Encinitas-Solana Beach Coastal Storm Damage Reduction Project

San Diego County, California

Appendix B

Coastal Engineering

U.S. Army Corps of Engineers
Los Angeles District

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1 INTRODUCTION

This coastal engineering appendix summarizes the modeling effort, analysis, and evaluation that has been performed to assess project impacts associated with alternatives of the Encinitas-Solana Beach Shoreline, San Diego County, Feasibility Study. Specifically, problems of shoreline and coastal bluff erosion in the Cities of Encinitas and Solana Beach, and the coastal flooding potential along a low lying coastal segment at Cardiff, Encinitas are analyzed for present and expected future scenarios. The following chapters discuss the relevant storm wave climate, coastal processes, and model simulations designed to statistically predict future shoreline evolution, episodic bluff failures and random wave overtopping scenarios to the Highway 101 corridor over a projected design life of 50 years. In addition, estimates of impacts on lagoon sedimentation, surfing, and sand burial of near shore habitats are discussed.

1.1 Purpose and Scope

The purpose of this report is to describe the bluff (e.g., cliff and seacliff), beach, lagoon, and nearshore conditions within the coastal region of the Cities of Encinitas and Solana Beach for both with and without Project scenarios.

1.1.1 Bluff Retreat

The historical oceanographic and climatic environments were characterized over the existing geologic conditions within the study area to assess the vulnerability of the coastal bluffs to episodic failure. The episodic failures are evaluated in terms of the distance of retreat of the upper bluff, herein defined as the bluff retreat, resulting from wave and tidal undercutting at the bluff base for each reach. The estimated upper bluff retreat for each subject reach and the wave overtopping potential at the Highway 101, determined within this appendix, is incorporated into the damage assessment developed within the economic analysis **(Appendix E).** The potential costs to public and private property and infrastructure for the future without Project condition is evaluated along with various alternatives to address identified problems.

1.1.2 Shoreline Evolution

Various beach fill sizes and replenishment rates were modeled with historical coastal geologic traits and historical wave conditions to estimate future without Project and with Project shoreline evolution. Differences between the with Project and without Project shoreline estimates result in project induced net shoreline changes. These net shorelines were used in various subsequent analyses for the following purposes:

1. Net shorelines were used by the economist to estimate recreation and shore protection benefits.
2. Net shorelines were used to estimate the necessary replenishment sand volume associated with various beach nourishment intervals and sea level rise scenarios which, in turn, were used to estimate construction volumes for cost estimates.
3. Net shorelines were used as input to a profile analysis to estimate changes to the gross longshore sediment transport (gross transport) rates which were, in turn, used to conduct a lagoon sedimentation analysis.
4. Net shorelines were used as input to a profile analysis model to estimate sand thicknesses at discreet offshore distances to estimate changes in profile volume for a surfing impact analysis.

5. Net shorelines were used as input to a profile analysis to estimate sand thicknesses at discreet offshore distances which were, in turn, used by the biologist to perform a habitat impact analysis.

1.1.3 Lagoon Sedimentation

A lagoon sedimentation analysis was performed to estimate Project induced changes to the amount and rate of sedimentation and subsequent dredging costs that would be expected with various beach fill projects. The lagoon sedimentation analysis assumes a proportional relationship between changes in gross transport and changes in lagoon sedimentation. As gross transport increases with increasing beach nourishment, lagoon sedimentation is expected to increase. An increase in lagoon sedimentation is a negative project impact, and the estimated costs of removing the sedimentation by dredging provide a valuation of this impact.

1.1.4 Surfing Impact

A surfing impact analysis was performed to estimate Project induced changes to surfing resources within the Project domain. These include positive and negative impacts that could possibly arise in the form of changes to backwash, wave breaking intensity, reef coverage, wave peel angles, wave ride distances, and surfability frequencies. The analysis was quantitative where feasible and qualitative elsewhere, providing sufficient results for reviewers to make judgments as to the quality and extent of Project induced impacts.

1.1.5 Habitat Impact

A habitat impact analysis was performed to estimate the Project induced impacts and subsequent mitigation costs for beach nourishments that have significant impacts. This analysis is briefly described in **Chapter 9 of the Integrated Report** and **Appendix H**.

2 PHYSICAL SETTING

2.1 <u>Geographic Setting</u>

The Cities of Encinitas and Solana Beach are located along the central coast of San Diego County, as shown in **Figure 1.5-1 in the Integrated Report**. San Elijo Lagoon is the dividing feature separating Encinitas to the north from Solana Beach to the south.

Encinitas is approximately 10 miles south of Oceanside Harbor and 17 miles north of Point La Jolla. The City's shoreline is approximately 6 miles long and is bounded by Batiquitos Lagoon in the City of Carlsbad to the north and the City of Solana Beach to the south. The major portion of the shoreline within the City can be characterized as consisting of narrow sand and cobble beaches backed by seacliffs. The southernmost segment at Cardiff, which is approximately 4,920 feet long, is a low lying tidal spit that fronts the San Elijo Lagoon.

The City of Solana Beach is approximately 20 miles north of San Diego and is bordered by the San Elijo Lagoon in the City of Encinitas to the north and the City of Del Mar to the south. The City's shoreline, which is approximately 2 miles in length, is comprised almost solely of narrow sand and cobble beaches fronting coastal bluffs.

2.2 Reach Discretization

To better characterize the coastal bluff and shoreline morphology as well as oceanographic conditions, the entire Encinitas/Solana Beach study area was divided into nine reaches as illustrated in **Figure 2.2-1, Figure 2.2-2, Figure 2.2-4, Figure 2.2-5, Figure 2.2-6, Figure 2.2-7,** and **Figure 2.2-8**. The distinction between reaches is based on differences in seacliff geology, topography, coastal development and beach conditions. **Table 2.2-1** describes the locations and limits of each reach and is detailed below.

Table 2.2-1 Study Area Reaches

Reach	Range		Approx. Length (mi)
	From	To	
1	Encinitas City Limit	Beacon's Beach	1.1
2	Beacon's Beach	700 Block, Neptune Ave.	0.3
3	700 Block, Neptune Ave.	Stone Steps	0.5
4	Stone Steps	Moonlight Beach	0.5
5	Moonlight Beach	Swami's	1.0
6	Swami's	San Elijo Lagoon Entrance	1.1
7	San Elijo Lagoon	Table Tops	1.2
8	Table Tops	Fletcher Cove	0.8
9	Fletcher Cove	Solana Beach City Limit	0.8

2.2.1 Reach 1 – Encinitas Northern City Limit to Beacon's Beach

The northernmost shoreline segment between Batiquitos Lagoon and Beacon's Beach (**Figure 2.2-1**) is approximately 6,200 feet in length and can be characterized as having a narrow to medium sized beach (50 to 150 feet) backed by high seacliffs (approximately 70 feet in height). The bluff top is densely developed with residential structures varying from multiple-family residences to low-density private homes.

The seacliffs along Reach 1 are comparatively stable because the bluff base is resistant to erosion, a relatively flatter upper bluff slope, vegetation cover, and presence of a continuous protective cobble berm. After the 1997-1998 El Nino season, the extent of the existing protective cobble berm was somewhat diminished. The narrow beach has been somewhat widened as a result of upcoast sand replenishment generated from the sedimentation of Batiquitos Lagoon in 1998 and 2000 and sand nourishment placed at Leucadia in 2001 under SANDAG's Regional Beach Sand Project (RBSP).

Small notches developed at the base of the bluff in the mid-1990's and have subsequently been covered by the presence of sand berm resulting from small beach nourishments prior to 2001 (sand from disposal operations of other projects). A site investigation conducted on February 6, 2002 indicated that approximately 18 percent of the properties located along the bluff top have constructed private seawalls for toe protection, many which are made to look “natural” for aesthetic and permitting reasons.

2.2.2 Reach 2 – Beacon's Beach to 700 Block, Neptune Avenue

The shoreline segment between Beacon's Beach and the 700 Block, Neptune Ave (**Figure 2.2-2**) is approximately 1,700 feet in length and includes two inactive ancient faults, namely the Beacons and Seawall Faults. The bluff top is densely developed with residential low-density private homes. This reach can be characterized as having a narrow sandy beach backed by high, steep sea cliffs that consist of hard siltstone and claystone and extend approximately 80 to 100 feet in height. The low bluff face of the southern section (south of 794 Neptune) represents an active landslide and is covered by a wide, thick zone of vegetation extending approximately 40 to 60 feet up from the bluff base.

The stability of the upper bluff is highly questionable along this portion of the reach as severe landslides are evident throughout. Several homes located along the bluff ledge have instituted emergency upper and lower bluff stabilization measures to protect against the catastrophic loss of the entire structure and to prevent the further erosion of the bluff base and the associated landslides that ensue as a result. Examples of upper bluff stabilization include shotcrete tie-back walls and terracing. In addition, several bluff top seaward facing decks extend beyond the ledge of recent bluff failures

The beach was narrow after the 1982-1983 El Nino season as sand was stripped away and deposited too far offshore to return. The sand replenishment from both maintenance dredging at Batiquitos Lagoon and the SANDAG Regional Beach Sand Project at Leucadia has slightly widened the beach and formed a small protective berm at the bluff base. Within this reach, more than one half of the properties are armored with a privately constructed seawall at the bluff base or a reinforced shotcrete wall on the upper bluff.

2.2.3 Reach 3 – 700 Block, Neptune Avenue to Stone Steps

The shoreline segment between the 700 Block, Neptune Ave. and Stone Steps **(Figure 2.2-2)** is approximately 2,600 feet in length and can be characterized as possessing a narrow to medium (approximately 50 to 150 foot wide) beach backed by a high, steep sedimentary sandstone sea cliff (approximately 100 feet high), similar to that of Reaches 1 and 2. The bluff top is fully developed with residential homes along the entire length of this reach.

Seacliffs are comprised of the slightly less erosion resistant Torrey Sandstone Formation. There are several bluff failure areas and wave cut notches, ranging from 2 to 6 feet deep, along the entire reach at the base of the bluff in areas where seawalls are absent. The upper bluff, comprised of weakly cemented terrace deposits, is oversteepened along much of this reach with the exception of intermittent sections where protective seawalls have been constructed along the bluff base and in areas where heavy vegetation throughout the bluff face is visible.

The beach width is much narrower here as compared to Reaches 1 and 2; and, as a result, privately constructed seawalls have been instituted to protect the majority of the homes located along the edge of the bluff top. Along the northern section of the reach, a hybrid co-mixture of seawalls and upper bluff retention structures exist that are not particularly aesthetically sensitive. Some of these upper bluff stabilization techniques include shotcrete walls, as well as a terraced approach coupled with vegetation. Within the southern section (south of 560

Neptune Ave.), several sections of 15-foot high-engineered seawalls were constructed after 1996 when this sub-area experienced severe bluff toe erosion.

2.2.4 Reach 4 – Stone Steps to Moonlight Beach

The shoreline section between Stone Steps and Moonlight Beach (**Figure 2.2-3**) is approximately 2,600 feet in length. Similar to the physical characteristics and urban development of Reaches 1 through 3, the narrow sandy beach along much of this reach is backed entirely by the slightly more erodible Torrey Sandstone. The bluff top ranges in height from approximately 30 feet in the southern portion of the reach, adjacent to Moonlight Beach, and quickly transitions to approximately 80 to 100 feet. Along most of the reach, except for the southern portion of the reach immediately adjacent to Moonlight Beach, an approximate 2 to 4-foot notch exists at the base of the bluff. The prevalent notch development coupled with the already over-steepened upper bluff zone is prone to future bluff failures, some of which could be catastrophic. In fact, it was along this coastal segment where a bluff failure resulted in the unfortunate loss of a human life in 2000.

Within the northern section, two small sections of bluff base are armored with seawalls that were constructed after 1996. Spotty notch fills are also used to protect the bluff base. However, some of the notch fills have been compromised as the bluff has since eroded out from behind them. Within the southern portion adjacent to Moonlight Beach, two patches of non-engineered revetment, probably constructed after the 1982-1983 El Nino season, protect the bluff toe from being eroded away.

The beach conditions are narrow on the northern portion and gradually widen toward Moonlight Beach. The sandy pocket beach that delineates Moonlight Beach is backed by a floodplain that gradually transitions into a cliff formation. Recreational facilities such as a lifeguard building and restrooms are located within the floodplain. The low lying plain and the associated beach width within Moonlight Beach are highly subject to wave attack particularly in response to large storm events. During these events, the back beach is subject to flooding and structures are susceptible to damage, as was the case during the winter of 1982-83. As a mitigation measure, the City constructs a protective temporary sand berm annually during the winter months to prevent flooding and potential damage to the City's facilities.

2.2.5 Reach 5 – Moonlight Beach to Swami's

The shoreline segment extending from Moonlight Beach to Swami's (**Figure 2.2-4**) is approximately 5,400 feet in length and contains a narrow to nonexistent sandy beach with a very thin sand lens backed by the predominant high, steep sea cliffs representative of the Encinitas shoreline. The development along the bluff top consists of high-density residential structures and the Self Realization Fellowship (SRF) property (Swami's) is located at the southern boundary of the reach.

The bluff ranges in height from approximately 30 to 80 feet and is comprised of different low cliff-forming formations. The northern one-third section is comprised of Torrey Sandstone, while the remaining section is comprised of the Del Mar formation, which is slightly more resistant to wave abrasion. The upper most sedimentary formations are comprised of moderately consolidated, weakly cemented marine and non-marine terrace deposits. This formation has a sloped face as it typically becomes highly unstable at angles steeper than 60 degrees. In addition, groundwater percolates through the porous upper weakly cemented sandstone and then flows along the contact between the more resistant Del Mar Formation. Evidence of groundwater seepage is prevalent along the lower vertical sea cliff from approximately E Street south.

Historically, the beach within this reach is narrow and low in elevation. Even after the SANDAG Beach Sand Project was completed in 2001, the beach was still narrow. Only several small patches of cobble berm exist in certain sections of the reach. As a result, wave and tidally induced notching exists at the base of the bluff as the toe is frequently exposed to seawater. In certain specific locations these notches are rather large, extending as deep as 8 feet or more and ranging in height from approximately 10 to 15 feet. Essentially, these large notches form seacaves that are often large enough to crawl, and sometimes walk, into. Due to the deteriorated nature of the bluff face along this reach, numerous bluff top failures have occurred in the last few years.

No recent bluff toe protective devices have been constructed within this reach; however, a long revetment structure section is present at the Self Realization Fellowship (SRF) property providing additional bluff slope protection. The bluff at the SRF has had a long history of slope stability issues, as the area is highly susceptible to landslides. In fact, following the severe winter of 1941, the existing SRF temple, which had been built 30 feet from the edge of the cliff, collapsed onto the beach below as a result of a massive landslide (Kuhn and Shepard, 1984).

2.2.6 Reach 6 – Swami's to San Elijo Lagoon Entrance

The shoreline segment between Swami's and San Elijo Lagoon (**Figure 2.2-5**) is approximately 7,400 feet in length and can be characterized by its narrow beach, varying presence of cobble, decreasing lower bluff topography, and relatively low development density. Although a small number of private homes occupy the northern end, most of the reach segment contains the Highway 101 right-of-way and the San Elijo State Beach, which includes recreational campsites and associated infrastructure.

The narrow beach is backed by cliffs ranging in height from approximately 60 to 80 feet in the northern portion of the reach dropping down to the contemporary beach level associated with the northerly edge of Escondido Creek (San Elijo Lagoon). The sea cliffs within this reach are in varying states of stability. The lower portion of the cliffs are comprised of the Del Mar Formation and groundwater seeps and springs are common, particularly in the northern and middle section of the cliffs near Sea Cliff County Park (Swami's), and appear to be contributing to the slope instability. In fact, a 300-foot length of Highway 101 failed along this section in 1958 and was subsequently stabilized with improved drainage. In addition, a robust rock revetment was installed to protect the highway from future storm and tidal impacts in 1961. The southern portion of the reach is backed by the San Elijo State Beach Campground and contains non-engineered riprap that protects five beach access points.

2.2.7 Reach 7 – San Elijo Lagoon to Table Tops

The low lying shoreline segment extending from San Elijo Lagoon to Table Tops (**Figure 2.2-6**) is approximately 5,900 feet in length and essentially forms a sand barrier between the Pacific Ocean and the San Elijo Lagoon. Development within this reach consists of three popular restaurants at the northern end of the reach with vehicular parking and highway right-of-way sections comprising the majority of improvements over the remaining portions of the reach.

This reach possesses a narrow sandy and cobble spit beach backed by Highway 101, which is protected by a non-engineered rock and concrete rubble revetment. The combination of natural and artificial shoreline protection along this reach results in the reduced exposure to storm-induced wave damage and flooding. However, the close proximity of the restaurants, located in the northern section of the reach, to the water's edge has rendered, and will continue to render, them susceptible to periodic episodes of incidental inundation and structural damage. Moreover, severe storms also cause flooding along Highway 101. For the most part, this is limited to only partial lane closures for limited time periods; however, the most severe storm occurrences result in rare instances of complete road closure for several days due to both coastal flooding and the time required to remove debris from the roadway.

2.2.8 Reach 8 –Table Tops to Fletcher Cove

The shoreline segment between Table Tops and Fletcher Cove (**Figure 2.2-7**) is approximately 3,500 feet in length and represents the northern reach located in the City of Solana Beach. The bluff top is fully developed throughout the reach with large multi-story private residences. The cliffs are approximately 80 feet high and are comprised of Torrey Sandstone over the lower 10 to 15 feet of the cliff face with the remaining 60 feet comprised of weakly cemented terrace deposits.

The shoreline may be presently characterized as consisting of a narrow to non-existent sandy beach backed by high, wave cut cliffs. In addition, small pockets of cobble exist in the back beach area at various locations. Fletcher Cove is located at the southern boundary of this reach and represents a small pocket beach with good public access. Prior to the 1997-1998 El Nino season, the moderate beach condition provided a buffer in preventing the bluff face from being directly exposed to storm wave attack and, as a result, only limited bluff erosion was reported. During the 1997-1998 winter months, sand was stripped away and the bluff face became directly exposed to wave abrasion. Severe toe erosion subsequently developed and bluff failures have been continuously reported since. Presently, notches, on the order of 4 to 8 feet, and large seacaves exist throughout the lower bluff region.

Several bluff top residences have instituted lower bluff stabilization measures to protect against the impingement of waves and tides. These stabilization measures include concrete seawalls, some of which have employed the use of textured artistic surfaces to appear more natural, ranging in height approximately 15 feet to 35 feet, as well as concrete notch infills designed to fill in the voids created by the abrasive forces of waves and tides. However, at several notch infill locations, erosion has since taken place in the lee of the infill resulting in flanking and continued erosion around the end of the infill. The existing notching at the base of the bluff, when combined with the already over steepened upper bluff, is indicative of future and potentially catastrophic block failures.

2.2.9 Reach 9 – Fletcher Cove to Solana Beach Southern City Boundary

The shoreline segment between Fletcher Cove and the Solana Beach Southern City Boundary (**Figure 2.2-8**) is approximately 4,000 feet in length. The bluff top, ranging in height from approximately 60 to 80 feet, is fully developed with private residential houses, as well as multiple family town homes and condominiums. The seacliffs are comprised of an erosive Torrey Sandstone lower bluff and a weakly consolidated sandstone layer throughout the remaining upper portions of the bluff, which are prone to both sliding and block failure.

The shoreline within this reach can presently be characterized as consisting of a narrow to non-existent sandy beach backed by high, steep sea cliffs. Various small pockets of natural cobble berm exist in the southern half of the reach that provides limited protection to the bluff face.

Similar to those of Reach 8, the bluffs within this reach are also susceptible to the repeated exposure of waves and tides after the 1997-1998 El Nino season during which time the beach was depleted. The developed notches range in depth from approximately 2 to 8 feet and fractures that extend through the upper bluff are evident above, and adjacent to, the deeper notches. Evidence of several major bluff failures exists within the reach and a recent large block failure in the center of the reach had occurred just prior to a field investigation conducted on February 6, 2002. Sea caves, several of which extend as deep as 20 to 30 feet, are present in several areas near the southern portion.

Several properties have instituted stabilization measures in the form of seawalls, rock revetments, and notch infills to protect the base of the bluff from eroding. However, the cliff face in the lee of older constructed notch infills and plugs has since eroded leaving the notch infill intact in its original position while the bluff face continues to erode from behind it. In places this has been measured to be as much as 3 to 4 feet. This is indicative of the fairly aggressive erosive nature of the base of the bluff in this shoreline segment.

It is apparent that without corrective action, this reach will continue to have episodic sea cliff and upper bluff failures. The narrow winter and spring beach provides no buffer zone between wave and tidal impacts and the base of the bluff, and as a result, the bluff face bears the full brunt of this energy. In fact, the bluff toe is exposed even during mid-tide levels, which is exacerbated further during storm events. This repeated exposure has resulted in the continued erosion of the bluff face and the associated recession of the upper bluff. It is expected that without corrective action, the magnitude of the upper bluff recession will most likely accelerate in this reach until the upper bluffs have fully equilibrated with the ongoing erosion occurring at the base of the bluff.

1


2 **Figure 2.2-1 Reach 1 - Encinitas Northern City Limit to Beacon's Beach**

Figure 2.2-2 Reach 2 & 3 - Beacon's Beach to Stone Steps

Figure 2.2-3 Reach 4 - Stone Steps to Moonlight Beach

Figure 2.2-4 Reach 5 - Moonlight Beach to Swami's

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2 **Figure 2.2-5 Reach 6 - Swami's to San Elijo Lagoon**

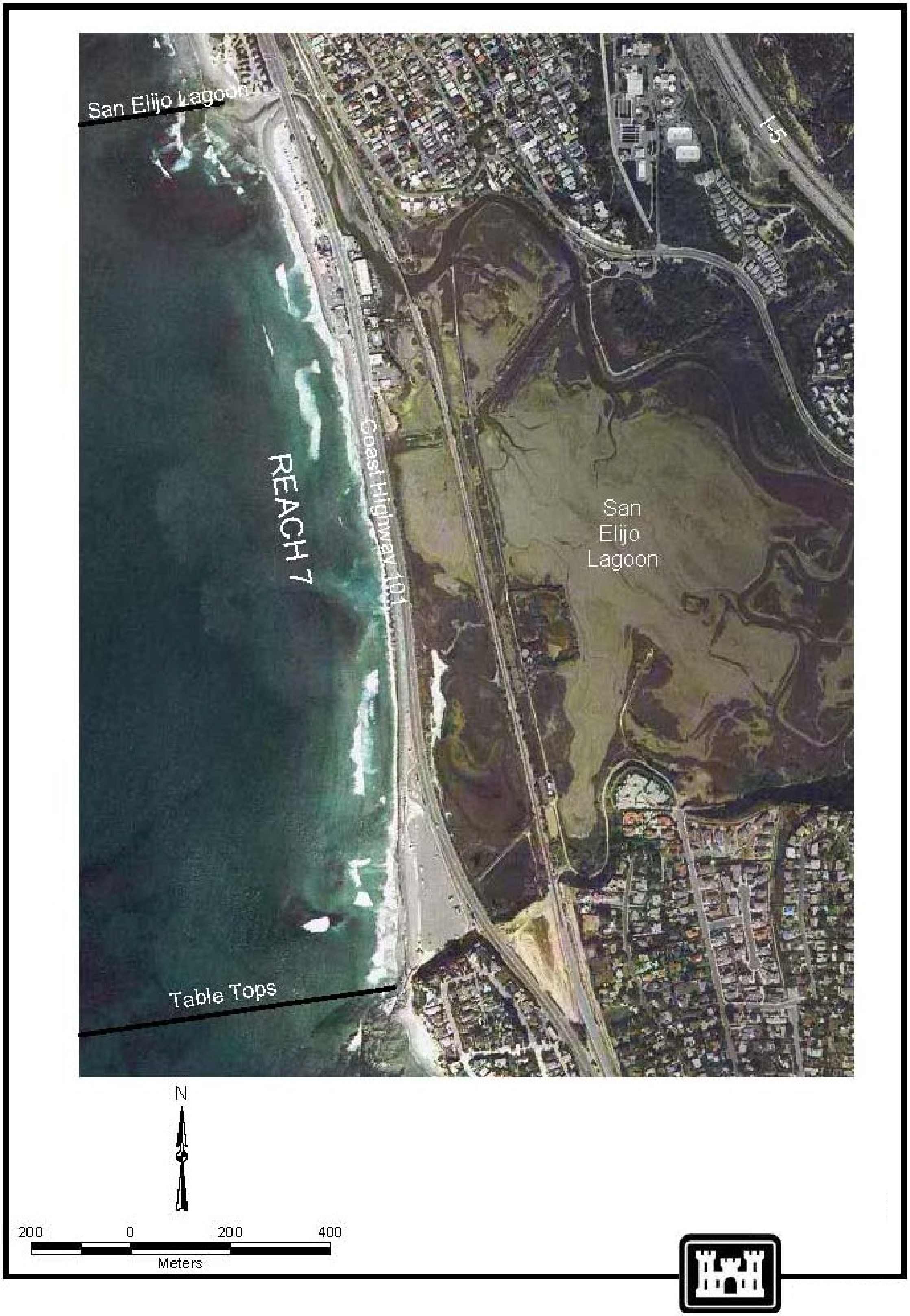

Figure 2.2-6 Reach 7 - San Elijo Lagoon to Table Tops

Figure 2.2-7 Reach 8 - Table Tops to Fletcher Cove

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2 **Figure 2.2-8 Fletcher Cove to Solana Beach Southern City Limit**

2.3 Beach Morphology

Evidence from historical ground and aerial photographs (USACE- LAD, 1996) indicates that the beach conditions can be divided into pre-1980 and post-1980 periods. Prior to 1980, the shoreline experienced cyclic advance and retreat. The beaches received more fluvial delivery and were occasionally replenished in the 1950's, 1960's and 1970's as placed sands from a series of beach nourishments conducted at Oceanside and Carlsbad were gradually transported downcoast to the Encinitas and Solana Beach region. Conversely, the beaches were depleted during rough weather years in which the beach sands were carried offshore into deeper depths and/or transported out of this littoral subcell. Historically, the moderate beaches provided a buffer zone against waves directly impinging upon the bluff face. As a result, little bluff toe erosion occurred prior to the 1980's.

From the late 1970's to present, southern California has experienced a series of severe weather patterns when compared to the rest of this century. Monthly precipitation totals from 1953 to 2002 recorded at the Oceanside Marina also show more frequent occurrence of extreme monthly precipitation for a single winter month since 1978. Fluvial delivery has also been significantly reduced due to river damming and mining activities as well as inland urbanization. The two rivers that contribute littoral drift to the south of Oceanside Harbor are the San Luis Rey and San Dieguito. The Coast of California Storm and Tidal Wave Study (USACE, 1991) report reviewed prior studies that estimated the annual yield of sands and gravels, pre and post dam construction, to drop from 86,000 to 28,000 cubic meters/year (112,000 to 33,000 cubic yards/year) for the San Luis Rey; and from 53,000 to 5,000 cm/yr (69,000 to 6,000 cubic yards/year) for San Dieguito River. The cumulative effects of these impacts have resulted in sand loss on the beaches. As a result of the severe winter storms in the 1982-1983 El Nino year and the extreme storm of 1988, most of the sand on the Encinitas beaches was lost even prior to the 1997-1998 El Nino season. Along the Solana Beach shoreline, the chronically depleted beach condition was worsened after the 1997-1998 season. It is apparent that beach sands were stripped away and lost from the littoral system during the stormy winter season of 1997-1998.

Presently, the depleted beaches within the Encinitas and Solana Beach shoreline have been widened as a result of recent sand replenishment activities. Sands dredged from Batiquitos Lagoon were placed at Batiquitos Beach in 1998 and 2000 to establish a feeder beach that can provide sand to the downcoast shoreline. The SANDAG's Regional Beach Sand Project conducted in 2001 also placed approximately 600,000 cubic yards at Batiquitos Beach, Leucadia, Moonlight Beach, Cardiff and Fletcher Cove (Noble Consultants, 2001). Recent beach profile surveys indicate that the placed sediment has dispersed alongshore both upcoast and downcoast of the beach-fill sites. The aforementioned activities have not only enhanced the recreational activities along the subject shoreline but have also provided the much-needed buffer to prevent the seacliff face from being directly exposed to storm wave attack.

It is anticipated that the Encinitas and Solana Beach beaches, without being regularly nourished, will be depleted again in the future. The depleted beaches will once again provide little protection to the bluff toe. Waves will constantly attack the bluff toe even during low tide periods. Accelerated bluff toe erosion will likely occur in the absence of protective beach sands throughout the Encinitas and Solana Beach shoreline. In Cardiff, without a moderate beach fronting the restaurant buildings and Highway 101, the dwellings and highway will remain vulnerable to coastal flooding and storm damage.

2.4 Site Geology

2.4.1 Onshore Geology

Geologic units in the Encinitas and Solana Beach coastal bluffs include dune sands and marine terrace deposits that form the sloping, upper coastal bluffs above the sea cliffs and three older Eocene “bedrock” geologic units. The sequence of formational material from north to south of the Encinitas segment is the Santiago, Torrey Sandstone and Delmar Formations. Within the Solana Beach area, the geological units exposed are the Delmar formation on the northern segment and the Torrey Sandstone on the southern portion.

The bluff-forming units overlie a wave-cut abrasion platform formed on the Eocene bedrock approximately 125,000 years ago when sea level was 20 feet higher (Lajoie and others, 1992). The sloping, upper portion of the Encinitas and Solana Beach bluffs is comprised predominantly of late Pleistocene, moderately-consolidated, silty-fine sands. Sand dune deposits locally cap the coastal terrace.

2.4.2 Offshore Geology

Offshore from the bluffs, a shore platform extends 500 to 900 feet seaward at a slope of 1V: 46H to a depth of 12 feet, followed by a steeper slope of 1V: 33H to depths of over 60 feet. This surface is an active wave-cut abrasion platform subject to erosion in the present wave environment. The platform is underlain by the same Eocene-age claystone, shale, and sandstone bedrock formations exposed in the sea cliffs. Gentle folding of the bedrock has imparted a northwestward inclination of a few degrees. As a result, the outcrops of individual bedrock formations in the shore platform are located southerly of their position in the coastal bluffs. Where the less erosion-resistant Torrey Sandstone underlies the platform, deeper water extends closer to the bluffs.

2.4.3 Seismicity

The geologic structure of the Encinitas and Solana Beach region is the result of faulting and folding in the current tectonic regime, which began approximately five million years ago when the Gulf of California began to open in association with renewed movement on the San Andreas fault system (Fisher and Mills, 1991). The tectonic forces are also evident in the localized folding and faulting of the Eocene-age sediments. Some of the faults locally control the contact between formations.

The study area is located in a moderately-active seismic region of southern California that is subject to significant hazards from moderate to large earthquakes. Ground shaking resulting from an earthquake can impact the Encinitas and Solana Beach study area. The estimated peak site acceleration for the maximum probable earthquake is approximately 45 percent of the gravitational acceleration (0.45g) from a magnitude 6.9 earthquake on the Rose Canyon fault zone, occurring at a distance of 2.5 miles.

2.4.4 Sources of Material

With the exception of the Delmar Formation, all of the other materials exposed in the coastal bluffs are comprised predominantly of slightly- to moderately-cemented, medium- to coarse-grained sand which contributes littoral material to the beach. The marine-terrace deposits, which form the upper sloping portion of the coastal bluff, represents the largest source of sand-sized sediments. The medium-grain size ranges from 0.2 to 0.5 millimeters, and the fine fraction ranges from 5% to approximately 30% (USACE-LAD, 1996).

The sandy fraction of the Eocene-age Formations have a similar range in the medium-grain size, with the Torrey Sandstone being the coarsest, and the sandy fraction of the Santiago being the finest. The Torrey Sandstone has a well-indurated, white-gray to light yellow-brown color, with the percent fines ranging from less than 5%, to upwards of 20%. The Santiago Formation, a well-indurated, light yellow-brown sandstone, is somewhat darker than the Torrey Sandstone with fines ranging from about 20% to 35%.

A number of available offshore sand sources were explored during the SANDAG sand project study (SANDAG, 2000). Specifically, the closest borrow sources to the Encinitas and Solana Beach region are located offshore of Batiquitos Lagoon (SO-7) at depths from -50 to –100 feet, MLLW and offshore of San Elijo Lagoon (SO-6) at depths from -60 to –100 feet, offshore of Del Mar (SO-5). Results of grain-size analyses show that the average medium grain sizes of the potential sand sources within the Batiquitos Lagoon and San Elijo Lagoon sites are approximately 0.62 and 0.34 mm, respectively. Although total volumes of 972,249, and 102,400 cy of sand were dredged from these two borrow sites to replenish the beach areas located within the Cities of Oceanside, Carlsbad, and Encinitas, significant volumes of coarse sand at these two borrow sites are still potentially available for beach nourishment. It is noted that additional exploration was recently conducted under the RSBP II project that was funded by SANDAG to identify more offshore sand sources. The results of these studies are summarized in the **Appendix C** and on

Table 12.1-1.

2.4.5 Bathymetry

In general, the offshore bathymetric contours within the Encinitas and Solana Beach coastal region are gently curving and fairly uniform. In addition, the nearshore contours are relatively straight and parallel. On average, the shoreline can be characterized by an approximate beach face slope of 45:1 (horizontal feet to vertical feet) extending from the base of the coastal bluffs to about -10.0 feet below the mean lower low water, MLLW, vertical datum. The nearshore slope extending seaward to approximately the -40-foot elevation contour is about 70:1. It should be noted that the beach face and nearshore slopes at Leucadia in the City of Encinitas are on average somewhat steeper than those to the south. The bathymetry seaward of the subject coastlines is presented in **Figure 2.4-1**.

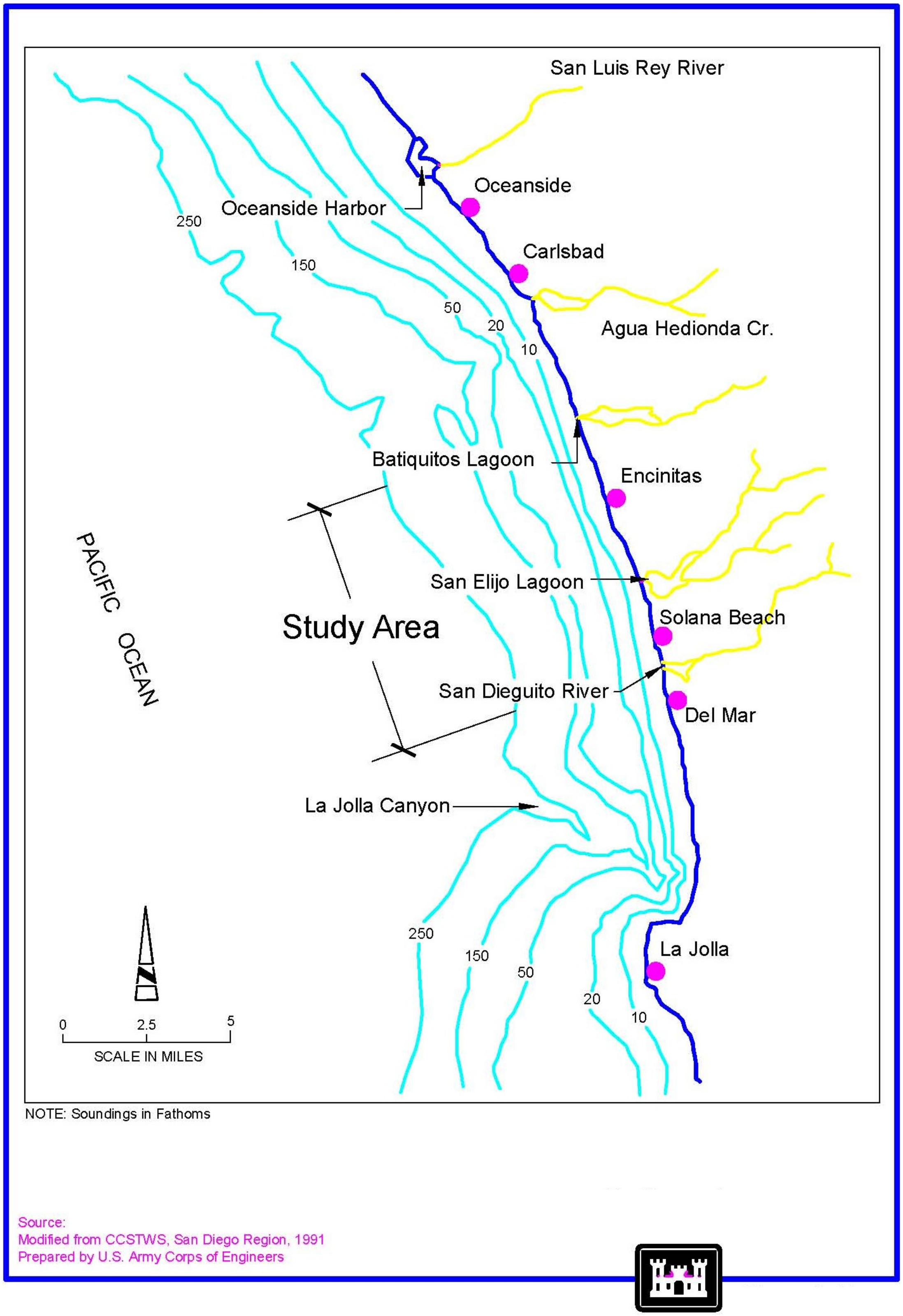


Figure 2.4-1 Bathymetry

3 OCEANOGRAPHIC CHARACTERISTICS

3.1 Climate

3.1.1 General Climatic Conditions

The study area has a semi-arid Mediterranean type climate that is maintained through relatively mild sea breezes over the cool waters of the California Current. Winters are usually mild with rainfall totals around the coast averaging approximately 10 to 20 inches per year. The rainfall increases in the inland areas ranging from approximately 20 inches per year to as much as 60 inches per year in the coastal mountains. **Table 3.1-1** presents the climate summary at an adjacent meteorological station (Station Number 046377 at Oceanside Marina).

Table 3.1-1 Monthly Climatic Summary at Oceanside Marina

Month	Average Maximum Temperature F^{O}	Average Minimum Temperature F^{O}	Average Total Precipitation inches
January	63.9	44.5	2.18
February	64.0	47.6	1.98
March	64.0	47.4	1.83
April	65.4	50.3	0.96
May	66.8	54.7	0.22
June	68.7	58.2	0.09
July	72.5	62.1	0.03
August	74.5	63.3	0.08
September	74.1	60.9	0.28
October	71.8	55.7	0.30
November	68.3	48.8	1.10
December	65.1	44.6	1.24

Typically, the wind climate in the offshore area within 50 to 100 miles of the study area is characterized by northwesterly winds averaging between 10 to 30 miles per hour. The predominant winds within the coastal region during October through February are from the east-northeasterly direction, while the winds during March through September are from the west-northwesterly direction. Average wind speeds during the summer and winter months along the coast range approximately between 5 and 7 miles per hour, respectively. Exceptions in these wind velocities occur during occasional winter storms in which wind strength and direction may vary and during Santa Ana conditions when winds are usually strong from the northeast.

3.1.2 Southern Oscillation El Nino (SOEN) Events

Southern Oscillation El Nino (SOEN) events are global-scale climatic variations with a frequency of approximately two to seven years. They represent an oscillatory exchange of atmospheric mass as manifest by a decrease in sea surface pressure in the eastern tropical Pacific Ocean, a decrease in the easterly trade winds, and an increase in sea level on the west coast of North and South America (USACE-LAD, 1986). The interaction between the atmospheric and oceanic environment during these events drive climatic changes that can result in significant modifications of wave climate along the world's coasts.

The severe winters of 1982-1983 and 1997-1998, which produced some of the most severe storms to ever impact the study area, were the result of intense El Niño events. The atmospheric disturbance associated with these two events caused abnormally warm water temperatures, a reversal of the westerly trade winds, and increased monthly mean sea levels (MSL) by as much as 0.42 feet in 1982-1983 season and 0.52 feet in 1997-1998 season at La Jolla, San Diego (Flick, 1998).

3.2 Coastal Processes

Water levels within the surf zone consist of four primary factors: 1) astronomical tides, 2) storm surge and wave set-up, 3) climatic variation related to El Niño, and 4) long-term changes in sea level. Each of these factors is briefly described in the following sections.

3.2.1 Tides

Tides along the southern California coastline are of the mixed semi-diurnal type. Typically, a lunar day (about 24 hours) consists of two high and two low tides, each of different magnitudes. A lower low tide normally follows the higher high tide by approximately seven to eight hours while the time to return to the next higher high tide (through higher low and lower high water levels) is usually approximately 17 hours. Annual tidal peaks typically occur during the summer and winter seasons following a solstice. The increased tidal elevations during the winter season can exacerbate the coastal impacts of winter storms.

Tides along the open coast of California have a spatial scale on the order of a hundred miles; hence, the prevailing tidal characteristics measured in La Jolla may be considered representative of the tidal elevations within the study area. The National Oceanic and Atmospheric Administration (NOAA) has established tidal datum at La Jolla in San Diego County. The current tidal epoch of approximately 19 years is inclusive of the time period from 1983 to 2001. The tidal characteristics are shown in **Table 3.2-1**.

Table 3.2-1 Tidal Characteristics at Scripps Pier in La Jolla, California

NOAA Station 9410230 **La Jolla, CA**	**Elevation relative to MLLW in feet** **Epoch: 1983-2001**
Highest observed water level (Jan 11, 2005)	7.66
Mean Higher High Water (MHHW)	5.33
Mean High Water (MHW)	4.60
Mean Tide Level (MTL)	2.75
Mean Sea Level (MSL)	2.73
Mean Low Water (MLW)	0.90
North American Vertical Datum -1988 (NAVD)	0.19
Mean Lower Low Water (MLLW)	0.00
Lowest observed water level (Dec.17, 1933)	-2.87

Source: http://tidesandcurrents.noaa.gov

3.2.2 Storm Surge and Wave Setup

Storm surge results from storms that induce fluctuations in the wind speed and atmospheric pressure. Storm surge is usually fairly small on the west coast of the United States when compared to storm surge on the east and gulf coasts of the United States. The decreased impact of storm surge on the west coast is due primarily to the relatively narrow continental shelf. It was estimated that the average increase in the water level resulting from storm surge effects ranges from approximately 0.3 to 0.5 feet within the San Diego coastal zone (USACE-LAD, 1991). The average positive tide residual, defined as the difference between the measured and predicted tide, usually occurs on a temporal scale of approximately six days; however, storm surges of significant magnitudes rarely continue for longer than two days.

Wave setup is the super-elevation of water levels that occur primarily in the surf zone where waves break as they approach a beach and reach their limiting wave steepness. The magnitude of the wave setup depends on the height of breaking waves occurring in the surf zone. The elevated water levels allow waves of increased magnitude to impinge onto the bluff face during a storm event.

3.2.3 Sea Level Rise

Long-term changes in the elevation of sea level relative to the land can be engendered by two independent factors: (1) global changes in sea level, which might result from influences such as global warming, and (2) local changes in the elevation of the land, which might result from subsidence or uplift. The ocean level has never remained constant over geologic time, but has risen and fallen relative to the land surface. A trendline analysis of yearly Mean Sea Level (MSL) data recorded at La Jolla in San Diego County 1924 to 2006 indicates that the MSL upward trend is approximately 0.0068 feet per year, as shown in **Figure 3.2-1**.

According to the Intergovernmental Panel on Climate Change (IPCC), global average sea levels have risen approximately 0.3 feet to 0.8 feet over the last century and are predicted to continue to rise between 0.6 ft and 2.0 ft over the next century (IPCC, 2007). In a 2009 study performed by the Pacific Institute on behalf of the California State Coastal Conservancy (SCC) scientific data gathered from 1980 to 1999 suggests that global sea level rise has outpaced the IPCC predictions (Rahmstorf, 2007). To the contrary, an analysis of U.S. Tide Gauge records spanning from 1930 to 2010 found the rate of sea level rise for this period to be decelerating (Houston and Dean, 2011). Potential effects from an acceleration of sea level rise on coastal environments, such as erosion, net loss of shorefront, increased wetland inundation, and storm surge have the potential to displace coastal populations, threaten infrastructure, intensify coastal flooding, and ultimately lead to loss of recreation areas, public access to beaches, and private property.

Given the potential for substantial effects that sea level rise could have on coastal environments, both federal and state agencies have prepared guidance for incorporating sea level rise into the planning and design of projects and these guidance have been incorporated into the current analyses.

The Engineer Circular 1165-2-212 on sea level rise (USACE, 2011) provides Corps guidance for incorporating the potential direct and indirect physical effects of projected future sea level change in the engineering, planning, design, and management of Corps projects. The guidance states that potential sea level rise must be considered in every Corps coastal activity as far inland as the extent of estimated tidal influence. This guidance recommends a multiple scenario approach to address uncertainty and help develop better risk-informed alternatives. Planning studies and engineering designs should consider alternatives that are developed and assessed for the entire range of possible future rates of sea level rise. The alternatives should be evaluated using “low”, “intermediate”, and “high” rates of future sea level rise for both “with” and “without” Project conditions. The local historical rate of sea level rise should be used as the low rate. The intermediate rate of local mean sea level rise should be estimated using the modified Curve I from the National Research Council (1987). The high rate of local sea level rise should be estimated using the modified Curve III from the National Research Council report. This high rate exceeds the upper bounds of the 2007 IPCC estimates 2007, thus allowing for the potential rapid loss of ice from Antarctica and Greenland. The sensitivity of alternative plans and designs to the rates of future local mean sea level rise should be determined. Design or operations and maintenance measures should be identified to minimize adverse consequences while maximizing beneficial effects. For each alternative sensitive to sea level rise, potential timing and cost consequences are evaluated.

These Corps recommended curves as are shown in **Figure 3.2-2** exhibiting the high (Curve III), intermediate (Curve I), and low (local historical trend) estimates. The estimates were adjusted to a year 2000 baseline for direct comparison with other sea level rise projections. The high and intermediate curves are based on the following formula.

$$SLR(t) = E_{local} t + bt^2$$

Where *SLR(t)* is the amount of sea level rise in meters from the 1986 baseline,
E_{local} is the historic trend at a local gage station per year,
b = 0.0001005 meters/year^2 is a constant for Curve III,
b = 0.0000236 meters/year^2 is a constant for Curve I, and
t is the year difference between 1986 and the subject year
(note that this study was performed with constant values provided in EC 1110-2-211 (2009) which has since been revised, however, the results are not appreciably different).

The low sea level rise is represented by a trendline analysis of yearly MSL data recorded at La Jolla in San Diego County from 1924 to 2006. This indicates an upward trend of approximately 0.0068 feet per year (2.07 millimeters per year), as shown in **Figure 3.2-1**.

In addition to USACE guidance, various agencies within the State of California have released guidance for their respective projects. Governor Arnold Schwarzenegger issued Executive Order S-13-08 (Office of the Governor, 2008) to enhance the State's management of potential climate effects from sea level rise, increased temperatures, shifting precipitation and extreme weather events. There are directives for four key actions including:

1. initiate California's first statewide climate change adaptation strategy that will assess the state's expected climate change impacts, identify where California is most vulnerable and recommend climate adaptation policies by early 2009;
2. request the National Academy of Science to establish an expert panel to report on sea level rise impacts in California to inform state planning and development efforts;
3. issue interim guidance to state agencies for how to plan for sea level rise in designated coastal and floodplain areas for new projects; and
4. Initiate a report on critical existing and planned infrastructure projects vulnerable to sea level rise.

Executive Order S-13-08 directs that, prior to release of the final sea level rise assessment report from the National Academy of Science, all California agencies that are planning construction projects in areas vulnerable to future sea level rise shall, for the purposes of planning, consider a range of sea level rise scenarios for the years 2050 and 2100 in order to assess project vulnerability and, to the extent feasible, reduce expected risks and increase resiliency to sea level rise. Sea level rise estimates should also be used in conjunction with appropriate local information regarding local uplift and subsidence, coastal erosion rates, predicted higher high water levels, storm surge and storm wave data.

Since release of Executive Order S-13-08, various California agencies have provided recommended sea level rise projections (California Climate Change Center, 2009a & 2009b; California State Coastal Conservancy, 2009; Coastal and Ocean Working Group of the California Climate Action Team, 2010; California Climate Action Team, 2010; California State Lands Commission, 2009; California Ocean Protection Council, 2011; California Department of Transportation, 2011), as summarized in Table 3-3 and shown in **Figure 3.2-2**. Sea level rise projections from a year 2000 baseline are provided for the years 2030, 2050, 2070, and 2100. Projections for the years 2070 and 2100 include three ranges of values for low, medium, and high greenhouse gas emissions scenarios corresponding to IPCC greenhouse gas emissions scenarios. In **Figure 3.2-2**, the data points identified as "COPC: Average, High" are the high range of the average of the models as recommended by the California Ocean Protection Council and repeated in **Table 3.2-2**.

Table 3.2-2 State of California Interim Guidance Sea Level Rise Projections

Year	Description	Average of Models Inches	Range of Models inches
2030		7	5-8
2050		14	10-17
2070	Low	23	17-27
	Intermediate	24	18-29
	High	27	20-32
2100	Low	40	31-50
	Intermediate	47	37-60
	High	55	43-69

Projections from year 2000 baseline. Source: California Ocean Protection Council, 2011

Assuming that the Project base-year (i.e., year 0) is set to be in 2015, the resultant sea level rise at the end of the 50 year Project life will occur in 2065. The analysis for the years 2015 to 2065 would cover the year 2050; therefore, it would implicitly satisfy the California requirement. Additionally, in order to satisfy California requirements pursuant to Executive Order S-13-08, the EIS/EIR should include a qualitative analysis for the year 2100. The projected sea level rise according to California projections in 2065 lies within the range of intermediate and high sea level rise scenarios per Corps guidance, so is captured by an analysis of the Corps sea level rise estimates. Thus only the Corps high, intermediate and low sea level rise projections were used in the current study.

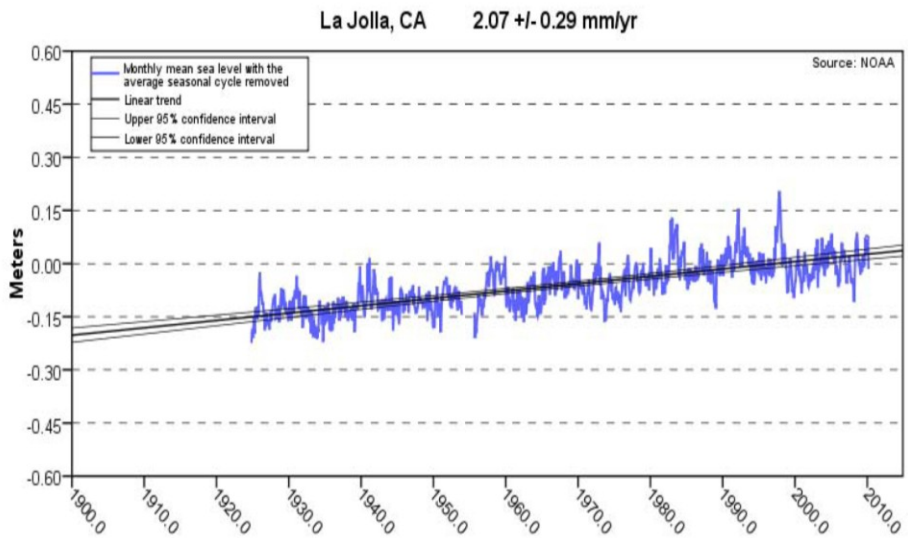


Figure 3.2-1 Historic Mean Sea Level Rise at La Jolla

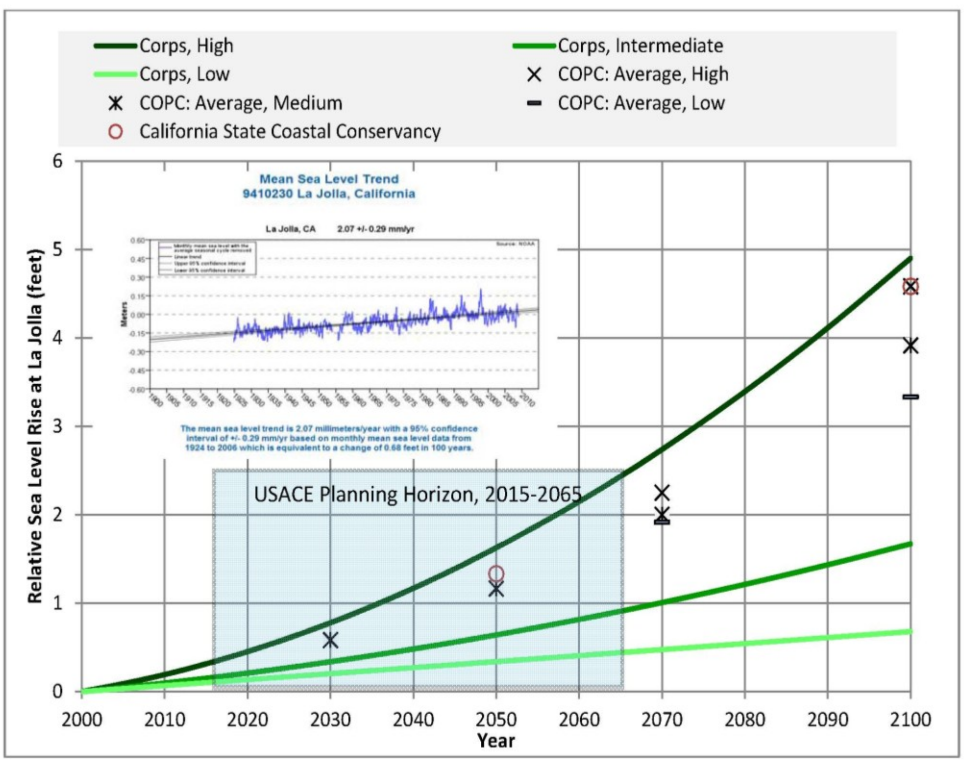


Figure 3.2-2 Relative Sea Level Rise Estimates

3.3 Waves

Waves that impinge on the shoreline, perhaps more than any other oceanographic factor, determine the fate of sediment movement and the associated impacts to the coastal environment. Essentially, waves are the driving force in generating the alongshore currents that are responsible for moving sand, suspended by wave action, along the coast, which ultimately results in changes to the shoreline. This section describes the regional wave climate within study area.

3.3.1 Wave Origin and Exposure

Wind waves and swell within the study area are produced by six basic meteorological weather patterns. These include extratropical cyclone swells in the northern hemisphere in the Pacific Ocean, swells generated by northwest winds in the outer coastal waters, westerly seas and southeasterly sea seas, storm swells from tropical storms and hurricanes off the Mexican coast, and southerly swells originating in the southern Pacific Ocean. **Figure 3.3-1** illustrates these identified weather patterns and their associated wave propagating directions.

<u>Extratropical Cyclone of the Northern Hemisphere:</u> This weather system represents the category of the most severe waves reaching the California Coast. Northern hemisphere swell waves are usually produced by remote meteorological disturbances, including Aleutian storms, subtropical storms north of Hawaii, and strong winds in the eastern North Pacific Ocean. These produce north or northwest swell on the California Coast. Deep water significant wave heights rarely exceed 10 feet, with wave periods ranging from 12 to 18 seconds. Significant wave height is defined as the average height of the one-third highest waves within a wave train. During extreme northern hemisphere storms, wave heights may exceed 20 feet with periods ranging from 18 to 22 seconds.

<u>Northwest Winds in the Outer Coastal Waters:</u> One of the predominant wave sources along the study area is the prevailing northwest winds north and west of the southern California coastal waters. This is particularly true during the spring and summer months. Wave heights are usually low, less than 3 feet; but on occasion, with superposition of a strong surface high and an upper level trough, the northwesterlies increase, becoming strong from about Point Sal to San Nicolas Island. Moderate northwestern winds will produce breaker heights of 4 to 6 feet, while strong events can generate breaking wave heights ranging from 6 to 9 feet with typical periods ranging from 6 to 10 seconds.

<u>West to Northwest Local Sea:</u> Westerly winds can be divided into two types: 1) temperature-induced sea breezes, and 2) gradient winds, both producing a west to northwest local sea. The former exhibits a pronounced seasonal and diurnal variation. The strongest sea breezes occur during the late spring and summer months, while the lightest sea breezes occur during December and January. The summer sea breeze usually sets in during the late morning and peaks in the mid-afternoon. In winter months, sea breeze conditions are limited to a few hours during early afternoon with a wind speed on the order of 10 knots. The summer sea breezes, on the other hand, will average about 15 knots and occasionally reach 20 knots or more. Gradient winds, lasting for a maximum duration of three days, are typically confined to the months of November through May with the peak occurring in March or early April. They usually occur following a frontal passage or with the development of a cold low pressure area over the southwestern United States. Under such conditions, locally generated wind waves combined with components of the northwest swell produce large waves that can potentially cause coastal damage within the region.

Pre-frontal Local Sea: The study area is vulnerable to storm conditions from strong winds blowing from the southeast to southwest along the coast prior to a frontal storm passage. These winds typically come from the south-southeast to south a short distance offshore. Wind waves, with peak wave periods of between 6 and 8 seconds, reach the shore with minimal island sheltering or refraction with directions coming from the southwest. Significant wave heights are generally in the range of 4 to 8 feet. Large wave heights are rare because the fetch and duration of these wind waves are short-lived.

Tropical Storm Swell: Tropical storms and hurricanes develop at low latitudes off the west coast of Mexico from June through October. These storms first move west as they depart mainland Mexico, then curve north and sometimes northeast before dissipating in the colder waters off Baja California. The swell generated by these storms usually do not exceed 6 feet in significant wave height. However, on rare occasions the offshore waters are warm enough to facilitate hurricane migration to more northern latitudes than usual. In September 1939, a hurricane passed directly over southern California generating recorded wave heights of 27 feet. This storm caused widespread damage along the coast.

Extratropical Cyclone of the Southern Hemisphere: From the months of April through October, and to a lesser extent the remainder of the year, large South Pacific storms traversing between south latitude 40° and 60° from Australia to South America send south swell to the west coast of Central and North America. Typical southern hemisphere swell rarely exceeds 4 feet in height in deep water, but with periods ranging between 18 and 21 seconds, they can break at over twice that height when they reach the coast. The south swell also causes a reversal in the predominantly littoral southward flow. During summer months, these waves dominate the littoral processes of the region driving alongshore currents northward as the northern-hemisphere swells are less frequent.

Figure 3.3-2 illustrates the wave exposure windows for the study area. The Channel Islands (San Miguel, Santa Rosa, Santa Cruz, and Anacapa), Santa Catalina Island, San Nicolas Island, and San Clemente Island provide some sheltering to the coastal region depending on the swell approach direction. The swell window, which is open to severe extratropical storms of the northern hemisphere, extends from approximately 277 to 284 degrees. The exposure window open to south swell and tropical storm swell extends from approximately 190 to 257 degrees. The study area is also open to west to northwest local sea and pre-frontal local sea from southwest to southeast.

3.3.2 Deep Water Wave Characteristics

Storms have an impact on the southern California coast now and in the past. The waves adversely impacting the study area are from mainly extratropical winter storms that, when combined with spring high tides, can cause severe beach and bluff erosion. The 1982-1983 El Niño winter storms resulted in permanent beach sand loss within the Encinitas coast that subsequently had a detrimental impact to the bluff stability as bluffs became directly exposed to storm wave attack. Accelerated bluff toe erosion occurred in Solana Beach after the already limited beach sand was completely stripped away during the 1997-1998 El Niño season.

Extreme storm events were selected primarily on the basis of their potential to generate damaging waves to the study area. This placed the emphasis on long period swells approaching from their respective exposure windows, dictated in large part by the offshore islands. Deep water wave characteristics of extreme storms have been hindcasted and measured in deep water. Pertinent hindcasted extratropical storm waves in deep water were selected to characterize the extreme deep water ocean wave conditions, as presented in **Table 3.3-1**.

3.3.3 Nearshore Wave Characteristics

Deep water waves that enter within the nearshore coastal area of the study area are altered by offshore island sheltering, refraction, diffraction, and shoaling effects as they propagate towards the shoreline. The offshore islands, as illustrated in **Figure 3.3-1**, provide some sheltering from waves approaching from the deep ocean. As waves continue to propagate shoreward, the combined effects of refraction and shoaling must be accounted for when determining the nearshore wave characteristics.

Transformation of deep water ocean waves to the nearshore coastal area near the study site was performed using a spectral back-refraction model (O'Reilly and Guza, 1991). The numerical model accounts for island sheltering, wave refraction and wave shoaling. **Table 3.3-2** shows the transformed nearshore extreme wave characteristics at Cardiff (Reach 7). The representative nearshore station, where the hindcasted deep water wave characteristics were transformed to, is at 33^{o}0'30.5" N and 117^{o}17'3.9"W in a water depth of approximately 32.5 feet.

3.3.4 Tsunamis

Tsunamis are long period waves caused by a large underwater disturbance such as an earthquake, volcanic eruption or landslide. Tsunamis cross the deep ocean as very long waves of low amplitude. Waves produced by tsunamis typically have a wavelength in excess of 100 miles with an amplitude of 3 feet or more. The waves resulting from a tsunami can be significantly amplified by shoaling, diffraction, refraction, convergence, and resonance as they propagate towards the coast, namely due to the immense traveling wave speeds and lengths.

Historically, tsunamis have not significantly affected the study area. It is believed that local earthquake events will not produce underwater disturbances capable of generating significant tsunamis within this coastal region. Although historically tsunamis originating off the coasts of Chile and Alaska have threatened the southern California coastline, the impacts to the study area have been negligible. Therefore, the threat of coastal flooding resulting from tsunamis along the study area is considered low.

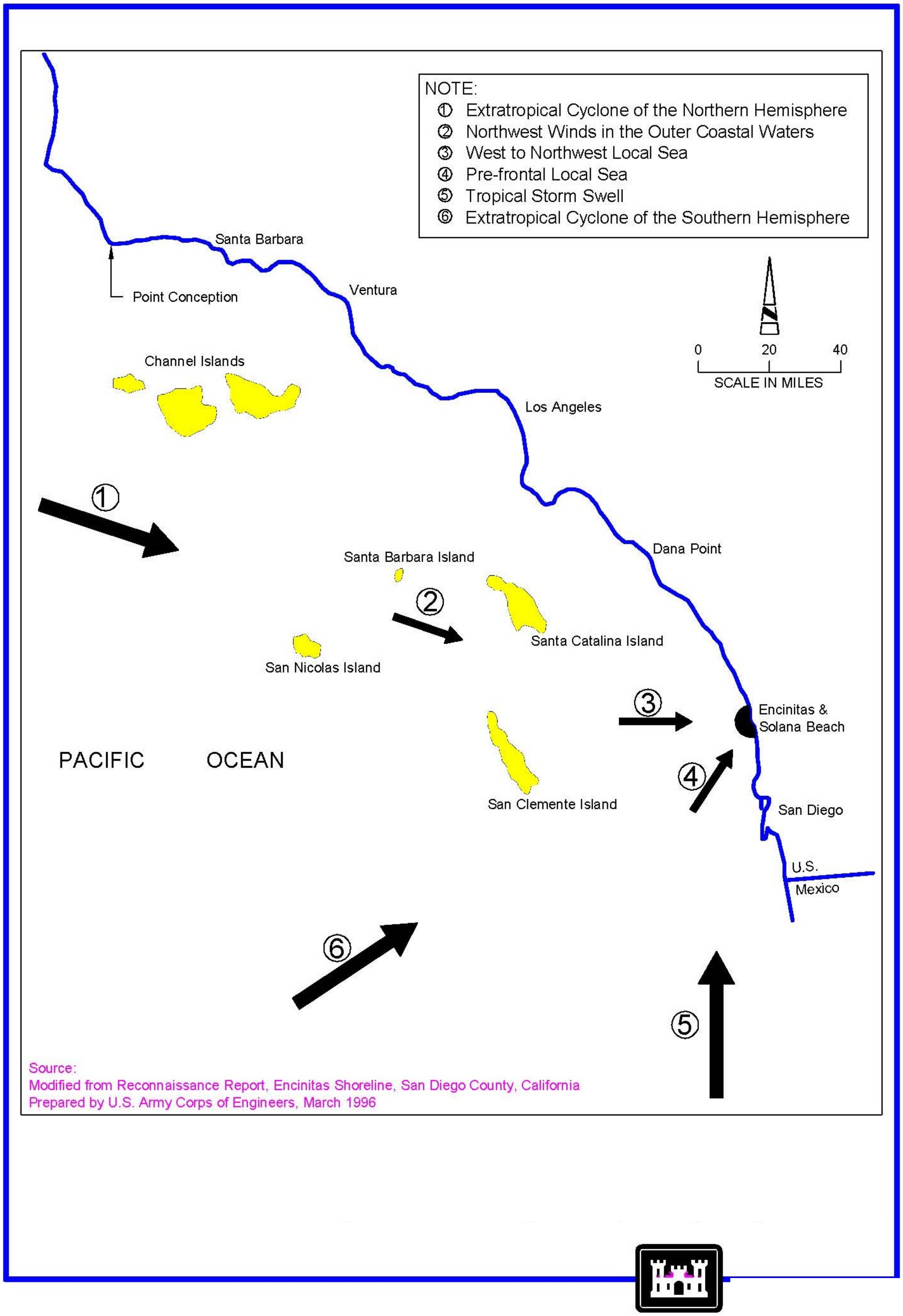


Figure 3.3-1 Meteorological Wave Origins Impacting Project Area

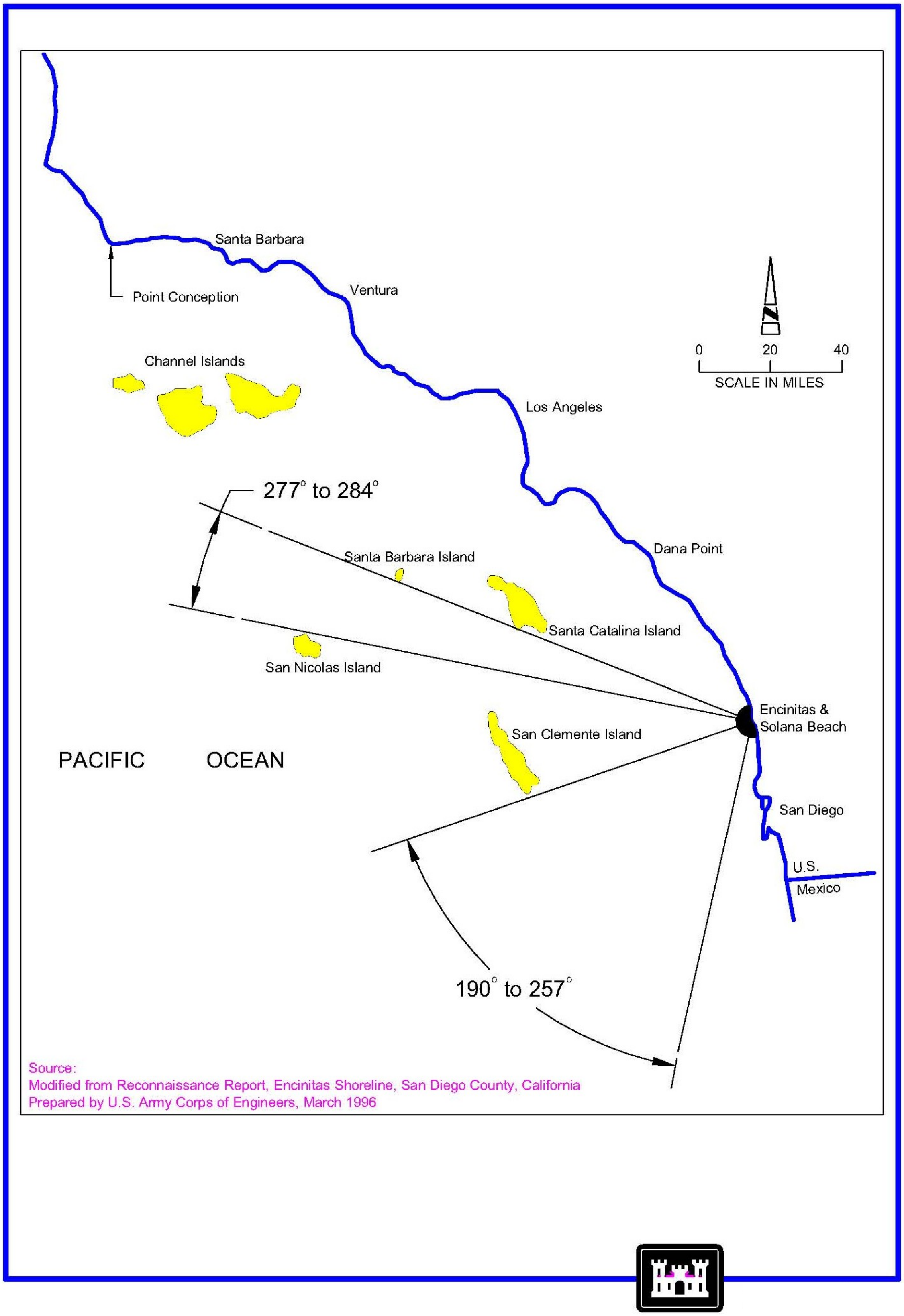


Figure 3.3-2 Wave Exposure Windows

Table 3.3-1 Hindcasted Extreme Extratropical Deep Water Wave Characteristics

Date of Storm	Hs (feet)	Ts (sec)	Dir (deg)	Date of Storm	Hs (feet)	Ts (sec)	Dir (deg)
12/31/79	17.4	16.9	286	3/1/91	16.4	12.7	277
2/17/80	17.8	12.7	254	2/11/92	14.8	12.7	269
2/20/80	21.4	15.3	265	1/18/93	14.4	10.5	241
1/22/81	18.2	16.9	277	2/9/93	14.2	15.3	277
1/29/81	19.4	12.7	275	1/5/95	18.1	8.7	288
12/1/82	22.3	12.7	298	1/11/95	16.5	13.9	280
1/27/83	22.9	15.3	287	2/3/95	14.1	16.9	278
2/13/83	19.4	16.9	278	3/12/95	19.3	15.3	273
3/2/83	30.3	16.9	270	2/1/96	13.8	10.5	257
12/3/85	18.6	15.3	286	12/7/97	13.2	9.5	229
2/1/86	17.7	16.9	282	1/30/98	21.7	16.9	287
2/16/86	24.7	16.9	258	2/1/98	16.9	16.9	279
3/11/86	22.2	16.9	286	2/4/98	23.0	16.9	280
3/5/87	13.4	13.9	267	2/7/98	19.3	13.9	266
12/17/87	17.0	16.9	283	2/18/98	22.5	16.9	282
1/18/88	32.3	13.9	290	2/21/00	17.5	12.7	280
2/4/91	14.8	16.9	277				

Notes: Hs denotes significant wave height, Ts denotes wave period

Table 3.3-2 Hindcasted Extreme Extratropical Nearshore Wave Characteristics At Reach 7

Date of Storm	Hs (ft)	Ts (sec)	Dir (deg)	Date of Storm	Hs (ft)	Ts (sec)	Dir (deg)
12/31/79	9.2	16.9	265	3/1/91	10.8	12.7	235
2/17/80	12.5	12.7	240	2/11/92	9.8	12.7	255
2/20/80	15.4	15.3	265	1/18/93	10.5	10.5	225
1/22/81	13.1	16.9	265	2/9/93	9.8	15.3	265
1/29/81	11.8	12.7	260	1/5/95	10.5	8.7	225
12/1/82	8.9	12.7	255	1/11/95	12.8	13.9	260
1/27/83	12.1	15.3	265	2/3/95	9.8	16.9	265
2/13/83	13.1	16.9	265	3/12/95	12.8	15.3	260
3/2/83	22.6	16.9	285	2/1/96	9.2	10.5	235
12/3/85	9.2	15.3	265	12/7/97	9.2	9.5	220
2/1/86	9.8	16.9	265	1/30/98	10.5	16.9	265
2/16/86	18.4	16.9	260	2/1/98	10.8	16.9	265
3/11/86	11.5	16.9	260	2/4/98	14.8	16.9	265
3/5/87	10.2	13.9	265	2/7/98	12.5	13.9	250
12/17/87	9.8	16.9	260	2/18/98	12.5	16.9	265
1/18/88	16.4	13.9	260	2/21/00	9.5	12.7	255
2/4/91	9.5	16.9	265				

Notes: Hs denotes significant wave height,
Ts denotes wave period

3.4 Currents

This section details the coastal and oceanographic currents affecting the water circulation patterns within the study area. These include currents offshore of the study area, alongshore currents (currents flowing parallel to the shoreline), and cross-shore currents (currents flowing perpendicular to the shoreline).

3.4.1 Offshore Currents

The offshore currents, including the California Current, the California Undercurrent, the Davidson Current, and the Southern California Countercurrent (also known as the Southern California Eddy), consist of major large-scale coastal currents, constituting the mean seasonal oceanic circulation with induced tidal and event specific fluctuations on a temporal scale of 3 to 10 days (Hickey, 1979).

<u>The California Current:</u> The California Current is the equatorward flow of water off the coast of California and is characterized as a wide, sluggish body of water that has relatively low levels of temperature and salinity. Peak currents with a mean speed of approximately 25 to 49 feet per minute occur in summer following several months of persistent northwesterly winds (Schwartzlose and Reid, 1972).

<u>The California Undercurrent:</u> The California Undercurrent is a subsurface northward flow that occurs below the main pycnocline and seaward of the continental shelf. The mean speeds are low, on the order of 10 to 20 feet per minute (Schwartzlose and Reid, 1972).

<u>The Davidson Current:</u> The Davidson Current is a northward flowing nearshore current that is associated with winter wind patterns north of Point Conception. The current, which has average velocities between 30 and 60 feet per minute, is typically found off the California coast from mid-November to mid-February, when southerly winds occur along the coast (Schwartzlose and Reid, 1972).

<u>The Southern California Countercurrent:</u> The Southern California Countercurrent is the inshore part of a large semi-permanent eddy rotating cyclonically in the Southern California Bight south of Point Conception. Maximum velocities during the winter months have been observed to be as high as 69 to 79 feet per minute (Maloney and Chan, 1974).

3.4.2 Alongshore Currents

Alongshore Currents are those nearshore currents that travel parallel to the shoreline extending throughout, and slightly seaward of, the surf zone. The alongshore currents in the coastal zone are driven primarily by waves impinging on the shoreline at oblique angles. The longshore sediment transport rate varies in proportion to characteristics of the regional wave climate and the directional predominance. The surf zone alongshore currents within the study area are nearly balanced between northerly and southerly flows and can attain maximum velocities of approximately 3 feet per second. Typically, summer swell conditions produce northerly drifting currents, while the winter swell from the west and northwest produce southerly alongshore currents. Overall, the persistence of the northerly drift occurs more frequently; but the greater wave energy associated with the winter storms generally results in a net southerly littoral drift.

3.4.3 Cross-shore Currents

Cross-shore currents exist throughout the study area, particularly at times of increased wave activity. These currents tend to concentrate at creek mouths and structures, but can occur anywhere along the shoreline in the form of rip currents and return flows of complex circulation. To date, no information is available that quantifies the velocities of these currents within the study area; however, studies have shown that the velocity of rip currents, in general, can exceed 6 feet per second (Dean and Dalrymple, 1999).

4 LITTORAL PROCESSES

This chapter identifies the various sediment transport and littoral processes that are responsible for the movement of sediment along the coastlines of both the Cities of Encinitas and Solana Beach. Identifying the littoral processes and determining a realistic sediment budget for the project study locale requires an understanding of the quantification of sediment sources, sinks, and transport characteristics, the quantification and interpretation of past shoreline changes, as well as the shoreline response to artificial beach nourishment activities. The net rate of sand supply to a beach is one of the most important factors in determining the health of a given beach. The influx of sediment to the shoreline represents one element of the local sand budget while the loss of sediment represents the other. The difference between these two elements determines whether a beach is erosive or accretive. Knowing where the regional sand supply sources are and quantifying the contribution of each source is critical in fully understanding beach erosion issues such that viable strategic alternatives can be formulated and designed to alleviate them.

A littoral cell is defined as a geographically limited coastal compartment that contains sand inputs, sand outputs, and sand transport paths. The littoral cell is one of the most important concepts to utilize when analyzing the littoral processes of a coastal region. This is due to the fact that the geographic topography, the littoral sand supply, and the wave forcing are all inherent in its definition. Ideally, cells are isolated from each other to insure no exchange of sediment in either the upcoast or downcoast direction; thereby, simplifying the tracking of sand movement. However, in reality a proportion of sediment is typically transported between upcoast and downcoast cells. In instances where this occurs, it is important to quantify the net transport volume bypassed between adjacent cells.

4.1 Encinitas – Leucadia Subcell

The coastal zone of the project study area is located within the Encinitas – Leucadia subcell of the Oceanside Littoral Cell, which extends approximately 7.5 miles from the south jetty of the Batiquitos Lagoon entrance to the southern boundary of the City of Solana Beach, as illustrated in **Figure 4.1-1**. The encompassing Oceanside Littoral Cell is a 51-mile long coastal reach bounded on the north by Dana Point Harbor and the south by Pt. La Jolla. This littoral cell contains a wide variety of coastal features including coastal cliffs, headlands, beaches composed of sand and/or cobblestone, rivers, creeks, tidal lagoons and marshes, submarine canyons, man-made shore and bluff protection devices, and major harbor structures. Within the Encinitas-Leucadia subcell, the shoreline is mostly characterized as consisting of narrow sandy beaches backed by high seacliffs. During the past 20 years or so, the backshore and bluff tops of this subcell have experienced rapid residential and commercial development and artificial beach nourishment has been performed periodically at many locations as well.

Seasonal variations in beach width are typical within the Encinitas-Leucadia subcell. During the winter season, when the wave environment is energetic, sediment is transported from the beach area and is stored in an offshore bar formation. These sands then return to the beach throughout the summer when a more benign wave environment is present. During the Coast of California Storm and Tidal Waves Study for the San Diego County Region (CCSTWS-SD), beach profile data (USACE-LAD, 1991) indicated that the beaches experienced seasonal winter erosion in excess of 100 feet. A loss of beach width of this magnitude, when combined with the already narrow beaches, could lead to the seasonal disappearance of many of the sandy beaches within this subcell.

Historically, the net alongshore sediment transport in this region has been considered to be from north to south; however, recent increased wave activity from the south over the past 10 to 15 years has resulted in an increase in the northerly littoral transport, as compared with previous decades, thus decreasing the net flow of southerly littoral transport materials.

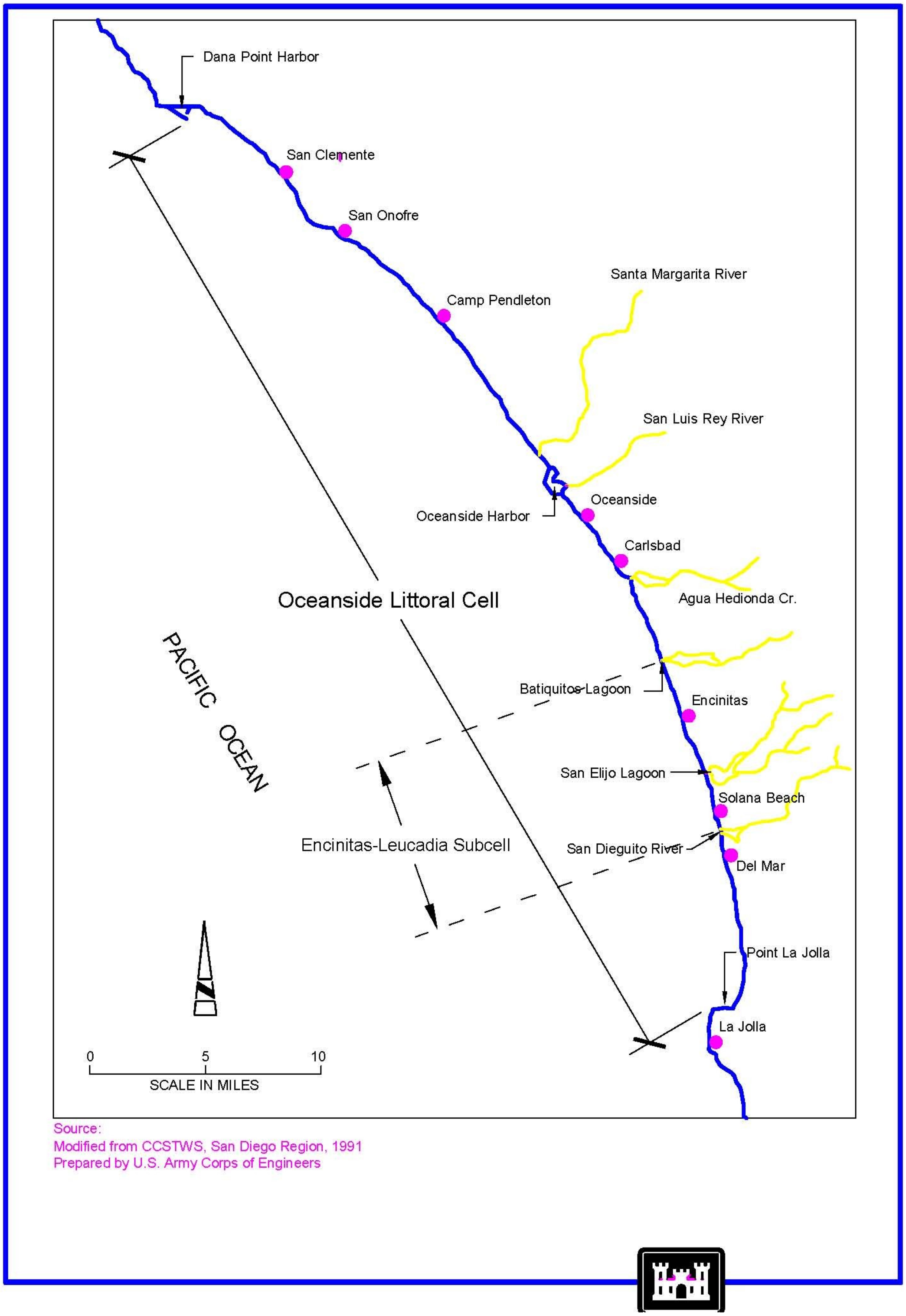


Figure 4.1-1 Oceanside Littoral Cell

4.2 Shoreline Changes

Beach profiles within the study area have been surveyed along 15 transects. Historically, most surveys were performed through the Los Angeles District Army Corps of Engineers in support of beach erosion studies and the CCSTWS-SD. This effort resulted in data spanning from 1934 through 1989 at four distinct transects within the study area. These transects include (from north to south) CB-720, SD-670, SD-630, and DM-590 (USACE-LAD, 1991). In addition to the CCSTWS-SD transects, the City of Carlsbad sponsored spring and fall surveys along transect CB-720 from 1988 to 1996. From 1996 through the San Diego Association of Governments (SANDAG) Regional Beach Sand Project I (RBSPI) in 2001, the SANDAG has continued the surveying efforts initiated through CCSTWS-SD, with additional support from the Cities of Encinitas and Solana Beach.

Table 4.2-1 presents the beach profile transect locations and their respective sponsors within the study shoreline, while **Figure 4.2-1** illustrates the survey transect locations in relation to the coastal zone of the study area and the nine established reach boundaries. The sporadic historical profiles range from 1934 to 1983. With the advent of the CCCSTWS-SD surveying efforts, beginning in 1984, surveys for each calendar year typically include a spring survey showing a depleted sand beach and a fall survey showing a well-developed sand beach. Each survey transect extends from the designated baseline to water depths of approximately 50 to 65 feet, MLLW. The complete plots of the surveyed profiles for each transect are presented in **Appendix BB**.

4.2.1 Mean Sea Level Beach Widths

The Mean Sea Level (MSL) beach widths were estimated from four of the CCSTWS-SD transects (CB-720, SD-670, SD-630, and DM-590) within the confines of the project study area of influence. The change in the MSL beach width over time for each CCSTWS-SD transect analyzed is shown in **Figure 4.2-2,** plotted in meters. The beach widths presented begin with the earliest known recorded survey performed in 1934 and extend through all survey efforts up until the year of 2001, which represents the comprehensive evolution of the MSL shoreline position for each respective transect.

The MSL beach width for the above referenced analyzed transects ranged between approximately 32 and 400 feet, respectively. The shoreline trends exhibited at Moonlight Beach (SD-670), Chart House (SD-630), and San Dieguito River (DM-590) appear to be comparable in both magnitude and seasonal variation while the MSL shoreline position at Batiquitos Beach (CB-720, the northernmost transect) is wider on a fairly consistent basis, although the seasonal variation follows a similar trend. The wider MSL shoreline trend of the Batiquitos Beach transect is consistent with the fact that the lagoon was once a historical fluvial contributor to Batiquitos Beach. As a result of urbanization and the completion of the Batiquitos Lagoon jetty construction in the 1990's, Batiquitos Beach is now a feeder beach where entrapped lagoon sediment is placed to ultimately nourish downcoast beaches. In fact, a portion of sediment dredged from the lagoon in 1998 and 2000 was placed on Batiquitos Beach.

