J Street Drain / Ormond Beach Lagoon
Coastal Engineering Report

Prepared for:

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This report incorporates revised and supplemental analysis by reference to the Coastal Engineering Report Addendum, dated November 2008.

Table of Contents

List of Figures

List of Tables

1. Introduction

HDR is conducting a Phase 1 preliminary design for Ventura County Watershed Protection District (VCWPD) to reconstruct approximately 2.2 miles of J Street Drain in the City of Oxnard, California. The combined flows of J Street Drain, Hueneme Drain, and the Oxnard Industrial Drain (OID) currently discharge into Ormond Beach Lagoon (lagoon). Following rainfall events, water levels in the lagoon rise and occasionally overtop the beach, forming a channel or breach that drains the lagoon and temporarily connects it to the Pacific Ocean. The overall goals of the project are to (1) reduce local flooding within the City of Oxnard by increasing the capacity of J Street Drain and reducing backwater effects in the lagoon during significant rain storms, and (2) minimize adverse ecological impacts to the lagoon. Because mechanical breaching benefits the first goal but is considered a detriment to the second, alternative management options and designs were investigated.

1.1. Purpose and Use

This report is intended primarily to document and inform the design of improvements to J Street Drain; it is not intended to provide a comprehensive analysis of the many hydrologic, hydraulic, and coastal processes historically and currently affecting the lagoon and its resources.

It is recognized that the lagoon is a unique and dynamic ecological resource and that human activities continue to influence its development and evolution. J Street Drain is an important source of freshwater for the lagoon, and thus modifications to the drain and its connection to the Pacific Ocean could cause beneficial and/or negative ecological impacts. Therefore, proposed modifications to J Street Drain were investigated for potential impacts to the lagoon dynamics and key physical processes. This report qualitatively describes historical lagoon morphology, key physical processes affecting lagoon breaching and closing, project specific data collection, and anticipated impacts to lagoon processes and morphology associated with proposed modifications to J Street Drain.

The Environmental Impact Report to be prepared by HDR will also utilize the findings of this report to frame the existing environment and assess any impacts related to physical changes documented herein.

1.2. Project Approach

Readily available site data and relevant previous studies were first gathered and reviewed. A reconnaissance site visit was then conducted and key physical processes were evaluated based on existing data and information. Following review of the existing data, supplemental field data were collected to fill critical data gaps. Analysis focused primarily on developing and modeling outlet alternatives to achieve the project goals as stated above.

1.3. Report Organization

Section 2 presents conclusions and key findings of the study. Previous relevant work by others is summarized in Section 3. Physical processes affecting the lagoon are summarized in Section 4. More detailed information on physical processes can be found in Appendix A. Supplemental field data collected for the project and field data provided by others are discussed in Section 5. Section 6 includes the analysis of various outlet alternatives followed by a discussion of potential project implications and design recommendations in Section 7. References are listed in Section 8. Appendix B summarizes the numerical modeling performed.

1.4. Project Vertical Datum

The project vertical datum is the National Geodetic Vertical Datum of 1929 (NGVD). However, the analysis presented herein references the North American Vertical Datum of 1988 (NAVD88) for consistency with more recent available data. The approximate correlation between the two datums is 0.0 ft NAVD ≈ -2.4 ft NGVD. Correlated values for key results are provided in the report conclusions.

2. Conclusions

The following conclusions are provided based on interpretation of readily available information, collection of supplemental field data, analysis of key physical process, and in consideration of the overall project goals.

1. Existing site conditions at Ormond Beach differ substantially from historic conditions. The project site historically contained a large wetland complex and coastal lagoons fronted by a beach. One lagoon was located north of J Street and fed by what is now the Hueneme Drain; the second was located south of J Street and fed by what is now the OID. Historical lagoons have been mostly filled and/or their drainages modified. Only a small remnant of the historical lagoon and wetlands at OID remains.

2. The existing lagoon began to develop following construction of J Street Drain, the Hueneme Pump Station in 1961, re-routing of the OID, and bypassing of littoral sediments initiated by the port of Port Hueneme in the early 1960s. J Street Drain and OID originally discharged separately to the ocean; as storm water ponded behind the growing dunes and beach, the two drains became connected in the late 1980s and began to function as a unit with a single outlet to the ocean during breaching events.

3. The geometry of the existing lagoon is naturally dynamic, varying considerably from season to season and year to year. Breaching during the design storms considered herein is dominated by the flows from OID and, more recently, breaches have tended to form near the OID and migrate south in the direction of net longshore sediment transport.

4. If sand bypassing at Port Hueneme were to cease, more rapid erosion at Ormond Beach would occur. Beach erosion and associated landward shoreline migration could create a lower and narrower beach, resulting in more frequent breaching. Similarly, ongoing sea level rise is expected to cause gradual shoreline recession and could conceptually result in more frequent breaching.

5. If J Street and OID were both provided permanent outlets through the beach to the ocean and freshwater not allowed to pond, the aerial extent of the lagoon would likely decrease due to infilling from wind blown sand and resulting in a lagoon more similar to the one that existed in the 1970s. If the ecology of the existing lagoon is to be maintained or improved, permanent outlets through the beach should not be considered.

6. When the lagoon is closed to the ocean, surface water and groundwater inflows cause the lagoon water level to rise, while evaporation and percolation through the beach tend to decrease water levels. Maximum water level in the lagoon prior to a breach is regulated by the lowest beach elevation between the lagoon and ocean. Beach elevation during closed conditions has been observed to be as high as 11.6 ft NGVD (14 ft NAVD) in isolated dunes. Based on available surveys and water level observations from 1975 to 2008, a nominal minimum beach elevation of 7.6 ft NGVD (10 ft NAVD) was applied for breaching analysis.

7. Lagoon breach formation is characterized as seaward breaching caused by freshwater overtopping the beach. Following a major breach, some limited wave-induced landward breaching may also occur. Breaches may occur multiple times each year. Wave-induced sediment transport is the main process for closing the breach and rebuilding the beach face. The tidal exchange through the breached inlet does not create velocities high enough to counteract the accretion caused by waves. Long-shore transport, aeolian transport, inflow from J Street, Hueneme and OID, and tidal exchange through the breach also contribute to the closure process. The time required for the breach to close is not expected to increase by emergency breaching.

8. Water levels in the lagoon, prior to a breach, range from about 1.6 ft to 6.6 ft NGVD (4 ft to 9 ft NAVD) and have been reported to reach up to 7.6 ft NGVD (10 ft NAVD). Based on numerical modeling for the 100-year storm, peak water levels within the lagoon at J Street are expected to reach about 9.6 ft NGVD (12 ft NAVD) in the absence of emergency breaching.

9. Channel design should consider lagoon water levels of no less than 9.6 ft NGVD (12 ft NAVD). If hydraulic analysis of the improved channel indicates that this water level cannot be accommodated by the design, or if the future unanticipated events change the condition of the beach/lagoon such that the design conditions analyzed herein may be exceeded, then emergency breaching must be considered.

10. Creating an emergency breach near OID prior to the 100-year storm is unlikely to decrease the peak water level in the lagoon near J Street, but would reduce initial lagoon water levels. A natural breach is expected to form early in the hydrograph for the 100-year storm, creating a breach well before peak flow arrives.

11. Creating an emergency breach near J-Street prior to the 100-year storm would decrease the peak water level in the lagoon near J Street from approximately 9.4 ft to 6.1 ft NGVD (11.8 ft to 8.5 ft NAVD). Emergency breaching would become less effective if performed further south of J Street.

12. Any emergency breaching alternative should include continuous monitoring of the lagoon water levels, a system for accurate prediction of storm events, a well defined emergency management plan, and the ability to rapidly mobilize excavation equipment. The emergency breach should be at least 50 ft wide, but would likely form on its own after initial excavation. Locating the breach near OID would maintain the existing lagoon configuration but does not achieve the lowest design water levels for the 100-year event at J Street.

3. Project Review

Various sources of existing information were gathered and reviewed to develop an understanding of the original project and recent analyses of J Street Drain and the Lagoon performed by others.

3.1. Physical Description

J Street drain was originally constructed in 1960 and discharged directly to the Pacific Ocean. As a result of storm water ponding behind the dunes and beach, two shallow lagoons formed at both J Street and OID, eventually becoming hydraulically connected and forming the existing lagoon. The lowest (deepest) areas of the lagoon are now located at the ends of J Street Drain and OID. These low areas are connected by a higher and relatively flat channel that varies in width from 50 to 100 ft. The surface area of the lagoon varies with freshwater inflow, the configuration of ephemeral side channels, and condition of the breach (open or closed). The majority of the wetted lagoon is located near to and south of the OID. The lagoon and adjacent wetlands intermittently connect to the Pacific Ocean following rain events when the lagoon water levels exceed the elevations of Ormond Beach. Bottom elevations in the lagoon are high compared to tide levels, thus limiting tidal range and prism. Figure 3.1 shows a three dimensional plot of the lagoon based on March 2008 aerial survey data. The water level shown is representative of mean higher high water (MHHW).

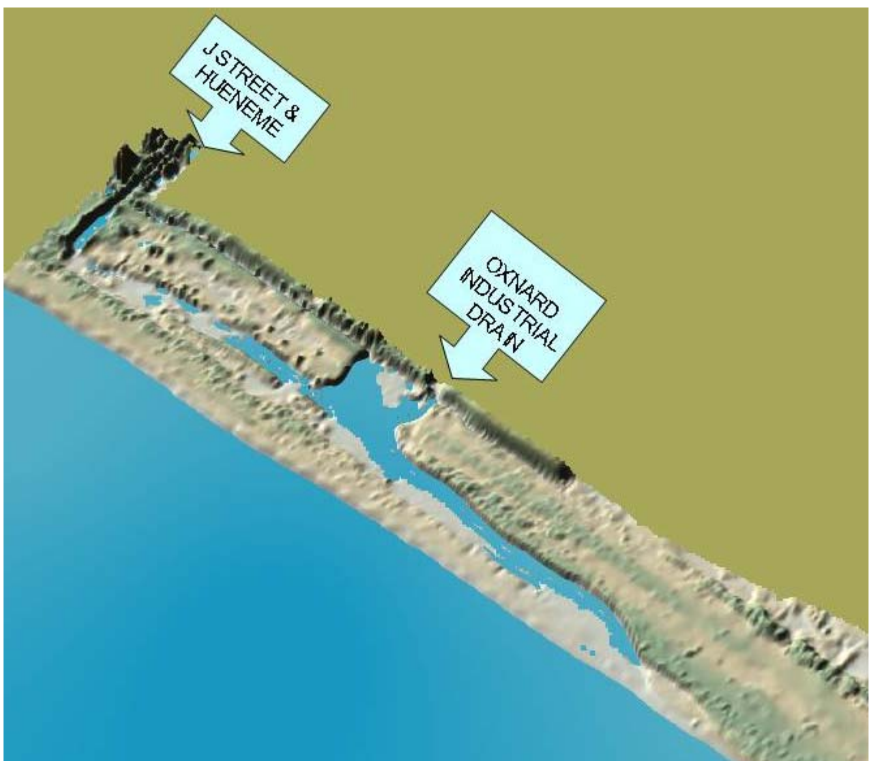

Figure 3.1. Three dimensional plot of the lagoon, Marsh 2008.

3.2. Previous Technical Studies

Previous hydraulic modeling and analysis of the J Street Drain and/or the lagoon has been conducted by the VCWPD (Su 2007, Tuan 1995), URS (2005), Tetra Tech (2005), and Phillip Williams and Associates (PWA 2007), among others. These studies differ in their assumptions regarding design flow rates and downstream boundary conditions in the lagoon but are otherwise consistent.

Prior to 1992, VCWPD routinely breached the lagoon mechanically near the J Street outfall. This practice was halted because of environmental concerns associated with decreasing water level and hydroperiod in the lagoon. Su's (2007) memorandum provides information on frequency of berm breaching and the modeling work of Tetra Tech (2005). Assuming that the berm completely breaches when the storage capacity of the lagoon is reached, Su (2007) indicates that the lagoon would breach during a 2-year storm event. During stronger storms the backwater effect of the lagoon will not substantially increase flooding because the breach occurs early in the storm. However, during the 2-year event, localized flooding may occur prior to breaching.

URS (2005) developed a plan and preliminary design to reduce flooding in the City of Oxnard by improving flow in J Street Drain. URS concluded that along with drainage system modifications, the backwater effect in the lagoon should be managed to reduce flooding. URS (2005) applied HEC-RAS, a one dimensional flow model, to analyze the existing and proposed hydraulic conditions. Rather than modeling the breaching process, URS assumed that the water level in the lagoon was either at the elevation of the beach or at normal depth based on channel slope, depending on the return period of the storm considered.

Tetra Tech (2005) applied FLO-2D, two dimensional flood analysis model, to define flooding in the City of Oxnard associated with a 100-year storm event; this modeling followed work by Pacific Advanced Civil Engineering. Tetra Tech also modeled the 2-year and 100-year storms to estimate when the lagoon breach occurs, assuming initial water levels in the lagoon of 4.0 ft and 6.5 ft Mean Sea Level (MSL) (6.7 ft and 9.2 ft NAVD). The analysis of flow in the lagoon was more detailed than that performed by URS (2005), but did not include an analysis of breaching processes that might elucidate potential project impacts for the improved channel. Tetra Tech suggested that lack of capacity in the existing drainage channels contributes more to local flooding than the condition of the beach elevation or lagoon water level.

Tuan's (1995) memorandum describes a VCRAT (Ventura County Rational Method) study to analyze the lagoon water level, backwater flooding effects, and rainfall intensity associated with flooding near the lagoon. The report indicates that the elevation of the beach varies from 5.5 ft to 8.5 ft MSL and that in September 1994 water levels in the lagoon reached approximately 7.5 ft MSL before breaching occurred. The analysis indicated that flooding begins when water levels in the lagoon reach 5.5 ft MSL and when rainfall intensity exceeds 1 inch/hour.

The California State Coastal Conservancy (SCC) is developing a comprehensive environmental restoration plan at Ormond Beach, including the lagoon and adjacent wetlands. To evaluate project feasibility, Philip Williams and Associates, Ltd (PWA 2007) prepared a hydrologic and geomorphic conditions report. The report summarizes key processes affecting Ormond Beach, the lagoon, and surrounding wetlands based primarily on existing data sources, surveys of the upper beach, and studies by others. The report also describes concepts for maintaining a tidal inlet at the lagoon by modifying elevations of adjacent lands and installing jetties or similar structures.

Previous studies have characterized stormwater flows into Ormond Lagoon and generally describe processes affecting lagoon breaching. Detailed analyses of coastal processes, lagoon hydrodynamics, hydroperiod, lagoon breaching dynamics, and lagoon closure have not been previously performed for assessment of potential project-related impacts or for design purposes.

4. Physical Processes

Modifications to J Street Drain and outlet configuration could potentially alter breaching frequency and duration, tidal prism, lagoon hydroperiod, and other characteristics of the lagoon. Therefore, HDR investigated the history of the lagoon, behavior of similar systems, and key physical processes affecting the coupled behavior of the drains, lagoon, beach, and the nearshore Pacific Ocean. The following sections provide a summary of key physical processes affecting the lagoon. More detailed information on physical processes can be found in Appendix A and within the references listed in Section 8.

4.1. Coastal Processes Summary

- Tides in the project area are semi-diurnal with a 5.4 ft range between MHHW and MLLW. For reference, the elevation of MHHW is approximately 5.3 ft NAVD. Tides exceed MHHW approximately 5% of the time.

- Tidal Prism varies considerably with changing topography of the lagoon and connected channels. PWA (2007) estimated the tidal prism to be between 15 and 17 acre-ft. Based on the models presented in Appendix B, the tidal prism ranges from approximately 9 to 24 acre-ft, depending on the tide and geography of the lagoon.

- Winds are predominantly from the west and are characterized in detail in Appendix A. Wind plays three primary roles in coastal processes by driving nearshore waves, surface currents, and aeolian transport (wind blown sand).

- Average rainfall in the area is only about 0.04 inch/day with higher rates from October through April.

- Waves nearshore are predominantly from the southwest. Significant wave height ranges from 1.5 to 5.0 ft 85% of the time. Wave period is between 12 and 18 seconds 75% of the time. Waves drive longshore and cross shore sediment transport at Ormond Beach. Wave-driven sediment transport coupled with tides, wind and lack of strong discharge from the drainage system causes closure of the breach.

- Net longshore transport is to the south at about 1,000,000 CY per year. Disruption of ongoing artificial bypassing at Port Hueneme would result in significant erosion at Ormond Beach and thus potentially alter the nature of the lagoon.

- Cross-shore transport is the dominant process responsible for breach closure and has not been previously studied.

- Due to the arid climate, aeolian transport along Ormond Beach is significant and capable of partially filling and dividing the lagoon in the absence of significant freshwater discharge.

- Measured long-term mean sea level rise at nearby tidal stations is less than 1 ft per century. IPCC (2007) predicts eustatic sea level rise of 1.1 ft over the next 100 years. Sea level rise likely will not significantly affect the elevation of the beach adjacent the lagoon over the next 100 years. Rise of MHHW by 1.3 ft should be considered to determine the lower limit for lagoon water level during a storm.

- Combined sediment load from J Street and OID is approximately 320 CY per year (HDR 2008B) with about 95% of the sediment contributed by OID.

- Hydroperiod of the lagoon has not previously been studied and/or documented in detail.

4.2. Coastal Lagoon Inlets and Breaching Review

A review of available literature was conducted to provide background on work completed at other sites and methods applied to analyze lagoon breaching dynamics. The reader is referenced to the following sources for more information. Most of the literature relevant to coastal breaching focuses on landward breaching occurring during extreme marine events from elevated storm surge levels so that water is flowing from the ocean rather than to the ocean.

Coastal breaching models have been developed by Basco and Shin (1999), Kraus (2003), Tuan, Verhagen, and Visser (2006), Tuan (2007), Mohamed (2001), Faeh (2007), Srinivas and Dean (1996), Odd, Roberts, and Visser (1998), Maddocks (2000), and others. Examples of applicable general sediment transport work include Madsen and Wood (2002), Myrhaug and Holmedal (2003), Fredsøe and Deigaard (1992), Davies *et al* (2002), Baldock *et al* (2005), Ogston and Sternberg (2002), Smith (2002), Soulsby and Damgaard (2005), and Yu, Sternberg, and Beach (1993).

Case studies evaluating lagoon/estuary breaching on the California coast and around the world are documented by Kraus (2002), Hansen *et al* (2007), PWA (1993A, 1993B), and Battalio *et al* (2006), among others. Stone Lagoon (along the coast of northern California) is similar to Ormond Beach Lagoon in that it breaches seaward (Kraus 2002). Kraus (2008) presents detailed discussion of breaching at multiple northern California lagoons/estuaries.

4.3. Breaching at Ormond Lagoon

Breaching at the Ormond Lagoon is caused by buildup of freshwater originating from J Street, Hueneme and Industrial Drains and can be characterized as seaward breaching. Tuan (2007), following Gordon (1990), describes the breaching process of coastal lagoon barriers due to overflow induced by heavy rain as follows:

> "The lagoon breakout stage is observed to consist of three distinct stages. In the first stage, a preferred scour channel (initial channel) is formed and cuts backwards across the barrier. The flow is subcritical in the breach section and

supercritical on the down slope. The second stage commences when a crescent-shaped weir forms in the main sand plug followed by a series of steps in the channel. The breach width increases rapidly as the breach flow is highly turbulent and supercritical. Once the main sand plug has been washed out completely, the final stage begins with a slower rate of breach deepening and widening."

After the breach is established and upland discharge has significantly decreased, tidal exchange between the lagoon and ocean acts to maintain the breach. Waves transport sediment onshore and alongshore and the varying tide and wave run-up distribute the sediment along the shoreface. Sediment transport in the swash zone, similar to transport under a small bore, effectively carries sediment into the breach. As tidal flow in the inlet becomes insufficient to remove all of the sand being transported by the waves, the breach will begin to close.

Based on field observation of Ormond Lagoon by HDR, the breaching process appears to be consistent with that described by Tuan (2007) and Gordon (1990).

For reference, Figure 4.1 shows the location of previous historic breaches along Ormond Beach, based on historic surveys and aerial photographs. Breaches tend to occur along the lagoon south of J Street and, more recently, near the outlet of the OID. The figure shows that the breach has occured at many different locations. Recent breaches have been located closer to OID than to J-Street.

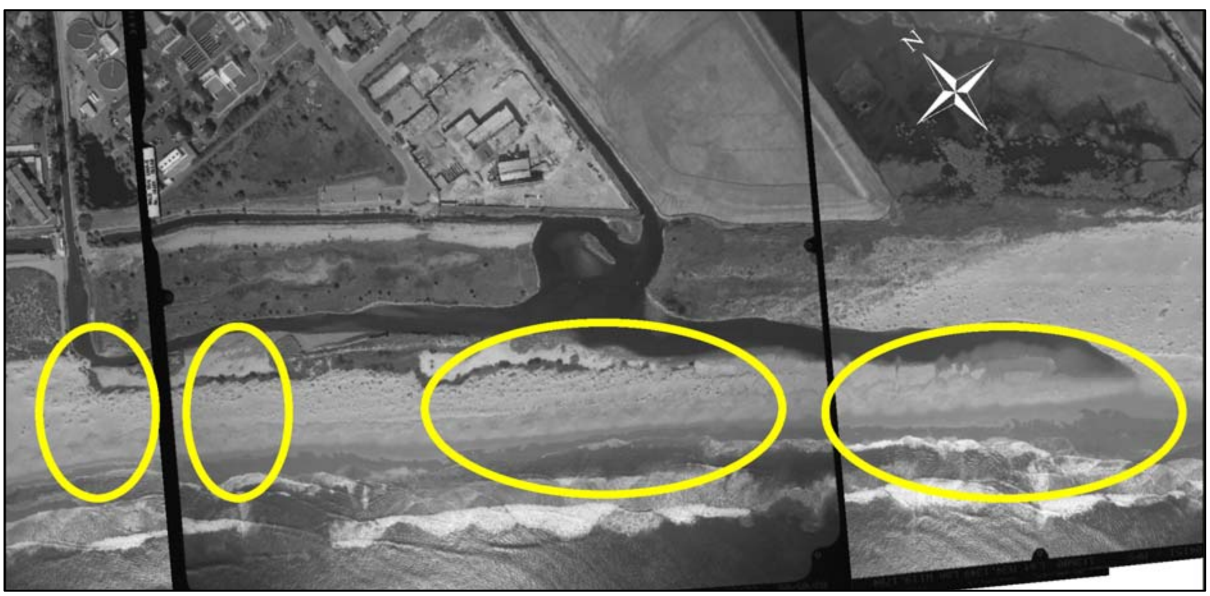

Figure 4.1. Locations of previous breaches.

Except at the existing breach area, which is the southeastern most area circled in Figure 4.1, the water level along the beach must exceed about 10 ft NAVD before landward breaching is likely to occur. There is no record of the tide level exceeding this elevation since the Santa Barbara gauge was installed in 1933. For reference, the highest tide recorded at the Santa Barbara gauge is 7.3 ft NAVD. Wave run-up in combination with extreme high tides could overtop the beach and dunes when the beach is low, for example following a seaward breaching event. However, the volume of water contributed to the lagoon in this case would be relatively small compared to the storage area of the lagoon. In any case, landward breaching is controlled by the ocean tide and wave conditions, which cannot be affected by the J Street Drain modifications.

5. Data Collection and Interpretation

The primary objective of the data collection and interpretation was to fill critical data gaps and compare new data with existing data for more detailed assessment of lagoon dynamics and project alternatives. This section describes new data collected for the project and related data collected and provided by others.

5.1. Critical Data Gaps

Recent aerial photographic surveys and LiDAR surveys were not capable of providing bottom elevations for areas of the wet lagoon or beach. Therefore, a bathymetric survey was performed along the beach face, nearshore, and within the wetted lagoon to supplement existing survey data. Surface sediment grab samples were collected along the beach face from the surf zone to the dunes and within the lagoon and breach. Grain size distributions were then determined by laboratory testing. Concurrently, suspended sediment samples were collected and later tested. An electronic gauge was also installed within the lagoon to continuously monitor water levels and salinity near the main body of the lagoon for 30 days.

The data collection timing and duration was selected to meet the needs of the project schedule. At the direction of HDR, data collection and analyses were completed by Coastal Frontiers Corporation, as described in the *J Street Drain Coastal Engineering 2008 Beach and Lagoon Monitoring Program* report (Coastal Frontiers 2008).

5.2. Surveys

5.2.1. Aerial Photogrammetric Survey

An aerial photogrammetric survey was performed by Mercator Photogrammetric Systems (MCS) on March 5, 2008. The survey provided geo-referenced ortho-photos and topography for J Street Drain and areas surrounding the lagoon. The survey compared well with the traditional land surveys described in the following sections, with elevations generally varying by less than 0.5 ft.

5.2.2. Beach Profile Survey

Beach profiles were collected on March 21, 2008, along 13 transects located between Port Hueneme Beach (north of pier) and Arnold Road (approximately 2 miles south of the lagoon). Transects extended from the dry beach out to a depth of approximately 40 ft. The purpose of the beach profile survey was to provide detailed cross-shore profile data along the beach fronting the lagoon. Changes in two-dimensional beach profiles provide a means for estimating the closure depth and net cross-shore sediment transport patterns. Closure depth is an important parameter in sediment transport modeling, and is the depth beyond which sediment transport is less active.

