

Construction Related Impacts

The construction of HDD intake systems will require specialized equipment and materials that are not readily available in the US. Drilling methods are, however, similar to other HDD operations. When performing HDD in an unconsolidated media like sand, it will be necessary to pressurize the bore hole and stabilize the walls by coating them with drilling mud (e.g., bentonite) or another similar drilling fluid. Because of the pressure involved to coat the hole walls with mud, frac-out of the drilling mud has occurred in other HDD operations. Such a prospect would result in the potential release of drilling fluid into the ocean environment. Furthermore, conventional HDD pipeline installation (and all HDD intake well construction to date) involves daylighting the drill offshore resulting in the release of drilling fluid to the ocean. However, Neodren™ claims that they have a new technique that avoids daylighting the drill offshore. As of late 2015, this method of construction has yet to be demonstrated successfully for an intake well.

The area required for construction and the associated impacts will likely be similar to slant wells.

Construction Schedule

The estimated time for construction of an HDD intake system is approximately two years, based on construction of other wells of the same size built in Spain.

3.2.4 Subsurface Properties

Available literature that describes subsurface properties and characteristics in the vicinity of the shoreline at each project site were reviewed, summarized, and used as a basis to model production of water from SSI alternatives. This information and modeling output is used to identify potential areas for focused evaluation and analyze subsurface intake feasibility, including capacity, and potential environmental impacts.

Appendix B provides a detailed review and analysis of subsurface properties, which is briefly summarized in the following subsections.

Hydrogeologic Setting

The study area lies within the Santa Barbara Groundwater Basin (California Department of Water Resources Basin 3-17), which includes two sub-basins referred to as Storage Units I and III.[24] The unconsolidated deposits (i.e., loose sediments) of Storage Unit I are subdivided into four zones including, from top to bottom:

[24] Martin, P. 1984. Ground-Water Monitoring at Santa Barbara, California: Phase 2-Effects of Pumping on Water Levels and on Water Quality in the Santa Barbara Ground-Water Basin. U.S. Geological Survey Water Supply Paper 2197.

1. Shallow Zone,
2. Upper Producing Zone (UPZ),
3. Middle Zone, and
4. Lower Producing Zone (LPZ).

The target zone within Storage Unit I for SSI alternatives that utilize wells is limited to the Shallow Zone. The Shallow Zone is approximately 200 feet thick in the study area and generally thickens towards the south (seaward), presumably continuing offshore. The UPZ and LPZ are not considered target zones for SSIs due to their limited hydraulic connection to seawater and because these aquifers are the primary aquifers supporting City production wells.

There is very little groundwater production in Storage Unit III (a single well) due to poor production capability and quality. Due to the very low permeability of sedimentary bedrock beneath the Shallow Zone, the target zone within Storage Unit III for SSI alternatives that utilize wells is limited to the Shallow Zone.

Geologic Cross Section Development

To understand the geology in the study area and determine specific target zones within the Shallow Zone for each of the SSI alternatives, seven geologic cross sections were developed based on a review of existing subsurface data including: borehole logs, cone penetration tests (CTPs), test pile drive analyses, surficial geologic maps, other published cross sections, offshore geophysical survey data, a coastal bathymetry survey, historical aerial photographs and various geologic reports from public and private sources. Based upon this information, sediments were characterized using the Unified Soil Classification System (USCS) and were further simplified to be sand, silt, or clay in the final cross sections. The results of the geologic cross section development were integral to the development of the numeric hydrogeologic model of the study site and also to illustrate the occurrence and distribution of subsurface geological materials both parallel and perpendicular to the shoreline.

Numeric modeling (MODFLOW) and analytical modeling results based upon the output of the geologic cross section development are presented in Section 3.3 of this report and summarize the yield, spacing, and number of wells required.

3.2.5 Coastal Hazards and Sediment Transport Analysis

Coastal hazards (tsunami hazard, sea level rise, and wave runup) and sediment transport (i.e., erosion or deposition) were evaluated to assess a subsurface intake alternative's susceptibility to oceanographic hazards. This information establishes a design basis used to determine applicable protective features or maintenance that may be required for each subsurface intake alternative. For the purposes of the coastal hazards and sediment transport analysis, only shallow subsurface intake technologies were discussed in detail.

These technologies are most significantly affected by oceanographic hazards such as erosion, sea level rise, and tsunamis due to their shallow construction and proximity to the ocean. However, because of their close proximity to the ocean and similar construction for their associated surface facilities, coastal hazards for vertical and slant wells constructed at or near East, West, or Leadbetter Beach would be similar to the hazards and conclusions presented for horizontal collector well type intakes at these locations.

The following subsections present the technical results for each of these analyses, which are presented in greater detail in Appendix C.

Methodology Overview

As described in the Work Plan (Appendix A), a sediment budget analysis was performed on using the Coastal Evolution Model (CEM) developed at the Scripps Institution of Oceanography. The sediment budget analysis provided far field inputs (large scale - i.e., regional, Santa Barbara Littoral Cell) to a near field (localized to areas of interest - i.e., East, West and Leadbetter Beach) seafloor stability and coastal hazards analysis of the site specific conditions and infrastructure associated with the subsurface intake alternatives evaluated as part of this study. The viability of each subsurface intake option was evaluated based upon the results of the seafloor stability and erosion analysis; the coastal hazards analysis evaluated vulnerabilities of all shore-side and offshore structures associated with the various subsurface intake alternatives.

Sediment Transport Evaluation

Results of the far-field (regional) analysis showed shoreline sediment drift flowing eastward. The flow of sediments is away from sources such as the creeks, streams, and bluffs of the Santa Ynez Mountains to the west, and flowing toward the Santa Barbara Harbor, which acts as a local sediment sink (i.e., where sediment accumulates). With regional sediment transport quantified, CEM solutions were obtained for a higher resolution grid in the near field of the candidate sites for subsurface intake facilities (i.e., East, West, and Leadbetter Beach). Key findings are presented for each beach site considered in the following subsections.

East Beach:

- Sediment transport is highly variable, but modeling results generally indicate an erosion condition. High variability exists due to dredge disposal activities that use East Beach as a receiver beach; however, erosion also occurs as a result of wave refraction and the sediment trapping effect that the harbor breakwater and entrance exert. Thus, East Beach is unstable and only exists because of regular nourishment by dredge material disposal.

- The depth of sediments are highly variable, changing by as much as 9.5 ft. This will require the top elevation of engineered fill for an onshore infiltration gallery to be at

least -9.5 ft below mean sea level (MSL) (i.e., requiring excavation to a depth of -24.5 ft MSL); construction of an offshore SIG could be placed at or beyond a depth of -51.5 ft MSL, the depth at which bottom elevation changes diminish. Such an installation would require a total excavation depth of 15 ft below existing grade, however, this bottom depth of -51.5 ft MSL doesn't occur within the 1/2-mile area offshore that was established in the Work Plan as a basis for this study.

- East Beach is an exposed open-coast site subject to high-energy Gulf of Alaska and El Nino storm waves. Therefore, both an onshore infiltration gallery and SIG technologies would be subject to construction and operational challenges.

- Neodren is feasible but not likely to be a reliable long-term source water supply alternative for this site because sediment cover is limited and comprised of a high percentage of silts and clays, which may cause reduced infiltration rates and potential clogging of drains. Because the sediment is highly variable, Neodren drains could be placed as shallow as 10 ft below existing grade in the bar-berm back beach section and as shallow as 8.5 ft below grade in offshore portions of bottom profile.

West Beach:

- Sediment transport modeling indicates deposition of sediment across the harbor entrance and West Beach. Beach and bottom profiles off of West Beach are controlled by dredge cuts and are not natural equilibrium formations due to this deposition of sediment.

- Sediment depth within the inner 400 ft of West Beach has a very high variability, changing by as much as 16 ft; the outer 800 ft is considerably more stable, and is estimated to vary by only 2-4 ft of thickness. Because it is sheltered by harbor breakwater, the inner portion of West Beach (i.e., inner 400 ft) appears to be a good site for an onshore infiltration gallery. The top elevation of engineered fill would have to be placed at least -8 ft MSL, or the depth of the deepest dredge cut would require the excavation of a hole to -23 ft MSL. Construction and operation of a SIG would be problematic because construction and periodic dredging activities would interfere with harbor use and navigation. Beach depth variability will result in inconsistent source water quality.

- Neodren technology is not particularly favorable because the quiet harbor waters allow significant fractions of fine grained sediments to settle, which may cause reduced infiltration rates and potential clogging of drains.

Leadbetter Beach:

- Sediment transport modeling indicates deposition of sediment across Leadbetter Beach.

- The sediment cover is abundant due to the formation of a fillet beach and the stabilization action of the harbor breakwater. It does, however, experience seasonal beach profile shifts and estimated beach profile changes of up to 11 ft in depth can occur. The top elevation for engineered fill for an onshore infiltration gallery would have to be placed at least -11 ft MSL (i.e., requiring excavation to a depth of -26 ft MSL); an offshore SIG could be placed at or beyond a depth of -49.2 ft MSL where shifts in sediment elevations are no longer an issue. Such an installation would require excavation of a 15 ft deep hole in the seabed; however, this bottom depth of -49.2 ft MSL doesn't occur within the 1/2-mile area offshore that was established in the Work Plan as a basis for this study.

- Leadbetter beach is an exposed open-coast site subject to the erosion action caused by the high-energy Gulf of Alaska and El Nino storm waves. Therefore, both onshore infiltration gallery and SIG technologies would be subject to construction and operation challenges.

- A Neodren system can be constructed entirely from landside launch points using HDD techniques. Leadbetter site appears favorable because its depositional environment assures adequate and continuous sediment cover comprised of predominately sand-sized particles. Neodren drains could be placed as shallow as 12 ft below existing grade in the bar-berm back beach section and as shallow as 6 ft below grade in offshore portions of bottom profile.

Sea Level Rise and Wave Runup Evaluation

Wave runup and overtopping was analyzed for possible shore-side infrastructure associated with subsurface intake alternatives assuming present conditions and future scenarios that included sea level rise. The following site locations were considered in the analysis:

1. Existing Charles Meyer Desalination Plant @ 525 Yanonali Ave. (elevation + 10 ft NGVD)
2. Pump station @ 420 Quinientos St. (elevation + 8 ft NGVD)
3. Collector well site @ 401 E. Yanonali Ave. (elevation + 12 ft NGVD)
4. Collector well site @ 103 S. Calle Cesar Chavez (Elevation + 10 ft NGVD)

The analysis presented in Appendix C was based on the 32-year wave record combined with a tidal hydroperiod function. A 50-year sea level rise projection (corresponding to Year 2065) was obtained using methodology presented in the 2013 California Coastal Commission’s Draft Sea Level Rise Guidance document. A low estimate of sea level rise of 7 inches and a high estimate of 35 inches was derived from these methods.

Results of the full analysis reveal that shoreside facilities located near East, West and Leadbetter Beach will be threatened by flooding from wave runup, as will the pump station

located at 420 Quinientos St. and collector well site located at 103 S. Calle Cesar Chavez. It is noted that these facilities are only threatened at future sea levels; the most serious threat are the possible beach facilities and the pump station at 420 Quinientos, where wave runup can reach as high as 11.1 ft NGVD, or 3.1 ft above the site elevation.

Tsunami Hazard Evaluation

Tsunami induced erosion, runup, and inundation was analyzed assuming present conditions and future scenarios that consider sea level rise described in the section above. Results showed that a tsunami is capable of eroding as much as 4 to 6 ft of seabed offshore, and could erode as much as 12 ft of beach sediment cover in a single wave breaking event. Thus, offshore subsurface intake components could be damaged by the erosion caused by such an event.

Furthermore, tsunami runup and inundation calculations indicate that every shore facility associated with potential subsurface intake alternatives would suffer serious degrees of overtopping during a tsunami event. Results are consistent with the FEMA tsunami flood map that indicates all of the East Beach corridor extending several miles inland along Mission Creek and Laguna Channel will be inundated by a shoaling tsunami solitary wave.

Summary

Based upon the results of the coastal hazards and sediment transport analysis summarized above and presented in Appendix C, the following conclusions can be made:

- West Beach is well suited for an onshore infiltration gallery[25], but is not well suited for a SIG or Neodren HDD technology. It should be noted however, that due to the nature of the beach bottom sediments (i.e., fine sands with low permeability) there is a high potential for an onshore infiltration gallery to become plugged with silt over time. Therefore it is very unlikely that West Beach could sustain reliable long-term collection of the large intake volumes needed for the City's desalination plant operations.

- The Leadbetter Beach intake site was found to be feasible for onshore infiltration gallery and SIG technologies; however, both technologies may experience construction and operational challenges due to exposure to high energy wave climates. Because of the depth required to reach stable bottom conditions (i.e., -49.2 ft MSL), a SIG cannot be constructed within the 1/2-mile area offshore that was established in the Work Plan as a basis for this study.

- Neodren was found to be a viable technology for Leadbetter Beach, and was also the only viable option for East Beach.

[25] Also referred to as lateral beach wells, beach infiltration galleries, and BIG in other reports and appendices.

- None of the shore-side facilities will be flooded by wave run up at present sea levels, although future sea level scenarios will cause flooding from wave run up at the pump station site (420 Quinientos St.) and one well collector site (103 S. Calle Cesar Chavez). All shore-side facilities will be inundated by a tsunami event; and, if located at a depth greater than 12 ft, only Neodren has the potential to be unaffected by tsunami erosion.

3.2.6 Water Quality and Treatment Needs

By examining the subsurface properties presented in Section 3.2.4, the geochemistry data presented in Appendix B, and reviewing experience at other SSI facilities, subsurface intake water quality and treatment needs can be estimated for each subsurface intake technology alternative. In some cases, subsurface intakes may eliminate or significantly reduce the need for a pretreatment system that would be needed to produce an equivalent RO feed water quality if a surface intake were used. However, because the City's desalination plant is already equipped with pre-filtration to remove suspended solids, it is assumed that filtration technology will continue to be used to remove suspended solids. Experience at other desalination facilities utilizing SSIs has shown that iron and manganese concentrations may be elevated, so additional pre-treatment may be needed to prevent fouling of the membranes.

The quality of seawater and groundwater was evaluated to estimate the amount and concentration of suspended solids, iron, and manganese and the relative percentage of seawater and groundwater that may be expected in produced water. Data was compared against other installations to determine the basis of design water quality that can be expected. The need for additional pretreatment was evaluated for dissolved iron and manganese. Table 3.1 presents a summary of the design basis water quality and pretreatment needs for the various intake alternatives.

3.2.7 Project Life

As discussed in the Work Plan, a 20-year project life will be assumed for a subsurface intake system. A 20-year project life was selected based upon the time that is assumed to be required for repayment of any loan used to finance a subsurface intake project. It may be difficult for the City to finance replacement facilities before they are finished paying for and using the original project.

Table 3.1 Estimated Intake Water Quality and Treatment Needs						
Parameter Concentration = mg/L	**Vertical Wells**(1)	**Onshore Infiltration Gallery**(2)	**Radial Collector Wells**(1)	**Slant Wells**(1)	**SIG**(3)	**HDD Wells**(3)
Estimated Turbidity (NTU)	<1	1 - 20	<1	<1	<1 - 3[5]	<1 - 3[5]
Estimated Total Fe^{2+} (mg/L)	1 - 3	1 - 3	1 - 3	1 - 3	< 0.5	< 0.5
Estimated Total Mn^{2+} (mg/L)	0.5 - 1.2	0.5 - 1.2	0.5 - 1.2	0.5 - 1.2	<0.001	<0.001
Estimated Percentage of Seawater (4)	<50%	95%	61-70%	92-95%	100%	100%
Pre-treatment Required? (6)	Y	Y	Y	Y	Y	Y

Notes:

(1) USGS, 2015, average of 5 shallow USGS monitoring wells along Santa Barbara waterfront.
(2) CH2MHill, 1990, Sand Lens report, Beach sand lens pumping test water quality analytical results
(3) CH2MHill, 1990, Sand Lens report, Santa Barbara Seawater analytical results
(4) Percentage of seawater estimated using numerical modeling as described in Appendix B.
(5) Results from experience at other facilities referenced in Section 3.2.3.
(6) Pre-treatment for iron and manganese or turbidity will likely be required for all SSI alternatives.

3.2.8 Reliability Features

Based upon the intake type, hydrogeology, geochemistry, and other factors including experience at other SSI facilities, safety factors have been assumed as a basis of design requirement to determine the redundancy required to address downtime for replacement, maintenance and repairs, as well as a possible decrease in production capacity due to plugging. Portions of facilities that were assumed to require some level of redundancy in the design included individual vertical, slant, and HDD wells, length of screen (all SSI facilities), and pumps placed in SSI facilities utilizing caissons or wet wells (collector wells, onshore infiltration gallery, SIG, and HDD). Table 3.2 presents a summary of the reliability features and assumed safety factors for each intake technology alternative.

Table 3.2 Intake Reliability Features

Parameter	Vertical Wells	Onshore Infiltration Galleries	Radial Collector Wells	Slant Wells	SIG	HDD Wells
Number of Facilities	N+1	N	N	N+1	N	N+1
Screened Area Safety Factor	X 1.2	X 1.2	X 1.2	X 1.2	X 1.2	X 1.2
Additional Pumps	NA	N+1	N+1	NA	N+1	N+1

Notes:

(1) Number of facilities (N) was increased by one for facilities (vertical wells, slant wells, HDD wells) with individual well casings. No redundancy was assumed for other facilities.
(2) Screen lengths were assumed to be increased by 20% to account for clogging.
(3) Additional pumps were added (N+1) in facilities with caissons and wet wells for water collection. Pump redundancy was not assumed for other facilities with individual pumps in well casings.

3.3 Hydrogeological Analysis of Subsurface Intake Systems

Based upon the BOD criteria presented in Section 3.2, each subsurface intake alternative was analyzed to determine conceptual design criteria that can be used to determine the following:

- Individual facility yield, spacing for multiple locations of given subsurface intake type, and length of beach required to produce 15,898 gpm and 10,000 AFY, or the maximum yield achievable if the 10,000 AFY production cannot be met.
- Percentage of ocean water captured by the subsurface intake
- Impacts to local groundwater supplies and sensitive habitats
- Potential to capture or mobilize known groundwater contamination

A detailed hydrogeological analysis of these parameters is presented in Appendix B, but is summarized in the following subsections.

3.3.1 Yield, Intake Facility Spacing, and Length of Beach Required

The yield, spacing, and number of wells required to produce the target capacity for each SSI alternative were evaluated using analytical or numerical (MODFLOW) methods based on understanding of the geometry and nature of the aquifer materials discussed in Section 3.2.4. Vertical wells, onshore infiltration galleries, radial collector (Ranney) wells, and slant wells were analyzed using simplified numerical modeling methods. SIG and HDD alternatives were evaluated using analytical methods. The simplified model layers and assumed hydraulic properties used for numerical and analytical methods were based on the geologic cross sections discussed previously. Refer to Appendix B for additional details regarding hydrogeological modeling for each of the SSI alternatives.

Results from the hydrological modeling conducted during this study are summarized in Table 3.3. Of the six SSI alternatives considered, only the SIG and HDD technologies are able to satisfy the study production goal of 15,898 gpm.

Table 3.3 Hydrogeological Analysis - Summary of SSI Alternatives

Intake Type	Shallow Zone Layer	No. Facilities Required[1]	Approx. Spacing (feet)	Beach Length Required (miles)[1]	Yield per Facility (gpm)	Potential Total Yield (gpm)[2]
Vertical Wells	Lower Sand	40 - 160	600 - 750	5.5 – 18	100-400	1,500 - 4,800
Onshore Infiltration Gallery	Upper Sand	6	N/A	3	Varies with length of available beach	10,100
Radial Collector Wells	Upper Sand	43	600	5	375	5,625
	Lower Sand	16 - 58	600 – 1,500	4 - 6	275 - 1,000	4,125 - 7,000
Slant Wells	Lower Sand	16 - 58	560 - 1,125	3.5 - 6	275 - 1,000	4,400 - 8,000
SIG	Upper Sand	1	N/A[3]	N/A[3]	15,898	15,898
HDD	Upper Sand	11	N/A	0.1	1,500	15,898

Notes:
(1) Total required to meet 15,898 gpm.
(2) Potential yield within available beach.
(3) SSI is constructed offshore.

3.3.2 Percentage of Ocean Water Inflow and Impact to Local Groundwater

The summary of yield for SSI alternatives on available beach including contribution of water from ocean and inland sources is presented in Table 3.4. Of SSI alternatives considered, only the SIG and HDD technologies derive all of their flow from offshore sources and do not have an impact on inland groundwater or coastal sensitive habitats.

The offset distance that each facility has to the mean tide line influences seawater contribution to the yield (i.e., vertical wells have higher contribution of inland water than slant wells because slant wells are closer in position to mean tide line). Furthermore, the estimation of the seawater contribution assumes continuous operation of each intake facility. Intermittent operation of the facility would decrease the contribution from offshore because of time to establish the offshore hydraulic connection.

3.3.3 Impacts to Sensitive Habitats

Numerical modeling estimated drawdown in the sensitive habitats and construction setback areas identified in this study caused by operating vertical wells, onshore infiltration galleries, radial collector wells, and slant wells at the maximum feasible production flow previously identified in Section 3.3.1 (i.e., summarized in Table 3.3). The maximum yield determined in Section 3.3.1 was based upon available beach area for facilities, as well as the maximum production that could be achieved without interfering with well operation (e.g., drawing down into the screened area and/or affecting pump operation). Results of the SSI drawdown modeling work are presented in Table 3.5. The following findings can be concluded:

- As expected all onshore well and the onshore infiltration gallery alternatives are estimated to impact the sensitive habitat areas.
- The onshore infiltration is estimated to have the most significant impact (i.e., up to 4 feet of drawdown).
- As presented previously in Table 3.3, vertical wells, onshore infiltration galleries, radial collector wells, and slant wells did not meet the capacity goals of this study due to limited beach area and hydrogeologic properties of the candidate sites. The potential yield within available beach is estimated to result in negative impacts to sensitive areas due to water level drawdown. In order to reduce impacts to sensitive areas, the potential yield could be further reduced by moving well facilities further from the sensitive areas and possibly reducing the flow rate from each type of SSI facility. However, to conduct further analysis of this nature, a de minimize impact to the sensitive areas must be established.
- Due to their distance and position away from the sensitive areas, the operation of SIG and HDD technologies are not expected to have significant impact on groundwater or sensitive habitats.

Table 3.4 Feasible Yield and Ocean/Inland Water Contribution Summary

Intake Type	Shallow Zone Layer	No. Facilities Required[1]	Yield Per Facility (gpm)	Potential Total Yield (gpm)[2]	Approx. Spacing (feet)	Screen Length (feet)	Inland Contribution[3]	Offshore Contribution[4]
Vertical Wells	Lower Sand (high K)	12	400	4,800	750	60	18%	82%
	Lower Sand (low K)	15	100	1,500	550	60	47%	53%
Onshore Infiltration Gallery	Upper Sand	6	Varies with Length	10,100	NA	9,000	5%	95%
Radial Collector Wells	Upper Sand	15	375	5,600	600	750	30%	70%
	Lower Sand (high K)	7	1,000	7,000	1,500		30%	70%
	Lower Sand (low K)	15	275	4,125	700		39%	61%
Slant Wells	Lower Sand (high K)	8	1,000	8,000	1,125	175	8%	92%
	Lower Sand (low K)	16	275	4,400	560		5%	95%
SIG	Upper Sand	1	15,898	15,898	N/A	5,000	0%	100%
HDD	Upper Sand	11	1,500	15,898	N/A	11,000	0%	100%

Notes:

(1) Total required to meet 15,898 gpm.
(2) Potential yield within available beach.
(3) Percentage of total produced flow derived from inland sources (groundwater).
(4) Percentage of total produced flow derived from offshore sources (seawater).

Table 3.5 Impacts to Local Groundwater and Sensitive Habitats	
Intake Type	**Drawdown Beneath Sensitive Habitats**
Vertical Wells	1 to 3 feet
Onshore Infiltration Gallery	1 to 4 feet[1]
Radial Collector Wells	0.5 to 3 feet
Slant Wells	1 to 3 feet
SIG	0 feet
HDD	0 feet

Note:
(1) Measured at a distance 250 feet from end of trench.

3.3.4 Capture of Known Groundwater Pollutants

SSI alternatives that derive a portion of their flow from inland sources can mobilize groundwater contaminants and carry them into the source water for the desalination plant or change gradients that affect ongoing remediation activities (e.g., pump and treat). A total of 75 contaminated sites were identified within the study area between Highway 101 and the coast; nine of these sites are listed as open and include contamination from heavy metals, gasoline, diesel, waste oil, solvents, polynuclear aromatic hydrocarbons, and total petroleum hydrocarbons.

Other potential sources of contamination are known to have impacted groundwater quality in the Shallow Zone near Santa Barbara. Soils contaminated with trash, lead, and hydrocarbons are routinely found in this area and are likely to influence groundwater quality. Because of the extent and prevalence of contamination, the City's building department requires soils testing for issuance of building permits in this area. Therefore, groundwater monitoring in the Shallow Zone is recommended prior to proceeding with development of vertical wells, radial collector wells, onshore infiltration galleries, and/or slant wells, as these technologies were shown to derive a portion of their flow from inland groundwater sources.

3.4 Conceptual Design Summary

This section presents the conceptual design for each SSI alternative that will be used in the initial screening analysis. The basis of design and the analysis of subsurface intake systems presented in Sections 3.2 and 3.3, respectively, were used to inform and develop this conceptual design. It is noted that only alternatives meeting basis of design requirements established in this TM were used to develop these conceptual designs. For alternatives that are unable to meet the production requirements set forth in study goals (e.g., produce 15,898 gpm), conceptual designs were developed based on the greatest production capacity that could be obtained given the available beach length.

The total beach length determined to be available for SSI development is 9,000 feet, and, excluding habitat areas that cannot be built upon, consists of the following:

- East Beach – 5,300 feet
- West Beach – 1,300 feet
- Leadbetter Beach – 2,400 feet

None of the conceptual designs presented in the following subsections require condemnation of property for facilities. Each of the alternatives presented in this report use City-owned property, rights of way, or existing easements. Two project site alternatives (401 E. Yanonali Street and 103 S. Calle Cesar Chavez) were not required for conceptual design purposes since designs assume the re-use of the City's existing intake pipeline.

The following subsections provide a summary of the conceptual designs developed for each intake alternative.

3.4.1 Vertical Wells

Table 3.6 and Figure 3.18 present the conceptual design summary and layout for vertical wells, respectively. Vertical wells located along East, West, and Leadbetter beaches are estimated to be 120 feet deep with 12-inch diameter PVC or fiberglass casings. The well screen would extend from -60 to -120 feet. Individual wells would be equipped with submersible pumps and are estimated to yield between 100 and 400 gpm based upon available drawdown and aquifer properties. Therefore, the total number of vertical wells required to produce 10,000 AFY of desalinated water is 40 to 160 wells. Assuming 550 to 750 feet between each well, which is needed to minimize drawdown interference and associated loss of production, the total number of vertical wells that can be constructed on the available beach is 12-15, with a total production rate of 1,400 to 4,800 gpm. To minimize beach impacts and extend the facility life by increasing the surface facility elevation to avoid effects of sea level rise, vertical wells are to be constructed on the south side of Cabrillo Boulevard, approximately 150 feet from the mean high tide line.[26]

The discharge from each well would be connected to a common pipeline and directed to the City's desalination facility. To minimize the aesthetic impact of these well facilities, submersible pumps and well heads may be enclosed in vaults; however, access to each vault will be required for crane equipment to periodically service the submersible pumps. Electrical facilities to support the well operation may be grouped into five centers (i.e., spaced to minimize voltage drop in electrical cables to the furthest well) with a building height of 12 to 15 feet. Significant considerations regarding the conceptual design for vertical wells are summarized below:

[26] Setback of 150 feet to minimize impacts from tsunami erosion and sea level rise. Location also avoids installation of equipment that requires routine maintenance in recreational sandy beach areas.

- Vertical wells will impact shallow inland groundwater. It is estimated that approximately 47 percent of total flow produced will be obtained from inland sources.
- Vertical well operation will impact sensitive habitats and potentially be impacted by contamination in the area.
- Beach facilities would be susceptible to inundation and erosion as a result of tsunami and would also be increasingly impacted by seawater rise over the 20 year project life. Electrical buildings will need to be constructed in a manner that provides flood protection but this would result in a negative visual impact to a culturally sensitive park area.
- As shown in the attached hydrogeological analysis (Appendix B), construction does not require crossing any known faults.

Table 3.6 Conceptual Design: Vertical Wells

Description	Units	Criteria
Number of Wells[1]	No.	15
Well Structure	-	Vault with Access Road
Well Structure Dimensions (L x W x H)[2]	feet	8 x 8 x 1
Capacity, each	gpm	100
Capacity, total	gpm	1,400[4]
Pump Type	-	Submersible
Depth	feet	-120
Screen Location	feet	-60 to -120
Casing Diameter	inches	12
Well Spacing	feet	600
Set-back from mean high tide	feet	150
Maximum Electrical Cable Run[3]	feet	700[4]
Electrical Buildings Required	No.	5
Electrical Building Structure	-	Flood Protected Concrete Masonry Unit (CMU) Building
Electrical Building Dimensions (L x W x H)[2]	feet	12 x 24 x 15

Notes:
(1) Includes one standby well.
(2) Height dimension represents height above grade.
(3) Spacing based on assumed voltage drop for pump motors with a VFD.
(4) Assuming lower value of range of hydraulic conductivity.
(5) Electrical cable run includes spacing between wells and 100 feet of distance into the well.

Figure 3.18 - Vertical Wells Conceptual Layout

carollo

3.4.2 Onshore Infiltration Galleries

Table 3.7 and Figure 3.19 present the conceptual design summary and layout for onshore infiltration galleries, respectively. Onshore infiltration galleries would be constructed on each feasible segment of beach. The estimated total yield would be 5,000 gpm, with a total of three onshore infiltration galleries to be located on Leadbetter Beach (2), and West Beach(1). East Beach was determined an infeasible site for an onshore infiltration gallery due to sediment transport conditions and coastal hazards. To produce the required volume of water meeting study goals (15,898 gpm), ten onshore infiltration galleries would be required on approximately three miles of beachfront.

Each gallery would consist of 18-inch diameter HDPE drains buried in a 30-ft deep trench located 100 feet from the mean high tide line along each beach. The trench would be backfilled with engineered filter pack sand to reduce infiltration of fines and maximize production. Each end of the buried HDPE drain would be completed into a buried concrete wet well that would allow access for periodic jetting and redevelopment to reduce clogging effects. Wet wells are assumed to be constructed 150 feet back from the mean high tide line. [27]

Wet wells may be located off the sandy beach areas and will be equipped with pit mounted vertical turbine pumps that discharge to the desalination facility. The wet well/pump house will also contain electrical facilities. Estimated height of this structure is 12 to 15-feet and will require access for cranes to remove the vertical turbine pumps for periodic maintenance.

Significant considerations regarding the conceptual design for onshore infiltration galleries are summarized below:

- Sediment transport issues at East Beach make the implementation of an onshore infiltration gallery infeasible. Leadbetter Beach is feasible, however, problematic to construct. West Beach can support an onshore infiltration gallery.
- Onshore infiltration galleries will impact shallow inland groundwater. It is estimated that approximately 5 percent of total flow produced will be obtained from inland sources.
- Onshore infiltration gallery operation will impact sensitive habitats and potentially be impacted by contamination in the area.

[27] Setback of 150 feet to minimize impacts from tsunami erosion and sea level rise. Location also avoids installation of equipment that requires routine maintenance in recreational sandy beach areas.

- Infiltration galleries will be susceptible to inundation and erosion as a result of tsunami and would also be increasingly impacted by seawater rise over the 20 year project life. Electrical buildings and wet wells will need to be constructed in a manner that provides flood protection but this would result in a negative visual impact to a culturally sensitive park area.

- As shown in the attached hydrogeological analysis (Appendix B), construction does not require crossing any known faults.

Table 3.7 Conceptual Design: Onshore Infiltration Gallery

Description	Units	Criteria
Number of Pump Stations	No.	2[3]
Pump Station Structure	-	Flood Protected CMU Building
Pump Station Dimensions (L x W x H)[1]	feet	24 x 24 x 15
Pump Type	-	Pit Mounted Vertical Turbine
Capacity, total	gpm	5,000
Depth of Excavation[1]	feet	30
Drain Diameter	inches	18
Length of Drain, total	feet	4,500
Set-back from mean high tide	feet	150

Notes:
(1) Height dimension represents height above grade.
(2) Maximum depth of excavation.
(3) Includes one pump station for two onshore infiltration galleries at Leadbetter Beach; One Pump station for West Beach.

Figure 3.19 - Onshore Infiltration Gallery Conceptual Layout

carollo

3.4.3 Radial Collector (Ranney) Wells

Table 3.8 and Figure 3.20 present the conceptual design summary and layout for radial collector wells, respectively. Collector wells located along East, West, and Leadbetter Beach will be 25 to 30-ft deep (assuming collection in the Upper Sand aquifer unit)[28] with a 20-ft diameter caisson. Each caisson will contain 5 horizontal, 12-inch diameter laterals that are 150-ft long, extending in a direction radiating toward the ocean. Individual collector wells are estimated to yield approximately 375 gpm. Based upon this yield, the 43 collector wells required to produce 10,000 AFY of desalinated water. Assuming a well spacing of 600 feet to minimize interference and loss of production, the total number of collector wells that can be constructed on the available beach is 15, with a total production capacity of 5,600 gpm. Collector wells are assumed to be constructed back from the shoreline (south side of Cabrillo Boulevard), approximately 150 feet from the mean high tide line. [29]

Individual collector wells will be equipped with pit mounted vertical turbine pumps. The discharge from each collector well would be connected to a common pipeline and directed the desalination facility. Electrical service and associated facilities will be located at each collector well. Estimated height of the pump station structure is 12 to 15-feet and will require access for cranes to remove the vertical turbine pumps for periodic maintenance. Significant considerations regarding the conceptual design for radial collector wells are summarized below:

- Radial collector wells will impact shallow inland groundwater. It is estimated that approximately 30 to 40 percent of total flow produced will be obtained from inland sources.
- Radial collector well operation will impact sensitive habitats and potentially be impacted by contamination in the area.
- Radial collector well facilities would be susceptible to inundation and erosion as a result of tsunami and would also be increasingly impacted by seawater rise over the 20 year project life. Electrical buildings and wet wells will need to be constructed in a manner that provides flood protection but they would result in a negative visual impact to a culturally sensitive park area.
- As shown in the attached hydrogeological analysis (Appendix B), construction does not require crossing any known faults.

[28] Radial collector wells constructed in the Upper Sand aquifer unit have a higher production capacity. Constructing 15 collector wells in the Lower Sand aquifer unit would result in a yield of 275 gpm per facility, with a total production of 4,125 gpm. Target depth is -60 to -120 ft.
[29] Setback of 150 feet to minimize impacts from tsunami erosion and sea level rise. Location also avoids installation of equipment that requires routine maintenance in recreational sandy beach areas.

Table 3.8 Conceptual Design: Radial Collector Wells		
Description	**Units**	**Criteria**
Number of Wells[1]	No.	15
Well Structure	-	Flood Protected CMU Building
Well Dimensions (Diameter x H)[2]	feet	20 x 15
Capacity, each	gpm	375
Capacity, total	gpm	5,600
Depth	feet	-25 to -30
Number of Radial Screens	No.	5
Radial Screen Length	feet	150
Radial Screen Diameter	inches	12
Caisson Diameter	feet	20
Well Spacing[3,4]	feet	600
Set-back from mean high tide	feet	150
Number of Pumps, per well	No.	3
Pump Type	-	Pit Mounted Vertical Turbine

Notes:

(1) Conceptual design based on targeting the upper sand layer.
(2) Height dimension represents height above grade.
(3) Spacing based on assumed voltage drop for pump motors with a VFD.
(4) Electrical cable run includes spacing between wells and 100 feet of distance into the well.

Figure 3.20 - Radial Collector Wells Conceptual Layout

carollo

3.4.4 Slant Wells

Table 3.9 and Figure 3.21 present the conceptual design summary and layout for slant wells, respectively. Slant wells located along East, West, and Leadbetter Beach will have a depth of 60 to 120 ft, installed at a 22 degree angle with 175-ft of stainless steel screen. Two to three slant wells will be grouped into common facilities where the slant well casings will intercept a wet well/pump station. Each slant well will have a submersible pump that discharges into the wet well. The wet well will have a pump house over the top with pit mounted vertical turbine pumps that transfer the water to the desalination plant. Electrical service and associated facilities will also be local to each slant well pumping station.

Individual slant wells are estimated to yield 275 to 1,000 gpm. Assuming a well spacing of 560 feet to minimize interference and loss of production, the total number of single slant well pumping stations can be constructed on the available beach is between 8 to 16 (depending on soil properties) with an estimated production rate of 4,400 to 8,000 gpm. Slant wells are assumed to be constructed back from the shoreline (south side of Cabrillo Boulevard), approximately 150 feet from the mean high tide line. [30] Estimated height of the pump station structure is 12 to 15-feet and will require access for cranes to remove the vertical turbine pumps for periodic maintenance.

Significant considerations regarding the conceptual design for slant wells are summarized below:

- Slant wells will impact shallow inland groundwater. It is estimated that approximately five percent of total flow produced will be obtained from inland sources.
- Slant well operation will impact sensitive habitats and potentially be impacted by contamination in the area.
- Beach facilities would be susceptible to inundation and erosion as a result of tsunami and would also be increasingly impacted by seawater rise over the 20 year project life. Pump stations wet wells and electrical facilities will need to be constructed in a manner that provides flood protection but the buildings would result in a negative visual impact to a culturally sensitive park area.
- As shown in the attached hydrogeological analysis (Appendix B), construction does not require crossing any known faults.

[30] Setback of 150 feet to minimize impacts from tsunami erosion and sea level rise. Location also avoids installation of equipment that requires routine maintenance in recreational sandy beach areas.

Table 3.9 Conceptual Design: Slant Wells		
Description	**Units**	**Criteria**
Number of Wells[1]	No.	16
Number of Slant Well Sites	No.	8
Slant Well Site Structure	-	Flood Protected CMU Building
Slant Well Site Dimensions (L x W x H)[2]	feet	24 x 30 x 15
Capacity, each	gpm	275
Capacity, total	gpm	4,400[3]
Well Diameter	inches	18
Well Depth[4]	feet	-60 to -120
Screen Length	feet	175
Slant Well Angle	degrees	22
Well Spacing	feet	650
Set-back from mean high tide	feet	150
Number of Pumps, per well	No.	1
Pump Type	-	Submersible

Notes:

(1) Assuming slant well layout of 8 sites with 2 slant wells per site. Slant wells at each site are angled to minimize interference between wells.
(2) Height dimension represents height above grade.
(3) Assuming lower value of range of hydraulic conductivity.
(4) Vertical depth.

Figure 3.21 - Slant Wells Conceptual Layout

carollo

3.4.5 Subsurface Infiltration Galleries (SIGs)

The total infiltration area required to achieve a target yield of 15,898 gpm (10,000 AFY of desalinated water) is seven acres, assuming a design infiltration velocity of 0.05 gpm/ft^2. Construction of a SIG would require excavation to a depth of 15 feet. Engineered filter media and collection piping would consist of three distinctive layers. However, when considering the basis of design requirements presented in prior sections, it is apparent that the construction of a SIG at any of the available project sites is infeasible, due to the following reasons.

- East Beach:
  - Construction of a SIG must be placed offshore at a depth where a stable ocean bottom is achieved (i.e., stable sediment transport conditions - a.k.a., "closure depth"), which is located at -51.5 ft MSL. As presented in Figure 3.22, this depth does not occur until a distance of 8,500 ft from the shoreline. This distance is well beyond the 1/2 mile (i.e., 2,640 ft) area offshore that was established in the Work Plan as a basis for this study.
  - As shown in the attached hydrogeological analysis (Appendix B), construction of a SIG would require crossing the possible offshore fault referenced in the report.
- West Beach:
  - Construction of a SIG would be problematic as construction activities would interfere with harbor use and navigation.
  - As shown in the attached hydrogeological analysis (Appendix B), construction of a SIG has the potential of crossing portions of the Rincon Creek Fault.
- Leadbetter Beach:
  - Construction of a SIG must be placed offshore at a depth where stable ocean bottom is achieved (i.e., "closure depth"), which is located at -49.2 ft MSL. Referring to Figure 3.23 below, this closure depth is attained at a distance of 6,750 ft from the shoreline. This distance is well beyond the 1/2 mile area offshore that was established in the work plan as a basis for this study.
  - As shown in the attached hydrogeological analysis (Appendix B), construction of a SIG would require crossing the Rincon Creek Fault.

Although the SIG does not affect shallow groundwater, sensitive habitats, or contaminant mobilization, a conceptual design was not developed because implementation of this technology did not meet the basis of design requirements established in this technical memorandum.

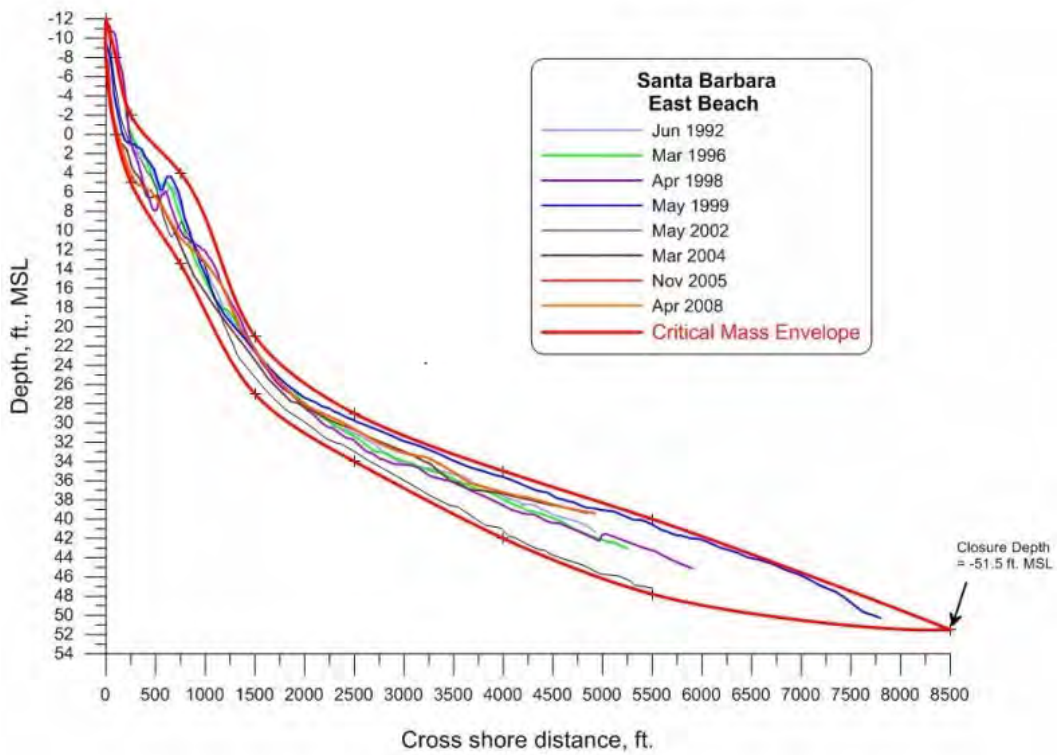

Figure 3.22 East Beach Ocean Bottom Profile

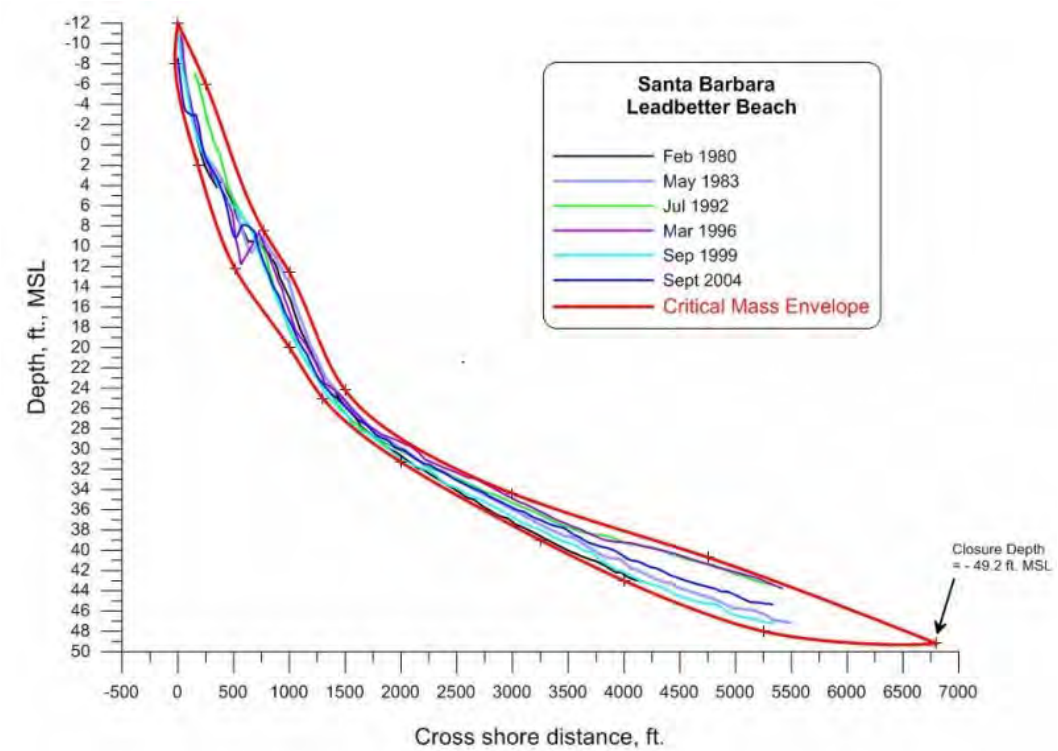

Figure 3.23 Leadbetter Beach Ocean Bottom Profile

2.4.6 Horizontal Directionally Drilled (HDD) Wells

Table 3.10 and Figure 3.24 present the conceptual design summary and layout for HDD wells, respectively. Individual HDD wells are estimated to yield 1,500 gpm and the total number required to produce 15,898 gpm is estimated to be 11, which can be constructed on the available beach. Sediment transport issues preclude this technology at West Beach, but the remaining beaches are candidate sites for this intake technology. East beach was the only beach considered for HDD well conceptual design purposes because of the close proximity and potential to repurpose the existing open ocean intake piping.

It may be possible to cluster all of the individual HDD wells in a splayed pattern at a single location feeding back to a common wet well and pumping station. Therefore, all 11 HDD wells required can be constructed from one location. HDD wells are assumed to be constructed back from the shoreline (south side of Cabrillo Boulevard), approximately 150 feet from the mean high tide line. [31] HDD wells are constructed of perforated high density polyethylene (HDPE) pipes in a cluster configuration and extend at depths of 10 to 30 feet below the ocean bottom. Each HDD well is 18 inches in diameter and 1,500 feet long with 500 feet of HDPE casing and 1,000 feet of perforated HDPE drain pipe. Each HDD wet well would be equipped with pit mounted vertical turbine pumps and connected to a common pipeline that conveys intake water to the City's desalination facility. Estimated height of the wet well/pump house with electrical facilities is 12 to 15 feet.

Significant considerations regarding the conceptual design for HDD wells are summarized below:

- Sediment transport issues at West Beach make the implementation of HDD wells infeasible. Both West Beach and Leadbetter beach can support HDD wells.
- HDD wells do not impact shallow inland groundwater.
- HDD well operation will not impact sensitive habitats or mobilize contamination in the area.
- Beach facilities would be susceptible to inundation and erosion as a result of tsunami and would also be increasingly impacted by seawater rise over the 20 year project life. Electrical buildings and wet wells will need to be constructed in a manner that provides flood protection but this would result in a negative visual impact to a culturally sensitive park area.

[31] Setback of 150 feet to minimize impacts from tsunami erosion and sea level rise. Location also avoids installation of equipment that requires routine maintenance in recreational sandy beach areas.

- As shown in the attached hydrogeological analysis (Appendix B), construction does not require crossing any known faults. At East Beach, the possible offshore fault is approximately 2,000 feet offshore, which will not impact the HDD drains which extend 1,500 feet offshore.

- HDD wells are an unproven intake technology with limited full-scale installations; the technology has not been widely accepted because of various performance challenges.

Table 3.10 Conceptual Design: HDD

Description	Units	Criteria
Number of Drains	No.	11
Capacity, each	gpm	1,500
Capacity, total	gpm	15,898
Well Length	feet	1,500
Well Diameter	inches	18
Length of Drain/Screen, each	feet	1,000
Length of Drain/Screen, total	feet	11,000
Drain Cover, minimum[1]	feet	10
Pump Station Building Type	-	Flood Protected CMU Building
Pump Station Building Dimensions (L x W x H)[2]	feet	24 x 24 x 15
Pump Type	-	Pit Mounted Vertical Turbine
Set-back from mean high tide	feet	150
Location	-	East Beach

Note:
(1) Minimum ocean bottom cover over HDD well.
(2) Height dimension represents height above grade.

Figure 3.24 - HDD Wells Conceptual Layout

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3.5 Initial Screening Analysis

This section presents results from an initial screening analysis performed to assess the technical feasibility for each SSI alternative. As described in the Work Plan, technical feasibility criteria were defined based upon the 2012 California Environmental Quality Act (CEQA) Statute and Guidelines and the California Ocean Plan Amendments that were adopted by the State Water Resources Control Board on May 6, 2015. The Ocean Plan Amendments identify 13 factors that should be used to determine feasibility for subsurface intakes:

1. Geotechnical data
2. Hydrogeology
3. Benthic topography
4. Oceanographic conditions
5. Presence of sensitive habitats
6. Presence of sensitive species
7. Energy use
8. Impact on freshwater aquifers, local water supply and existing water users
9. Desalinated water conveyance
10. Existing infrastructure
11. Design constraints (engineering constructability)
12. Project life cycle costs
13. Other site and facility-specific factors

For the purposes of this study, these factors were characterized by the four main aspects of the CEQA definition of "feasible": i.e., economic, environmental, social, and technological factors. Therefore, initial screening consists of only those factors identified in the Ocean Plan Amendments considered "technological" feasibility criteria. Design information for each SSI alternative presented in Sections 3.2, 3.3, and 3.4 were developed considering this required information for technical feasibility screening (i.e., initial screening).

3.5.1 Initial Screening Criteria

The technical factors used for initial screening are a starting point to determine if an SSI alternative should be considered for further evaluation - e.g., before economic, environmental, and social factors are considered. Intake alternatives that were judged to have technical feasibility criteria in conflict with the project objectives (i.e., defined in the Work Plan) fail initial screening and will not considered further in this study. For alternatives that passed initial screening, each SSI alternative will also evaluated for feasibility based upon the economic, environmental, social, and technological factors.

For the purposes of this study, "Initial Screening Criteria" was defined as follows:

> *Initial Screening Criteria: Those technical factors that would not allow a full-scale system to be successfully constructed or operated, would result in a high risk of failure or unacceptable performance, or would not produce water supply required to replace the use of the desalination plant's screened open ocean intake per Study goals.*

Table 3.11 presents initial screening criteria that were used in this study. Initial screening criteria were analyzed concurrent to the design basis development presented in Section 3.2 through 3.4 to avoid carrying forward alternatives for further study that are not technically feasible.

Table 3.11 Initial Screening Criteria

Screening Criteria	Failure to meet criteria
Geotechnical Hazards	
Seismic hazard	Project facilities would cross a known fault line, or be exposed to a seismic hazard that could otherwise not be protected from loss by design
Hydrogeologic Factors	
Operation of subsurface intake adversely impacts existing fresh water aquifers, local water supplies, or existing water users.	• Volume of groundwater in storage is reduced due to subsurface intake pumping, impacting drought supply and requiring additional desalination to make up for loss of groundwater. • Operation of subsurface intake causes salt water intrusion into groundwater aquifers.
Operation of subsurface intake adversely impacts sensitive habitats such as marshlands, drainage areas, etc.	Operation of subsurface intake drains surface water from sensitive habitat areas or adversely changes water quality.
Insufficient length of beach available for replacing full yield derived from the existing open ocean intake.	Small individual facility yield, large number of facilities required, and minimum spacing between facilities requires more shoreline than is available.
Benthic Topography	
Land type makes intake construction infeasible	Depth to bedrock too shallow (i.e., less than 40-feet deep); rocky coastline; cliffs
Oceanographic Factors	
Erosion, sediment deposition, sea level rise or tsunami hazards	Oceanographic hazards make aspects of the project infrastructure vulnerable in a way that cannot be protected and/or would prevent the City from being able to receive funding or insurance for this concept
Presence of Sensitive Habitats	
Proximity to marine protected areas	Location would require construction within a marine protected area

Table 3.11 Initial Screening Criteria	
Screening Criteria	**Failure to meet criteria**
Design and Construction Constraints	
Adequate capacity	Subsurface material lacks adequate transmissivity to meet target yield of at least 15,898 gpm (i.e., build-out intake capacity necessary to produce 10,000 AFY).
Lack of adequate linear beach front for technical feasibility	Length of beachfront available is not sufficient for construction of the required number of wells of all or portion of intake to meet target yield
Lack of adequate land for required on-shore facilities	• Surface area needed for on-shore footprint of an intake unit is greater than the available onshore area • Requires condemnation of property for new on-shore intake pumping facilities
Lacking adequate land for on-shore construction staging	The amount of land available to stage construction does not meet need
Precedent for subsurface intake technology	Intake technology has not been used before in a similar seawater or fresh water application at a similar scale.

Initial screening renders each SSI alternative to be categorized as:

1. Not feasible (NF),
2. Potentially feasible, but does not meet study goals (PF*), or
3. Potentially feasible (PF).

3.5.2 Initial Screening Results

As indicated by the results of the initial screening criteria are summarized in Table 3.12, none of the SSI alternatives considered in this study were determined to be potentially feasible based upon the study objectives. These findings are the result of the basis of design, hydrogeological analysis, and conceptual design information presented previously in Sections 3.2, 3.3 and 3.4. Where an SSI alternative was determined to be "not feasible" based upon this study's initial screening criteria, this "not feasible" finding is explained further in Tables 3.13 through 3.18, below. Discussion is grouped by the project technology and the initial screening criteria presented in Table 3.11.

Table 3.12 Subsurface Desalination Intake Initial Screening Results

		Subsurface Intake Alternative					
	Initial Screening Criteria	**Vertical Beach Wells**	**Onshore Infiltration Gallery**	**Radial Collector Wells**	**Slant Wells**	**Subsurface Infiltration Galleries**	**HDD Wells**
Geotechnical Hazards							
1	**Seismic Hazard**						
a.	Project facilities would cross a known fault line, or be exposed to a seismic hazard that could otherwise not be protected from loss by design	**PF**	**PF**	**PF**	**PF**	**NF**	**PF**
Hydrogeologic Factors							
2	**Impact on existing freshwater aquifers, local water supplies, or existing water users**						
a.	Volume of groundwater in storage is reduced due to subsurface intake pumping, impacting drought supply and requiring additional desalination to make up for loss of groundwater.	**PF**	**PF**	**PF**	**PF**	**PF**	**PF**
b.	Operation of subsurface intake causes salt water intrusion into groundwater aquifers.	**PF**	**PF**	**PF**	**PF**	**PF**	**PF**
3	**Impact to sensitive habitats such as marshlands, drainage areas, etc.**						
a.	Operation of subsurface intake drains surface water from sensitive habitat areas or adversely changes water quality.	**NF**	**NF**	**NF**	**NF**	**PF**	**PF**
4	**Insufficient length of beach available for replacing full yield derived from existing open ocean intake.**						
a.	Small individual facility yield, large number of facilities required, and minimum spacing between facilities requires more shoreline than is available.	**PF***	**PF***	**PF***	**PF***	**PF**	**PF**
Benthic Topography							
5	**Land type makes intake construction infeasible.**						
a.	Depth to bedrock too shallow (i.e., less than 40-feet deep); rocky coastline; cliffs	**PF**	**PF**	**PF**	**PF**	**PF**	**PF**
Oceanographic Factors							
6	**Erosion, sediment deposition, sea level rise, or tsunami hazards.**						
a.	Oceanographic hazards make aspects of the project infrastructure vulnerable in a way that cannot be protected and/or would prevent the City from being able to receive funding or insurance for this concept.	**PF**	**PF** (4)	**PF**	**PF**	**NF**	**PF**

Table 3.12 Subsurface Desalination Intake Initial Screening Results

		Subsurface Intake Alternative					
Initial Screening Criteria		**Vertical Beach Wells**	**Onshore Infiltration Gallery**	**Radial Collector Wells**	**Slant Wells**	**Subsurface Infiltration Galleries**	**HDD Wells**
Presence of Sensitive Habitats							
7	**Proximity to marine protected areas**						
a.	Location would require construction within a marine protected area.	**PF**	**PF**	**PF**	**PF**	**PF**	**PF**
Design and Construction Constraints							
8	**Adequate capacity**						
a.	Subsurface material lacks adequate transmissivity to meet target yield of at least 15,898 gpm (i.e., build-out intake capacity necessary to produce 10,000 AFY).	**NF**	**NF**	**NF**	**NF**	**PF**	**PF**
9	**Lack of adequate linear beach front for technical feasibility**						
a.	Length of beachfront available is not sufficient for construction of the required number of wells of all or portion of intake to meet target yield.	**NF**	**NF**	**NF**	**NF**	**PF**	**PF**
10	**Lack of adequate land for required on-shore facilities**						
a.	Surface area needed for on-shore footprint (i.e., pump house) of an intake unit is greater than the available onshore area.	**PF**	**PF**	**PF**	**PF**	**PF**	**PF**
b.	Requires condemnation of property for new on-shore intake pumping facilities.	**PF**	**PF**	**PF**	**PF**	**PF**	**PF**
11	**Lack of adequate land for required on-shore construction staging**						
a.	The amount of land available to stage construction does not meet need.	**PF**	**PF**	**PF**	**PF**	**PF**	**PF**
12	**Precedent for subsurface intake technology**						
a.	Intake technology has not been used before in a similar seawater or fresh water application at a similar scale.	**PF**	**PF**	**PF**	**PF**	**PF**	**NF**
Passes Initial Screening? Yes (Y) or No (N)		**N**	**N**	**N**	**N**	**N**	**N**

Notes:
(1) NF = Not Feasible
(2) PF = Potentially Feasible
(3) PF* = Potentially Feasible, but does not meet current study goals
(4) Potentially feasible at Leadbetter and West Beach only. Sediment transport conditions at East Beach make the implementation of an onshore infiltration gallery infeasible (refer to Section 3.4.2).

Vertical Beach Wells

Information supporting the determination of vertical wells to be “not feasible” is presented in Table 3.13, below.

Table 3.13 Initial Screening Supporting Information: Vertical Wells

No. [1]	Description	Discussion [2]
3	Impact to Sensitive Habitats	• As presented in Section 3.3 and 3.4, vertical wells will cause 1 to 3 feet of drawdown in sensitive habitat areas. Results are supported by numeric modeling presented in Appendix B. [3]
8	Adequate Capacity	• Refer to Table 3.4. Even when assuming the most optimistic hydrogeologic parameters, vertical wells can only supply 4,800 gpm, well below the 15,898 gpm target yield. • Even at this low yield, the operation of vertical wells will impact wetlands, suggesting an even lower production rate is required.
9	Lack of Beach Front	• Refer to Table 3.3. To achieve the capacity required to meet this study's objectives, 5.5 to 18 miles of beachfront are required. As presented in Section 3.4, only 1.7 miles of beach front is available for SSI development.

Notes:

(1) Corresponds to initial screening criteria number listed in Table 3.12.
(2) Definitions of initial screening criteria are present in Table 3.11.
(3) As discussed in Section 3.3.3, further reduction to feasible yield will be required to potentially reduce impacts to sensitive habitat areas. To evaluate this possibility, a de minimis impact to the sensitive areas needs to be established.

Onshore Infiltration Galleries

Information supporting the determination of onshore infiltration galleries to be “not feasible” is presented in Table 3.14, below.

Table 3.14 Initial Screening Supporting Information: Onshore Infiltration Galleries

No. [1]	Description	Discussion [2]
3	Impact to Sensitive Habitats	• As presented in Section 3.3 and 3.4, onshore infiltration galleries have the most significant impact to sensitive habitat areas (i.e., can result in up to 4 ft of drawdown). Results are supported by numeric modeling presented in Appendix B. [3]
8	Adequate Capacity	• Refer to Table 3.4. Onshore infiltration galleries can supply 10,100 gpm, which is less than the 15,898 gpm yield required for this study. [4]
9	Lack of Beach Front	• Refer to Table 3.3. To achieve the capacity required to meet this study's objectives, 3 miles of beachfront are required. As presented in Section 3.4, only 1.7 miles of beach front is available for SSI development. [5]

Notes:

(1) Corresponds to initial screening criteria number listed in Table 3.12.
(2) Definitions of initial screening criteria are present in Table 3.11.
(3) As discussed in Section 3.3.3, further reduction to feasible yield will be required to potentially reduce impacts to sensitive habitat areas. To evaluate this possibility, a de minimis impact to the sensitive areas needs to be established.
(4) Assumes construction at all three beaches. In Section 3.4, conceptual design proved construction at East Beach was not feasible due to sediment transport conditions. Actual intake total capacity is therefore, reduced to only 5,000 gpm.
(5) Excluding East Beach, which was determined to be an infeasible site for onshore infiltration galleries due to sediment transport conditions, only 0.7 miles of beachfront is available.

Radial Collector Wells

Information supporting the determination of radial collector wells to be “not feasible” is presented in Table 3.15, below.

Table 3.15 Initial Screening Supporting Information: Radial Collector Wells

No. [1]	Description	Discussion [2]
3	Impact to Sensitive Habitats	• As presented in Section 3.3 and 3.4, radial collector wells will cause 0.5 to 3 ft of drawdown in sensitive habitat areas. Results are supported by numeric modeling presented in Appendix B. [3]
8	Adequate Capacity	• Refer to Table 3.4. Even when assuming the most optimistic hydrogeologic parameters, radial collector wells can only supply 7,000 gpm, which is less than the 15,898 gpm yield required for this study.
9	Lack of Beach Front	• Refer to Table 3.3. To achieve the capacity required to meet this study's objectives, 4 to 6 miles of beachfront are required. As presented in Section 3.4, only 1.7 miles of beach front is available for SSI development.

Notes:

(1) Corresponds to initial screening criteria number listed in Table 3.12.
(2) Definitions of initial screening criteria are present in Table 3.11.
(3) As discussed in Section 3.3.3, further reduction to feasible yield will be required to potentially reduce impacts to sensitive habitat areas. To evaluate this possibility, a de minimis impact to the sensitive areas needs to be established.

Slant Wells

Information supporting the determination of slant wells to be “not feasible” is presented in Table 3.16, below.

Table 3.16 Initial Screening Supporting Information: Slant Wells

No. [1]	Description	Discussion [2]
3	Impact to Sensitive Habitats	As presented in Section 3.3 and 3.4, slant wells will cause 1 to 3 ft of drawdown in sensitive habitat areas. Results are supported by numeric modeling presented in Appendix B. [3]
8	Adequate Capacity	Refer to Table 3.4. Even when assuming the most optimistic hydrogeologic parameters, slant wells can only supply 8,000 gpm, which is less than the 15,898 gpm yield required for this study.
9	Lack of Beach Front	Refer to Table 3.3. To achieve the capacity required to meet this study's objectives, 3.5 to 6 miles of beachfront are required. As presented in Section 3.4, only 1.7 miles of beach front is available for SSI development.

Notes:

(1) Corresponds to initial screening criteria number listed in Table 3.12.
(2) Definitions of initial screening criteria are present in Table 3.11.
(3) As discussed in Section 3.3.3, further reduction to feasible yield will be required to potentially reduce impacts to sensitive habitat areas. To evaluate this possibility, a de minimis impact to the sensitive areas needs to be established.

Subsurface Infiltration Galleries

Information supporting the determination of a SIG to be “not feasible” is presented in Table 3.17.

Table 3.17 Initial Screening Supporting Information: SIG		
No. [1]	**Description**	**Discussion [2]**
1	Seismic Hazard	As presented in Section 3.4.5, a SIG constructed at West or Leadbetter Beach would require crossing portions of the Rincon Creek Fault. Construction of a SIG at East Beach would require crossing the possible offshore fault referred to in this TM and in various other reports. Results are supported by the hydrogeologic analysis presented in Appendix B.
6	Oceanographic Factors	Refer to Section 3.4.5. At East and Leadbetter Beach, the depth where a stable ocean bottom exists does not occur within the 1/2 mile area offshore that was established as a basis for the study. At West Beach, construction of a SIG is problematic because construction activities will interfere with harbor use and navigation. Results are supported by the coastal hazards and sediment transport study presented in Appendix C.

Notes:

(1) Corresponds to initial screening criteria number listed in Table 3.12.
(2) Definitions of initial screening criteria are present in Table 3.11.

HDD Wells

Information supporting the determination of HDD wells to be “not feasible” is presented in Table 3.18.

Table 3.18 Initial Screening Supporting Information: HDD Wells		
No. [1]	**Description**	**Discussion [2]**
12	Precedent for Technology	As presented in Section 3.2.3, HDD wells have been used as an intake technology for only 10 years with no applications in California or even the U.S. This "lack of precedent" determination is supported by the ISTAP that was convened by the California Coastal Commission to evaluate the feasibility of subsurface intake alternatives for the Huntington Beach Project, referenced in Section 3.6.

Notes:
(1) Corresponds to initial screening criteria number listed in Table 3.12.
(2) Definitions of initial screening criteria are present in Table 3.11.

3.6 Conclusions and Recommendations

This section presents the basis of design, conceptual design, and initial screening analysis for each of the six SSI alternatives considered in this study. As defined in the Work Plan, only those alternatives that are determined to be technically feasible through initial screening analysis shall be subjected to a further feasibility analysis that also considers social, environmental, and economic factors. None of the SSI alternatives met study goals and survived initial screening analysis.

Out of the alternatives considered in this study, HDD wells passed all initial screening criteria except for one – HDD wells lack of a precedent for SSI technology. As discussed in Section 3.2.3, HDD wells have been used as an intake technology for only 10 years, with very limited experience (i.e., no experience in California or the United States) and high variability in performance. The basis for the "not feasible" conclusion presented in this study is supported by the findings of the Independent Scientific Technical Advisory Panel (ISTAP) that was assembled by the California Coastal Commission to evaluate technical feasibility of SSI designs for the proposed water desalination facility at Huntington Beach, California.[6] The ISTAP report for Huntington Beach found that:

> “There is inadequate data on the long-term reliability and maintainability of the HDD wells/drains. This subsurface intake design option is considered to be technically infeasible at the Huntington Beach site because of a high performance risk. There is too great uncertainty that a system could be constructed that would reliably provide the water volume over the operational life of the desalination facility.”

Also, as presented in Section 2.3, global experience further supports this as demonstrated by the HDD installation at the San Pedro de Pinatar plant located in Almeria, Spain where the HDD wells have lost significant capacity and produced poor water quality. This plant was expanded using a screened open ocean intake to address reliability issues associated with the HDD well intake.

APPENDIX A – SSI STUDY WORK PLAN

CITY OF SANTA BARBARA

SUBSURFACE DESALINATION INTAKE AND POTABLE REUSE FEASIBILITY STUDIES

WORK PLAN
SUBSURFACE DESALINATION INTAKE

FINAL
August 2015

3150 BRISTOL STREET, SUITE 500 • COSTA MESA, CALIFORNIA 92626 • P. 714.593.5100 • F. 714.593.5101

City of Santa Barbara

Subsurface Desalination Intake and Potable Reuse Feasibility Studies

WORK PLAN
Subsurface Desalination Intake

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1.0 INTRODUCTION

The purpose of this document is to present a Work Plan that will be followed to evaluate the feasibility of using a subsurface intake to supply seawater to the City of Santa Barbara's (City) Charles Meyer Desalination Plant (Desal Plant), in order to replace the use of a screened open ocean intake.

1.1 Background

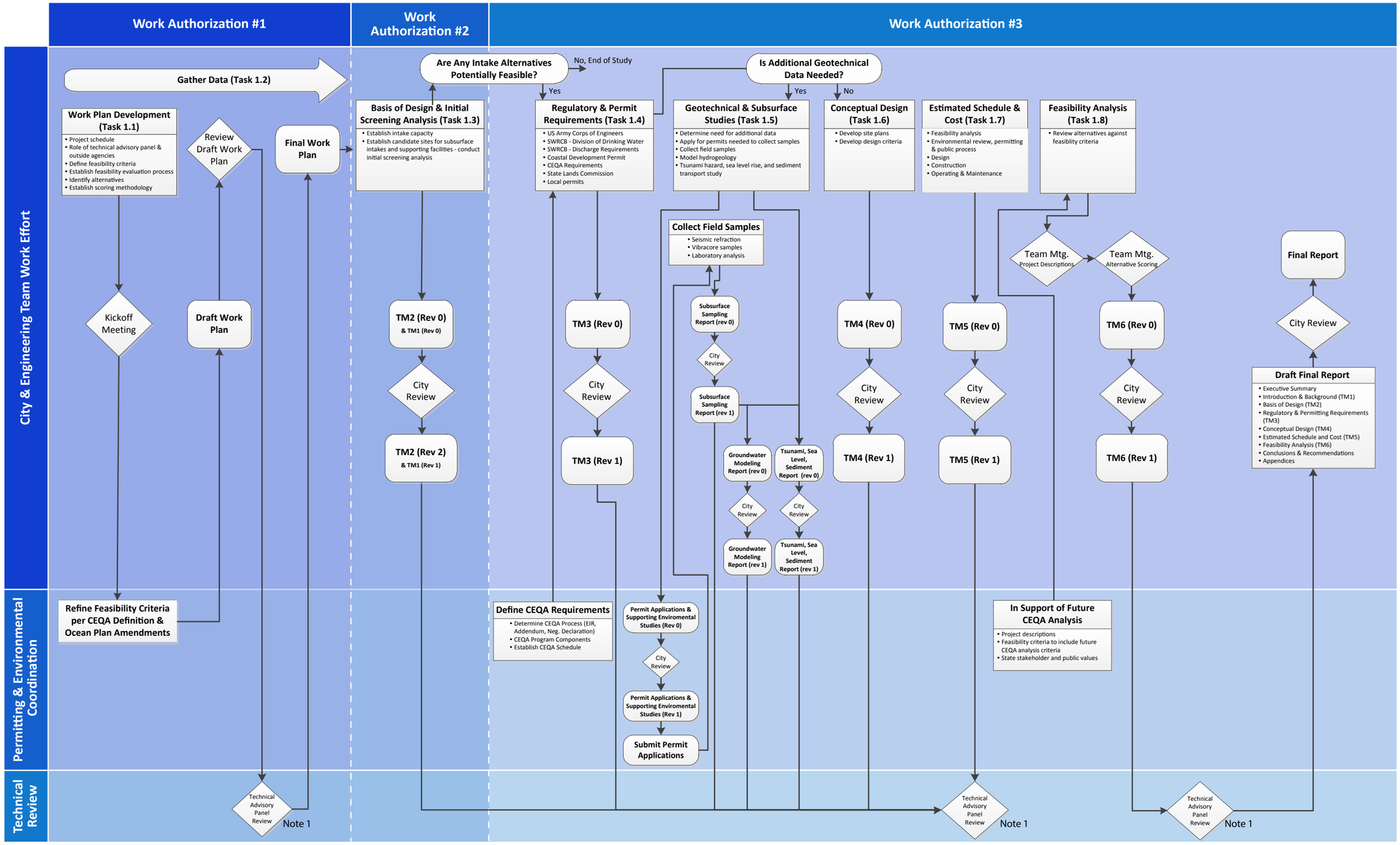
On September 23, 2014 the City of Santa Barbara City Council directed Public Works Department staff to report back with a plan to evaluate the feasibility of subsurface desalination intakes (subsurface intake) and potable reuse, including indirect and direct potable reuse options. The direction given by City Council was to report back with a plan for this evaluation following award of the desalination plant contract in April 2015. Furthermore, on January 30, 2015, the Central Coast Regional Water Quality Control Board (RWQCB) adopted an amendment to the City's El Estero Wastewater Treatment Plant (WWTP) Waste Discharge Requirements (WDR) that included a condition that the City should report back to the RWQCB by August of 2015 with a Work Plan that will result in completed feasibility studies by June 2017.

The City subsequently retained the services of Carollo Engineers, Inc. (Carollo) to complete these studies. Carollo will deliver the work for these feasibility studies under three work authorizations:

- **Work Authorization 1:** The Work Plans for both the subsurface intake and potable reuse studies. (**Note:** *The subsurface intake Work Plan is the subject of this document*)
- **Work Authorization 2:** Subsurface intake initial screening analysis and potable reuse feasibility study.
- **Work Authorization 3:** Subsurface intake feasibility study.

Each subsequent work authorization would be performed at the direction of City Council in accordance with the RWQCB's requirement.

A programmatic workflow diagram for all three authorizations for the subsurface intake study is presented in Figure 1. A copy of the fully executed scope of work for Work Authorization 1 is presented in Appendix A.



Notes:

1. It is envisioned that the technical advisory process includes a public meeting where stakeholders will be given a chance to state their interests in the City's study effort and comment upon the direction of the City's work product.

Figure 1 - Subsurface Desalination Intake Feasibility Study Programmatic Work Plan

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1.2 Objectives

The City is required to submit a Work Plan for evaluating subsurface intakes to the RWQCB by August 2015. The overall objective of this Work Plan is to present the methodology and procedures that will be used to perform the subsurface intake feasibility study. Objectives of this Work Plan include:

1. Establish the project schedule.
2. Establish the methods by which the design basis will be established. Design basis includes intake capacity and site alternative evaluation.
3. Establish the types of subsurface intakes that will be studied.
4. Establish procedure to identify sites for subsurface intakes and raw water conveyance piping
5. Establish a procedure to determine subsurface properties
6. Establish procedure to model subsurface intake's influence on the sustainability of the City's drinking water aquifer (capacity and water quality).
7. Establish procedure to estimate subsurface intake water quality and any additional treatment needs.
8. Establish the scope of cost estimates and cost estimating procedures.
9. Establish and define feasibility screening criteria.
10. Establish and define initial screening criteria that may limit further consideration of project alternatives.
11. Establish technical advisory panel role, procedures, and objectives.
12. Establish the role of outside agencies (e.g., RWQCB, California Coastal Commission, etc.) and City residents.

1.3 Scope

The City Council meeting minutes from September 23, 2014, Agenda Item 16: *Authorize Actions and Adopt a Resolution for Reactivating the Charles E. Meyer Desalination Facility*, state that there was an additional motion "to direct staff to return to the City Council after the [Desalination Plant Reactivation] contract decision is made in April [2015] to begin exploring a range of alternatives, including subsurface intake and potable reuse options." Relative to determine City Council's intent as to the scope of this study, the verbal transcript of the meeting was examined. In review of this transcript, the verbal intent was to "direct staff…[to evaluate the] feasibility, cost, and timeline associated with both converting the offshore facility to a subsurface intake and look at the options about potable reuse".[1]

[1] Mayor Schneider, as documented on September 23, 2014 City Council Meeting video recording (available on the City's website): http://media-07.granicus.com:443/OnDemand/santabarbara/santabarbara_d2343df5-8a20-499d-b1fb-5dda1f9e0414.mp4 at 2 hours and 33 minutes.

This motion was further adopted by the Central Coast RWQCB, who on January 30, 2015 amended the City's NPDES Permit (AMENDED ORDER NO. R3-2010-0011, NPDES NO. CA0048143) and in Section VI Paragraph C.6.c.iii (Special Provisions, Desalination Facility) adopted a provision to require the City to "Analyze the feasibility of a range of alternatives, including subsurface intake and potable reuse options."

Therefore, the direction given by both the City Council and RWQCB, relative to the scope of this study was to evaluate:

1. A replacement of the City's open ocean intake using a subsurface intake.
2. Potable reuse alternatives, also in the context of a replacement of desalination plant's open ocean intake use.

1.4 Work Plan Organization and Sequence of Work

This Work Plan focuses only on the City's subsurface intake feasibility study and is organized into the following sections:

- Introduction
- Basis of Design
- Feasibility and Initial Screening Criteria
- Implementation Schedule Development
- Cost Estimating Methodology
- Feasibility Analysis
- Technical Advisory Process

The City's potable reuse feasibility study is addressed as separate Work Plan.

The programmatic workflow diagram presented in Figure 1 shows the chronology that project work product will be developed and reviewed for each of the three work authorizations. As noted in Figure 1, only potentially feasible alternatives will be evaluated in Work Authorization 3. Initial screening will be performed in Work Authorization 2 and if enough data is available to determine that the alternative does not pass initial screening, no further feasibility analysis will be performed for that intake alternative.

A complete project schedule including the anticipated dates of all project milestones and deliverables is presented in Figure 2.