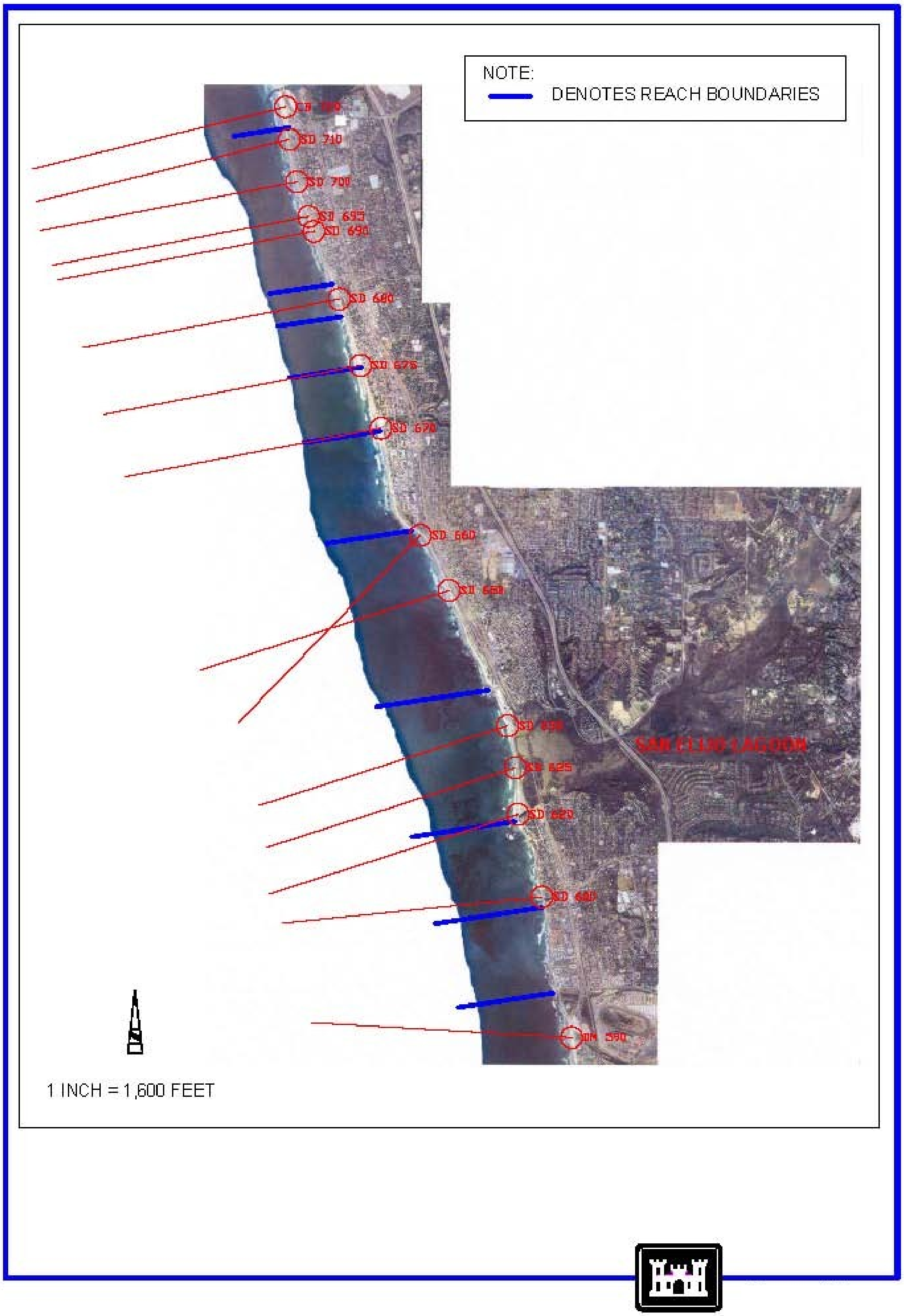
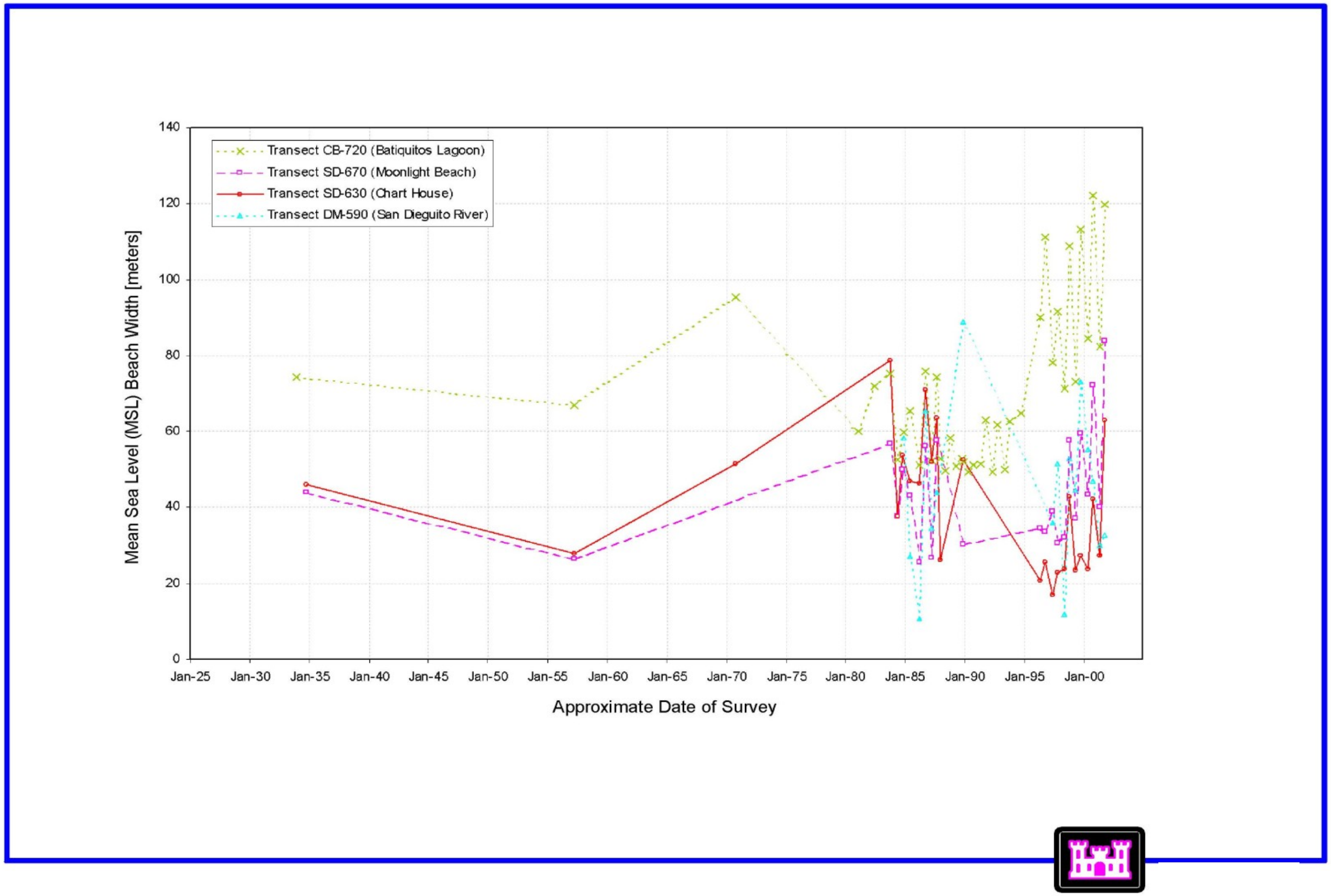


Figure 4.2-1 Survey Transect Locations



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2 **Figure 4.2-2 Mean Sea Level (MSL) Shoreline Evolution**

Table 4.2-1 Beach Profile Transect Locations, Sponsor and Period of Survey

Transect	Location (Reach No.)	Sponsor and Survey Period		
		CCSTWS-SD	Period (City Sponsor)	SANDAG
CB-720	Batiquitos Lagoon (North of Reach 1)	1934 – 1989	1988 – 1996 (Carlsbad)	1996 – present
SD-710 *	**Parliament Road** (Reach 1)	-----	-----	2001 – present
SD-700	Grandview Street (Reach 1)	-----	2000 – present (Encinitas)	2008
SD-695 *	**Jupiter Street** (Reach 1)	-----	-----	2001 – 2005
SD-690 *	**Jason Street** (Reach 1)	-----	2005 – present (Encinitas)	2001 – 2005
SD-680	Beacons Beach (Reach 2)	-----	-----	1999 – present
SD-675 *	Stone Steps (Reach 3, 4)	-----	-----	2001 – present
SD-670	**Moonlight Beach** (Reach 4, 5)	1934 – 1989	-----	1996 – present
SD-663	J Street (Reach 5)	-----	-----	2010 – present
SD-660	Swamis (Reach 6)	-----	2000 – present (Encinitas)	----
SD-650	San Elijo Park (Reach 6)	-----	2000 – present (Encinitas)	----
SD-630	**Chart House** (Reach 7)	1934 – 1989	-----	1996 – present
SD-625	Cardiff by the Sea (Reach 7)	-----	2000 – present (Encinitas)	----t
SD-620	Seaside (Reach 7, 8)	-----	2000 – present (Encinitas)	----
SD-610	Tide Park (Reach 8)	-----	2002 - present(Solana)	
SD-600	**Fletcher Cove** (Reach 8)	-----	-----	1996 – present
DM-595	Seascape Surf (Reach 9)		2002 - present (Solana)	
DM-590	San Dieguito Lagoon (South of Reach 9)	1984 – 1989	-----	1997 – present

Notes: All surveys performed subsequent to CCSTWS-SD were conducted by Coastal Frontiers Corporation. Transects in bold text were RBSPI Receiver Sites. * denotes added transects in support of RBSPI monitoring efforts.

With the exception of the Batiquitos Beach transect, the MSL shoreline position across the study area indicate widths range between approximately 65 and 200 feet. During depleted spring profile conditions, the MSL beach width typically ranges between 60 and 130 feet. When considering the gently sloping foreshore profile and the fact that high tide levels are several feet above the MSL elevation of +2.75 feet MLLW, the width of the dry beach above high tide is narrow to non-existent across a large proportion of the study area. Consequently, the toe of the coastal bluffs backing the sandy beach along most of the study area reaches are exposed to tidal and wave impacts over the potentially storm laden winter and spring months.

4.2.2 Mean Sea Level Shoreline Beach Widths from 1996 through 2009

The SANDAG and City of Encinitas sponsored transects that were surveyed during the spring of 1996 to 2009 were further analyzed in more detail to provide a better understanding of the more recent MSL shoreline fluctuations within the study area.

Table 4.2-2 presents the MSL beach widths for each surveyed transect within the study area. Of particular note is the shoreline recession, and the associated shoreline rebound, exhibited after the El Nino season of 1997-98, which is evident in the Spring 1998 and the Fall 2000 MSL shoreline positions, respectively. Furthermore, the Spring 2001 MSL shoreline position represents the pre-nourishment condition prior to construction of the SANDAG Regional Beach Sand Project, and the Fall 2001 MSL beach width represents the initial post-nourishment monitoring survey.

For a more adequate visual representation of the points mentioned above, **Figure 4.2-3** presents the seasonal change in MSL shoreline position for several SANDAG transects across the study area relative to the initial survey performed at each respective transect. Positive beach width changes represent accretion while negative beach width changes represent erosion relative to their initial survey. The seasonal fluctuations of the shoreline become more evident as the accreted foreshore sands surveyed during the fall season move offshore forming a nearshore bar during the winter months resulting in the landward migration of the MSL shoreline position. For a clearer representation of the annual changes in the MSL shoreline position as opposed to the seasonal, **Figure 4.2-4** and **Figure 4.2-5** presents the depleted spring and wide fall beach conditions, respectively, for five study area transects (CB-720, SD-680, SD-670, SD-630, and SD-600).

From **Figure 4.2-4**, it is evident that the shoreline leading up to the 1997-98 El Nino event consisted of erosion ranging from approximately 65 feet followed by a subsequent rebound through the Spring 2000 survey. After the Spring of 2000, it appears as though the erosional trend has again resurfaced as almost all of the Spring 2001 MSL shoreline positions have migrated landward of their Spring 2000 locations. It is noted that at Moonlight Beach (SD-670), the City of Encinitas typically imports approximately 1,000 cubic yards to renourish the beach each spring (which may have been included in some of these surveys) and a rip rap revetment protects the Chart House (SD-630) transect, somewhat limiting the back beach shoreline position.

Moreover, it is interesting to note that at both Batiquitos Beach (CB-720) and Fletcher Cove (SD-600), the shoreline recovery exhibited after the passing of the 1997-98 El Nino season did not fully rebound to their respective Spring 1996 locations. Considering the fact that Batiquitos Beach acts as a feeder beach to the Encinitas and Solana Beach shoreline, sand deficits exhibited at this location typically results in the short-term accretion of downcoast beaches followed by a more substantial duration of erosion as the sediment supply from Batiquitos Beach becomes more depleted. The loss of beach width at Fletcher Cove in Solana Beach, approximately 20 feet since 1996, is also of particular concern as beach widths here are typically narrow to begin with and Fletcher Cove represents the main beach area in Solana Beach designed for recreational purposes.

From **Figure 4.2-5**, it is clear that the variation of the MSL shoreline position for the summer profiles within the project area are somewhat stable; although, the shoreline position eroded between 6 and 65 feet between the October 1996 and October 1997 surveys. Directly following the severe El Nino winter of 1997-98, the summer profile rebounded from the previous year approximately 66 feet. However, in the period ranging between October 1998 and October 2000, the shoreline position appears to have been in a recession by an average magnitude of approximately 15 feet per year. The relatively benign wave environment of the 2000-01 winter and summer seasons is evident as the summer profiles rebounded for all transects except for the Batiquitos Lagoon transect (CB-720).

Table 4.2-2 Recent Mean Sea Level Shoreline Beach Widths Within The Encinitas and Solana Beach Study Area

Transect	Mean Sea Level (MSL) Beach Widths [feet]											
	Spring 1996	Spring 1998	Spring 2000	Fall 2000	Spring 2001	Fall 2001	Spring 2004	Spring 2005	Spring 2006	Spring 2007	Spring 2008	Spring 2009
CB-720 Batiquitos Lagoon	271	213	254	375	248	371	295	286	296	287	326	291
SD-710 * **Parliament Road**	---	---	---	---	140	**220**	145	118	206	143	121	130
SD-700 (ENC-01) Grandview Street	---	---	---	90	82	94	88	88	71	---	71	86
SD-695 * **Jupiter Street**	---	---	---	---	78	**119**	116	114	---	---	---	---
SD-690 * **Jason Street**	---	---	---	---	76	**108**	89	85	76	---	---	---
SD-680 Beacons Beach	---	---	96	144	84	168	152	148	111	126	130	127
SD-675 * Stone Steps	---	---	---	---	93	116	117	155	105	86	111	93
SD-670 **Moonlight Beach**	106	101	136	227	124	271	148	130	174	158	180	187
SD-660 (ENC-02) Swami's	---	---	---	136	122	141	135	123	89	---	---	---
SD-650 (ENC-03) San Elijo Park	---	---	---	142	113	149	137	141	117	---	---	---
SD-630 **Chart House**	66	77	75	132	87	**204**	123	183	135	133	126	131
SD-625 (ENC-04) Cardiff by the Sea	---	---	---	106	74	119	115	118	107	---	---	---
SD-620 (ENC-05) Seaside	---	---	---	99	88	100	142	121	93	---	---	---
SD-600 **Fletcher Cove**	110	71	101	108	90	**171**	93	107	112	82	110	84
DM-590 San Dieguito Lagoon	---	18	158	117	59	84	69	63	114	46	110	153

Note:
SANDAG Regional Beach Sand Project Receiver Sites are denoted in bold type
Fall 2001 Bold type widths are SANDAG RBSP post construction survey.
** Transects added in support of the SANDAG Regional Beach Sand Project

Spatial shoreline fluctuations within the Encinitas and Solana Beach coastal zone were also analyzed. **Figure 4.2-4** illustrates the MSL shoreline position for each spring survey subsequent to, and including, the 1996 survey from Batiquitos Beach (CB-720) to the San Dieguito River (DM-590). The results indicate that the MSL beach width is rather narrow, as the MSL shoreline location along 95 percent of the study area ranges between 60 and 130 feet.

The annual spring fluctuation in the shoreline position between 1996 and 2001 was approximately 30 feet across the study area. In addition, it is interesting to note that the three transects exhibiting the narrowest MSL shoreline position are located at Beacon's Beach (SD-680), the Chart House in Reach 7 (SD-630), and Fletcher Cove (SD-600). Moreover, it may be inferred from the figure that the annual nourishment efforts performed by the City of Encinitas at Moonlight Beach (SD-670) have had a positive impact on the beach width in that location.

Finally, the entrapped sediment point source locations of both Batiquitos Beach and the San Dieguito River delta have exhibited wide fluctuations in the MSL shoreline position, comparatively speaking. For both transects (CB-720 and DM-590, respectively), the spring 1998 survey exhibited the most landward erosion followed by varying degrees of shoreline accretion leading up to the spring 2000 survey. Between the spring 2000 survey and the spring 2001 survey, the shoreline at both Batiquitos Beach and San Dieguito River delta eroded 7.5 and 83.0 feet, respectively. **Figure** essentially verifies that the shoreline erosion and accretion trends within the study area are directly related to the shoreline fluctuations and the nourishment activities occurring at these two entrapped sediment point source locations. Therefore, the health of the Encinitas and Solana Beach shoreline is dependent upon the magnitude of storm activity and the influx of sediment from both Batiquitos Beach and the San Dieguito River delta.

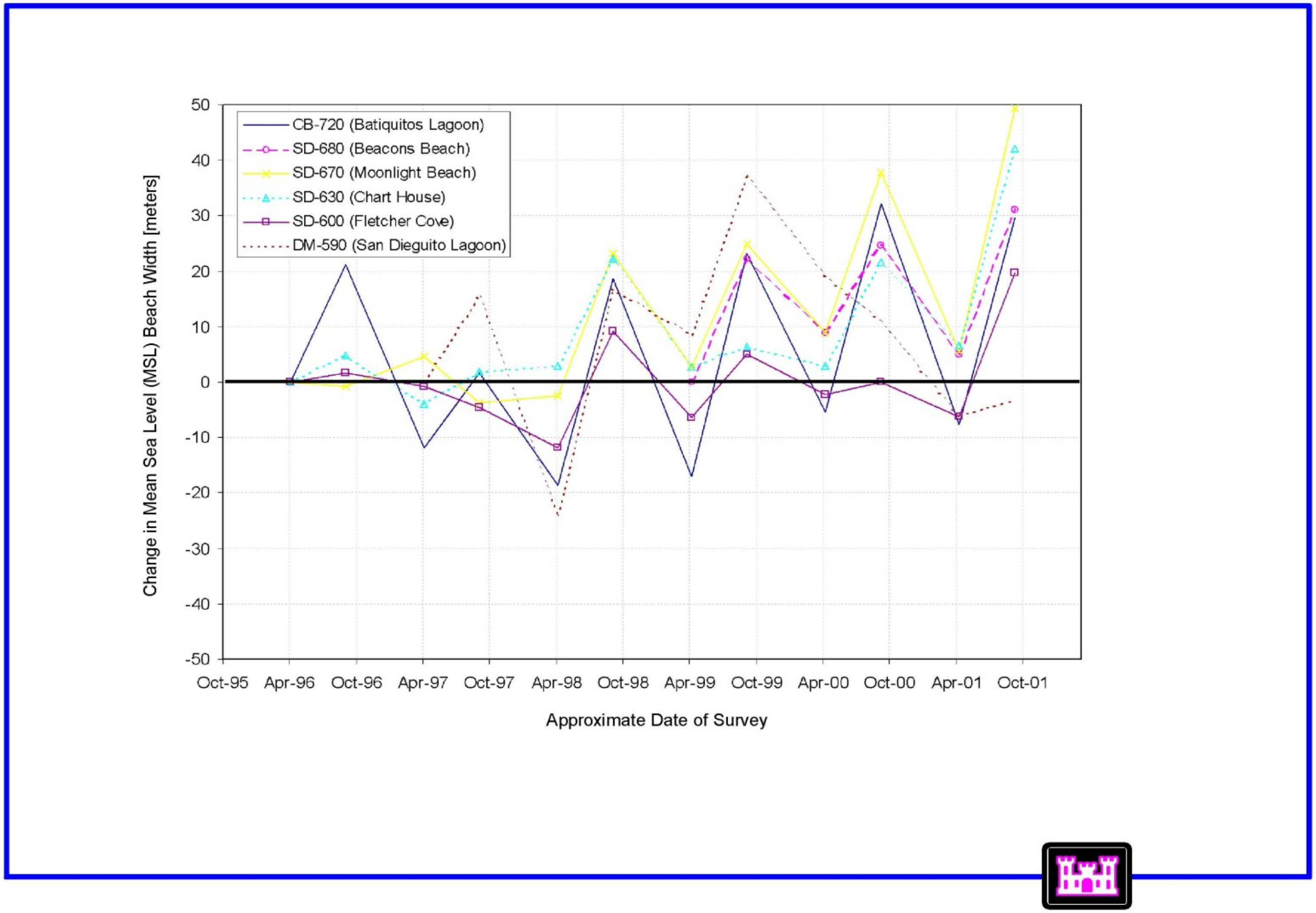


Figure 4.2-3 Seasonal Mean Sea Level (MSL) Shoreline Changes

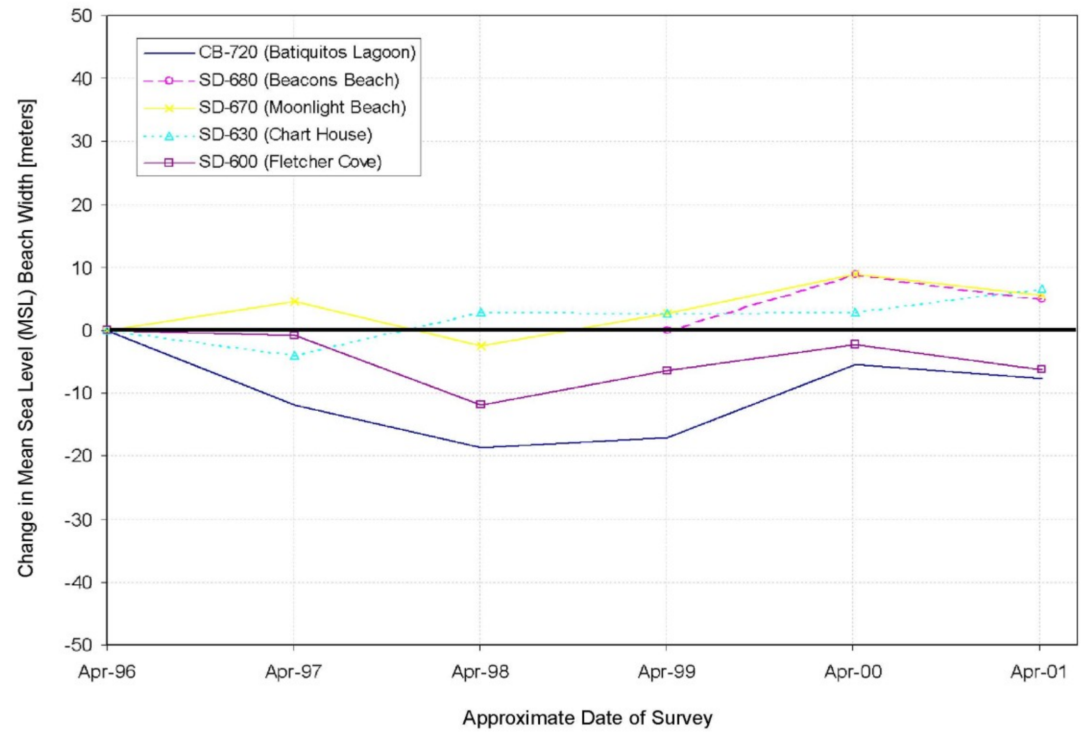


Figure 4.2-4 Annual Spring Mean Sea Level (MSL) Shoreline Changes

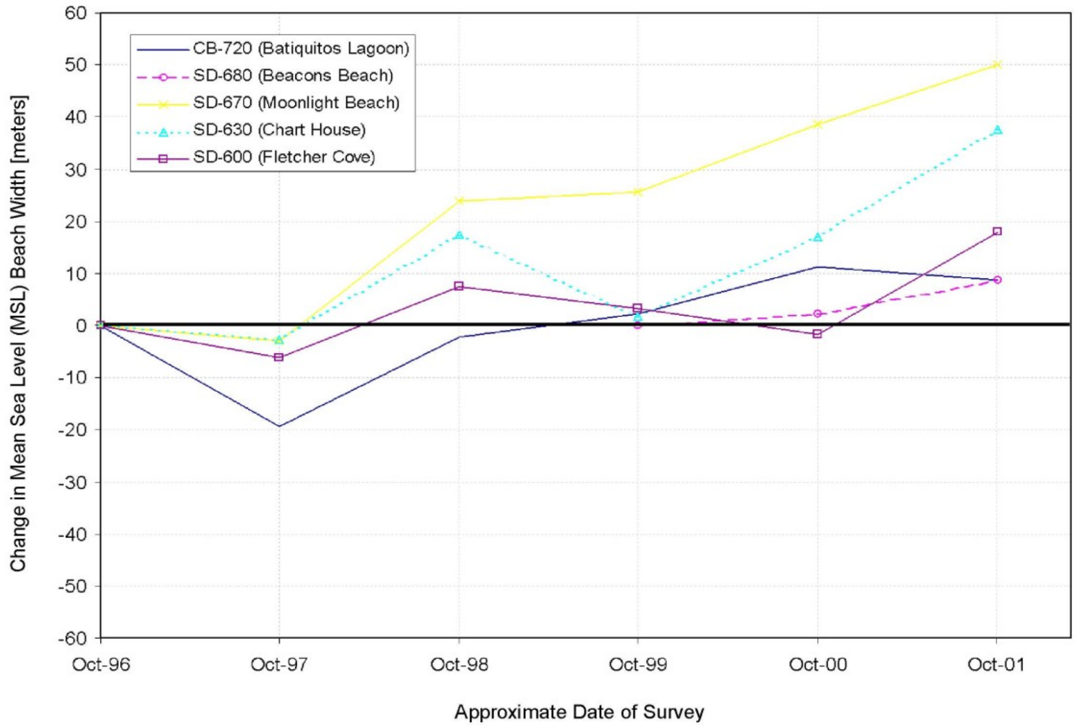
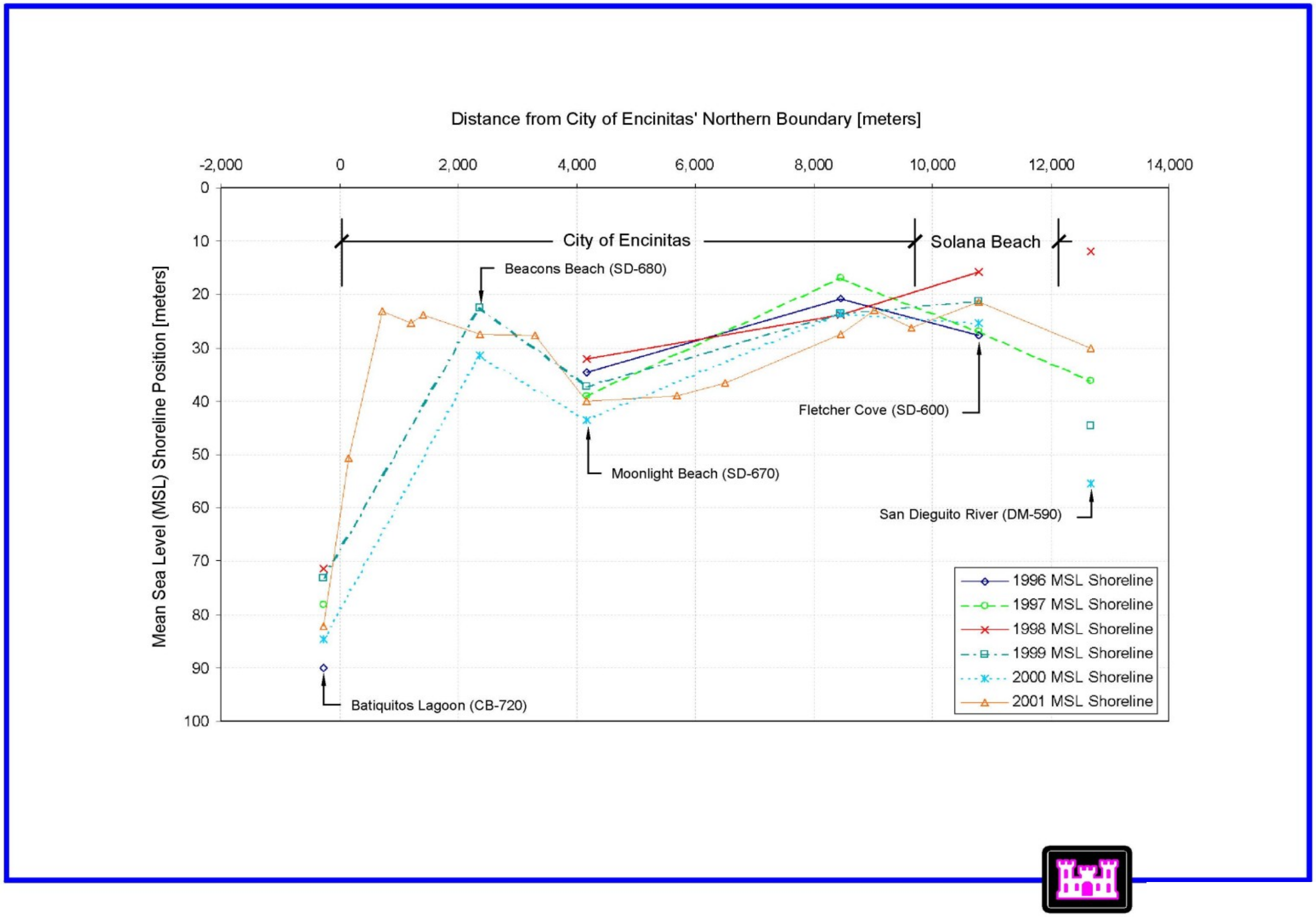


Figure 4.2-5 Annual Fall Mean Sea Level (MSL) Shoreline Changes



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2 **Figure 4.2-6 Annual Spatial MSL Shoreline Evolution**

4.3 Sediment Sources

This section details the various sediment sources including river, stream and lagoon discharge, coastal bluff erosion, beach erosion, and artificial beach nourishment within the Encinitas-Leucadia subcell.

4.3.1 River, Stream and Lagoon Sediment Discharge

There are several river and lagoon sediment discharge points within the Encinitas-Leucadia littoral subcell. Moreover, numerous rivers and small streams discharge sediment into the surrounding Oceanside Littoral Cell as well. However, due to inland urbanization and the population growth of the region, the largest drainage basins are extensively regulated by the presence of dams and reservoirs; thereby, drastically limiting their coastal sediment delivery potential. It has been estimated that a fluvial delivery reduction of approximately 75 percent has occurred within the Oceanside Littoral cell as a result of these flood control restrictions (California Department of Boating and Waterways (CDBW) and SANDAG, 1994). Fluvial delivery of sands and gravels between the Carlsbad submarine canyon and La Jolla was estimated to have decrease from 65,000 cy/yr to 5,000 cy/yr (USACE-LAD, 1991).

Three fluvial sources including the Batiquitos and San Elijo Lagoons, as well as the San Dieguito River are located within the study area or immediately adjacent to the study area. At Batiquitos and San Elijo Lagoons, it was estimated that the tributaries deliver approximately 820 and 6,900 cubic yards of sediment into the lagoon back basins, respectively (USACE-LAD, 1988). The current fluvial delivery is expected to be much less due to upland urbanization within the region. Furthermore, the delivered sediment settling in the backbay without migrating through the inlet areas does not provide any sand source to this littoral sub-cell. The maintenance dredging performed within the west and central basins of Batiquitos Lagoon and the inlet entrance at San Elijo Lagoon is primarily due to the entrapment of the tidalflood shoals developing in these areas. The volume of fluvial delivery to the project study area from the San Dieguito River was estimated to range from 620 to 13,000 cubic yards per year (Simons & Li, 1988 & 1985). Based upon the present drainage conditions resulting from urbanization and the associated construction of riverine control structures, the volume delivery would be at the low end of the estimated range.

4.3.2 Coastal Bluff Erosion

A large proportion of the steep coastal cliffs within the study area are geologically unstable due to the fact that most of them are comprised of sedimentary structures and not hard metamorphic and igneous rocks. However, a byproduct of coastal cliff failures resulting from the instability of the bluff is that sediment is directly supplied to the beach face; thereby, contributing a source of littoral sediment.

Previous estimates for the contribution of sediment from coastal bluff erosion differ; as failures are rather episodic in nature and the geological makeup of the cliffs vary depending upon their respective location within the project area. Based on literature review, the historical coastal cliff erosion rate within the project area range between approximately 0.2 and 0.4 feet per year. This corresponds to an erosion rate of approximately 20 to 40 feet per 100 years (AMEC, 2002 & USACE-LAD, 1996). Young and Ashford (2006) used airborne LiDAR to measure sea cliff retreat rates of 6 and 12 cm/yr for Leucadia and Solana Beach, respectively, with an average beach-sediment yield from the cliffs in the Oceanside littoral cell of 1.8 cubic meter/m-yr (0.8 cy/ft/yr).

The actual annual sediment contribution resulting from coastal cliff retreat may be estimated from the historic average bluff retreat rate, sand content of the bluff material, and the extent of any bluff toe protective devices. **Table 4.3-1** presents the projected annualized volume of sediment contribution to the study area as well as the required information used to calculate the estimated volume.

The estimated annual volume of sediment contribution resulting from bluff erosion, presented in **Table 4.3-1**, was calculated by multiplying the average retreat rate, bluff length, and bluff height for each reach. During the analysis, it was assumed that the bluff top would retreat and ultimately equilibrate to a more stable slope, as opposed to a total shearing off of the bluff face. As such, the estimated volumes were calculated accordingly. Once calculated, the volumes were adjusted to account for the percentage of sand within the bluff, as well as the percentage of existing toe protective devices.

The total estimated annual bluff retreat contribution of sediment for the entire study area is approximately 12,650 cubic yards per year. However, it should be noted that the sand percentages presented in **Table 4.3-1** includes a certain percentage of fine-grained material (e.g. less than 0.1 mm) that would most probably be suspended and carried offshore once exposed to wave and tidal activity. Fine-grained material could comprise as much as 10 to 20 percent of the sand percentages presented. It is noted that due to recent armoring at the bluff base, the annual sediment contribution from bluff erosion has been somewhat reduced.

Table 4.3-1 Estimated Annual Bluff Sediment Contribution

Reach	Average Retreat Rate (ft/yr)	Average Length of Bluff (ft)	Average Height of Bluff (ft)	Percent of Sand Content (%)	Percent of Toe Protective Device (%)	Annual Sediment Contribution (cy/yr)
1	0.25	6,500	65	69	18	1,100
2	0.36	1,800	90	67	45	400
3	1.20	580	90	78	70	1,200
4	1.0	2,500	80	79	10	2,800
5	0.56	5,200	90	61	30	2,100
6	0.62	5,800	80	50	60	1,100
7	N/A	N/A	N/A	N/A	N/A	N/A
8	1.0	3,500	80	79	50	1,900
9	1.0	4,100	75	78	50	2,100

Source: USACE-LAD, Appendix D, 2003

4.3.3 Artificial Beach Nourishment/Sand Bypassing

Artificial beach nourishment and sand bypassing have occurred on numerous occasions within the Encinitas-Leucadia subcell. In 1997, the Batiquitos Lagoon Enhancement Project was completed in order to restore the natural environmental lagoon habitat. This project placed about 1.8 MCY of sandy dredge material within the Encinitas-Leucadia subcell. In addition, on-going maintenance dredging of the lagoon for this ecosystem restoration project, has placed approximately 161,000 cubic yards of sand downcoast at Batiquitos Beach (SD-680). **Table**

4.3-2 presents the volume of dredged material, as well as the placement quantity for each dredging cycle at Batiquitos Lagoon.

Table 4.3-2 Maintenance Dredging and Beach nourishment Volumes Near Batiquitos Lagoon

Year	Bypass Volume (yd^3)	Note
1994-1997	1,800,000	Lagoon Restoration
1999	6,000	Placed south of entrance
2000	4,000	Placed south of entrance
2001	45,000	Placed south of entrance
2007	66,000	Placed south of entrance
2009	40,000	Encinitas Resort Hotel

Source: Coastal Frontiers Corporation

The San Diego Association of Governments (SANDAG) Regional Beach Sand Project I (RBSPI) was constructed during the summer of 2001. This project resulted in the placement of approximately 600,138 cubic yards of beach nourishment sands within the Encinitas and Solana Beach project study area. **Table 4.3-3** presents the SANDAG RBSPI beach nourishment placement locations and quantities within the study area.

SANDAG's RBSPII is expected to place up to 2.3 million cubic yards of sand at 10 receiver sites in San Diego County, with 587,000 cubic yards proposed for the study area. **Table 4.3-4** show the RBSPII preferred Alternative 2-R beach nourishment locations and quantities within the study area (AECOM et. al, 2011).

Table 4.3-3 SANDAG Regional Beach Sand Project Nourishment Characteristics

Receiver Site	Reach	Volume cy	Fill Length ft
Batiquitos Beach	1	116,923	1,600
Leucadia Beach (Beacon's)	1/2	131,837	2,300
Moonlight Beach	4/5	105,211	1,200
Cardiff Beach	7	100,510	900
Fletcher Cove	8/9	145,657	1,900

Source: NCI, 2001

Table 4.3-4 RBSPII Nourishment Characteristics

Receiver Site	Reach	Volume (yd^3)	Nourishment Length (ft)
Batiquitos Beach	1	118,000	Identical to RBSPI
Leucadia Beach (Beacon's)	1/2	117,000	Identical to RBSPI
Moonlight Beach	4/5	105,000	Identical to RBSPI
Cardiff Beach	7	101,000	Identical to RBSPI
Solana Beach (Fletcher Cove)	8/9	146,000	Identical to RBSPI

Source: AECOM

Figure 4.3-1 presents the pre-nourishment and 3-month post-nourishment MSL beach widths surveyed in May and October of 2001, respectively, as well as the previous October 2000 MSL beach width to better differentiate between the seasonal shoreline fluctuations and the beach nourishment accretions. A notable increase in MSL beach width is evident at Batiquitos Beach

(CB-720), Beacon's Beach (SD-680), Moonlight Beach (SD-670), Cardiff Beach (SD-630), and Fletcher Cove (SD-600) between the pre-nourishment (May 2001) and the 3-month post nourishment (October 2001) surveys. Furthermore, the post nourishment (October 2001) shoreline position is seaward of that of the previous October 2000 survey for the entire study area. This figure illustrates the immediate benefits of beach nourishment within this shoreline segment.

A number of smaller scale localized nourishment projects have also been performed within the study area. The City of Encinitas provides an annual beach nourishment of approximately 1,000 yd^3 to Moonlight Beach each spring and the mouth of the San Elijo Lagoon is periodically dredged to maintain adequate tidal flushing on an as-needed basis. This typically results in approximately 5,000 yd^3 of material placed south of the Lagoon each episode. Moreover, since October 1986, the San Elijo Lagoon has supplied an approximate average annual bypassing volume of 14,860 cubic yards to the immediate downcoast adjacent shoreline. **Table 4.3-5** shows the annual volume of the past downcoast beach nourishment related to the maintenance of the San Elijo Lagoon entrance. A detailed log of each dredging episode is presented in **Appendix C2**. It should be noted that the sediment dredged at the lagoon entrance cannot be credited as a sediment source as the deposited sediment originates from the partial reduction of the natural longshore sediment transport and not from upland fluvial sources. In addition, in the spring of 1999, approximately 51,000 yd^3 of sand was placed at Fletcher Cove as a result of the Lomas Santa Fe Grade Separation Project (AMEC, 2002).

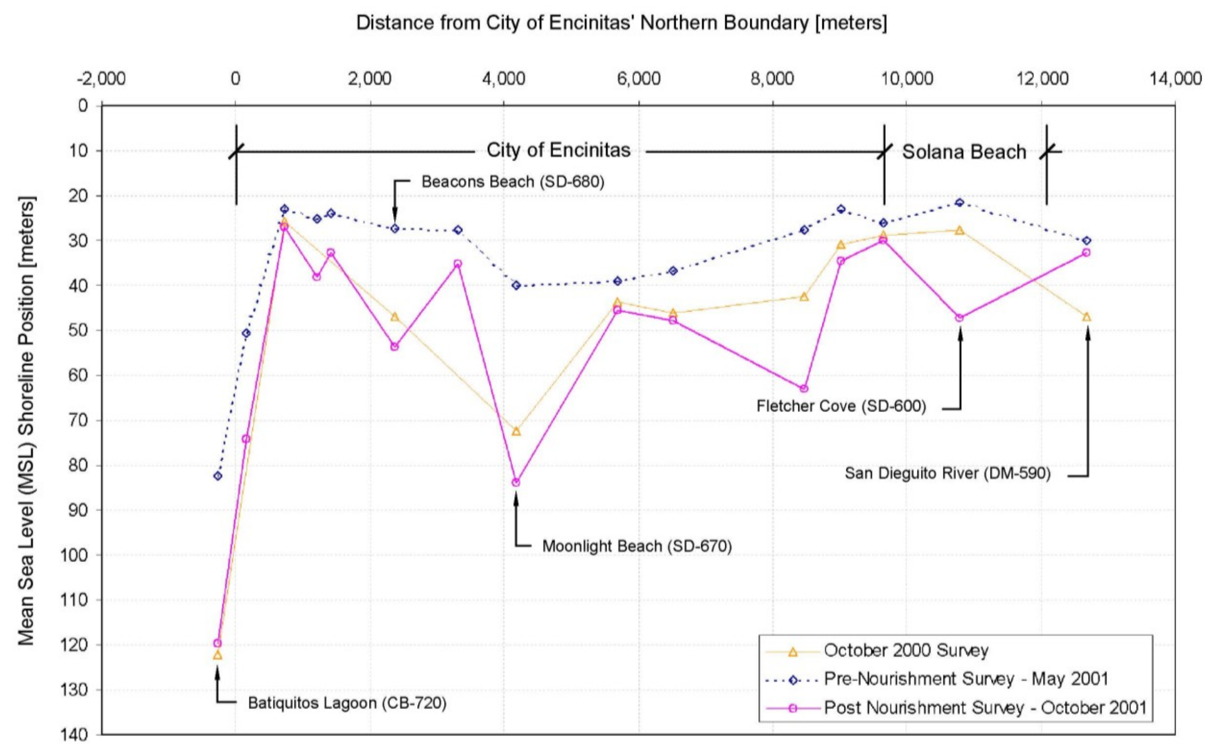
Table 4.3-5 Estimated Annual Volume Dredged From San Elijo Lagoon Entrance

Year	Annual Volume (yd^3)	Year	Annual Volume (yd^3)	Year	Annual Volume (yd^3)
1986	2,000	1995	6,000	2004	30,000
1987	4,000	1996	8,000	2005	17,000
1988	4,000	1997	31,000	2006	18,000
1989	3,000	1998	12,000	2007	19,000
1990	4,000	1999	17,000	2008	23,000
1991	4,000	2000	23,000	2009	19,000
1992	3,500	2001	23,000	2010	21,000
1993	7,500	2002	18,000		
1994	20,000	2003	32,000		

Source: San Elijo Lagoon Conservancy, 2002 and Coastal Frontiers Corporation, 2010

4.3.4 Beach Erosion

Beach erosion is typically associated with the landward migration of the shoreline and the associated reduction of dry beach width. The corresponding sediment losses on a beach can actually provide a sand source for downdrift beaches. Quantifying the magnitude of the sand volume fluctuations across each profile transect is critical in determining the rate of beach erosion within the study area, which thereby allows for an adequate representation of the associated sediment budget.



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2 **Figure 4.3-1 SANDAG Nourishment Project Impact**

During the CCSTWS-SD investigation, it was estimated (USACE-LAD, 1991) that the beaches within the vicinity of the Encinitas-Leucadia subcell experienced an average retreat rate of 1.0 to 2.0 feet per year from 1940 to 1960, an average annual advance of 3.0 to 4.0 feet per year between 1960 and 1980, and an average retreat of 1.0 to 2.0 feet per year after 1980. These findings are consistent with the environmental characteristics and the human interventions that occurred along this littoral cell during their respective time periods.

In order to quantify the change in sand volume density across the project study area, the annual depleted spring MSL shoreline beach widths at Batiquitos Beach (CB-720), Beacon's Beach (SD-680), Moonlight Beach (SD-670), Chart House (SD-630), and Fletcher Cove (SD-600) were analyzed for the period ranging from 1996 to 2001. This period was chosen to illustrate the volumetric fluctuations occurring as a result of the 1997-98 El Nino event, as well as the intermediate-term volumetric fluctuations subsequent to the relative rebound of the MSL shoreline position after the spring 1998 survey.

The changes in volume density between relevant surveys at each above-referenced transect were analyzed by employing the volume change-to-shoreline advance or retreat ratio (V/S) developed during the CCSTWS-SD study (1991). A V/S value of one implies that there is one cubic yard of volume change for one-foot of beach advancement or retreat per lineal foot of shoreline. In the CCSTWS-SD analysis, the shoreline movements (S) were referenced to the MHHW location (+5.4 feet, MLLW) while the volume changes (V) were measured from the profile baseline location to various water depths. The V/S ratio for both all available data and extreme event data exclusively was estimated for all of the different shoreline reaches in San Diego County. Within the Encinitas-Leucadia sub-reach, the V/S ratio to reference depths of -10, -30 and -40 feet were between 0.222 to 0.463 cubic yards per foot for averaged long-term conditions and between 0.629 and 0.726 cubic yards per foot for short-term extreme events (USACE-LAD, 1991, Table 3-6).

Based on both the previous CCSTWS-SD surveys and the recent SANDAG surveys within the study area, the average depth of closure (or depth at which net sand movement in the cross-shore direction does not produce measurable depth change) is approximately -30 feet, MLLW. For this reason, the V/S ratio corresponding to this reference depth for the Encinitas-Leucadia sub-reach was employed.
Table 4.3-6 presents the results of the volumetric density changes across the Encinitas and Solana Beach project study area from Spring 1996 to Spring 2001.

Table 4.3-6 Estimated Average Annual Sediment Contribution Due to Beach Erosion/Accretion (1996 to 2001)

Transect	Location	Annual Cross-Sectional Volume (CY/ft/yr)	Annual Volume (CY/yr)
CB-720	Batiquitos Beach	-0.338	-1,500
SD-680	Beacon's Beach	+3.000	+22,000
SD-670	Moonlight Beach	+0.241	+2,400
SD-630	Chart House	+0.289	+3,000
SD-600	Fletcher Cove	-0.272	-1,900

The annual volumes presented in **Table** 4.3-6 are based upon a V/S ratio of 0.222 cubic yards/foot for all available data. Shoreline advance is denoted by a plus (+) sign while shoreline retreat is represented by a minus (-) sign. Summing the estimated annual volumes calculated between 1996 and 2001 for the project study area yields a net beach accretion of 24,141 cubic yards per year. The beach accretion at Beacon's Beach (Transect SD-680) is probably due to the dispersive effect of the feeder beach that was established at Batiquitos Beach after the 2000 maintenance dredging at Batiquitos Lagoon, as stated in **Section 4.3.3**.

In order to assess the coastal erosion impacts resulting from the 1997-98 El Nino event, a similar set of calculations was performed from Spring 1996 to Spring 1998. **Table 4.3-7** presents the results of this analysis. The annual volumes presented in **Table 4.3-7** have been annualized for the interim 2-year (1996-1998) period of record and are based upon the extreme event V/S ratio of 0.629 cubic yards per foot. Summing the estimated annual volumes yields a net beach erosion of 68,315 cubic yards per year occurring over the storm laden 1997-98 El Nino event. However, it should be noted that surveys were not performed at Beacon's Beach (SD-680) until 1999; therefore, potential volumetric gains, resulting from the feeder beach at Batiquitos Beach, are not represented in this extreme event analysis.

Table 4.3-7 Estimated Average Annual El Nino Event Sediment Contribution Due to Beach Erosion/Accretion (1996 to 1998)

Transect	Location	Annual Cross-Sectional Volume (cy/ft/yr)	Annual Volume (cy/yr)
CB-720	Batiquitos Beach	-5.81	-42,500
SD-680	Beacon's Beach	no data	no data
SD-670	Moonlight Beach	-0.75	-10,700
SD-630	Chart House	+0.90	+10,100
SD-600	Fletcher Cove	-3.67	-25,400

4.4 <u>Sediment Sinks</u>

This section details the various sediment sinks located within the Encinitas and Solana Beach study area, which are ultimately responsible for the loss of sediment within the system. When sand enters into a sediment sink, the material is lost and will not return to the beach without some form of human intervention. For this reason, it is important to quantify the deficit imposed on the system. The sediment sinks located within the Encinitas-Leucadia subcell include entrapment caused by lagoons and offshore losses.

4.4.1 Lagoon Entrapment

As described previously, several lagoons and marshes exist along the Encinitas-Leucadia subcell, namely Batiquitos and San Elijo Lagoons and the San Dieguito River delta to the south. With the exception of small storm-induced overwash and the formation of small flood-tide deltas, the quantity of entrapped alongshore transported sediment updrift of the tidal entrances is not presently significant in this littoral subcell. However, due to sedimentation, the lagoon and river mouths are periodically dredged to ensure adequate tidal flushing; thereby, resupplying good quality beach sand to adjacent beaches.

4.4.2 Offshore Losses

The offshore transport of sediment typically results from large storms that carry sediment offshore through unusually large cross-shore currents. It is possible that the sediment has been deposited so far offshore that the sediment does not migrate back to the shoreline. The fact that the San Diego shoreline erosion began after 1983 probably demonstrates the above-described offshore sediment transport that resulted from the clustering extreme storms occurring during the 1982-1983 El Nino year.

Estimates of the actual quantity of sediment carried offshore by the processes defined above are difficult to quantify; however, it has been estimated that as much as 26,000 to 113,000 cubic yards of sand per year could be deposited offshore as a result of rip currents (Tekmarine, 1987). In addition, based on an extensive evaluation of bathymetric information obtained from survey data extending from 1934 to 1972 presented in CCSTWS-SD, it appears as though approximately 1,000,000 cubic yards of sediment has been deposited at water depths ranging from 30 to 120 feet offshore of the project study area (USACE-LAD, 1991). This correlates to an approximate annual offshore sand loss of approximately 25,650 cubic yards per year across the Encinitas and Solana Beach study area.

4.5 Alongshore Littoral Transport

This section summarizes the alongshore transport rate potential for the Encinitas-Leucadia subcell developed, in part, during the Coast of California Storm and Tidal Waves Study for San Diego County. As discussed previously, the net alongshore transport rate within the study area has been substantially impacted over the years through human intervention. Prior to 1978, these impacts were not readily noticeable due to the relatively benign wave climate extending from approximately 1945 through 1978. Coincidentally, this time period also corresponded with an unprecedented degree of coastal development along the Encinitas and Solana Beach study shoreline, as well as the entire San Diego County coastal region. This development included the rapid urbanization of coastal bluffs, the development of two harbors (Oceanside and Dana Point), one coastal power plant (Encinitas at Agua Hedionda Lagoon), and the construction of numerous groins, jetties, seawalls, and blufftop residences.

The benign wave environment heading into the late 1970’s, coupled with the relatively large quantity of nourishment sands placed along the coast during the 1960’s, yielded a somewhat healthy and stable regional shoreline until the early 1980’s. The relatively mild and seasonably predictable wave climate of the uniform epoch of 1945 to 1978 was followed by a period of more variable and, at times, far more intense wave events. Most notably, these events occurred during the winters of 1979-80, 1982-83, and 1997-98. As stated previously, the winter of 1982-83 was particularly severe as a series of clustering storm events occurred. In addition, the yield of sediment from upland rivers and streams decreased dramatically due to the construction of dams and the concretization of flood control channels. Consequently, sand depletion alongshore the study shoreline area began after the 1982-1983 El Nino season.

Estimates suggest that an average net southerly littoral alongshore transport rate of between approximately 100,000 to 250,000 cubic yards per year occurred from 1945 to 1977 (Techmarine, 1987 & USACE-LAD, 1991). It was also estimated under the same study that from 1978 to the late 1980’s, the net southerly transport rate decreased to between 0 and 40,000 cubic yards per year. The reduction of the net alongshore littoral transport is probably attributed to the increasing occurrence of the southerly swell pattern during the 1980’s period or the historical wave data prior to 1978 did not fully comprise all wave patterns that include both the northwest and southerly swells. During a recent study, conducted by the City of Encinitas, for the relocation of the San Elijo Lagoon inlet, the average net southerly littoral transport potential at Cardiff was estimated to be 56,175 cubic yards per year, which was based upon wave climate data extending from 1978 to 1994 (Coastal Environments, 2001). It should be noted that the ability of these estimated rates to move sand is severely limited by the overall deficit of sand available for transport. Therefore, the natural alongshore transport potential in response to the regional oceanographic environment is not performing at its true capacity.

4.6 Cross-Shore Littoral Transport

The cross-shore transport of sand refers to the seasonal and episodic fluctuations of the beach profile as sands shift to equilibrate with the incoming wave environment. The offshore location where little net sediment transport occurs beyond is known as the depth of closure.

While the alongshore sediment transport is primarily due to the wave-induced alongshore current, the cross-shore sediment transport is a result of the water particle motions under the influence of waves and the formation of near shore circulation cells and rip currents. Seasonal shoreline changes are considered to be in response to the greater incidence of storms during winter and the associated seaward sand transport and storage in near shore bar formations (Dean and Dalrymple, 1999). With the increased wave heights associated with storms, the bar typically forms farther offshore and is larger in size. The larger offshore bar formation requires a greater volume of sediment, which is provided in part by erosion of the subaerial portion of the beach.

Evidence indicating the transport of sediment across the shore face within the study area is illustrated in the beach profile surveys presented in **Appendix C1**. For the most part, the shapes of these beach profile surveys show the seasonal cross-shore sand fluctuation. In addition, possibly contributing to the cross-shore sand transport within the study area is the contribution of cross-shore currents that could transport sediment offshore during storm events. Cross-shore currents are essentially jets of water that emanate through the breaker line of the surf zone that have the ability to carry with them wave suspended sediment. It was estimated in the CCSTWS-SD study that as much as 25,650 cubic yards of sand could be lost each year within the study area as stated in **Section 0**.

4.7 Sediment Budget

The shoreline trends along the beach essentially dictate the conceptual sediment budget for the region of interest. If beaches are eroding the sediment budget has a net deficit of sand (i.e., more sediment is being lost than gained); however, if beaches are accreting, the sediment budget has a net surplus of sand (i.e., more sediment is being gained than lost). When beaches are stabilized and no net accretion or erosion is occurring along the shoreline, the sediment budget is balanced. In order to develop the sediment budget for the Encinitas and Solana Beach project study area, all of the sand inputs (sources), outputs (sinks), littoral transport paths, and storage capacities quantified in the previous sections have been compiled and combined.

4.7.1 Historical

Prior to 1940, the San Diego County coast experienced periods of relatively abundant sand supply following large sand injections from river floods due to the upland absence of channel concretization and damming. In addition, since the alongshore sediment transport was not disrupted by shore perpendicular coastal structures, the beaches within the Encinitas and Solana Beach coastal zone were relatively stable. Between 1960 and 1978, the effects of man-made coastal structures, namely at Oceanside Harbor and Agua Hedionda Lagoon, had a subtle impact on the stability of the coastal beaches within the project study area as the predominant storm and wave events during this period were fairly benign. However, from 1978 through to the present, a period during which extreme wave episodes have been well above average when compared to other periods over the past century, human intervention in the form of coastal structures and upstream dams on major rivers has had a profound impact on the now erosive nature of the beaches of Encinitas and Solana Beach. As a result, the average net transport rate was estimated to be between 40,000 and 56,175 cubic yards per year to the south in the project study area since the early 1980's (USACE-LAD, 1991& Coastal Environments, 2001). The CCSTWS (USACE – LAD, 1991) report estimates net transport alongshore into this sub-cell as 270,000 cy/yr for the two pre-1980 sediment budget time periods.

4.7.2 Present

The above referenced historical sediment budget quantities indicate that the health of the Encinitas and Solana Beach coastal region is largely dependent upon the wave climate and the degree of human intervention. It is evident from the analysis of the sediment budget that human activity within the influence of the coastal zone has had both negative and positive effects on the beach width within the study area. The negative impacts have been due primarily to poor watershed management practices and, to a lesser extent, the construction of Oceanside Harbor, which have significantly reduced the sand supply within the Encinitas and Solana Beach study area by curtailing both the flood waters and by disrupting the natural flow of the alongshore littoral transport. In order to mitigate the losses associated with the reduction in the delivery of sediment to the coastal zone, beach nourishment efforts have been instituted at several locations within the study area. These nourishment efforts have resulted in the placement of approximately 783,200 cubic yards of sand along the Encinitas/Solana Beach shoreline to date. The replenishment includes the regular sand-bypassing at Batiquitos Lagoon since 1998, annually imported material at Moonlight Beach for the past ten years, an opportunistic sand placement at Fletcher Cove, and the 2001 SANDAG RBSPI project.

Although these artificial nourishment efforts have had some positive effects, the sediment budget is currently in a net deficit, which is expected to continue into the future without some form of remediation. In fact, for the period ranging between 1996 and 2001, but prior to the SANDAG Regional Beach Sand Project, the project study area beaches exhibited a net deficit of approximately 9,767 cubic yards per year, assuming that the fluvial delivery from the San Dieguito River contributed to this subcell. The total sediment deficit within the project area was first derived by summing the total annual volumes for the fluvial contribution, coastal bluff contribution, and the artificial beach nourishment contribution, which yields a value of 33,900 cubic yards per year. It is noted that the by-passing volume at Batiquitos Lagoon in 2002 and the nourished material from the SANDAG Sand Project are not included in the computation as the beach profile comparison is from April 1996 to April 2001. The SANDAG Sand Project in the Encinitas/Solana Beach shoreline segment did not commence until June 2001.

Table 4.7-1 details the itemized sediment budget quantities over the course of this 5-year period.

Table 4.7-1 Encinitas and Solana Beach Sediment Budget Analysis (1996 to 2001)

Coastal Process Component	Estimated Annual Volume (cy/yr)
Fluvial Contribution	+621
Coastal Bluff Contribution	+12,700
Artificial Beach Nourishment/Sand Bypassing	+20,600
Total sand sources	+33,900
Net Beach Gain from 1996 to 2001	+24,200
Sediment Loss within Subcell	-9,700

Notes: + denotes gain and – implies loss

As a result of the sand deficient beaches, storm and wave events impinge directly upon the base of the bluffs causing them to erode and eventually fail. Over the years, numerous blufftop homeowners have constructed bluff stabilization structures in the form of seawalls to maintain the integrity of the bluffs, thereby protecting their homes. In addition, severe bluff failures resulting in a total shearing off of the bluff face are extremely dangerous to recreational beach users as well as the blufftop residents. In the year 2000, a severe block failure resulted in a fatality. For these reasons, it is important to mitigate for the loss of sediment that historically was present along the Encinitas and Solana Beach shoreline.

4.7.3 Future

The health of the Encinitas and Solana Beach shoreline is dependent upon the magnitude of storm activity and the influx of sediment from both Batiquitos Beach and the San Dieguito River delta. The Coast of California Storm and Tidal Waves Study for the San Diego County Region (1991) predicted that extensive damage and loss of property would occur over the next 50 years resulting from the loss of beach width and the associated coastal bluff retreat. With the fairly thin sand lens, measured in the nearshore and offshore zone (USACE-LAD, 1988), that is likely to be severely depleted during the winter season, it is almost certain that the bluff toe erosion will continue along the Cities of Encinitas and Solana Beach in the absence of protective beach sands at the base of the bluff. Furthermore, in Cardiff, without a moderate sandy beach fronting the restaurant buildings and Highway 101, the dwellings and highway are vulnerable to storm damage and wave overtopping. As a result, this coastal engineering analysis models the potential without project future erosion scenarios within each reach of the study area over the next 50 years.

5 WITHOUT PROJECT CONDITIONS

5.1 Statement of the Problem

Prior to the 1982-1983 El Nino season, which resulted in an unprecedented number of severe winter storms that impacted the southern California coastline, a moderate beach with a sandy berm existed along the shorelines of Encinitas and Solana Beach. The sandy berm provided a buffer that prevented the base of coastal bluffs from being exposed to direct wave and tidal impingement. During the severe 1982-1983 El Nino winter season, shore morphology was altered in that beach sands were stripped off the beach and deposited offshore. A large proportion of these sands were either transported beyond the depth of closure or carried southward (downcoast) via alongshore currents. Consequently, a sand-limited beach condition was observed in the subsequent years within Encinitas and Solana Beach. It is noted that the depth of closure is defined as the most landward depth at which no significant cross-shore sand movement occurs seaward of this location.

As the beach with little sandy berm was unable to provide a natural buffer for protecting the bluff base against wave action, erosion along the bluff base occurred under wave and tidal actions, undercutting the bluff, resulting in notches and sea caves at the toe of the bluff. These notches extend for hundreds of feet along the bluff base and several sea caves grew 30 to 40 feet deep. As a result of the deep notches reducing the support at the base, the upper bluff failed and sheared off. Detailed logs of historic bluff failures that were reported by both Cities of Encinitas and Solana Beach are respectively presented in **Appendix C3**. In total, there were 203 reported bluff failures for both Cites between 1990 and 2008.

A bluff failure occurs when a portion of bluff material separates from the bluff and falls on the beach below. After the bluff failure occurs, the remaining upper bluff slope becomes over-steepened beyond the angle of repose. This further induces additional bluff retreat at the top as the upper bluff slope gradually declines to a more stable angle. As the bluff collapses, the material falls onto the beach face below reducing lateral beach access and further endangering the safety of beachgoers. Moreover, with each successive episodic upper bluff failure, the public infrastructure and private dwellings located at the bluff top become increasingly threatened. The damage and collapse of the bluff-top structures, due to episodic and unpredictable bluff failure, have occurred in the past and recently. It is expected that the aforementioned bluff failures will continue to worsen if no measures to prevent bluff failure are implemented.

At Cardiff (Reach 7), the shoreline consists of a low-lying narrow beach backed by the San Elijo Lagoon, coastal development and Highway 101 that is protected by a non-engineered revetment. The highway corridor is occasionally flooded owing to wave overtopping during severe storm events. For the most part, this is limited to only partial lane closures for a short duration due to road inundation and the time required to clear debris. Since 1988, there have been numerous road closures of different magnitudes and durations, translating to approximately four (4) road closures per year. The data compiled by the City of Encinitas for each road closure during this period is presented in **Appendix C4**.

In addition to periodic Highway 101 road closures, several oceanfront restaurants and parking facilities located just downcoast (south) of the entrance of the San Elijo Lagoon are also prone to storm-related inundation. Although an engineered riprap revetment protects the restaurants, flooding and content damages have occurred in the past as a result of storm-induced wave overtopping and projectile debris. It is noted that during the 2009-2010 El Nino season, bank erosion at an isolated location along Highway 101 occurred even with the presence of the existing riprap revetment.

5.2 Analysis of The Problems

Analyses in the past to assess the above-identified bluff retreat for any damage potential always resorted to the average rate over a project design life (USACE-LAD, 1996). Though the annualized rate of coastal bluff erosion is a good indicator of the gradual retreat at the bluff top, it does not adequately represent the episodic nature of bluff failure, when almost instantaneously several feet of bluff top can fail and fall onto the beach below. An annualized retreat rate essentially accounts for the long-term average bluff retreat of various episodic failures and periods of little or no erosion activity. As a result, the annualized retreat rate, when averaged over a long period (e.g. 50 years), tends to yield a misleading picture of bluff erosion and the resulting damage related to the bluff-top development. Therefore, this analysis employs the Monte Carlo Simulation technique to statistically characterize each unpredictable and episodic bluff failure event within the study area over a 50-year design life cycle.

The formulation of the benefits are based primarily on avoided seawall construction cost and the "trigger" for when these private investments would occur is tied to set-back distance between top of the bluff edge and the nearest structure. Many of these set-back distances are not large compared to the retreat experienced in one episodic block failure; however, the set-backs are large relative to the long-term average bluff retreat rate. The discounting of when the investments occur over the economic life has a significant impact on the Benefit Cost Ratio.

For the low-lying narrow sandy and cobble beach at Cardiff (Reach 7), a detailed wave runup analysis was performed to determine the magnitude of waves overtopping the non-engineered riprap revetment that protects the Highway 101 Corridor. Past recurrence events indicate that the majority of wave overtopping occurs during storm events coinciding with high water levels. Due to the randomness of water levels and the intensity of a particular wave event, a probabilistic approach of jointly defining the occurrence of high water levels and severe wave events was applied to this wave overtopping analysis.

5.2.1 Future Sea Level Rise Scenarios

Global average sea levels have risen approximately 0.3 ft. to 0.8 ft. over the last century and are predicted to continue to rise between 0.6 ft and 2.0 ft over the next century (IPCC 2007). In 2009, a study titled "The Impacts of Sea-Level Rise on the California Coast" was performed by the California Climate Change Center with funding from the California Ocean Protection Council (OPC), California Energy Commission (CEC), California Environmental Protection Agency, Metropolitan Transportation Commission, and California Department of Transportation (Caltrans). Scientific data gathered as part of this study from 1993 to 2006 suggests that global sea level rise has outpaced the IPCC predictions (California Climate Change Center 2009). Houston and Dean (2011) analyzed U.S. tide gage data and showed the rate of sea level rise to have been decelerating. Never the less, the potential effects of an acceleration in sea level rise on coastal environments include erosion, net loss of shorefront, increased wetland inundation, and storm surge have the potential to displace coastal populations, threaten infrastructure, intensify coastal flooding, and ultimately lead to loss of recreation areas, public access to beaches, and private property.

A large degree of uncertainty exists in the models of future sea level rise (SLR), particularly when projected far into the future. However, sea level rise effects during the project's evaluation period should be considered and it is in this study by evaluating scenarios of future accelerating rates of SLR. The bluff retreat model, discussed further in this Section, is driven by wave attack intensity and duration which increases with higher relative sea levels. The limited volume of littoral drift within the area will be re-distributed across the profile providing even less bluff toe protection than its present day condition. Project alternatives also have different requirements for different SLR scenarios if they are to provide consistent shore erosion risk reduction over time. The "With-Project" is discussed in **Chapter 6**, Plan Formulation.

USACE interim policy on future SLR was issued in EC 1165-2-211, INCORPORATING SEA-LEVEL CHANGE CONSIDERATIONS IN CIVIL WORKS PROGRAMS (1 July 2009). (This guidance was updated in 2011 with EC-1165-2-212 with slight changes in the equations that would have an insignificant effect on this studies results). This guidance includes consideration of sea level rise by evaluating scenarios of three projections of SLR:

1) An extrapolation of local, historic relative sea level rise, which for the study area is taken from NOAA tide station measurements at the La Jolla tide gage (USACE Low);
2) An intermediate sea level rise based on Curve I from the National Research Council (NRC 1987, USACE Intermediate); and
3) A high estimate of high sea level rise based on Curve III from the NRC study (USACE High).

The NRC eustatic SLR projections are adjusted for local land movements to approximate a relative SLR. These projections are shown in the solid lines of **Figure 5.2-1**. For comparison, the more recent projections published in IPCC(2007) are also shown. The recent projections are bounded by the older NRC curves. **Table 5.2-1** show the projected mean sea level rise relative to the current NOAA tidal epoch (1983-2001) over the project planning horizon.

Table 5.2-1 Future Sea Level Rise Scenario

Year	Low (Historic extrapolation)	Intermediate (NRC Curve I)	High (NRC Curve III)
1992 (mid-point 1983-2001 epoch)	0.0 ft	0.0 ft	0.0 ft
2015 (start of planning horizon)	0.2 ft	0.4 ft	0.4 ft
2065 (end of planning horizon)	0.5 ft	1.8 ft	2.5 ft

In response to the U.S. Army Corps of Engineers' Engineer Circular, EC 1165-2-211 "Water Resource Policies and Authorities Incorporating Sea-level Change Considerations in Civil Works Programs" on July 1, 2009, the Encinitas-Solana Beach Feasibility Study Project Development Team (PDT) agreed to develop a White Paper describing the approach to incorporating EC 1165- 2-211 into the feasibility study. The Sea Level Rise White Paper (Everest/EDAW, 2009) was reviewed by the USACE Coastal Planning Center of Expertise (PCX), South Pacific Division (SPD), and Sea Level Rise Review Panel.

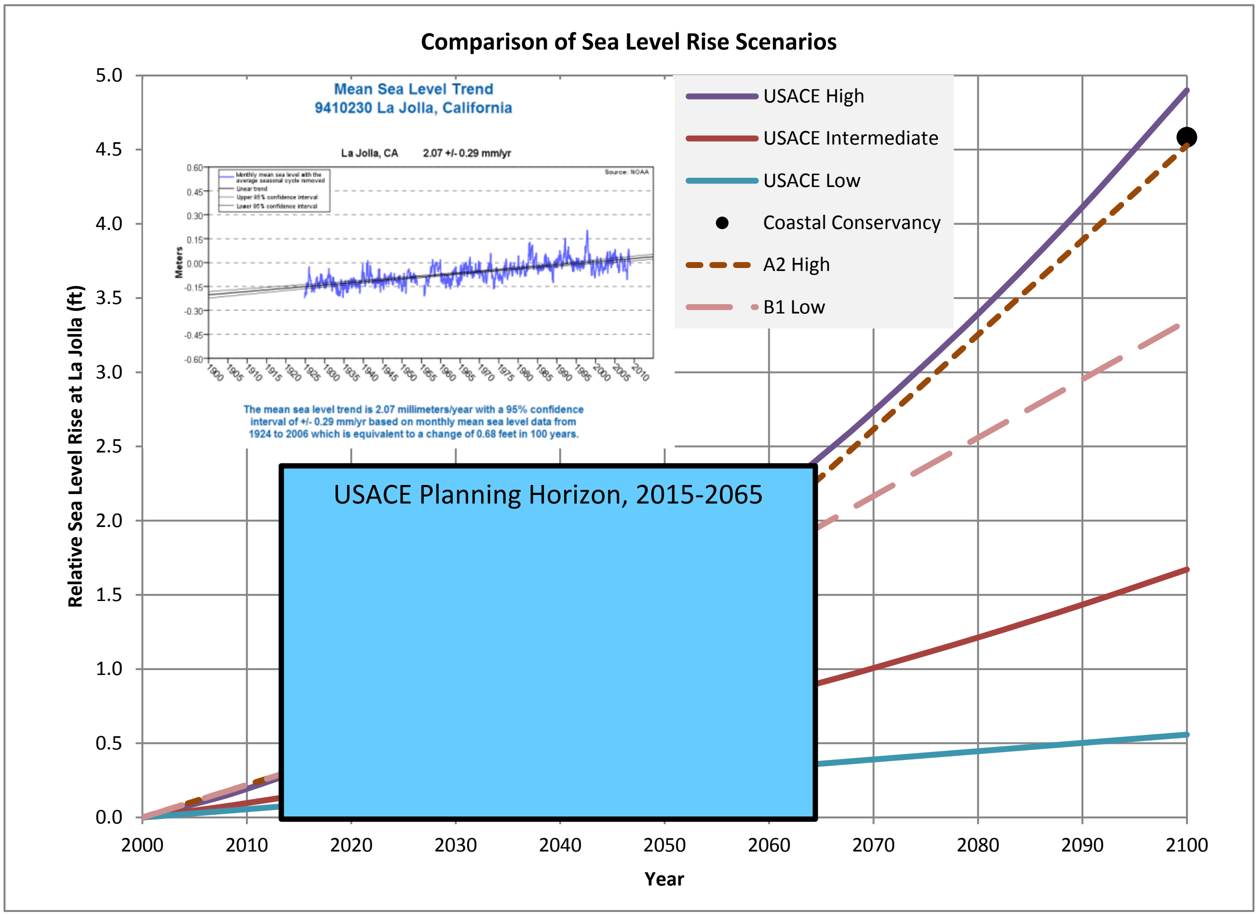


Figure 5.2-1 Sea Level Rise Estimates using USACE and California Climate Change Center 2009z, Values

5.2.2 Future Without Project Beach Conditions

The SANDAG Regional Beach Fill Project I was completed in fall of 2001. In excess of 600,100 cy of sand were placed at five different beach locations within the two cities (**Table 4.3-3**) somewhat alleviating the beach's sand-starved conditions. In addition, past sand replenishment projects using sands outside the Oceanside Littoral Cell have supplied small volumes of sediment to this subcell. However, it is still expected that the sand deficient conditions within the entire study area will continue, as previously stated in Section 4.6.5, without implementing a regular sand replenishment program similar to the one completed in 2001 (Noble Consultants, 2001). It is noted that the subject shoreline was severely eroded during the 2009-2010 El Nino season and returned to the depleted beach conditions prior to the 2001 beach nourishment project.

Therefore, it is assumed that for the entire project life (i.e., 50 years), the study area will be represented by the depleted beach conditions observed prior to the SANDAG replenishment. Only a thin lens of sand topping the natural bedrock planform exists during the summer and fall months. In some shoreline segments, sand is nonexistent even in the summer. In the winter and spring seasons, a depleted beach condition, exposing the natural bedrock, occurs and thus is the basis for the Monte Carlo simulation to statistically characterize the episodic bluff failures. Although no underlying bedrock formation exists at Cardiff, a similar beach-depleted condition also applies to this low-lying shoreline segment for the wave overtopping analysis.

Consideration of two sea level rise scenarios under the depleted beach conditions in the future, was also included in the bluff failure and wave overtopping (Reach 7 only) analyses. The two SLR scenarios that were considered are the historic upward trend of sea level and the projected sea level rise of the NRC-III curve, as respectively illustrated in **Figure 5.2-1**.

5.2.3 Monte Carlo Simulation for Bluff Failure

In the past, engineers have resorted to use the existing deterministic synoptic summaries for characterizing uncertain future behaviors. However this methodology cannot provide information on probability or in the variability in the time history of bluff failures in the future. This information in necessary for risk-based economic evaluation. In this study, the Monte Carlo technique was, therefore, applied to simulate the random process of storm waves impinging upon the bluff base, inducing toe erosion, and subsequently triggering a bluff failure. The same technique was also used to simulate the magnitude of the upper bluff failure when it occurs.

Bluff toe erosion occurs mostly during severe storm events when waves, impinging upon coastal bluffs, induce mechanical abrasion at the base and force impacts on small joints and fissures in rock units, and hydraulic action on the bluff face. When the bluff toe erosion extends to a certain depth, the upper bluff loses its support at the base and consequently fails. Thus, characterization of a bluff failure requires 1) an understanding of the bluff toe erosion induced by wave attack at the base; and 2) a direct correlation between the threshold value of the toe erosion and the upper bluff failure.

A semi-empirical numerical model was developed by Sunamura (Sunamura, 1982) to quantify the short-term bluff erosion as a function of the rock resistance of a coastal bluff and the wave force acting at the bluff base. The analyzed results from the past field applications indicate that only large waves during a storm event are responsible for inducing bluff erosion. On the other hand, no analytic or empirical approach has been proposed to quantitatively formulate the correlation between toe erosion and bluff top failure (bluff retreat). Thus, a direct and deterministic computation to predict the bluff retreat in the future under the without-project conditions is not feasible.

The Monte Carlo Simulation technique combined with the Sunamura's short-term toe erosion model was, therefore, employed in this analysis to statistically quantify the bluff retreat scenarios for a 50-year project design life throughout the entire study area, except Reach 7. The simulations consisted of two Monte Carlo types of random sampling, based on two formulated statistical distributions: 1) impinging wave height at the bluff base and 2) the sheared-off size of bluff failure on the top, if it occurs. Although wave climate in the future is uncertain and unpredictable as it depends strongly on the meteorological conditions, a statistic representation can be derived, based upon the wave environment observed in the past 20 to 30 years during which a rougher than normal wave climate was recorded. Bluff failures can also be statistically formulated from a detailed, comprehensive, historic database that was assembled since 1990 when episodic bluff failures began to frequently occur.

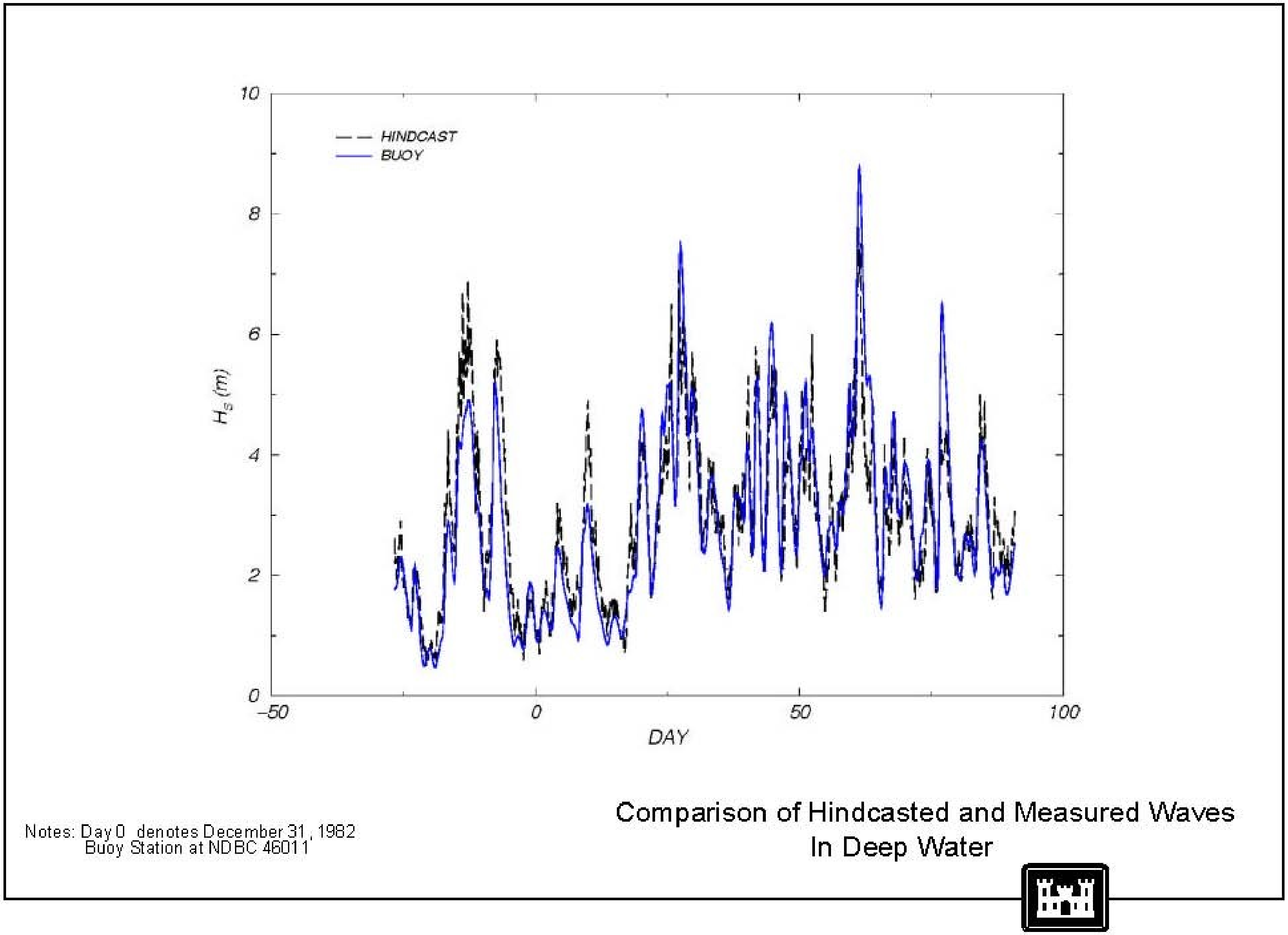
In the following sections, two deterministic sub-model systems, namely wave generation and propagation model, and Sunamura's short-term toe erosion model are briefly addressed. Subsequently, the randomness that was generated from this statistic model (Monte Carlo Simulation) is discussed, followed by the implementation of the entire model system, as well as the modeled results.

Wave Characteristic at Bluff Toe

Day-to-day wave characteristics at the bluff toe for all reaches, except Reach 7, were obtained from 1) defining deep water waves via a hindcast wave model; 2) propagating generated waves to the nearshore water region via a back-refraction model; and 3) continuing the wave propagation until waves arrive at the bluff base in three different forms (non-breaking, breaking or broken).

A full-spectral wind-wave generation model was applied to define the deepwater wave climate. The model is commonly used by the National Centers for Environmental Prediction (NCEP) of the National Oceanic and Atmospheric Administration (NOAA). The hindcast spatial domain covers 66°S to 61.5°N and 100°E to 68°W with a resolution of 1.5° latitude by 2.0° longitude. Twenty frequency bins were used (covering a range in period from approximately 4 to 27 seconds) with 72 directional bins, giving a directional resolution of 5°. Surface winds from the reanalyzed NCEP dataset (Kalnay et al, 1996) were used to drive the model over the period from January 1, 1979 to June 30, 2001. **Figure 5.2-2** shows the comparison of the synthetic waves and the measured data at a NOAA buoy station (NDBC 46011), located 21 nautical miles offshore of Point Conception, for the period from December 1982 to March 1983 during the 1982-1983 El Nino year. The results illustrate a relatively good agreement between the hindcasted and recorded wave data.

The O'Reilly spectral back-refraction model (O'Reilly and Guza, 1991), a well-applied model in southern California coastal zone, was used to perform a linear spectral refraction transformation from deep water to the shallow water region. The wave energy and direction were transformed by back-refracting rays from a target site to the offshore deepwater locations. Each frequency bin is treated separately, with wave rays transmitted from the target site at different initial directions. Wave rays that eventually reach the boundaries of the domain (deep water location) represent solutions that can potentially contribute to the wave field at the target site. Wave energy, frequency, and initial and final directions along the ray line are recorded. Wave rays reaching only to offshore islands are assumed to represent the frequency/direction pairs that cannot contribute energy to the target site. **Figure 5.2-3** illustrates a deduced correlation coefficient of 0.86 between the transformed and measured waves at the CDIP Oceanside gage from December 1997 to March 1998 during the 1997-1998 El Nino season. A correlation coefficient of 0.80 or the high correlation between the two data sets. In addition, **Figure 5.2-4** shows the cumulative occurrence of hindcasted (at the Stone Steps nearshore location) and measured (at Oceanside Buoy) waves from 1979 to 1994 for the months between December and May (winter and spring seasons). The Oceanside wave gage location (CDIP, NO. 004) is at a depth of 34 feet, while the hindcasted location at Stone Steps in Encinitas is at a depth of approximately 30 feet. The discrepancy of the cumulative probability distribution is probably attributed to the variation of bathymetry at the two sites. Nevertheless, the comparisons of the statistic distribution, time series, and correlation of the hindcasted and measured waves are indicative of the validity and applicability of the combined wave hindcast and propagation model.



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2 **Figure 5.2-2 Comparison of Hindcasted and Measured Waves in Deep Water**

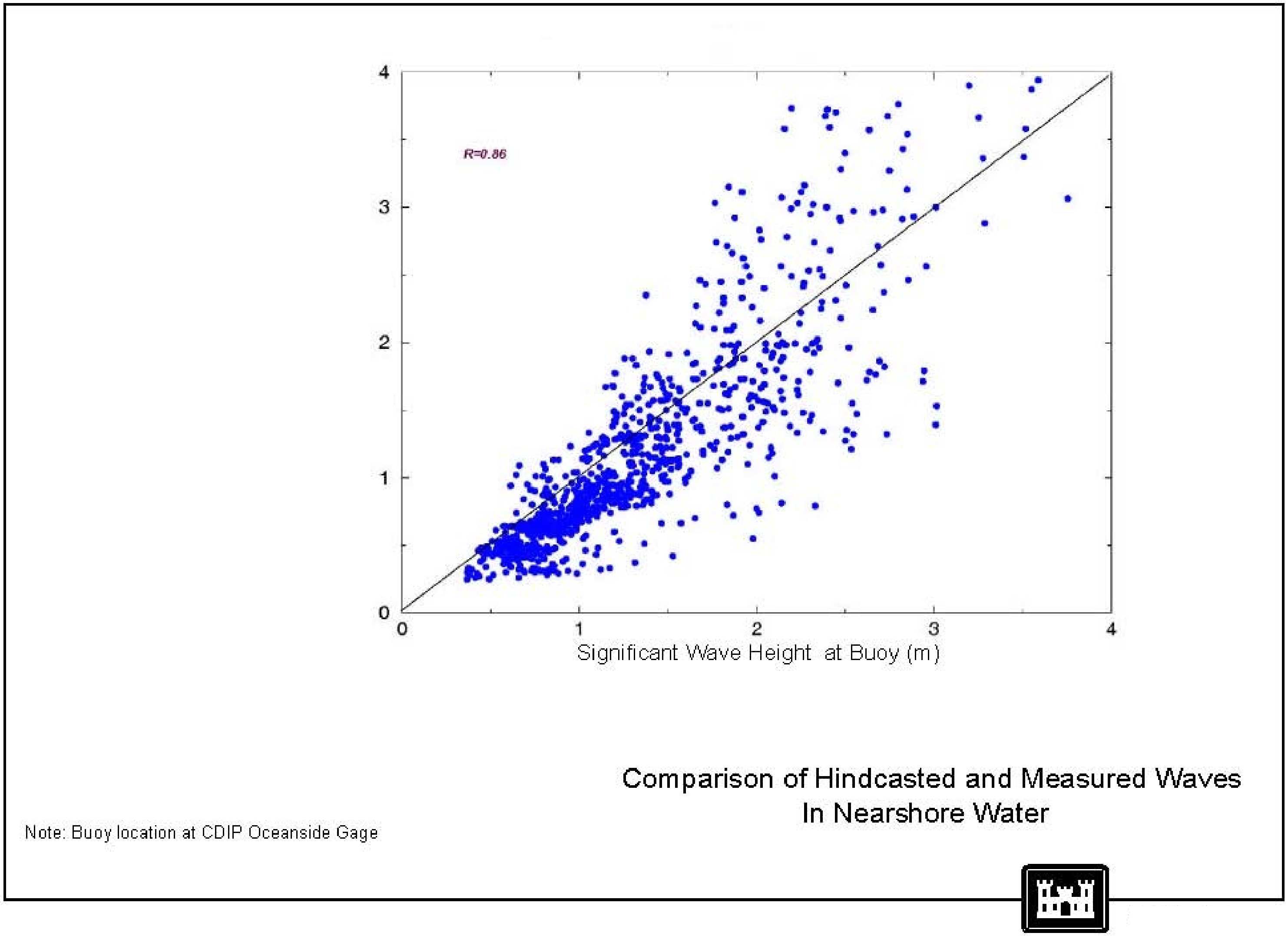


Figure 5.2-3 Comparison of Hindcasted and Measured Waves in Nearshore Waters

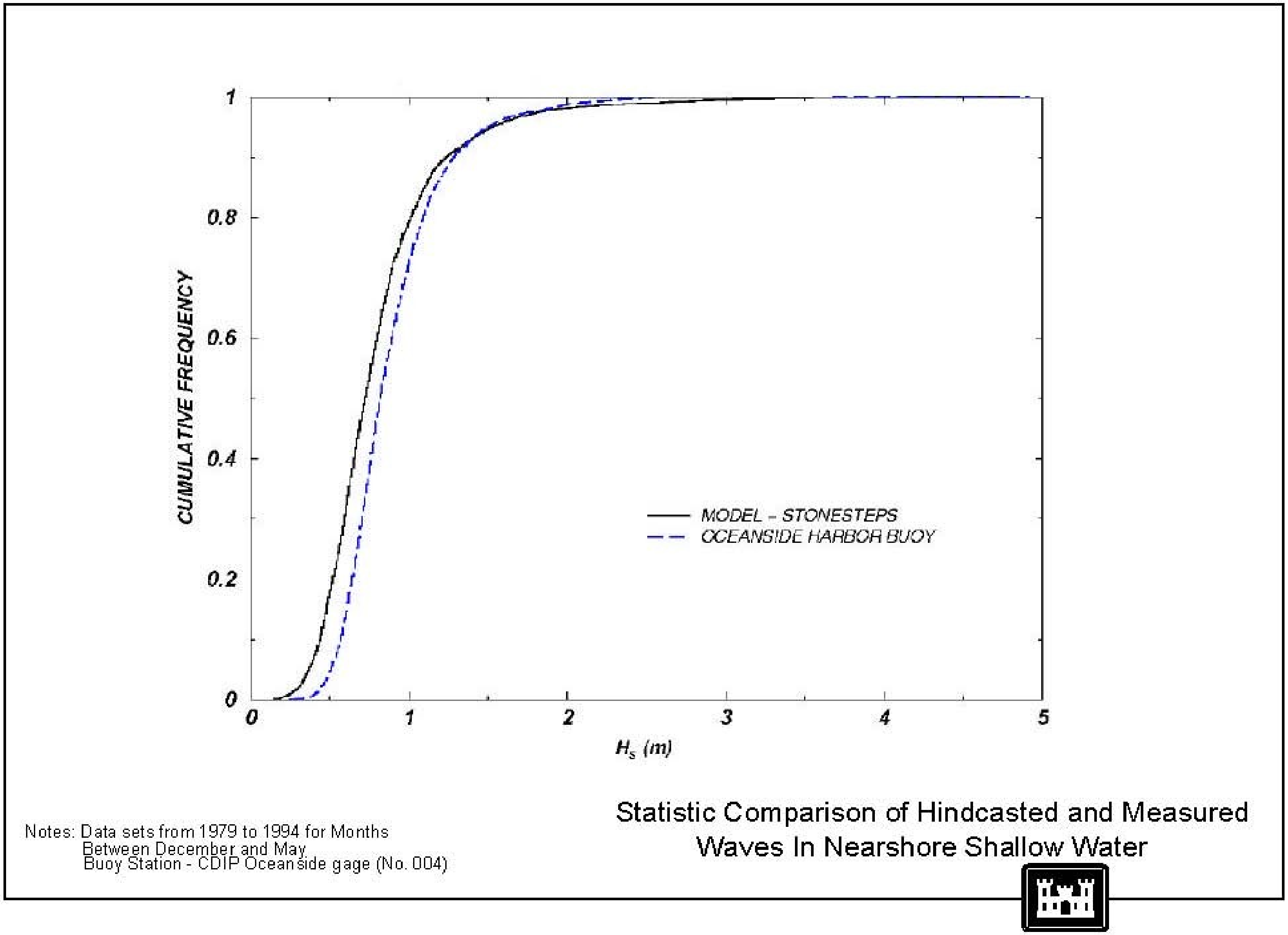


Figure 5.2-4 Statistic Comparison of Hindcasted and Measured Waves in Nearshore Shallow Water

In this analysis, the hindcasted deepwater wave spectra, including both energy and direction, were transformed to the nearshore water region by 1) discretizing the deepwater spectra into a one-second period increment and a one-degree directional segment, respectively; 2) computing the transformed energy at the shallow water target point for each component; 3) assembling the transformed wave components for all included frequencies and directions; and 4) estimating the wave height, wave period and approach direction from the transformed spectra. In each of the eight reaches considered for the bluff erosion study, except Reach 7, transformation functions were developed for a set of 20 shallow water target points (a "line") extending seaward from the shoreline at depths ranging from 3 to 66 feet. Using the maximum energy period from the shallow water spectrum, breaker heights were also calculated using the empirical formula developed by Kaminsky and Kraus (1993). The deduced nearshore wave characteristics were further transformed to the bluff base in accordance with three possible wave conditions at the base as presented in the following:

1) Reformed waves after they were broken - If the water depth at the bluff base was shallower than the computed breaker depth, it was considered to be a broken wave condition. A simplistic breaker decay model (Dally, et. al, 1984) was employed to calculate the reformed wave height as a function of the breaker height, water depth and beach slope. The inshore platform slope and the elevation at the bluff base, employed for the wave computations in the modeled reaches, are presented in **Table 5.2-2.**

2) Breaking waves - If the depth at the bluff base was equal to the breaking depth, the computed breaking wave height was used.