Locations of the cross-shore beach profile transects are depicted in Figure 5.1. Three of the transects were located on transects previously established by the Beach Erosion Authority for the Clean Oceans and Nourishment (BEACON) monitoring program, while ten new transects, Transects OL01 through OL10, were established specifically for the present project. Alignments of the new transects were chosen based on transect BCN23. Transect OL01 was aligned with J

Street, approximately 575 ft east of transect BCN23, while transects OL02 through OL10 were taken at 500-ft intervals. The transect establishment activities, data collection procedure, and reduction processes are described by Coastal Frontiers (2008). The accuracy of the soundings vertically and horizontally is approximately ±0.5 ft and ±2.0 ft, respectively.

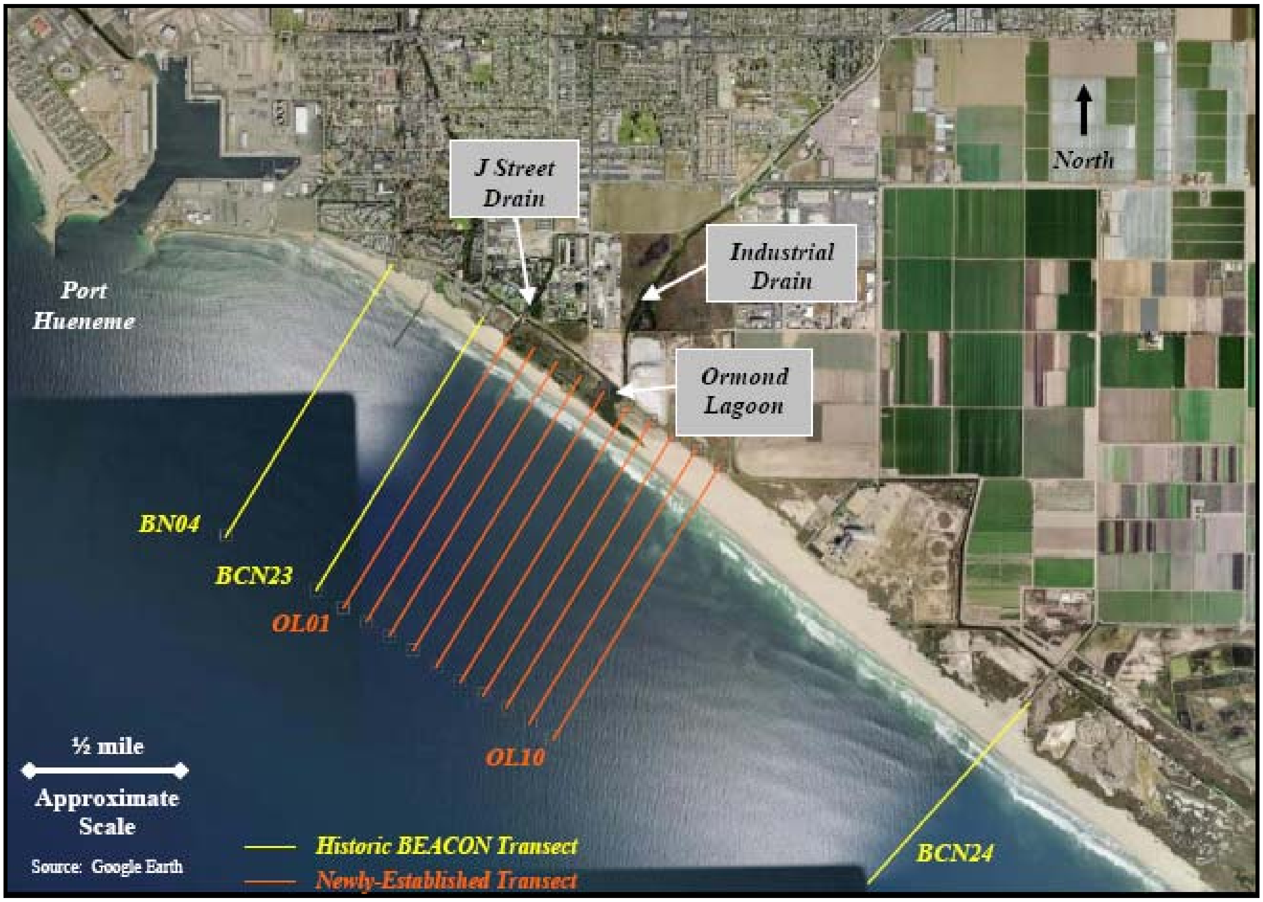

Figure 5.1. Location of beach profile survey transects (Coastal Frontiers 2008).

Figure 5.2 provides a beach profile plot for transect BCN23 and includes selected profiles obtained between October 1987 and November 2003 as provided by BEACON. Also shown is the observed depth of closure based on the beach profiles shown in Figure 5.2 (refer to Appendix B for additional details on depth of closure). All beach profile plots are provided in Coastal Frontiers (2008).

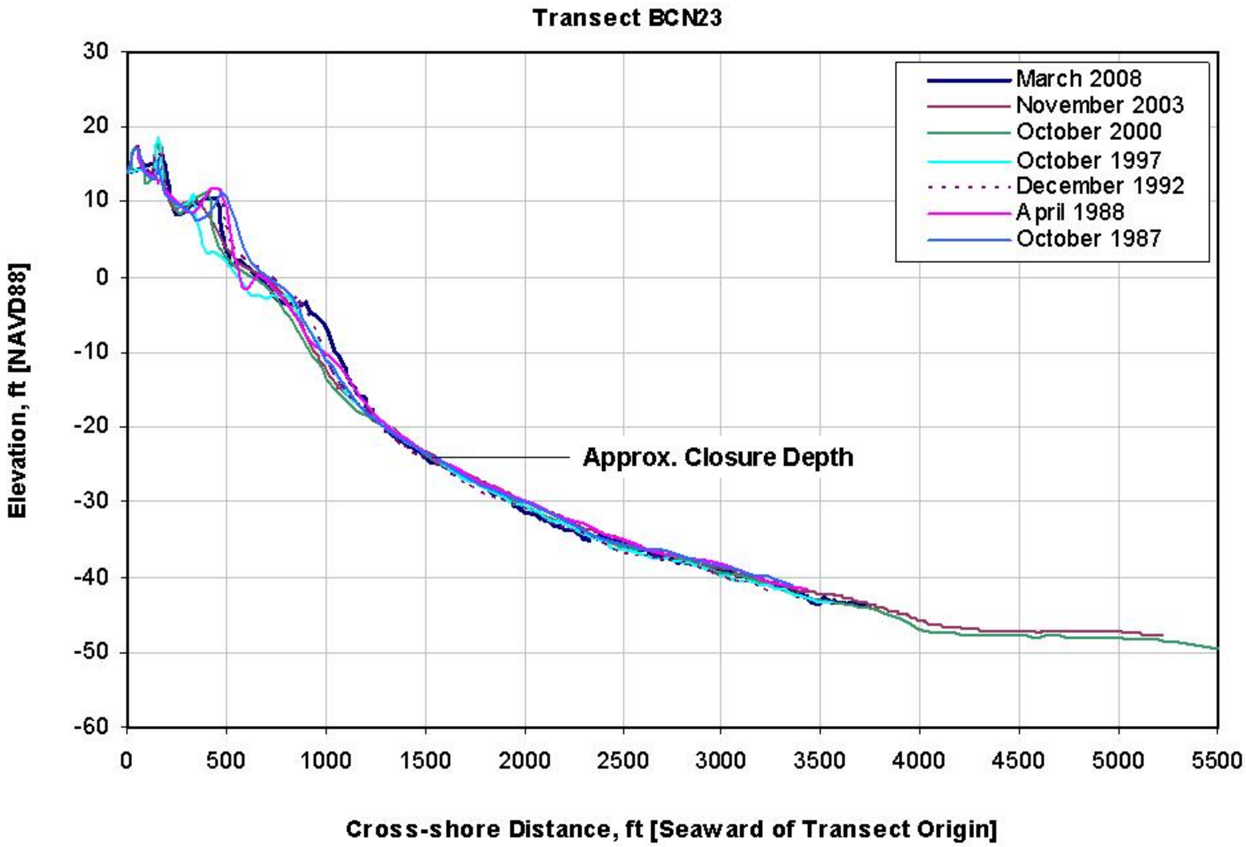

Figure 5.2. Historical cross-shore profiles for Transect BCN23.

5.2.3. Lagoon Bathymetric Survey

The bathymetric survey of Ormond Lagoon was conducted on March 24 and 27, 2008. The purpose of the survey was to establish bottom elevations of the lagoon and transitions between the lagoon and adjacent vegetated uplands. Surveys were conducted in U.S. survey feet relative to North American Vertical Datum of 1988 (NAVD88) in California State Plane Zone 5, NAD 83, respectively. The topographic and bathymetric data were acquired along cross-sections of the lagoon. A small breach had closed just prior to the bathymetric survey.

5.2.4. Other Previous Surveys

Numerous other topographic surveys were gathered and reviewed, as summarized in Table 5.1. Figure 5.3 shows the result of the March, 2008 Mercator Photogrammetric Systems survey.

Table 5.1. Summary of available aerial surveys.

Date	Source	Type	Vertical Datum
Mar-08	Mercator	Photogrammetry	NGVD
Jul-01	Towill, Inc.	LIDAR	NAVD
Apr-98	NOAA	LIDAR	NAVD
Oct-97	NOAA	LIDAR	NAVD
Dec-75	Toups Eng.	Photogrammetry	NGVD

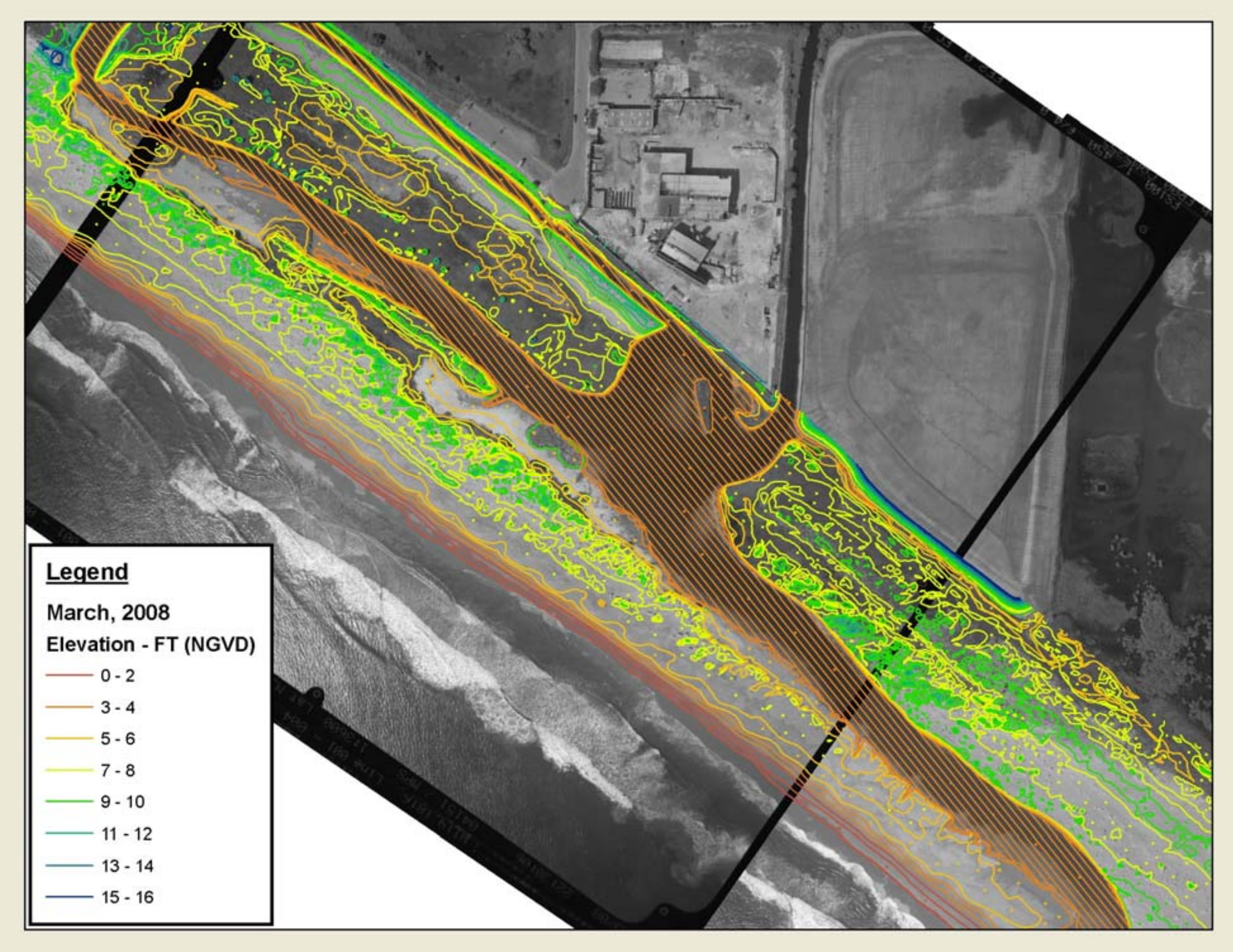

Figure 5.3. Aerial survey flown 3/5/2008 by Mercator Photogrammetric Systems.

5.3. Longshore Current

The combination of waves and currents in the nearshore create a longshore current that transports sediment along Ormond Beach. Characterization of the longshore current at Ormond Beach is important for determining sediment transport related to breach closure and calibrating the hydrodynamic and sediment transport models. The survey team estimated the longshore current to be approximately 2 ft/s during the sediment and water sample collection. Because there were no readily available means of measuring the longshore current, the survey rodman floated along shore near Transect OL09 for approximately 1 to 2 minutes. The position of the rodman was measured and the mean velocity calculated (Table 5.2). Although somewhat crude, this measurement technique provided a rough approximation of typical longshore current velocity at the site. Measurements of longshore current are otherwise unavailable for Ormond Beach.

Table 5.2. Estimation of longshore current at Ormond Beach.

Description	Location			Total Distance [ft]	Elapsed Time [sec]	Current Velocity [ft/sec]
	Northing [U.S. Ft]	Easting [U.S. Ft]	Elevation [Ft-NAVD88]			
Transect OL09 - Longshore Current Estimate A						
Start	1,872,501	6,204,841	1.4	161	74	2.2
Stop	1,872,378	6,204,946	0.8			
Transect OL09 - Longshore Current Estimate B						
Start	1,872,483	6,204,832	0.6	148	89	1.7
Stop	1,872,376	6,204,935	0.5			

5.4. Sediment Samples

Sediment size is a controlling factor of beach morphology. Thus, surface sediments were collected from the beach and lagoon. Suspended samples of total sediment concentration were also collected and analyzed.

5.4.1. Bottom Sediment Samples

A total of 14 sediment samples were collected at different locations along Transects OL01 and OL09 and inside Ormond Lagoon (Figure 5.4). Five samples were collected along each transect, two samples were collected in the lagoon, and the remaining two samples were collected at the site of the recent breach. Each sample consisted of 30 cm^3 of sand collected in two glass containers. A sieve analysis was performed on each of the sediment samples to determine the median grain size and particle size distribution. Results show that grain sizes along the Ormond Beach range from fine to medium sand with a median grain size between 0.23 and 0.57 mm, depending on location. Detailed sieve analysis results for each sample are provided separately in Coastal Frontiers (2008).

Figure 5.4. Location of water and sediment grab samples.

5.4.2. Suspended Sediment Samples

A total of 31 water samples were collected at 10 sites throughout the project area to analyze suspended sediment concentration. The samples were taken along Transects OL01 and OL09 and also inside Ormond Lagoon (Figure 5.4). Four locations along both Transects OL01 and Transect OL09 were sampled at depths of approximately 4, 3, 2, and 1 ft. The remaining two samples were taken inside the lagoon at mid-depth at approximately the same location of the bottom grab samples. Results from the sampling analysis show that the greatest concentrations of suspended sediments were located near the seabed at water depths of approximately 1-2 ft.

5.5. Beach Elevation

Survey data summarized in Table 5.3 were evaluated to determine representative maximum and minimum elevations of the beach during each respective survey. The expected maximum water level in the lagoon is regulated by the lowest beach crest elevation. Aerial surveys provided maximum coverage of the beach allowing reasonably accurate estimation of the minimum and maximum beach elevations adjacent the lagoon. The beach transects surveyed by BEACON do not provide coverage at the breach and instead provided estimates of the maximum dune crest elevation near the lagoon. The 1975 survey shows that J Street and OID had separate discharge locations, similar to the configurations in the photographs presented in Appendix A.

The survey data suggest that the beach reaches its maximum elevation of approximately 14 ft NAVD in the vegetated dunes. Elevation across a beach is not uniform in space or constant in time. To support numerical modeling, described later in this report, it is important to define the beach in an idealized uniform manner. A nominal beach elevation is determined through analysis of available survey data to represent the existing beach elevation. The nominal elevation of the beach for the purposes of this discussion is defined as the elevation at which the beach is likely to be overtopped. The nominal elevation is significantly lower than the maximum beach elevation.

Tetra Tech (2005) assumed representative beach elevations of 10 to 11 ft MSL (12.7 to 13.7 ft NAVD) based on a single available survey. Review of the additional survey data listed in Table 5.3 indicates that elevations range from 4 to 14 ft NAVD, depending on storm activity, that 4 ft is representative for a breached condition, and 14 ft is representative for a beach that has not breached in well over one year. The aerial surveys show the representative nominal beach/dune crest elevation adjacent the lagoon to be approximately 10 ft NAVD.

Based on the aerial surveys and other available data and under typical conditions, a representative elevation for the beach prior to breaching is approximately 10 ft NAVD.

Table 5.3. Summary of historic beach elevation data.

Survey Date	Source	Maximum Dune Elev. ft NAVD	Nominal Beach Elev. ft NAVD	Minimum Beach Elev. ft NAVD
Mar-08	Coastal Frontiers	12	10	4
Mar-08	Mercator	14	10	4
Nov-03	BEACON - BCN24	12	NA	NA
Jul-01	Towill, Inc.	12	10	6
Apr-98	NOAA	13	8	4
Oct-97	BEACON - BCN24	12	NA	NA
Oct-97	NOAA	12	10	10
Dec-92	BEACON - BCN24	14	NA	NA
Apr-88	BEACON - BCN24	11	NA	NA
Oct-87	BEACON - BCN24	11	NA	NA
Dec-75	Toups Eng.	12	10	9

5.6. Water Level and Salinity Observations

Water level data is available from a number of sources and augmented by a 32-day data set of water level and salinity collected as part of the field program for the project.

5.6.1. April 2008

Salinity and water level data were collected over a 32 day period from March 27 to April 28, 2008 in the lagoon utilizing a MacroCTD sensor located as shown in Figure 5.5. The sensor was configured to record conductivity, temperature, and pressure every six minutes based on an average of 60 samples taken at 2 Hz. Salinity measurements of the ocean and inside the lagoon were collected to attempt to measure the salinity transport during a breaching event; however, the lagoon remained closed through the monitoring period. Measurements were taken at a single point at mid-depth to obtain a time series history for model input.

During the data collection period, the lagoon remained closed and water levels were not tidally influenced; however, some exchange of salinity between the ocean and lagoon did occur. The measured salinity in the lagoon, water level in the lagoon, and tidal elevation are shown in Figure 5.6. Salinity in the lagoon responded to fluctuating water levels in the ocean, with high tides resulting in an increase of salinity at the gage location. Even though the lagoon was not open to tidal exchange, waves in combination with high tides appear to have overtopped the recently closed breach, resulting in slightly higher water levels and salinity spikes measured at the gage. The overall trend of decreasing salinity combined with rising water level is attributed to industrial and agricultural runoff, and not precipitation. Significant rainfall was not recorded during the monitoring period.

Figure 5.5. MacroCTD location in Ormond Lagoon.

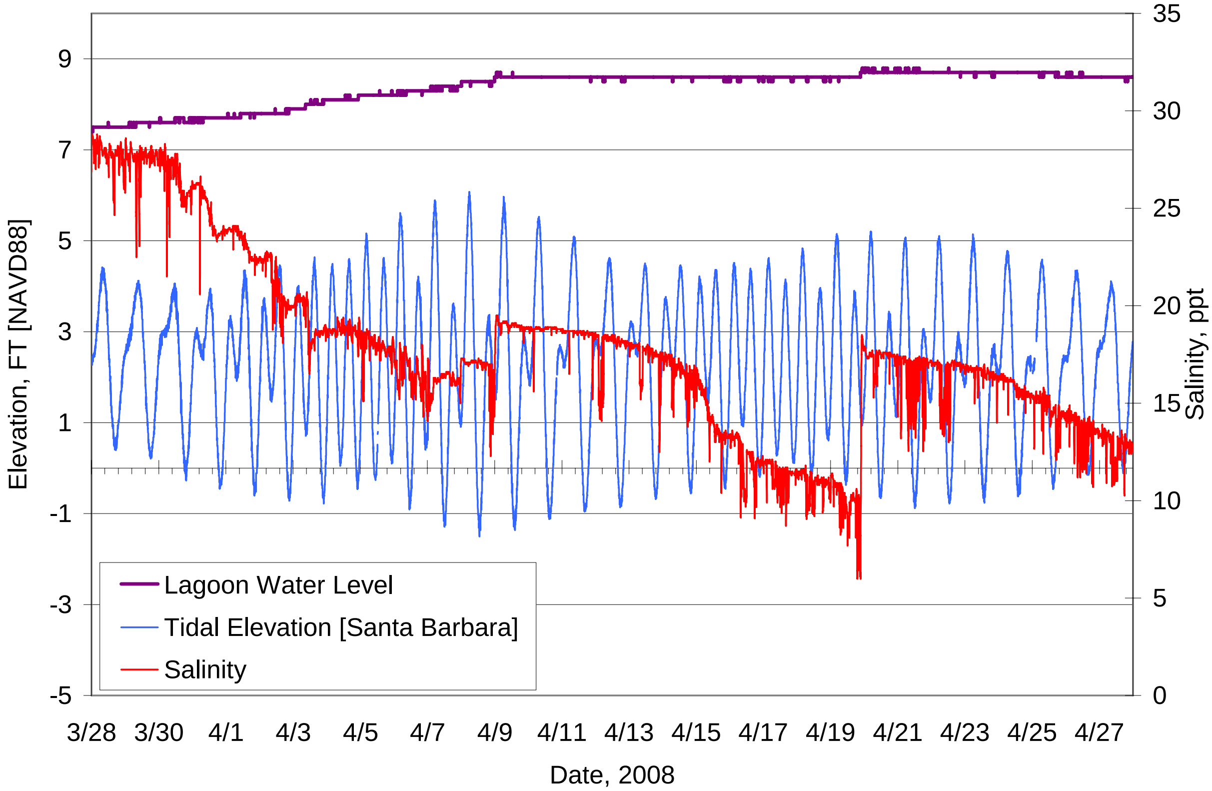

Figure 5.6. Measured lagoon water level, salinity and tide.

5.6.2. VCWPD J Street Water Level Gauge

A water level gage was installed by VCWPD near the Hueneme pump station in J Street Drain and operated from 2002 to 2005. Figure 5.7 presents the gage data from 2003. Calibration information was not available and the gage datum could not be determined by comparison with overlapping data sets. However, the data qualitatively indicate that the lagoon water levels generally rise and remain elevated above tidal levels during the summer months from May to September, and then rapidly decrease by 2 to 3 ft following breaches in early fall through the spring. The gage data also demonstrate that breaches form and close multiple times each year, from the early fall through the spring. Duration of lagoon closure may range from one week to five months or more. The data also indicate that the lagoon empties within 30 minutes to two hours following a breach, depending on the tide at the time of the breach.

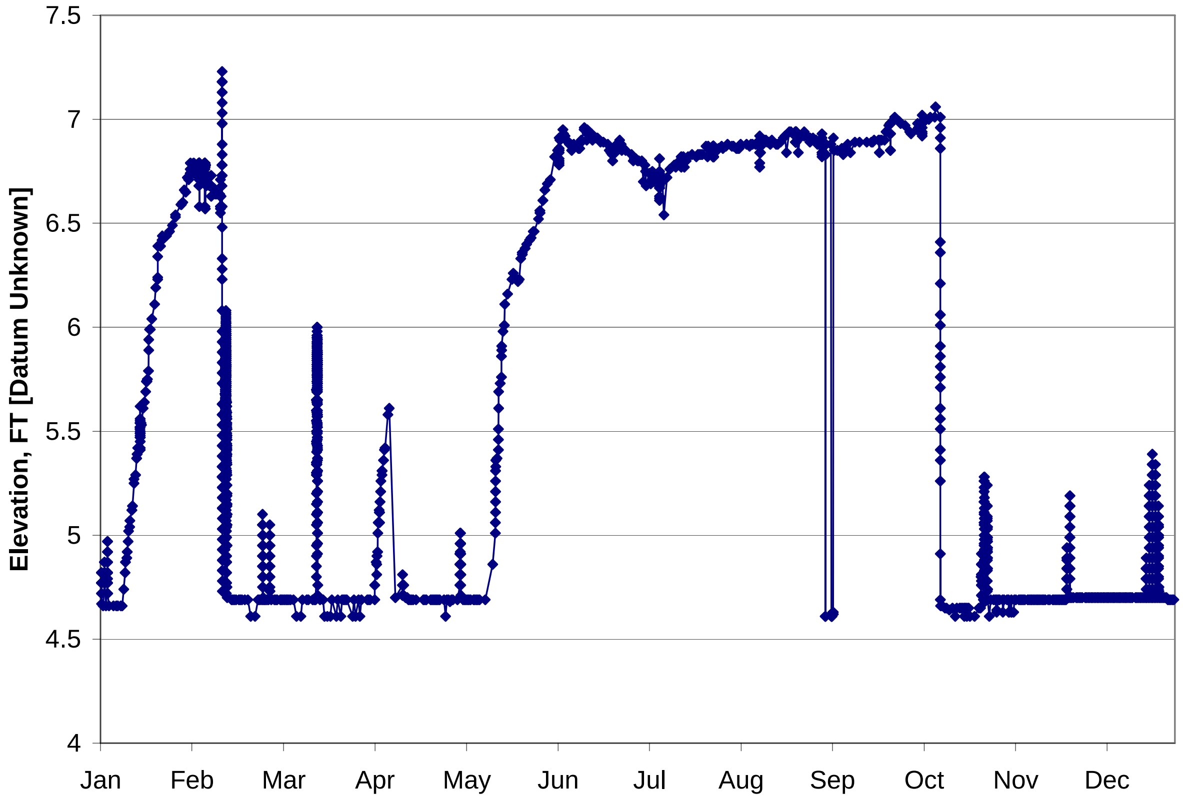

Figure 5.7. J Street 793 Alert Gage stage data for 2003.