Subsurface Desalination Intake & Potable Reuse Feasibility Studies

ID	Task Name	Duration	Start	Finish
1	**Notice to Proceed (Work Authorization 1)**	1 day	Fri 5/1/15	Fri 5/1/15
2	Notice to Proceed (Work Authorization 2)	1 day	Tue 9/22/15	Tue 9/22/15
3	Notice to Proceed (Work Authorization 3)	1 day	Mon 11/23/15	Mon 11/23/15
4	**TASK 1 - SUBSURFACE DESALINATION INTAKE FEASIBILITY STUDY**	**526 days**	**Fri 5/1/15**	**Fri 5/5/17**
5	**1.1 Work Plan Development**	**114 days**	**Fri 5/1/15**	**Wed 10/7/15**
6	Kickoff Meeting	1 day	Wed 5/20/15	Wed 5/20/15
7	Draft Work Plan	36 days	Fri 5/1/15	Fri 6/19/15
8	City Review	5 days	Mon 6/22/15	Fri 6/26/15
9	Submit Draft Work Plan to RWQCB	1 day	Mon 6/29/15	Mon 6/29/15
10	RWQCB Approval/Comments	1 day	Tue 7/21/15	Tue 7/21/15
11	Final Work Plan	10 days	Thu 8/13/15	Wed 8/26/15
12	TM1 (Revision 0): Intro, Background & Project Alternatives	15 days	Thu 8/27/15	Wed 9/16/15
13	City Review	5 days	Thu 9/17/15	Wed 9/23/15
14	TM1 (Revision 1): Intro, Background & Project Alternatives	10 days	Thu 9/24/15	Wed 10/7/15
15	**1.2 Literature Review**	**106 days**	**Fri 5/1/15**	**Fri 9/25/15**
16	Collect Literature	99 days	Fri 5/1/15	Wed 9/16/15
17	Data Collection List Updates	**105 days**	**Mon 5/4/15**	**Fri 9/25/15**
18	Weekly Data Collection Updates	16 days	Mon 5/4/15	Mon 5/25/15
23	Monthly Data Collection Updates	66 days	Fri 6/26/15	Fri 9/25/15
28	**1.3 Basis of Design & Initial Screening Analysis**	**55 days**	**Wed 9/23/15**	**Tue 12/8/15**
29	**1.3.1 Prelim. Geophysical Studies (Tsunami, Sea Level Rise & Sediment Transport)**	**35 days**	**Wed 9/23/15**	**Tue 11/10/15**
33	**1.3.2 Basis of Design and Initial Screening**	**35 days**	**Wed 10/21/15**	**Tue 12/8/15**
37	**1.4 Regulatory Requirements**	**30 days**	**Wed 9/23/15**	**Tue 11/3/15**
41	**1.5 - Geotechnical & Subsurface Studies**	**220 days**	**Tue 11/24/15**	**Mon 9/26/16**
42	**1.5.1 Permitting for Subsurface Data Collection/Sampling**	**35 days**	**Tue 11/24/15**	**Mon 1/11/16**
52	**1.5.2 Field Activities for Geotechnical and Subsurface Data Collection/Sampling**	**125 days**	**Tue 3/8/16**	**Mon 8/29/16**
57	**1.5.3 Groundwater Modeling**	**35 days**	**Tue 8/9/16**	**Mon 9/26/16**
61	**1.6 Conceptual Design**	**45 days**	**Tue 9/6/16**	**Mon 11/7/16**
65	**1.7 Estimated Schedule and Cost**	**25 days**	**Tue 10/4/16**	**Mon 11/7/16**
69	**1.8 Feasibility Analysis**	**113 days**	**Tue 11/8/16**	**Thu 4/13/17**
79	**1.9 Subsurface Desalination Intake Feasibility Report**	**35 days**	**Fri 4/14/17**	**Thu 6/1/17**
83	**TASK 2 - NOT USED**	**333 days**	**Tue 11/24/15**	**Thu 3/2/17**
108	**TASK 3 - POTABLE REUSE FEASIBILITY STUDY**	**248 days**	**Fri 5/1/15**	**Tue 4/12/16**
109	**3.1 Work Plan Development**	**109 days**	**Fri 5/1/15**	**Wed 9/30/15**
110	Kickoff Meeting	1 day	Wed 5/20/15	Wed 5/20/15
111	Draft Work Plan	36 days	Fri 5/1/15	Fri 6/19/15
112	City Review	5 days	Mon 6/22/15	Fri 6/26/15
113	Submit Draft Work Plan to RWQCB	1 day	Mon 6/29/15	Mon 6/29/15
114	RWQCB Approval/Comments	1 day	Tue 7/21/15	Tue 7/21/15
115	Final Work Plan	10 days	Thu 8/13/15	Wed 8/26/15
116	TM1 (Revision 0): Intro, Background & Project Alternatives	15 days	Thu 8/27/15	Wed 9/16/15
117	City Review	5 days	Thu 9/17/15	Wed 9/23/15
118	TM1 (Revision 1): Intro, Background & Project Alternatives	5 days	Thu 9/24/15	Wed 9/30/15
119	**3.2 Literature Review**	**106 days**	**Fri 5/1/15**	**Fri 9/25/15**
120	Collect Literature	99 days	Fri 5/1/15	Wed 9/16/15
121	Data Collection List Updates	105 days	Mon 5/4/15	Fri 9/25/15
122	Weekly Data Collection Updates	16 days	Mon 5/4/15	Mon 5/25/15
127	Monthly Data Collection Updates	66 days	Fri 6/26/15	Fri 9/25/15
132	**3.3 Regualtory and Permit Requirements**	**30 days**	**Thu 8/27/15**	**Wed 10/7/15**
136	**3.4 Conceptual Design**	**45 days**	**Thu 8/27/15**	**Wed 10/28/15**
140	**3.5 Estimated Schedule and Cost**	**25 days**	**Thu 9/24/15**	**Wed 10/28/15**
144	**3.6 Feasibility Analysis**	**70 days**	**Thu 12/31/15**	**Wed 4/6/16**
154	**3.7 Potable Reuse Feasibility Report**	**7 days**	**Mon 4/4/16**	**Tue 4/12/16**
158	**TASK 4 - TECHNICAL ADVISORY PROCESS**	**453 days**	**Wed 8/12/15**	**Fri 5/5/17**
159	Workshop 1: Work Plan	1 day	Wed 8/12/15	Wed 8/12/15
160	Workshop 2: Initial Screening	1 day	Wed 12/30/15	Wed 12/30/15
161	Workshop 3: Feasibility Analysis (Potable Reuse Study)	1 day	Fri 5/5/17	Fri 5/5/17
162	Workshop 4: Feasibility Analysis (Subsurface Study)	1 day	Fri 4/1/16	Fri 4/1/16
163	**RWQCB FINAL REPORT DEADLINE**	1 day	Fri 6/2/17	Fri 6/2/17

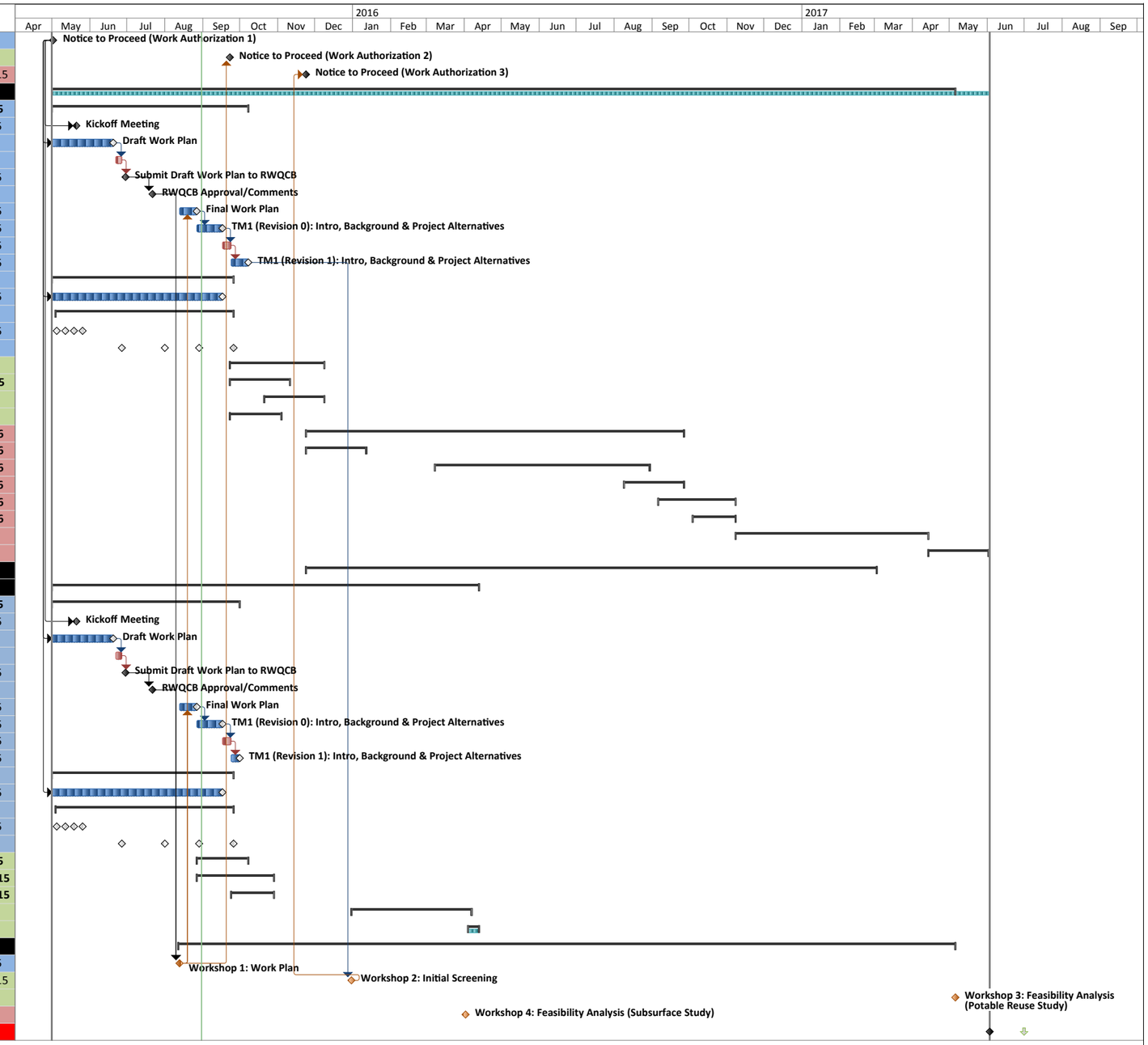

Figure 2 - Project Schedule

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1.5 Goal of Study

The goal of this study is to meet the requirements set forth by City Council and the RWQCB that were described in Section 1.3. However, this study may also inform future studies including future updates to the City's Long Term Water Supply Plan. The City's primary water source is Cachuma Reservoir, which provides over 50 percent of the City's water supply during a normal (non-drought) year. The City's water supply allocation from Cachuma could be reduced in the future due to pending federal environmental decisions on a revised Biological Opinion for the Cachuma Project, reduced operational yield due to siltation in the reservoir, and reduced drought yield as a result of the current historic drought. The City's supply planning will need to be updated to address shortages caused by such reductions to the City's existing Cachuma supply. Options for replacing a reduced Cachuma supply may include desalination and potable reuse.

Because the amount of the reduction from the City's Cachuma supply is unknown at this time, it is premature for the City to evaluate exact desalination and potable reuse capacity options that may or may not meet the City's needs. The timing for this analysis would be more appropriate following the final federal environmental decisions and operational yield analyses that determine the future Cachuma allocations. Therefore, the direction given by City Council and the RWQCB (as presented in Section 1.3) is appropriate at this time because it will determine the maximum capacity that is technically feasible from subsurface intakes and potable reuse without requiring the City to invest in developing many project concepts that may or may not meet the City's future needs pending forthcoming environmental and operational yield decisions.

Thus, the goal of this study is to understand the maximum yield that is technically feasible for subsurface intake alternatives (subject of this Work Plan) and potable reuse alternatives (subject of a separate Work Plan). The maximum yield will provide information on whether the alternatives could replace the open ocean intake independently, and potentially combined. How the City will use of these technically feasible maximum yields needs to be informed by the City's need, which will follow at a later date. Therefore, the information developed in this study will inform future studies, such as an update to the City's Long Term Water Supply Plan.

Feasibility and initial screening criteria are presented in Section 3 of this Work Plan. Alternatives are first subjected to initial screening criteria, which are based on technical feasibility criteria and capacities defined under current project objectives. It is anticipated that alternatives may end up in the following three general categories, defined further as follows:

1. **Infeasible** – The alternative does not pass the initial screening criteria and is not feasible due to technical criteria.

   Action: The alternative shall not be considered further in this study and is not recommended for inclusion in future studies.

2. **Potentially feasible, does not meet current Study goals** – The alternative meets technical screening criteria and is potentially feasible. However, the alternative's capacity does not meet the current Study goals.

   Action: The alternative shall not be considered further in this study but is potentially feasible and may be considered in future studies. Information collected during the screening process is useful to inform future studies.

3. **Potentially feasible** – The alternative passes through the initial screening stage and is considered potentially feasible.

   Action: The alternative shall be considered further in this study under current objectives and is subject to the work sequence laid out in the Work Plan.

2.0 BASIS OF DESIGN

To focus the efforts of this study on only those options that are at least potentially feasible, it is important to establish a clear definition of the basis of design for the subsurface intake alternatives. Raw water production capacity, project site alternatives, intake technology alternatives, subsurface properties, and water quality and treatment needs determine the basis of design for the various subsurface intake alternatives.

As noted previously in the programmatic work flow diagram presented in Figure 1, the basis of design will be established in Work Authorization 2. Once the design basis is established, initial screening criteria can be assessed based upon available information. Where sufficient information is not available, an alternative will be determined "potentially feasible" and the study will recommend the collection of additional data. By screening alternatives in this manner, only potentially feasible alternatives are considered for the feasibility analysis. Therefore, the definitions for basis of design criteria presented in the subsequent subsections of this Work Plan are intended to guide the project's work effort and the initial screening analysis that will be conducted in Work Authorization 2.

2.1 Capacity

As described earlier, the goal of this Study is to understand the maximum yield that is technically feasible for subsurface desalination intake alternatives and potable reuse alternatives, and to evaluate the feasibility of alternatives to replace the City's existing screened open ocean intake. All alternatives will go through technical evaluation to determine the maximum yield achievable. The target yield for each alternative will be based on the City's permitted capacity for screened open ocean intake, which is 10,000 acre-feet per year (AFY) of finished desalinated water supply. Each subsurface intake shall therefore be designed to produce 15,898 gallons per minute (gpm) of seawater to meet such needs. This intake flow rate accounts for a 45 percent RO recovery and the volume of raw water required for backwashing any pretreatment filters when the City's desalination plant is operated to produce 10,000 AFY. Because it is unknown if a subsurface intake can produce the quality of water required to completely eliminate pretreatment, and the City's desalination plant is existing and uses pretreatment filters that require backwash, the volume of intake water required for backwash is included in the intake capacity required. Consistent with the existing facility operation, backwash water is not recycled to reduce intake flow required.

2.2 Project Site Alternatives

Project site alternatives for a subsurface intake shall include the following areas due to their proximity to the City's desalination plant, the proximity to the existing intake line and its existing easement for a railroad crossing, and the availability of prior geotechnical data. [2,3,4,5,6]

1. East Beach
2. West Beach
3. Leadbetter Beach
4. 401 E. Yanonali Street (i.e., City Corporation Yard, APN #017-540-006), and
5. 103 S. Calle Cesar Chavez (APN #017-113-020)

These locations are identified in Figure 3.

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[2] Outfall pipeline easement granted by Southern Pacific Railroad Company: Recording Instrument, Book 902, pages 111 through 120, dated November 28, 1949.
[3] CH2M Hill. 1989. Draft Technical Memorandum No. 3: Report on Preliminary Hydrogeologic Testing on East Beach, Santa Barbara. Prepared for City of Santa Barbara, California.
[4] CH2M Hill. 1990. Desalination Feasibility Study Summary Report. Prepared for City of Santa Barbara and Goleta Water District, California.
[5] CH2M Hill. 1990. Draft Technical Memorandum: Report on Hydrogeologic Testing of Beach Sand Lens, Santa Barbara. Prepared for City of Santa Barbara.
[6] Martin, P., Berenbrock, C., 1986. Ground-Water Monitoring at Santa Barbara, California: Phase 3 – Development of a Three-Dimensional Digital Ground-Water Flow Model for Storage Unit I of the Santa Barbara Ground-Water Basin, U.S. Geological Survey Water-Resources Investigations Report 86-4103.

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Figure 3 - Subsurface Intake Project Site Alternatives

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At these locations, this study will focus on the areas onshore and offshore, depending upon the intake technology that is being considered. For offshore areas, only the submerged tideland areas that fall within the sovereign lands legislatively granted to the City, pursuant to Chapter 78, Statutes of 1925, as amended (Grant) will be considered. The seaward limit of this Grant is the U.S. pierhead line, established by the Secretary of the Navy and located one-half (1/2) mile offshore.[7] Consideration of only this offshore area simplifies property acquisition requirements (i.e., lease from the California State Lands Commission (CSLC)) for any lands required by subsurface intake facilities.

2.2.1 Site Access and Security

As part of the evaluation of project site alternatives, consideration will be given to the site access for maintenance procedures (such as pump replacement and well rehabilitation). Industry standards and precedent projects for each alternative will be uses as the basis for estimation of the frequency of these efforts. Furthermore, additional security features of the sites will be addressed as part of the basis of design.

2.3 Intake Technology Alternatives

Based upon the state of intake technology and recent studies conducted by others, the following intake technology alternatives will be considered for this study.[8,9,10,11,12]

1. Vertical wells
2. Lateral beach wells (onshore infiltration galleries)
3. Horizontal collector wells (i.e., Ranney wells)
4. Slant wells
5. Subsurface infiltration galleries (SIG) – offshore
6. Horizontal directionally drilled (HDD) wells (i.e., Neodren)

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[7] CSLC. 2014. Correspondence between California State Lands Commission (CLSC) and Joe Monaco (Dudek), Subject: Request for Consistency Determination for the Reactivation of a Desalination Plant with Lease No. PRC 4942.9, a General Permit - Public Agency Use to the City of Santa Barbara Channel, City of Santa Barbara, Santa Barbara County. August 20, 2014.
[8] Mackey. E.D., et al. 2011. Assessing Seawater Intake Systems for Desalination Plants. Water Research Foundation. Denver, CO.
[9] Kennedy/Jenks Consultants. 2011. scwd^2 Seawater Desalination Intake Technical Feasibility Study. Prepared for scwd^2 Desalination Program. September 2011.
[10] SWRCB. 2012. Mitigation and Fees for the Intake of Seawater by Desalination and Power Plants, Final Report. March 12, 2012.
[11] Missmer. 2013. Subsurface Intakes for Seawater Reverse Osmosis Facilities: Capacity Limitation, Water Quality Improvement, and Economics. Desalination. Elsevier. 322 (2013) 37-51.
[12] ISTAP. 2014. Final Report: Technical Feasibility of Subsurface Intake Designs for the Proposed Poseidon Water Desalination Facility at Huntington Beach, California. Published under the Auspices of the California Coastal Commission and Poseidon Resources (Surfside) LLC. October 9, 2014.

For each alternative, this study will develop a project description to assist in the comparison of potentially feasible alternatives. Project descriptions will contain the following:

- Physical description of the intake system and required infrastructure/facilities.
- Potential yield and water quality produced.
- History of use (i.e., California, U.S. and global) for both seawater and freshwater intake applications. History will include capacity as well as information regarding site, design, and performance.
- Regulatory requirements affecting design, construction and operation.
- Required construction equipment, resources, and procedures to assist in the subsequent evaluation of constructability and construction impacts.
- Reliability
- Maintenance requirements.

2.4 Subsurface Properties

This section presents Work Plan elements associated with reviewing available literature and publications that describe subsurface properties and characteristics in the vicinity of the shoreline at each project site alternative. This information will be used to identify potential areas for focused evaluation and analyze subsurface intake feasibility, capacity, and potential impacts. Also presented is a discussion of data gaps and collection of new data to address data gaps that will then be used to further evaluate feasibility and capacity of subsurface intake alternatives that are not eliminated from consideration due to failure to pass initial screening.

2.4.1 Literature Review

Available literature describing geologic and hydrogeologic properties of the beach and near shore areas will be reviewed. Sources of information include:

- Published geologic/hydrogeologic studies in the area, including:
  - USGS reports
  - Prior hydrologic and geotechnical studies conducted by the City.
- The City's 1989 and 1990 subsurface intake studies conducted on East, West, and Leadbetter Beaches.
- Geotechnical data associated with the design and installation of piles supporting Stearns Wharf.
- Any data related to sand movement (i.e., erosion and deposition) in the areas of East, West and Leadbetter Beaches that may be associated with harbor dredging and mooring (data provided by the City Waterfront Department).

- Hydrologic data and studies on existing wells and the groundwater aquifer used for drinking water production, including various USGS hydrogeologic and modeling studies.
- Studies relating to tsunami hazard, sediment transport, and projected sea level changes in the Santa Barbara area: California State Waters Map Series—Offshore of Santa Barbara, California. http://pubs.usgs.gov/sim/3281/
- Barnard, P.L., Revell, D.L., Hoover, D., Warrick, J., Brocatus, J., Draut, A.E., Dartnell, P., Elias, E., Mustain, N., Hart, P.E., and Ryan, H.F. 2009. Coastal Processes Study of Santa Barbara and Ventura Counties, California: U.S. Geological Survey Open-File Report 2009-1029, 904 p.
- Bechtel Corporation. 1990. Alternative Water Supplies. Submitted to the City of Santa Barbara. April 1990.
- City of Santa Barbara Public Works Department. Groundwater Data Collection files provided by Kelley Dyer on June 8, 2015.
- State Water Resources Control Board Geotracker contamination inventory tool. http://geotracker.waterboards.ca.gov/
- ISTAP. 2014. Final Report: Technical Feasibility of Subsurface Intake Designs for the Proposed Poseidon Water Desalination Facility at Huntington Beach, California. Published under the Auspices of the California Coastal Commission and Poseidon Resources (Surfside) LLC. October 9, 2014
- Johnson, S.Y., Dartnell, P., Cochrane, G.R., Golden, N.E., Phillips, E.L., Ritchie, A.C., Greene, H.G., Krigsman, L.M., Kvitek, R.G., Dieter, B.E., Endris, C.A., Seitz, G.G., Sliter, R.W., Erdey, M.D., Gutierrez, C.I., Wong, F.L., Yoklavich, M.M., Draut, A.E., Hart, P.E., and Conrad, J.E. (S.Y. Johnson and S.A. Cochran, eds.), 2013. California State Waters Map Series—Offshore of Santa Barbara, California: U.S. Geological Survey Scientific Investigations Map 3281, pamphlet 45 p., 11 sheets, scale 1:24,000.
- National Water Research Institute (NWRI), 2015. West Basin Municipal Water District's Ocean Water Desalination Subsurface Intake Study – Guidance Manual Review, Bureau of Reclamation Project No. R14AP00173.
- Nishikawa, T., 1997. A Simulation-Optimization Model for Water Resources Management, Santa Barbara, California, U.S. Geological Survey Water-Resources Investigations Report 97-4246.
- Nishikawa, T., 1998. Water-Resources Optimization Model for Santa Barbara, California. J. Water Resource Planning Management, 124(5), 252–263.
- Martin, P., 1984. Ground-Water Monitoring at Santa Barbara, California: Phase 2 – Effects of Pumping on Water Levels and on Water Quality in the Santa Barbara Ground-Water Basin, U.S. Geological Survey Water Supply Paper 2197.
- Martin, P., Berenbrock, C., 1986. Ground-Water Monitoring at Santa Barbara, California: Phase 3 – Development of a Three-Dimensional Digital Ground-Water

Flow Model for Storage Unit I of the Santa Barbara Ground-Water Basin, U.S. Geological Survey Water-Resources Investigations Report 86-4103.

- Mustain, N. 2007. Grain Size Distribution of Beach and Nearshore Sediments of the Santa Barbara Littoral Cell: Implications for Beach Nourishment. MS Thesis in Earth Sciences, University of California, Santa Cruz, 107 pp.
- SWRCB. 2015. Proposed Desalination Amendment: Creating a Consistent Permitting Process. State Water Resources Control Board. April 24, 2015.
- SWRCB. Draft Final Amendment to the Water Quality Control Plan for Ocean Waters of California Addressing Desalination Facility Intakes, Brine Discharges, and Incorporating other Non-substantive Changes. State Water Resources Control Board. May 5, 2015.
- Swarzenski, P.W., Izbicki, J.A. 2009. Coastal Groundwater Dynamics off Santa Barbara, California: Combining geochemical tracers, electromagnetic seepmeters, and electrical resistivity. Estuarine, Coastal and Shelf Science, 83, 77-89.
- Todd, D. K. 1978. Groundwater Basin Data: Status and Needs. A Report to the City of Santa Barbara, California.
- U.S. Department of the Interior, 1969. Geology, Petroleum Development, and Seismicity of the Santa Barbara Channel Region, California. Geological Survey Professional Paper 679.
- U.S. Geological Survey. Groundwater Watch, Santa Barbara County. http://groundwaterwatch.usgs.gov/countymap.asp?sa=CA&cc=083
- Wong, F.L., Phillips, E.L., Johnson, S.Y., Sliter, R.W., 2012. Modeling of Depth to Base of Last Glacial Maximum and Seafloor Sediment Thickness for the California State Waters Map Series, Eastern Santa Barbara Channel, California, U.S. Geological Survey Open-File Report 2012-1161, 16p.

Additional relevant geological reports and studies prepared for other subsurface intake projects in California will also be reviewed to obtain information about the applicability of comparable near shore conditions that may be relevant to this study.

Information that will be collected and reviewed in this task will be used to develop an understanding of the stratigraphy of the beach and near shore environment, locations and depths that could be targeted for potential development of subsurface intake facilities, hydraulic properties of the various geologic units, location of confining layers and faults that would limit the yield of subsurface intake facilities, groundwater quality considerations (including the location of known sources of contamination), and location of water supply wells and sensitive habitats that could be impacted by subsurface intake development.

2.4.2 Additional Data Collection

It is anticipated that there will be a number of uncertainties regarding subsurface conditions. Therefore, identification of associated data gaps that will developed during this feasibility study. In some cases, it may be appropriate to make assumptions or translate subsurface information from other similar locations when conducting the feasibility analysis. Safety factors will be applied to these assumptions based upon the quality of information available – and where necessary, written justification will be made to substantiate these assumptions. In other cases, it may not be possible to make assumptions and, in such cases, a range of focused field data collection activities may be suggested to improve the understanding of site conditions and subsurface intake feasibility for any given subsurface intake alternative at a given location. This information may also be helpful in identifying locations along the beach where subsurface conditions are better than other locations (e.g., locations where there is coarse material associated with ancestral stream channels). Because the nature and significance of the data gaps is not yet known, it is not possible to develop a specific Work Plan for data collection at this time. For this reason, this Work Plan outlines several potential data collection activities that may be performed, depending on the type of subsurface intake facility being evaluated. These activities fall into the following general categories that may include (least invasive and costly listed first):

1. Geophysical survey conducted along the beach and into near shore area to aid in defining stratigraphy, target zones, and depth to bedrock.
2. Drilling of coreholes and installation of piezometers to refine subsurface stratigraphy, target zones, and groundwater levels at the shoreline in specific areas.
3. Installation of one or more test wells and observation wells and performance of aquifer tests to measure aquifer productivity, hydraulic conductivity, and water quality.
4. Offshore drilling and coreholes to collect ocean bottom samples for determination of permeability and seawater infiltration gallery feasibility, SIG basis of design, and depth to low permeability materials associated with an offshore fault.

Based upon these data collection alternatives, if desired, at the direction of City Council, it is anticipated that a more specific data collection program (i.e., Phase 3 of the City's work program presented in Figure 1) would be developed for specific locations and specific subsurface intake alternatives, once the initial feasibility study work is completed. This data collection Work Plan will be prepared for review prior to implementation of the program.

2.4.3 Field Program Permitting

There are a number of permits/approvals that will be needed prior to conducting data collection field work. Permits/approvals may include:

- Coastal Development Permit
- Army Corps of Engineers, General Permit 7

- Regional Water Quality Control Board, Section 404 determination
- CEQA/NEPA
- City of Santa Barbara Parks Department

Activity descriptions associated with permit applications may include:

- Description of sampling technique and what the data will be used for.
- Description of equipment used and access that is required.
- Number of people required to collect the samples.
- Duration of sampling event.
- Description of any restoration that may be required following sampling.

It is anticipated that a more specific and detailed description of permits that would be required would be prepared as part of the field program discussed in the feasibility study.

2.5 Tsunami (Coastal) Hazards and Sediment Transport

Tsunami (coastal) hazards and sediment transport (i.e., erosion or deposition) will be evaluated to assess a subsurface intake alternative's susceptibility to oceanographic and geophysical hazards. This information will help to determine any applicable protective features, site alternatives, or required maintenance that should be considered when establishing a basis of design for each subsurface intake alternative. The following subsections present the technical approach for performing these evaluations.

2.5.1 Tsunami (Coastal) Hazard Analysis

The coastal hazard analysis consists of two phases of input analyses (i.e., water and land), delineation of elevations and assets on the landside of the shoreline, and delineation of water level extremes:

- *Landside Analysis* involves compilation of stationary databases, including drawings of the structural components of the Desal Plant that are part of the shoreline facilities (e.g., beach weir box), beach profiles and elevations of neighboring structures that might interact with local wave shoaling dynamics.

- *Waterside Analysis* includes a determination of extreme water levels, inundation, and recurrence probabilities using an assimilation of non-stationary databases. Because of the nature of the Santa Barbara shoreline, the coastal hazard analysis of the project structures and supporting facilities must be coupled to a sea level rise, tides, and storm wave hazard analysis of the harbor breakwater system. To perform these complex wave analyses, numerical refraction-diffraction computer codes called OCEANRDS will be utilized.

The fundamental inputs to the coastal hazards analysis are:

- Extreme wave height.
- Local water depth.
- Depth and slope of sediment cover over bedrock at the toe of the project structures.

The basis of design must consider a domain that extends far beyond the project boundaries to account for wave climate variability, wave propagation, shelf bathymetry, etc. The coastal hazard analysis will utilize a research tool recently developed at the Scripps Institution of Oceanography, referred to as the Coastal Evolution Model (CEM). The Coastal Evolution Model employs algorithms consistent with the U.S. Army Corps of Engineers Coastal Engineering Manual, but employs the latest generation equilibrium beach profile algorithms 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. Once the model has been calibrated, the design tsunami associated with the project will be incorporated.

The information developed as a result of this coastal hazard analysis will help to determine the design basis for locating subsurface intake facilities to avoid loss as a result of sea level rise or tsunami inundation and if any control features can be provided to protect the facilities.

2.5.2 Sediment Transport Analysis

The sediment transport and sediment budget analysis will support hazard assessment of the subsurface intake alternatives for the project. The characteristic of an optimal subsurface intake site is one that is neither erosional nor depositional, and one that is within a feasible hydraulic pathway to the desalination facility. Evaluating long term erosional or depositional tendencies and predicting shoreline evolution requires analyzing the sediment budget of the littoral cell in which the subsurface intake candidate sites reside. A littoral cell is a coastal compartment that contains the complete cycle of sedimentation, including sources, transport paths, and sinks. The Santa Barbara Littoral Cell (containing the sites to be evaluated) was one of the sites used during CEM validation, which required assembly and formatting of a full range of databases for the CEM – all of which will be available for use during this project.

The data developed as a result of the sediment transport analysis will help to determine the design basis for locating subsurface intake facilities to avoid plugging or erosion, if any control features can be provided to protect the facilities from erosion or deposition, and what maintenance may be expected.

2.6 Water Quality and Treatment Needs

Subsurface intake systems may reduce the concentration of the following parameters found in seawater:

- Suspended particulate matter (i.e., total suspended solids (TSS) and turbidity) and remove virtually all of the algae,
- Up to 98 percent of bacteria,
- Up to 50 percent of the natural organic carbon with a higher percentage of organic polymers removed, and
- A significant concentration of transparent exopolymer particles (TEP).[13]

Reduction of these constituents in seawater intake water may have the following advantages:

- *Suspended solids* - reduced concentrations of suspended solids may reduce or in some cases eliminate the need for additional pretreatment (i.e., filtration) to meet RO feed water silt density index (SDI) specifications. Where filtration is still required, a reduction in solids generation from backwash waste treatment is realized.
- *Biopolymer and TEP* - reduced concentrations of biopolymers and TEP may decrease the risk of membrane biofouling and may increase the time between required membrane cleanings - possibly allowing longer operating life for the membranes.[14]

Therefore, in some cases, a subsurface intake may eliminate or significantly reduce the need for a pretreatment system that would be needed to produce an equivalent RO feed water quality if a surface intake were used.

Although most existing literature provides the expectation that a subsurface intake will eliminate the need for pretreatment, some California projects have demonstrated otherwise[15]:

- **Long Beach, California:** A demonstration facility operating at the Hayes Generating Station in Long Beach showed that a subsurface infiltration gallery intake did not eliminate the need for pretreatment. The pilot study showed cartridge filters required weekly (or more frequent) replacement.[16]

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[13]ISTAP. 2014. Final Report: Technical Feasibility of Subsurface Intake Designs for the Proposed Poseidon Water Desalination Facility at Huntington Beach, California. Published under the Auspices of the California Coastal Commission and Poseidon Resources (Surfside) LLC. October 9, 2014.
[14] Dehwah. A.H.A., Li, S., Al-Mashharawi, S., Winters, H., Missimer, T.M. 2014. Influence of beach well and deep ocean intakes on TEP and organic carbon reduction in SWRO systems.
[15] SWRCB. 2012. Mitigation and Fees for the Intake of Seawater by Desalination and Power Plants, Final Report. March 12, 2012.
[16] Allen, J., Cheng, R., Tseng, T.J., Wattier, K., 2009. Update for the Pilot and Demonstration-Scale Research Evaluation of Under-Ocean Floor Seawater Intake and Discharge. 2009 AWWA Annual Conference & Exposition. June 16, 2009.

- **Morro Bay, California:** Dissolved iron concentrations from vertical beach wells were found to range from 1 to 10 mg/L necessitating pretreatment using sulfuric acid and filtration.[17]
- **Doheny Beach, California:** Significant concentrations of iron and manganese (Fe: 10 mg/L; Mn: 5 mg/L) were found in source water from a slant well intake. It is believed that these concentrations will decrease over time to non-detect levels, however, the rate at which this will occur cannot be reliably predicted. Additional treatment is recommended for implementing the desalination process and to treat brine before discharge to the South Orange County Water Authority's ocean outfall.[18]

As a result, careful study of subsurface geochemistry must be completed to demonstrate the possible production of suspended solids, iron, and manganese. The available data will be used to compare against other installations to determine a basis of design water quality that can be expected. However, because the City's desalination plant is already equipped with filtration, it is assumed that filtration technology will continue to be used to remove suspended solids. The need for additional pretreatment to address dissolved iron and manganese will be estimated through this study.

Additionally, raw water characterization will consider capture of high concentrations of carbon dioxide (i.e., greater than that seen in seawater) that may require additional mitigation to offset greenhouse gas (GHG) to meet the City's GHG limits.[19]

2.7 Analysis of Subsurface Intake Systems

This section presents the methodology that will be used to evaluate each subsurface intake alternative in terms of various hydrogeologic feasibility screening criteria. The following hydrogeologic feasibility screening criteria are included in the analysis:

- Individual facility yield, spacing for multiple locations of given subsurface intake type, and length of beach required to produce 15,898 gpm and 10,000 AFY
- Percentage of ocean water captured by the subsurface intake
- Impacts to local groundwater supplies and sensitive habitats
- Potential to capture or mobilize known groundwater contamination

Methodologies for conducting the technical analyses are described below.

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[17] Kartinen.E., et al. 2003. Solving Morro Bay's Seawater Reverse Osmosis Plant's Iron Problem. AWWA Membrane Technology Conference. Atlanta, GA. March 2003.
[18] MWDOC. 2014. Final Summary Report - Doheny Ocean Desalination Project Phase 3 Investigation. Prepared by the Municipal Water District of Orange County. January 2014.
[19] City of Santa Barbara. 2012. Climate Action Plan. September 2012.

2.7.1 Methodology for Evaluating Yield, Intake Facility Spacing, and Length of Beach Required

The number of facilities required to meet the target flow of up to 15,898 gpm (10,000 AFY) and the length of beach required to accommodate these facilities is an important feasibility consideration. The infrastructure (i.e., facilities) required to accommodate each type of subsurface intake (presented in Section 2.3) will have a different configuration and expected flow rate. When multiple facilities are required to achieve the target yield, the facilities must be spaced far enough apart from one another to prevent significant interference with each other, thus reducing yield and potentially impacting local groundwater supplies, other groundwater users and/or sensitive habitats. There are three potential project site alternatives under consideration for subsurface intakes: East, West, and Leadbetter Beach (refer to Figure 3). Following are approximate lengths of each beach, measured using the City of Santa Barbara Map System at the approximate high tide mark[20]:

- Leadbetter Beach: 3,230 feet; 0.61 miles.
- West Beach: 1,395 feet; 0.26 miles.
- East Beach: 8,130 feet; 1.54 miles.

These are total lengths; the actual length available to a project would likely be less because of site access issues, presence of creek and estuary discharges, setback requirements from environmentally sensitive areas, locations of existing facilities (pier, recreational areas, dredge system piping, etc.), and utilities.

To determine the overall beach area required for a subsurface intake alternative, the first step is to estimate the rate of flow that each type of subsurface intake facility would be expected to produce. Standard analytical equations and numerical methods will be used to estimate flow. The type of analytical approach will depend upon the type of subsurface intake (e.g., well versus subsurface infiltration gallery) and the aquifer unit penetrated by the subsurface intake. Table 2.1 presents the methods that will be used to calculate yield.

Assumed aquifer water levels (high and low), groundwater gradient (high and low), and a range of transmissivity values for the target aquifer zone(s) will be estimated using available data obtained in Section 2.4.1. A maximum amount of allowable water level drawdown at the subsurface intake (e.g., inside the well) will be established so that the water level does not fall below the top of a confining layer or assumed screen depth. The calculations will be iterated until the production rate does not result in drawdown exceeding the allowable maximum.

[20] City of Santa Barbara. Map Analysis and Printing System. http://gismaps.santabarbaraca.gov/SilverlightViewer/Viewer.html?Viewer=CityOfSantaBarbaraPublic

Table 2.1 Methods Used to Compute SSI Yield		
Type of SSI	**Method to Estimate Flow**	**Reference**
Vertical Wells	Theis Method (confined) Neuman and Witherspoon (unconfined, leaky)	Theis, C.V. 1935. The relation between the lowering of the piezometric surface and the rate and duration of discharge of a well using groundwater storage, Am. Geophys. Union Trans., vol. 16, pp. 519-524. Neuman, S.P. and P.A. Witherspoon. 1972. Field determination of the hydraulic properties of leaky multiple aquifer systems, Water Resources Research, vol. 8, no. 5, pp. 1284-1298.
Lateral beach wells (onshore infiltration galleries)	Yield is assumed and is a function of the length of intake screen required to achieve the yield and permeability of surrounding media	Driscoll, 1986. Groundwater and Wells, Second Edition, pg. 765. On-land infiltration galleries.
Collector wells (i.e. Ranney wells)	Hantush and Papadopulos, 1962	Hantush, M.S., Papadopulos, I.S., 1962. Flow of ground water to collector wells. J. Hydraulics Div., Proc. Am. Soc. Civil Engrs HY 5, 221–244.
Slant wells	Universal Drawdown Equation or Numerical Model	Williams, D. E., 2013. Drawdown distribution in the vicinity of nonvertical wells. Groundwater, vol. 51, no. 5, pp. 745-751 MODFLOW 2000
Subsurface infiltration galleries (SIG) – offshore	Yield is assumed and is a function of the length of intake screen, permeability of the bed material, and area required to achieve the yield	Driscoll, 1986. Groundwater and Wells, Second Edition, pg. 763. Flow into bed-mounted infiltration galleries.
Horizontal directionally drilled (HDD) wells (i.e., Neodren)	Universal Drawdown Equation or Numerical Model	Williams, D. E., 2013. Drawdown distribution in the vicinity of nonvertical wells. Groundwater, vol. 51, no. 5, pp. 745-751 MODFLOW-2000

Once the yield from a single subsurface intake facility is estimated, the number of subsurface intake facilities required to achieve the target yield can be computed. The next

step is to calculate the distance needed between subsurface intake facilities in order to minimize interference and reduction in yield caused by water level drawdown from the neighboring intake. This step will not be needed for the subsurface infiltration gallery type intakes because this type of intake does not rely on horizontal flow through an aquifer.

For subsurface intakes that require multiple facilities to achieve the target yield, the applicable analytical equations presented in Table 2.1 will be used with the calculated flow rate to estimate the amount of drawdown that would be expected at various distances from the subsurface intake. It is assumed that drawdown caused by interference from the neighboring subsurface intakes (combined total from all neighboring subsurface intakes) is unacceptable if it exceeds 20 percent of the available drawdown in the affected well. On the basis of this analysis, the acceptable spacing between subsurface intake facilities can be estimated. Lastly, the length of beach required to achieve the target yield with the estimated number of subsurface intake facilities can be estimated and compared to the length of beach that is available.

2.7.2 Methodology for Evaluating Percentage of Ocean Water Inflow into Subsurface Intake Systems

Subsurface intake's that draw a significant percentage of total flow from groundwater rather than seawater may have greater impact on the local groundwater basin, City wells, and sensitive habitats. The percentage of seawater relative to groundwater brought into the subsurface intake will depend on the alternative being considered, location relative to ocean, and the aquifer that is penetrated by the intake. For example, vertical wells would be expected to produce a mixture of native groundwater and seawater because they are typically located some distance back from the beach, while subsurface infiltration galleries produce primarily seawater because they are constructed on the ocean floor. In addition, if the aquifer material penetrated by a well is separated from the ocean bottom by a confining layer or the ocean bottom lacks permeability, or the presence of the known offshore fault limits the flow of seawater, wells would be expected to produce proportionately more groundwater than seawater and hence, would have a greater potential impact on the groundwater basin.

As discussed in Section 2.4, the City will estimate subsurface properties based upon existing literature, or supplemental information will be collected as determined necessary and if authorized by City Council. This information will include an assessment of the production capacity of intake facilities subsurface properties and also the connectivity of subsurface formations to the local groundwater supplies and sensitive habitats. Based upon an initial review of the data, there appears to be an unconfined shallow zone (upper 100-200 feet) and deeper, underlying zones referred to as the upper and lower producing zones. Percent ocean water yield from the shallow unconfined and upper production zone will be estimated using applicable analytical methods such as WINFLOW (2-D), a simplified numerical flow model with particle tracking, or the USGS groundwater flow model currently under development for the Santa Barbara basin (refer to Section 2.6.3). Preference will be

given to using the USGS groundwater flow model if it can be readily adapted to evaluate this issue. Aquifer water levels, groundwater gradient, and transmissivity values for the target aquifer zone will be estimated using available data obtained in Section 2.4.1. Percent ocean water yield from the lower production zone will not be estimated.

2.7.3 Methodology for Evaluating Impacts to Local Groundwater Supplies and Sensitive Habitats

As shown in Figure 4, most City wells are completed in Storage Unit 1 that underlies much of the City center and several are completed in the Foothill Basin on the north and west side of the City (Storage Unit 2). Wells are screened in the upper, middle, and/or lower producing zones at various depths. In general, groundwater in Storage Unit 1 flows from north to south toward the ocean where it discharges. During periods of heavy pumping (e.g., drought periods), there is evidence that seawater intrusion occurs. Subsurface intakes that produce a significant amount of local groundwater may impact the amount of groundwater in storage and affect water levels and production at City wells, particularly during drought. Water level drawdown caused by a subsurface intake may also lower water levels and increase the potential for salt water intrusion. There are also several sensitive habitats in the vicinity of the beach areas that are the subject of this study. These include, but may not be limited to:

- Sycamore Creek and tidal pool that seasonally discharges to the East Beach area
- Clark bird refuge and pond that discharges to East Beach
- Mission Creek and tidal pool that discharges to East Beach

A numerical model will be used to determine potential impacts to the Storage Unit 1, the Foothills Basin and local sensitive habitats. For many years, the City has been working with the USGS on developing a numerical groundwater flow model for the Santa Barbara Basin as part of its Long Term Water Supply Program, adopted in 1994. The model is based on MODFLOW-2000, with the addition of SEAWAT-2000 to model salt water intrusion. USGS staff working on the model have indicated that the model is appropriate for evaluating reductions in available groundwater storage in the basin and impacts to City production wells resulting from subsurface intakes, including salt water intrusion. The model can also be used to simulate water level drawdown near sensitive habitats that are hydraulically-connected to the shallow groundwater system. Some modifications to the model may be needed to simulate water levels in the shallow zone. The updated model report is undergoing technical review at the USGS and will not be published until early 2016. However, USGS staff have indicated that the model is completed and calibrated and they are willing to use the model and to generate preliminary results on an informal basis until the model update is published.

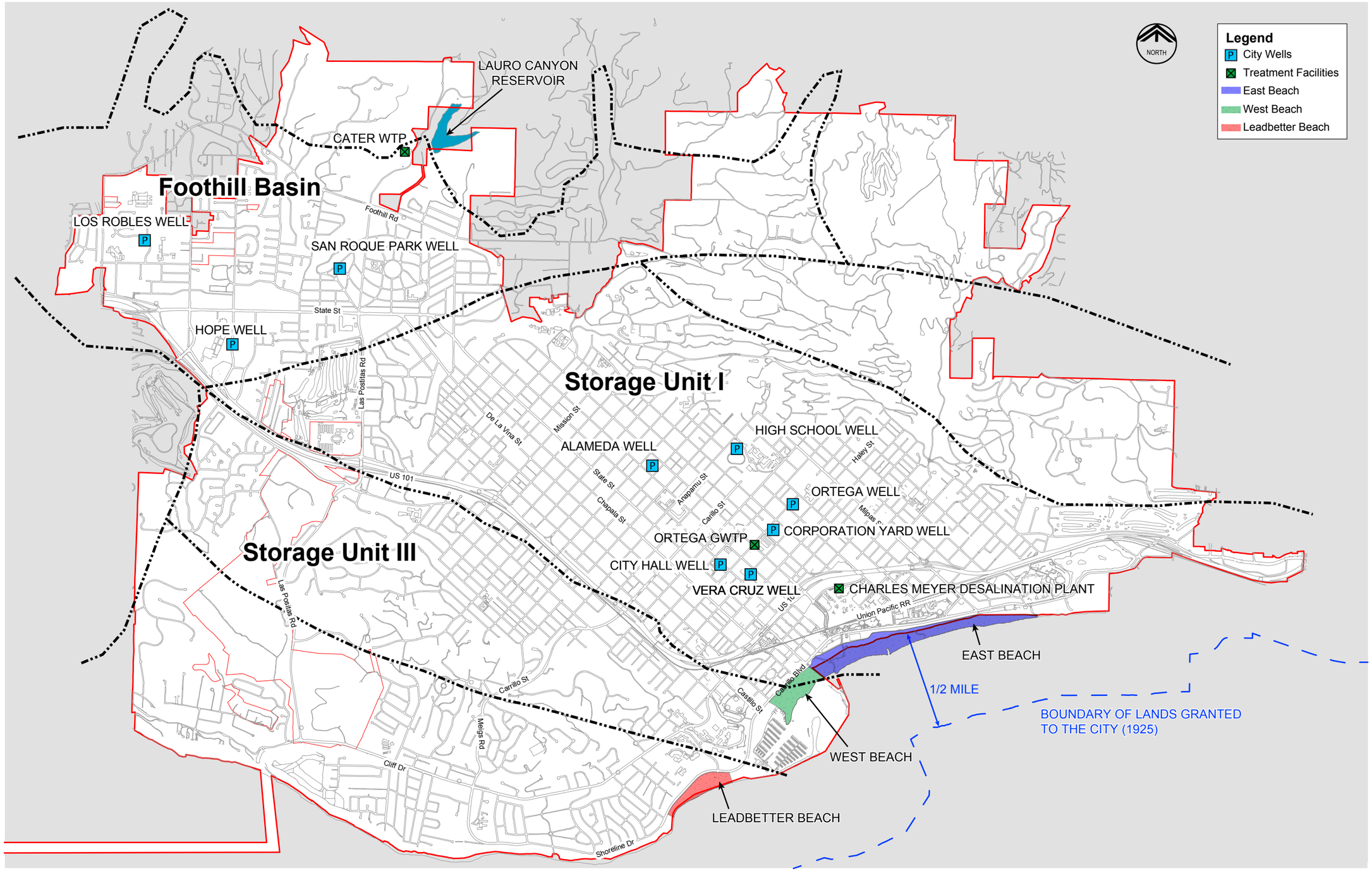

Figure 4 - City Water Wells and Groundwater Basins

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The model can simulate pumping from subsurface intake vertical wells along the coast. Collector wells can be simulated as large diameter production wells. Onshore infiltration galleries can be simulated with the model using a drain package. Slant wells and HDD wells cannot be directly simulated and so some adjustments to the model or simplifying assumptions will be necessary because these intakes are completed at an angle and do not align with the model grid arrangement. The subsurface infiltration gallery alternative will likely not be modeled using the groundwater flow model because it is assumed that all of the flow into this intake is derived from seawater and so there is no anticipated impact on the groundwater basin or sensitive terrestrial habitats.

2.7.4 Methodology for Evaluating Potential Capture of Known Groundwater Contamination

The uppermost portions of shallow zone that underlie the City are 100 - 200 feet thick and extend toward and beneath the shoreline. Groundwater quality in this shallow zone has been impacted by a number of sources of contamination, which are regulated by the Regional Water Quality Control Board. Subsurface intakes that are open to this zone may capture or mobilize this contamination. It is also possible, depending on degree of confinement, that subsurface intake pumping from deeper water producing zones could draw contamination into deeper units and impact subsurface intake water quality. Steps that will be taken to evaluate the potential for this to occur include the following:

- Prepare a contaminant source inventory for the area within 2 miles of the beach areas.
- Identify known and documented sources of groundwater contamination that have the potential, due to proximity and mobility characteristics, to impact subsurface intake water quality in the shallow zone or deeper producing zones.
- Utilize the results from the numerical modeling assessment performed in Section 2.7.3 to assess whether water level drawdown from subsurface intake operations is likely to cause movement of known sources of contamination toward the subsurface intake. Some modifications to the model may be needed to simulate water levels in the shallow zone.

Based upon this information, the project team will summarize the relative risk (high, medium, or low) of capturing known sources of contamination into subsurface intake alternatives completed in the shallow zone and deeper producing zones.

2.8 Project Life

A 20 year project life will be assumed for a subsurface intake system. This is also the time that is assumed to be required for repayment of any loan used to finance a subsurface intake project.

2.9 Reliability Features

Reliability of maintaining the required intake capacity and water quality will also be addressed in the design basis. Based upon the intake type, hydrogeology, geochemistry, and other factors, using the literature data, a safety factor will be substantiated and established as a basis of design requirement used to determine the redundancy required to address downtime for maintenance and repairs, as well as a possible decrease in production capacity due to plugging.

3.0 FEASIBILITY CRITERIA AND INITIAL SCREENING

As presented in Figure 1 and in Section 2 of this Work Plan, as the design basis is developed, initial screening criteria (i.e., based upon technical criteria) are considered. However, before the initial screening analysis can proceed, it is necessary to first identify feasibility criteria that can be used to analyze the subsurface intake alternatives.

For this project, "feasibility" will be defined by industry standard procedures for projects in California, as documented in the 2012 California Environmental Quality Act (CEQA) Statute and Guidelines. The act provides the following definition:

> *"Feasible" means capable of being accomplished in a successful manner within a reasonable period of time, taking into account economic, environmental, social, and technological factors."*

Consistent with this definition, the Ocean Plan Amendments that were adopted by the State Water Resources Control Board on May 6, 2015 identifies 13 factors that should be used to determine feasibility for subsurface intakes:

1. Geotechnical data
2. Hydrogeology
3. Benthic topography
4. Oceanographic conditions
5. Presence of sensitive habitats
6. Presence of sensitive species
7. Energy use
8. Impact on freshwater aquifers, local water supply and existing water users
9. Desalinated water conveyance
10. Existing infrastructure
11. Design constraints (engineering constructability)
12. Project life cycle costs
13. Other site and facility-specific factors

For the purposes of this study, these factors can be identified by the four main components of the CEQA definition of "feasible" as presented in Table 3.1: i.e., economic, environmental, social, and technological factors. As indicated in Table 3.1, some of the Ocean Plan criteria affect one or more of the CEQA factors of feasibility. Expanded definitions for each of the subsurface intake feasibility screening criteria presented in Table 3.1 are presented in following subsections.

Table 3.1 Feasibility Criteria

Feasibility Criteria	CEQA Feasibility Criteria			
	Technological Factors	Social Factors	Environmental Factors	Economic Factors
Geotechnical factors				
1 Geochemistry				
a. Risk of adverse geochemical interactions due to fluid mixing	x			
b. Risk of well clogging	x			
c. Risk of changes to inorganic water chemistry	x			
2 Seismic hazards				
a. Project facilities would cross a known fault line, or be exposed to a seismic hazard that could otherwise not be protected from loss by design	x			
Hydrogeology factors				
3 Impact on freshwater aquifers, local water supplies and existing water users	x	x	x	
4 Impact to sensitive habitats such as marshlands, drainage areas, etc.	x	x	x	
5 Potential yield per installation	x			
6 Proximity to sources of underground water contamination (i.e., will mobilize or capture contamination)	x		x	
Benthic Topography				
7 Suitability of bottom conditions (e.g., rocky bottom, presence of sensitive environments such as kelp beds, etc.)	x			
Oceanographic Factors				
8 Sensitivity to sea level rise (i.e., bathymetry)	x			x
9 Sensitivity to erosion or sedimentation (e.g., able to protect against erosion or sedimentation, able to maintain permeability of ocean bottom without entrainment of fine sediment (i.e., armoring), etc.)	x			x
10 Sensitivity to tsunami inundation	x			x
Presence of Sensitive Habitats				
11 Proximity to marine protected areas	x		x	
12 Proximity to on-shore habitats such as marshlands, or environmental sensitive habitat areas (ESHAs)	x		x	
Energy Use				
13 Project requires more or less energy than other alternatives, accounting for any possible reduction in treatment requirements.	x		x	x
14 Project energy use exceeds City's Greenhouse Gas Emission Threshold as identified in the City's 2012 Climate Action Plan				
Design and Construction Constraints				
15 Proximity to existing infrastructure (e.g., existing intake line, railroad crossing, desalination facility)	x			x
16 Number of units required for design capacity	x			x
17 Linear feet of beachfront required	x			x
18 Onshore footprint for facilities	x			x

Table 3.1 Feasibility Criteria

Feasibility Criteria	CEQA Feasibility Criteria			
	Technological Factors	Social Factors	Environmental Factors	Economic Factors
19 Onshore footprint required for construction activities	x			x
20 Complexity of off-shore construction (e.g., uneven topography, wave energy, depth to seabed, environmental monitoring requirements, etc.)	x		x	x
21 Scope and complexity of property, easement, or right of way acquisitions (e.g., State Lands lease, property condemnation, rail road crossing, etc.)	x	x		x
22 Reliability and performance				
a. Precedent (i.e., demonstration of intake technology in similar seawater or freshwater applications at a similar scale)	x			
b. Performance risk (i.e., stable yield and quality over project life)	x			
c. Maintainability (i.e., can yield or quality be restored by standard means that won't significantly impact the facility operation or availability)	x			
d. Material of construction performance	x			x
23 Sustainability (e.g., labor, chemicals, mechanical equipment use to sustain performance)				
a. Frequency of maintenance	x	x	x	x
b. Complexity of maintenance	x	x	x	x
Other Site-Specific Factors				
24 Impact to recreational uses of land or ocean		x		x
25 Impact to commercial uses of land or ocean		x		x
26 Certainty of implementation schedule and costs (i.e., as affected by affected by permitting, demonstration or pilot testing, environmental requirements, monitoring, etc.)		x		x
Economic Factors				
27 Cost impacts to water rate payers		x		x
28 Impact of project construction schedule on recreational and commercial use as it relates to the local economy		x		x

Geotechnical Factors

1. Geochemistry
    a. Risk of adverse geochemical interactions due to fluid mixing: The risks of adverse fluid mixing are greatest where waters from different directions within an aquifer (landwards vs. seawards), aquifers, or aquifer depths enter an intake (or enter different intakes and later mixing within piping system). Systems with the lowest risk of adverse fluid mixing are constructed subsea and produce water largely by vertical infiltration.
    b. Risk of well clogging: Loss of intake capacity by clogging (or plugging) can be cause by a variety of chemical, biological, and physical processes. The greatest risk of clogging occurs where there is mixing of dissimilar water or a change in water chemistry (e.g., introduction of dissolved oxygen). Clogging is of greatest concern where rehabilitation is complex and expensive.
    c. Risk of changes to inorganic water chemistry: Long-term changes in water chemistry caused, for example, by different fractions of landward derived freshwater could interfere with the reliable performance of the reverse osmosis process. The risk is lowest where intakes produce water predominantly by vertical infiltration of seawater (e.g., subsea galleries).
2. Seismic Hazards
    a. Project facilities located near a fault line: Project facilities that would cross a known fault line, or be exposed to a seismic hazard that could otherwise not be protected from loss by design are considered. Active faults pose a risk of liquefaction and settlement at the facility.

Hydrogeology Factors

1. Impact on freshwater aquifers, local water supplies, and existing water users: Groundwater pumping of saline water may result in abstraction of freshwater from a groundwater basin, adversely impacting the basin's water budget and causing additional drawdown that causes groundwater to flow seaward.
2. Impact to sensitive habitats: This criterion considers impacts from water drawdown or changes in hydrology due to the location of a subsurface intake relative to sensitive habitats such as marshlands, wetlands, and drainage areas.
3. Potential yield per installation: Potential yield per installation are best estimates of unit yield per well, acre of gallery subsurface area, and per foot of HDD well or water tunnel. In the absence of site-specific data, these values were estimated based on local hydrogeology and the performance of similar systems constructed elsewhere.
4. Proximity to sources of underground water contamination: The potential to uncover or release potential underground water contaminants is addressed in this criterion

including the potential of the contaminants to mobilize and spread to other areas or to feasibly capture and abate contamination.

Benthic Topography

1. Suitability of bottom conditions: Suitability of bottom environmental conditions is applicable to only seabed and surf zone infiltration galleries. Unsuitable conditions would be a rocky bottom or the presence of sensitive environments such as kelp beds.

Oceanographic Factors

1. Sensitivity to sea level rise (bathymetry): Sensitivity to sea level rise relates to the effects of changes in water depth and landwards beach migration on constructed intakes. The location of intake structures needs to consider the projected rise of seawater and beach migration over their operational lives. This includes design considerations of locating different subsurface intake technologies further inland or offshore to avoid the impacts of future sea level rise that would place them in a sub-optimal setting. Sensitivity to sea level rise based on local bathymetry including both current and potential post-sea level rise future conditions.
2. Sensitivity to erosion or sedimentation: Sensitivity to erosion or sedimentation – e.g., able to protect against erosion or sedimentation, able to maintain permeability of ocean bottom without entrainment of fine sediment (i.e., armoring). Sedimentation rate, whether natural or anthropogenically influenced, may impact subsurface intakes by either burying or exhuming them. The sensitivity of an intake design alternative is evaluated based on local sedimentation rates and likely intake locations and designs. This criterion also includes beach maintenance through artificial replenishment or deposition of dredge material.
3. Sensitivity to tsunami inundation: This criterion considers the location of facilities associated with each subsurface intake technology that may result in disruptions to operation or increased maintenance due to potential tsunami inundation. The ability to protect intake facilities through feasible design and construction is also considered as part of this criterion.

Presence of Sensitive Habitats

1. Proximity to California Marine Protected Areas (MPAs), California State Water Quality Protection Areas of Special Biological Significance (SWQPAs), and other offshore sensitive habitats. Pursuant to the Marine Life Protection Act it is unlawful to injure, damage, take, or possess any living or non-living, geological, or cultural marine resource within MPAs that would compromise protection of species, natural communities, habitats, or geologic features. The intent behind this Act was to protect sensitive marine resources within these MPAs and consistency with this intent will be evaluated in respect to all actions associated with construction, operation, and

maintenance of a subsurface intake. The SWQPAs are ocean areas monitored and maintained for water quality by the California State Regional Water Quality Control Board with the intent of protecting water quality within these potentially sensitive areas and the impacts from a subsurface intake to the protection of these areas is considered in this criterion. Impacts caused by construction, operation or maintenance of subsurface intakes to other offshore habitats including kelp beds, seagrass and eelgrass beds, and soft-bottom benthic habitat are also considered.

2. Proximity to onshore sensitive habitats: Sensitive onshore habitats such as estuaries, marshlands, wetlands, or environmental sensitive habitat areas (ESHAs) may experience direct and indirect impacts from construction, operations, and maintenance of a subsurface intake and associated facilities based on the location of these facilities.

Energy Use

1. Project energy requirements: This criterion addresses if the project's operational energy requirements would be more or less impactful than current site conditions, accounting for any possible reduction in treatment or pumping requirements.
2. Project greenhouse gas emissions (GHG): The project's increase or decrease in GHG emissions from operational energy use above the current site conditions and City's approved GHG policy is considered in this criterion.

Design and Construction Constraints

1. Proximity to existing infrastructure: Proximity to existing infrastructure to decrease the amount of construction associated with connecting the subsurface intake to an existing intake pipeline and the desalination facility. This can also include proximity to important infrastructure, such as a railroad crossing, that could increase or decrease the project implementation schedule and complexity of construction.
2. Number of units: Number of units (e.g., number of wells, gallery acres, and feet of well pipelines) required for design capacity, which can correlate to amount and complexity of construction and maintenance as well as the onshore and offshore space foregone.
3. Linear feet of beachfront required for facilities associated with a subsurface intake: The linear beachfront requirement gives an indication of how spread out a system will be and is an important cost and logistical factor. This is calculated by multiplying the number of units by anticipated minimum spacing for each subsurface intake technology.
4. Onshore footprint for facilities: The onshore footprint is the area permanently required for the facilities associated with the subsurface intake and do not include temporary construction easements.

5. Onshore footprint required for construction activities: The offshore footprint of a seafloor or surf zone infiltration gallery is determined by the size and number of filter units required for production capacity and reliability purposes.

6. Construction complexity: Complexity of construction refers to the potential for difficulties to occur during construction including:

    a. The local availability of contractors who are qualified to perform the work and that have the specialty equipment and experience with this specific type of work.

    b. Construction challenges and risks due to uneven topography, the depth to the seabed, and unfavorable wave energy conditions

    c. Consideration of factors that may impede or delay construction including uncertainties and extended duration for obtaining or complying with construction permits, seasonal restrictions on beach construction due to public use, seasonal restrictions of offshore operations due to sea conditions, as well as environmental review impacts from construction.

7. Scope and complexity of property, easement, or right of way acquisitions required to connect the subsurface intake to the desalination plant: This includes (but may not be limited to) obtaining California State Lands Commission lease, property condemnation, and railroad crossing.

8. Reliability and performance

    a. Precedent of technology implementation: Precedent of technology implementation of the subsurface intake technology in similar seawater or freshwater applications at a similar scale to the project. Confidence in the feasibility of a subsurface intake technology is greatest where there is a track record of successful implementation of that technology at other sites with geological conditions similar the project area.

    b. Performance risk: Performance risk is the potential for the subsurface intake system to not meet project performance expectations in terms of water yield and quality over the project life. A large amount of uncertainty with regard to the likelihood of successful implementation of a subsurface intake technology is considered a high performance risk. Performance risk also relates to the opportunities to pilot test an intake option or accurately estimate system performance using other means or data, including the operational history of comparable systems constructed in similar geologies.

    c. Maintainability: Maintainability of the subsurface intake system to restore yield or quality of product water by standard means that won't significantly impact the facility operation or availability of product water. This can include replacement of pumps, pipelines, and other intake facilities and also chemically or mechanically cleaning the subsurface intake to restore capacity.

d. Materials of construction performance: Material constraints address construction materials requirements for intake types. In general, seawater intakes should be constructed of corrosion resistant materials.

9. Sustainability (e.g., labor, chemicals, mechanical equipment use to sustain performance)
    a. Frequency of maintenance: Frequency of maintenance is the relative frequency at which an intake option is expected to require maintenance activities to either address breakdowns (e.g., pump failure) or restore system performance (e.g., well rehabilitation).
    b. Complexity of maintenance: Complexity of maintenance addresses both technical difficulties associated with potential maintenance activities and logistical issues that may make maintenance more complex. For example, offshore maintenance is considered more complex than onshore maintenance activities.

Other Site-Specific Factors

1. Impact to recreational uses of land or ocean: Impact to recreational uses of land or ocean including temporarily restricting recreational activities to beach users, surfers, and recreational boating during construction and/or maintenance activities, or permanent restrictions during the facility operation.
2. Impact to commercial uses of land or ocean: Impact to commercial uses of land or ocean including temporarily or permanently restricting fishing, aquaculture activity, commercial port and harbor activities, and ocean tourism.
3. Certainty of implementation: Certainty of implementation schedule and costs that accounts for effects from permitting, demonstration or pilot testing, environmental requirements, and monitoring.

Economic Factors

1. Cost to water rate payers: Cost impacts to water rate payers from estimated product water price accounting for construction, operational, and maintenance costs.
2. Impact of construction schedule on recreational and commercial uses: Impact of project construction schedule on recreational and commercial use as it related to the local economy. This factor primarily focusses on the restriction of recreational and commercial uses both onshore and offshore due to construction, operation, and maintenance of a subsurface intake.

3.1 Initial Screening Criteria

The technical factors identified in Table 3.1 will be a starting point to determine if each option should be further considered for evaluation - e.g., before economic, environmental and social factors are considered. Intake alternatives that are judged to have technical feasibility criteria in conflict with the project objectives will be determined to fail initial screening, and will not be considered further in this study. For alternatives that pass initial screening, each subsurface intake alternative will also be evaluated for feasibility based upon the economic, environmental, social, and technological factors identified in Table 3.1.

For the purposes of this study, "Initial Screening Criteria" will be defined as follows:

> *Initial Screening Criteria: Those technical factors that would not allow a full-scale system to be successfully constructed or operated, would result in a high risk of failure or unacceptable performance, or would not produce water supply required to replace the use of the desalination plant's screened open ocean intake per Study goals.*

Table 3.2 presents initial screening criteria that will be used in this study. Initial screening criteria will be analyzed concurrent to the design basis development presented in Section 2 to avoid carrying forward alternatives for further study that are not technically feasible.

Table 3.2 Initial Screening Criteria

Screening Criteria	Failure to meet criteria
Geotechnical Hazards	
Seismic hazard	Project facilities would cross a known fault line, or be exposed to a seismic hazard that could otherwise not be protected from loss by design
Hydrogeologic Factors	
Operation of subsurface intake adversely impacts existing fresh water aquifers, local water supplies, or existing water users.	• Volume of groundwater in storage is reduced due to subsurface intake pumping, impacting drought supply and requiring additional desalination to make up for loss of groundwater. • Operation of subsurface intake causes salt water intrusion into groundwater aquifers.
Operation of subsurface intake adversely impacts sensitive habitats such as marshlands, drainage areas, etc.	Operation of subsurface intake drains surface water from sensitive habitat areas or adversely changes water quality.

Table 3.2 Initial Screening Criteria	
Screening Criteria	**Failure to meet criteria**
Insufficient length of beach available for replacing full yield derived from the existing open ocean intake.	Small individual facility yield, large number of facilities required, and minimum spacing between facilities requires more shoreline than is available.
Benthic Topography	
Land type makes intake construction infeasible	Depth to bedrock too shallow (i.e., less than 40-feet deep); rocky coastline; cliffs
Oceanographic Factors	
Erosion, sediment deposition, sea level rise or tsunami hazards	Oceanographic hazards make aspects of the project infrastructure vulnerable in a way that cannot be protected and/or would prevent the City from being able to receive funding or insurance for this concept
Presence of Sensitive Habitats	
Proximity to marine protected areas	Location would require construction within a marine protected area
Design and Construction Constraints	
Adequate capacity	Subsurface material lacks adequate transmissivity to meet target yield of at least 15,898 gpm (i.e., build-out intake capacity necessary to produce 10,000 AFY).
Lack of adequate linear beach front for technical feasibility	Length of beachfront available is not sufficient for construction of the required number of wells of all or portion of intake to meet target yield
Lack of adequate land for required on-shore facilities	• Surface area needed for on-shore footprint of an intake unit is greater than the available onshore area • Requires condemnation of property for new on-shore intake pumping facilities
Lacking adequate land for on-shore construction staging	The amount of land available to stage construction does not meet need
Precedent for subsurface intake technology	Intake technology has not been used before in a similar seawater or fresh water application at a similar scale)

After the initial screening, projects will be further categorized as 1) Infeasible, 2) Potentially feasible, but does not meet Study goals, or 3) Potentially feasible. The next steps or actions for each of these categories is described in Section 1.

4.0 IMPLEMENTATION SCHEDULE DEVELOPMENT

In conjunction with the cost estimate, an implementation schedule for each of the project alternatives will be developed. Major components that will be included in the implementation schedule are summarized below:

- Planning Phase (Feasibility Studies)
  - Work Plan Development: A Work Plan outlining the feasibility study is developed
  - Initial Screening Analysis: Technical feasibility criteria are used to determine if a subsurface intake alternative passes initial screening.
  - Regulatory and Permit Requirements: Alternatives passing initial screening are subjected to an analysis of regulatory and permit requirements.
  - Conceptual Design: Site plans and design criteria are established for each surviving alternative.
  - Feasibility Analysis: Surviving alternatives are screened against all feasibility criteria outlined in Table 3.1.
- Test Well Demonstration
  - Design: A full set of bid set construction documents are created for the test well.
  - Permitting: Test well is put through a permitting process in which all required permits are obtained for test well – Coastal Development Permit, Army Corps Permit, City Building Permit, etc.
  - Environmental: CEQA and/or NEPA for test well
  - Bid Phase: Contractors are solicited for bids for construction of the test well.
  - Construction: Test well is constructed by selected contractor.
  - Test Well Demonstration: Test well is operated while data is collected and analyzed.
  - Report/Recommendation: All findings resulting from the test well demonstration are summarized and reported in the final report.

** *Assumes property or easement acquisition is not necessary for the Test Phase.*

- Implementation
  - Property and easement acquisition: Any property or easements that are needed will be attained.
  - Design: A full set of bid set construction documents are created for the full scale subsurface intake.
  - Permitting: Subsurface intake is put through a permitting process in which all required permits are obtained – Coastal Development Permit, Army Corps Permit, City Building Permit, etc.
  - Environmental: CEQA and/or NEPA for full scale subsurface intake.

- Bid Phase: Contractors are solicited for bids for construction of the full scale subsurface intake.
- Construction: Full scale subsurface intake is constructed by selected contractor.
- Operation: Full scale subsurface intake is operated and serves as a source or raw water to be treated at the Desal Plant.

5.0 COST ESTIMATING METHODOLOGY

As demonstrated in the programmatic work flow diagram presented in Figure 1, intake alternatives surviving initial screening shall proceed to various additional study phases, which provide the basis for a cost estimate. Alternatives lacking sufficient data for analysis may be recommended for additional data collection, resulting in the potential for samples and other studies. The following studies shall be performed on subsurface intake alternatives surviving initial screening analysis:

- Regulatory and Permit Requirements
- Geotechnical and Subsurface Studies (for alternatives requiring additional data collection)
- Conceptual Design

Aforementioned studies will be used as basis to perform a Class 4 feasibility cost estimate, as defined by the American Association of Cost Engineers (AACE), on each surviving subsurface intake alternative. Typical estimating methodologies for this level of cost estimate include parametric models, specific analogy, expert opinion, and trend analysis. A review of similar projects will be used as the basis for the cost estimate. As defined by the AACE, the expected accuracy range of a Class 4 cost estimate is as follows:

- Low: -15% to -30%
- High: +20% to +50%

The cost estimate will represent the total cost for implementation of the subsurface intake alternative. The estimated cost shall include the following:

- Feasibility analysis
- Environmental review, permitting, and public process
- Property and easement acquisition
- Design fees
- Construction costs
- Operation and maintenance

Results from the cost estimate will be used during the feasibility analysis of surviving alternatives. The feasibility analysis process is described in the following section.

6.0 FEASIBILITY ANALYSIS

As presented in Figure 1, each alternative that survives the initial screening analysis shall be subjected to a feasibility analysis after the estimated schedules and costs are complete. Whereas the initial screening analysis only considered certain technological factors presented in Table 3.2, the feasibility analysis will consider all technological, social, environmental, and economic factors presented in Table 3.1. For each alternative, advantages and disadvantages with respect to each of the 28 feasibility criteria will be ascribed. Table 6.1 below provides an example summary that will be used to present the feasibility criteria analysis for each intake alternative**.**

Once the feasibility analysis has been completed, it will be reviewed by the technical advisory panel. The final report deliverable will consist of all technical memoranda associated with this work.

7.0 TECHNICAL ADVISORY PROCESS

The technical advisory process described in this Work Plan provides an independent, third party review of the project work product at key intervals throughout the project duration, as the work product is developed. The technical advisory process shall achieve the following objectives:

1. Provide timely review of project work product by experts in the required subject matter to advise and guide the City's feasibility study.
2. Facilitate input from project stakeholders that can be used to inform the City's comparison of potentially feasible alternatives.
3. Create a record of the review and stakeholder process to be included as an appendix to the feasibility study report.

Table 6.1 Sample Feasibility Analysis Summary Table

Feasibility Criteria	Advantages	Disadvantages
Geotechnical factors		
1 Geochemistry a. Risk of adverse geochemical interactions due to fluid mixing b. Risk of well clogging c. Risk of changes to inorganic water chemistry **2 Seismic hazards** a. Project facilities would cross a known fault line, or be exposed to a seismic hazard that could otherwise not be protected from loss by design	**[Insert advantages for alternative here]**	**[Insert disadvantages for alternative here]**
Hydrogeology factors		
3 Impact on freshwater aquifers, local water supplies and existing water users		
4 Impact to sensitive habitats such as marshlands, drainage areas, etc.		
5 Potential yield per installation		
6 Proximity to sources of underground water contamination (i.e., will mobilize or capture contamination)		
Benthic Topography		
7 Suitability of bottom conditions (e.g., rocky bottom, presence of sensitive environments such as kelp beds, etc.)		
Oceanographic Factors		
8 Sensitivity to sea level rise (bathymetry)		
9 Sensitivity to erosion or sedimentation (e.g., able to protect against erosion or sedimentation, able to maintain permeability of ocean bottom without entrainment of fine sediment (i.e., armoring), etc.)		
10 Sensitivity to tsunami inundation		
Presence of Sensitive Habitats		
11 Proximity to marine protected areas		
12 Proximity to on-shore habitats such as marshlands, or environmental sensitive habitat areas (ESHAs)		
Energy Use		
13 Project requires more or less energy than other alternatives, accounting for any possible reduction in treatment requirements.		
14 Project energy use exceeds City's Greenhouse Gas Emission Threshold as identified in the City's 2012 Climate Action Plan		
Design and Construction Constraints		
15 Proximity to existing infrastructure (e.g., existing intake line, railroad crossing, desalination facility)		
16 Number of units required for design capacity		

Table 6.1 Sample Feasibility Analysis Summary Table

Feasibility Criteria	Advantages	Disadvantages
17 Linear feet of beachfront required		
18 Onshore footprint for facilities		
19 Onshore footprint required for construction activities		
20 Complexity of off-shore construction (e.g., uneven topography, wave energy, depth to seabed, environmental monitoring requirements, etc.)		
21 Scope and complexity of property, easement, or right of way acquisitions (e.g., State Lands lease, property condemnation, rail road crossing, etc.)		
22 Reliability and performance		
a. Precedent (i.e., demonstration of intake technology in similar seawater or freshwater applications at a similar scale)		
b. Performance risk (i.e., stable yield and quality over project life)		
c. Maintainability (i.e., can yield or quality be restored by standard means that won't significantly impact the facility operation or availability)		
d. Material of construction performance		
23 Sustainability (e.g., labor, chemicals, mechanical equipment use to sustain performance)		
a. Frequency of maintenance		
b. Complexity of maintenance		
Other Site-Specific Factors		
24 Impact to recreational uses of land or ocean		
25 Impact to commercial uses of land or ocean		
26 Certainty of implementation schedule and costs (i.e., as affected by affected by permitting, demonstration or pilot testing, environmental requirements, monitoring, etc.)		
Economic Factors		
27 Cost impacts to water rate payers		
28 Impact of project construction schedule on recreational and commercial use as it relates to the local economy		

To assist the Central Coast Regional Water Quality Control Board administer the technical advisory process, the City will retain the services of the National Water Research Institute (NWRI). NWRI is a California non-profit organization whose activities include ensuring safe, reliable sources of water now and for future generations through a variety of research, education, and public out-reach activities. NWRI has facilitated similar technical advisory programs on subsurface intake and potable reuse feasibility projects in California, including programs for both municipal and state regulatory agencies. NWRI will retain the services of the experts that will review the work, facilitate the project meetings (i.e., that will include an opportunity for stakeholder comments) and complete the documentation of the technical review and stakeholder process.

Participants in the technical advisory process shall consist of:

- A moderator: Jeff Mosher, National Water Research Institute,
- Technical Advisory Panel (TAP): Consultants retained by NWRI; The composition of the TAP shall consist of up to four individuals whose qualifications may include:
  - Hydrogeologists, geotechnical, civil engineers, and/or contractors experienced in the design, construction, and costs of subsurface desalination plant intakes.
  - CEQA consultant experienced in coastal development.
  - Public agency representative experienced with the implementation of seawater desalination.
  - Former regulators with experience in permitting.
- Project stakeholders (e.g., regulators and city residents),
- The City's public works staff, and
- The City's consultant team: Carollo Engineers.

This section of the Work Plan provides the guidelines for how the technical advisory process will be conducted. The qualifications and role of the technical advisors, the format for the technical advisory meetings, stakeholder process, and documentation will be explained.

7.1 Technical Advisory Panel

NWRI shall select and retain approximately four technical advisors to review the work product developed by the City’s consultant team. It is anticipated that the technical advisory panel may consist of the following types of experts:

- Hydrogeologists, technical, civil engineers, and/or contractors experienced in the design, construction, and costs of subsurface desalination plant intakes.

7.2 Technical Advisory Panel Meetings

The following technical advisory workshops will be held at the intervals described in the programmatic Work Plan diagram and project schedule - i.e., Figures 1 and 2:

1. TAP Workshop No. 1: Work Plan
2. TAP Workshop No. 2: Initial Screening Analysis
3. TAP Workshop No. 4: Subsurface Desalination Intake Feasibility Study

Note: *TAP Workshop No. 3 is associated with the Potable Reuse study.*

The City will provide NWRI with the necessary work product for review at least 15 working days prior to a technical advisory workshop. NWRI will be responsible for distributing the work product to the technical advisory panel, and posting the material to the project website (also managed by NWRI). The project website will be open to the public and NWRI shall post the work product no less than 5 days prior to a technical advisory workshop.

NWRI will create and distribute an agenda for each technical advisory workshop, however, each technical advisory workshop will consist of two parts and follow the format described in Table 7.1.

Table 7.1 TAP Workshop Format

Part 1	
1. Moderated by NWRI 2. Presentation by City highlighting key material from the work product that is the subject of the workshop 3. TAP questions and answers on presentation material and work product that is the subject of the workshop 4. Stakeholder comment period 5. Meeting minutes, including TAP questions, City responses, and stakeholder comments will be recorded by NWRI staff.	Participants include: • City and City's consultant team • NWRI moderator and staff • TAP members • Project Stakeholders
Part 2	
1. Moderated by NWRI 2. City and City consultant team will be provided an opportunity to ask the TAP questions regarding the comments received. 3. TAP will be allowed to ask additional questions. 4. Meeting minutes will be prepared by NWRI staff consisting of the final TAP comments on the work product developed by the City's team.	Participants include: • City and City's consultant team • NWRI moderator and staff • TAP members

7.2.1 Stakeholder Process

As indicated in Table 7.1, a portion of Part 1 of each TAP Workshop will consist of a stakeholder comment period where:

1. Stakeholders (e.g., regulatory agencies, City residents) will be provided the opportunity to fill out comment cards related to issues, feedback, or comments regarding the work product.
2. Comment cards must be submitted to NWRI staff 10 minutes before the stakeholder comment period begins.
3. Each stakeholder shall have 120 seconds to deliver their comments.
4. Stakeholders that have successfully completed their comment cards are able to yield their time to another individual to speak on their behalf.

NWRI's moderator will administer the stakeholder process in accordance with this procedure.

The entire workshop shall be recorded for reference and made available on the NWRI managed project website. It is the responsibility of NWRI to produce meeting minutes from the workshop, which will be reviewed by the consultant team and posted on NWRI's project website. The comment cards will require stakeholders to fill out the following information:

Name

Affiliation (e.g., regulatory agency, City resident, other)

City Resident? ☐ Yes or ☐ No

Comment

Stakeholders are not required to attend a technical advisory workshop to submit comments for the record. Comments may be submitted to NWRI within 5 working days of the technical advisory workshop. NWRI is responsible for recording comments and comment cards as part of the Workshop meeting record (i.e., meeting minutes).

7.3 Project Stakeholders

Anticipated stakeholders associated with this project are presented in Table 7.2. This list of stakeholders was adapted from the noticing list included in the City's 2014 Coastal Development Application for Repair and Maintenance Activities at the Charles Meyer Desalination Facility Offshore Intake Structure. This list includes those residents and businesses that are in close proximity to the areas affected by the work on the City's intake, and parties that have expressed interest in the City's desalination plant reactivation project. For the purposes of this study, this list should not be considered exhaustive and will require periodic updates as project alternatives are clearly defined and updated.

Table 7.2 Project Stakeholders	
Name	**Location**
Environmental Defense Center	Santa Barbara, CA
Santa Barbara Arts and Crafts Show	Santa Barbara, CA
California Department of Fish and Game,	San Diego, CA
Central Coast Regional Water Quality Control Board	San Luis Obispo, CA
Army Corps of Engineers Regulatory Division	Ventura, CA
Santa Barbara County Flood Control District	Santa Barbara, CA
City of Santa Barbara Waterfront Department	Santa Barbara, CA
City of Santa Barbara Creeks Division, Attention	Santa Barbara, CA
City of Santa Barbara Parks Division	Santa Barbara, CA
Santa Barbara Trolley Company	Santa Barbara, CA
Wheel Fun Rentals of Santa Barbara	Santa Barbara, CA
Land and Sea Tours	Santa Barbara, CA
Mtd Santa Barbara	Santa Barbara, CA
Rusty's Pizza	Santa Barbara, CA
Santa Barbara Visitor Center	Santa Barbara, CA
Santa Barbara Fish House	Santa Barbara, CA
City of Santa Barbara Recreation Division	Santa Barbara, CA
El Torito	Santa Barbara, CA
Segway of Santa Barbara	Santa Barbara, CA
Surf N Wear	Santa Barbara, CA
Mountain Air Sports	Santa Barbara, CA
Harbor View Inn, Eladio's Restaurant, State Street Coffee	Santa Barbara, CA
Montecito Assn	Santa Barbara, CA
Montecito Planning Commission	Santa Barbara, CA
Santa Barbara Waterfront Division	Santa Barbara, CA
City of Goleta	Goleta, CA
Santa Barbara Parks	Santa Barbara, CA
Central Coast Water Authority	Buellton, CA
Environmental Defense Center	Santa Barbara, CA
Janet Martorana	Santa Barbara, CA
Metropolitan Transit District	Santa Barbara, CA

Table 7.2 Project Stakeholders	
Name	**Location**
S.B. Co Air Poll Cont Dist	Santa Barbara, CA
S.B. Unified School Districts	Santa Barbara, CA
SBCAG	Santa Barbara, CA
Surfrider Foundation	Santa Barbara, CA
Caltrans District 5	San Luis Obispo, CA
Central Coast Regional	San Luis Obispo, CA
David Matson, Deputy Director	Santa Barbara, CA
Goleta Water District	Goleta, CA
Carpinteria Valley Water District	Carpinteria, CA
Montecito Water District	Santa Barbara, CA
Union Pacific Railroad	Omaha, NE
US Fish and Wildlife Service	Ventura, CA
Union Pacific Railroad	Roseville, CA
Santa Barbara Channelkeeper	Santa Barbara, CA
Heal the Ocean	Santa Barbara, CA
Sweetwater Collaborative	Santa Barbara, CA
Phil Walker	Santa Barbara, CA
Robert H Sulnick	Santa Barbara, CA

7.3.1 Regulators

Regulators that have been identified as project stakeholders are presented in Table 7.3. This list is not final, and may be expanded as the project develops.

7.4 Documentation Requirements

A list of the documents that will be developed as part of the technical advisory process is presented in Table 7.4. These documents will be made available via NWRI's project website at times indicated.

Table 7.3 Key Regulators

Agency	Name	Office	Phone	Email
Division of Drinking Water	Jeff Densmore	Carpinteria	(805) 566-1326	jeff.densmore@waterboards.ca.gov
Division of Drinking Water	Kurt Souza	Carpinteria	(805) 566-4745	kurt.souza@waterboards.ca.gov
Central Coast RWQCB	Peter von Langen	San Luis Obispo	(805) 549-3688	peter.vonlangen@waterboards.ca.gov
California Coastal Commission	Tom Luster	San Francisco	(415) 904-5400	tluster@coastal.ca.gov

Table 7.4 Technical Advisory Process Documents and Publication Procedures	
Document Title	**Publication Procedure**
Draft Work Plan	• Provided to NWRI 15 working days prior to TAP Workshop 1. • Posted to NWRI website at least 5 days prior to TAP Workshop 1.
TAP Workshop 1 Agenda	• Distributed to stakeholder list 15 days prior to TAP Workshop 1. • Posted to NWRI website at least 5 days prior to TAP Workshop 1.
TAP Workshop 1 Meeting Minutes	• Draft provided to project team and TAP panel for review 10 days following TAP Workshop 1. Will include all stakeholder comments received at the workshop or by correspondence to NWRI. • Posted to NWRI website within 30 days following TAP Workshop 1.
Tech Memo 2 (Basis of Design & Initial Screening Analysis) – Subsurface Tech Memo 3 (Permit & Regulatory Req.) – Subsurface Draft groundwater modeling report – Subsurface Draft tsunami hazard, sea level rise & sediment transport report	• Provided to NWRI 15 working days prior to TAP Workshop 2. • Posted to NWRI website at least 5 days prior to TAP Workshop 2.
TAP Workshop 2 Agenda	• Distributed to stakeholder list 15 days prior to TAP Workshop 2. • Posted to NWRI website at least 5 days prior to TAP Workshop 2.
TAP Workshop 2 Meeting Minutes	• Draft provided to project team and TAP panel for review 10 days following TAP Workshop 2. Will include all stakeholder comments received at the workshop or by correspondence to NWRI. • Posted to NWRI website within 30 days following TAP Workshop 2.
Draft subsurface sampling report Tech Memo 4 (Conceptual Design) Tech Memo 5 (Estimated Schedule & Cost) Tech Memo 6 (Feasibility Analysis)	• Provided to NWRI 15 working days prior to TAP Workshop 4. • Posted to NWRI website at least 5 days prior to TAP Workshop 4.