3) Non-breaking waves - If the depth at the base was greater than the breaking depth, the computed shallow water wave height was used and was then propagated to the bluff base via the shoaling process.

Table 5.2-2 Inshore Bathymetry

Reach	Inshore Platform Slope	Bluff base Elevation (ft, MLLW)
1	0.019	3.7
2	0.020	2.7
3	0.020	1.7
4	0.020	1.7
5	0.020	1.7
6	0.016	2.7
8	0.016	1.7
9	0.016	1.7

Wave hindcasts in a 3-hour interval, extending from January 1979 to June 2001, were performed in this analysis. The historically recorded tidal levels were selected to temporally synchronize with the wave-hindcasted calendar dates and times so as to account for the random nature of combining tides and waves. In addition, adjustments to the water levels were considered to include the effects of surfbeat and wave setup (USACE, 2002) that were induced by wave breaking and uprush over the inshore zone. For each analyzed reach, one data set consisting of 65,736 hindcasted wave heights at the bluff base over the 22-year period was deduced. Wave conditions at the bluff base under the two projected SLR scenarios (i.e., the historic trend and NRC-III curve) were characterized by raising the synchronized historic tides with the projected sea level rises in individually analyzed project years (i.e., from 2015 and 2065) and following the same wave transformation process to propagate hindcasted waves to the bluff base.

Representativeness of Hindcasted Wave Climate

Since the 1979 to 2001 hindcasted wave set was used to develop the Monte-Carlo statistics and as input for the numerical shoreline modeling (**Section 7)**, it is worthwhile to attempt to understand what this data represents in a historical and future context. Within the climate modeling community there is presently a high level of confidence in the potential for human induced climate change increasing tropical cyclone wave activity (IPCC, 2007). In addition studies have concluded that North Pacific winter storm wave heights, and storm frequencies have been increasing over the last fifty years and are trending upward (Graham et. Al., 2002; Inman et. Al., 2006, Graham, 2005). They have also found that the approach direction of these winter swells impacting southern California have trended from more northwesterly to more westerly over time. As part of their analyses these studies have shown how these waves were larger over the 1980's and 1990's (during the latest Pacific Decadal Oscillation warm phase) than they were from 1940's through the 1970's (the latest Pacific Decadal Oscillation cool phase). This recent history of the North Pacific winters is clear. Whether it is part of a longer-term upward trend or just part of an ongoing cycle is still being debated.

Most of the studies that predict a trend of increasing North Pacific wave activity are limited to data records that only extend back to the 1940's. Studies that use North Pacific Ocean data extending back to the previous century have more mixed conclusions. Bromirski et. al., (2002) showed that the higher than normal North Pacific wave activity of the 1980's and 1990's are part of a longer-term cyclical pattern and the heightened wave activity of those recent decades are shown to be "very active, but not extraordinarily so compared to the pre-1948 epochs." Also, Chang and Fu (2003) suggest that global storm track activity during the last part of the 20th century may not be more intense than the activity prior to the 1950s. In contrast, Seymour (2011) found a long term trend of decreasing north pacific index dating back to 1900. This index is inversely correlated with wave activity; hence a long-term increase of wave activity was concluded.

In addition to reviews of historical wave climates, models of future wave activity are available. One such model by the California Climate Change Center (2009a) predicted reduced future wave activity in California and concluded that "the positive trends in eastern North Pacific winter wave heights noted over the latter half of the twentieth-century are very likely due to natural climate variability rather than anthropogenic warming."

The two different conclusions based on North Pacific waves tell two different possible stories about how representative the last two decades of North Pacific wave activity were. If North Pacific wave activity is trending upward, then the last two decades were higher than previous and are lower than the expected future wave climates. If North Pacific wave activity is experiencing no long-term trend or decreasing, then the heightened wave activity during the last two decades should subside for the next decade or so.
The types of studies that are available for the North Pacific are less common for the tropical Pacific and South Pacific Ocean regions. This is likely due to a lack of long-term data and due

to the relative importance these regions have on the North American coastline, where much research is done. With the paucity of knowledge about these wave climates, the representativeness of the hindcasted wave set used in this study cannot be known with regards to these components.

Given the difficulties of placing the hindcasted wave set into an accurate historical context, and the difficulties inherent in long term weather predictions, it would be speculative to attempt to extrapolate that data set into any future context. Therefore, it is unclear whether the hindcasted wave data will be representative of future wave conditions. This uncertainty is not unprecedented however. A common assumption for coastal studies is that future weather and wave conditions will be similar to historical conditions used to support the analyses. This assumption applies for the current study as well.

Wave Induced Bluff Toe Erosion Model

The previously mentioned Sunamura model computes the short-term bluff toe erosion induced by the wave force (function of wave height) acting at the base. This simplistic model was applied to predicting bluff toe erosion induced by wave attack for several field cases. The fundamental equation of this model is written as:

$$X = \sum_{i=j}^{N} X_i = \sum_{i=j}^{n} k\left(C + \ln\frac{\rho g H_i}{S_c}\right)\Delta t_i$$

where X is the accumulated bluff toe erosion depth from N waves at bluff toe,

X_i is the individual erosion by the ith wave with height of H_i and duration of Δt_i,

S_c is the compressive strength of the bluff material,

ρ is the density of water,

g is the gravitational acceleration,

C is a non-dimensional constant,

k is a constant with dimension of Length over time [L/T], and

Subscript j is the group number of the critical wave height H_j to initiate the toe erosion, which is given by $H_j = S_c e^{-c} / \rho g$.

The equation implies that the resulting toe erosion is proportional to the magnitude of wave height and is inversely related to the compressive strength (S_c) of bluff material. After replacing constant C with critical wave height H_j, the equation can be rewritten as:

$$X = \sum_{i=j}^{n} k\left(\ln\frac{H_i}{H_j}\right)\Delta t_i$$

It is noted that two unknown constants *k* and H_j (or *C*) should be determined prior to the model application to predict bluff toe erosion and, in practice, at least two sets of field data are required to calibrate *k* and H_j.

The calibration, performed for constants *k* and H_j in Reach 8, was based on the temporally measured notch depths and hindcasted wave heights at the bluff base during the same measurement period. **Table 5.2-3** lists the maximum bluff notch depths and individual periods measured by TerraCosta (2002) between 1997 and 2000.

Table 5.2-3 Measured Maximum Notch Depths at Reach 8

Event period	Maximum measured notch depth (ft)
Nov. 1997 – Jun. 1998	7
Nov. 1998 – Feb. 15, 2000	3
Nov. 1998 – Dec. 15, 2000	4
Nov., 1997 – Feb. 15, 2000	10

It should be noted that a notch configuration has dimensions of height, width and depth. Thus, depending on its dimensional configuration, the average notch depth over a formed toe-eroded segment is most likely narrower than the maximum value measured in the field. The ratios of the average to the maximum notch depth for a rectangular-, elliptic-, parabolic-, and triangular-shape notch were calculated to be 1.0, 0.78, 2/3, and 0.5, respectively. The calibration process utilizing the maximum measured notch depths presented in **Table 5.2-3** would over-predict the extent of toe erosion. Therefore, the constants, *k* and H_j, were calibrated from the average notch depths.

Table 5.2-4 lists the calibrated values of k and H_j for different notch configurations based upon the average notch depth. The calibrated constant k is sensitive to the notch configuration, as compared to no change in H_j. From past field observations, it was determined that a parabolic configuration represents the most realistic shape of the observed notches. **Figure 5.2-5** shows the calibrated results for Reach 8, based on the assumption of a parabolic notch configuration. Hence, k = 1,045 m/year and H_j = 1.08 m were used in the model simulations to predict bluff failure in Reach 8.

Since no measured notch depth data is available for the remaining reaches, it is impossible to directly calibrate k and H_j via the same procedure as described above for Reach 8. The critical wave heights at the bluff base for the remaining reaches are likely to vary from the one calibrated in Reach 8. In lieu of field measurements, the k values for the remaining reaches were estimated in relation to the calibrated k_8 value in Reach 8 (TerraCosta, 2002), based upon the geologic conditions of the bluff formation and its related rock resistance force, as presented in **Table 5.2-5**. The critical wave height was assumed to remain unchanged throughout the entire study area.

Table 5.2-4 Values of calibrated C and Hj for different notch shapes

Notch shape	**Ratio of average to maximum depth**	H_j **(m)**	***k*** **(m/year)**
Rectangle	1	1.08	1,560
Ellipse	0.78	1.08	1,215
Parabola	0.67	1.08	1,045
Triangle	0.5	1.08	780

Table 5.2-5 Ratio of k Value to k8 for Remaining Reaches

Reach	**1**	**2**	**3**	**4**	**5**	**6**	**8**	**9**
k / k_8	0.1	0.5	0.75	0.625	0.5	0.5	1.0	1.0

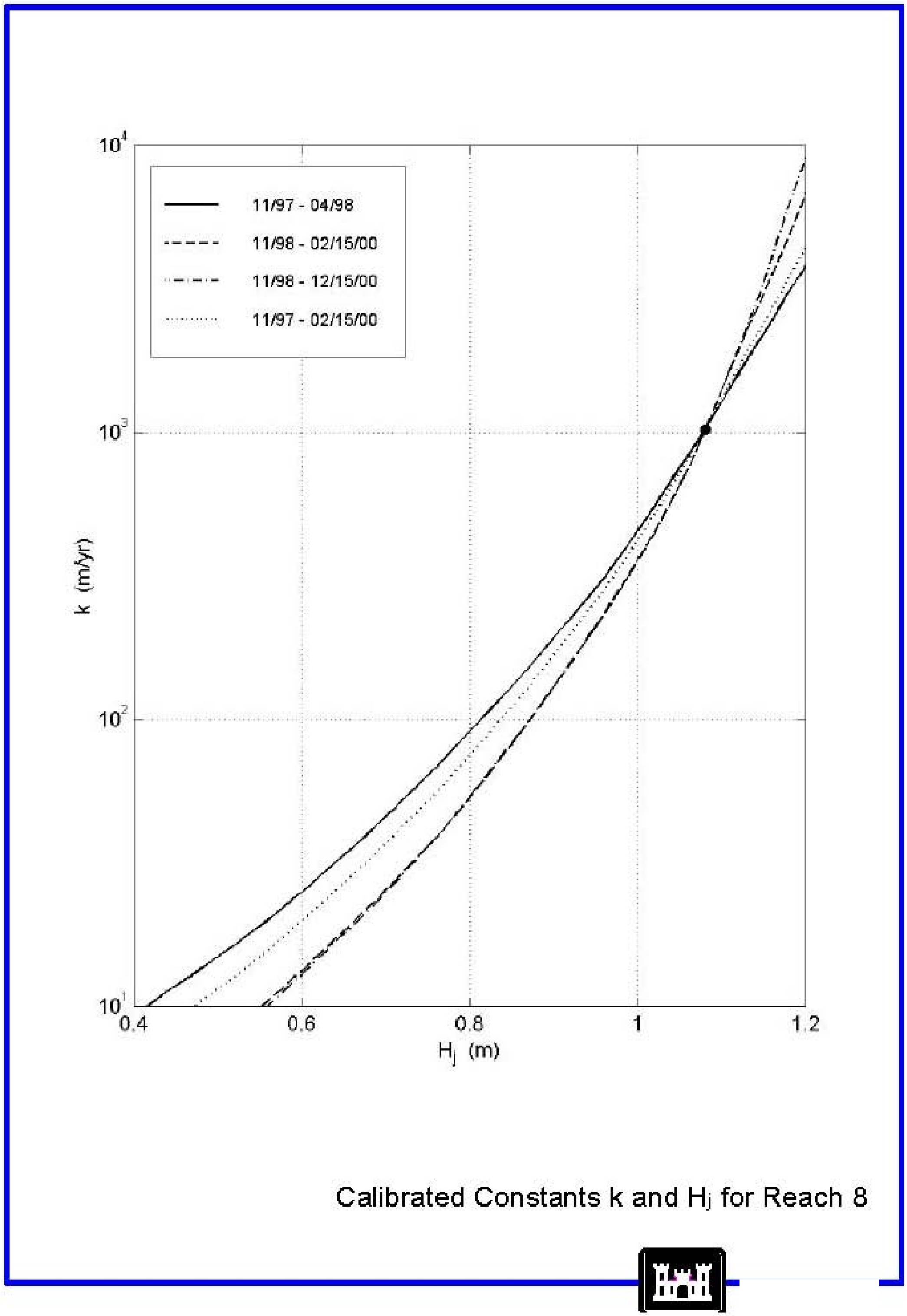


Figure 5.2-5 Calibrated Constants k and Hj for Reach 8

Randomness of Impinging Waves and Bluff Failure

As stated previously, two types of random populations, namely wave height and bluff failure, are required for this Monte Carlo Simulation. The frequency occurrence of wave height at the bluff base for each reach was developed based on the time history of hindcasted wave heights extending from 1979 to 2001. The wave height at the bluff base depends significantly on not only the deepwater wave climate but also the water level. Peak storm waves lasting for 12 to 24 hours arriving at the bluff base can be small in magnitude if the arrival coincides with a low water level. On the other hand, approaching waves at the base can be fairly sizeable under a moderate wave condition if they arrive during high tides.

To ensure the combined randomness of waves and tides, individual 3-hour significant wave heights at the bluff base were computed via the propagation of deepwater waves coinciding with the water level measured at the precise wave-hindcasted time for the entire 22-year period. Under the two previously-identified SLR scenarios, the corresponding SLR values were added to the synchronized historic water levels in individual project years for deducing wave heights at the bluff base. Thus, each computed wave height at the bluff base takes into account the variation of the still water elevation that includes the astronomic tide level, wave-induced setup and sea level rise. The calculated wave heights were then categorized (totally about 65,736 data points for each reach) in accordance with four meteorological seasons: a four-month winter season (December, January to March), a two-month spring season (April and May), and two 3-month seasons for summer (June to August) and fall (September to November).

Past field investigations indicate that bluff toe erosion mainly occurs in the winter and spring seasons when the beach conditions are most depleted. Even with the assumed future depleted beach conditions, a thin sand lens that provides a buffer to prevent the bluff toe from wave exposure may exist during the summer and fall months, particularly in the City of Encinitas. Furthermore, long swells occurring during these two seasons (June to November) are generally benign. As a consequence, little bluff toe erosion occurs during the summer and fall months. Therefore, the toe-erosion model only applies to the winter and spring seasons (December to May) when wave energy is high and the sand lens fronting the bluff toe is almost nonexistent. Wave heights at the bluff base in different reaches vary in accordance with the beach slope and bluff base elevation. The higher the bluff base elevation is, the lower the impinging wave heights are. The base elevation at Reach 1 is the highest (**Table 5.2-2**) and thus the impinging wave heights are the smallest as compared to the remaining reaches. The impinging wave heights at Reaches 3, 4, 5, 8 and 9 are generally greater than Reaches 1, 2 and 6.

Eight frequency distributions of wave height occurrence at the bluff base for the analyzed eight reaches were derived from the compilations of the winter and spring data subsets. **Figure 5.2-6 to Figure 5.2-13** illustrate the deduced frequency distributions (occurrence and cumulative frequency) of wave heights at the bluff base in the spring and winter seasons for the eight analyzed reaches without inclusion of sea level rise, while **Figure 5.2-14** shows the frequency distribution of the deepwater wave height. For the two considered sea level rise scenarios, the cumulative frequency distributions of wave height at the bluff base during the spring and winter seasons in individual reaches were similarly deduced. **Figure 5.2-15** through **Figure 5.2-22** present the distribution curves for a series of project years under the SLR scenario of the historic trend, while **Figure 5.2-23** through **Figure 5.2-30** illustrate the derived cumulative distributions in the same project years for the SLR scenario that is based on the high rate of sea level rise (i.e., NRC-III curve).

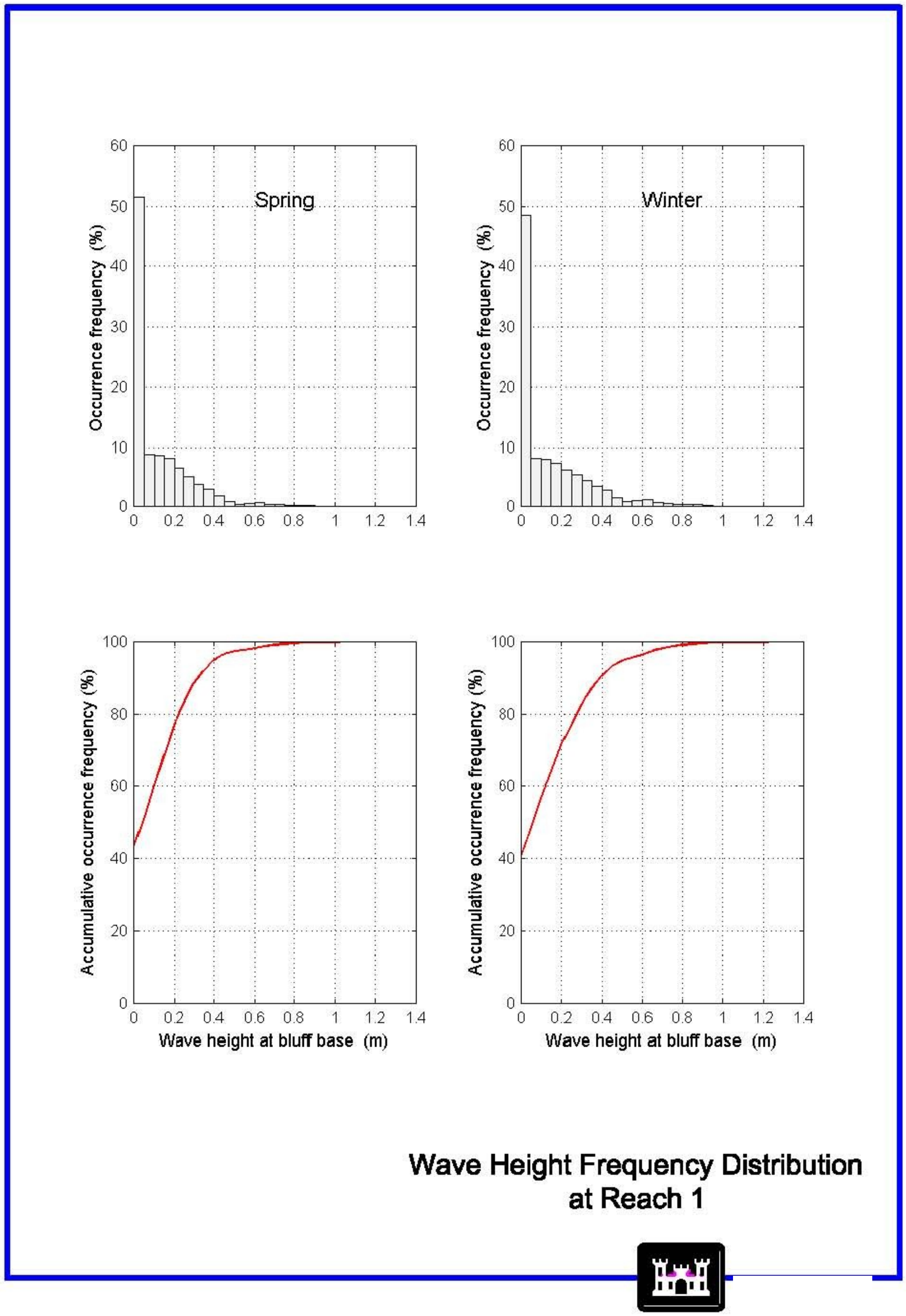


Figure 5.2-6 Wave Height Frequency Distribution at Reach 1

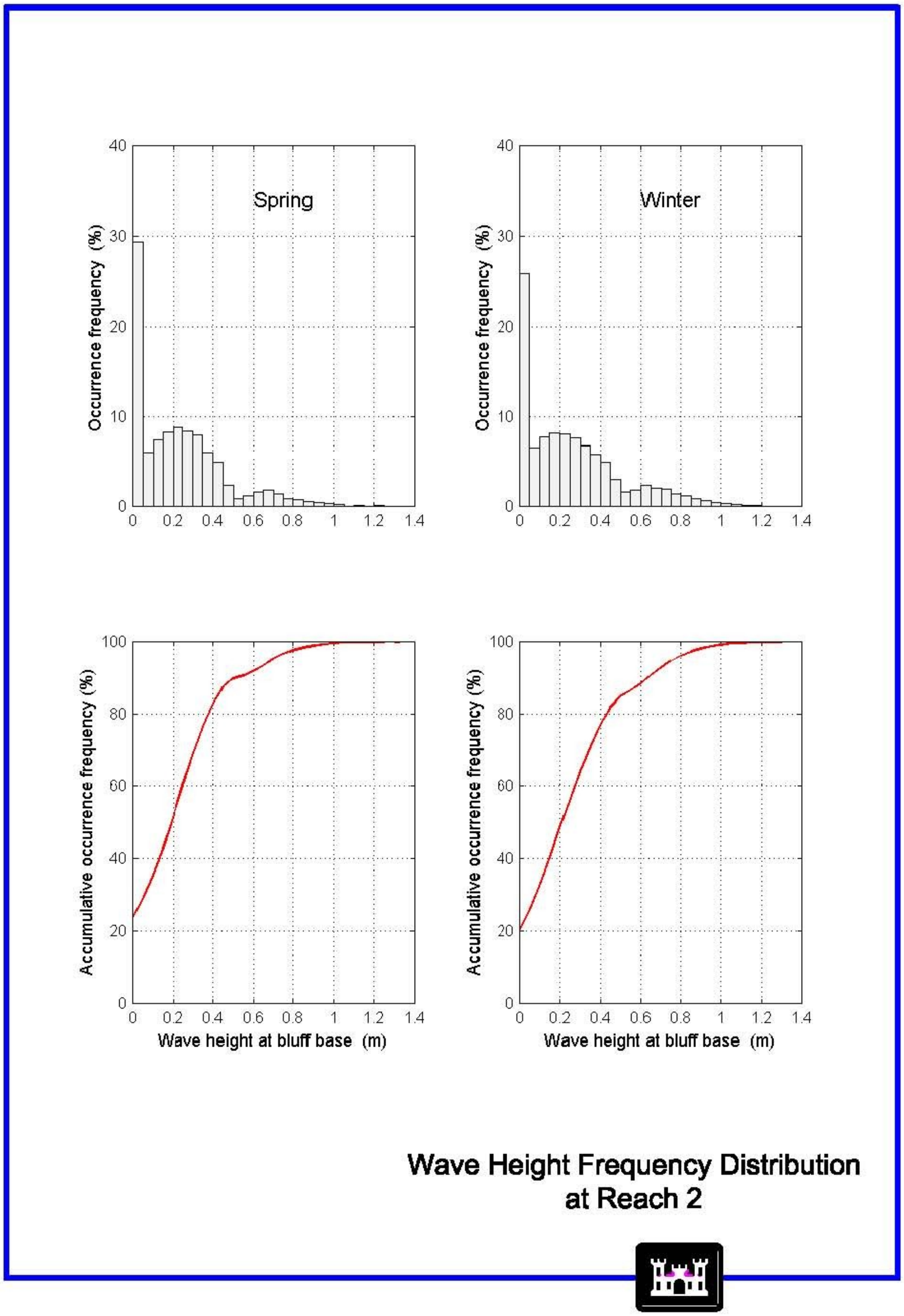


Figure 5.2-7 Wave Height Frequency Distribution at Reach 2

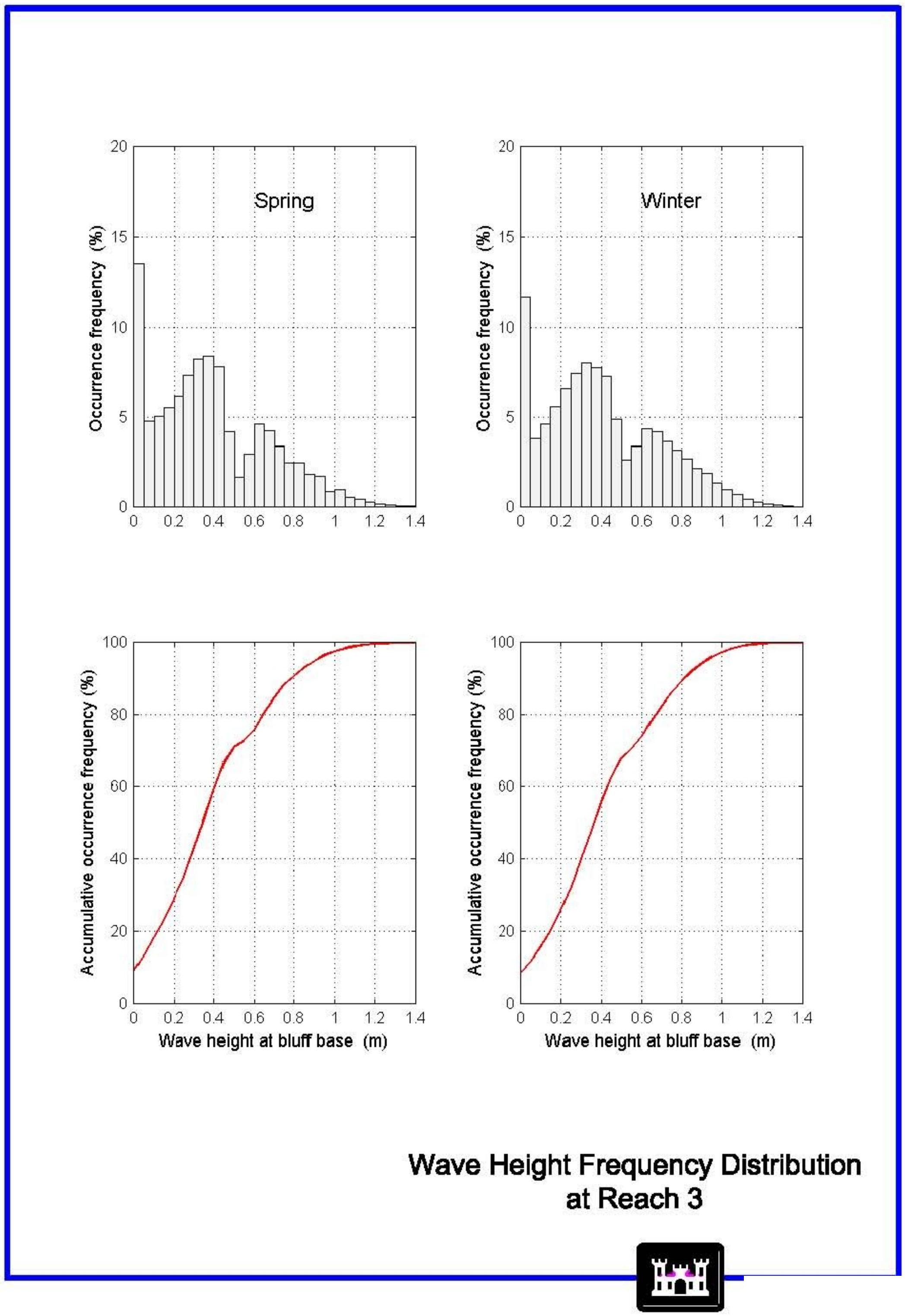
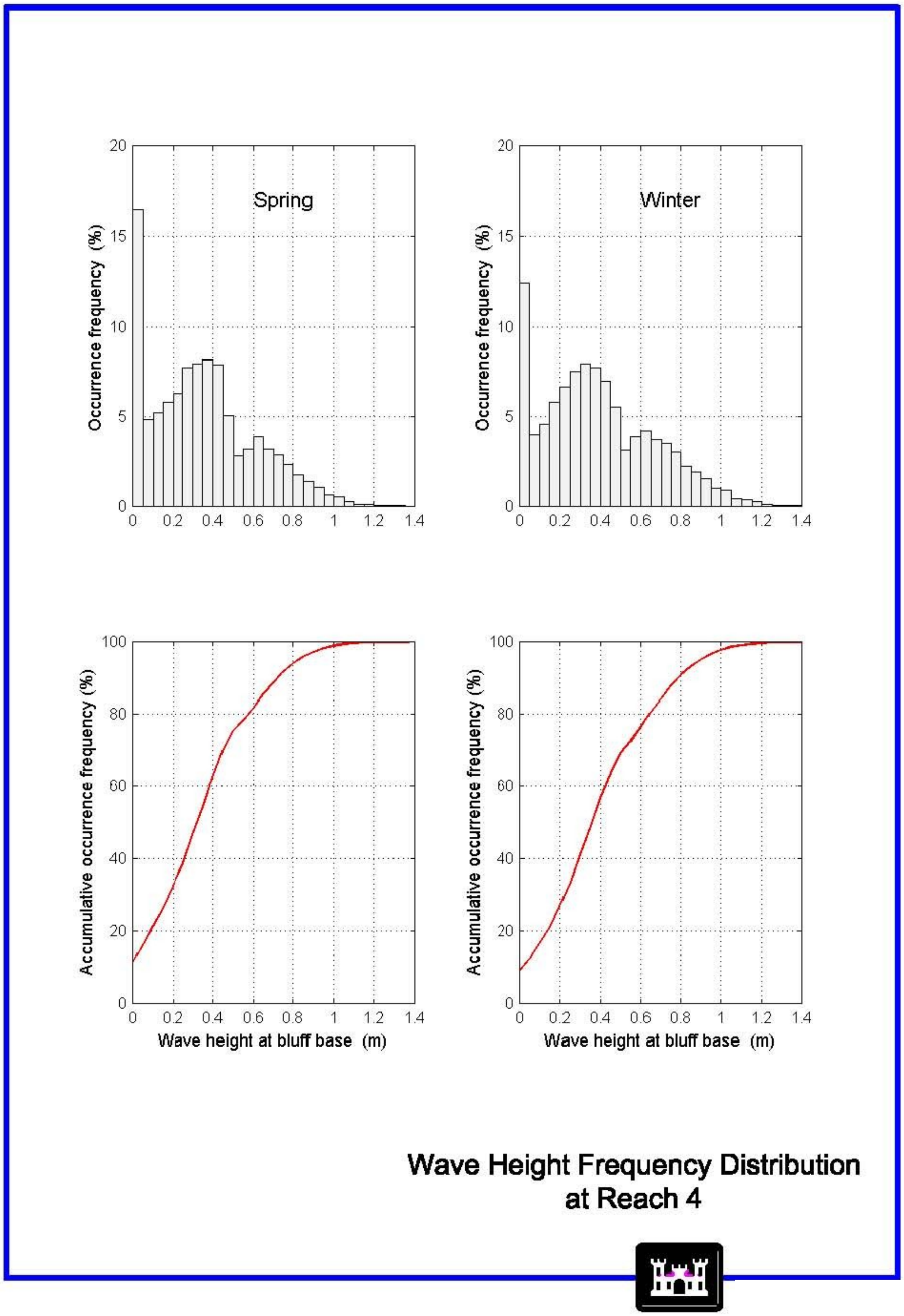


Figure 5.2-8 Wave Height Frequency Distribution at Reach 3



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2 **Figure 5.2-9 Wave Height Frequency Distribution at Reach 4**

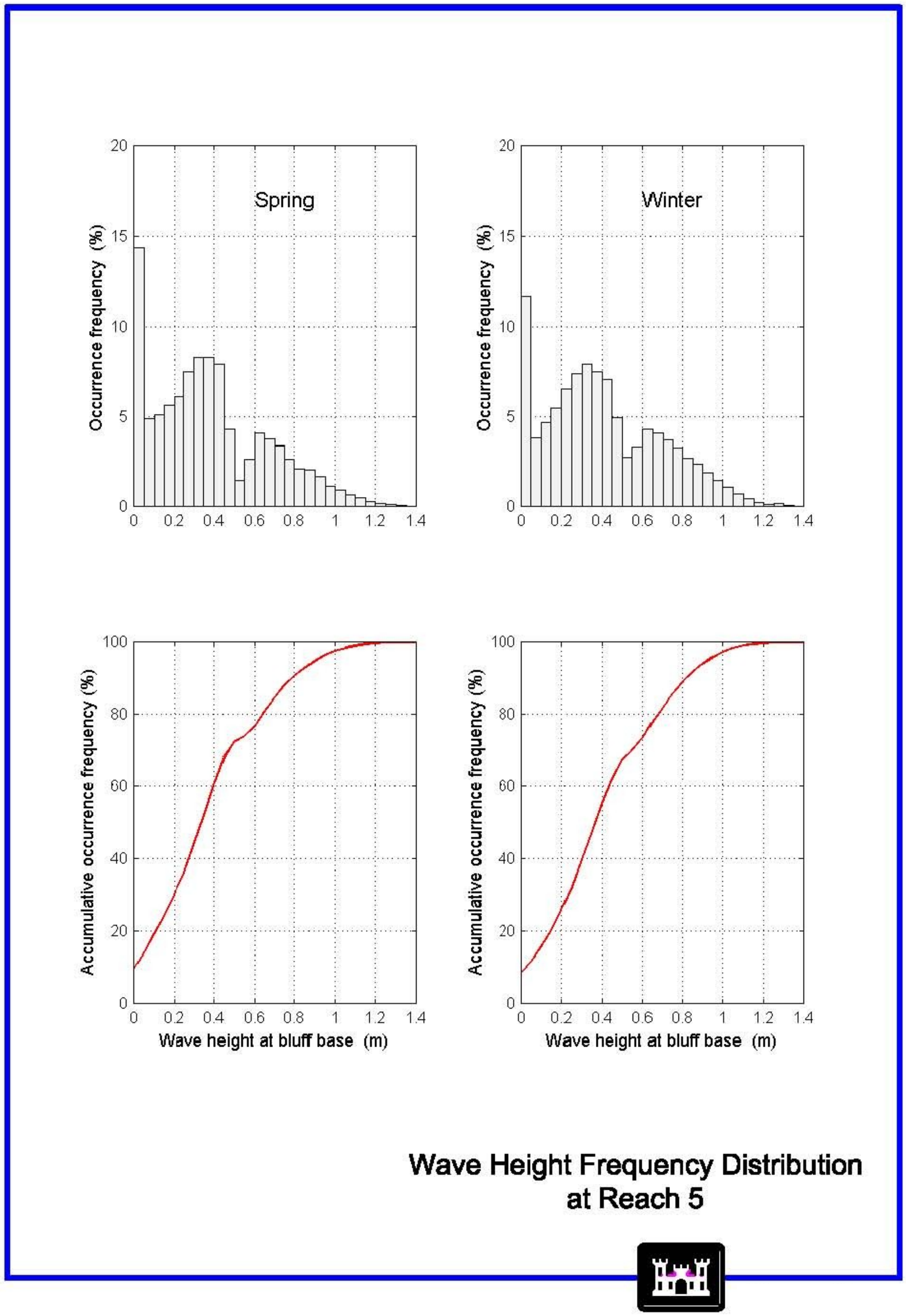


Figure 5.2-10 Wave Height Frequency Distribution at Reach 5

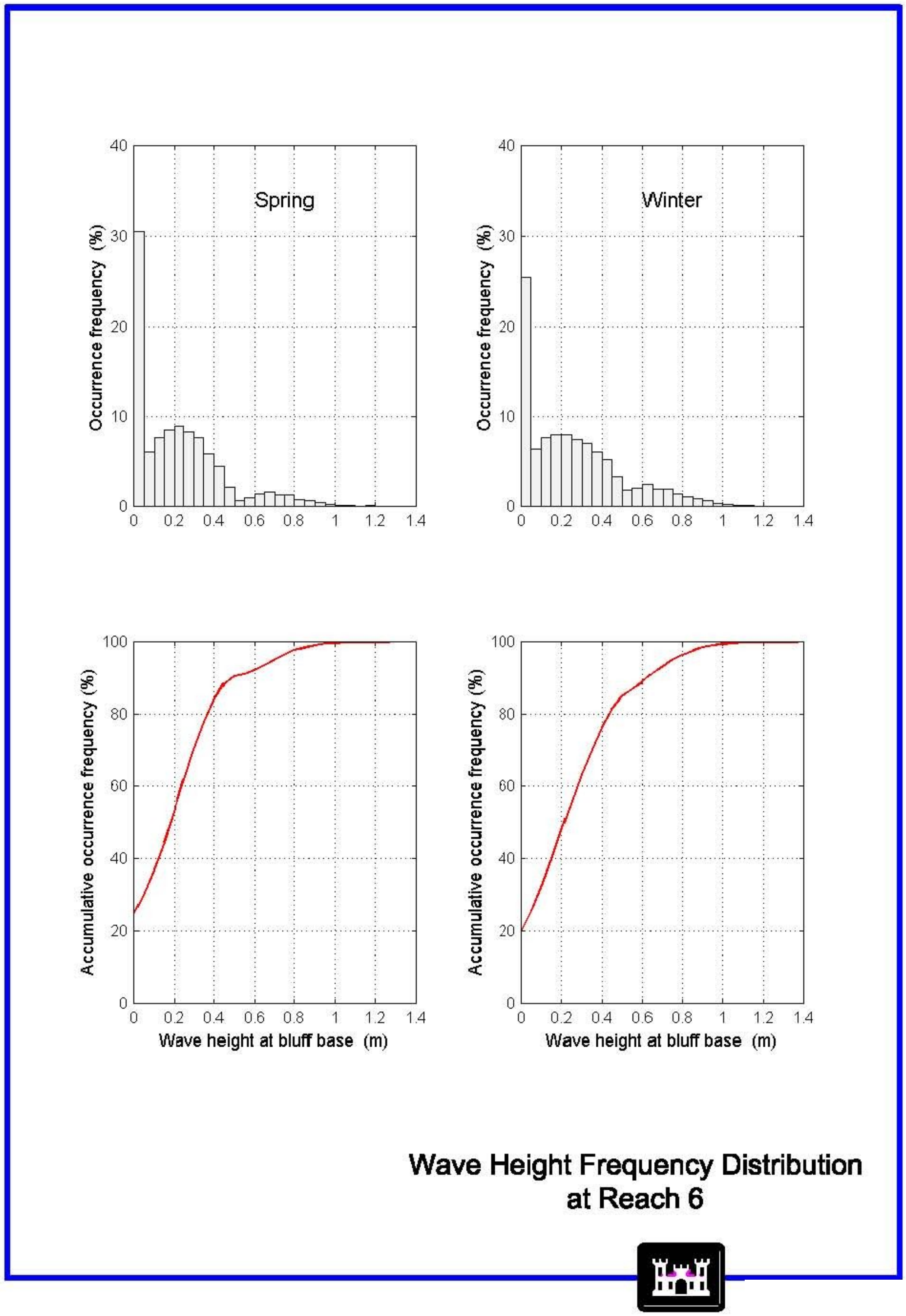
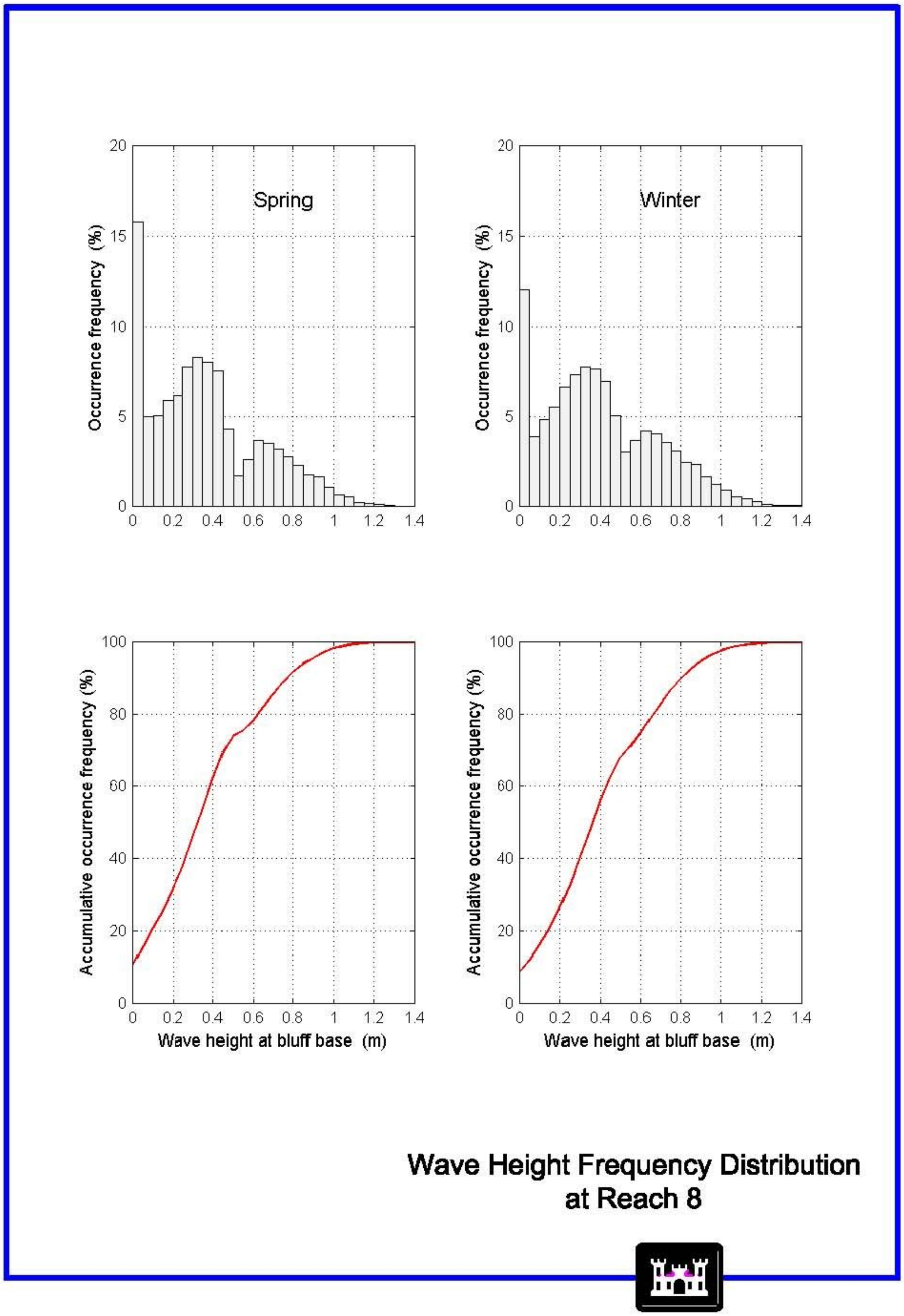


Figure 5.2-11 Wave Height Frequency Distribution at Reach 6



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2 **Figure 5.2-12 Wave Height Frequency Distribution at Reach 8**

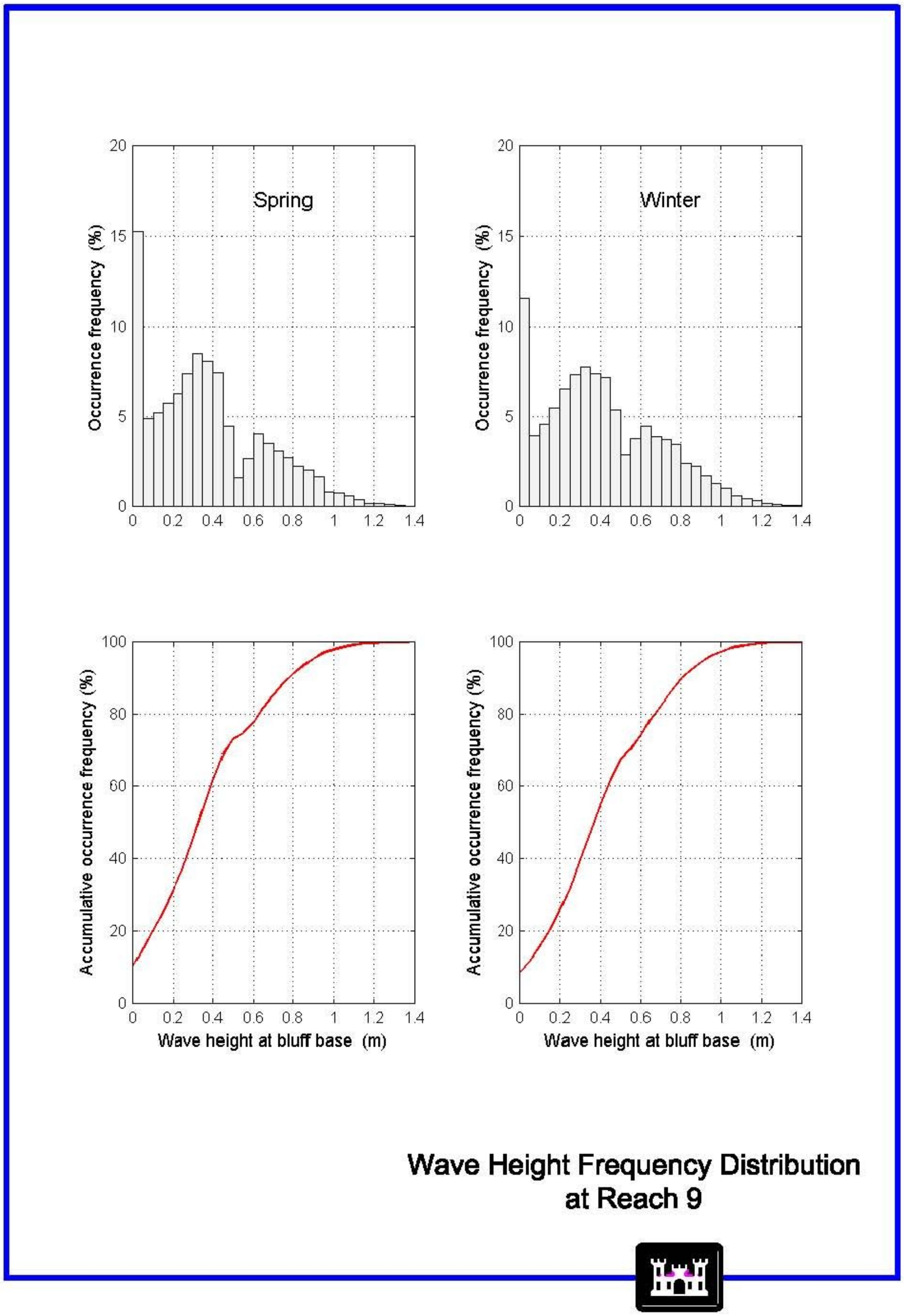


Figure 5.2-13 Wave Height Frequency Distribution at Reach 9

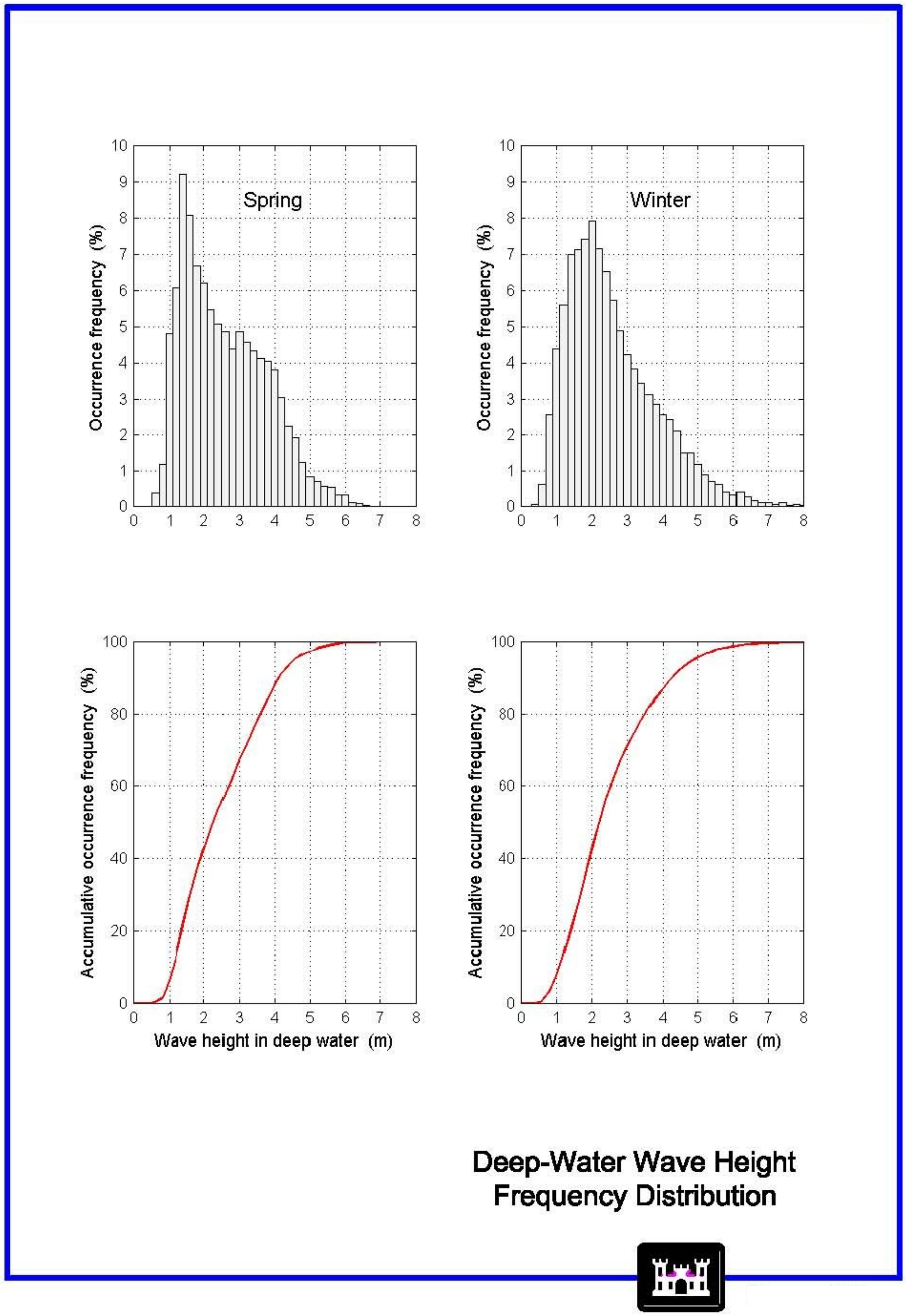


Figure 5.2-14 Deep-Water Wave Height Frequency Distribution

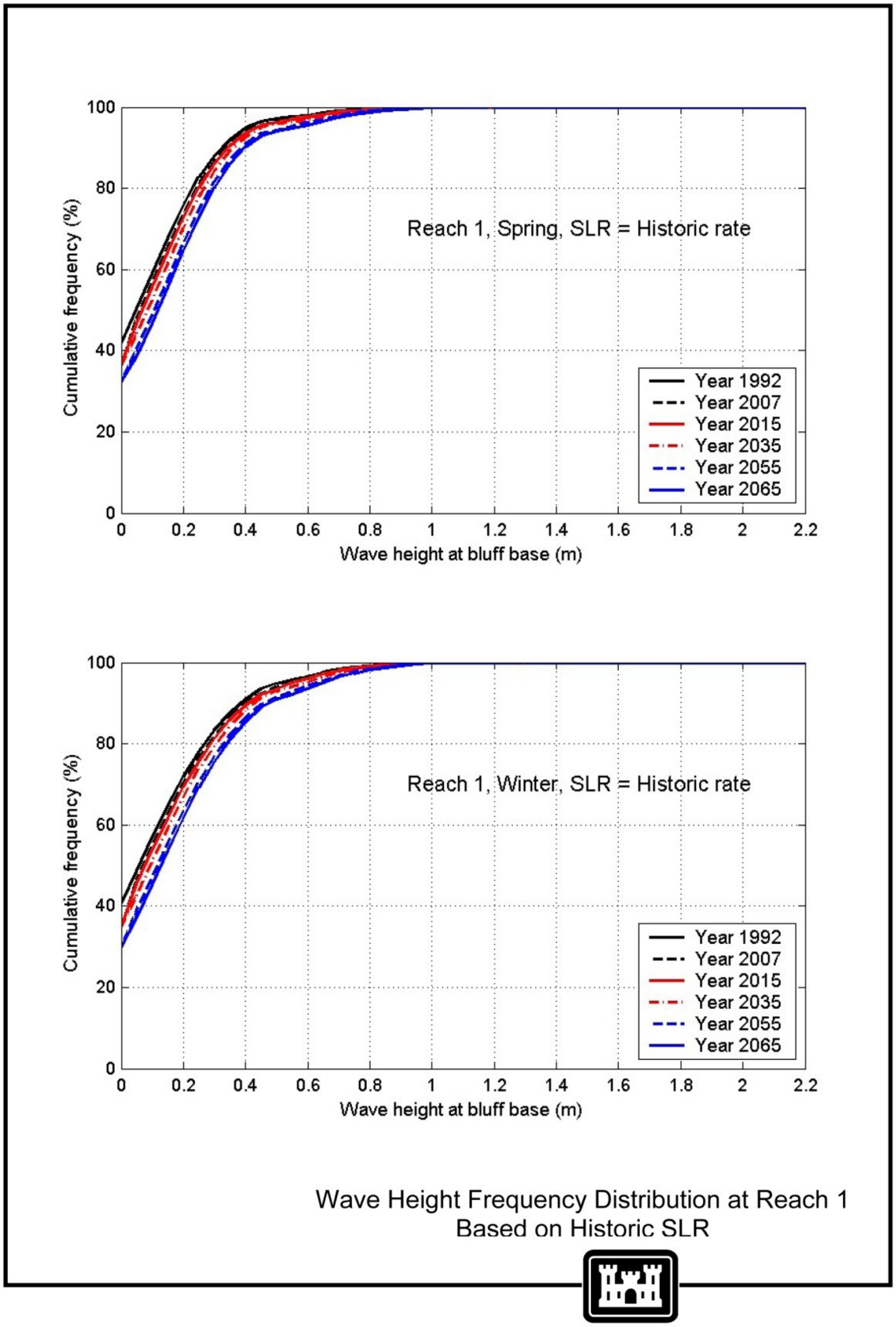


Figure 5.2-15 Wave Height Frequency Distribution at Reach 1 Based on Historic SLR

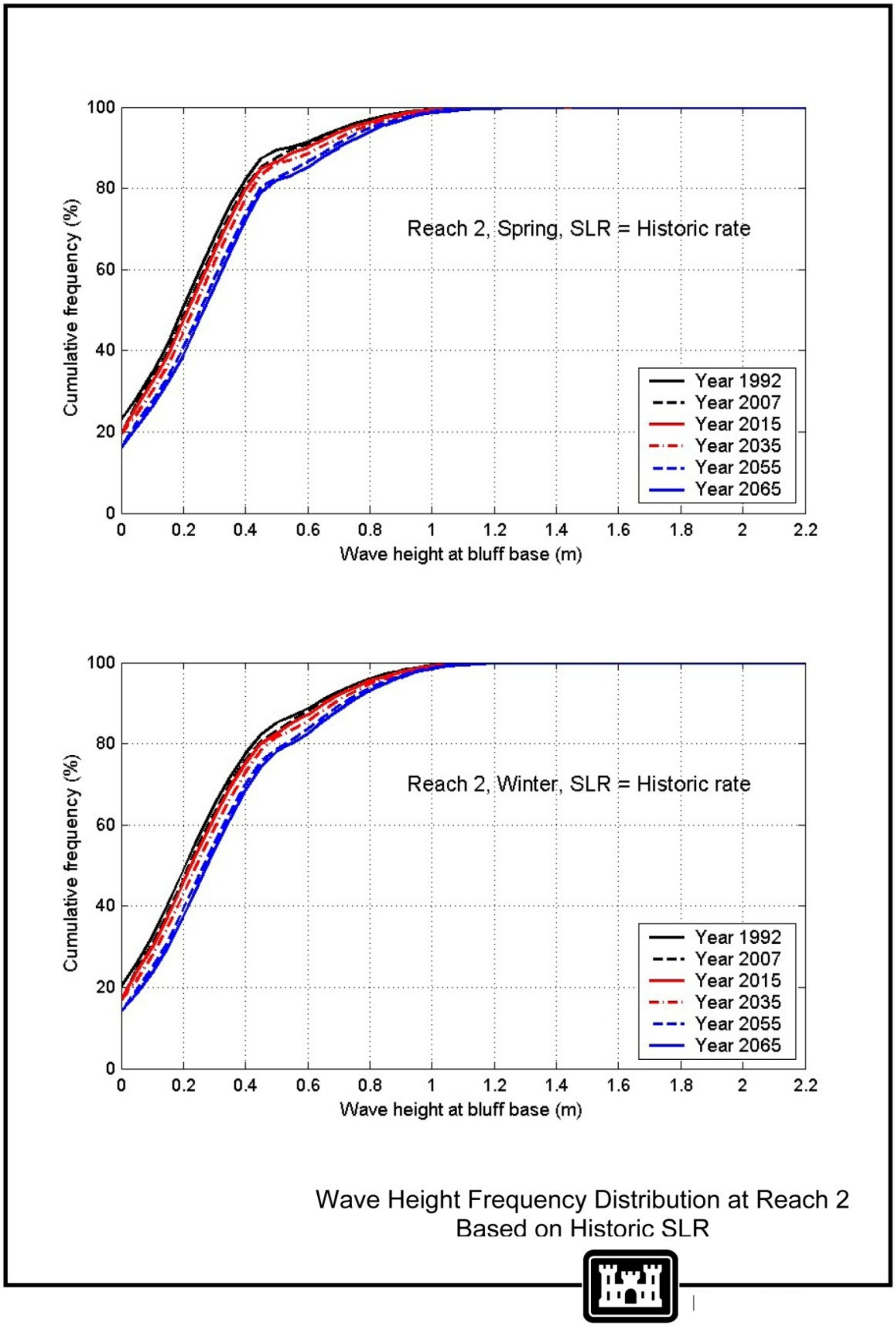
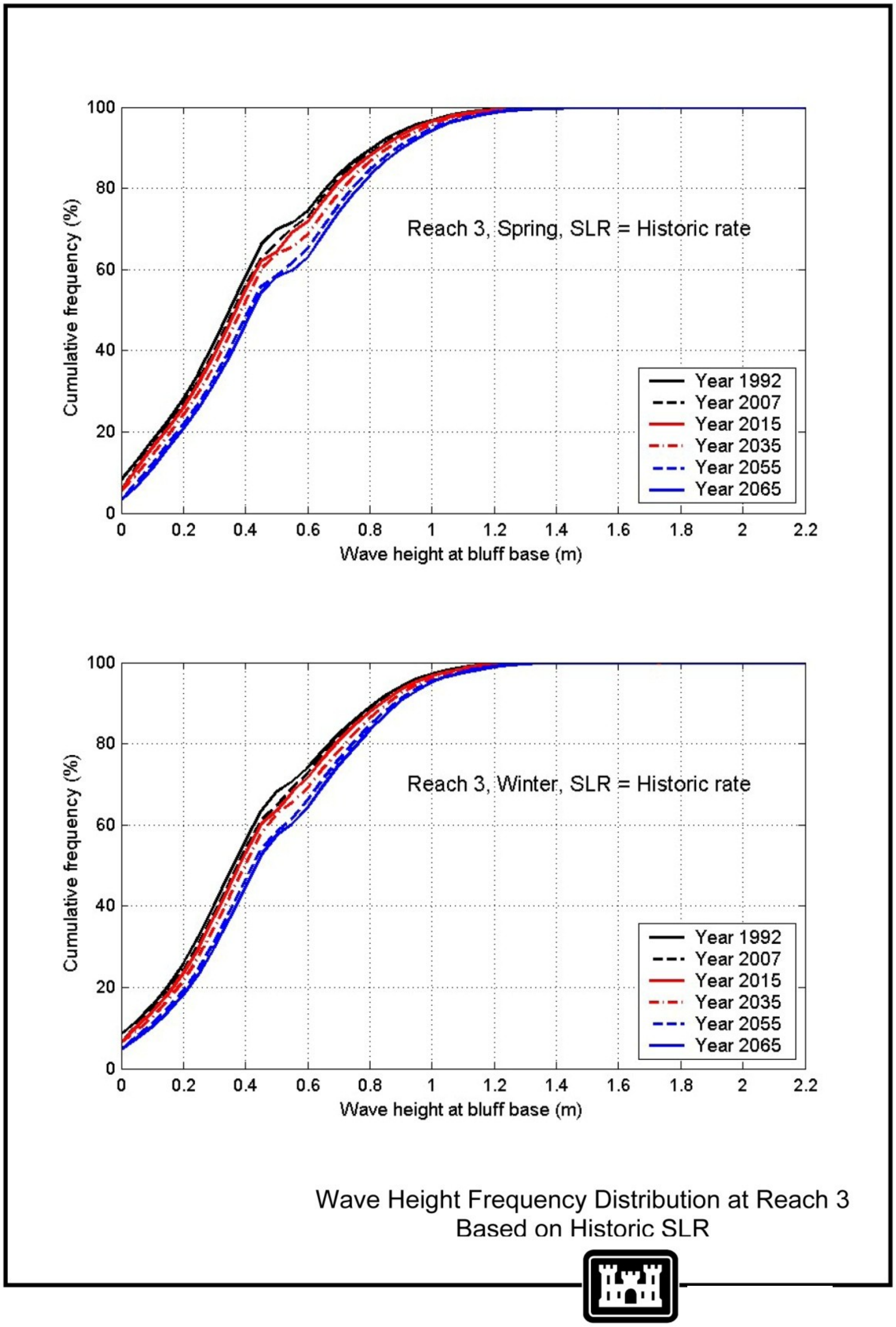


Figure 5.2-16 Wave Height Frequency Distribution at Reach 2 Based on Historic SLR



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2 **Figure 5.2-17 Wave Height Frequency Distribution at Reach 3 Based on Historic SLR**

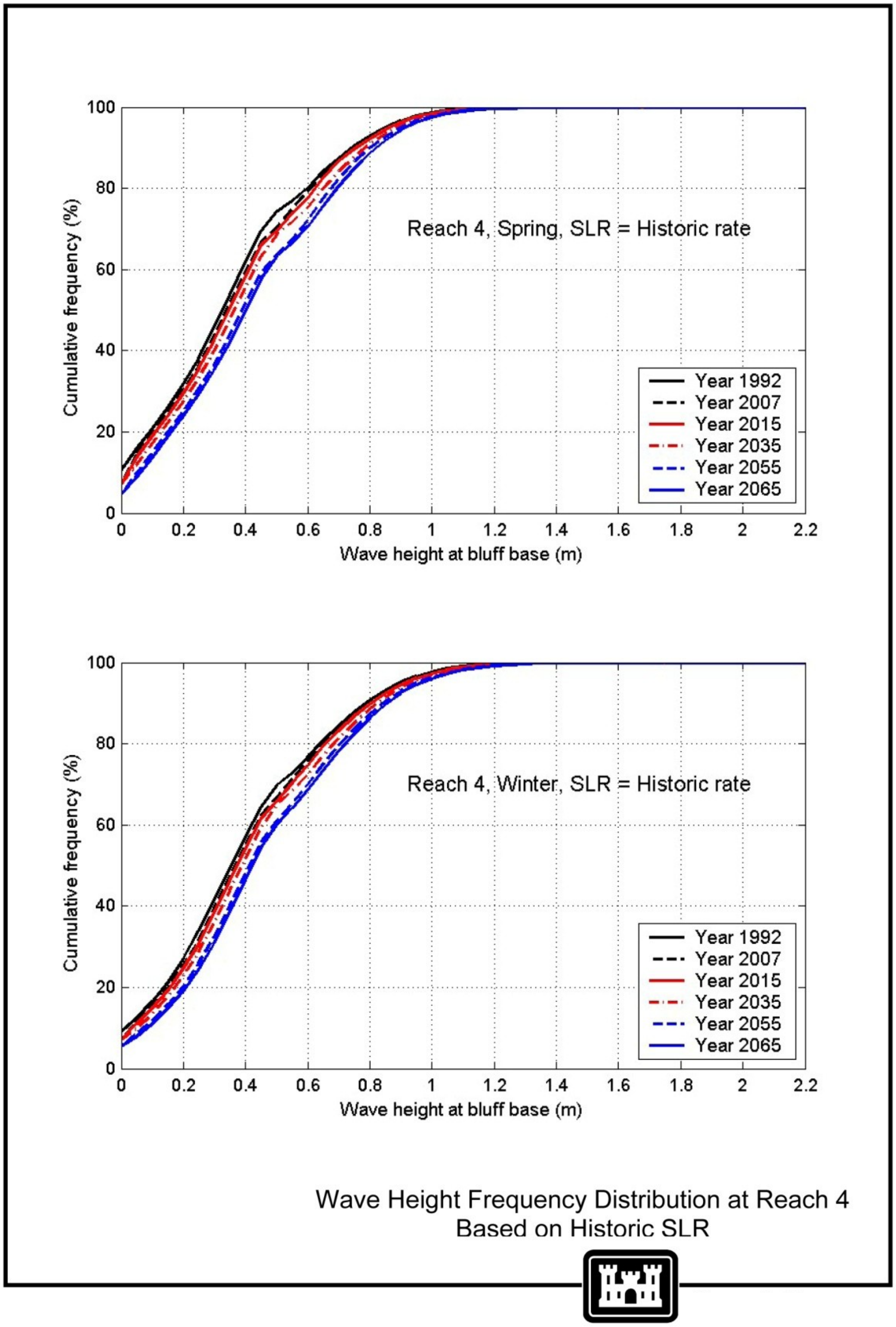


Figure 5.2-18 Wave Height Frequency Distribution at Reach 4 Based on Historic SLR

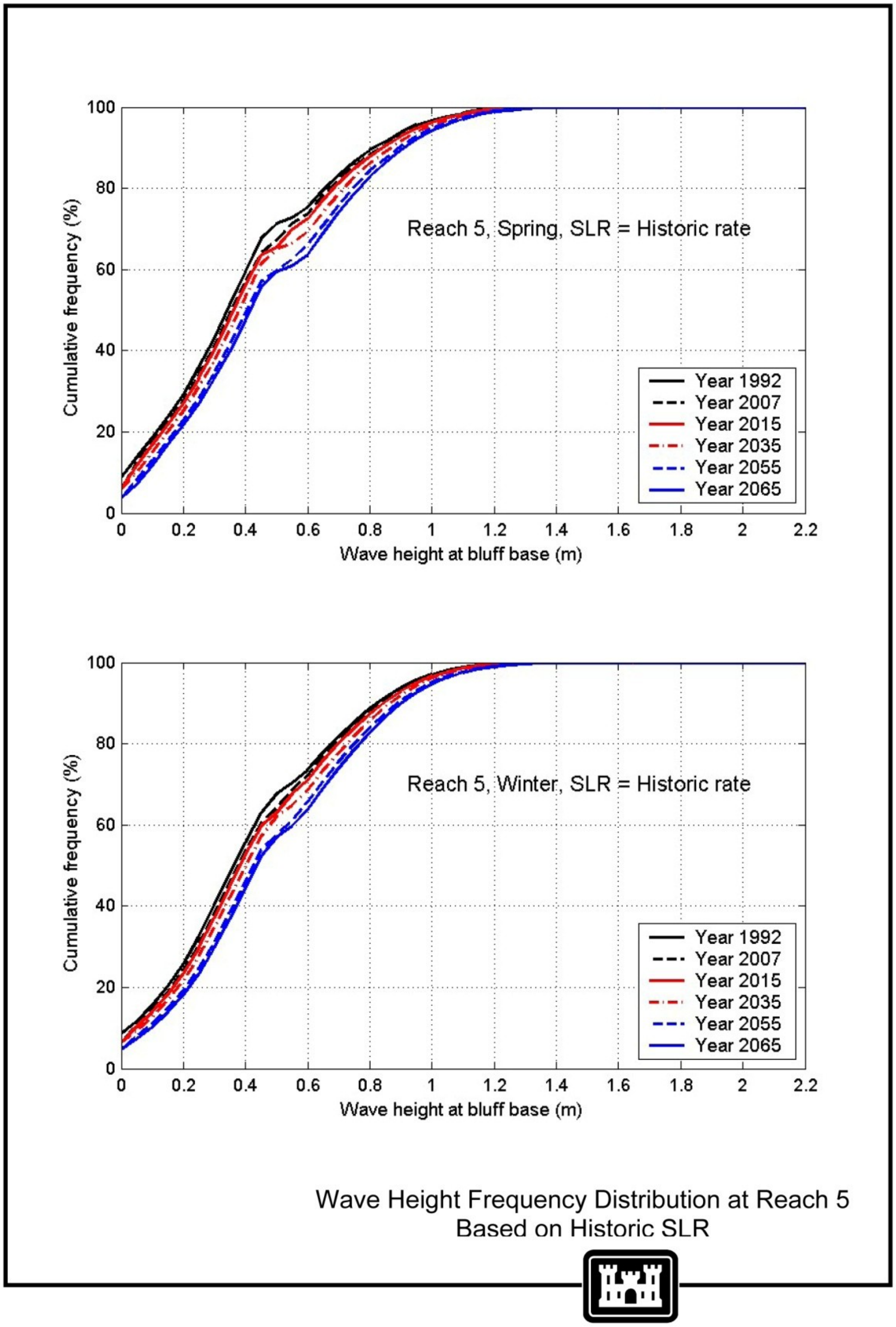


Figure 5.2-19 Wave Height Frequency Distribution at Reach 5 Based on Historic SLR

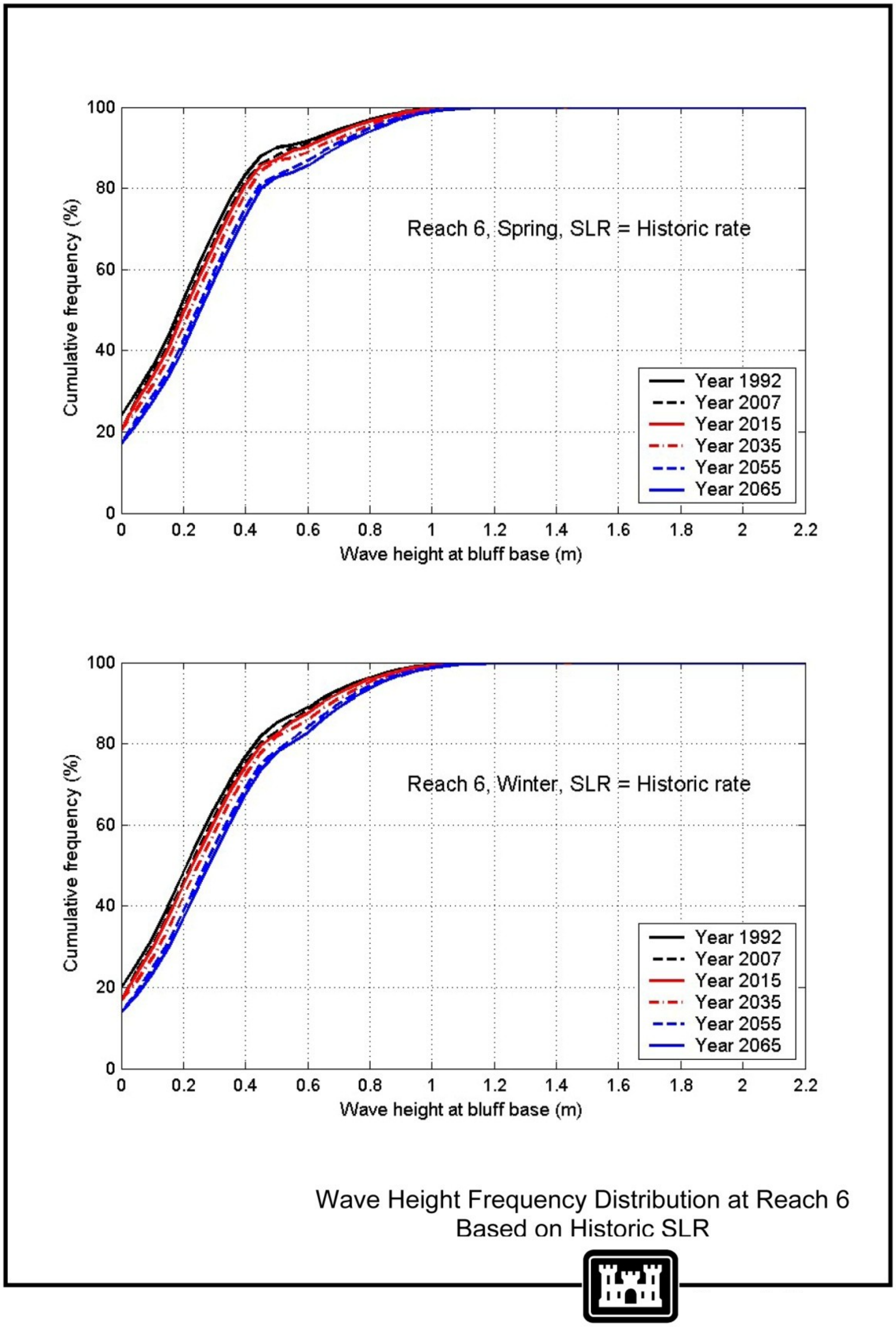


Figure 5.2-20 Wave Height Frequency Distribution at Reach 6 Based on Historic SLR

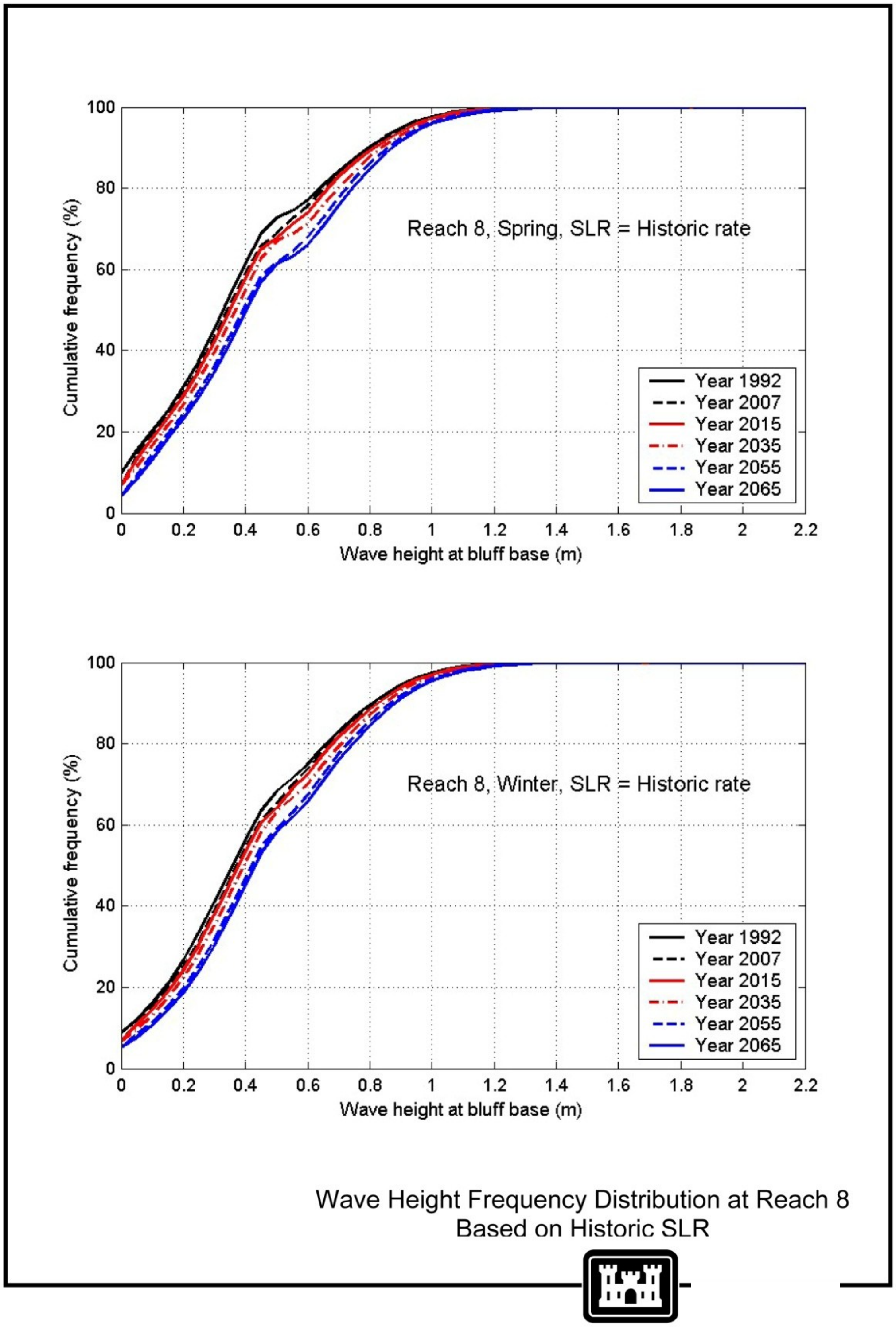


Figure 5.2-21 Wave Height Frequency Distribution at Reach 8 Based on Historic SLR

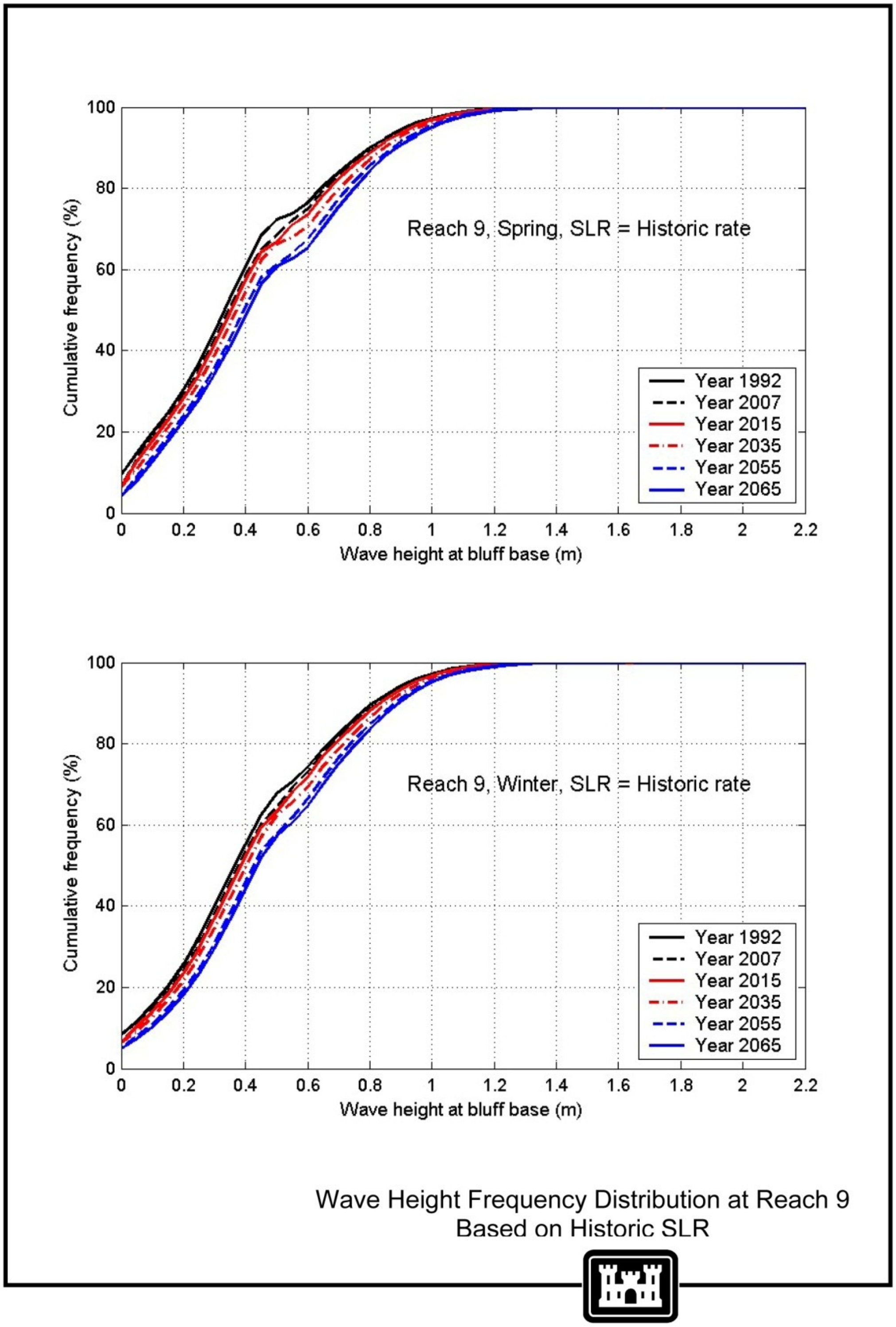


Figure 5.2-22 Wave Height Frequency Distribution at Reach 9 Based on Historic SLR