5.6.3. CH2M Hill Water Level Data

CH2M Hill (2008) collected water level data in OID near the lagoon from mid-October 2007 through the first week of January 2008 and provided these to the VCWPD. Plotted in Figures 5.8 and 5.9 are measured water level at OID, NOAA tide data averaged between the Santa Monica and Santa Barbara stations, and precipitation. Two breaches occurred during the monitoring period, first on December 17, 2007 and again on January 4, 2008. The December breach appears to have occurred with an initial lagoon water level near 8.3 ft NAVD. Freshwater inflow caused the water level to rise to around 9.2 ft NAVD before the lagoon breached and water level dropped. From the plots, it appears that the lagoon emptied and equilibrated with the tide in less than 6 hours, consistent with observations at the J Street gauge discussed above.

Following the breach, water levels at OID remained tidally dominated, with no appreciable phase lag. However, the low tide level is clipped. This may have resulted from either drying of the water level gage or more likely impoundment of water within the lagoon above the tidal level due to the bathymetry of the lagoon or tidal channel. The breach closed during a neap tidal cycle around January 1, 2008, and water levels in the lagoon began to rise until the lagoon breached again on January 4.

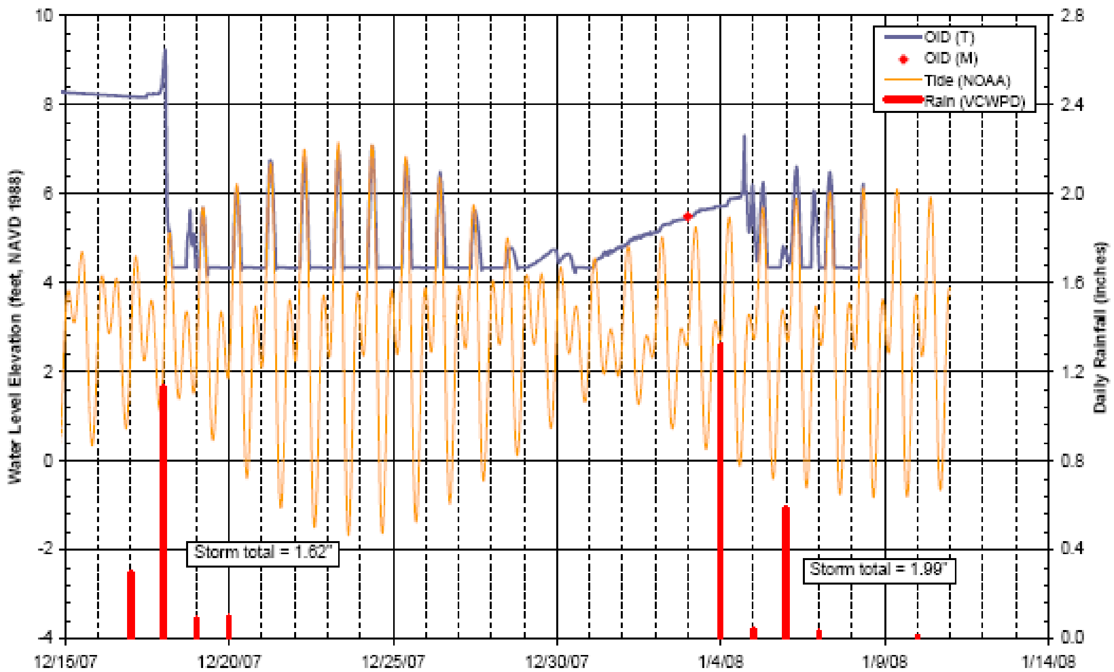

Figure 5.8. Water level in the lagoon 12/5/2007 – 1/10/2008 (CH2M Hill 2008).

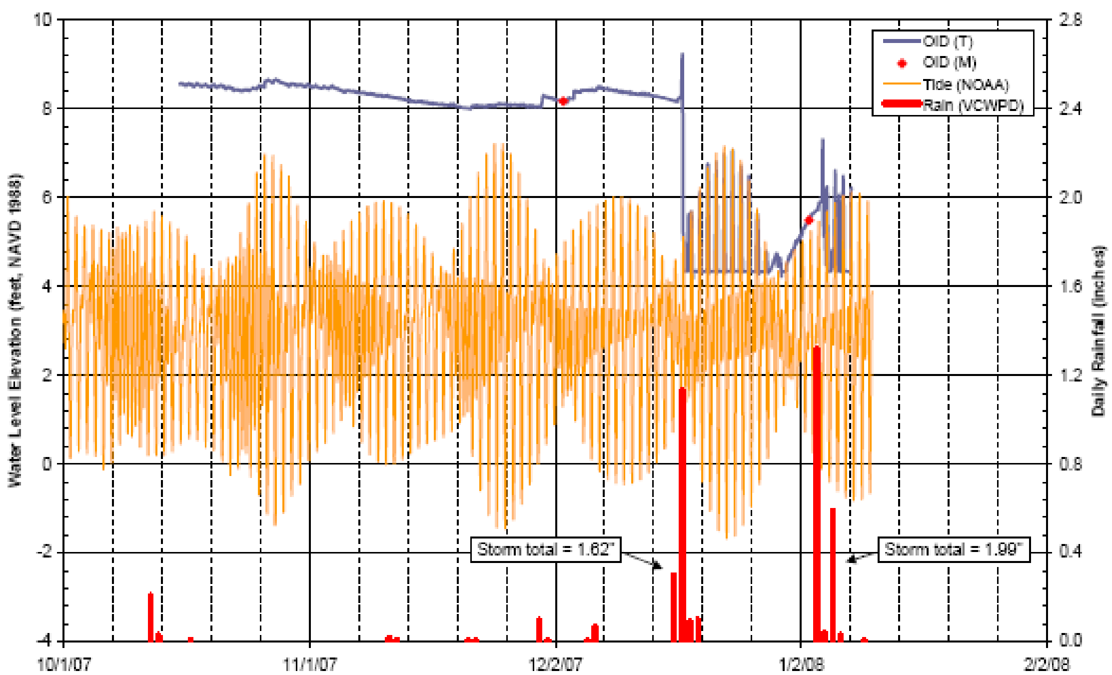

Figure 5.9. Water level in the lagoon 10/2007 – 1/2008 (CH2M Hill 2008).

5.6.4. Summary of Water Level Data

Water level data indicate that the breaches may form multiple times each year, by stormwater flows or by gradual water level rise. Where the drains meet the lagoon, a water level spike of about 1 ft has been observed just prior to breaching for moderate storm events. During recent breaches, water levels exceeded approximately 9.0 ft NAVD before breaching commenced and water level crests during breaching lasted from 30 minutes to a few hours. The water level was observed to peak above the nominal elevation of the beach.

5.7. Storm Hydrographs

Hydrographs were provided by VCWPD for the 2, 5, 10, 50 and 100-year events. Figures 5.10 and 5.11 plot the provided 2-year and 100-year event hydrographs, respectively. The storm hydrographs were applied as inputs to the lagoon hydrodynamic model to simulate storm events and evaluate outlet alternatives discussed in the next section.

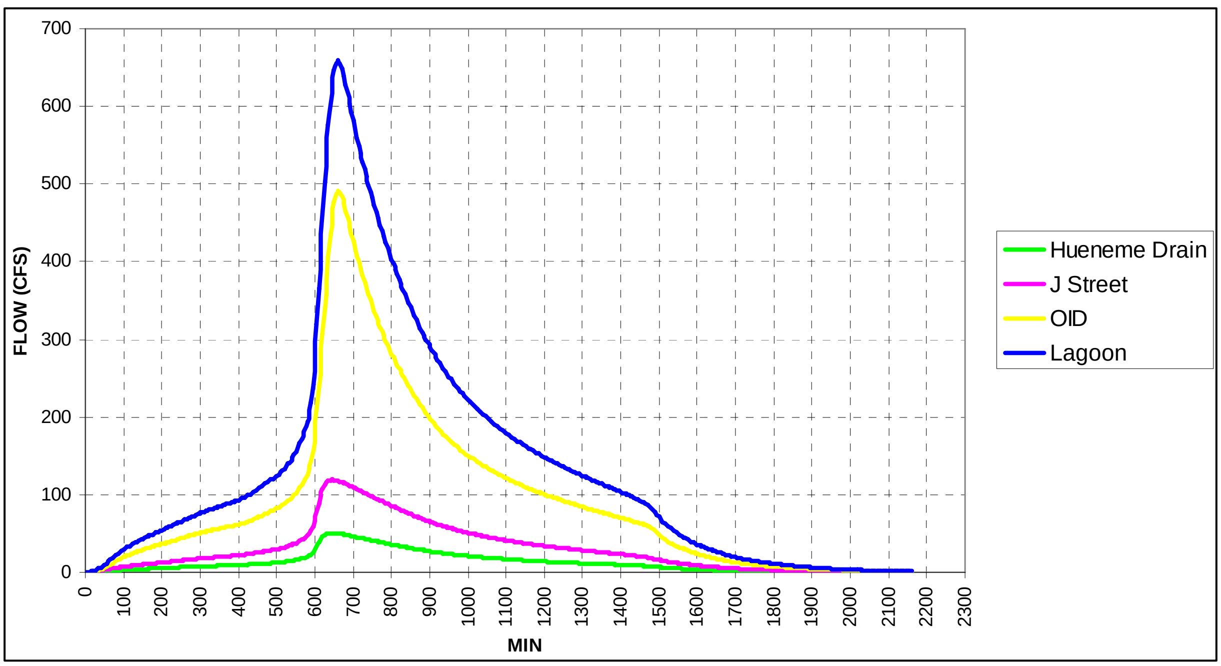

Figure 5.10. Hydrograph for 2-year event.

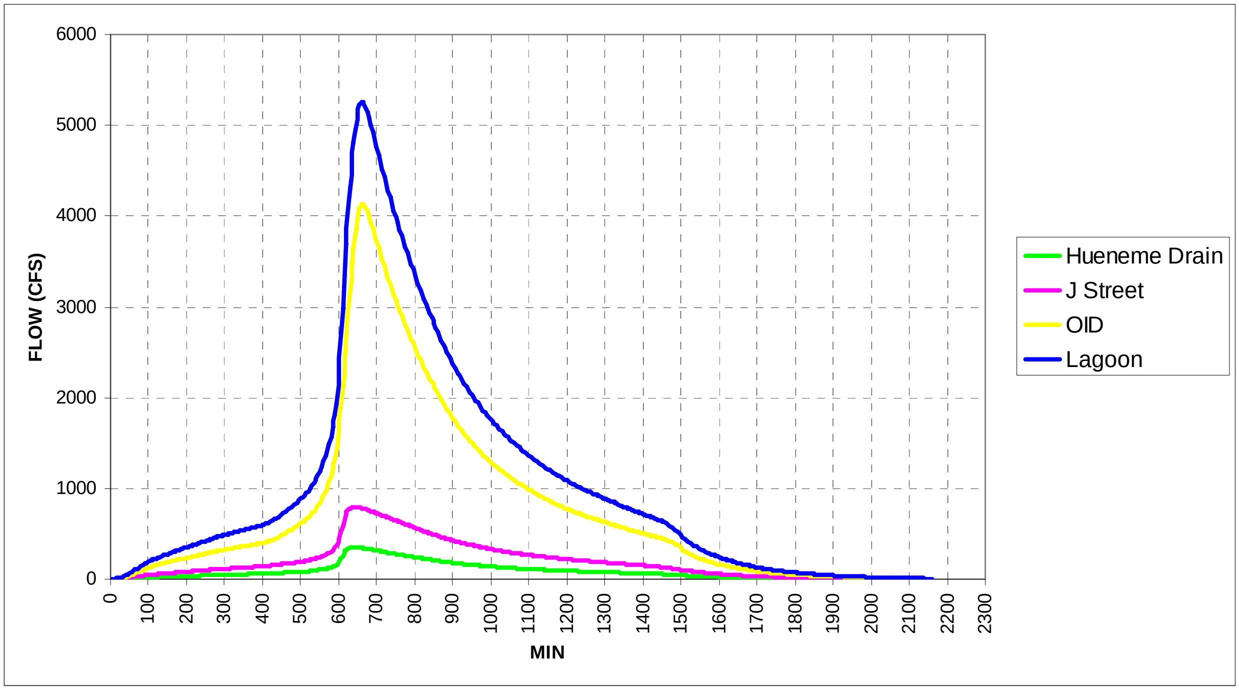

Figure 5.11. Hydrograph for 100-year event.

6. Alternatives Analysis

Modifications to J Street Drain channel and its outlet to the ocean may be required to reduce upstream flooding during design storms. Specifically, it is expected that the channel will be lowered by approximately 4 feet and a cutoff wall and a riprap apron installed at the channel transition to the lagoon. More active management of lagoon water levels may be required, in an emergency, to reduce impacts to the adjacent beach and lagoon dynamics. The following three outlet alternatives were therefore evaluated:

1. No Action
2. Beach Maintenance
3. Emergency Mechanical Breaching

Evaluation of a permanent installation, such as a jettied inlet or weir structure, was not considered because of the desire to avoid significant alteration of lagoon dynamics such as hydroperiod and geomorphology and the excessive cost of maintaining an inlet. This section describes numerical modeling and analysis that were undertaken to evaluate the feasibility, potential benefits, and impacts of these three alternatives.

6.1. Description of Outlet Alternatives

6.1.1. No Action Alternative

This alternative represents the unmanaged outlet condition, where the lagoon is allowed to breach and close as it has since mechanical breaching ceased in 1992. This alternative would result in no direct impacts to lagoon dynamics since breaching and closure of the lagoon would not be directly changed. Lagoon hydroperiod, which varies with beach elevation and water level, would be unaffected. Due to the dynamic nature of the lagoon, the geometry and depth of the lagoon will continue to change.

The No-Action alternative is disadvantageous because it offers no method for VCWPD to respond to unanticipated changes in the lagoon and upstream hydrology.

6.1.2. Beach Maintenance Alternative

Maintaining a section of the beach below the nominal beach elevations as a “breach corridor” would encourage the breach to form earlier in a storm and thus would conceptually reduce the maximum water levels in the lagoon during the design storm, reducing potential flooding. The elevation of the maintained section would need to be above extreme high tide, approximately 7 ft NAVD.

Maintenance is a common practice on recreational beaches and can be performed with standard construction equipment such as a bulldozer or front-end loader. Effective maintenance requires constant action to remove wind blown sand that would accumulate within the maintained area.

Maintaining a breach corridor would increase the frequency of breaching and likely result in lower water levels in the lagoon even when the breach is closed, decreasing the existing lagoon hydroperiod. This alternative would result in impacts to the lagoon.

6.1.3. Emergency Mechanical Breaching Alternative

Creating a mechanical breach, as was performed prior to 1992, would decrease the lagoon water level to the minimum possible elevation prior to significant rainfall events and provide more rapid discharge of stormwater from the lagoon to the ocean. To minimize reduction in lagoon hydroperiod and ecological function, the emergency breach would be made prior to storm events expected to both cause flooding and naturally form a breach. Thus, this alternative would decrease the lagoon water levels sooner (likely on the order of 24 hours) than natural breaching.

Effective execution of this alternative requires continuous monitoring of the lagoon water levels, advanced prediction of significant stormwater events, a well defined emergency management plan, and the ability to rapidly mobilize equipment.

6.2. Numerical Modeling

Incorporating the site data described in previous sections, MIKE 21 numerical models were applied to simulate hydrodynamics and lagoon morphology during breaching events for the existing condition and for hypothetical lagoon conditions based on available survey and water level data. First, a hydrodynamic model of the existing lagoon system was developed. Then the hydrodynamic model was coupled with a sediment transport model capable of simulating sediment transport and bed morphology. Finally, a qualitative analysis of breach closure was performed to evaluate closure time and closure processes. The objective of the modeling was to quantify existing conditions and investigate implications for design of the J Street Drain channel and the impacts of alternatives on the lagoon.

6.2.1. Breach Formation

The model applied to quantify breaching was forced from the landward side by design hydrographs provided by VCWPD. Ocean tides were applied on the seaward model boundaries to simulate spring tide conditions. The primary sediment transport mechanism during breaching is high velocity flow in the lagoon and over the beach. Therefore, waves were not included in the breaching simulations. Additional details on the numerical models are documented in Appendix B.

The breaching model was run for the scenarios outlined in Table 6.1. Table 6.2 summarizes the maximum water level near J Street and OID, as well as the time from model start (start of the hydrographs) to the time the breach begins to take place for each scenario. Calibration and verification information is presented in Appendix B. Based on comparison with available water level data, actual breaching occurs more rapidly than simulated by the model. Thus, simulated water levels in the lagoon are slightly higher than would be observed, so the modeled breaching is a conservative approximation of actual breaching.

Table 6.1. Model scenarios.

Run	Return Period	Lagoon Initial Water Level, ft (NAVD)	Description
1	2	9	Rough channel bottom to prove ability to erode channel that has started to fill in.
2	2	9	Existing representative beach.
3	2	6	Shows breaching time with lower initial water level with existing representative beach.
4	100	9	Existing representative beach.
5	2	6	50 m maintained section near OID
6	2	6	30 m maintained section near OID
7	2	6	10 m maintained section near OID
8	2	tide	Inlet near OID
9	2	tide	Inlet near J-Street
10	100	tide	Inlet near OID
11	100	tide	Inlet near J-Street
12	100	tide	Inlet near J-Street and near OID

Table 6.2. Model result summary.

Run	Time to Breach	Peak Water Level Near J-Street, ft (NAVD)
1	9:30	11.5
2	9:30	11.5
3	10:55	11.5
4	3:40	11.8
5	4:45	8.5
6	4:35	8.7
7	5:10	9.8
8	0:00	7.9
9	0:00	7.2
10	0:00	11.8
11	0:00	8.5
12	0:00	7.2

6.2.1.1. No Action

An idealized beach/lagoon system was developed for the model domain to represent the no-action condition. Because of the dynamic nature of the lagoon, the lagoon configuration at the time of the March 2008 survey is not considered representative of typical conditions. The idealized domain was created by altering the March 2008 data to match conditions observed in the historic data, creating a relatively uniform beach (berm) with a low elevation of about 10 ft NAVD.

Idealized representative conditions were simulated in Runs 1 through 3 for both the 2-year and 100-year events. Initial water levels in the lagoon ranged from 6 ft to 9 ft NAVD for the 2-year event. The maximum water level within the lagoon is a function of beach elevation and inflow. Supercritical flow during initial breach formation in the model is handled through introduction of numerical dissipation; therefore, the rate of erosion at incipient breaching was limited to improve model stability.

Consistent with previous analysis by others, the simulations indicate that the breach would occur significantly earlier in the 100-year event than during the 2-year event and that the maximum water level is about the same for both events for the idealized representative case. Greater flow during the 100-year event causes earlier breaching than the 2-year event and leads to significantly greater sediment transport, providing greater forcing to reshape the lagoon/beach system. Figure 6.1 shows the bed level at peak flow during the 100-year event. Because the modeled beach was relatively uniform, overwash and seaward deposition occurred over a broad area near OID. The simulated flows ultimately eroded a single dominant channel. The same results were observed in the 2-year storm simulations, but on a smaller scale.

As a sensitivity test to assess the role of aeolian transport, the model was run with variable elevation along the lagoon channel between OID and J Street to represent wind-blown sand deposits. The model showed that the high sections tend to erode and the low sections tend to fill, maintaining the lagoon in its current configuration. This exercise helps show how variable infilling from aeolian transport would typically be offset by flow during storms.

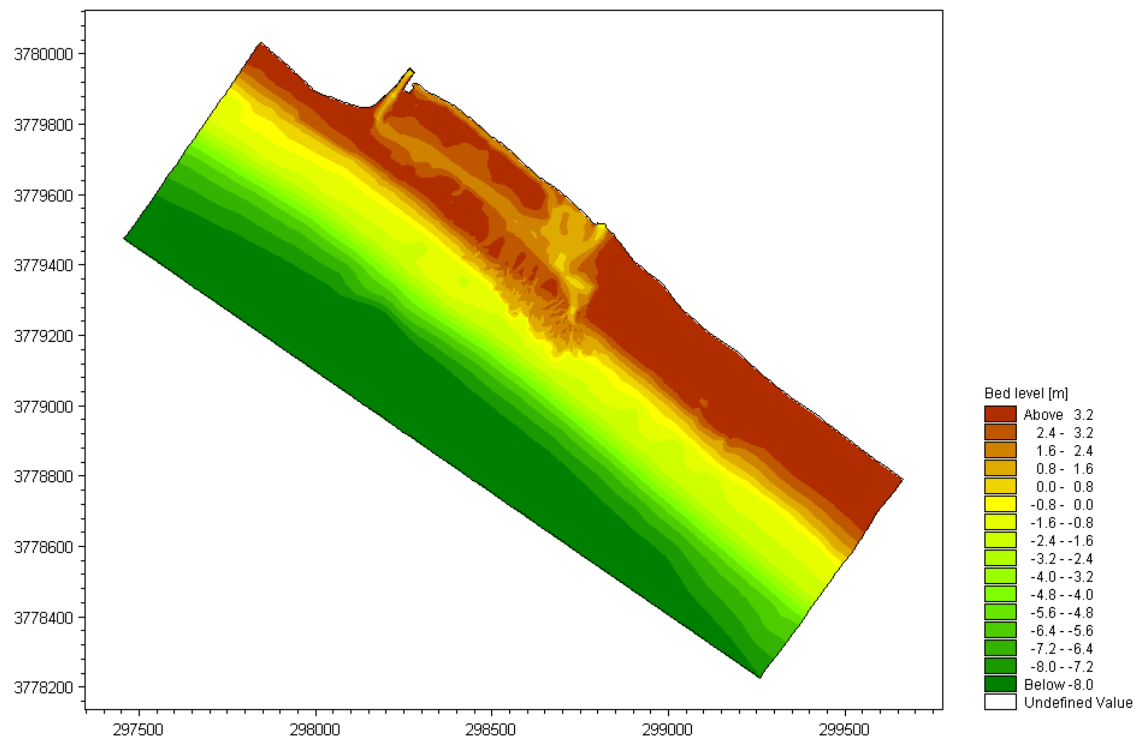

Figure 6.1. Elevation above NAVD at peak flow during the 100-year event.

6.2.1.2. Beach Maintenance

The minimum elevation at which a breach corridor could be maintained without being overtopped by the tide was judged to be approximately 7 ft NAVD. A maintained breach corridor was modeled in Runs 5 through 7. The initial water level was set about 1 ft below the beach elevation to allow sufficient difference between the wet and dry areas within the model at model start. Comparison between the existing conditions and a maintained breach corridor indicated that maintaining the corridor near OID would decrease the maximum water level in the lagoon during a 2-year storm by about 3 ft (see Run 6). The model results suggest that reducing the width of the breach corridor to less than 100 feet would significantly reduce the benefits of maintenance.

Different locations for the breach corridor were also tested with the numerical model. The analysis indicates that the water level is about 1.3 ft lower at J Street when the breach is near J Street as opposed to the breach being located near OID. Historic data and model results indicate that the beach could overtop near OID for less frequent events, even if the beach is lowered adjacent J Street. Maintaining a breach corridor at J Street may lead to loss of connection between J Street and OID and is therefore not recommended.

6.2.1.3. Emergency Mechanical Breaching

Creating a lagoon breach just prior to a storm would provide the lowest water surface elevations in the lagoon at J Street. Creating the emergency breach near OID would most likely maintain the lagoon in its current configuration. However, an emergency breach would temporarily impact hydroperiod.

The initial water level was set to the tidal boundary based on the assumption that the breach had allowed the entire lagoon to drain to the tidal elevation, as described in Appendix B. This assumption states that the lagoon drains to tidal elevation just as it would after discharge from a storm has ceased. There could be areas within the lagoon at higher elevations based on actual bathymetry of the lagoon at time of breach.

At peak discharge with a breach simulated near OID, modeled lagoon water levels at J Street reached about 7.9 ft NAVD for the 2-year storm. With the inlet near J Street the peak water levels at J Street reached 7.2 ft NAVD for the 2 year storm.

During the 100-year storm for the idealized representative beach the inlet forms early in the hydrograph, well before peak flow, essentially creating the same case as modeled for the emergency breach with the inlet near OID. At peak discharge with a breach simulated near OID, modeled lagoon water levels at J Street reached about 11.8 ft NAVD for the 100-year storm. With the inlet near J Street the peak water levels at J Street reached 8.5 ft NAVD for the 100-year storm.

Further investigation into inlet location was carried out to determine the impact of closure of the channel between J-Street and OID. At peak discharge for this case, modeled lagoon water levels at J Street reached about 7.2 ft NAVD for the 100-year storm.

6.2.2. Breach Closure

In addition to developing a better understanding of how the breach forms, we set out to better understand how it closes. There are several processes (long-shore transport, cross-shore transport, wind-blown transport, tidal exchange, runoff, etc.) that likely contribute to breach closure. Breach closure is dynamic occurring over a duration controlled by the ambient forcing. It is hypothesized that cross-shore transport is the predominant mechanism and have employed SBEACH as a tool to test that hypothesis.