Table 7.4 Technical Advisory Process Documents and Publication Procedures	
Document Title	**Publication Procedure**
TAP Workshop 4 Agenda	• Distributed to stakeholder list 15 days prior to TAP Workshop 4. • Posted to NWRI website at least 5 days prior to TAP Workshop 4.
TAP Workshop 4 Meeting Minutes	• Draft provided to project team and TAP panel for review 10 days following TAP Workshop 4. Will include all stakeholder comments received at the workshop or by correspondence to NWRI. • Posted to NWRI website within 30 days following TAP Workshop 4.

APPENDIX A – WORK AUTHORIZATION 1
SCOPE OF SERVICES

SANTA BARBARA CITY AGREEMENT NO. 25,191

With

Carollo Engineers, Inc., for a Work Plan for Subsurface Desalination Intake and Potable Reuse Feasibility Studies

This Contract is entered into on May 5, 2015, by and between:

The City of Santa Barbara, a Municipal Corporation, referred to herein as the "City,"

and,

Carollo Engineers, Inc., a California Corporation, referred to herein as the "**Contractor,**"

WITNESSETH:

WHEREAS, Contractor has the special background, training and experience required by City, and in consideration of the mutual covenants, conditions, promises and agreements, herein, the City and Contractor **AGREE**:

1. SCOPE OF CONTRACTOR SERVICES

a. Contractor agrees to provide a Work Plan for Subsurface Desalination Intake and Potable Reuse Feasibility Studies as described in more detail in the attached scope of services (Exhibit A) dated April 20, 2015.

On January 30, 2015, the Regional Water Quality Control Board adopted an amendment to the City's El Estero Wastewater Treatment Plant water discharge requirements that included a special provision for the City to "analyze the feasibility of a range of alternatives, including subsurface intake and potable reuse options" and "submit a feasibility study work plan for the Regional Water Board, by August 31, 2015."

b. The City has been advised and enters into this Contract understanding that Tom Seacord has been designated the project manager for provide Subsurface Desalination Intake and Potable Reuse Feasibility Studies and that the Project Manager will have direct responsibility for interacting with City staff and delivering Contractor's services to the City under this Contract. Contractor shall not substitute nor otherwise allow any other person to serve in place of the Project Manager without the written consent of the Department Head, who shall have sole discretion as to whether the proposed substitution is acceptable. Should Contractor substitute or allow any unauthorized person to serve as project manager, Contractor shall have no right to any monies for services provided by that unauthorized person and City shall also have the right to immediately terminate this Contract.

2. COMPENSATION

a. The total compensation for all services provided pursuant to this Contract, including all extra services as defined in Section 3 hereof and reimbursable expenses,

shall not exceed the sum of **$343,925** without the express written approval of City Council of the City of Santa Barbara. The basic contract is for **$312,659** and the total that may be claimed for Extra Services under Section 3 of this Contract shall not exceed **$31,266.** This Contract provides the exclusive means of payment and reimbursement for costs to Contractor by the City.

b. Changes in personnel or in rates of compensation set forth in Exhibit A may be made only after written notice to and written approval by the Department Head, **Rebecca J. Bjork**, ("Department Head").

c. Where travel costs are included in Exhibit A, only the actual travel costs (at fare, rate per mile or lump sum approved), and/or actual expenses pursuant to the provisions of the Contract and within guidelines approved by the City Finance Director will be reimbursed.

d. Contractor may be reimbursed for such other necessary costs, including actual costs of copies, printing, postage, shipping and documents expense, and all costs of other materials, equipment, services and supplies, as approved and required to complete the work, according to the attached Exhibit A.

e. Compensation for Extra Services of Contractor authorized in accordance with Section 2 shall be paid to Contractor by City in accordance with the fee schedule set forth in Exhibit A. Contractor shall only be entitled to payment for Extra Services under this Contract if Contractor has obtained authorization required under Section 3 below.

f. Contractor shall submit itemized statements, which shall include a detailing of the number of hours spent on each task and copies of all subcontractors' invoices, to request payment in accordance with the standard billing format issued by the City Department. Contractor shall keep records concerning payment items on a generally recognized accounting basis and such records shall be maintained for a period of 3 years following the completion of the work assigned. Such records shall be made available for copying, inspection or audit by City employees or independent agents during reasonable business hours.

3. EXTRA SERVICES OF CONTRACTOR

Prior to performing any services other than those described in Exhibit A ("Extra Services"), Contractor shall submit a written request for Extra Services and obtain the written approval of the Department Head or his/her designee. The request for Extra Services shall at minimum include a description of the services to be performed, the reason why the Extra Services are needed or required, a schedule for completion of the proposed Extra Services, and a not-to-exceed amount for performance of the proposed Extra Services. Each approved Extra Services request shall be billed separately.

4. TIME OF BEGINNING AND COMPLETION

Services shall begin upon full execution of this Contract by the City, and delivery of a fully executed copy of the Contract to the Contractor. Contractor shall adhere to schedules and deadlines agreed to by City and Contractor shown in Exhibit A. The Contractor shall submit to the City a feasibility study work plan to "analyze the feasibility of a range of alternatives, including subsurface intake and potable reuse options" for review by the Regional Water Quality Control Board (RWB), by August 31, 2015. Contractor's failure to complete the above services within the time specified, due to avoidable delays, may at the City's discretion be considered a material breach of this Contract. Contractor shall review the remaining work and schedule of performance at least monthly and shall confirm that completion may be expected within the schedule

approved, or in the alternative, give immediate notice when it shall first appear that the approved schedule will not be sufficient, together with an explanation for any projected insufficiency of delays in the schedule. No extension of time to complete any portion of the services called for in the Contract shall be allowed except upon the express, written approval of the Department Head. Contractor shall request, in writing, a time extension for approval by City, promptly upon the occurrence of any action causing delay in Contractor's prosecution of the services. The nature of the delay, the corrective actions taken and the impacts on the project schedule shall be described in each request for a time extension.

5. OWNERSHIP OF DOCUMENTS

All documents, computer programs, plans, renderings, charts, designs, drafts, surveys and other intellectual property which is originally developed by Contractor pursuant to this Contract shall become the property of City upon full and complete compensation to Contractor for services performed herein. Contractor will take such steps as are necessary to perfect or to protect the ownership interest of the City in such property. Contractor may retain copies of said documents for Contractor's file.

6. ASSIGNMENT OF CONTRACT

Contractor shall not assign, sublet or transfer any right, privilege or interest in this Contract, or any part thereof, without prior written consent of City. Contractor shall not substitute personnel designated in the proposal of Contractor without the written consent of City.

7. OFFICIAL NOTICES

Notices to either party shall be provided by personal delivery or by depositing them in the United States mail, first class postage prepaid, and addressed as identified at the signature page of this Contract. A party may change mailing address for all purposes under this Contract, by written notice.

8. DEFENSE, INDEMNITY AND HOLD HARMLESS

a. Contractor shall, to the extent permitted by law, investigate, defend, indemnify and hold harmless the City, its officers, employees and agents from and against any and all loss, damage, liability, claims, demands, detriments, costs, charges and expenses (including reasonable attorney fees) and causes of action of whatsoever character which the City may incur, sustain or be subjected to on account of loss or damage to property or loss of use thereof, or for bodily injury to or death of any persons (including but not limited to property, employees, subcontractors, agents and invitees of each party hereto) arising out of or in any way connected with the work to be performed under this Agreement other than as such work relates to Professional Liability Insurance.

b. With respect to Professional Liability Insurance, Contractor shall investigate, defend, indemnify and hold harmless the City, its officers, agents and employees from and against any and all loss, damage, liability, claims, demands, detriments, costs, charges and expenses (including reasonable attorney's fees) and causes of action of whatsoever character which City may incur, sustain or be subjected to

on account of loss or damage to property or loss of use thereof, or for bodily injury to or death of any persons (including but not limited to property, employees, subcontractors, agents and invitees of each party hereto) arising out of or due to the acts, errors or omissions of Contractor.

9. INSURANCE REQUIREMENTS

As part of the consideration of this Agreement, Consultant agrees to purchase and maintain at its sole cost and expense during the life of this agreement insurance coverage against claims for injuries to persons or damages to property which may arise from or in connection with the performance of the work hereunder by the Consultant, its agents, representatives, or employees.

MINIMUM SCOPE AND LIMIT OF INSURANCE

Coverage shall be at least as broad as:

A. **Commercial General Liability (CGL): Insurance Services Office Form CG 00 01 covering CGL on an "occurrence" basis, including products and completed operations with limits of no less than Two Million Dollars ($2,000,000) per occurrence for bodily injury, personal injury and property damage. If a general aggregate limit applies, either the aggregate limit shall apply separately to this project or the general aggregate limit shall be twice the required occurrence limit.**

B. **Automobile Liability: Insurance Services Office Form Number CA 0001 covering Code 1 (any auto), or if Consultant has no owned autos, Code 8 (hired) and Code 9 (non-owned), with limits of no less than One Million Dollars ($1,000,000) per accident for bodily injury and property damage.**

C. **Workers' Compensation: In accordance with the provisions of the California Labor Code, Consultant is required to be insured against liability for Workers' Compensation or to undertake self-insurance. Statutory Workers' Compensation and Employers' Liability of at least $1,000,000 shall cover all Consultant's staff while performing any work incidental to the performance or this agreement.**

D. **Professional Liability: Professional Liability (Errors and Omission) Insurance appropriate to the Consultant's profession, with limit no less than One Million Dollars ($1,000,000) per occurrence or claim and Two Million Dollars ($2,000,000) aggregate to cover all services rendered by the Consultant pursuant to this Agreement.**

If the Consultant maintains higher coverage limits than the amounts shown above, then the City requires and shall be entitled to coverage for the higher coverage limits maintained by the Consultant. Any available insurance proceeds in excess of the specified minimum limits of insurance and coverage shall be available to the City.

OTHER INSURANCE PROVISIONS

Each insurance policy shall contain, or be endorsed to contain, the following five (5) provisions:

1) Additional Insured Status

The City of Santa Barbara, its officers, employees, and agents, shall be covered as additional insureds on the Commercial General Liability and the Automobile Liability policy with respect to liability arising out of work or operations performed by or on behalf of the Consultant including materials, parts, or equipment furnished in connection with such work or operations and automobiles owned, leased, hired, or borrowed by or on behalf of the Consultant. Additional Insured coverage shall be provided in the form of an endorsement to the Consultant's insurance (at least as broad as Insurance Services Office Form CG 20 10 11 85). A copy of the endorsement evidencing that the City of Santa Barbara has been added as an additional insured on the policy, must be attached to the certificate of insurance.

2) Subcontractors

Consultant shall require and verify that all subcontractors maintain insurance meeting all the requirements stated herein, and Consultant shall ensure that the City is an additional insured on insurance required from subcontractors. For Commercial General Liability coverage subcontractors shall provide coverage with a format at least as broad as Insurance Services Office form CG 20 38 04 13.

3) Notice of Cancellation

A provision that coverage will not be cancelled or subject to reduction without written notice given to the City Clerk, addressed to P.O. Box 1990, Santa Barbara, California 93102-1990.

4) Primary Coverage

For any claims related to this contract, the Consultant's insurance coverage shall be primary insurance as respects the City, its officers, officials, employees, and volunteers. Any insurance or self-insurance maintained by the City shall be excess of the Consultant's insurance and shall not contribute with it.

5) Waiver of Subrogation

Consultant hereby agrees to waive rights of subrogation which any insurer of Consultant may acquire from Consultant by virtue of the payment of any loss. Consultant agrees to obtain any endorsement that may be necessary to affect this waiver of subrogation. Consultant agrees to obtain any endorsement that may be necessary to affect this waiver of subrogation, but this provision applies regardless of whether or not the City has received a waiver of subrogation endorsement from the insurer.

The Workers' Compensation policy shall be endorsed with a waiver of subrogation in favor of the City for all work performed by the Consultant, its employees, agents and subcontractors.

ACCEPTABILITY OF INSURERS

All insurance coverage shall be placed with insurers that have a current rating from AM Best of no less than A: VII; and are admitted insurance companies in the State of California. All other insurers require prior approval of the City.

CLAIMS MADE POLICIES

If the required Professional Liability (Errors and Omissions) policy provides coverage on a claims-made basis:

1. **The Retroactive Date must be shown and must be before the date of the contract or the beginning of contract work.**
2. **Insurance must be maintained and evidence of insurance must be provided for at least five (5) years after completion of the contract of work.**
3. **If coverage is canceled or non-renewed, and not replaced with another claims-made policy form with a Retroactive Date prior to the contract effective date, the Consultant must purchase "extended reporting" coverage for a minimum of five (5) years after completion of contract work.**

COVERAGE LIMITS SPECIFICATIONS

Approval of the insurance by City or acceptance of the certificate of insurance by City shall not relieve or decrease the extent to which the Consultant may be held responsible for payment of damages resulting from Consultant's services or operation pursuant to this Agreement, nor shall it be deemed a waiver of City's rights to insurance coverage hereunder.

If, for any reason, Consultant fails to maintain insurance coverage which is required pursuant to this Agreement, the same shall be deemed a material breach of contract. City, at its sole option, may terminate this Agreement and obtain damages from the Consultant resulting from said breach. Alternately, City may purchase such required insurance coverage, and without further notice to Consultant, City may deduct from sums due to Consultant any premium costs advanced by City for such insurance.

DEDUCTIBLES AND SELF-INSURED RETENTIONS

Any deductibles or self-insured retentions must be declared to and approved by the City. At the option of the City, either: the Consultant shall cause the insurer to reduce or eliminate such deductibles or self-insured retentions as respects the City, its officers, officials, employees, and volunteers; or the Consultant shall provide a financial guarantee satisfactory to the City guaranteeing payment of losses and related investigations, claim administration, and defense expenses.

EVIDENCE OF COVERAGE

Consultant must provide evidence that it has secured the required insurance coverage before execution of this agreement. A Certificate of Insurance supplied by the City or the appropriate ACORD and Insurance Services Office forms evidencing the above shall be completed by Consultant's insurer or its agent and submitted to the City prior to execution of this Agreement by the City.

Consultant shall furnish the City with original certificates and amendatory endorsements or copies of the applicable policy language effecting coverage required by this clause. All certificates and endorsements are to be received and approved by the City before work commences. However, failure to obtain the required documents prior to the work beginning shall not waive the Consultant's obligation to provide them. The City reserves the right to require complete, certified copies of all required insurance policies, including endorsements required by these specifications, at any time.

10. TERMINATION

This Contract may be terminated with or without cause by either party at any time by giving the other no less than thirty (30) days notice in writing. In the event of such termination, Contractor shall deliver all programs, drawings, surveys, drafts, plans, work in progress and other documents related to the project to the City within five (5) days of the notice of termination. In the event of such termination, Contractor shall be compensated for such services as are performed and work product delivered to the City up to the point of termination.

11. RIGHT TO PERFORM SIMILAR SERVICES

Nothing in this Contract shall restrict the City from providing the same or similar services through City employees, other contractors, other resources, or by arrangements with other agencies. Contractor may engage in similar activities to the extent that such work does not conflict with the proper performance of services under this Contract.

12. CONFLICT OF INTERESTS

Contractor warrants by execution of this Contract that no person or selling agent has been employed or retained to solicit or secure this Contract upon an agreement or understanding for commission, percentage, brokerage or contingent fee, and that Contractor maintains no agreement, employment, or position which would be in conflict with the duties to be performed for City under this Contract. Contractor further agrees that during the term of this Contract, Contractor will not obtain, engage in, or undertake any interests, obligations or duty that would be in conflict with, or interfere with, the services or duties to be performed under the provisions of this Contract.

13. ADMINISTRATION OF EMPLOYMENT

Contractor shall obtain and administer the employment of personnel having the background, training, experience, licenses and registration necessary for the work assigned, including all coordination, the withholding of proper taxes and benefits, the

payment of wages, employer's contributions for FICA, and Federal and State unemployment payments, and the review and maintenance of any necessary licenses, certificates, memberships and other qualifications necessary for the services to be provided. Contractor is an independent contractor and shall not be considered an agent or employee of the City for any purpose. Contractor and its employees and agents are not entitled to any of the benefits or privileges that the City provides its employees.

14. BUSINESS TAX CERTIFICATE

Prior to the execution of the Contract, Contractor shall obtain a business tax certificate from the City at Contractor's expense. Contractor shall maintain a business tax certificate as required by the City Finance Director during the term of this Contract.

15. NO WAIVER OF PROVISIONS

No waiver of a breach of any provision of this Contract shall be construed to be a continuing waiver of that provision, nor a waiver of any breach of another provision of this Contract.

16. APPLICABLE LAWS, PARTIAL INVALIDITY

This Contract shall be subject to the Santa Barbara City Charter, and the laws, rules, regulations and ordinances in effect within the City of Santa Barbara, County of Santa Barbara, California, and any interpretation of the law that may be necessary shall be pursuant to the laws applicable within that jurisdiction. If any provision of this Contract is determined to be invalid, illegal or unenforceable for any reason, that provision shall be deleted from this Contract and such deletion shall in no way affect, impair, or invalidate any other provision of this Contract, unless it was material to the consideration for the performance required. If a provision is deleted which is not material to such consideration, the remaining provisions shall be given the force and effect originally intended.

17. NON-DISCRIMINATION ORDINANCE

Contractor shall perform all work pursuant to this Contract in compliance with Section 9.126.020 of the Santa Barbara Municipal Code (a copy of which is attached as Exhibit B), prohibiting unlawful discrimination in employment practices, and shall be bound by the terms of such ordinance.

18. CITY SERVICE CONTRACTOR MANDATORY MINIMUM WAGE

a. Chapter 9.128 of the Santa Barbara Municipal Code establishes a mandatory minimum wage for employees of contractors providing services to the City. In the performance of this Agreement, Contractor and any subcontractor, agent, or assignee of Contractor under this Agreement shall comply with the provisions of Chapter 9.128 of the Municipal Code as such Chapter existed upon the adoption of this Agreement or the last date this Agreement was amended.

b. Current Living Wage Certificates on forms supplied by the City shall be completed by Contractor, submitted to City prior to execution of this Contract by City, and attached as Exhibit C. Contractor shall require any and all subcontractors and all tiers of

such subcontractors to provide Living Wage Certificates as required by Santa Barbara Municipal Code Chapter 9.128.

19. NONAPPROPRIATIONS OF FUNDS

Notwithstanding any other provision of this Agreement, in the event that no funds or insufficient funds are appropriated or budgeted by the City, or funds are not otherwise available for payments in the fiscal year(s) covered by the term of this Agreement, then City will notify Contractor of such occurrence and City may terminate or suspend this Agreement in whole or in part, with or without a prior notice period. Subsequent to termination of this Agreement under this provision, City shall have no obligation to make payments with regard to the remainder of the term.

IN WITNESS WHEREOF, the parties have executed this contract as of the date and year first written above.

CITY OF SANTA BARBARA
A Municipal Corporation

Rebecca Bjork
Public Works Director

ATTEST:

Gwen Peirce, CMC
City Clerk Services Manager

APPROVED AS TO CONTENT:

Joshua Haggmark
Water Resources Manager

APPROVED AS TO FORM:
Ariel Pierre Calonne
City Attorney

By

Business Tax Compliance:
Certificate No. 32030

By

Approved as to Insurance:

Mark Howard
Risk Manager

CONTRACTOR:
Carollo Engineers, Inc.

Signature

Jim Meyerhofer
Type or Print Name

Sr. Vice President
Title

5075 Shoreham Pl. Suite 120
Address

San Diego	CA	92122
City	State	Zip

858-505-1020
Telephone Number

EXHIBIT A

Scope of Services

SCOPE OF SERVICES

PRELIMINARY DESIGN SERVICES FOR
RECOMMISSIONING THE CITY'S DESALINATION PLANT
(Subsurface Desalination Intake and Potable Reuse Feasibility Studies).
(Project)

AUTHORIZATION #1: WORK PLAN DEVELOPMENT

BACKGROUND

On September 23, 2014 the City of Santa Barbara City Council directed Public Works Department staff to report back on a plan to evaluate the feasibility of subsurface desalination intake and potable reuse, including indirect and direct potable reuse options. The direction given by CITY Council was to report back on a plan for this evaluation following award of the desalination plant contract in April 2015. Furthermore, on January 30, 2015, the Central Coast Regional Water Quality Control Board (RWQCB) adopted an amendment to the CITY's El Estero Wastewater Treatment Plant (WWTP) Waste Discharge Requirements (WDR) that included a condition that the CITY should report back to the RWQCB by August of 2015 with a Work Plan for these studies that will have the work completed by June 2017. This scope of services was therefore developed to satisfy the direction of City Council and the RWQCB by preparing a feasibility analysis for subsurface desalination intakes and potable reuse alternatives. This information can be used as part of future planning efforts designed to help the CITY plan for future drought emergencies.

PURPOSE

The purpose of this scope of services is to present the initial tasks required for evaluating the feasibility of subsurface desalination plant intakes and potable reuse alternatives. The feasibility study work product will be developed in a manner so as to accomplish the following objectives:

- Satisfy the requirements of the CITY's amended Waste Discharge Requirements for the El Estero WWTP.
- Support a future updates to the CITY's Long Term Water Supply Plan.

This scope of services is the first of three authorizations required to complete this study effort. The three separate authorizations include:

- Authorization #1: Work plan development and literature review.
- Authorization #2: Subsurface desalination intake basis of design and fatal flaw analysis; Potable Reuse Feasibility Study.
- Authorization #3: Subsurface desalination intake feasibility study.

The steps involved in this study are described graphically in the programmatic work flow diagrams presented in Figures 1 and 2.

CAROLLO'S SERVICES

TASK 1 – SUBSURFACE DESALINATION INTAKE FEASIBILITY STUDY
The tasks required to perform the services associated with ***TASK 1 – Subsurface Desalination Intake Feasibility Study*** are presented below and summarized graphically in the attached Programmatic Work Plan.

1.1 – Work Plan Development
The CITY of Santa Barbara is required to submit a Work Plan for evaluating subsurface desalination intakes to the Central Coast Regional Water Quality Control Board (RWQCB) by August 2015. As part of CAROLLO's services included in this Scope of Work, CAROLLO will conduct a kickoff meeting with the CITY to develop a Work Plan that has the following objectives:

- Establish the project schedule.
- Establish technical advisory panel role, procedures and objectives.
  - It is anticipated that the technical advisory process will be facilitated by the National Water Research Institute (NWRI). The technical advisors will include a panel of approximately four (4) experts (chosen and retained by NWRI).
  - It is anticipated that up to three (3) technical advisory workshops will be included in the Work Plan. These workshops will occur throughout the project at points that coincide with project work product development and completion. The points at which these technical advisory workshops occur will be established in the Work Plan.
  - It is anticipated that the panel will review and advise on technical studies and conclusions of CAROLLO and CITY.
  - It is anticipated that public comments will be facilitated by NWRI as part of the Technical Advisory Panel meetings. Applicable public comments/sentiment (i.e., consistent with regulatory framework) will be incorporated thereafter into the feasibility screening analysis.
- Establish the role of outside agencies (e.g., RWQCB, California Coastal Commission, etc.) and City residents.
- Establish the methods by which the design basis will be established. Design basis includes:
  - Intake capacity
  - Candidate sites for study
- Establish the types of subsurface intakes that will be studied (e.g., vertical wells, lateral beach wells, horizontal collector wells (i.e., Ranney wells), slant wells, subsurface infiltration galleries (SIG), and horizontally directionally drilled (HDD) wells (i.e., Neodren))
- Establish and define fatal flaws that may limit further consideration of project sites, which may include: available land, known geologic hazards/conditions (e.g., proximity of faults, depth to bedrock, transmissivity of soils, etc.), proximity to marshes that may be affected by intake use, anticipated loss of facilities due to erosion, etc.
- Establish and define feasibility screening criteria (e.g., constructability, permitability, impact to the CITY's drinking water aquifers, estimated environmental impacts during construction, impact to rate payers, etc.).
- Establish sequencing of analyses and application of feasibility screening criteria.
- Establish procedure to identify sites for subsurface intakes and raw water conveyance piping
- Procedure to determine subsurface properties (if applicable). Examples include:

  - Review literature data to establish sites to focus study.
  - Collect new data:
    - Identify permits required and establish application procedure
    - Sequence of subsurface data collection.
- Establish procedure to model subsurface intake's influence on the sustainability of the CITY's drinking water aquifer.
- Establish procedure to estimate subsurface intake water quality and additional treatment needs.
- Establish and define metrics to compare subsurface intake alternatives to the CITY's current open ocean intake. These metrics may include:
  - Reliability of intake water supply
  - Benefit to treatment process costs
  - Impact to CITY's groundwater supply
  - Construction phase environmental impact (monetized)
  - Operation phase treatment impacts (monetized)
  - Impact to rate payers
- Establish scoring methodology to use.

To maximize the Work Plan development / kickoff meeting's potential, CAROLLO will distribute kickoff meeting agenda, which will include some assignments for CITY staff to consider before the meeting date (e.g., possible feasibility screening criteria and their definitions, site alternatives for facilities and conveyance of raw water, etc.). Following the kickoff meeting, CAROLLO will:

- Prepare meeting minutes to identify action items and data needs.
- Develop a draft Work Plan that will be submitted to the Central Coast RWQCB staff for review.
  - Upon receipt of the RWQCB's comments, CAROLLO will prepare a final Work Plan.

Following acceptance of the Work Plan by the RWQCB, in concert with the data collected as part of Task 1.2 (Literature Review), CAROLLO will prepare a draft Technical Memorandum (TM) that will be used as part of the technical advisory process (TM 1 (Revision 0)) and summarizes the pertinent background information, definitions for feasibility screening criteria that were established, and the subsurface desalination intake alternatives that will be evaluated. The CITY will review this TM and provide comments back to CAROLLO within 1 week following submission of the draft TM. CAROLLO will incorporate any comments into a revised TM (TM 1 (Revision 1)) that will be used as supporting material for the technical advisory process discussed in Task 4.1 (Technical Advisory Process). After the technical advisory process is complete, the final TM contents will be used as chapters in the Desalination Subsurface Intake Feasibility Report.

Task 1.1 Deliverables by CAROLLO

1. Kickoff Meeting Agenda and Assignments
2. Kickoff Meeting Minutes
3. Draft Work Plan
4. Final Work Plan
5. TM 1 (Revision 0): Introduction, Background and Project Alternatives
6. TM 1 (Revision 1): Introduction, Background and Project Alternatives

1.2 – Literature Review

CAROLLO will collect, review and prepare data to evaluate the subsurface intake alternatives identified. CAROLLO will prepare a formal list of data needs that are either independently collected or requested from the CITY. CAROLLO will provide weekly data gathering list updates during the first month of the project and monthly updates thereafter. It is anticipated that the list will include items such as, but not limited to:

- Published geologic/hydrogeologic studies in the area, including:
  - USGS reports
  - Prior hydrologic and geotechnical studies conducted by the CITY.
- The CITY's 1989 and 1990 subsurface intake studies conducted on East, West and Ledbetter Beaches.
- Geotechnical data associated with the design and installation of piles supporting Stearns Warf.
- Any data related to tsunami hazards, sea level rise and sediment transport (i.e., erosion and deposition) in the areas of East, West and Ledbetter Beaches that may be associated with harbor dredging and mooring. The CITY Waterfront Department may be consulted for this information.
- Basis of design reports for the CITY's desalination plant (after reactivation and associated improvements).
- Water, sewer, (existing) recycled water and stormwater atlas data in GIS format
- Current or anticipated flood plain maps
- Hydrologic data and studies on existing wells and the groundwater aquifer used for drinking water production, including various USGS hydrogeological and modelling studies..
- Data related to baseline environmental conditions that could be affected by one or more of the intake options.
- California State Waters Map Series—Offshore of Santa Barbara, California http://pubs.usgs.gov/sim/3281/

CAROLLO will use the CITY's Map Analysis and Printing System (http://gismaps.santabarbaraca.gov/) to facilitate development of scaled site plans and graphics.

Task 1.2 Deliverables by CAROLLO

1. Data collection lists and updates

Summary of TASK 1 Project Meetings

Task No.	Meeting Name	Purpose
1.1	Kickoff Meeting	CAROLLO and CITY will meet to: • Discuss CAROLLO's proposal for Work Plan content • Identify candidate project sites for initial feasibility screening ***(This meeting will be held at the same time as the kickoff meeting in Task 3.1)***
	Draft Work Plan Meeting	CAROLLO and CITY will meet to review draft work plan before it is submitted to the RWQCB and Technical Advisory Panel. ***(This meeting will be held at the same time as the Draft Work Plan Meeting in Task 3.1)***

TASK 3 – POTABLE REUSE FEASIBILITY STUDY

The tasks required to perform the services associated with ***TASK 3 – Potable Reuse Feasibility Study*** are presented below and summarized graphically in the attached Programmatic Work Plan.

3.1 – Work Plan Development

The CITY of Santa Barbara is required to submit a Work Plan for evaluating potable reuse alternatives to the Central Coast Regional Water Quality Control Board (RWQCB) by August 2015. As part of CAROLLO's services included in this Scope of Work, CAROLLO will conduct a kickoff meeting with the CITY to develop a Work Plan that has the following objectives:

- Establish the project schedule.
- Establish technical advisory panel role, procedures and objectives.
  - It is anticipated that the technical advisory process will be facilitated by the National Water Research Institute (NWRI). The technical advisory panel will include a panel of approximately four (4) experts (chosen and retained by NWRI) as well as representatives from various regulatory agencies as determined appropriate.
  - It is anticipated that up to three (3) technical advisory panel workshops will be included in the Work Plan. These workshops will occur throughout the project at points that coincide with project work product development and completion. The points at which these technical advisory panel workshops occur will be established in the Work Plan.
  - It is anticipated that the panel will review and advise on technical studies and conclusions of CAROLLO and CITY.
  - It is anticipated that public comments will be facilitated by NWRI as part of the Technical Advisory Panel meetings. Public comments/sentiment will be incorporated thereafter into the feasibility screening analysis.
- Establish the role of outside agencies (e.g., RWQCB, California Coastal Commission, etc.) and City residents.
- Establish and define fatal flaws that may limit further consideration of project sites, which may include: poor aquifer tranmissivity, known geologic hazards, small site, etc.
- Establish and define feasibility screening criteria (e.g., estimated environmental impacts, permitability, improves the reliability of the CITY's water supply, impact to rate payers, potable water quality benefits, etc.).
- Establish sequencing of analyses and application of feasibility screening criteria.
- Procedure to identify the capacity of potable reuse supply that is available.
- Identify possible sites for treatment, storage and distribution facilities to evaluate when considering both Direct Potable Reuse (DPR) and Indirect Potable Reuse (IPR) alternatives.
  - It is anticipated that up to twenty (20) site/process/routing/size alternatives may be considered for further analysis against the feasibility screening criteria identified. Possible treatment facility location options may include (but may not be limited to):
    - 401 E. Yanonali Street (i.e., City Corporation Yard, APN #017-540-006), and
    - 103 S. Calle Cesar Chavez (APN #017-113-020)
    - Repurposing the Charles Meyer Desalination Plant located at 525 E. Yanonali Street

    Possible indirect potable recharge locations may include (but may not be limited to):
    - Recharge wells in the foothills basin (near Route 154 and Highway 101)
    - Recharge wells in groundwater basin referred to as "Unit 1" (north of Highway 101)

    - Infiltration of water (i.e., like a spreading basin) in Mission Creek before Oak Park.

    Possible potable reuse options may include (but may not be limited to):
    - Discharge of advanced treated wastewater into Lauro Canyon Reservoir (a.k.a., raw water production).
    - Dilution and off-setting the intake volume of seawater flowing to the Charles Meyer Desalination Plant
- Establish and define metrics to compare potable reuse alternatives to the CITY's current drought plan (i.e., desalination).
- Establish scoring methodology to use

To maximize the Work Plan development / kickoff meeting's potential, CAROLLO will distribute kickoff meeting agenda, which will include some assignments for CITY staff to consider before the meeting date (e.g., possible feasibility screening criteria and their definitions, site alternatives for facilities and distribution of treated water, etc.). Following the kickoff meeting, CAROLLO will:

- Prepare meeting minutes to identify action items and data needs.
- Develop a draft Work Plan that will be submitted to the Central Coast RWQCB staff for review.
    - Upon receipt of the RWQCB's comments, CAROLLO will prepare a final Work Plan.

Following acceptance of the Work Plan by the RWQCB, in concert with the data collected as part of Task 3.2 (Data Gathering), CAROLLO will prepare a draft Technical Memorandum (TM) that will be used as part of the technical advisory process (TM 1 (Revision 0)) and summarizes the pertinent background information, definitions for feasibility screening criteria that were established, and the IPR/DPR alternatives that will be evaluated. The CITY will review this TM and provide comments back to CAROLLO within 1 week following submission of the draft TM. CAROLLO will incorporate any comments into a revised TM (TM 1 (Revision 1)) that will be used as supporting material for the technical advisory process discussed in Task 4.1 (Technical Advisory Process). After the technical advisory process is complete, the final TM contents will be used as chapters in the Potable Reuse Feasibility Report.

Task 3.1 Deliverables by CAROLLO

1. Kickoff Meeting Agenda and Assignments
2. Kickoff Meeting Minutes
3. Draft Work Plan
4. Final Work Plan
5. TM 1 (Revision 0): Introduction, Background and Project Alternatives
6. TM 1 (Revision 1): Introduction, Background and Project Alternatives

3.2 – Data Gathering

CAROLLO will collect, review and prepare data to evaluate the study alternatives identified. CAROLLO will prepare a formal list of data requested and provide weekly data gathering list updates during the first month of the project and monthly updates thereafter. It is anticipated that the list will include items such as, but not limited to:

- Basis of design reports for the CITY's recycled water treatment system, which should include:
    - Historical effluent flow data (hourly flows over previous 10 years, including drought periods)

  - Existing and projected recycled water demands
  - Recycled water quality, including regulated primary and secondary drinking water standards, non-regulated treatment goals, and irrigation water standards (e.g., boron and sodium adsorption ratio).
- Water, sewer, (existing) recycled water and stormwater atlas data in GIS format
- Current or anticipated flood plain maps
- Hydrologic data, aquifer characteristics (thickness, orientation, extent, degree of confinement), estimates of aquifer properties (e.g., T, S), confining layer extent and properties, production well pumping rates, water quality, well logs, and studies on existing wells and the groundwater aquifer used for drinking water production.
- Location and nature of all existing wells in the study area including well logs, geophysical logs, water quality data, water level data, and yield data.
- Soil infiltration rates (for use in estimating infiltration basin capacities)
- Hydrologic data and studies on existing wells and the groundwater aquifer used for drinking water production (e.g., USGS Report).
- Regional (County-wide) reports on potable reuse opportunities that describe:
  - Regional groundwater balance that addresses sustainable yield of groundwater aquifers that are shared by more than one agency.
  - Aquifer adjudication between the City and neighboring agencies.
  - How excess groundwater use by one agency may be balanced by IPR.

CAROLLO will use the CITY's Map Analysis and Printing System (http://gismaps.santabarbaraca.gov/) to facilitate development of scaled site plans and graphics.

Task 3.2 Deliverables by CAROLLO

1. Data collection lists and updates

Summary of TASK 3 Project Meetings

Task No.	Meeting Name	Purpose
3.1	Kickoff Meeting	CAROLLO and CITY will meet to: • Discuss CAROLLO's proposals for Work Plan content • Identify candidate project sites for initial feasibility screening ***(This meeting will be held at the same time as the kickoff meeting in Task 1.1)***
	Draft Work Plan Meeting	CAROLLO and CITY will meet to review draft work plan before it is submitted to the RWQCB and Technical Advisory Panel. ***(This meeting will be held at the same time as the Draft Work Plan Meeting in Task 1.1)***

TASK 4 – PROJECT REVIEW

4.1 – Technical Advisory Process

CAROLLO will retain the services of NWRI to facilitate a technical advisory process that will be defined in the Work Plan developed in Task 1.1 and Task 3.1. It is anticipated the technical advisory process will consist of:

- Approximately four (4) technical advisors (selected and retained by NWRI) with the following qualifications in the areas of both potable reuse and subsurface desalination intakes:
    - Hydrogeologist
    - Engineer or contractor
    - Regulatory/permitting (e.g., CEQA consultant)
    - Water Quality
- Up to four (4) technical advisory panel workshops consisting of the following topics:
    - Workshop 1: Work Plan Review (Task 1 and Task 3) – ***Included in this Scope of Services (Authorization #1)***
    - Workshop 2: Fatal Flaw Analysis (Task 1 and Task 3) – ***Included in a separate Scope of Services (Authorization #2)***
    - Workshop 3: Feasibility Analysis (Potable Reuse) – ***Included in a separate Scope of Services (Authorization #2)***
    - Workshop 4: Feasibility Analysis (Subsurface Desalination Intake) – ***Included in a separate Scope of Services (Authorization #3)***

  CAROLLO will prepare technical materials and make presentations to the technical advisory panel.
- NWRI will facilitate technical advisory workshops and public comment.

The Technical Advisory Process will be formally adopted in the Work Plan developed in Task 3.1.

CAROLLO will be responsible for:

- Coordination of workshop dates with workshop participants.
- Preparing and distributing workshop materials to the technical advisors and workshop participants a minimum of 2 weeks prior to the workshop meetings.

CAROLLO's subconsultant (NWRI) will be responsible for:

- Facilitating the technical advisory panel workshops and public comment.
- Preparing draft and final meeting minutes.

Task 4.1 Deliverables

1. Meeting Agendas and Workshop Materials
2. Draft meeting minutes
3. Final meeting minutes

4.2 –CITY Council Workshops/Meetings

CAROLLO will attend up to three (3) CITY Council workshops or meetings to assist CITY staff in presenting the progress and findings of the studies completed in Tasks 1, 2 and 3.

Task 4.2 Deliverables by CAROLLO

1. Powerpoint presentation for City Council workshop or meeting

Summary of TASK 4 Project Meetings (Authorization #1)

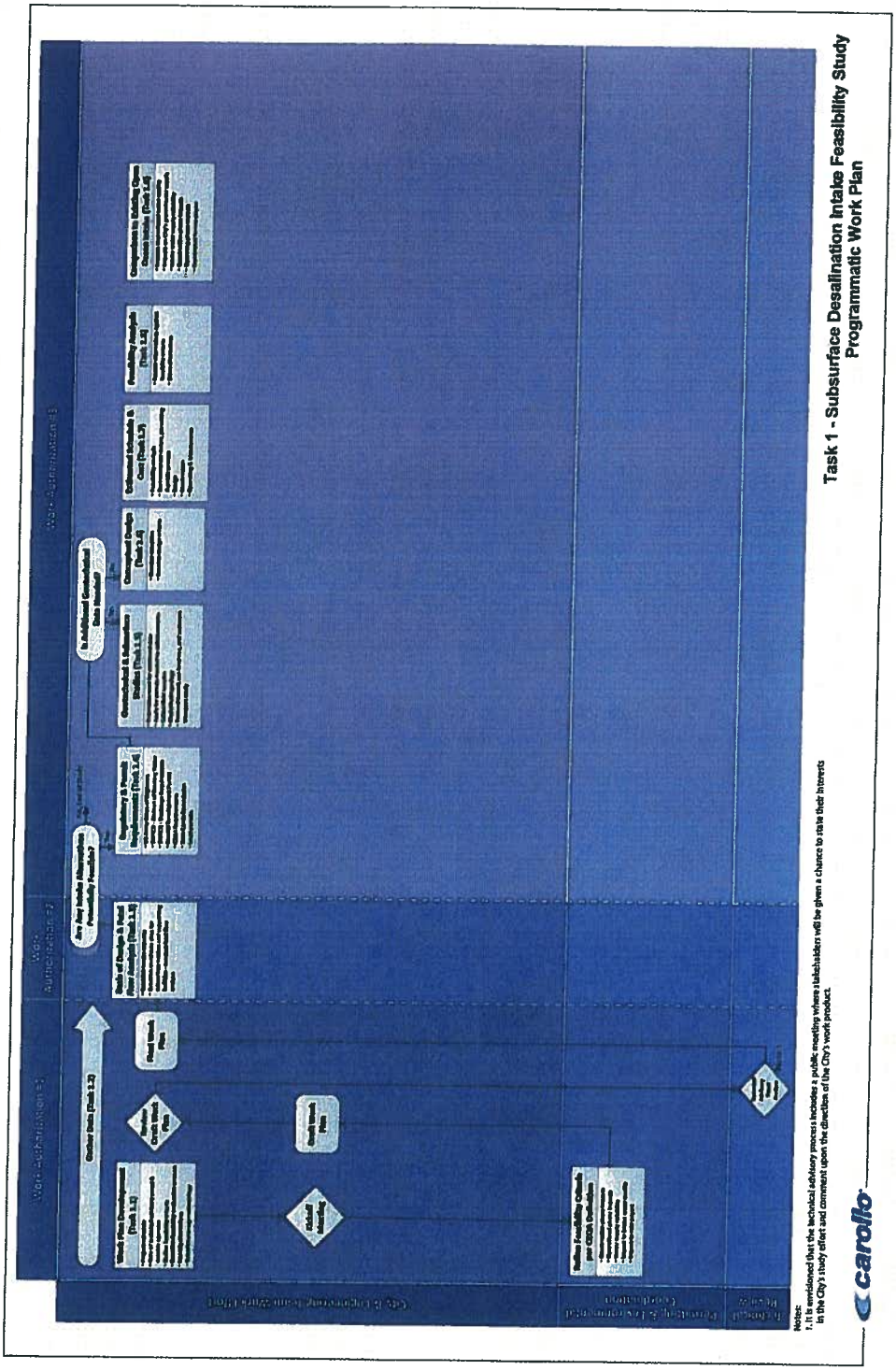
Task No.	Meeting Name	Purpose
4.1	Technical Advisory Panel Workshop #1 (Task 1.1 and 3.1)	This meeting will be facilitated by NWRI and have the following objectives: • Following meeting with CITY staff to review Draft Work Plans submitted under Tasks 1.1 and 3.1, CAROLLO and CITY will meet with Technical Advisory Panel to review Draft Work Plans. CAROLLO will present Draft Work Plan to technical advisory panel and interested parties. • NWRI will facilitate and receive comments from the audience on work plan approach.

TIME OF PERFORMANCE

The project schedule will be further refined during the work plan development, however the CITY is required to submit a draft work plan to the RWQCB by August 2015 and a final Feasibility Study Report by June 2017. It is anticipated that CAROLLO will provide services based upon the schedule presented in Attachment A.

PAYMENT

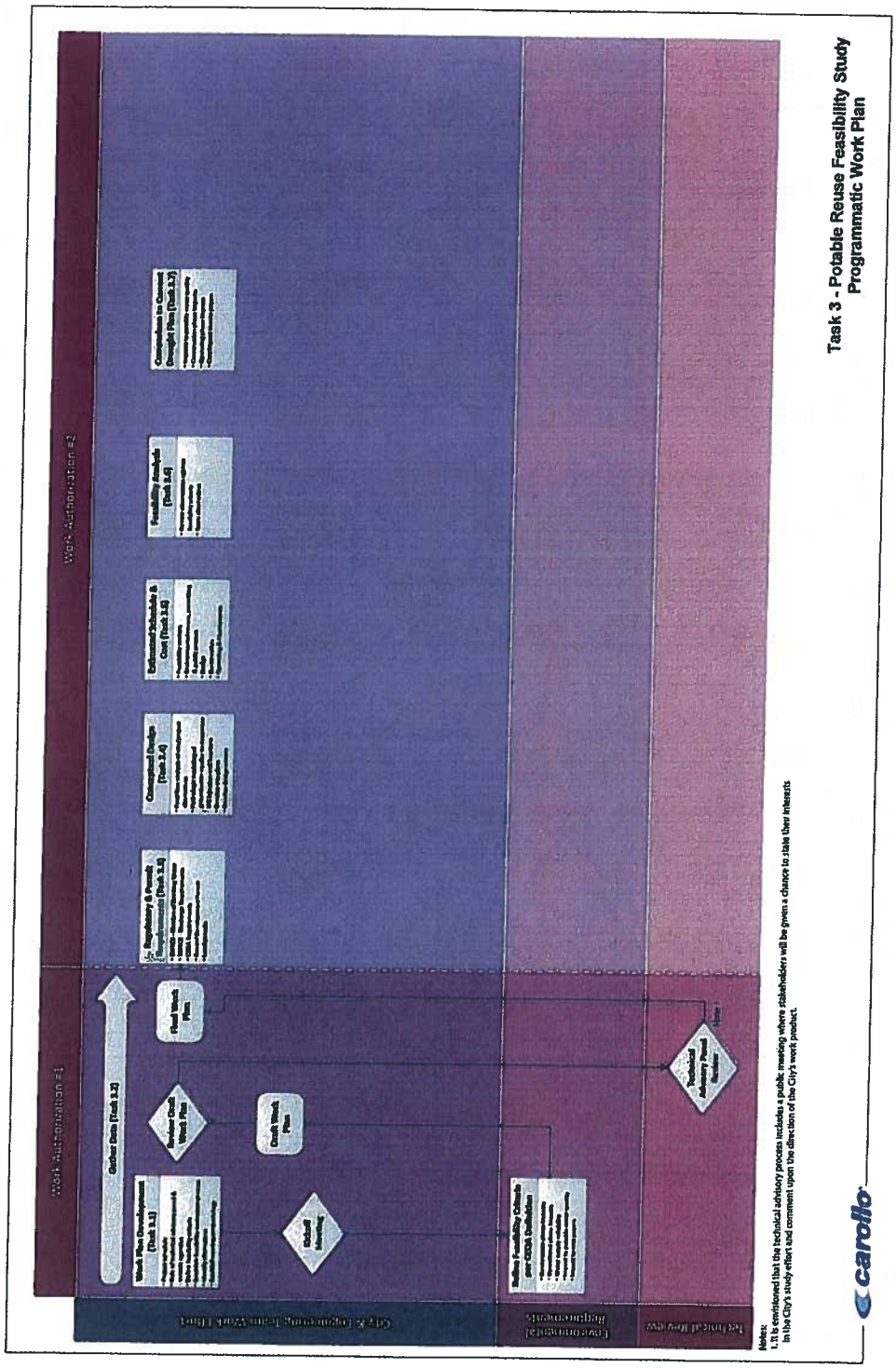
Payment will be based upon the terms stated in the contract Agreement between CAROLLO and the CITY. Invoices will be submitted by CAROLLO to the CITY on a monthly basis and will include CAROLLO's labor hours and direct costs, along with supporting invoices and the CITY's invoice cover sheet. Refer to the attached table for a schedule of fees involved with this Scope of Services.



Notes:
1. It is envisioned that the technical advisory process includes a public meeting where stakeholders will be given a chance to state their interests in the City's study effort and comment upon the direction of the City's work product.

carollo

Task 1 - Subsurface Desalination Intake Feasibility Study
Programmatic Work Plan



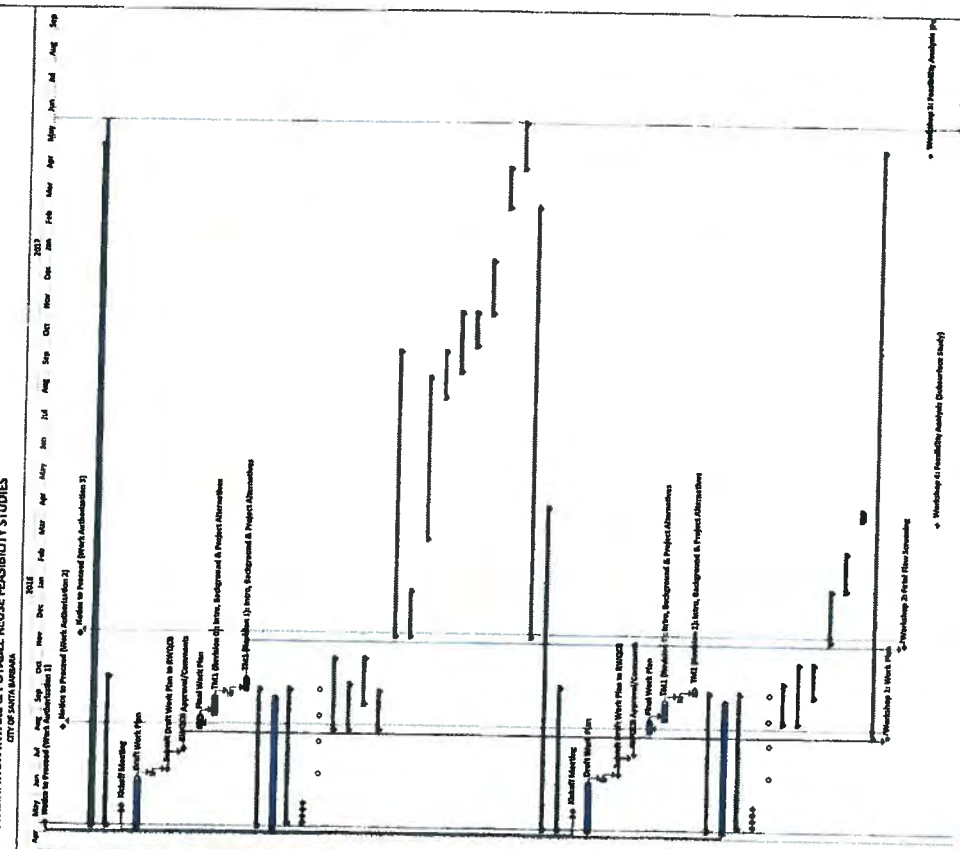
Task 3 - Potable Reuse Feasibility Study
Programmatic Work Plan

carollo

ATTACHMENT A

SUBSURFACE DESALINATION INTAKE & POTABLE REUSE FEASIBILITY STUDIES

CITY OF SANTA BARBARA



CAROLLO ENGINEERS

PRELIMINARY DESIGN SERVICES FOR
RECOMMISSIONING THE CITY'S DESALINATION PLANT
(Subsurface Intake and Potable Reuse Studies)

AUTHORIZATION #1: WORK PLAN DEVELOPMENT

		Jim Meyerhofer Principal-in-Charge	Tom Sizemore Senior Manager/Project Engineer	Ali Chenaie Senior Project Engineer	Ryan Troop Project Engineer	Matthew Johnson Senior Advisor	Jamie Rosen - Technical Advisor	Tom Kenyon Numerical Analyst (Cost Estimator)	Jim Thompson Senior Manager	Technician	Total Personnel	Total	Tetra Tech	Carollo Rosen	Hydro Geo Consulting	Tetra Environmental	Richard Walker Inorganic Chemist	Water Research Institute (WRI) Technical Advisory Panel	Subtotal Water Reuse Mark-up (10%) Tasks 1, 2, 3	Travel	Total	Subtotal Labor	Subtotal ODCs	Total
													OTHER DIRECT COSTS (ODCs)											
													Subcontracts											
1.0	Work Plan Development	1	40	80					1	8	130	$ 1,551	$ 17,115	$ 30,020	$ 8,000				$ 6,855	$ 120	$ 8,000	$ 25,540	$ 33,660	$ 57,214
1.0	Alternatives Analysis		4	24								$ 200		$ 20,000					$ 2,000	$ -	$ -	$ 4,000	$ 22,000	$ 26,000
1.0	Work Plan Development	1	40	80					1	8	130	$ 1,551	$ 17,115	$ 30,020					$ 6,855	$ 120	$ 8,000	$ 25,540	$ 47,660	$ 73,199
2.0	Data Gathering		4	24								$ 200		$ 20,000					$ 2,000	$ -	$ -	$ 4,000	$ 22,000	$ 22,000
4.0	Technical Advisory Panel		20	20						6	46	$ 400	$ 3,000	$ 4,000			$ 12,000		$ 1,600	$ -	$ 1,700	$ 8,500	$ 19,700	$ 28,200
4.0	City Council Meeting Attendance		8							4	12	$ 200							$ -	$ -	$ 1,200	$ 2,200	$ 1,200	$ 3,400

Notes:
1. Tetra Environmental is a subconsultant to Dudek who will manage their contract and will apply a 10% mark-up to Tetra's fee. Carollo has not applied an second mark-up to the Tetra Environmental fee.
2. 10% mark-up applies to subconsultant fees only. No mark-up has been applied to ODCs (Printing, Travel).

EXHIBIT B

Contractor's Nondiscriminatory Employment Certificate

CONTRACTOR'S NONDISCRIMINATORY EMPLOYMENT CERTIFICATE
Santa Barbara Municipal Code § 9.126.020

A. Certificate Generally

Consistent with a policy of nondiscrimination in employment on contracts of the City of Santa Barbara and in furtherance of the provisions of Section 1735 and 1777.6 of the California Labor Code a "contractor's obligation for nondiscriminatory employment certificate" as hereinafter set forth shall be attached and incorporated by reference as an indispensable and integral term of all bid specifications and contracts of the City for purchases, services, and the construction, repair, or improvement of public works.

B. Contents of Certificate

The Contractor's obligation for nondiscriminatory employment is as follows:

1. The Contractor will not discriminate against any employee or applicant for employment because of race, creed, color, national origin, ancestry, sexual orientation, political affiliation or beliefs, sex, age, physical handicap, medical condition, marital status or pregnancy (as those terms are defined by the California Fair Employment and Housing Act -- Government Code Section 12900-12996), except where such discrimination is based on a bona fide occupational qualification. The Contractor will take positive action or ensure that applicants are employed, and that employees are treated during employment, without regard to their race, creed, color, national origin, ancestry, sexual orientation, political affiliation or beliefs, sex, age, physical handicap, medical condition, marital status or pregnancy (as those terms are defined by the California Fair Employment and Housing Act -- Government Code Section 12900-12996), except where such discrimination is based on a bona fide occupational qualification. Such action shall include but not be limited to the following: Employment, upgrading, demotion, or transfer; recruitment or recruitment advertising; layoff or termination; rates of pay or other forms of compensation; and selection for training, including apprenticeship. The Contractor agrees to post in conspicuous places, available to employees and applicants for employment, notices to be provided by the City setting forth the provisions of this nondiscrimination clause.

2. The Contractor will, in all solicitations or advertisements for employees placed by or on behalf of the Contractor, state that all qualified applicants will receive consideration for employment without regard to race, color, national origin, ancestry, sexual orientation, political affiliation or beliefs, sex, age, physical handicap, medical condition, marital status or pregnancy (as those terms are defined by the California Fair Employment and Housing Act -- Government Code Section 12900-12996), except where such discrimination is based on a bona fide occupational qualification.

3. The Contractor will send to each labor union or representative of workers, with which he has a collective bargaining agreement or other contract or understanding, a notice to be provided by the City advising the said labor union or workers' representative of the Contractor's commitments under this provision, and shall post copies of the notice in conspicuous places available to employees and applicants for employment.

4. The Contractor will permit access to his records of employment, employment advertisements, application forms, and other pertinent data and records by the City, the Fair Employment Practices Commission, or any other appropriate agency of the State designated by the City for the purposes of investigation to ascertain compliance with the Contractor's Obligation for Nondiscriminatory Employment provisions of this contract, or Fair Employment Practices statute.

5. A finding of willful violation of the nondiscriminatory employment practices article of this contract or of the Fair Employment Practices Act shall be regarded by the City as a basis for determining that as to future contracts for which the Contractor may submit bids, the Contractor is a "disqualified bidder" for being "nonresponsible".

   The City shall deem a finding of willful violation of the Fair Employment Practices Act to have occurred upon receipt of written notice from the Fair Employment Practices Commission that it has investigated and determined that the Contractor has violated the Fair Employment Practices Act and has issued an order under Labor Code Section 1426 or obtained an injunction under Labor Code Section 1429.

   Upon receipt of any such written notice, the City shall notify the Contractor that unless he demonstrates to the satisfaction of the City within a stated period that the violation has been corrected, he shall be declared a "disqualified bidder" until such time as the Contractor can demonstrate that he has implemented remedial measures, satisfactory to the City, to eliminate the discriminatory employment practices which constituted the violation found by the Fair Employment Practices Commission.

6. Upon receipt from any person of a complaint of alleged discrimination under any City contract, the City Administrator shall ascertain whether probable cause for such complaint exists. If probable cause for the complaint is found, the City Administrator shall request the City Council to hold a public hearing to determine the existence of a discriminatory practice in violation of this contract.

   In addition to any other remedy or action provided by law or the terms of this contract, the Contractor agrees that, should the City Council determine after a public hearing duly noticed to the Contractor that the Contractor has not complied with the nondiscriminatory employment practices provisions of this contract or has willfully violated such provisions, the City may, without liability of any kind, terminate, cancel, or suspend this contract, in whole or in part. In addition, upon such determination the Contractor shall, as a penalty to the City, forfeit a penalty of $25.00 for each calendar day, or portion thereof, for each person who was denied employment as a result of such noncompliance. Such moneys shall be recovered from the Contractor. The City may deduct any such penalties from any moneys due the Contractor from the City.

7. The Contractor certifies to the City that he has met or will meet the following standards for positive compliance, which shall be evaluated in each case by the City:

   a. The Contractor shall notify all supervisors, foremen and other personnel officers in writing of the content of the nondiscrimination provision and their responsibilities under it.
   b. The Contractor shall notify all sources of employee referrals (including unions, employment agencies, advertisements, Department of Employment) of the content of the nondiscrimination provision.
   c. The Contractor shall file a basic compliance report as required by the City. Willfully false statements made in such reports shall be punishable as provided by law. The compliance report shall also specify the sources of the work force and who has the responsibility for determining whom to hire, or whether or not to hire.
   d. The Contractor shall notify the City of opposition to the nondiscrimination provision by individuals, firms or organizations during the period of this contract.

8. Nothing contained in this Contractor's Obligation for Nondiscriminatory Employment Certificate shall be construed in any manner to prevent the City from pursuing any other remedies that may be available at law.

9. The Contractor certifies to the City that he will comply with the following requirements with regard to all subcontractors and suppliers:

   a. In the performance of the work under this contract, the Contractor will include the provisions of the foregoing paragraphs (1) through (8) in all subcontracts and in any supply contract to be performed within the State of California, so that such provisions will be equally binding upon each subcontractor and each supplier.
   b. Contractor will take such action with respect to any subcontract or purchase order as the City may direct as a means of enforcing such provisions including sanctions for noncompliance: Provided, however, that in the event the Contractor becomes involved in, or is threatened with, litigation with a subcontractor or supplier as a result of such direction by the City, the Contractor may request the City to enter into such litigation to protect the interests of the City.

EXHIBIT C

Contractor's Living Wage Certificate

LIVING WAGE CERTIFICATION

Official notification to: _____.

_____.

_____.

The service contract that is pending between your company and the City of Santa Barbara is subject to the City of Santa Barbara Living Wage Ordinance, SBMC Chapter 9.128 (hereinafter referred to as "the Ordinance"). Pursuant to this ordinance, you are hereby notified that your company is required to demonstrate compliance by completing and returning the attached compliance statement. This statement must be completed and returned before contract commencement. **You may fax the compliance statement to: either the requesting department or to the City of Santa Barbara Finance Department (Purchasing) at (805) 897-1977.**

Please Note: Current living wage rates will apply to all subsequent contracts and amendments during the remainder of the current fiscal year ending June 30, 2015.

The City of Santa Barbara Living Wage Ordinance was adopted on April 4, 2006 (Ordinance number 5384). All capitalized terms used herein are used as defined in the Ordinance. The Ordinance requires that persons directly working on City of Santa Barbara contracts, for services specified in the ordinance, are to be paid a living wage while working on the City of Santa Barbara contract. The Ordinance only applies to those persons directly providing services to the City and does not apply to administrative or support staff employees of a Service Contract, such as administrators, payroll, personnel, or similar employees. The Ordinance also does not apply to employees who are Handicapped, Apprentices, Learners, or Student Interns, who are otherwise part of an employer's training program as those terms are defined in the Ordinance. The Ordinance also states that employees have the right to expressly negotiate and agree to wage and benefit levels different than those required by the Ordinance.

The Ordinance requires that employees working for your firm on this contract be notified that the City of Santa Barbara Living Wage Ordinance applies to them. As part of compliance for this contract, you are required to notify affected employees.

Effective from July 1, 2014, through June 30, 2015, the current rate for minimum compensation to employees is:

1. If benefits are not provided to an Employee, a wage of no less than $16.70 per hour.

2. If Basic Medical Insurance and Compensated Holidays are provided to the Employee, a wage of no less than $14.32 per hour.

3. If Supplemental Employee Benefits are provided to the Employee, a wage of no less than $13.12 per hour.

(All capitalized terms used herein are used as defined in the Ordinance, SBMC Chapter 9.128)

Also be advised that the City may request any or all certified payrolls associated with this contract, however, any such request will be made to your firm in writing and provide fourteen calendar days to respond. The City may also conduct on-site audits to verify compliance. These audits may include, but are not limited to, employee interviews.

Direct questions regarding this Ordinance to General Services Manager, City of Santa Barbara Finance Department, P.O. Box 1990, Santa Barbara, CA 93102.

1. * Select A, B C or D below.

☐ A. **The Living Wage Ordinance does not apply to this contract because:**

☐ **Exemption for Handicapped Individuals and Apprentices**. For the purposes of this form, an employee shall not include a "handicapped employee" employed pursuant to a special license issued under Sections 1191 and 1191.5 of the state Labor Code or an "apprentice" or "learner" employed pursuant to a special license issued under Section 1192 of the state Labor Code.

☐ **Exemption for Student Interns**. For the purposes of this form, an employee shall also not include a student intern which shall be defined as a person receiving educational or school credit at a duly licensed and accredited school or educational institution as part of or in connection with his or her employment or service with the City Service Contractor.

☐ **Public Entity.**

☐ **Non-profit exemption**.

☐ **Workers are part of a bona fide collective bargaining agreement.**

☐ **Persons employed are defined as executive or professional as used in the federal Fair Labors Standards Act of 1938 (29 USC Section 201 et. seq.).**

☐ Services are incidental. Explain: _____

_____

Complete the certification portion on page 3.

☒ B. Employees working on City of Santa Barbara contracts receive a pay rate that meets or exceeds the City of Santa Barbara Living Wage requirement of $16.70 per hour without benefits.

** Complete items #2, #3, #4, #5 and the certification portion on page 3.*

☐ C. Employees working on City of Santa Barbara contracts receive a pay rate that meets the City of Santa Barbara Living Wage requirement of $14.32 per hour with the following benefits:

1. A combined twelve days compensated leave time annually for full-time employees, and prorated leave for employees working less than full time
2. Basic Medical Insurance Coverage for the Employee.

** Complete items #2, #3, #4, #5, #6 and the certification portion on page 3.*

☐ D. Employees working on City of Santa Barbara contracts receive a pay rate that meets the City of Santa Barbara Living Wage requirement of $13.12 per hour with all of the following benefits:

1. A combined twelve days compensated leave time annually for full-time employees, and prorated leave for employees working less than full time
2. Basic Medical Insurance Coverage for the Employee.

3. Basic Medical Insurance Coverage for the Employee's spouse, domestic partner or family.

4. One additional Supplemental Benefit as defined in the Ordinance.

   ☐ Pension or deferred compensation retirement plan.

   ☐ Childcare or dependent care.

   ☐ Equivalent of ten (10) eight hour days of compensated leave over and above the compensated leave in item 1.

   ☐ Other: _____

** Complete items #2, #3, #4, #5, #6 and the certification portion on page 3.*

2. Will any subcontractors perform work on this contract? ☒ Yes ☐ No
   If yes, please indicate company(s) on an additional page.

3. Will you post employee notification form in an area accessible to employees working on City of Santa Barbara contracts? ☒ Yes ☐ No

4. You may be required to provide certified payroll records, time cards, and other records any time during the contract period to demonstrate compliance. These payroll records must include the following information for each employee working on this contract: employee name, job classification, employer benefit contribution, and hourly pay under this contract.

   Do you agree to provide this information within 14 calendar days when requested? ☒ Yes ☐ No

   The City may also perform on site payroll audits that may include, but are not limited to, employee interviews.

5. a) Please provide the total affect that the Living Wage requirements had on your bid price (i.e., no cost affect, increase bid price by $..., etc.)?

   No Affect

   b) How many employees benefited from the living wage requirement? None

   c) How much did the above employees benefit in aggregate during the contract:
   $0.00

6. The City has several insurance plans. To qualify for a lower wage tier, you must offer insurance at no cost to your employees and match one of the following plans in terms of co-pays/out-of-pocket expenses.

   ☐ **Aetna HMO:** No deductible, $100 co-pay for emergency room visits, no charge for preventative care, $25 co-pay for office visits to Primary Care Physicians/$35 co-pay to Specialists; Prescriptions: $20 co-pay for generics; $30 co-pay for brand, & $45 co-pay for non-formulary.

   ☐ **Kaiser HMO:** No deductible, $35 co-pay for emergency room visits, no charge for preventative care, $10 co-pay for office visits; Prescriptions: $5 co-pay for generics; $15 co-pay for brand & non-formulary is not covered.

☐ **Aetna Open Access Managed Care PPO:** Deductibles: $500/individual $1,000/family, $100 co-pay + 20% coinsurance for emergency room visits, no charge for preventative care, $25 co-pay for office visits; Prescriptions: $20 co-pay for generics; $30 co-pay for brand, & $45 for non-formulary.

☐ **Aetna Health Reimbursement PPO:** Deductibles: $2,000/individual $4,000/family, 20% coinsurance for emergency room visits, no charge for preventative care, 20% coinsurance for office visits; Prescriptions: $10 co-pay for generics; $20 co-pay for brand, & $35 for non-formulary.

☐ **Aetna Health Savings Account PPO:** Deductibles: $2,500/employee only coverage, $5,000/family, 20% coinsurance for emergency room visits, no charge for preventative care, 20% coinsurance for office visits; Prescriptions: $15 co-pay for generics; $25 co-pay for brand, & $40 for non-formulary.

The signatory below hereby certifies, under penalty of perjury, that the forgoing information is correct:

CAROLLO ENGINEERS, INC.
Company Name

5075 SHOREHAM PL., SUITE 120, SAN DIEGO, CA 92122
Company Address **City, State, Zip**

THOMAS F. SEACORD 858-505-1020 858-505-1015
Contact Name **Phone number** **Fax number**

THOMAS F. SEACORD, VICE PRESIDENT
Name and Title (Please print)

Signature

APRIL 17, 2015
Date

You may fax the compliance statement to: City of Santa Barbara Finance Department (Purchasing) at (805) 897-1977.

List of Subcontractors/Subconsultants

DUDEK
605 Third Street
Encinitas, CA 92024
Phone: (760) 479-4296
Contact: Joe Monaco

FUGRO WEST, INC.
660 Clarion Court, Suite A
San Luis Obispo, CA 93401
Phone: (805) 542-0797
Contact: Paul Sorensen

GSI WATER SOLUTIONS, INC.
418 Chapala St. Suite F
Santa Barbara, CA 93101
Phone: (805) 895-3956
Contact: Jeff Barry

MICHAEL BAKER INTERNATIONAL
9755 Claremont Mesa Blvd
San Diego, CA 92124
Phone: (858) 614-5000
Contact: Scott Jenkins

NATIONAL WATER RESEARCH INSTITUTE
18770 Ward St
Fountain Valley, CA 92708-0896
Phone: (714) 378-3278
Contact: Jeff Moshier

TENERA
971 Dewing Ave, Suite 101
Lafayette, CA 94549
Phone: (915) 962-9769
Contact: David Mayer

WATER GLOBE CONSULTING
824 Contravest Lane
Winter Springs, FL 32708
Phone: (203) 253-1312
Contact: Nikolay Voutchkov

APPENDIX B – HYDROGEOLOGIC ANALYSIS OF SSI ALTERNATIVES

FINAL

Technical Feasibility Evaluation

Hydrogeologic Analysis of SSI Alternatives Santa Barbara Desalination Project

Prepared for

Carollo Engineers

December 29, 2015

Prepared by

GSI Water Solutions, Inc.
418 Chapala Street, Suite F Santa Barbara, CA 93101
P: 805.895.3956 info@gsiws.com www.gsiws.com

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Technical Feasibility Evaluation
Hydrogeologic Analysis of SSI Alternatives

Executive Summary

In coordination with Carollo Engineers and Fugro Consultants, this technical report provides the results of an evaluation of six (6) subsurface intake (SSI) alternatives to the City of Santa Barbara's existing screened open ocean intake for the City's Charles Meyer Desalination Plant (Desal Plant). In accordance with the Subsurface Desalination Intake Work Plan (Carollo, 2015), this report provides technical analyses of whether any single SSI alternative can fully replace the capacity of the ocean intake of 15,898 gallons per minute (10,000 acre-feet per year) within the available beach area. The study area focused on City owned property along East Beach, West Beach, and Leadbetter Beach. The total length of the beach that may be available for SSI development is approximately 9,000 feet (1.7 miles). The SSIs are all assumed to be completed in the Shallow Zone sediments as defined by the USGS (Martin, 1984). Because all the SSIs considered in this study are completed in the Shallow Zone, impacts are not anticipated on the City's groundwater resources.

The work performed included development of seven site-specific geologic cross sections based on subsurface properties of the sedimentary horizons and evaluation of the yield, spacing, and number of each type off SSI alternative that would be required. The subsurface intake alternatives considered included:

1. Vertical wells,
2. Beach infiltration galleries (BIG)[1]
3. Radial collector wells (also known as 'Ranney wells')
4. Slant wells
5. Seabed infiltration gallery (SIG)[2] and
6. Horizontal directionally drilled wells (HDD)

Of the six types of subsurface intakes evaluated, only the seabed infiltration gallery (SIG) and horizontal directionally drilled (HDD) wells are able to satisfy the requirement to produce a flow rate of 15,898 gpm from within the City-owned beach front. In addition, these intake alternatives are the only two that derive all of their flow from offshore sources and therefore do not impact onshore groundwater resources. The other SSIs evaluated are capable of producing between 9 and 64 percent of the required flow. The following table provides a summary of the results of this hydrogeologic evaluation.

[1] Referred to in Work Plan as Lateral Beach Wells or Onshore Infiltration Galleries.
[2] Referred to in Work Plan as Subsurface Infiltration Galleries

Table ES-1.

Intake Type	Shallow Zone Layer	Number of Facilities Required [1]	Approximate Spacing (feet)	Length of Beach Required (Miles)[1]	Yield per Facility (gpm)	Potential Yield [2] (gpm)	Percentage of Required Desal Plant Flow
Vertical Wells	Lower Sand	40 - 160	600 - 750	5.5 – 18	100-400	1,500 - 4,800	9 – 30%
Beach Infiltration Gallery	Upper Sand	6	N/A	3	Varies with length of available beach	10,100	64%
Radial Collector Wells	Upper Sand Lower Sand	43 16 - 58	600 600 – 1,500	5 4 - 6	375 275 - 1,000	5,625 4,125 - 7,000	35% 26 - 44%
Slant Wells	Lower Sand	16 - 58	650 - 1,250	3.5 - 6	275 - 1,000	4,400 - 8,000	28 – 50%
SIG	Upper Sand	1	One facility only	One facility only; located offshore	15,898	15,898	100%
HDD	Upper Sand	11	N/A[3]	0.1	1,500	15,898	100%

Notes:
1. Total required to meet 15,898 gpm.
2. Potential yield within available beach.
3. HDD wells constructed as multi-well clusters from one location.

Vertical Wells

The total potential yield from vertical wells is estimated to be the lowest of all intake alternatives. A total of up to 15 vertical wells installed on the available beach would have a combined pumping yield of from 1,500 to 4,800 gpm, which is 9 to 30 percent of the total required yield. Water produced from vertical wells would consist of as much as 47 percent water produced from inland sources and only 53 percent from offshore (seawater) sources. In this case, inland sources refers to groundwater present in the Shallow Zone only. At the maximum yield based upon beach frontage, water level drawdown of 1 to 3 feet is predicted in areas with sensitive habitat, such as Mission Creek and Laguna drain outfalls.

A total of between 5.5 and 18 miles of similar beach and as many as 160 wells would be required from vertical wells to produce the total water required for the plant.

Beach Infiltration Gallery (BIG)

Of the intake alternatives which cannot satisfy the full project flow requirements, the Beach Infiltration Gallery has the highest potential yield. If galleries are constructed across the full length of the available beaches, the yield is estimated to be approximately 10,000 gpm. BIGs are calculated to derive approximately 95 percent of their flow from offshore sources, with very little inland contribution; this is due to the predominance of clay in the uppermost saturated zone inland of the beach. To satisfy the entire yield required, the BIG would require 3 miles of similar beach with the upper sand layer that is at least 30 feet deep. BIGs producing 10,000 gpm could induce drawdown impacts in beach areas where sensitive habitats exist; impacts could be reduced by reducing the length of the BIG and locating the BIG farther from the sensitive habitat area(s), both of which would result in a lower yield.

Radial Collector Wells

Three collector well scenarios were evaluated: (a) upper (beach) sand, (b) lower sand layer assuming a relatively high hydraulic conductivity, and (c) lower sand layer assuming a relatively low hydraulic conductivity. For the upper beach sand, it may be possible to produce up to 5,600 gpm from 15 collector wells constructed on the available beach assuming the upper beach sand is sufficiently thick and permeable across the entire beach area. For the lower sand with high hydraulic conductivity, seven collector wells spaced 1,500 feet apart could produce 7,000 gpm from the available beach area. For the lower sand with low hydraulic conductivity, fifteen radial collector wells spaced 700 feet apart could produce up to 4,125 gpm from the available beach areas.

Because the geologic cross sections indicate that the lower sand layer extends inland, radial collectors in this layer may produce water comprised of as much as 40 percent inland groundwater and 60 percent from offshore sources. As much as 70 percent of the produced water would be from offshore sources if collectors are constructed in the upper (beach) sand. Areas with sensitive habitat, such as Mission Creek and Laguna drain outfalls, could be affected by pumping from collector wells constructed in either the upper or lower sand layers.

To satisfy the total flow required, a total of between 4 and 6 miles of similar beach would be required.

Slant Wells

Slant wells each have an estimated yield of 275 to 1,000 gpm per well. A total of 8 to 16 slant wells could be constructed within the available beach area with a total yield of 4,400 to 8,000 gpm. Because the slant wells are installed under the ocean and are not parallel to the shoreline, they may produce water comprised of as much as 95 percent from offshore sources and as little as 5 percent from inland groundwater sources. At the maximum yield based upon beach frontage, water level drawdown of 1 to 3 feet is predicted in areas with sensitive habitat, such as Mission Creek and Laguna drain outfalls.

Four to six miles of similar beach would be required to satisfy the project's required flow rate of 15,898 gpm.

Seabed Infiltration Gallery (SIG)

A seabed infiltration gallery located offshore will likely satisfy the project yield requirement for desalination as specified in study goals. Further, the produced water would likely consist entirely of seawater without any contribution from inland groundwater. The area required to construct these beds and allow sufficient infiltration to produce the target yield is estimated to be 7 acres.

This intake alternative could fit in the offshore area within the 0.5 mile City of Santa Barbara jurisdictional area if bedrock is not present and other factors including longshore current velocity and seafloor erosion are satisfactory. This SSI alternative is not anticipated to impact sensitive onshore habitat areas.

Horizontal Directionally-Drilled Wells (HDD)

As with SIGs, HDD wells may be able to satisfy the total project yield requirement. To achieve this flow, approximately 11,000 feet of screen would be required from eleven HDD wells. It may be possible to install 11 HDD facilities from a single location. As with SIGs, the produced water would likely consist entirely of water from offshore sources.

This intake alternative could fit in the offshore area within the 0.5 mile City of Santa Barbara jurisdictional area if bedrock is not present and other factors including longshore current velocity and seafloor erosion are satisfactory. Areas with sensitive habitat, such as Mission Creek and Laguna drain outfalls, are not expected to be impacted.

Technical Feasibility Evaluation
Hydrogeologic Analysis of SSI Alternatives

1 Introduction

On behalf of the City of Santa Barbara (City) and in coordination with Carollo Engineers and Fugro Consultants, this technical report provides the results of an evaluation of six (6) subsurface intake (SSI) alternatives to the City's existing screened open ocean intake for the City's Charles Meyer Desalination Plant. This work was completed in accordance with the Subsurface Desalination Intake Work Plan (Carollo, 2015). The assumed basis of design for the desalination plant intake and the full permitted capacity of the City's existing ocean intake is 15,898 gallons per minute (gpm) (22.9 million gallons per day (MGD)), which is the intake flow required to produce 10,000 acre-feet per year (AFY) of desalinated water. This report provides technical analyses of whether any single SSI alternative can fully replace the capacity of the ocean intake.

Detailed hydrogeologic data were collected and reviewed to establish site-specific subsurface properties of the sedimentary horizons that would be tapped by the various SSI alternatives. These subsurface characteristics strongly govern the yield of each facility, potential site locations and spacing between sites, water quality, and constructability.

For each SSI alternative, calculations were conducted to determine the facilities needed to deliver the 15,898 gpm of water from the subsurface. Additionally, given the available length of beachfront, the maximum yield of each SSI alternative within the available beach area was calculated.

2 Background

The study area for the hydrogeologic analysis of the SSI alternatives includes the site alternatives identified in the Work Plan and extends inland approximately 4,500 feet to facilitate the development of the hydrogeologic framework for the technical analyses. The SSI project site alternatives include the onshore portions of Santa Barbara City property along Leadbetter, West and East Beach areas, extending ½ mile offshore, but excluding construction setbacks. These exclusion areas where SSI facilities should not be constructed were defined based on several factors including: 1) sensitive habitat areas such as where Sycamore Creek, Mission Creek, and Estero Drain form ponded areas on the beach, 2) setbacks from Stearns Wharf, and 3) setbacks from known unstable or easily eroded areas (e.g., locations on Leadbetter Beach). A map of the project location, and study area is presented as Figure 1.

FIGURE 1

Project Location Map

Hydrogeologic Analysis of
SSI Alternatives
Santa Barbara, California

LEGEND

- Study Area
- Santa Barbara City Limit
- Highway 101
- Major Road

N

0 1,000 2,000 3,000 4,000
Feet

MAP NOTES:
Date: December 28, 2015
Data Sources: USDA/NRCS, County of Santa Barbara, CA

GSI Water Solutions, Inc.

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3 Subsurface Properties

3.1 Hydrogeologic Setting

The study area lies within the Santa Barbara Groundwater Basin (California Department of Water Resources Basin 3-17), which includes two sub-basins referred to as Storage Units I and III (Martin, 1984).

The western portion of the study area, encompassing the onshore area to the southwest of the Mesa Fault (Leadbetter Beach and westernmost portion of West Beach), lies within Storage Unit III (Figure 2). The generalized hydrogeologic setting for the Storage Unit III portion of the study area is 25 to 100 feet of unconsolidated deposits (Shallow Zone) that overly sedimentary bedrock (Martin, 1984 and Freckleton et al., 1998). The Shallow Zone unconsolidated deposits in Storage Unit III are Holocene-age alluvial deposits that consist of discontinuous, coarse-grained water-bearing deposits inter-fingered with fine-grained deposits of lower hydraulic conductivity (Freckleton et al., 1998), but may also contain a basal layer of Santa Barbara Formation, just above bedrock (Martin, 1984). There is very little groundwater production in Storage Unit III (a single well) due to poor production capability and quality. Due to the very low permeability of sedimentary bedrock beneath the Shallow Zone, the target zone within Storage Unit III for SSI alternatives that utilize wells is limited to the Shallow Zone.

The eastern portion of the study area lies within Storage Unit I. The generalized hydrogeologic setting for the portion of the study area located in Storage Unit I is based upon USGS Water-Supply Paper 2197 (Martin, 1984). Storage Unit I is a fault bounded block that is down-dropped relative to Storage Unit III along the Mesa Fault (Martin, 1984). The greatest thickness of unconsolidated deposits in Storage Unit I is approximately 1,000 feet in the area adjacent to the northeast side of the Mesa Fault near the Santa Barbara Harbor (Martin, 1984). Although the unconsolidated deposits in Storage Unit I area progressively thin to the northwest and northeast, the consolidated bedrock contact occurs far beneath the zone of interest for this study.

The unconsolidated deposits of Storage Unit I are subdivided into four zones including, from top to bottom: the Shallow Zone, the Upper Producing Zone (UPZ), the Middle Zone, and the Lower Producing Zone (LPZ). The Shallow Zone is composed of Holocene and Pleistocene-aged unconsolidated deposits. Water-bearing deposits are present in the Shallow Zone, but are laterally discontinuous. Fine-grained deposits are prevalent in the Shallow Zone, which confine or partly confine the underlying Upper Producing Zone (UPZ) (Martin, 1984). The Shallow Zone is approximately 200 feet thick in the study area and generally thickens towards the south (seaward), presumably continuing offshore in the study area. The bottom portion of the Shallow Zone is fine-grained and confines the underlying Upper Producing Zone (UPZ). The Shallow Zone is not a primary aquifer and is not developed for water supply purposes.

The UPZ underlies the Shallow Zone and is composed of medium to coarse sand with some fine gravel and is generally continuous throughout Storage Unit I (Martin, 1984). The UPZ is underlain by the Middle Zone, a fine-grained confining layer that makes up the upper part of the Santa Barbara Formation. The Lower Producing Zone (LPZ) is composed of medium to coarse sand with fine gravel and shell fragments (Martin, 1984). Most of the groundwater pumping in Storage Unit I is from the LPZ at inland wells located north of Highway 101.