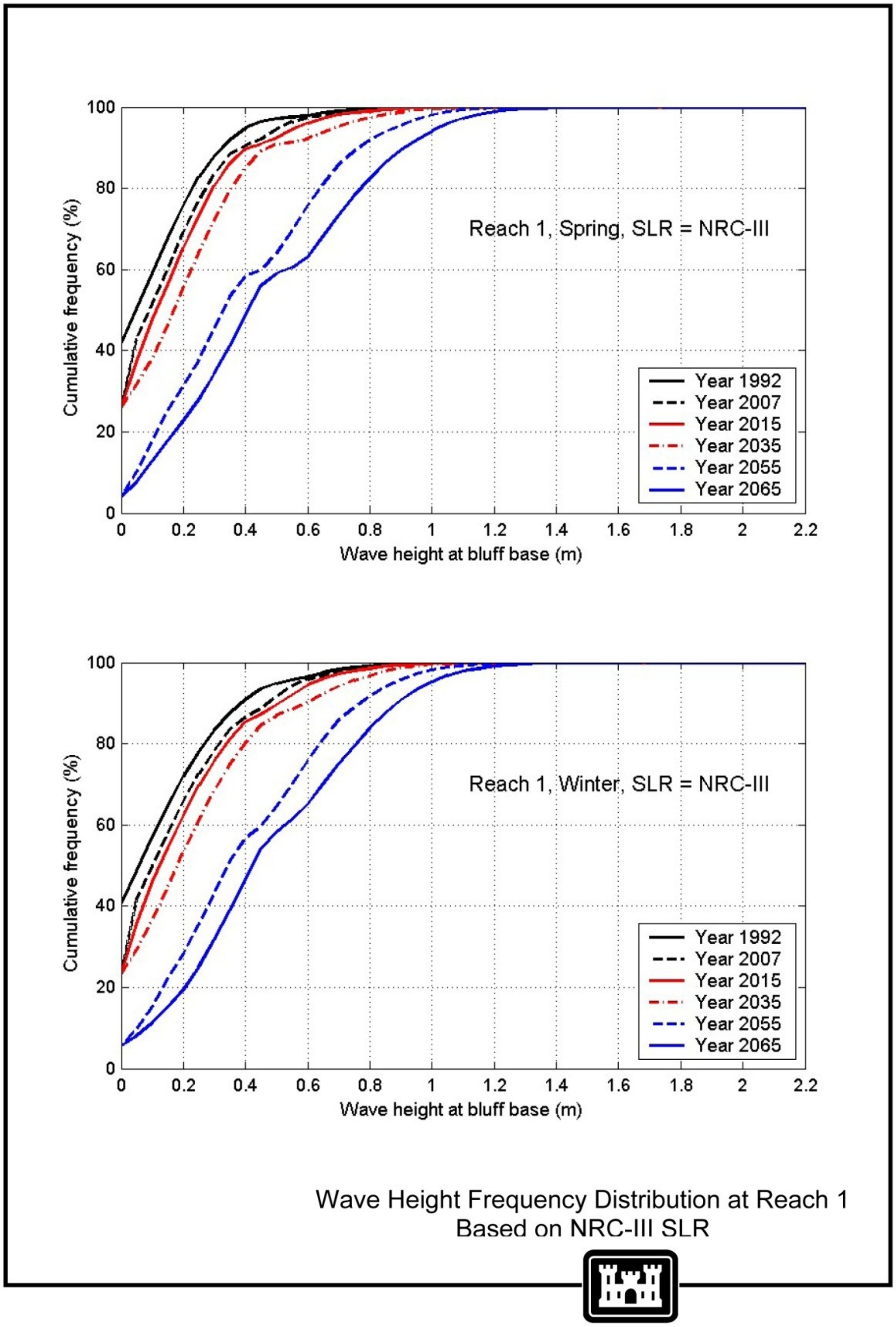
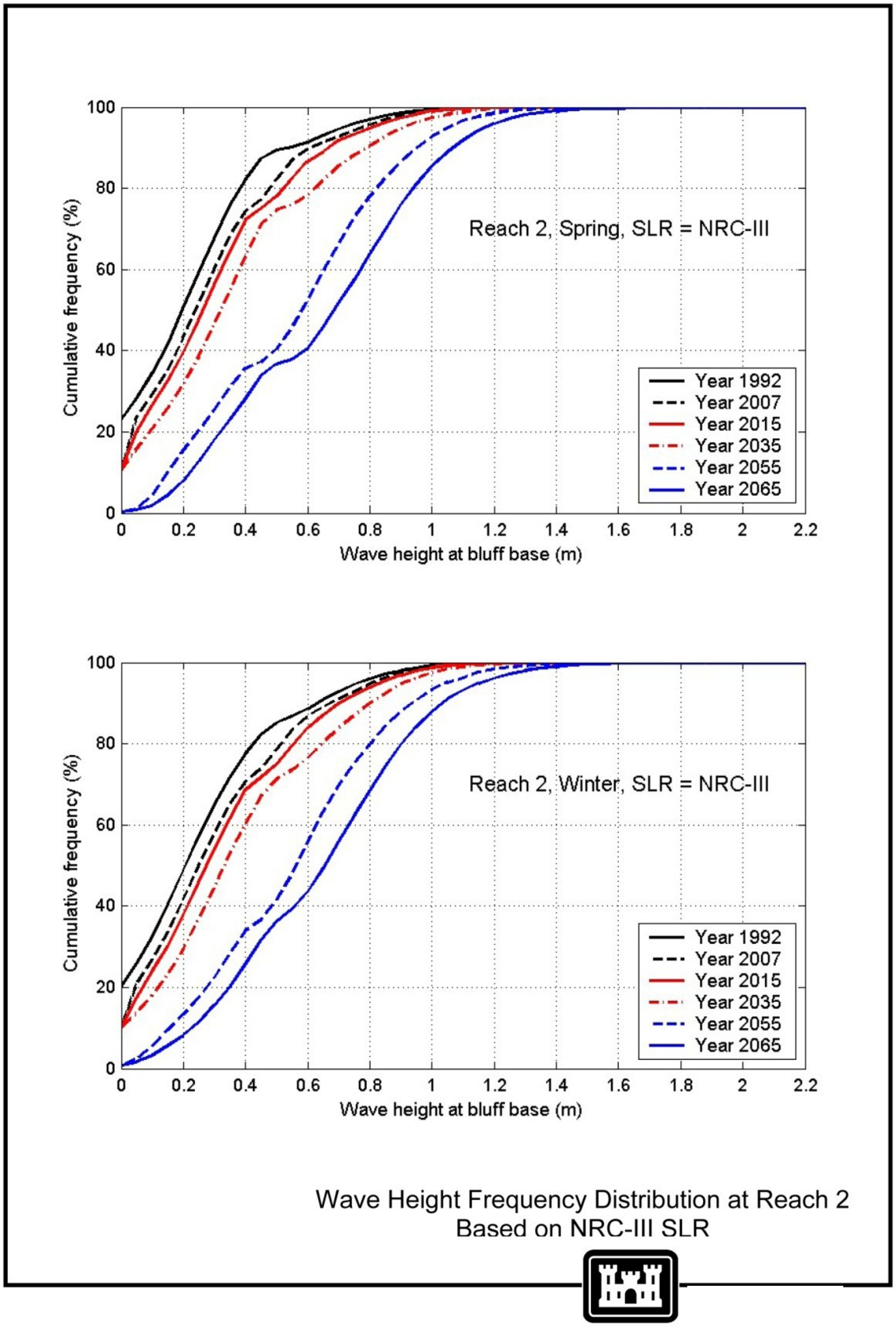


Figure 5.2-23 Wave Height Frequency Distribution at Reach 1 Based on NRC-III SLR



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2 **Figure 5.2-24 Wave Height Frequency Distribution at Reach 2 Based on NRC-III SLR**

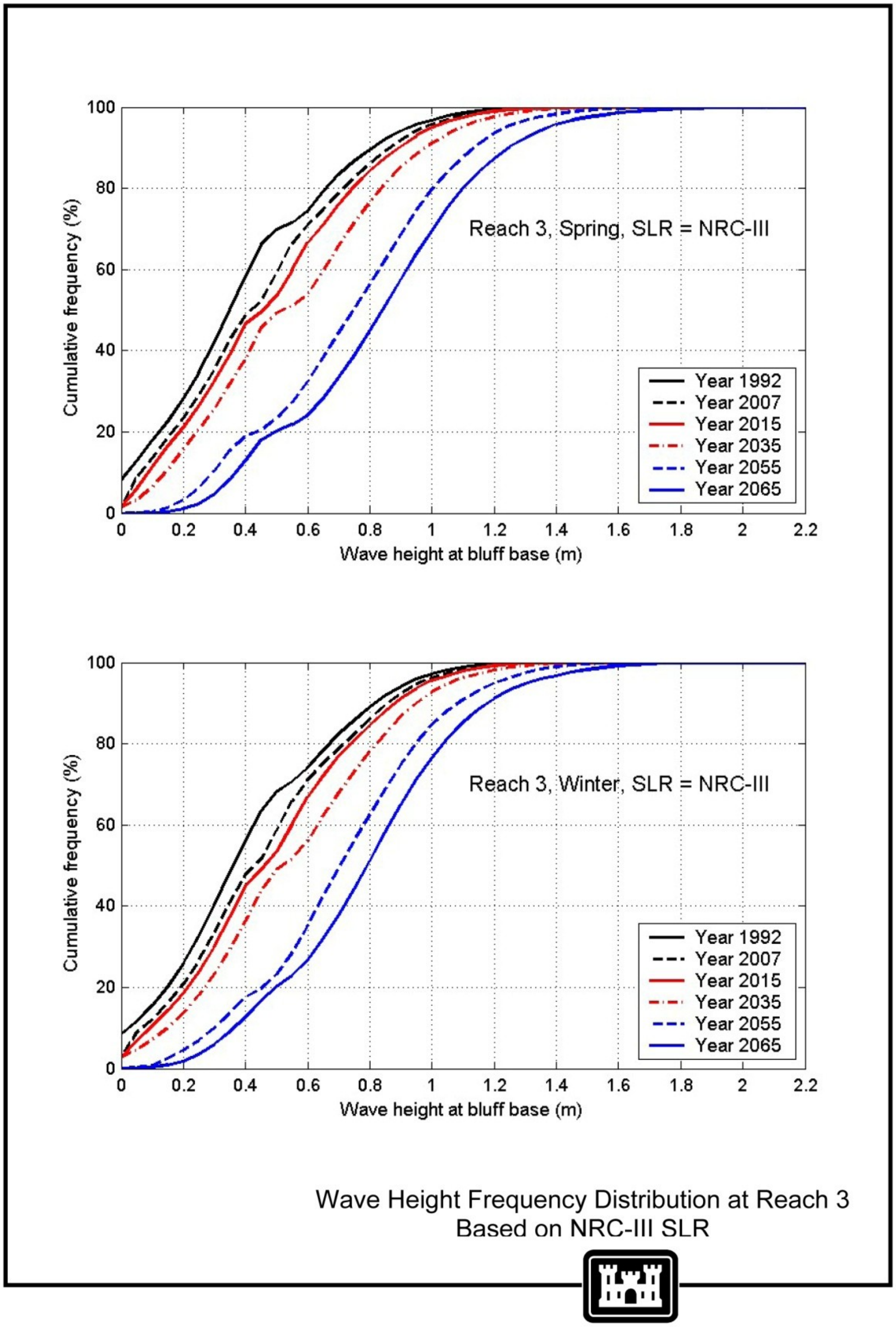


Figure 5.2-25 Wave Height Frequency Distribution at Reach 3 Based on NRC-III SLR

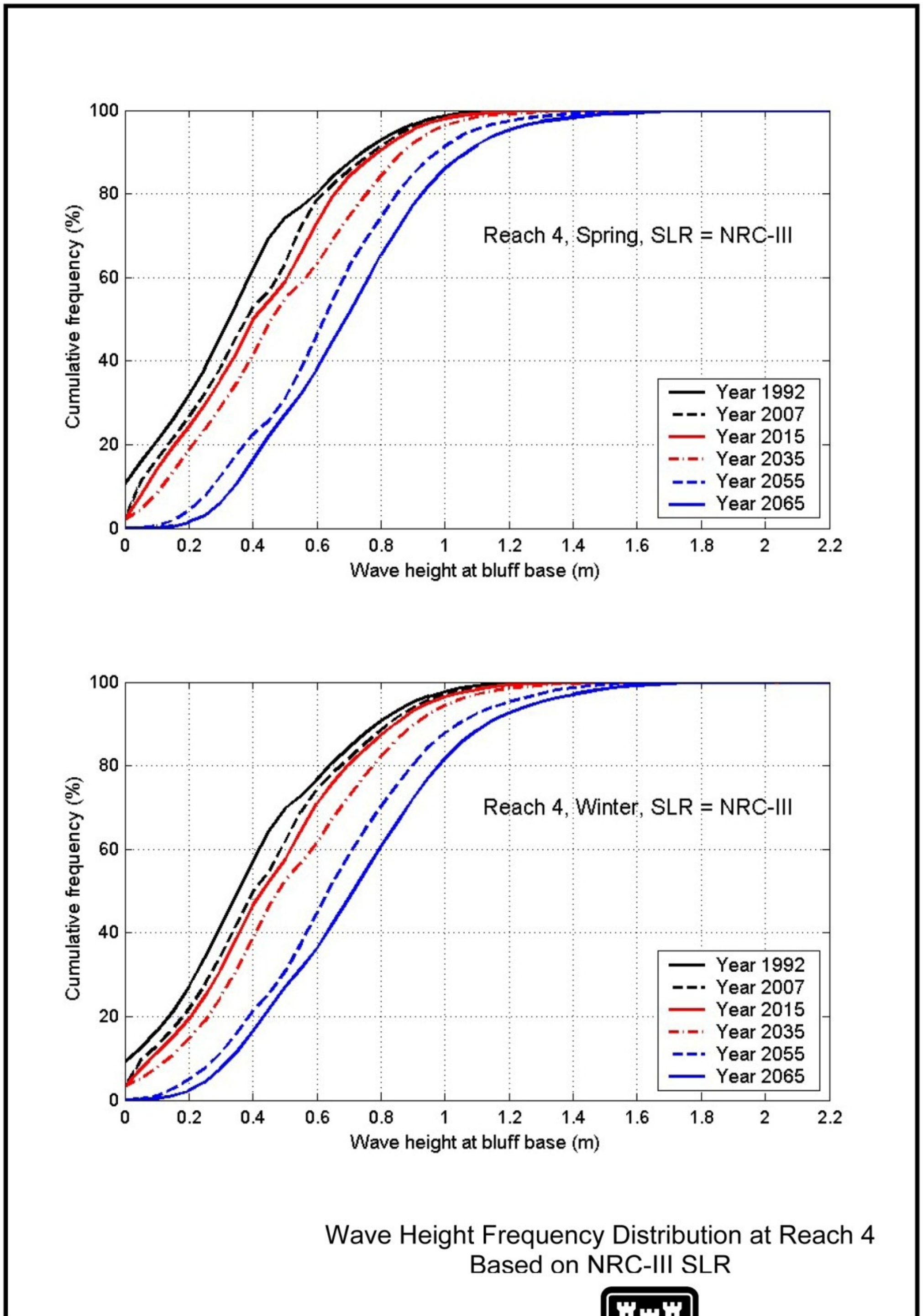
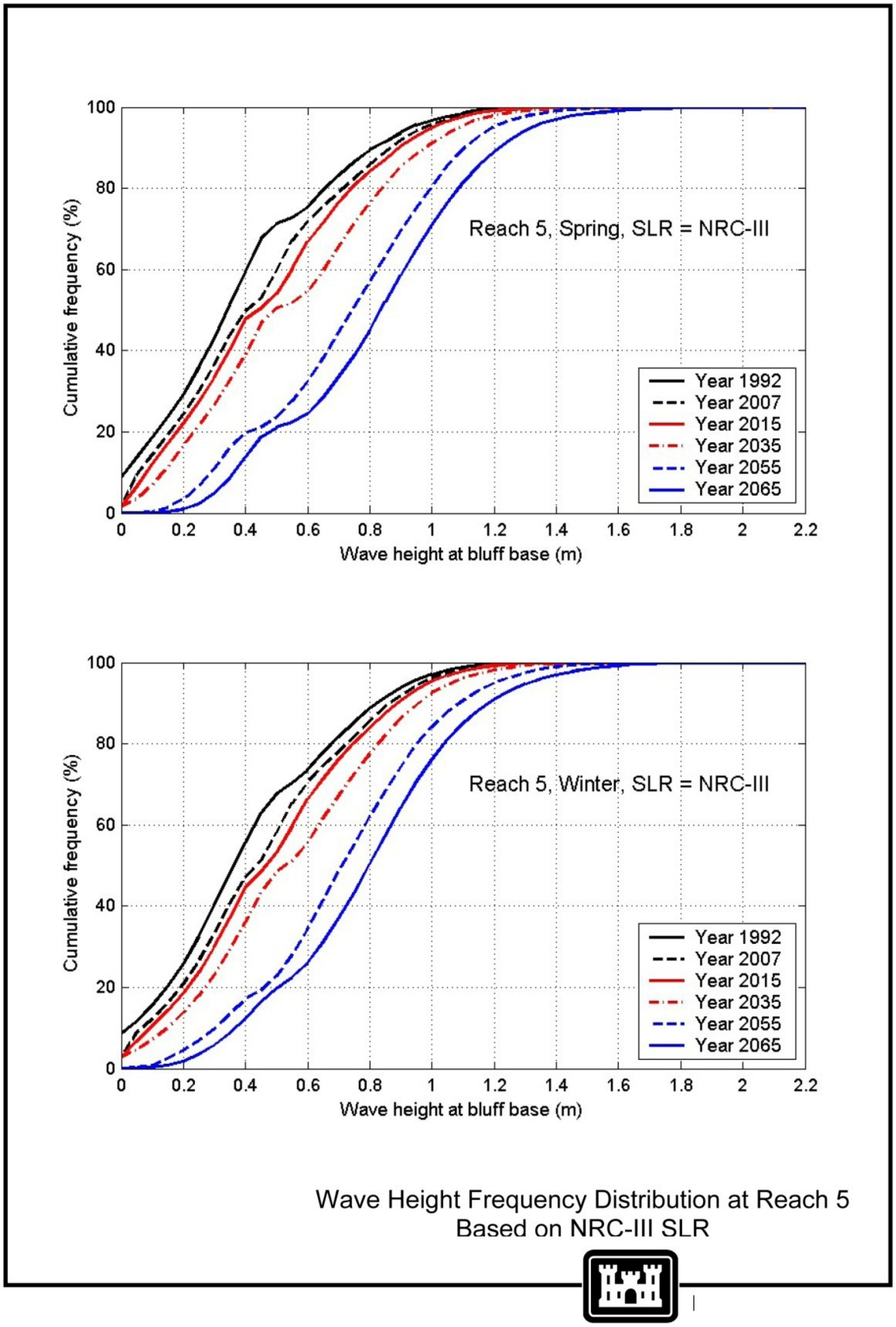


Figure 5.2-26 Wave Height Frequency Distribution at Reach 4 Based on NRC-III SLR



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2 **Figure 5.2-27 Wave Height Frequency Distribution at Reach 5 Based on NRC-III SLR**

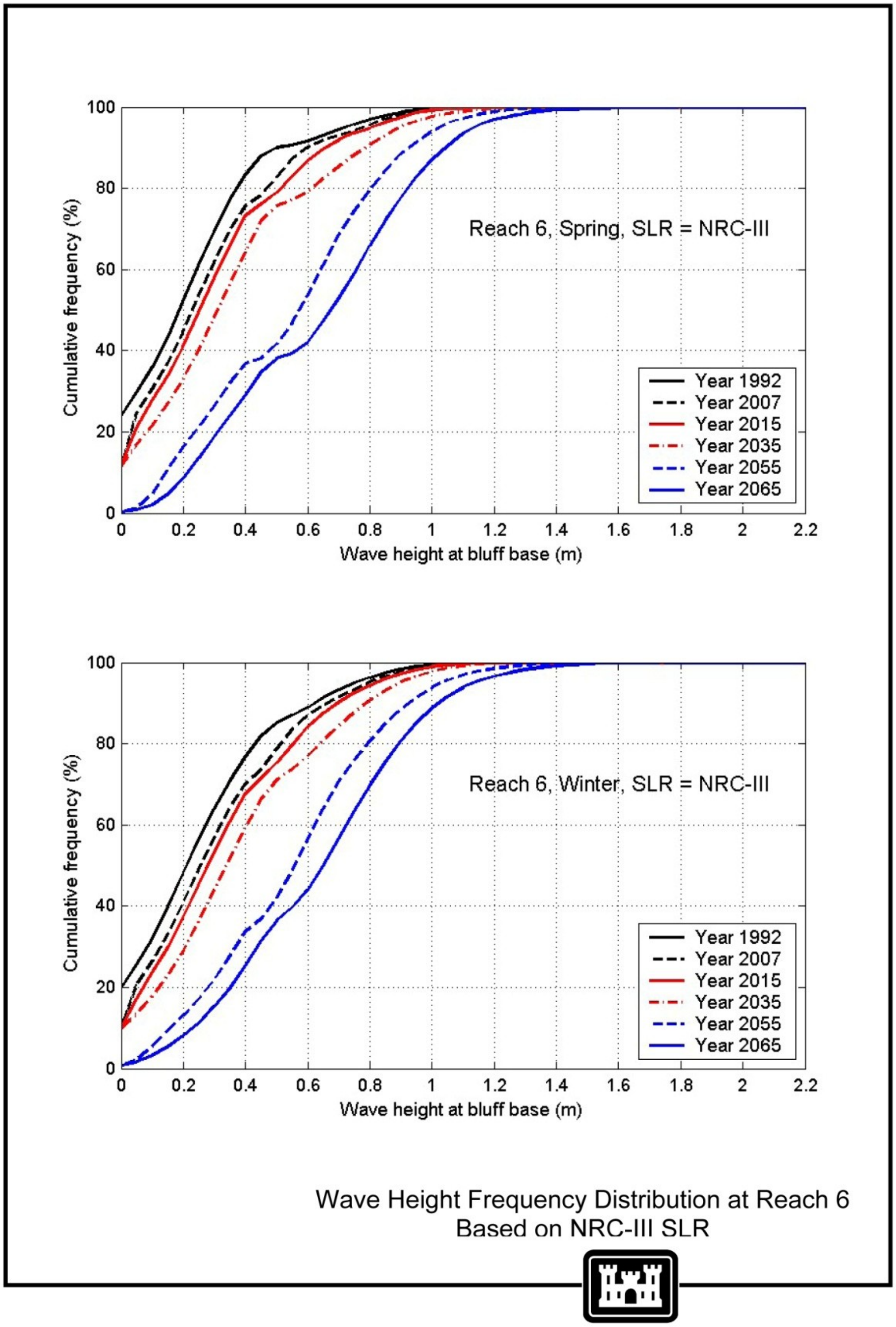
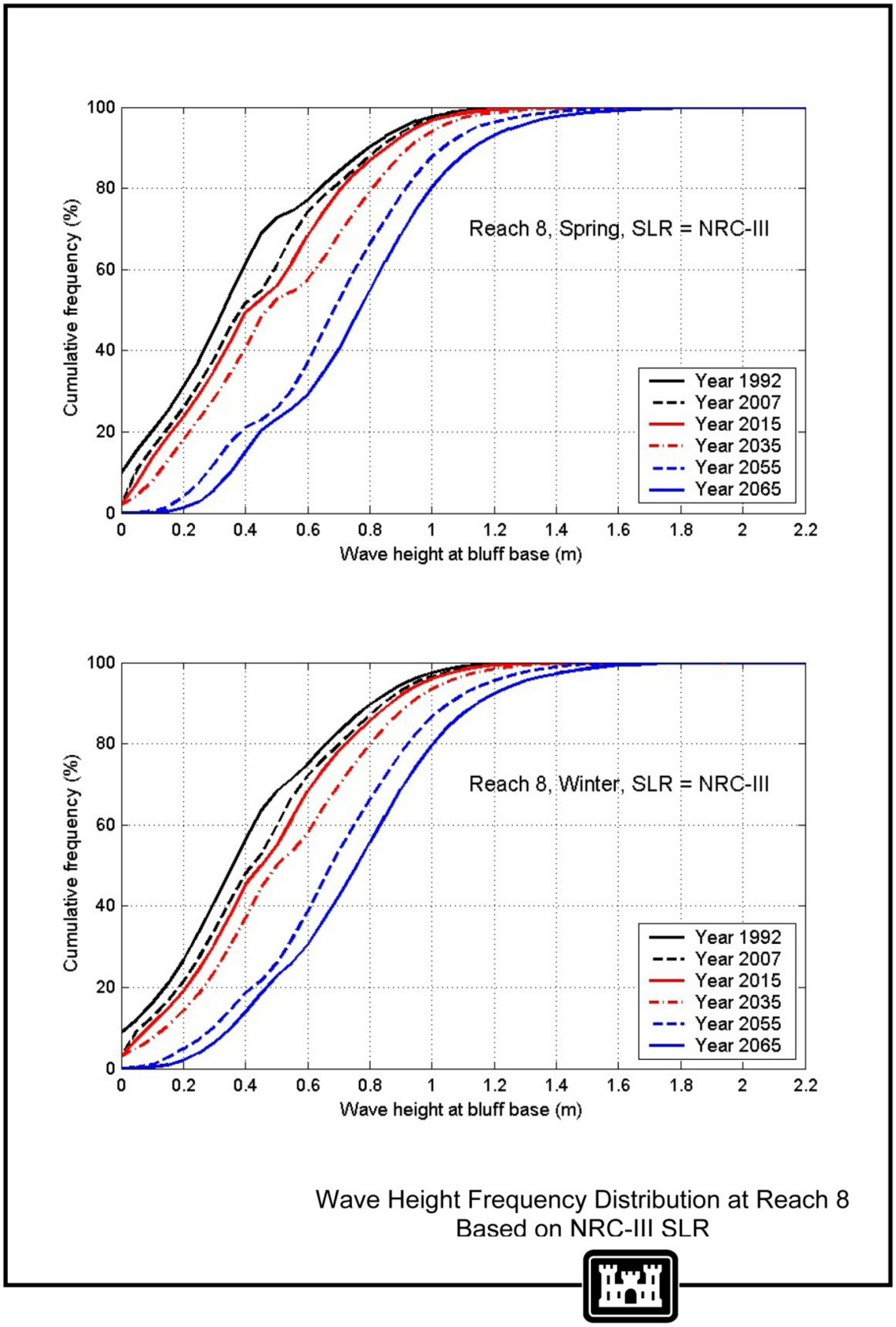


Figure 5.2-28 Wave Height Frequency Distribution at Reach 6 Based on NRC-III SLR



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2 **Figure 5.2-29 Wave Height Frequency Distribution at Reach 8 Based on NRC-III SLR**

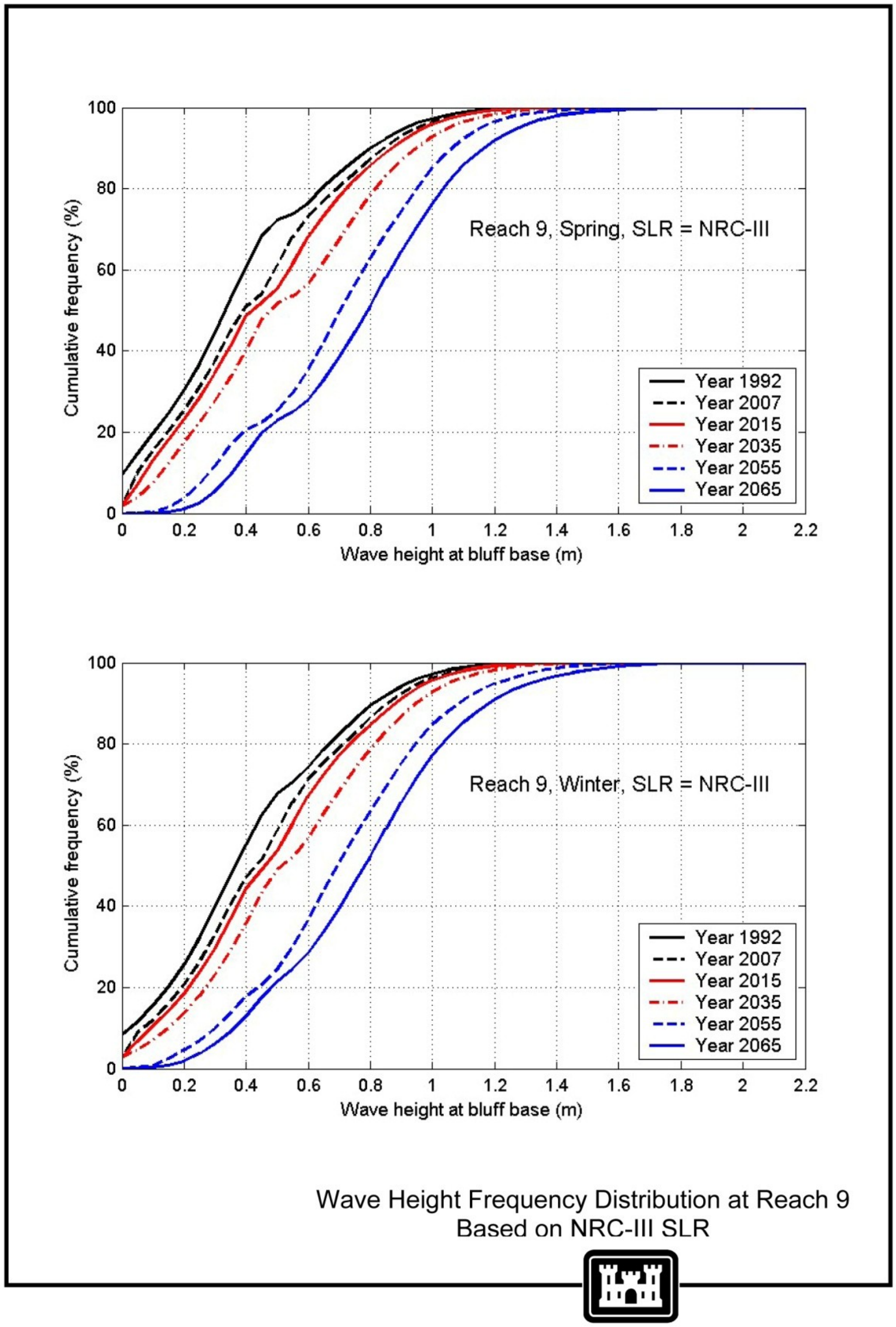


Figure 5.2-30 Wave Height Frequency Distribution at Reach 9 Based on NRC-III SLR

In the simulations, random waves at the bluff base were selected from each corresponding frequency distribution of wave height. Thus, the model does not follow the strict chronology of the approaching wave sequence, but randomly samples the impinging waves at the bluff base from the compiled statistic database. The wave selection process captures the total impinging wave energy for the two seasons in a given year, though not in the same exact sequence.

The statistic representation in terms of the magnitude of bluff failure (referred to as the erosion of the bluff crest), as shown in **Figure 5.2-31**, was derived from the observed field data reported since the 1990's. Among the 203 reported historic bluff failures, 137 events that had the detailed information including length, height and depth (thickness) were used to deduce the frequency distribution of bluff failure. Although the maximum bluff retreat did exceed 30 feet (9 meters) in depth, the majority of bluff failure (approximately 90 percent) had a magnitude of 3 to 10 feet (0.8 to 3.2 meters) in depth.

Model Implementation

The Monte Carlo simulation is a statistical approach to predict an uncertain system by using sequences of random numbers. This technique allows for the random sampling of a pre-defined (known) occurrence distribution of each individual element to statistically characterize the behavior of the uncertain system.

After formulation of the frequency distributions of wave height and bluff retreat, and the calibration of Sunamura's empirical coefficients (k and H_j), future bluff failures for a project design life of 50 years were statistically predicted. The entire modeling system consisted of the deterministic Sunamura submodel and a series of random numbers generated via the Monte Carlo technique. Each individual wave height or bluff retreat was then referred to a randomly selected number in accordance with the deduced frequency distribution that was formulated in each reach.

In each simulation, two uncorrelated data sets were respectively generated for the wave height at the bluff base and the magnitude of the upper bluff retreat, if a bluff failure occurs. The random numbers represented random populations of the entire 50-year simulation period in a 3-hour interval during the winter and spring seasons. Each simulated time step, the bluff toe erosion was calculated from the Sunamura submodel, based upon a randomly selected wave height. If the cumulative notch depth exceeded the threshold value (i.e., 8 feet for triggering a bluff failure, the individual upper bluff retreat was then determined by a randomly selected value from the second set of random populations. Subsequently, the cumulative bluff retreat and the new notch depth were updated. This procedure continued until the end of the 50th year. **Figure 5.2-32** illustrates the flowchart of the model structure for each simulation.

Sufficient simulations were required to generate a statistic representation of the modeled results. The range (deviation) and average (mean) values of the bluff retreat were derived from the total required simulations. Although the random sequence of wave height selected in the Monte Carlo Simulation cannot physically resemble a storm wave condition, the modeled bluff retreat resulting from the accumulation of individual wave in each time step does statistically represent the bluff failure scenarios over the simulated period.

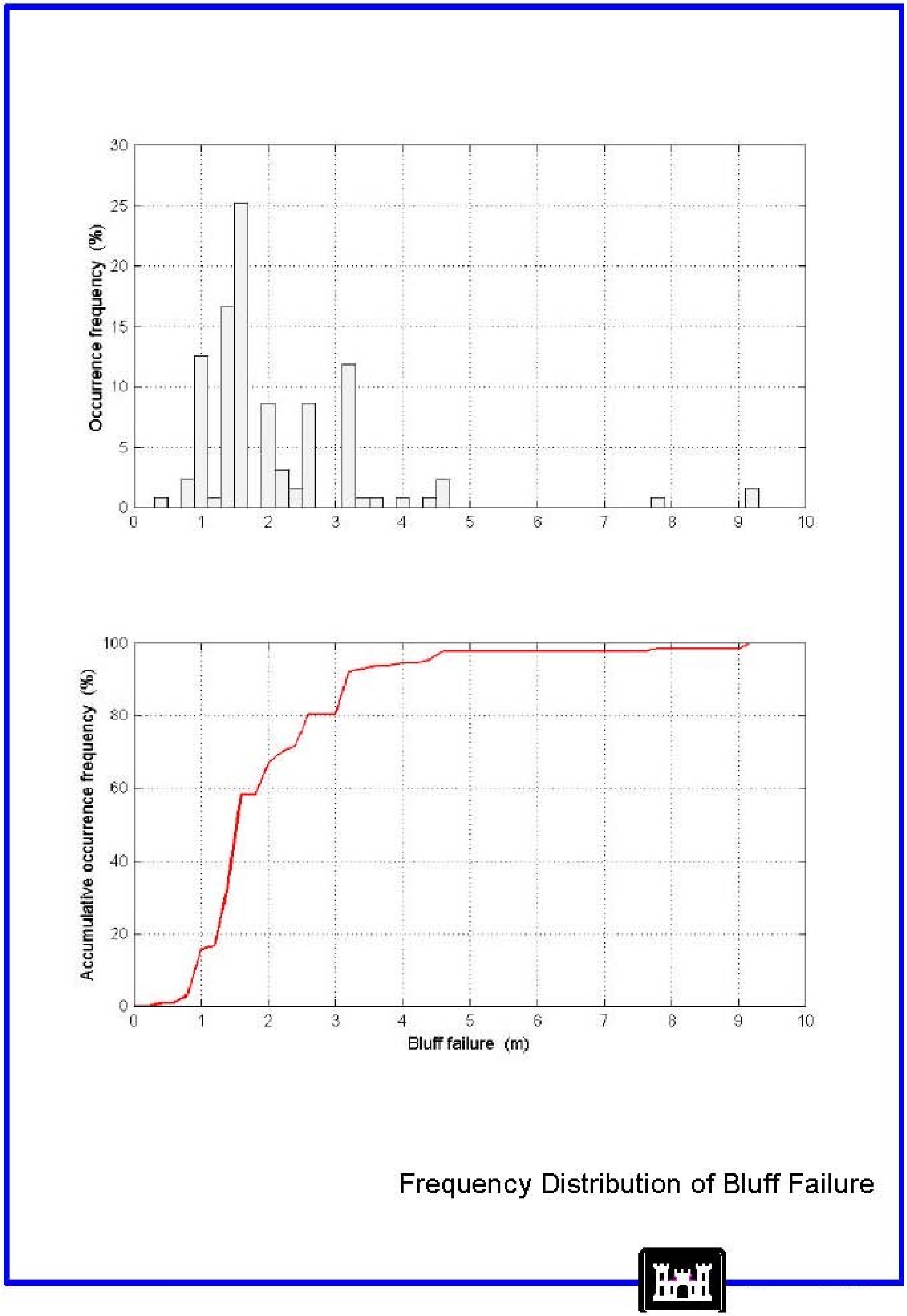


Figure 5.2-31 Frequency Distribution of Bluff Failure

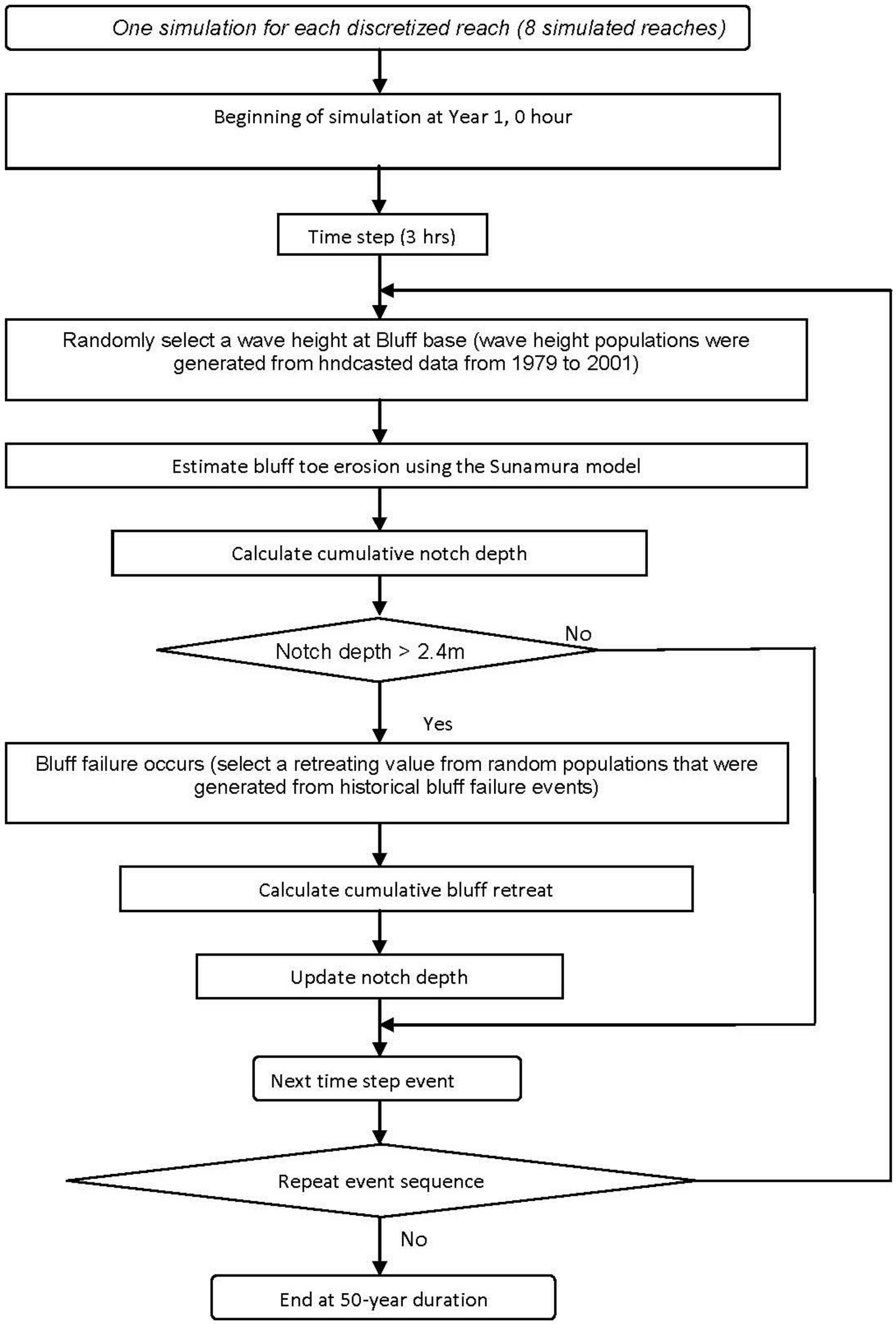


Figure 5.2-32 Flowchart of the Model Structure for One Simulation

5.2.3.1 Simulated Results Without Sea Level Rise

Since the most recent field investigation of the bluff was conducted by the Corps of Engineers in 2007 to update the setback distance at the bluff-top development and other pertinent geophysical conditions of the bluff, it is necessary for the bluff retreat simulation to extend the time period from 2007 and 2065, although the project starting year is designated to be in 2015 (i.e., Year 0). The notch depth was last updated in 2007 and the future notch condition in 2015 is not obtainable as any economic "events" that occur before the evaluation period are not counted as benefits. It is expected that different initial notch depths will result in the variation of the modeled bluff retreat at the end of the 50-year simulation. Considering the possible range of the observed notch depths, four cases with different initial notch depths of 0, 2 feet, 4 feet, and 6 feet, were included in the simulation. **Figure 5.2-33** shows, for example, the predicted mean bluff retreats averaged over 200 simulations for the four initial notch depths in Reach 8. It can be observed that the simulation-averaged bluff retreat is directly proportional to the selected initial notch depth. The discrepancy in the cumulative bluff retreat at the end of the 50-year period is approximately equal to the difference of the initial notch depth. The initial condition affects the timing when the notch will reach its threshold depth of 8 ft. The different starting point show in **Figure 5.2-33** provides a series of values of top of bluff retreat time with different initial conditions. The economic model simulation subdivided each reach into lengths with different initial notch depths and sampled corresponding bluff retreat rates.

To achieve a better statistic representation of the random process, sufficient Monte Carlo simulations were executed. The time history of each simulation resembles the likely individual scenario of bluff failure within the study area. **Figure 5.2-34** shows the simulation-averaged bluff retreats in Reach 8 for simulations of 10, 100, 200 and 1,000 runs, respectively. The discrepancy of the simulation-averaged results reduces, as the number of simulations increases. The discrepancy becomes negligible for 200 simulations or more. Therefore, 200 simulations should be sufficient to obtain a reliable estimate of the averaged bluff retreat. Nevertheless, the modeled results of 1,000 simulations were provided for an economic evaluation to account for the potential variation of the development damage at the bluff top.

It is noted that the computed wave heights are generally smaller in Reach 1, as compared to that in other reaches, due to the elevated bluff base (**Table 5.2-2**). In addition, the rock formation of the bluff face is more resistant to wave abrasion (see *k* value in **Table 5.2-5**). Therefore, no resulting bluff retreat was modeled in Reach 1. Past bluff failure records indicate that little bluff failure occurred within this reach, probably due to the high elevation of the bluff base and the natural armoring of a backbeach cobble berm. Various degrees of the resultant bluff retreat (from minor to severe) were computed for the remaining reaches. **Figure 5.2-35** to **Figure 5.2-41** show the time histories of 1000 simulated results from 2007 to 2065 in Reaches 2, 3, 4, 5, 6, 8 and 9, except Reach 1. It is noted that the project starting year is in 2015. A time history of the mean bluff retreat is also presented in each figure. **Table 5.2-6** lists the modeled mean bluff retreat at the end of the 50-year cycle, which agrees relatively well to the average annual retreat rate that was previously adopted in the engineering evaluation, as presented in **Appendix D**. Much higher erosion rates estimated in Reaches 3, 8 and 9 are due to poor rock resistance of the bluffs and low base elevations that result in more exposure to direct wave impingement on the bluff base.

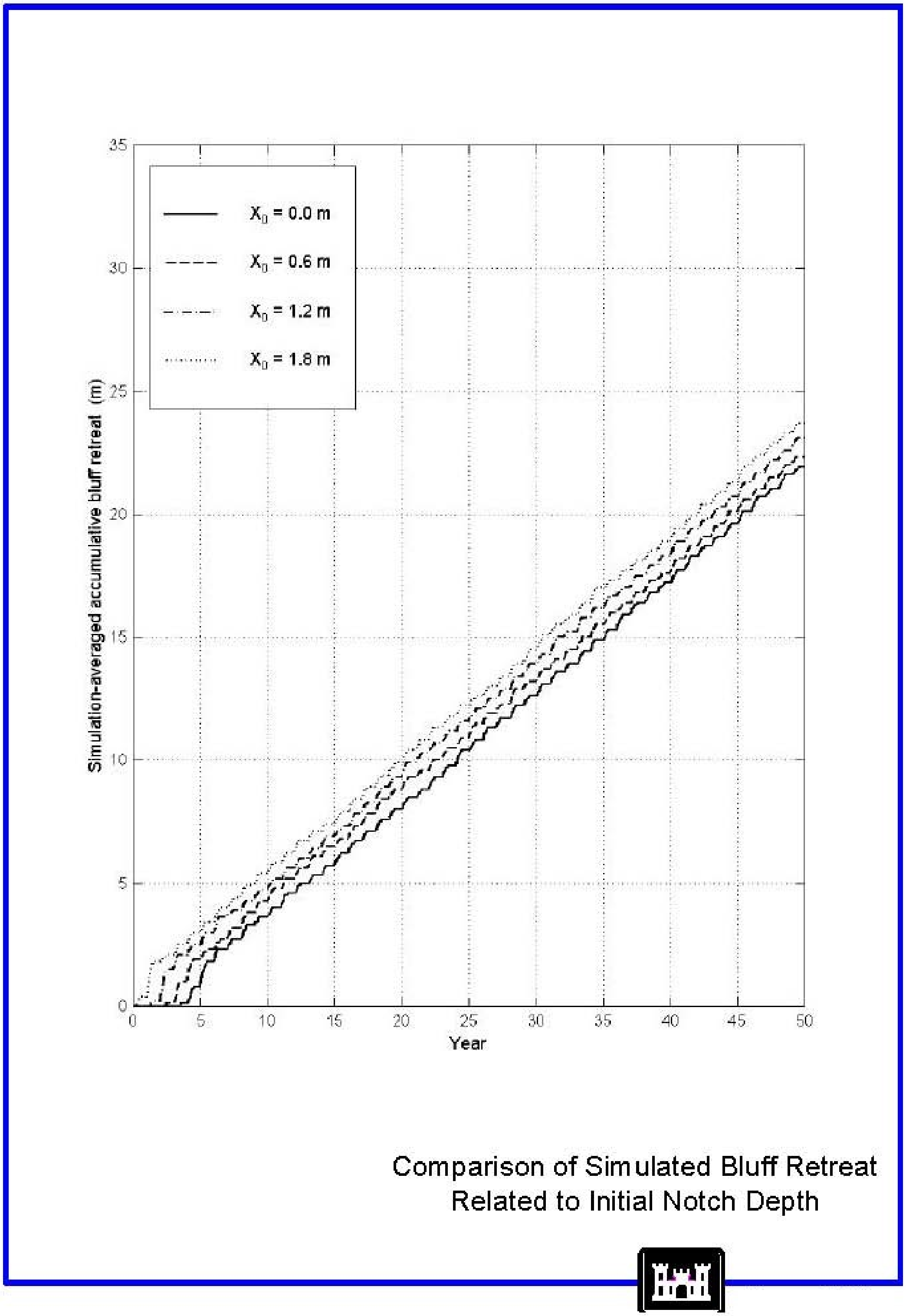


Figure 5.2-33 Comparison of Simulated Bluff Retreat Related to Initial Notch Depth

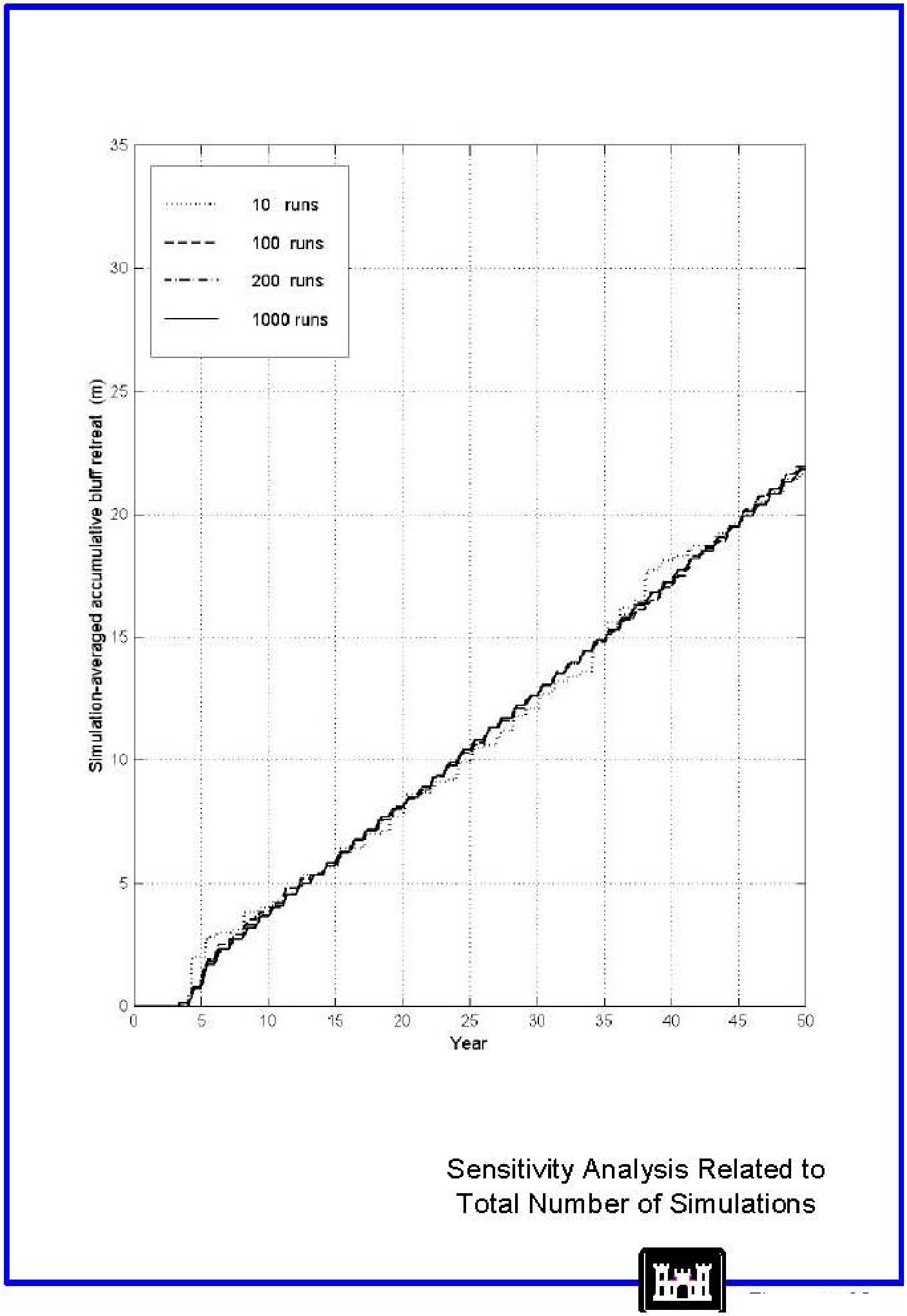
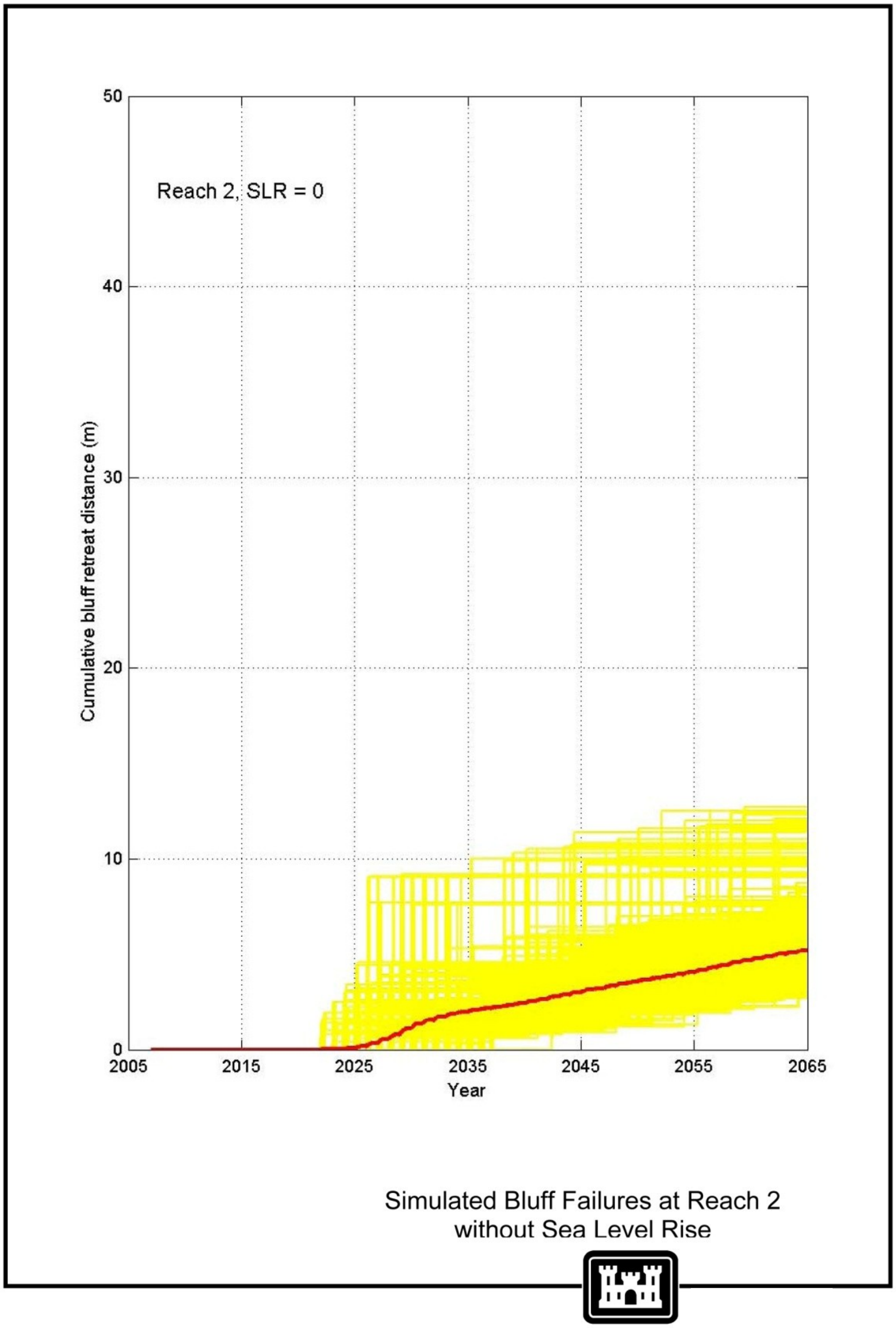


Figure 5.2-34 Sensitivity Analysis Related to Total Number of Simulations



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2 **Figure 5.2-35 Simulated Bluff Failures at Reach 2 without Sea Level Rise**

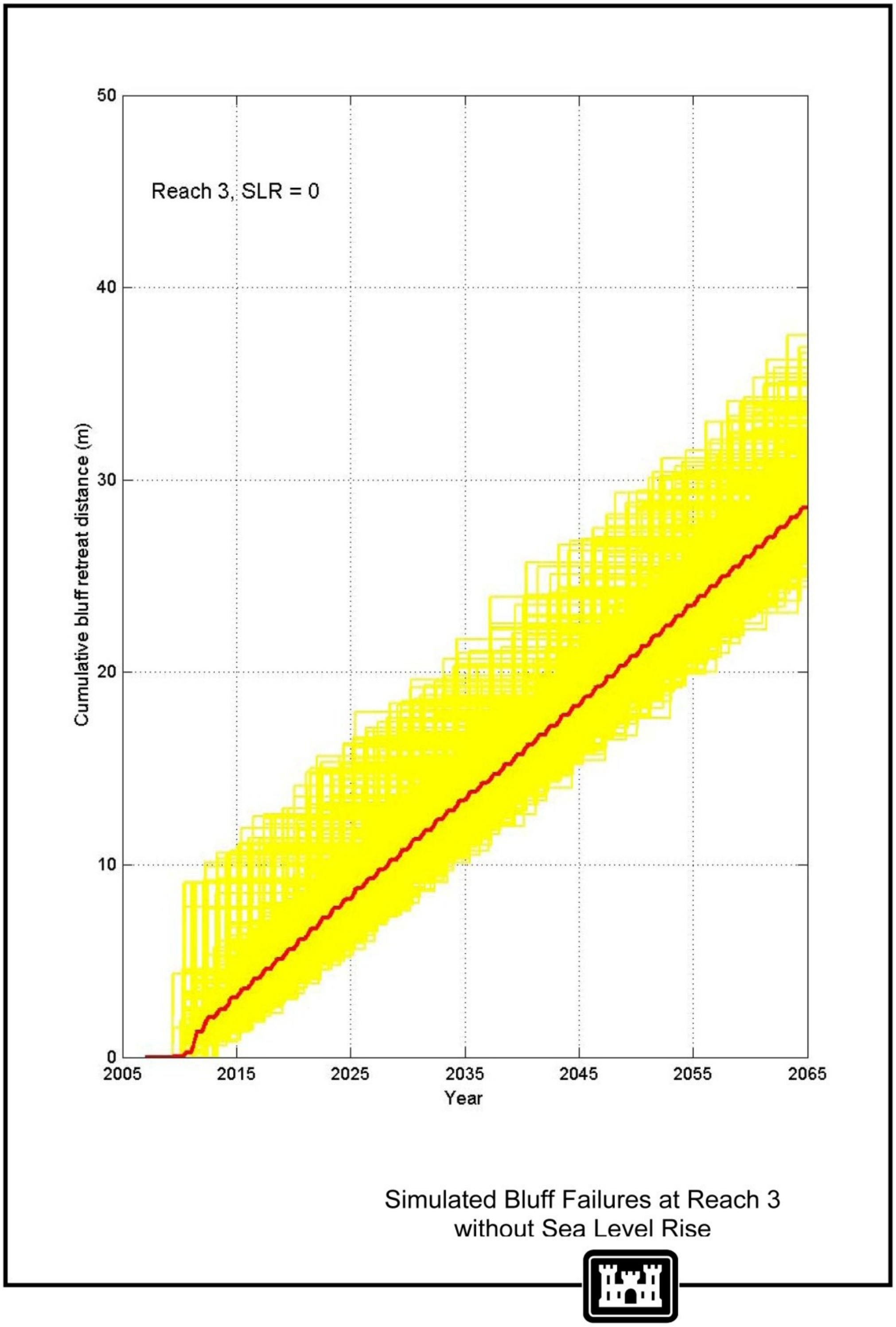


Figure 5.2-36 Simulated Bluff Failures at Reach 3 without Sea Level Rise

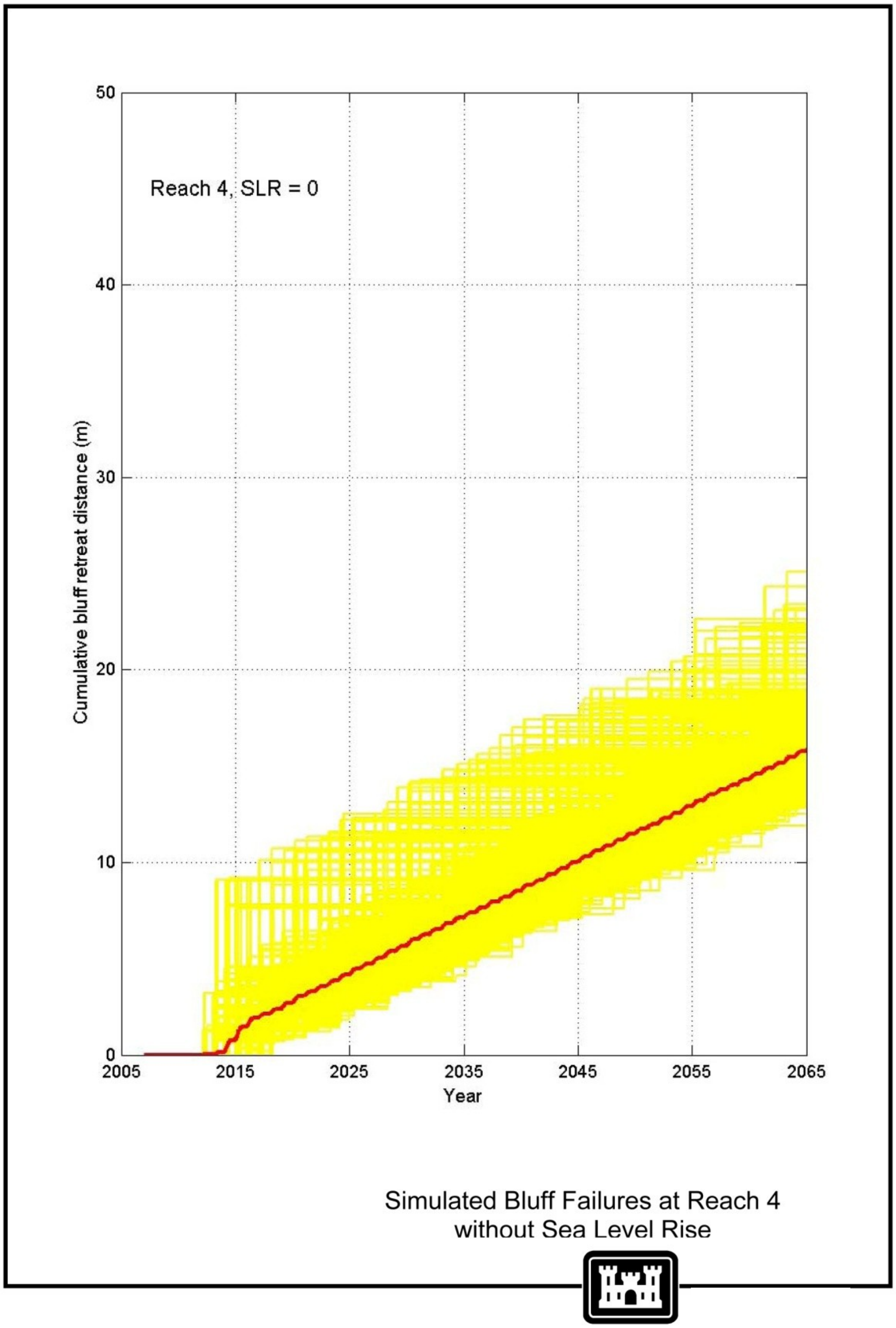


Figure 5.2-37 Simulated Bluff Failures at Reach 4 without Sea Level Rise

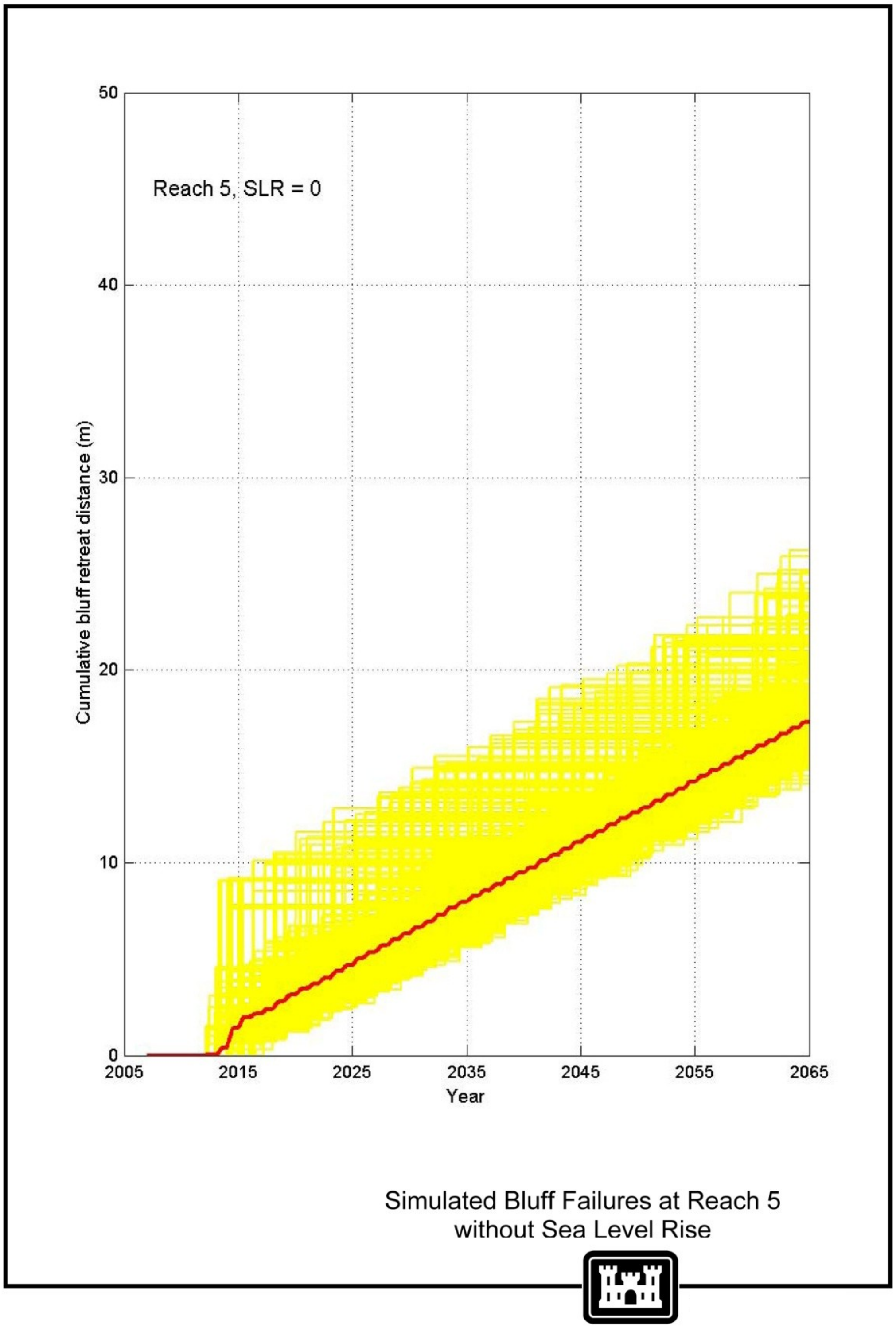


Figure 5.2-38 Simulated Bluff Failures at Reach 5 without Sea Level Rise

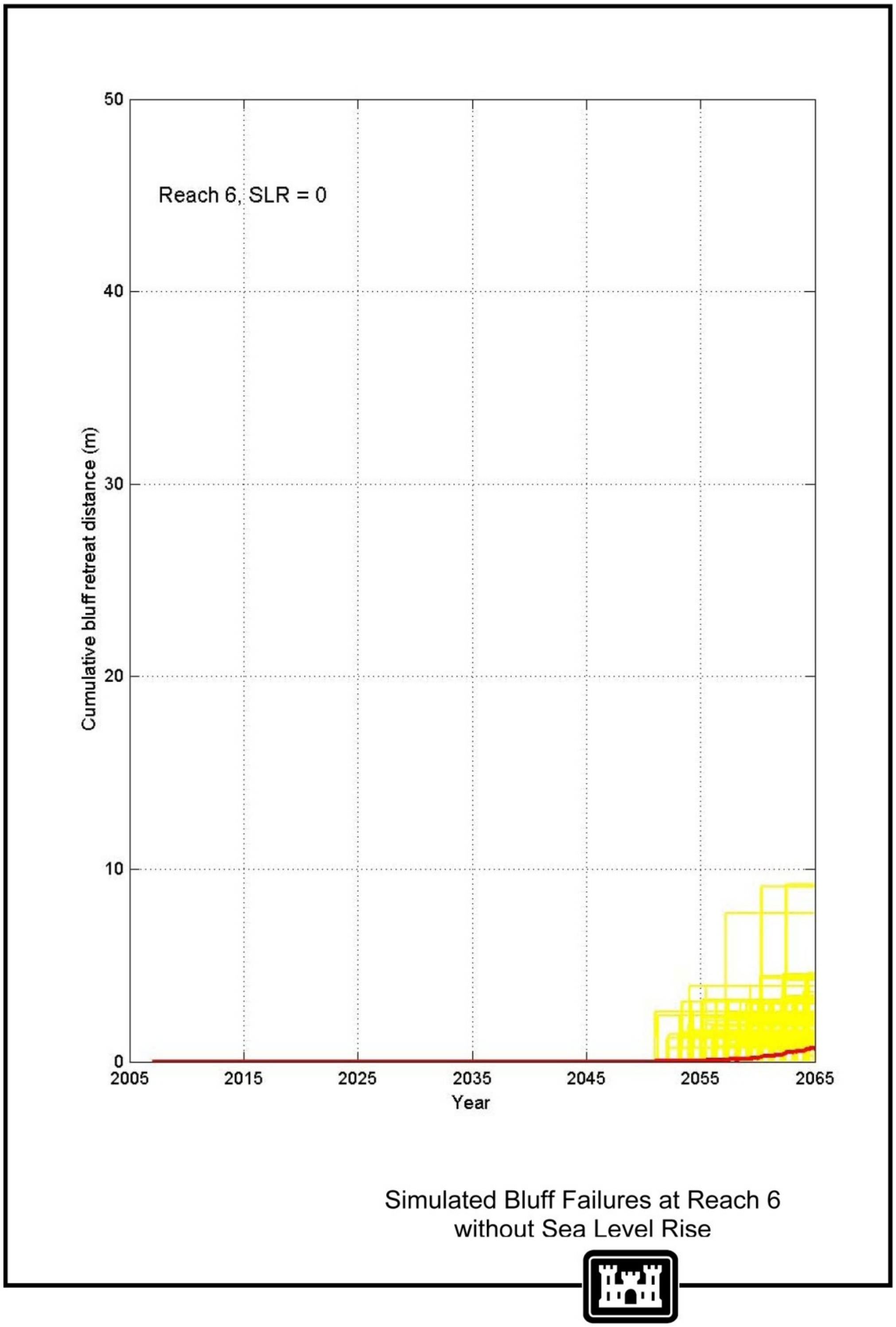


Figure 5.2-39 Simulated Bluff Failures at Reach 6 without Sea Level Rise

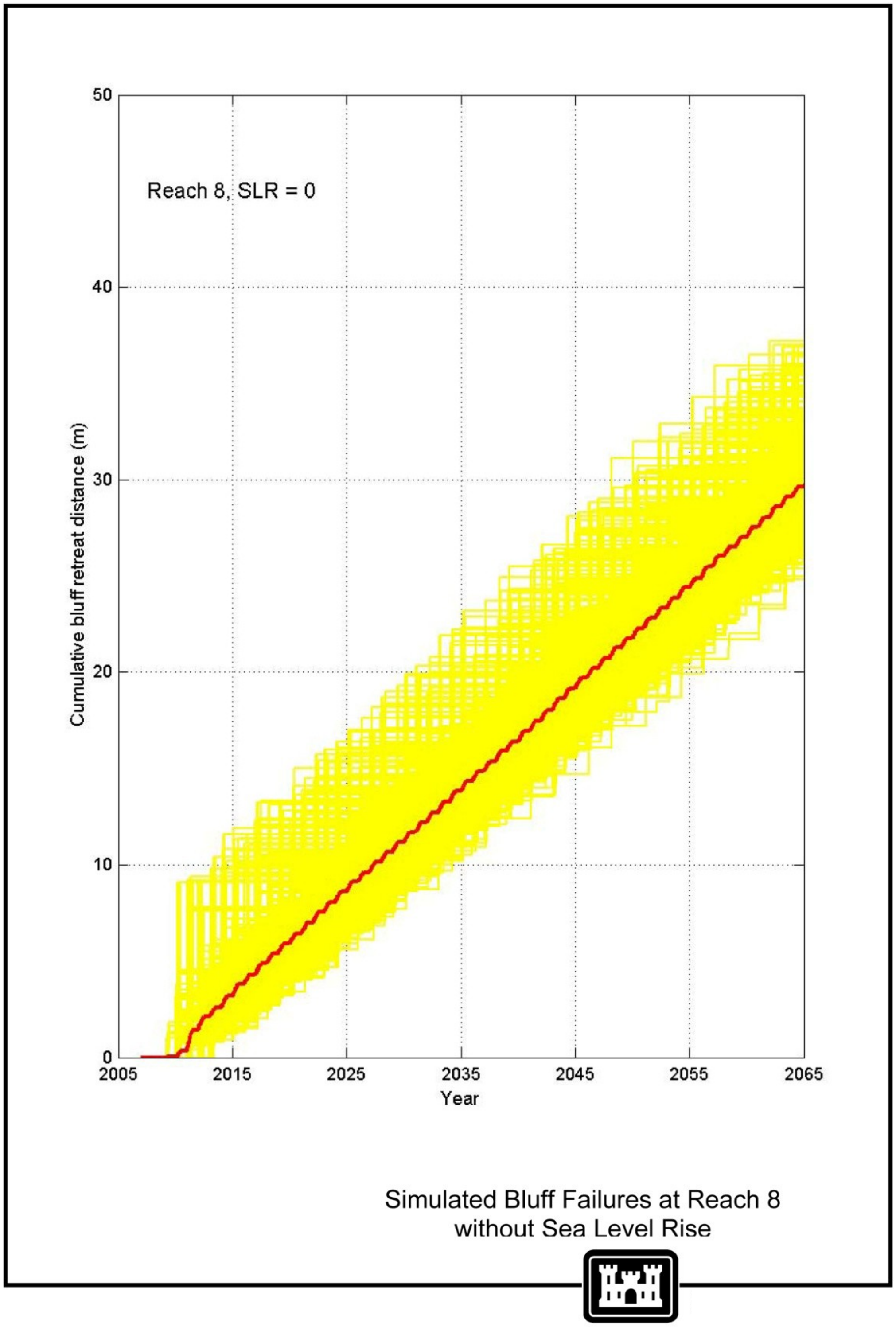


Figure 5.2-40 Simulated Bluff Failures at Reach 8 without Sea Level Rise

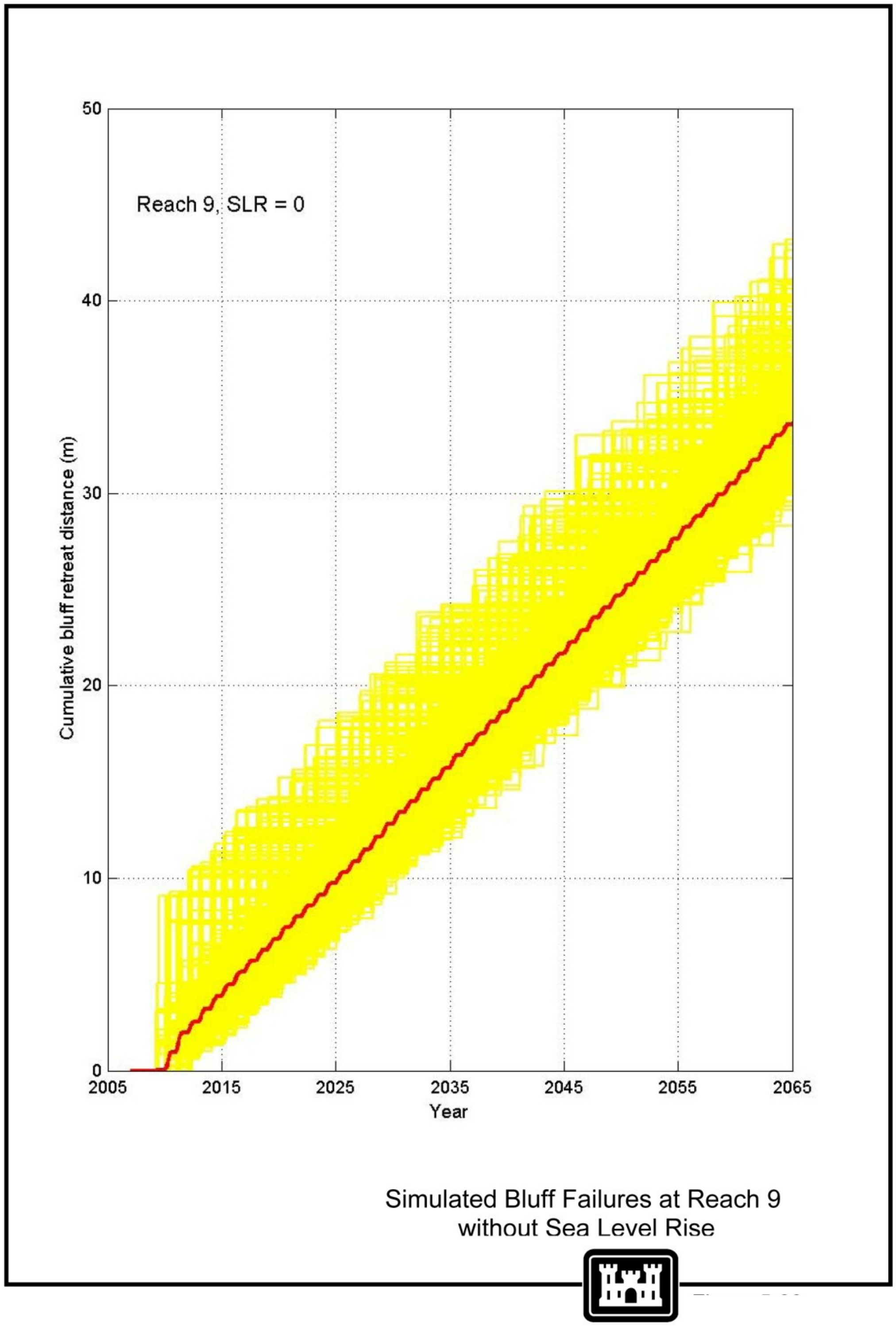


Figure 5.2-41 Simulated Bluff Failures at Reach 9 without Sea Level Rise

To further delineate the statistical representation of the simulated results, **Figure 5.2-42** illustrates the cumulative probability occurrence of the predicted resultant bluff retreat at the end of a 50-year project period in Reach 8. The figure implies that only a 5-percent chance for the cumulative bluff retreat of 82 feet or greater would occur at the end of the 50th year. The similar statistical representation can also be deduced for the remaining reaches.

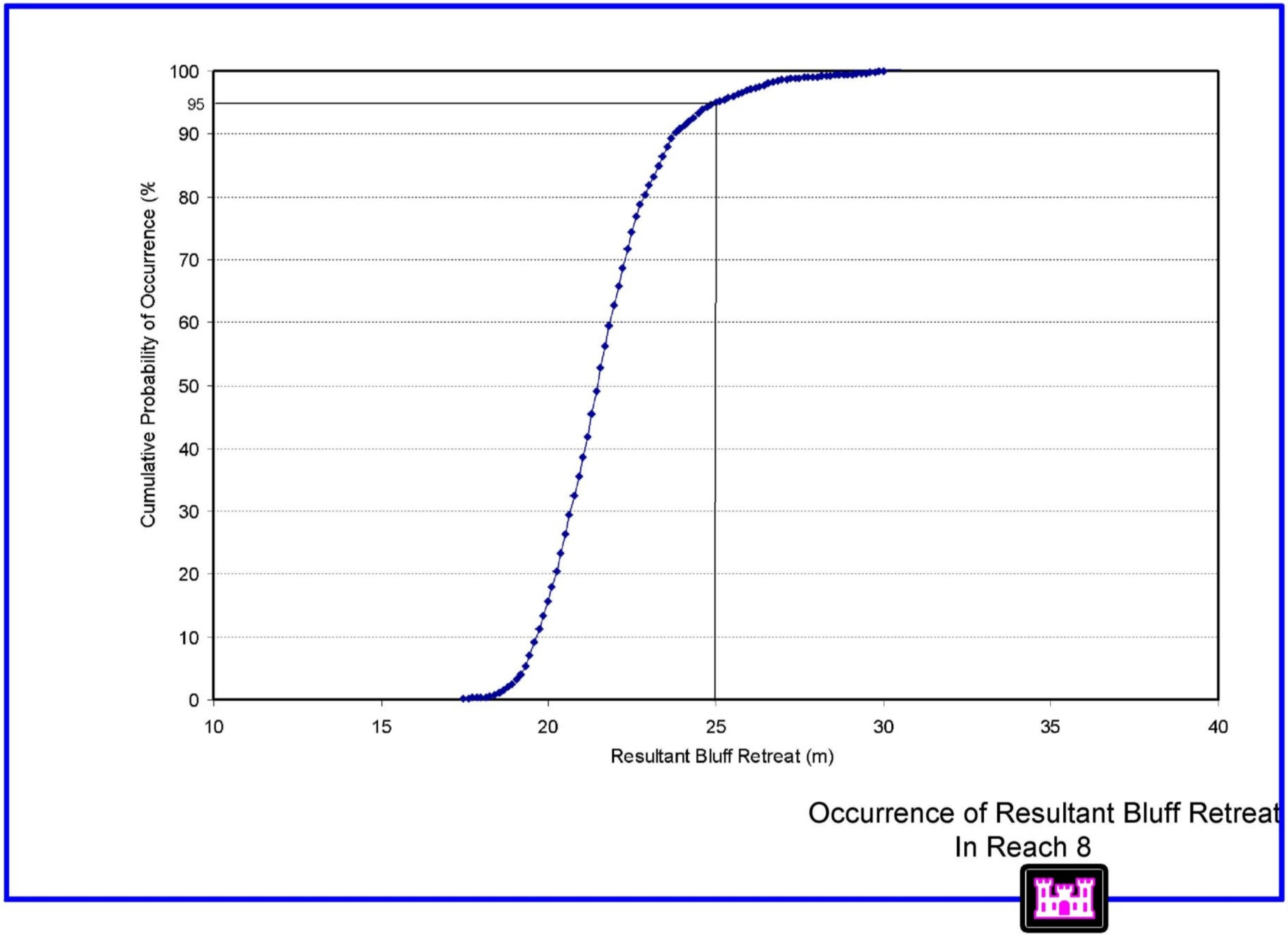
The numerical modeling that combines both the semi-empirical formulation developed by Sunamura and the Monte Carlo simulation technique enables a systematic, statistical analysis to incorporate a variety of physical variables. These include offshore wave environment, climatological changes, sea-level rise, variation in rock resistance of bluffs, the elevation of the shore platform, and the presence of transient sands or shingles that form a buffer to protect the bluff toe against wave abrasion. The significance of this Monte Carlo simulation is to allow for the characterization of each individual episodic event that closely resembles the natural process of bluff failure. The bluff retreat may occur gradually or episodically. A minor bluff failure can be immediately followed by another one with varying magnitudes over a short period. Conversely, a severe bluff retreat may require a long period for another potential bluff failure to occur when the re-eroded notch reaches the critical depth again.

Table 5.2-6 Modeled Bluff Retreat Averaged Over 1000 Simulations Under Without SLR conditions

Reach	Cumulative Bluff Retreat Over 50 years (ft)	Annualized Bluff Retreat (ft/yr)	Geologically Averaged Bluff Retreat Rate (ft/yr)*
1	0.0	0.0	0.2
2	14.1	0.3	0.3 – 0.5
3	80.4	1.6	1.2
4	44.3	0.9	1.0
5	48.6	1.0	0.2 – 0.6
6	0.3	0.007	0.1 – 1.0
7	N/A	N/A	N/A
8	83.7	1.7	0.4 – 1.2
9	92.5	1.9	0.4 – 1.2

*: from USACE-LAD, 2003

A considerable discussion, based upon the geologic morphology, is presented in **Appendix C** in estimating an annualized bluff retreat over a long-term basis. While it discusses the benefits and shortcomings of contemporary methodology used in assessing relative rates of bluff erosion, there remains a reliance on historic data, which may possibly underestimate future erosion rates. Moreover, when one attempts to assess changes in the future climate, or the effect of high sea-level rise, empirical estimates become even more tenuous. For example, Reach 1 would likely have some measurable erosion over the next 50 years, and Reach 6 may likewise experience more erosion than the numerical simulations suggest.



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2 **Figure 5.2-42 Occurrence of Resultant Bluff Retreat in Reach 8**

5.2.3.2 Simulated Results With Project Sea Level Rise

The Monte Carlo bluff retreat simulations were also carried out for the two sea level rise scenarios, including the historic trend and the high rate (NRC-III curve), respectively, to assess the potential impact of sea level rise on the predicted bluff erosion in the future. The time series of the 1,000 simulated results for the cumulative bluff top retreat distances are shown in **Figure 5.2-43** through **Figure 5.2-49** for the low SLR scenario following the historic trend. **Figure 5.2-50** through **Figure 5.2-57** show the simulated results for the high SLR scenario, based on the NRC-III curve. The time histories of the predicted mean bluff retreat over 1,000 simulations are also presented as combined figures for comparison. It is noted that the predicted bluff failure in Reach 1 will only occur for the high SLR scenario based on the NRC-III Curve projection.

A comparison of modeled results was made for the three predicted water level conditions (without SLR, low SLR following the historic trend and high SLR based on the NRC-III curve), as shown in **Figure 5.2-58** to **Figure 5.2-61** for all eight simulated reaches. It can be seen that the prediction from the NRC-III curve yields extremely large cumulative bluff retreats (e.g., exceeding 200 meters over 59 years in Reach 9), as compared to the other two scenarios. Whether it represents a realistic prediction or an overestimated model simulation is debatable. The Monte Carlo simulations were based on the assumption that the bluff base elevation is unchanged even with the continuous landward bluff retreat in the future. However, it may be reasonable to expect that the bedrock layer at the bluff base is elevated as the bluff retreats landward. Therefore, considering the uncertainty of the statistical prediction, a range of potential bluff retreat, as also shown in **Figure 5.2-58** to **Figure 5.2-61**, was estimated between the upper bound (a constant elevation at the bluff base) and the lower bound (an elevated bedrock layer approximately following the upward slope of the inshore platform slope (**Table 5.2-2**). The shady area shown in each figure can be considered as a likely range of the future bluff retreat that was predicted under the high sea level rise scenario (i.e., NRC-III curve).

5.2.4 Randomness of Wave Related Flooding

The flooding potential resulting from wave overtopping at Highway 101 within Reach 7 depends on the impinging storm waves and water levels. Wave overtopping is likely to occur during the events of large waves and high water levels. The road closures presented in **Appendix C4** were evaluated to determine the approximate nearshore oceanographic conditions (i.e., wave height and maximum tidal elevation) during each respective documented road closure. These results are presented in **Table 5.2-7**. Although there is some variability in the significant wave height, there appears to be a closer correlation between the road closures and the water levels as approximately 70 percent of the Highway 101 closures occurred during periods of elevated high tides exceeding +5.5 feet, MLLW. Furthermore, this also suggests that moderate wave conditions will have a greater wave overtopping potential under high sea levels that include sea level rise in the future (i.e., two identified sea level rise scenarios).