The SBEACH model only considers closure due to waves. Tide was included so that the waves acted on a variable water level. Considering the small tidal prism, waves are a more dominant forcing than tidal currents. Therefore, tidal currents in the breach were excluded, although they may act to change the length of time the breach is open. The model helped with examination of the long term wave-induced sediment transport into the breached section of the beach.

6.2.2.1. Closure Model Results

The closure model presents qualitative insight into the process of breach closure. The model shows that waves can cause closure of the breach within a relatively short period of time (within 2 to 4 weeks). Observations also indicate that this time scale is accurate. The dynamic nature of cross shore transport, coupled with variability in waves and tides, makes prediction of actual closure time complex. The model also assumes that there is no additional discharge from the drain or tidal currents. Both of which could act to increase time the breach remains open.

These model results serve to indicate that wave forced cross-shore transport would fill the breach within the timescale witnessed. More exact time for the breach to close and whether or not the breach would stay closed is dependent on the wave conditions at the time as well as other physical processes not included in the model. More detailed model results are included in Appendix B.

7. Discussion

7.1. Lagoon Geography and Morphology

Historic aerial photography, surveys, and maps reveal that the lagoon is naturally dynamic and that flows from OID, J Street and Hueneme drains continue to shape the lagoon. Human modifications to upstream hydrology and beach sediment transport have contributed to the formation of the existing lagoon.

Lagoon morphology is forced by upstream inflow, waves, tides, aeolian transport, and anthropogenic factors. A lagoon appears to have been historically present as part of the natural drainage system of the now channelized OID. That lagoon did not extend to the limits of the current lagoon between J Street and OID.

Both two-inlet and single-inlet lagoon configurations are apparent from the available information. Lack of recent mechanical breaching has contributed to tendency of the lagoon to breach at one location. The breach tends to form near OID where the largest volume of flow originates.

Flow rates peak in the lagoon during breaching, when greater velocities last until the water surface in the lagoon has equilibrated with the ocean tide. After the water surface in the lagoon is at the same level as the ocean, flow is controlled by the tides and ambient flow from the drains. Areas of rapid sediment transport are primarily confined to the breach and narrow sections (channels) in the lagoon.

7.2. Water Level in the Lagoon

Water levels in the lagoon during rain storms is a function of the initial water level, beach conditions (elevation, width), and inflow. Flow rates through the lagoon peak at more than 5000 CFS for the 100-year return period. The initial water level in the lagoon can range from the tidal (ocean) water level up to about 10 ft NAVD (lagoon full). Extreme high tide levels in the ocean frequently exceed 7 ft NAVD but rarely exceed 8 ft NAVD.

Analyses of existing conditions, including results of numerical models and analysis of available data, show that the water level in the lagoon generally exceeds 9 ft NAVD prior to breaching. Lowering the maximum elevation of the beach by creating a breach corridor would decrease the pre-storm water level in the lagoon. In comparison, creating an emergency mechanical breach before a storm provides an even lower pre-storm water level. As the intensity of the storm increases, the upstream flood-reduction benefits of decreasing the pre-storm water level in the lagoon decrease because flow during the onset of the storm acts to create a breach well before peak flow.

7.3. Potential Impacts of Proposed Alternatives

This list of considerations is not intended to be an exhaustive list but rather a brief discussion of some conceptual impacts. The environmental impact assessed at a later time based in part on results of this analysis.

Construction of the drainage system, cessation of mechanical breaching, and other human activities actively influence physical processes in the existing lagoon. Leaving the lagoon in this condition would seem to have the least physical or environmental impact. However, the dynamic nature of the lagoon suggests that it may undergo significant changes regardless of human influences.

7.3.1. No Action

Allowing the lagoon system to continue to breach in an unmanaged manner would result in no significant change to lagoon dynamics or hydroperiod. However, the No-Action alternative does not guarantee that the future lagoon configuration (geometry, water depth, hydroperiod, etc.) would be the same as the existing configuration. For channel design purposes, water levels within the lagoon at 10 ft NAVD must be considered prior to breaching and peak water level of 12 ft NAVD must be considered during a 100-year storm.

7.3.2. Beach maintenance

Maintaining a section of beach at a lower than naturally occurring elevation as a “breach corridor” would control the range of water levels within the lagoon, decreasing the lagoon hydroperiod. Maintaining breach corridors at multiple locations would encourage multiple breaches, reducing the flow between OID and J Street. The flow between OID and J Street is essential to maintaining the shallow lagoon environment that connects the drains.

7.3.3. Emergency Mechanical Breaching

Emergency mechanical breaching allows management of lagoon water levels when needed, reduces impacts to lagoon hydroperiod, and appears to be the most feasible outlet alternative.

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Appendix A – Coastal Processes Review

Various sources of existing information were gathered and reviewed to develop an understanding of the project design, history of the lagoon, the behavior of similar systems, and key physical processes affecting the coupled behavior of J Street & Oxnard Industrial Drains, the lagoon, Ormond Beach, and the nearshore Pacific Ocean.

A1. Wind

Wind data were obtained from the National Data Buoy Center (NDBC). Average hourly winds were analyzed for NDBC Station 46053 offshore Santa Barbara (Figure A.1) for the period of 1996 through 2007 and NDBC Station 46025 offshore Santa Monica (Figure A.2) for the period of 1997 through 2007. Daily average winds at Naval Base, Port Hueneme (Figure A.3) were analyzed for the period from 1996 through 2008 with significant gaps in coverage. Wind directions are displayed following standard meteorological convention.

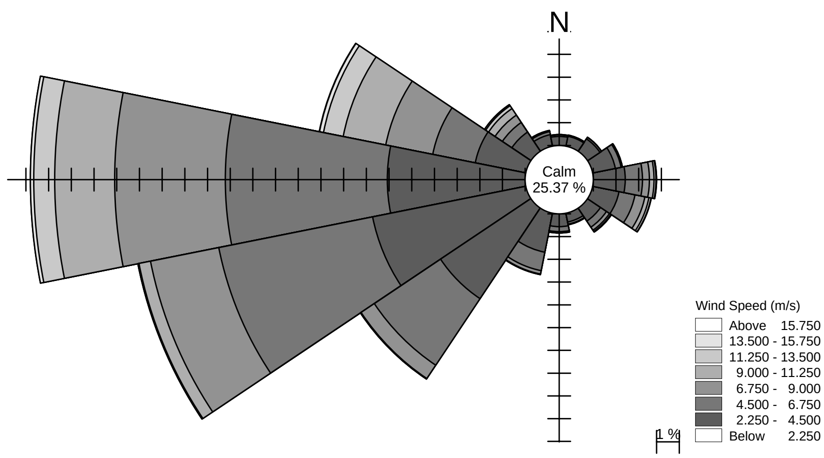

Figure A.1. Wind rose for Santa Barbara, Station 46053.

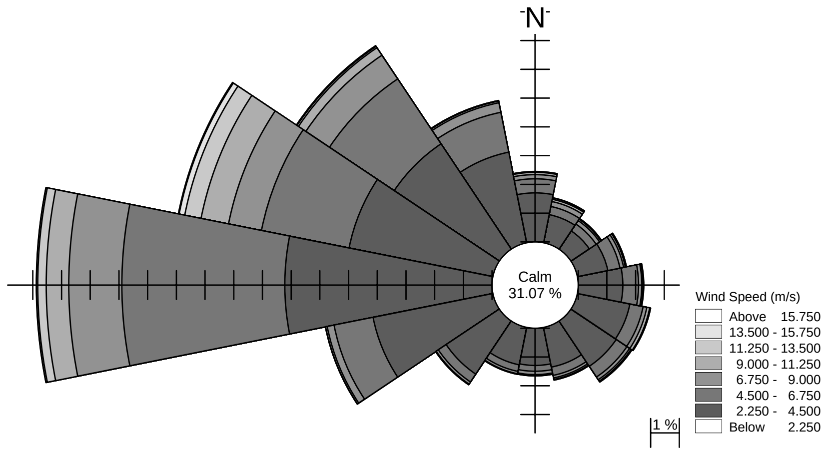

Figure A.2. Wind rose for Santa Monica, Station 46025.

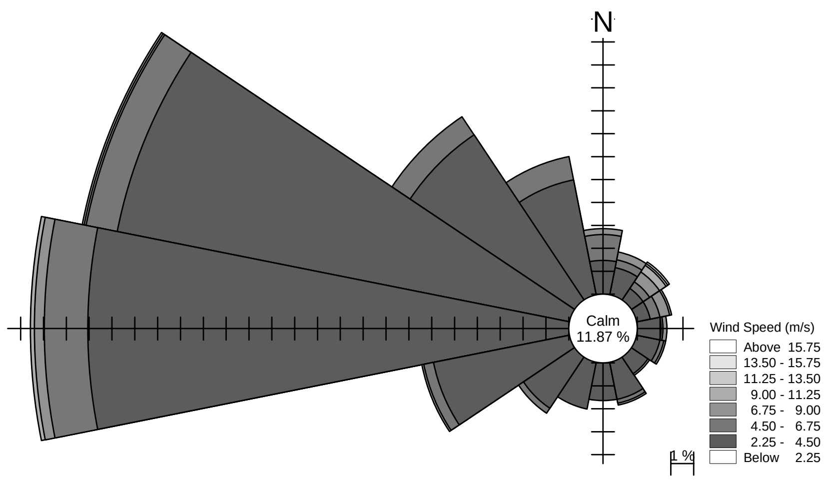

Figure A.3. Wind rose for Port Hueneme.

A2. Waves

Ten years of historical wave data were collected from the NDBC Station 46053 in 1,370-ft deep water offshore of Santa Barbara and Station 46025 in 2,900-ft deep water offshore in the Santa Monica Basin. Wave direction data are not available for Stations 46053 and 46025. Directional wave measurements are available for a one year historical wave record from the Coastal Data Information Program (CDIP) Station 141 in 67-ft deep water offshore of Port Hueneme; these data are plotted in Figure A.4. The Port Hueneme buoy was commissioned in 2007. Typically, a longer record is preferred for wave analysis. Data at CDIP Station 141 were compared and applied with the data at NDBC 46053 and 46025 to develop the statistical distribution of waves at the site.

Wave Information Studies (WIS) hindcast data at Station 91 in 14,500-ft deep water are available for the period from 1981 through 2004 (Tracy 2004). As plotted in Figure A.5, the WIS data show that the waves offshore are predominantly from the northwest.

Due to shoaling and refraction, local wave direction at Ormond Beach does not typically match that of offshore waves. An obvious reason for this is the shape of the coast from Point Conception to the Mexican border. Waves from the north are limited to local generation by the sheltering effect from the coastline. Waves from the west are also limited by the Channel Islands. Winds are predominantly from the west but the longest available fetch is from the southwest. The Channel Islands also cause waves from the west to refract, increasing the percentage of time waves are from the southwest.

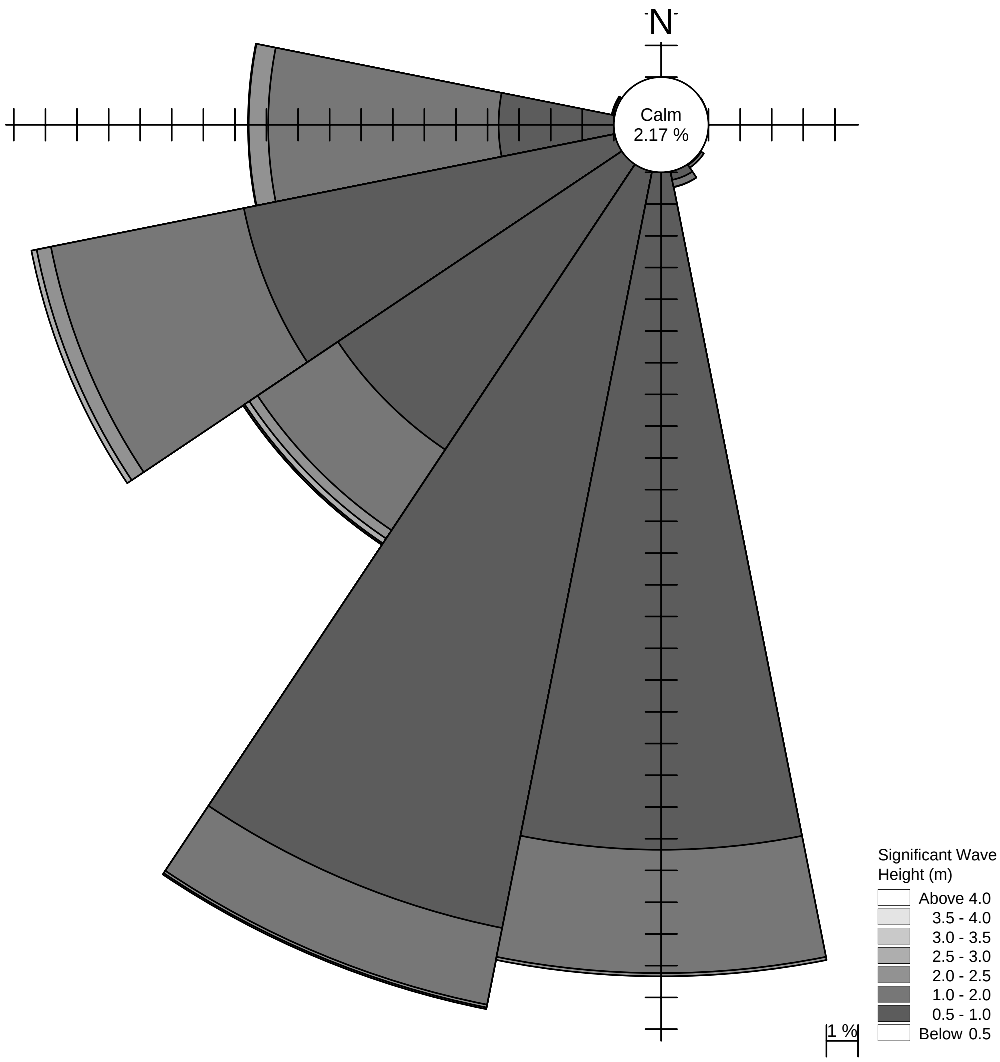

Figure A.4. Wave rose for CDIP Station 141.

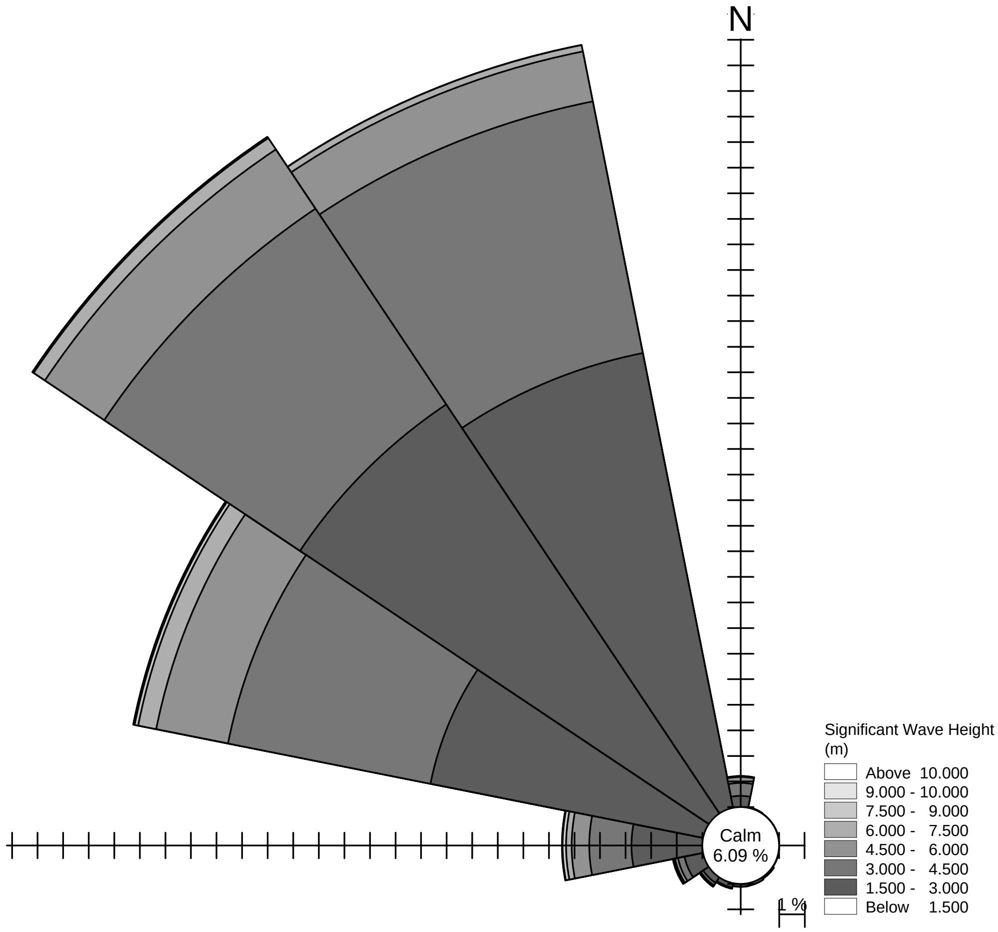

Figure A.5. Wave rose for WIS Station 91.

A3. Tides

Tidal elevation and datum information were obtained from the NOAA tide gauge in Santa Barbara, CA and at the NOAA tide gauge in Santa Monica, CA. The water level analysis, shown in Figure A.6, is based on four years of verified historical data at Santa Barbara, and ten years of verified historical data from Santa Monica. Water level statistics were calculated using the average hourly water level reported at each station. Based on these data, percent exceedance of water level was calculated. Tides in the region are predominately semi-diurnal, with two high tides and two low tides occurring per day. Tidal datums and the greater diurnal tidal range, defined as the difference between MHHW (mean higher high water) and MLLW (mean lower low water), at both gauges are summarized in Table A.1.

Water level data was collected inside the lagoon in an effort to compare with water level at the tide gauges and to calibrate the numerical model. The water level data inside the lagoon would have provided a time history of flow into the lagoon depending on the weather during deployment. Water level in the ocean is generally much lower than the elevation of the beach crest.

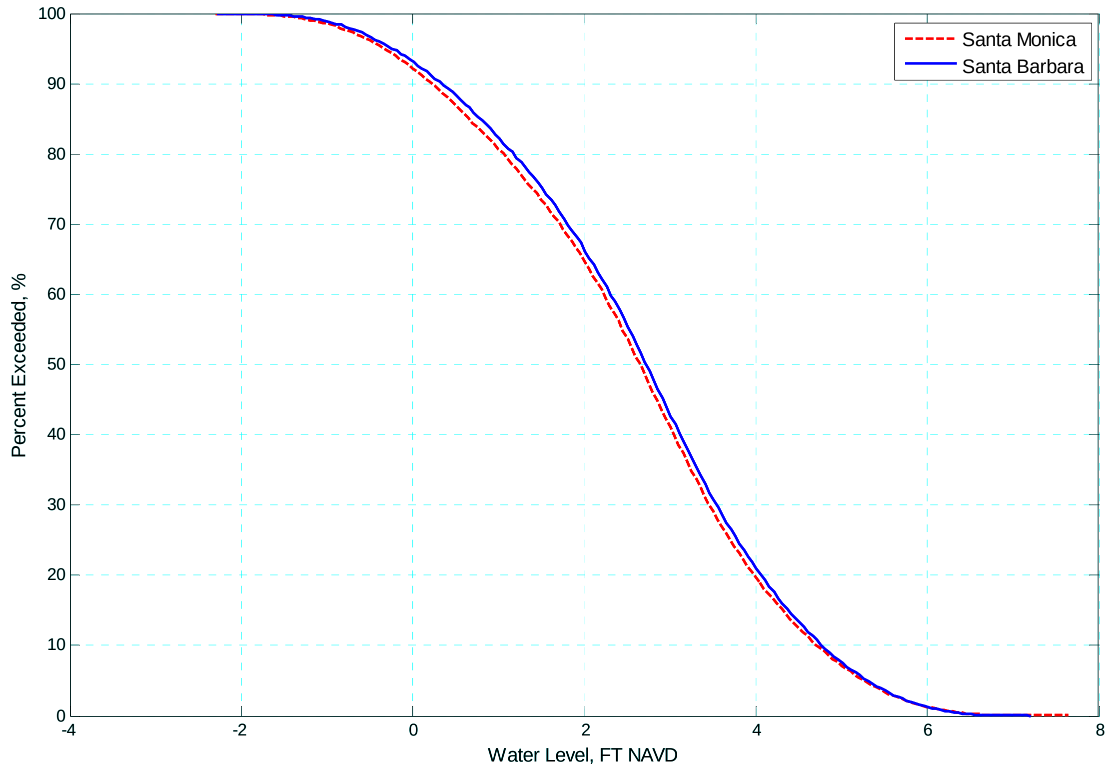

Figure A.6. Water level frequency of exceedance.

Table A.1. Tidal datums and range.						
Station Name	**MHHW, ft NAVD**	**MHW, ft NAVD**	**MSL, ft NAVD**	**MLW, ft NAVD**	**MLLW, ft NAVD**	**Tide Range, ft**
Santa Barbara	5.30	4.54	2.69	0.89	-0.09	5.39
Santa Monica	5.24	4.50	2.60	0.74	-0.19	5.43

A4. Precipitation

Precipitation data was gathered from the University of California Climate Station #156 in Oxnard. Based on eight years of data the average daily precipitation rate near the proposed project site is approximately 0.95 mm/day. Monthly fluctuations are shown in Figure A.7.

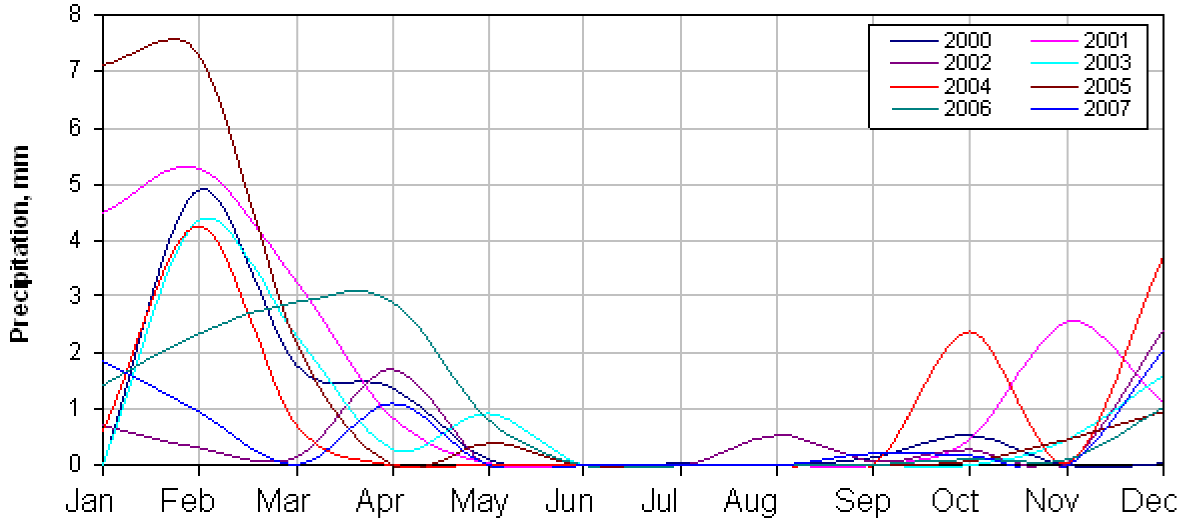

Figure A.7. Monthly precipitation fluctuations in Oxnard, CA.

A5. Upstream Sediment Load

Approximately 332 cubic yards (CY) of sediment per year enter the lagoon from the drainage channels (HDR 2008B). Industrial and Rice Road Drains contribute 95% of the sediment load to the lagoon. Details on the sedimentation study are provided in HDR (2008B).

A6. Longshore Transport

Longshore transport is the movement of sand along a coastline, forced by waves and currents. Greater wave height and/or angle of approach generally cause greater longshore sediment transport. Sediment is carried from sources updrift of Ormond Beach and from Ormond Beach to downdrift areas. Between Santa Barbara and Point Mugu, the net direction of sediment transport along the coast is to the south. Gradients in the rate of longshore transport, such as offshore losses to coastal canyons or deep draft navigation channels, are primarily responsible for beach erosion and accretion trends.

Ormond Beach is located immediately southeast of Port Hueneme, a jettied inlet that interrupts longshore transport. Sand supply for beaches in Ventura County has historically been from Ventura and Santa Clara Rivers. Longshore transport can be significantly interrupted by tidal inlets, especially during the initial phases of inlet development. Approximately 1,100,000 cubic yards (CY) of sand per year are mechanically bypassed south around Port Hueneme (Coastal Sand Management Plan 1989). Campbell and Benedet (2004) report that from 1959 to 1987 about 910,000 CY were bypassed annually around the Port in the direction of net transport. Weigel (1994) has also confirmed a similar magnitude of bypassing and further describes longshore transport in the area.

Without a continuous supply of bypassed sand to Ormond Beach from the north, ongoing transport to the south would lead to more rapid erosion at Ormond Beach and impact the lagoon system. Ormond Beach would switch from being a stable beach (not significantly accreting or eroding) to one with significant erosion. This erosion would likely narrow the beach and expose the lagoon to more frequent inundation from the Pacific Ocean. Lagoon morphology would be altered as the beach retreated and the lagoon became more frequently exposed to wave action and tidal influence.