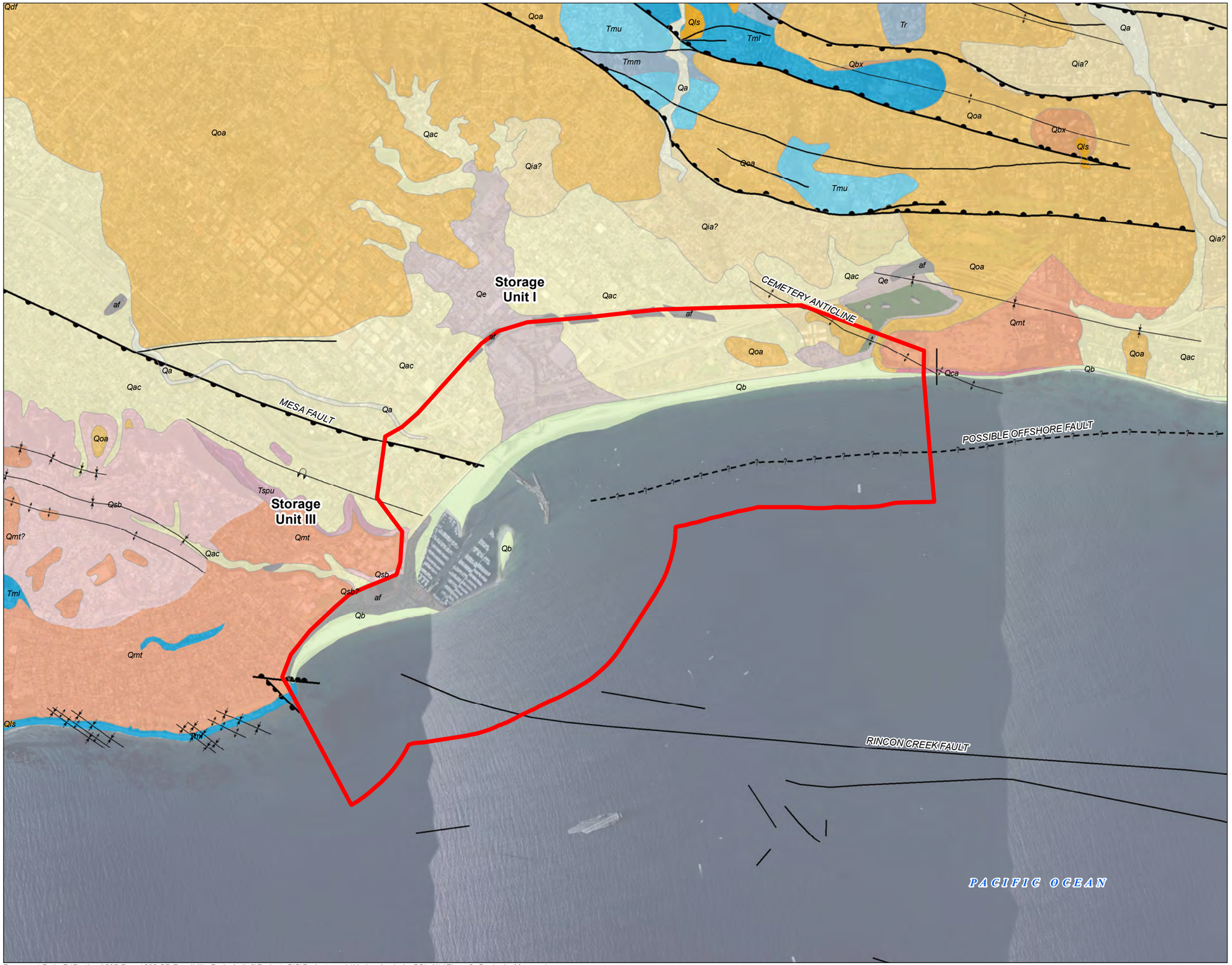

FIGURE 2

Geologic Map

Hydrogeologic Analysis of
SSI Alternatives
Santa Barbara, California

LEGEND

Study Area

Folds

- Anticline
- Overturned anticline
- Syncline

Faults

- Normal Fault
- Reverse Fault

Surficial Geology

- af - Artificial fill
- Qa - Active channel alluvium
- Qb - Beach deposits (Holocene)
- Qbx - Shale-clast sedimentary breccia (middle Pleistocene)
- Qdf - Debris flow deposits (Holocene and upper Pleistocene?)
- Qe - Estuarine deposits (Holocene)
- Qac - Alluvium and colluvium (Holocene and upper Pleistocene)
- Qia - Intermediate alluvial deposits (upper Pleistocene)
- Qls - Landslide deposits (Holocene and Pleistocene)
- Qmt - Marine-terrace deposits (upper Pleistocene)
- Qoa - Older alluvial deposits (upper and middle Pleistocene)
- Qca - Casitas Formation (upper and middle Pleistocene)
- Qsb - Santa Barbara Formation (middle and lower Pleistocene)
- Tml - Monterey Formation, lower calcareous unit (middle and lower Miocene)
- Tmm - Monterey Formation, middle shale unit (upper and middle Miocene)
- Tmu - Monterey Formation, upper siliceous unit (upper Miocene)
- Tr - Rincon Shale (lower Miocene)
- Tspu - Sespe Formation, upper sandstone and mudstone unit (upper Oligocene)

N

0 1,000 2,000
Feet

MAP NOTES:
Date: December 28, 2015
Data Sources: USGS (Minor et al, 2009), USGS (Johnson et at, 2013)

GSI Water Solutions, Inc.

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The target zone within Storage Unit I for SSI alternatives that utilize wells is limited to the Shallow Zone (upper [beach] sand and lower sand layers). The UPZ and LPZ are not considered target zones for SSIs due to their limited hydraulic connection to seawater and because these aquifers are the primary aquifers supporting City production wells. The vertical connectivity of the UPZ to the ocean is greatly limited by a series of confining layers at the base of the Shallow Zone that limit vertical infiltration of seawater. This is corroborated by the low vertical conductivity of this unit (approximately 0.02 to 0.04 feet per day (feet/day) utilized in the calibrated USGS Santa Barbara Groundwater Model (USGS, in press). The vertical connectivity of LPZ to the ocean is further limited by the Middle Zone, which is another aquitard. The ability to extract seawater laterally through the UPZ or LPZ may be limited by an offshore fault that is believed to juxtapose these aquifers against low permeability bedrock. Calibration of the USGS Santa Barbara Groundwater Model required the use of such a very low conductance boundary at the inferred offshore fault location (USGS, in press). While the presence of the offshore fault remains in question (see discussion in Section 4.2.3), the model calibration results suggest that lateral continuity of the UPZ and LPZ offshore is limited by some sort of geologic feature, whether that is a fault or a lithologic change. In either case, the result is limited lateral connectivity of the UPZ and LPZ to the ocean, which inhibits the potential seawater production capacity of these zones. For these reasons, the focus of the SSI evaluation was on the Shallow Zone.

3.2 Development of Geologic Cross Sections

To understand the geology in the study area, and determine specific target zones within the Shallow Zone for each of the SSI alternatives, seven geologic cross sections were developed based on a review of existing subsurface data. The results of the geologic cross section development were integral to the development of the hydrogeologic conceptual model of the study site. The spatial arrangement of available data was analyzed to establish cross section alignments that would illustrate the geometry and lithology of the Shallow Zone, and the distribution of aquifer materials therein. Lines of cross section are arranged to present as much information at the SSI project area as the data allow. In general, the spatial arrangement of available data is denser inland from the beach, especially at depths greater than 30 feet below ground surface (bgs). Therefore several cross sections running perpendicular to the coast are presented to assess changes in lithology/geometry moving from the onshore to the offshore environment. The arrangement of cross sections and available data locations are provided in Figure 3; the cross sections are provided in Figures 4 to 10.

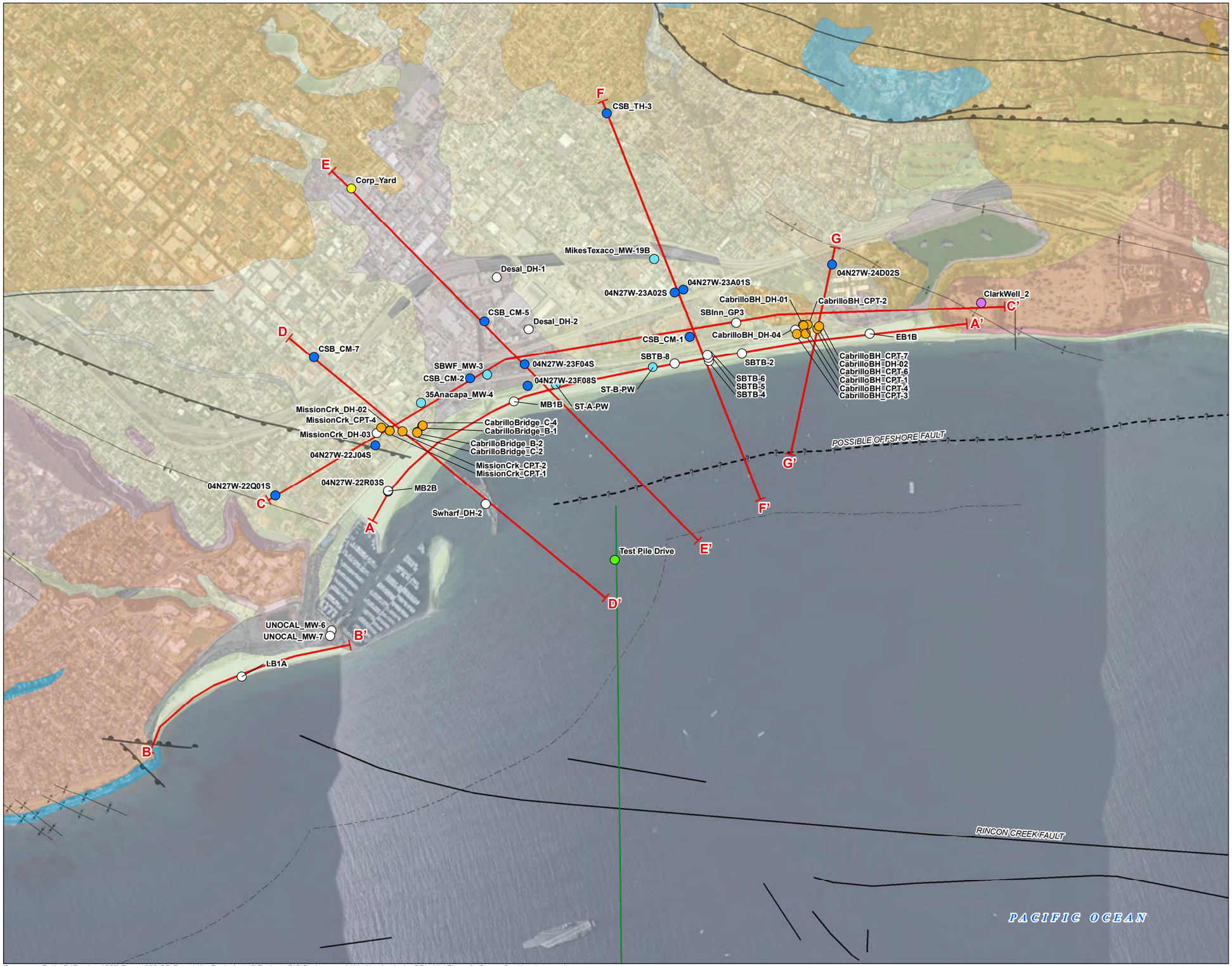
3.2.1 Data Sources Utilized

A literature review was conducted to identify available data sources for the development of geologic cross sections in the study area and to derive hydraulic properties for the SSI yield analysis. Data sources include; borehole logs, cone penetration tests (CPTs), test pile drive analyses, surficial geologic maps, other published cross sections, offshore geophysical survey data, a coastal bathymetry survey, historical aerial photographs, and various geologic reports from public and private sources. Each data source received a thorough review and was then ranked by data quality. Data sources ranked as “1” are considered empirical, primary data sources of high quality and are given the highest priority (i.e., borehole logs completed by a geologist). Data sources ranked as “2” are considered either inferred, primary data sources of high quality (i.e., CPT or seismic data) or empirical, primary data sources of unknown quality (i.e., Bache, 1853). Data sources ranked as “3” are considered secondary data sources or as a ‘working theory’ without direct supporting evidence (i.e., Muir’s possible offshore fault). A complete list of data sources consulted for this study and their rankings are shown in Table 1.

FIGURE 3

Cross Section Locations

Hydrogeologic Analysis of
SSI Alternatives
Santa Barbara, California



LEGEND

- Cross Section Line
- Boring
- CPT
- Pile Drive
- City Production Well
- Monitoring Well
- USGS Monitoring Well
- Private Well

All Other Features

- Santa Barbara City Limits 1/2 Mile Offshore
- SBC-109 Seismic Line

N

0 750 1,500 2,250
Feet

MAP NOTES:
Date: December 28, 2015
Data Sources: See Table 1

GSI Water Solutions, Inc.

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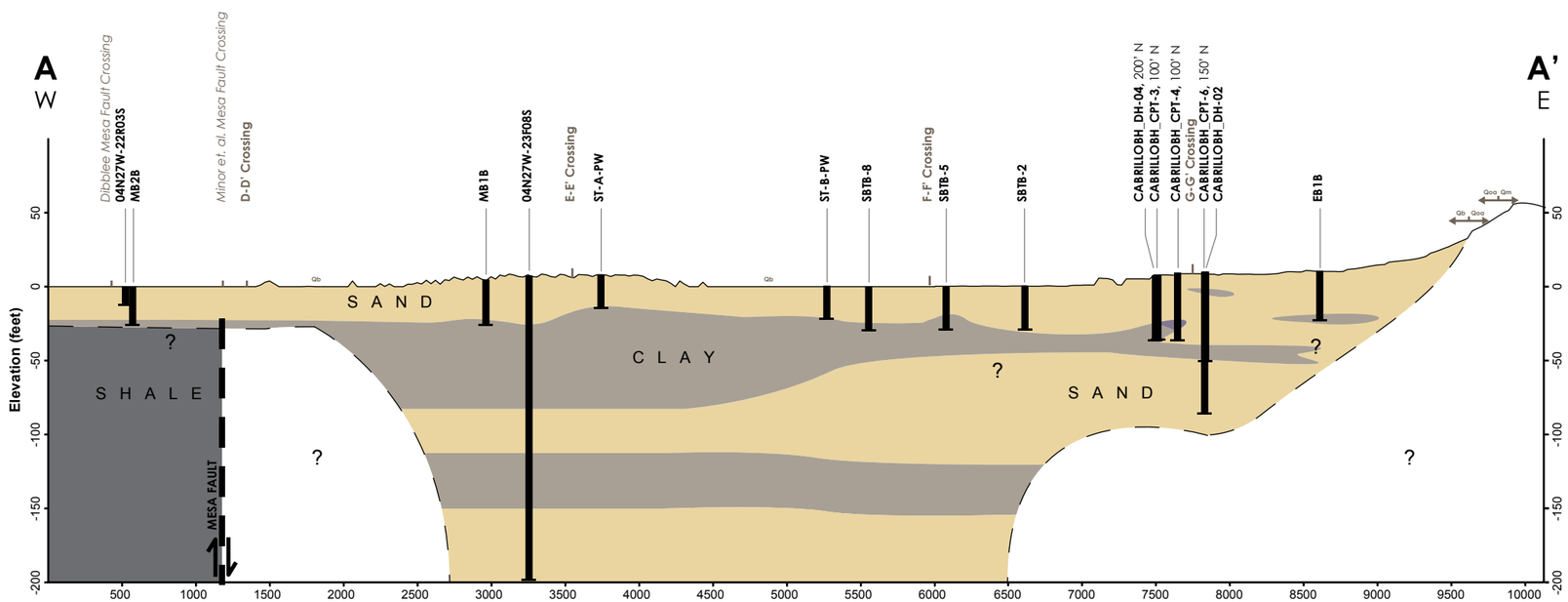

FIGURE 4

Cross Section A-A'

Hydrogeologic Analysis of
SSI Alternatives
Santa Barbara, California

LEGEND

Ground Surface

Boring/Well

Geology

Sand

Silt

Clay

Shale (Bedrock)

NOTES:

Qb - Beach deposits (Holocene)

Qmt - Marine-terrace deposits (upper Pleistocene)

Qoa - Older alluvial deposits (upper and middle Pleistocene)

SCALE

10x Vertical Exaggeration

100 feet

1000 feet

NOTES:

Date: December 28, 2015

Data Sources: See Table 1

GSI Water Solutions, Inc.

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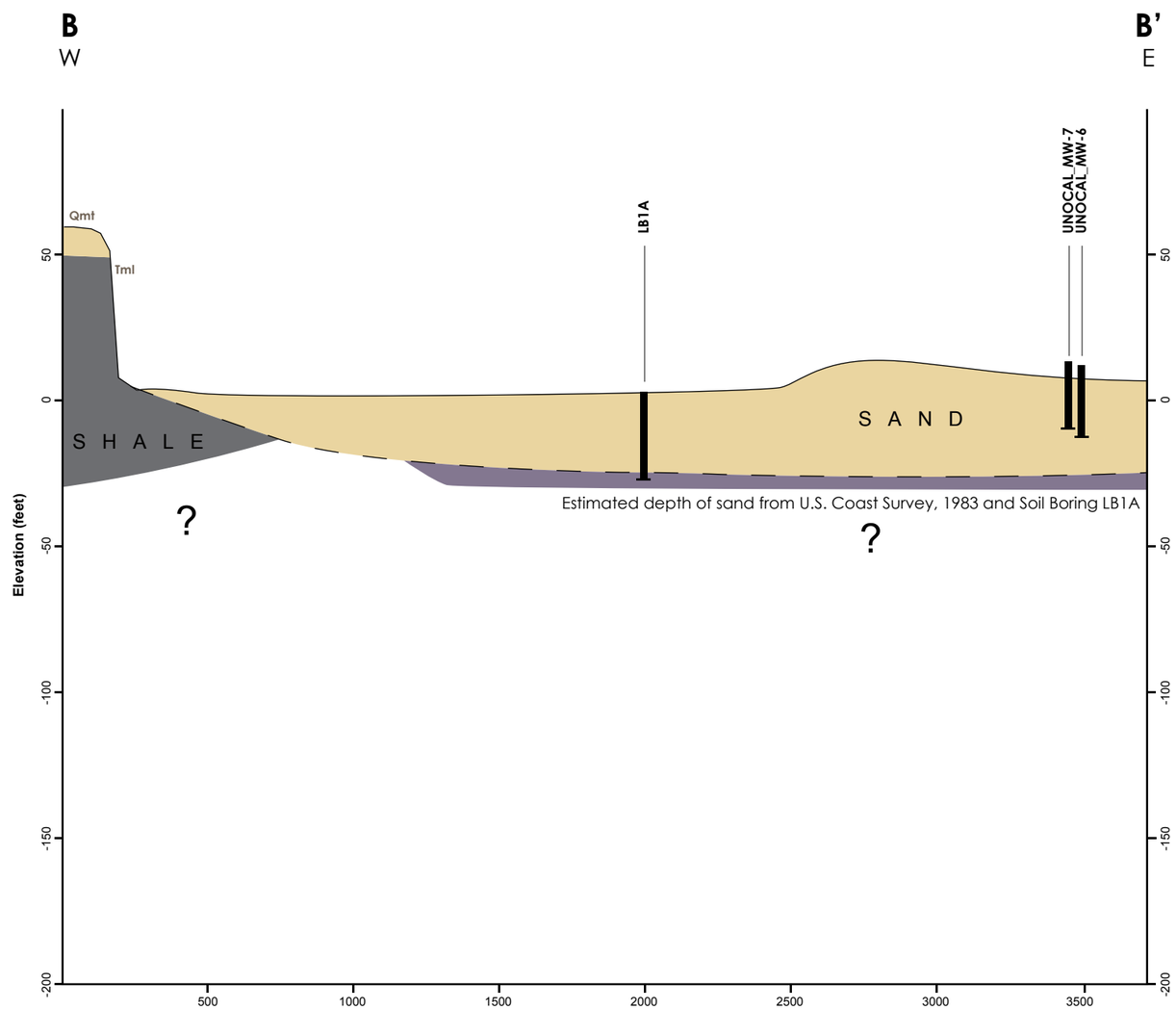

FIGURE 5

Cross Section B-B'

Hydrogeologic Analysis of
SSI Alternatives
Santa Barbara, California

LEGEND

Ground Surface

Boring/Well

Geology

Sand

Silt

Clay

Shale (Bedrock)

NOTES:

Qmt - Marine-terrace deposits (upper Pleistocene

Tml - Monterey Formation, upper siliceous unit (upper Miocene)

SCALE

50 feet

10x Vertical Exaggeration

500 feet

NOTES:

Date: December 7, 2015

Data Sources: See Table 1

GSI Water Solutions, Inc.

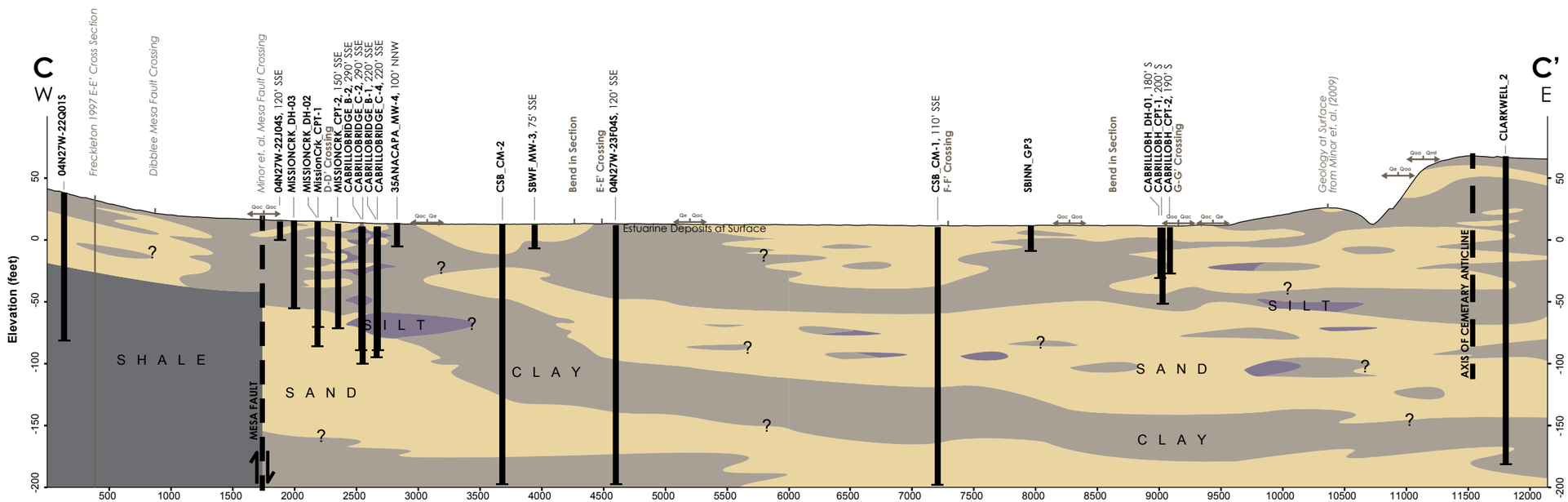

FIGURE 6

Cross Section C-C'

Hydrogeologic Analysis of
SSI Alternatives
Santa Barbara, California

LEGEND

Ground Surface

Boring/Well

Geology

Sand

Silt

Clay

Shale (Bedrock)

NOTES:

Qe - Estuarine deposits (Holocene)

Qac - Alluvium and colluvium (Holocene and upper Pleistocene)

Qmt - Marine-terrace deposits (upper Pleistocene)

Qoa - Older alluvial deposits

SCALE

100 feet

10x
Vertical
Exaggeration

1000 feet

NOTES:

Date: December 28, 2015

Data Sources: See Table 1

GSI Water Solutions, Inc.

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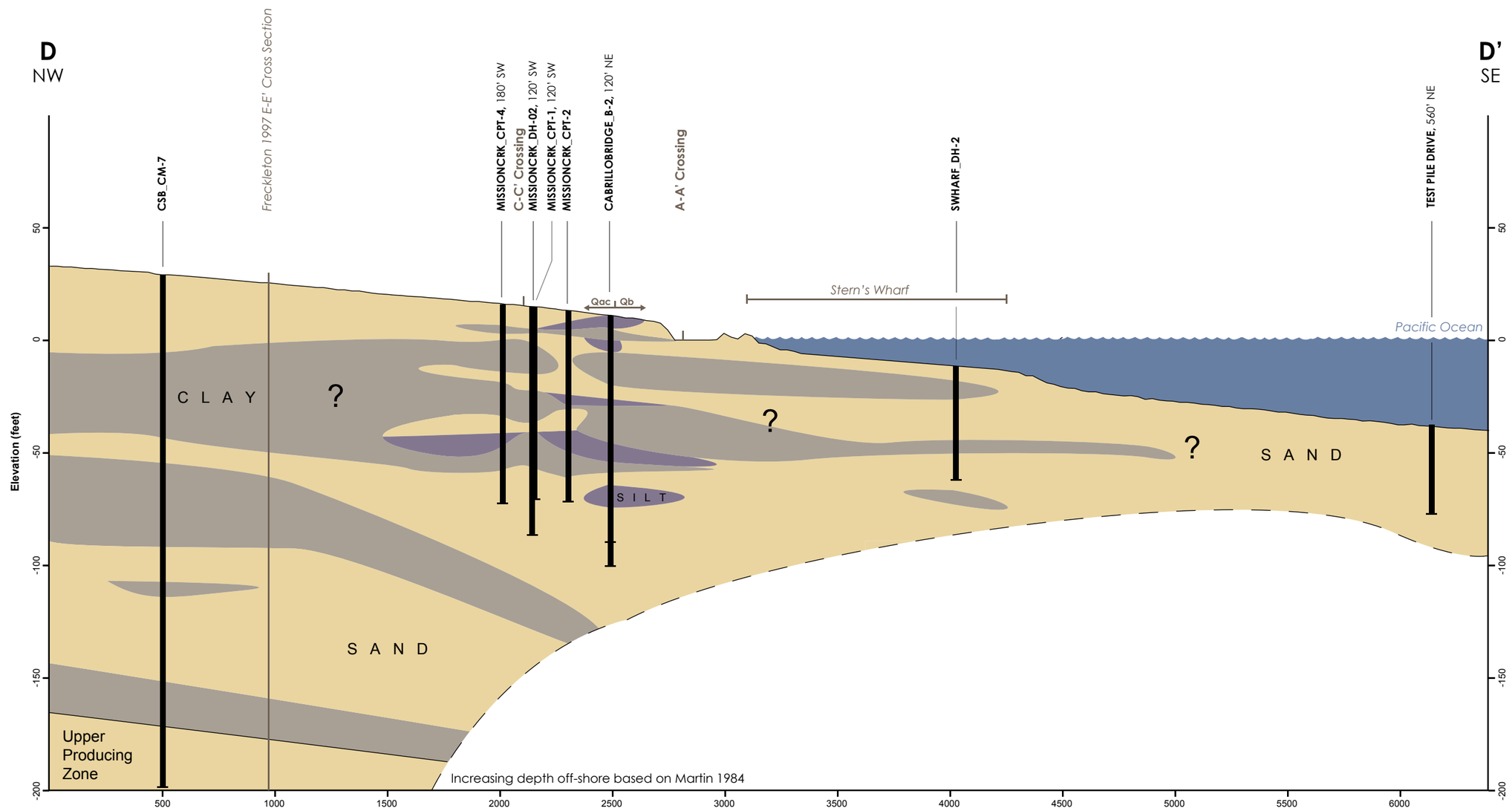

FIGURE 7

Cross Section D-D'

Hydrogeologic Analysis of
SSI Alternatives
Santa Barbara, California

LEGEND

- Ground Surface
- Boring/Well

Geology

- Sand
- Silt
- Clay
- Shale (Bedrock)

NOTES:
Qb - Beach deposits (Holocene)
Qac - Alluvium and colluvium (Holocene and upper Pleistocene)

SCALE

50 feet
10x Vertical Exaggeration
500 feet

NOTES:
Date: December 7, 2015
Data Sources: See Table 1

GSI Water Solutions, Inc.

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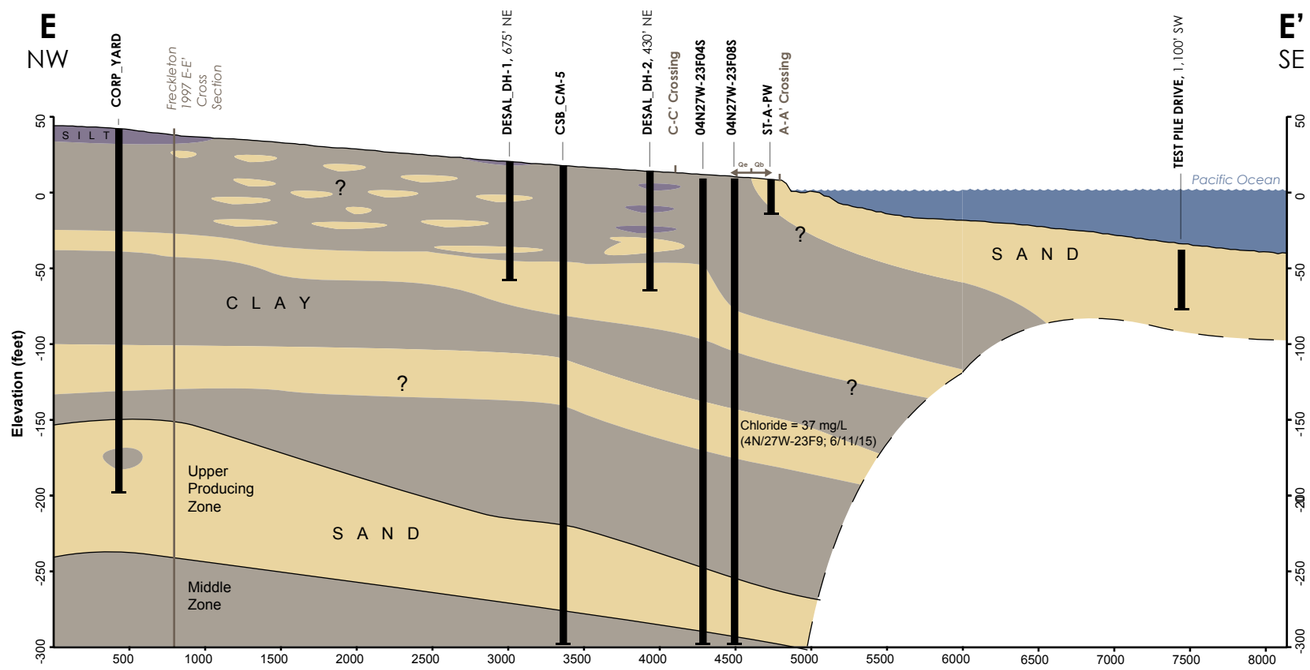

FIGURE 8

Cross Section E-E'

Hydrogeologic Analysis of
SSI Alternatives
Santa Barbara, California

LEGEND

- Ground Surface
- Boring/Well

Geology

- Sand
- Silt
- Clay
- Shale (Bedrock)

NOTES:
Qb - Beach deposits (Holocene)
Qe - Estuarine deposits (Holocene)

SCALE

100 feet

10x
Vertical
Exaggeration

1000 feet

NOTES:
Date: December 28, 2015
Data Sources: See Table 1

GSI Water Solutions, Inc.

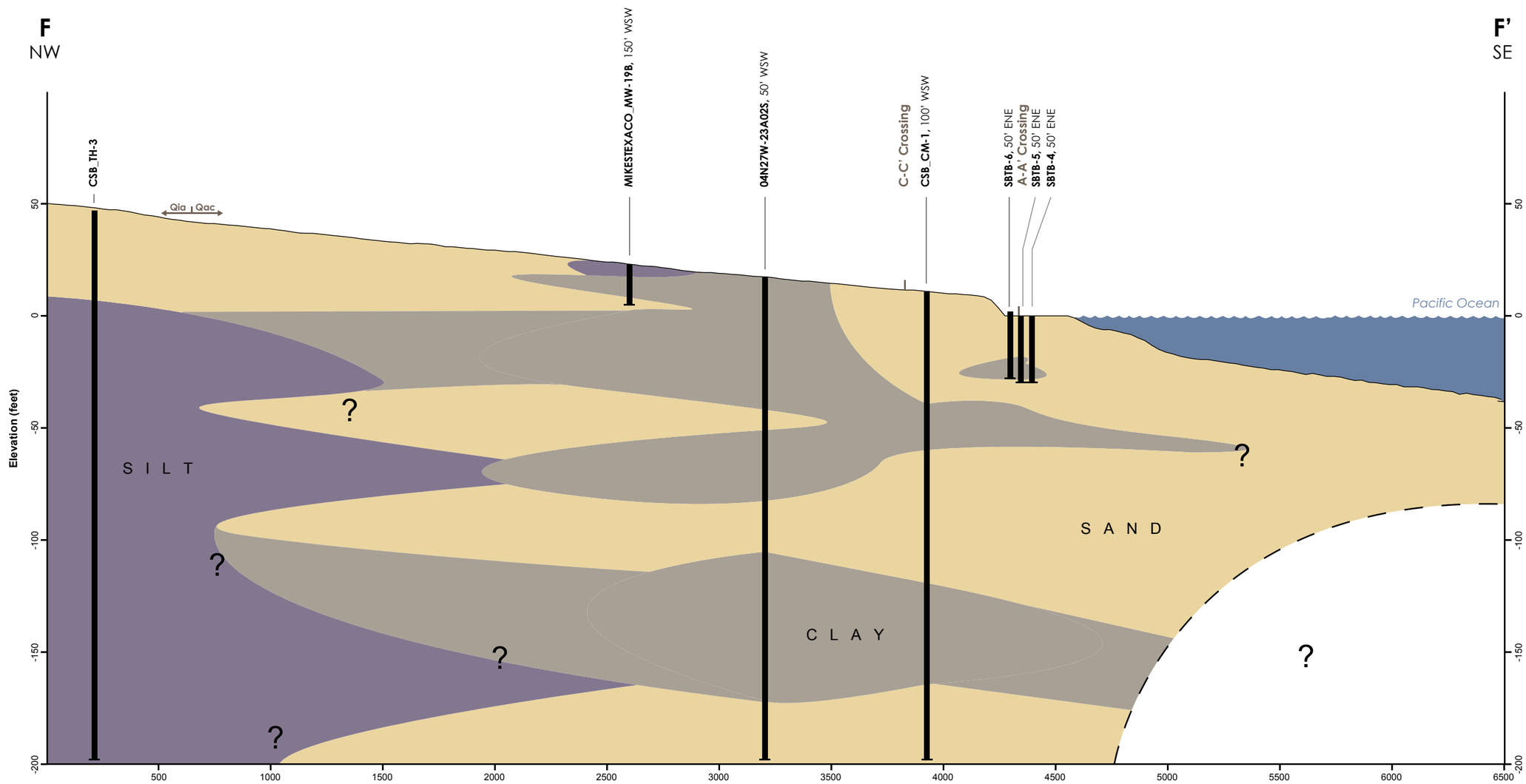

FIGURE 9

Cross Section F-F'

Hydrogeologic Analysis of
SSI Alternatives
Santa Barbara, California

LEGEND

- Ground Surface
- Boring/Well

Geology

- Sand
- Silt
- Clay
- Shale (Bedrock)

NOTES:
Qac - Alluvium and colluvium (Holocene and upper Pleistocene)
Qia - Intermediate alluvial deposits (upper Pleistocene)

SCALE

50 feet
10x Vertical Exaggeration
500 feet

NOTES:
Date: December 7, 2015
Data Sources: See Table 1

GSI Water Solutions, Inc.

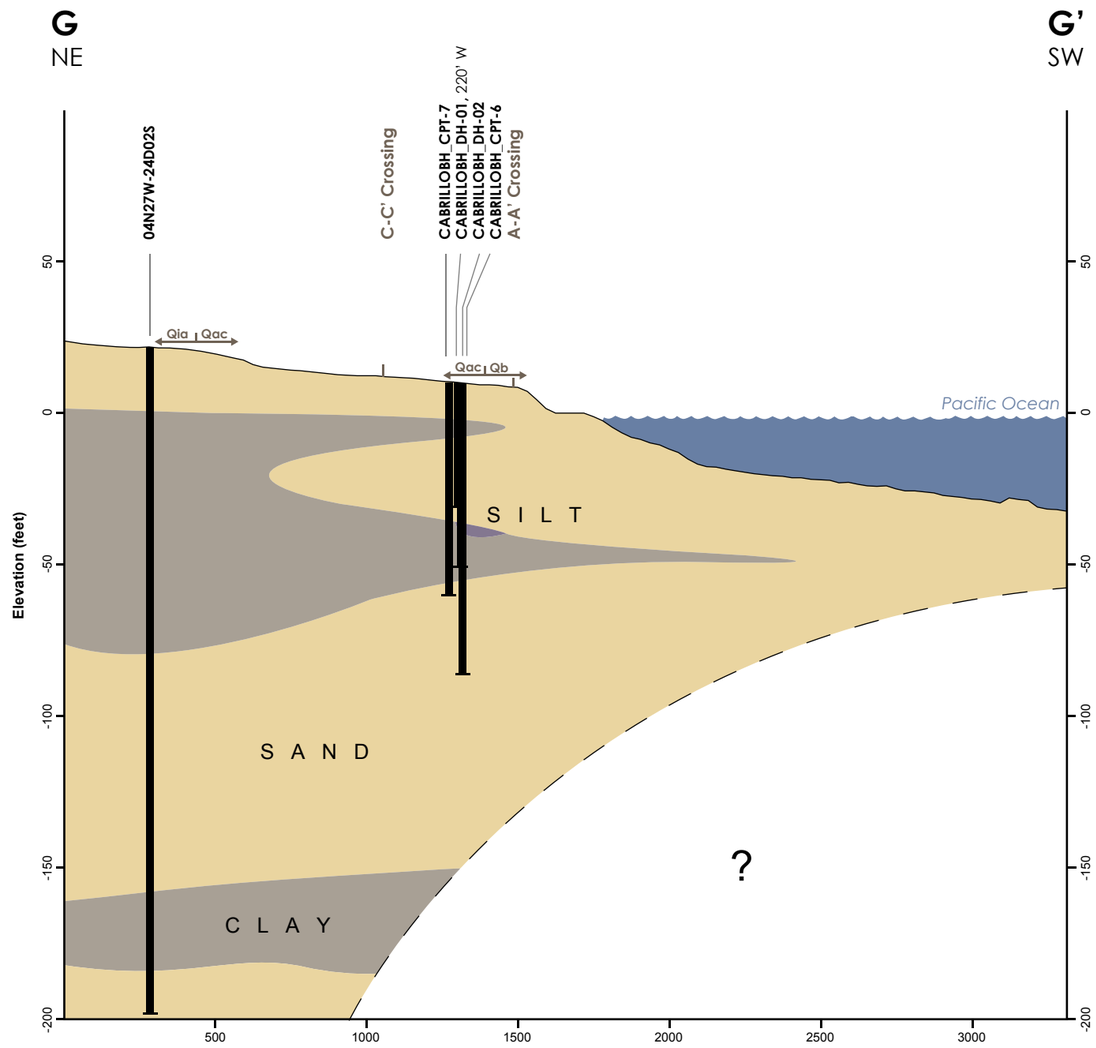

FIGURE 10

Cross Section G-G'

Hydrogeologic Analysis of
SSI Alternatives
Santa Barbara, California

LEGEND

Ground Surface

Boring/Well

Geology

Sand

Silt

Clay

Shale (Bedrock)

NOTES:

Qb - Beach deposits (Holocene)

Qac - Alluvium and colluvium (Holocene and upper Pleistocene)

Qia - Intermediate alluvial deposits (upper Pleistocene)

SCALE

50 feet

500 feet

10x Vertical Exaggeration

NOTES:

Date: December 7, 2015

Data Sources: See Table 1

GSI Water Solutions, Inc.

Table 1. Data Sources Used for Development of Geologic Cross Sections

Data Source	Information Provided		Data Rank[1]
General Data Sources			
Minor et al., 2009	Ground surface geology and orientation of the Mesa Fault and cemetery anticline		1,2
Martin, 1984	Upper Producing Zone (UPZ) picks on the CM-2 borehole log and general geometry of unconsolidated materials within Storage Unit I (thickening/increasing depth toward ocean and Mesa Fault)		1,2
Johnson et al., 2013	Figure 7 (SBC-109 seismic line) shows approximately 50 feet of horizontally layered material in area just seaward of Possible Offshore Fault. The entire SBC-109 dataset was used to define seafloor bathymetry.		2
Freckleton et al., 1998	UPZ depths from E-E' cross section and general geometry of unconsolidated materials in Storage Unit III		2
Dibblee, 1986	Orientation of the Mesa Fault		2
Bache, 1853	Contoured depths to seafloor in area of Leadbetter Beach prior to installation of harbor breakwater structures		2
Muir, 1968	Possible Offshore Fault		3
Special Collections - UCSB Library, 1928, 1929, and 1938	Aerial photos showing kelp beds in approximate alignment with "Muir's Fault" alignment		3
Borehole Log Data Sources			
		Borehole Log ID	
USGS borehole logs, various dates	Lithologic log of boreholes within study area (9 total)	CSB_CM-1 (1978) CSB_CM-5 (?) CSB_CM-7 (1987) CSB_TH-3 (1979) 04N27W-22J04S (2005) 04N27W-22Q01S (1979) 04N27W-22R03S (2005) 04N27W-23A02S (1976) 04N27W-23F04S (1986)	1
Hutchinson, 1979	Borehole log for CM-2	CSB_CM-2	1
CH2M Hill, 1989	Borehole logs on East Beach to 30 feet depth and hydrologic testing results.	SBTB-2 SBTB-4 SBTB-5 SBTB-6 SBTB-8	1
CH2M Hill, 1990	Borehole logs on Leadbetter, West, and East Beaches	EB1B LB1A MB1B MB2B ST-A-PW ST-B-PW	1

Data Source	Information Provided		Data Rank[1]
SGD, 1991a	Lithologic log of boreholes at Santa Barbara Desalination Plant	Desal_DH-1 Desal_DH-2	1
SGD, 1991b	Analysis of off-shore test pile drive	Test Pile Drive	2
DWR Water Well Completion Report No. 487527	Lithologic log of borehole for Clark Well #2	ClarkWell_2	1
Dames & Moore, 1992	Lithologic log of borehole at Chase Palm Park	SBWF_MW-3	1
GeoResearch, 1995	Lithologic log of boreholes at Santa Barbara Harbor	Unocal_MW-6 Unocal_MW-7	1
Fugro, 1999	Lithologic log of offshore borehole on Stearns Wharf	Swharf_DH-2	1
DMI-EMK, 2005	Lithologic log of borehole at Mike's Texaco	MikesTexaco_MW-19B	1
Bengal Engineering, 2005	Lithologic log of boreholes (2 total) and CPTs (2 total) at Cabrillo bridge over Mission Creek	CabrilloBridge_B-1 CabrilloBridge_B-2 CabrilloBridge_C-2 CabrilloBridge_C-4	1,2
Fugro, 2009	Lithologic log of boreholes (2 total) and CPTs (3 total) near Mission Creek between Mason and State Street	MissionCrk_DH-2 MissionCrk_DH-3 MissionCrk_CPT-1 MissionCrk_CPT-2 MissionCrk_CPT-4	1,2
Pueblo, 2013	Lithologic log, geophysical logs, and picks of UPZ, Middle Zone, and LPZ for Corporation Yard Replacement Well	Corp_Yard	1
LE&A, 2013	Lithologic log of borehole at Rand Trust Property	35Anacapa_MW-4	1
DMI-EMK, 2014	Lithologic log of borehole at Santa Barbara Inn	SBInn_GP3	1
Fugro, 2015	Lithologic log of boreholes (3 total) and cone penetration tests (CPTs) (6 total)	CabrilloBH_DH-1 CabrilloBH_DH-2 CabrilloBH_DH-4 CabrilloBH_CPT-1 CabrilloBH_CPT-2 CabrilloBH_CPT-3 CabrilloBH_CPT-4 CabrilloBH_CPT-6 CabrilloBH_CPT-7	1,2

Notes: [1] Data Rank: (1) empirical primary data source of high quality, (2) inferred primary data source of high quality or empirical primary data source of unknown quality, (3) secondary data source

3.2.2 Methods

Borehole logs were compiled from published sources, unpublished consultant reports, and individual well completion reports, and entered into a lithologic database using the Unified Soil Classification System (USCS). USCS material assignments were either copied directly from the borehole log, if available, or assigned based on material descriptions and/or sieve analysis results. All interpretations based on material descriptions were considered with regard to hydrologic material properties, therefore only clean sands and gravels were assigned as sands and gravels (SP, SW, or GP, GW) while descriptions indicating greater than 15% fines of either silt or clay were assigned as silts (ML, MH, SM, or GM) or clays (CL, CH, SC, or GC), respectively. USCS material assignments were simplified further as either sand, silt, or clay during lithologic correlation of the cross sections as shown in Figures 4 through 10. A summary of the materials assigned to each material type is presented as Table 2. The lithologic database and lines of cross section were input into GIS to produce section profiles with ground surface elevations and lithologic contact depths in each borehole.

Table 2. USCS Material Descriptions Simplified for Cross Section Correlation.

USCS Assignment	Maps to
SP SW GP GW	Sand
ML MH SM GM	Silt
CL CH SC GC	Clay

Lithologic contact depths based on CPT data are also plotted on section profiles and used to fill in data gaps between boreholes (Bengal Engineering, 2005 and Fugro, 2009 and 2015). Because lithologic materials from CPT signatures are inferred based on indirect material measurements, these data were used primarily for determination of depths of lithologic contacts, while descriptions of those materials were assigned based on lithologic samples from borehole logs for the cross sections.

Other data sources considered include UPZ depths from the CM-2 borehole (Martin, 1984) and from Freckleton's E-E' cross section (Freckleton, et al, 1998). Surficial geology and inferred orientation of the Mesa Fault and cemetery anticline are considered based on Minor et al., (2009). Offshore high-resolution seismic data (Johnson et al., 2013) and an offshore test pile drive analysis (SGD, 1991b) provide data seaward of Stearn's Wharf and are considered in relation to Muir's "possible offshore fault" (1968). Historical aerial photos of the Santa Barbara waterfront showing kelp beds in approximate alignment with Muir's "possible offshore fault" were also considered (UCSB Library, 1928, 1929, and 1938).

Leadbetter Beach has changed significantly in response to construction of the Santa Barbara Harbor breakwater, finished in 1930, and now extends approximately 1,000 feet further offshore at its eastern end due to sand deposition from the longshore currents slowed by the Harbor breakwater. A US Coast Survey from 1853 provides the only known pre-breakwater bathymetry in the Leadbetter Beach area (Bache, 1853) and has been used to approximate the volume of infilled sand.

The geologic interpretation presented on the cross sections was correlated based on existing data while honoring the ranking of each data type. Locations of the cross sections are presented as Figure 3 and the cross sections are presented as Figures 4 through 10.

3.2.3 Discussion of Geologic Cross Sections

Lithologic correlation between deeper boreholes in cross sections running perpendicular to the shore (D-D' through G-G') provide the overall geometric concept of plunging/thickening of the Shallow Zone toward offshore, in agreement with Martin (1984). The cross sections running perpendicular to shore provide insight into the data-sparse deeper portions of cross section A-A'. In general, the geologic cross sections document the beach sand wedge as uninterrupted along the beach fronts to a depth of approximately 30 feet (A-A' and B-B') and inland 200 feet (E-E') to 1,000 feet (F-F'). Inland from the beach sand wedge the geologic cross sections show discontinuous water bearing sand units interspersed with fine-grained materials, possibly related to historical estuarine deposits and laterally migrating stream channels. The distribution of unconsolidated materials in inland areas is consistent with published descriptions of the Shallow Zone (Martin, 1984 and Freckleton et al., 1998).

The base of the Shallow Zone is represented on geologic cross sections D-D' and E-E' at approximately 200 – 250 feet bgs in the West and East Beach areas. In general, the Shallow Zone consists of a beach sand wedge or "upper sand" that is approximately 30 feet thick that is underlain by two to three sand layers (referred to in this report as "lower sand"), separated by fine-grained layers. Although the thicker sand and clay layers appear to be generally continuous inland, their spatial arrangement and continuity as they extend offshore is unknown.

Of particular interest to the offshore environment is the presence or absence of the possible offshore fault (Figures 2 and 3). This fault, if it exists, would bring low permeability bedrock closer to the ocean bottom and limit the hydraulic connection of the Shallow Zone with the ocean. Arguments in support of the presence of this fault include the apparent historical presence of kelp beds in alignment with the proposed location of the fault (UCSB Library, 1928, 1929, and 1938)[3] and the fact that calibration of the USGS Santa Barbara Groundwater Model required a low conductivity boundary condition in the vicinity of this fault to aid in calibration (USGS, in press). However, data evaluated as part of this study suggests that the fault may not exist or at least does not affect the upper 50 feet of unconsolidated materials on the seafloor based on a high-resolution offshore seismic survey (Johnson et al., 2013) and a test pile drive analysis (SGD, 1991b), that were both completed just offshore of the possible fault. Together, the seismic and test pile studies indicate that at least 50 feet of unconsolidated materials exist on the apparent up-thrown side (south side) of this possible fault where bedrock would be exposed on the seafloor according to Muir (1968). It should be noted that the seismic and test pile studies are not able

[3] Kelp can only root on hard substrate such as bedrock, not sediments or unconsolidated seafloor sands. Kelp shown in historical aerial photos indicates the presence of either bedrock substrate or cobbles large enough to provide kelp anchorage.

to conclusively differentiate between unconsolidated material types, but are considered strong evidence for the lack of bedrock in the upper approximately 50 feet of materials beneath the seafloor at these locations.

4 Hydrogeologic Analysis of Subsurface Intake Systems

The yield, spacing, and number of wells required to produce 15,898 gpm for each SSI alternative were evaluated using analytical or numerical (MODFLOW) methods based on our understanding of the geometry and nature of the aquifer materials, as described in the following sections. The subsurface intake alternatives considered include:

1. Vertical wells,
2. Beach infiltration galleries (BIG),
3. Radial collector wells (also known as 'Ranney wells'),
4. Slant wells,
5. Seabed infiltration gallery (SIG), and
6. Horizontal directionally drilled wells (HDD).

4.1 Evaluation Approach

Vertical wells, (onshore) beach infiltration galleries (BIGs), radial collector wells and slant wells were analyzed using simplified numerical modeling methods to estimate achievable yields, optimal number and spacing of facilities, proportion of inland groundwater versus seawater capture, as well as potential impacts on sensitive habitats. The two remaining intake alternatives (SIGs and HDDs) were evaluated using analytical methods. The numerical model was utilized where flow paths are expected to involve some degree of horizontal flow, while the analytical modeling was used where flow is expected to be primarily vertical.

Based on cross sections and conceptual groundwater model described in Section 4, the simplified numerical model layers and assumed hydraulic properties used for both the numerical and analytical methods for areas inland of the beach, on the beach, and offshore are presented in Table 3.

Table 3 Groundwater Model Layering

					Area Inland of Beach			On the Beach & Offshore		
Layer Number	**Elevation Top (feet)**	**Elevation Bottom (feet)**	**Thickness (feet)**	**Intake Alternative**	**Aquifer Unit**	**Kh (ft/d)**	**Kz (ft/d)**	**Aquifer Unit**	**Kh (ft/d)**	**Kz (ft/d)**
1-6	0 (Water Table)	-30	30	BIG Radial Wells SIG HDD	Clay and Sand	9	0.9	Upper Sand	55	11
7-8	-30	-60	30	None	Clay	1	0.1	Clay	1	0.1
9-11	-60	-120	60	Vertical wells Radial Wells Slant Wells	Clay and Sand	9	0.9	Lower Sand	55, 5	11, 0.5
12	-120	-150	30	None	Clay and Sand	1	0.1	Clay	1	0.1
13	-150	-190	40	None	Sand	9	0.9	Sand	55, 5	11, 0.5
14	-190	-200	10	None	Clay and Sand	1	0.1	Clay	1	0.1
Underlying	Upper Producing Zone (Not Modeled)									

Notes: Kh: Horizontal Hydraulic Conductivity
Kz: Vertical Hydraulic Conductivity
K values for the lower sands were modelled at 55 and 5 feet per day (refer to Section 4.1.1)

The intake depth of each SSI alternative varies based on the specific technology and the Shallow Zone hydrostratigraphy. Generally:

- Beach infiltration galleries (BIG) are assumed to consist of lateral screens that are designed to draw in seawater laterally within shallow sediments adjacent to the ocean. BIG's are therefore assumed to be completed at the base of the upper sand layer to a depth of no more than 30 feet (the typical depth of the first underlying clay unit).
- Vertical wells and slant wells are assumed to be completed in the lower sand layer of the Shallow Zone, which in the beach area typically lies at an elevation interval from -60 feet (top of sand) to -120 feet. These SSI alternatives cannot be completed in the shallow sand because the sand layer is too thin to provide for adequate pumping drawdown and a sufficiently long surface sanitary seal.
- Radial collector wells were assumed to be completed in either the upper or lower sand layer.
- For the analytical modeling, the lateral screens of the SIG and the entirety of the HDD wells are assumed to be completed within the upper sand layer.

4.1.1 Numerical Modelling

A simplified numerical model was constructed using the U.S. Geological Survey (USGS) MODFLOW-NWT numerical modelling package (Niswonger and others, 2011) and the Groundwater Vistas (GV) graphical user interface (ESI, 2011). The remainder of this section discusses the numerical modeling approach and construction of the model.

The four subsurface intake systems evaluated in the simplified model rely on some degree of horizontal flow in the aquifer to capture seawater. This horizontal flow aspect also results in these facilities capturing a portion of groundwater derived from upgradient inland areas because of the existence of fine grained confining layers between the ocean bottom and sand layers. Because these intakes are located either nearshore or onshore, they have the potential to affect nearby sensitive beach habitats due to the drawdown that they create.

Simplified numerical modeling was performed for these intake alternatives because the method:

1. Accounts for spatial differences in aquifer transmissivity between inland, beach, and offshore areas;
2. Provides a three-dimensional representation of groundwater flow conditions within the Shallow Zone, which is necessary to account for the placement of subsurface intake screens at different depths intervals; and
3. Accounts for drawdown interference effects between each constructed facility in a more realistic manner than can be achieved by applying two-dimensional analytical methods (which cannot readily account for spatial variability in the physical and hydraulic properties of an aquifer system).

The numerical model simulated the length of beach in the study area, which allowed for simultaneous modeling of multiple locations where intakes could be located outside of construction setback areas where no intakes will be constructed (to avoid existing facilities, and to provide protection of nearby sensitive habitats) (Figures 11 through 13). Exclusion areas surrounding sensitive habitats, existing facilities, and areas prone to erosion are presented on Figures 11 through 13. Five of the available beach construction areas ranged between 500 feet and 1,300 feet in length, while the sixth area (on East Beach) was simulated as 4,400 feet long. The total length of available beach (outside of exclusion areas) is estimated to be up to 9,000 feet.

The model domain extends from a distance 2,000 feet inland of the mean tide line (generally north) to a distance 2,500 feet offshore. The grid spacing is 25 feet by 25 feet throughout the model domain, but is reduced to 12.5 feet near the shoreline to better simulate the narrow width of onshore beach infiltration galleries. No-flow boundaries are used around the model perimeter, except at the upgradient inland model boundary, where a specified head of 15.5 feet is established to provide a horizontal hydraulic gradient of approximately 0.007 feet per foot, which is at the upper end of the estimated range (0.003 to 0.007) as inferred from water level data from environmental site wells downloaded from GeoTracker. The offshore no-flow boundary is not intended to represent a geologic boundary; rather, the boundary is the limit of the calculation domain. The Shallow Zone is believed to extend further offshore, but the model domain was truncated to simplify the model and reduce simulation runtimes. This truncation does not create a material impact on the simulation results.

FIGURE 11

Leadbetter Beach Construction Setbacks and Contaminated Sites

Hydrogeologic Analysis of
SSI Alternatives
Santa Barbara, California

LEGEND

- GeoTracker Sites (closed)
- Construction Setbacks

N

0 200 400 600
Feet

MAP NOTES:
Date: December 28, 2015
Data Sources: CA SWRCB GeoTracker (2015)

GSI Water Solutions, Inc.

Document Path: P:\Portland\597-Fugro\002-SB Feasibility Proj - Auth 2\Project_GIS\Project_mxds\Hydro_Analysis_SSI_Alts\Figure11_Leadbetter_Beach.mxd

FIGURE 12

West Beach Construction Setbacks and Contaminated Sites

Hydrogeologic Analysis of
SSI Alternatives
Santa Barbara, California

LEGEND

- GeoTracker Sites (open)
- GeoTracker Sites (closed)
- Sensitive Habitat
- Construction Setbacks

N

0 250 500
Feet

MAP NOTES:
Date: December 28, 2015
Data Sources: CA SWRCB GeoTracker (2015)

GSI Water Solutions, Inc.

Document Path: P:\Portland\597-Fugro\002-SB Feasibility Proj - Auth 2\Project_GIS\Project_mxds\Hydro_Analysis_SSI_Alts\Figure12_West_Beach.mxd

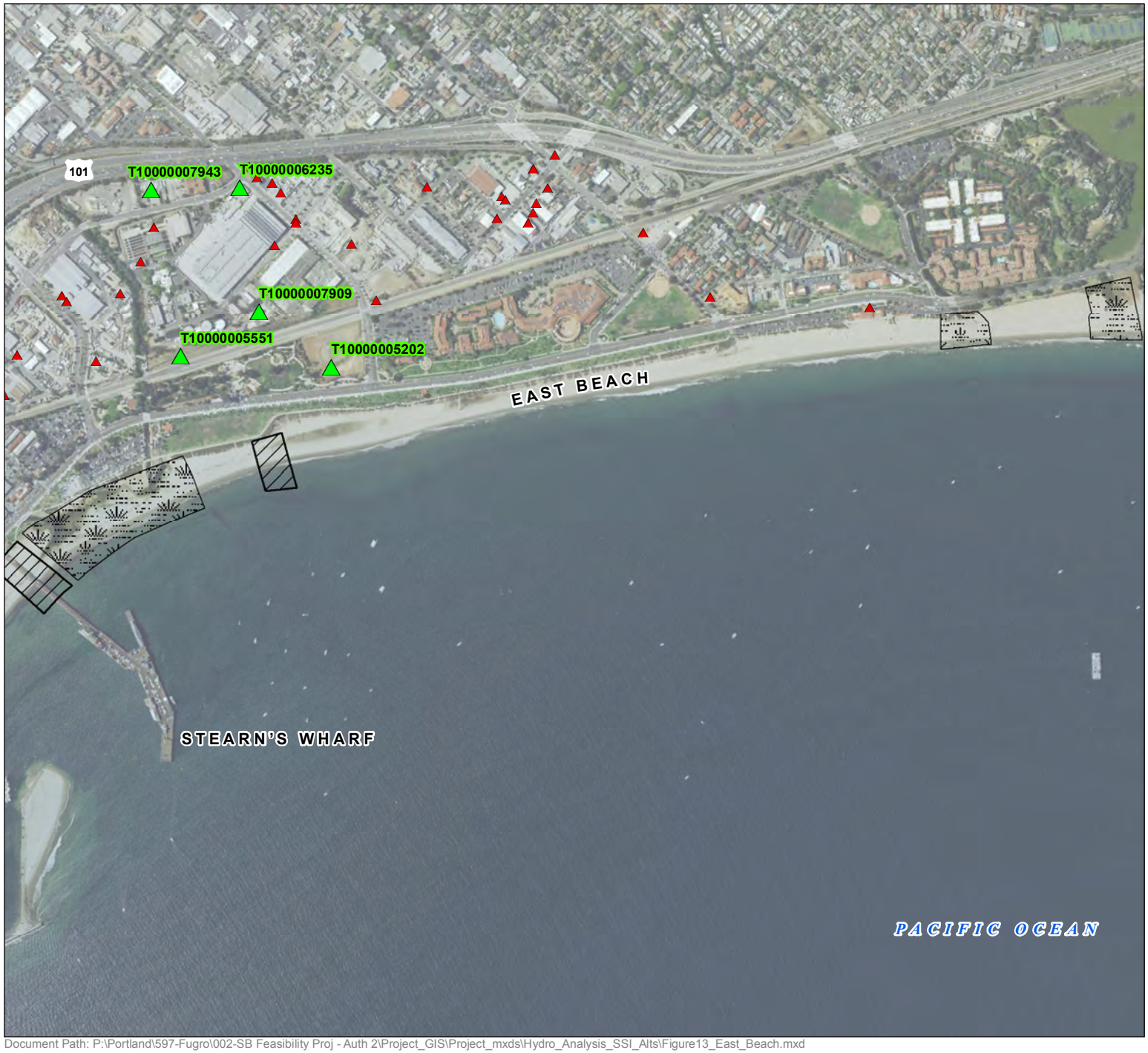

FIGURE 13

East Beach Construction Setbacks and Contaminated Sites

Hydrogeologic Analysis of
SSI Alternatives
Santa Barbara, California

LEGEND

- GeoTracker Sites (open)
- GeoTracker Sites (closed)
- Sensitive Habitat
- Construction Setbacks

N

0 500 1,000
Feet

MAP NOTES:
Date: December 28, 2015
Data Sources: CA SWRCB GeoTracker (2015)

GSI Water Solutions, Inc.

Document Path: P:\Portland\597-Fugro\002-SB Feasibility Proj - Auth 2\Project_GIS\Project_mxds\Hydro_Analysis_SSI_Alts\Figure13_East_Beach.mxd

The thickness of each layer was defined from typical depths and thicknesses of the upper sand unit, and the underlying clay and sand units (lower sand), which together comprise the regionally-defined Shallow Zone. The upper sand unit is subdivided into six layers, each 5 feet thick. The top of the uppermost model layer (Layer 1) was established as sea level at the mean tide line, then was specified to slope downward from the mean tide line towards the edge of the model domain. The slope of the ocean floor between the mean tide line and this offshore fault was defined from published bathymetric contour maps (Johnson and others, 2013). In the inland direction, the top of the uppermost model layer was specified from estimates of the hydraulic gradient from a review of environmental site reports downloaded from GeoTracker, and the surfaces of underlying layers were assumed to follow this same slope. All layers were specified as being active throughout the model domain, to provide a three-dimensional representation of groundwater movement towards each intake type from inland and offshore areas alike.

Geologic cross sections prepared for this study indicate that the Shallow Zone aquifer consists predominantly of sand on the beach and offshore, but includes significant clay lenses that appear to extend inland from near the back of the beach. Two of the cross sections (D-D' and G-G') show that an approximately 10-foot thick saturated zone, consisting predominantly of sand, may extend inland, though it may be imbedded with intermittent clay lenses. In contrast, two other cross sections (E-E' and F-F') show a predominance of clay and little to no sand inland from the beach. While there appears to be considerable variability both horizontally and vertically, cross sections D-D' and G-G' were used as the basis for simulating flow from the inland area.

The hydraulic conductivity value used to represent the medium- to fine-grained sand in the beach sand lens and offshore (model layers 1-6) was assumed to be 55 feet per day (feet/day), based on three data sources: 1) four pumping tests performed on East Beach in 1989 and 2) 1990 (CH2M Hill, 1989 and 1990) and 3) shallow percolation tests performed at the Cabrillo Bath House (Fugro, 2015).

1. The results of the short-term pumping test performed in 1989, indicating a horizontal hydraulic conductivity of 22 feet/day, represented what the investigators believed was "the least favorable hydrogeologic conditions along the beach" and therefore indicative of the lower end in hydraulic conductivity variability of beach sands (CH2M Hill, 1989).
2. The hydraulic conductivity results from the pumping tests performed on east beach range from 22 feet/day to 55 feet/day, with the long-term (2 week) pumping test indicating the highest hydraulic conductivity value (CH2M Hill, 1989 and 1990).
3. At the Cabrillo Bath House, the (vertical) percolation rate in the beach sand was approximately 59 feet/day (Fugro 2015).

The value of 55 feet/day assigned to the upper sand (model layers 1-6) is within the reasonable range for medium sand of between 20 and 70 feet/day (Bouwer, 1978 and Domenico and Schwartz, 1990). The value of 55 feet/day for hydraulic conductivity used to represent the upper sands may overestimate the hydraulic conductivity and water-bearing capability of portions of the beach that consist of finer sands or sands interbedded with silt and/or clay deposits. Numerical model simulations were conducted using a vertical hydraulic conductivity value (Kz) of 11 feet/day, which provides a vertical anisotropy ratio (Kh:Kz) of 5:1.

The hydraulic conductivity of the sand at deeper depths offshore (model layers 9-11) are simulated at 55 feet/day. Sensitivity runs using a value of 9 feet/day were also performed to evaluate the possibility

that the lower sands contain a greater fraction of fine-grained material than the beach sands. The lower hydraulic conductivity used in the sensitivity runs is based on the hydraulic conductivity of the sandy horizons in the onshore area, as descried below. Using the vertical anisotropy ratio of 5:1, the vertical hydraulic conductivity values for the lower sand unit were 11 and 1 feet/day for the primary and sensitivity runs, respectively.

The aquitard separating the upper sand from the lower sand (model layers 7-8) was modeled using a modest permeability (1 foot/day) to reflect local discontinuities and its potential to act as a leaky aquitard between these two sand units (see also Table 3). The other aquitards (model layers 12 and 14) were also modeled using a hydraulic conductivity of 1 foot/day.

As presented in Table 3, for the onshore areas, the equivalent upper sand and lower sand layers (model layers 1-6 and 9-11) were modeled using a hydraulic conductivity of 9 feet/day. This value was derived from inspection of the cross sections as follows. As described above, the 30 feet thick beach sand (upper sand) is assigned a horizontal hydraulic conductivity of 55 ft/day based on aquifer tests, which corresponds to a transmissivity of 1,650 square feet per day (ft2/day). The cross sections show that this sand sequence becomes much more heterogeneous a short distance inland of the beach, with significant fines and lenticular sand lenses that likely are interconnected to varying degrees. The thickest sand package visible on the cross sections in the upper most portion of the onshore area is on the order of 5 feet thick, or about one-sixth of the thickness of the 30-foot sand sequence on the beach. Because the thickness of each model layer is uniform throughout the model domain, the hydraulic conductivity of the upper six layers (the upper 30 feet of the aquifer) was decreased by a factor of one-sixth, from 55 ft/day to 9 ft/day. This adjustment provided lower bulk values of the hydraulic conductivity and transmissivity for the upper six layers, to reflect the greater predominance of fine-grained materials, while also considering that some groundwater movement likely does occur from the uplands through sand lenses with varying degrees of lateral continuity and tortuosity. As with the beach and offshore areas, the lower sand units in the onshore area were assumed to have the same hydraulic conductivity as the upper sand unit, except for the sensitivity runs that had lower assumed hydraulic conductivity values to reflect consolidation and potential presence of fines.

For each model layer the interface with the ocean was modeled as follows:

- At cells where the ocean floor lies at an elevation within the layer, a head-dependent boundary condition was applied, using the River package in MODFLOW-NWT. The width and length of the "river" boundary condition was set equal to the full dimensions of the cell (25 feet by 25 feet), and an assumed vertical hydraulic conductivity of 11 feet/day was applied to the calculation of the bed conductance for the ocean floor.
- At cells where the ocean floor lies at or below the bottom elevation of the cell, a constant-head boundary was applied, with the head set to mean sea level.

Given the slope of the ocean floor, near the southern model boundary the deepest constant head cell was placed in model layer 5 and the deepest river cell was placed in model layer 6. However, a constant head boundary was placed in all model layers in the southern-most two rows of the model so that seawater could enter every model layer. This approach was used because without this deeper constant head boundary, the lower sand unit might have had little to no connection to ocean water in the model,

whereas a connection likely exists offshore (even though no offshore borings exist in that area to help map the exact location and thickness of the connection).

4.1.1.1 Vertical Wells

In the numerical model, a series of vertical wells were simulated located at the back of the beach, which at most locations is approximately 150 feet inland from the mean tide line. Each well was assumed to be screened through the full thickness of the lower sand layer between an elevation of approximately -60 and -120 feet (in model layers 9 through 11) through a total of 60 feet of saturated permeable deposits. The allowable drawdown was assumed to be between 50 and 55 feet, in order to provide 10 feet of water column above the screen and to minimize (if not avoid) the potential to draw down the water level into the top of the lower sand layer.

4.1.1.2 Beach Infiltration Gallery (BIG)

Within the numerical model, each BIG was placed along the beach centered mid-way between the mean tide line and the back of the beach. Accordingly, these facilities were simulated as lying 50 feet inland from the mean tide line. Each gallery was assumed to have its intake screen 30 feet below the water table. The target available drawdown was simulated as 25 feet in order to maximize the potential yield while also preventing dewatering of the screen.

4.1.1.3 Radial Collector Wells

As with the vertical wells, the radial collector wells were simulated within the numerical model at locations at the back of the beach, which at most locations is approximately 150 feet inland from the mean tide line. The assumption for simulating the radial collector well locations at the back of the beach is because the upper sand layer was too thin and so the laterals needed to be placed in the lower sand layer. Therefore, there is no benefit of locating the collector in the middle of the beach. Each radial collector was simulated as containing five 150-foot long laterals arranged in a semi-circular pattern. Two laterals were placed parallel to the shoreline (perpendicular to the ambient inland groundwater flow direction); one lateral extended perpendicular towards the shoreline (parallel with the ambient inland groundwater flow direction); and two laterals placed at angles between the three aforementioned laterals, for a total screen length of 750 feet.

The radial collector wells were simulated separately in the lower sand and the upper sand units to assess whether there would be a difference in yield. The simulations in the lower sand layer were performed with each lateral placed in model layer 10, which is simulated at an elevation of between -80 and -100 feet, to represent its placement in the middle of the lower sand layer. Each lateral was simulated as pumping from six of the 25-foot cells. The allowable drawdown in each model cell representing a portion of a radial collector lateral was assumed to be between 50 and 55 feet, in order to minimize (if not avoid) the potential to draw down the water level below the base of the overlying clay layer and into the top of the lower sand layer. For the upper (beach) sand simulations, the laterals were placed in model layer 6 between an elevation of approximately -25 and -30 feet at the beach. The target drawdown for the upper (beach) sand simulations was limited to 25 feet in order to maximize the potential yield while also prevent dewatering of the laterals.

4.1.1.4 Slant Wells

The slant wells were simulated with their wellheads set back approximately 150 feet inland from the mean tide line. These wells are assumed to be drilled at an angle of 20 degrees from horizontal. Based on this angle, the top and bottom of the lower sand are encountered at drilled distances of 205 feet and

380 feet from the wellhead (as measured inside the casing), resulting in a penetrated thickness of 175 feet. Geographically, these distances correspond to 192 feet and 357 feet from the wellhead. Each slant well site is assumed to have 3 separate drilled casings per drilling site, with one casing being drilled towards the mean tide line and the other two casings being drilled at -45 and +45 degree angles horizontally from the casing being drilled towards the mean tide line. Each casing was simulated as pumping from six of the 25-foot cells, with two pumping cells in layer 9, two pumping cells in layer 10, and two pumping cells in layer 11. The allowable drawdown was assumed to be between 50 and 55 feet, in order to provide at least 10 feet of vertical hydraulic head pressure above the screen and to minimize (if not avoid) the potential to draw down the water level into the top of the lower sand unit.

4.1.2 Analytical Modeling

The two remaining SSI alternatives (SIGs and HDDs) were evaluated by analytical methods for estimation of achievable yields and area required for intakes. The evaluation for these two alternatives was conducted using analytical methods because these intake alternatives involved the screen placements directly beneath the ocean in the upper sand layer, through which flow will primarily be vertical. The simplified conceptual model of this shallow sand allowed the estimation of achievable yields using industry-standard calculations based on those presented in Driscoll (1986, pages 763 to 764) for bed-mounted infiltration galleries. These calculations estimated the total required length of screen for intakes, the configuration for which was presumed based on typical screen configurations and design criteria by Driscoll.

4.1.2.1 Seabed Infiltration Gallery (SIG)

Horizontal laterals for bed-mounted SIGs were simulated at a location entirely under the ocean floor. In accordance with the typical configuration requirements of screens for SIGs, we assumed that the total length of screen would be contained within rectangular beds located within the jurisdictional limits of the City of Santa Barbara, which extends ½ mile offshore. Each bed will need to extend offshore beyond the surf zone. For the calculation, the lengths of screen would be buried 15 feet below the seafloor where the depth of water above the screens is equal to or greater than 25 feet. The diameter of each pipe is assumed to be 18 inches. Flow to the submerged SIG intake screens would occur vertically.

In order to achieve water quality benefits from natural biological treatment that occurs in the upper portion of the sand bed associated with both SIG and HDD, Missimer et al (2013) suggests the following possible design infiltration rates for achieving biologic treatment within a SIG/HDD facility:

> "A classical gravity fed slow sand filter, depending on the turbidity of the water being treated, can operate at infiltration rates ranging from 0.1 to 0.4 m/h [0.04 to 0.16 gpm/ft^2] with minimal need to clean the upper layer of the filter. Modern design criteria for slow and rapid sand filtration tend to have a lower range for the recommended design filtration rate at 0.05 to 0.2 m/h [0.02 – 0.08 gpm/ft^2]."

For the analytical yield calculations, we assumed that the engineered filter pack placed around the intake pipes would have a uniform hydraulic conductivity value of greater than or equal to the native beach sands (assumed 55 feet/day) placed in the top 5 feet of the constructed bed. As constructed, the bottom and middle layers of the constructed beds would likely consist of coarser engineered materials with higher hydraulic conductivity rates.

4.1.2.2 Horizontal Directionally Drilled Wells (HDD)

The analytical modeling methods performed for HDDs are similar to those used for SIGs described above, with the principal difference being that installation method for HDD is by drilling instead of excavation of beds. The perforated intake pipe is assumed to be 18 inches in diameter and will be entirely buried at least 10 feet below the ocean floor. Flow to the submerged HDD intake screens is assumed to occur vertically.

4.2 Yield, Intake Facility Spacing, and Length of Beach Required

The principal findings of the SSI Analysis are presented on Tables 4 and 5 followed by a summary:

Table 4 Summary of Intake Alternatives.

Intake Type	Shallow Zone Layer	Number of Facilities Required [1]	Approximate Spacing (feet)	Length of Beach Required (Miles)[1]	Yield per Facility (gpm)	Potential Yield [2] (gpm)	Percentage of Required Desal Plant Flow
Vertical Wells	Lower Sand	40 - 160	600 - 750	5.5 – 18	100-400	1,500 - 4,800	9 – 30%
Beach Infiltration Gallery	Upper Sand	6	N/A	3	Varies with length of available beach	10,100	64%
Radial Wells	Upper Sand	43	600	5	375	5,625	35%
	Lower Sand	16 - 58	600 – 1,500	4 - 6	275 - 1,000	4,125 - 7,000	26 – 44%
Slant Wells	Lower Sand	16 - 58	650 - 1,250	3.5 - 6	275 - 1,000	4,400 - 8,000	28 – 50%
SIG	Upper Sand	1	One facility only	One facility only; located offshore	15,898	15,898	100%
HDD	Upper Sand	11	N/A[3]	0.1	1,500	15,898	100%

Notes: 1. Total required to meet 15,898 gpm.
2. Potential yield within available beach.
3. HDD wells constructed as multi-well clusters from one location.

Table 5 Summary of Feasible Yield for Intake Alternatives on Available Beach

Intake Type	Shallow Zone Layer	Number of Facilities*	Yield Per Facility (gpm)	Potential Yield* (gpm)	Approximate Spacing (feet)	Length of screen (Feet)	Inland Contribution	Offshore Contribution
Vertical Wells	Lower Sand (high K)	12	400	4,800	750	60	18%	82%
	Lower Sand (low K)	15	100	1,500	600	60	47%	53%
Beach Infiltration Gallery	Upper Sand	6	Varies with Length	10,100	N/A	9,000	5%	95%
Radial Collector Wells	Upper Sand	15	375	5,600	600	750	30%	70%
	Lower Sand (high K)	7	1,000	7,000	1,500		30%	70%
	Lower Sand (low K)	15	275	4,125	700		39%	61%
Slant Wells	Lower Sand (high K)	8	1,000	8,000	1, 250	175	8%	92%
	Lower Sand (low K)	16	275	4,400	650		5%	95%
Seabed Infiltration Gallery	Upper Sand	1	15,898	15,898	N/A	5,000	0%	100%
HDD	Upper Sand	11	1,500	15,898	N/A	11,000	0%	100%

Note: * Potential yield within available beach

Of the six types of subsurface intakes evaluated, only the seabed infiltration gallery and horizontal directionally drilled wells are able to satisfy the requirement to produce 15,898 gpm for desalination. In addition, these intake alternatives are the only ones that derive all of the flow from offshore sources. The other intakes evaluated are capable of producing between 9 and 64 percent of the required flow.

Generally, the distance each facility is located from the mean tide line has a small influence on the amount of seawater contribution to the yield of each type of facility because of the limited hydraulic connection between the ocean and the lower sand layers of the Shallow Zone. In the case of the onshore beach infiltration gallery, its closer position to the mean tide line combined with its placement in the shallow sands is expected to result in a high contribution from seawater (estimated to be on the order of 95 percent). The percentages of seawater relative to groundwater are expected to vary depending on the tide. Notably, the estimation of the contribution from onshore and offshore discussed below assumes continuous operation of each intake facility. Intermittent operation of the facility would decrease the contribution from offshore because it takes time to establish the offshore hydraulic connection.

4.2.1.1 Vertical Wells

A total of up to 15 vertical wells installed on the available beach would have a combined pumping yield of from 1,500 to 4,800 gpm, which is 9 to 30 percent of the total required yield. A total of 16 vertical wells installed to a depth of 120 feet on the available beach spaced 550 feet apart and each pumping at a rate of 100 gpm continuously would have a combined pumping yield of 1,500 gpm, which is only 10% of the total required yield of 15,898 gpm. Water produced from vertical wells would consist of as much as 47 percent water produced from inland sources and only 53 percent from offshore (seawater) sources. In this case, inland sources refers to groundwater present in the Shallow Zone only. A total of between 5.5 and 18 miles of similar beach and as many as 160 wells would be required from vertical wells to produce the total water required for the plant.

4.2.1.2 Beach Infiltration Gallery (BIG)

Of the intake alternatives which cannot satisfy the full project flow requirements, the Beach Infiltration Gallery has the highest potential yield. If galleries are constructed across the full length of the available beaches, the yield is estimated to be approximately 10,000 gpm. BIG's are calculated to derive approximately 95 percent of their flow from offshore sources, with very little inland contribution; this is due to the predominance of clay in the uppermost saturated zone inland of the beach. To satisfy the entire yield required, the BIG would require 3 miles of similar beach with the upper sand layer that is at least 30 feet deep.

4.2.1.3 Radial Collector Wells

Three collector well scenarios were evaluated: (a) upper (beach) sand, (b) the lower sand layer assuming a relatively high hydraulic conductivity, and (c) the lower sand layer assuming a relatively low hydraulic conductivity. For the upper beach sand, it may be possible to produce up to 5,600 gpm from 15 collector wells constructed on the available beach assuming the upper beach sand is sufficiently thick and permeable across the entire beach area. For the lower sand with high hydraulic conductivity, seven collector wells spaced 1,500 feet apart could produce 7,000 gpm from the available beach area. For the lower sand with low hydraulic conductivity, fifteen radial collector wells spaced 700 feet apart could produce up to 4,125 gpm from the available beach areas.

Because the geologic cross sections indicate that the lower sand layer extends inland, radial collectors in this layer may produce water comprised of as much as 40 percent inland groundwater and 60 percent from offshore sources. As much as 70 percent of the produced water would be from offshore sources if collectors are constructed in the upper (beach) sand. To satisfy the total flow required, a total of between 4 and 6 miles of similar beach would be required.

4.2.1.4 Slant Wells

Slant wells each have an estimated yield of 275 to 1,000 gpm per well. A total of 8 to 16 slant wells could be constructed within the available beach area with a total yield of 4,400 to 8,000 gpm. Because the slant wells are installed under the ocean and are not parallel to the shoreline, they may produce water comprised of as much as 95 percent from offshore sources and as little as 5 percent from inland groundwater sources. Four to six miles of similar beach would be required to satisfy the project's required flow rate of 15,898 gpm.

4.2.1.5 Subsurface Infiltration Gallery (SIG)

A seabed infiltration gallery located offshore will likely satisfy the project yield requirement for desalination as specified in study goals. Further, the produced water would likely consist entirely of seawater without any contribution from inland groundwater. The area required to construct these beds and allow sufficient infiltration to produce the target yield is estimated to be 7 acres. To accomplish this would require installation of approximately 7,000 feet of drain spaced approximately 10 feet apart. For our calculation, each individual screen was 325 feet long, although many configurations would satisfy the total length requirements. Such an installation would likely satisfy recommended flow velocities for slow sand filter design (i.e., 0.05 gpm/ft^2) (Missimer, 2013).

This intake alternative could fit in the offshore area within the 0.5 mile City of Santa Barbara jurisdictional area if bedrock is not present and other factors including longshore current velocity and seafloor erosion are satisfactory.

4.2.1.6 Horizontal Directionally Drilled Wells (HDD)

As with SIGs, HDD wells may be able to satisfy the total project yield requirement. To achieve this flow, approximately 11,000 feet of screen would be required from approximately eleven HDD wells, assuming the HDD can be advanced to at least 1,000 feet offshore. It may be possible to advance all eleven HDD wells from a single location. As with SIGs, the produced water would likely consist entirely of seawater from offshore sources.

This intake alternative could fit in the offshore area within the 0.5 mile City of Santa Barbara jurisdictional area if bedrock is not present and other factors including longshore current velocity and seafloor erosion are satisfactory.

4.3 Impacts to Local Groundwater and Sensitive Habitats

The contribution of local groundwater to the SSI alternatives was estimated for the vertical wells, beach infiltration gallery, radial collectors, and slant wells based upon the maximum yield achievable with the available beach frontage. These SSI alternatives derive some portion of flow from inland sources. Both the SIG and HDD SSI alternatives would derive all of their water from offshore sources. As presented in Table 5, radial collector wells and vertical wells appear to have the lowest percentage of seawater contribution. Testing of the slant well at Dana Point, California indicates that this slant well is producing on the order of 50 percent seawater (Martin Feeney, Personal Comm., 2015). In Monterey, testing

indicates that the slant well is producing greater than 90 percent seawater (Martin Feeney, Personal Comm., 2015). The difference between Dana Point and Monterey may be attributable to the Dana Point slant well penetrating aquifers that receive inland recharge, whereas the Monterey slant well does not.

Table 6 Drawdown Impacts to Local Groundwater and Sensitive Habitats

Intake Type	Drawdown Beneath Sensitive Habitats
Vertical Wells	1 to 3 feet
Beach Infiltration Gallery	~ 1 to 4 feet at a distance 250 feet from end of trench
Radial Collector Wells	0.5 to 3 feet
Slant Wells	1 to 3 feet
Seabed Infiltration Gallery	0
HDD	0

Numerical modeling simulated drawdown related to pumping of the vertical wells, beach infiltration galleries, radial collector wells and slant wells in the sensitive habitats within the construction setback areas presented on Figures 11 through 13. The results of the numerical modeling simulations are presented on Table 6 and indicate that groundwater levels are predicted to decline by one to three feet beneath the sensitive habitat areas (i.e., based upon the maximum yield achievable using the available beach frontage; for the purposes of this study well spacing was not optimized to reduce this type of drawdown). The beach infiltration gallery created the greatest amount of drawdown beneath sensitive habitat areas (up to 4 feet) because of its placement in the shallow surficial sands nearest the sensitive habitats.