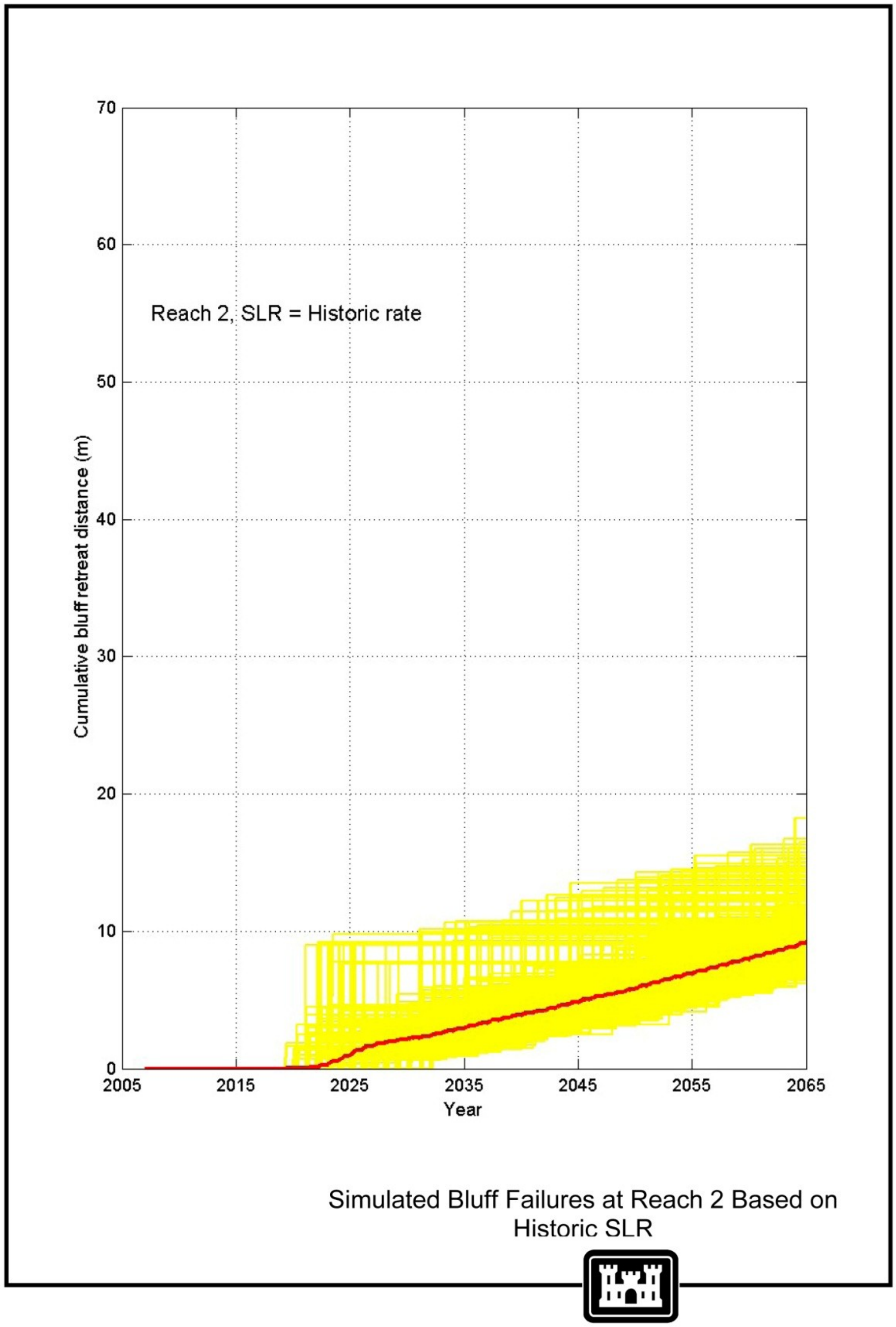


Figure 5.2-43 Simulated Bluff Failures at Reach 2 Based on Historic SLR

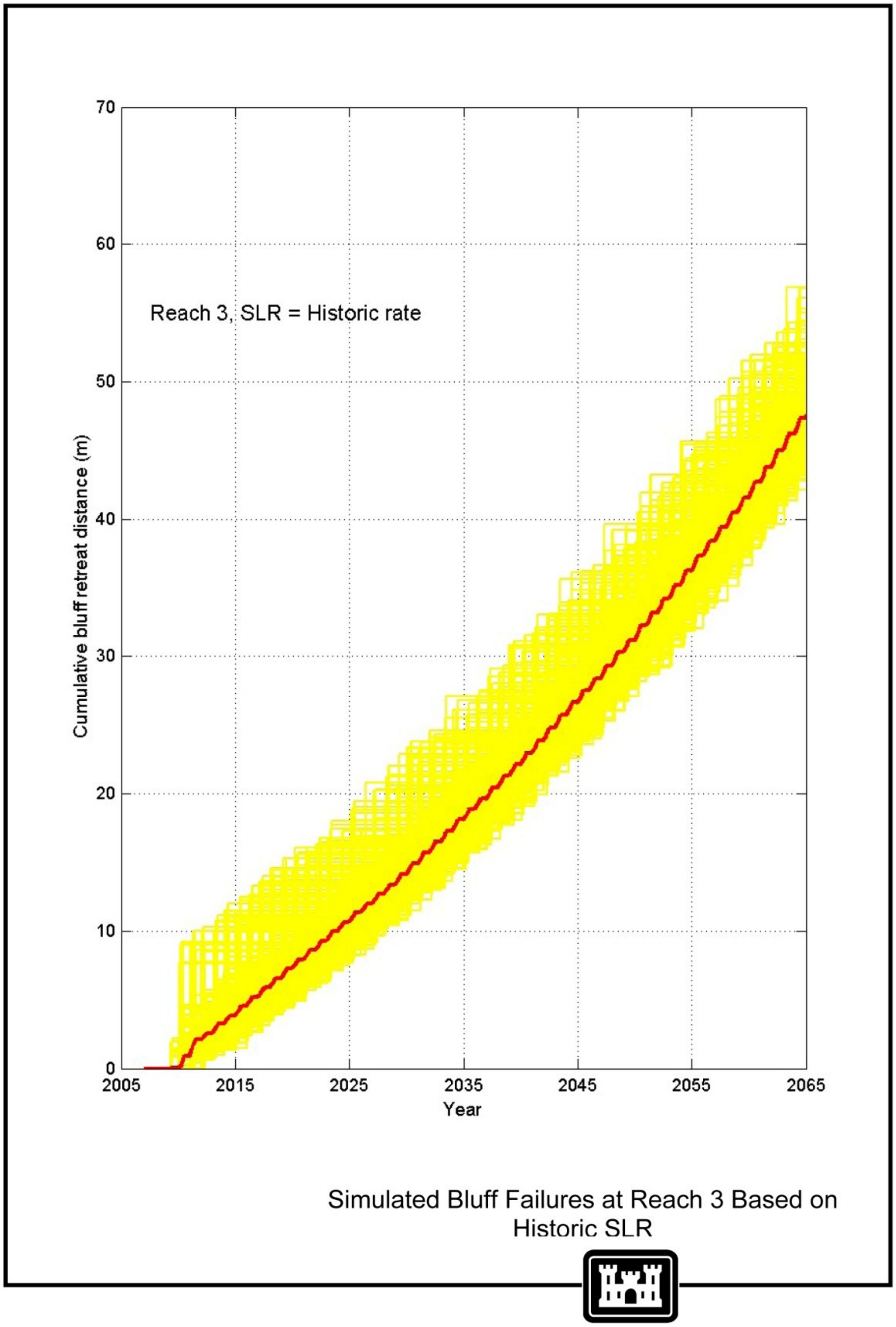


Figure 5.2-44 Simulated Bluff Failures at Reach 3 Based on Historic SLR

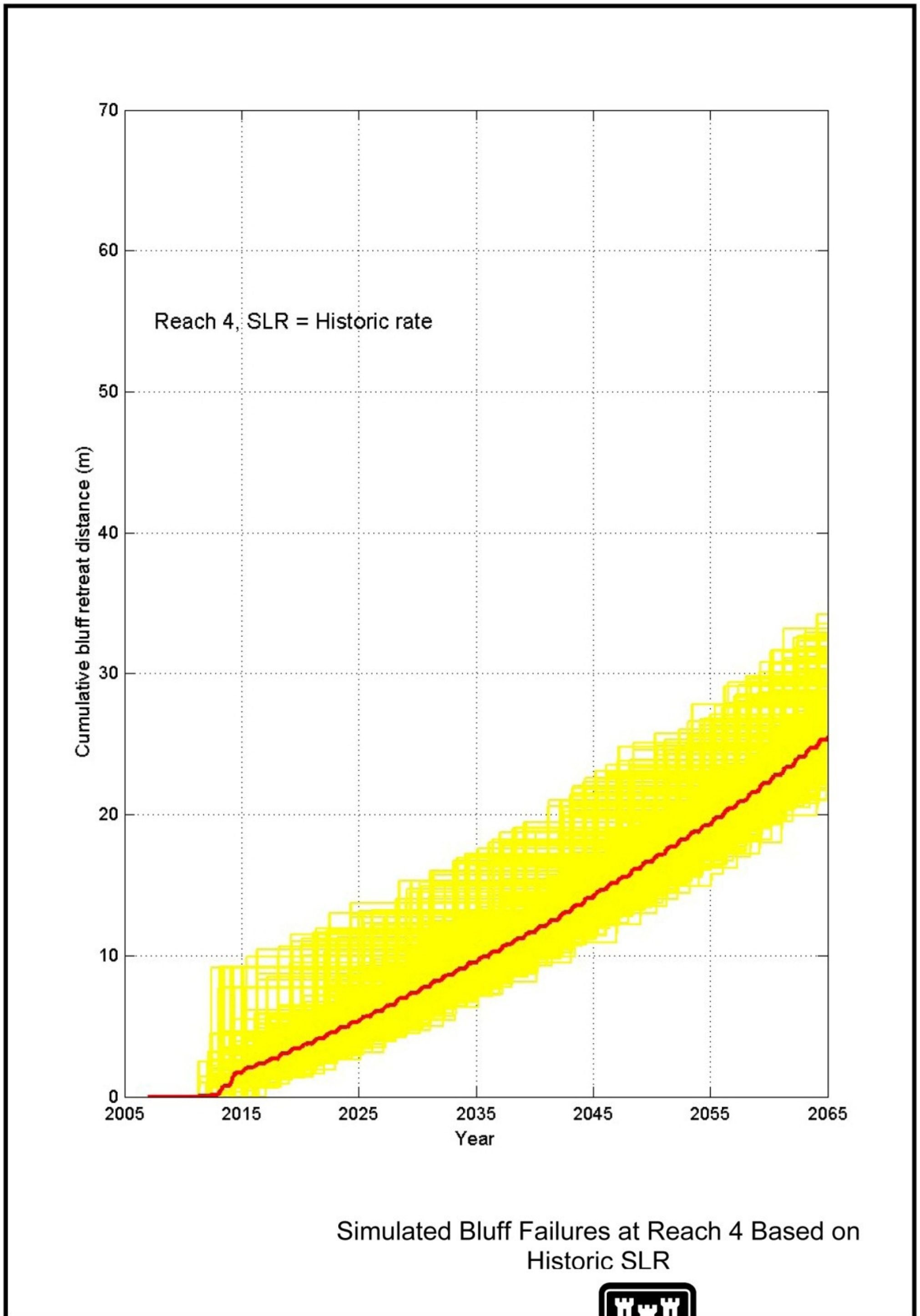


Figure 5.2-45 Simulated Bluff Failures at Reach 4 Based on Historic SLR

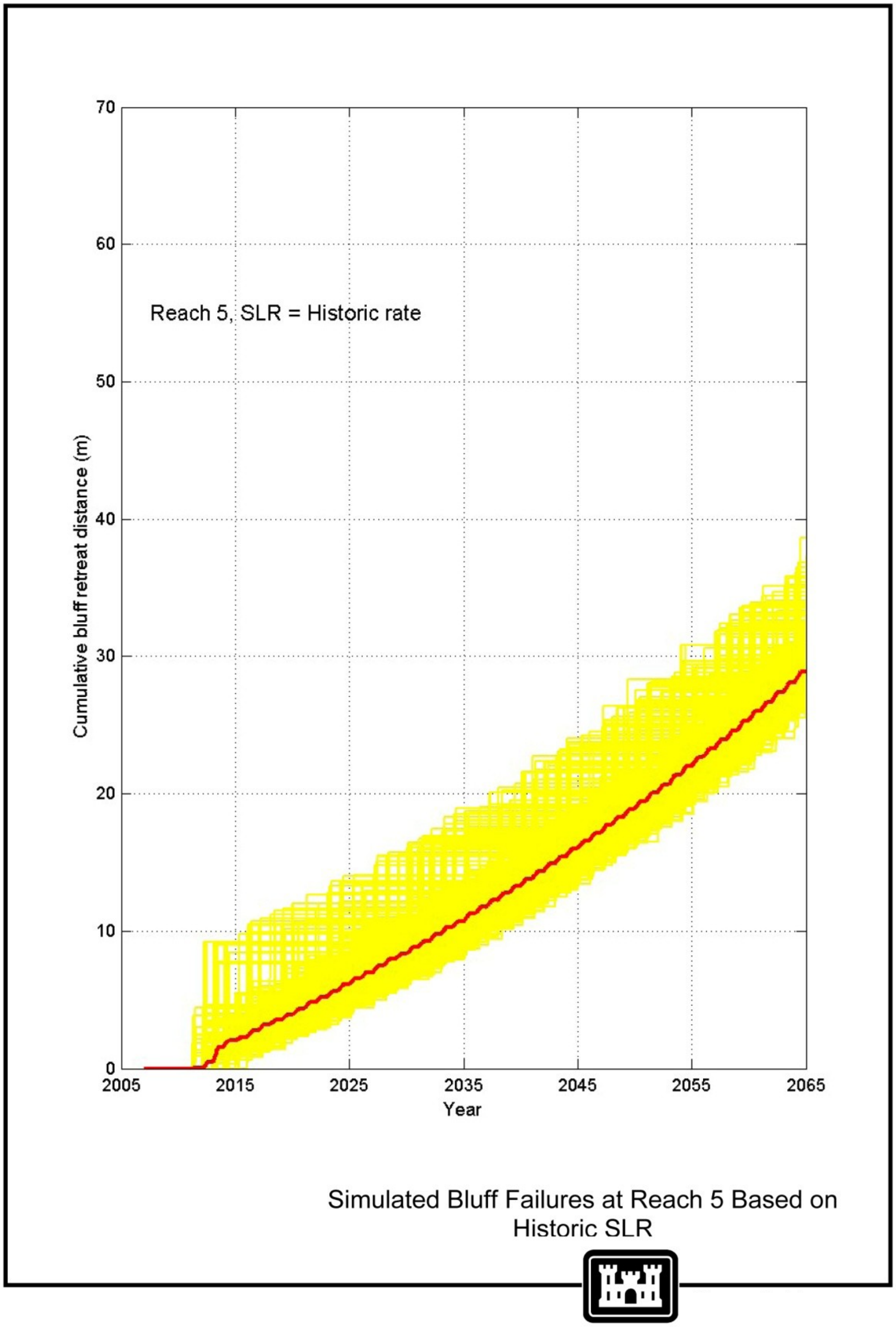


Figure 5.2-46 Simulated Bluff Failures at Reach 5 Based on Historic SLR

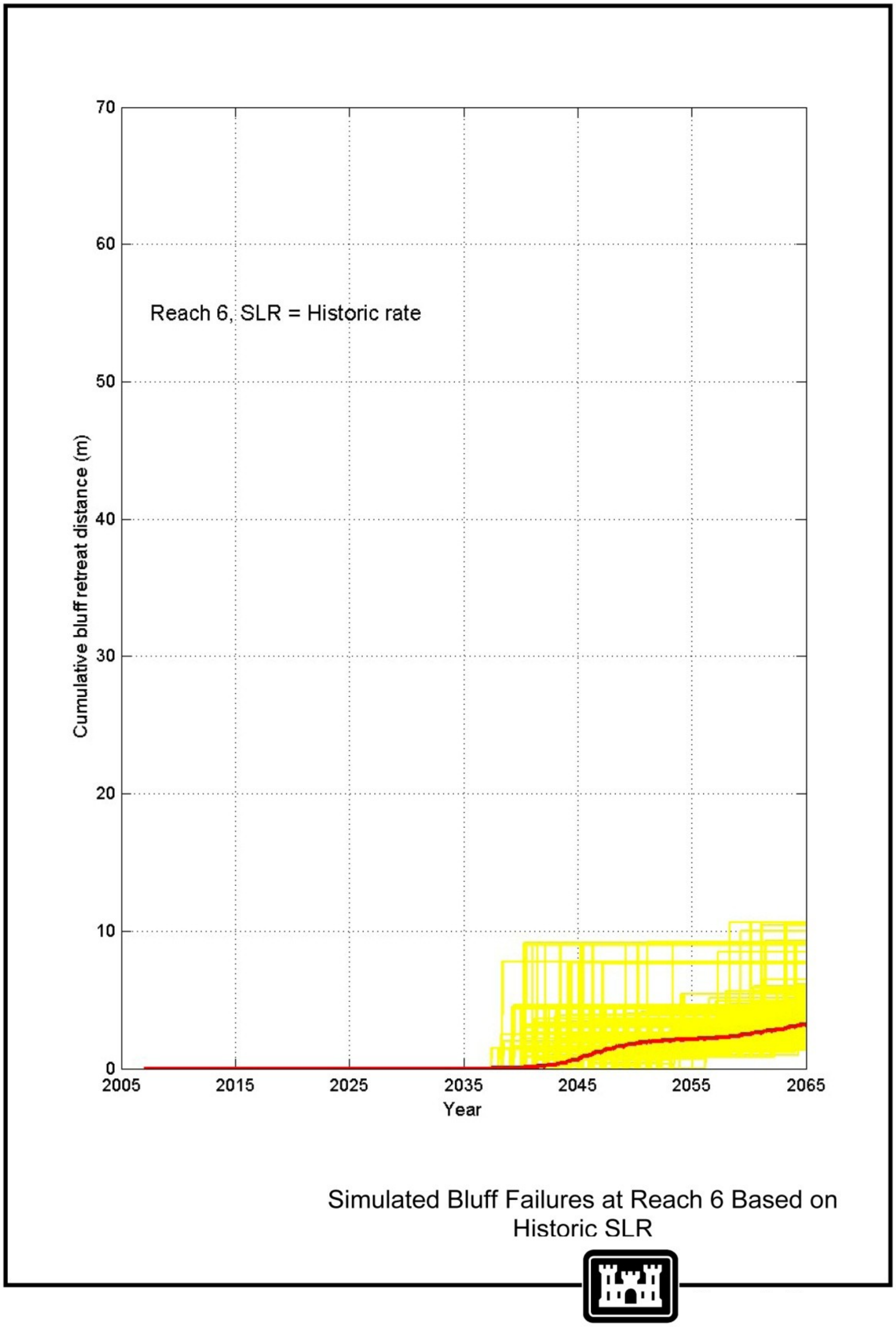
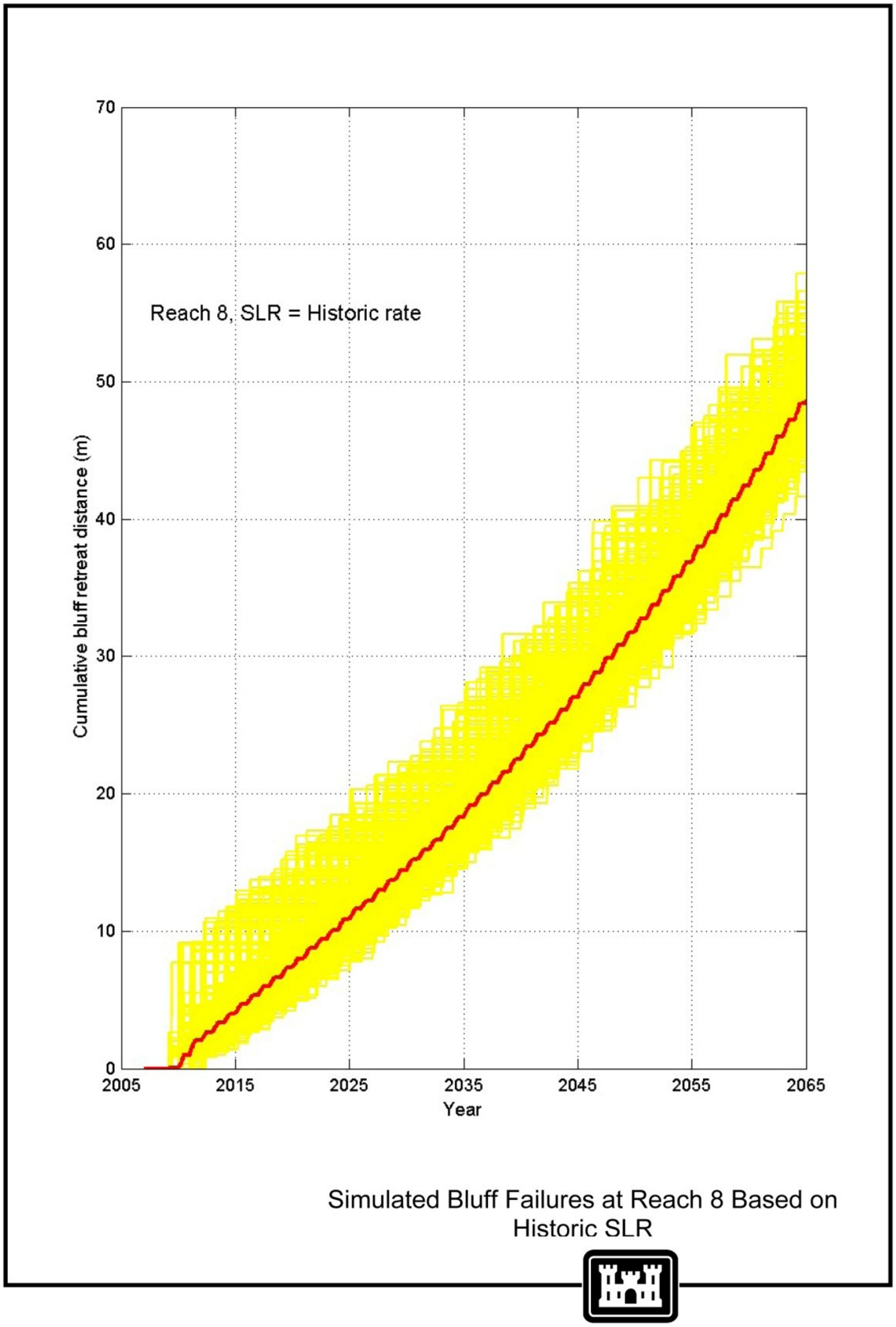


Figure 5.2-47 Simulated Bluff Failures at Reach 6 Based on Historic SLR



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2 **Figure 5.2-48 Simulated Bluff Failures at Reach 8 Based on Historic SLR**

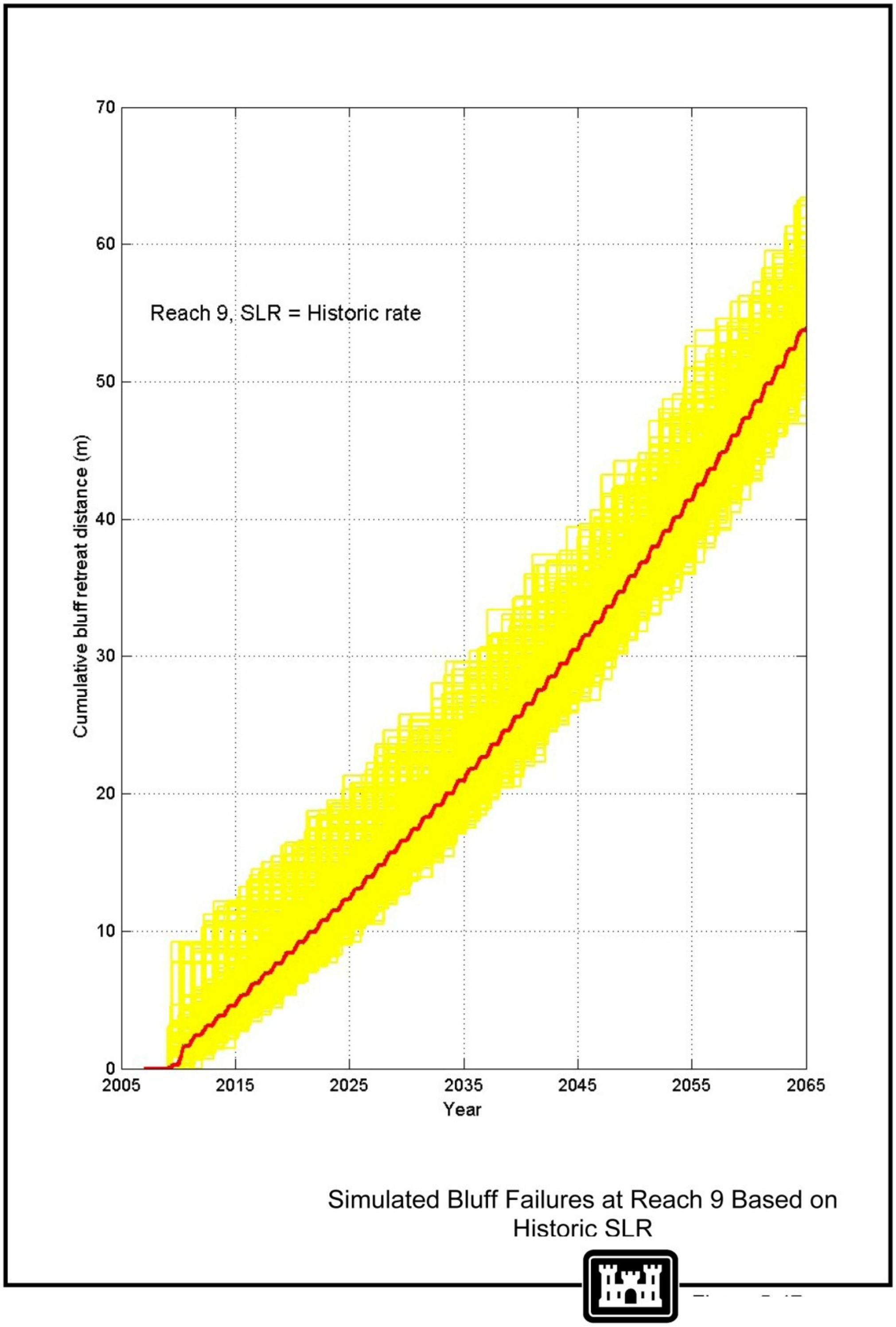


Figure 5.2-49 Simulated Bluff Failures at Reach 9 Based on Historic SLR

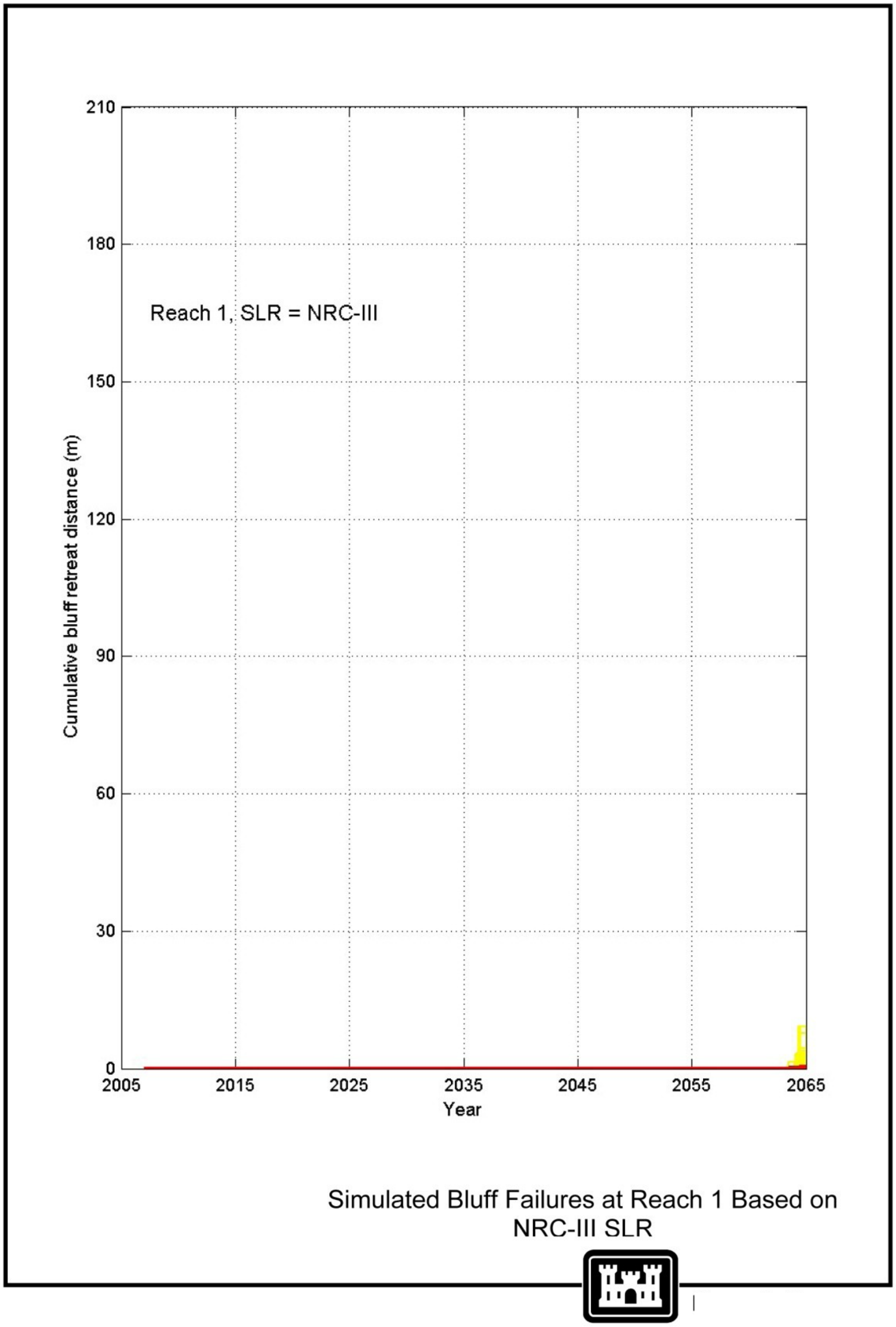


Figure 5.2-50 Simulated Bluff Failures at Reach 1 Based on NRC-III SLR

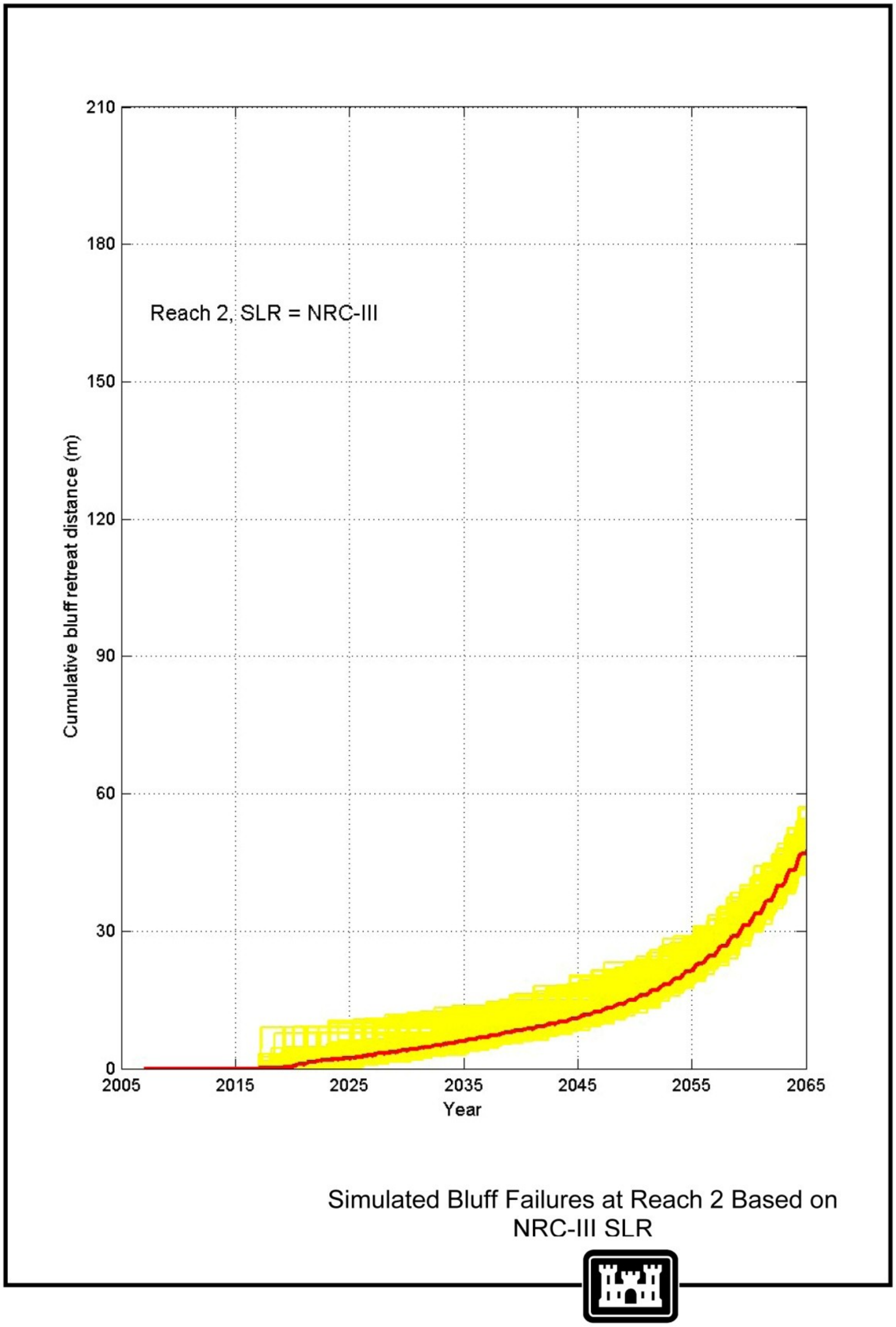
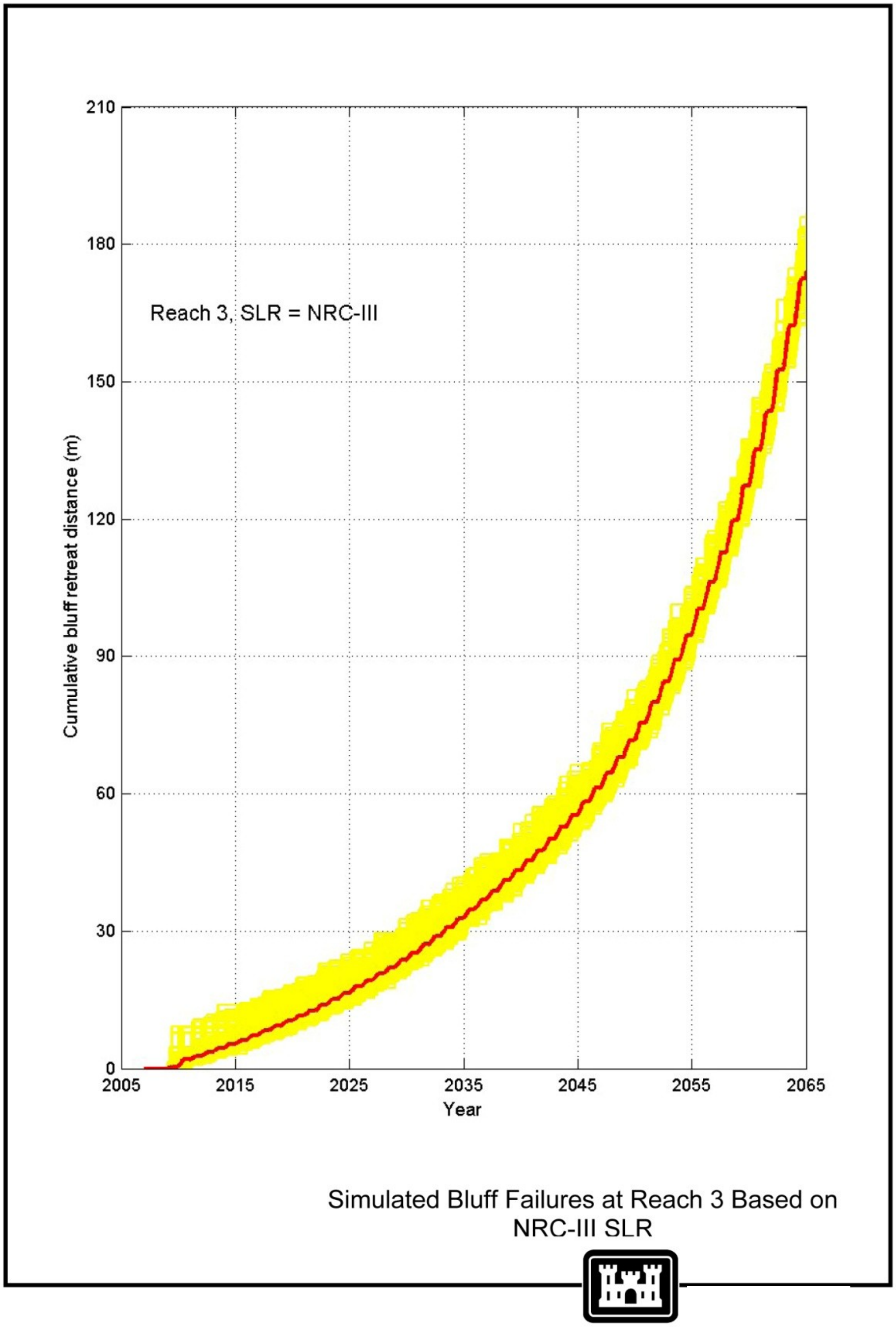


Figure 5.2-51 Simulated Bluff Failures at Reach 2 Based on NRC-III SLR



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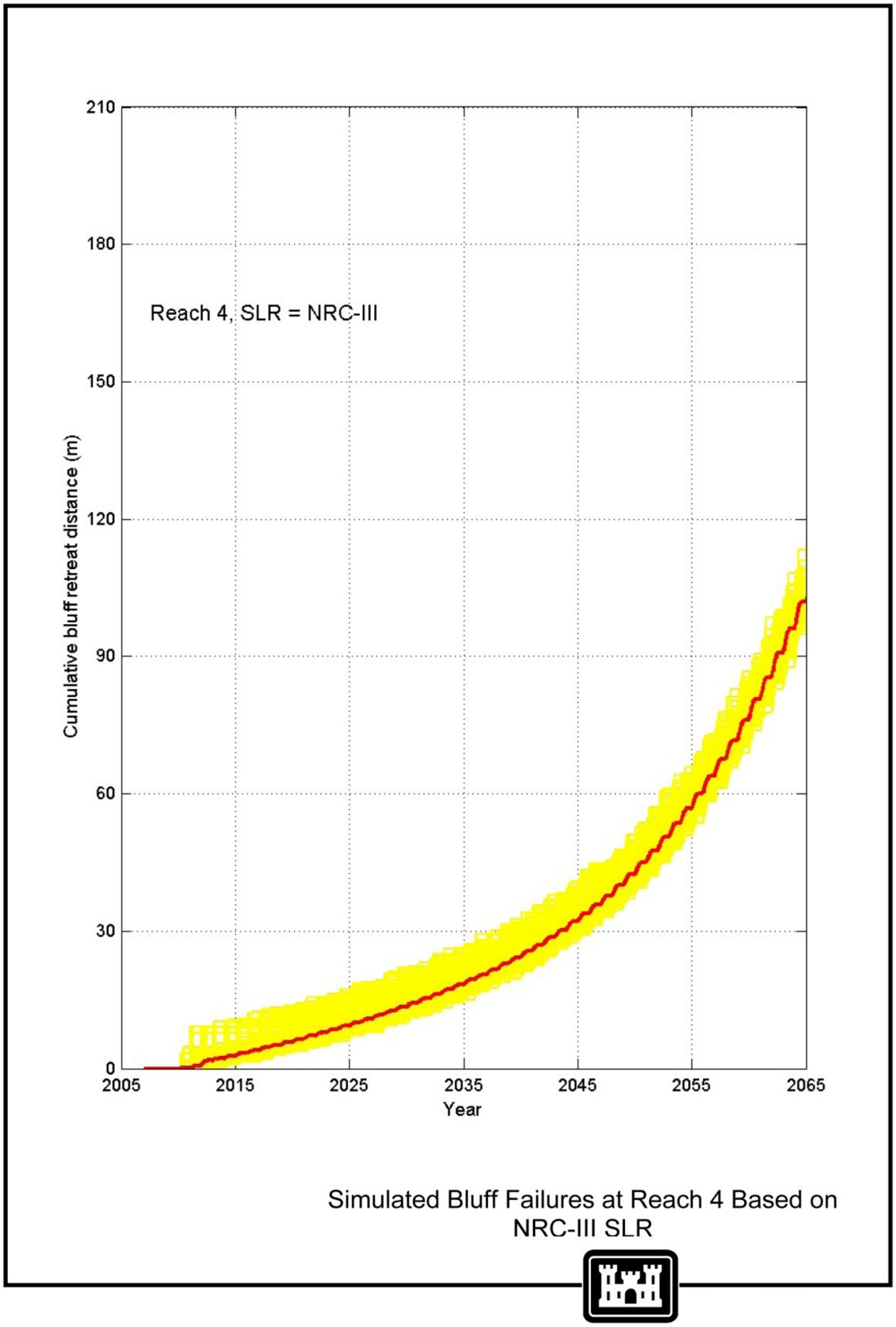
2 **Figure 5.2-52 Simulated Bluff Failures at Reach 3 Based on NRC-III SLR**



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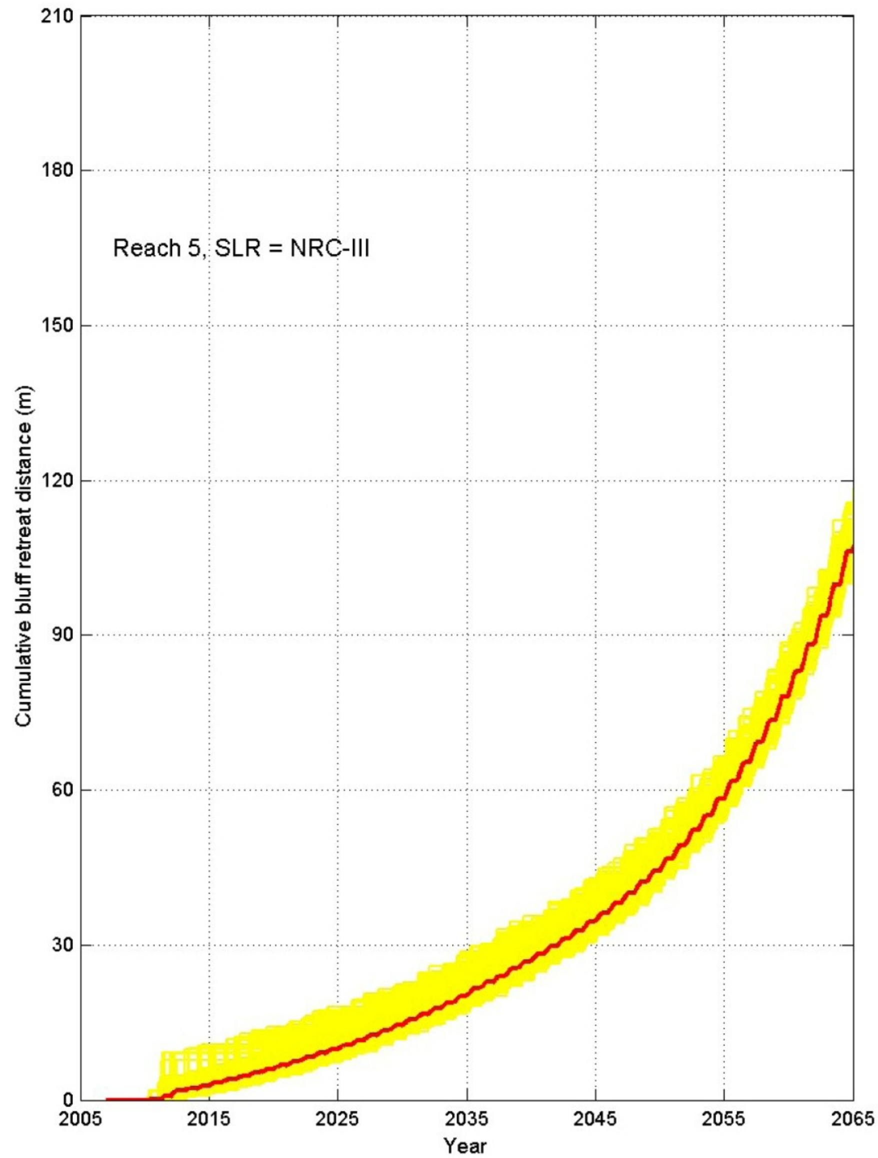
2 **Figure 5.2-53 Simulated Bluff Failures at Reach 4 Based on NRC-III SLR**



Simulated Bluff Failures at Reach 5 Based on NRC-III SLR

Figure 5.2-54 Simulated Bluff Failures at Reach 5 Based on NRC-III SLR

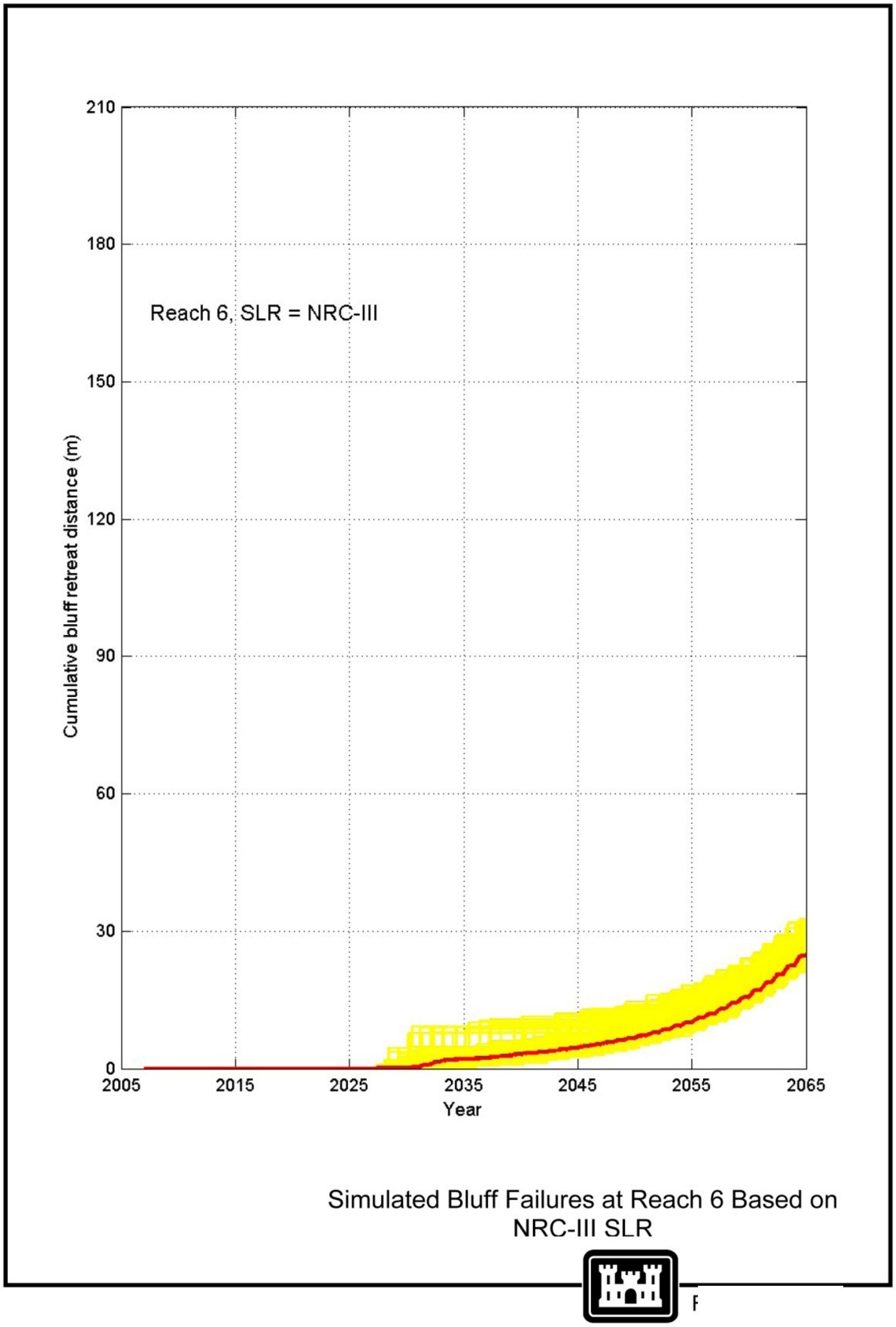


Figure 5.2-55 Simulated Bluff Failures at Reach 6 Based on NRC-III SLR

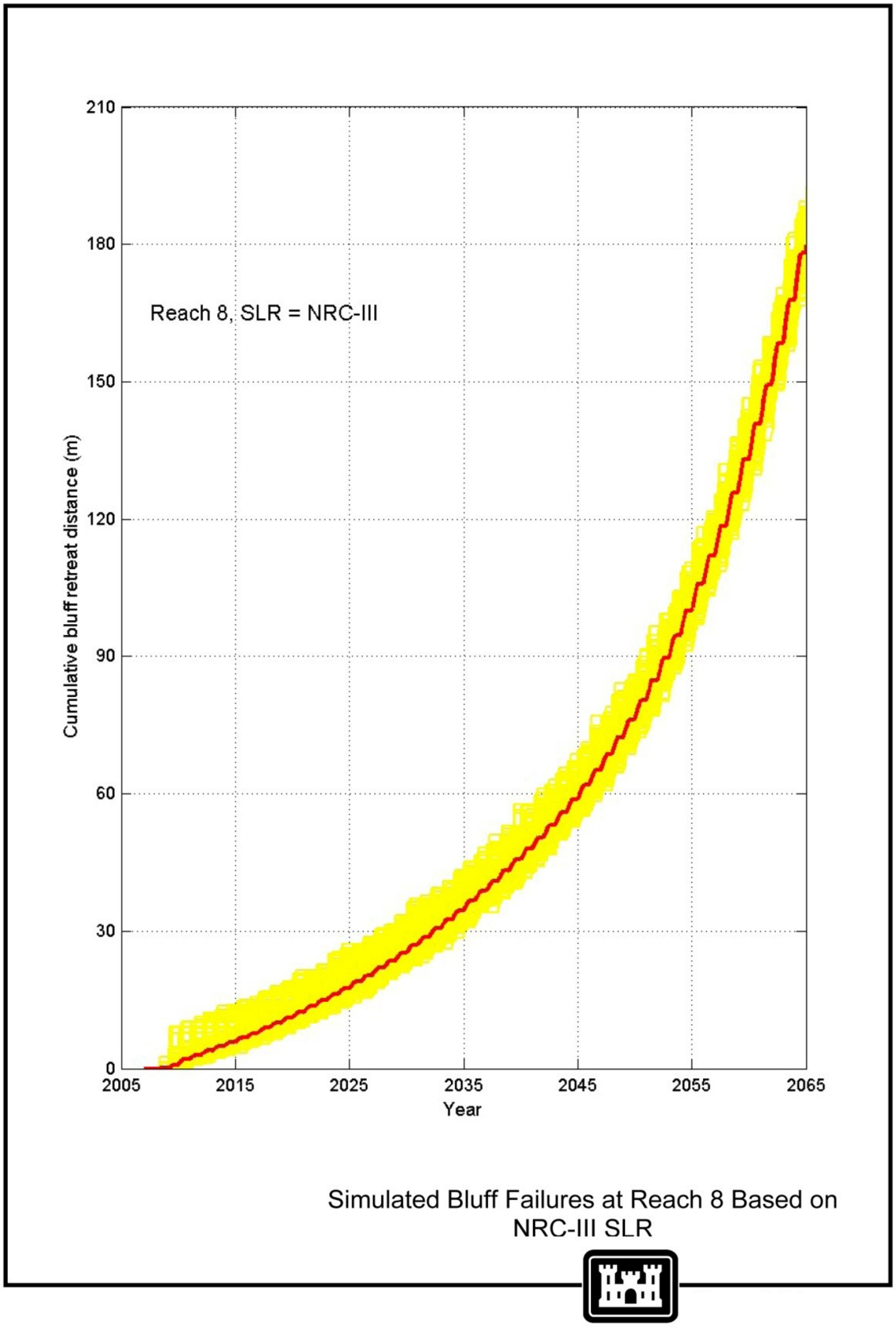


Figure 5.2-56 Simulated Bluff Failures at Reach 8 Based on NRC-III SLR

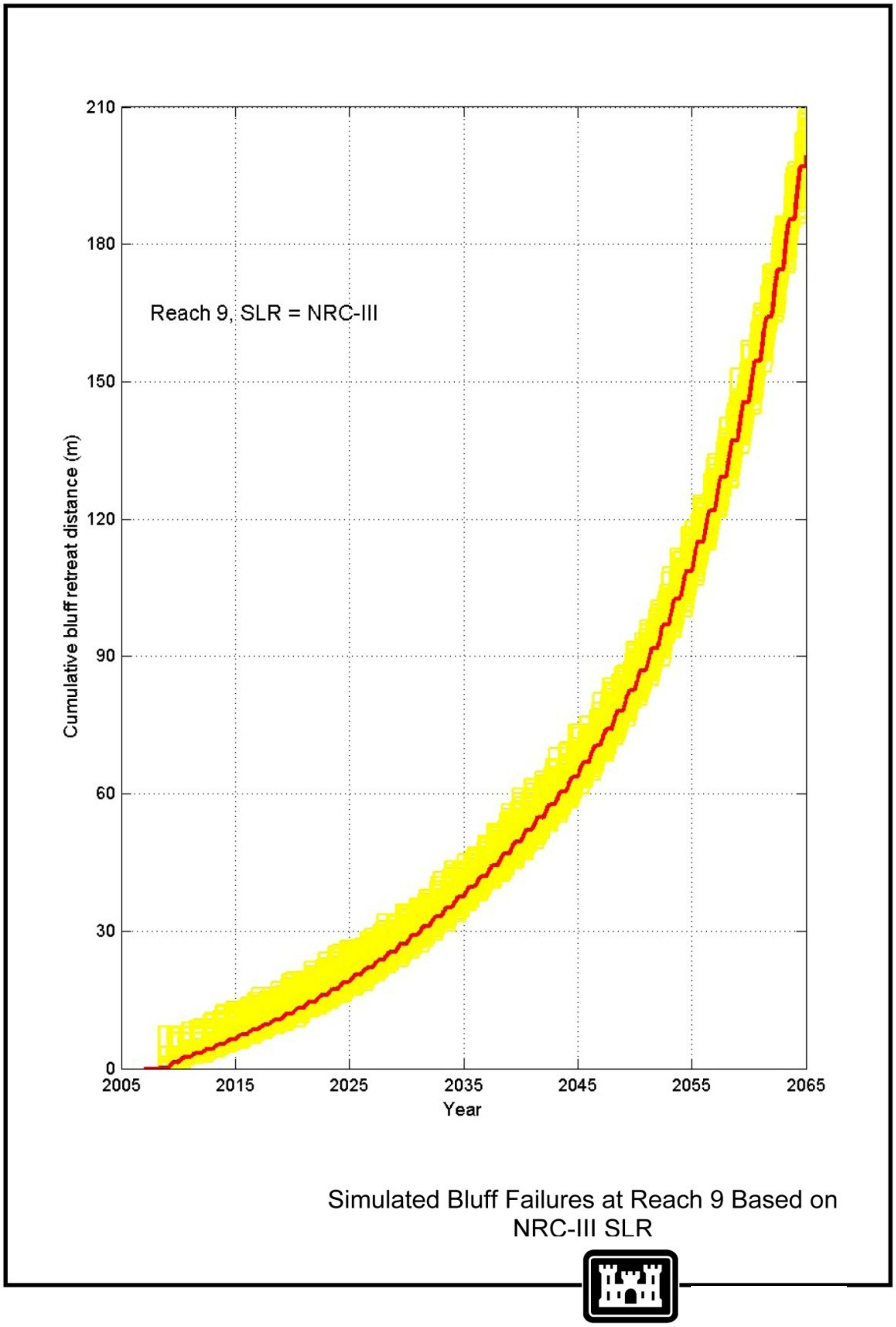


Figure 5.2-57 Simulated Bluff Failures at Reach 9 Based on NRC-III SLR

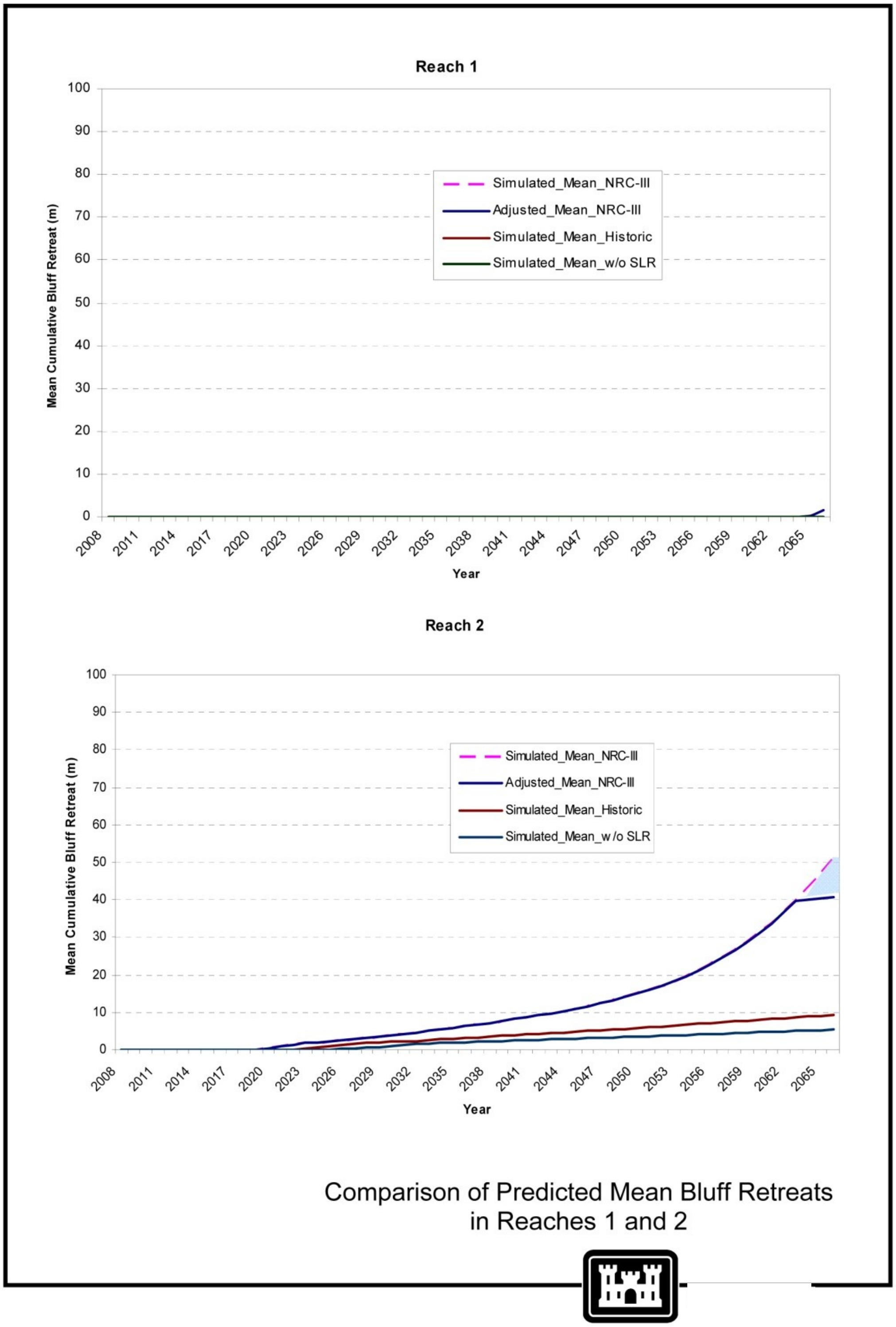


Figure 5.2-58 Comparison of Predicted Mean Bluff Retreats in Reaches 1 and 2

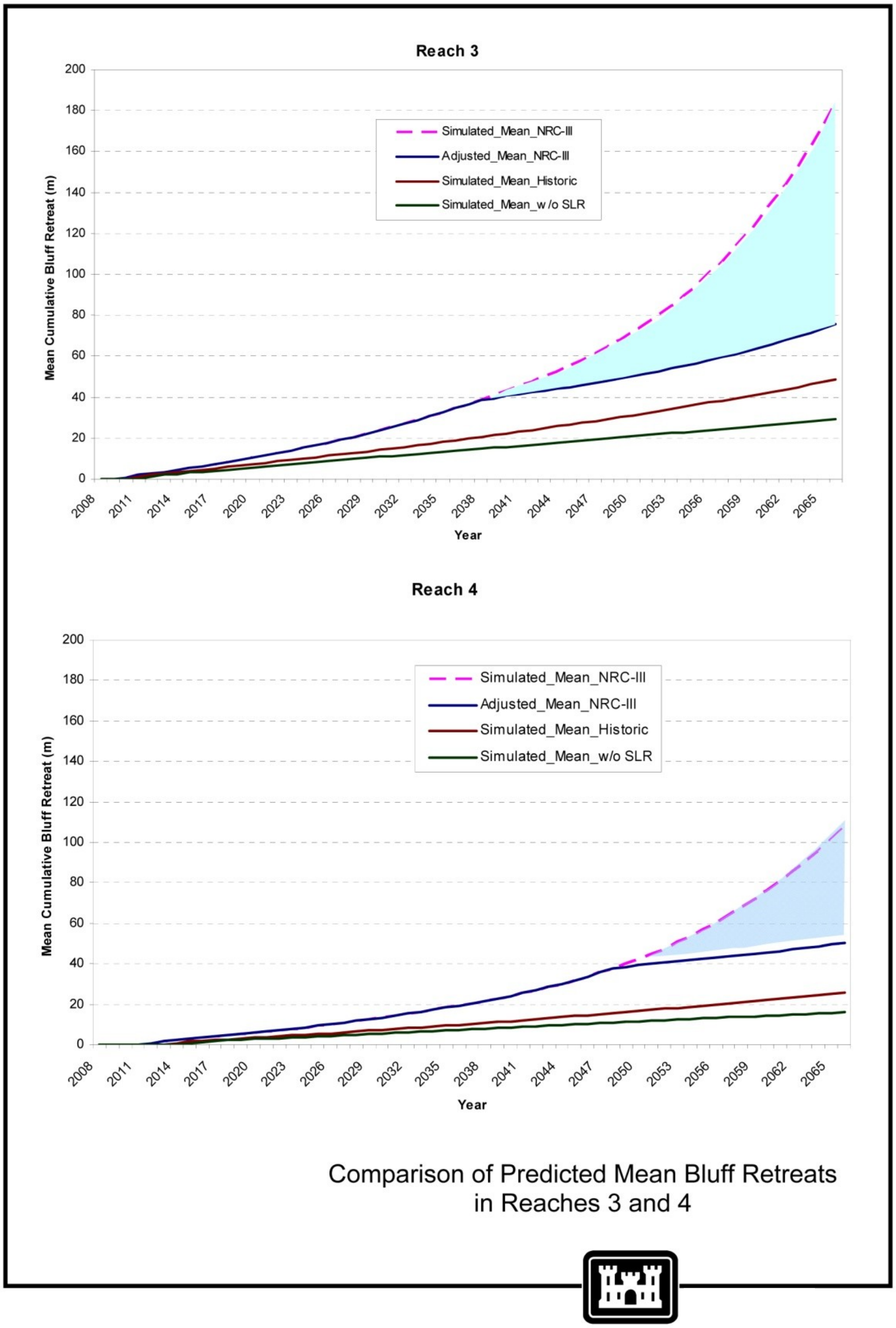


Figure 5.2-59 Comparison of Predicted Mean Bluff Retreats in Reaches 3 and 4

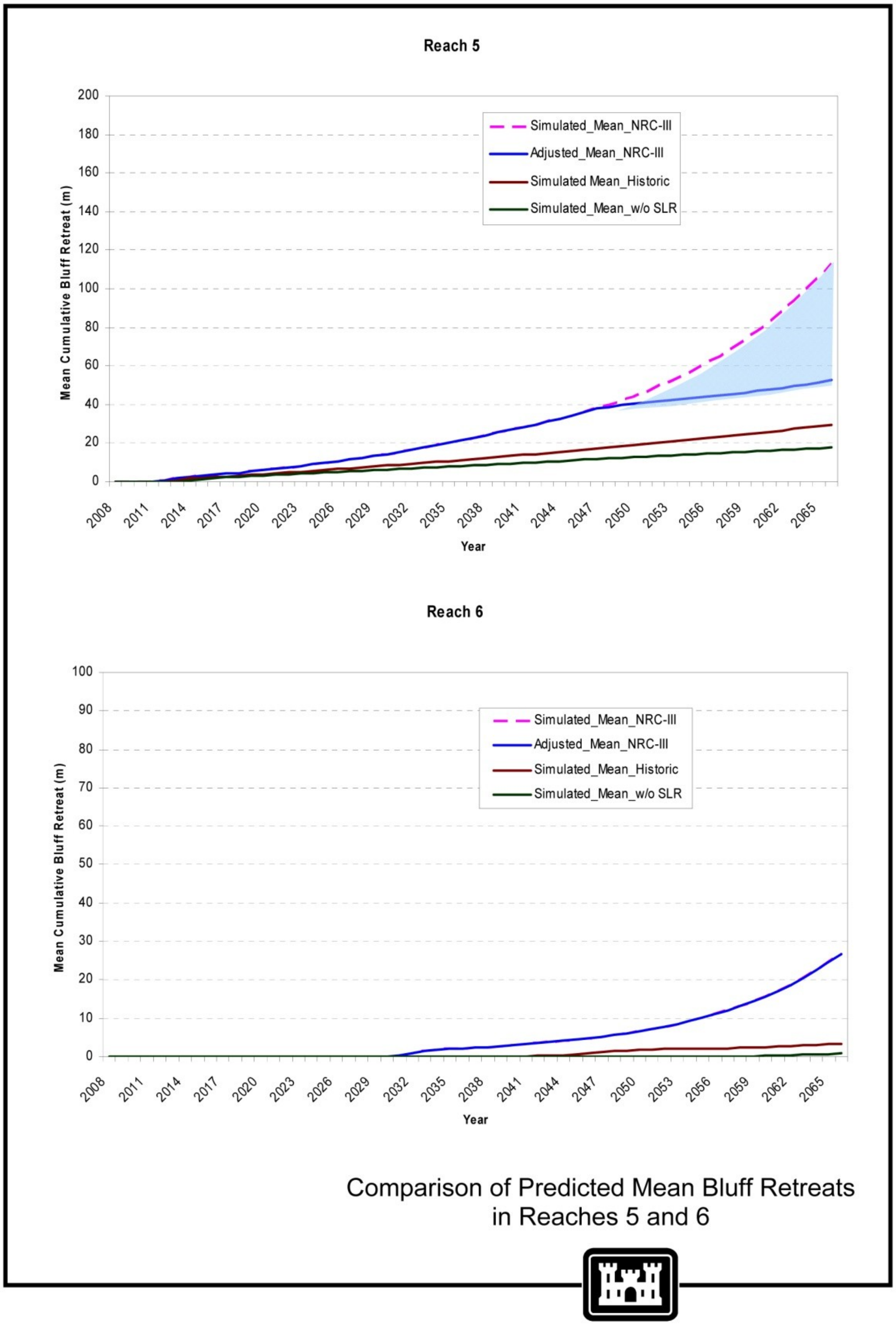


Figure 5.2-60 Comparison of Predicted Mean Bluff Retreats in Reaches 5 and 6

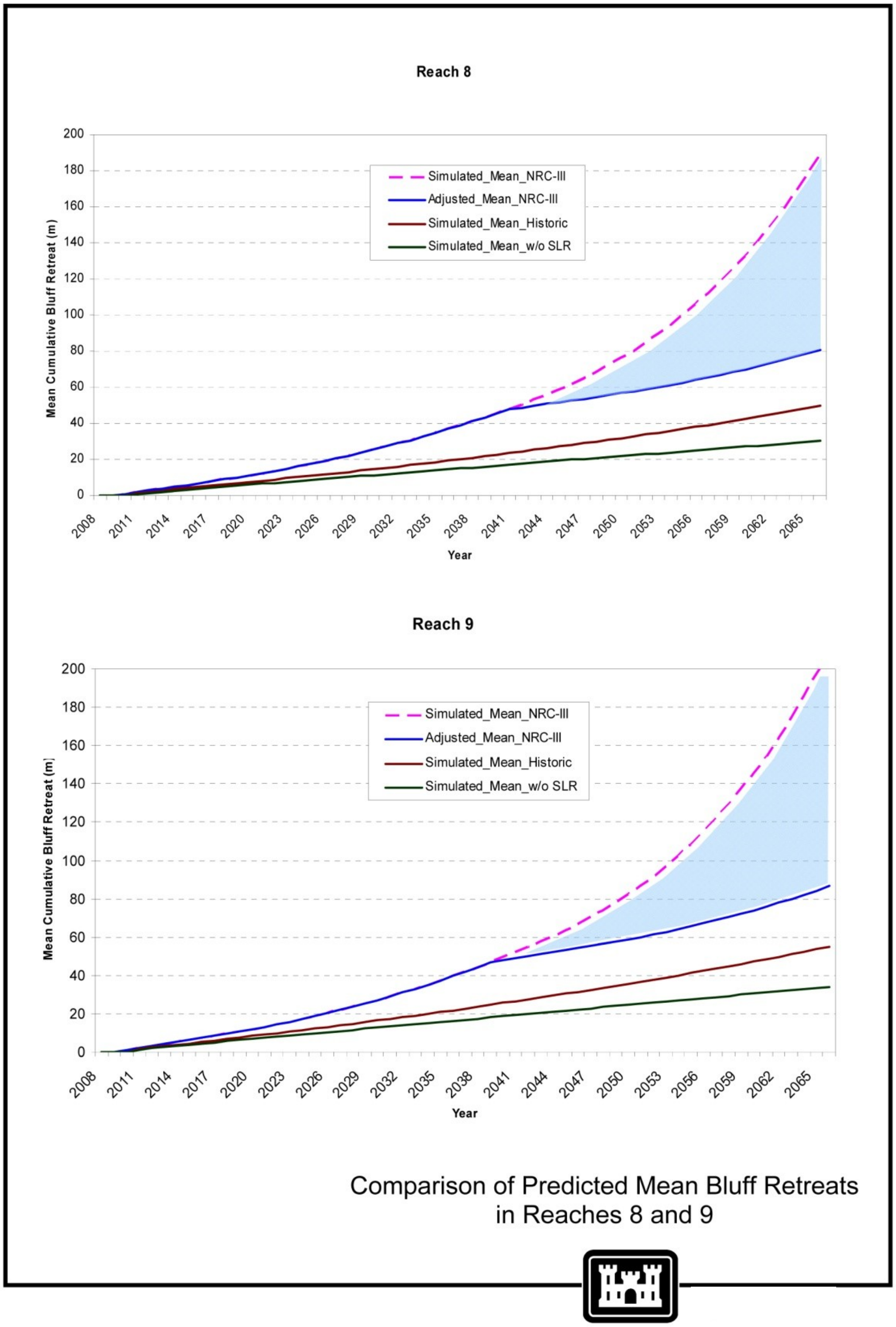


Figure 5.2-61 Comparison of Predicted Mean Bluff Retreats in Reaches 8 and 9

Table 5.2-7 Approximate Oceanographic Conditions during HWY 101 Reach 7 Road Closures

Date	Duration of Closure (hrs)	Wave Height, H_s (ft)	Maximum Tidal Elevation, (ft), MLLW
1/22/88	40.0	16.4	6.92
3/1/91	8.0	10.8	6.23
3/12/92	4.0	4.6	5.05
3/25/92	4.0	4.6	5.31
11/3/92	5.0	3.0	6.56
1/18/93	6.0	10.5	6.36
1/19/93	5.0	10.5	6.43
2/6/93	3.0	8.9	6.79
3/3/93	8.0	3.0	4.72
12/13/93	3.2	5.2	6.82
2/7/94	1.0	6.2	6.17
9/30/94	4.0	2.6	5.22
1/2/95	2.5	4.3	6.96
1/3/95	2.5	10.5	5.41
1/12/95	7.0	12.8	5.54
1/13/95	2.5	12.5	5.68
1/16/95	3.0	7.2	5.97
1/23/95	2.5	8.2	5.22
1/24/95	2.5	8.2	5.28
1/30/95	10.5	3.3	6.59
2/3/95	4.0	14.1	5.38
2/9/95	7.0	6.6	4.86
12/17/95	2.5	8.5	5.38
12/19/95	8.0	8.5	6.40
12/20/95	12.0	8.5	7.02
12/20/95	5.0	8.5	7.02
12/22/95	15.0	8.5	7.25
1/1/96	4.0	3.9	5.61
2/7/96	2.5	7.9	5.09
2/18/96	3.0	7.5	6.63
10/26/96	1.0	3.6	6.53
2/6/97	3.0	3.6	6.76
2/7/95	3.5	6.6	6.69
2/21/97	4.0	4.3	5.48
10/30/97	13.0	1.3	6.69
11/12/97	3.0	4.6	7.71
11/13/97	2.0	4.6	7.84
11/14/97	1.0	5.2	7.71
11/15/97	3.0	5.9	7.35
11/28/97	3.5	7.2	6.89
12/4/97	4.0	9.2	6.86

Since storm water levels combining the astronomical tides with wave-induced setup vary during a storm event, the peak waves of a severe storm impinging onto the Cardiff shoreline (Reach 7) can coincide with water levels ranging from high to low. The maximum wave runup elevations for two storm events with the same intensity can be vastly different, depending on the resultant water levels at the time when the storm waves arrive. Therefore, a probabilistic representation on wave overtopping scenarios is presented in this analysis. Subsequently, the economic analysis for road closures can be deduced through a random process using a similar Monte Carlo Simulation technique. In the following sections, return storm waves, frequency occurrences of various water levels, and wave runup calculations are addressed to characterize the wave-related flooding (induced by large waves and high water levels) at Highway 101 within Reach 7.

<u>Return Storm Wave Heights</u>

The selected extreme extratropical wave events that were hindcasted to the nearshore coastal zone in the Encinitas and Solana Beach study area (**Table 3.3-1**) were statistically analyzed to determine their respective extreme recurrence intervals.

The Automated Coastal Engineering System (ACES), developed by the Corps of Engineers (USACE, 1992), was employed to perform the extreme significant wave height analysis. This application provides significant return wave height estimates of 2, 5, 10, 25, 50, and 100 years for a given input data array of extreme significant wave heights. The ACES program utilizes the approach developed by Goda (1988) to fit five candidate probability distributions. The candidate distribution function chosen to best represent the extreme return wave heights impacting the Cardiff shoreline (Reach 7) is a Weibull distribution with an exponent value of 1.0. **Table 5.2-8** presents the estimated representative extreme return wave heights for the selected extratropical storms, as presented in **Table 3.3-1**.

Table 5.2-8 Estimated Extreme Return Wave Heights for Extratropical Storms (Reach 7)

Depth ft, (MLLW)	Extreme Return Significant Wave Heights, in feet					
	2 -yr	**5-yr**	**10-yr**	**25-yr**	**50-yr**	**100-yr**
32.5	12.5	15.1	17.4	20.3	22.3	24.6

The largest hindcasted nearshore wave height of 22.6 feet occurred during the March 1983 storm event. According to the return storm wave heights presented in **Table 5.2-8**, the March 1983 storm event has a return frequency of approximately once every 50 years. The El Nino season of 1983 is typically considered to meet the coastal and oceanographic design criteria. However, it is important to note that this is due primarily to the severity of various clustering storm events that impacted the southern California coast during the 1982-1983 period, which resulted in extreme high water elevations and high beach erosion in addition to large nearshore wave heights.

The analysis conducted for this study includes return wave heights for storms recurring over the 2, 5, 10, 25, 50, and 100-year period, as presented in **Table 5.2-8**. However, large return waves typically break in deeper water zones, which effectively increases the distance that the broken waves must travel before impinging upon the shoreline. The larger the wave height is, the farther offshore waves will break. For this reason, an additional forced wave breaking condition was also evaluated to account for waves breaking close to the toe of the non-engineered riprap revetment.

5.2.4.1 Storm Water Levels

The prevailing tidal characteristics exhibited within the project site are presented in **Table 3.2-1**. For the purposes of analyzing the exposure of Reach 7 to wave-induced inundation, the tidal elevations measured at the Scripps Pier in La Jolla were quantified for a 23-year period, extending from 1979 to 2001 (i.e., the same period of wave hindcast). The tides observed at the Scripps Pier NOS Tidal Station are considered to be the representative tidal characteristics within the Cardiff coastal zone. The tidal records were analyzed to determine the duration of a particular tidal range (e.g., between +1.00 and +1.25 meters, MLLW) within the entire period of record. The percentage of each tidal range occurrence is presented in **Table 5.2-9**.

Table 5.2-9 Percentage of Tidal Elevation Occurrences for Cardiff Coastal Zone (Reach 7)

Tidal Elevation Range (m, MLLW)	Percentage of Occurrence [%]
< 0.0	4.7
0.0 – 0.25	8.2
0.25 – 0.50	11.1
0.50 – 0.75	15.4
0.75 – 1.00	20.2
1.00 – 1.25	18.0
1.25 –1.50	12.2
1.50 –1.75	6.7
1.75 – 2.00	2.8
2.00 – 2.25	0.6
2.25 – 2.50	<0.1
>2.50	0

Typical maximum storm surge on the order of 0.3 to 0.5 feet in the San Diego region is insignificant as compared to the wave-induced setup (USACE-LAD, 1991). Use of the measured tides at the Scripps Pier station from 1979 to 2001 automatically takes into account, to a certain extent, the effect of storm surge during a storm event within this measured period that includes the 1982-1983 and 1997-1998 El Nino seasons. In this analysis, storm water levels were computed from the measured astronomical tidal elevations superimposed by wave-induced setup depending on the intensity of each storm event. Wave-induced setup were computed for various return storm wave heights and their corresponding breaking wave condition in accordance with the formulations presented in the Coastal Engineering Manual (USACE, 2002). The formulations were based upon the variation of the radiation stress varying within the surf zone. **Table 5.2-10** presents the estimated wave setup for various return wave heights as listed in **Table 5.2-8**.

Table 5.2-10 Estimated Return Wave Setups

Return Frequency (yrs)	Estimated Wave Setup (ft)
2	1.6
5	1.9
10	2.1
25	2.3
50	2.5
100	2.8

5.2.4.2 Wave Runup Analysis

In order to determine the maximum wave runup elevations impacting the Highway 101 corridor, it was assumed that the storm waves attack on a pre-existing eroded profile. Transect (SD-625) located almost directly in the mid-section of the Cardiff shoreline was chosen to best represent the beach profile characteristics seaward of Highway 101. The previous City of Encinitas and SANDAG sponsored surveys at Station SD-625 (**Figures C1-31** of **Appendix C1**) and two additional beach profile surveys, C6 and C7, as defined by the City of Encinitas during the Feasibility Study and Conceptual Plan for the Relocation of the San Elijo Lagoon Inlet (Coastal Environments, 2001), were chosen to determine the eroded storm beach profile.

Based on the three available depleted spring profiles, the historical information regarding storm scour at this particular site, and the known geomorphologic characteristics adjacent to, and seaward of the San Elijo Lagoon, the design scour elevation within the Cardiff (Reach 7) coastal segment was estimated to be approximately -1.0 feet, MLLW. As evidenced in the SD-625 surveys, the non-engineered riprap revetment that protects Highway 101 maintains an average slope of 4 to 1 (horizontal to vertical) and terminates at the shoulder of the roadway/bike lane at an average elevation of +17.7 feet, MLLW. The inshore slope extending from -1.0 to -6.0 feet, MLLW is approximately 80 to 1 (horizontal to vertical). Seaward of -6.0 feet, MLLW, the offshore slope is approximately 40 to 1. The eroded scour profile employed during the course of this wave runup analysis is illustrated in **Figure 5.2-62**.

A wave runup analysis was performed to assess the future without-project vulnerability of Highway 101 to wave-induced inundation and projectile debris resulting from hazardous storm events of varying frequencies. The design criteria described and detailed above were imported into the “WRUP” computer program, developed by Noble Consultants, Inc., to calculate the wave runup elevations. The technical methodology that WRUP employed is strictly based on the equations, curves and methods contained within the Shore Protection Manual (SPM) and its referenced publications (USACE, 1984).

Wave runup simulations were executed for significant nearshore wave heights associated with return extratropical cyclonic storms events ranging from 2 to 100 years, as well as a forced breaking wave condition, with wave periods ranging from 14 to 20 seconds. The design water level elevations ranged from +3.0 to +9.0 feet, MLLW and were incrementally increased by 0.25 meters for each significant wave height simulation. This exercise was performed to assess the potential exposure duration of Highway 101 during extreme return storm events.

5.2.4.3 Randomness of Wave Overtopping

Based upon the depth-limited breaking wave criteria and various return wave conditions, the wave runup computations indicate that waves will overtop the protected revetment at a minimum storm water level of +6.6 feet, MLLW. During tide levels of +6.6 ft MLLW concurrent with depth limiting wave conditions, the roadway will experience overtopping. Accounting for the storm wave setup as described in **Section 5.2.3**, storm waves overtopping Highway 101 at an elevation of approximately +17.5 feet, MLLW would vary in accordance with different astronomical tidal levels under varying return storm wave conditions. **Table 5.2-11** presents the deduced minimum tidal elevations for the analyzed return storm events to result in Highway 101 wave overtopping. For example, under a 5-year return storm event, the non-engineered revetment will be overtopped during the period in which the tide levels are higher than the elevation at +4.7 ft, MLLW.

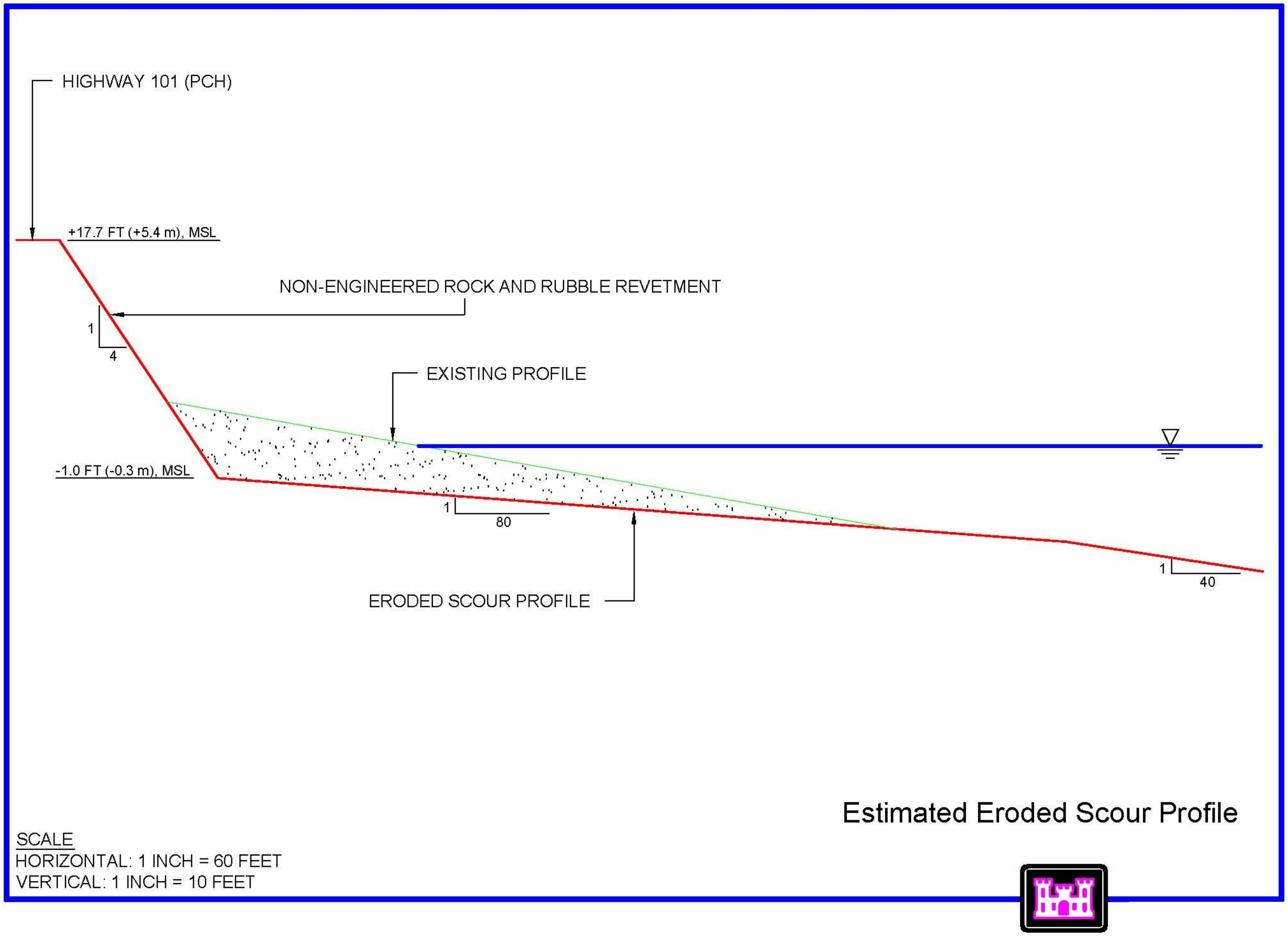
Table 5.2-11 Deduced Minimum Tidal Elevations for Highway 101 Wave Overtopping

Return Frequency (yrs)	Minimum Tidal Elevation Meters MLLW	ft, MLLW
2	1.51	5.0
5	1.43	4.7
10	1.36	4.6
25	1.29	4.2
50	1.23	4.0
100	1.15	3.8

Figure 5.2-63 through **Figure 5.2-65** respectively present the deduced probability for waves overtopping the protective revetment with a crest elevation at approximately 17.7 feet, MLLW under three different sea level scenarios :1) no sea level rise, 2) the historic trend, and 3) the projected sea level rise following the NRC-III curve. The wave overtopping occurrence will increase from about 20% under the present-day conditions, to approximately 30% and 73% in Year 2065 respectively under the historic trend and the projected high sea level rise scenario (i.e., the NRC-III curve) for a 10-year return wave height.

To characterize road closures along the Highway 101 corridor (i.e. waves overtopping the protected revetment) for a project life of 50 years under the without project conditions, two primary oceanographic parameters, namely return storm waves and astronomical tides, need to be randomly selected to prescribe the uncertain nature of wave overtopping events. Therefore, the Monte Carlo Simulation technique **(Appendix E)** used in the modeling of bluff failure scenarios can also be applied to provide the statistical representation of the road closure analysis for assessing the potential economic impact. This task was performed in the economic analysis and is presented in **Appendix E**.

In addition, it is also noted that a small section of the embankment of Highway 101 at Cardiff was damaged during the 2009-2010 El Nino season. It further demonstrates the need to upgrade the existing non-engineered protective revetment to provide an adequate protection for the road embankment from wave-induced scouring under the future sea level rise conditions.