During a site visit to Ormond Beach on February 12, 2008, HDR observed waves breaking on a small ebb shoal that had developed seaward of the channel following a breaching event. The small shoal was reworked into the beach system, as was observed on March 20, 2008. When a tidal inlet first opens, ebb and/or flood shoal typically forms. Sediment from these shoals comes from the adjacent beach and from the adjacent coastlines as fed by longshore transport. It appears that while the breach at Ormond Lagoon is open, some small component of the longshore transport is trapped in both the ebb shoal and lagoon. The small ephemeral ebb shoal created by episodic breaching has no significant effect on downdrift beaches. However, a permanent inlet (for flood control or restoration) at Ormond Beach would affect longshore transport by temporarily trapping sand in the ebb shoal and lagoon until natural bypassing commences, or by impounding sand against a hard structure such as a rock jetty.

Experience at Port Hueneme indicates that a significant effort would be required to maintain a permanent tidal inlet at Ormond Beach. Therefore, alternatives for improving J Street Drain that include a more permanent connection between the lagoon and the ocean are not advised.

A7. Cross-shore Transport

Cross shore transport refers to sand moving across the wet beach perpendicular to the shoreline (onshore and offshore). On an engineering time scale, cross shore transport is limited to a conceptual depth of closure beyond which waves do not cause significant sediment transport (Dean, Kriebel, and Walton 2002). Depth of closure is a function of the sand gain size, shape of the beach profile, and waves. According to methods by Dean, Kriebel, and Walton (2002), depth of closure can be estimated based on an annual 12-hour exceedance significant non-breaking wave height. Based on a wave record of 2 years, this definition results in a conceptual depth of closure at Ormond Beach of approximately 25 feet. Observations of profile data further support this estimate of depth of closure.

Cross-shore transport plays an important role in moving sand across the shore face, eventually closing the lagoon breach and rebuilding the beach. There are no readily available data or studies documenting cross shore transport at Ormond Beach. However, historic beach profiles at locations immediately north and south of the project site are available as part of monitoring by the BEACON. As presented in Appendix B, SBEACH was applied during the present investigation to estimate cross-shore transport and its impact on profile morphology.

A8. Aeolian Transport

Aeolian (wind-blown) sand transport is likely a significant component of the long term evolution of the lagoon. In the absence of freshwater discharge, wind-blown sand fills shallow areas of the lagoon and narrow tidal channels during dry summer months. From review of aerial photographs and surveys, this process of infilling contributes to the range of locations where the breach has been observed to occur along the beach. Aeolian transport is known to have significantly contributed to closing of small inlets at other locations such as Mustang Island Fish Pass in Texas (Kraus and Heilman 1997).

Visual observations made during the February 12, 2008 site visit indicated that the back beach may be quickly reformed by aeolian transport after breach closure. Representative wind-blown sand transport rates were calculated based on nearby offshore wind data and representative dry beach sand samples based on multiple methods outline by Hopf and Sherman (2007). Figure A.8 shows representative calculations of wind blown sand transport by four different methods for 2006. The analysis indicated that the bulk of wind transport occurs in the winter through spring, and that gradual transport of sand occurs during the summer months.

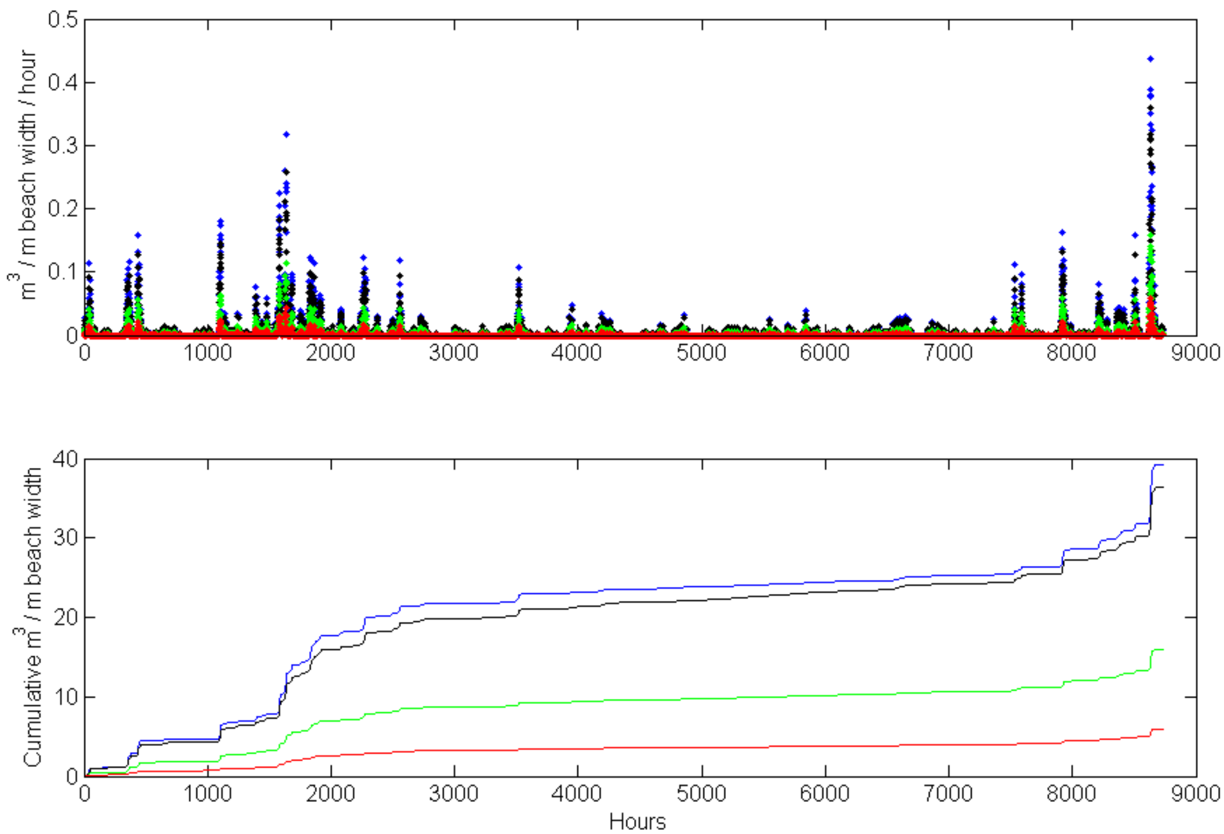

Figure A.8. Four methods for estimating wind-blown sand transport for the year 2006.

A9. Relative Sea Level Rise

IPCC (2007) predicts eustatic sea level rise over the next 100 years between 0.6 ft and 1.9 ft with a central value of 1.1 ft. The National Oceanic and Atmospheric Administration's (NOAA) Center for Operational Oceanographic Products and Services (2008) reports that historic measured relative sea level rise at the Santa Barbara tide gauge is approximately 0.91 feet per century and approximately 0.52 feet per century at Santa Monica. Relative sea level rise is the combined relative change in water level including effects of subsidence or uplift. Ground water withdrawal and oil and gas production have been named as the primary source of subsidence within the Oxnard Plain (Hanson 1992). Hanson (1992) also indicates that tectonic activity is a minor contributor to subsidence and uplift within the Oxnard Plain. Available data indicates that MHHW is rising 19% faster than MSL for a mean anticipated rise of 1.3 ft over then next century (Coastal Conservancy Undated-B).

Sea level anomalies occur when the 5 month average of the interannual variation of mean sea level is greater than 0.3 ft (NOAA 2008). The greatest anomalies on the California coastline are attributed to the El Niño and the 2000-2001 La Niña events (NOAA 2008). The anomaly during the El Niño event raised the sea level for an extended period by as much as 1.2 ft (Ryan *et al* 1999). El Niño has occurred 5 times in the observed data record and its influence is accounted for in the representative beach developed to assess flooding in the lagoon.

Historically, eustatic sea level is rising and the trend is expected to continue in the coming decades. Irrespective of improvements at J Street and Ormond Beach, sea level rise will result in long-term retreat of shorelines in the absence of additional sediment sources. One method to calculate shoreline retreat due to sea level rise was proposed by Bruun (1962). Bruun (1962) suggests that volume within the active beach will be conserved over a long period and provides an equation to calculate shoreline retreat given an equilibrium beach profile. Based on Bruun (1962) the shoreline at Ormond Beach will retreat between 40 ft to 50 ft for every 1 ft of sea level rise.

Naturally, shorelines retreat due to rising relative sea level but the beach profile will generally remain unchanged relative to MSL and shoreline position. If the supply of sediment is unchanged and the beach is not limited on its landward side, then the entire beach system will simply fall back. In this case, the elevation of the dune will be relative to sea level, meaning that if sea level rises by 1 ft then the dune crest will rise by 1 ft as will every other part of the active beach profile. If however, the sediment supply is reduced or development has been allowed to take place behind the beach, the character of the future beach becomes much more uncertain.

Development north of Port Hueneme and the considerable amount of material that is bypassed every year make it likely that sediment supply will be reduced if sea level rises considerably. A reduction in sediment being bypassed at Port Hueneme will lead to shoreline erosion at Ormond Beach. If the beach begins to erode due to lack of available sediment supply, it is likely that the lagoon will be filled with sand as the beach tries to migrate landward, requiring more rainfall to maintain the current lagoon configuration. If climate change results in lower precipitation then it is likely that the lagoon will fill to a state similar to that seen in photos taken in the 1970's.

Considering, that short term (such as El Niño events) sea level rise of 1 ft is already present in the data used to develop the representative beach for analysis purposes, the nominal elevation of the beach adjacent the lagoon will not appreciably accrete over the next 100 years if sea level rises by only 1 ft. It is more likely that shoreline erosion will become a problem over the next 100 years as coastal development continues and beaches attempt to retreat landward. Significant shoreline erosion will likely change the character of the lagoon and increase frequency of breaching but will not likely raise the elevation of the beach.

Sea level rise likely will not significantly affect the elevation of the beach adjacent the lagoon over the next 100 years. Rise of MHHW by 1.3 ft should be considered as the lower limit for lagoon water level during a storm.

A10. Aerial Photography

Historic aerial photography was obtained for the vicinity of the lagoon and photos from 1945, 1950, 1972, 1979, 1989, 1994, 2004, 2006 and 2007 were qualitatively reviewed. A shoreline survey from 1855 and navigation chart from 1945 were also reviewed. Historical conditions indicate that the lagoon was created in its current form by drainage system construction. Continued evolution and growth of the lagoon is expected to be similar to the recent past.

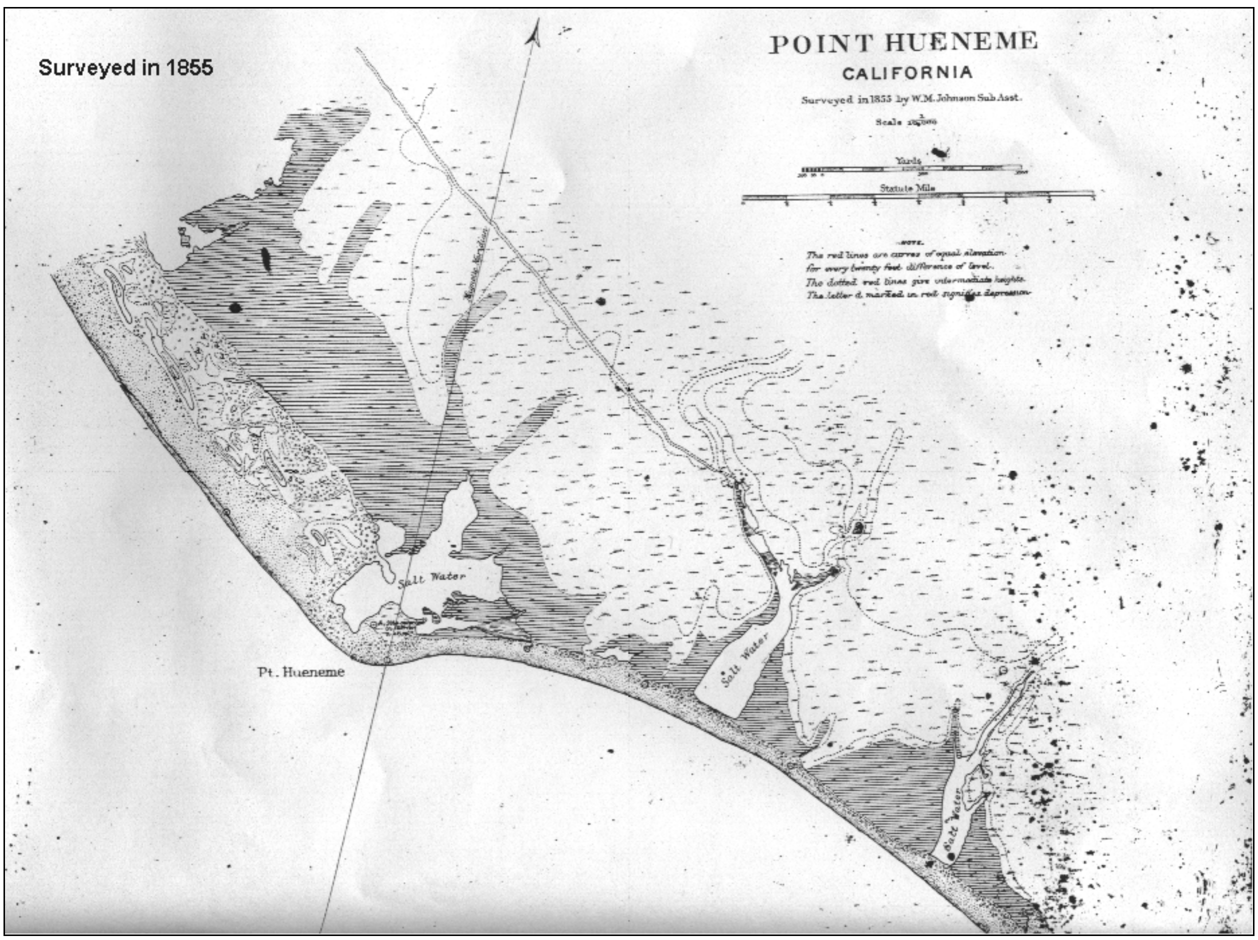

Figure A.9. 1855 shoreline survey (Johnson 1855).

Figure A.9 shows the 1855 survey, indicating that three nearby lagoons existed in 1855. Historically, a natural drainage, wetland, and lagoon existed at the current location of the OID. Portions of this lagoon and wetland features are still evident at the south end existing lagoon. The central lagoon has been filled and no longer exists. Historical drainage is now captured by the Hueneme and J Street Drains. The central and south lagoons likely breached to the Pacific Ocean following large rainfall events but did not remain permanently connected. However, the lagoons may have been open to tidal influence longer than under existing conditions because of the larger tidal prism of the lagoons and adjacent wetlands. The large lagoons shown in Figure A.9 are similar to one of the alternatives being considered by the California State Coastal Conservancy to restore wetland habitat in the area.

Figure A.10. Aerial photograph, 1945 (unknown).

Figure A.10 shows a 1945 aerial photograph, indicating significant development in the region, including filling of the lagoon north of J Street and creation of the Hueneme Drain and Port Hueneme. The lagoon at OID and Hueneme Drain has been channelized, with drainage routed south towards Mugu Lagoon. Prior to construction of J Street Drain in the early 1960's, flows were collected within Hueneme Drain and passed south to the Industrial Drain and beyond. The lagoon only existed at the location of the OID and did not extend northwest to the existing location of J Street Drain. It is unlikely that the OID discharged directly to the Pacific Ocean via Ormond Beach at the time of the photograph.

Figure A.11. Aerial photograph facing J Street Drain, 1972, Copyright © 2002-2007 Kenneth & Gabrielle Adelman, California Coastal Records Project, www.Californiacoastline.org.

Figure A.11 shows the early formation of the lagoon at the outfall of J Street Drain in 1972, ten years after construction in 1962. Available information suggests that during this time the breach was periodically maintained to promote flow from J Street directly to the ocean and mechanically excavated breach is evident in the photograph. The figure also shows that water frequently drained behind the dunes and within dune swales, forming what were likely ephemeral wetlands and channels. Sediment from the dunes was likely transported from the lagoon to the ocean following storm events. Comparison between the 1972 photos and the 1945 photo indicate that discharge from the J Street Drain created the northern portions of the lagoon because direct flow from the drain to the ocean wasn't maintained at all times. Lateral spreading of the impounded water, both north and south, occurred until the J Street drainage flows scoured a path south to the Oxnard Drain, connecting the two systems.

Figure A.12. Aerial photograph between J Street and Industrial Drains, 1972, Copyright © 2002-2007 Kenneth & Gabrielle Adelman, California Coastal Records Project, www.Californiacoastline.org.

Figure A.12 shows the between J Street and Industrial Drains in 1972. Stormwater was no longer routed south, instead it flowed over Ormond Beach. The backwater at J Street Drain had not yet connected to the lagoon at Oxnard Industrial Drain (OID). Channelization of what is now OID likely increased the capacity of the lagoon to convey floodwater directly to the Pacific Ocean and prevented flooding of inland areas. These changes also likely increased the frequency and intensity of breaching by conveying more flow directly to the lagoon and beach rather than allowing for local storage in uplands and adjacent wetlands. The result was the formation of a well defined lagoon channel.

Figure A.13. Aerial photograph facing Industrial Drain, 1972, Copyright © 2002-2007 Kenneth & Gabrielle Adelman, California Coastal Records Project, www.Californiacoastline.org.

Figure A.13 shows Industrial Drain and a small developing lagoon, including portions of Hueneme Drain. Breaching has recently occurred at the lagoon formed by Industrial Drain.

Figure A.14. Aerial photograph showing J Street Drain, 1979, Copyright © 2002-2007 Kenneth & Gabrielle Adelman, California Coastal Records Project, www.Californiacoastline.org.

Figure A.14 reflects the dynamic nature and recent evolution of the lagoon system. The lagoon appeared to have been growing in 1972, but by 1979 only a weak hydrologic connection between J Street and OID had developed. Few permanent wetlands appear in the photographs from 1979.

Figure A.15. Aerial photograph showing the area between J Street and Industrial Drain, 1979, Copyright © 2002-2007 Kenneth & Gabrielle Adelman, California Coastal Records Project, www.Californiacoastline.org.

Figure A.15 shows that, in 1979, Industrial Drain had not recently flowed directly to the Pacific Ocean. The overwash features on the beach evident in 1972 are less evident, although still visible. At the time of this photograph, it had likely been at least a few months since Industrial Drain last breached without mechanical assistance. From the photograph, it appears that mechanical breaching was occurring at OID as well as at J Street.

Figure A.16. Aerial photograph, 1994 (USGS 1994).

This low resolution 1994 aerial photograph in Figure A.16 shows a significantly larger connection/lagoon between J Street and OID. This can be attributed to the lack of mechanical breaching at both J Street and OID. The lagoon appears most similar to the current configuration. Flow from J Street and OID converge at the lagoon in this photograph. The lagoon exists on the beach bounded on its landward side by development and infrastructure.

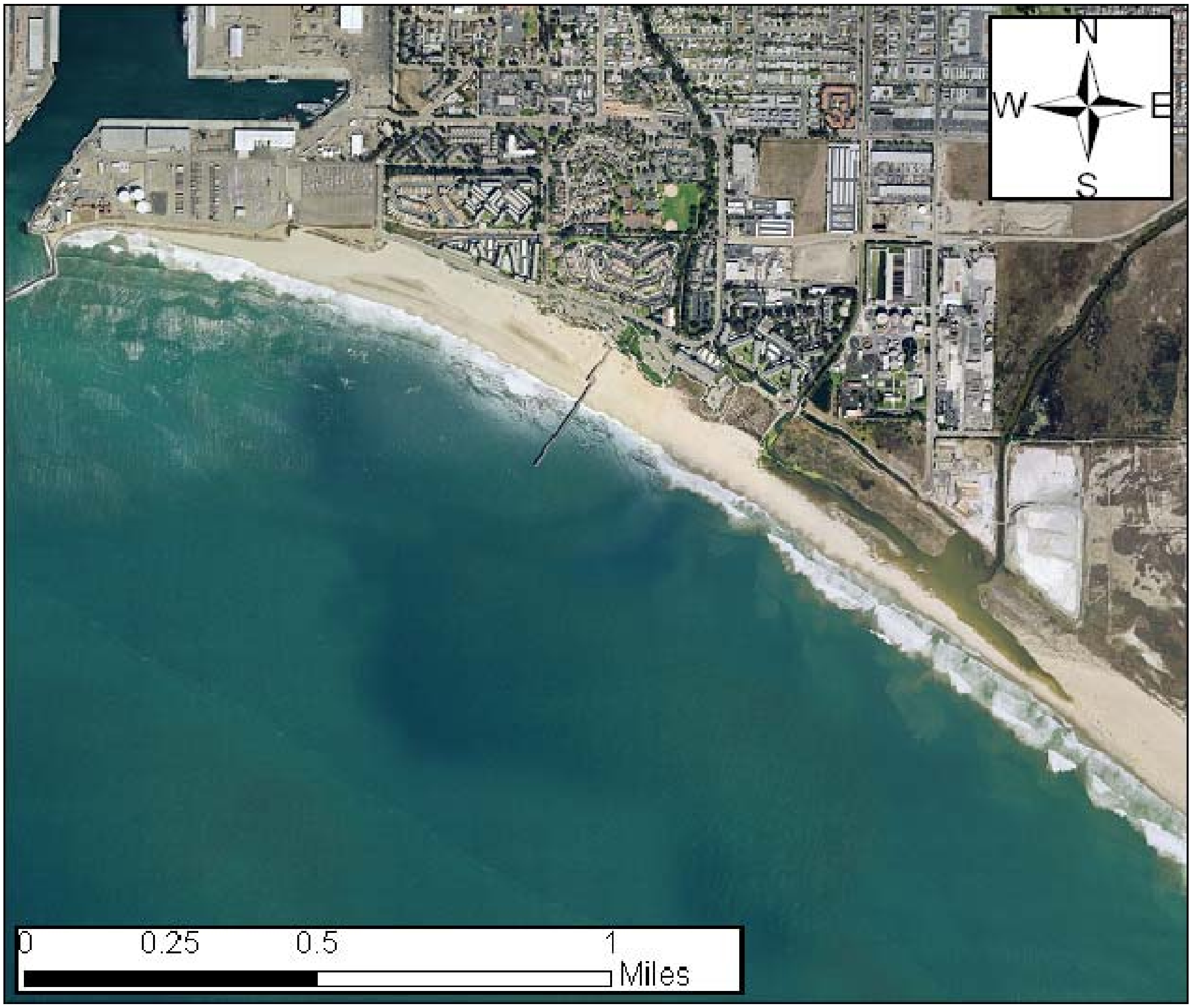

Figure A.17. Aerial photograph, 2007 (VCWPD 2007).

Figure A.17 shows that in December 2007, it had been some time since the system breached to the Ocean as evidenced by the lack of overwash features. In mid December 2007 the beach breached, connecting the lagoon to the ocean near OID. Figure A.18 shows a photograph of the breach taken on December 22, 2007. The photograph reflects significant beach scouring since December, 2007.

The historical aerial photographs support the assumption that the existing lagoon developed as a result of flow from J Street and Industrial Drains ponding on the back side of the beach until a breach occurred. Flow during the breaching events is strong enough across the entire lagoon area to transport sand from the dunes to the ocean. That frequent process, along with groundwater seepage, appears to have caused the gradual formation of a semi-permanent lagoon and wetlands evident today.

Without strong flow within the lagoon during breaching, much of the lagoon area would likely never have developed. It's probable that if both drains were provided permanent, constricted flow pathways through the beach, the lagoon would rapidly decrease in size, ultimately resulting in short channels across the beach instead of a lagoon.

Figure A.18. Breach on December 22, 2007.

Appendix B – Numerical Modeling

B1. Model Approach and Conventions

B1.1. Units, Coordinate System, Datum

All units for modeling results are presented in the Standard International (SI) system in order to maintain consistency within the MIKE21 code. Units in this appendix are all shown in SI to conform to the model standards. Results are converted for the main body of the report. Horizontal coordinates shown are in UTM Zone 11. The project horizontal datum is NAD'83. The project vertical datum is NAVD '88.

B1.2. Direction Convention

Water currents and transport are calculated as component velocities in the positive x and y directions. Current direction refers to direction of propagation (the direction towards which the current flows). Wind and wave direction follow the meteorological convention for input and output, indicating direction of origin (direction from which it travels).

B2. Seaward Breaching

The model was applied to quantify breaching is forced by hydrographs of flow from the channels into the lagoon. Tides are applied on the ocean side of the beach to simulate typical tidal conditions. The primary transport mechanism during breaching is high velocity flow in the lagoon and over the beach. Therefore, waves are not included in the breaching simulation.

B2.1. Software Description

B2.1.1. MIKE 21 Coupled Flexible Mesh Flow Model

MIKE 21 software, developed by DHI, was applied to simulate water circulation, waves, sediment transport and morphology in the lagoon-beach system. MIKE 21 is generally applicable to the simulation of hydraulic and environmental phenomena in lakes, estuaries, bays, coastal areas and seas (DHI 2008A, B, and C).

B2.1.2. Software Limitations

The following model limitations collectively increase the time the model takes to completely form the breach.

- Impermeable Bed
  - Flow can not enter the bed, weakening the sediment and increasing the rate of transport at breaching.
- Supercritical flow
  - The initial breaching flow process is supercritical. Flow during this time is numerically dissipated to simulate dissipation caused by turbulence. Increased

turbulence is not included in the sediment transport solution, leading to reduced transport during initial breaching.

- Wetting and drying
  - o The model requires that the water level in adjacent cells reach a preset level prior to breaching. This acts to slow down the simulated breaching process.