The operation of the SIG and HDD are not expected to cause measureable drawdowns effects in the sensitive habitats.

4.4 Capture of Known Groundwater Pollutants

The potential exists for SSI alternatives that derive a portion of their flow from inland sources to draw contamination toward them or change gradients and affect ongoing remediation activities (e.g., pump and treat) if they are occurring. The California State Water Resources Control Board (SWRCB) GeoTracker database was used to catalog contaminated sites located within the study area between Highway 101 and the coast. A total of 75 sites were identified, of which nine sites are listed as 'Open'. The nine open sites range in status from 'Site Assessment' to 'Eligible for Closure' and include contamination from heavy metals, gasoline, diesel, waste oil, solvents, polynuclear aromatic hydrocarbons (PAH), and total petroleum hydrocarbons (TPH). A summary of the open contaminated

sites and their respective constituents of concern (COC) is shown in Table 6 and the contaminated site locations are shown on Figures 11 through 13.

Table 7 Open Contaminated Sites Located in Study Area between Highway 101 and the Coast

GeoTracker ID	Site Name	COCs	Status	Status Date
T10000003790	Sri Padma LLC (formerly known as City Block and Phantom Cargo)	Solvents	Open - Site Assessment	5/19/2012
T10000005202	CHASE PALM PARK EXTENSION PROJECT	TPH and Lead	Open - Site Assessment	7/1/2015
T10000006225	HWY 101 EXPANSION PROJECT	Lead	Open - Site Assessment	3/1/2007
T10000006235	Former Standard Oil Bulk Plant	TPH	Open - Site Assessment	9/19/2014
T10000007909	Fire Training Facility	TPH	Open - Assessment & Interim Remedial Action	10/27/2015
T10000007943	City Desalination Plant	TPH	Open - Assessment & Interim Remedial Action	11/6/2015
T10000005551	El Estero Turtle Pond (A.K.A. El Estero Drain)	Arsenic, Lead, Mercury, PAH, TPH	Open - Remediation	1/8/2014
T10000000467	Parking Lot	Gasoline/Diesel & Waste Oils	Open - Eligible for Closure	4/3/2015
T10000004141	Freeman & Wood Properties	Gasoline/Diesel & Waste Oils	Open - Eligible for Closure	9/23/2013

Note: All data from SWRCB GeoTracker <http://geotracker.waterboards.ca.gov>

Other potential sources of contamination are known to have impacted groundwater quality in the Shallow Zone near Santa Barbara. A magnitude 6.9 earthquake hit Santa Barbara on June 29, 1925 and leveled much of the City. Rubble and debris from that quake was moved to areas between this study's beach areas and Highway 101, including the City's El Estero Wastewater Treatment Plant and Charles Meyer Desalination Plant properties. Soils contaminated with trash, lead and hydrocarbons are routinely found in this area and are likely to influence groundwater quality. Because of the extent and prevalence of contamination, the City's building department requires soils testing for any building permits in this area. Therefore, groundwater monitoring in the Shallow Zone is recommended prior to proceeding with development of vertical wells, collector wells, beach infiltration galleries, and/or slant wells.

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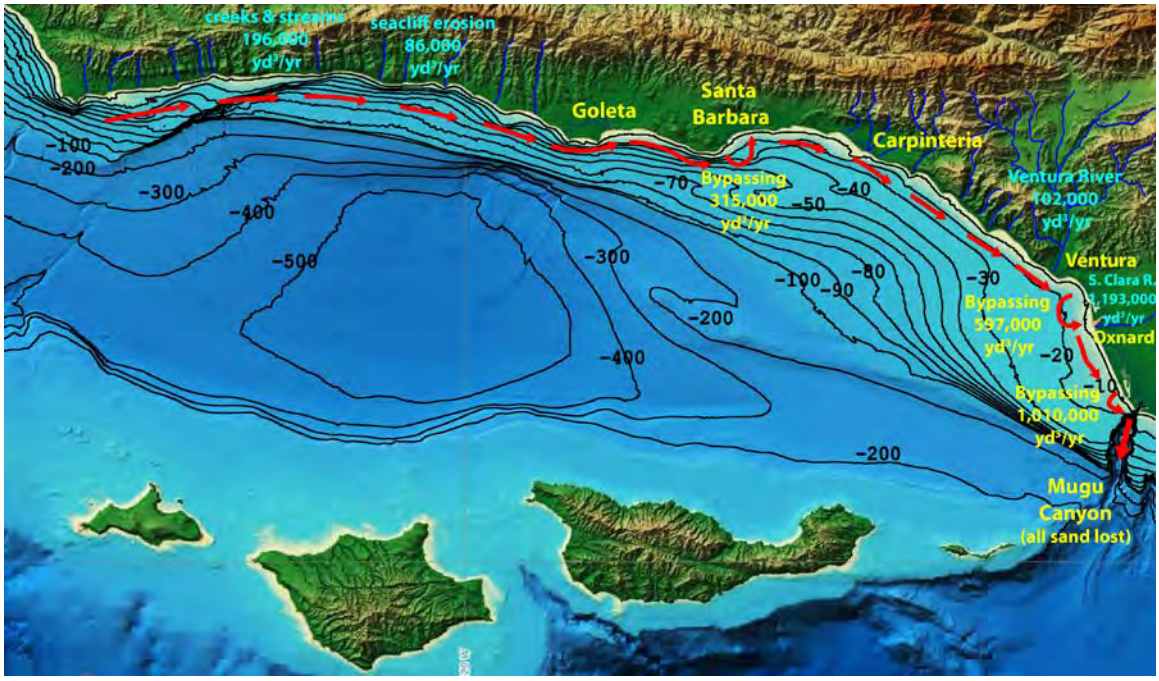
APPENDIX C – COASTAL HAZARDS AND SEDIMENT TRANSPORT ANALYSIS

Coastal Hazards and Sediment Transport Analysis for the City of Santa Barbara Subsurface Desalination Intake Feasibility Study



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Draft: 30 November, 2015; Final: 29 December 2015

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ABSTRACT: Subsurface intake options for the City of Santa Barbara's (City) Subsurface Desalination Intake Feasibility Study are reviewed and the site requirements for each alternative were evaluated by performing a sediment transport and coastal hazards evaluation. A sediment budget analysis is performed on the Santa Barbara Littoral Cell using the Coastal Evolution Model developed at the Scripps Institution of Oceanography. The sediment budget analysis provided critical far field inputs to a near field seafloor stability and coastal hazards analysis of the site specific conditions and infrastructure of the City's Subsurface Desalination Intake Feasibility Study. The viability of each subsurface intake option is evaluated with respect to the results of the seafloor stability and erosion analysis; while the coastal hazards analysis evaluated vulnerabilities of all shore-side and offshore structures associated with this study. It was concluded that the West Beach intake site is well suited for a beach infiltration gallery (BIG) but is not optimal for a subsurface infiltration gallery (SIG) or for Neodren horizontal well technology. The Leadbetter Beach intake site was found to be feasible for SIG or BIG type intake systems but both are problematic to construct at this site due to exposure to high energy wave climate, The Neodren intake technology was found to be the best option for the Leadbetter Beach site and the only viable option for East Beach. None of the shore-side facilities will be significantly flooded by wave run up at present sea levels, although future sea level scenarios will cause flooding from wave run up at the pump station site. All shore-side facilities will be inundated by tsunami and only Neodren will be unaffected offshore by tsunami erosion. All conclusions must be considered within the context of this report’s scope (i.e., sediment transport and coastal hazards evaluation only). Overall feasibility of subsurface intake alternatives must consider other technical factors and will be evaluated by others. These additional technical factors should include, but not limited to hydrogeology, constructability, reliable performance history, etc.

1) Introduction:

This study provides a coastal hazards and sediment transport analysis for the City of Santa Barbara's (City) Subsurface Desalination Intake Feasibility Study. The analysis includes assimilation of long-term wave climate data bases and the construction of a sediment budget for the study's sites to evaluate nearshore and offshore erosion and accretion cycles and inundation by extreme wave and tsunami run-up that may affect stability and operations of subsurface desalination plant intake structures, 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.
The City's desalination plant is currently permitted by the Central Coast Regional Water Quality Control Board (RWQCB) and the California Coastal Commission to discharge brine to the El Estero Wastewater Treatment Plant (EEWWTP) and to draw seawater from the Pacific Ocean using a screened open ocean intake. On January 30, 2015, the Central Coast RWQCB amended the City's permit (NPDES Permit CA0048143), requiring the City to evaluate the feasibility of subsurface desalination intakes. In accordance with this amendment, on August 31, 2015 the City submitted a Work Plan to the RWQCB, which was approved on October 20, 2015. While this Work Plan closely follows the methodology presented in the "Desalination Amendments" to the California Ocean Plan that were adopted in May 2015, the requirements presented in these Amendments do not apply to the City's Subsurface Desalination Intake Feasibility Study because the City's desalination plant is not "a new or expanded facility".

For the purposes of this study, only shallow subsurface intake technology will be discussed in detail because - due to their shallow construction and proximity to the ocean - these technologies are most affected by oceanographic hazards such as erosion, sea level rise and tsunami. Any conclusions made for shallow collector well alternatives can apply to deeper intake technology alternatives (e.g., slant wells and vertical beach wells). There are three geomorphic conditions of the site location for successful operation shallow subsurface intake systems. These are: 1) adequate sediment cover, 2) the proper grain size distribution within that sediment cover (no lenses of silts and clays that would otherwise retard seawater infiltration rates), and 3) a stable seabed. To evaluate the adequacy of the site conditions in Santa Barbara for these requirements, this report presents the results of a sediment budget and erosion analysis using the Coastal Evolution Model (CEM). The Coastal Evolution Model was developed under a $1 million grant by the Kavli Foundation to make forecast predictions of the effects of sea level rise on the coastline of California, and was validated in the Oceanside and Santa Barbara Littoral Cells for the same period of record used in the present study.

1.1) General Project Description:

Figure 1.1 provides a site plan for the City's Subsurface Desalination Intake Feasibility Study, which will use the discharge infrastructure of the El Estero Wastewater Treatment Plant (EEWWTP) for disposal of brine from the seawater reverse osmosis (SWRO) facilities located at 525 Yanonali Ave (elevation + 10 ft NGVD).

The source water for Charles Meyer Desalination Plant is currently derived from an existing screened open ocean intake located 2,500 ft. offshore of East Beach (Figure 1.1) and operates in accordance with the City's amended NPDES permit (CA0048143) and Coastal Development Permit 4-96-119. The purpose of this study is to evaluate subsurface intake systems sited at several possible landside locations (Figure 1.2) including:

- Leadbetter Beach west of Santa Barbara Harbor,
- West Beach inside Santa Barbara Harbor, or
- East Beach east of Santa Barbara Harbor.

Figure 1.1: Site plan for the City of Santa Barbara's Desalination Plant (from Carollo, 2014).

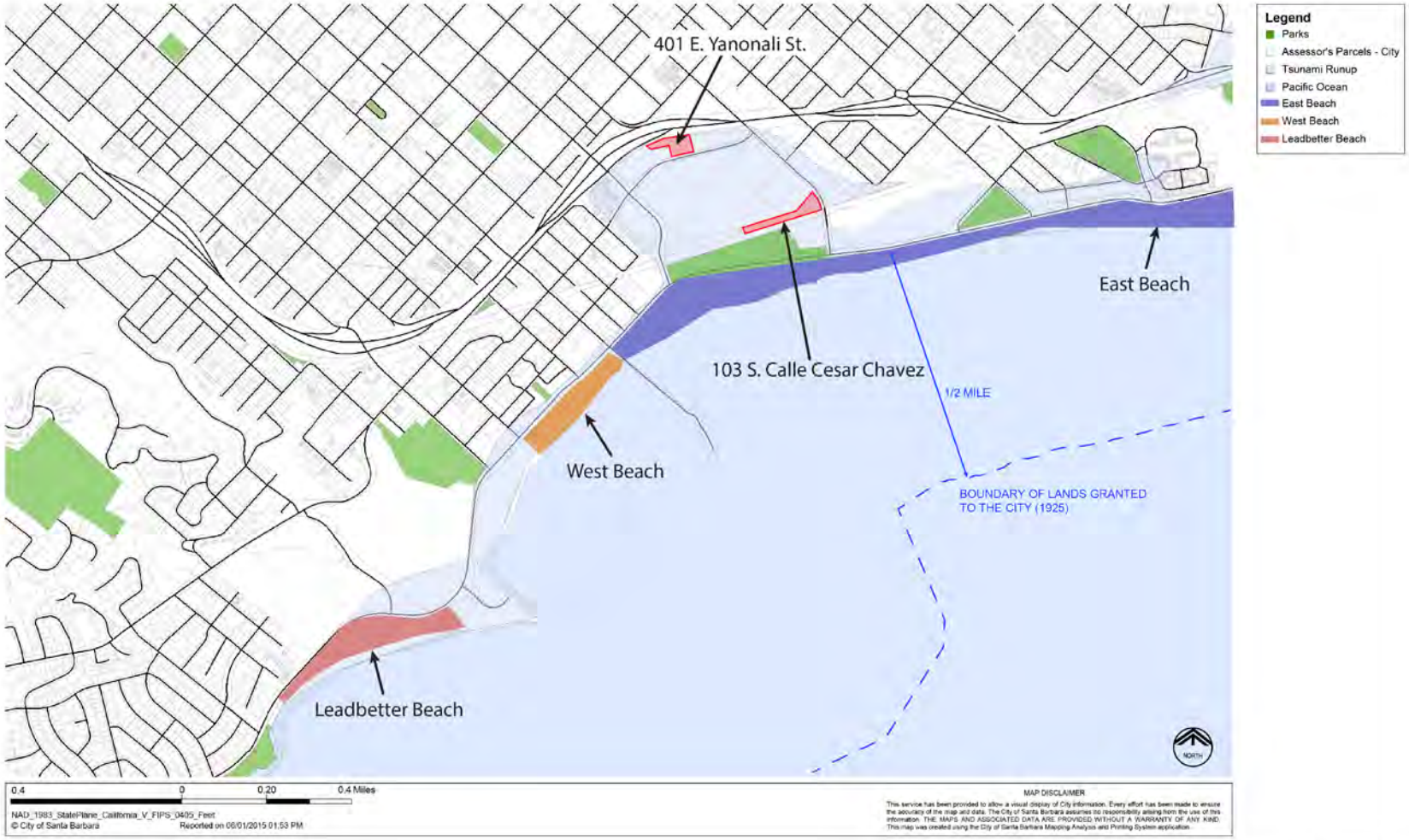

Figure 1.2: Subsurface Intake Project Site Alternatives

A pump station to support source water inflows from a collector well type subsurface intake alternatives (e.g., SIG or Neodren™) to the the desalination plant may be located at the following locations (Figure 1.1):

- An existing pump station / chemical storage area at 420 Quinientos St (elevation + 8 ft NGVD).
- The City's corporation yard annex, 401 E. Yanonali Ave (elevation + 12 ft NGVD), or
- 103 S. Calle Cesar Chavez (Elevation +10 ft. NGVD);

The vulnerability of the landside facilities to inundation by wave and tsunami run-up and offshore intake structures to seafloor instability will be assessed by this study.

2) Literature Review of Shallow Subsurface Intake Technology

There are four types of shallow sub-seabed intake technologies: subsurface infiltration galleries (SIG), beach infiltration galleries (BIG); advanced horizontal well technology (e.g. Neodren™ system), and radial collector wells (Ranney collectors). Only a very small amount of the literature on this prior art is found in peer-reviewed journals or technical reports from resource agencies. Most of it is found in conference proceedings where the objectivity and efficacy of the information can be questionable. Irrespective of the quality of the literature, it is clear that reportedly successful sub-seabed intake technologies (SIG in particular) have only been demonstrated in fetch-limited environments, those without open ocean exposure to distant swell waves. Consequently, vulnerability to wave erosion and vigorous littoral sediment transport has not been a factor. This is in sharp contrast to the Santa Barbara environment where long-period, high-energy waves from the Gulf of Alaska storms in winter, and from the Mexican tropical hurricanes and southern hemisphere storms in summer, have historically resulted frequent periods of high sea-states and massive beach and nearshore erosion, (USACE, 1993, Inman and Jenkins, 2004, a, b,& c)

2.1) Subsurface Infiltration Gallery (SIG), Fukuoka, Japan:

There is only one example in the world of *Sub-surface (seabed) Infiltration Galleries* being used on prototypic production scales as intakes for an operational desalination facility. That example is at the Uminonakamichi Nata Seawater Desalination Center on the island of Kyushu in Fukuoka, Japan (referred to herein as the *Fukuoka Seabed Infiltration Gallery Intake*). The Fukuoka Seabed Infiltration Gallery Intake was designed and constructed by the Obayashi Corporation, and is considered a proprietary intake system by that company. The infiltration gallery has an intake capacity of 27 mgd to meet Uminonakamichi Nata Seawater Desalination Center's 13 mgd production capacity. Although it has been in continuous use since beginning production in June 2005, it has not operated at full capacity. It consists of infiltration branch pipe segments connected to an infiltration main (Figure 2.1). The 64.2 m wide (210 ft), 313.6 m (1,030 ft) long gallery consists of a non-metallic header-lateral arrangement with 0.6 m (2 ft) diameter laterals (Figure 2.2) attached to two 1.8 m (6 ft) diameter headers. The headers are attached to a central concrete collection vault from which a single 1.58 m (5.2 ft) pipe conveys the water to the plant. The gallery is located 650 m

(2,132 ft) offshore at a water depth of 11.5 m (38 ft). The intake pipes themselves are located about 3.9 m (12.8 ft) below the seabed, under 1.5 m (5 ft) of graded sand, 0.3 m (1 ft) of graded gravel and 2.1 m (7 ft) of coarse gravel, (Kawaguchi, A., 2007; Pankratz, 2014)

At full production, the gallery operates at a rate of 5.1m/d (0.087 gpm/ft^2). The infiltration branch pipe segments are merely examples of a"*French Drain*" (buried pipes with holes along the pipe lengths), but in Japan, these are referred to as "*Toyo Drains*". The gallery is installed below layers of imported sand and gravel (engineered fill) in a pit excavated to about 13 ft below the ambient seabed. The excavated area of seabed is approximately 215,280 square ft (4.9 acres). The infiltrated water flows from the branch pipes to the infiltration main, and, then is conveyed to the onshore intake tank (located below ground) by a transmission pipe. Water collected in the intake tank is then pumped to the desalination center. The infiltration system flows using the difference between the sea level and the water level in the intake tank, and does not require pumping (other than the pumps for the intake tank).

The Fukuoka Seabed Infiltration Gallery Intake began testing and start-up in 2005 and full scale operation in 2006. A number of Japanese newspaper articles were published the first year after full scale operation proclaiming unqualified success for the Fukuoka Seabed Infiltration Gallery Intake; but since that time industry professionals who have visited the site have privately expressed concerns the Toyo Drains are clogging and intake water production is declining, (Kawaguchi, 2007). Because the Obayashi Corporation regards its Seabed Infiltration Gallery Intake as a proprietary technology, it has been less than forthcoming on technical information; and has released very few details on maintenance issues and operational sustainability. However, since 2007, there has been a recent update of on-the-ground intelligence of the Fukuoka Seabed Infiltration Gallery Intake, (Pankratz, 2014). Pankratz, reported that operations manager, (Taketo Tanaka), confirmed that virtually no maintenance of the infiltration gallery has been required, and that the headloss across the system remains almost unchanged from the day the plant was commissioned. The feed water has never been chlorinated, and neither the sand bed nor the piping network has required any cleaning. Divers inspect the surface of the seabed above the gallery one or two times per year, and have noted that the scouring action of the sea currents appears to keep the surface of the sand relatively clean.

Operations manager, (Taketo Tanaka) also claims here has never been evidence of an accumulation of fish eggs or larvae on the seabed, and the low head loss across the system indicates a lack of biofouling, although the gallery piping has never been inspected, either by divers or cameras, and the filter bed media has never been cored and analyzed for larval entrainment.

Pankratz,(2014) concludes his recent visit did not add to the hard data available on the Fukuoka intake, or infiltration galleries in general, but it did confirm that the system has performed as it was intended, requiring virtually no maintenance and providing a reliable and consistent volume of almost particulate-free seawater. However, some operating data has been previously published (Missimer, et. al., 2013). Monitoring

of the Fukuoka feed-water pumped from the gallery shows a very significant improvement in water quality with the silt density index (SDI) being reduced from background levels exceeding 10 to consistently below 2.5 to the beginning of 2010 and mostly below 2.0 thereafter (Figure 2.3).

Amount of Intake water	Approx. 103,000 m3/day (27.2 mgd)
Desalination System	RO system: High-pressure Hollow-fiber type RO Low-pressure Spiral-wound type RO
Amount of Water Produced	Approx. 50,000m3/day (13.2 mgd)

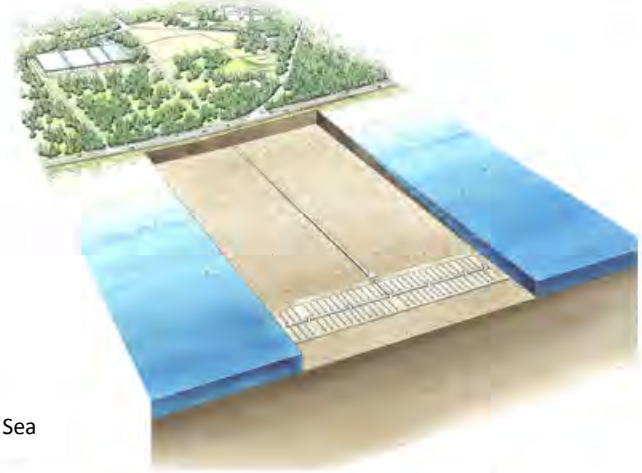

Fig. 2.1.A - View from Sea

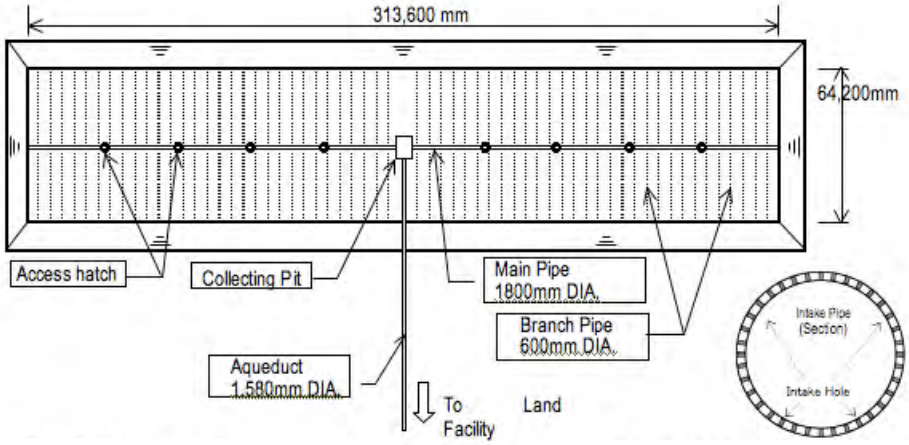

Fig. 2.1.B - Intake Filtration (Plan)

Fig. 2.1.C - Intake Filtration Pipe (Section)

Figure 2.1: The Obayashi SIG Sub-surface (seabed) Infiltration Gallery as deployed at the Uminonakamichi Nata Seawater Desalination Center on the island of Kyushu in Fukuoka, Japan (referred to herein as the *Fukuoka Seabed Infiltration Gallery Intake*).

Figure 2.2: Section of 0.6m (2 ft) diameter perforated lateral branch pipe used in the Fukuoka Seabed Infiltration Gallery Intake, referred to as "*Toyo Drains*". The large infiltration holes are large in comparison to the micron-scale infiltration holes of the Neodren horizontal well technology.

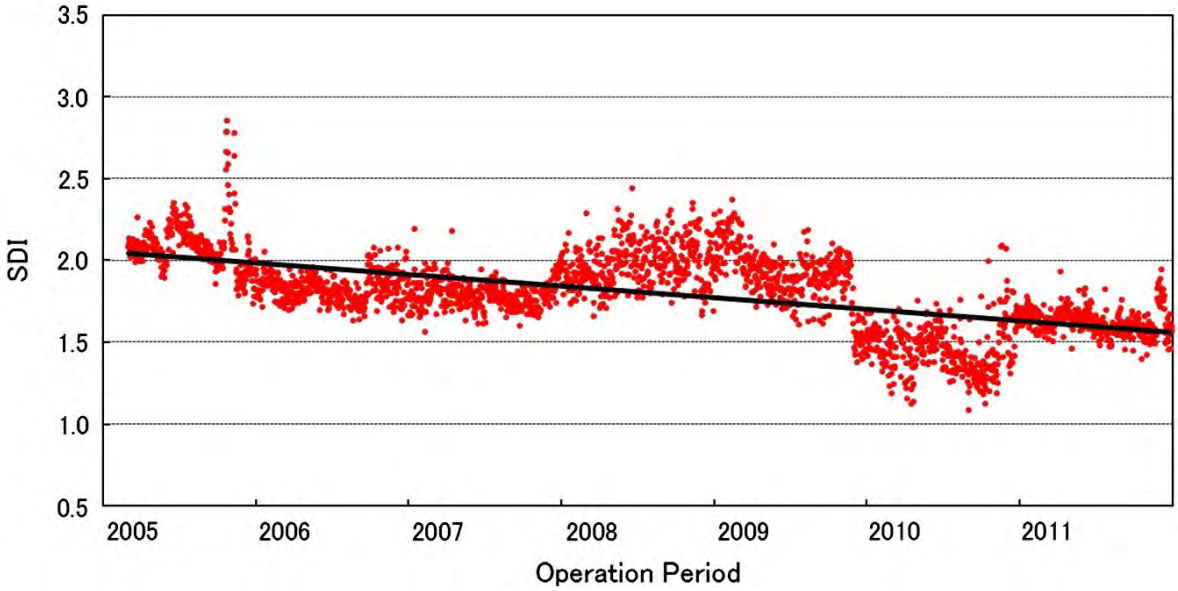

Figure 2.3: Long-term variation in the silt density index SDI of water coming from the seabed gallery at Fukuoka, Japan. The water quality has been consistently good and has improved during the life of the facility.

Another seabed infiltration gallery has been designed and constructed the City of Long Beach, CA, and installed inside the breakwater system of the Long Beach Harbor (Wang, et. al., 2007). The Long Beach installation is an excellent proxy for the proposed subsurface intake at West Beach inside Santa Barbara Harbor (Figure 1.2), and was in the testing phase for a significant time period with infiltration rates ranging from 2.9 to 5.8 m/d (0.05 to 0.1 gpm/ft^2) (Allen, et. al., 2008). This testing revealed substantial reduction in turbidity, SDI_{15}, total dissolved carbon (TDC), and heterotrophic total plate counts (mHPCs) with initially some reduction in concentrations of DOC and AOC before the system was shot down due to filter clogging (Missimer, et. al.,2013).

Fukuoka is located on the north-west side of the island of Kyushu Japan on the Korea Straits that connects the East China Sea to the southwest with Sea of Japan to the northeast. Ocean waves at the site of the Fukuoka Seabed Infiltration Gallery Intake are fetch limited (not exposed to long-period, open ocean swell waves[1]) due to the narrows of the Korea Straits; and the coastal oceanography and sediment transport is dominated by the Tsushima Warm Current (TWC) flowing through the Korea Straits into the semi-enclosed Sea of Japan. The fetch limited offshore environment off Fukuoka promotes long periods of calm sea states which diminish the rigors of offshore construction of a SIG in 11.5 m (37.7 ft) of local water depth. These calm sea-states allow the 10 ft. deep dredged hole in which the SIG piping is installed to be maintained without wave-induced scour and erosion collapsing the hole or infilling it before piping installation is complete, and also allows the engineered fill to be subsequently placed without loss of the fill material. The calm sea-states also maximize the half-life of the engineered fill after placement because wave erosion is minimal. Such fortuitous and persistent calm sea-sates do not exist offshore at Santa Barbara where calm sea-states seldom persist for any significant length of time, (Inman and Jenks, 2004 a & b; see Section 6 for more detail). There are also climatological differences that are relevant to the post construction sustainability of a SIG. The City of Santa Barbara is subjected to deep El Nino cycles, with long periods of dry conditions, followed by powerful winter-time El-Nino storms. The El Nino winter storms cause massive erosion and sediment delivery from the semi-arid (and highly erodible) watershed (Inman and Jenkins, 1999, 2004c).

Fukuoka on the other hand has a humid subtropical climate with hot humid summers and relatively mild winters. Fukuoka's weather, as well as the sediment yield of the regional watersheds, is controlled by the Korean Monsoon that produces on average about 1,600 mm (63 in) of precipitation per year, with a stretch of more intense precipitation between the months of June and September. These high rainfall amounts falling on the high relief topography surrounding Fukuoka, result high inter-annual yields of sediment flux into the local coastal ocean, particularly fluxes of fine-grained sediments derived from the volcanic clays that predominate in the regional watersheds. The formation of the TWC-influenced sediment deposits shifted towards shallower water regions during postglacial sea-level rise, (Nishida and Ikehara, 2006); and this long-term shift in combination with the high seasonal fluxes of fine-grained sediments from the local watersheds has produced a highly dissimilar set of conditions relative to Santa Barbara, (see Section 6 for more detail on Santa Barbara comparisons).

Conditions at the Long Beach experimental SIG are climatologically, geomorphically and oceanographically quite similar to Santa Barbara due to the close proximity of one to the other. The Long Beach experimental SIG is located inside the breakwater system of the Long Beach/Los Angeles Harbor, where it is completely sheltered from wave exposure, and is most similar to the proposed installation at West Beach inside Santa Barbara Harbor. Because of this wave sheltering. The Long Beach experimental SIG was built on the bar-berm section of the beach profile, allowing shoreline access of the construction equipment for excavation of the gallery and placement of the piping. However, this is profoundly different from the wave conditions and construction environment at the Leadbetter and East Beach sites presented in Figure 1.2, which lie on the exposed open coast of Santa Barbara. At these two exposed beaches in Santa Barbara, a SIG will have to be built far offshore of the beach to avoid wave erosion, in rough water conditions with exposure to both local and distant open ocean storm waves. At the Leadbetter and East Beach sites in Santa Barbara, constructability of a SIG will be significantly more challenging than what was experienced at the Long Beach experimental SIG, and potentially problematic (see Section 6 for more detail on Santa Barbara comparisons).

To deal with the problems of constructing a SIG along exposed high energy coastlines, Dr. Robert Bittner of the Independent Science and Technology Advisory Panel (ISTAP) appointed by the California Coastal Commission suggested looking at installing the SIG drain and piping system in precast concrete boxes and then excavating a trench offshore with a dredge, followed by dropping precast concrete boxes into the trench. This is a very intriguing idea. The closest proxy to this idea is probably the precast concrete boxes used to build the tactical harbor breakwater referred to as *Mulberry* for the Normandy Invasion. Figure 2.4 shows a construction photo of some of the precast concrete boxes used in Mulberry in a shipyard in the south of England prior to the Normandy Invasion. From the scale of ladders shown in the photo, the Mulberry concrete box modules appear to be roughly comparable in size to what Dr. Bittner is proposing for an SIG to be installed at the Huntington Beach Desalination Facility, (HBDF), a site very similar to Santa Barbara in terms of wave exposure. Figure 2.5 shows the Mulberry concrete box modules in a neatly deployed detached breakwater system off Normandy, 5 days after D-Day, creating a tactical harbor known as *Port Winston*. Figures 2.6 & 2.7 show how the Mulberry concrete boxes look 60 years later resting on the seabed off Normandy France. It is apparent that the orderly arrangement of these boxes has been completely disrupted by the ensuing English Channel storms, and many of the boxes have also been tilted by the action of non-uniform subsidence and burial. There is also evidence of pronounced scour around some of the boxes that have subsided and tilted (Figure 2.7).

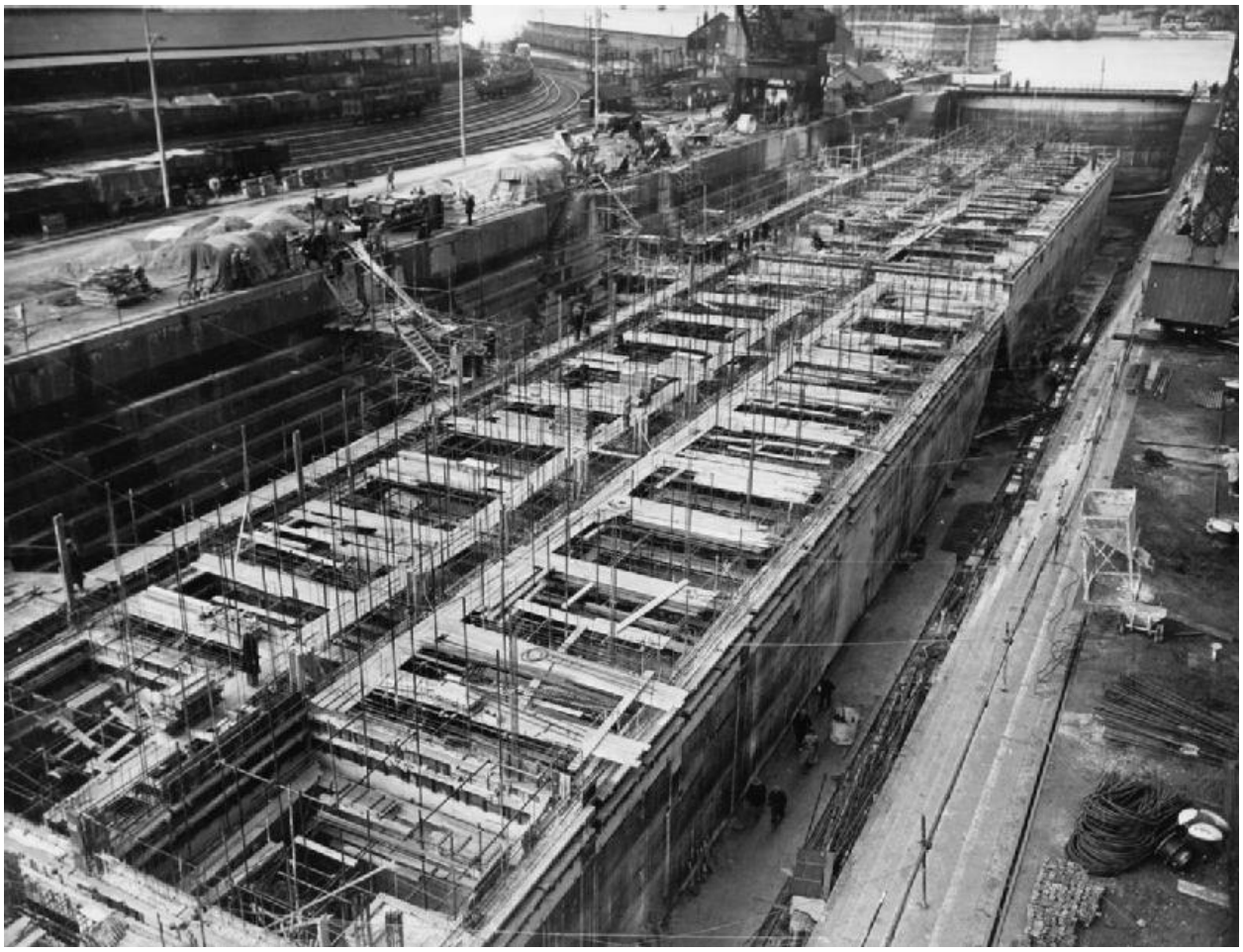

Figure 2.4: Construction of precast concrete boxes used in Mulberry prior to the Normandy Invasion, circa March, 1944.

Figure 2.5: Deployment of the Mulberry modules to form a detached breakwater system off Normandy France on D-Day plus 5 (11 June 1944).

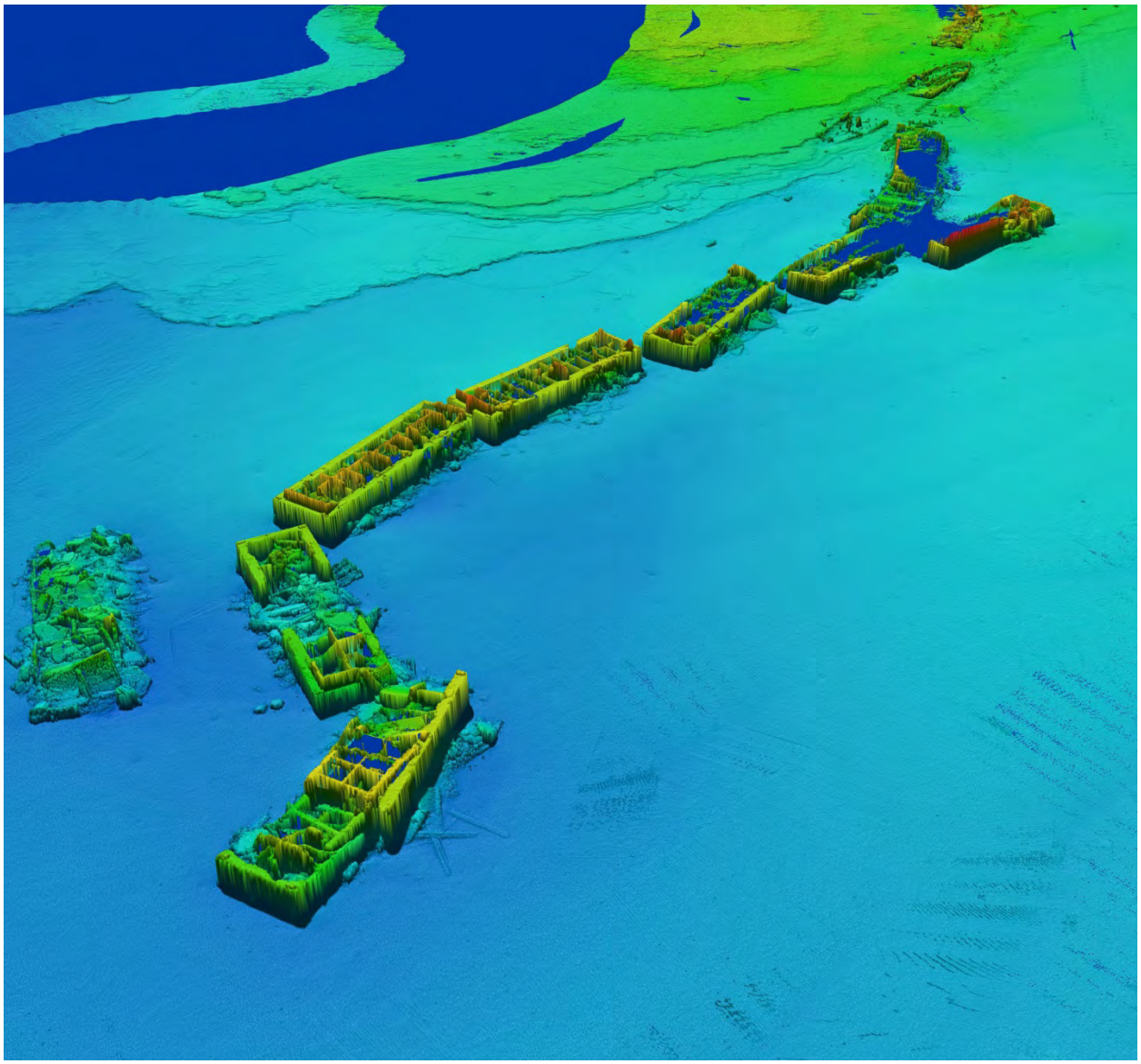

Figure 2.6: Multi-beam 3-dimensional sonar imagery of the Mulberry concrete box modules (upper) sunk off Normandy France in 2004. Figure courtesy of Prof. Larry Meyer, University of New Hampshire.

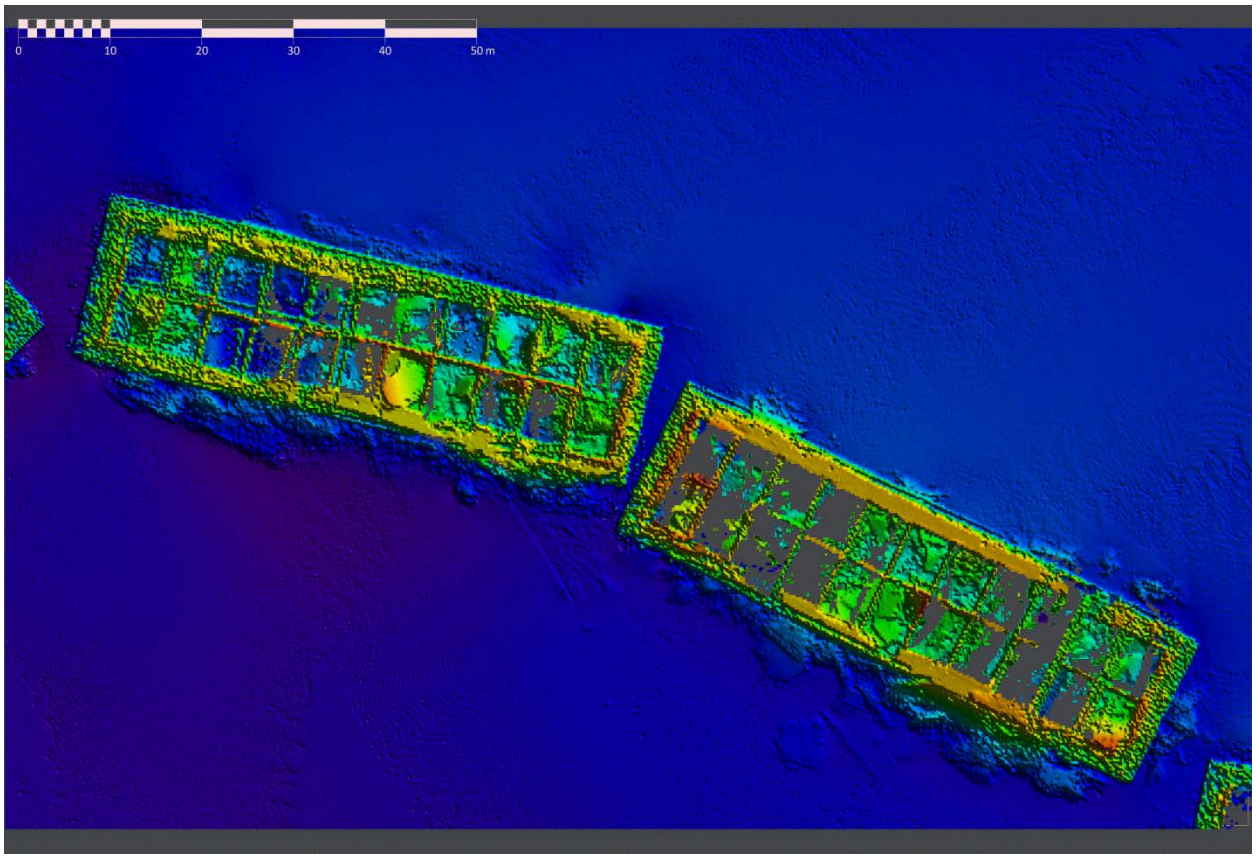

Figure 2.7: High-resolution multi-beam sonar imagery of two of the Mulberry concrete box modules sunk off Normandy France in 2004. Figure courtesy of Prof. Larry Meyer, University of New Hampshire.

From these examples off Normandy, the primary concern with the concrete boxed modular SIG is how to keep it level and well-ordered over time, regardless of whether it is set in a trench or is simply lying proud on the seabed. Non-uniform subsidence over time can arise from a variety of factors, including cyclical liquefaction and scour by the shoaling wave pressure and velocity fields, non-uniformities in the seabed sediment stratigraphy, and large scale bedforms that apply uneven dispersive (granular) pressures around the sides of the boxes, as found around the ship wreck off Normandy in Figure 2.8. The scour, liquefaction, and bedform factors can be largely remediated by moving the SIG offshore into deeper water beyond influences of shoaling wave pressure and motion. Offshore of Santa Barbara, wave effects should vanish at water depths of between 70 ft. and 90 ft. given the wave periods typical of the highest 13 % of incident waves. At those depths, it would be difficult to dredge a trench, and the project is focusing on City owned lands which extend off shore only about ½ mile. But if that constraint is relaxed, there seems to be no reason why the concrete boxed modular SIG couldn't simply rest proud on the seafloor, where it would create a very substantial artificial reef to attract sea life. However, there are several down-sides to the deep water solution. First, it will require a longer conveyance pipeline to the shore-side desalination facilities, a cost increase factor. Second, the seabed at water depths of between 70 ft. and 90 ft. is typically comprised of gray or green muds, indicating that the absence of wave motion allows for fine sediment deposition of washload from river floods which could eventually put a capping layer of mud on top of the engineered fill that was placed in the SIG box modules. That would reduce infiltration rates to the branch pipe network and degrade SIG source water production rates. Finally, the attraction of marine life to the SIG box modules by the artificial reef effect will ultimately lead to benthic organisms recruiting to and living in the engineered fill; and because that fill is confined by the boxes, its organic content will increase over time, ultimately reducing infiltration rates and degrading source water quality produced by the SIG. So, while an intriguing idea, it is not clear whether or not the precast concrete box modular SIG would actually reduce construction costs relative to the fully buried SIG concept built off a temporary pier.

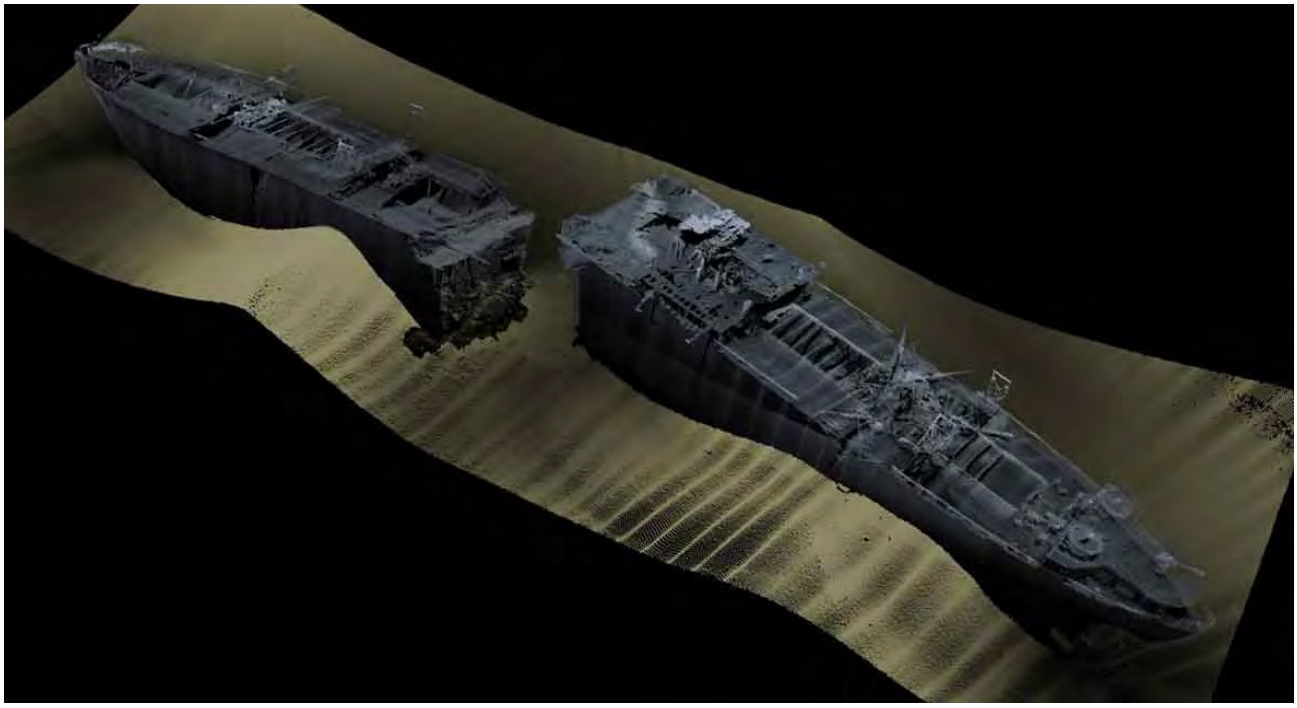

Figure 2.8: High-resolution multi-beam sonar imagery of sand waves around a ship wreck off Normandy France in 2004. Figure courtesy of Prof. Larry Meyer, University of New Hampshire.

Seabed infiltration galleries (SIG) and other shallow subsurface intakes are relatively innocuous. They are favored by environmental and permitting agencies for their perceived benefits in avoiding entrainment/impingement impacts, although no peer-reviewed studies have been done to definitively prove the minimizing effects on marine life (Foster, et. al., 2012). They are also less vulnerable to upsets from sporadic jellyfish runs and red tide occurrences, which could otherwise upset desalination plant operations (Pankratz, 2014).

In summary, the drawbacks to SIG designs are that their productivity and sustained reliability is highly site specific and determined by seabed sediment characteristics, underlying site geology and the wave and tidal activity. The construction of a SIG can have significant water quality and marine life impacts due to the need to dredge and remove a large section of ocean bottom habitat, obliterating the benthic communities of about 8.2 acres of seabed in the case of the Santa Barbara. Operation of the SIG could also result in marine life impacts due to periodic maintenance activities that disrupt benthic habitat and produce turbidity in the water column, (e.g., activities such as seabed raking, spot dredging and fill replacement). And, even when hydrologic conditions are favorable, the costs of a large offshore construction project may prove infeasible for many, especially smaller, projects (Pankratz, 2014). Large-scale seabed infiltration galleries can be technically complex to construct. The technical complexity of a SIG is compounded during long-term operation by the difficulty to adequately clean the laterals and distribution piping when they become partially clogged. All well types

require periodic maintenance and cleaning which can be easily accomplished in conventional vertical wells, but can be quite complex for a SIG because of its long distance from the shoreline, particularly at Santa Barbara where an SIG must be sited far offshore to avoid wave erosion (see Section 6 for more detail). In offshore locations where the bottom sediment is unconsolidated, (as is the case offshore of Santa Barbara), construction requires the use of sheet piling. The handling and placement of large sheet pile sections in water depths on the order of 12 m (39 ft) would be extremely challenging in the high energy sea-states which regularly occur offshore of Santa Barbara (see Section 6 for more detail).

2.2) Beach Infiltration Gallery (BIG), Long Beach & Huntington Beach:

When a SIG is moved close to shore or inside the surf zone, it is referred to as a *beach infiltration gallery* (BIG).[1] Recently, the Independent Science and Technology Advisory Panel (ISTAP) appointed by the California Coastal Commission considered several coastal processes and construction aspects for implementing BIG intake technology at the Huntington Beach Desalination Facility (HBDF). Like the Santa Barbara site, the HBDF is also sited on an exposed high energy coast with very active beach and shoreline variability. In this regard, the ISTAP addressed several specific questions:

"What are the potential shoreline stability impacts on a Beach Infiltration Gallery (BIG)? How much vertical movement of the sand level and horizontal movement of the surf zone could be anticipated as a consequence of seasonal and episodic shifts in the beach profile?"

Using the Huntington Beach Desalination Facility (HBDF) as a surrogate to answer this question, Figures 2.9 and 2.10 show the measured beach and shore-rise profiles at the SA-180 range line, (located 191 m south of the HBDF), that has been monitored by the U.S. Army Corps of Engineers, Los Angeles District, between October 1918 and January 1994, (USACE, 1994). This historically surveyed range line is in the approximate neighborhood of the optimal SIG site identified in Jenkins and Wasyl, 2014. The envelope of variability defined by these profiles (*critical mass envelope*) reveal the potential range of variability in the beach profiles as a consequence of seasonal and tidal effects and climate cycles such as El Nino Southern Oscillation (ENSO), as well as episodic effects such as accretion/erosion waves propagating through the HBDF area from the beach nourishment activities associated with the *San Gabriel River to Newport Bay Erosion Control Project.* To a certain degree, these profiles also reflect the effects of sea level rise over a 76 year period, but certainly not to the degree anticipated by 2050, when sea level is expected to rise another 4 and 24 inches (10.1 to 61 cm), according to California State recommended projections. Figures 2.9 and 2.10 are both annotated for the tidal elevations of MHHW and MLLW according to the NOAA tide gage #941-0660 at the Port of Los Angeles. We find that the mean diurnal tidal range overlaid on the historic variability in the beach profiles leads immediately to a 240 m uncertainty in the on/offshore location of the shoreline at any given time. The surf zone begins at the

[1] Also referred to as *Onshore Infiltration Gallery* and *Lateral Beach Wells* in literature data.

shoreline and extends seaward to the wave breaking point, which from Hunt (1959), is a function of the local water depth:

$$h_b = H(x)/\gamma \tag{1}$$

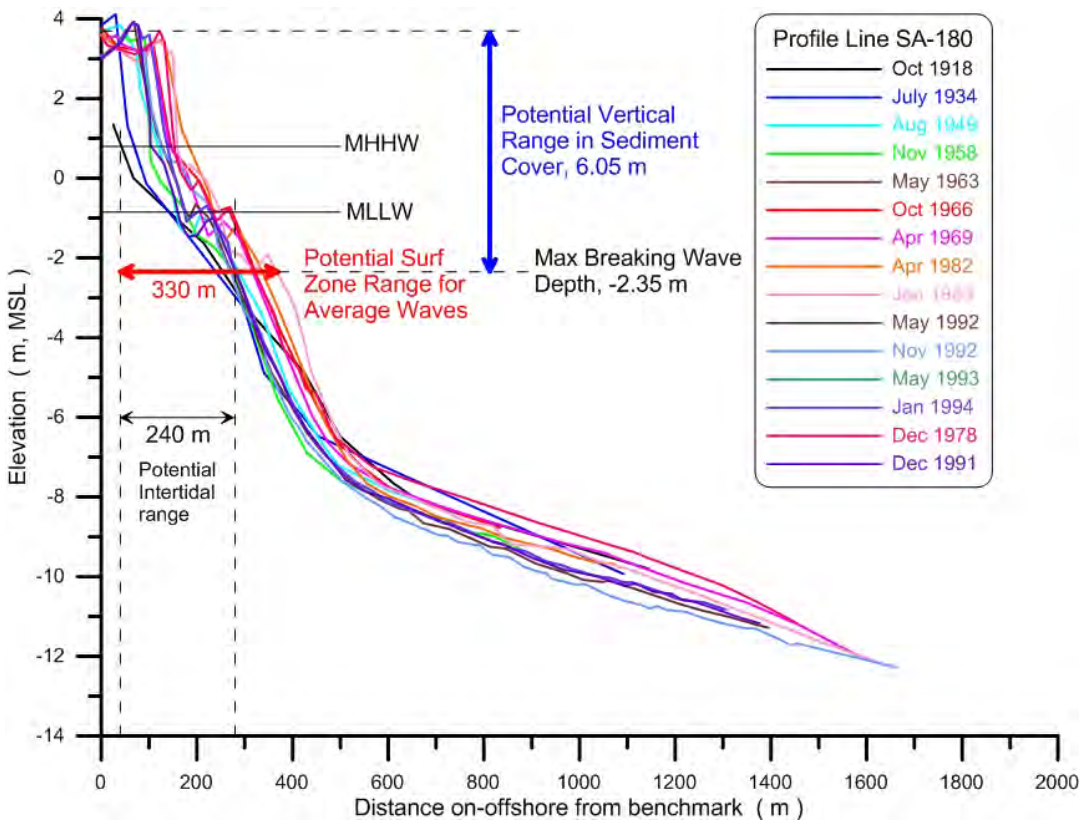

Figure 2.9: Measured beach and shore-rise profiles at the SA-180 range line, (located 191 m south of the HBDF), monitored by the U.S. Army Corps of Engineers, Los Angeles District, between October 1918 and January 1994. Data from USACE, (1994). Annotations are given for average wave climate with deep water incident wave heights in the range of 0.9 m and 1.2 m.

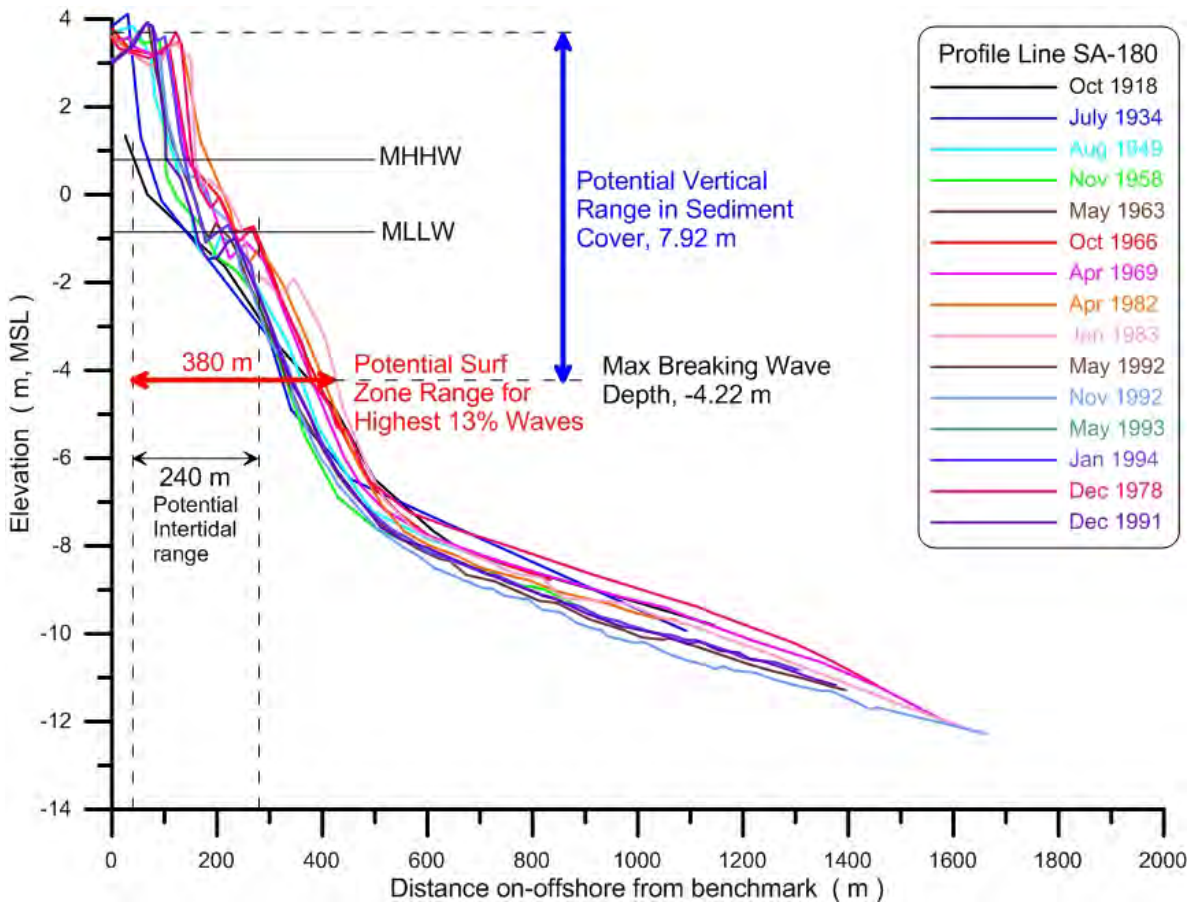

Figure 2.10: Measured beach and shore-rise profiles at the SA-180 range line, (located 191 m south of the HBDF), monitored by the U.S. Army Corps of Engineers, Los Angeles District, between October 1918 and January 1994. Data from USACE, (1994). Annotations are given for the highest 13 % waves with deep water incident wave heights in the range of 2.4 m to 2.7 m, with some waves reaching significant heights as large as 4 m to 6m.

Where h_b is the depth of wave breaking, γ is the breaker factor, and $H(x)$ is the shoaling wave height calculated from the incident wave height H_∞ and period T using Stokes theory:

$$H(x) = \frac{H_\infty}{\sqrt{2\sigma}}\left(\frac{g}{h(x)}\right)^{1/4} \quad and \quad \sigma = \frac{2\pi}{T} \qquad (2)$$

For average waves, (with deep water incident wave heights in the range of 0.9 m to 1.2 m) the maximum depth of wave breaking calculates at -2.35 m MSL (Figure 1); and for the highest 13% waves (with deep water incident wave heights in the range of 2.4 m to 2.7 m) the maximum depth of wave breaking is -4.22 m MSL. This means that the on/off shore variability of the surf zone can be as much as 330 m between the most eroded beach profile and the most accreted profile under average wave climate conditions

(Figure 2.9), and as much as 380 m for the highest 13% of incident waves (Figure 2.10). If we examine the vertical variation in the beach sand levels across these ranges of surf zone variability, we find as much as 6.05 m of vertical variation under average wave climate (Figure 2.9) and 7.92 m of vertical variation for the highest 13% of incident waves. This means that one would probably have to excavate as much as 8 m of sediment overburden to completely bury and level a Beach Infiltration Gallery in the surf zone. If one merely looks at the profiles in Figures 2.9 and 2.10 from afar, it is apparent that the profile envelope (*critical mass envelope*) is much steeper and thicker in the surf zone than offshore near closure depth where the SIG was optimally sited in Jenkins and Wasyl, (2014), indicating that the challenges of burying and leveling an infiltration gallery diminish as one goes further offshore. Ideally, a Beach Infiltration Gallery should be built when the beach and shore-rise profiles are in their most eroded state. This would lessen the likelihood of exposure of the BIG by future erosion. This opportune construction scheduling would most likely coincide with cessation of winter waves during an El Nino year.

One of the expected advantages of a BIG over a SIG is that construction costs could be reduced by moving closer to shore because a shorter temporary pier would be required for construction. However, mobilization, labor and time on-job are major cost factors. Moving the gallery closer to shore puts the construction work in a regime of higher waves and greater wave induced currents as a consequence of wave shoaling. As waves propagate into shallower water, they shoal and increase in height according to Equation (2); and eventually break once the water depth becomes roughly 5/4 the shoaling wave height. It is difficult to see how construction costs are reduced by moving shoreward into a more difficult construction environment. At offshore locations near closure depth, it is estimated there would be 13% loss in construction time due to high sea states that would cause excessive pendulation to crane operations from the temporary pier, or loss of engineered fill as a consequence of excessive water motion (Jenkins and Wasyl, 2014). That *down-time* number would undoubtedly increase to perhaps 18 % or 20 % if the preponderance of work is performed further inshore where sea states are higher. The reduced materials costs of a shorter construction pier must be weighed against loss of on-job time and perhaps heightened risk of component damage while trying to work in the higher states encountered near shore.

2.3) Advanced Horizontal Well Technology:

Horizontal well construction has rarely been used in the water industry, but has a variety of potential applications. A key issue is matching the technology to the specific geologic conditions at a given site to maximize the efficiency of withdrawal within the framework of the fundamental groundwater hydraulics. Most un-lithified sediments are deposited in horizontal layers that make vertical wells very effective because the screens can be placed perpendicular to the bedding planes and tend to take advantage of the generally high horizontal to vertical ratio of hydraulic conductivity. If it is the purpose of a horizontal well to induce vertical flow, such as in the case of drilling beneath the seabed, then use of the technology does have the advantage of producing high yields per individual well. If the aquifer to be used is semi-confined or not well-connected vertically to the overlying sea, then the wells may not be effective in producing high, sustainable

yields. Also, great care must be taken in use of horizontal wells beneath the seafloor in terms of water quality because the well may pass through zones of sediments containing varying oxidation conditions along the axis of the well. Mixing of oxygenated seawater with anoxic seawater within the well, especially where hydrogen sulfide is present, can lead to the precipitation of elemental sulfur that would require removal before entry into the membrane reverse osmosis (RO) treatment process. Also, the oxidation issue can cause precipitation of ferric hydroxide or manganese dioxide. The con- figuration of using horizontal wells as intakes for SWRO plants appears to have considerable advantages (Missimer, et. al., 2013; Delhomme, et. al., 2005).

In recent years horizontal well intakes have been installed in several facilities in Spain at San Pedro del Pinatar, Aguilas, and Alicante, with the highest capacity reported at 45.6 mgd; where the schematic layout is shown in Figure 2.11a, (Malfeito,J.,2006; Malfeito, J. and A. Jimenez, 2007; Peters, T. and D. Pinto, 2007 & 2010). It features a German manufactured infiltration pipe with micron-size infiltration holes that are reportedly immune to clogging, and minimize the need for periodic maintenance in long-term operation. With the Neodren™ directional drilling technology, sub-seabed installation of infiltration piping as large as 20 inch diameter can reportedly be accomplished over distances of 2500 ft from the shore-side drill entry point. Unfortunately, there have been few operating data reported from the larger capacity SWRO facilities currently using this intake type. The standard Neodren™ method (which is protected by international patent) requires a “pop-up” or access hole that penetrates the seafloor at the seaward end for each well branch in order to install the infiltration pipe by means of pulling it landward through the pop-up hole toward the shore-side HDD entry point of the well branch. This pulling operation, conducted from a barge or ship moored offshore, requires calm sea-states, an oceanographic condition at Huntington Beach that seldom persists for any length of time. During high sea-states, heave/roll responses of the service the service vessel during the pulling operation can result in significant strain on the infiltration pipe, potentially damaging the pipe or compromising the pop-up hole and its end-works. However, recent developments in HDD technology may allow installation of Neodren™ pipe by pushing it seaward from the shore-side HDD entry point of the well branch. By this method (which is not restricted under the Neodren™ patent) there is no seafloor disturbance of any kind, thereby greatly diminishing challenges of securing a Coastal Development Permit, however, there is currently no known or citable industry experience with this method of construction for a water plant intake domestically or abroad.

Data on silt density index (SDI) for a Neodren™ system compared to multi-media filtration and ultrafiltration show a value of 5.1 compared to 3.4 and 3.2, respectively, on one system and 4.6 compared to 2.6 and 2.4, respectively, on another system with the locations of the systems not given (Peters, T. and D. Pinto, 2007). Typical seawater SDI values commonly are greater than 10 (both SDI_{10} and SD_{I5}), which suggest that the horizontal well system does improve water quality somewhat, but not sufficiently to preclude the need for a pre-treatment train in the SWRO facility. However, no data on organic carbon or bacteria removal are presented in the literature touting this technology.

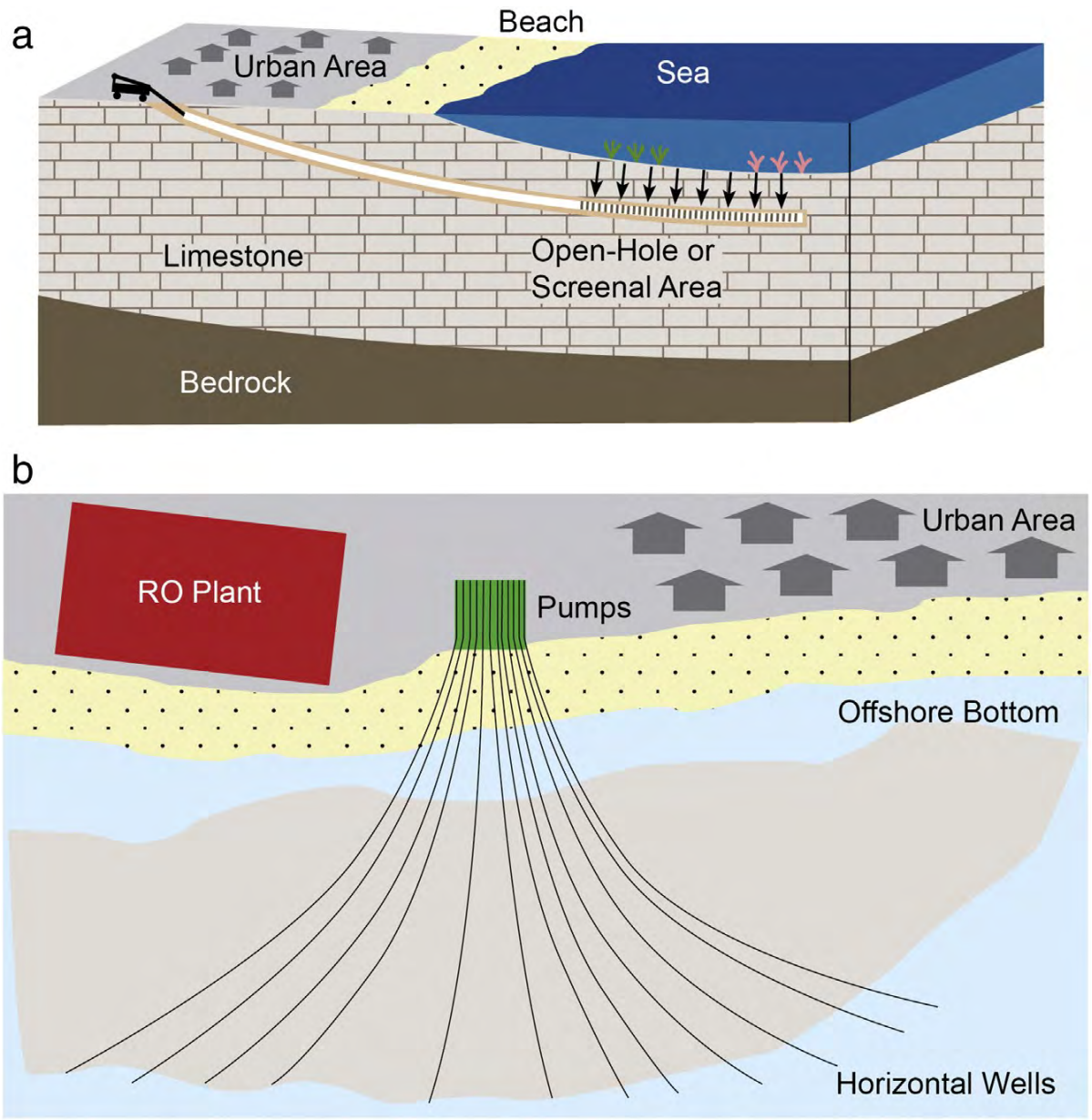

Figure 2.11. Horizontal wells can be drilled from the shoreline using older mature technology or the Neodren™ system. (a) General configuration of a horizontal system. (b) Horizontal well systems can be configured to allow multiple wells to be drilled from a compact location, saving land cost and allowing pumps to be housed in a single building. (Missimer, et. al., 2013).

An issue requiring consideration in the selection of a horizontal well intake is the elimination of feasibility and operational risk. While the assessment of groundwater sources adjacent to the shoreline is rather well established, the hydro-geologic characterization of the offshore sub-bottom requires specialized equipment and methods which are expensive and may still leave questions that cannot be easily answered, such as on sub-bottom oxidation state of the water and horizontal geological variations that

could reduce or eliminate productivity of the well(s). The drilling of test borings and obtaining accurate water quality samples can be difficult if not impossible under some conditions, where the offshore bottom slope is very steep or where wave action is intense, not allowing use of barge-mounted drilling equipment.

Another important issue concerning the long-term operation of any horizontal well system is the ability to adequately clean the well when it becomes partially clogged (Missimer, 2009). All well types require periodic maintenance and cleaning which can be easily accomplished in conventional vertical wells using weak acid and various redevelopment processes, such as air or water surging, sonic disaggregation and redevelopment, or some combination of processes depending on the nature of the clogging, such as calcium carbonate scaling, iron nodule precipitation, or biofouling (Driscoll, 1986). Maintenance work on a horizontal well can be quite complex because of its long distance from the shoreline and the presence of screen in the well that could be damaged during maintenance due to the cleaning pipe traveling on the lower screen surface of the well. Neodren™ recommends maintaining an offshore pop-up access hole to service periodic maintenance. In a high energy wave environment, such as exists at Santa Barbara where there is significant littoral transport, (Inman and Jenkins, 2004c), the access hole and its end-works could become buried. This represents a maintenance challenge simply to maintain access to, and prevent burial of, the pop-up holes and their end works; only after which the subsequent maintenance of the infiltration branches is possible. However, a cleaning tool has been recently developed to clean the Neodren™ pipe using landside high pressure feed water; thereby eliminating the need for a pop-up hole to perform maintenance.

There are additional considerations for the feasibility of implementing horizontal well technology at Santa Barbara. Given the infiltration rate estimates and source water requirements of the City's desalination plant, an array of 5 horizontal well branches, each 2500 ft. in length, is probably required; and each branch requires a certain minimum separation at the shore-side drilling location (Figure 2.11b). Given these issues and concerns, the determination of the feasibility of implementing advanced horizontal well technology at the City's desalination plant site will require additional oceanographic, geotechnical / hydro-geological, and marine construction analysis.

2.4) Radial Collector Wells (Ranney Collectors) at Landside Sites:

Radial collector wells are characterized by a central caisson typically having a 10 ft. to 16.5 ft diameter with a series of laterals which are screened to allow water flow to move into the caisson during pumping as presented in Figure 2.12. Radial wells are commonly used to provide large-capacity intake capability along rivers in parts of the United States and in some European locations. Operational radial collector well capacities range from 0.1 mgd to 13.6 mgd, (Missimer, 1997; Hunt, 2002). The only known operating collector well system used for a SWRO intake is located at the PEMEX Salina Cruz refinery in Mexico (Voutchkov, 2005), which has three wells, each with a capacity of 4 mgd.

The geologic conditions that favor a radial collector well design over a conventional or horizontal well design are the occurrence of thick gravel beds at a relatively shallow depth that have a preferentially high hydraulic conductivity compared

to the overlying sediments. High-yield radial collector wells could be successfully developed in the gravel unit by installing the collector laterals in the gravel that extend under the seabed. Collector laterals could be installed only on the seaward side of the well to eliminate impacts to fresh groundwater resources occurring in the landward direction and to also eliminate the potential for drawing contaminated water or water having high concentrations of undesirable metals, such as iron and manganese, into the well field (Figure 2.12).

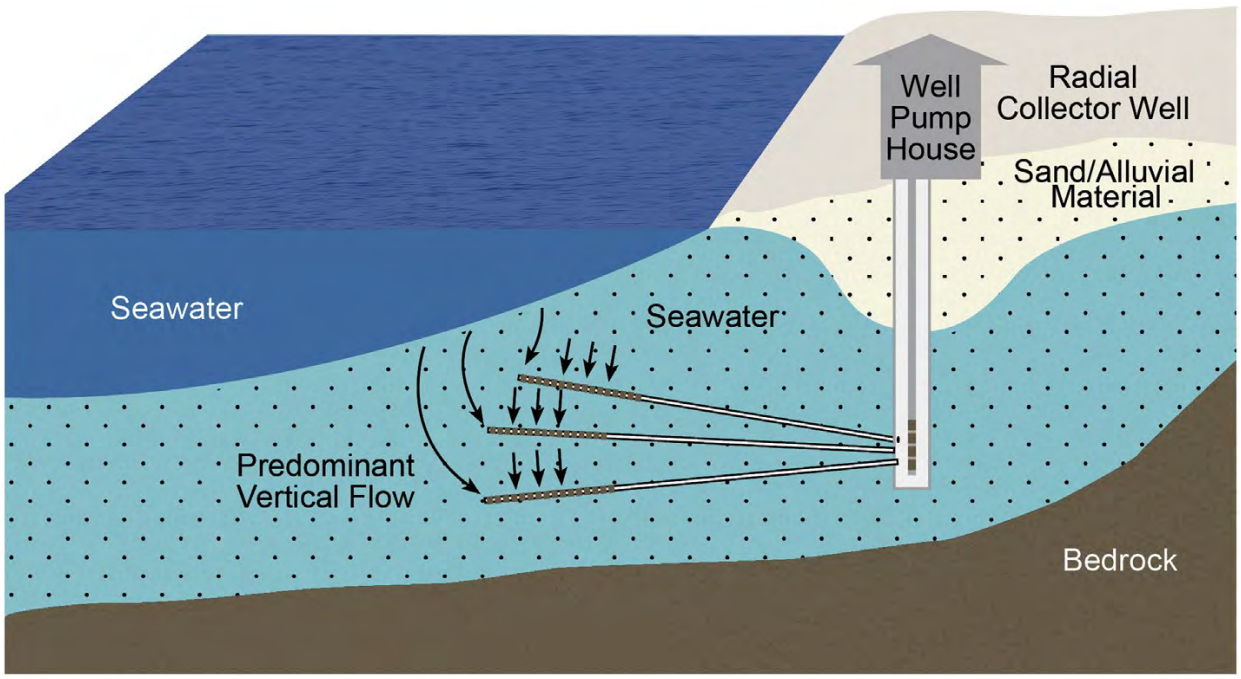

Figure 2.12: Typical design from a radial collector or Ranney well. The laterals can be designed to extend beneath the seabed to only vertical recharge through the seabed, precluding landward impacts. Note that the laterals occur on a single plane and many can be installed. The well pump house would be replaced by submerged pumps located in the intake caisson of the HBDF. (Missimer, et. al., 2013).

Proper aquifer characterization is required in the design of a radial collector well intake system. While the test program to determine potential yield of individual wells and the required space between them is relatively easy to perform (same as conventional wells), the assessment of water quality within the sediments can be more complex. It is quite important to assess the redox state of the water to be pumped because radial wells have a caisson that allows air to come in contact with the water originating in the laterals. If the water flowing into the well from the coastal aquifer contains hydrogen sulfide, iron (Fe^{2+}), or manganese (Mn^{2+}), it could react with the dissolved oxygen in the water temporarily stored in the caisson and precipitate elemental sulfur, ferric hydroxide, or manganese dioxide respectively, any of which can foul the cartridge filters and membranes (Missimer, 1997 & 2009).

Radial collector wells have an advantage over conventional vertical wells in that the individual well yields can be very high. However, they do require location near the

shoreline and are therefore subject to beach erosion and storm wave damage. They could be used to produce large quantities of feed water in areas where the geology is supportive and the tidal water is relatively calm with low wave action. Since individual wells can yield up to about 13 mgd, they could be used to supply feed water to SWRO systems operated from land-based platforms. However radial collector wells have never been operated from offshore platforms anywhere in the world, not even as small experimental installations. Using best land-based estimates, at least 5 Rainey wells on offshore platforms would be required for the generation of the approximately 20 mgd of maximum source water capacity needed to operate the City's Desalination Plant. In addition, no long-term operating data are available on the radial collector wells used for SWRO intakes, either on or offshore. There are potentially greater risks associated with radial collector wells because a substantially large investment in their construction occurs before their performance can be known with certainty.

2.5) Applicability of Shallow Sub-Seabed Intakes to the Santa Barbara Site:

In contrast to the sites where previous sub-surface intakes have been built for small-scale seawater desalination plants, (e.g., Fukuoka Japan, Long Beach Harbor, San Pedro del Pinatar, Aguilas, and Alicante, Spain, and Salina Cruz, Mexico), Santa Barbara is located on the exposed open coast of the Santa Barbara Channel in the northern arc of the Southern California Bight, fully open to long period swells from the Gulf of Alaska winter storms, Mexican Hurricanes in summer and long period swells from the Southern Hemisphere in late summer and early autumn (The Southern Winter). This entire geologic province is an eroding collision coast with a major sediment sink for the Santa Barbara Littoral Cell (Figure 2.13) located in the neighborhood of Pt. Mugu (the Mugu Submarine Canyon). Sediment cover over the project site (Figure 2.14) is highly variable due to strong littoral drift rates moving sandy sediments down-channel from west to east. Much of the net littoral drift is captured by the Santa Barbara Harbor, but is subsequently removed by periodic maintenance dredging of the harbor. The harbor dredging activities result in bypassing 314,000 cubic yards of sandy sediments annually around the harbor with post-dredge disposal of these quantities on East Beach, (Figure 1.2). This action insures that the sandy sediments at the Leadbetter Beach, West Beach, and East Beach sites are frequently removed by either erosion or dredging and replaced by new inflows of littoral drift and/or dredge disposal. Consequently, SIG or BIG subsurface intake systems that rely on engineered fill consisting of gradated layers of sands and gravels must be designed such that the engineered fill is placed beneath the mobile sediment cover at these candidate sites.

The major drainage basins supplying sediment to the Santa Barbara Littoral cell are numerous small streams and creeks draining the west faces of the Santa Ynez Mountains up-drift of Santa Barbara Harbor, and the major Ventura and Santa Clara Rivers, down-drift of Santa Barbara. The small Santa Ynez watersheds supply 196,000 cubic yards of sandy sediments annually to the littoral drift directly up-drift from the Santa Barbara Harbor, which is augmented by another 86,000 cubic yards annually from cliff and bluff erosion (Figure 2.13). These same processes of watershed and bluff erosion and harbor dredging/bypassing also occur further down-drift from Santa Barbara in the lower reaches of the littoral cell around the cities of Oxnard and Ventura, but at

even greater rates than at Santa Barbara. However, these down-drift processes do not impact the seafloor stability around Santa Barbara because the littoral drift is predominately a one-way sediment transport pathway that is maintained by the island sheltering effects on incident waves caused by the Channel Islands archipelago. Island sheltering limits incident waves to a predominantly westerly direction, so that waves always shoal at a very oblique angle relative the shoreline of the Santa Barbara Littoral Cell, producing a string gradient in wave radiation stress directed toward the east. Consequently, all sediment transport in the Santa Barbara Littoral Cell proceeds from west to east and is lost to the action of turbidity currents flowing down the Hueneme Submarine Canyon, Mugu Submarine Canyon and into the abyssal fan beyond the continental shelf.

Ultimately the Santa Barbara Littoral Cell is a constant loss sedimentary system that is only maintained by a continually source of new sediment input provided by erosion of the Santa Ynez watersheds and coastal bluffs. These watershed and bluff formations lie within and between structurally complex folds and thrust faults with appreciable vertical slip and overturned beds. These formations are predominantly Cenozoic sediments of Pliocene through Eocene age that are relatively unconsolidated and easily eroded. While the Santa Ynez creeks provide locally marginal sediment cover for a SIG or BIG in the nearshore areas at Santa Barbara, that sediment cover is not especially thick, and is constantly being replaced by seasonal depositional/erosion cycles ever since the Holocene period up to and including present time (Inman and Jenkins, 1999; USGS, 2004). The isopach map in Figure 2.14 (delineating contours of constant sediment cover thickness), shows that the sediment cover over bedrock in the vicinity of the proposed intake sites off Leadbetter Beach, and East Beach are only 2 m to 4 m (6.5 to 13 ft) thick, with areas of sediment cover thickening further inshore. These areas of localized thickening are the shorerise and bar-berm beach formations that are in a constant state of flux. Although the Santa Ynez watersheds that supply most of the nearshore sediment cover around Santa Barbara are largely influenced by a semi-arid Mediterranean type climate, the periodic occurrence of El Nino floods throughout the last 6000 years of the Holocene have resulted in episodic flood-induced depositional formations (ephemeral deltas, sand waves, bars and hummocks) with layers of sandy and silty material in the offshore sediment stratigraphy. These broad-scale and continuing geomorphic and climatic processes are highly unfavorable for the future maintenance and sustainability of the engineered fill material of a SIG or BIG at Santa Barbara, because engineered fill that is lost during an erosion cycle will at some point be replaced by the more silty sediments during post-El Nino flood deposition and dispersion.