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2 **Figure 5.2-62 Estimated Eroded Scour Profile**

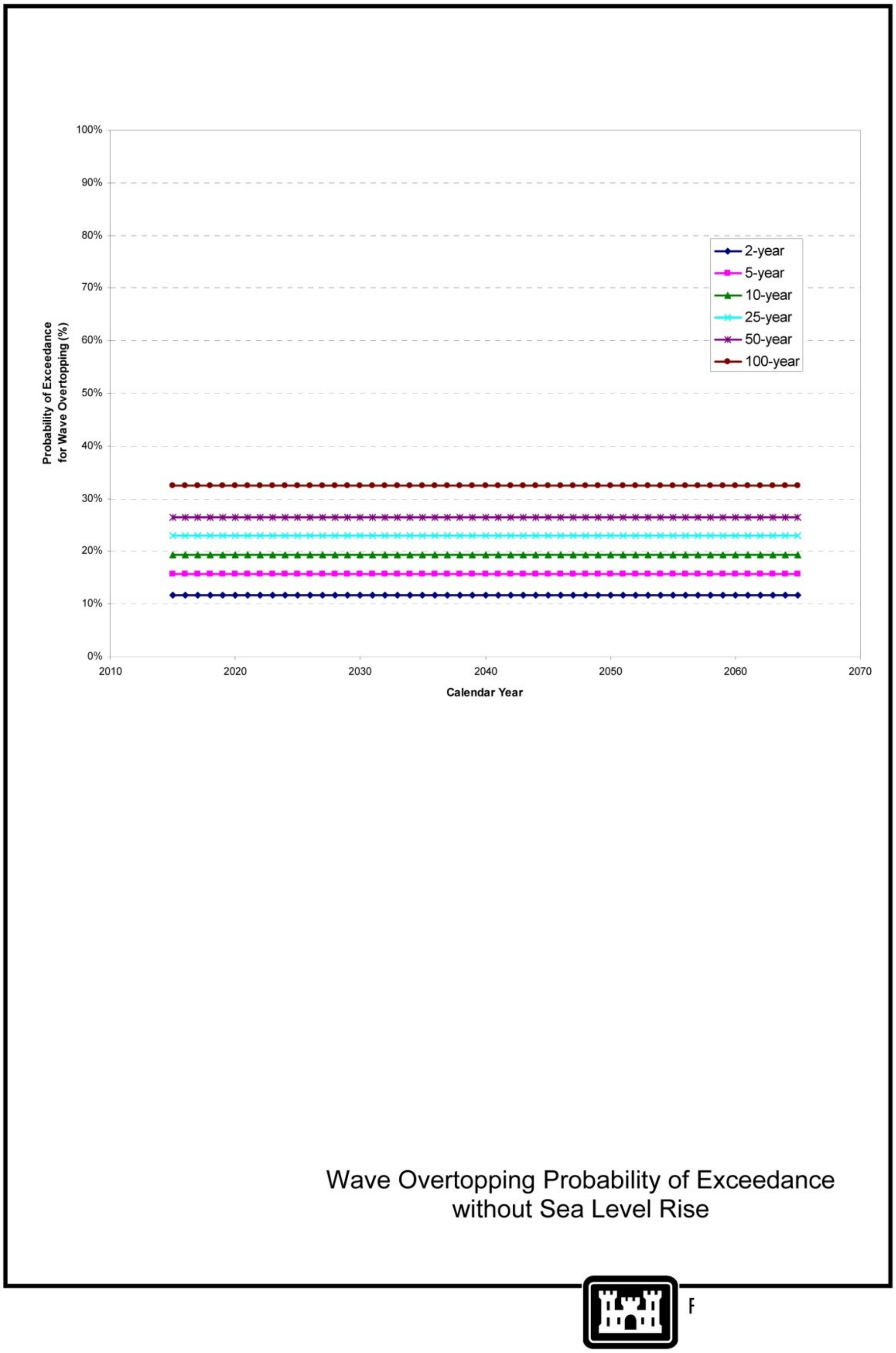
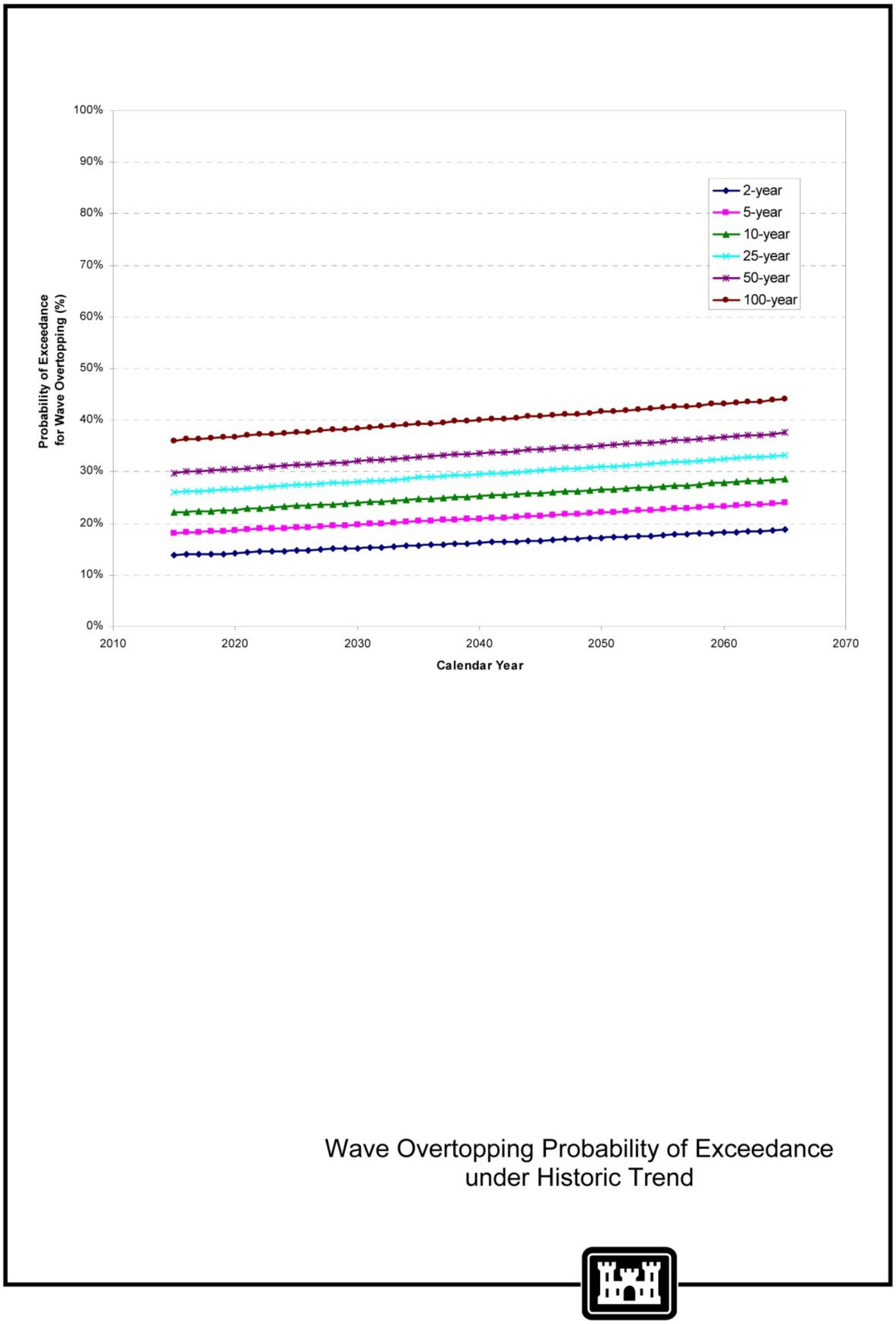


Figure 5.2-63 Wave Overtopping Probability of Exceedance without Sea Level Rise



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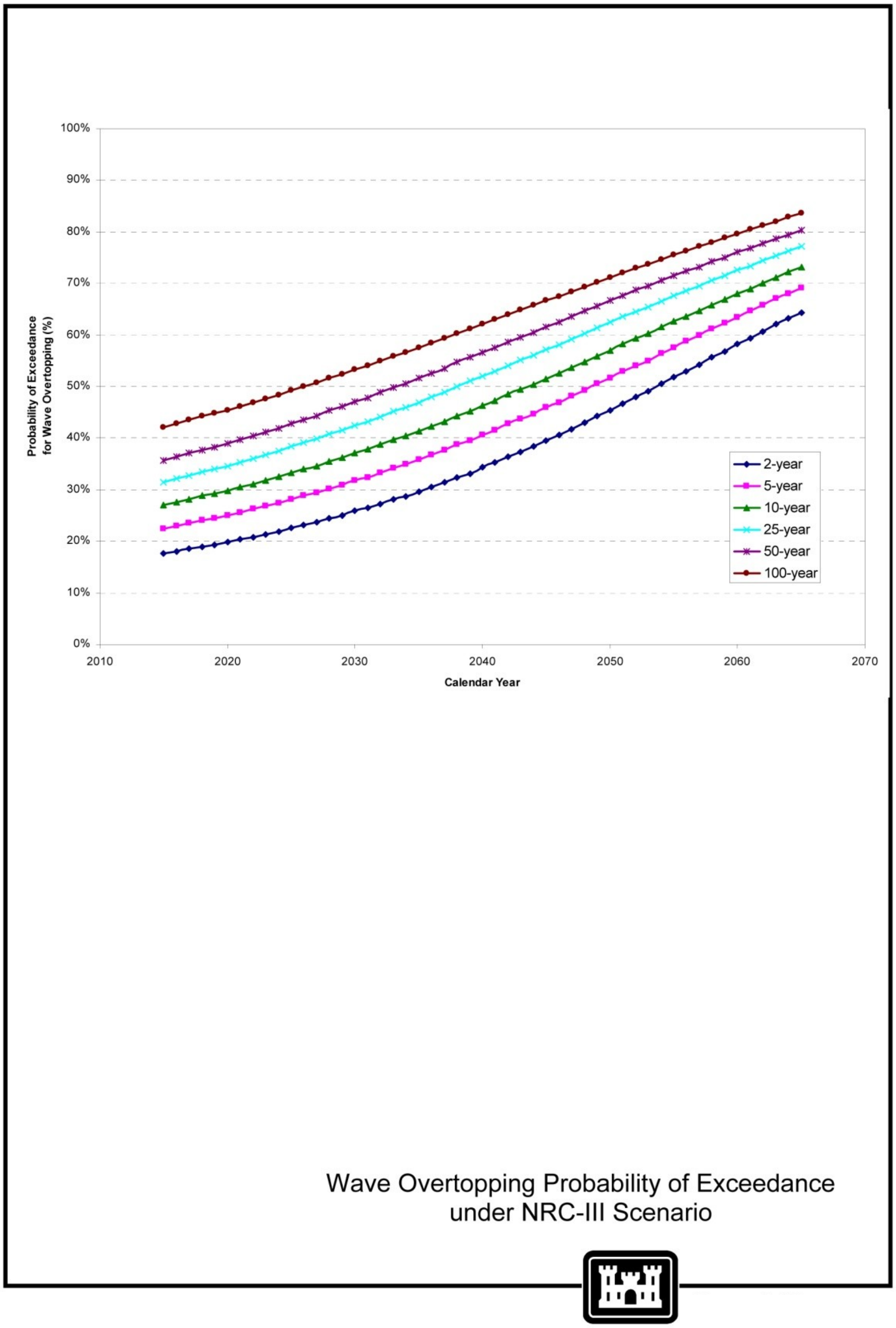
2 **Figure 5.2-64 Wave Overtopping Probability of Exceedance under Historic Trend**



1
2 **Figure 5.2-65 Wave Overtopping Probability of Exceedance under NRC-III Scenario**

6 PLAN FORMULATION

The without-project-conditions analysis indicates that portions of the study area are prone to continuous bluff base erosion and resulting bluff failure in Encinitas and Solana Beach. The persistently occurring bluff failure will threaten the existing land development along the bluff top. Based upon these high value developments and the likelihood that local interest will expend efforts in avoiding future structure damage and land loss, the anticipated future without project is continued construction of emergency seawalls with some bluff top structure losses resulting from bluff failure. Reaches 3, 4 and 5 in Encinitas and Reaches 8 and 9 in Solana Beach warrant alternate measures to mitigate further bluff failure resulting from storm wave attack at the bluff base. Two shoreline segments are identified where protective beach fills plans are approximately 10,600 feet in length from Reach 3 thru Reach 5 and 7,500 feet for the entire Solana Beach shoreline (i.e., Reaches 8 and 9).

This chapter discusses alternative measures that can provide storm damage protection in the Encinitas/Solana Beach shoreline area. A preliminary screening of an array of alternative plans identified several pertinent alterative measures (USACE-LAD, 2003). These alternative measures will be further detailed in this chapter, following a generic overview of various fundamental engineering techniques for shoreline protection against wave attack and for beach sand preservation to mitigate shoreline retreat.

6.1 Engineering Techniques for Shore Protection

The engineering techniques for shore protection can be classified into two major categories identified as the soft-structural and hard-structural methods. The soft-structural method includes beach fills, sand scraping, or sand bypassing/recycling. Hard structures consist of the sand retention features that impede alongshore sand movement (e.g., groins, jetties, artificial reefs, or detached breakwaters), and the storm-protective features, which directly prevent shoreline or upland erosion (e.g., coastal armoring, seawalls or revetments). Detailed summaries of engineering methods, techniques, and data pertinent to the sand preservation strategies and shore erosion problems can be referenced to the Shore Protection Manual (USACE, 1984), a coastal engineering reference prepared by Dean and Dalrymple (2002), and the Coastal Engineering Manual (USACE, 2002), as well as other Corps publications. These shoreline protection techniques are briefly reviewed as follows.

6.1.1 Soft Structural Approaches

Beach Nourishment

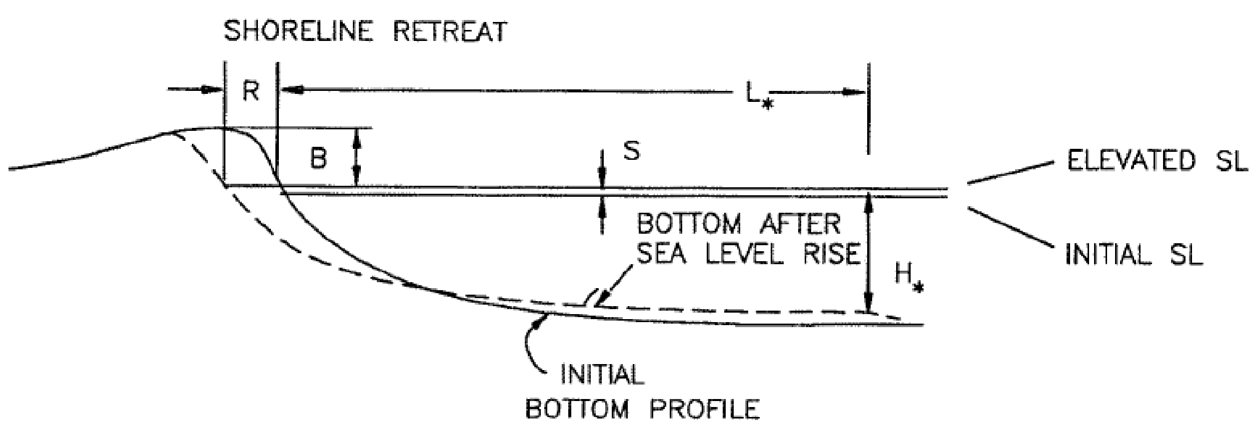
Beach nourishment is the most non-intrusive technique available for shoreline protection. A beach fill, with the widened beach, offers storm protection to the shoreline and upland both by reducing wave energy nearshore and by creating a sacrificial beach to be eroded during a storm. Other benefits of the beach fill include creating additional recreational area and providing, in some cases, environmental habitats for endangered species. This approach directly addresses the deficit of sand in the system with the least potential of causing adverse effects on adjacent property. It is a benign and acceptable approach to beach erosion mitigation. This practice is supported by the National Research Council (1995), which has strongly endorsed the beach fill measure and has issued substantial design guidelines.

Sands dredged from the offshore or onshore borrow sites can be transported and placed on the beach mechanically or hydraulically. However, the hydraulic means has been used in most of the beach fill projects in the United States, in which the sands are scraped from the offshore borrow site by a hydraulic or hopper dredge, and is pumped via floating pipelines to the receiver site where it is discharged onto the beach. The RBSP conducted in 2001 is an example of this application (Noble Consultants, 2001). The RBSP included the restoration of 12 beaches in San Diego County between Oceanside and Imperial Beach, California. More than two million cubic yards of sand were dredged from six offshore borrow sites, transported to each of the 12 beach sites, and carefully placed within the designated beach limits.

However, a beach sand fill represents the replacement of a sand resource, but does little to avoid the need for subsequent replenishment. Thus, the use of nourishment as an erosion control technique requires a continuous financial commitment. The sand nourishment practice is not without potential consequences, which can include: 1) increasing the offshore transport of sand during storms that may impact the nearshore marine habitats; 2) forming nearshore bars resulting from the increase of cross-shore sand movement that alters incoming wave dynamics affecting recreational surfing; 3) increase sand shoaling at tidal inlets affecting lagoon circulation and inlet closure; and 4) sand burial of surf-zone rocky habitat.

Adjustments for Sea Level Rise

Under the scenarios of future sea level rise, the amount of sand required to be placed with each beach-fill to obtain and sustain a fixed shoreline position will vary over time depending on the rate and acceleration in the sea level changes. Beach-fill alternatives account for this change by changing the re-nourishment volumes over the period of analysis to hold the proposed shorelines steady to account for sea level rise. This results in a steady risk reduction for shore protection over the project life. The increase in nourishment volumes is estimated through application of the Bruun Rule applied over the period of analysis using the ranges of sea level rise increases described for the NRC scenarios.



Shoreline Response to Sea Level Rise per Brunn Rule (USACE, 2002)[1]

R= shoreline retreat; S= increase in sea level; L= cross shore distance to water depth H*; B= berm height of eroded area; and H*= closure depth.

Beach Scraping

Beach scraping is the removal of material from the lower part of the beach for deposition on the higher part. Beach scraping is usually performed by a scraper pan or front-end loader, which removes or skims the uppermost layer of the beach. Scrapped sands are used to construct a temporary protective berm on narrow beaches. The winter sand berm constructions at Carpinteria, Seal Beach and Surfside/Sunset Beach illustrate this type of practice. After each winter season passes by, the berms are moved and the beaches are restored to their without-berm conditions.

Beach scraping is different from artificial nourishment. Artificial nourishment is the placement of new material imported from off-site sources. Beach scraping redistributes the available beach material in a manner that improves the coastal protection capabilities of the overall beach profile without providing any new beach material. A technically responsible beach-scraping program that skims no more than one foot of the upper beach surface will not induce any adverse effects on adjacent beaches (Brunn, 1983). Brunn (1983) also stated that beach scraping should only be done where beach material is available in relative surplus in the profile. This is the area of regional sand deficit and active fluctuation of the beach profile where ridges build up by swells following a storm or during the spring and summer seasons.

Sand Bypassing/Recycling

Sand bypassing involves the mechanical transfer of sand around littoral barriers such as jetties and breakwaters. Sand from the accretion area updrift of the barrier is used to nourish the eroded downdrift beaches and maintain the natural littoral transport. In other situations, sand traps are excavated in inlet areas. These traps are periodically dredged to remove the sand, which is deposited there by the tidal currents and impinging waves in the inlet. Effective bypassing can be accomplished when the dredged sands are placed on the downdrift beaches. This has been done on a regular basis at Santa Barbara, Ventura, and Channel Islands harbors located within the Santa Barbara and Ventura Counties, and at Batiquitos Lagoon in San Diego County.

Sand recycling is performed to transfer beach material from a sand-abundant segment to a sand-deficient one within a well-defined littoral drift cell via a mechanical means. If the sand-abundant beach segment is located downdrift of the sand-deficient reach, the replenished sand material will eventually be moved back to the original beach segment. Thus, the sand recycling process can continue on a regular basis as long as the surplus of sand material is available. Since the overall sediment budget within the littoral cell remains unchanged, no long-term adverse effect will result.

6.1.2 Hard Structural Approaches

Hard structures are built to prevent further shoreline erosion or to impede the motion of sand along a beach. Based on structure objectives and features, hard-structures for shoreline protection can be divided into three categories: cross-shore sand retention structure such as groins and jetties that mimic headlands; shore-parallel sand retention structures including submerged or non-submerged artificial reefs and offshore breakwaters; and shore-parallel protective structures consisting of seawalls, bulkheads and revetments.

Cross-Shore Sand Retention Structures

Cross-shore sand retention structures, such as groins and jetties, are constructed perpendicular to the shore to form fillets that protect or retard beach erosion. The structures typically extend from a point landward of the predicted shoreline recession to an offshore location that is far enough to trap a portion of littoral transport. Most of the littoral transport moves in a zone landward of the typical breaker line under the prevailing wave conditions (usually about the 10 foot water depth). Hence, extension of sand retention structures beyond that depth is generally uneconomical (USACE, 1984). The groin field constructed in Newport Beach has demonstrated the sand retention purpose in maintaining an adequate beach width for storm protection of the shorefront properties.

A cross-shore sand retention structure acts as a barrier to alongshore sediment transport. The amount of sand trapped by the structure depends on the permeability, height and length of the structure, and the background net to gross longshore transport ratio. As material accumulates on the updrift side of the structure, supply to the downdrift side is reduced. These result in a local beach accretion on the upside of the structure (fillet) at the expense of an erosion of the beach for some distance downdrift, as sketched in **Figure 6.1-1**. The upcoast fillet is sometimes pre-filled to mitigate any loss of material on the updrift side. After the shoreline or beach nearby the structure adjusts to an "equilibrium" stage in accordance with the wave conditions, littoral drift will pass the structure either directly over it or be diverted around the seaward end of the structure. Because of the adverse effects on the downdrift shoreline, the cross-shore sand retention structures should be used as a protective feature only after careful consideration of the many factors involved.

Shore-Parallel Sand Retention Structures

Shore-parallel sand retention structures, such as submerged or emergent artificial reefs and offshore breakwaters, are built parallel to the shoreline to provide dual purposes of protecting shore areas from direct wave action and of trapping littoral sand on landward beaches. The structures induce wave reflection, diffraction, breaking and energy dissipation, leading to a "shadow zone" shoreward of the structures where the wave energy is reduced. As wave energy is the primary driver of littoral transport, the significant reduction in wave energy results in the deposition of sediment behind the structure, as shown schematically in **Figure 6.1-2**.

As sand is deposited, a seaward projecting shoal is formed in the still water behind the breakwater. This projecting shoal in turn acts as a groin, which tends to cause an advance in the updrift shoreline. The shoal projection will grow until either a new equilibrium stage (e.g., Salient) is reached in accordance with the littoral transport or a tombolo is formed connecting the breakwater to the shore.

The effectiveness of a shore-parallel retention structure acting as a sand trap in providing a protected area depends on its height and length in relation to the wave action and variation in water levels at the site and on its offshore location. If it is desirable for an offshore sand retention structure to not disturb the view of the sea, the structure can be designed submerged, allowing a shallow water depth atop the structure. The rubble-mound Santa Monica breakwater located in Los Angeles County illustrates the benefit of these structures resulting in a moderate and stable beach gain. The Venice breakwater, also in Los Angeles County, is an example of an emergent structure that provides a moderate and stable beach gain.

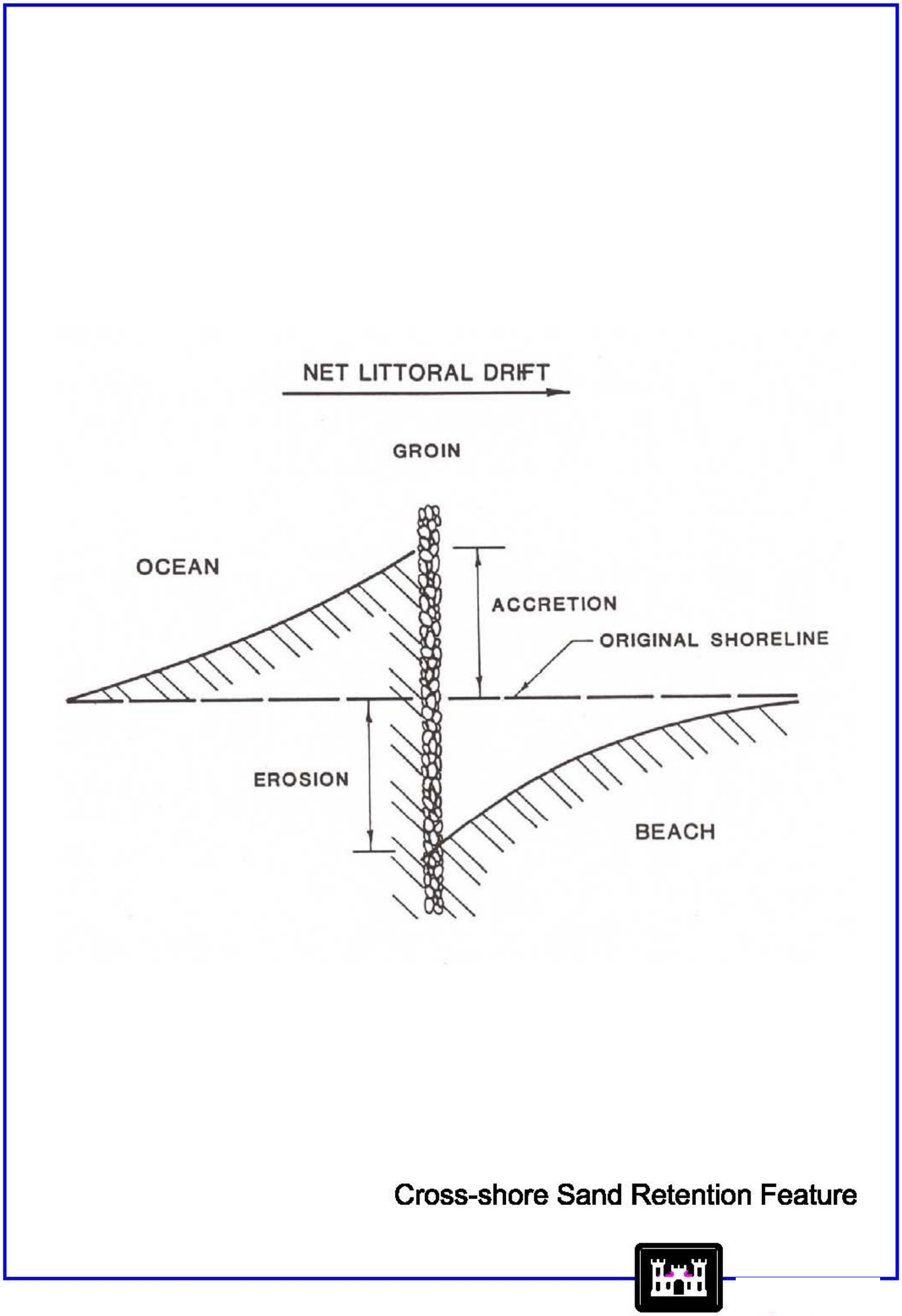


Figure 6.1-1 Cross-shore Sand Retention Feature

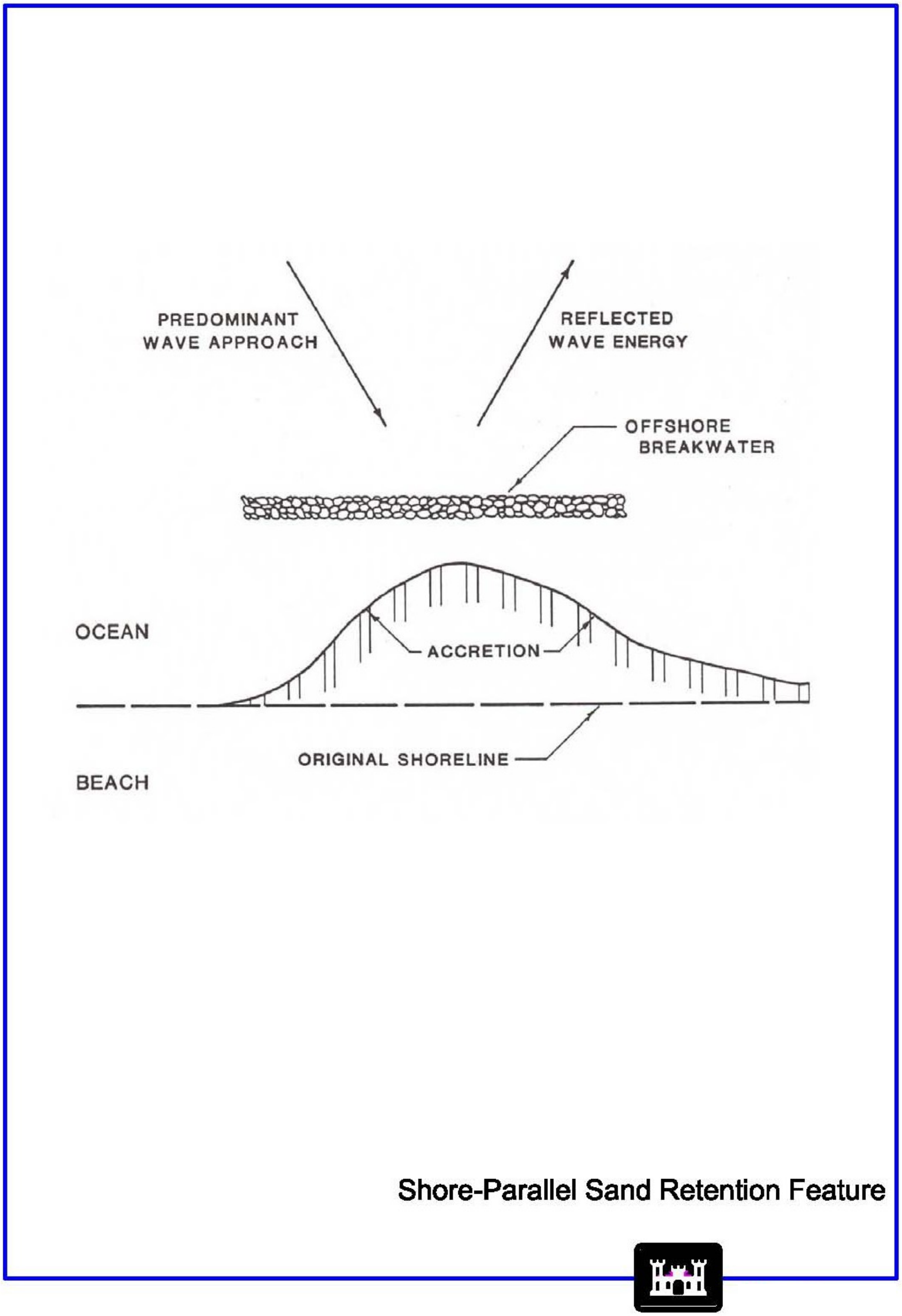


Figure 6.1-2 Shore-Parallel Sand Retention Features

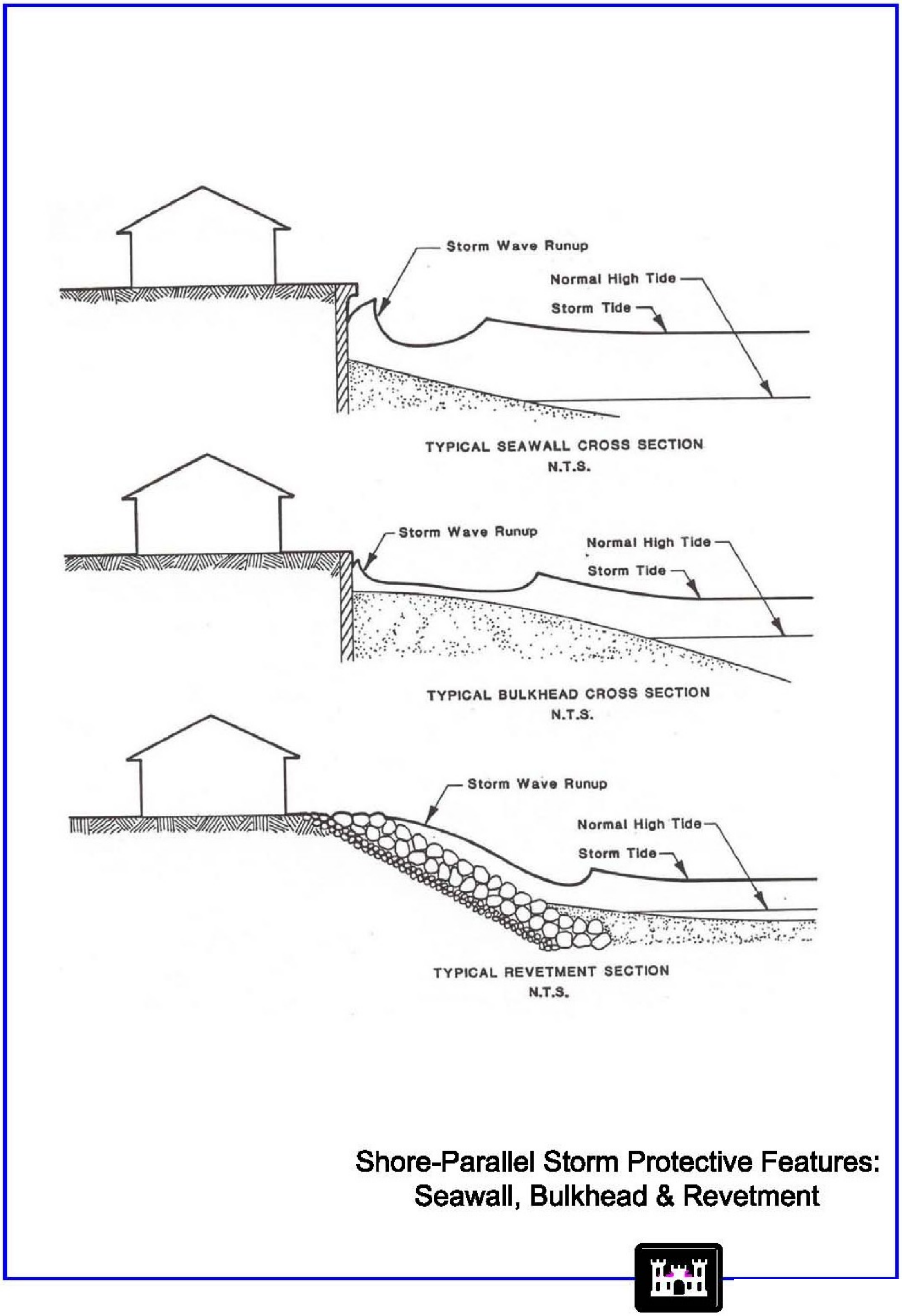


Figure 6.1-3 Shore-Parallel Storm Protective Feature: Seawall, Bulkhead & Revetment

Shore-Parallel Storm Protective Structures

Shore-parallel storm protective features such as seawalls, bulkheads, and revetments are structures placed parallel to the shoreline to separate a land area from the ocean, as shown in **Figure 6.1-3**. These structures are generally constructed to protect buildings, infrastructure, and uplands (dunes, bluffs, cliffs and wetlands) from wave attack. Seawalls are designed to resist the full forces of waves while bulkheads are designed to retain fill, and are generally not designed for direct exposure to wave action. Revetments are flexible structures designed to protect shorelines against erosion by currents or wave action.

Shore-parallel storm protective structures protect only the land immediately behind them. These structures provide no protection to either upcoast or downcoast shoreline and provide no benefits in trapping nearshore sand or in protecting beach from erosion.

6.1.3 Innovative Structure Approaches

Everts (Everts and Eldon, 2000) introduced a concept of naturally occurring beach-retention structures that are responsible for preservation of sandy beaches. The features are generally classified according to their mechanism of beach retention: those that block sediment, block wave energy, or beneficially alter incident surf patterns. Studies are still undergoing to better quantify the characteristics of the naturally occurring rocky features, formational outcrops, or deltaic substrates and to investigate how they might be applied to mimic similar conditions on eroding shores. Strategies under review include construction of artificial headlands, artificial reefs, enhancement of existing outcrops, and nearshore and foreshore placement of gravel, boulders, and cobble.

6.2 Alternative Measures Considered

The application of any specific engineering technique for shore protection requires a systematic and thorough study. In particular, the selection of project alternatives for a given environment and location entails a detailed site-specific consideration of needs and littoral transport dynamics as well as a multidiscipline appraisal of the induced impacts including environmental quality, cost and economic benefits. After reviewing all possible shore protection techniques, a preliminary screening of alternative measures was performed to narrow the field by eliminating those measures that prove unacceptable or infeasible at a second glance (USACE-LAD, 1996). Measures passing this screening were analyzed and screened further via thorough discussions with federal, state, and local agencies, and local residents until several candidate alternative measures were selected. The alternative measures considered for further detailed analyses during this plan formulation phase are limited to 1) beach fills; 2) a hybrid plan consisting of sealing up the toe notches at the bluff base prior to the construction of scaled-back beach fills; and 3) seawalls.

6.2.1 Beach Fills

The most desirable protection for the project shoreline that all stakeholders seem to agree on is a wide protective beach as a direct consequence of an artificial beach fill. The beach fill is the most non-intrusive technique available for shoreline protection while also enhancing recreational opportunities for beach goers -- which potentially provides NED benefits and induced regional benefits for local governments. The RBSP conducted in 2001 received widespread public support in San Diego County. The effectiveness in preventing bluff toe erosion and the associated economic benefits in the study area have been well documented by local governments, even though a portion of placed sands has been lost from the littoral system since placement.

Beach fills considered under this plan formulation phase consist of two separate shoreline segments in Encinitas and Solana Beach: 1) Segment 1- extending from 700 Block, Neptune Ave to Swami's Reefs (Reaches 3 to 5), and 2) Segment 2 - stretching from Table Tops Reefs to the southern city limit in Solana Beach (Reaches 8 and 9). Swami's and Table Tops Reefs, acting as natural sediment entrapment barriers, are designated as one of the boundary ends for each respective beach fill. Environmental constraints of potential impacts on the existing rock habitats (e.g., surfgrass) and surfing breakers in the reef areas also preclude any sand placement within the immediately adjacent reef areas. Therefore, the proposed alongshore length of each beach fill is shorter than the individual segment length. The beach fill proposed for Segment 1 is approximately 7,800 feet long, while artificial beach widening in Segment 2 extends for approximately 7,200 feet in length. **Figure 6.2-1** shows the alongshore extent of beach fill in Segment 1, while **Figure 6.2-2** illustrates the beach fill boundary within the City of Solana Beach (i.e., Reaches 8 and 9).

Beach fills spread laterally alongshore in an upcoast and downcoast directions as waves rework on the artificial deposits. A filled beach width would gradually narrow to a stage that sand replenishment is necessary to restore the required width for the protection against storm wave attack. Thus, a repetitive sand replenishment program is essential to ensure a successful beach fill project. The period and the volume for each replenishment cycle can be estimated via a numerical simulation using the Corps GENEralized Model for Simulating Shoreline Change (GENESIS) and in the evaluation of measured shoreline data at the specific and/or similar project sites. The GENSIS modeling effort performed under this feasibility study is presented in the **Chapter 7**.

The without-project-future conditions is a sediment starved beach profile where the winter-spring condition is completely denuded at the bluff toe along both shoreline segments, even though moderate to narrow beaches have been observed as recently as 2009 as a direct consequence of the SANDAG beach nourishment project in 2001. The required width of the beach fills was derived from the seasonal variation in beach width that has been observed in the field, the anticipated seasonal and severe storm cross-shore erosion, and coincident wave-runup at the bluff base. Historical observations within the Encinitas/Solana Beach shoreline indicate that the typical seasonal variation in MHHW beach width is about 40 to 70 feet (USACE-LAD, 1991). The storm-induced short-term shoreline retreats measured from past severe storm events (e.g., 1988 January storm) were approximately 100 feet (see Appendix B of USACE-LAD, 1991). And a long history of seasonal beach profiles provide data to quantify the likely cross-shore distribution of littoral drift on the profile that can protect the bluff toe from wave and tide impact.

The width of protective beach and its periodic re-nourishment period is optimized through an economic NED analysis discussed in the **Appendix E**. Alternate widths were developed in 50-foot increments up to an increased width of 400-feet or until the analysis demonstrated a decline in net benefits. The affects of additional beach fill on reducing bluff top erosion is discussed in **Section 6.6**, Beach Fill Affects on Bluff Failure. This analysis is in accordance with the Corps' planning guidelines to select an optimal beach width, and is further described in **Chapter 12**. These optimal beach fills were based on the overall project net benefits and include details such as initial beach nourishment width and sand replenishment cycles. The design sand placement densities, or volume of sand placed per alongshore length (cy/ft) is based on the analysis of site specific beach profiles and V/S ratios. The construction beach fill prism dimensions are typical for the California coasts with crest height at +10 feet MLLW, foreshore slope of 15:1 (horizontal to vertical), and tapering to the back beach elevation ranging from about +12 to +18 feet above MLLW. **Figure 6.2-3** and **Figure 6.2-4** illustrate the beach fill profiles for the two beach-fill segments, respectively.

6.2.2 Hybrid Plan

Regulatory permit applications to the federal and state agencies are required prior to initiating any beach fill activity. Therefore, the environmental review process as well as the availability of funds appropriations can affect the timing of each sand replenishment cycle. In addition, the cyclic variation of annual wave climate in a short time span (e.g., 4 to 7 years) may accelerate or slow down sediment loss during a particular replenishment cycle as compared to the average projection derived from historical observations or model simulations. Further, SANDAG may implement another RBSP in the future. As a consequence, there exist some risk that a protective beach may be eroded away before the next designated sand replenishment cycle is carried out. This can be due to either deficiency of available funding, delay due to environmental review, or severe storm events occurring. Under such incidents, the bluff base would again be vulnerable to direct wave attack. Bluff failure may be triggered from additional toe erosion, if a substantial toe notch has previously been developed.

The comprehensive beach fill based strictly on a minimum storm-protective beach width criterion, as previously described in **Section 6.2.1**, may not be achieved due potentially to 1) funding availability in a particular replenishment cycle year, 2) unexpected severe climatologic environment occurring during a replenishment cycle. The full beach width required for the bluff protection may not be maintained throughout the entire project life cycle. Therefore, with the increasing severe storm occurrence predicted in Southern California (Graham, 2001), the denuded beach conditions similar to those observed prior to the 2001 SANDAG beach nourishment project may occur between replenishment cycles within the 50-year project design life.

To prevent the bluff base from toe erosion during a short period in which the beach is almost or completely depleted, a hybrid plan combining notch fill and a beach fill with a narrower beach fill than a beach only plan is an alternative. The plan provides the flexibility of a required beach width necessary for bluff base protection. It can optimize the design width of a beach fill that is potentially constrained by the limitation of available funding and associated environmental impacts.

The hybrid plan consists of an extensive notch fill with erodible concrete at the bluff base along with a beach fill. The initial berm width in each segment would be narrower than the one proposed for the beach fill only alternative. The crest elevation of the placed berm is at approximately +10 feet, MSL with a front face slope of 10:1 (horizontal : vertical) similar to the cross section described in the beach fill alternative (see **Figure 6.2-3 and Figure 6.2-4**). The detailed description of the optimization process to determine the designated berm width is also presented in the **Chapter 8**. Similarly, the GENESIS program was applied to assess the shoreline evolution after each beach fill cycle, as delineated in **Chapter 7**.

Figure 6.2-1 Alongshore Extent of Beach Fill in Segment 1

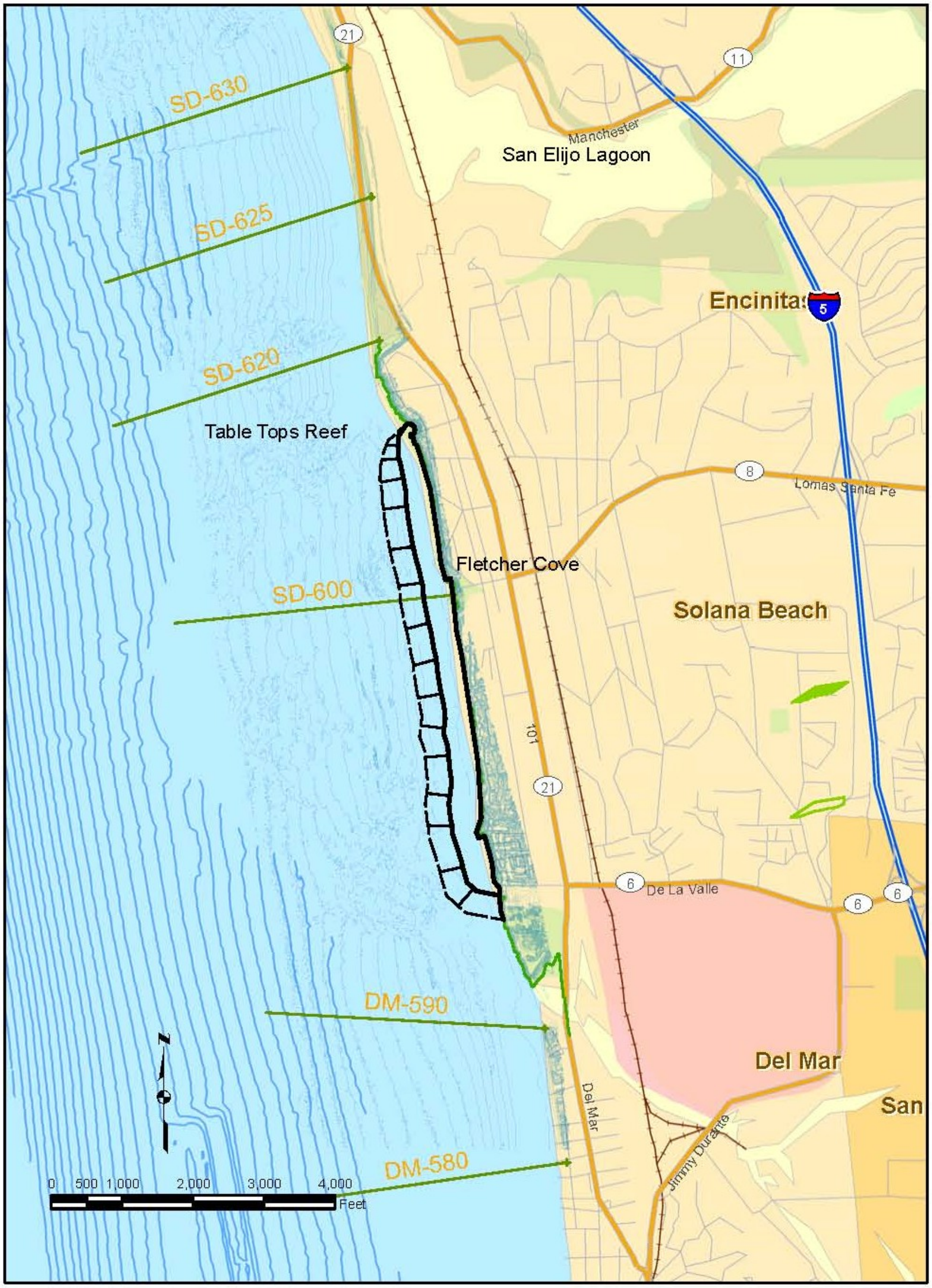
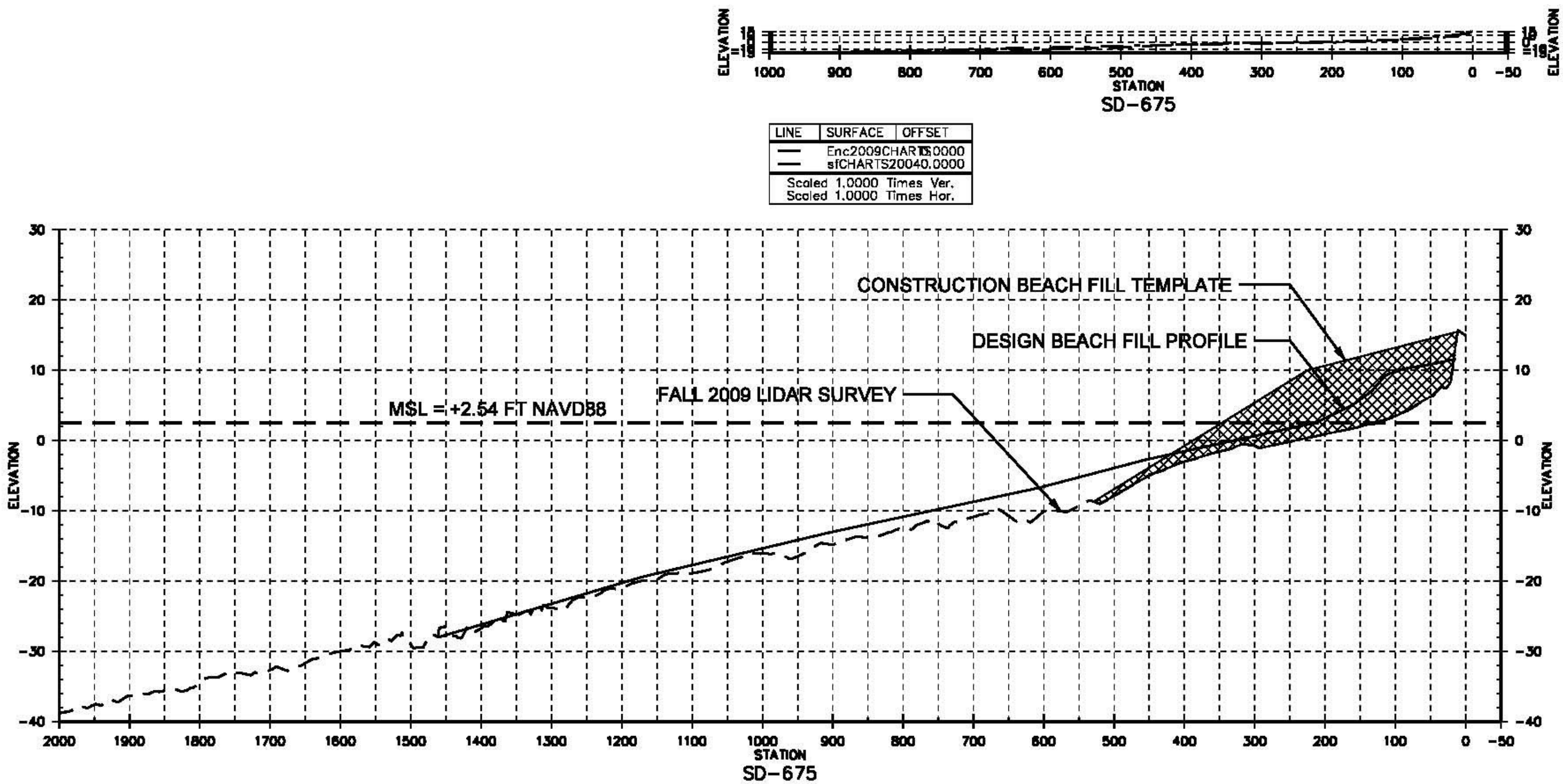


Figure 6.2-2 Alongshore Extent of Beach Fill in Segment 2

6.2.3 Seawalls

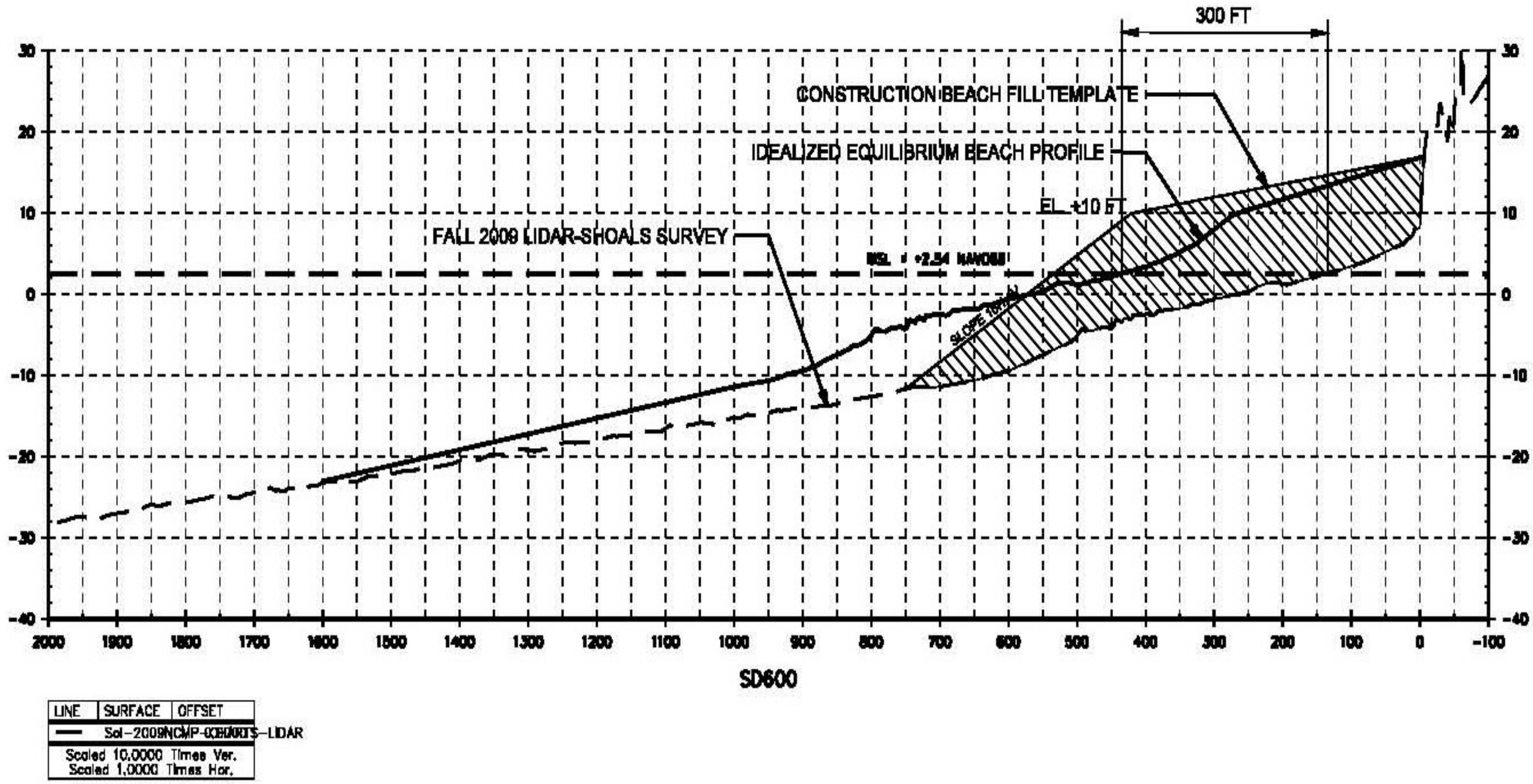
Because of site constraints related to construction access, seawalls that have been constructed in Encinitas and Solana Beach to protect the bluff base against wave attack are either tied-back shotcrete or cast-in-place walls, depending on the required height of the proposed seawall structures. In Encinitas, historical seawalls installed in 1980's are 30 to 40 feet in height above the MLLW line. However, the cast-in-place walls constructed since 1996 have a top elevation at +16 feet, MLLW only. Although wave overtopping can still occur under an extreme storm condition, the overtopping storm water appears to induce insignificant abrasion to the Torrey Sandstone bluff face. Thus, the existing low seawalls indeed provide an adequate protection to the bluff base. Therefore, the proposed seawall alternative applicable to Reaches 3, 4 and 5 would be similar to the recently constructed walls within these reaches. The proposed seawall consists of a continuous cast-in-place wall panel that is 24 inches thick on the bottom and is gradually reduced to 18 inches on the top. The wall panel that is embedded 2 feet into bedrock is anchored deep into the bluff with tied-back rods. **Figure 6.2-5** illustrates the cross-section view of the 16-foot wall in relation to the to-be-protected bluff and the detailed wall section.

In Solana Beach a continuous shotcrete wall with a crest elevation at +35 to +40 feet above MLLW with tied-back anchors embedded deep into the bluff is proposed. The additional height is required due to the geological formation that consists of a 10-foot thick sand layer beginning at an elevation of approximately +25 feet, MLLW for Reaches 8 and 9. The shotcrete wall is embedded 2 feet into the bedrock layer, has a thickness of 30 inches on the bottom and is gradually tapered to 18 inches wide at the top. **Figure 6.2-6** shows the cross section view of the wall and the detailed wall section itself.



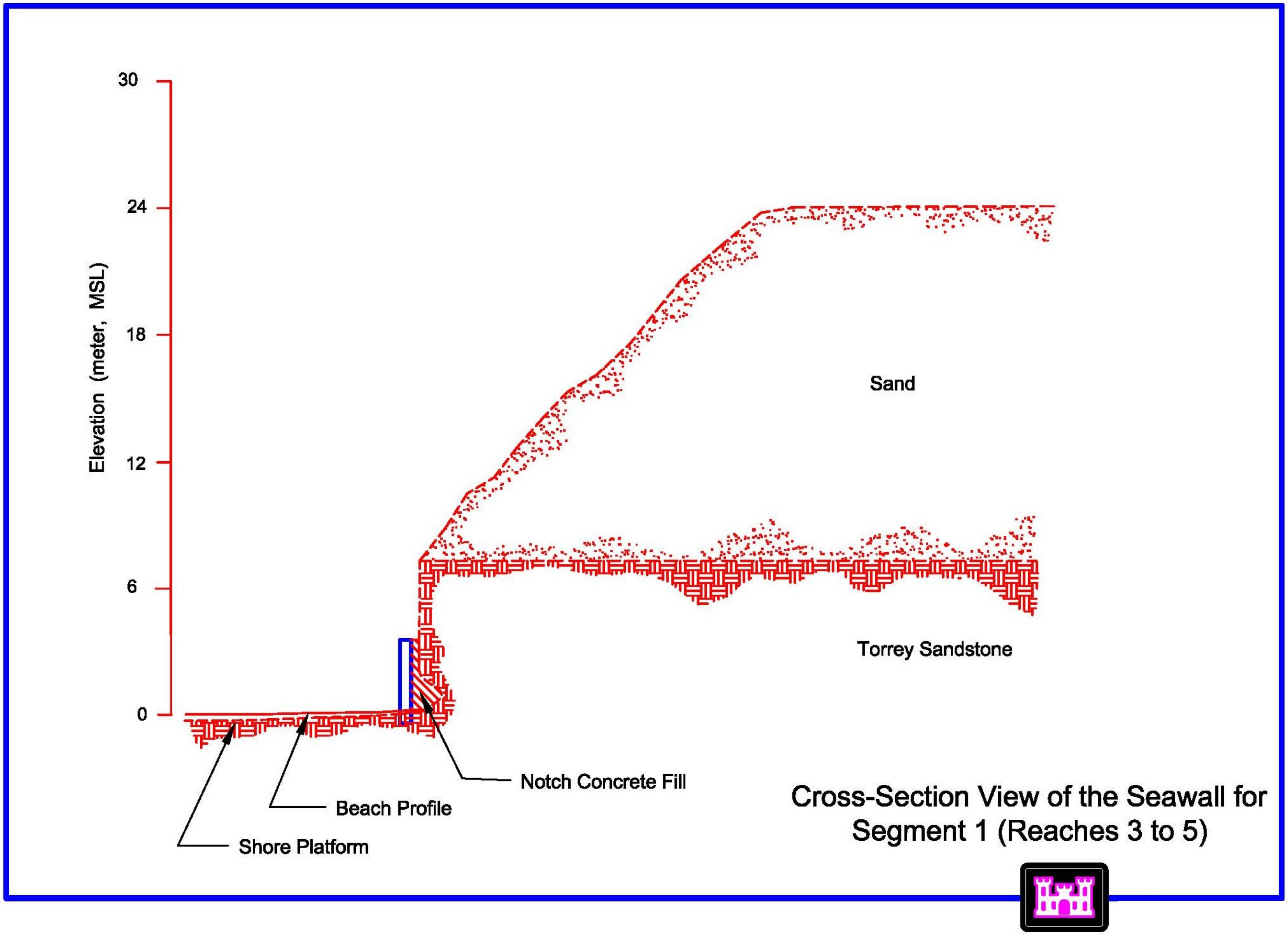
TYPICAL BEACH FILL PROFILE

Figure 6.2-3 Beach Fill Section for Segment 1

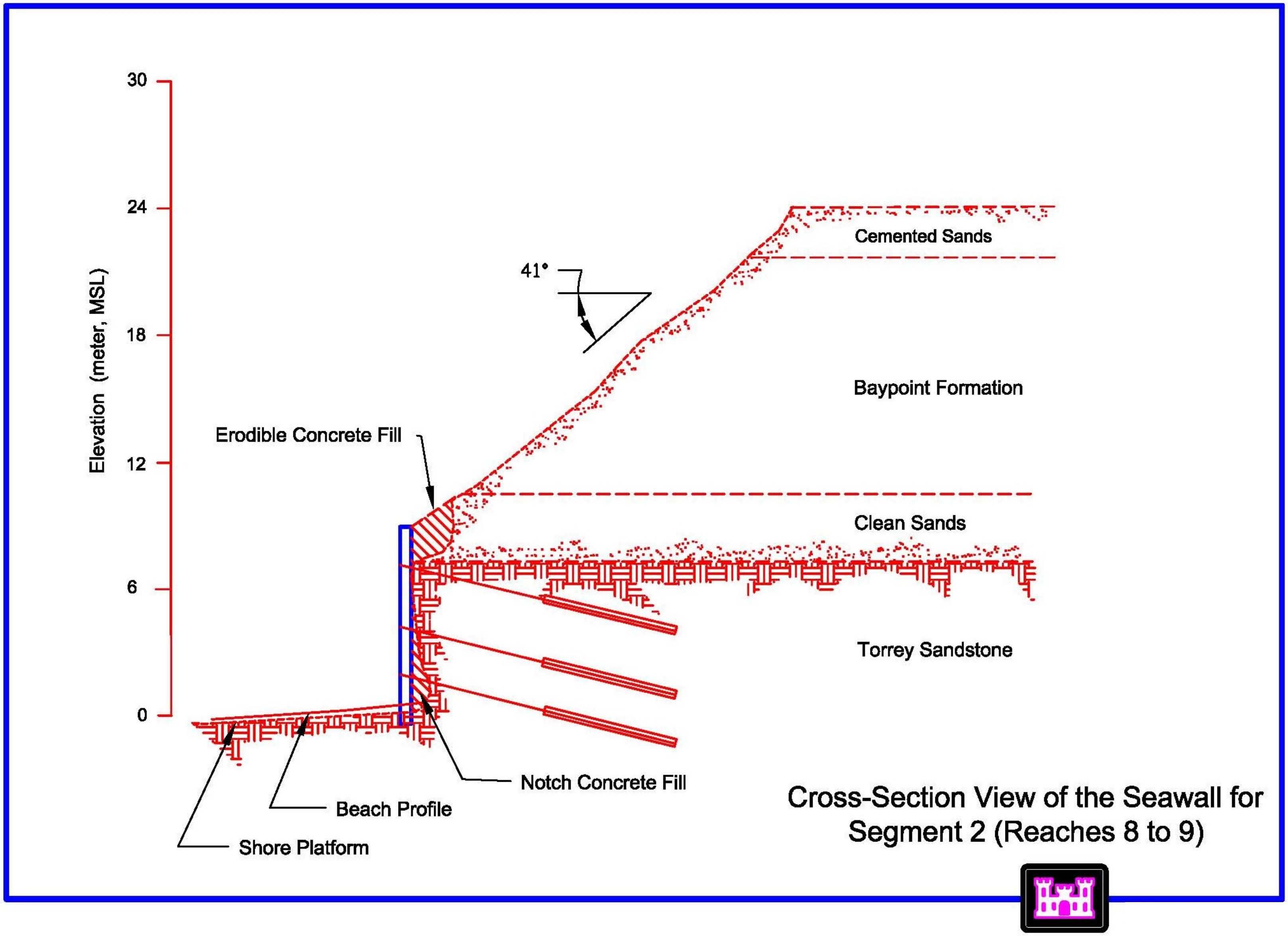


1

2 **Figure 6.2-4 Beach Fill Section for Segment 2**



1
2 **Figure 6.2-5 Cross-Section View of the Seawall for Segment 1 (Reaches 3 to 5)**



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2 **Figure 6.2-6 Cross-Section View of the Seawall for Segment 2 (Reaches 8 to 9)**

6.3 Upper Bluff Stabilization

Upper bluff stabilization will be required to arrest surface and ground water erosion and mitigate geotechnical instabilities. Suitable alternatives to stabilize the upper bluff are limited due to the highly friable and erodible soils that comprise the upper terrace deposits. This activity is presumed to occur independent of the type of shore protection measure at the base of the bluffs, and would be needed for both with and without project. Conventional gravity and cantilevered structures are viewed to be unacceptable unless integrated with an entire bluff-height stabilization solution, as it is not feasible to construct a requisite foundation to support these structures within the upper bluff. Therefore, upper-bluff stabilization is limited to a tied-back structural shotcrete wall extending down to the design stable slope angle or a geogrid reinforced fill built up layer-by-layer from the lower bluff.

The tied-back wall does not rely upon foundation soils beneath, or in front of, the wall for any of its stability. A temporary construction backcut that has a sloping angle of 35 degrees can be made to prepare the upper bluff for a structural shotcrete wall. Tied-back anchors that are installed to restrain the structural shotcrete skin would be placed on 8 foot centers with various rows of anchors depending on the wall height locations. The estimated construction cost would be approximately $30,000 to $50,000 per linear foot. For the geogrid reinforced fill, the keys and benches are one foot minimum into formational or firm material. All fills are keyed and benched through all compacted topsoil on a layer-by-layer basis. Based on several geogrid reinforced fills constructed in Solana Beach, the average construction cost is approximately $50,000 to $75,000 per linear foot.

6.4 Initial Beach-Fill Volumes

Alternate beach fill plans are formulated to extend the MSL seaward from the without project position in increments of 50-ft, initially, with varying replenishment intervals and quantity to reestablish that initial MSL position. Projected loss rates of the beach-fill were estimated with the GENESIS shoreline modeling and consideration of the performance of prior beach-fills in the project area. The degree, or effectiveness, of the beach to protect the bluffs from tides and wave action is discussed in **Section 6.6**.

Alternatives are evaluated under two scenarios of rising sea levels. **Table 6.4-1** and **Table 6.4-2** show the initial beach fill volumes for widening beach from 50 to as much as 400 feet for the Encinitas and Solana segments, respectively. Sand volume is also increased to offset rising sea levels in the initial placement, hence the longer replenishment intervals and accelerating sea level rise require larger volumes for equal shore protection effectiveness at the end of the cycle.

The sand borrow source is expected to be from the near shore areas in the vicinity of SO-5 and SO-6 for initial construction, and possibly off of Mission Bay or Oceanside for future replenishment. An overfill factor is the ratio of the volume removed from the borrow site and the volume added to the active beach profile. This overfill factor is dependent on the geotechnical properties of both the borrow site and receiving beach fill site, principally bulk densities and grain size distribution, and to some extent the method of construction. For this study, an overfill factor of 1.20 was applied based on the long term experience of the recurring beach fill project at Surfside-Sunset Beach in southern California's Orange County (see Gadd et al, 1996) where 34-years of beach-fills and monitoring of the nourished profile volume could be accounted for if approximately 20 percent of the borrow site volume is presumed lost to the offshore. The lost material is presumed to occur during construction. Construction fill volumes can be updated during design based on future receiving beach surveys and detailed geotechnical evaluation of the borrow site. **Table 6.4-3** and **Table 6.4-4** show the initial dredge volumes from near-shore borrow sites considered for the Encinitas and Solana segments, respectively.

Beach profile conditions that existed prior to the SANDAG I Regional Beach-Fill Project (RBSP I) was taken to represent the without project condition. Profile conditions that existed between the period of 1997 to 2000, at the two data rich profiles, SD670 and SD600, were used to characterize the active littoral volume. SD670 is representative of the Encinitas Segment 1 and SD600 of the Solana Segment 2. The without project active profile volumes were 100 cy/ft for Segment 1 and 75 cy/ft for Segment 2, respectively.

RBSP I added approximately 237,000 cy in the general vicinity of Segment I in the fall of 2001: 132,000 cy at Leucadia and 105,000 cy at Moonlight State Beach. The measured profile response at SD670 displayed an increase in the active profile volume of 25 cy/ft as a result of this fill. The active profile volume at SD670 over the eight years between 2002 and 2010 decreased from about 200 to 140 cy/ft, a loss of 60 cy/ft and loss rate of 7.5 cy/ft/yr.

RBSP I added approximately 146,000 cy at Solana Beach at Fletcher Cove. The measured profile response at SD600 also displayed an increase in the active profile volume of 25 cy/ft as a result of this fill. The active profile volume at SD600 over the eight years between 2002 and 2010 decreased from about 85 to 65 cy/ft, a loss of 20 cy/ft and loss rate of 2.5 cy/ft/yr.

A second SANDAG Regional Beach-Fill Project (RBSP II) is expected in 2012 and is projected to add 222,000 cy to Segment 1 and 146,000 cy to Segment 2. Scaling from the measured performance of the RBSP 1and using a base year of 2015 for the federal project was used to estimate the affects of the RBSP II on the active profile sand volume in the base-year. This estimate resulted in 9,000 cy of the RBSP II fill remaining in the active profile volume for Segment 1 and 102,200 cy remaining in the base year for Segment 2. The majority of the RBSP II beach fill in Encinitas is in Reach 1 which is north of the proposed project. The 9,000 cy is based on scaling of the observed profile volume change over the proposed project form RBSP I project.

Table 6.4-1 Initial Beach-Fill Placement Quantities for Encinitas Reach Alternatives in cubic yards

LOW SEA LEVEL RISE SCENARIO															
Added MSL Beach Width	**Renourishment Interval in years**														
	22	**33**	**44**	**55**	**66**	**77**	**88**	**99**	**110**	**111**	**112**	**113**	**114**	**115**	**116**
50'	334,626	337,766	340,906	344,046	347,186	350,326	353,465	356,605	359,745	362,885	366,025	369,165	372,305	375,445	378,584
100'	671,673	674,812	677,952	681,092	684,232	687,372	690,512	693,652	696,792	699,931	703,071	706,211	709,351	712,491	715,631
150'	1,008,719	1,011,859	1,014,999	1,018,139	1,021,278	1,024,418	1,027,558	1,030,698	1,033,838	1,036,978	1,040,118	1,043,258	1,046,397	1,049,537	1,052,677
200'	1,345,765	1,348,905	1,352,045	1,355,185	1,358,325	1,361,465	1,364,605	1,367,744	1,370,884	1,374,024	1,377,164	1,380,304	1,383,444	1,386,584	1,389,724
HIGH SEA LEVEL RISE SCENARIO															
50'	352,919	365,663	378,712	392,065	405,724	419,687	433,955	448,529	463,407	478,589	494,077	509,870	525,967	542,370	559,077
100'	689,966	702,710	715,758	729,112	742,770	756,734	771,002	785,575	800,453	815,636	831,124	846,916	863,014	879,416	896,124
150'	1,027,012	1,039,756	1,052,805	1,066,158	1,079,817	1,093,780	1,108,048	1,122,621	1,137,499	1,152,682	1,168,170	1,183,963	1,200,060	1,216,463	1,233,170
200'	1,364,059	1,376,802	1,389,851	1,403,205	1,416,863	1,430,826	1,445,095	1,459,668	1,474,546	1,489,729	1,505,216	1,521,009	1,537,107	1,553,509	1,570,216

Table 6.4-2 Initial Beach-Fill Placement Quantities for Encinitas Reach Alternatives in cubic yards (continued)

LOW SEA LEVEL RISE SCENARIO															
Added MSL Beach Width	Renourishment Interval in years														
	22	33	44	55	66	77	88	99	110	111	112	113	114	115	116
50'	160,676	163,578	166,480	169,382	172,284	175,186	178,088	180,990	183,892	186,794	189,697	192,599	195,501	198,403	201,305
100'	417,748	420,650	423,552	426,454	429,356	432,259	435,161	438,063	440,965	443,867	446,769	449,671	452,573	455,475	458,377
150'	674,821	677,723	680,625	683,527	686,429	689,331	692,233	695,135	698,037	700,939	703,841	706,743	709,645	712,547	715,449
200'	931,893	934,795	937,697	940,599	943,501	946,403	949,305	952,207	955,109	958,011	960,913	963,815	966,717	969,619	972,521
250'	1,188,965	1,191,867	1,194,769	1,197,671	1,200,573	1,203,475	1,206,377	1,209,279	1,212,181	1,215,083	1,217,985	1,220,887	1,223,789	1,226,691	1,229,593
300'	1,446,037	1,448,939	1,451,841	1,454,743	1,457,645	1,460,547	1,463,449	1,466,351	1,469,253	1,472,155	1,475,057	1,477,959	1,480,861	1,483,763	1,486,665
350'	1,703,109	1,706,011	1,708,913	1,711,815	1,714,717	1,717,619	1,720,521	1,723,423	1,726,325	1,729,227	1,732,129	1,735,031	1,737,933	1,740,835	1,743,738
400'	1,960,181	1,963,083	1,965,985	1,968,887	1,971,789	1,974,691	1,977,593	1,980,495	1,983,397	1,986,300	1,989,202	1,992,104	1,995,006	1,997,908	2,000,810

Table 6.1-2 Initial Beach-Fill Placement Quantities for Solana Reach Alternatives in cubic yards (completed)

HIGH SEA LEVEL RISE SCENARIO															
Added MSL Beach Width	**Renourishment Interval in years**														
	22	**33**	**44**	**55**	**66**	**77**	**88**	**99**	**110**	**111**	**112**	**113**	**114**	**115**	**116**
50'	177,584	189,362	201,422	213,764	226,388	239,294	252,481	265,950	279,701	293,734	308,049	322,645	337,523	352,683	368,125
100'	434,656	446,434	458,495	470,837	483,460	496,366	509,553	523,023	536,774	550,806	565,121	579,717	594,596	609,755	625,197
150'	691,728	703,506	715,567	727,909	740,533	753,438	766,626	780,095	793,846	807,879	822,193	836,790	851,668	866,828	882,269
200'	948,800	960,579	972,639	984,981	997,605	1,010,510	1,023,698	1,037,167	1,050,918	1,064,951	1,079,265	1,093,862	1,108,740	1,123,900	1,139,342
250'	1,205,872	1,217,651	1,229,711	1,242,053	1,254,677	1,267,582	1,280,770	1,294,239	1,307,990	1,322,023	1,336,337	1,350,934	1,365,812	1,380,972	1,396,414
300'	1,462,944	1,474,723	1,486,783	1,499,125	1,511,749	1,524,655	1,537,842	1,551,311	1,565,062	1,579,095	1,593,410	1,608,006	1,622,884	1,638,044	1,653,486
350'	1,720,017	1,731,795	1,743,855	1,756,197	1,768,821	1,781,727	1,794,914	1,808,383	1,822,134	1,836,167	1,850,482	1,865,078	1,879,956	1,895,116	1,910,558
400'	1,977,089	1,988,867	2,000,927	2,013,269	2,025,893	2,038,799	2,051,986	2,065,456	2,079,207	2,093,239	2,107,554	2,122,150	2,137,028	2,152,188	2,167,630

Table 6.4-3 Initial Dredge Borrow Quantities for Encinitas Reach Alternatives in cubic yards

LOW SEA LEVEL RISE SCENARIO															
Added MSL Beach Width	**Renourishment Interval in years**														
	22	**33**	**44**	**55**	**66**	**77**	**88**	**99**	**110**	**111**	**112**	**113**	**114**	**115**	**116**
50'	403,291	407,059	410,827	414,595	418,363	422,131	425,898	429,666	433,434	437,202	440,970	444,738	448,506	452,273	456,041
100'	807,747	811,515	815,283	819,051	822,818	826,586	830,354	834,122	837,890	841,658	845,426	849,193	852,961	856,729	860,497
150'	1,212,203	1,215,971	1,219,738	1,223,506	1,227,274	1,231,042	1,234,810	1,238,578	1,242,346	1,246,113	1,249,881	1,253,649	1,257,417	1,261,185	1,264,953
200'	1,616,658	1,620,426	1,624,194	1,627,962	1,631,730	1,635,498	1,639,266	1,643,033	1,646,801	1,650,569	1,654,337	1,658,105	1,661,873	1,665,640	1,669,408
HIGH SEA LEVEL RISE SCENARIO															
50'	425,243	440,536	456,194	472,218	488,609	505,365	522,486	539,974	557,828	576,047	594,633	613,584	632,901	652,584	672,633
100'	829,699	844,991	860,650	876,674	893,064	909,820	926,942	944,430	962,283	980,503	999,088	1,018,039	1,037,357	1,057,039	1,077,088
150'	1,234,155	1,249,447	1,265,106	1,281,130	1,297,520	1,314,276	1,331,398	1,348,886	1,366,739	1,384,959	1,403,544	1,422,495	1,441,812	1,461,495	1,481,544
200'	1,638,610	1,653,903	1,669,561	1,685,585	1,701,976	1,718,732	1,735,854	1,753,341	1,771,195	1,789,414	1,808,000	1,826,951	1,846,268	1,865,951	1,886,000

Table 6.4-4 Initial Dredge Borrow Quantities for Solana Reach Alternatives in cubic yards (continued)

LOW SEA LEVEL RISE SCENARIO															
Added Beach MSL Width	Renourishment Interval in years														
	22	33	44	55	66	77	88	99	110	111	112	113	114	115	116
50'	213,251	216,734	220,216	223,699	227,181	230,664	234,146	237,628	241,111	244,593	248,076	251,558	255,041	258,523	262,006
100'	521,738	525,220	528,703	532,185	535,668	539,150	542,633	546,115	549,598	553,080	556,562	560,045	563,527	567,010	570,492
150'	830,225	833,707	837,189	840,672	844,154	847,637	851,119	854,602	858,084	861,567	865,049	868,531	872,014	875,496	878,979
200'	1,138,711	1,142,194	1,145,676	1,149,158	1,152,641	1,156,123	1,159,606	1,163,088	1,166,571	1,170,053	1,173,536	1,177,018	1,180,500	1,183,983	1,187,465
250'	1,447,198	1,450,680	1,454,163	1,457,645	1,461,128	1,464,610	1,468,092	1,471,575	1,475,057	1,478,540	1,482,022	1,485,505	1,488,987	1,492,469	1,495,952
300'	1,755,684	1,759,167	1,762,649	1,766,132	1,769,614	1,773,097	1,776,579	1,780,061	1,783,544	1,787,026	1,790,509	1,793,991	1,797,474	1,800,956	1,804,438
350'	2,064,171	2,067,653	2,071,136	2,074,618	2,078,101	2,081,583	2,085,066	2,088,548	2,092,030	2,095,513	2,098,995	2,102,478	2,105,960	2,109,443	2,112,925
400'	2,372,658	2,376,140	2,379,622	2,383,105	2,386,587	2,390,070	2,393,552	2,397,035	2,400,517	2,403,999	2,407,482	2,410,964	2,414,447	2,417,929	2,421,412

Table 6.1-4. Initial Dredge Borrow Quantities for Solana Reach Alternatives in cubic yards (completed)

HIGH SEA LEVEL RISE SCENARIO															
Added Beach MSL Width	Renourishment Interval in years														
	22	33	44	55	66	77	88	99	110	111	112	113	114	115	116
50'	233,540	247,675	262,147	276,957	292,106	307,593	323,418	339,581	356,082	372,921	390,099	407,614	425,468	443,660	462,190
100'	542,027	556,161	570,633	585,444	600,592	616,079	631,904	648,067	664,568	681,408	698,585	716,101	733,955	752,147	770,677
150'	850,514	864,648	879,120	893,930	909,079	924,566	940,391	956,554	973,055	989,894	1,007,072	1,024,587	1,042,441	1,060,633	1,079,163
200'	1,159,000	1,173,134	1,187,607	1,202,417	1,217,566	1,233,052	1,248,877	1,265,040	1,281,542	1,298,381	1,315,558	1,333,074	1,350,928	1,369,120	1,387,650
250'	1,467,487	1,481,621	1,496,093	1,510,904	1,526,052	1,541,539	1,557,364	1,573,527	1,590,028	1,606,867	1,624,045	1,641,561	1,659,414	1,677,606	1,696,136
300'	1,775,973	1,790,107	1,804,580	1,819,390	1,834,539	1,850,026	1,865,850	1,882,013	1,898,515	1,915,354	1,932,532	1,950,047	1,967,901	1,986,093	2,004,623
350'	2,084,460	2,098,594	2,113,066	2,127,877	2,143,025	2,158,512	2,174,337	2,190,500	2,207,001	2,223,841	2,241,018	2,258,534	2,276,388	2,294,579	2,313,110
400'	2,392,946	2,407,081	2,421,553	2,436,363	2,451,512	2,466,999	2,482,824	2,498,987	2,515,488	2,532,327	2,549,505	2,567,020	2,584,874	2,603,066	2,621,596

6.5 Replenishment Volumes

Error! Not a valid bookmark self-reference. and **Table 6.5-2** show the replenishment beach fill volumes considered for the Encinitas and Solana segments, respectively. These volumes re-establish the MSL position of the initial beach-fill by replacing losses to alongshore and offshore transport, and to offset the effects from rising sea level as described in **Section 6.1.2**

Table 6.5-1 Replenishment Dredge Borrow Quantities for Encinitas in cubic yards (continued)

LOW SEA LEVEL RISE SCENARIO - 2-YEAR REPLENISHMENT INTERVAL				
YEAR	Initial MSL Beach Width Added			
	50 feet	100 feet	150 feet	200 feet
Initial Borrow 1/	403,291	807,747	1,212,203	1,616,658
2017-2	83,556	115,085	166,380	235,285
2019-2	83,556	115,085	166,380	235,285
2021-2	83,556	115,085	166,380	235,285
2023-2	83,556	115,085	166,380	235,285
2025-2	83,556	115,085	166,380	235,285
2027-2	83,556	115,085	166,380	235,285
2029-2	83,556	115,085	166,380	235,285
2031-2	83,556	115,085	166,380	235,285
2033-2	83,556	115,085	166,380	235,285
2035-2	83,556	115,085	166,380	235,285
2037-2	83,556	115,085	166,380	235,285
2039-2	83,556	115,085	166,380	235,285
2041-2	83,556	115,085	166,380	235,285
2043-2	83,556	115,085	166,380	235,285
2045-2	83,556	115,085	166,380	235,285
2047-2	83,556	115,085	166,380	235,285
2049-2	83,556	115,085	166,380	235,285
2051-2	83,556	115,085	166,380	235,285
2053-2	83,556	115,085	166,380	235,285
2055-2	83,556	115,085	166,380	235,285
2057-2	83,556	115,085	166,380	235,285
2059-2	83,556	115,085	166,380	235,285
2061-2	83,556	115,085	166,380	235,285
2063-2	83,556	115,085	166,380	235,285

1/ Adjusted for remaining volume of RBSPII Project

Table 6.5-2 Replenishment Dredge Borrow Quantities for Encinitas in cubic yards (continued)

HIGH SEA LEVEL RISE SCENARIO 2-YEAR REPLENISHMENT INTERVAL				
YEAR	**Initial MSL Beach Width Added**			
	50 feet	**100 feet**	**150 feet**	**200 feet**
Initial Borrow 1/	425,243	829,699	1,234,155	1,638,610
2017-2	106,972	138,500	189,796	258,701
2019-2	108,435	139,964	191,259	260,164
2021-2	109,899	141,427	192,722	261,627
2023-2	111,362	142,891	194,186	263,091
2025-2	112,825	144,354	195,649	264,554
2027-2	114,289	145,817	197,113	266,018
2029-2	115,752	147,281	198,576	267,481
2031-2	117,216	148,744	200,040	268,945
2033-2	118,679	150,208	201,503	270,408
2035-2	120,143	151,671	202,967	271,872
2037-2	121,606	153,135	204,430	273,335
2039-2	123,070	154,598	205,893	274,799
2041-2	124,533	156,062	207,357	276,262
2043-2	125,997	157,525	208,820	277,725
2045-2	127,460	158,989	210,284	279,189
2047-2	128,923	160,452	211,747	280,652
2049-2	130,387	161,915	213,211	282,116
2051-2	131,850	163,379	214,674	283,579
2053-2	133,314	164,842	216,138	285,043
2055-2	134,777	166,306	217,601	286,506
2057-2	136,241	167,769	219,065	287,970
2059-2	137,704	169,233	220,528	289,433
2061-2	139,168	170,696	221,991	290,897
2063-2	140,631	172,160	223,455	292,360

1/ Adjusted for remaining volume of RBSPII Project

Table 6.5-2. Replenishment Dredge Borrow Quantities for Encinitas in cubic yards (continued)

LOW SEA LEVEL RISE SCENARIO 3-YEAR REPLENISHMENT INTERVAL				
YEAR	Initial MSL Beach Width Added			
	50 feet	100 feet	150 feet	200 feet
Initial Borrow 1/	407,059	811,515	1,215,971	1,620,426
2018-3	154,555	204,513	267,158	346,251
2021-3	154,555	204,513	267,158	346,251
2024-3	154,555	204,513	267,158	346,251
2027-3	154,555	204,513	267,158	346,251
2030-3	154,555	204,513	267,158	346,251
2033-3	154,555	204,513	267,158	346,251
2036-3	154,555	204,513	267,158	346,251
2039-3	154,555	204,513	267,158	346,251
2042-3	154,555	204,513	267,158	346,251
2045-3	154,555	204,513	267,158	346,251
2048-3	154,555	204,513	267,158	346,251
2051-3	154,555	204,513	267,158	346,251
2054-3	154,555	204,513	267,158	346,251
2057-3	154,555	204,513	267,158	346,251
2060-3	154,555	204,513	267,158	346,251
2063-3	150,787	200,745	263,390	342,483

1/ Adjusted for remaining volume of RBSPII Project

Table 6.5-2 Replenishment Dredge Borrow Quantities for Encinitas in cubic yards (continued)

HIGH SEA LEVEL RISE SCENARIO 3-YEAR REPLENISHMENT INTERVAL				
YEAR	Initial MSL Beach Width Added			
	50 feet	100 feet	150 feet	200 feet
Initial Borrow 1/	440,536	844,991	1,249,447	1,653,903
2018-3	191,324	241,282	303,927	383,020
2021-3	194,617	244,575	307,220	386,313
2024-3	197,910	247,868	310,513	389,605
2027-3	201,203	251,161	313,806	392,898
2030-3	204,496	254,453	317,099	396,191
2033-3	207,788	257,746	320,391	399,484
2036-3	211,081	261,039	323,684	402,777
2039-3	214,374	264,332	326,977	406,069
2042-3	217,667	267,625	330,270	409,362
2045-3	220,959	270,917	333,562	412,655
2048-3	224,252	274,210	336,855	415,948
2051-3	227,545	277,503	340,148	419,240
2054-3	230,838	280,796	343,441	422,533
2057-3	234,130	284,088	346,733	425,826
2060-3	237,423	287,381	350,026	429,119
2063-3	207,862	257,820	320,465	399,557

1/ Adjusted for remaining volume of RBSPII Project

Table 6.5-2 Replenishment Dredge Borrow Quantities for Encinitas in cubic yards (continued)

LOW SEA LEVEL RISE SCENARIO - 4-YEAR REPLENISHMENT INTERVAL				
YEAR	Initial MSL Beach Width Added			
	50 feet	100 feet	150 feet	200 feet
Initial Borrow 1/	410,827	815,283	1,219,738	1,624,194
2019-4	224,395	275,636	337,963	421,871
2023-4	224,395	275,636	337,963	421,871
2027-4	224,395	275,636	337,963	421,871
2031-4	224,395	275,636	337,963	421,871
2035-4	224,395	275,636	337,963	421,871
2039-4	224,395	275,636	337,963	421,871
2043-4	224,395	275,636	337,963	421,871
2047-4	224,395	275,636	337,963	421,871
2051-4	224,395	275,636	337,963	421,871
2055-4	224,395	275,636	337,963	421,871
2059-4	224,395	275,636	337,963	421,871
2063-4	216,860	268,100	330,427	414,336

1/ Adjusted for remaining volume of RBSPII Project

Table 6.5-2 Replenishment Dredge Borrow Quantities for Encinitas in cubic yards (continued)