B2.2. Model Domain

The breaching model domain extends from Industrial and J Street drains across the beach to approximately 10 m water depth. Figures B.1 – B.4 show the domain for the representative existing condition, a maintained section of beach elevation, a lagoon with an inlet near OID and a lagoon near J Street, respectively. The breaching model domain was developed to capture morphology of the lagoon system. Very small elements are required across the breach to accurately model the process, significantly increasing computation time. The flexible meshes contain approximately 40,500 elements ranging in size from about 500 m^2 to 1 m^2.

The lagoon is not included in the verification domain to reduce computational time. Verification of sediment transport requires a much finer mesh on the beach face to capture wave induced transport and does not require elements in the lagoon. Figure B.5 shows the domain for the transport calibration model. The flexible mesh contains 36,403 elements ranging in size from about 500 m^2 to 1 m^2.

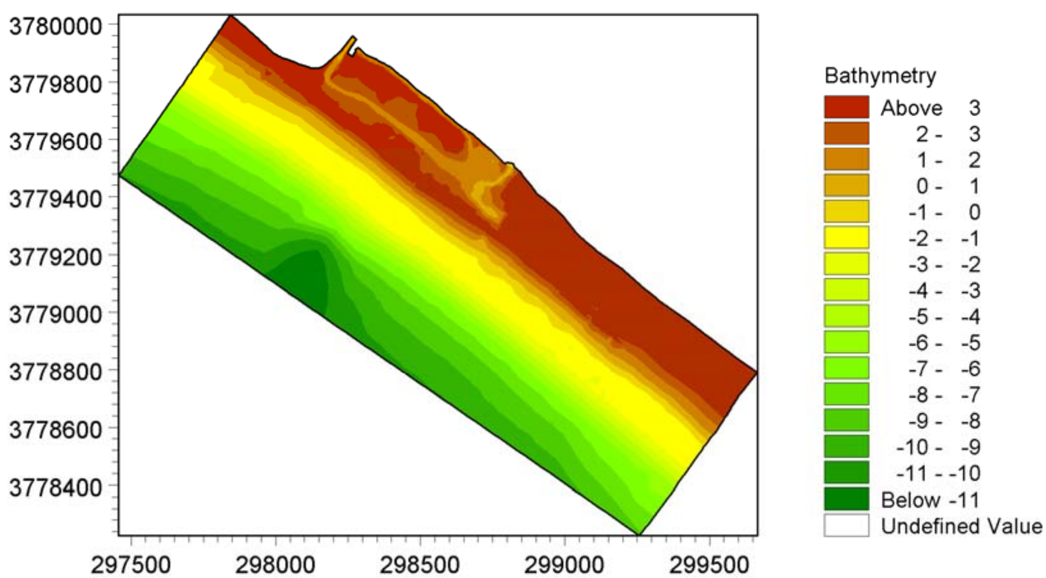

Figure B.1 - Breaching Model Domain, no action idealized beach.

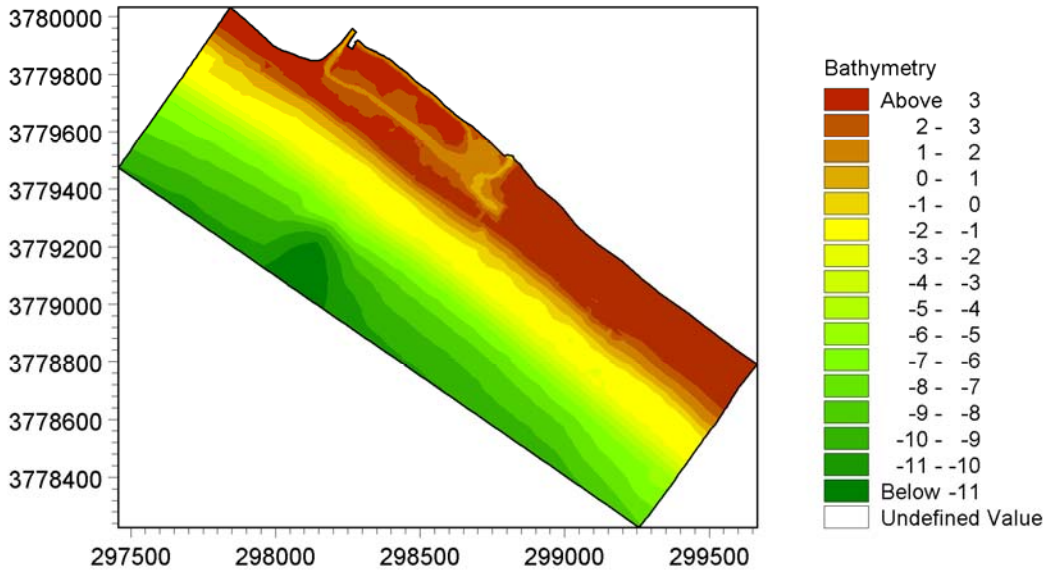

Figure B.2 - Breaching Model Domain, maintained beach.

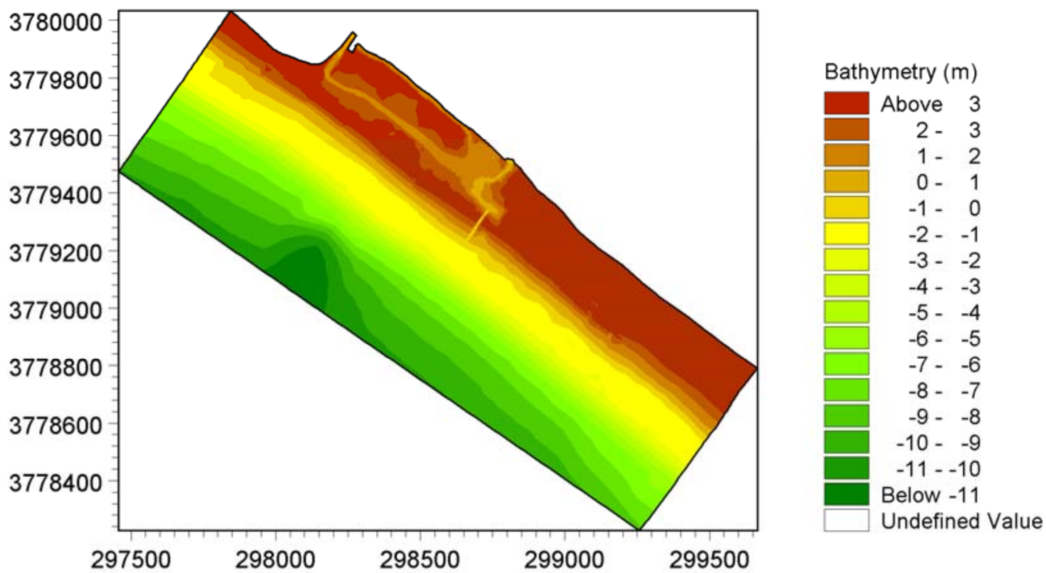

Figure B.3 - Breaching Model Domain, emergency mechanical breach near OID.

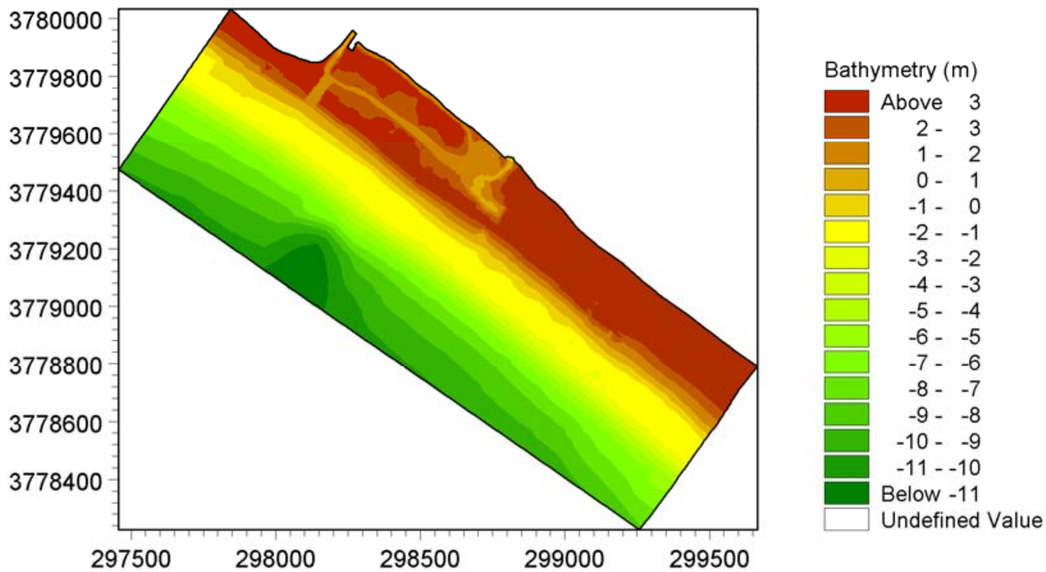

Figure B.4 – Breaching model domain, emergency mechanical breach near J Street.

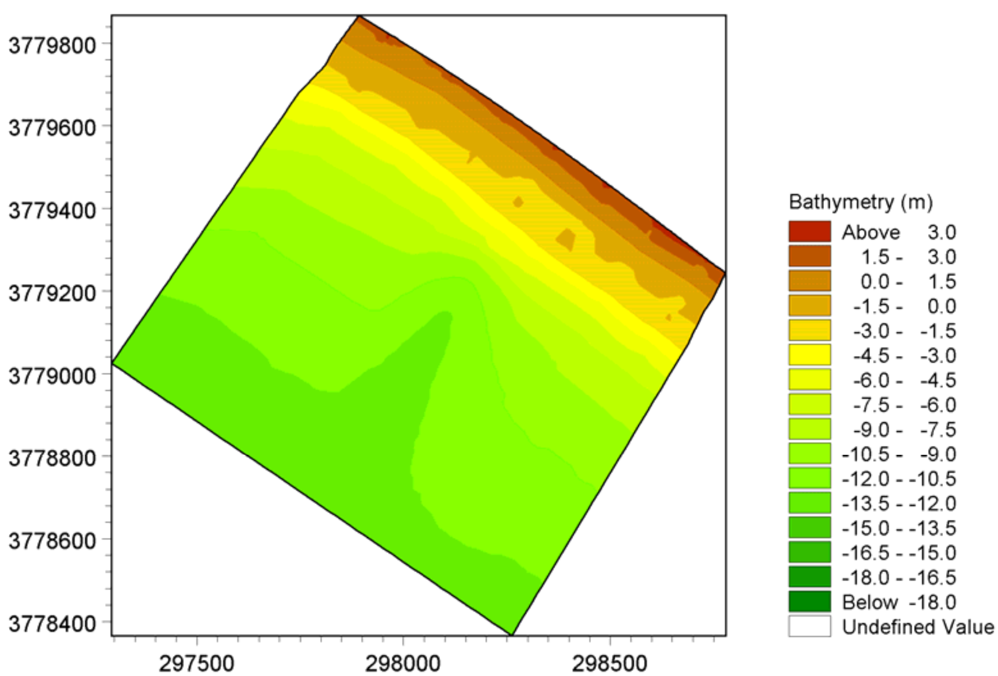

Figure B.5 – Calibration Model Domain.

B2.3. Boundary Conditions

B2.3.1. Hydrographs

Hydrographs for the existing and proposed conditions (Figures 5.10 and 5.11) were provided by VCWPD and applied as boundary conditions within the model at J Street and OID. Flows for J Street and Hueneme Drains were combined.

B2.3.2. Tidal

The open boundary on the ocean side of the domain was forced with a typical spring tide based on tidal observations at Santa Barbara shown in Figure B.6. The tidal boundary condition represents the range of tidal elevations exceeded over 90% of the time. The tidal elevation plays a role in determining the absolute depth to which the breach cuts but has little effect on the breaching process since the lagoon is generally perched above the tide.

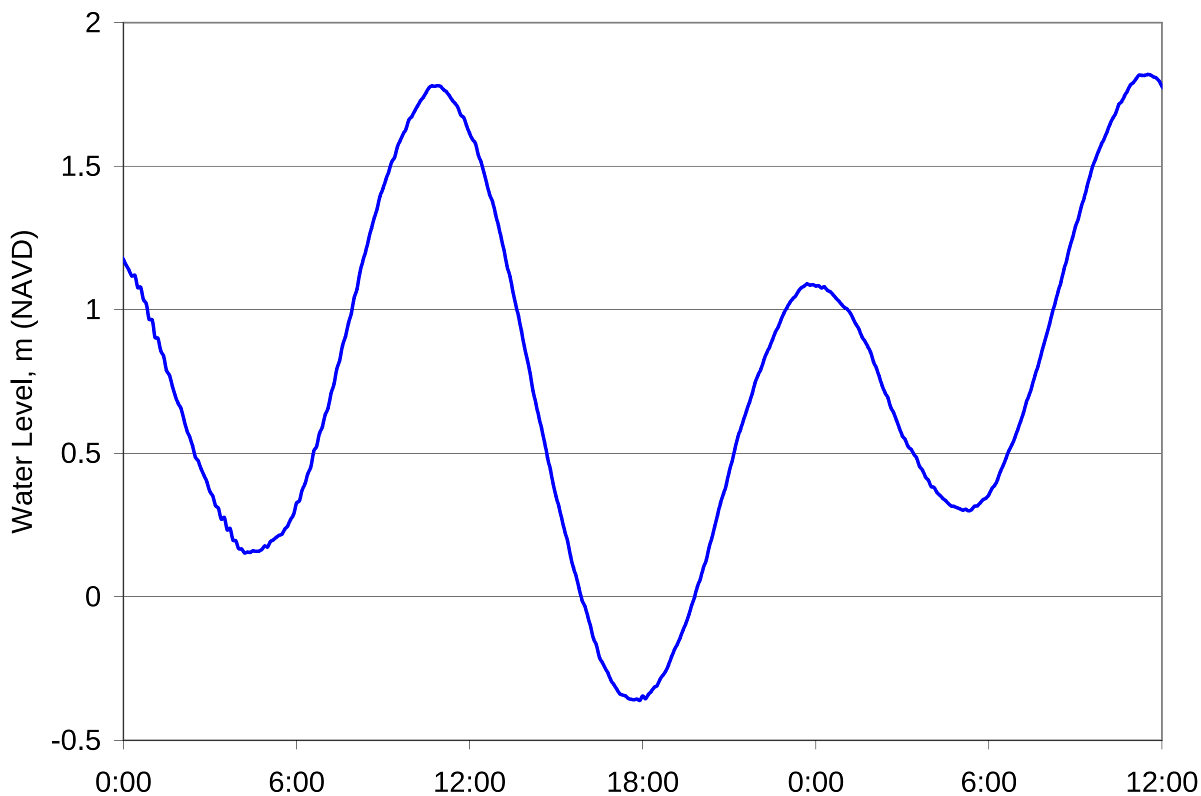

Figure B.6. Water level boundary applied during breaching.

B2.3.3. Sediment Transport

Boundary concentrations for sediment transport are set to zero such that only sediment existing within the domain at model start is available for transport. A deterministic bottom concentration boundary condition is applied (DHI 2008D).

B2.3.4. Waves

Waves from CDIP Station 141 are specified at the open ocean boundary for verification. Lateral ocean boundaries are specified as such.

B2.3.5. Salinity

Ocean boundary conditions in the salinity model are specified at 33.8 ppt, based on collected data. Boundaries at the drains are specified as having 0 ppt for the breaching model. No data is available on the salinity of water discharged from the drains to the lagoon so 0 ppt was assumed to demonstrate the salinity change during storm events.

Detailed analysis of salinity in the lagoon when the breach is closed is not possible with available data. While the breach is closed, other sources affecting salinity dominate (such as ground water, seepage through the beach, heat exchange and rain) making detailed analysis impossible with the existing data.

B2.4. Model Parameters

The horizontal eddy viscosity for the circulation models was applied using the Smagorinsky formulation (DHI 2007A). The Smagorinsky coefficient was set to 0.28 with a minimum eddy viscosity of 1.8e-6 m2/s and a maximum eddy viscosity of 1.0e10 m2/s. Density was included as a function of salinity in the seaward breaching model and was assumed to be barotropic for the closure models.

B2.5. Other Forcing

Wind was not included. The domain is small enough that wind induced flow is insignificant compared to the flow induced by the hydrographs. Closed boundaries (land) were specified as those areas above the maximum expected elevation at which water might reach or areas outside the interest of this report. Elements are allowed to flood and dry as the water level varies.
Since the boundaries to the model are at the exit of the drainage channels rainfall is accounted for by the upstream modeling effort. Local effects of rainfall will likely have little impact when compared to forcing from the drainage channels.

B2.6. Initial Conditions

Water level is initially set to the tidal elevation on the ocean side of the beach, 0.86 m NAVD. The initial water level in the lagoon was set at 6 ft NAVD or 9 ft NAVD and is specified for each completed run in Table B.2, conditions that agree with historically observed conditions in the lagoon and available beach survey data. Initial suspended sediment concentration is set to zero everywhere within the domain.

B2.7. Model Verification

Limited data were available for model verification. Verification was sufficient to develop models to assess changes in the lagoon system based on modifications to lagoon configuration and drainage channels.

B2.7.1. Waves

Waves from CDIP Station 141 are specified at the open ocean boundary. Lateral ocean boundaries are specified as such. Waves from CDIP 141 and tides at Santa Barbara were applied at the boundary of the wave model domain to simulate waves over the sampling interval. Waves were observed across the surf zone and compared to calculations. Calculated wave height and period were within the range of the observations. Figure B.7 shows wave height calculated in the simulation at the time data collection was ongoing.

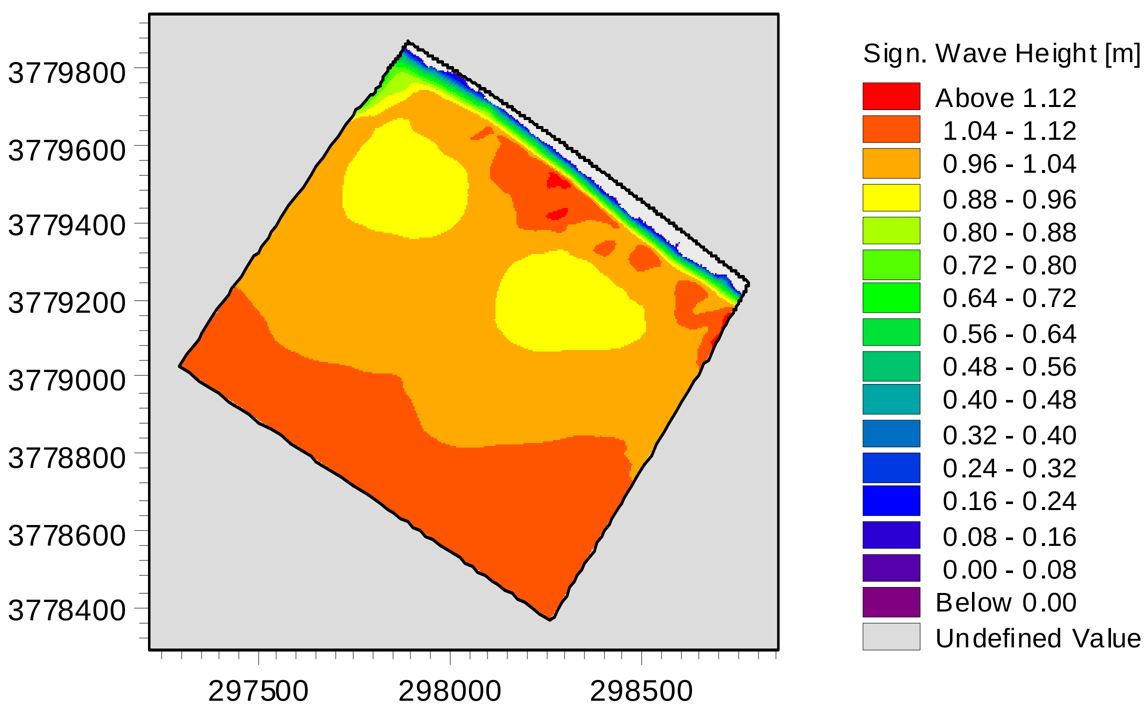

Figure B.7 – Waves during model verification.

B2.7.2. Hydrodynamics

Water level in the lagoon was measured in the hope that a breach would form during the data collection period and allow proper verification of the model: this did not occur. A sensitivity analysis of the velocity over the breach to the specified roughness coefficient was conducted to determine the appropriate Manning's n value. Sensitivity analysis suggested that the roughness would have little effect. A value of 30 was specified based on prior experience with more available data and the results of the sensitivity analysis.

The hydrodynamic model, coupled with waves and sediment transport, calculates wave setup and longshore current. Observations of longshore current ranged from 0.50 m/s to 0.65 m/s. Simulated longshore current during the observation period is shown in Figures B.8 and B.9. The region within which transport was measured is calculated to vary within 0.40 to 0.75 m/s from north to south. The simulated value is within the range of the observations.

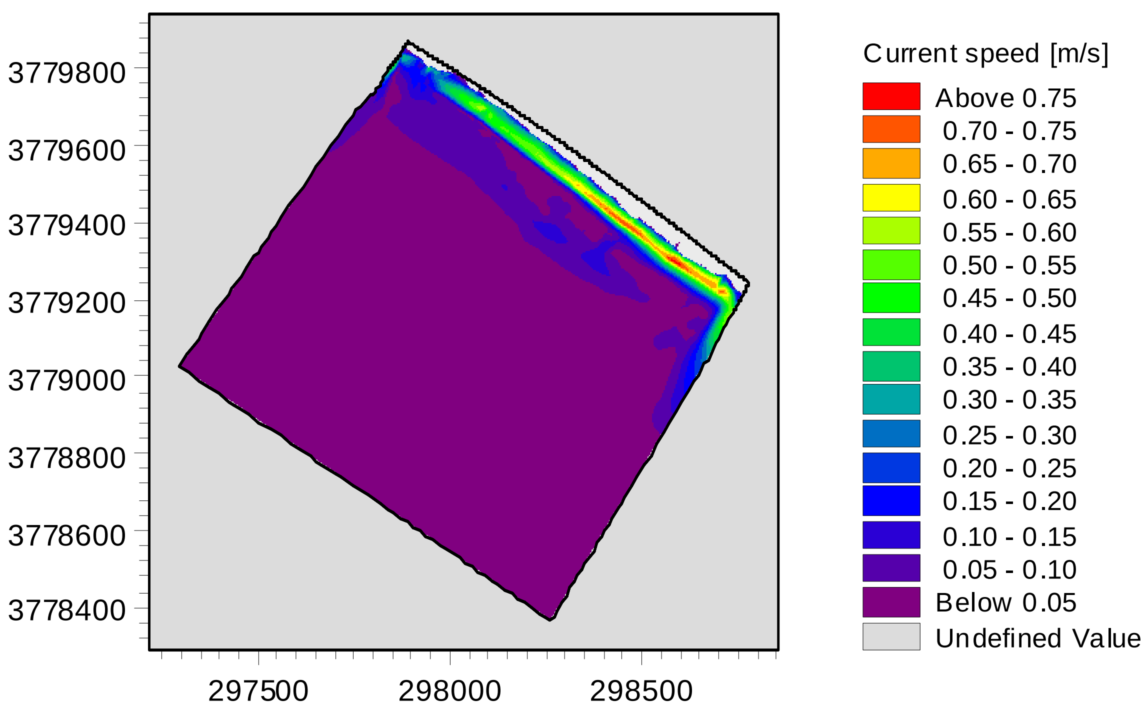

Figure B.8 – Wave and tide induced currents during model verification.

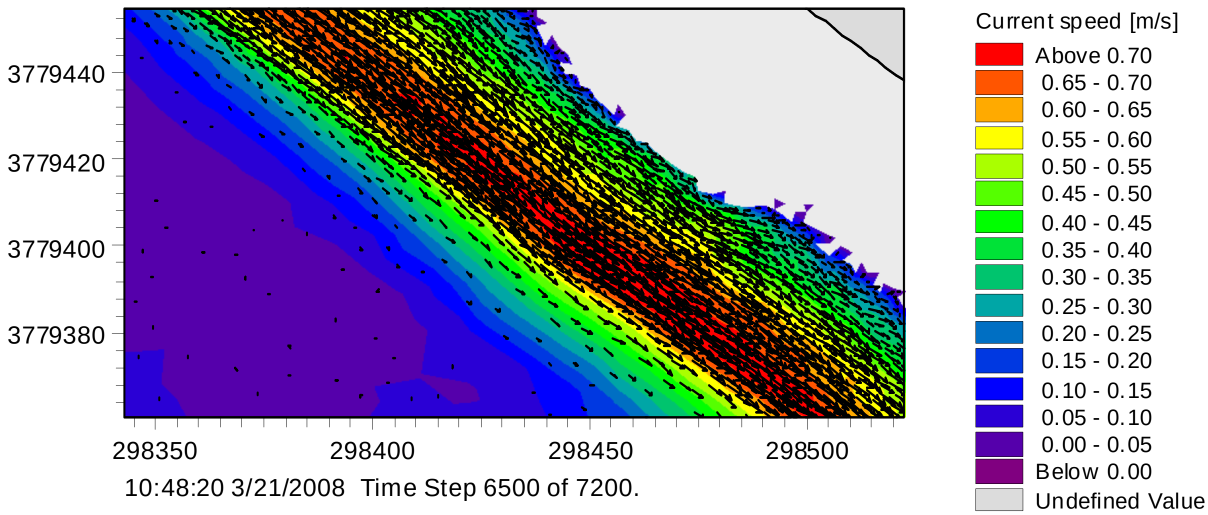

Figure B.9 – Enlarged wave and tide induced currents during model verification.