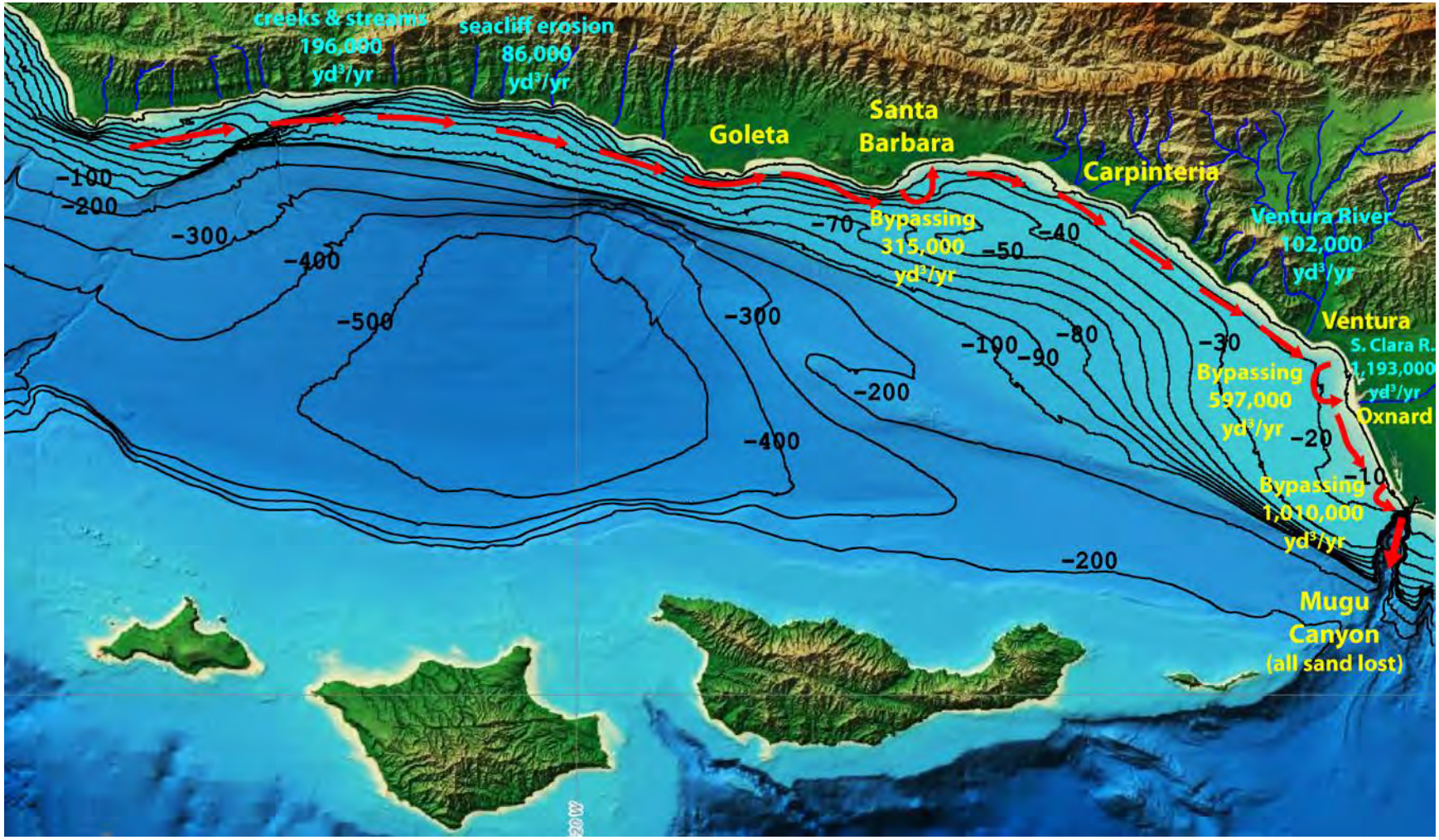

Figure 2.13: Santa Barbara Littoral Cell. Farfield bathymetry depth contours (black) in meters MSL from NGDC (2103). Dashed red line shows littoral drift pathway from natural sediment sources along the south faces of the Santa Ynez Mountains and from dredging of dredging and bypassing from Santa Barbara Harbor, Ventura Harbor and Port Hueneme. Sediment yield and transport rates from BEACON, (2009)

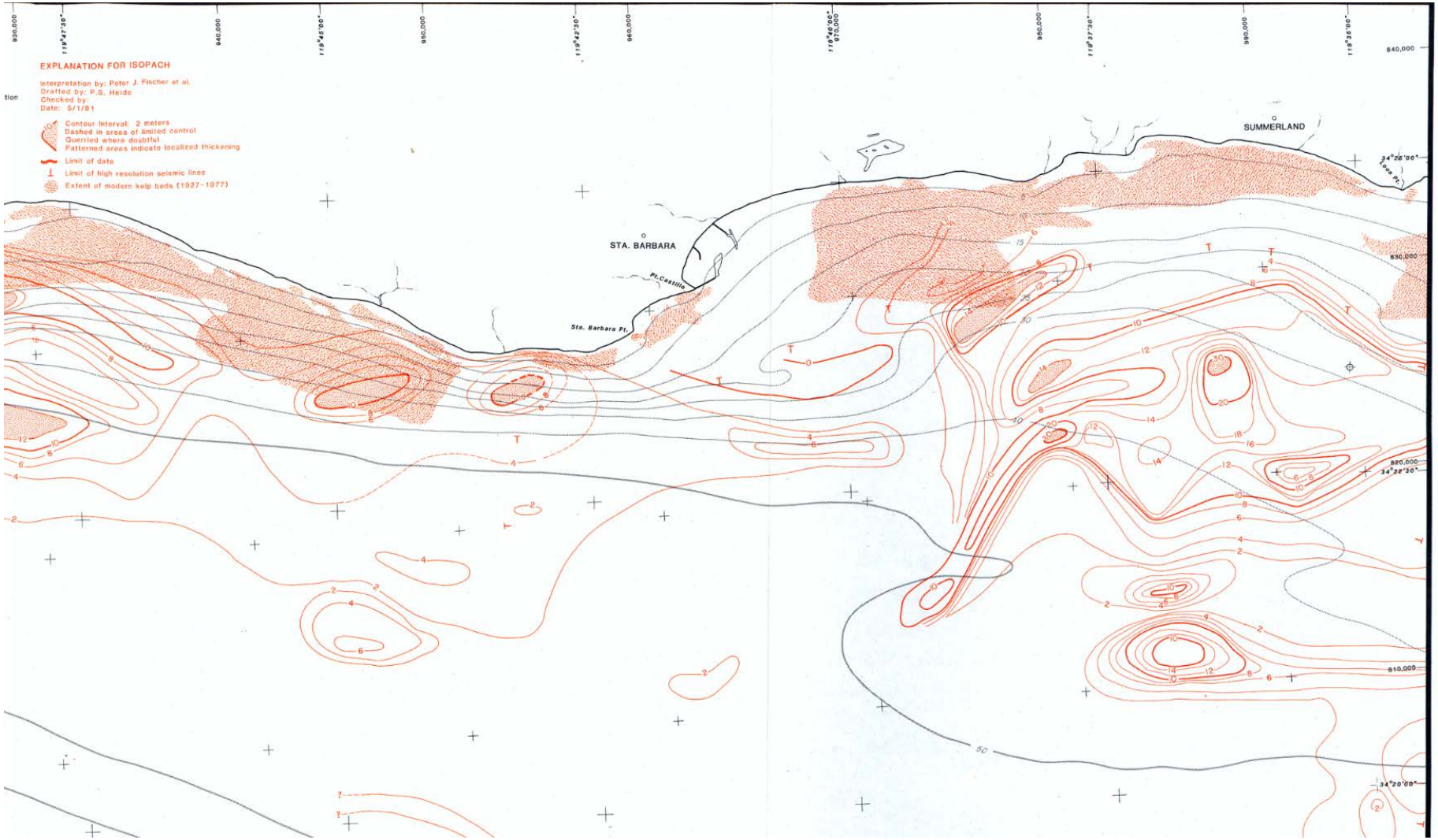

Figure 2.14: Bathymetry/isopach map of the area around Santa Barbara. Bathymetry depth contours are in black (meters MSL) and sediment thickness contours (isopachs in meters) are in red. The shaded red areas indicate areas of localized thickening, usually due to seasonal beach profile changes. The T's denote the limits of seismic data. From, USGS, 2004.

Seabed Infiltration Galleries (SIG), the Beach Infiltration Galleries (BIG), and the Neodren™ Seawater Intake System all require three precise geomorphic conditions of the site location for successful operation. These are: 1) adequate sediment cover, 2) the proper grain size distribution within that sediment cover (no lenses of silts and clays), and 3) a stable seabed. All are vulnerable to exposure by erosion; and conversely all are vulnerable to impaired infiltration rates due to new deposition of silts and clays on the seabed following construction. If the sediment cover becomes capped with lenses of newly deposited fine grained silts and clays, the permeability of the sediment cover will be inadequate to provide required/design feed water. The SIG and BIG intake technologies must have on the order of 10 ft of sediment cover or more that is predominantly sands and/or gravels to provide adequate seabed permeability and insure high infiltration rates of seawater. While the Obayashi Seabed Infiltration Gallery can be made to provide that type of sediment cover through the use of engineered fill, the Neodren Seawater Intake can operate with substantially less native sediment cover, on the order of only 2 ft. to 4 ft., provided that sediment cover is not lost to wave erosion.

At Santa Barbara, the constructability of the Obayashi Seabed Infiltration Gallery at RBGS and ESGS is questionable because it requires excavation of a dredged pit to elevations of 10 ft below ambient seabed in which the infiltration branch pipe segments and engineered fill are subsequently placed. From Figure 2.14 it does not appear that 10 ft. of sediment cover is available continuously over large nearshore areas. Moreover, such offshore excavation activity is surely a time consuming process in high-energy sea states, as are common off Santa Barbara (Inman and Jenkins, 1996, 2004c). Therefore, it would be exceedingly problematic to get a calm sea state of sufficient length of time to complete this kind of construction, and the dredged pit is likely to collapse before the infiltration pipes and engineered fill can be placed. To avoid this, the Obayashi Seabed Infiltration Gallery must be constructed a considerable distance off shore, beyond closure depth, (the depth beyond which seabed erosion or accretion ceases, typically at about –15 meters MSL depth). Construction in such deep water is undoubtedly more difficult from a mechanical perspective, and consequently more expensive and problematic (Inman and Jenkins, 1996). For this reason, the only sensible construction option for either a SIG or a BIG is to first build a temporary pier from which the SIG and BIG holes can be dredged and the piping and engineered fill subsequently placed. On the other hand, the Neodren™ Seawater Intake is insulated from these construction problems (over distances of no more than 3000 ft from the shore-side drill entry point) due to its directional drilling techniques. However, it is probably desirable to place the Neodren™ Seawater Intake close to shore where wave induced bottom stresses are large and capable of re-suspending or even preventing deposition of lenses of fine grained silts and clays. Based on these considerations we proceed with a sediment budget and seafloor stability analysis tailored to the Neodren™ system, as the SIG and BIG alternatives are more costly and difficult to construct.

3) Technical Approach:

To quantitatively evaluate the problems of implementing subsurface intake technology at Santa Barbara, we invoke a numerical seabed stability analysis utilizing the ***Coastal Evolution Model*** applied to the Santa Barbara Littoral Cell (Figures 2.13) and to the Goleta Subcell (Figure 3.1). 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).

3.1 General Description and Architecture:

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 3.2) 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 Santa Barbara Littoral Cell (the region of coastline between Point Conception and Point Mugu, Figure 2.13) is divided into a series of coupled control cells (Figure 3.3). 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 3.2, upper) has been used to predict the change in shoreline width and beach profile resulting from 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.

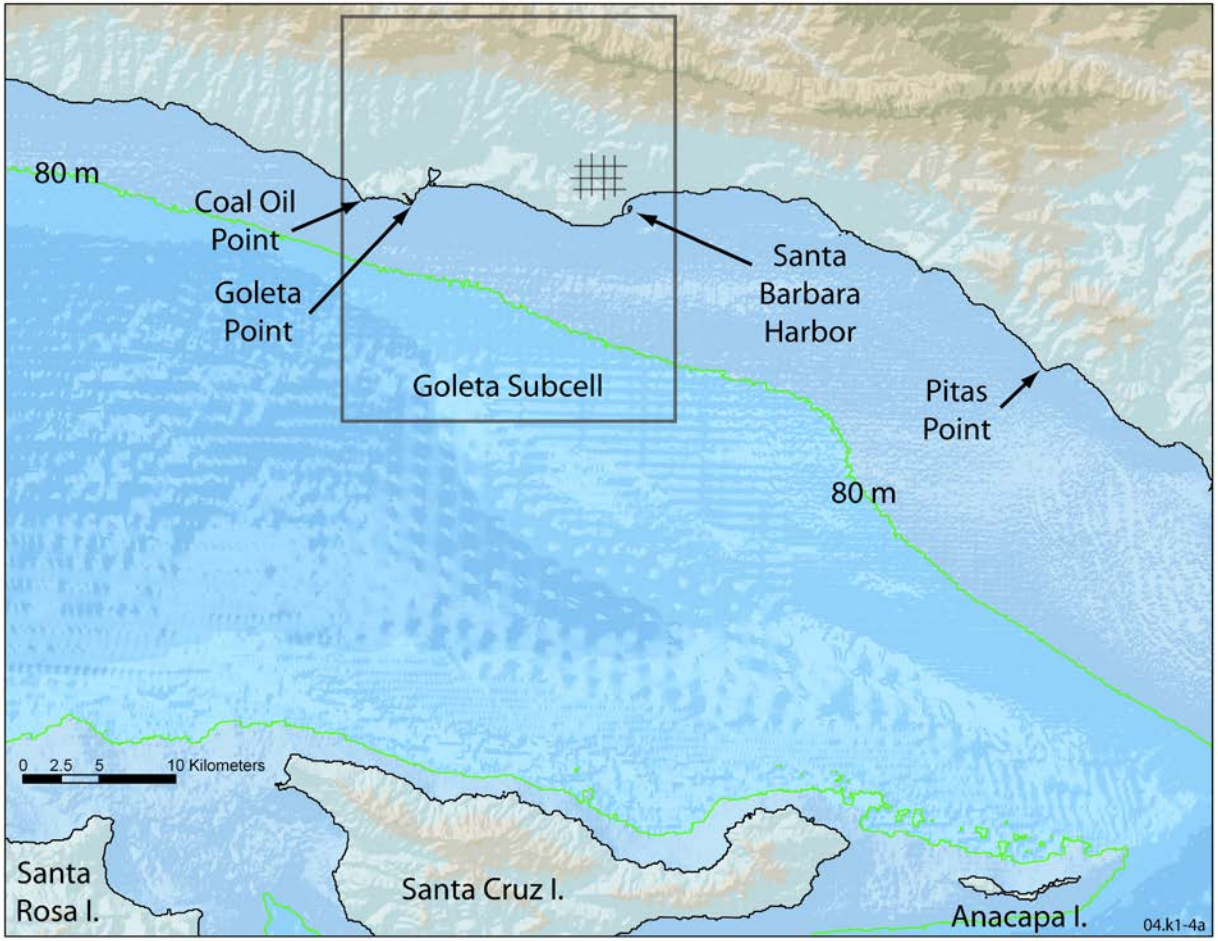

Figure 3.1: The Goleta Subcell used as CEM computational control cell for numerical seabed stability analysis of the City's Santa Barbara Desalination Plant study area.

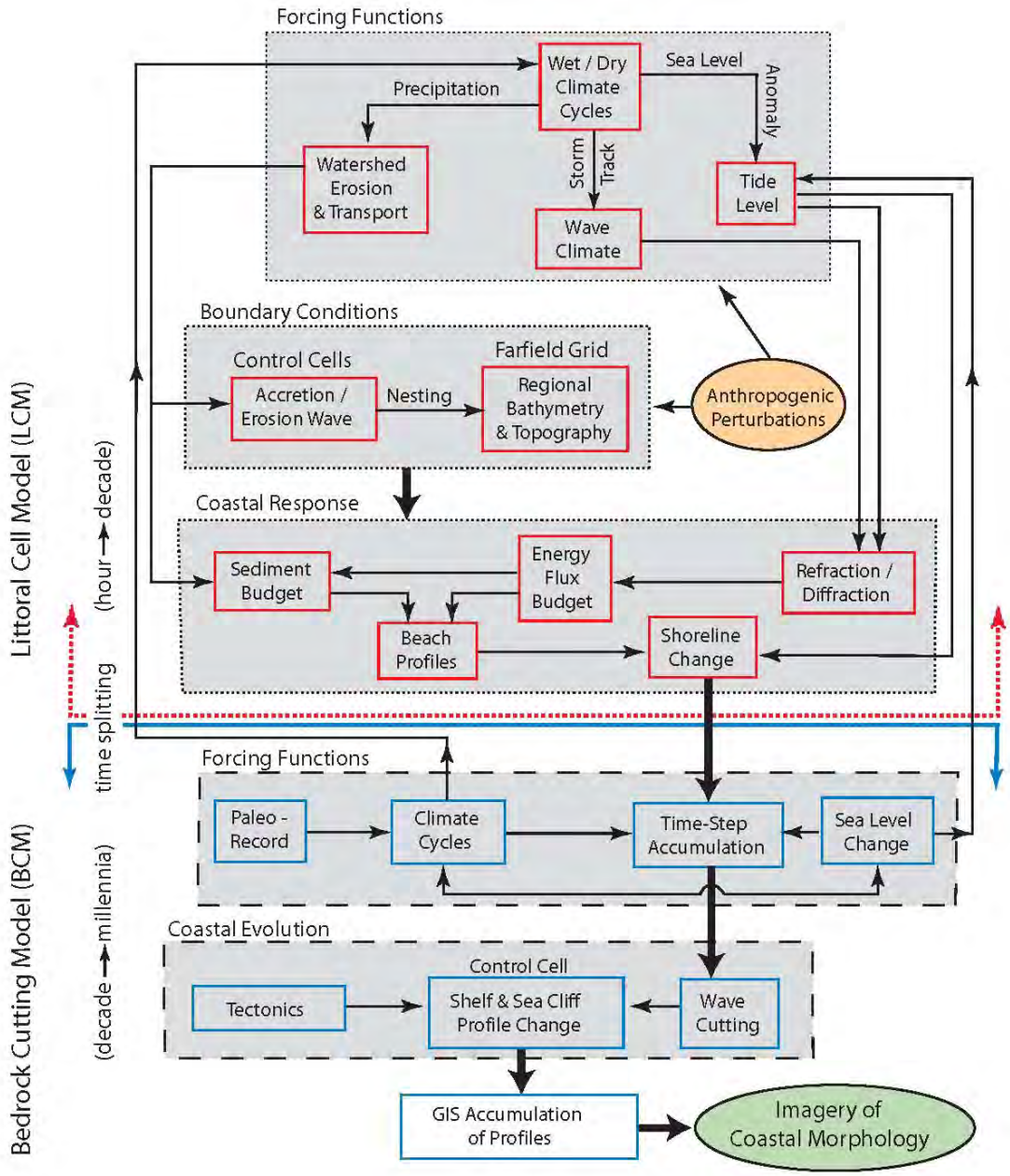

Figure 3.2: Architecture of the Coastal Evolution Model consisting of the Littoral Cell Model (above) and the Bedrock Cutting Model (below). Modules (shaded) are formed of coupled primitive process models. (Jenkins and Wasyl, 2005).

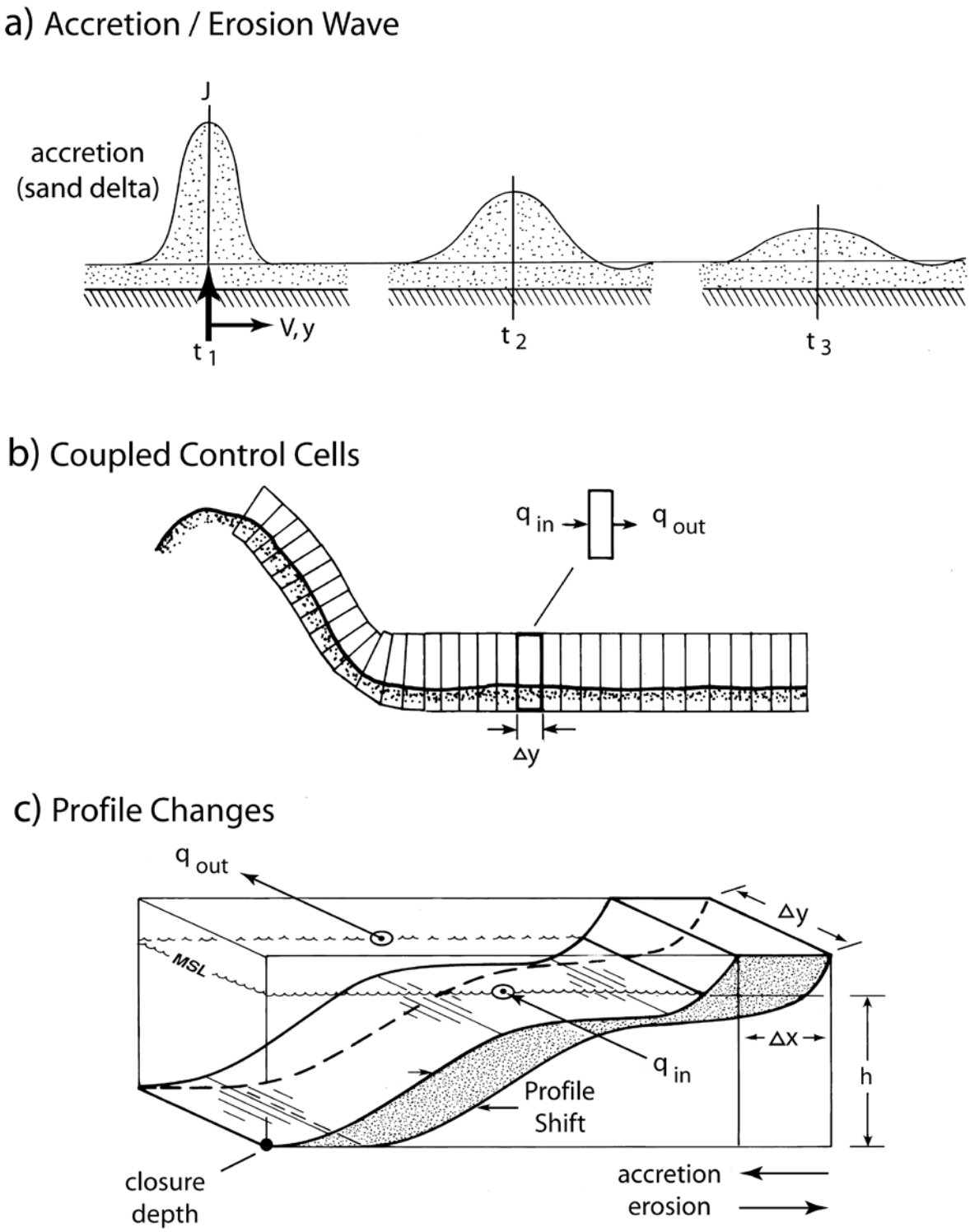

Figure 3.3: Computational control cell approach for modeling shoreline change after Jenkins, et. al., (2007).

In the LCM, the variation of the sediment cover with time is modeled by 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 3.3. The sediment continuity equation is written (Jenkins, et al, 2007):

$$\frac{\partial q}{\partial t} = \frac{\partial}{\partial y}\left(\varepsilon \frac{\partial q}{\partial y}\right) - V_l \frac{\partial q}{\partial y} + J(t) - R(t) \qquad (3)$$

Where 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 into the littoral cell from watersheds or beach disposal of dredge material, and R(t) is the flux of sediment lost to sinks , in this case, the Mugu Submarine Canyon. The first term in (3) is the surf diffusion term while the second is the advective term due to the longshore current. For any given control cell inside the reach from Point Conception and Point Mugu, (3) may be discretized in terms of the rate of change of "beach volume", Λ, in time increment Δt , given by:

$$\frac{d\Lambda}{dt} = J(t) + \frac{q_{in} + q_{out}}{\Delta t} \qquad (4)$$

Sediment is supplied to the control cell 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 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:

$$\frac{dV}{dt} \cong \frac{d\Lambda}{dt} = \frac{dX}{dt} \cdot Z \cdot l \qquad (5)$$

where $$Z = Z_1 + h_c \qquad (6)$$

Here, Z is the height of the shoreline flux surface equal to the sum of the closure depth below mean sea level, h_c, 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 (4) sum to zero; otherwise the shoreline will move during any time step increment as:

$$\Delta x(t) = \frac{1}{\Delta y(Z_1 + h_c)} \int \left(\frac{\partial}{\partial y}\left(\varepsilon \frac{\partial q}{\partial y} \right) - V \frac{\partial q}{\partial y} + J(t) \right) dt \qquad (7)$$

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

$$J = \xi\, Q^{\omega} \qquad (8)$$

where ξ, ω are empirically derived power law coefficients of the sediment rating curve from best fit (regression) analysis (Inman and Jenkins,1999). When river 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 3a). 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 = qin - qout , see Figure 3b and 3c. 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 (7).

It is well known that beach and nearshore bottom profiles change seasonally in response to seasonal wave climate variations as shown in Figure 3.4, (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). 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 long-term wave records applied to the well-tested elliptic cycloid solutions published in Jenkins and Inman (2006).

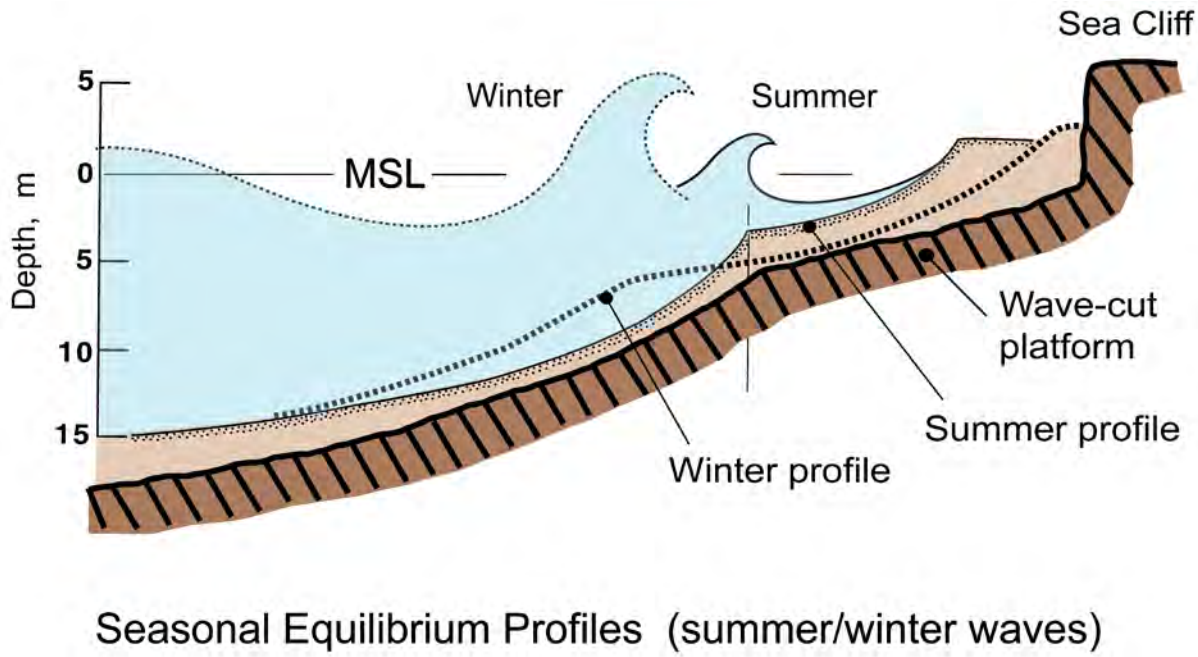

Figure 3.4: Schematic of summer and winter equilibrium beach profiles, from Inman, et al (1993).

When a long term collection of summer and winter beach equilibrium profiles for a broad range of wave heights, a well-defined envelope of variability becomes apparent as illustrated in Figure 3.5 and 3.6a. Figure 3.5 combines 12 measured bottom profiles over a 37 year period from two adjacent beaches near Oceanside Harbor, CA. These beaches have geomorphic similitude with the beaches near Santa Barbara Harbor, and are shown here to illustrate a fundamental principle. In Figure 3.5, elliptic cycloid solutions for equilibrium profiles are also overlaid as colored traces to further define this envelop of variability. The cycloid solutions are from Jenkins and Inman, (2006), and are based on average summer and winter wave heights and periods. Comparison of the measured profiles in grey with the cycloid solutions indicates that the volume of sand associated with long term beach profile variations are directly calculable by integration of the cycloid solutions between the limits of wave climate. This integration is shown in Figure 3.6b, and the volume of sand is referred to as the ***critical mass***. The critical mass represents the minimum volume of sediment cover required to maintain equilibrium bottom profiles and a stable seabed over the long-term, (where long-term is on the order of decades). Figure 3.6b indicates that the critical mass increases with wave height, and decreases with sediment grain size. Thus, the critical mass requirements become very large for finer-grained sediments in high energy wave climate environments. Furthermore, the total mass of sand in the littoral cell, (as specified by the sediment budget in Equation 4), must exceed the critical mass in order for the beach and nearshore sediment cover to remain sustainable over time. If the sediment budget declines to less than the critical mass, then the beach and nearshore will denude down to bedrock, and all the sediment cover is quickly lost. This occurred in many places in Southern California during the El Nino winter of 1983 (Inman and Jenkins, 1993, 2004), and would be disastrous for a SIG or BIG intake system if it happened at the Santa

Barbara site in the future. Only the Neodren™ technology would be able to survive a repeat of the 1983 El Nino winter conditions due to its ability to be placed below the critical mass envelope by means of horizontal directional drilling (HDD).

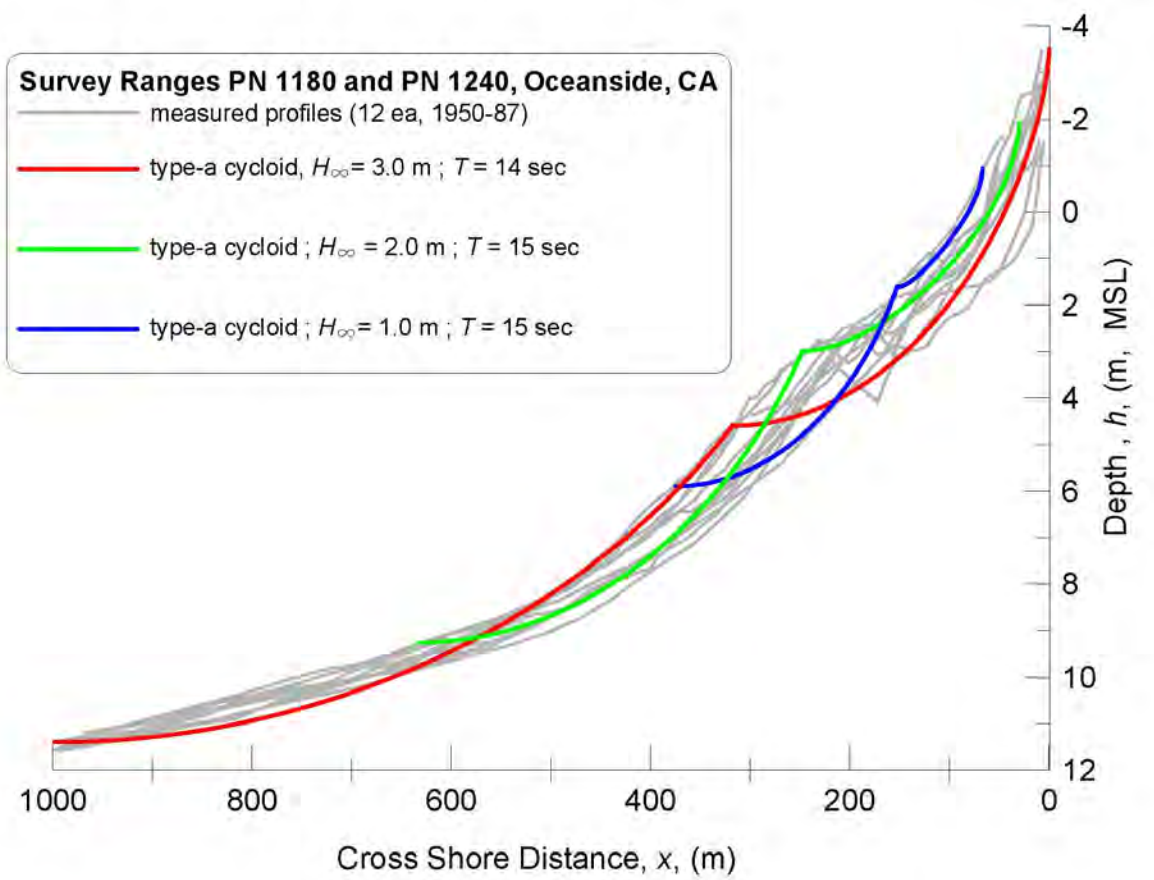

Figure 3.5: Envelope of variability of measured beach profiles (1950- 1987) at Oceanside CA (shown in grey), compared to an ensemble of elliptic cycloid solutions (colored) for selected wave heights and periods for average summer and winter wave climate; (from Jenkins and Inman, 2006)

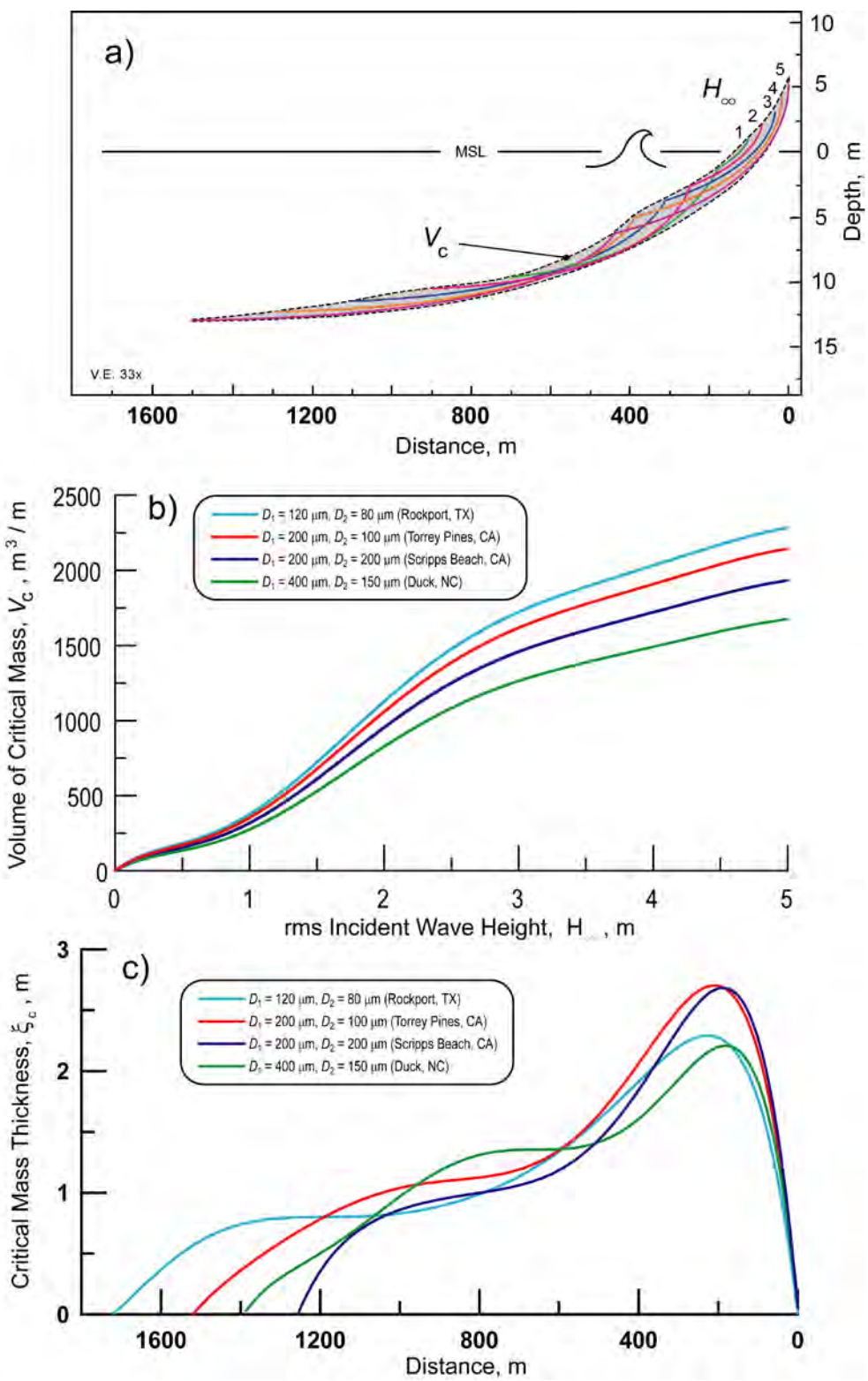

Figure 3.6: Features of the critical mass of sand: a) critical mass envelope for waves of 1m to 5m in height; b) volume of critical mass as a function of wave height and sediment grain size; c) variation in the thickness of the critical mass as a function of distance offshore; (from Jenkins, et. al., 2007)

3.2) Closure Depth:

This is the most important parameter in the optimal siting of shallow sub-seabed intake technology. Closure depth 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 also represents the outer limit of the critical mass. If a SIG were located inshore of closure depth, the engineered fill would suffer seasonal or episodic erosion, and subsequently be replaced by seasonal or episodic deposition of native sediments whose grain size may or may not be compatible with the fill material.

Hallermeier [1978, 1981] derived a relation for closure depth, by assuming a relationship for the energetics of sediment suspensions based on a critical value of the Froude number, giving:

$$h_c \cong 2.28 H_{ss} - 6.85 \left(H_{ss}^2 / gT^2 \right) \qquad (9)$$

where H_{ss} is the nearshore storm wave height that is exceeded only 12 hours each year and T is the associated wave period.

Birkemeier [1985] suggested different values of the constants and found that the simple relation $h_c = 1.57\ H_{ss}$ provided a reasonable fit to his profile measurements at Duck, North Carolina. *Cowell et al*. [1999] reviews the Hallermeier relation for closure depth h_c and limiting transport depth h_i and extends the previous data worldwide to include Australia. Their calculations indicate that h_c ranges from 5 m (Point Mugu California) to 12 m (SE Australia), while h_i ranges from 13 m (Netherlands) to 53 m (La Jolla, California). They conclude that discrepancies in data and calculation procedures make it "pointless to quibble over accuracy of prediction" in h_c and h_i. In the context of planning for beach nourishment, *Dean* [2002] observes that "although closure depth…..is more of a concept than a reality, it does provide an essential basis for calculating equilibrated…beach widths."

While it may be reasonable to apply the Hallermeier relation or its simpler form after *Birkemeier* [1985] to the shorerise boundary condition, comparisons with the *Inman et al.* [1993] beach profile data set show that these relations tend to underestimate closure depth. We propose an alternative closure depth relation. This relation is based on two premises: 1) closure depth is the seaward limit of non-zero net transport in the cross-shore direction; and, 2) closure depth is a vortex ripple regime in which no net granular exchange occurs from ripple to ripple. *Inman* [1957] gives observations of stationary vortex ripples in the field and *Dingler and Inman* [1976] establish a parametric relationship between dimensions of stationary vortex ripples and the Shield's parameter $\tilde{\Theta}$ in the range $3 < \tilde{\Theta} < 40$. Using the inverse of that parametric relation to solve for the depth gives (Jenkins and Inman, 2006):

$$h_{\mathbf{c}} = \frac{K_{\mathbf{e}} H_{\infty}}{\sinh k h_{\mathbf{c}}} \left(\frac{D_o}{D_{\mathbf{2}}} \right)^{\psi} \qquad (10)$$

where K_e and ψ are non-dimensional empirical parameters, D_2 is the shorerise median grain size; and D_o is a reference grain size. With $K_e \sim 2.0, \psi \sim 0.33$ and $D_o \sim 100\mu\text{m}$, the empirical closure depths reported in Inman et al. [1993] are reproduced by (10). From (10) 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 (10), closure depth also increases with increasing wave period. Using (10), the distance to closure depth X_{c2} can be obtained from (Jenkins and Inman, 2006),

$$X_{c2} = \frac{h_c I_e^{(2)}}{\varepsilon} \cong \frac{\pi\, h_c}{2\varepsilon}\sqrt{\frac{2-e^2}{2}} \qquad (11)$$

Where X_{c2} is measured from the origin of the shorerise located a distance X_2 from the berm and a distance $X_3 - X_2$ inside the breakpoint (Figure 3.8a), $I_e^{(2)}$ is the elliptic integral of the second kind, and ε is a stretching factor proportional to the Airy wave mild slope factor *N*, and

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

3.3) Elliptic Cycloid Solutions for the Shore-rise and Beach Profiles:

The elliptic cycloid was proven to be the mathematical representation of a shore-rise or bar-berm beach profile by Jenkins and Inman, 2006. This mathematical relation is embedded in the algorithms of the CEM and used to calculate the bottom profile of the beach and seabed offshore of Santa Barbara for any given point in time based on the incident wave height, period, direction, and sediment grain size. The elliptic cycloid solutions were developed for beach profiles by Jenkins 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. 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.

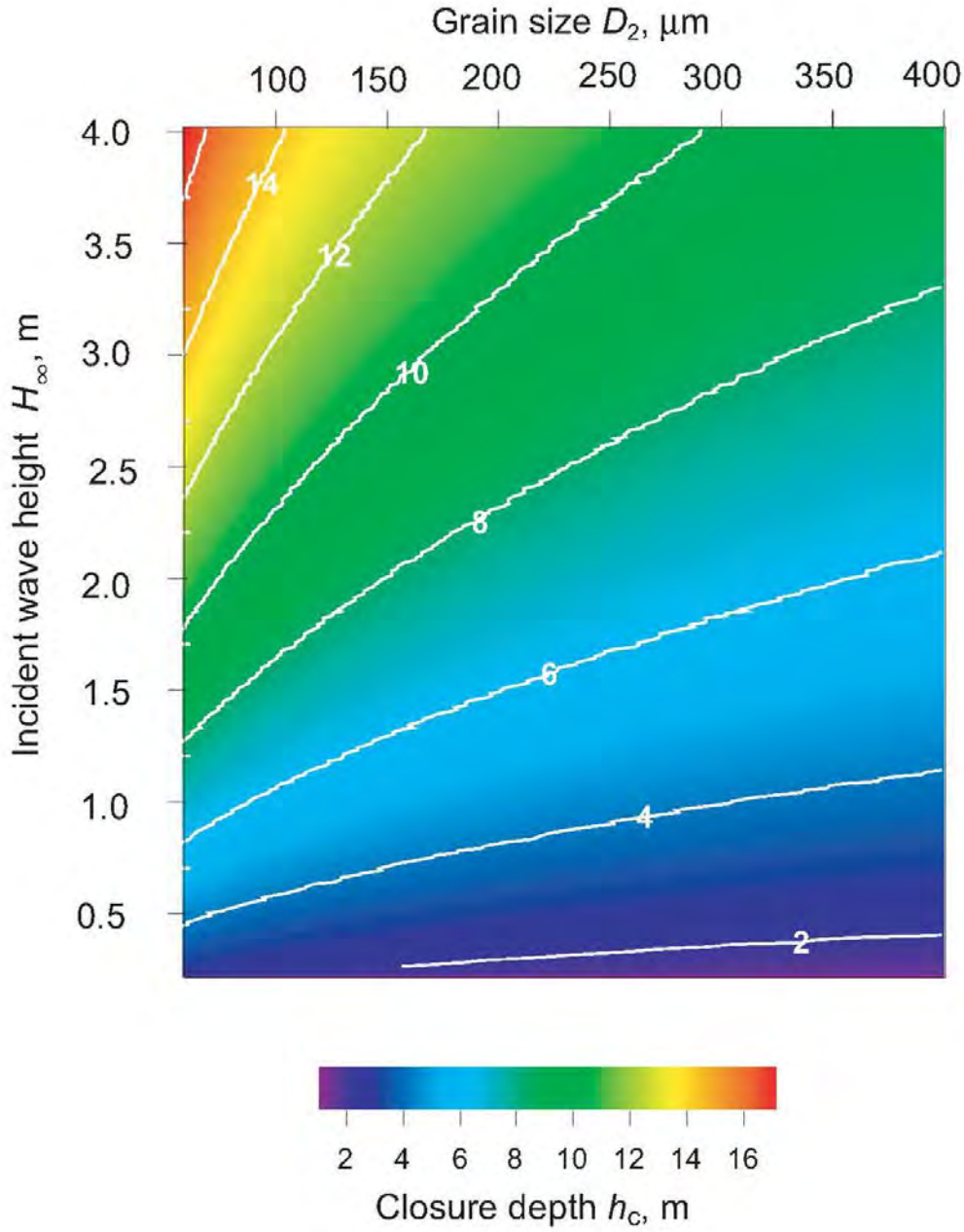

Figure 3.7: Closure depth contoured versus incident wave height and sediment grain size for waves of 15 second period, with $K_e \sim 2.0, \psi \sim 0.33$ and $D_o \sim 100\mu\text{m}$. D_2 is the shorerise median grain size; and D_o is a reference grain size; (from Jenkins and Inman, 2006) .

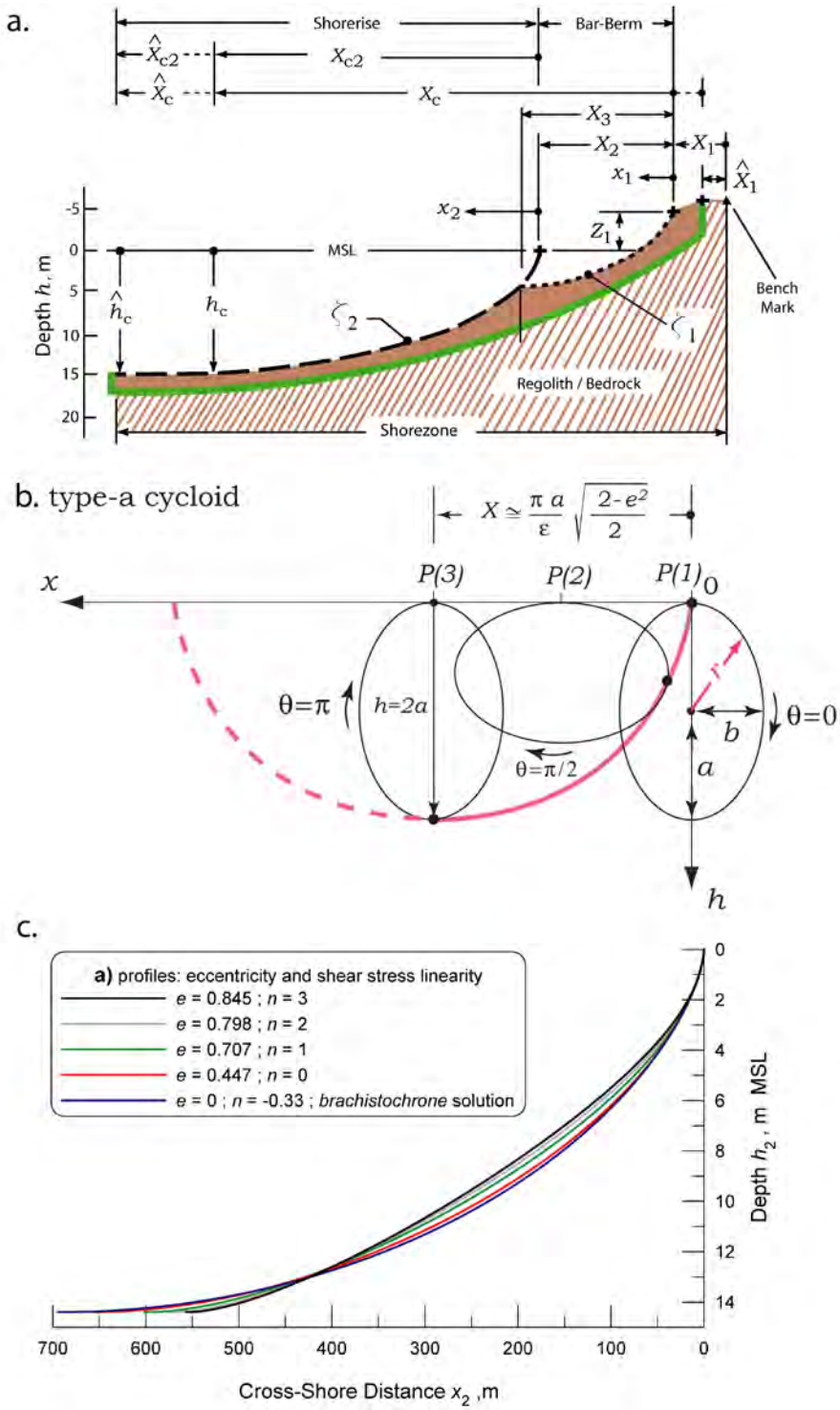

Figure 3.8. Equilibrium beach profile a) nomenclature, b) elliptic cycloid, c) Type-a cycloid solution; (from Jenkins and Inman, 2006).

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 3.8. 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:

$$x = x_2 = \frac{2r\, I_e^{(k_{1,2})}}{\pi\, \varepsilon}\,(\theta - \sin\theta) \qquad (12)$$

where *r* is the radius vector measured from the center of an ellipse whose semi-major and semi-minor axes are *a*, *b* and $I_e^{(k)}$ 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:

$$r = r_a = \left[\frac{a^2 b^2}{a^2 \sin^2\theta + b^2 \cos^2\theta}\right]^{1/2} = \frac{a\sqrt{1-e^2}}{\sqrt{\sin^2\theta + (1-e^2)\cos^2\theta}} \qquad (13)$$

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 3.8b is based on the elliptic integral of the second kind that has an analytic approximation, $I_e^{(2)} = (\pi/2)\sqrt{(2-e^2)/2}$, see *Hodgman* [1947]. The inverse of (13) for the type-a elliptic cycloid gives the companion solution in terms of local water depth, *h*, as:

$$h = h_2 = \frac{\pi\varepsilon\; x_2}{2I_e^{(k_{1,2})}}\left(\frac{1-\cos\theta}{\theta - \sin\theta}\right) = r\left(1-\cos\theta\right) \qquad (14)$$

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,

$$X = \frac{2aI_{\mathrm{e}}^{(2)}}{\varepsilon} \cong \frac{\pi\ a}{\varepsilon}\sqrt{\frac{2-e^2}{2}} \qquad \text{at} \quad \theta = \pi \quad \text{(type-a cycloid)} \qquad (15)$$

3.4) Critical Mass:

The critical mass determines the volume of sediment that can be potentially eroded, and the depth below existing grade that erosion might extend, due to extreme storms and seasonal change or shoreline recession. The critical mass of sand on a beach is that required to maintain equilibrium beach shapes over a specified time, usually ranging from seasons to decades. The critical mass for a seasonal beach is determined from the volume of the envelope of sand necessary to maintain continuous beach forms during the many changes in shape from one equilibrium state to another over a period of seasons (Jenkins and Inman, 2003). Generally, changes in profile shape between equilibrium states involve transitional shapes that are non-equilibrium in form. However, as a first order approximation, we assume the critical mass envelope consists of a set of incremented equilibrium profiles, and the associated set of transitional profiles occurring between successive equilibrium states. Each profile in this set corresponds to a particular rms breaker height H_{b} that varies between some seasonal minimum H_{bo} and the critical wave height $\hat{H}_{\mathrm{b}}$, the highest wave condition for which the existing sand supply can accommodate equilibrium and transitional profile adjustments. The equilibrium profiles are incremented by infinitesimal changes in wave height, $H_{\mathrm{bo}} \leq H_{\mathrm{b}} + \mathrm{d}H_{\mathrm{b}} \leq \hat{H}_{\mathrm{b}}$, giving a continuous envelope of beach profile change. The volume of this envelope can be calculated from the thermodynamic solutions for the bar-berm profile, ζ_1, and the shorerise profile ζ_2 to solve for the volume of critical mass V_{c} per meter of shoreline (m^3/m):

$$V_c = \int_{H_\infty}^{\hat{H}_b}\int_{X_1}^{X_3}\frac{\partial\zeta_1}{\partial H_b}dxdH_b + \int_{H_\infty}^{\hat{H}_b}\int_{X_3}^{X_c}\frac{\partial\zeta_2}{\partial H_b}dxdH_b \qquad (16)$$

Analytic solutions to V_{c} are difficult because the thermodynamic solutions for the curvilinear coordinates (ζ_1, ζ_2) using elliptic cycloids are transcendental. Therefore solutions for the V_{c} envelope are obtained by numerical integration of (16) based on long term wave climate (cf. Section 5). We use the number crunching capabilities of the CEM for this purpose.
Figure 3.9 gives the critical mass solution resulting from numerical integrations of (16). Because equilibrium and transitional profiles are grain size dependent through the closure depth condition, the volume of critical mass has a certain degree of sensitivity to grain size. Sensitivity analyses of (16) based on numerical integration show that finer grain sizes, particularly in the shorerise, tend to result in larger volumes of critical mass. This is shown in in Figure 3.10 with the wave period fixed. Longer curvilinear length ζ1, ζ2 and deeper closure depths hc arise from finer grained sediment, thus resulting in physically larger critical mass envelops. However, the sensitivity of the volume of critical mass to grain size is second order relative to the dependence on wave height and period. A polynomial fit to the wave height dependence averaged over all grain sizes gives the following analytic approximation:

$$V_c \cong 500 H_b^{0.9} \qquad (17)$$

where H_b is in meters, giving V_c in m^3 per meter of beach length.

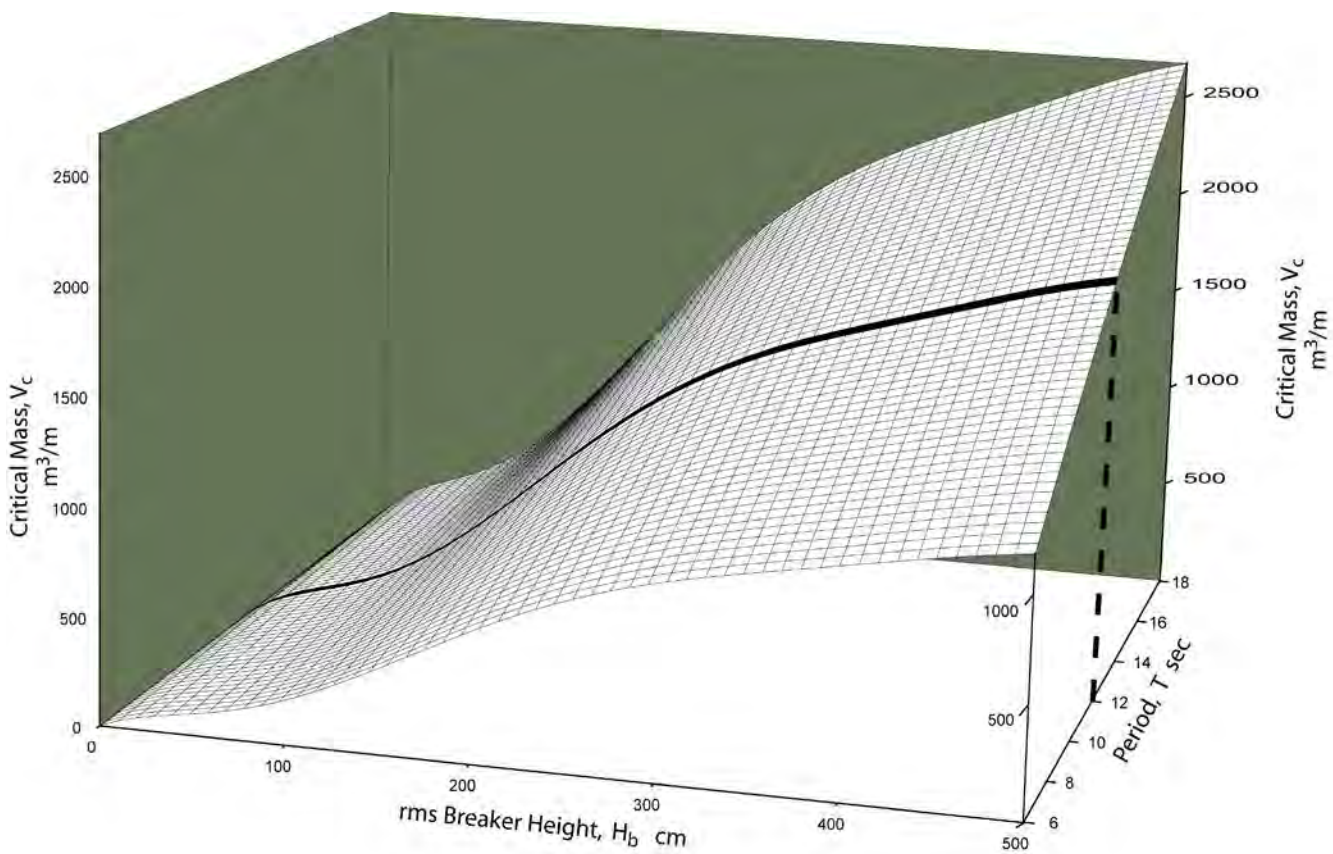

Figure 3.9: Three dimensional rendering of the total solution space of the critical mass. Black line corresponds to the solution in Figure 3.10 for D_1 = 225 microns and D_2 = 125 microns.

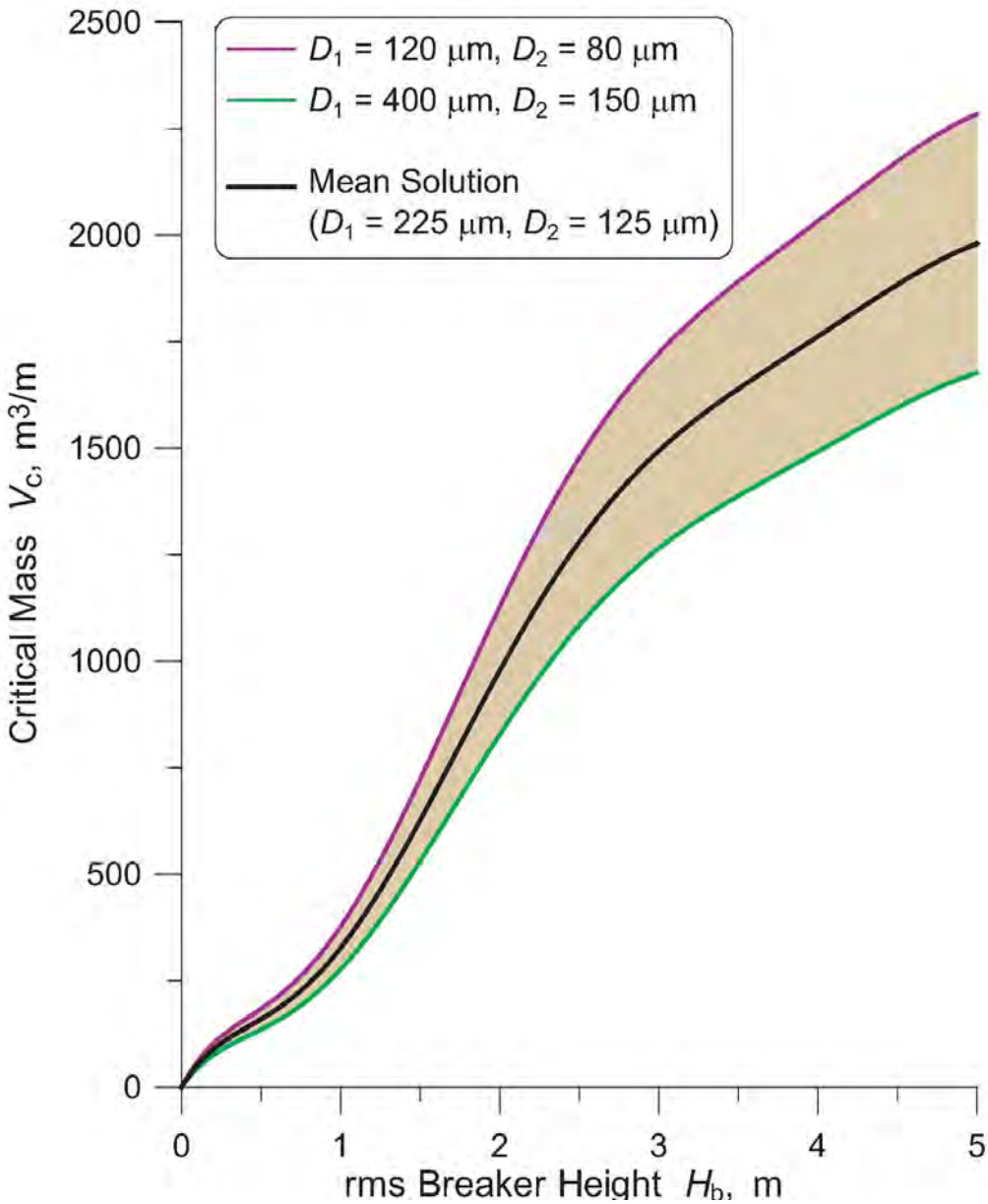

Figure 3.10: Critical mass solution as a function of rms breaker height for 12 sec waves breaking on variable sediment grain size in the bar-berm D_1 and shore-rise D_2 portions of the seabed profile. Curves generated from numerical integration of elliptic cycloid solutions.

3.5 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 run-up 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 run-up exceeds the profile crest elevation, which can result in flooding landward of the crest. Run-up 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 run-up and the SWL, referenced to an established vertical datum. The results in this study for run-up exposure of landside facilities are referenced to NGVD 1929 vertical datum. 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}.

Wave setup and run-up are typically computed at hourly time steps from an historic record of wave monitoring, (see Section 3.0). Wave setup and run-up are combined with coincident 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:

$$\overline{\eta} = 4.0 F_H F_T F_{Gamma} F_{Slope} = 4.0 \left(\frac{H'_0}{26.2}\right)^{0.8} \left(\frac{T_P}{20.0}\right)^{0.4} \left(\frac{m_{DIM}}{0.01}\right)^{0.2} \tag{18}$$

Dynamic wave setup is calculated as:

$$\eta_{rms} = 4.0\, G_H G_T G_{Gamma} G_{Slope} = 4.0\left(\frac{H'_0}{26.2}\right)^{0.8}\left(\frac{T_P}{20.0}\right)^{0.4}(Gamma)^{0.16}\left(\frac{m_{DIM}}{0.01}\right)^{0.2} \qquad (19)$$

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:

$$Gamma = 2047\nu^4 - 3083\nu^3 + 1782\nu^2 - 4769.9\nu + 507.1 \qquad (20)$$

Values of are computed directly from the spectral moments (m_0, m_1 and m_2) based on the Longuet-Higgins (1973) definition of the spectral narrowness:

$$\nu = \left[\frac{m_0 m_2}{m_1} - 1\right]^{1/2} \qquad (21)$$

Gamma values are limited from 1 to 38, based on the range of wave data used (Section 4.2) to relate the spectral narrowness, ν, to the peak parameter, *Gamma*, as shown in Figure 3.11.

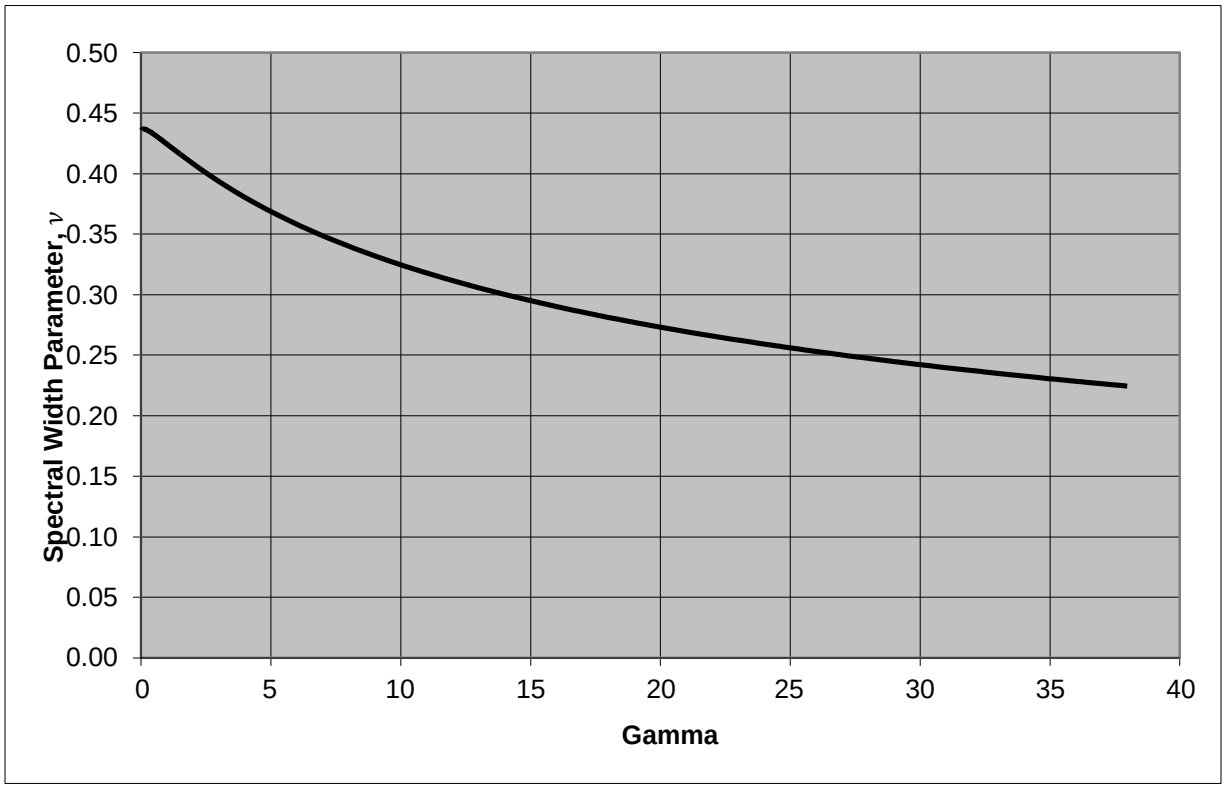

Figure 3.11: Spectral Width Parameter versus Spectral Peak Parameter

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 18 and 19. The nearshore slope, m_{DIM}, is taken from the elliptic cycloids (Section 3.3) and is calculated from 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 deepwater significant wave height would be subject to depth-limited breaking (van der Meer, 2002). The landward limit of wave run-up is calculated iteratively, with the initial approximation being the SWL.

3.6 Wave Run-up:

Wave run-up 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 run-up for transects with natural, gently sloping (m_{DIM} < 0.125) profiles. For shorelines with shore protection structures and steeply sloping (m_{TAW} ≥ 0.125) natural shorelines where the dynamic water level exceeds the toe of the structure, the TAW method was used to calculate run-up. If the dynamic water level does not reach the toe of the structure or bluff face, the DIM is used. The total run-up, including wave setup and incident wave run-up, is added to the *still water level* (SWL) to determine the *total water level*, (TWL), see Figure 3.12). Each of these methods is described in detail in the following subsections

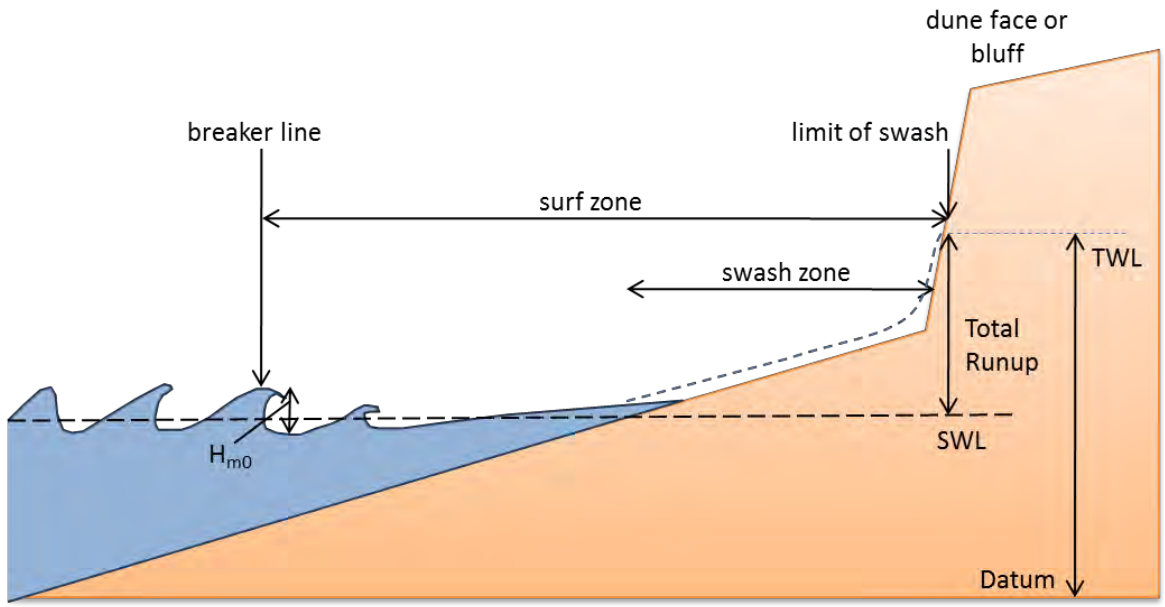

Figure 3.12: Conceptual Model Showing the Components of Wave Run-up Associated with Incident Waves

3.7 DIM Run-up Calculations:

Run-up on gently sloping, natural shorelines, and beaches seaward of a structure or bluff toe, is calculated using the DIM. The run-up calculation is based on the standard deviations of the oscillating wave setup and the incident wave run-up 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 19. The standard deviation of the incident wave oscillations (wave run-up), σ_2, on natural beaches is:

$$\sigma_2 = 0.3\xi_0 H'_0 \qquad (22)$$

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 run-up, $\hat{\eta}_T$, is determined from the combination of the two standard deviations of the fluctuating components:

$$\hat{\eta}_T = 2.0\sqrt{\eta_{rms}^2 + \sigma^2} \qquad (23)$$

Combining the results from Equations 18 & 23 yields the total wave run-up, which when superimposed with the SWL yields the total water level, TWL:

$$TWL = \overline{\eta} + \hat{\eta}_T + SWL \tag{24}$$

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

3.8 TAW Run-up Calculations:

Run-up 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 run-up 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 run-up 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 run-up 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 run-up. The dynamic setup, however, varies insignificantly across the surf zone and requires no adjustment.

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

$$\left(H_b\right)_{rms} = 0.714\left(\frac{\kappa}{g}\right)^{1/5}\left(\frac{H_0'^2 C_0}{2}\right)^{2/5} \tag{25}$$

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:

$$\overline{\eta}_0 = 0.189\left(H_b\right)_{rms} \tag{26}$$

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

$$\overline{\eta}(h) = 0.189\left(H_b\right)_{rms} - 0.186h \tag{27}$$

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:

$$\gamma_\eta = \frac{\overline{\eta}(h)}{\overline{\eta}_0} \tag{28}$$

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,

$$\overline{\eta}' = \gamma_\eta \overline{\eta} \tag{29}$$

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

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

(30)

The next step is to compute the wave height at the toe of the barrier and the resultant wave run-up 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 $H_{m0} = 0.78h_{toe}$. 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 run-up 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 run-up limit is initially unknown, the still water level plus $1.5H_{m0}$ is chosen

as a first estimate (Figure 3.13). If the run-up limit exceeded the selected crest, the run-up limit was set at the crest.

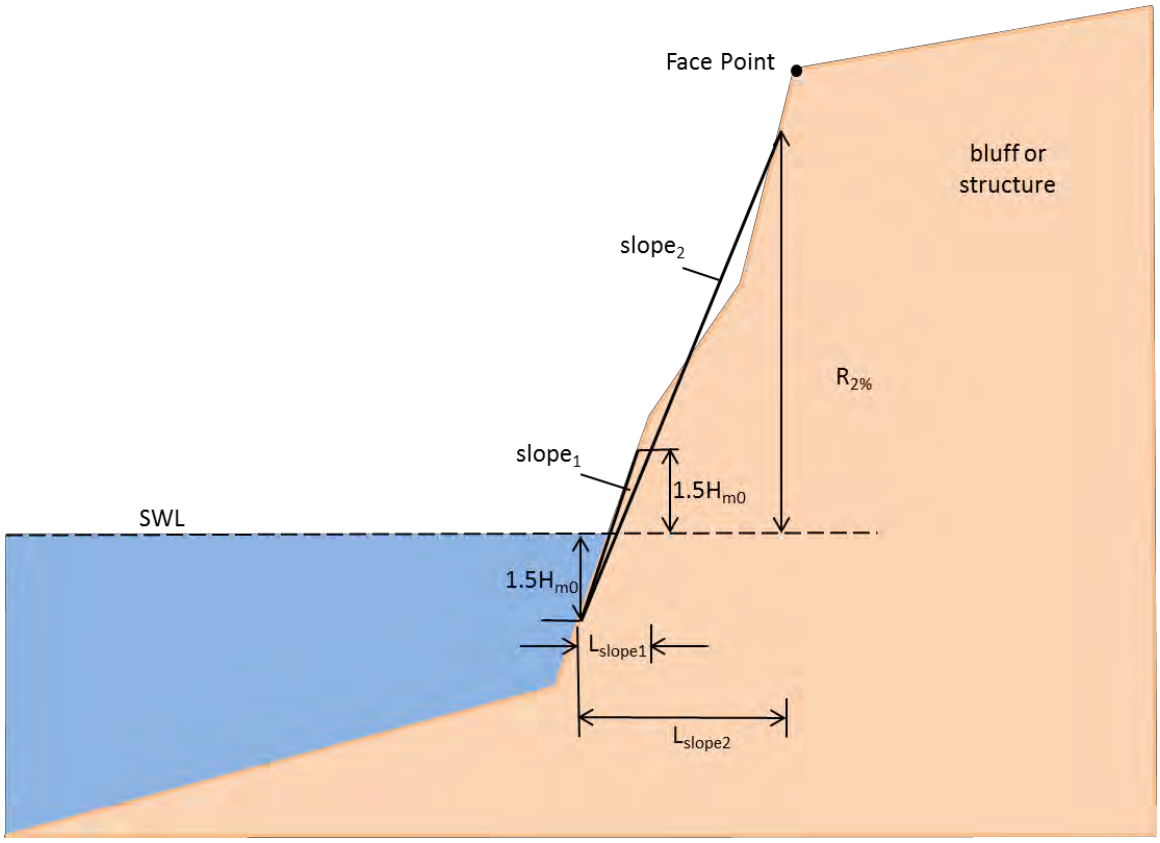

Figure 3.13: Determination of an Average Slope of Hard Back-Shore Formations (Bluff or Barriers) Based on an Iterative Approach, (Corrected from van der Meer, 2002)

The general formula of TAW for calculating the 2-percent wave run-up 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:

$$R_{2\%} = H_{m0}\, \gamma_r \gamma_b \gamma_\beta \left(4.3 - \frac{1.6}{\sqrt{\xi_{0m}}} \right) \qquad \text{if: } 1.8 \le \gamma_\beta\, \xi_{0m} \tag{31}$$

Where, $R_{2\%} = 2\sigma_2$ is the wave run-up 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.

The influence factor for oblique wave attack is calculated at each time step in the CDIP wave record (see Section 4.2). 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 run-up, $R_{2\%} = 2\sigma_2$ 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:

$$TWL = SWL + \overline{\eta}' + 2.0\sqrt{\eta_{rms}^2 + \left(\frac{R_{2\%}}{2}\right)^2} \tag{32}$$

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 run-up 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:

$$\gamma_v = 1.35 - 0.0078 \tan^{-1} m_{face} \tag{33}$$

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 run-up 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 run-up elevation exceeds a structure's profile crest elevation. When wave run-up 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 run-up and extreme value analyses is a statistical value and is not associated with either a specific wave height or DWL2%. Therefore, the maximum wave height at the structure toe and the maximum and average DWL2% associated with the 32 annual maximum TWLs were chosen for use with the 1-percent TWL to estimate the 1-percent overtopping hazard.

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

$$Q' = 0.82\sqrt{gH_s^3}\exp\left(-\frac{3.0R_c}{H_s\gamma_\beta\gamma_s}\right) \tag{34}$$

Where H_s, is the significant wave height at the structure, R_c is the freeboard, γ_β is the influence factor for oblique wave attack, and γ_s is the influence factor for porosity. To conservatively maximize the overtopping potential, H_s and R_c are selected as the maximum wave height at the structure and the minimum freeboard between the highest DWL2% and the barrier crest elevation based on the 32 annual maxima. This maximum overtopping potential is based on a single hourly time step from the 32-year wave record (Section 4.2) and likely exceeds the 1-percent overtopping hazard. For comparison, the maximum wave height at the structure is also paired with the freeboard between the average DWL2% from the 32 annual maxima and the crest elevation to estimate the overtopping rate expected over a full tidal cycle during a peak storm event. The influence factor for wave obliqueness is assumed to be 1.0.

3.9 Statistical Analysis:

The preferred approach for determining the X-percent-annual-chance wave-induced flood elevation involves utilizing a reasonably long observational (or continuous model) record to establish a probability distribution that can be used to evaluate the flood elevation for any frequency. A general rule of thumb is that a historical record at least one-third the length of the return period of interest is the minimum record needed to produce statistically reliable results. The extremal probability distribution can be used to establish any flood elevation frequency, but the levels of confidence in the values decrease with the length of record. In this case, a modeled continuous record of 32 years of offshore and nearshore wave conditions (see Section 4.2) are used to derive estimates of TWLs. This hindcast period is long enough that an extreme value distribution can be applied to it, in order to estimate the TWL elevation for a 1-percent-annual-chance condition. An annual maxima/Generalized Extreme Value (GEV) fit is used in the extreme value analysis to determine the 1- percent-annual-chance event for existing conditions and for two sea level rise example scenarios.

The cumulative distribution function of the GEV family of distributions is given by:

$$F(z) = \exp\left\{ -\left[1 + \xi \left(\frac{z-\mu}{\sigma} \right) \right]^{-1/\xi} \right\} \tag{35}$$

The GEV model has three parameters: μ is the mode of the extreme value distribution (referred to as location parameter); σ is the dispersion (also known as the scale parameter), and ξ , (not to be confused with the Iribarren number in wave run-up equations), is a shape parameter that determines the type of extreme value distribution. These parameters were determined using routines for GEV statistical analysis within the Wave Analysis for Fatigue and Oceanography, Version 2.1.1 (WAFO) toolbox for Matlab, which contains tools for fatigue analysis, sea state modeling, statistics, and numerics (WAFO-group, 2000). The three parameters, μ, σ, ξ and the fit of the resulting cumulative distribution function to the annual maxima are evaluated for the maximum likelihood solutions.

4) Model Initialization:

4.1) Bathymetry:

Bathymetry provides a controlling influence on all of the coastal processes that effect wave shoaling, sediment transport, erosion, accretion, seafloor stability and shoreline evolution. 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 Santa Barbara Littoral Cell, (Inman et al. 1993). The stationary bathymetry was derived from the National Ocean Survey (NOS) digital database compiled by NGDC (2103). 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.

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 and have been compiled by Noble, (1997) and Moffatt and Nichol, (2002). 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. In the very nearfield of the City's desalination plant intake and discharge, Tierra (2013) performed high resolution bathymetric survey on 5 m grid cell

resolution. These data were incorporated in the nearfield grid and co-registered with the NOS data along the deep water boundary (Figures 2.13 and 4.1).

To perform both the required wave shoaling and transport computations in the farfield of the project's possible subsurface intake systems, resolution of the bottom bathymetry must be sufficient to provide at least two grid points per wavelength of the highest frequency wave to be shoaled. The farfield grid computes the effects of island sheltering and regional scale refraction and circulation due to the shallow banks of the continental margin (Figure 2.13). A nearfield grid in the immediate neighborhood of the possible subsurface intake systems is nested inside the farfield grid and is plotted in Figure 4.1, where again the depth contours are shown as the black contour lines scaled in meters MSL. The nearfield grid is used to calculate the changes in the seabed bottom profiles and the rates of erosion and deposition around the study area's subsurface intake sites.

4.2) Wave Forcing:

The physical oceanography of the Santa Barbara Coast is generally well understood from numerous previous studies. The details of wave refraction into the Santa Barbara Channel and into the near field of the study area (Figures 2.13 & 4.1), were derived from measurements during the Coastal Data Information Program (CDIP). This program routinely monitored waves at several locations in the Southern California Bight since 1980. Wave records were reconstructed over a 30 year period of record (1980-2010) using CDIP monitoring stations at Santa Barbara, Harvest Platform, and Begg Rock. Data for these CDIP wave monitoring sites is available beginning January 1980 through March 2012 [CDIP, 2012].

The data in the preliminary file represent partially shoaled wave data specific to the local CDIP monitoring sites. To correct these data to the intake and run-up sites, the data are entered into a refraction/diffraction numerical code, back-refracted out into deep water, and subsequently brought onshore into the immediate neighborhood of the proposed intake sites. An example of a reconstruction of the back-refracted wave field throughout the Bight is shown in Figure 4.2 using the CDIP data from the Santa Barbara array. 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 the Channel Islands have induced longshore variations in wave height throughout the Southern California Bight. These variations (referred to as shadows and bright spots) induce longshore transport away from areas of high waves (bright spots, red) and toward areas of low waves (shadows, dark blue). Figure 4.3 shows the deep water significant wave heights, periods and directions resulting from the series of back-refraction calculations for the complete CDIP and SIO data set at $\Delta t = 6$ hour intervals over the 1980-2012 period of record. The data in Figure 4.3 are the values used as the deep water boundary conditions of the forward refraction computations into the Santa Barbara Sub Cell (Figures 4.4 and 4.5). The deep water wave angles in Figure 4.3 are plotted with respect to the direction (relative to true north) from which the waves are propagating at the deep water boundary of Figures 4.4 and 4.5. Inspection of Figure 4.3 reveals that a number of large swells lined up with the wave windows open to the Torrey

Pine Sub-Cell during the El Niño's of 1980-83, 1986-88, 1992-95, and 1997-98. The largest of these swell events was the 18 January 1988 storm, producing 6.5 m water swells off Santa Barbara, (see event #6 in Figure 4.3).