HIGH SEA LEVEL RISE SCENARIO - 4-YEAR REPLENISHMENT INTERVAL				
YEAR	Initial MSL Beach Width Added			
	50 feet	100 feet	150 feet	200 feet
Initial Borrow 1/	456,194	860,650	1,265,106	1,669,561
2019-4	275,616	326,857	389,184	473,092
2023-4	281,470	332,711	395,038	478,946
2027-4	287,324	338,564	400,892	484,800
2031-4	293,178	344,418	406,745	490,654
2035-4	299,032	350,272	412,599	496,507
2039-4	304,885	356,126	418,453	502,361
2043-4	310,739	361,980	424,307	508,215
2047-4	316,593	367,833	430,161	514,069
2051-4	322,447	373,687	436,014	519,923
2055-4	328,301	379,541	441,868	525,777
2059-4	334,155	385,395	447,722	531,630
2063-4	273,935	325,175	387,502	471,410

1/ Adjusted for remaining volume of RBSPII Project

Table 6.5-2 Replenishment Dredge Borrow Quantities for Encinitas in cubic yards (continued)

LOW SEA LEVEL RISE SCENARIO 5-YEAR REPLENISHMENT INTERVAL				
YEAR	**Initial MSL Beach Width Added**			
	50 feet	**100 feet**	**150 feet**	**200 feet**
Initial Borrow 1/	414,595	819,051	1,223,506	1,627,962
2020-5	263,991	336,469	406,725	500,779
2025-5	263,991	336,469	406,725	500,779
2030-5	263,991	336,469	406,725	500,779
2035-5	263,991	336,469	406,725	500,779
2040-5	263,991	336,469	406,725	500,779
2045-5	263,991	336,469	406,725	500,779
2050-5	263,991	336,469	406,725	500,779
2055-5	263,991	336,469	406,725	500,779
2060-5	263,991	336,469	406,725	500,779
HIGH SEA LEVEL RISE SCENARIO 5-YEAR REPLENISHMENT INTERVAL				
YEAR	**Initial MSL Beach Width Added**			
	50 feet	**100 feet**	**150 feet**	**200 feet**
Initial Borrow 1/	472,218	876,674	1,281,130	1,685,585
2020-5	330,761	403,239	473,495	567,549
2025-5	339,907	412,385	482,642	576,696
2030-5	349,054	421,532	491,789	585,843
2035-5	358,201	430,679	500,935	594,989
2040-5	367,347	439,825	510,082	604,136
2045-5	376,494	448,972	519,228	613,282
2050-5	385,640	458,118	528,375	622,429
2055-5	394,787	467,265	537,522	631,576
2060-5	403,933	476,411	546,668	640,722

1/ Adjusted for remaining volume of RBSPII Project

Table 6.5-2 Replenishment Dredge Borrow Quantities for Encinitas in cubic yards (continued)

LOW SEA LEVEL RISE SCENARIO 6-YEAR REPLENISHMENT INTERVAL				
YEAR	**Initial MSL Beach Width Added**			
	50 feet	**100 feet**	**150 feet**	**200 feet**
Initial Borrow 1/	418,363	822,818	1,227,274	1,631,730
2021-6	319,867	435,615	523,777	629,478
2027-6	319,867	435,615	523,777	629,478
2033-6	319,867	435,615	523,777	629,478
2039-6	319,867	435,615	523,777	629,478
2045-6	319,867	435,615	523,777	629,478
2051-6	319,867	435,615	523,777	629,478
2057-6	319,867	435,615	523,777	629,478
2063-6	304,796	420,544	508,706	614,406
HIGH SEA LEVEL RISE SCENARIO 6-YEAR REPLENISHMENT INTERVAL				
YEAR	**Initial MSL Beach Width Added**			
	50 feet	**100 feet**	**150 feet**	**200 feet**
Initial Borrow 1/	488,609	893,064	1,297,520	1,701,976
2021-6	403,284	519,032	607,194	712,895
2027-6	416,455	532,203	620,365	726,066
2033-6	429,626	545,375	633,536	739,237
2039-6	442,797	558,546	646,707	752,408
2045-6	455,969	571,717	659,878	765,579
2051-6	469,140	584,888	673,049	778,750
2057-6	482,311	598,059	686,220	791,921
2063-6	361,871	477,619	565,780	671,481

1/ Adjusted for remaining volume of RBSPII Project

Table 6.5-2 Replenishment Dredge Borrow Quantities for Encinitas in cubic yards (continued)

LOW SEA LEVEL RISE SCENARIO 7-YEAR REPLENISHMENT INTERVAL				
YEAR	Initial MSL Beach Width Added			
	50 feet	100 feet	150 feet	200 feet
Initial Borrow 1/	422,131	826,586	1,231,042	1,635,498
2022-7	380,048	573,404	682,460	798,547
2029-7	380,048	573,404	682,460	798,547
2036-7	380,048	573,404	682,460	798,547
2043-7	380,048	573,404	682,460	798,547
2050-7	380,048	573,404	682,460	798,547
2057-7	380,048	573,404	682,460	798,547
2064-7	357,441	550,797	659,853	775,940
HIGH SEA LEVEL RISE SCENARIO 7-YEAR REPLENISHMENT INTERVAL				
YEAR	**Initial MSL Beach Width Added**			
	50 feet	**100 feet**	**150 feet**	**200 feet**
Initial Borrow 1/	505,365	909,820	1,314,276	1,718,732
2022-7	481,209	674,566	783,622	899,709
2029-7	499,137	692,493	801,549	917,636
2036-7	517,064	710,420	819,476	935,563
2043-7	534,991	728,348	837,404	953,491
2050-7	552,919	746,275	855,331	971,418
2057-7	570,846	764,202	873,258	989,345
2064-7	386,161	579,518	688,574	804,660

1/ Adjusted for remaining volume of RBSPII Project

Table 6.5-2 Replenishment Dredge Borrow Quantities for Encinitas in cubic yards (continued)

LOW SEA LEVEL RISE SCENARIO 8-YEAR REPLENISHMENT INTERVAL				
YEAR	Initial MSL Beach Width Added			
	50 feet	100 feet	150 feet	200 feet
Initial Borrow 1/	425,898	830,354	1,234,810	1,639,266
2023-8	403,931	648,662	758,472	877,714
2031-8	403,931	648,662	758,472	877,714
2039-8	403,931	648,662	758,472	877,714
2047-8	403,931	648,662	758,472	877,714
2055-8	403,931	648,662	758,472	877,714
2063-8	381,324	626,055	735,865	855,107

HIGH SEA LEVEL RISE SCENARIO 8-YEAR REPLENISHMENT INTERVAL				
YEAR	Initial MSL Beach Width Added			
	50 feet	100 feet	150 feet	200 feet
Initial Borrow 1/	522,486	926,942	1,331,398	1,735,854
2023-8	523,934	768,665	878,475	997,717
2031-8	547,349	792,081	901,890	1,021,132
2039-8	570,765	815,496	925,305	1,044,548
2047-8	594,180	838,911	948,721	1,067,963
2055-8	617,595	862,327	972,136	1,091,378
2063-8	438,398	683,130	792,939	912,182

1/ Adjusted for remaining volume of RBSPII Project

Table 6.5-2 Replenishment Dredge Borrow Quantities for Encinitas in cubic yards (continued)

LOW SEA LEVEL RISE SCENARIO 9-YEAR REPLENISHMENT INTERVAL				
YEAR	**Initial MSL Beach Width Added**			
	50 feet	**100 feet**	**150 feet**	**200 feet**
Initial Borrow 1/	429,666	842,822	1,238,578	1,643,033
2024-9	423,215	698,157	845,823	976,020
2033-9	423,215	698,157	845,823	976,020
2042-9	423,215	698,157	845,823	976,020
2051-9	423,215	698,157	845,823	976,020
2060-9	408,143	683,086	830,752	960,949
HIGH SEA LEVEL RISE SCENARIO 9-YEAR REPLENISHMENT INTERVAL				
YEAR	**Initial MSL Beach Width Added**			
	50 feet	**100 feet**	**150 feet**	**200 feet**
Initial Borrow 1/	539,974	944,430	1,348,886	1,753,341
2024-9	563,158	838,100	985,766	1,115,963
2033-9	592,793	867,735	1,015,401	1,145,598
2042-9	622,428	897,370	1,045,036	1,175,233
2051-9	652,063	927,005	1,074,671	1,204,868
2060-9	548,086	823,029	970,695	1,100,891

1/ Adjusted for remaining volume of RBSPII Project

Table 6.5-2 Replenishment Dredge Borrow Quantities for Encinitas in cubic yards (continued)

LOW SEA LEVEL RISE SCENARIO - 10-YEAR REPLENISHMENT INTERVAL

YEAR	Initial MSL Beach Width Added			
	50 feet	100 feet	150 feet	200 feet
Initial Borrow 1/	433,434	837,890	1,242,346	1,646,801
2025-10	427,639	717,732	908,667	1,057,192
2035-10	427,639	717,732	908,667	1,057,192
2045-10	427,639	717,732	908,667	1,057,192
2055-10	427,639	717,732	908,667	1,057,192

HIGH SEA LEVEL RISE SCENARIO 10-YEAR REPLENISHMENT INTERVAL

YEAR	Initial MSL Beach Width Added			
	50 feet	100 feet	150 feet	200 feet
Initial Borrow 1/	557,828	962,283	1,366,739	1,771,195
2025-10	588,619	878,712	1,069,647	1,218,172
2035-10	625,206	915,299	1,106,233	1,254,759
2045-10	661,792	951,885	1,142,819	1,291,345
2055-10	698,378	988,471	1,179,406	1,327,932

1/ Adjusted for remaining volume of RBSPII Project

Table 6.5-2 Replenishment Dredge Borrow Quantities for Encinitas in cubic yards (continued)

LOW SEA LEVEL RISE SCENARIO 11-YEAR REPLENISHMENT INTERVAL				
YEAR	Initial MSL Beach Width Added			
	50 feet	100 feet	150 feet	200 feet
Initial Borrow 1/	437,202	841,658	1,246,113	1,650,569
2026-11	437,533	760,367	974,462	1,151,674
2037-11	437,533	760,367	974,462	1,151,674
2048-11	437,533	760,367	974,462	1,151,674
2059-11	418,694	741,528	955,623	1,132,835
HIGH SEA LEVEL RISE SCENARIO 11-YEAR REPLENISHMENT INTERVAL				
YEAR	**Initial MSL Beach Width Added**			
	50 feet	**100 feet**	**150 feet**	**200 feet**
Initial Borrow 1/	576,047	980,503	1,384,959	1,789,414
2026-11	620,648	943,482	1,157,577	1,334,789
2037-11	664,917	987,751	1,201,846	1,379,058
2048-11	709,187	1,032,021	1,246,116	1,423,328
2059-11	585,528	908,362	1,122,457	1,299,669

1/ Adjusted for remaining volume of RBSPII Project

Table 6.5-2 Replenishment Dredge Borrow Quantities for Encinitas in cubic yards (continued)

LOW SEA LEVEL RISE SCENARIO 12-YEAR REPLENISHMENT INTERVAL				
YEAR	Initial MSL Beach Width Added			
	50 feet	100 feet	150 feet	200 feet
Initial Borrow 1/	440,970	845,426	1,249,881	1,654,337
2027-12	440,246	812,485	1,023,433	1,202,739
2039-12	440,246	812,485	1,023,433	1,202,739
2051-12	440,246	812,485	1,023,433	1,202,739
2063-12	402,568	774,806	985,755	1,165,061
HIGH SEA LEVEL RISE SCENARIO 12-YEAR REPLENISHMENT INTERVAL				
YEAR	**Initial MSL Beach Width Added**			
	50 feet	**100 feet**	**150 feet**	**200 feet**
Initial Borrow 1/	594,633	999,088	1,403,544	1,808,000
2027-12	646,593	1,018,832	1,229,780	1,409,086
2039-12	699,278	1,071,516	1,282,465	1,461,771
2051-12	751,962	1,124,201	1,335,149	1,514,455
2063-12	459,642	831,881	1,042,830	1,222,135

1/ Adjusted for remaining volume of RBSPII Project

Table 6.5-2 Replenishment Dredge Borrow Quantities for Encinitas in cubic yards (continued)

LOW SEA LEVEL RISE SCENARIO 13-YEAR REPLENISHMENT INTERVAL				
YEAR	Initial MSL Beach Width Added			
	50 feet	100 feet	150 feet	200 feet
Initial Borrow 1/	444,738	849,193	1,253,649	1,658,105
2028-13	451,397	812,554	1,065,363	1,256,230
2041-13	451,397	812,554	1,065,363	1,256,230
2054-13	443,861	805,018	1,057,827	1,248,694
HIGH SEA LEVEL RISE SCENARIO 13-YEAR REPLENISHMENT INTERVAL				
YEAR	**Initial MSL Beach Width Added**			
	50 feet	**100 feet**	**150 feet**	**200 feet**
Initial Borrow 1/	613,584	1,018,039	1,422,495	1,826,951
2028-13	682,074	1,043,231	1,296,040	1,486,907
2041-13	743,905	1,105,062	1,357,871	1,548,738
2054-13	739,662	1,100,819	1,353,628	1,544,495

1/ Adjusted for remaining volume of RBSPII Project

Table 6.5-2 Replenishment Dredge Borrow Quantities for Encinitas in cubic yards (continued)

LOW SEA LEVEL RISE SCENARIO 14-YEAR REPLENISHMENT INTERVAL				
YEAR	Initial MSL Beach Width Added			
	50 feet	100 feet	150 feet	200 feet
Initial Borrow 1/	448,506	852,961	1,257,417	1,661,873
2029-14	457,133	831,052	1,108,036	1,337,857
2043-14	457,133	831,052	1,108,036	1,337,857
2057-14	434,526	808,445	1,085,429	1,315,249
HIGH SEA LEVEL RISE SCENARIO 14-YEAR REPLENISHMENT INTERVAL				
YEAR	**Initial MSL Beach Width Added**			
	50 feet	**100 feet**	**150 feet**	**200 feet**
Initial Borrow 1/	632,901	1,037,357	1,441,812	1,846,268
2029-14	713,238	1,087,157	1,364,141	1,593,961
2043-14	784,947	1,158,866	1,435,850	1,665,670
2057-14	654,044	1,027,963	1,304,947	1,534,768

1/ Adjusted for remaining volume of RBSPII Project

Table 6.5-2 Replenishment Dredge Borrow Quantities for Encinitas in cubic yards (continued)

LOW SEA LEVEL RISE SCENARIO 15-YEAR REPLENISHMENT INTERVAL				
YEAR	**Initial MSL Beach Width Added**			
	50 feet	**100 feet**	**150 feet**	**200 feet**
Initial Borrow 1/	452,273	856,729	1,261,185	1,665,640
2030-15	460,971	853,950	1,160,022	1,439,360
2045-15	460,971	853,950	1,160,022	1,439,360
2060-15	423,293	816,272	1,122,343	1,401,682
HIGH SEA LEVEL RISE SCENARIO 15-YEAR REPLENISHMENT INTERVAL				
YEAR	**Initial MSL Beach Width Added**			
	50 feet	**100 feet**	**150 feet**	**200 feet**
Initial Borrow 1/	652,584	1,057,039	1,461,495	1,865,951
2030-15	743,601	1,136,580	1,442,651	1,721,990
2045-15	825,920	1,218,899	1,524,971	1,804,309
2060-15	563,236	956,215	1,262,286	1,541,625

1/ Adjusted for remaining volume of RBSPII Project

Table 6.5-2 Replenishment Dredge Borrow Quantities for Encinitas in cubic yards (completed)

LOW SEA LEVEL RISE SCENARIO 16-YEAR REPLENISHMENT INTERVAL				
YEAR	**Initial MSL Beach Width Added**			
	50 feet	**100 feet**	**150 feet**	**200 feet**
Initial Borrow 1/	456,041	860,497	1,264,953	1,669,408
2031-16	464,250	857,236	1,184,959	1,483,041
2047-16	464,250	857,236	1,184,959	1,483,041
2063-16	411,500	804,486	1,132,209	1,430,291
HIGH SEA LEVEL RISE SCENARIO 16-YEAR REPLENISHMENT INTERVAL				
YEAR	**Initial MSL Beach Width Added**			
	50 feet	**100 feet**	**150 feet**	**200 feet**
Initial Borrow 1/	672,633	1,077,088	1,481,544	1,886,000
2031-16	774,503	1,167,488	1,495,212	1,793,294
2047-16	868,164	1,261,149	1,588,873	1,886,955
2063-16	468,575	861,561	1,189,284	1,487,366

1/ Adjusted for remaining volume of RBSPII Project

Table 6.5-3 Replenishment Dredge Borrow Quantities for Solana in cubic yards (continued)

LOW SEA LEVEL RISE SCENARIO 2-YEAR REPLENISHMENT INTERVAL								
YEAR	Initial MSL Beach Width Added							
	50 feet	100 feet	150 feet	200 feet	250 feet	300-feet	350-feet	400 feet
Initial Borrow 1/	213,251	521,738	830,225	1,138,711	1,447,198	1,755,684	2,064,171	2,372,658
2017-2	76,627	117,719	170,821	226,671	282,087	336,286	388,045	437,239
2019-2	76,627	117,719	170,821	226,671	282,087	336,286	388,045	437,239
2021-2	76,627	117,719	170,821	226,671	282,087	336,286	388,045	437,239
2023-2	76,627	117,719	170,821	226,671	282,087	336,286	388,045	437,239
2025-2	76,627	117,719	170,821	226,671	282,087	336,286	388,045	437,239
2027-2	76,627	117,719	170,821	226,671	282,087	336,286	388,045	437,239
2029-2	76,627	117,719	170,821	226,671	282,087	336,286	388,045	437,239
2031-2	76,627	117,719	170,821	226,671	282,087	336,286	388,045	437,239
2033-2	76,627	117,719	170,821	226,671	282,087	336,286	388,045	437,239
2035-2	76,627	117,719	170,821	226,671	282,087	336,286	388,045	437,239
2037-2	76,627	117,719	170,821	226,671	282,087	336,286	388,045	437,239
2039-2	76,627	117,719	170,821	226,671	282,087	336,286	388,045	437,239
2041-2	76,627	117,719	170,821	226,671	282,087	336,286	388,045	437,239
2043-2	76,627	117,719	170,821	226,671	282,087	336,286	388,045	437,239
2045-2	76,627	117,719	170,821	226,671	282,087	336,286	388,045	437,239
2047-2	76,627	117,719	170,821	226,671	282,087	336,286	388,045	437,239
2049-2	76,627	117,719	170,821	226,671	282,087	336,286	388,045	437,239
2051-2	76,627	117,719	170,821	226,671	282,087	336,286	388,045	437,239
2053-2	76,627	117,719	170,821	226,671	282,087	336,286	388,045	437,239
2055-2	76,627	117,719	170,821	226,671	282,087	336,286	388,045	437,239
2057-2	76,627	117,719	170,821	226,671	282,087	336,286	388,045	437,239
2059-2	76,627	117,719	170,821	226,671	282,087	336,286	388,045	437,239
2061-2	76,627	117,719	170,821	226,671	282,087	336,286	388,045	437,239
2063-2	76,627	117,719	170,821	226,671	282,087	336,286	388,045	437,239

1/ Adjusted for remaining volume of RBSPII Project

Table 6.5-3 Replenishment Dredge Borrow Quantities for Solana in cubic yards (continued)

HIGH SEA LEVEL RISE SCENARIO 2-YEAR REPLENISHMENT INTERVAL								
YEAR	Initial MSL Beach Width Added							
	50 feet	100 feet	150 feet	200 feet	250 feet	300-feet	350-feet	400 feet
Initial Borrow 1/	233,540	542,027	850,514	1,159,000	1,467,487	1,775,973	2,084,460	2,392,946
2017-2	98,268	139,360	192,462	248,312	303,729	357,928	409,686	458,880
2019-2	99,621	140,713	193,815	249,665	305,082	359,281	411,039	460,233
2021-2	100,974	142,066	195,167	251,017	306,434	360,633	412,391	461,586
2023-2	102,326	143,418	196,520	252,370	307,787	361,986	413,744	462,938
2025-2	103,679	144,771	197,873	253,723	309,139	363,338	415,097	464,291
2027-2	105,031	146,123	199,225	255,075	310,492	364,691	416,449	465,643
2029-2	106,384	147,476	200,578	256,428	311,845	366,044	417,802	466,996
2031-2	107,737	148,829	201,930	257,780	313,197	367,396	419,154	468,349
2033-2	109,089	150,181	203,283	259,133	314,550	368,749	420,507	469,701
2035-2	110,442	151,534	204,636	260,485	315,902	370,101	421,859	471,054
2037-2	111,794	152,886	205,988	261,838	317,255	371,454	423,212	472,406
2039-2	113,147	154,239	207,341	263,191	318,608	372,807	424,565	473,759
2041-2	114,500	155,592	208,693	264,543	319,960	374,159	425,917	475,112
2043-2	115,852	156,944	210,046	265,896	321,313	375,512	427,270	476,464
2045-2	117,205	158,297	211,399	267,248	322,665	376,864	428,622	477,817
2047-2	118,557	159,649	212,751	268,601	324,018	378,217	429,975	479,169
2049-2	119,910	161,002	214,104	269,954	325,371	379,570	431,328	480,522
2051-2	121,263	162,355	215,456	271,306	326,723	380,922	432,680	481,875
2053-2	122,615	163,707	216,809	272,659	328,076	382,275	434,033	483,227
2055-2	123,968	165,060	218,162	274,011	329,428	383,627	435,385	484,580
2057-2	125,320	166,412	219,514	275,364	330,781	384,980	436,738	485,932
2059-2	126,673	167,765	220,867	276,717	332,134	386,333	438,091	487,285
2061-2	128,026	169,118	222,219	278,069	333,486	387,685	439,443	488,638
2063-2	129,378	170,470	223,572	279,422	334,839	389,038	440,796	489,990

1/ Adjusted for remaining volume of RBSPII Project

Table 6.5-3. Replenishment Dredge Borrow Quantities for Solana in cubic yards (continued)

LOW SEA LEVEL RISE SCENARIO 3-YEAR REPLENISHMENT INTERVAL								
YEAR	**Initial MSL Beach Width Added**							
	50 feet	**100 feet**	**150 feet**	**200 feet**	**250 feet**	**300-feet**	**350-feet**	**400 feet**
Initial Borrow 1/	216,734	525,220	833,707	1,142,194	1,450,680	1,759,167	2,067,653	2,376,140
2018-3	87,743	122,996	181,675	250,459	315,061	378,451	439,930	497,976
2021-3	87,743	122,996	181,675	250,459	315,061	378,451	439,930	497,976
2024-3	87,743	122,996	181,675	250,459	315,061	378,451	439,930	497,976
2027-3	87,743	122,996	181,675	250,459	315,061	378,451	439,930	497,976
2030-3	87,743	122,996	181,675	250,459	315,061	378,451	439,930	497,976
2033-3	87,743	122,996	181,675	250,459	315,061	378,451	439,930	497,976
2036-3	87,743	122,996	181,675	250,459	315,061	378,451	439,930	497,976
2039-3	87,743	122,996	181,675	250,459	315,061	378,451	439,930	497,976
2042-3	87,743	122,996	181,675	250,459	315,061	378,451	439,930	497,976
2045-3	87,743	122,996	181,675	250,459	315,061	378,451	439,930	497,976
2048-3	87,743	122,996	181,675	250,459	315,061	378,451	439,930	497,976
2051-3	87,743	122,996	181,675	250,459	315,061	378,451	439,930	497,976
2054-3	87,743	122,996	181,675	250,459	315,061	378,451	439,930	497,976
2057-3	87,743	122,996	181,675	250,459	315,061	378,451	439,930	497,976
2060-3	87,743	122,996	181,675	250,459	315,061	378,451	439,930	497,976
2063-3	84,261	119,514	178,193	246,977	311,579	374,968	436,448	494,494

1/ Adjusted for remaining volume of RBSPII Project

Table 6.5-3 Replenishment Dredge Borrow Quantities for Solana in cubic yards (continued)

HIGH SEA LEVEL RISE SCENARIO 3-YEAR REPLENISHMENT INTERVAL								
YEAR	Initial MSL Beach Width Added							
	50 feet	100 feet	150 feet	200 feet	250 feet	300-feet	350-feet	400 feet
Initial Borrow 1/	247,675	556,161	864,648	1,173,134	1,481,621	1,790,107	2,098,594	2,407,081
2018-3	121,727	156,980	215,659	284,443	349,045	412,435	473,914	531,960
2021-3	124,770	160,024	218,703	287,487	352,089	415,478	476,957	535,003
2024-3	127,814	163,067	221,746	290,530	355,132	418,521	480,001	538,047
2027-3	130,857	166,110	224,789	293,573	358,175	421,565	483,044	541,090
2030-3	133,900	169,154	227,833	296,617	361,219	424,608	486,087	544,133
2033-3	136,944	172,197	230,876	299,660	364,262	427,651	489,131	547,177
2036-3	139,987	175,240	233,919	302,703	367,305	430,695	492,174	550,220
2039-3	143,030	178,284	236,963	305,747	370,349	433,738	495,218	553,263
2042-3	146,074	181,327	240,006	308,790	373,392	436,781	498,261	556,307
2045-3	149,117	184,370	243,049	311,833	376,435	439,825	501,304	559,350
2048-3	152,160	187,414	246,093	314,877	379,479	442,868	504,348	562,394
2051-3	155,204	190,457	249,136	317,920	382,522	445,911	507,391	565,437
2054-3	158,247	193,501	252,180	320,963	385,566	448,955	510,434	568,480
2057-3	161,290	196,544	255,223	324,007	388,609	451,998	513,478	571,524
2060-3	164,334	199,587	258,266	327,050	391,652	455,041	516,521	574,567
2063-3	137,012	172,265	230,944	299,728	364,330	427,719	489,199	547,245

1/ Adjusted for remaining volume of RBSPII Project

Table 6.5-3 Replenishment Dredge Borrow Quantities for Solana in cubic yards (continued)

LOW SEA LEVEL RISE SCENARIO - 4-YEAR REPLENISHMENT INTERVAL								
YEAR	Initial MSL Beach Width Added							
	50 feet	100 feet	150 feet	200 feet	250 feet	300-feet	350-feet	400 feet
Initial Borrow 1/	220,216	528,703	837,189	1,145,676	1,454,163	1,762,649	2,071,136	2,379,622
2019-4	104,047	129,862	191,951	271,455	356,577	439,317	518,027	591,076
2023-4	104,047	129,862	191,951	271,455	356,577	439,317	518,027	591,076
2027-4	104,047	129,862	191,951	271,455	356,577	439,317	518,027	591,076
2031-4	104,047	129,862	191,951	271,455	356,577	439,317	518,027	591,076
2035-4	104,047	129,862	191,951	271,455	356,577	439,317	518,027	591,076
2039-4	104,047	129,862	191,951	271,455	356,577	439,317	518,027	591,076
2043-4	104,047	129,862	191,951	271,455	356,577	439,317	518,027	591,076
2047-4	104,047	129,862	191,951	271,455	356,577	439,317	518,027	591,076
2051-4	104,047	129,862	191,951	271,455	356,577	439,317	518,027	591,076
2055-4	104,047	129,862	191,951	271,455	356,577	439,317	518,027	591,076
2059-4	104,047	129,862	191,951	271,455	356,577	439,317	518,027	591,076
2063-4	97,082	122,897	184,986	264,490	349,612	432,352	511,062	584,112

1/ Adjusted for remaining volume of RBSPII Project

Table 6.5-3 Replenishment Dredge Borrow Quantities for Solana in cubic yards (continued)

HIGH SEA LEVEL RISE SCENARIO - 4-YEAR REPLENISHMENT INTERVAL								
YEAR	Initial MSL Beach Width Added							
	50 feet	100 feet	150 feet	200 feet	250 feet	300-feet	350-feet	400 feet
Initial Borrow 1/	262,147	570,633	879,120	1,187,607	1,496,093	1,804,580	2,113,066	2,421,553
2019-4	151,388	177,203	239,292	318,796	403,918	486,658	565,368	638,417
2023-4	156,798	182,613	244,702	324,207	409,328	492,068	570,778	643,828
2027-4	162,209	188,024	250,112	329,617	414,739	497,478	576,188	649,238
2031-4	167,619	193,434	255,523	335,027	420,149	502,889	581,599	654,649
2035-4	173,030	198,845	260,933	340,438	425,560	508,299	587,009	660,059
2039-4	178,440	204,255	266,344	345,848	430,970	513,710	592,419	665,469
2043-4	183,850	209,665	271,754	351,259	436,380	519,120	597,830	670,880
2047-4	189,261	215,076	277,164	356,669	441,791	524,530	603,240	676,290
2051-4	194,671	220,486	282,575	362,079	447,201	529,941	608,651	681,701
2055-4	200,082	225,897	287,985	367,490	452,612	535,351	614,061	687,111
2059-4	205,492	231,307	293,396	372,900	458,022	540,762	619,471	692,521
2063-4	149,834	175,649	237,737	317,242	402,364	485,103	563,813	636,863

1/ Adjusted for remaining volume of RBSPII Project

Table 6.5-3 Replenishment Dredge Borrow Quantities for Solana in cubic yards (continued)

LOW SEA LEVEL RISE SCENARIO 5-YEAR REPLENISHMENT INTERVAL								
YEAR	Initial MSL Beach Width Added							
	50 feet	100 feet	150 feet	200 feet	250 feet	300-feet	350-feet	400 feet
Initial Borrow 1/	223,699	532,185	840,672	1,149,158	1,457,645	1,766,132	2,074,618	2,383,105
2020-5	135,728	158,139	210,776	290,377	383,181	475,105	561,736	641,325
2025-5	135,728	158,139	210,776	290,377	383,181	475,105	561,736	641,325
2030-5	135,728	158,139	210,776	290,377	383,181	475,105	561,736	641,325
2035-5	135,728	158,139	210,776	290,377	383,181	475,105	561,736	641,325
2040-5	135,728	158,139	210,776	290,377	383,181	475,105	561,736	641,325
2045-5	135,728	158,139	210,776	290,377	383,181	475,105	561,736	641,325
2050-5	135,728	158,139	210,776	290,377	383,181	475,105	561,736	641,325
2055-5	135,728	158,139	210,776	290,377	383,181	475,105	561,736	641,325
2060-5	135,728	158,139	210,776	290,377	383,181	475,105	561,736	641,325

1/ Adjusted for remaining volume of RBSPII Project

Table 6.5-3. Replenishment Dredge Borrow Quantities for Solana in cubic yards (continued)

HIGH SEA LEVEL RISE SCENARIO 5-YEAR REPLENISHMENT INTERVAL								
YEAR	Initial MSL Beach Width Added							
	50 feet	100 feet	150 feet	200 feet	250 feet	300-feet	350-feet	400 feet
Initial Borrow 1/	276,957	585,444	893,930	1,202,417	1,510,904	1,819,390	2,127,877	2,436,363
2020-5	197,440	219,851	272,488	352,090	444,894	536,817	623,448	703,037
2025-5	205,894	228,305	280,942	360,543	453,347	545,271	631,902	711,491
2030-5	214,348	236,758	289,396	368,997	461,801	553,725	640,356	719,944
2035-5	222,802	245,212	297,850	377,451	470,255	562,179	648,810	728,398
2040-5	231,255	253,666	306,303	385,905	478,709	570,632	657,263	736,852
2045-5	239,709	262,120	314,757	394,358	487,162	579,086	665,717	745,306
2050-5	248,163	270,573	323,211	402,812	495,616	587,540	674,171	753,759
2055-5	256,617	279,027	331,664	411,266	504,070	595,994	682,625	762,213
2060-5	265,070	287,481	340,118	419,720	512,524	604,447	691,078	770,667

1/ Adjusted for remaining volume of RBSPII Project

Table 6.5-3 Replenishment Dredge Borrow Quantities for Solana in cubic yards (continued)

LOW SEA LEVEL RISE SCENARIO 6-YEAR REPLENISHMENT INTERVAL								
YEAR	Initial MSL Beach Width Added							
	50 feet	100 feet	150 feet	200 feet	250 feet	300-feet	350-feet	400 feet
Initial Borrow 1/	227,181	535,668	844,154	1,152,641	1,461,128	1,769,614	2,078,101	2,386,587
2021-6	168,941	198,032	239,154	304,112	381,557	468,817	552,465	631,489
2027-6	168,941	198,032	239,154	304,112	381,557	468,817	552,465	631,489
2033-6	168,941	198,032	239,154	304,112	381,557	468,817	552,465	631,489
2039-6	168,941	198,032	239,154	304,112	381,557	468,817	552,465	631,489
2045-6	168,941	198,032	239,154	304,112	381,557	468,817	552,465	631,489
2051-6	168,941	198,032	239,154	304,112	381,557	468,817	552,465	631,489
2057-6	168,941	198,032	239,154	304,112	381,557	468,817	552,465	631,489
2063-6	155,011	184,102	225,225	290,183	367,627	454,888	538,536	617,559

1/ Adjusted for remaining volume of RBSPII Project

Table 6.5-3 Replenishment Dredge Borrow Quantities for Solana in cubic yards (continued)

HIGH SEA LEVEL RISE SCENARIO 6-YEAR REPLENISHMENT INTERVAL								
YEAR	Initial MSL Beach Width Added							
	50 feet	100 feet	150 feet	200 feet	250 feet	300-feet	350-feet	400 feet
Initial Borrow 1/	292,106	600,592	909,079	1,217,566	1,526,052	1,834,539	2,143,025	2,451,512
2021-6	246,039	275,130	316,253	381,211	458,655	545,915	629,564	708,587
2027-6	258,212	287,303	328,426	393,384	470,828	558,089	641,737	720,760
2033-6	270,386	299,476	340,599	405,557	483,002	570,262	653,910	732,934
2039-6	282,559	311,650	352,773	417,731	495,175	582,436	666,084	745,107
2045-6	294,733	323,823	364,946	429,904	507,348	594,609	678,257	757,280
2051-6	306,906	335,997	377,119	442,077	519,522	606,782	690,430	769,454
2057-6	319,079	348,170	389,293	454,251	531,695	618,956	702,604	781,627
2063-6	207,763	236,853	277,976	342,934	420,378	507,639	591,287	670,310

1/ Adjusted for remaining volume of RBSPII Project

Table 6.5-3 Replenishment Dredge Borrow Quantities for Solana in cubic yards (continued)

LOW SEA LEVEL RISE SCENARIO 7-YEAR REPLENISHMENT INTERVAL								
YEAR	Initial MSL Beach Width Added							
	50 feet	100 feet	150 feet	200 feet	250 feet	300-feet	350-feet	400 feet
Initial Borrow 1/	230,664	539,150	847,637	1,156,123	1,464,610	1,773,097	2,081,583	2,390,070
2022-7	202,486	229,318	269,495	334,366	420,834	510,700	596,146	677,932
2029-7	202,486	229,318	269,495	334,366	420,834	510,700	596,146	677,932
2036-7	202,486	229,318	269,495	334,366	420,834	510,700	596,146	677,932
2043-7	202,486	229,318	269,495	334,366	420,834	510,700	596,146	677,932
2050-7	202,486	229,318	269,495	334,366	420,834	510,700	596,146	677,932
2057-7	202,486	229,318	269,495	334,366	420,834	510,700	596,146	677,932
2064-7	181,591	208,423	248,601	313,471	399,939	489,806	575,251	657,037

1/ Adjusted for remaining volume of RBSPII Project

Table 6.5-3 Replenishment Dredge Borrow Quantities for Solana in cubic yards (continued)

HIGH SEA LEVEL RISE SCENARIO 7-YEAR REPLENISHMENT INTERVAL								
YEAR	Initial MSL Beach Width Added							
	50 feet	100 feet	150 feet	200 feet	250 feet	300-feet	350-feet	400 feet
Initial Borrow 1/	307,593	616,079	924,566	1,233,052	1,541,539	1,850,026	2,158,512	2,466,999
2022-7	295,984	322,816	362,994	427,864	514,332	604,199	689,644	771,430
2029-7	312,554	339,386	379,563	444,434	530,901	620,768	706,214	788,000
2036-7	329,123	355,955	396,132	461,003	547,470	637,337	722,783	804,569
2043-7	345,692	372,524	412,702	477,572	564,040	653,907	739,352	821,138
2050-7	362,262	389,094	429,271	494,142	580,609	670,476	755,922	837,708
2057-7	378,831	405,663	445,840	510,711	597,178	687,045	772,491	854,277
2064-7	208,136	234,968	275,146	340,016	426,484	516,350	601,796	683,582

1/ Adjusted for remaining volume of RBSPII Project

Table 6.5-3 Replenishment Dredge Borrow Quantities for Solana in cubic yards (continued)

LOW SEA LEVEL RISE SCENARIO 8-YEAR REPLENISHMENT INTERVAL								
YEAR	Initial MSL Beach Width Added							
	50 feet	100 feet	150 feet	200 feet	250 feet	300-feet	350-feet	400 feet
Initial Borrow 1/	234,146	542,633	851,119	1,159,606	1,468,092	1,776,579	2,085,066	2,393,552
2023-8	224,280	248,771	290,463	342,317	425,448	524,241	622,447	717,662
2031-8	224,280	248,771	290,463	342,317	425,448	524,241	622,447	717,662
2039-8	224,280	248,771	290,463	342,317	425,448	524,241	622,447	717,662
2047-8	224,280	248,771	290,463	342,317	425,448	524,241	622,447	717,662
2055-8	224,280	248,771	290,463	342,317	425,448	524,241	622,447	717,662
2063-8	203,385	227,876	269,569	321,423	404,553	503,346	601,552	696,768

1/ Adjusted for remaining volume of RBSPII Project

Table 6.5-3. Replenishment Dredge Borrow Quantities for Solana in cubic yards (continued)

HIGH SEA LEVEL RISE SCENARIO 8-YEAR REPLENISHMENT INTERVAL								
YEAR	Initial MSL Beach Width Added							
	50 feet	100 feet	150 feet	200 feet	250 feet	300-feet	350-feet	400 feet
Initial Borrow 1/	323,418	631,904	940,391	1,248,877	1,557,364	1,865,850	2,174,337	2,482,824
2023-8	335,193	359,684	401,376	453,230	536,361	635,154	733,360	828,575
2031-8	356,834	381,325	423,018	474,872	558,003	656,796	755,001	850,217
2039-8	378,476	402,967	444,659	496,513	579,644	678,437	776,643	871,858
2047-8	400,117	424,608	466,301	518,155	601,286	700,079	798,285	893,500
2055-8	421,759	446,250	487,943	539,797	622,927	721,720	819,926	915,141
2063-8	256,136	280,627	322,320	374,174	457,305	556,098	654,304	749,519

1/ Adjusted for remaining volume of RBSPII Project

Table 6.5-3 Replenishment Dredge Borrow Quantities for Solana in cubic yards (continued)

LOW SEA LEVEL RISE SCENARIO 9-YEAR REPLENISHMENT INTERVAL								
YEAR	**Initial MSL Beach Width Added**							
	50 feet	**100 feet**	**150 feet**	**200 feet**	**250 feet**	**300-feet**	**350-feet**	**400 feet**
Initial Borrow 1/	237,628	546,115	854,602	1,163,088	1,471,575	1,780,061	2,088,548	2,397,035
2024-9	253,864	281,375	322,342	386,008	471,998	565,706	664,114	757,154
2033-9	253,864	281,375	322,342	386,008	471,998	565,706	664,114	757,154
2042-9	253,864	281,375	322,342	386,008	471,998	565,706	664,114	757,154
2051-9	253,864	281,375	322,342	386,008	471,998	565,706	664,114	757,154
2060-9	239,934	267,445	308,412	372,078	458,069	551,777	650,185	743,225
HIGH SEA LEVEL RISE SCENARIO 9-YEAR REPLENISHMENT INTERVAL								
YEAR	**Initial MSL Beach Width Added**							
	50 feet	**100 feet**	**150 feet**	**200 feet**	**250 feet**	**300-feet**	**350-feet**	**400 feet**
Initial Borrow 1/	339,581	648,067	956,554	1,265,040	1,573,527	1,882,013	2,190,500	2,498,987
2024-9	383,206	410,717	451,684	515,350	601,341	695,048	793,456	886,497
2033-9	410,596	438,107	479,074	542,740	628,731	722,439	820,847	913,887
2042-9	437,986	465,497	506,464	570,130	656,121	749,829	848,237	941,277
2051-9	465,376	492,888	533,854	597,520	683,511	777,219	875,627	968,667
2060-9	369,276	396,787	437,754	501,420	587,411	681,119	779,527	872,567

1/ Adjusted for remaining volume of RBSPII Project

Table 6.5-3. Replenishment Dredge Borrow Quantities for Solana in cubic yards (continued)

LOW SEA LEVEL RISE SCENARIO 10-YEAR REPLENISHMENT INTERVAL								
YEAR	Initial MSL Beach Width Added							
	50 feet	100 feet	150 feet	200 feet	250 feet	300-feet	350-feet	400 feet
Initial Borrow 1/	241,111	549,598	858,084	1,166,571	1,475,057	1,783,544	2,092,030	2,400,517
2025-10	283,219	309,358	351,371	411,684	500,760	596,978	689,456	787,127
2035-10	283,219	309,358	351,371	411,684	500,760	596,978	689,456	787,127
2045-10	283,219	309,358	351,371	411,684	500,760	596,978	689,456	787,127
2055-10	283,219	309,358	351,371	411,684	500,760	596,978	689,456	787,127
HIGH SEA LEVEL RISE SCENARIO 10-YEAR REPLENISHMENT INTERVAL								
YEAR	Initial MSL Beach Width Added							
	50 feet	100 feet	150 feet	200 feet	250 feet	300-feet	350-feet	400 feet
Initial Borrow 1/	356,082	664,568	973,055	1,281,542	1,590,028	1,898,515	2,207,001	2,515,488
2025-10	432,005	458,143	500,157	560,470	649,546	745,764	838,242	935,913
2035-10	465,820	491,958	533,972	594,285	683,361	779,579	872,056	969,728
2045-10	499,635	525,773	567,787	628,100	717,176	813,394	905,871	1,003,543
2055-10	533,450	559,588	601,602	661,915	750,991	847,209	939,686	1,037,358

1/ Adjusted for remaining volume of RBSPII Project

Table 6.5-3 Replenishment Dredge Borrow Quantities for Solana in cubic yards (continued)

LOW SEA LEVEL RISE SCENARIO 11-YEAR REPLENISHMENT INTERVAL								
YEAR	Initial MSL Beach Width Added							
	50 feet	100 feet	150 feet	200 feet	250 feet	300-feet	350-feet	400 feet
Initial Borrow 1/	244,593	553,080	861,567	1,170,053	1,478,540	1,787,026	2,095,513	2,403,999
2026-11	311,553	343,409	384,895	438,879	521,645	620,743	720,560	823,830
2037-11	311,553	343,409	384,895	438,879	521,645	620,743	720,560	823,830
2048-11	311,553	343,409	384,895	438,879	521,645	620,743	720,560	823,830
2059-11	294,141	325,996	367,483	421,467	504,233	603,331	703,148	806,418
HIGH SEA LEVEL RISE SCENARIO 11-YEAR REPLENISHMENT INTERVAL								
YEAR	Initial MSL Beach Width Added							
	50 feet	100 feet	150 feet	200 feet	250 feet	300-feet	350-feet	400 feet
Initial Borrow 1/	372,921	681,408	989,894	1,298,381	1,606,867	1,915,354	2,223,841	2,532,327
2026-11	480,797	512,652	554,139	608,123	690,889	789,987	889,804	993,074
2037-11	521,713	553,568	595,055	649,039	731,805	830,903	930,720	1,033,990
2048-11	562,629	594,485	635,971	689,955	772,721	871,819	971,636	1,074,906
2059-11	448,337	480,193	521,679	575,663	658,429	757,527	857,344	960,614

1/ Adjusted for remaining volume of RBSPII Project

Table 6.5-3 Replenishment Dredge Borrow Quantities for Solana in cubic yards (continued)

LOW SEA LEVEL RISE SCENARIO 12-YEAR REPLENISHMENT INTERVAL								
YEAR	**Initial MSL Beach Width Added**							
	50 feet	**100 feet**	**150 feet**	**200 feet**	**250 feet**	**300-feet**	**350-feet**	**400 feet**
Initial Borrow 1/	248,076	556,562	865,049	1,173,536	1,482,022	1,790,509	2,098,995	2,407,482
2027-12	331,294	358,861	401,597	464,048	557,123	672,974	781,530	897,016
2039-12	331,294	358,861	401,597	464,048	557,123	672,974	781,530	897,016
2051-12	331,294	358,861	401,597	464,048	557,123	672,974	781,530	897,016
2063-12	296,470	324,037	366,773	429,223	522,299	638,150	746,706	862,191
HIGH SEA LEVEL RISE SCENARIO 12-YEAR REPLENISHMENT INTERVAL								
YEAR	**Initial MSL Beach Width Added**							
	50 feet	**100 feet**	**150 feet**	**200 feet**	**250 feet**	**300-feet**	**350-feet**	**400 feet**
Initial Borrow 1/	390,099	698,585	1,007,072	1,315,558	1,624,045	1,932,532	2,241,018	2,549,505
2027-12	522,011	549,577	592,314	654,764	747,840	863,691	972,246	1,087,732
2039-12	570,704	598,271	641,007	703,458	796,533	912,384	1,020,940	1,136,425
2051-12	619,398	646,964	689,701	752,151	845,227	961,078	1,069,633	1,185,119
2063-12	349,221	376,788	419,524	481,975	575,050	690,901	799,457	914,943

1/ Adjusted for remaining volume of RBSPII Project

Table 6.5-3 Replenishment Dredge Borrow Quantities for Solana in cubic yards (continued)

LOW SEA LEVEL RISE SCENARIO 13-YEAR REPLENISHMENT INTERVAL								
YEAR	Initial MSL Beach Width Added							
	50 feet	100 feet	150 feet	200 feet	250 feet	300-feet	350-feet	400 feet
Initial Borrow 1/	251,558	560,045	868,531	1,177,018	1,485,505	1,793,991	2,102,478	2,410,964
2028-13	353,758	391,364	436,633	499,299	585,753	703,134	813,054	929,444
2041-13	353,758	391,364	436,633	499,299	585,753	703,134	813,054	929,444
2054-13	346,793	384,399	429,668	492,334	578,788	696,170	806,089	922,480
HIGH SEA LEVEL RISE SCENARIO 13-YEAR REPLENISHMENT INTERVAL								
YEAR	Initial MSL Beach Width Added							
	50 feet	100 feet	150 feet	200 feet	250 feet	300-feet	350-feet	400 feet
Initial Borrow 1/	407,614	716,101	1,024,587	1,333,074	1,641,561	1,950,047	2,258,534	2,567,020
2028-13	566,962	604,568	649,836	712,502	798,956	916,338	1,026,257	1,142,648
2041-13	624,109	661,715	706,983	769,649	856,103	973,485	1,083,405	1,199,795
2054-13	620,187	657,793	703,062	765,728	852,182	969,563	1,079,483	1,195,873

1/ Adjusted for remaining volume of RBSPII Project

Table 6.5-3 Replenishment Dredge Borrow Quantities for Solana in cubic yards (continued)

LOW SEA LEVEL RISE SCENARIO 14-YEAR REPLENISHMENT INTERVAL								
YEAR	**Initial MSL Beach Width Added**							
	50 feet	**100 feet**	**150 feet**	**200 feet**	**250 feet**	**300-feet**	**350-feet**	**400 feet**
Initial Borrow 1/	255,041	563,527	872,014	1,180,500	1,488,987	1,797,474	2,105,960	2,414,447
2029-14	357,241	429,098	481,410	544,105	625,866	719,575	815,739	924,053
2043-14	357,241	429,098	481,410	544,105	625,866	719,575	815,739	924,053
2057-14	336,346	408,203	460,516	523,210	604,971	698,680	794,844	903,158
HIGH SEA LEVEL RISE SCENARIO 14-YEAR REPLENISHMENT INTERVAL								
YEAR	**Initial MSL Beach Width Added**							
	50 feet	**100 feet**	**150 feet**	**200 feet**	**250 feet**	**300-feet**	**350-feet**	**400 feet**
Initial Borrow 1/	425,468	733,955	1,042,441	1,350,928	1,659,414	1,967,901	2,276,388	2,584,874
2029-14	593,945	665,802	718,115	780,809	862,570	956,280	1,052,443	1,160,758
2043-14	660,223	732,080	784,392	847,087	928,848	1,022,557	1,118,721	1,227,035
2057-14	539,236	611,093	663,405	726,100	807,861	901,570	997,734	1,106,048

1/ Adjusted for remaining volume of RBSPII Project

Table 6.5-3 Replenishment Dredge Borrow Quantities for Solana in cubic yards (continued)

LOW SEA LEVEL RISE SCENARIO 15-YEAR REPLENISHMENT INTERVAL								
YEAR	Initial MSL Beach Width Added							
	50 feet	100 feet	150 feet	200 feet	250 feet	300-feet	350-feet	400 feet
Initial Borrow 1/	258,523	567,010	875,496	1,183,983	1,492,469	1,800,956	2,109,443	2,417,929
2030-15	360,723	467,543	514,572	582,316	660,859	765,510	868,662	972,237
2045-15	360,723	467,543	514,572	582,316	660,859	765,510	868,662	972,237
2060-15	325,899	432,719	479,748	547,492	626,035	730,686	833,838	937,413
HIGH SEA LEVEL RISE SCENARIO 15-YEAR REPLENISHMENT INTERVAL								
YEAR	Initial MSL Beach Width Added							
	50 feet	100 feet	150 feet	200 feet	250 feet	300-feet	350-feet	400 feet
Initial Borrow 1/	443,660	752,147	1,060,633	1,369,120	1,677,606	1,986,093	2,294,579	2,603,066
2030-15	621,944	728,764	775,792	843,536	922,080	1,026,731	1,129,883	1,233,458
2045-15	698,027	804,847	851,876	919,620	998,164	1,102,814	1,205,966	1,309,541
2060-15	455,241	562,061	609,090	676,834	755,377	860,028	963,180	1,066,755

1/ Adjusted for remaining volume of RBSPII Project

Table 6.5-3. Replenishment Dredge Borrow Quantities for Solana in cubic yards (continued)

LOW SEA LEVEL RISE SCENARIO 16-YEAR REPLENISHMENT INTERVAL								
YEAR	**Initial MSL Beach Width Added**							
	50 feet	**100 feet**	**150 feet**	**200 feet**	**250 feet**	**300-feet**	**350-feet**	**400 feet**
Initial Borrow 1/	262,006	570,492	878,979	1,187,465	1,495,952	1,804,438	2,112,925	2,421,412
2031-16	364,206	486,771	533,523	598,697	676,628	769,870	879,699	992,872
2047-16	364,206	486,771	533,523	598,697	676,628	769,870	879,699	992,872
2063-16	315,451	438,017	484,768	549,943	627,874	721,116	830,945	944,118
HIGH SEA LEVEL RISE SCENARIO 16-YEAR REPLENISHMENT INTERVAL								
YEAR	**Initial MSL Beach Width Added**							
	50 feet	**100 feet**	**150 feet**	**200 feet**	**250 feet**	**300-feet**	**350-feet**	**400 feet**
Initial Borrow 1/	462,190	770,677	1,079,163	1,387,650	1,696,136	2,004,623	2,313,110	2,621,596
2031-16	650,956	773,522	820,273	885,448	963,379	1,056,621	1,166,449	1,279,623
2047-16	737,523	860,088	906,840	972,014	1,049,945	1,143,187	1,253,016	1,366,189
2063-16	368,203	490,768	537,520	602,694	680,625	773,867	883,696	996,870

1/ Adjusted for remaining volume of RBSPII Project

Table 6.5-4 ENCINITAS (SEGMENT 1) BORROW AND PLACEMENT VOLUMES

EN-1A: Encinitas-NED (100-ft at 5-years)

Year		Borrow (cy)		Placement (cy)	
		SLR Scenario			
		Low	High	Low	High
2015	0	820,000	880,000	680,000	730,000
2020	5	340,000	400,000	280,000	340,000
2025	10	340,000	410,000	280,000	340,000
2030	15	340,000	420,000	280,000	350,000
2035	20	340,000	430,000	280,000	360,000
2040	25	340,000	440,000	280,000	370,000
2045	30	340,000	450,000	280,000	370,000
2050	35	340,000	460,000	280,000	380,000
2055	40	340,000	470,000	280,000	390,000
2060	45	340,000	480,000	280,000	400,000
Total		3,850,000	4,840,000	3,200,000	4,030,000

EN-2A: Encinitas- (100-ft at 10-years)

Year		Borrow (cy)		Placement (cy)	
		SLR Scenario			
		Low	High	Low	High
2015	0	840,000	960,000	700,000	800,000
2025	10	720,000	880,000	600,000	730,000
2035	20	720,000	920,000	600,000	760,000
2045	30	720,000	950,000	600,000	790,000
2055	40	720,000	990,000	600,000	820,000
Total		3,710,000	4,700,000	3,090,000	3,910,000

EN-1B and EN-2B: (50-ft at 5-years)

Year		Borrow (cy)		Placement (cy)	
		SLR Scenario			
		Low	High	Low	High
2015	0	410,000	470,000	340,000	390,000
2020	5	260,000	330,000	220,000	280,000
2025	10	260,000	340,000	220,000	280,000
2030	15	260,000	350,000	220,000	290,000
2035	20	260,000	360,000	220,000	300,000
2040	25	260,000	370,000	220,000	310,000
2045	30	260,000	380,000	220,000	310,000
2050	35	260,000	390,000	220,000	320,000
2055	40	260,000	390,000	220,000	330,000
2060	45	260,000	400,000	220,000	340,000
Total		2,790,000	3,780,000	2,320,000	3,150,000

Table 6.5-5 SOLANA (SEGMENT 2) BORROW AND PLACEMENT VOLUMES

SB-1A: Solana NED under Low SLR (200-ft at 13-yrs)

Year		Borrow (cy)		Placement (cy)	
		Low	High	Low	High
2015	0	1,180,000	NA	960,000	NA
2028	13	500,000	NA	420,000	NA
2041	26	500,000	NA	420,000	NA
2054	39	490,000	NA	410,000	NA
Total		2,670,000	NA	2,210,000	NA

SB-1A: Solana NED under High SLR (300-ft at 14-years)

		Borrow (cy)		Placement (cy)	
Year	SLR Scenario=	Low	High	Low	High
2015	0	NA	1,970,000	NA	1,640,000
2029	14	NA	960,000	NA	800,000
2043	28	NA	1,020,000	NA	850,000
2057	42	NA	900,000	NA	750,000
Total		NA	4,850,000	NA	4,040,000

SB-1B and 2A: Solana (150-ft at 10-yrs)

Year		Borrow (cy)		Placement (cy)	
		Low	High	Low	High
2015	0	860,000	970,000	700,000	790,000
2025	10	350,000	500,000	290,000	420,000
2035	20	350,000	530,000	290,000	440,000
2045	30	350,000	570,000	290,000	470,000
2055	40	350,000	600,000	290,000	500,000
Total		2,260,000	3,180,000	1,870,000	2,630,000

SB-1C: Solana (100-ft at 10-years)

		Borrow (cy)		Placement (cy)	
Year	SLR Scenario=	Low	High	Low	High
2015	0	550,000	660,000	440,000	540,000
2025	10	310,000	460,000	260,000	380,000
2035	20	310,000	490,000	260,000	410,000
2045	30	310,000	530,000	260,000	440,000
2055	40	310,000	560,000	260,000	470,000
Total		1,790,000	2,700,000	1,470,000	2,230,000

6.6 Beach Fill Affects on Bluff Failure

With project benefits are estimated with a benefit capture curve that is shown on **Figure 6.6-1**. This curve defines the relationship between the mean sea level (MSL) beach width and the percentage of potential benefits realized from protecting the base of the bluff from coastal storm erosion. The curve is based on the following assumptions:

a. Captured benefits are inversely proportional to the rate of notch growth at the base of the bluff.

b. The rate of notch growth predicted in the Monte-Carlo simulation of bluff failures, using the formulation after Sunamura (see **Section 5.2.2**), is directly proportional to wave height and the number of waves impinging at the base of the bluff.

c. The number of waves impinging at the bluff base is proportional to exposure time and wave height is proportional to the water depth at the bluff base, where water depth is equal to tide elevation minus the sand level elevation.

d. The winter season is when sand level elevations under the “with-project” beach fill alternatives are low enough to where the base of the bluff will be exposed at higher tide stages.

e. The distribution of sand levels in the cross-shore dimension will behave as they have historically as shown in the profile behavior from long-term records. For the Encinitas Segment I reach, CCSTWS/SANDAG/City profile SD670 is used to evaluate the spatial distribution of sand thickness in the cross-shore dimension. For the Solana Beach Segment II reach, CCSTWS/SANDAG/City profile SD600 is used to evaluate the cross-shore sand thickness.

f. The hardpan substrate underlying the beach sand is comparatively non erosive and the elevation of the hardpan fronting the beach bluffs will remain constant over the 50-year evaluation period. At both locations, the elevation of this erosion resistant hardpan is +1.7 ft above MLLW at the toe of the bluff.

Beach profiles and the approximate location of the hardpan substrate is shown in **Figure 6.6-2** for the Moonlight location (SD-670) and **Figure 6.6-3** for the Fletcher Cove location (SD-600). Dependant on season and the profile’s available sand volume, the beach sand level, or top-of-sand elevation, at the base of the bluff has been equal to the hardpan elevation of +1.7 ft (MLLW) when all of the sand is scoured away from the base of the bluff to an elevation as high as +8 to + 12 feet (MLLW) when a full beach berm exist. The location of the base of the bluff is about 60 to 70 feet from the baseline zero station of **Figure 6.6-2** and **Figure 6.6-3**.

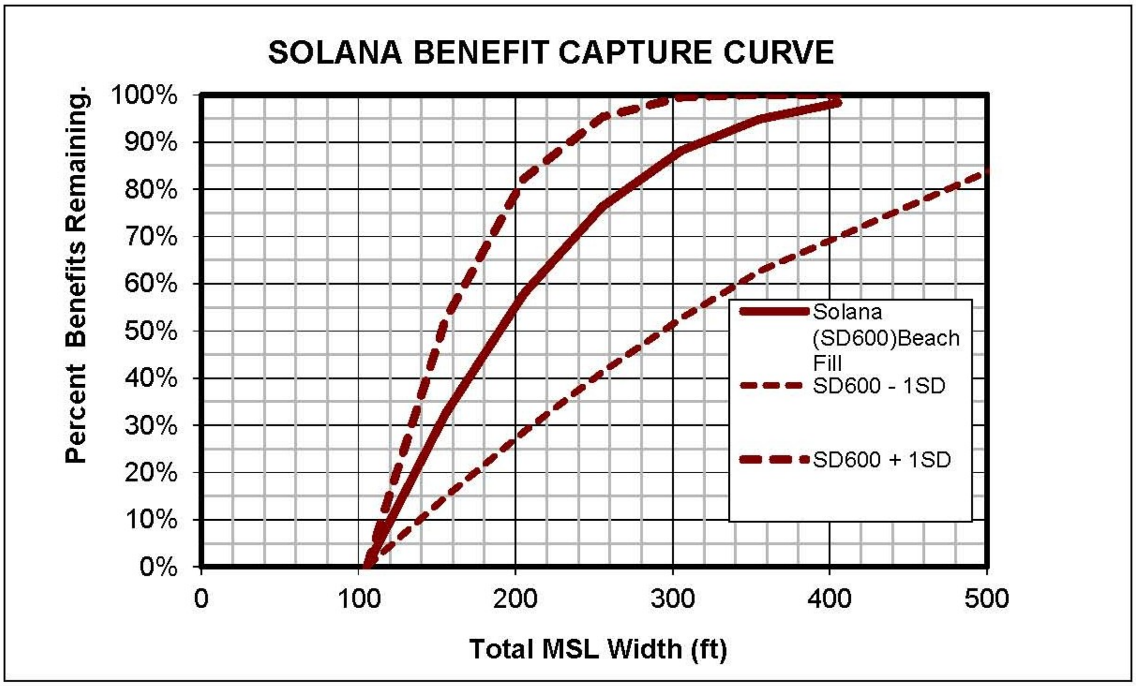
The relationship between the profile sand volume and mean sea level (MSL) position is shown on **Figure 6.6-4** and **Figure 6.6-5** for SD-670 and SD600, respectively. The least-squares fit of these data results in a 0.864 cubic yards/foot for SD-670 and 0.713 cubic yards/foot for SD-600 relationship between MSL beach width and profile sand volume per alongshore unit-width. The profile sand volume is computed from the elevation of the hardpan to the top-of-sand to an offshore distance of about 1600 feet. This corresponds to an effective depth of closure of about 28-feet at SD670 and 23-feet at SD-600, respectively. Historically, sand volume densities have ranged from about 50 cubic-yards/ft to 200 cubic yards/ft.

Figure 6.6-6 and Figure 6.6-7 display the cross-shore distribution of the profile sand volume by season. For SD-670, 21 percent of the profile sand volume is located within the first 200 feet from the back beach in the “with” project spring profile, while under with project fall conditions the percentage increases to 30 percent. This is a measure of the seasonal change of sand being pulled from the beach to create sand bars during the steep winter wave conditions and the migration of those bars to build the beach in summer during the period of relatively small long summer swell conditions. For SD-600, the corresponding percentages of profiles sand volume are 13 percent and 28 percent for the spring and fall conditions, respectively. The benefit capture curve assumes the spring condition distribution of sand across the profile to estimate the “with-project” sand thickness at the base of the bluff during the vulnerable winter season.

The “without-project” condition presumes a profile sand volume to be nil. “With-Project” beach alternatives uses the least-squares fit of profile sand volume versus MSL beach width described above. For example, an average 100-foot width between the bluff and MSL beach would be equivalent to a profile sand volume of 86.4 cubic yards / foot along Segment I and 71.3 cubic yards / foot along Segment II. Furthermore, along Segment I typical spring conditions would have 20 percent of the profile sand volume distributed within the closest 200 ft of the bluff toe resulting in an average sand thickness above the hardpan of 0.21x86.4cy/ftx27cy/cf/200ft/ft = 2.4 feet and sand elevation at the base of the bluff of 1.7 + 2.4 $\cong$ +4.1 ft (MLLW). At Segment II, the 100-foot MSL width beach results in a sand elevation of +3.0 ft (MLLW).

The tidal range at the project site, as represented by tidal data from NOAA’s La Jolla Tide gage, is from about -2-feet to +8 feet (MLLW). The hourly distribution of measured tides during the 1997-1998 El Nino is shown on **Figure 6.6-8** The tide level exceeded -2-feet all of the time, exceeded 3.2 feet half of the year, and never exceeded 8-feet (MLLW). The estimate of sand level described above and this El Nino year distribution of tide levels were used to estimate the time distribution of water depths near the bluff toe. Because these depths are quite shallow and the period of interest is the winter wave season, wave heights were assumed to be depth limited and therefore, proportional to the water depth.

The tide frequency distribution curve was binned into 0.2-foot increments and the annual sum of the product of wave height times time (Δt_i) computed for various bluff toe depths, shown on **Table BC-1**. The complement of the ratio of these annual sums forms the basis of the Benefit-Capture curve shown on **Figure 6.6-1.** For example, when the tide elevation is 6.5 feet and the sand elevation (winter toe depth) is 4.7 feet, the resulting water depth at the toe is 1.8 feet, and this occurs 0.52% of the time (%f) for a 6.5 foot tide. The water depth at the toe is multiplied by the frequency to produce a factor of time for this occurrence. For a given sand elevation (winter toe depth), summing all factors for each tide elevation produces a sum, when divided by 0.86 and then subtracted from 1, provides a percentage of project effectiveness. The without project sand elevation is 2.8 feet, and results in 100% damage.



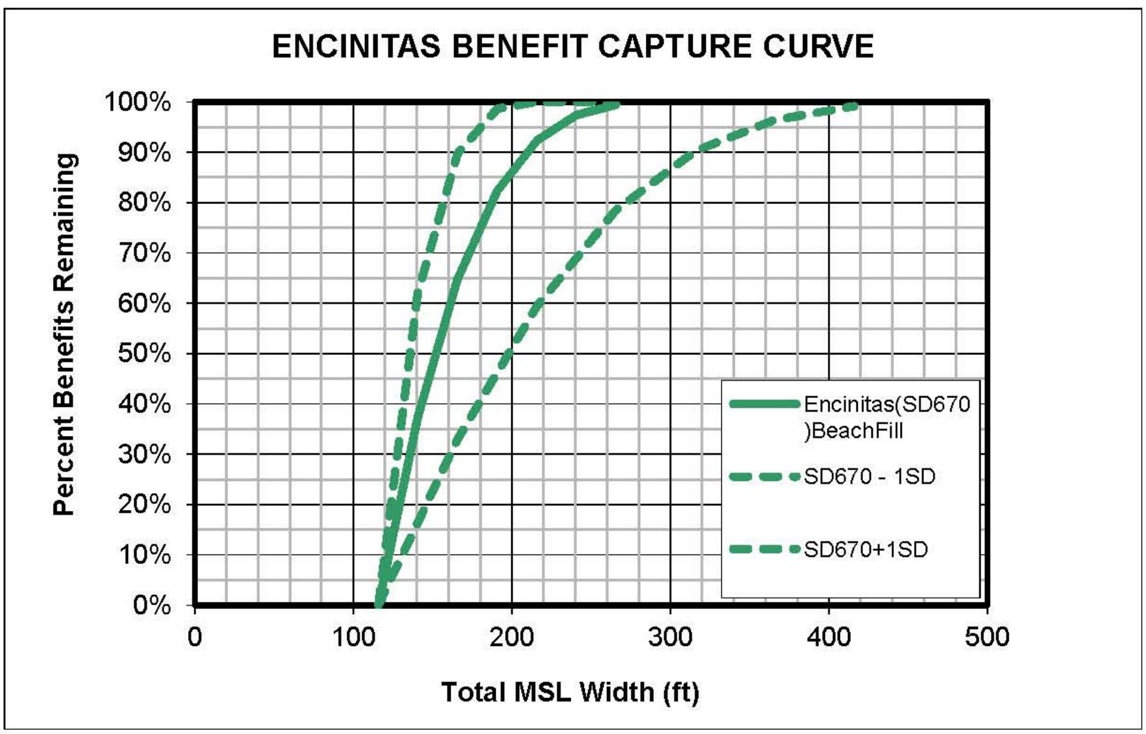


Figure 6.6-1 Solana Beach and Encinitas Benefit Capture Curves

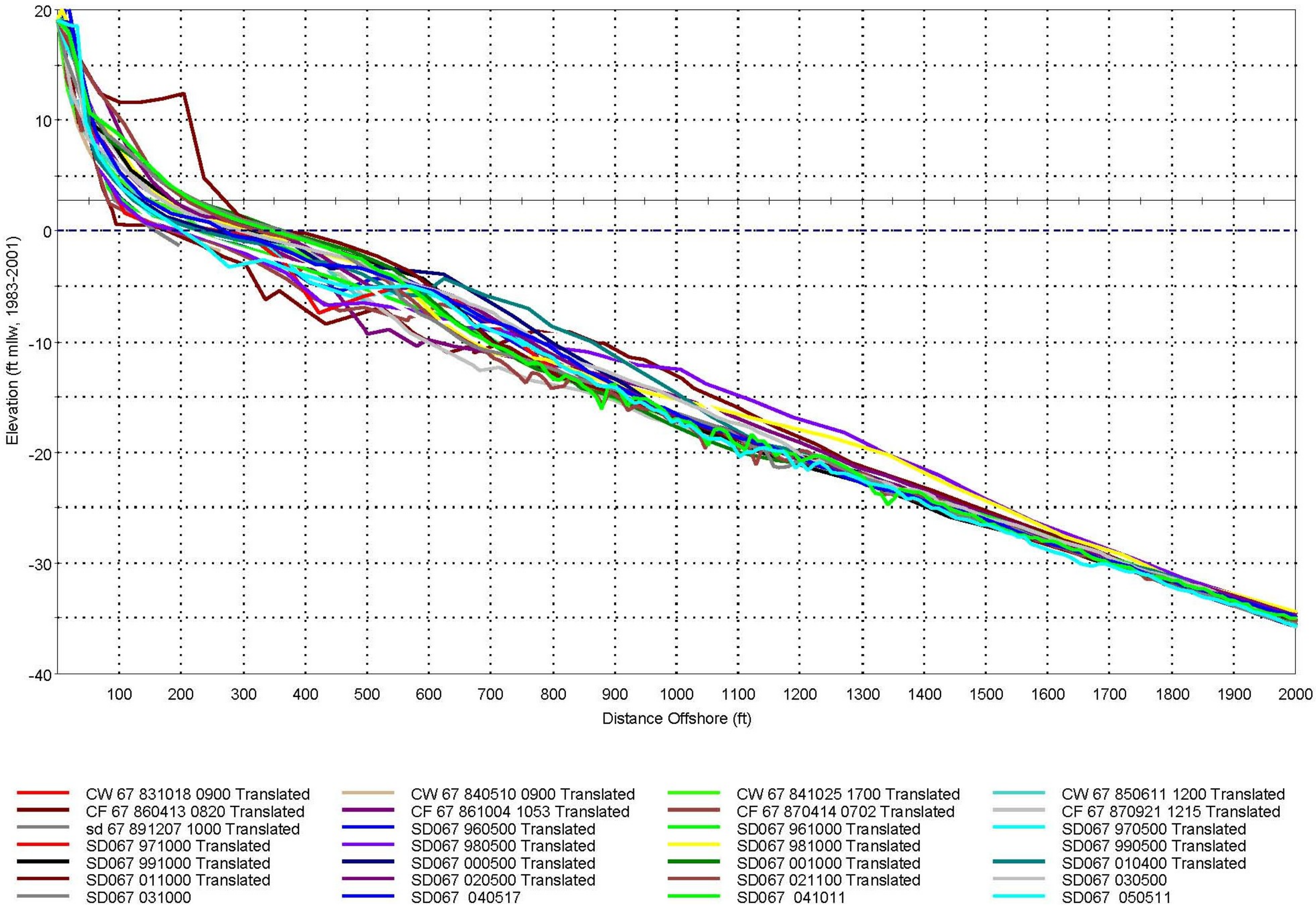


Figure 6.6-2 SD0670 Moonlight State Beach

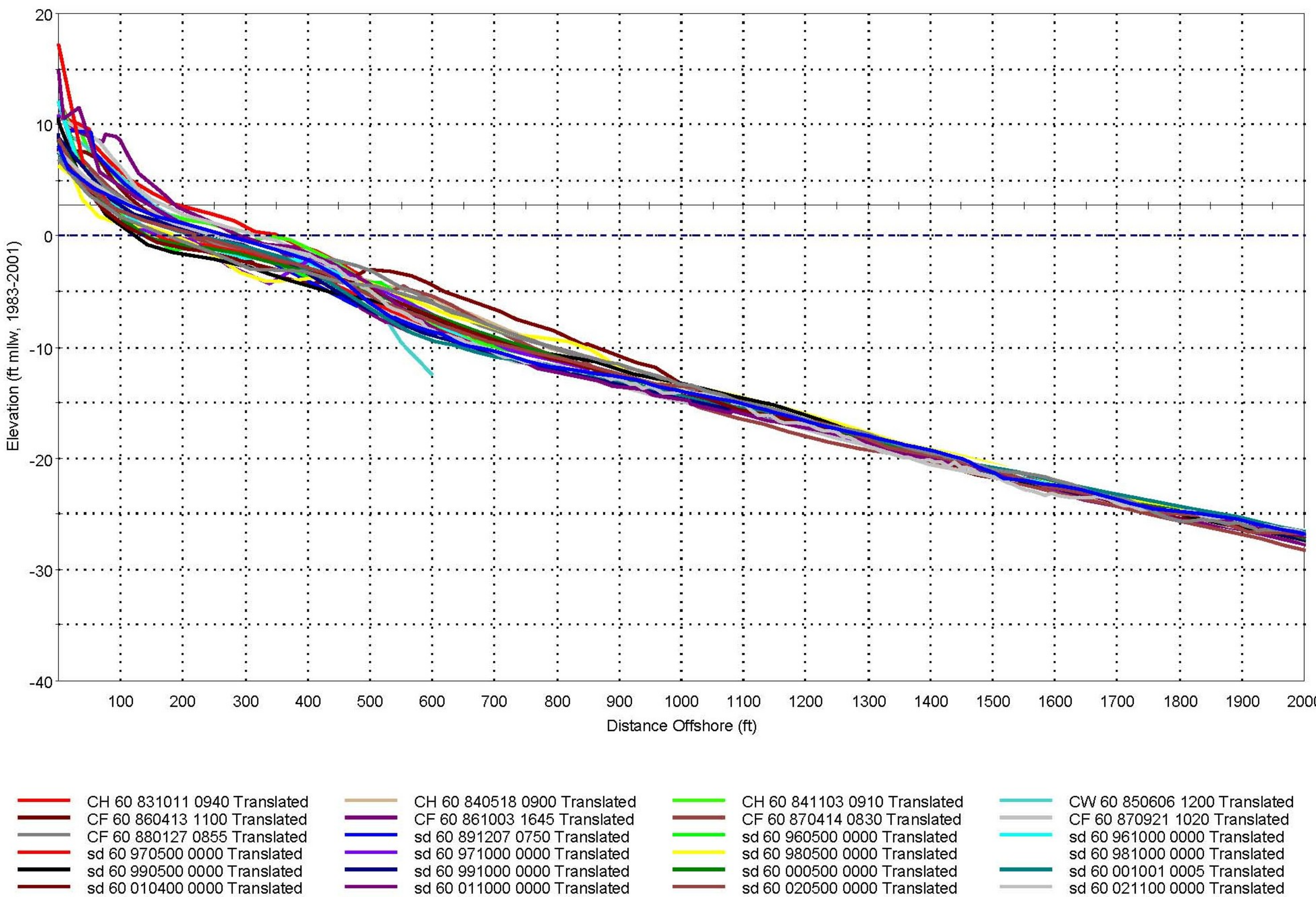
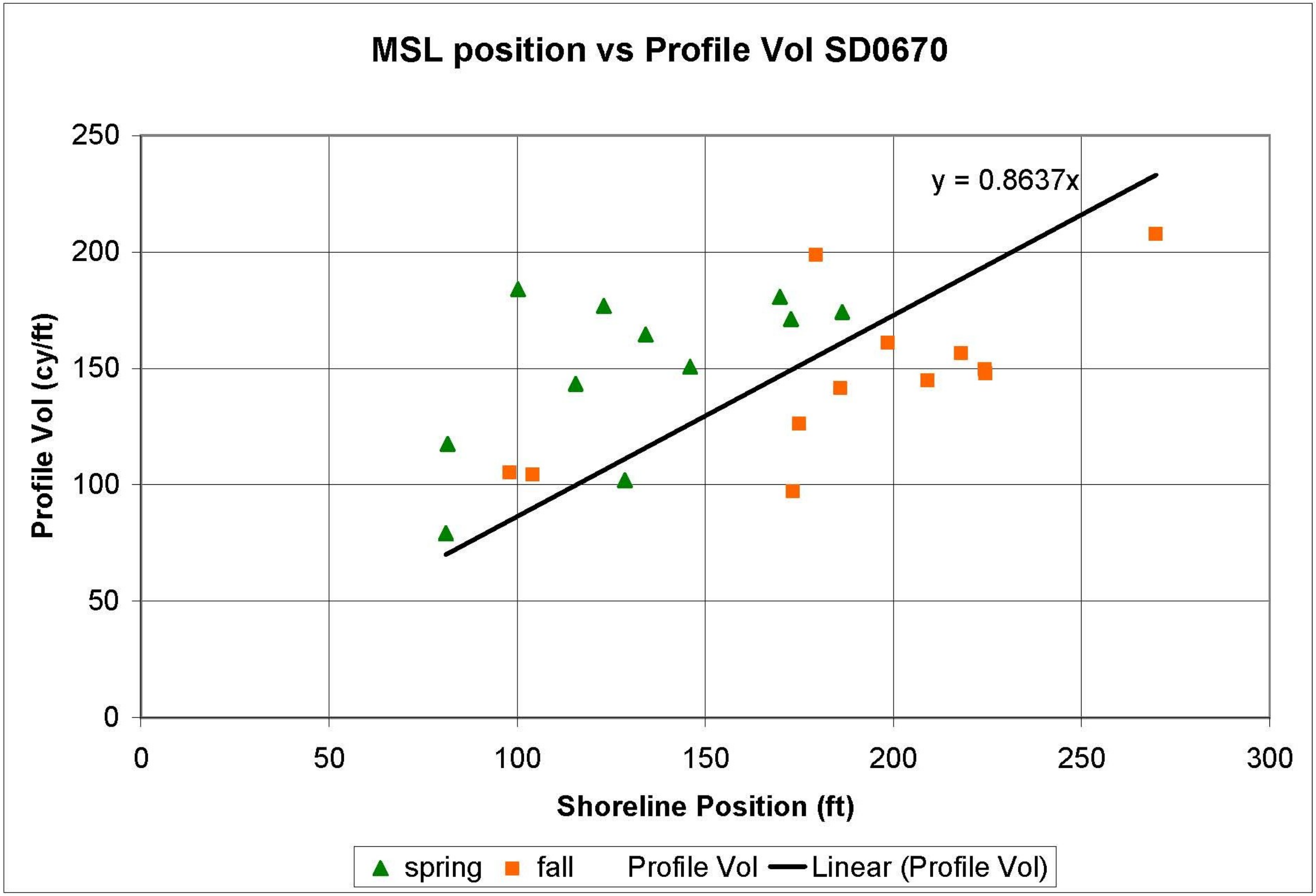


Figure 6.6-3 SD0600 Fletcher Cove Beach



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2 **Figure 6.6-4 MSL Position vs. Profile Vol SD0670**

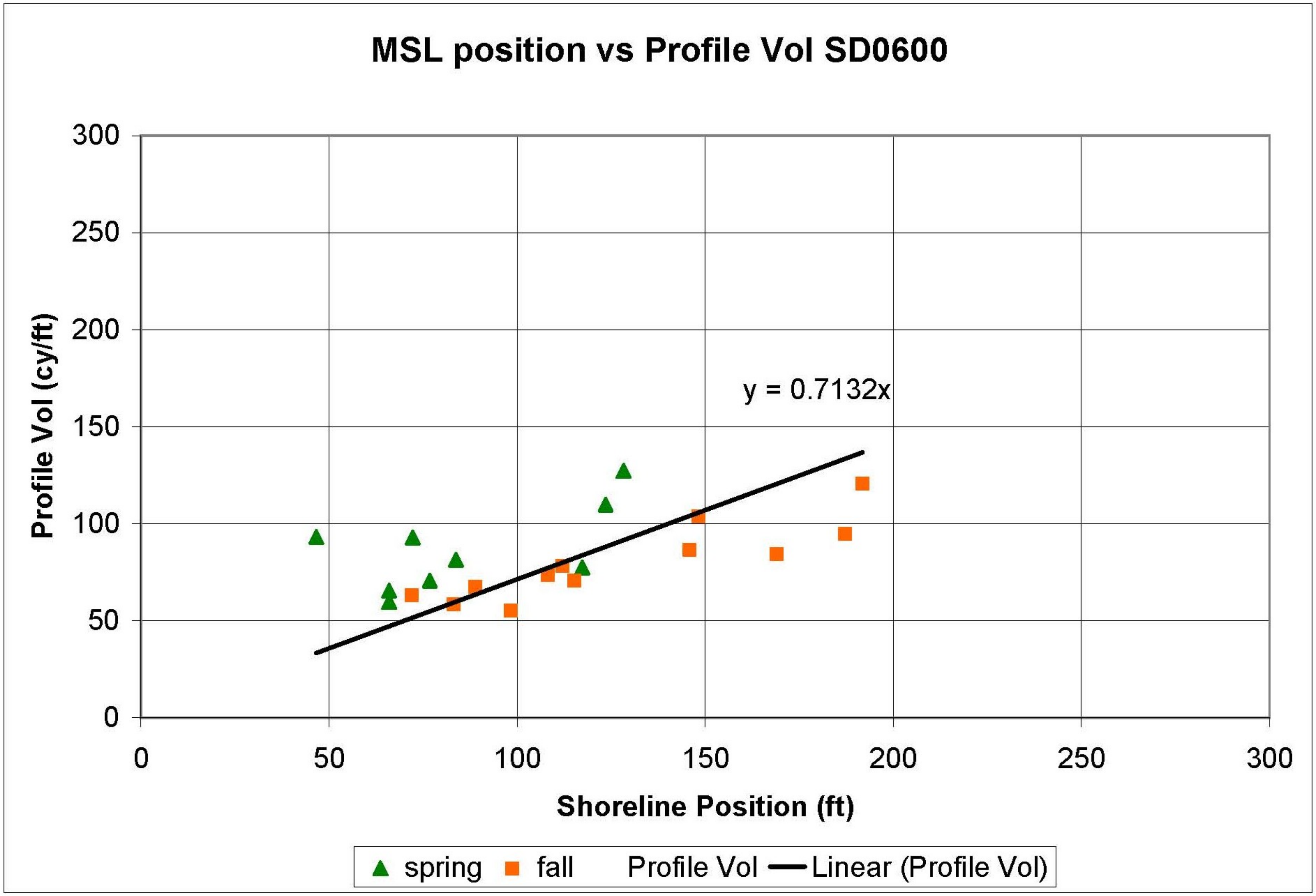
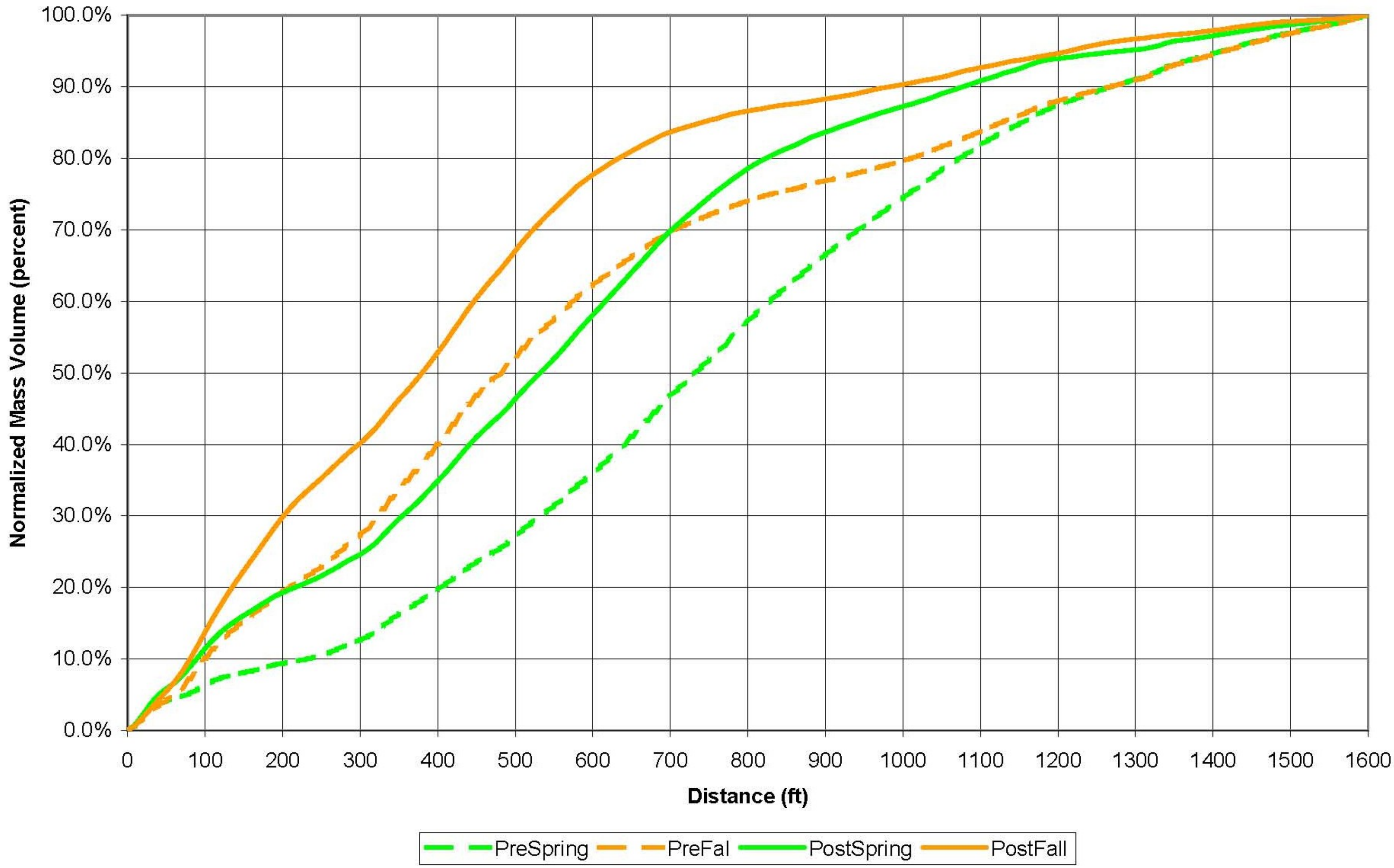


Figure 6.6-5 MSL Position vs. Profile Vol SD0600



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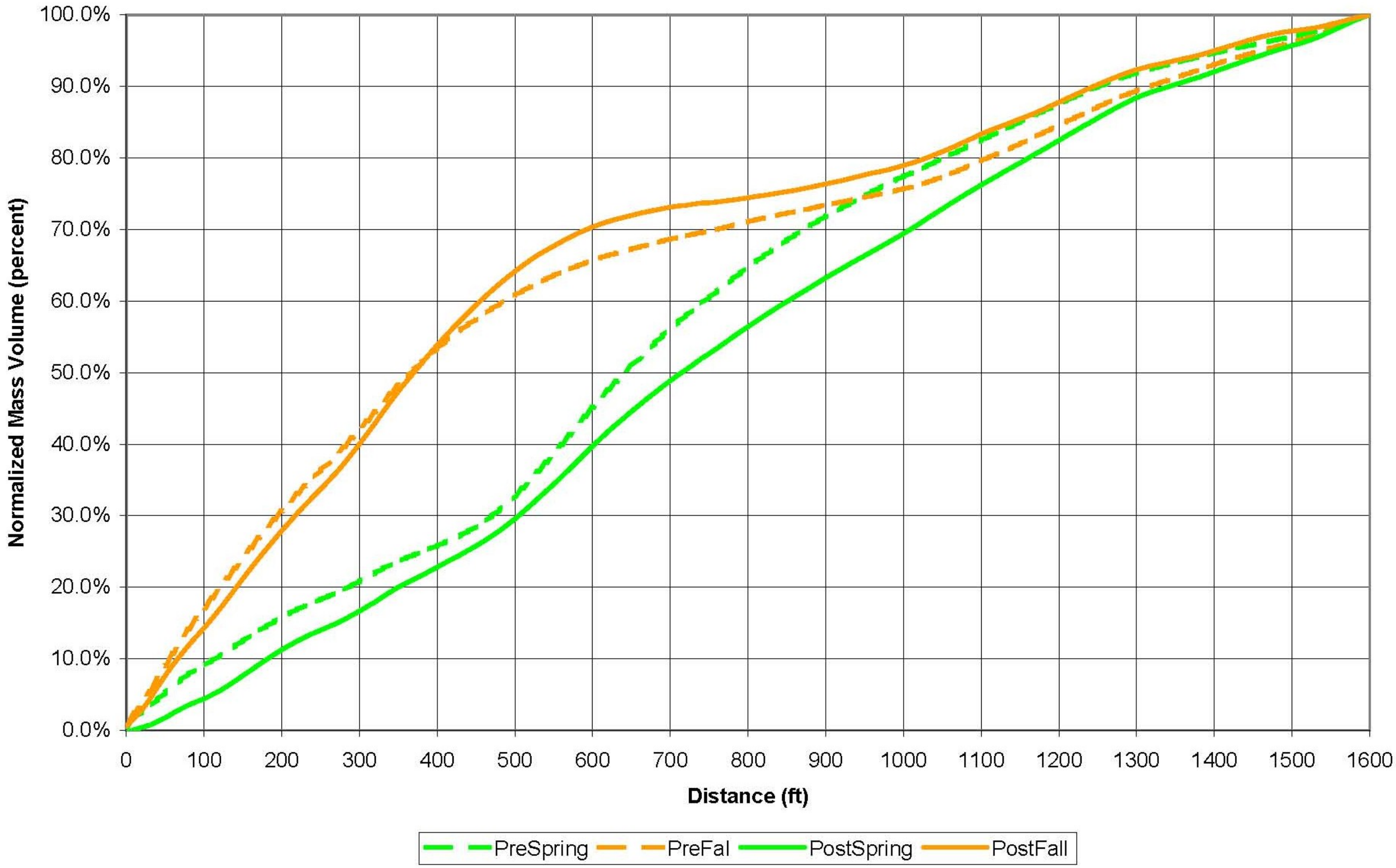
2 **Figure 6.6-6 SD0670 Normalized Profile Mass Diagram**



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2 **Figure 6.6-7 SD0600 Normalized Profile Mass Diagram**

range	count	frequency	F
8.5	0	0.00%	0.00%
8.3	0	0.00%	0.00%
8.1	0	0.00%	0.00%
7.9	0	0.00%	0.00%
7.7	2	0.02%	0.02%
7.5	2	0.02%	0.05%
7.3	2	0.02%	0.07%
7.1	12	0.15%	0.22%
6.9	26	0.32%	0.54%
6.7	39	0.48%	1.03%
6.5	42	0.52%	1.54%
6.3	71	0.88%	2.42%
6.1	88	1.09%	3.51%
5.9	111	1.37%	4.88%
5.7	120	1.48%	6.36%
5.5	135	1.67%	8.03%
5.3	155	1.91%	9.94%
5.1	224	2.77%	12.71%
4.9	203	2.51%	15.22%
4.7	274	3.38%	18.60%
4.5	284	3.51%	22.11%
4.3	337	4.16%	26.27%
4.1	355	4.38%	30.66%
3.9	358	4.42%	35.08%
3.7	397	4.90%	39.98%
3.5	361	4.46%	44.44%
3.3	358	4.42%	48.86%
3.1	398	4.92%	53.78%
2.9	351	4.34%	58.12%
2.7	386	4.77%	62.88%
2.5	369	4.56%	67.44%
2.3	323	3.99%	71.43%
2.1	301	3.72%	75.15%
1.9	262	3.24%	78.38%
1.7	246	3.04%	81.42%
1.5	220	2.72%	84.14%
1.3	217	2.68%	86.82%
1.1	172	2.12%	88.95%
0.9	162	2.00%	90.95%
0.7	134	1.66%	92.60%
0.5	136	1.68%	94.28%
0.3	129	1.59%	95.87%
0.1	101	1.25%	97.12%
-0.1	84	1.04%	98.16%
-0.3	55	0.68%	98.84%
-0.5	45	0.56%	99.39%
-0.7	28	0.35%	99.74%
-0.9	9	0.11%	99.85%
-1.1	11	0.14%	99.99%
-1.3	1	0.01%	100.00%
-1.5	0	0.00%	100.00%
-1.7	0	0.00%	100.00%
-1.9	0	0.00%	100.00%
-2.1	0	0.00%	100.00%
-2.3	0	0.00%	100.00%
-2.5	0	0.00%	100.00%
	8096		

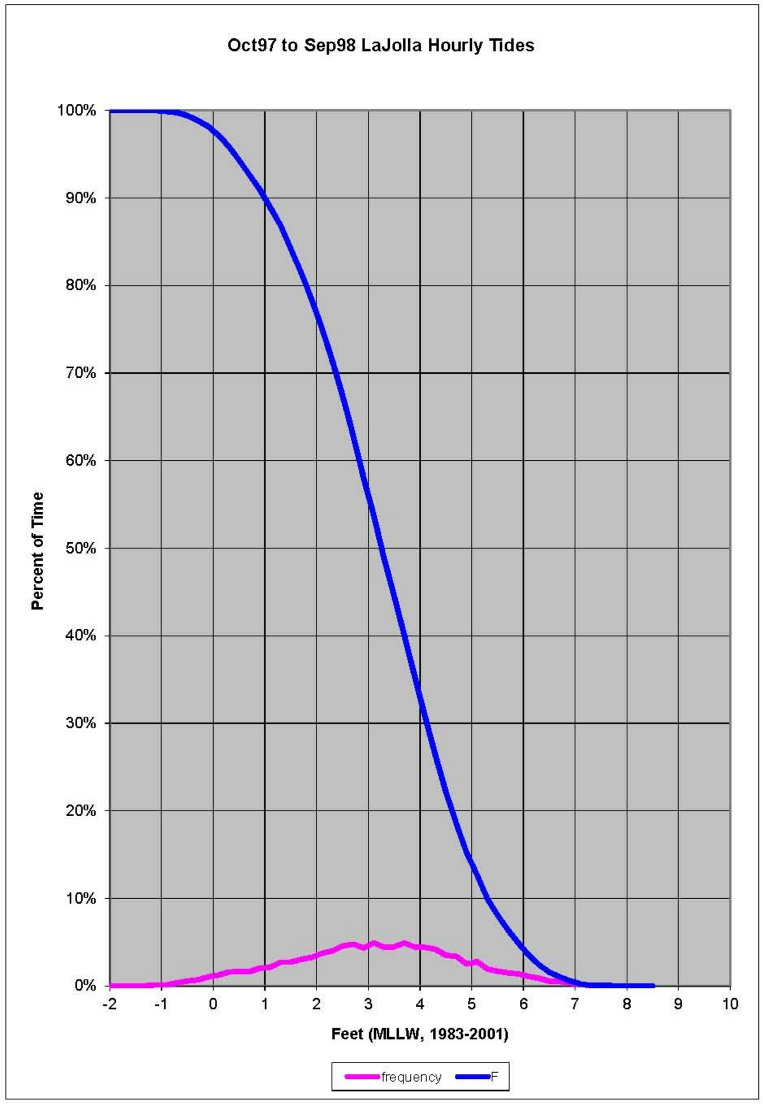


FIGURE 6-17 HOURLY DISTRIBUTION OF MEASURED TIDES DURING 1997-98 EL NINO, LA JOLLA, CALIFORNIA

Figure 6.6-8 Hourly Distribution of Measured Tides During 1997-1998 El Nino, La Jolla, California

7 NUMERICAL MODELING OF BEACH NOURISHMENTS

A numerical shoreline model was applied to predict the shoreline behaviors for various combinations of beach nourishment options. The modeled results were then analyzed to determine the nearshore habitat impacts, the optimal beach nourishment or hybrid plan alternative (based on the associated Project construction costs and resultant economic benefits), lagoon sedimentation impacts, and surfing impacts. A profile analysis was performed as described in **Chapter 8** of this report as an intermediate calculation step between the shoreline modeling and subsequent impact analyses. The nearshore habitat impacts analysis is presented in **Chapter 9**, the lagoon sedimentation analysis is presented in **Chapter 10**, and the surfing impact analysis is presented in **Chapter 11** of this report. The economic optimization analysis is presented in **Chapter 12** of this report. The numerical shoreline modeling methods and results are presented in the following.