B2.7.3. Sediment Transport

Suspended sediment samples were collected to verify the sediment transport model. The samples were collected across the surf zone rather than in the breach during a storm event. Verification based on sediment suspended across the surf zone was conducted. Data collected during a breach would be the best possible source for verification, but that data was not available.

The sediment transport model calculates total load which includes suspended and bed load (DHI 2008C) while the measurements only capture suspended load. Bed load must be calculated separately to compare the computed total load to the measured suspended load. Bed load is calculated outside of MIKE21 following the method proposed by Soulsby and Damgaard (2005). Bed load is then subtracted from the modeled total load to determine total suspended load. The calculated total suspended load is then compared to the measured suspended load to estimate model accuracy. Typical sediment transport accuracy is within about an order of magnitude of observations (Davies et al 2002).

Table B.1 shows the comparison between measured and simulated total suspended sediment transport. Figures B.10 and B.11 show the magnitude of total transport near shore. The samples are an instantaneous measurement of suspended transport. Suspended transport can vary significantly over a single wave period based on the periodic oscillations of velocity and acceleration under waves. Given the typical accuracy of sediment transport models, the analysis requirements and the available data and time allowed to collect additional data, the model is considered sufficiently accurate to meet the requirements of this analysis.

Table B.1. Comparison between measured and simulated suspended transport.

Sample	Depth	Wave Height	Total Suspended Load	
			Measured	Calculated
	[m]	[m]	[m3/m]	[m3/m/s]
OL-01-04	1.2	0.7	2.28E-04	6.74E-05
OL-01-03	0.9	0.6	6.30E-04	1.58E-04
OL-01-02	0.6	0.6	1.41E-03	1.80E-04
OL-09-04	1.2	0.9	9.36E-04	1.33E-04
OL-09-03	0.9	0.5	2.43E-03	2.41E-04
OL-09-02	0.6	0.3	4.39E-03	2.15E-04
OL-09-01	0.5	0.3	3.96E-04	1.61E-04

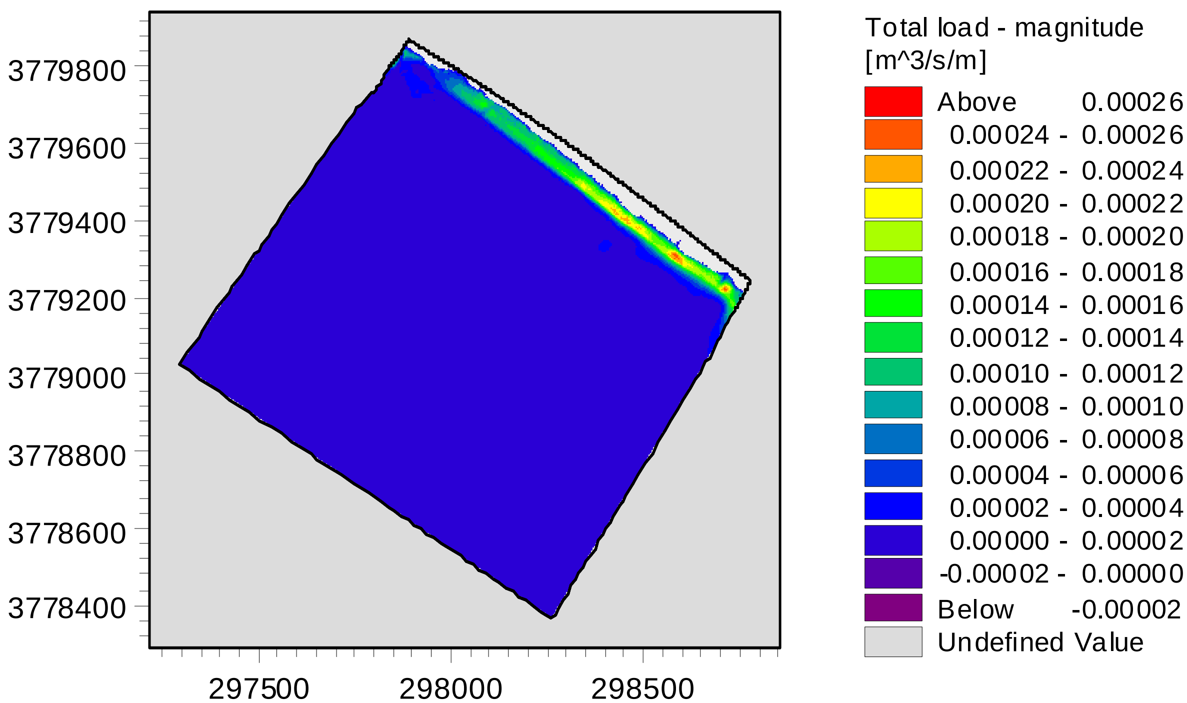

Figure B.10 – Wave and current induced sediment transport during model verification.

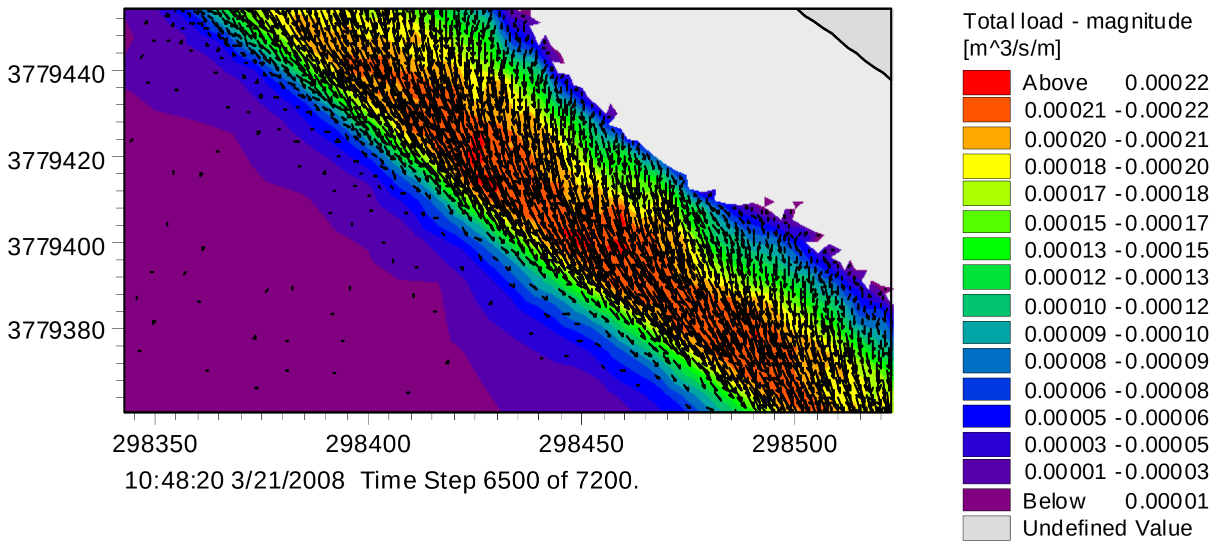

Figure B.11 – Enlarged sediment transport during model verification.

B2.7.4. Salinity

The salinity and water level collected allows a demonstration of salinity during the breaching events. The horizontal dispersion is formulated based on the scaled eddy viscosity (DHI 2008A). Figure B.12 shows simulation of salinity in the lagoon during a 2-year storm with the idealized representative beach just after peak flow.

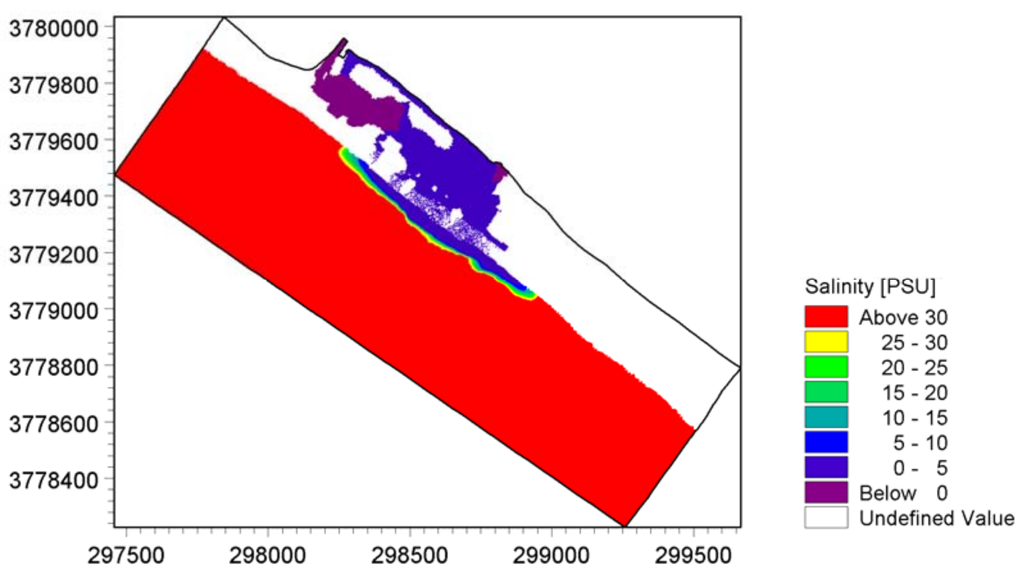

Figure B.12. Salinity at 2-year peak flow over idealized representative beach.

B2.8. Breaching Model Results

The breaching model was run for cases described in Table B.2. The time to breach from start of hydrograph and maximum water level near the outlet of J Street is documented in Table B.3.

Table B.2. Model run definition.

Run	Return Period	Lagoon Initial Water Level, m (NAVD)	Description
1	2	2.74 (9ft)	Rough channel bottom to prove ability to erode channel that has started to fill in.
2	2	2.74 (9ft)	Existing representative beach.
3	2	1.83 (6ft)	Shows breaching time with lower initial water level with existing representative beach.
4	100	2.74 (9ft)	Existing representative beach.
5	2	1.83 (6ft)	50 m maintained section near OID
6	2	1.83 (6ft)	30 m maintained section near OID
7	2	1.83 (6ft)	10 m maintained section near OID
8	2	tide	Inlet near OID
9	2	tide	Inlet near J-Street
10	100	tide	Inlet near OID
11	100	tide	Inlet near J-Street
12	100	tide	Inlet near J-Street and near OID

Table B.3. Model result summary.

Run	Time to Breach	Peak Water Level Near J-Street, m (NAVD)
1	9:30	3.5
2	9:30	3.5
3	10:55	3.5
4	3:40	3.6
5	4:45	2.6
6	4:35	2.65
7	5:10	3.0
8	0:00	2.4
9	0:00	2.2
10	0:00	3.6
11	0:00	2.6
12	0:00	2.2

B2.8.1. Idealized Representative Beach

An idealized beach/lagoon system was developed for the model domain to represent the no-action condition. The dynamic nature of the lagoon ensures that the lagoon configuration at the time of the latest survey was not representative of the longer term trend. The idealized domain was created by altering the March 2008 data through analysis of historic data, ultimately creating a uniform beach elevation where the March 2008 beach was below 10 ft NAVD.

An idealized representative beach was modeled for both the 2-year and 100-year events, with the initial water level varied between 6 ft and 9 ft NAVD for the 2-year event, and with modifications made within the domain to test erosive properties of the flow. Figure B.13 shows the water level at its peak for the 2 year event. Figure B.14 shows the bed level immediately after the 2 year event. Even though the flow overtopped a significant portion of the beach, one primary channel ultimately formed.

The results show that a significant section of the beach is overtopped during the storm and that as the discharge falls and the channel scours the flow tends toward one dominant channel. Figure B.15 shows the bed after the 100-year event, indicating the same trend as seen during the 2-year events but with significantly more transport over the beach. Aerial photos taken after the December 2007 breach indicate that the beach is in fact overtopped over a long section with a majority of the flow tending towards one dominant channel. Bed level change after the 2 year event for the domain with high and low sections included in the channel between J Street and OID shows that the deep sections are filled and the higher sections are eroded, effectively acting to maintain the current shape of the lagoon. Without flow between J Street and OID, natural channel maintenance will not occur.

Peak water level during the 100 year storm (Figure B.16) shows that the water level is slightly higher than the 2 year and overtopping occurs over more of the beach.

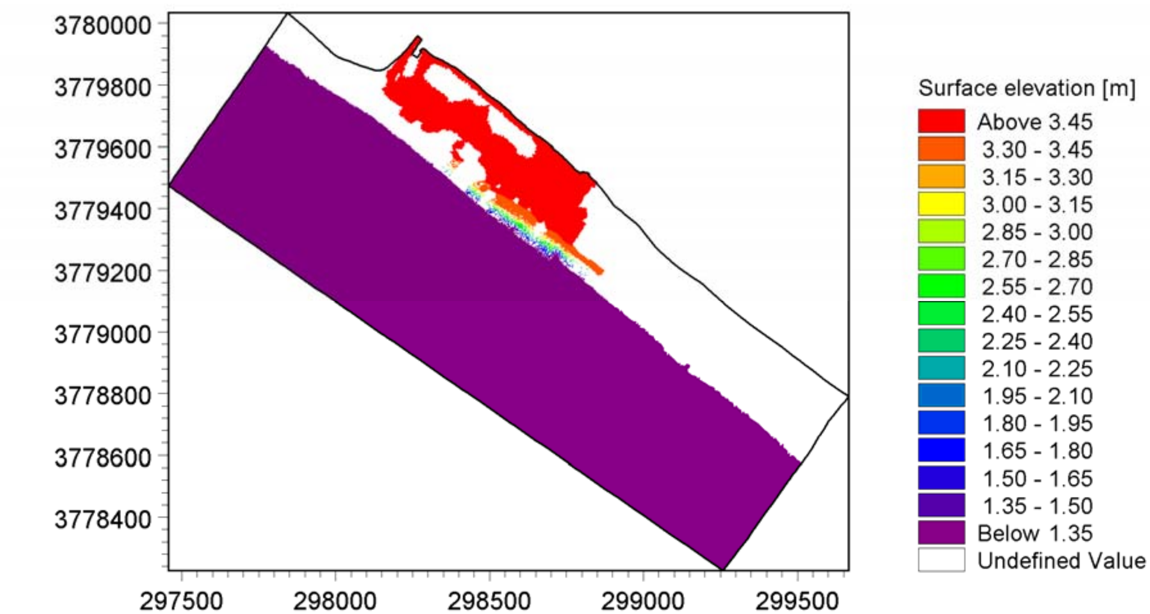

Figure B.13. Peak water level during 2-year event with representative beach.

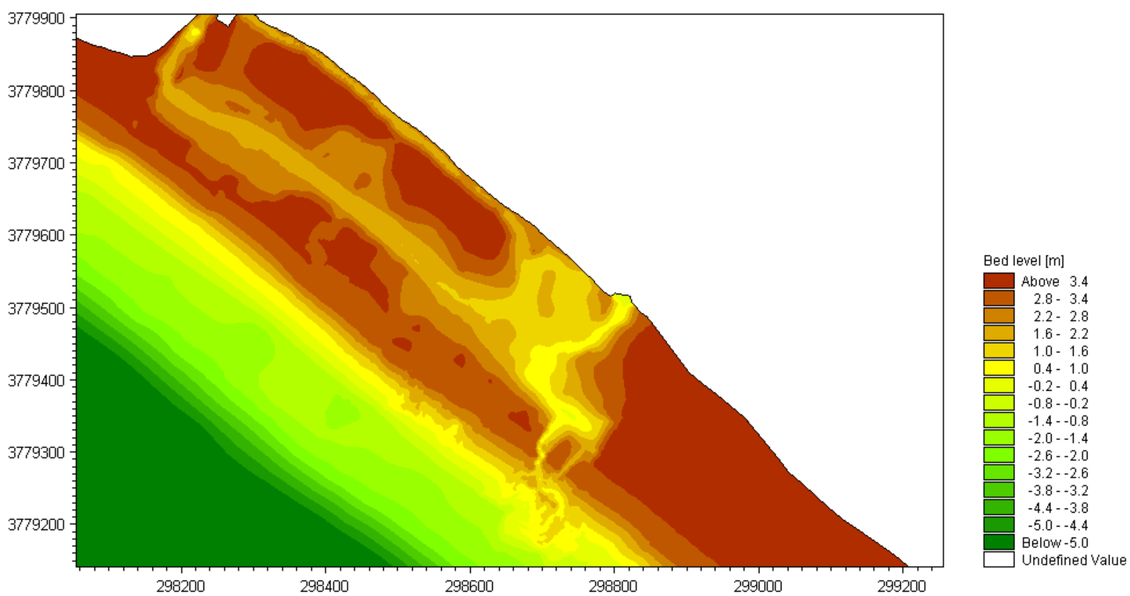

Figure B.14. Bed level at the end of the 2 year storm over the idealized beach.

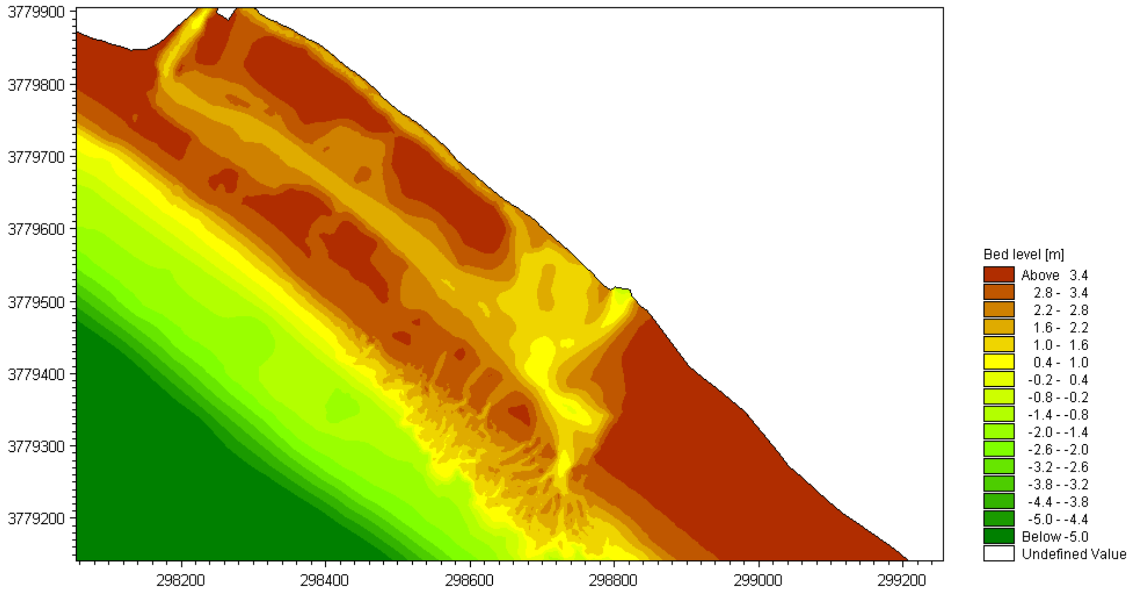

Figure B.15. Bed level at the end of the 100 year storm over the idealized beach.

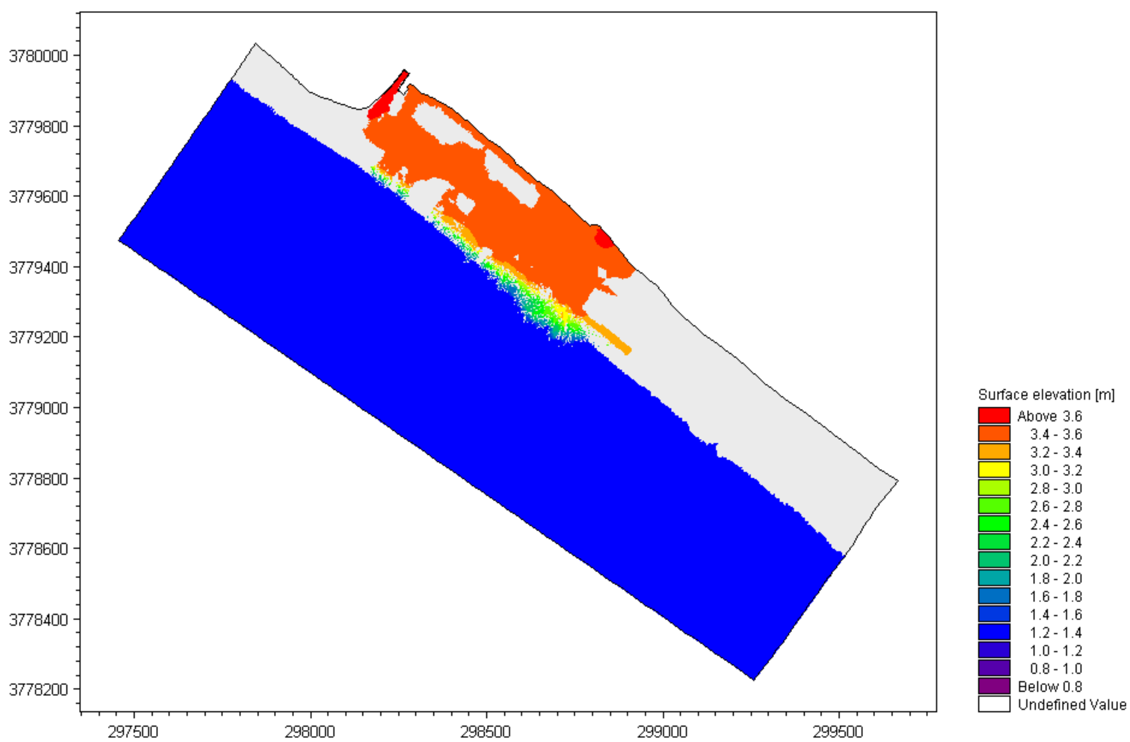

Figure B.16. Peak water level during the 100 year storm over the idealized beach.

B2.8.2. Beach Maintenance

Maintaining a section of the beach to a maximum elevation would cause a breach to form earlier in a storm and lower the maximum water surface elevation achieved at the beach. The minimum elevation to which the beach can be maintained without being opened by the tide is approximately 7 ft NAVD. The recommended location for the maintenance is near OID. Historic data and model results indicate that the beach will overtop at this location even if the beach is lowered adjacent J Street.

Comparison between the existing conditions and a 1 m (3 ft) lower beach section indicate that maintaining the beach near OID will lower the maximum water level in the lagoon during the 2 year event by about 1 m (3 ft). The model suggests that the width of the maintained section would impact the water level during the 2 year storm if the width is less than about 100 ft, but this dependency is affected by model inaccuracy related to the time it takes for the breach to form. Conservatively, the data shows that the low section of beach to be maintained needs to be about 100 ft wide. Figure B.17 shows the bed level after the 2 year event with the beach crest maintained at a low elevation. The result is similar to the existing condition with less overtopping of the overall beach section. Figure B.18 shows peak water level during the 2 year storm for a 30 m wide low section near OID.

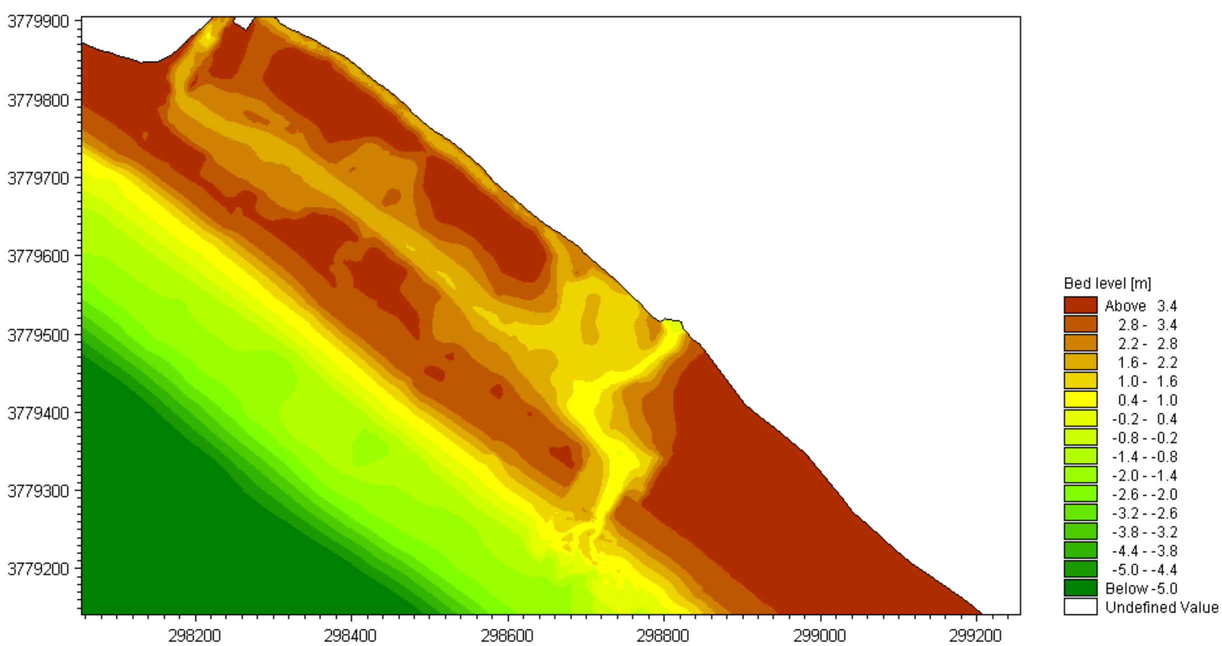

Figure B.17. Bed level at the end of the 2 year storm over the maintained beach.