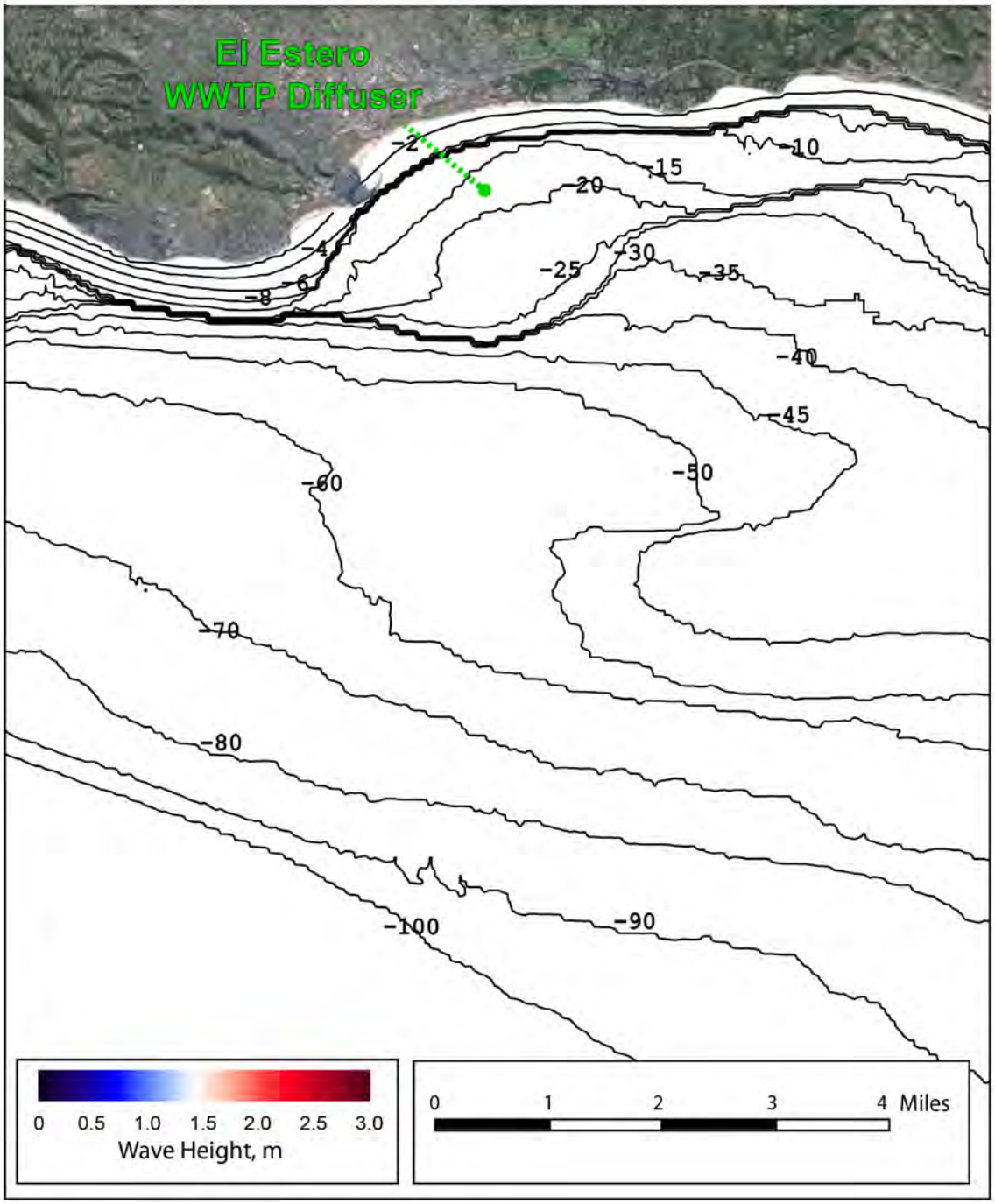

Figure 4.1: Nearfield bathymetry grid. Depth contours in meters MSL. Data from NGDC (2013)

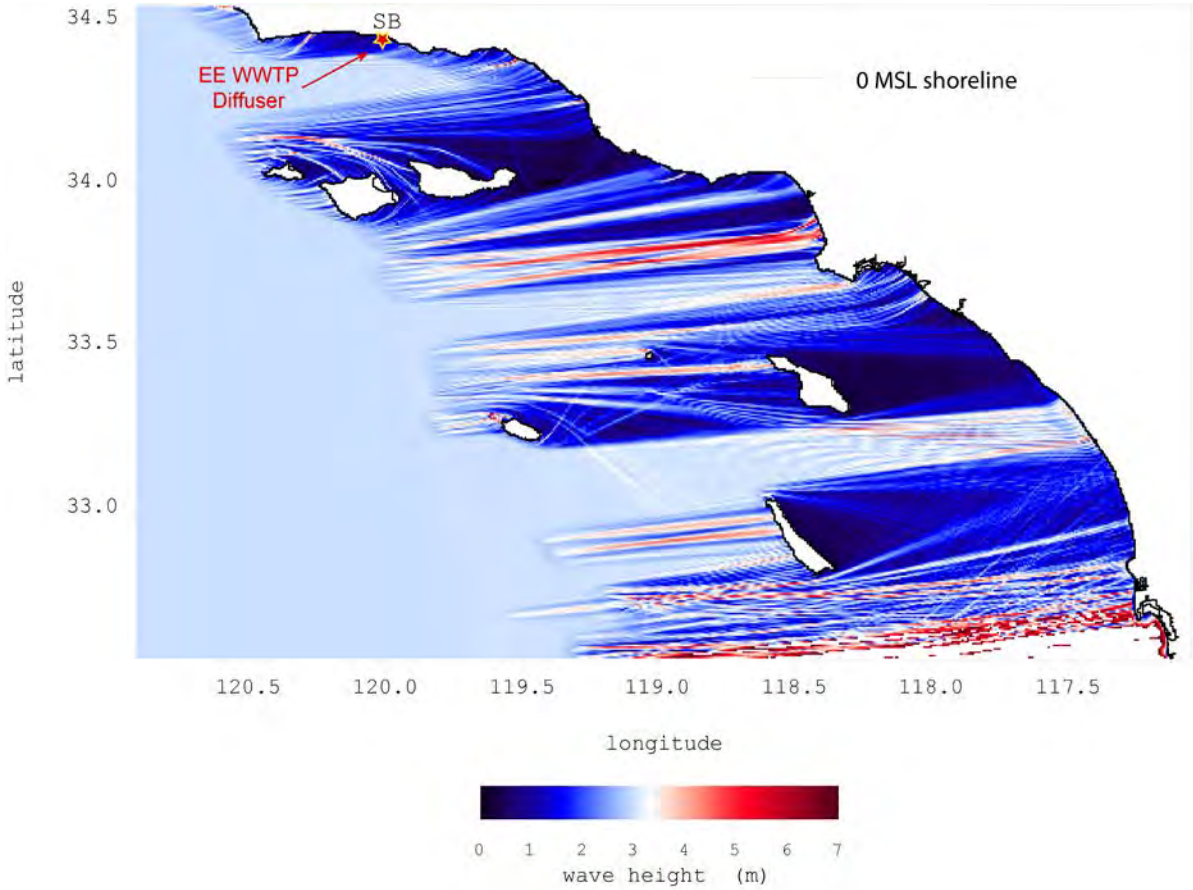

Figure 4.2: Farfield refraction/diffraction into the Southern California Bight and Santa Barbara Channel, (from Jenkins and Wasyl, 2008b).

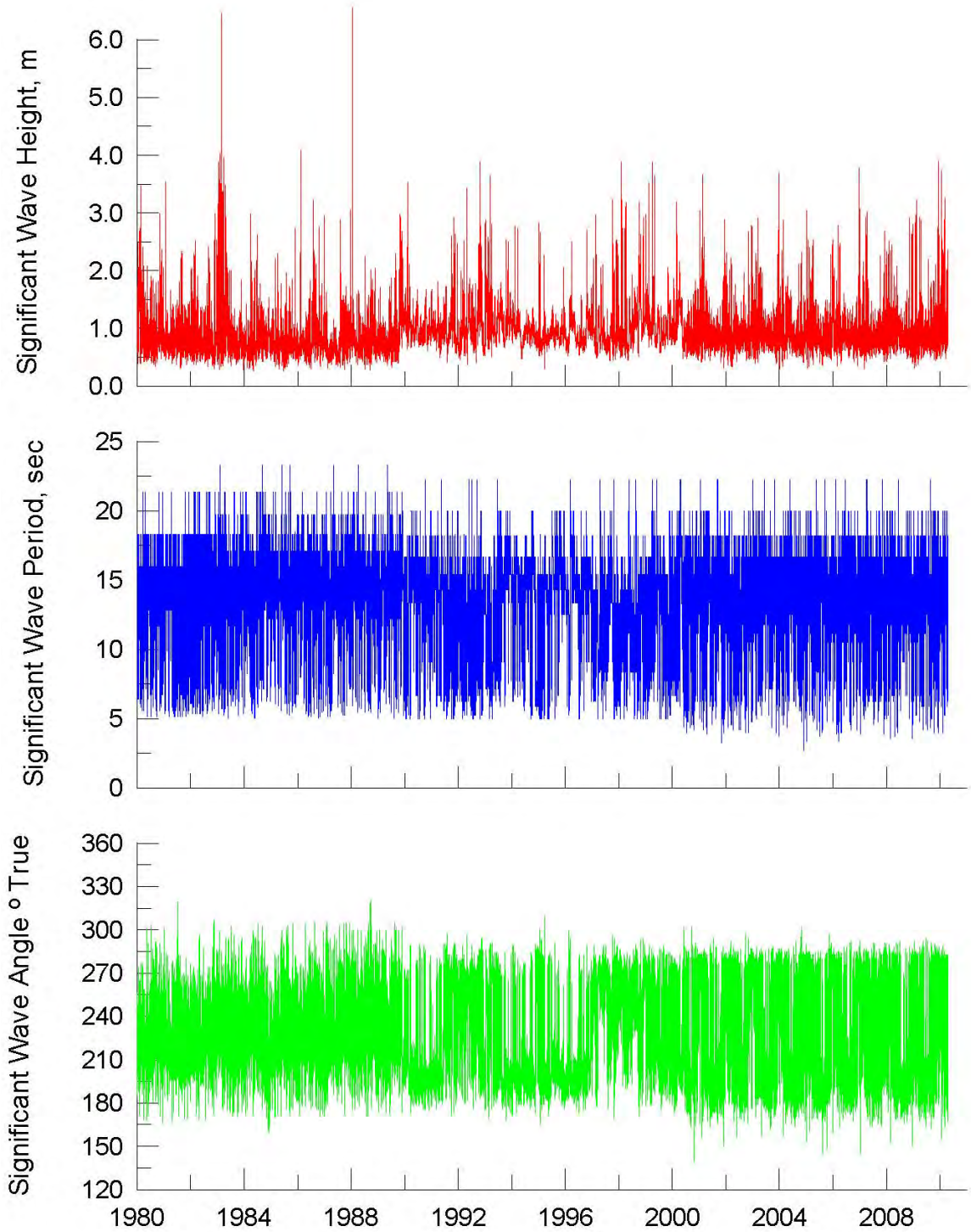

Figure 4.3: Deep water wave data for wave forcing at Santa Barbara derived from back refraction of CDIP monitoring data, 1980-2012. Record contains 11,842 daily observations.

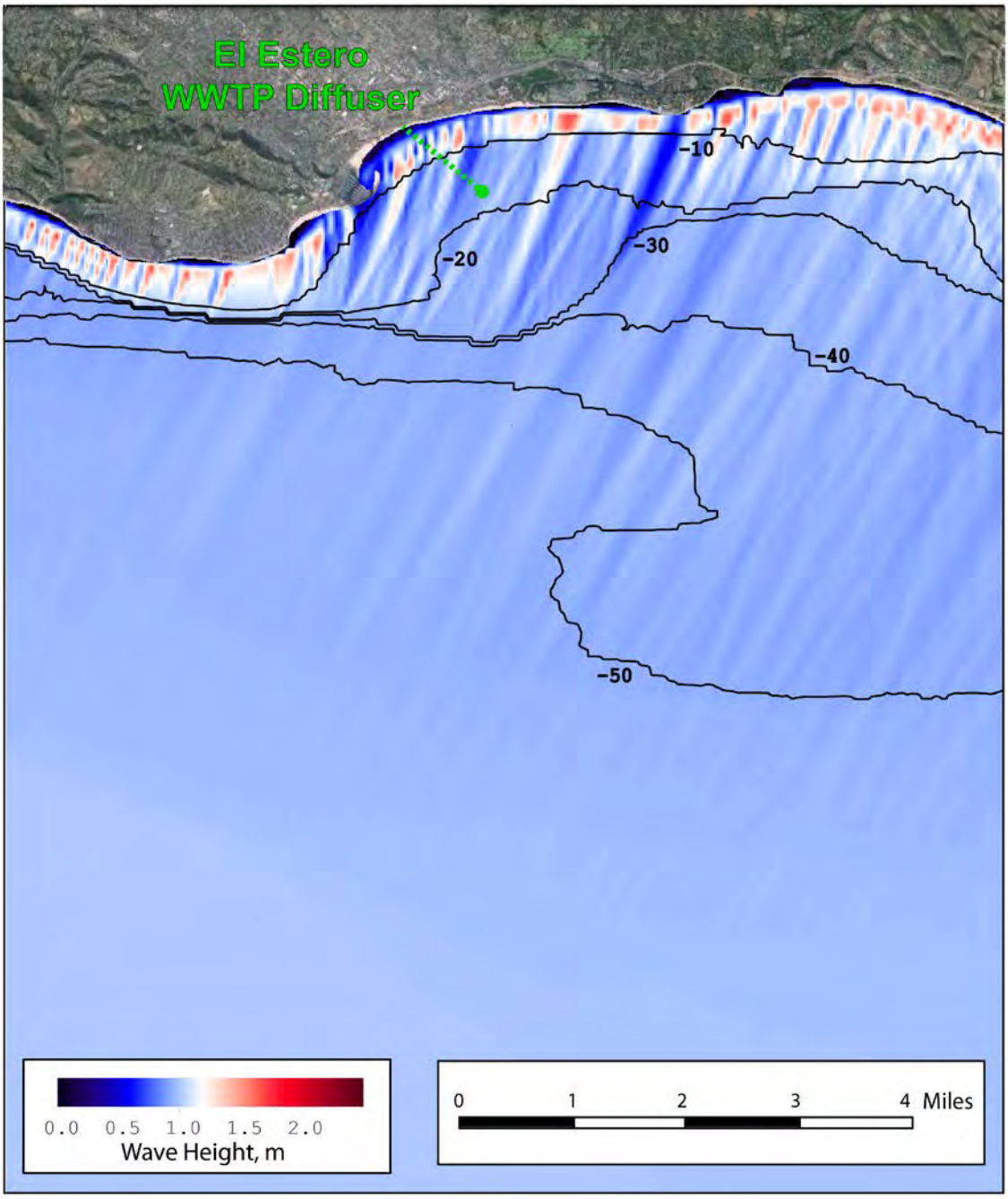

Figure 4.4: High resolution refraction/diffraction computation for average dry weather model scenario based on 1.0 m deep water wave height from 235^0 with 10 sec period.

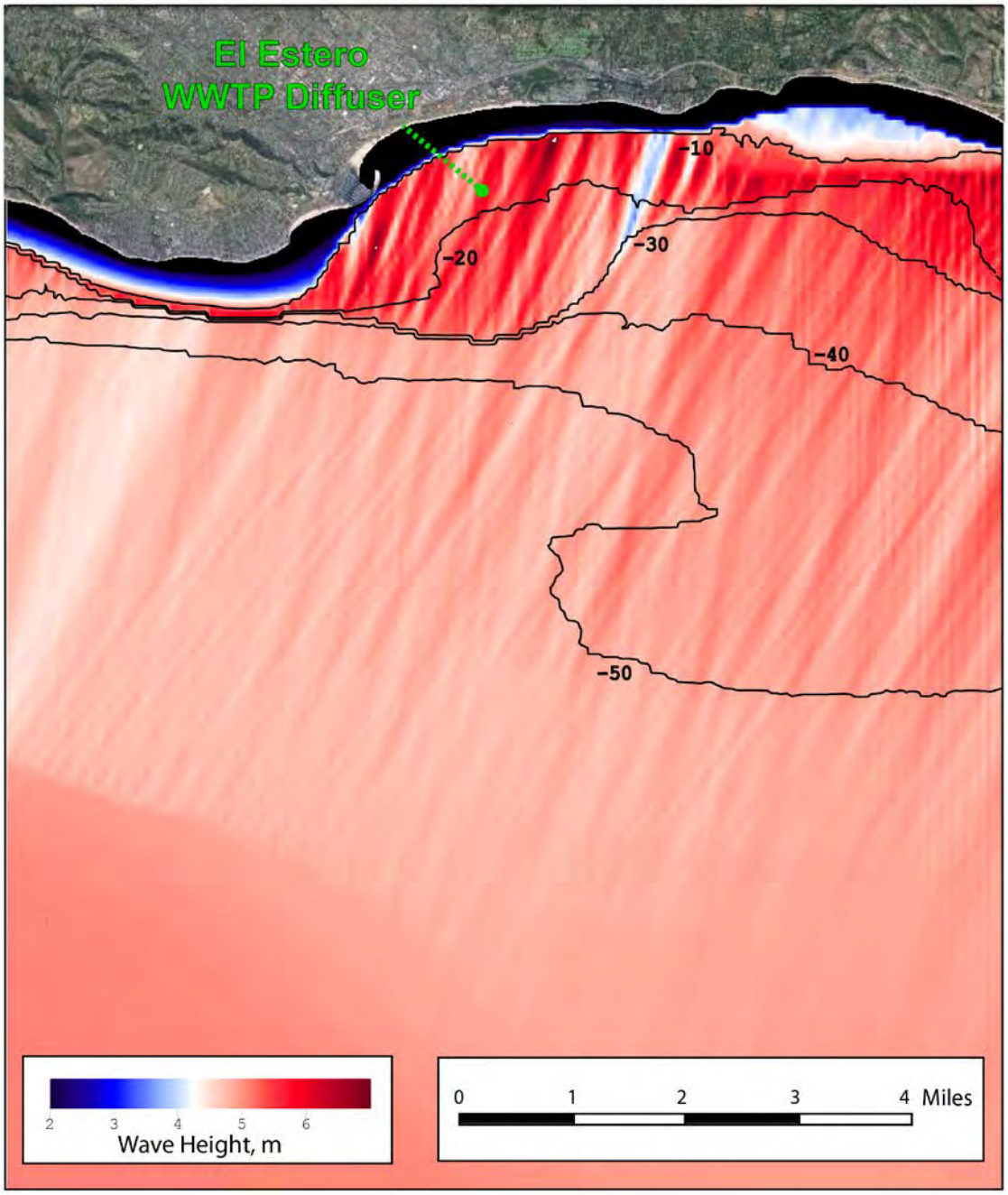

Figure 4.5: High resolution refraction/diffraction computation for a wet weather model scenario based on 5 m deep water wave height from 263^0 with 14 sec period, 21 November 2011.

Figure 4.5 gives an example of the forward refraction calculation over the Santa Barbara region for the low energy waves that characterize the low mixing conditions of a dry weather modeling scenario. These particular waves were observed on 22 August 2011, and had a daily mean wave height of only 1.0 m, approaching Santa Barbara from 235^0 with a 10 sec period. In contrast, the refraction/diffraction pattern for a wet weather scenario is shown in Figure 4.5 for 5.0 m high storm swells shoaling onto Santa Barbara from 265^0 with a 14 sec period during 21 November 2011. The longer period southwest swells of the stormy wet weather scenario produce a pronounced pattern of shadows (regions of locally smaller waves) and bright spots (regions of locally higher waves). Wave-driven nearshore currents flow away from bright spots and converge on shadows. Inspection of Figure 4.5 reveals that the open ocean intake site is in a bright spot, where oscillatory wave velocities scrubbing over the surfaces of the intake screens and supporting strucutures will induce large turbulent eddy formation and seabed scour; whereas likely subsurface intake sites closer to shore in in the surf zone (inshore of the wave break point for these extreme 6 m high breakers). For average wave conditions typical of Figure 4.4, the potential subsurface intake sites are found to be in a wave shadow zone where sediment deposition is more likely.

4.3) Current Forcing:

While waves dominate the initial dilution and dispersion of brine and secondary treated wastewater (effluent) discharges in the inshore domain, the tidal currents control dilution and dispersion in the offshore domain of the Santa Barbara Littoral Sub-cell. These currents also augment the scrubbing and vortex scour action of the local flow around the open ocean intake screens, and impart a net drift to the sediment suspended by wave and current scour of the seabed in the immediate neighborhood of those structures. The ADCP measurements taken during the 2012-2013 NPDES monitoring studies at mooring stations #RWS-2 and #RWS-3 shown in Figure 1.2, (Tierra Data, 2013), provide high resolution time series of these currents, which are subsequently used to calibrate the TIDEFEM codes of the SEDXPORT model to reconstruct long term currents from archival tidal constituents, as shown in Figure 4.10c. Figure 4.6 plots the near-bottom ADCP currents from profile cell #1 (2.4 m above seabed) at mooring RWS-3 during the NPDES monitoring period 11/14/11-11/24/12 for the east-west current velocity component (a); north-south velocity component (b); and total velocity amplitude (c). Figure 4.7 decomposes the near-bottom total velocity amplitudes into probability densities (red bars) and cumulative probability (blue). We find rather large maximum near bottom currents on the order of 50 cm/sec (~1.0 kt) in the inshore domain of the offshore intake site, while average near bottom currents are considerably less, on the order of only 5 cm/sec. Further up into the water column, in the vicinity of the intake screens, ADCP currents from profile cell #3 (6.4 m above seabed) in Figure 4.8 find many more instances of maximum currents on the order of 50 cm/sec to 55 cm/sec during the NPDES monitoring period 11/14/11-11/24/12. Accordingly, the probability densities (red bars) and cumulative probability (blue) of the interior currents in Figure 4.9 reveal higher average total current amplitudes, on the order of only 8 cm/sec to 10 cm/sec. These are favorable findings with respect to obtaining high along screen to through screen velocity ratios, which will provide high rejection ratios of potentially entrained eggs, larvae, juveniles and ichthyoplankton.

The ADCP current data collected for the 2012-2013 NPDES monitoring studies for EEWWTP in Figures 4.6-4.9 were used to calibrate the TIDEFEM codes of the SEDXPORT model in APPENDIX-A to reconstruct a long term current record from archival data of tidal elevation, based on algorithms from Long's Code. This reconstruction is shown in Figure 4.10c and allows a simultaneous current forcing record to be combined with long term records of the other controlling variables to ultimately calculate long term probability statistics on brine dilutions. In addition, this TIDEFEM calibration with ADCP data also permits a complete 2-d construction of the current field throughout the Santa Barbara Littoral Cell.

4.4) Tides and Extreme Water Levels:

The nearest ocean tide gage station is at Santa Barbara Harbor (NOAA # 941-0840). However continuous ocean water level measurements are only available at this station after 1995. The Santa Barbara tide gage (NOAA #941-0660) was last leveled using the 1983-2001 tidal epoch. Elevations of tidal datums referred to NAVD 88, in feet are listed in Table 4.1. In the wave run-up statistical analysis, the one percent recurrence interval for combinations of high waves and high tides is given by the condition when the compound frequency (joint probability) of the wave height and the tide stage is 1 in 100 years or 1%:

$$P[H_{\max}, \eta] = P(H_{\max}) \bullet P(\eta) = \frac{1}{100} \qquad (36)$$

Where $P_{H\max}$ is the return frequency of the maximum wave height $H_{\max}$ in T_R years or $P(H_{\max}) = 1/T_R$, where T_R is the wave return interval; and $P(\eta)$ is the probability of ocean water levels reaching an elevation of $\eta = Z_i$ feet NAVD 88. $P(\eta)$ is often referred to as the tidal hydroperiod function and is derived from the NOAA tide gage records of water elevation η based on the number of water elevation measurements $N(\eta \geq Z_i)$ that exceed any given reference level Z_i, or

Table 4.1: Elevations on Station
Datum Station: 9411340, Santa Barbara, CA **Status:** Accepted (Dec 5 2011) **Units:** Feet
T.M.: 120
Epoch: 1983-2001
Datum: STND

Datum	Value	Description
MHHW	8.56	Mean Higher-High Water
MHW	7.80	Mean High Water
MTL	5.97	Mean Tide Level
MSL	5.95	Mean Sea Level
DTL	5.86	Mean Diurnal Tide Level
MLW	4.14	Mean Low Water
MLLW	3.16	Mean Lower-Low Water
NAVD88	3.29	North American Vertical Datum of 1988
STND	0.00	Station Datum
GT	5.40	Great Diurnal Range
MN	3.66	Mean Range of Tide
DHQ	0.75	Mean Diurnal High Water Inequality
DLQ	0.98	Mean Diurnal Low Water Inequality
HWI	5.52	Greenwich High Water Interval (in hours)
LWI	11.57	Greenwich Low Water Interval (in hours)
Maximum	10.55	Highest Observed Water Level
Max Date & Time	01/19/1992 16:24	Highest Observed Water Level Date and Time
Minimum	0.27	Lowest Observed Water Level
Min Date & Time	12/17/1933 08:00	Lowest Observed Water Level Date and Time
HAT	10.36	Highest Astronomical Tide
HAT Date & Time	07/11/1987 04:30	HAT Date and Time
LAT	1.21	Lowest Astronomical Tide
LAT Date & Time	01/01/1987 00:18	LAT Date and Time

Tidal Datum Analysis Periods
01/01/1983 - 12/31/2001
04/01/2006 - 03/31/2010

EHW = 7.26 ft NAVD, (5.00 ft NGVD)
MHHW = 5.27 ft NAVD, (3.01 ft NGVD)
MHW = 4.51 ft NAVD, (2.25 ft NGVD)
MSL = 2.66 ft NAVD, (0.40 ft NGVD)
MTL = 2.68 ft NAVD, (0.42 ft NGVD)
ELW = -3.02 ft NAVD, (-5.28 ft NGVD)
NGVD = 2.26 ft NAVD, (0.0 ft NGVD)

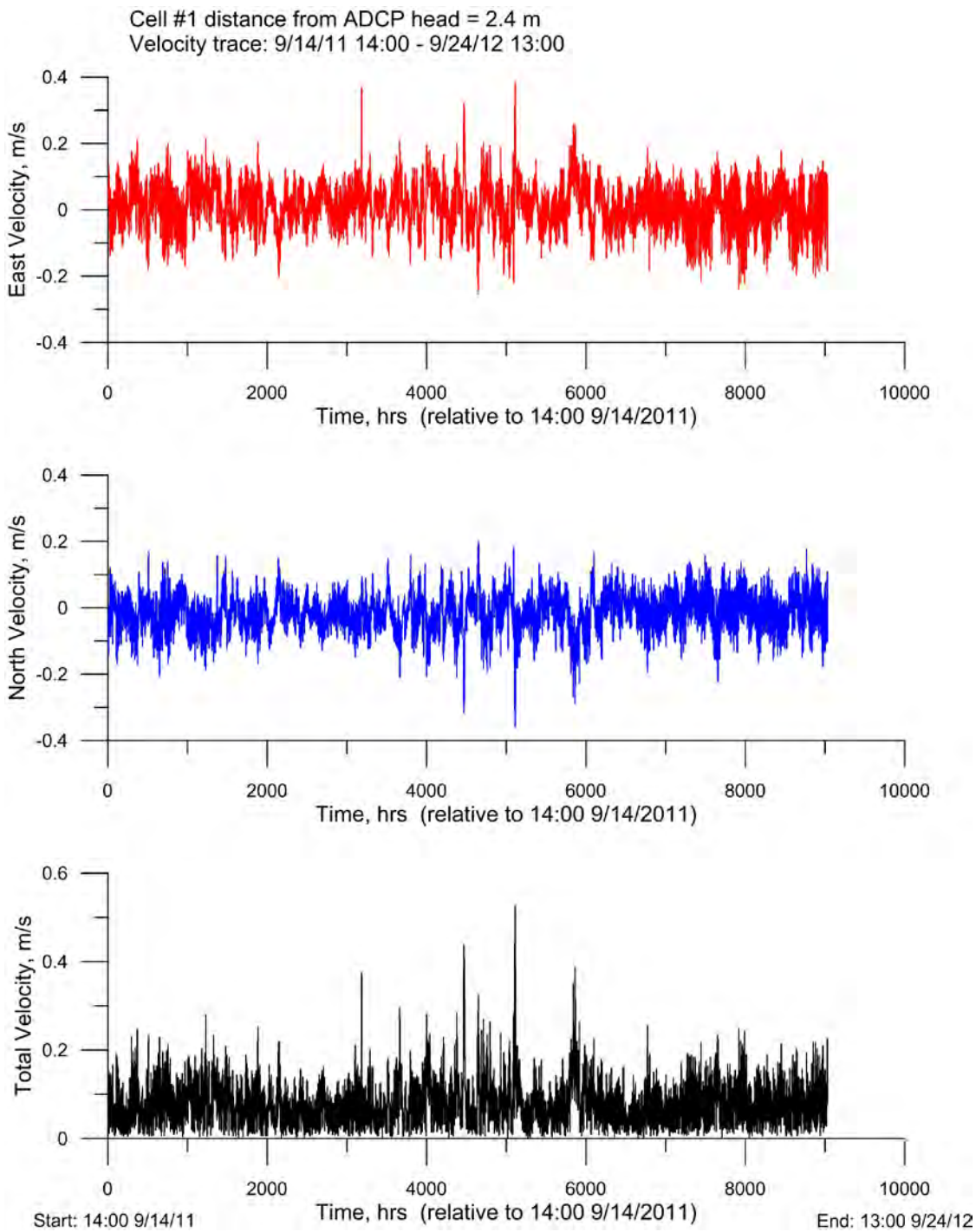

Figure 4.6: Near-bottom currents (2.4 m above seabed) near mooring location RWS-3 in Figure 1.2. Measurements by Acoustic Doppler Current Profiler under the 2012-2013 EEWWTP NPDES monitoring studies (SCCOOS, 2014, CalCOFI, 2014), 11/14/11-11/24/12. East-west current velocity component (a); north-south velocity component (b); total velocity amplitude (c).

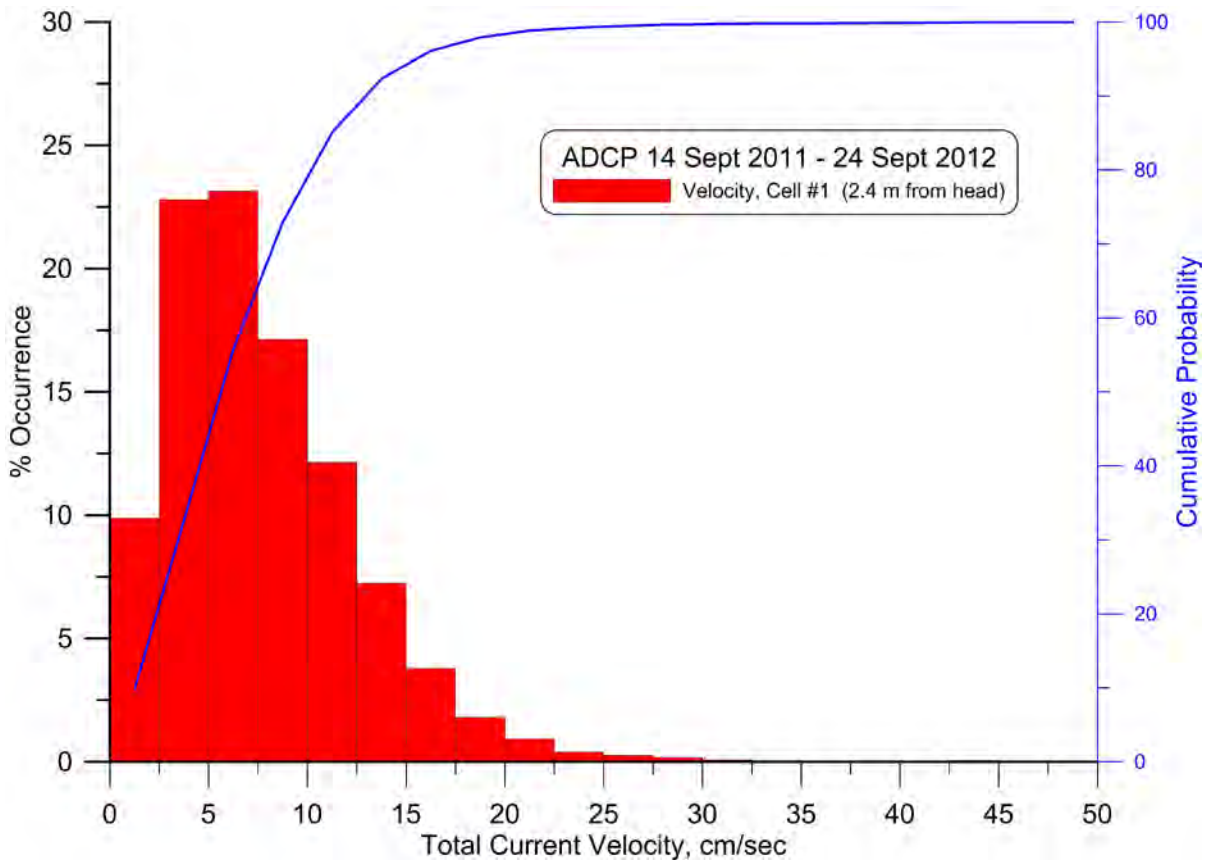

Figure 4.7: Histogram (probability density) and cumulative probability of near-bottom currents (2.4 m above seabed) near mooring location RWS-3 in Figure 1.2. Measurements by Acoustic Doppler Current Profiler under the 2012-2013 EEWWTP NPDES monitoring studies (SCCOOS, 2014, CalCOFI, 2014), 11/14/11-11/24/12.

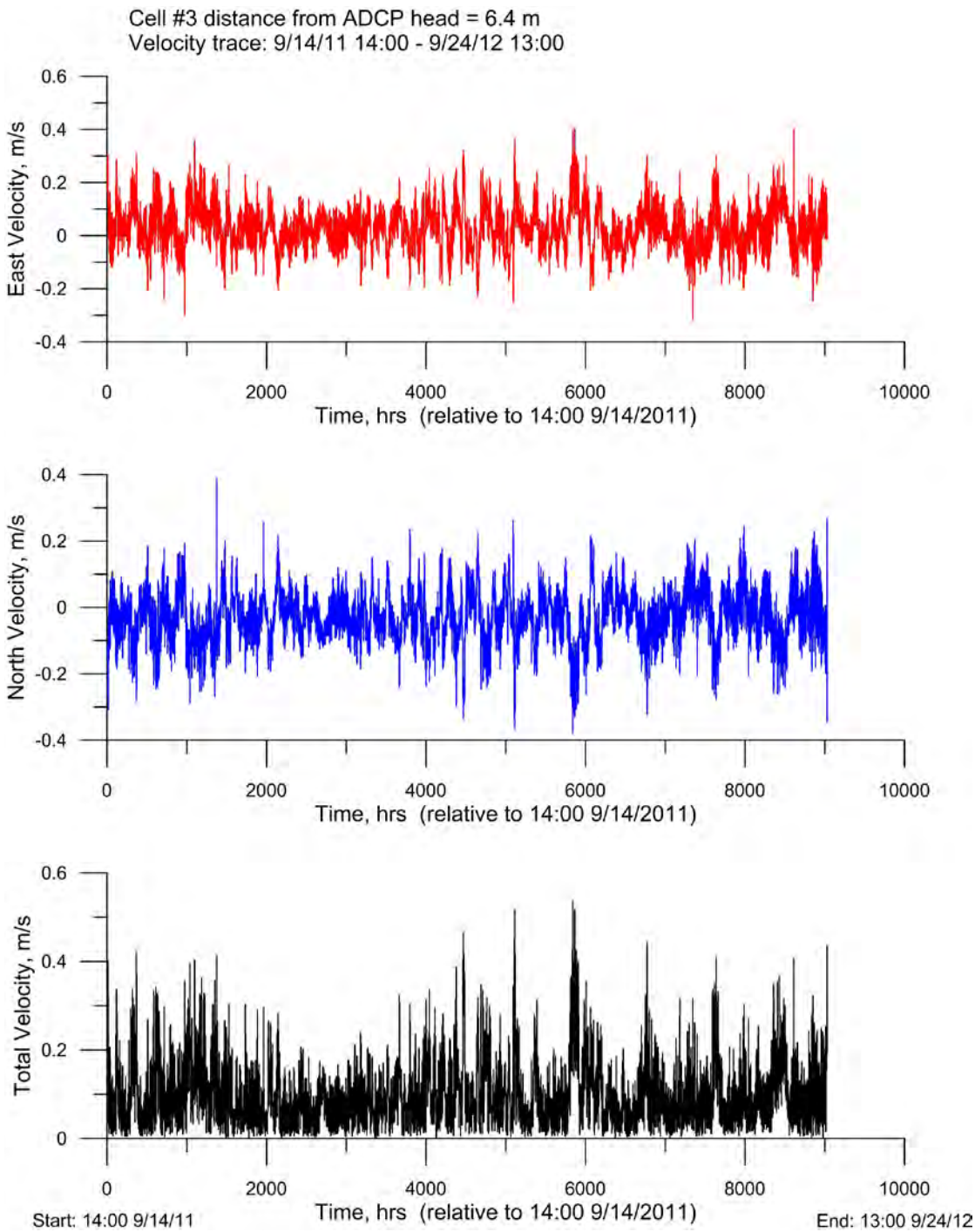

Figure 4.8: Interior currents near the offshore open ocean intake site (6.4 m above seabed) near mooring location RWS-2 in Figure 1.2. Measurements by Acoustic Doppler Current Profiler under the 2012-2013 EEWWTP NPDES monitoring studies (SCCOOS, 2014, CalCOFI, 2014), 11/14/11-11/24/12. East-west current velocity component (a); north-south velocity component (b); total velocity amplitude (c).

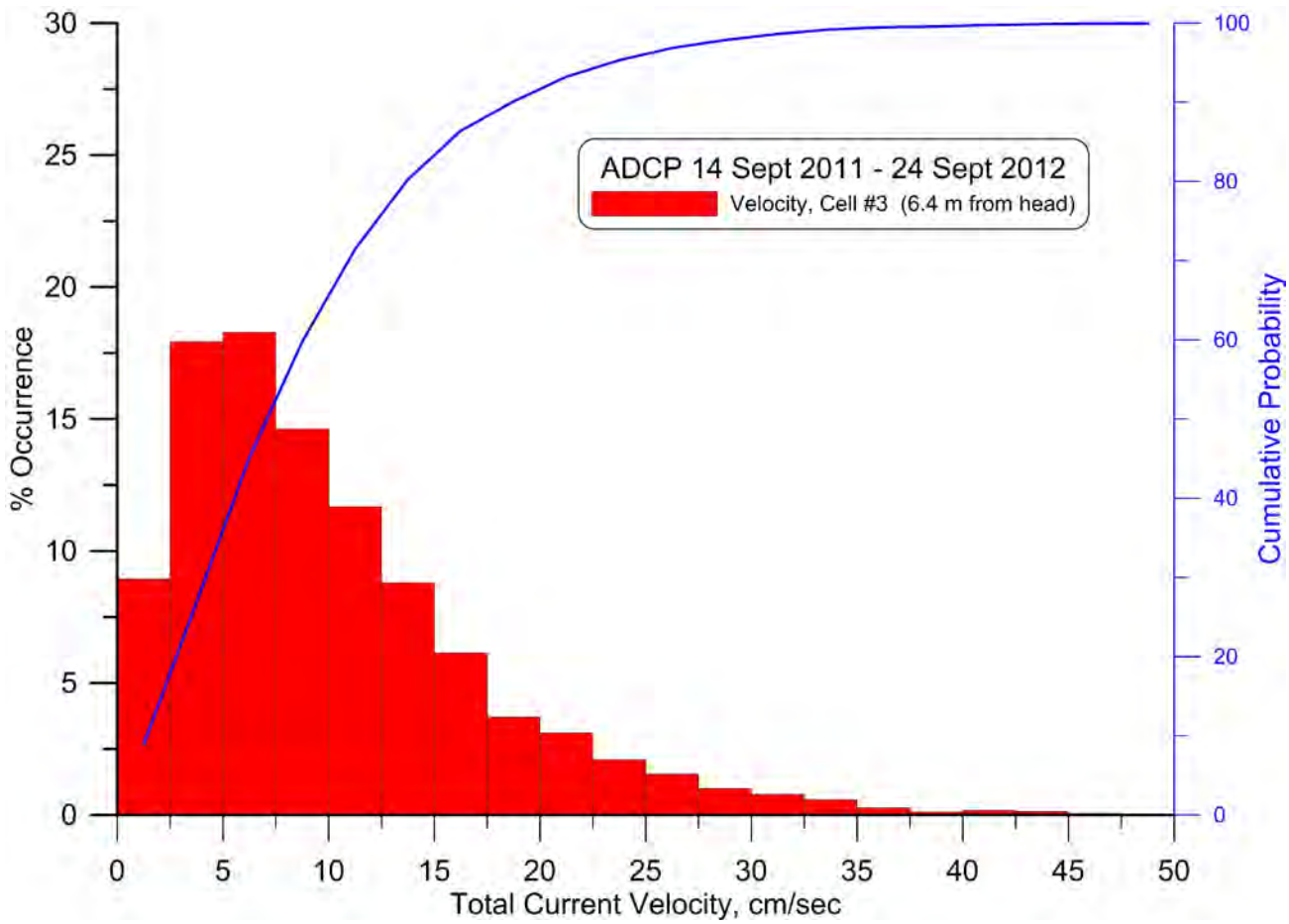

Figure 4.9: Histogram (probability density) and cumulative probability of interior currents (6.4 m above seabed) near mooring location RWS-2 in Figure 1.2. Measurements by Acoustic Doppler Current Profiler under the 2012-2013 EEWWTP NPDES monitoring studies (SCCOOS, 2014, CalCOFI, 2014), 11/14/11-11/24/12.

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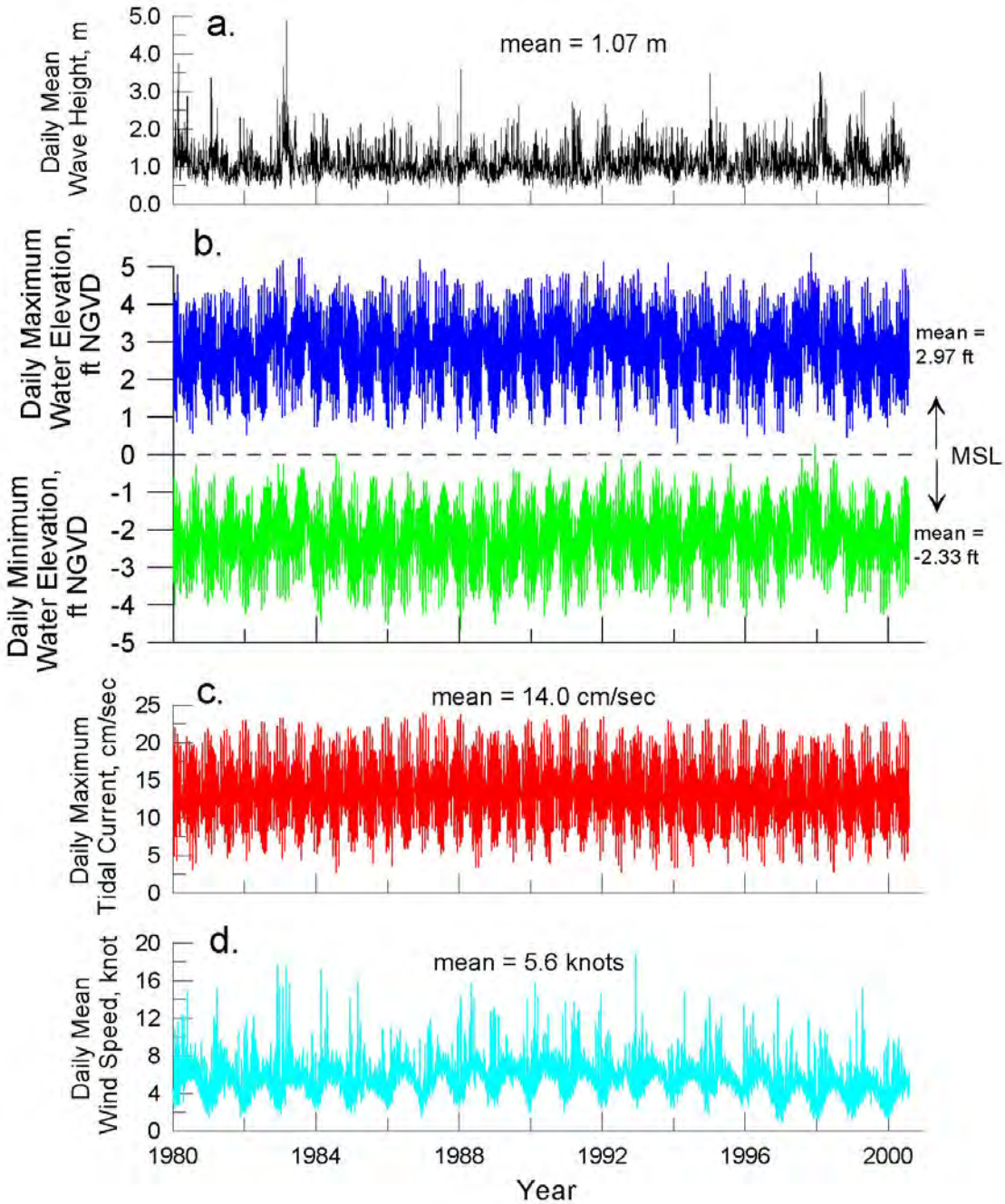

Figure 4.10: Long-term record of controlling environmental variables for long-term CEM analysis of seafloor stability.

$$P(\eta) = \frac{100\%}{N_o} \sum N(\eta \geq Z_i) \tag{37}$$

Where N_0 is the total number of tide gage measurements (typically at 6 minute intervals) in the NOAA tide gage record. Daily maximum water levels from the Santa Barbara tide gage are plotted in Figure 10b; and the hydroperiod function computed from Equation 37 from the NOAA Santa Barbara tide gage record is plotted in Figure 4.11.Inspection of Figure 4.11 reveal that the highest observed water level at Santa Barbara Harbor was EHW = 7.26 ft NAVD or +5.00 ft NGVD which occurred on 19 January 1992, while mean sea level is at MSL = 2.66 ft NAVD, or 0.40 ft NGVD. In an extreme run-up analysis for the 1% return frequency due to the compound occurrence of maximum waves and and maximum tides, Equation 36 would apply the EHW water level to a wave height having a 1 year return interval, while MSL water levels would be applied to a wave height having a 100 year return interval.

4.5) Sediment Flux from River Floods:

River sediment flux is the most persistent source term in the sediment budget of the Santa Barbara Littoral, and is due to the discharges from four major rivers: Santa Maria River, Santa Ynez River, Ventura River, and the Santa Clara River; two major creeks: San Antonio Creek and Calleguas Creek; and numerous smaller creeks in the Santa Ynez Mountain Watershed; particularly the reach between Pt Conception and Santa Barbara. The sediment flux into the mass balance of the Santa Barbara Littoral Cell is represented by the *J(t)* term in equation (3). The USGS has published annual mean flow volumes since 1940 and daily event based runoff volumes for these rivers and creeks during water years 1997-98 and 1998-99 (USGS, 2000). The annual mean flow volumes at the USGS gage stations on these creeks for the period of record of 1940-99 are listed in Inman and Jenkins, 1999. The peak flow events were in 1969 and 1983, and no comparable floods have occurred since 1998.

The sediment yield data induced by rainfall variation is derived by applying sediment rating curves to the annual mean stream flow of the rivers and creeks of the Santa Barbara Littoral Cell. The rating curves were derived in a two-step procedure [e.g., Brownlie and Taylor, 1981a&b]. This procedure utilized a limited amount of daily sediment flux measurements available under two separate USGS monitoring programs, namely: 1) the Hydrologic Benchmark Network; and 2) the National Stream Quality Accounting Network (USGS, 1997). Rather than seeking rating curves between annual flow volume and annual sediment flux per Brownlie and Taylor (1981a), better correlations are obtained between daily cumulative flow volume, (V_i, m^3/day) and daily sediment yield (J_i, tons/day), see Inman and Jenkins, (1999). These data were fitted to a power function $J_i = \xi Q^{\omega}$, where (ξ, ω) are statistically derived constants (per equation 9) that give daily estimates of sediment flux for each watershed over the period of record of the CEM simulations. For the Santa Maria River ξ = 5.23 x 10^{-9} and ω = 1.078; for the Santa Ynez River, ξ = 4.64 x 10^{-9} and ω = 1.765; for the Ventura River, ξ = 3.20 x 10^{-9} and ω = 1.539 while for the Santa Clara River ξ = 7.48 x 10^{-9} and ω = 1.502. For the San Antonio Creek ξ = 2.03 x 10^{-9} and ω = 1.163; for the Santa Ynez Creeks, ξ = 5.04

x 10^{-9} and ω = 1.872; while for the Calleguas Creek, ξ = 4.13 x 10^{-9} and ω = 1.892. Sediment flux data for these rivers and creeks are listed in Table-1. There it is shown that sediment flux from the Santa Clara river, is an order of magnitude greater than all other watersheds; but the combined fluxes from the watersheds that are up-drift from Santa Barbara (and therefore provide sediment cover to the study area) total 1,413,000 yd^3/yr or roughly equivalent to the sediment yield of the Santa Clara River.

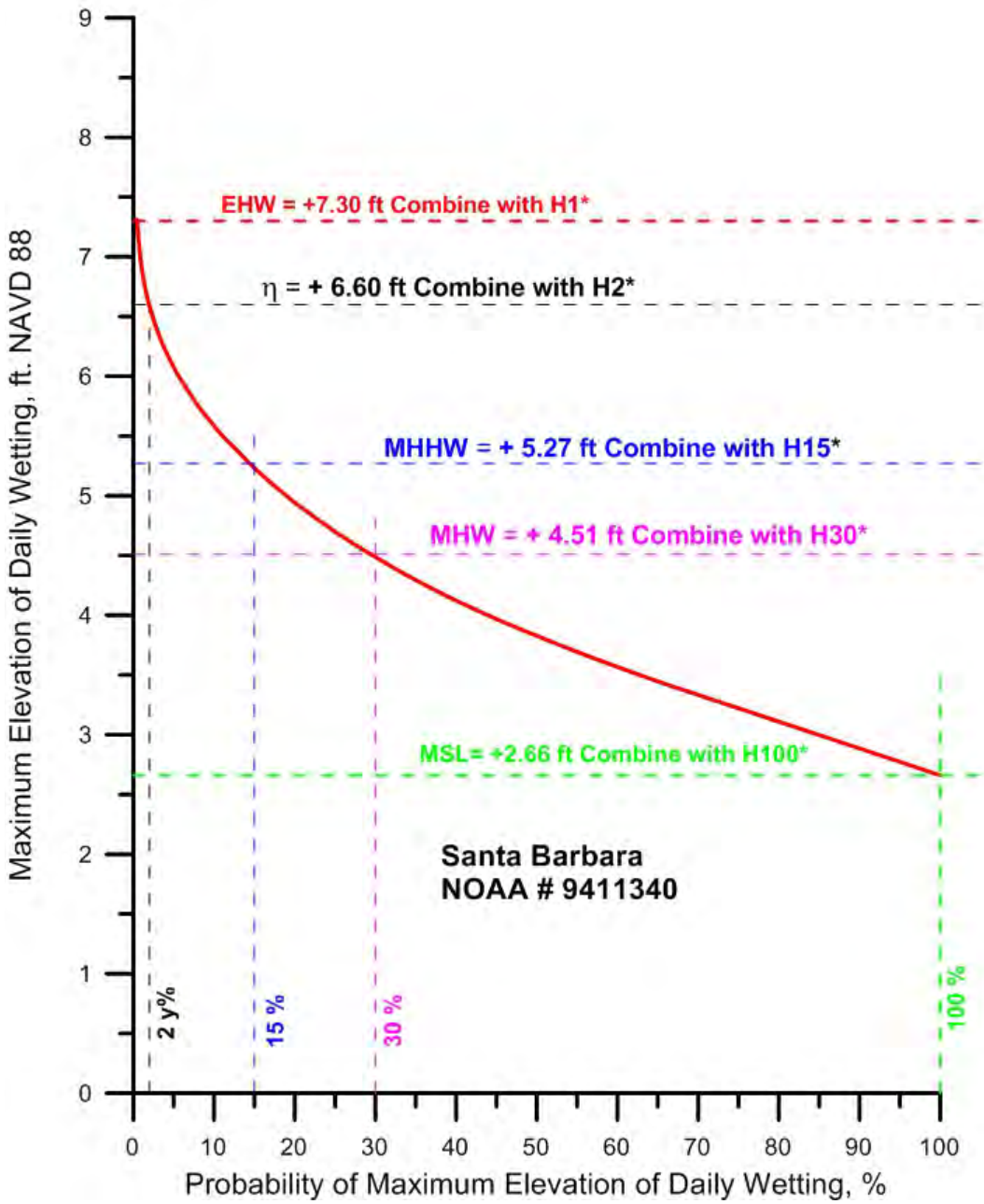

Figure 4.11: Tidal hydroperiod function giving the probability of maximum elevation of daily tidal inundation at Santa Barbara. Constructed from verified NOAA tide gage waterlevel record at Santa Barbara, NOAA tide gage # (941-1340). Note: 0.0 ft. NGVD 29 = +2.26 ft NAVD 88.

The current understanding of reductions in the natural supply of sediment to the coast that has grain size greater than 0.062 millimeters (fine sand) from the major rivers and streams within the Central and South Region are summarized in Table 2**.** Construction of dams has been the major reason for the reduced delivery of sand to the beaches. Based upon Farnsworth and Warrick's study (2007) the mean annual fine sediment contributions (silt and clay sized material) from rivers and streams can be at least as much as the corresponding sand delivery values or substantially higher. These values are used as sediment source inputs to the CEM sediment budget analysis for the Santa Barbara Littoral Cell, and are summarized in Figure 4.12.

Table 4.2: Annual Sediment Flux from Rivers and Creeks of the Santa Barbara Littoral Cell

(data from Inman and Jenkins, 1999; Willis and Griggs, 2003)

Watershed	Pre-Dam Sediment Flux, J_i, yd^3/yr	Post-Dam Sediment Flux, J_i, yd^3/yr	Reduction (%)
Santa Maria River	811,000	261,000	68%
San Antonio Creek	60,000	60,000	0%
Santa Ynez River	713,000	347,000	51%
Santa Ynez Mountains Watersheds	195,000	195,000	0%
Ventura River	216,000	102,000	53%
Santa Clara River	1,634,000	1,193,000	27%
Calleguas Creek	65,000	65,000	0%

4.6) Sediment Sources from Cliff and Bluff Erosion:

The episodic erosion of seacliffs that occurs primarily between Point Conception and Santa Barbara is the other significant source of sediment that is naturally delivered to the shoreline. Estimates of the quantity of sand that enters the littoral system over time vary between scientific studies. Runyan and Griggs (2003) have proposed that only sediment with grain sizes greater than 0.125 millimeters in diameter meaningfully contribute to nourishment of sandy beaches. Using this sediment size cutoff criteria the natural contribution from bluff erosion between Point Conception and Santa Barbara would be about 14,000 cubic yards per year under all natural conditions. Considering the effects of seacliff armoring and erosion protection which has reduced erosion by an estimated twenty percent, the present-day contribution may also be reduced to a volume on the order of 11,000 cubic yards per year. However, this volume constitutes only about 3.6 percent of the average annual maintenance dredging volume at Santa Barbara Harbor

(315,000 cubic yards per year). Other scientists differ on the amount of sand that seacliffs may contribute to the coast. Diener (2000) considers littoral sediments as fine sand (>0.0625 millimeters) in his research. Using his criteria and study results, the contribution of sand from seacliff erosion may be much greater (i.e., about 106,000 cubic yards per year). After accounting for seacliff armoring effects, a net contribution volume of about 86,000 cubic yards per year is estimated. This value represents over one-forth of the sand that is dredged on average from Santa Barbara Harbor. The natural supplies of sediment to the Santa Barbara Littoral Cell are summarized in Figure 4.12.

4.7) Sediment from Beach Disposal of Dredge Material:

Another important input to the sediment source term *J(t)* in the CEM sediment budget (equation 1) is beach disposal of dredge material, otherwise referred to as *beach nourishment*. Beach nourishment has been especially active in the Santa Barbara Littoral Cell for many years, principally due to beach disposal of dredge material from Santa Barbara, Ventura and Port Hueneme Harbors. In fact, the understanding of alongshore sand movement and littoral drift has been deduced mainly from study of the sand that accumulates at Santa Barbara, Ventura, and Channel Islands Harbors. Each harbor is a littoral sand trap, and regular maintenance dredging is required to maintain sand supply to the downcoast beaches. The annual average volume of sand that is dredged from each harbor indicates the increasing gradient of sand movement along the Santa Barbara Littoral Cell from west to east (cf. Figure 2.13):

Santa Barbara Harbor – 315,000 cubic yards per year.
Ventura Harbor – 597,000 cubic yards per years
Channel Islands Harbor – 1,010,000 cubic yard per year.

Port Hueneme Harbor requires little dredging since most of the sand is trapped immediately upcoast at Channel Islands Harbor and the harbor entrance is located at the head of the Hueneme Submarine Canyon. The eastern region of the littoral cell is considered to be sediment abundant which means there is always sand on the beach that can be moved regardless of the duration and intensity of the incident waves. In contrast, the beaches within the western portion of the Santa Barbara Littoral Cell are considered to be sediment limited. For this reason, the annual dredging volumes of Santa Barbara Harbor are the smallest of any harbor in the littoral cell. It also means that the amount of wave energy that impacts the shoreline around Santa Barbara is capable of moving more sand than exists on the beach. Under these conditions, the relatively thin deposits of sand that form the narrow sediment limited beaches can be quickly stripped away as the sand transport capability of the incident waves (potential sand transport) exceeds the smaller volume of sand that is present and moved (actual sand transport). This is another reason why the Neodren system is a preferable subsurface intake option at Santa Barbara than the SIG or BIG systems, as it requires substantially less sediment cover than the SIG or BIG systems.

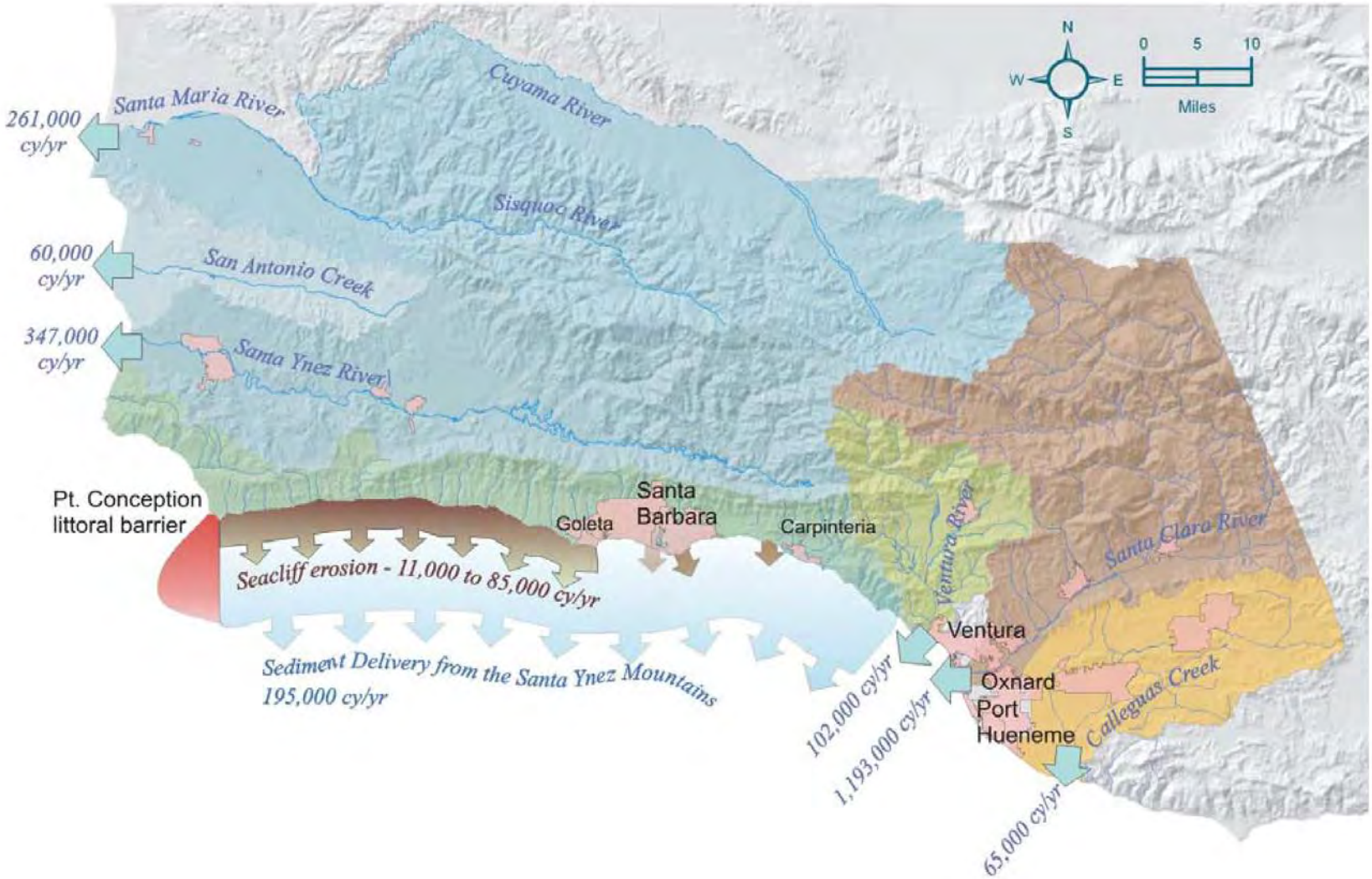

Figure 4.12: Natural supply of sediment to the Santa Barbara Littoral Cell. As previously discussed, all of the river and stream sediment that is discharged to the North Region is confined there and does not pass around the Point Conception littoral barrier; (from BEACON, 2011)

4.8) Sediment Grain Size and Stratigraphy:

Grain size of the sediments in the nearshore domain, and their variability with depth in the seabed (stratigraphy) is a leading order variable in both the closure depth and beach/shorerise profile algorithms of the Coastal Evolution Model. The model is initialized using beach and seafloor cores taken at in the nearfield of the study area. The closure depth solutions and elliptic cycloid profile solutions that determine the burial and erosion potential of the intake and discharge end-works are functions of the seabed sediment grain size (Jenkins and Inman, 2006). There is a unique solution for the volume of critical mass for any arbitrary selection of grain size in the bar-berm, D_1, and the shorerise, D_2 (Figures 3.7, 3.9, & 3.10). Seafloor sediment characterization by Calscience Environmental Laboratories for the Leadbetter, West and East Beach intake sites as well as the offshore diffuser and open ocean intake sites has produced the grain size distributions shown in Figures 4.13 – 4.16. These grain size values are inputs to the elliptic cycloid solutions in equations (10) – (17) to compute the long term shore rise and bar berm profile evolution, the closure depth and critical mass and the seafloor stability for the intake infrastructure associated with this study. A general inspection of Figures 4.13-4.16 reveals the coarsest median grain size and lowest percentage of silts and clays are found at Leadbetter Beach, where a persistent bright spot in the refraction/diffraction pattern of the largest waves (Figure 4.5) winnows out the finer size fraction. On the other hand, West Beach has the finest size median grain size and higher percentages of silts and clays (26.72 %), primarily because it is in a quiet water depositional area behind the Santa Barbara Harbor jetties. East Beach is intermediate, since it is a receiver beach for the sediments dredged from the Santa Barbara Harbor, but it is also an exposed erosional area where the finer grain sized fractions are winnowed away by shoaling waves.

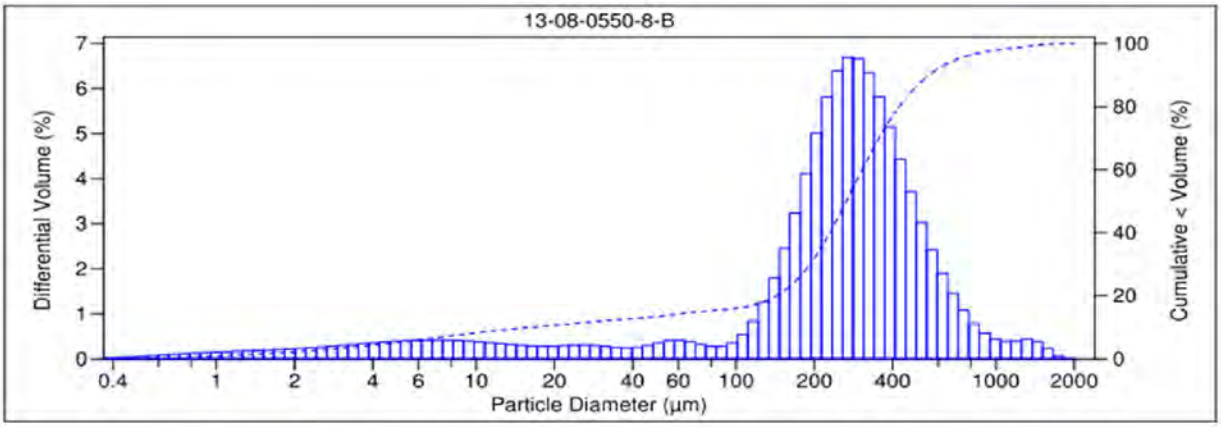

Figure 4.13: Grain Size Distribution at Leadbetter Beach, Santa Barbara, 6 July, 2013. Median grain size = 0.301 mm. Silt and clay = 14.36 %. Data from Calscience Environmental Laboratories in Tierra Data, (2013)

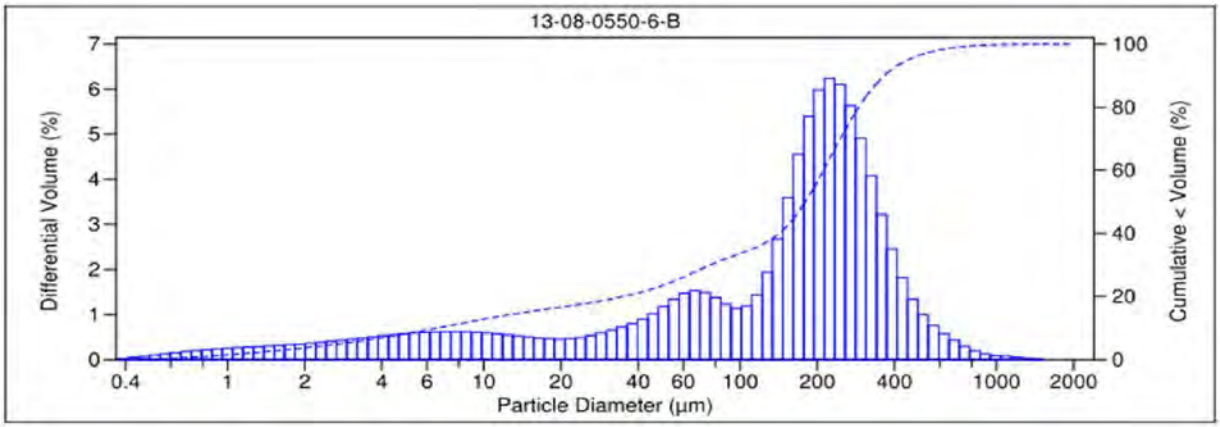

Figure 4.14: Grain Size Distribution at West Beach, Santa Barbara, 6 July, 2013. Median grain size = 0.186 mm. Silt and clay = 26.72 %. Data from Calscience Environmental Laboratories in Tierra Data, (2013)

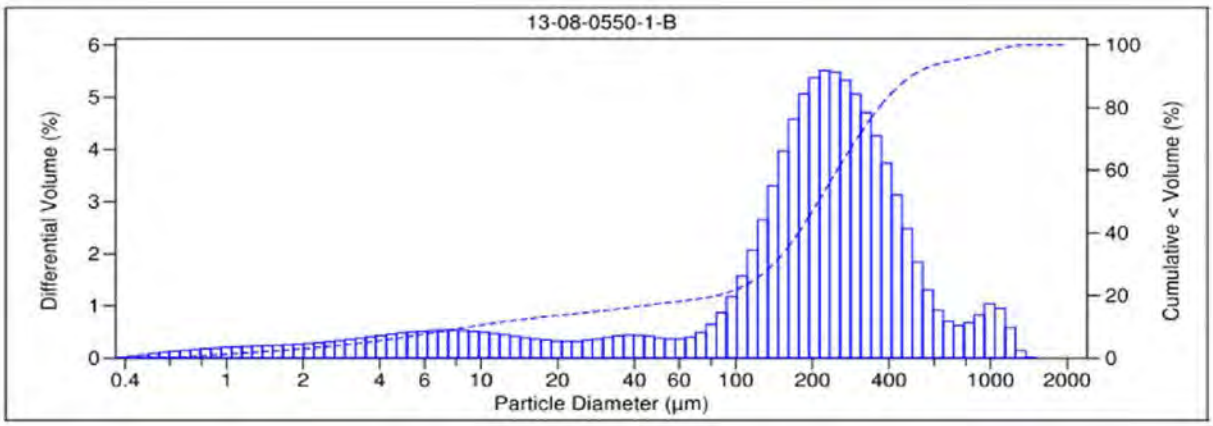

Figure 4.15: Grain Size Distribution at East Beach, Santa Barbara, 6 July, 2013. Median grain size = 0.253 mm. Silt and clay = 18.32 %. Data from Calscience Environmental Laboratories in Tierra Data, (2013).

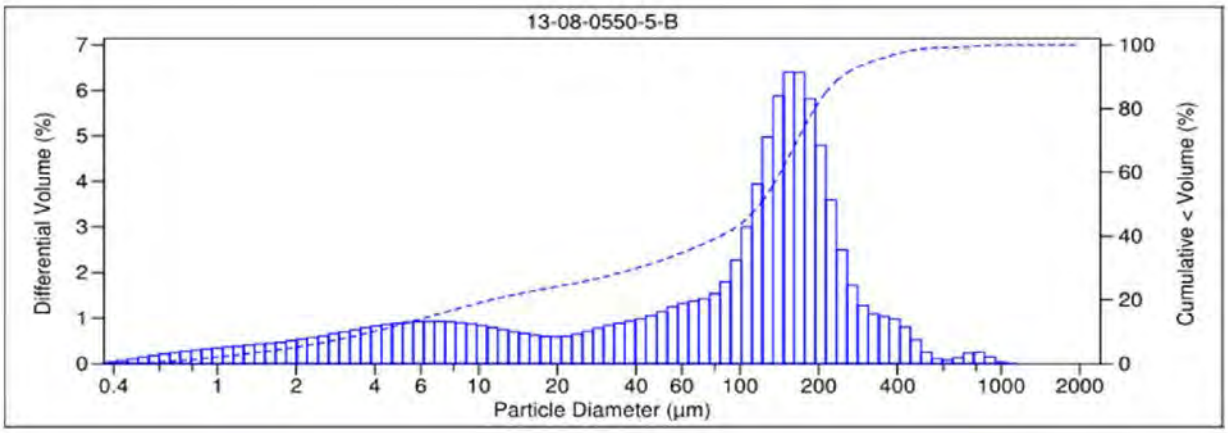

Figure 4.16: Grain Size Distribution at offshore of East Beach near offshore intake site at sample station RSW-3, Santa Barbara, 6 July, 2013. Median grain size = 0.125 mm. Silt and clay = 35.37 %. Data from Calscience Environmental Laboratories in Tierra Data, (2013).

4.9) Beach and Shorerise Profiles:

Non-Stationary bathymetry is the domain of seafloor inshore of closure depth that varies over time in response to beach erosion and accretion. It is measured periodically with beach and shorerise profiling conducted by contractors to the US Army Corps of Engineers (USACE) in fulfillment of requirements for issuance of dredge permits for the Santa Barbara Harbor. These measurements are archived in the dredge permits of the Los Angeles District USACE, (1980 -2008), and the profiles for the ranges relevant to the seabed stability around the intake sites are plotted Figures 4.17 - 4.19. These measurements are used to calibrate the beach and shorerise profile algorithms in the Coastal Evolution Model. Measured beach and shore-rise profiles across the harbor north fillet beach at Leadbetter Beach are plotted in Figure 4.17 between February 1980 and September 2004. This is the up-drift beach at the harbor breakwater and typically represents the most accreted profiles in the nearfield of the intake site; thereby capturing the best case scenario at this site with respect to resident sediment cover. Figure 4.19 shows the measured East Beach and shore-rise profiles across the receiver beach at the east of the harbor breakwater, monitored by the U.S. Army Corps of Engineers, Los Angeles District, between June 1992 and April 2008. These profiles show considerably more variation over time than those at Leadbetter Beach due to the accretion and subsequent erosion in between each harbor dredging cycle. The beach profiles inside the harbor at West Beach in Figure 4.18 show the most variation due to the action of dredge cuts in the depositional features that form inside the harbor. The measurements in figures 4.17-4.19 are used to calibrate the beach and shorerise profile algorithms in the Coastal Evolution Model.

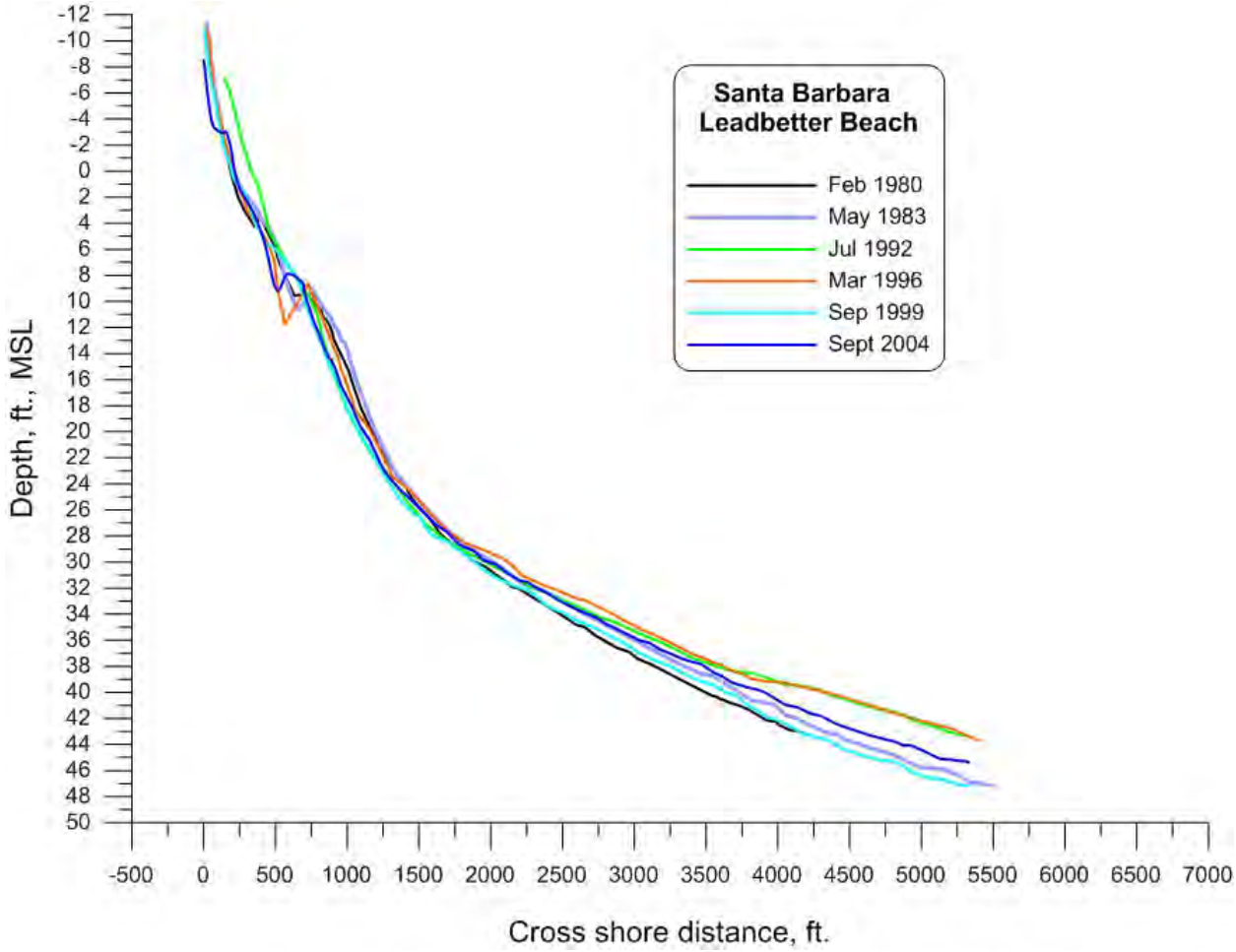

Figure 4.17: Measured beach and shore-rise profiles at the Leadbetter Beach intake site, (cf. Figure 1.2), monitored by the U.S. Army Corps of Engineers, Los Angeles District, between February 1980 and September 2004. Data from USACE Santa Barbara Harbor Dredge Permits, (1980 - 2008). Note depth is positive below MSL with units in feet MSL.

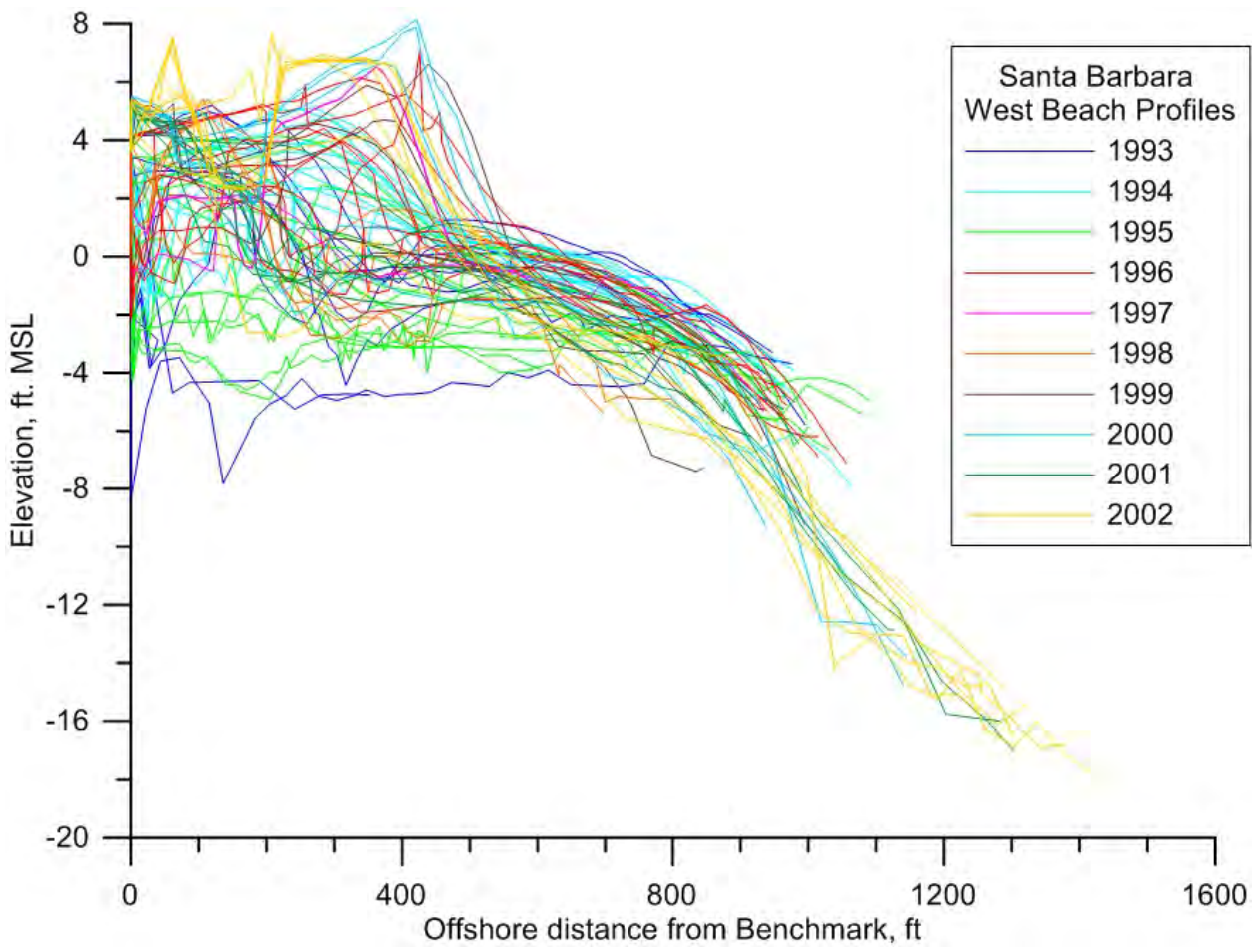

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Figure 4.18: Measured beach and shore-rise profiles at the West Beach intake site, (cf. Figure 1.2), monitored by the U.S. Army Corps of Engineers, Los Angeles District, between June 1992 and April 2008. Data from USACE Santa Barbara Harbor Dredge Permits, (1980 - 2008). Note elevations are negative below MSL with units in feet.

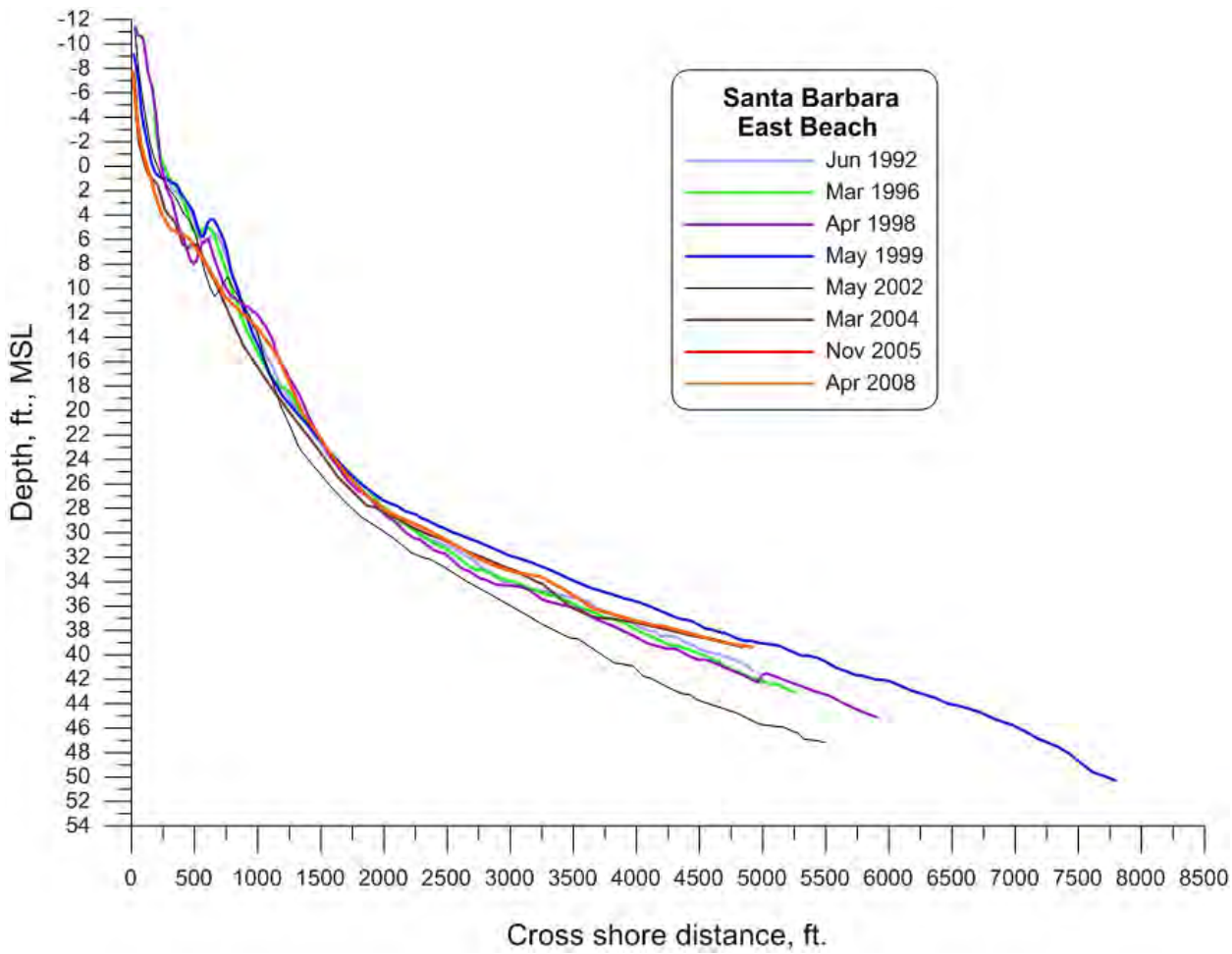

Figure 4.19: Measured beach and shore-rise profiles at the East Beach intake site, (cf. Figure 1.2), monitored by the U.S. Army Corps of Engineers, Los Angeles District, between June 1992 and April 2008. Data from USACE Santa Barbara Harbor Dredge Permits, (1980 - 2008). Note depth is positive below MSL with units in feet MSL.

5) Coastal Evolution Analysis of the Santa Barbara Littoral Cell:

The Coastal Evolution Model (CEM) was time-stepped through the 32 year period of record of input variables as detailed in Section 4, (1980-2012); producing 11,842 daily solutions at 1,800 coupled control cells (cf. Figure 3.3 b) along a 160 km reach of coast between Point Conception and the Mugu Submarine Canyon (Figure 2.13). Along a 6 km reach in the nearfield of the study area, computational precision was increased by using the nested inner nearfield grid with 1 arc-second resolution among 200 coupled control cells between the Cliff House and the Santa Barbara Cemetery. In the coarse outer grid, the control cells are assigned 90 m spacing along the coastline, and 30 m spacing in the high resolution inner grid. The keystone solutions in each control cell are: 1) the sediment volume flux, *dq/dt,* per unit length of shoreline (m^3/m/day), also referred to as the erosion-deposition flux; 2) the closure depth; and, 3) the critical mass envelope. The sediment volume flux, *dq/dt,* tells us whether the section of coast represented by a particular control cell is eroding ($dq/dt < 0$), or accreting through sediment deposition ($dq/dt > 0$). We use the sediment volume flux to assess the long-term seafloor stability of the sub-seabed intake sites at Leadbetter, West and East Beaches (Figure 1.2) as well as the offshore intake, (Figures 1.1 and 1.2, respectively). Ideally an optimal sub-seabed intake site will neither erode nor accrete; and so, we look for the closest places where, $dq/dt \rightarrow 0$.

The sediment volume flux is calculated by the CEM in each control cell using equation (1). The predominant term is the source term $J(t)$, due to the natural sediment sources from watershed and bluff erosion as quantified in Section 4.5 and 4.6. However, harbor dredging and beach disposal of dredge material produces large periodic pulses of sediment, especially in the nearfield of the study area. Here, sediment supply and resident beach sand volume are directly impacted by the sediment *hold-and-release effect* of the Santa Barbara Harbor and its maintenance dredging activity. This is especially evident from photographs taken before and after completion of the Santa Barbara Harbor breakwater system, (Figures 5.1 - 5.3). These figures clearly show how the prevailing west to east littoral drift has been intercepted and a large portion of it impounded by the harbor breakwater system. The sediment trapping effects of the harbor occur both inside and outside the harbor breakwater system, forming a new beach (West Beach) inside the harbor, and another (Leadbetter Beach) at the up-drift (west) end of the main breakwater. The littoral drift sands that were scavenged by the harbor breakwater to form these new beaches are no longer available to replenish wave erosion losses occurring on beaches down-drift of the harbor, resulting in progressive losses of beach width along the stretches of East Beach between the harbor and the Santa Barbara Cemetery (cf Figure 5.3). Moreover, the trapping of littoral drift sands inside the breakwater system reduces navigable water depths inside the harbor.

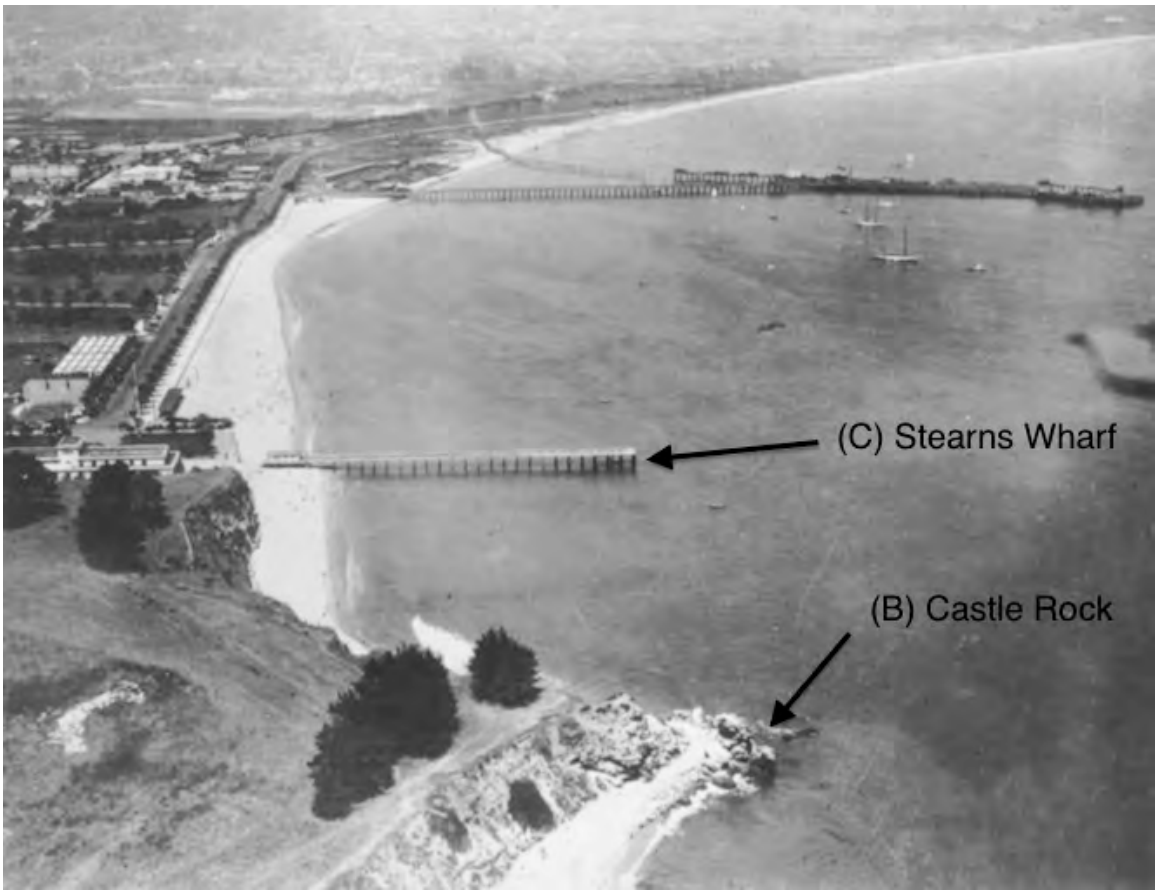

Figure 5.1: Photograph taken circa 1926 prior to construction of the Santa Barbara Harbor breakwater system. Note the very narrow section of beach west of Castle Rock in the lower foreground which is today known as Leadbetter Beach. Also note the very wide section of beach east of Stearns Wharf which is today known as East Beach.

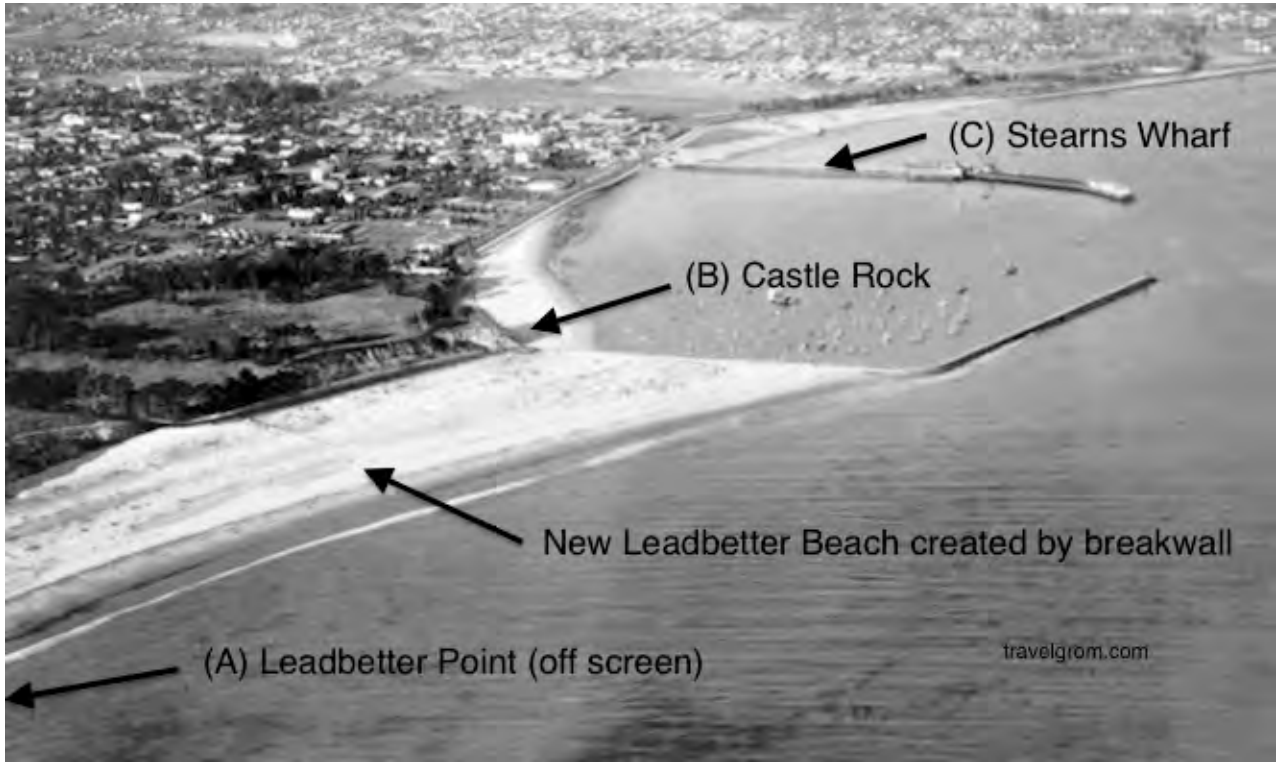

Figure 5.2: Photograph taken circa 1939 after completion of the Santa Barbara Harbor breakwater system in 1930. Note the formerly narrow section of beach west of Castle Rock (cf. Figure 5.1) has experienced significant accretion of sand forming a wide fillet beach on the west side of the harbor breakwater which is today known as Leadbetter Beach. The formerly wide section of beach east of Stearns Wharf (cf. Figure 5.1) has eroded and become very narrow (known as East Beach today). Also note the beginnings of the formation of a new beach immediately east of Castle Rock (known as West Beach today) produced by the trapping and impoundment of littoral drift sands within the harbor breakwater system.

Figure 5.3: Photograph taken circa May 1980 after the El Nino storms of the 1980 winter. Note the substantial erosion that has occurred on East Beach while Leadbetter Beach has remained essentially stationary in spite of wave overtopping and flooding of the Leadbetter Beach parking lot. Also note the prodigious beach widths the have been maintained through the winter storm series along the sheltered stretches of West Beach.

The US Army Corps of Engineers Los Angeles District recognized early on these sand trapping impacts of the harbor breakwater on navigation and the eastern beaches of Santa Barbara and instituted a maintenance dredging program which typically bi-annually or tri-annually dredges sands trapped inside the harbor breakwater and places it as beach fill on the eroding sections of East Beach. However dredged beach fill sediments do not stay where they were initially deposited, and will propagate down-drift over time as a lump of sediment known as an *accretion/erosion wave*, see Figure 3.3a and Inman and Jenkins (2004c). The formulation of this down-drift migration of the accretion/erosion wave is given by the second term in equation (3), the $V_l\,(dq/dy)$ term, known as *the advective term*. As the accretion/erosion wave migrates down-drift, it also spreads out laterally along the shore line and is reduced in amplitude by the action of the first term in equation (3), referred to as the surf-diffusion term, $\varepsilon\,(\partial^2 q/dy^2)$. The initial placement of a large amount of sediment in a relatively small area, (whether that be a river delta after a flood or a receiver beach after placement of dredged beach-fill), creates a large along-shore gradient in sediment volume, *dq/dy.* That gradient renders the sediment mass to be highly mobile under the influence of longshore currents, V_l, with additional spreading by surf diffusion. Longshore currents are generated when waves break at an angle to the shoreline, or when there is an along shore variation in wave height; where longshore currents flow down-coast in the direction of wave breaking and flow away from areas of high waves and towards areas of low waves. The formulation for the longshore transport rate of sediment, Q_L, due to the action of the longshore current, V_l, is taken from the work of Komar and Inman (1970) according to:

$$Q_L = K\left(C_n\, S_{yx}\right)_b \tag{38}$$

where C_n is the phase velocity of the waves; $S_{xy} = E\sin\alpha_b\cos\alpha_b$ is the along shore component of the onshore component of the radiation stress tensor; α_b is the breaker angle relative to the shoreline normal; $E = 1/8\rho g H_b^2$ is the wave energy density; ρ is the density of water; g is the acceleration of gravity; H_b is the breaking wave height; and, K is the transport efficiency equal to:

$$K = 2.2\sqrt{c_{rb}} \tag{39}$$

$$c_{rb} = \frac{2g\tan^2\beta_0}{H_b\sigma^2} \tag{40}$$

Here c_{rb} is the reflection coefficient which is calculated from the nearshore bottom slope, β_0 of the stationary bathymetry as determined from the break point coordinates and the position of the 0 MSL contour; and, σ is the radian frequency =

2π/T, where T is the wave period. The longshore transport velocity, $V_l = \overline{V_l(x)}$, is determined from the longshore current theories of Longuet-Higgins (1970), according to:

$$\overline{V}_l(x) = v_0\left(\frac{10x}{49X_b} - \frac{5}{7}\log\frac{x}{X_b}\right) \qquad \text{if} \qquad 0 \le x \le X_b$$

$$\overline{V}_l(x) = v_0\,\frac{10}{49}\left(\frac{x}{X_b}\right)^{5/2} \qquad \text{if} \qquad x > X_b \tag{41}$$

$$\text{where: } v_0 = \frac{0.256\pi\beta}{C_D}\sqrt{gh_b}\,\sin\alpha_b$$

Here, X_b is the width of the surf zone derived from the coordinates of the break points (x_b, y_b) that were computed from the CEM refraction analysis. Solutions from equations (38) - (41) give the highest rates of sediment flux in the neighborhood of the break point, $x = X_b$, where the longshore currents approach a maximum value of $\overline{V}_l(x) = v_0$. When the longshore transport rate is averaged over some extended length of time, t_0, the resultant is referred to as *potential littoral drift* $\overline{Q}_L$, where :

$$\overline{Q}_L = \frac{1}{t_0}\int_{t_0} KC_n S_{yx}\,dt \tag{42}$$

The net sediment volume flux out of or into a control cell (erosion or deposition, respectively) that results from the action of the advective term in equation (3) is related to the longshore transport rate Q_L by a functional known as the *divergence of drift*, $\nabla \bullet Q_L$, written as:

$$V_l\frac{dq}{dy} = \nabla \bullet Q_L \cong \int \frac{\partial Q_L}{\partial y}dy = KC_n\int\frac{\partial S_{yx}}{\partial y}dy \tag{43}$$

Therefore, the net erosion or deposition of sediment in a control cell due to advective transport by longshore currents (divergence of drift) is proportional to the along shore gradient of the radiation stress tensor component, $S_{xy} = E\sin\alpha_b\cos\alpha_b$. Positive values of radiation stress gradient indicate depositional tendencies, while negative values indicate erosion. Ideally, for a sub-seabed intake site we seek sections of coast where the radiation stress gradient is small and trending to zero. These equations (38-43) relate divergence of drift to the longshore flux of energy at the break point which can be obtained directly from the refraction/diffraction solutions of the CEM, (e.g., Figures 4.4 and 4.5); and is proportional to the square of the near breaking wave height and breaker angle. By this formulation, the CEM calculates a local sediment volume fluxes for

control cells in the far-field grid, and in the nearfield grid that are separated by great distances from the primary sources of sediment in the Santa Barbara Littoral Cell, in particular the Santa Ynez watersheds and bluffs between Pt. Conception and Santa Barbara (Figure 2.12).

The advective (divergence of drift) term of equation (3) is decisive to the sub-seabed intake siting analysis because it is the mechanism that spreads out the large volumes of river deposition and beach-fill over many kilometers of coastline of the Santa Barbara Littoral Cell, as well as diverting sand into and around the Santa Barbara Harbor. Divergence of drift and surf diffusion are wave driven, and their magnitudes and variations from place to place in the Santa Barbara Littoral Cell and around the harbor breakwater; and ultimately these mechanisms are driven by the wave refraction/diffraction pattern of the general region, beginning with the initial approach of waves into the Southern California Bight from distant storms. Figure 4.2 shows CEM computations of the refraction/diffraction patterns of the 5 largest storms to enter the Southern California Bight during the 1998 El Nino winter. Many areas of the Bight are sheltered from these waves by the break-water effect of the offshore islands (referred to as *island sheltering*); but the geometry of the Santa Barbara Channel leaves the western portion of the Santa Barbara Littoral Cell open to waves from the prevailing westerly storm track, while waves approaching from southern hemisphere storms and Mexican hurricanes can freely travel through the gaps between the Channel Islands to arrive at Santa Barbara.

Zooming in on local wave shoaling tendencies in the nearfield of Santa Barbara, Figure 4.4 reveals that an abrupt widening of the continental shelf seaward of Santa Barbara creates a large dog-leg in the -20 m to – 50 m depth contours, which in turn, gives rise to beams of intensified wave energy (red bright spots), that doubles shoaling wave heights at the western end of Leadbetter Beach and the eastern end of East Beach. Between these two bright spots, there is an area of greatly diminished wave energy (blue shadow zone) which shelters the east end of Leadbetter Beach, the harbor and open ocean intake sites. We will show from the wave transport and beach erosion simulations that these shadow zones promote seafloor stability in limited areas that could support intake infrastructure.

The CEM ran 11,842 daily refraction calculations over the 1980- July 2012 period of record, from which the littoral drift parameters of wave energy flux, long-shore current, and divergence of drift were obtained for 1,800 coupled control cells along a 160 km reach of coast between Point Conception and the Mugu Submarine Canyon. Model inputs for these calculations included CDIP monitored waves from Section 4.2, coastal currents from Section 4.3, tides and extreme water levels from Section 4.4, sediment flux from streams and creeks (Section 4.5), sediment flux from bluff erosion (Section 4.6), sediment flux from harbor dredging (Section 4.7) grain size distributions from Section 4.8. Using these inputs, the free parameters of the CEM were adjusted iteratively until the time averaged gross littoral transport rate computed at the four harbors could match the annual dredging volumes reported in Section 4.7. By this approach the harbors are used as control points in the littoral cell to calibrate the model,

under the assumption that the volumes of sand trapped by these harbors is a measure of the gross littoral transport rate.

Wave energy flux, potential littoral drift, and divergence of drift were averaged over the 32 year period of record and their variation along the coast is plotted in Figure 5.4 in terms of distance from Point Conception. Several striking trends are revealed. First, wave energy flux is not uniform throughout the Santa Barbara Littoral Cell (Figure 5.4a) due to island sheltering, local variations in shelf bathymetry and the angle of the local coastline relative to incident wave direction, all of which give rise to complex variations in wave refraction/diffraction patterns throughout the littoral cell. These variations in wave energy flux lead to variations in the potential littoral drift rate calculated from Equation (42) and plotted in terms of cubic meters of sand moving along shore per day in Figure 5.4b. Potential littoral drift represents the transport capacity of the incident wave field for moving sand along shore per day, and represents the actual littoral drift if there is adequate sand supply on the beach and shore rise. However, even in the nearfield of Santa Barbara, the sediment cover is very thin with rocky outcrops in some places, (cf. the isopach map in Figure 2.12); and in these places the actual littoral drift during storms in particular can be less than the potential littoral drift because there is more wave transport energy available to move sand than sand available to be moved. However one thing that is obvious from Figure 5.4b is that the littoral drift everywhere east of Pt. Conceptions flows eastward as one-way, unidirectional transport stream, or a *river of sand* so to speak, flowing away from sediment sources of the creeks, streams and bluffs of the Santa Ynez Mountains to the west, and flowing toward the Santa Barbara Harbor which acts as a local sediment sink. The rate of littoral drift reaching Santa Barbara Harbor averages about 650 – 700 m^3/day, but increases to about 3 times that rate at the Ventura and the Channel Islands Harbors further down-drift to the east where the largest sediment sources are found (Ventura and Santa Clara Rivers); indicating that the littoral drift in the western portions of the Santa Barbara Littoral Cell is probably sediment supply limited.

The divergence of drift calculated from Equation (43) and plotted in the lower panel of Figure 5.4 (c) adds another wrinkle to this transport mechanism. The divergence of drift is the dominant factor in determining whether a certain section of coast is erosional or depositional, where positive values are depositional and negative values are erosional. On the scale of the divergence of drift throughout the littoral cell, the area around Santa Barbara appears to be a complex mix of weakly erosional and depositional sections, with relatively small variations in divergence of drift on the order of +/- 1.0 to 2.0 m^3/day per meter of shoreline. This condition is referred to as *non-divergent littoral drift* and indicates a stable, steady-state condition that is neither erosional nor depositional, an optimal condition of sub-seabed intake site. Elsewhere in the Santa Barbara Littoral Cell (e.g. in the Ventura area), departures in divergence of drift are as large as +/- 10.0 to 20.0 m^3/day per meter of shoreline. Therefore, from the standpoint of seafloor stability, where one would like to have the divergence of drift as small as possible, the Santa Barbara area appears to be one of the more favorable regions in the Santa Barbara Littoral Cell for placing desalination intake infrastructure.

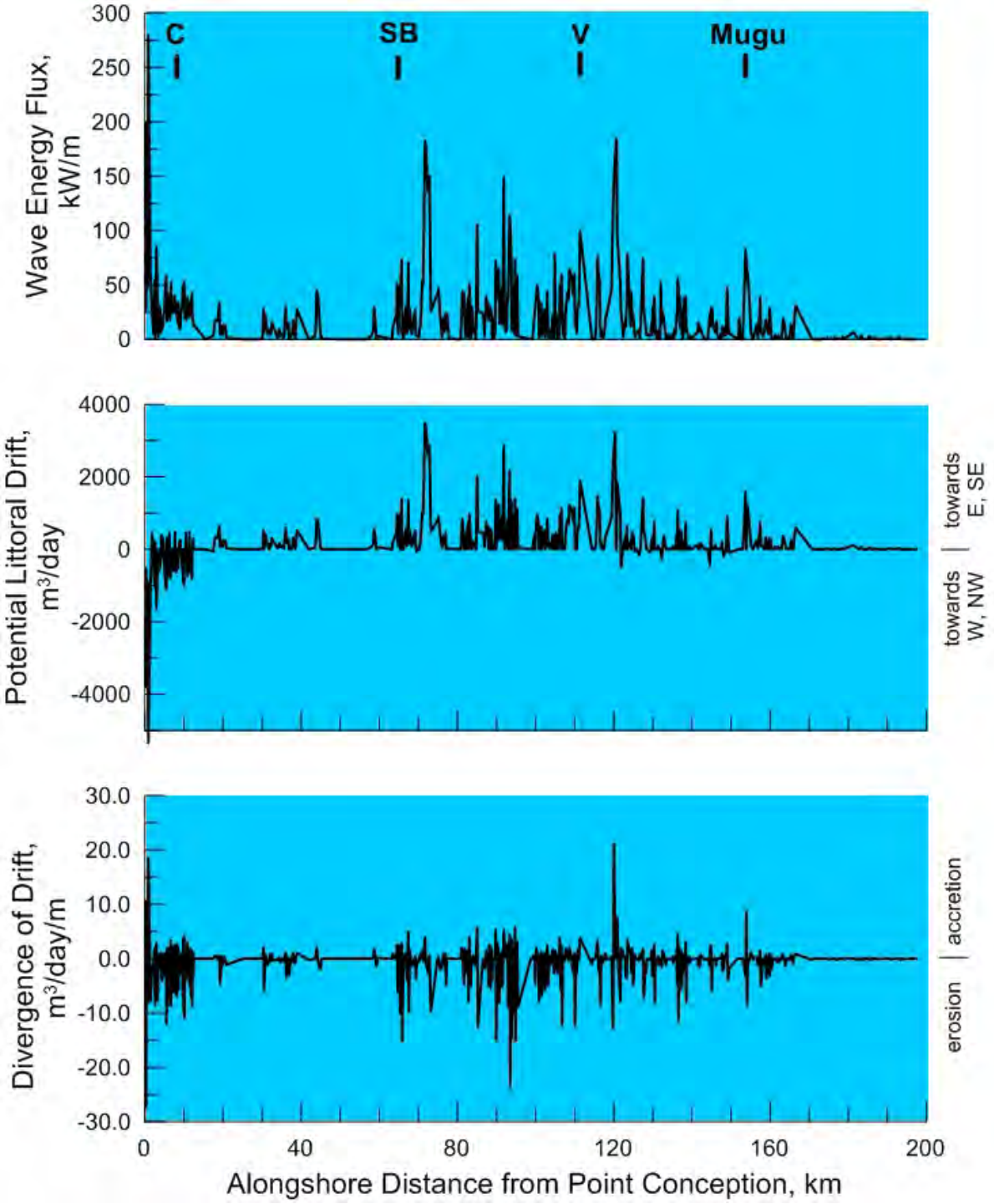

Figure 5.4: Littoral drift parameters at 1800 locations between Point Conception and the Mugu Submarine Canyon, calculated by the calibrated CEM and averaged over the 32-year period of record (1980-2012). Upper panel: wave energy flux. Middle panel: potential littoral drift (positive toward the east/southeast, negative toward the west/northwest). Lower panel: divergence of drift (positive values are depositional and negative values are erosional). Notations: C= Pt Conception; SB = Santa Barbara; V = Ventura; Mugu = Mugu Submarine Canyon.

With this insight into how the Santa Barbara site interacts with the large scale sediment transport system of the Santa Barbara Littoral Cell, we now turn to CEM solutions using the high-resolution inner grid along a 6 km reach in the nearfield of the study area, using the nested inner nearfield grid with 1 arc-second resolution with 200 coupled control cells between the Cliff House and the Santa Barbara Cemetery. In this inner grid, we perform the more complex calculations for sediment volume flux solutions to equation (3), constrained by the sediment budget of the Santa Barbara Littoral Cell. Divergence of drift (Figure 5.4c) is only one of 4 terms contributing to sediment volume flux solution. Two of the remaining 3 terms, the sediment source term *J(t)* and sink term *R(t)* are controlled by the large scale littoral cell processes while the remaining term, the surf diffusion, is a local mixing/diffusion process controlled by the magnitude of the incident wave energy and the along shore variation in wave height (Figure 5.4a).

Figure 5.5 gives the solution for the average daily sediment volume flux between Cliff House and the Santa Barbara Cemetery, averaged over the 32-year period of record (1980-2012). Here, the units for sediment volume flux are cubic meters of sand per day per meter of shoreline, with positive values indicating reaches of shoreline that are depositional and negative values indicate shorelines that are erosional. Inspection of Figure 5.5 reveals the sediment volume flux is very small, trending to zero over a 1600 m reach of coast west of Santa Barbara Harbor, between Cliff House and the middle of Leadbetter Beach, indicating this section of coast is stable with minimal erosional or depositional tendencies. Among other lesser factors, this condition arises because the divergence of drift is almost nil along this section of coast, i.e., the same amount of littoral drift that arrives at the western edge of this region near Cliff House also exits this region at the eastern edge in the central portion of Leadbetter Beach. Nowhere else is this stable condition found within 6 km to the west or to the east of the Santa Barbara Desalination facilities. However, the sediment volume flux trends positive along the eastern portions of Leadbetter Beach where a large sand fillet beach formation has built up against the western flank of the Santa Barbara Harbor breakwater (Figure 5.2). Over time, this depositional feature has become so pronounced and persistent that a parking lot with restroom and lifeguard facilities has been built on it (Figure 5.3). The positive sediment volume fluxes along the fillet beach section of Leadbetter Beach range between 0.2 and 0.6 m^3/day per meter of shoreline, which over time have proven to be sufficient to offset the erosion losses of even the worst El Nino storms and maintain a stable equilibrium fillet beach formation. When factored over the 32 year, period of record, the positive sediment volume fluxes along the fillet beach section of western Leadbetter Beach accumulate 4,675 m^3 of sand per meter of coast, on the order of 5 times the sediment volume in a critical mass envelope (Figures 5.6 and 5.7).

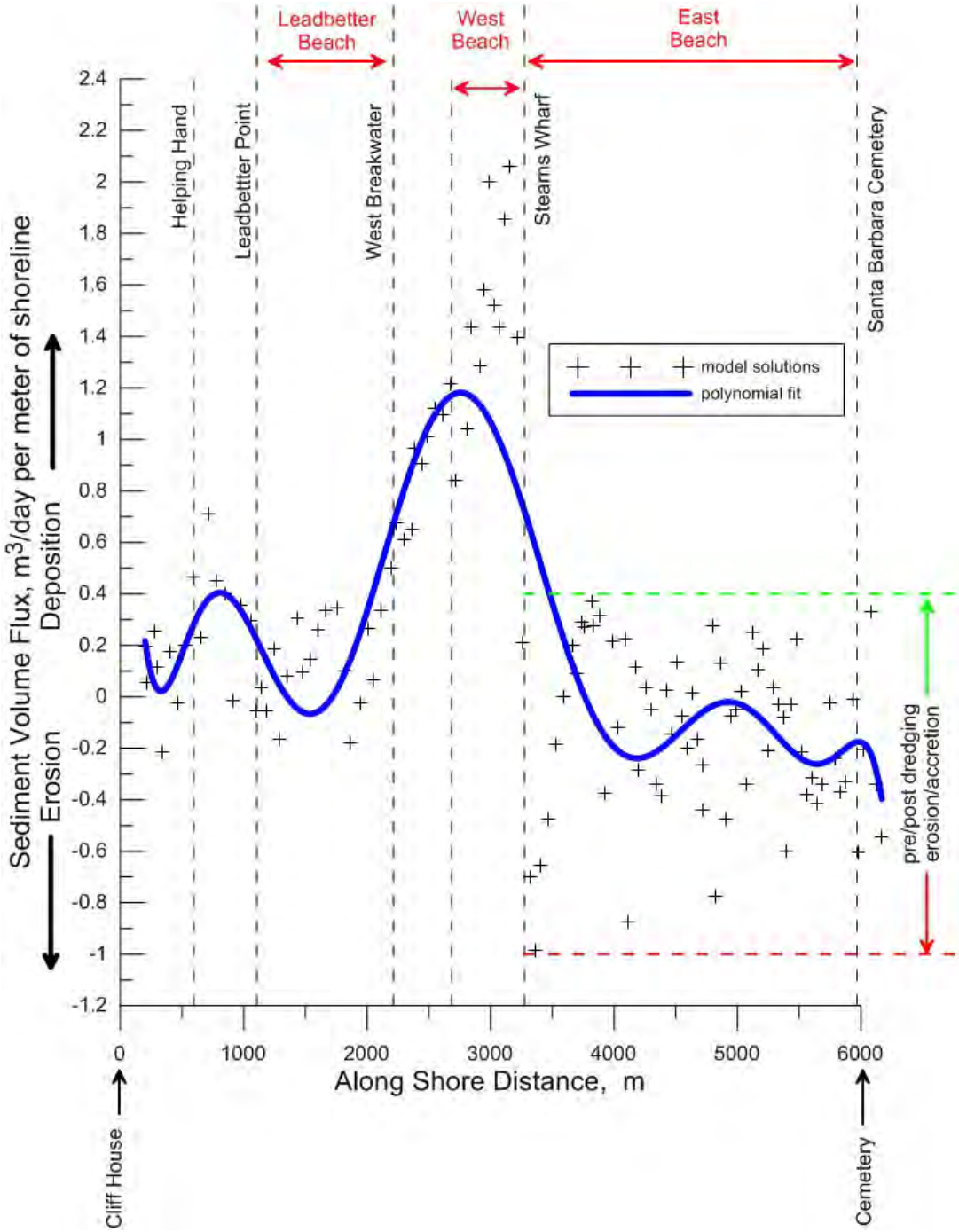

Figure 5.5: Daily sediment volume flux, *dq/dt,* calculated by the calibrated CEM from equation (3) and averaged over the 32-year period of record (1980-2012) for the reach between the Cliff House and the Santa Barbara Cemetery.

Based on 11,842 solutions over the 1980-2012 simulation period, the CEM calculates in Figure 5.6 that bottom profile perturbations caused by shoaling waves at the potential Leadbetter Beach intake site require a critical mass of at least 991 m^3/meter of shoreline; and that these profile perturbations cease seaward of the – 49.2 ft MSL (-15 m MSL) depth contour, referred to as *closure depth*. The critical mass determines the volume of sediment cover above a subsurface intake system that can be potentially eroded by the action of seasonal and episodic profile change or shoreline recession. The critical mass of sand on a beach is that required to maintain equilibrium beach shapes over a specified time, usually ranging from seasons to decades. Thus the positive sediment volume fluxes calculated in Figure 5.5 along the fillet beach section of Leadbetter Beach provide a cumulative safety factor of 5 for maintaining long term equilibrium of a bar-berm beach and shorerise profile at Leadbetter Beach. In addition, the critical mass envelope is relatively thin at the Leadbetter Beach site due to the stabilization action of the harbor breakwater. The variation in thickness of the critical mass envelope in Figure 5.7 (blue line) indicates that sand level variations due to beach profile changes can be as much as 11 ft. across the inner bar-berm beach profile at the Leadbetter Beach site, but no more than 5 ft across the shore rise profile off shore. Thus the engineered fill of a Beach Infiltration Gallery (BIG) at Leadbetter Beach would have to be placed at least 11 ft below existing grade; while the engineered fill of a Subsurface Infiltration Gallery (SIG) offshore at Leadbetter Beach would have to be placed at least 5 ft below existing grade. Construction of a BIG at the Leadbetter Beach site would require excavation of a 21 ft deep hole in the beach (down to about -9 ft to -15 ft. MSL), while a SIG would require excavating only a 10 ft. deep hole in the seabed if it were placed offshore at closure depth where existing grade is at -49.2 ft MSL and the bottom of the dredged SIG hole is at -59.2 ft.MSL. Because Leadbetter Beach is an exposed open-coast site subject to high-energy Gulf of Alaska and El Nino storm waves, construction of either a BIG or a SIG at this site would be subject to all of the challenges and difficulties discussed in Sections 2.1 and 2.2. However, a Neodren™ subsurface intake system is immune to these difficulties because it can be constructed entirely from landside launch points using horizontal directional drilling (HDD) techniques. The Leadbetter site appears quite favorable for the Neodren™ technology because its depositional environment assures adequate and continuous sediment cover comprised of predominately sand sized sediment, with on 14.36% silts and clays (Figure 4.13), thereby assuring excellent infiltration rates. Moreover, the Neodren™ drains would only have to be placed below the bottom of the critical mass envelope, which means at Leadbetter Beach the Neodren™ drains could be placed as shallow as 12 ft below existing grade in the bar-berm back beach section and as shallow as 6 ft below existing grade in the offshore shorerise portions of the bottom profile.

Proceeding along shore further east of Leadbetter Beach, the sediment volume fluxes in Figure 5.5 trend even more positive across the harbor entrance and West Beach, where $dq/dt \rightarrow$ 0.8 to 1.9 m^3/day per meter of shoreline, and averaging about 1.2 m^3/day/m of West Beach shoreline. This very high depositional flux indicates that significantly more littoral drift reaches the harbor entrance than flows around the harbor entrance, and results in 240,800 m^3/yr of sand entering the harbor and depositing in a massive shoal known as West Beach. The beach and bottom profiles of the West Beach

shoal in Figure 4.18 are largely controlled by dredge cuts, and are therefore not natural equilibrium formations. However, it is possible to apply the critical mass envelope concept to the envelope of these dredge cuts and deduce an equivalent critical mass for West Beach of 407 m^3/meter of shoreline. The inner 400 ft of the West Beach profiles are analogous to a bar-berm profile section in which the critical mass envelope is 16 ft thick. The outer 800 ft. of the West Beach profiles in Figure 4.18 exhibit wave-formed shorerise expressions for which the critical mass envelope is 2 ft to 4 ft thick. Because West Beach is sheltered by the harbor breakwater, it is an ideal site for a Beach Infiltration Gallery (BIG), which could have to have its engineered fill placed below – 8ft. MSL in the inner 400 ft. of beach profile; and requiring excavation of a hole to -18 ft. MSL to place the infiltration and branch piping. Construction of a SIG in the outer 800 ft. section of West Beach would be problematic because construction activities would interfere with harbor navigation. West Beach is also not a particularly favorable site for the Neodren™ subsurface intake system because the quiet harbor waters allow significant fractions of fine grained sediments to settle on the West Beach shoals. The percentage of fines in the West Beach sediments is 26.72 % silts and clays (Figure 4.13), which is sub-optimal for infiltration rates of Neodren™ drains; but nonetheless workable, although it would require longer or greater numbers of Neodren™ drains for a given source water production rate.

East of Santa Barbara Harbor, between Stearns Wharf and the Cemetery, Figure 5.5 indicates that sediment volume fluxes are highly variable, with $-0.1 \leq dq/dt \leq +0.4$, but generally trending negative. The high variability is due to dredge disposal activities which use East Beach as a receiver beach, but the negative trend is due to the wave refraction and sediment trapping effect which the harbor breakwater and entrance exert of the potential littoral drift. With the exception of the brief periods during which an average of 315,000 yds^3/yr are placed on East Beach during dredging, more sediment leaves the east end of this section of coast due to eastward flowing littoral drift than arrives at the west end as a consequence of a small fraction of littoral drift by-passing the harbor entrance. Therefore East Beach is intrinsically erosional, and the resident sediment volume there is sustained only by the action of harbor dredging, which persists only through repeated new funding authorizations by the US Congress for the indefinite future. The sediment volume fluxes along the 2,694 m section of East Beach range average -0.25 m^3/day per meter of shoreline, which over time are marginally balanced by the placement of dredge fill on East Beach by Santa Barbara Harbor dredging. When factored over the 32 year, period of record, this negative sediment volume flux accumulates to 2,862 m^3 of sand per meter of coast, about twice the sediment volume in a critical mass envelope (Figures 5.7 and 5.8). Therefore this beach is unstable and only exists because of regular beach nourishment. East Beach will disappear if Congress ceases funding for maintenance dredging of Santa Barbara Harbor; or this beach will become seriously eroded if Congress ever interrupts or delays funding for such dredging. However, under the present status quo, supported by Congressional funds, the CEM calculates in Figure 5.8 that bottom profile perturbations caused by shoaling waves at the potential East Beach intake site provide a critical mass of at least 1,345 m^3/meter of shoreline; and that these profile perturbations cease seaward of the – 51.5 ft MSL (-15.7 m MSL) depth contour at a distance of about 8,500 ft. offshore. Therefore the offshore

open ocean intake site is beyond closure depth (Figures 1.1 and 1.2), and immune to potential scour or burial from wave-induced bottom profile changes. (This is not true for tsunami induced bottom profile changes, see Section 6.2). The variation in thickness of the East Beach critical mass envelope in Figure 5.7 (red line) indicates that sand level variations due to beach profile changes can be as much as 9.5 ft. across the inner bar-berm beach profile at the East Beach site, and no more than 8 ft across the shore rise profile off shore. Thus the engineered fill of a Beach Infiltration Gallery (BIG) at East Beach would have to be placed at least 9.5 ft below existing grade; while the engineered fill of a Subsurface Infiltration Gallery (SIG) offshore at East Beach should be placed at or beyond closure depth. Construction of a BIG at the East Beach site would require excavation of a 19.5 ft deep hole in the beach (down to about -7.5 ft to -13.5 ft. MSL), while a SIG would require excavating only a 10 ft. deep hole in the seabed if it were placed offshore at closure depth where existing grade is at -51.5 ft MSL and the bottom of the dredged SIG hole is at -61.5 ft MSL. Like Leadbetter Beach, East Beach is also an exposed open-coast site subject to high-energy Gulf of Alaska and El Nino storm waves, and construction of either a BIG or a SIG at this site would be subject to all of the challenges and difficulties discussed in Sections 2.1 and 2.2. A Neodren™ subsurface intake system is immune to these difficulties because it can be constructed entirely from landside launch points using horizontal directional drilling (HDD) techniques. The East Beach site appears workable for the Neodren™ technology but not ideal, because its sediment cover is comprised of between 18% and 35% silts and clays (Figures 4.15 and 4.16); but again reduced infiltration rates with this high a percentage of silts and clays can be compensated with longer or more numbers of Neodren™ drains. Regardless, the Neodren™ drains would only have to be placed below the bottom of the critical mass envelope, which means at East Beach the Neodren™ drains could be placed as shallow as 10 ft below existing grade in the bar-berm back beach section and as shallow as 8.5 ft below existing grade in the offshore shorerise portions of the bottom profile. Based on the isopach map in Figure 2.14, it does not appear that sufficient sediment cover exists offshore of East Beach to place the Neodren™ drains much more than 8.5 ft below existing grade, giving a safety margin of only about 0.5 ft.

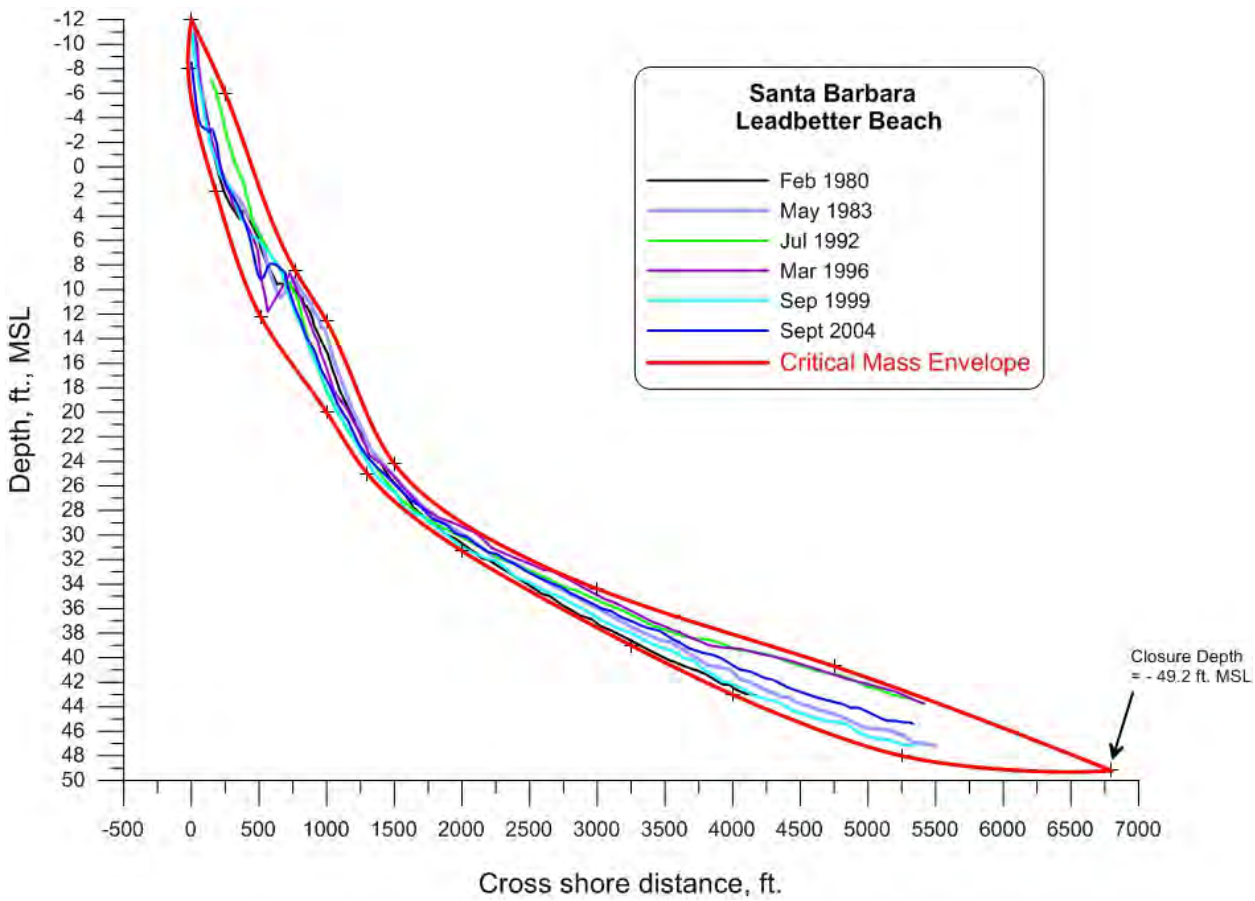

Figure 5.6: Critical mass envelope at historic survey range, Leadbetter Beach, calculated by the calibrated CEM sediment budget based on the 32-year period of record CDIP monitored waves, (cf. Figure 4.3). Beach surveys by the U.S. Army Corps of Engineers, Los Angeles District, between February 1980 and September 2004. Data from USACE Santa Barbara Harbor Dredge Permits, (1980 - 2008). Critical mass volume = 991 m^3 per meter of shoreline calculated from equation (16).

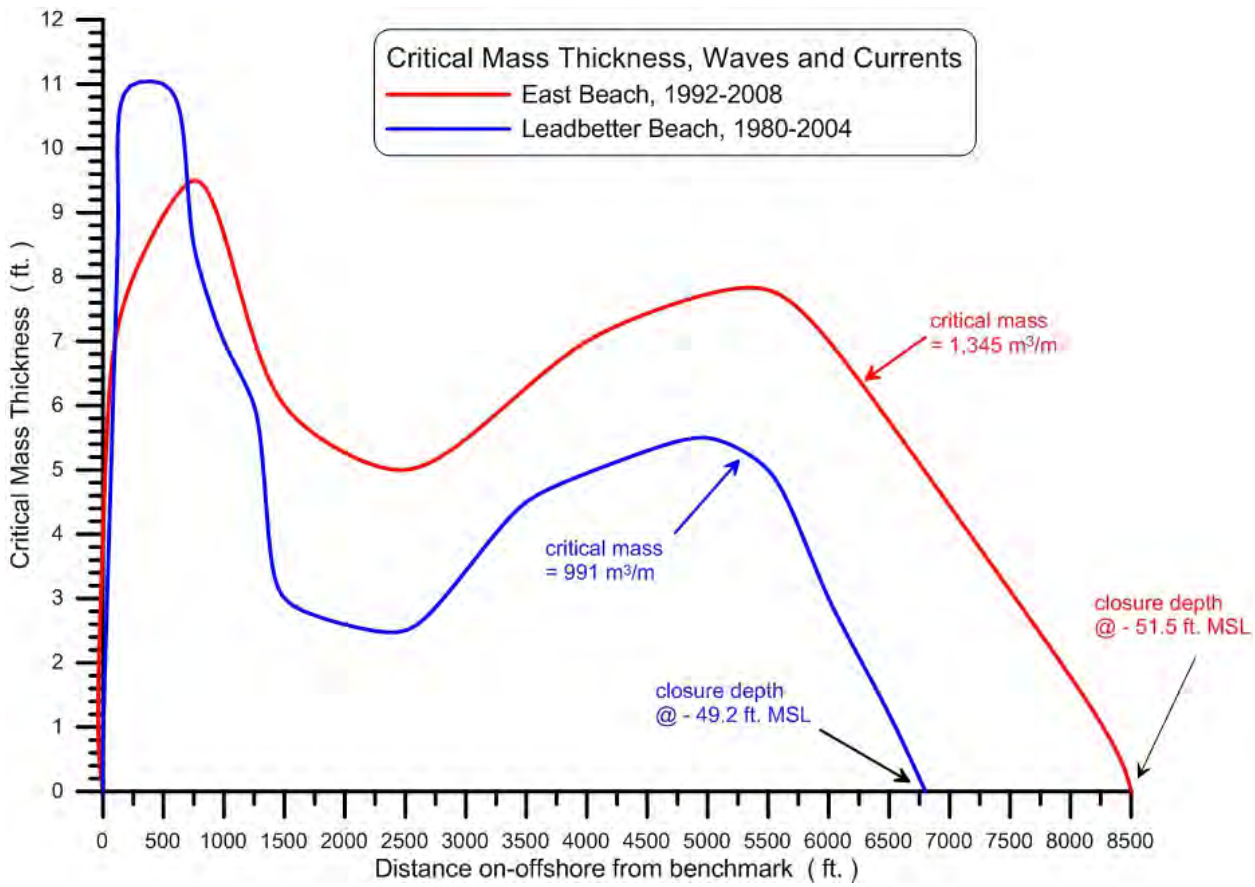

Figure 5.7: Thickness of critical mass envelope at historic survey ranges, Leadbetter Beach and East Beach, Santa Barbara Harbor, calculated by the calibrated CEM sediment budget based on the 32-year period of record CDIP monitored waves, (cf. Figure 4.3).

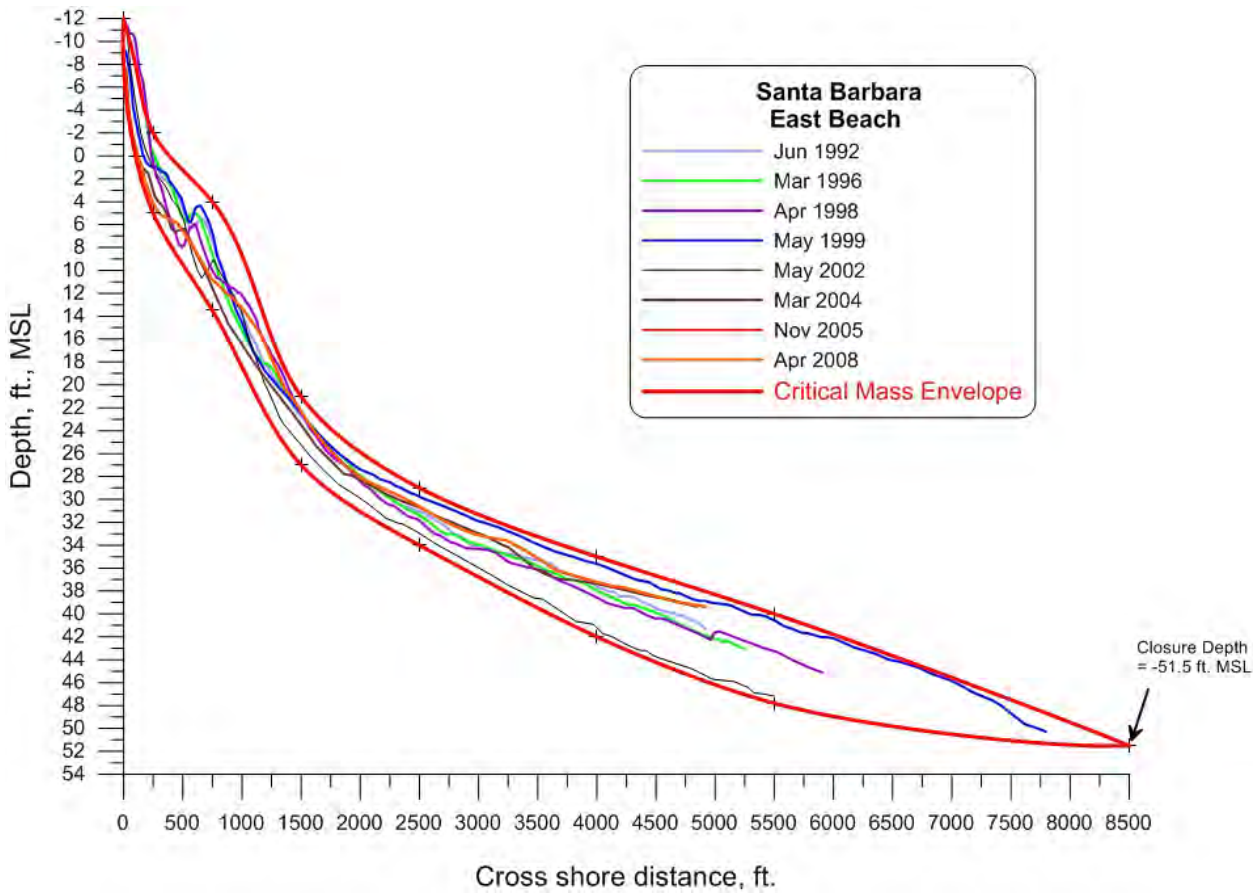

Figure 5.8: Critical mass envelope at historic survey range, East Beach, calculated by the calibrated CEM sediment budget based on the 32-year period of record CDIP monitored waves, (cf. Figure 4.3). Beach surveys by the U.S. Army Corps of Engineers, Los Angeles District, between February 1980 and September 2004. Data from USACE Santa Barbara Harbor Dredge Permits, (1980 - 2008). Critical mass volume = 1,345 m^3 per meter of shoreline calculated from equation (16).

6) Wave Run-up and Tsunami Hazard Analysis :

Wave run-up, and overtopping were analyzed at the shore-side facilities associated with the study area assuming present conditions and two future scenarios including sea level rise. These facilities included: the Charles Meyer Desalination Plant 525, Yanonali Ave (elevation + 10 ft NGVD), a pump station / chemical area at 420 Quinientos St (elevation + 8 ft NGVD), and a pair of collector well sites at 401 E. Yanonali Ave (elevation + 12 ft NGVD) and 103 S. Calle Cesar Chavez (Elevation +10 ft. NGVD). The analysis was based upon the 32- year CDIP wave record in Figure 4.10 and the tidal hydroperiod function in Figure 4.11 which provided inputs to the onshore analysis of setup, erosion, run-up, and overtopping.

To estimate sea level rise in the year 2065, which corresponds with a 50-year SLR projection per CAT-OPC guidance, estimates of 7 inches (low estimate) and 35 inches (high estimate) were projected based upon equations B-3 and B-4 in the 2013 California Coastal Commission's Draft Sea Level Rise Guidance document. For the low sea level rise estimate, 7 inches was added to each hourly SWL value prior to being paired with the wave conditions for that time step; these inputs were then used in the onshore analysis of setup, run-up, and overtopping. Similarly, for the high sea level rise estimate, 35 inches was added to each hourly SWL value prior to being paired with the wave conditions for that time step; these inputs were then used in the onshore analysis of setup, run-up, and overtopping.

For each of the three scenarios (present conditions and low and high sea level rise estimates for the year 2065), hourly TWL results from the thirty two year wave records were reviewed, and the annual maximum was selected for each year. The annual maxima were then plotted, and the 1-percent-annual-chance TWL was calculated statistically from these results. The 1-percent-annual-chance TWL for each facility scenario are summarized in Tables 6.1-6.4. For all scenarios, the fit of the resulting cumulative distribution function to the annual maxima were evaluated and the maximum likelihood solution was selected.

Tables 6.1-6.4 also includes the mean run-up slope calculated from the run-up slopes associated with the 32 annual maximum TWLs. The vertical wall reduction factor, γ_v, for the backshore revetment at East Beach was computed and the TAW run-up method was used in the annual TWL maxima output. The reduction factors for berms, and porosity were set to 1 for all scenarios, and the angle of wave attack reduction factor changed with each time step, based on the refracted wave direction along East Beach.

As the SWL increases across the three scenarios, the SWL and DWL2% intersect the beach profile at higher elevations. For the majority of the present day conditions annual maxima, the DWL2% intersects the beach below the toe of the East Beach revetment fronting East Cabrillo Ave. In these cases, the run-up slope calculation includes the milder foreshore slope. For annual maxima where the DWL2% exceeds the structure toe, the run-up slope does not include the milder foreshore and is restricted to the steeper slope on the revetment itself. As the SWL and DWL2% increase with sea

level rise, an increasing number of annual maxima DWL2% occur. This, in turn, results in steeper run-up slopes, larger wave heights, and higher wave run-up and total water level values.

Results from the erosion analysis are based on the elliptic cycloid formulation of the CEM and the sediment volume flux calculation in Figure 5.5. Erosion was limited to the beach berm and had little effect on the TWL results in the present conditions scenario, only leading to a slightly milder run-up slope on the present condition final eroded profile. In the 2065 high and low sea level rise estimate scenarios, the run-up area is increasingly located on the back beach revetment and less on the foreshore beach slope. For these scenarios, erosion did not impact the TWL results across the intact, Most Likely Winter Profile, and final eroded profiles. The revetment geometry remains constant across the scenarios, accounting for the constant crest height and vertical slope reduction factor, γ_v .

Table 6.1: One-Percent-Annual-Chance TWLs and Mean Run-up Slopes: Charles MeyerDesalination Plant, 525, Yanonali Ave (elevation + 10 ft NGVD).

Scenario	**Mean Run-up Slope**	East Beach γ_v	dq/dt **Beach Erosion Rate m³/m/day**	**Site Elevation (ft, NGVD)**	**1% Annual Chance TWL (ft, NGVD)**
Present	0.182	0.71	-1.0	10.0	7.1
2065, low	0.388	0.71	-1.0	10.0	8.4
2065, high	0.860	0.71	-1.0	10.0	9.8

Table 6.2: One-Percent-Annual-Chance TWLs and Mean Run-up Slopes: Pump Station, 420 Quinientos St (elevation + 8 ft NGVD).

Scenario	**Mean Run-up Slope**	East Beach γ_v	dq/dt **Beach Erosion Rate m³/m/day**	**Site Elevation (ft, NGVD)**	**1% Annual Chance TWL (ft, NGVD)**
Present	0.250	0.71	-1.0	8.0	8.0
2065, low	0.548	0.71	-1.0	8.0	9.5
2065, high	1.178	0.71	-1.0	8.0	11.1

Table 6.3: One-Percent-Annual-Chance TWLs and Mean Run-up Slopes, Collector Well Site, 401 E. Yanonali Ave (elevation + 12 ft NGVD), or

Scenario	Mean Run-up Slope	East Beach γ_v	dq/dt Beach Erosion Rate $m^3/m/day$	Site Elevation (ft, NGVD)	1% Annual Chance TWL (ft, NGVD)
Present	0.136	0.71	-1.0	12.0	6.5
2065, low	0.322	0.71	-1.0	12.0	7.8
2065, high	0.715	0.71	-1.0	12.0	9.1

Table 6.4: One-Percent-Annual-Chance TWLs and Mean Run-up Slopes, Collector Well Site: 103 S. Calle Cesar Chavez (Elevation +10 ft. NGVD)

Scenario	Mean Run-up Slope	East Beach γ_v	dq/dt Beach Erosion Rate $m^3/m/day$	Site Elevation (ft, NGVD)	1% Annual Chance TWL (ft, NGVD)
Present	0.284	0.71	-1.0	10.0	8.4
2065, low	0.622	0.71	-1.0	10.0	10.1
2065, high	1.338	0.71	-1.0	10.0	11.7

Inspection of Tables 6.1-6.4 reveals that the only shoreside facilities threatened by flooding from wave run-up are the pump station at 420 Quinientos St (Table 6.2) and one of the collector well sites a 103 S. Calle Cesar Chavez (Table 6.4), and these two facilities are only threatened at future sea levels. The most serious of threat during future sea level rise is at the pump station site where1% Annual Chance TWL can reach as high as 11.1 ft. NGVD, or 3.1 ft. above the site elevation. To understand further the persistence of potential flooding at the pump station site, the number of overtopping events in the 32 annual maxima was recorded. The 32 annual maxima for present day conditions contain no overtopping events, while the and the 2065 low sea level rise estimate contains 2 overtopping events. The 2065 high sea level rise estimate results contain wave overtopping in 24 of the 32 annual maxima. Although overtopping is currently infrequent, the 2065 high estimate of 35 inches of sea level rise is found here to result in wave overtopping of the pump station site becoming an annual occurrence. A summary of the number of overtopping events and the range of overtopping rates from the 32 annual maxima for each pump station scenario are presented in Table 6.5. Severity of wave overtopping events has been examined by quantifying the number of overtopping events which exceed certain overtopping rate thresholds.

Table 6.5: Summary of Overtopping Events at Pump Station, 420 Quinientos St (elevation + 8 ft NGVD).

Scenario	Number of Overtopping Events	Range of Overtopping Rates, *Q'* (cfs/ft)	Number of Events where Q' > 1.766 cfs/ft	Number of Events where Q'> 0.353 cfs/ft	Number of Events where Q' > 0.004 cfs/ft
Present	0	0.04-2.9	0	0	0
2065, low	2	0.20-2.8	1	1	2
2065, high	24	0.005-3.5	2	8	24

The maximum overtopping rate calculated for the 2065, low scenario is slightly more than the present conditions maximum overtopping rate due to a small difference in the maximum event selection. Given the difficulty in accurately estimating overtopping rates and volumes, the overtopping rates calculated for these two scenarios should be considered equal for all practical purposes.

Table-6.2 overtopping rate thresholds were compared to the EurOtop Wave Overtopping of Sea Defences and Related Structures: Assessment Manual (EurOtop Manual) (Pullen et al., 2007) which discusses physical implications of overtopping on both the structure and users of the structure during different conditions. According to field testing, it is expected that the structure crest and rear slopes should remain undamaged and resist erosion under all conditions of overtopping calculated for this highway revetment. However, the EurOtop Manual advises limiting casual pedestrian activity in the vicinity of the seawall under conditions when overtopping exceeds 0.004 cfs and trained staff when overtopping exceeds 0.353 cfs. Vehicular traffic should be restricted when overtopping exceeds 1.766 cfs. At the pump station, overtopping during major coastal storms will exceed safe pedestrian limits and measures to evacuate people and vehicles from this areas should be employed under severe conditions.

The pump station wave overtopping rates associated with the 1-percent-annual-chance TWLs shown in Table 6.2 are presented in Table 6.6. For each scenario, the maximum overtopping potential based on a single hourly time step which uses the maximum DWL2% is reported. For comparison, the overtopping rate expected over a full tidal cycle during a peak storm event which uses the average DWL2% are also provided. Similar to the results presented in Table 6.2, the severity of overtopping increases considerably for the 35 inch sea level rise estimate for the year 2065. Wave overtopping is not a constant discharge but rather a process which varies in both time and volume. Higher waves will pulse greater volumes of water into the pump station site while lower waves may not push any volume over the crest. The average overtopping volumes calculated are shown in Table 6.3. For the high 2065 conditions, approximately 3.5 cubic feet of water per second per linear foot is expected to overtop the pump station site for each wave during the highest tidal stages of the 1-percent-annual-chance storm.

Table 6.6: Overtopping Rates Pump Station Corresponding to the 1% TWLs

Scenario	Maximum Overtopping Rate based on maximum DWL2%, *q* (cfs/ft)	Overtopping Rate over Full Tidal Cycle based on mean DWL2%, *q* (cfs/ft)
Present	0	0
2065, low	2.8	0.1
2065, high	3.5	0.7

6.1 Tsunami Run-up and Inundation:

Tsunami induced erosion, run-up, and inundation were analyzed for the East Beach bottom profile and shore-side facilities associated with the study's subsurface intake infrastructure assuming present conditions and two future scenarios including sea level rise. The tsunami scenario is based on a 2m high solitary wave approaching East Beach from 165 degrees true, as could be anticipated for a catastrophic tsunami event arising from a majpor landside on the East side of San Clemente Island. The local refraction/diffraction pattern from the solitary wave is calculated in Figure 6.1. Inspection of Figure 6.1 reveals the tsunami wave height begins to increase at 50 m of water depth due to shoaling and reaches about 6m of height before breaking along the shores of East 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. The critical mass thickness computed by the CEM in Figure 6.2 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 6.2 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. Only a Neodren ™ intake system would be immune to tsunami seabed erosion. Tsunami run-up and TWL inundation calculations in Tables 6.7-6.10 also indicate that every shore facility associated with the study's subsurface intake infrastructure would also suffer serious degrees of overtopping. These findings are consistent with the FEMA tsunami flood map in Figure 6.3 which show that all of the East Beach corridor extending several miles inland will be inundate by a shoaling tsunami solitary wave.

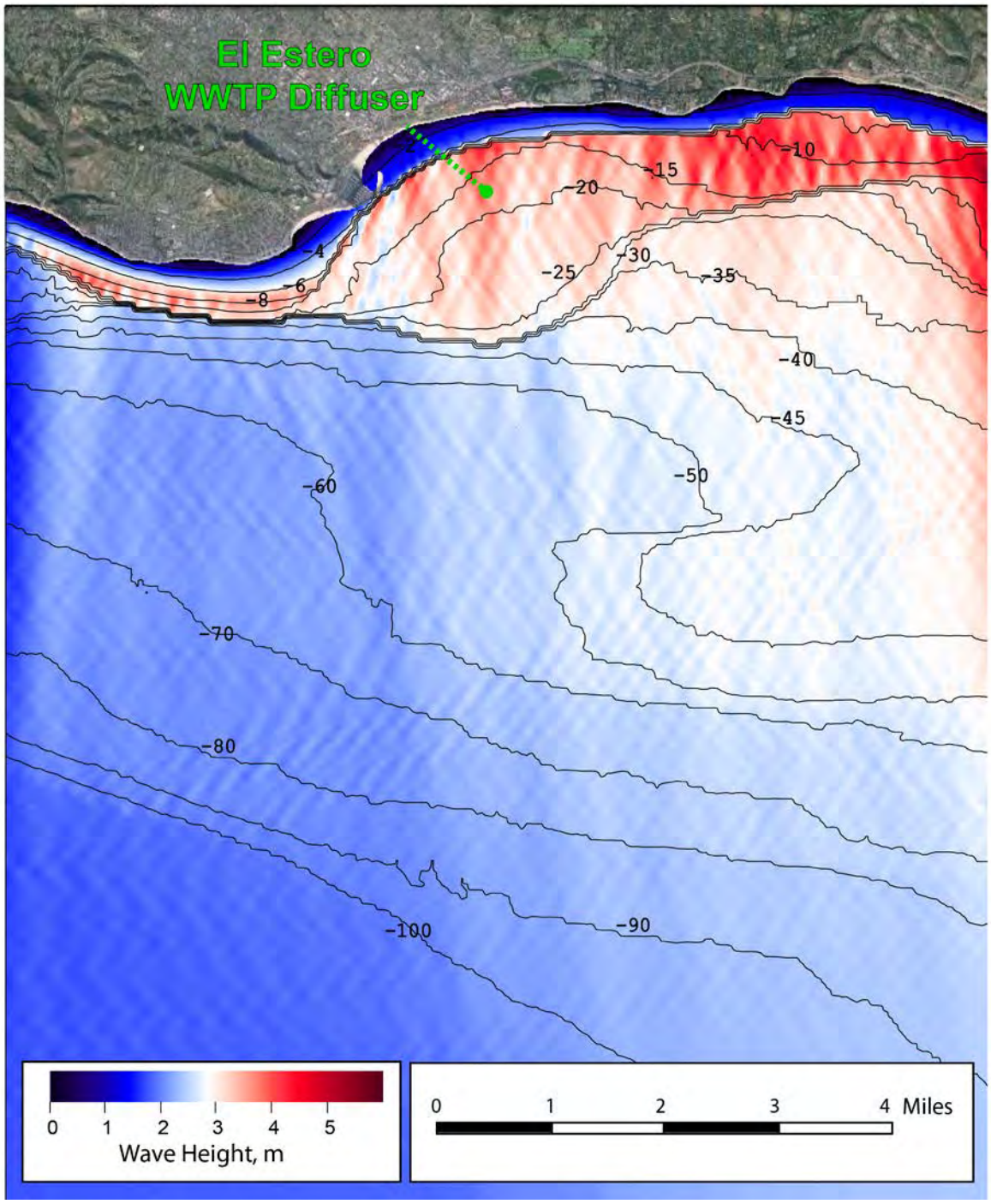

Figure 6.1: High resolution refraction/diffraction computation for a 2m high solitary tsunami wave approaching East Beach from 165 degrees true.

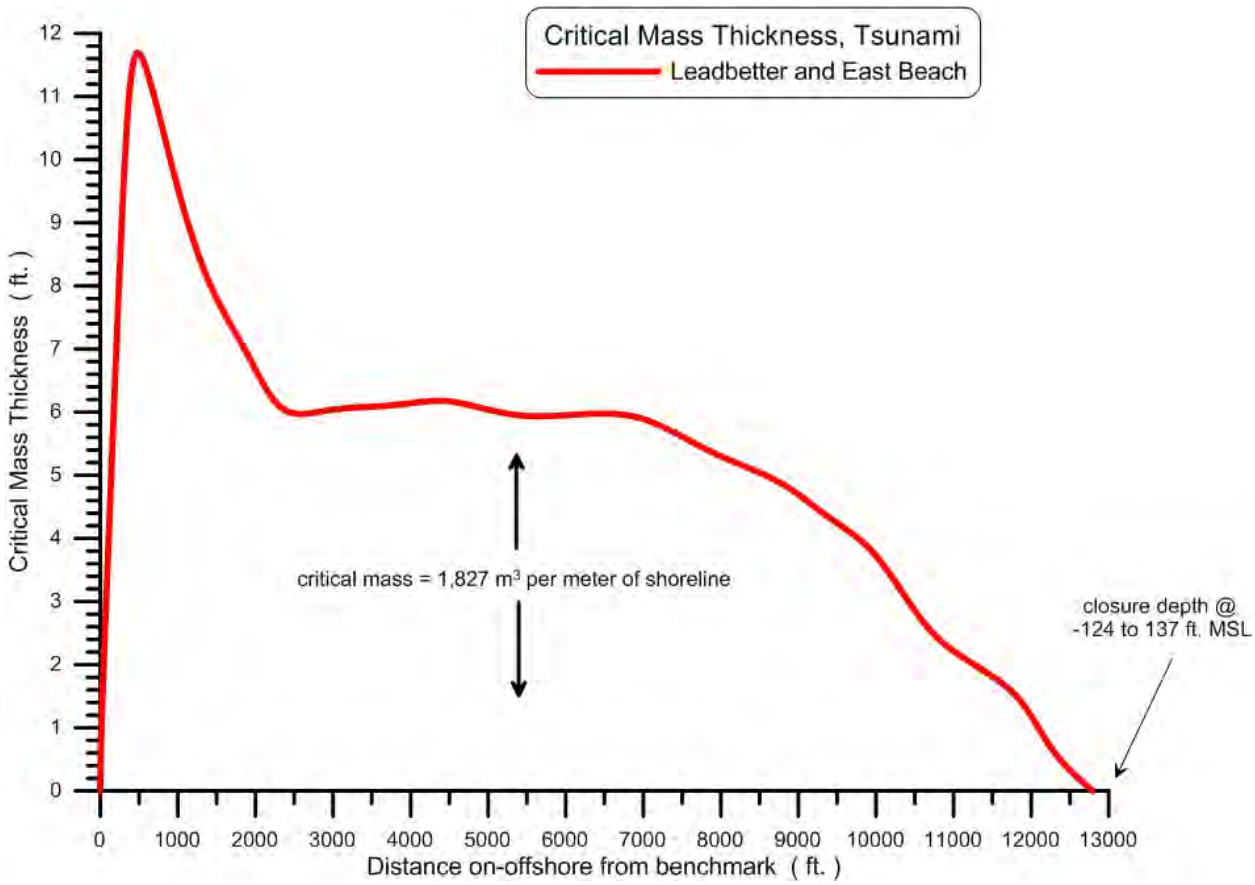

Figure 6.2: Thickness of critical mass envelope at historic survey ranges East Beach, Santa Barbara Harbor, calculated by the calibrated CEM sediment budget based a 2m high solitary tsunami wave approaching East Beach from 165 degrees true. Closure depth = -124 to -137 ft. MSL; critical mass volume = 1,827 m^3 per meter of shoreline.

Table 6.7: Tsunami TWLs and Mean Run-up Slopes: Charles Meyer Desalination Plant, 525, Yanonali Ave (elevation + 10 ft NGVD).

Scenario	Mean Run-up Slope	East Beach γ_v	dq/dt Beach Erosion Rate $m^3/m/day$	Site Elevation (ft, NGVD)	Tsunami TWL (ft, NGVD)
Present	0.182	0.71	-1827	10.0	13.6
2065, low	0.388	0.71	-1827	10.0	16.1
2065, high	0.860	0.71	-1827	10.0	18.8

Table 6.8: Tsunami TWLs and Mean Run-up Slopes: Pump Station, 420 Quinientos St (elevation + 8 ft NGVD).

Scenario	Mean Run-up Slope	East Beach γ_v	dq/dt Beach Erosion Rate $m^3/m/day$	Site Elevation (ft, NGVD)	Tsunami TWL (ft, NGVD)
Present	0.250	0.71	-1827	8.0	15.4
2065, low	0.548	0.71	-1827	8.0	18.2
2065, high	1.178	0.71	-1827	8.0	21.3

Table 6.9: Tsunami TWLs and Mean Run-up Slopes, Collector Well Site, 401 E. Yanonali Ave (elevation + 12 ft NGVD), or

Scenario	Mean Run-up Slope	East Beach γ_v	dq/dt Beach Erosion Rate $m^3/m/day$	Site Elevation (ft, NGVD)	Tsunami TWL (ft, NGVD)
Present	0.136	0.71	-1827	12.0	12.5
2065, low	0.322	0.71	-1827	12.0	15.0
2065, high	0.715	0.71	-1827	12.0	17.5

Table 6.10: Tsunami TWLs and Mean Run-up Slopes, Collector Well Site: 103 S. Calle Cesar Chavez (Elevation +10 ft. NGVD)

Scenario	Mean Run-up Slope	East Beach γ_v	dq/dt Beach Erosion Rate $m^3/m/day$	Site Elevation (ft, NGVD)	Tsunami TWL (ft, NGVD)
Present	0.284	0.71	-1827	10.0	16.1
2065, low	0.622	0.71	-1827	10.0	19.4
2065, high	1.338	0.71	-1827	10.0	22.5

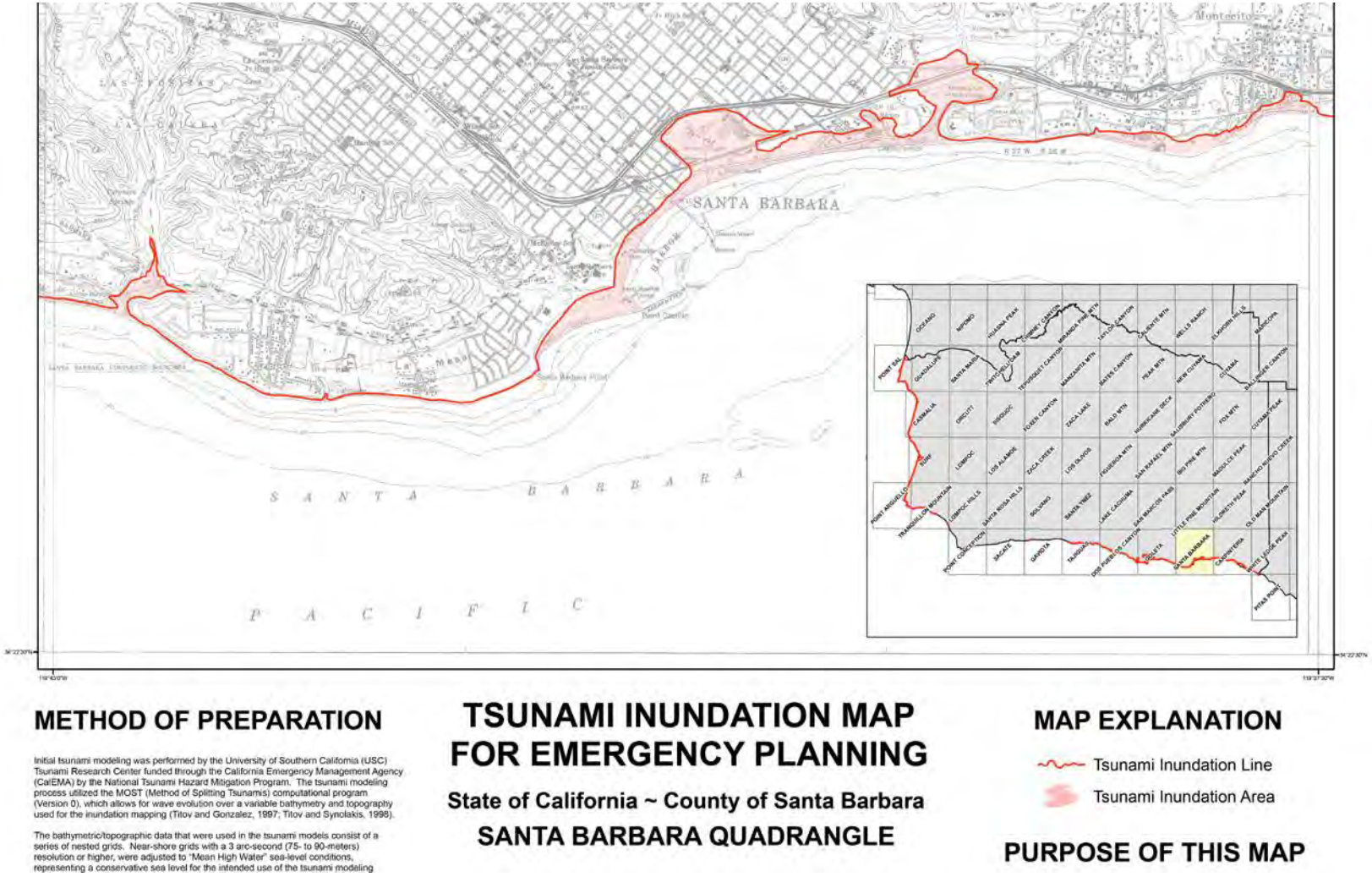

Figure 6.3: FEMA tsunami inundation map

7) Conclusions:

Subsurface intake options for the City of Santa Barbara's Subsurface Desalination Intake Feasibility Study are reviewed and the site requirements for each alternative evaluated by performing a sediment transport and coastal hazards evaluation. A sediment budget analysis was performed on the Santa Barbara Littoral Cell using the Coastal Evolution Model developed at the Scripps Institution of Oceanography. The sediment budget analysis provided critical far field inputs to a near field seafloor stability and coastal hazards analysis of the site specific conditions and infrastructure. The viability of each subsurface intake option was evaluated with respect to the results of the seafloor stability and erosion analysis; while the coastal hazards analysis evaluated vulnerabilities of all shore-side and offshore structures associated with the intake alternatives. It was concluded that the West Beach intake site is well suited for a beach infiltration gallery (BIG) but is not optimal for a subsurface infiltration gallery (SIG) or for Neodren horizontal well technology. The Leadbetter Beach intake site was found to be feasible for SIG or BIG type intake systems but both are problematic to construct at this site due to exposure to high energy wave climate, The Neodren intake technology was found to be the best option for the Leadbetter Beach site and the only viable option for East Beach. None of the shore-side facilities will be significantly flooded by wave run up at present sea levels, although future sea level scenarios will cause flooding from wave run up at the shore-side pump station sites. All shore-side facilities will be inundated by tsunami and only Neodren will be unaffected offshore by tsunami erosion. All conclusions must be considered within the context of this report’s scope (i.e., sediment transport and coastal hazards evaluation only). Overall feasibility of subsurface intake alternatives must consider other technical factors and will be evaluated by others. These additional technical factors should include, but not limited to hydrogeology, constructability, reliable performance history, etc.

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