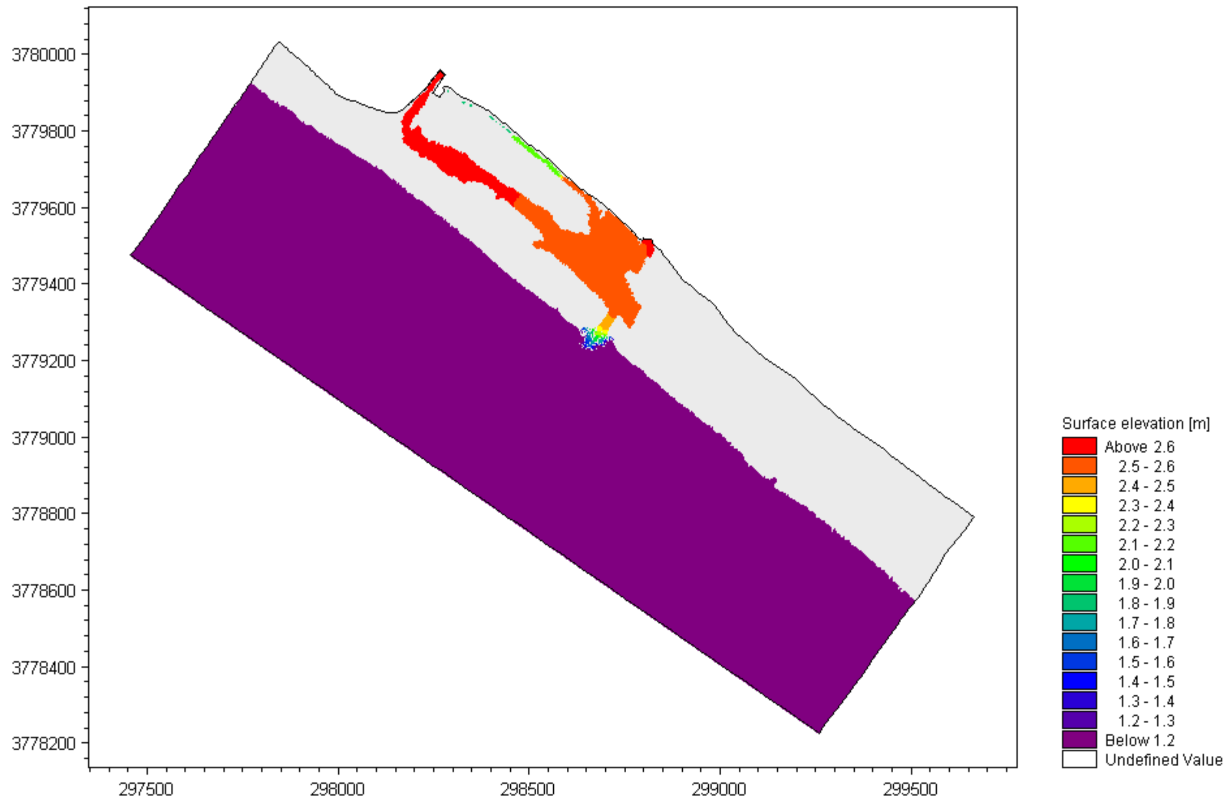

Figure B.18. Peak water level during the 2 year storm over the maintained beach crest.

B2.8.3. Emergency Breach

Figures B.19 and B.20 show the water level in the lagoon near J Street and OID for runs 8 and 9, respectively. The data show that the water level within the lagoon near the outlet fluctuates with the tide but the elevation of the bottom within the channel between J Street and OID prohibits tidal exchange beyond the channel except at the highest tides creating a perched section of the lagoon. Figures B.21 and B.22 show the bed level change in the lagoon after the 2-year event. Scour in the channel between J Street and OID is considerably more pronounced during run 9, driven by higher flow from OID than J Street. Figures B.23 and B.24 show the bed level change in the lagoon after the 100-year event

The breach/inlet will likely best be created by cutting a section of beach from the lagoon to the ocean, allowing the flow to create the breach. The model suggests that the breach would be between 10 m and 30 m wide to a depth determined by the elevation of the lagoon.

The model results show that, for a typical storm, the emergency breach produces the lowest water level of any of the assessed alternatives near J Street and that the closer the inlet is to J Street, the lower the water level there will be. The lagoon near the breach dries in the model runs below about 1.1 m, as evidenced by the missing data in Figure B.20.

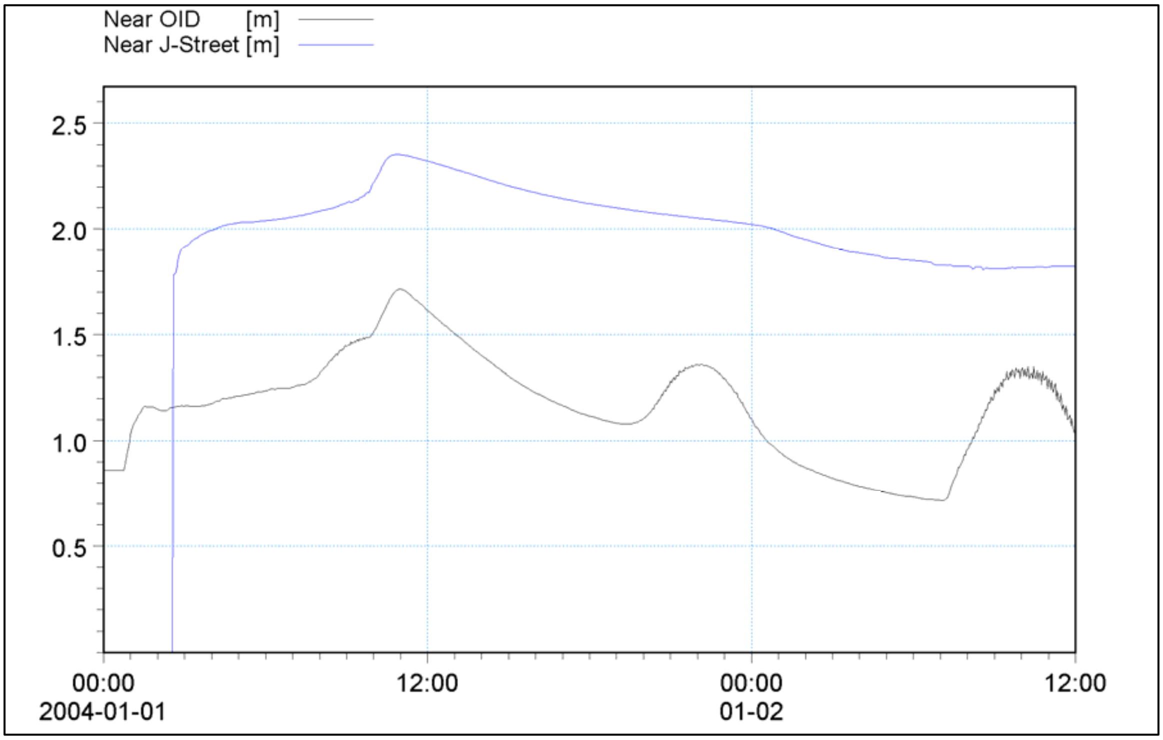

Figure B.19. Water level during the 2-year storm with an emergency breach near OID.

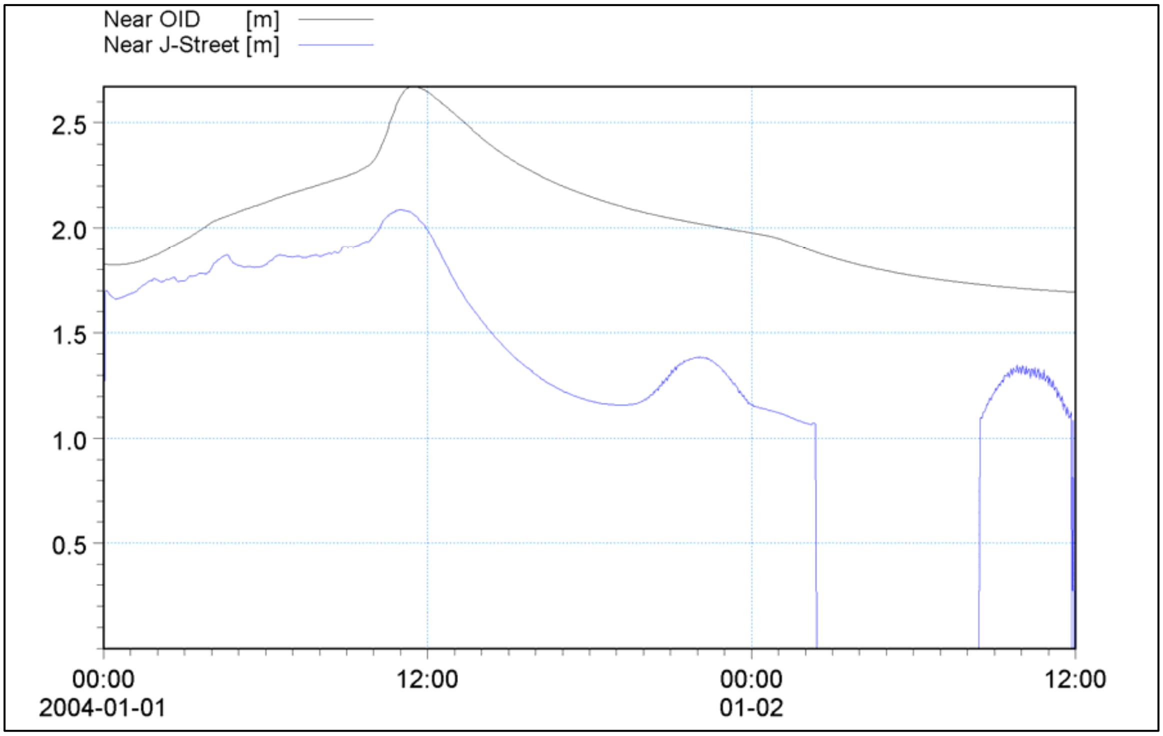

Figure B.20. Water level during the 2 year storm with an inlet near J Street.

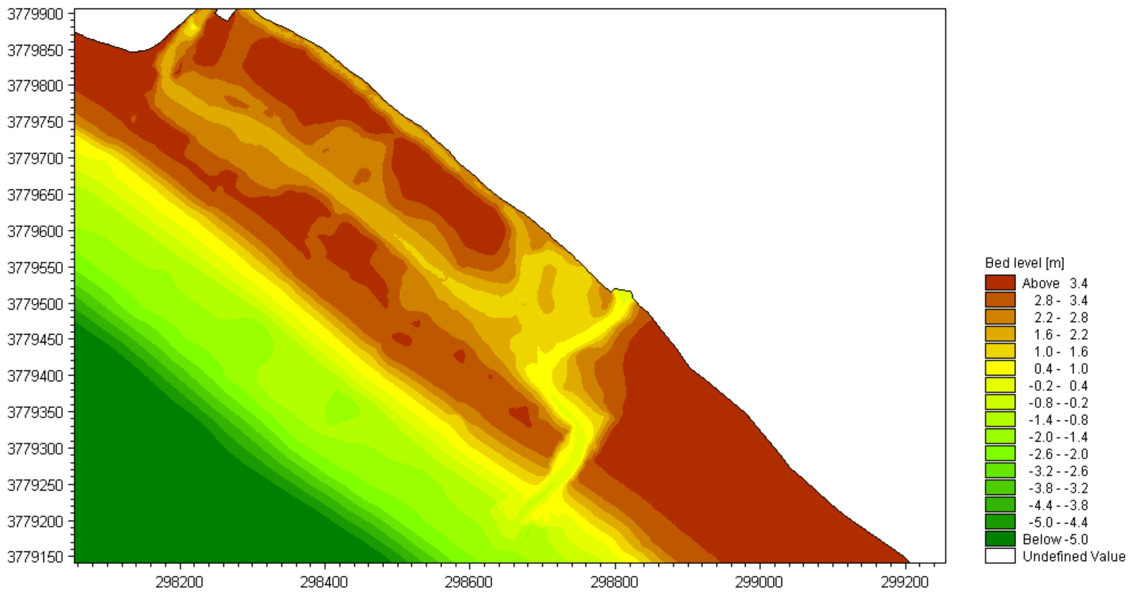

Figure B.21. Bed level after the 2 year storm with an inlet near OID.

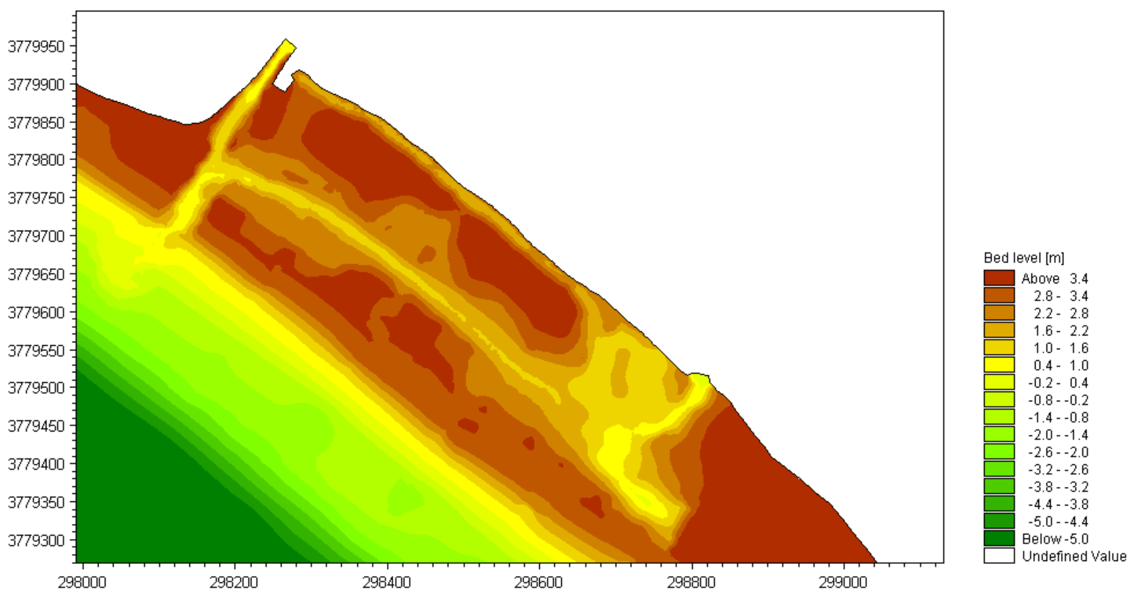

Figure B.22. Bed level after the 2 year storm with an inlet near J Street.

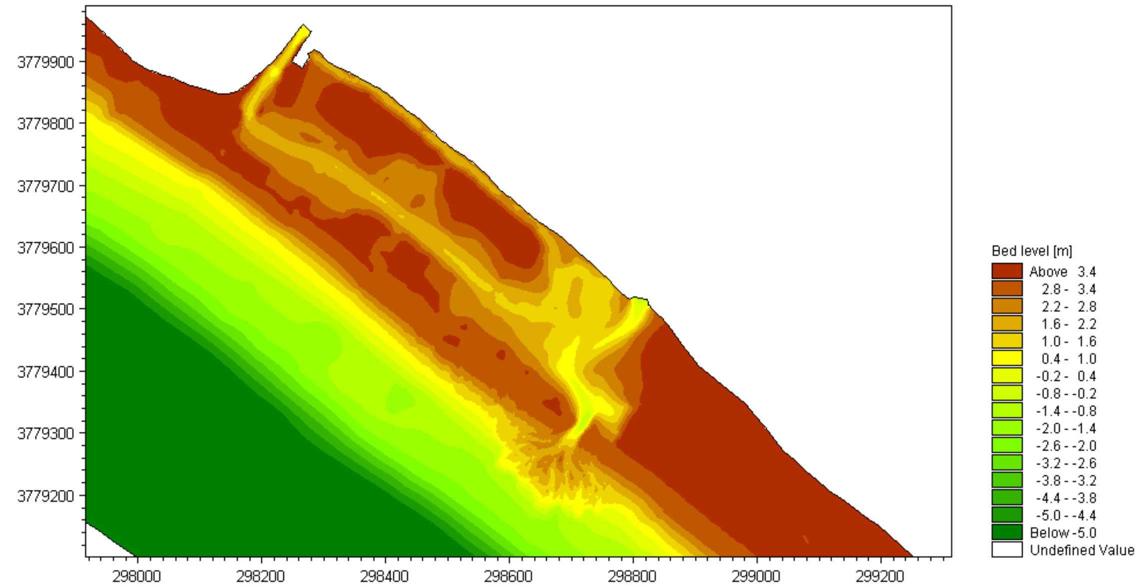

Figure B.23. Bed level after the 100 year storm with an inlet near OID.

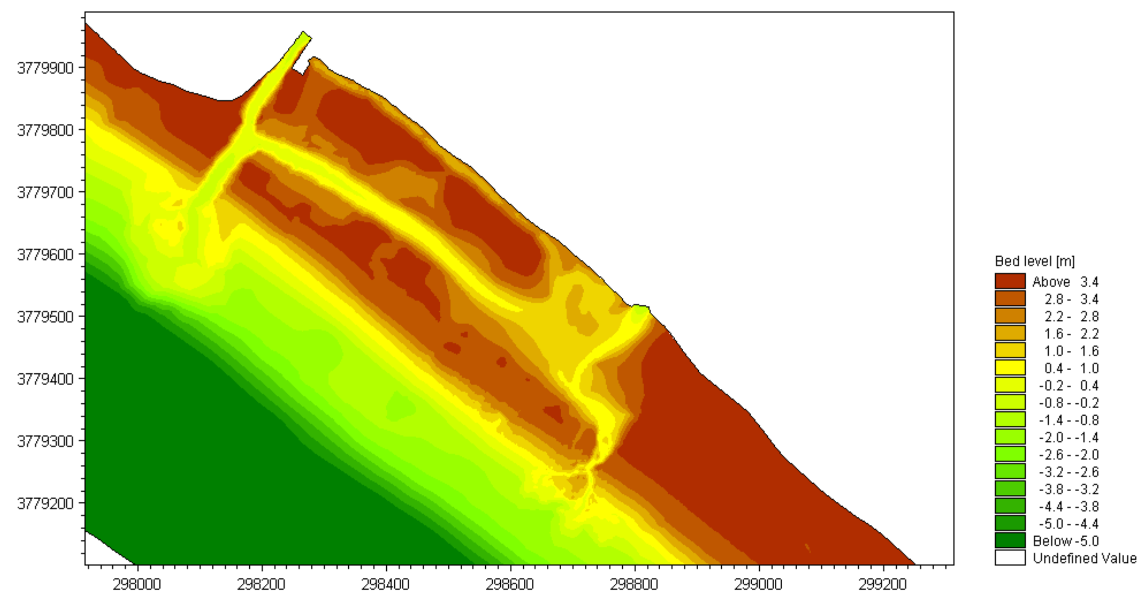

Figure B.24. Bed level after the 100 year storm with an inlet near J-Street.

B3. Breach Closure

In order to obtain a better understanding of the closure process, a cross-shore sediment transport model was applied. Since the closure of the breach is primarily due to the accretion of the beach driven by wave action, the model was forced with offshore waves and the water level varied according to tidal conditions. The model assisted with examination of the long term contribution of wave-induced sediment transport on the breached section of the beach.

B3.1. Software Description

B3.1.1. SBEACH (Storm –induced BEAch CHange)

SBEACH was developed and copyrighted by Veri-Tech, Inc., based on a version prepared by the U.S. Army Engineer Waterways Experiment Station. The model simulates cross-shore beach, berm, and dune erosion produced by storm waves and water levels (Veri-Tech 2008). SBEACH also simulates beach accretion, although it is described as highly qualitative and lacks field verification (James 1992).

B3.2. Model Domain

The closure model domain consists of a cross-shore profile of the beach, extending from an approximate elevation of 4 m to -10 m NAVD. The elevations of the model domain were chosen to extend below the depth of closure and above the highest elevation sediment would be disturbed due to wave interaction. The domain was split into 300 grid cells, each having a grid cell width of 2.5 m.

Sediment characteristics were assumed to be constant along the length of the profile. The maximum slope prior to avalanching was set to 30º and default sediment transport parameters were used.

B3.3. Boundary Conditions

B3.3.1. Waves

Waves are the primary forcing for the closure process as modeled in SBEACH. A time series of wave heights and associated peak periods were taken from Port Hueneme Nearshore wave buoy, NOAA Station 46234 (NOAA NDBC). A time series consisting of wave characteristics between February and March 2008 is used to compare with observations of the beach, performed by HDR and others, and the breach was intermittently open during that time period. Wave characteristics are listed at thirty minute intervals.

B3.3.2. Tidal

The fluctuation of water surface elevation due to tides was also represented in the model. A time series of water surface elevation was taken from the Santa Barbara, CA tide station (NOAA CO-OPS). Tidal forcing within SBEACH fluctuates along the cross-shore profile and affects wave characteristics along the profile. Tide levels are listed at one hour intervals.

B3.4. Initial Conditions

The initial profile input into the model is a cross-shore section of the lagoon-beach system including a breached portion of the beach. The initial profile is a representative cross-shore profile of the lagoon-beach system, after the lagoon water level rises, breaches the beach, and reaches quasi-equilibrium near zero net flow.

B3.5. Closure Model Results

The closure model presents qualitative insight of the process of breach closure. The model shows that waves and tides can force the closure of the breach in a relatively short period of time. As seen in Figures B.25 and B.26, cross-shore transport was simulated for approximately two weeks and one month, respectively. The plots show the initial and final cross section profiles. Difference between the profiles represents accretion of the beach. The two cases presented were both forced with a record of actual waves.

Both figures show significant accretion across the breach. The model results indicate that cross-shore transport would fill the breached section of the beach, although the time for the breach to close and whether the breach would stay closed is dependent on the wave conditions at the time as well as other physical processes not included in the model.

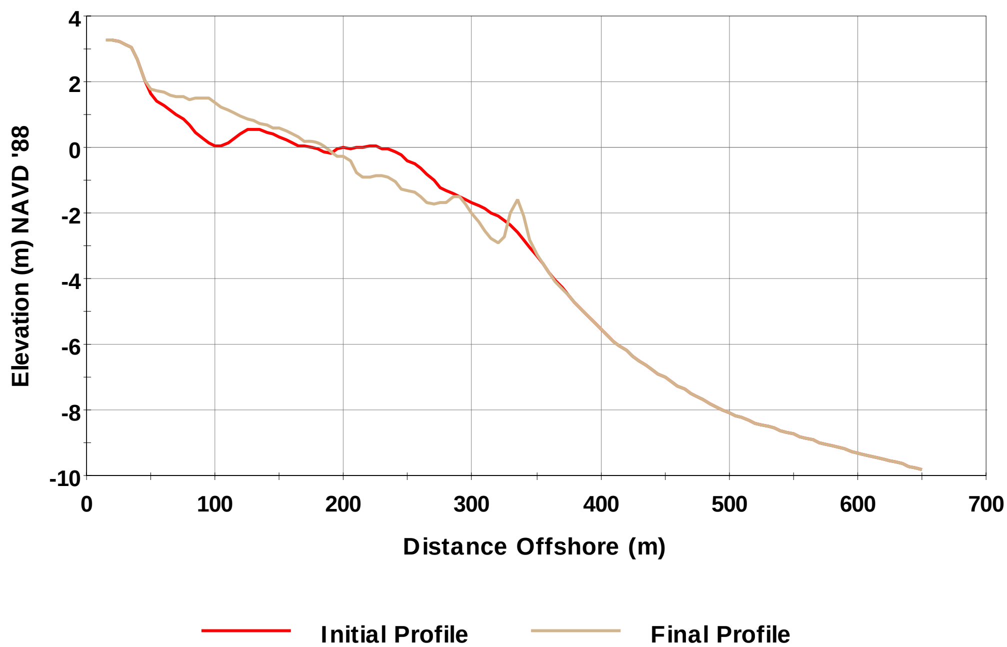

Figure B.25. Modeled cross-shore profile change, March 5 – March 20, 2008

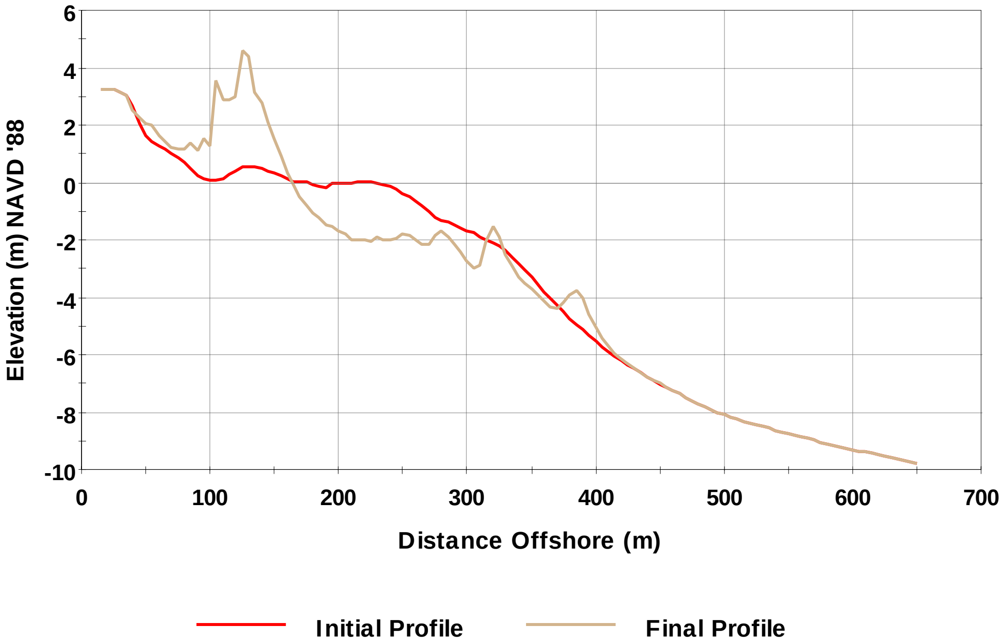

Figure B.26. Modeled cross-shore profile change, February 20 – March 20, 2008