7.1 Model Description

The NEMOS computer program, developed by the USACE, Coastal Hydraulics Laboratory (CHL), is a set of modules within the Coastal Engineering Design and Analysis Software (CEDAS) suite of programs that simulate the long-term shoreline evolution of a beach in response to imposed wave conditions, presence of coastal structures and other engineering activities such as beach nourishment. The numerical modules within NEMOS that were applied in this analysis are the GENEralized Model for Simulating Shoreline Change (GENESIS) (Hansen, 1987; Hansen, 1989; Hansen& Kraus, 1989; Veri-Tech, 2011) and the STeady State Irregular WAVE Model (STWAVE) (Smith et al., 2001; Veri-Tech, 2011).

GENESIS was developed to simulate long-term shoreline changes on an open coast as induced by spatial and temporal differences in longshore sediment transport. GENESIS is equipped with an internal wave transformation process sub-model and is generalized in that a wide variety of offshore wave inputs, initial beach planform configurations, coastal structures and beach nourishments can be included in the simulation. The main utility of GENESIS lies in simulating shoreline response to artificial beach nourishment with or without the presence of coastal structures such as detached breakwaters, groins, jetties and seawalls. Extensive testing and field verification for GENESIS have been conducted by the Corps before its release for public use. The model has continuously been updated and improved based on recent technical researches and field applications. It has been applied in the past to simulate shoreline changes for several projects proposed in southern California (Gravens, 1990; Moffatt & Nichol Engineers, 2000; Moffatt & Nichol et. al, 2011) with reasonable accuracy for engineering analyses and environmental evaluations.

It should be noted that GENESIS can only predict the long-term shoreline evolution induced by longshore sediment transport under the assumption that the cross-shore transport occurs mainly seasonally without any long-term net gain or loss across the beach profile. The short-term shoreline change that is significantly dependent on the cross-shore transport cannot be obtained from GENESIS model predictions, but was instead estimated using a separate tool as presented in **Chapter 8** of this report.

In the GENESIS simulations, the longshore sediment transport rate is computed based on the longshore wave energy flux method (USACE, 1984) with an additional contribution resulting from the longshore gradient of breaking wave heights. This additive component is relatively significant only in the vicinity of coastal structures. Either the internal wave transformation model or an external wave model can be used to deduce nearshore wave information for computing the longshore sediment transport rate. To account for the irregular bathymetry of the study area, STWAVE was used as the external wave model in this analysis. STWAVE calculates wave transformation from offshore deep water to a nearshore reference line. From that reference line, the internal wave model within GENESIS propagates the waves to the breaking point where the longshore sediment transport rate is calculated.

STWAVE is a robust numerical model which spatially quantifies the change in wave characteristics (wave height, period, direction and spectral shape) from offshore to the nearshore zone. It is formulated as a steady state model for the spectral wave propagation over irregular bathymetry using a 2-D finite-difference representation of a simplified form of the spectral balance equation (Smith et al., 2001). STWAVE is capable of simulating wave shoaling, refraction, diffraction and breaking, wave growth due to local sea breeze, and wave-wave interaction and white capping that redistribute and dissipate energy in a growing wave field.

7.2 Model Domain

The study area has a shoreline length of approximately 7.4 miles, running from the north end of Reach 1 to the south end of Reach 9. In order to minimize the impacts that might be induced by the artificially specified boundary conditions at both the upcoast and downcoast ends, the modeled domain was expanded to approximately 5.5 miles north of Reach 1 and 2.7 miles south of Reach 9. Thus, the total length of modeled shoreline is 15.5 miles. Extending the modeled domain to the north includes a portion of the Oceanside shoreline, for which the longshore sediment transport rate has previously been calculated. This value was used to calibrate the GENESIS model.

The STWAVE model domain covers an area of 15.5 miles alongshore and 3.2 miles seaward extending from the shoreline to a water depth of approximately 300 feet. Wave characteristics at this deep water condition were generated from the hindcasted wave model called WAVEWATCH III and the O'Reilly spectral back-refraction model that were previously presented in **Chapter 5** of this report. **Figure 7.2-1** illustrates the GENESIS and STWAVE model domains.

Since GENESIS only operates in metric units, the GENESIS modeling was performed in meters, but the results were converted to feet for reporting. GENESIS was operated using the MSL vertical datum, but values in this report are given relative to the MLLW vertical datum for consistency, except where noted. In the most recent 1983 to 2001 tidal epoch, MSL was 2.73 feet higher than MLLW at the La Jolla tide gage.

7.2.1 Modeling Grids

Two different model grids and coordinate systems were designated for GENESIS and STWAVE. In the GENESIS simulations, the 15.5 mile long, shoreline model domain was represented by 650 cells, each with a cell length of 40 meters (i.e., 131 feet). The alongshore axis (i.e., x-axis) was chosen to be approximately parallel to the shoreline with an orientation angle of 342 degrees, clockwise, from the true north. The positive alongshore direction is from the southeast to the northwest and the y-axis extends seawards. Differing from the GENESIS coordinate system, STWAVE is oriented with the x-axis extending landward. STWAVE used a uniform mesh over the model domain consisting of 625 cells in the alongshore direction (i.e., y-axis) and 130 cells in the cross-shore direction (i.e., x-axis), with a cell spacing of 40 meters (i.e.,131 feet).

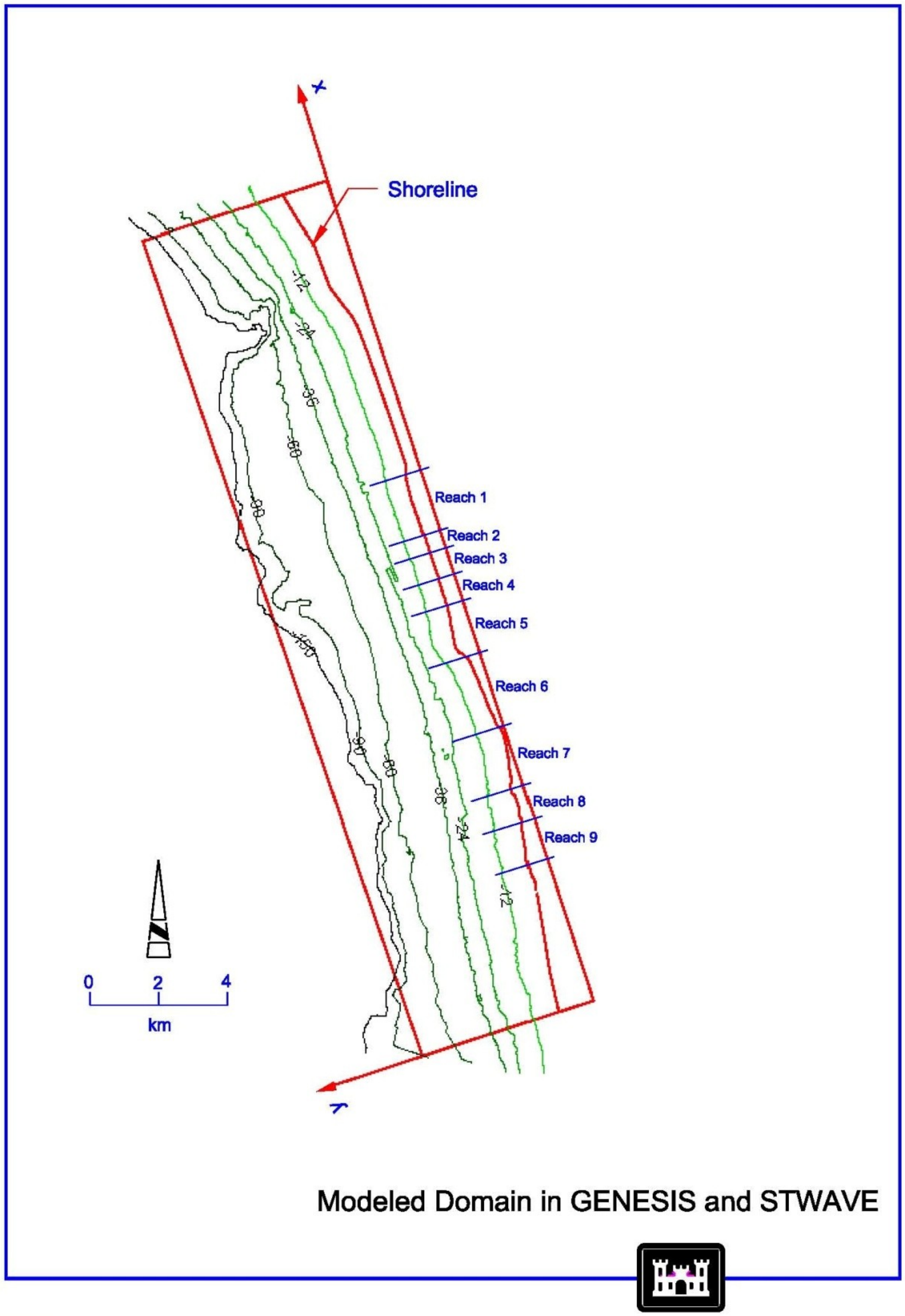


Figure 7.2-1 Modeled Domain in GENESIS and STWAVE

7.2.2 Shoreline Positions and Coastal Features

The MSL shoreline is typically used as the modeled shoreline for shoreline morphology modeling. Regardless of the beaches that presently exist and may temporarily exist within the study area as a consequence of the RBSPI and RBSPII, the future without Project beach fronting the coastal bluffs are expected to be denuded during the majority of the Project duration. Therefore, the initial shorelines used in the GENESIS simulations were modeled as backed by a non-erodible bluff. This also agrees with the denuded beach assumption used in the without Project bluff retreat analysis of **Chapter 5** of this report.

The bluff toe retreat rate is essentially zero in comparison to the shoreline changes from the beach nourishments. Therefore, the bluff toe locations backing the initial shoreline were also assumed to be non-erodible or fixed in the shoreline model simulations. This is equivalent to having a shoreline backed by seawalls as the starting shoreline. By doing this, the shoreline simulations were capable of isolating the shoreline impacts resulting from the proposed beach nourishments.

The bluff toe position was placed landward of and adjacent to the initial MSL shoreline. From a comparison of measured shorelines occurring in 2002 and 2004, the MSL shorelines were found to be relatively stable in comparison to the actual bluff toe. Throughout the GENESIS model domain the average of the absolute differences between these measured shorelines was 16 feet. The average of the absolute differences between either shoreline and the bluff toe was approximately 180 feet. This implies that the shoreline was relatively stable over that time even though there was a roughly 180 feet wide beach between the MSL shoreline and bluff toe. If the model was configured with a large distance between the bluff toe and shoreline, then the modeled shoreline would tend to retreat towards the bluff toe, even though in reality the sand-starved coastline has already retreated to the bluff toe and is somewhat stable. GENESIS only sees a one-dimensional shoreline (in this case the MSL shoreline), while in reality there are many possible shorelines ranging from MLLW to the beach berm. Solutions to this shortcoming were to either place the bluff toe just landward of the MSL shoreline (in the model) or model a shoreline that runs along and just seaward of the bluff toe (or seawall). Since results of the modeling were all netted out, either approach is acceptable. But since the MSL shorelines were easily available, the former approach was used.

Existing natural coastal features such as reefs and river deltas as well as man-made shore protective structures (e.g., jetties and groins) can play an important role in affecting shoreline evolution. The only man-made shore-perpendicular structures within the model domain were the jetties at the Batiquitos Lagoon entrance. Low relief reefs at Swamis (south end of Reach 5) and Table Tops (north end of Reach 8) were modeled as submerged breakwaters, simulating both sedimentation and longshore sediment transport in their lee.

7.3 Model Parameters

7.3.1 Wave Characteristics

In the GENESIS simulations, the characteristics of breaking waves were essential for determining the longshore sediment transport rate, from which the shoreline changes were predicted. Nearshore wave conditions as described below served as a primary input to the GENESIS modeling and were essential for accurate GENESIS predictions.

7.3.1.1 Wave Data Sources and Transformation

Potential data sources were reviewed to identify the appropriate wave data for the wave input to the GENESIS model execution. Two types of wave data sources were available for the study area, including 1) the Coastal Data Information Program (CDIP) data and 2) historically hindcasted wave data as described in **Chapter 5**.

The CDIP wave gages within the vicinity of the study area include nearshore stations at Oceanside (Station 004), Del Mar (Station 051) and Scripps Pier (Station 073) as well as the offshore stations at Oceanside (Station 045) and La Jolla (Station 095). Due to shoaling and refraction over the irregular nearshore bathymetry, spatial variation of the wave field is expected. Nearshore wave characteristics, particularly the wave approach directions, vary significantly at different gage locations even if they are deployed in the same water depth. Therefore, any nearshore wave data collected at a specific station cannot represent the wave characteristics at the model's offshore boundary.

On the contrary, wave alteration induced by bathymetry variation is negligible in deep water farther offshore and the wave field is more homogeneous in this area. It is reasonable and common to use the deep water wave climates along the offshore model boundary to drive shoreline morphology modeling. Given this, the CDIP offshore deep water wave data at Oceanside and La Jolla buoys were preferable sources. However, the offshore Oceanside buoy was only deployed in 1997 and the La Jolla buoy was deployed in 1999, so neither data record were long enough to be considered a long-term representative record. As an alternative, the hindcasted deep water wave record discussed in the bluff retreat analysis of **Chapter 5** were available so that data set was used for shoreline modeling. The main difference being that for the shoreline modeling, the waves were transformed to a nearshore location as opposed to the bluff toe as was done for the bluff retreat analysis. Several preliminary analyses were performed to determine the optimal deep water location, which was at coordinates 33^{o} 1' 41"N and 117^{o} 19' 45" W in a water depth of 300 feet. At this depth wave transformation caused by bathymetry variations were negligible.

These deep water wave characteristics were derived via the WAVEWATCH III hindcast tool and the O'Reilly wave propagation tool as discussed in the bluff retreat analysis of **Chapter 5**. These methods resulted in estimates of wave height, wave period, and wave direction, every three hours, covering the period from 1979 to 2000. The effects of island sheltering between the large hindcast spatial domain and the deep water location were accounted for in the O'Reilly back-refraction model. These hindcasted deep water waves were transformed to the nearshore zone using STWAVE. STWAVE was used to quantify the refraction and shoaling effects due to the localized irregularity of nearshore bathymetry. The deep water location was chosen to be landwards of all offshore islands so no island sheltering existed between it and the nearshore region specified for GENESIS.

7.3.1.2 Hindcasted Deep Water Wave Characteristics

The hindcasted deep water wave characteristics are discussed here. Incoming wave trains primarily consist of two primary patterns of north or northwest extratropical storm swells and southerly swells originating in the southern hemisphere. In addition, four secondary wave patterns observed in the region are swells generated by northwest winds in the outer coastal waters, westerly and southeasterly local seas, and swell from tropical storms and hurricanes off the Mexican coast. These are discussed in detail in **Section 3.1** of this report. **Figure7.3-1** shows the occurrence frequencies of the hindcasted wave height and approach angle, as well as the joint probability of these two parameters from 1979 to 2000 data set at the hindcasted deep water location. The same data is graphically represented in the wave roses of **Figure 7.3-2**. It can be seen that the offshore wave heights can be as high as 18 feet with wave approach angles ranging from 180 to 290 degrees (i.e., from south to northwest).

The shoreline in the GENESIS model domain is oriented from northwest to southeast with an azimuth of approximately 342 degrees, clockwise from true north. The shore normal direction is 252 degrees. The hindcasted deep water wave data indicates that approximately 37 percent of waves come from a direction north of the shore normal and propagate downcoast and 63 percent come from south of shore normal propagating upcoast. Although the prevailing wave direction is from south, most of the larger storm waves that drive longshore sediment transport are from west or northwest, driving transport downcoast. For example, the largest waves with the height reaching about 18 feet hindcasted in the 1983 El Niño winter were from the northwest or west, generated from extratropical storms in the northern Pacific Ocean. A similar pattern was observed in the wave record from CDIP's offshore Oceanside buoy between 2000 and 2003, during which approximate 60 percent of waves propagate from between 180 and 250 degrees. As a consequence of this broad spectrum of wave directions, the net transport direction can be either upcoast (northwest) or downcoast (southeast) in a given year.

7.3.1.3 Wave Simulation Groups

The sequential order of incoming waves is essential in modeling shoreline evolution. Incoming wave scenarios used in the GENESIS simulations were not constructed by randomly sampling wave characteristics from the statistical distributions, as was done for the bluff retreat analysis of **Chapter 5**. Instead, the sequential series of the entire 22 year record of offshore hindcasted deep water wave data was reassembled into five wave simulation groups representing different wave climate periods. Each group covers a period of eight consecutive years during which the hindcasted deep water wave climate represents a period of either stormy or benign wave conditions. By doing this, the behavior of beach nourishment was analyzed under various wave climates to estimate the broad spectrum of shoreline evolution after the beach nourishment. This procedure also provided a range of uncertainty resulting from the variation of wave environment. The final shorelines used in estimating the optimal beach nourishment option and replenishment intervals are called scenario-mean shorelines and were determined by averaging the shoreline positions from all five wave simulation groups.

The duration of each of these wave simulation groups was selected to be eight years, to capture the three to seven year El Niño period observed in southern California. Five wave simulation groups, were constructed from the 22 year wave record for the shoreline evolution modeling. The wave simulation groups included the sequential wave events from 1979 to 1986, 1983 to 1990, 1987 to 1994, 1991 to 1998 and 1993 to 2000. **Figures 7.3-2** through **Figure 7.3-7** show wave roses for the total record and for the individual wave simulation groups. The relative amount of wave storminess can be seen in the **Figure 7.3-8** wave height probability of exceedence curve. From this, it can be seen that the 1991-1998 and 1993-2000 wave simulation groups were stormier than the group as a whole and the other wave simulation groups were relatively benign.

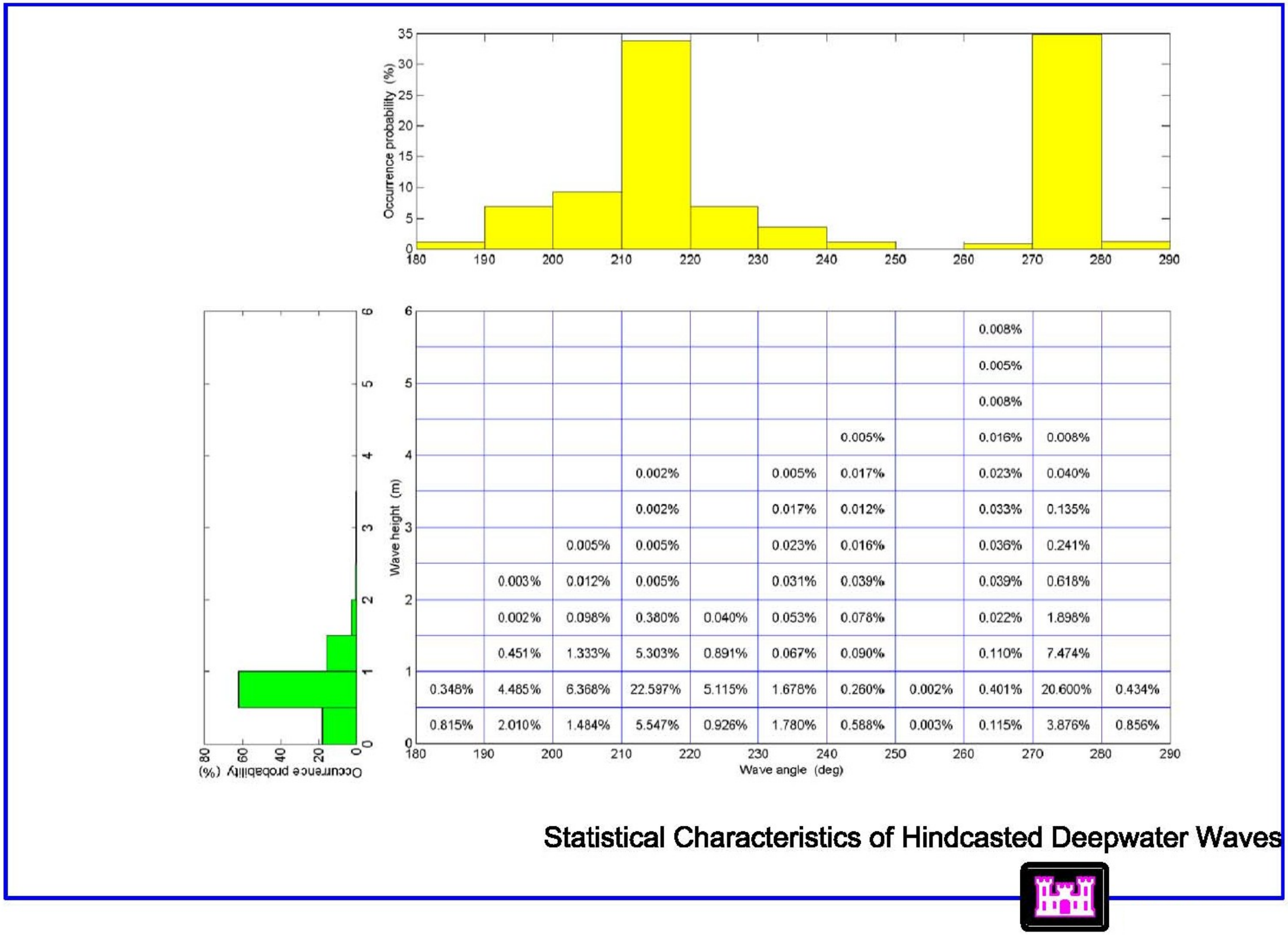


Figure 7.3-1 Statistical Characteristics of Hindcasted Deepwater Waves

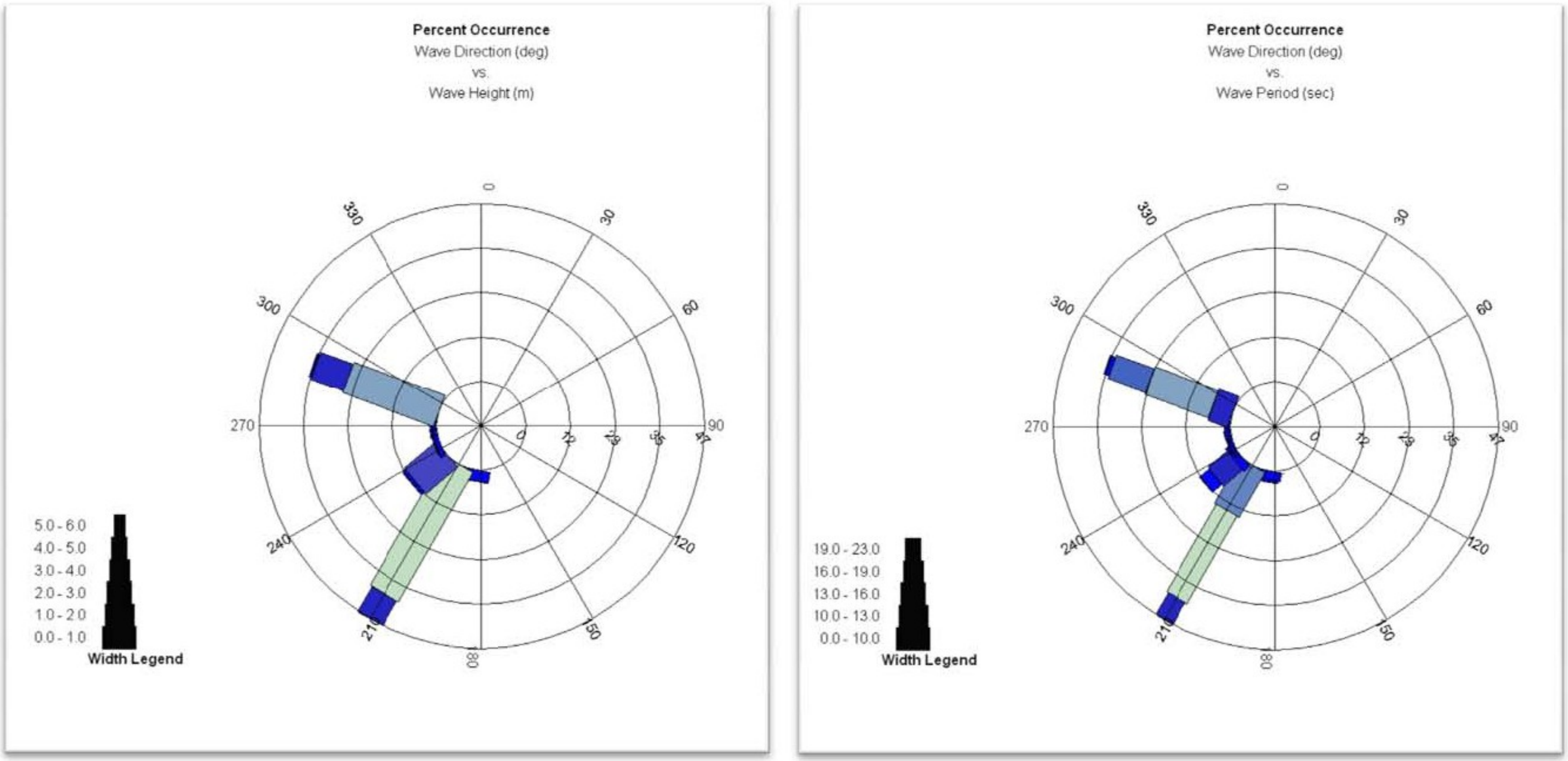


Figure 7.3-2 Wave Rose, 1979 - 2000

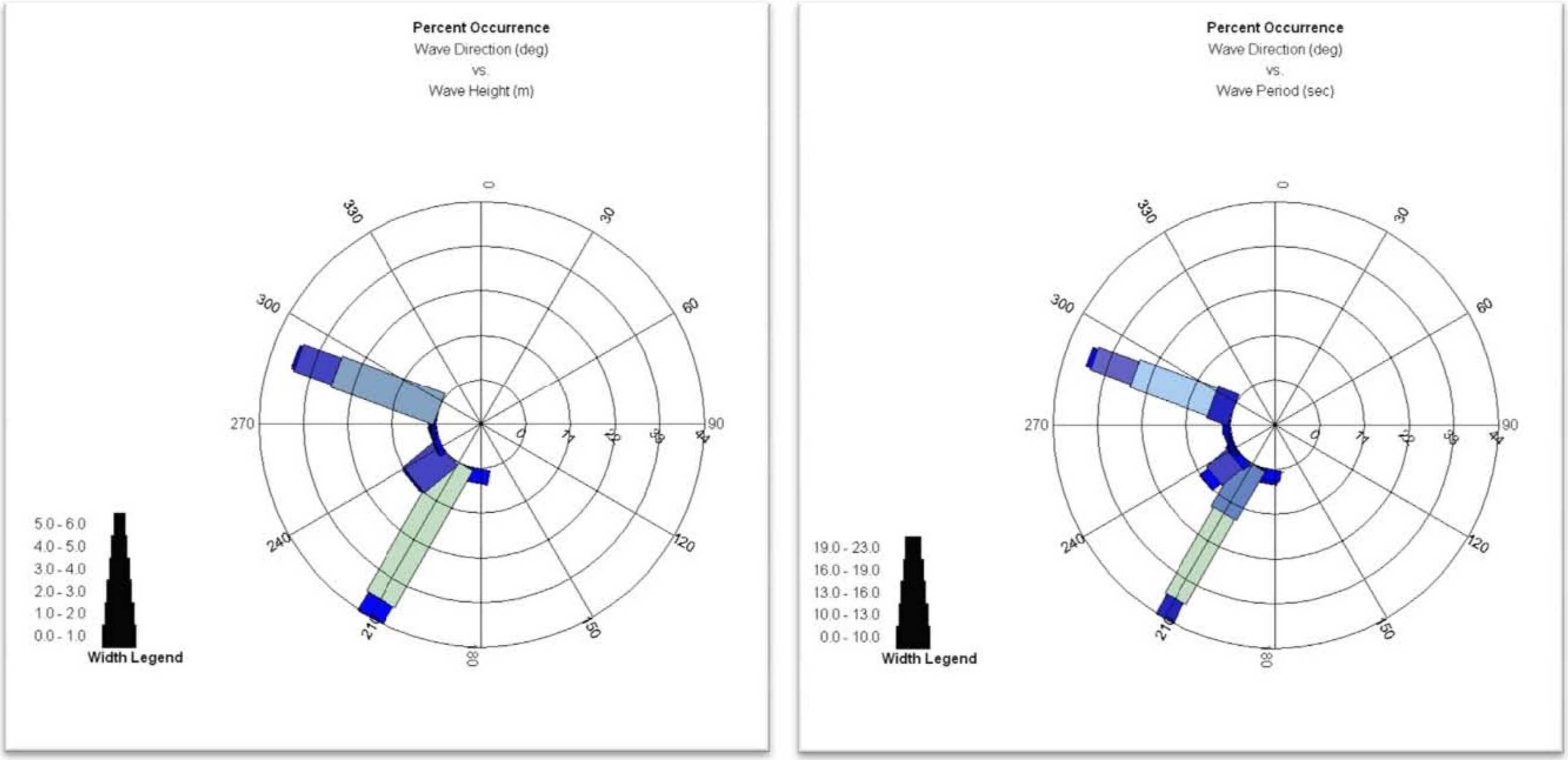
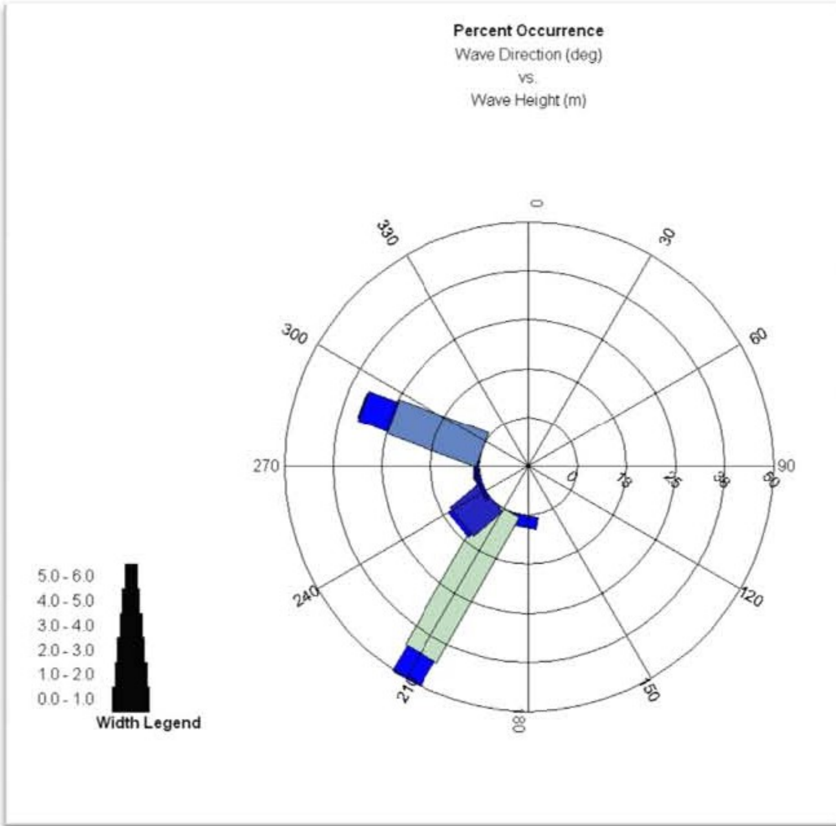


Figure 7.3-3 Wave Rose, 1979 - 1986



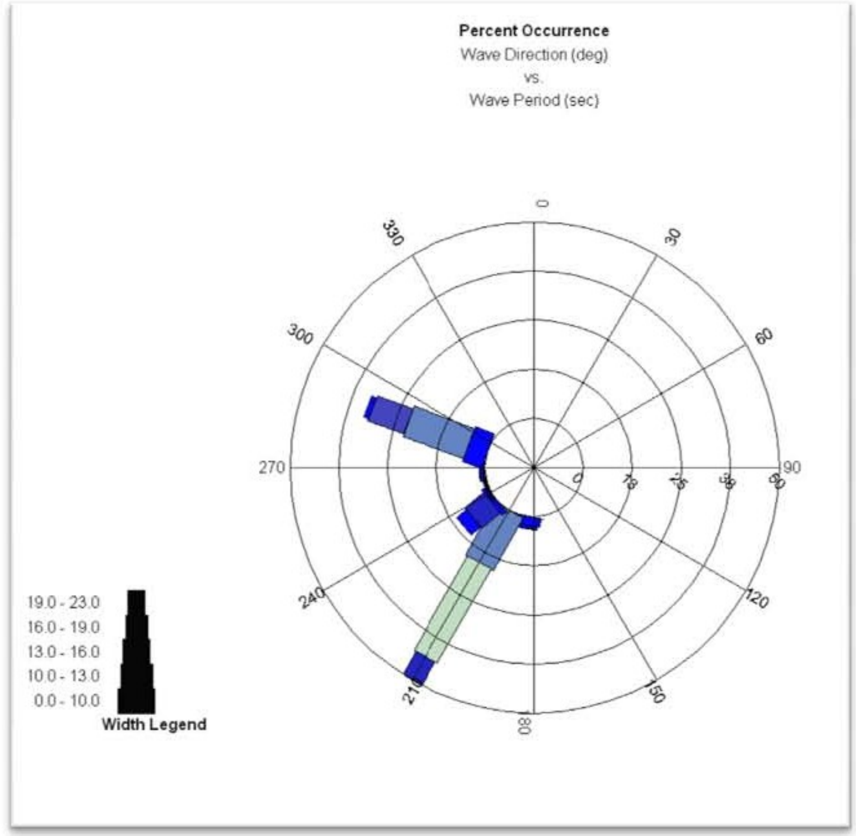


Figure 7.3-4 Wave Rose, 1983-1990

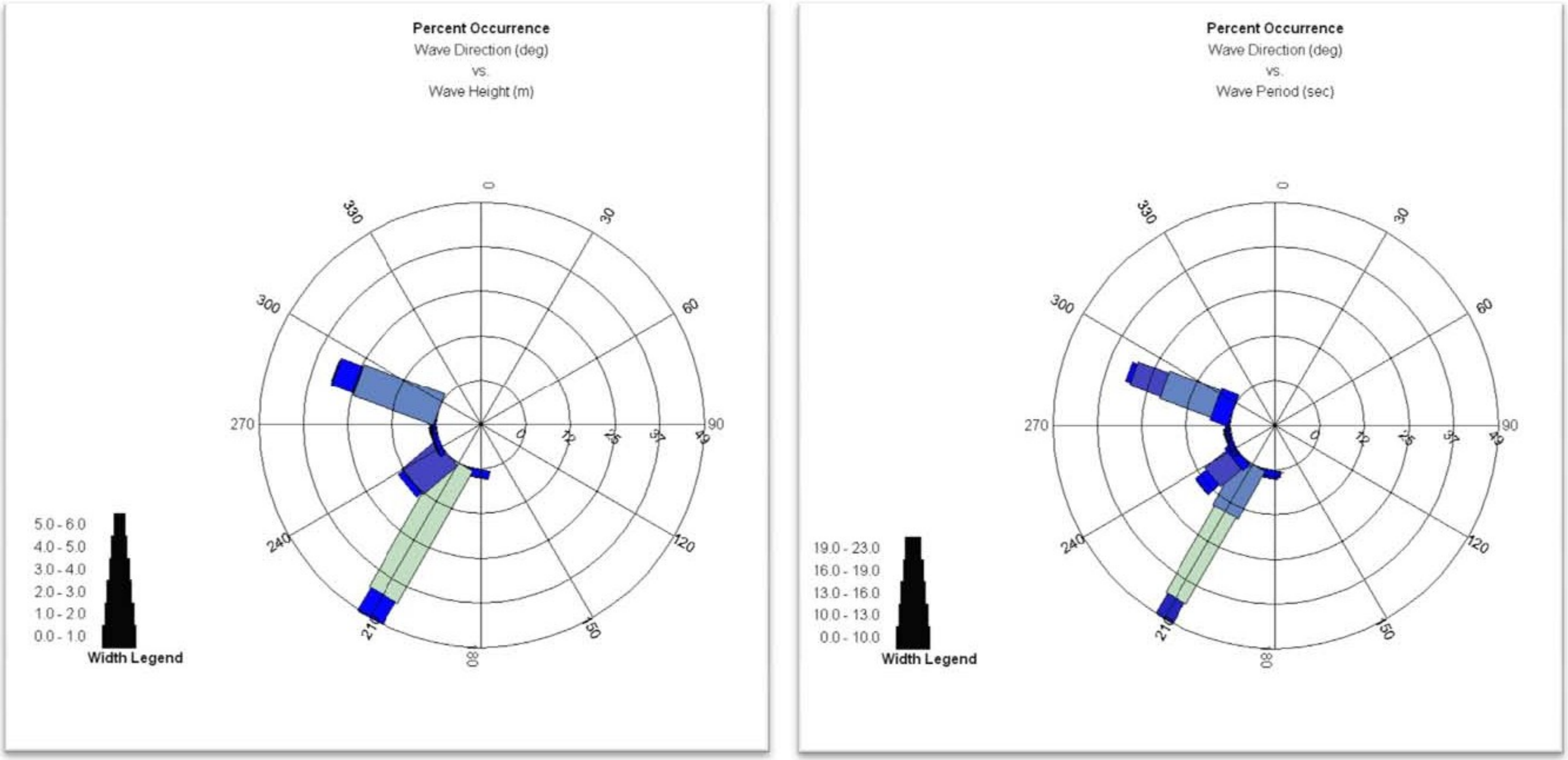
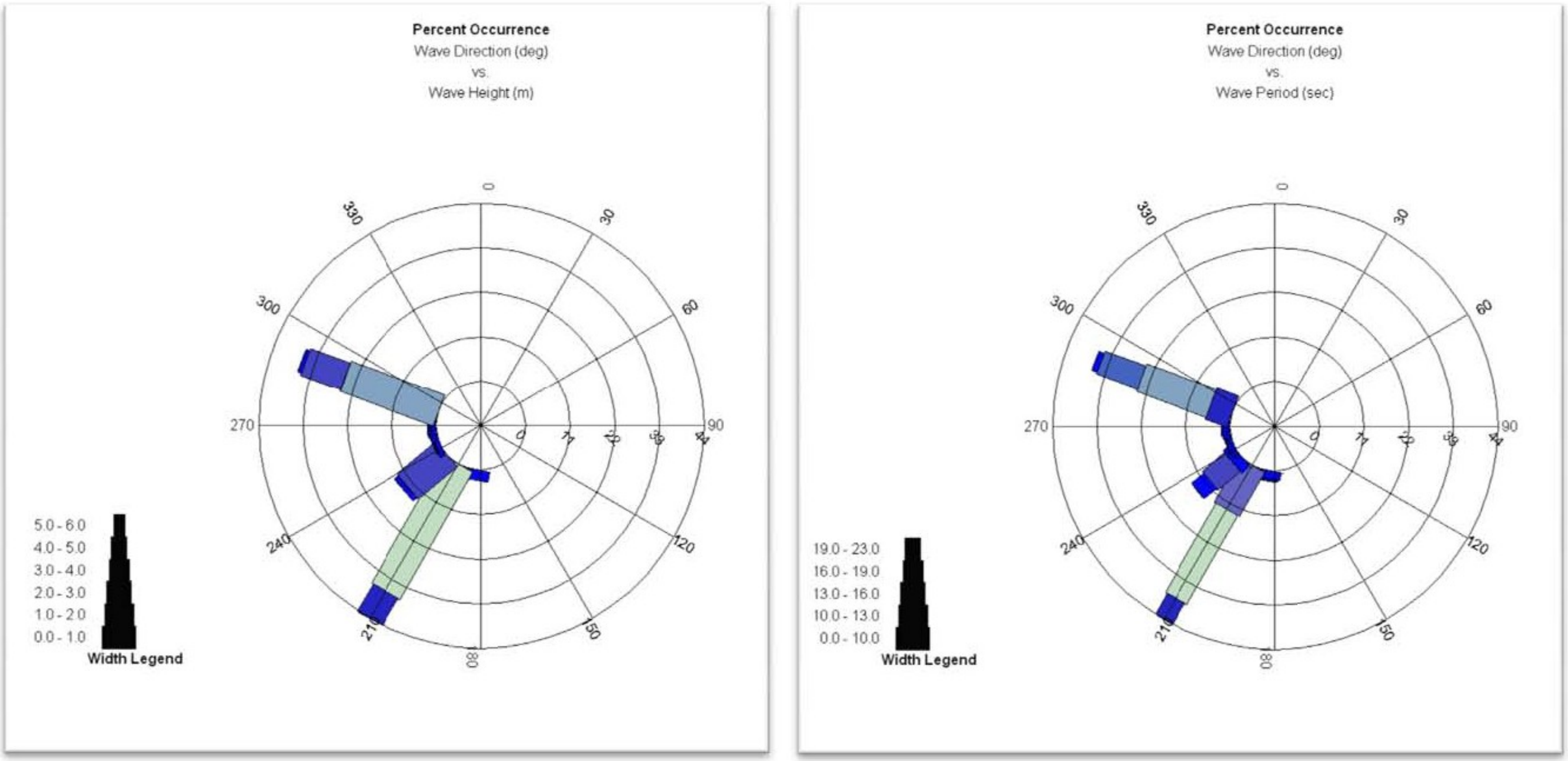


Figure 7.3-5 Wave Rose, 1987-1994



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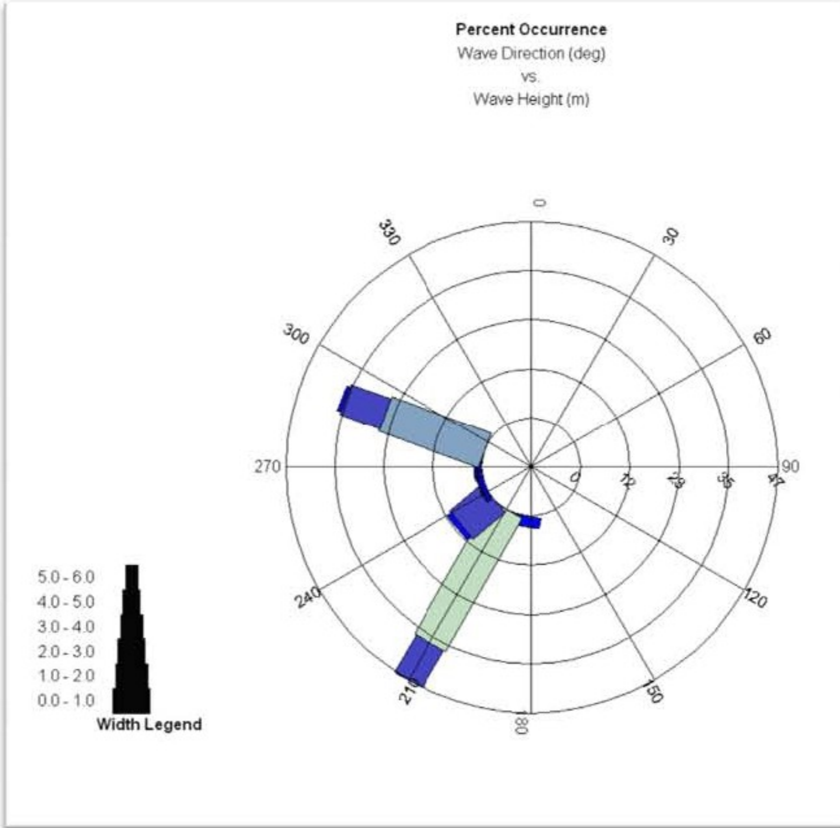
2 **Figure 7.3-6 Wave Rose, 1991-1998**



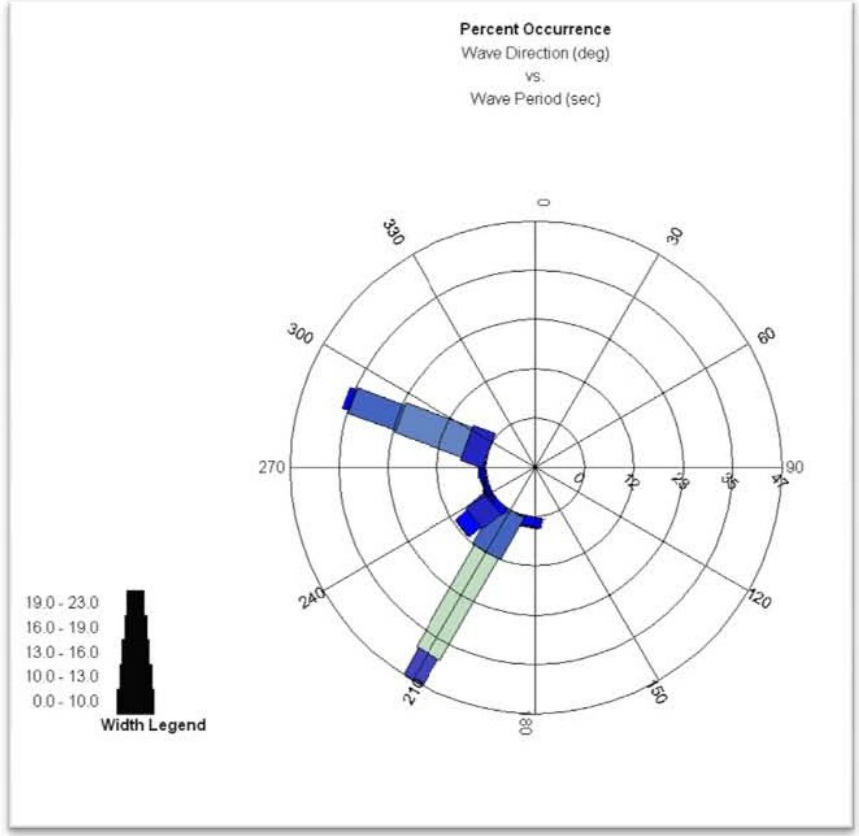
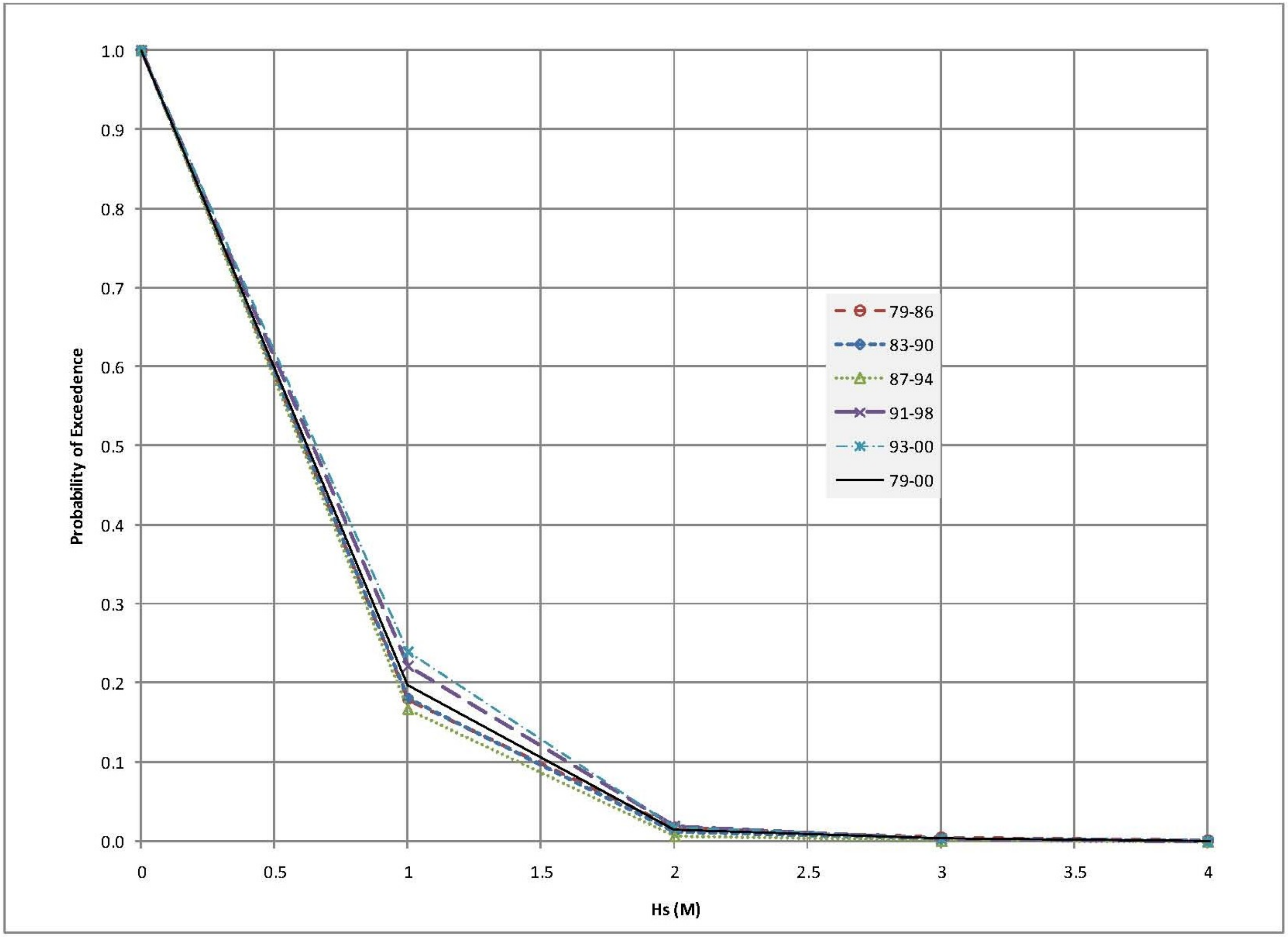


Figure 7.3-7 Wave Rose, 1993-2000



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2 **Figure 7.3-8 Wave Height Probability of Exceedence for All Hindcasted Deepwater Waves and for Wave Simulation Groups**

7.3.2 Sediment Grain Size

The GENESIS simulations depend heavily on the D_{50} interacting with the longshore breaking wave energy flux to drive longshore sediment transport. A review of measured beach and nearshore sediment sampling data was performed to characterize the existing beach grain sizes. The average of all D_{50} within the littoral cell from a 1983-1984 data set (USACE, 1991) was 0.19 mm. The average D_{50} from a 2009 data set developed to support the RBSPII was 0.17mm. Both of these were based on samples at elevations extending from 12 to -30 feet, MLLW and from profile DM-580 to SD-690 in the alongshore. Overall, the existing D_{50} within the study area littoral zone is between 0.17 to 0.19 mm.

Offshore sand sources exploited during the RBSPI had a D_{50} ranging from 0.14 to 0.62 mm. The potential borrow sand sources located within the San Diego region are generally characterized as medium sized sand with a D_{50} ranging from 0.34 mm to 0.62 mm (Noble Consultants, 2001). Since the primary purpose of the shoreline analysis is to investigate the post-nourishment shoreline evolution, a D_{50} of 0.34 mm is used in the GENESIS simulations to conservatively analyze the post-nourishment shoreline evolution in the model domain. Larger grain sizes stay where placed longer, which is conservative. Using a smaller grain size, which would be more representative of existing sand, would result in greater dispersal of the beach nourishment.

7.3.3 Depth of Closure and Berm Height

In the GENESIS model, the horizontal distance between the depth of closure and the backbeach berm height encompasses the limit within which the loss or gain of beach sand occurs. The depth of closure is defined as the seaward limit beyond which the beach profile exhibits negligible changes. The berm height occurs where the berm crest levels off. Based on the semi-annual beach profile surveys that were conducted between the fall of 1997 and the spring of 2002, the estimated depth of closure can be as high as -13 feet MLLW in Solana Beach, but typically ranges from -20 to -30 feet, MLLW throughout the study area (Coastal Frontiers Corporation, 2010). A representative elevation of -23.5 feet, MLLW was selected as the depth of closure for the GENESIS simulations. For the RBSPI, two berm heights of 11.75 and 12.5 feet MLLW were used in the study area (Moffatt & Nichol Engineers, 2000). It is expected that the Project beach nourishment alternative would have a similar beach nourishment configuration. Therefore, in this analysis the modeled beach nourishment berm height was 12.5 feet MLLW.

7.3.4 Reefs

Low relief reefs at Swamis (south end of Reach 5) and Table Tops (north end of Reach 8) were modeled as submerged breakwaters, simulating both sedimentation and longshore sediment transport in their lee. The reef at Swamis is located approximately 2,000 feet downcoast (southeast) of Encinitas-Segment 1. Table Tops is located at the north end of Solana-Segment 2. The reefs were modeled as permeable detached offshore breakwaters with transmission coefficients that match the sand bypassing capability of the reefs.

7.4 Model Calibration

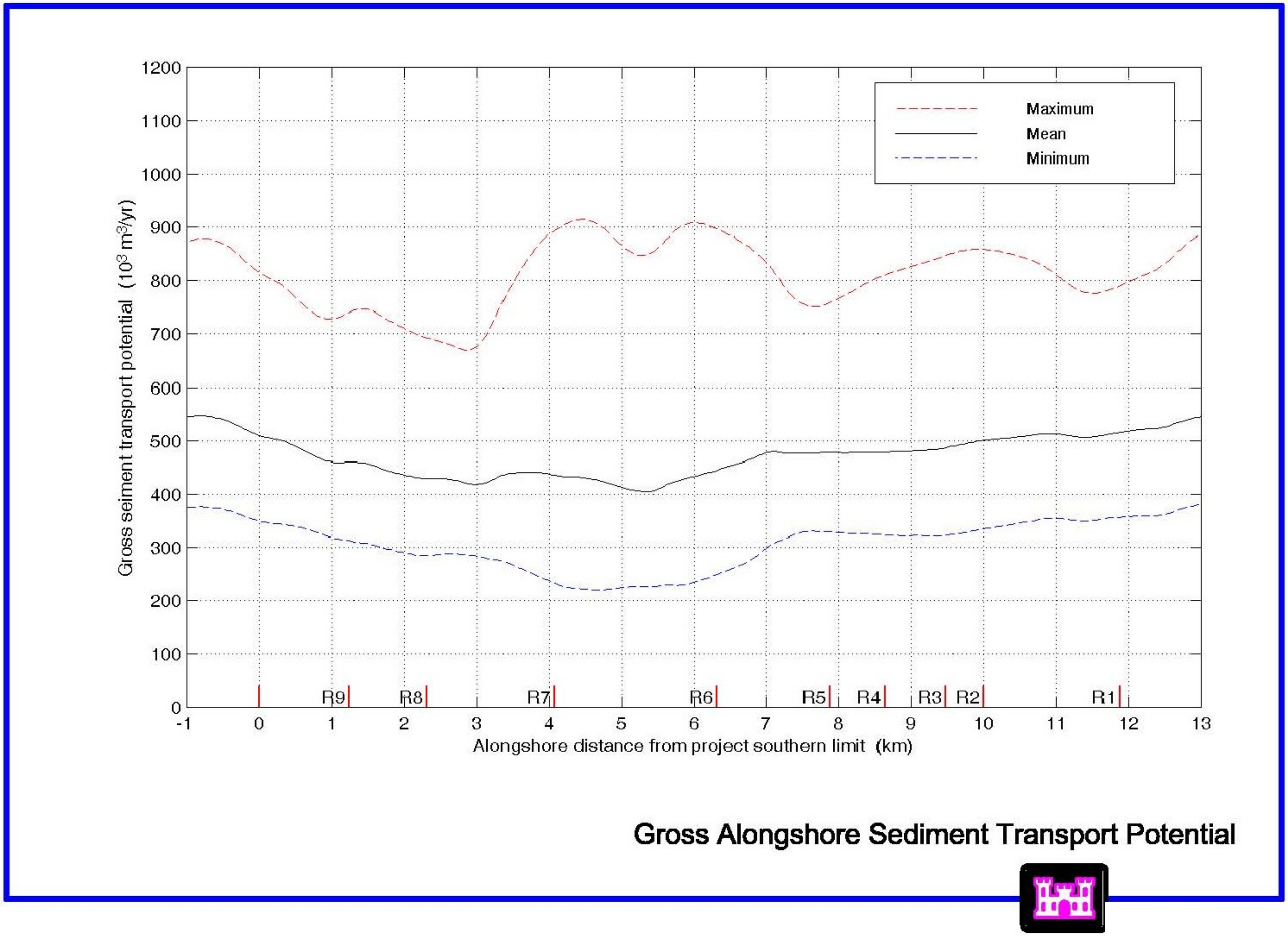
Model calibration is the process of adjusting model parameters by comparing modeling results against estimated or measured data. Several studies were prepared within the Oceanside Littoral Cell to assess and mitigate the Oceanside Harbor shoaling and adjacent beach erosion problems (Marine Advisors, 1960; Hales, 1978; Inman & Jenkins, 1983 and USACE-LAD, 1991). A general conclusion of these earlier studies was that within the Oceanside area (north portion of the GENESIS model domain) the gross transport rate is between 1.2 million and 1.4 million yd^3/year, with the net transports ranging from 102,000 to 253,000 yd^3/year to the southeast. The GENESIS model was calibrated to these previously estimated net and gross transport rates.

The K_1 and K_2 calibration coefficients were used to calculate the longshore sediment transport rate within the GENESIS model. These calibration coefficients were adjusted to obtain both the net and gross transport rates at the Oceanside reach within the ranges estimated by previous studies. K_1 was recommended to range from 0.58 to 0.77 by Hanson & Kraus (1989); from 0.1 to 1.0 by Gravens & Kraus (1991); and equal 0.39 by the Shore Protection Manual (USACE, 1984) Smaller K_1 values produce less transport and increase beach width longevity in the model. K_2 was recommended to range from 0.5 to 1.0 times K_1 by Hanson & Kraus (1989) and from 0.5 to 1.5 times K_1 by Gravens & Kraus (1991). Larger K_2 values tend to produce greater sedimentation in the lee of reefs or breakwaters, which can lead to tombolo development and model instability. Calibration coefficients used in other southern California projects are summarized in **Table 7.4-1.** This table shows that previous southern California projects used K_1 values ranging from 0.2 to 1.0 and K_2 values ranging from 0.3 to 1.0 times K_1.

Table 7.4-1 GENESIS Calibration Factors from Southern California Projects

	Current Study	RBSPI	RBSPII	Bolsa Chica Wetlands	Recommended
K1	0.55	0.8	1.0	0.2	0.1 – 1.0
K2	0.4	0.2	1.0	0.1	
K2/K1	0.7	0.3	1.0	0.5	0.5 - 1.5
Calibrated to	Sediment Transport	Sediment Transport	Shorelines	Shorelines	

The K_1 and K_2 calibration coefficients for the current GENESIS modeling were chosen to be 0.55 and 0.40 respectively. These yielded a net transport rate of 250,000 yd^3/year to the south and a gross transport rate of 1.3 million yd^3/year. These transport rates were averaged over the 22 year period from 1979 to 2000. Figure 7.4-1 and **Figure 7.4-2** show the calibrated, GENESIS model predicted, gross and net transport rates for the without Project conditions under the assumption that the future wave climate would be similar to that observed over the 22 year historical wave period. Negative values in **Figure 7.4-2** are southerly net transport. The net transport rate used in the calibration is consistent with the range used for the RBSPI study (Moffatt & Nichol Engineers, 2000). As shown in **Table 7.4-1**, K_1 in the current GENESIS modeling was in the recommended range and within the range of values used by other southern California projects. The ratio of K_2/K_1 was also within the recommended range and within the range of ratios used by other southern California projects.



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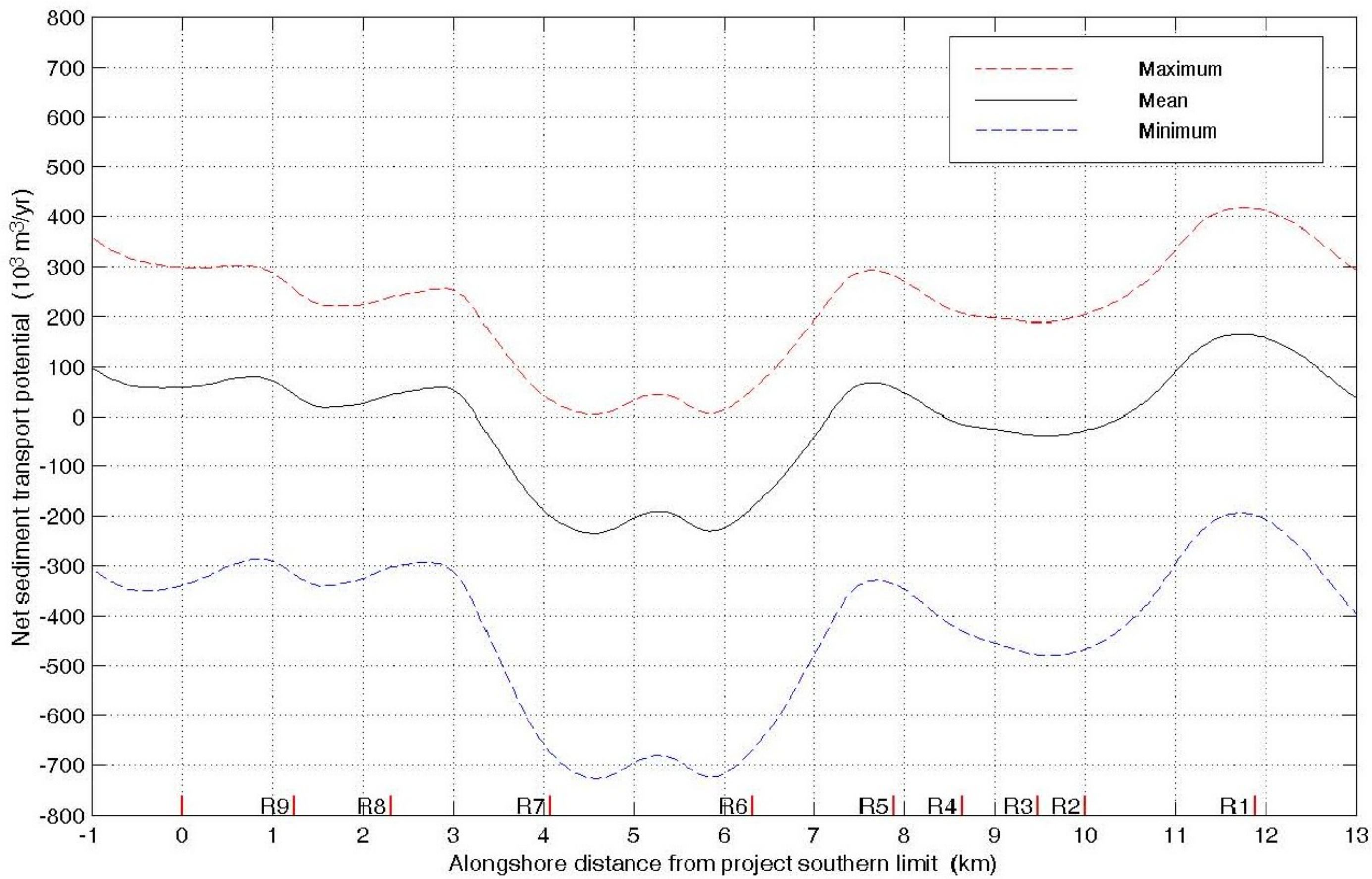
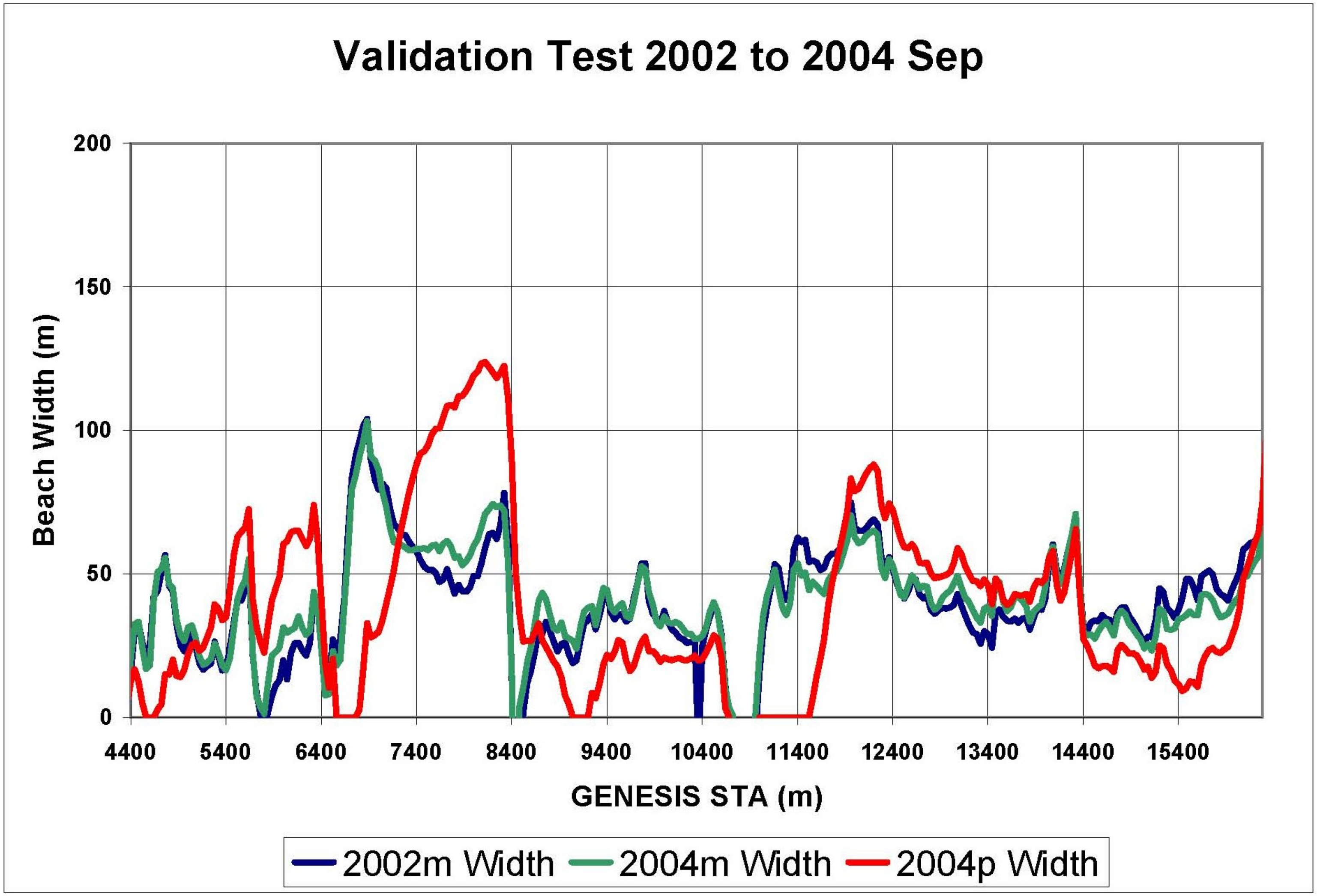
2 **Figure 7.4-1 Gross Alongshore Sediment Transport Potential**



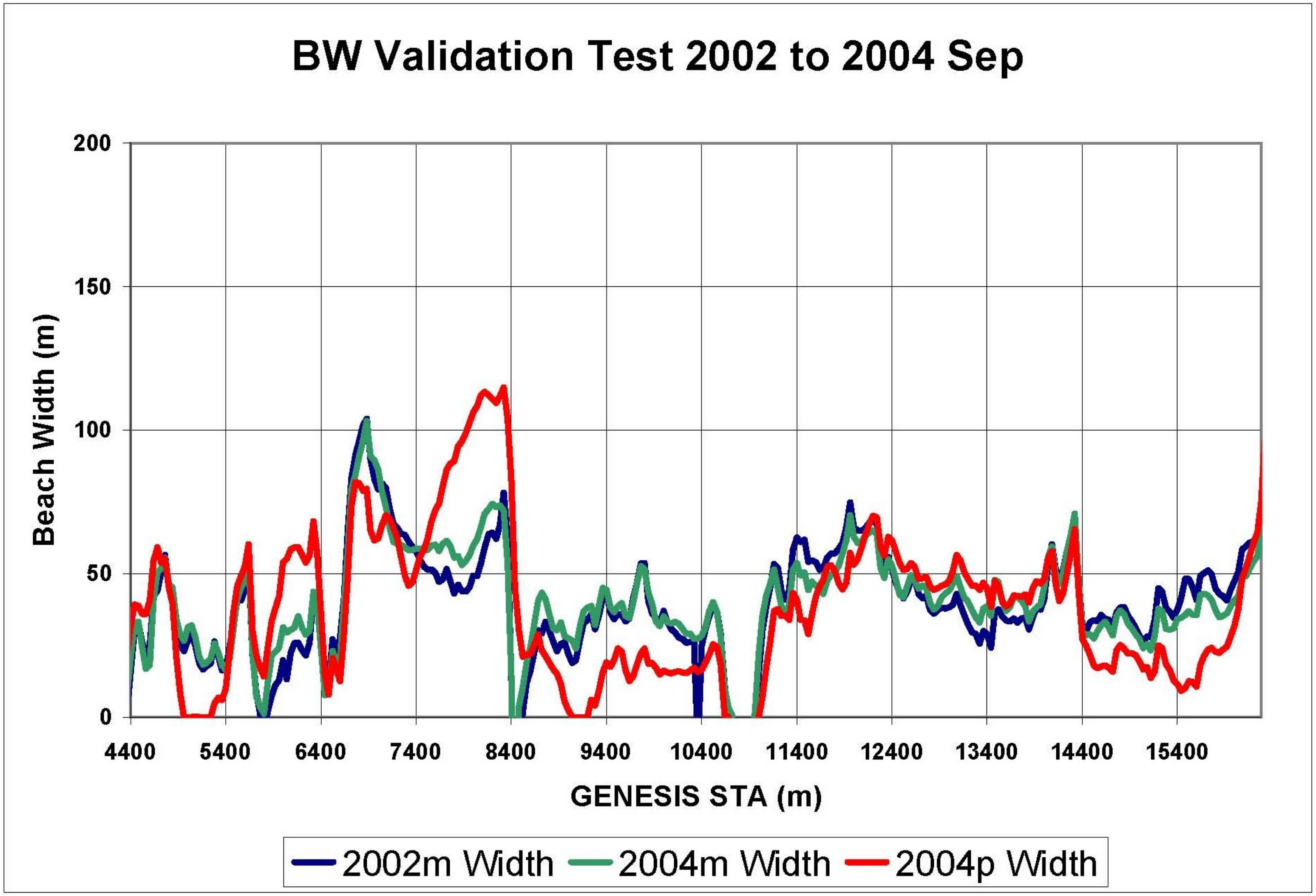
Net Alongshore Sediment Transport Potential

Figure 7.4-2 Net Alongshore Sediment Transport Potential



1 Diffracting Groins Scheme Validation of 2002 to 2004 Measured vs 2004 Predicted

2 **Figure 7.4-3 Validation Test 20002 to 2004 Sep**



Validation Test of 2002 to 2004measured vs 2004 predicted

Figure 7.4-4 BW Validation Test 2002 to 2004 Sep

The selection of whether to use shore normal groins or shore parallel submerged breakwaters to simulate reefs was based on a set of model runs compared to historical shoreline measurements. This calibration compared measured (M) and predicted (P) shoreline positions occurring between an initial 2002 shoreline and a final 2004 shoreline. Two types of structures were tested within the model for simulation of reefs. **Figure 7.4-3** shows results of using groins while **Figure 7.4-4** shows results from a model setup using detached, transmissive breakwaters to simulate the reefs. Breakwaters were chosen for simulating reefs for the remaining GENESIS modeling. The greatest model error (difference between predicted and measured shorelines) over this calibration period was 131 feet.

7.5 <u>Model Validation</u>

The above described GENESIS model configuration was validated by comparing a measured (M) shoreline against a GENESIS predicted (P) shoreline. This configuration used the above determined model calibration parameters as input. Details of the validation are as follows:

- The initial shoreline was the measured MSL shoreline from the May 2002 LiDAR survey (M 200205).
- The comparison shoreline was the measured MSL shoreline from the April 2004 LiDAR survey (M 200404).
- Since a wave record coincident with the validation period was not available in the hindcast deep water wave record, all five of the wave simulation groups were used and compared for both validity and sensitivity. One shoreline prediction was created for each wave simulation group.
-

GENESIS validation results are shown in Figure 7.5-1. In each graph, the x-axis is the distance along the GENESIS baseline in feet, and the y-axis is the shoreline position as measured normal to the GENESIS baseline in feet. The top panel shows the results across the entire GENESIS model domain. The middle panel shows results for the Encinitas-Segment 1 and the bottom panel shows results for the Solana-Segment 2. In these graphs, the non-erodible bluff toe is shown by the light grey line labeled “bluffline,” the measured starting shoreline is shown by the blue line labeled “M 200204,” the measured shoreline in year 2004 is black and labeled “M 200404.” The GENESIS predicted shorelines are various colors and labeled with a P for predicted, followed by the starting year of each wave simulation group that was used.

A perfect validation would show the predicted shoreline overlaying the measured shoreline (M 200404) exactly. Where and how much these two lines deviate indicates how much the model predictions differ from the measured shorelines. As can be seen, in most locations the measured shoreline is within the envelope of the predicted shorelines. Where the measured shoreline lies outside the predicted shorelines, some error exists beyond the differences attributed to choice of wave simulation group. One major source of model error that has been observed for other longshore sediment transport models is the cross shore variability. It has been observed that the shoreline can erode large distances over a single storm, chiefly driven by cross shore transport and this is captured in measured shoreline data, but not in the predicted shorelines. In addition, GENESIS is best at modeling long-term shoreline changes. When the validation period is short and the shoreline variation is on the same order of magnitude as the seasonal change, deviation from measured shorelines should be expected.

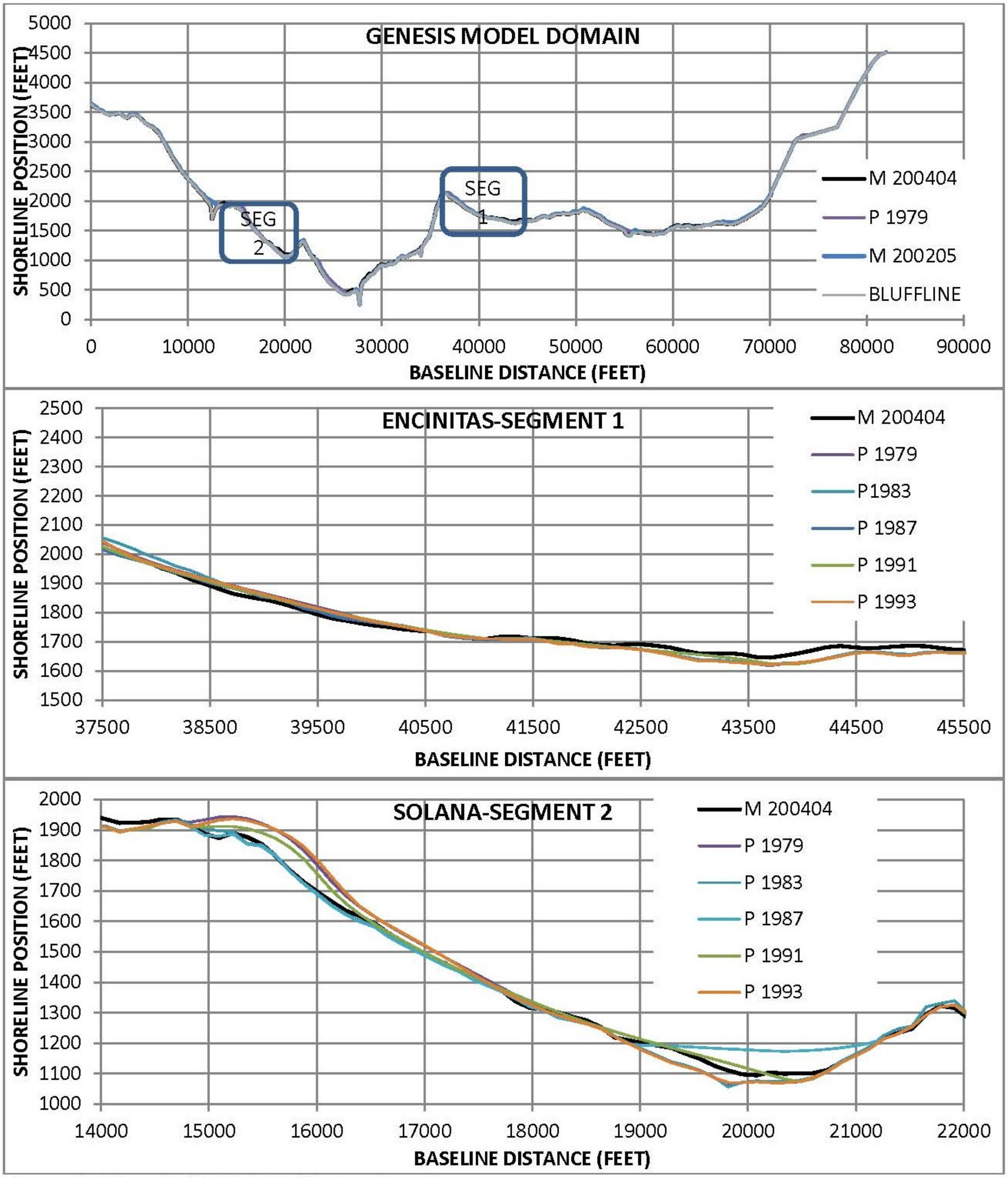


Figure 7.5-1 GENESIS Validation Results

7.6 Sensitivity Analysis

Uncertainties or natural fluctuations typically exist in the model parameters and inputs. The primary parameters and principal inputs that may potentially affect the predicted results of the longshore sediment transport rate and shoreline change include: empirical coefficients K_1 and K_2 for estimating the longshore sediment transport rate, D_{50} of beach sand, depth of closure, berm height, wave height, wave direction and permeability of reefs.

Three types of sensitivity analysis were performed to quantify the influence of these uncertainties on the predicted results. One was to investigate the predicted longshore sediment transport rates using the hindcasted waves on a yearly basis from 1979 to 2000. The second was for the resulting shoreline changes following a beach nourishment using the same wave characteristics for an 8 year span between 1993 and 2000. Although the sensitivity analysis was only performed for the 1993-to-2000 period, the variability of the modeled results and general conclusions are expected to be similar under different wave climatic periods. The third sensitivity analysis was performed to determine if mixing model results from different beach nourishment options at each segment was valid.

7.6.1 Longshore Sediment Transport Sensitivity

Since spatial variation of longshore sediment transport rate induces shoreline change in response to the conservation of sand volume, an accurate prediction of the longshore sediment transport rate is crucial to the predictability of shoreline evolution. Table 7.6-1 shows the sensitivity of the annual net and gross transport rates, averaged over 22 years from 1979 to 2000, to the various model parameters such as K_1, K_2 and D_{50} as well as nearshore wave height and direction (i.e., GENESIS's external wave inputs).

Table 7.6-1 Sensitivity Analysis of Longshore Sediment Transport Rates

Parameter	Model Value	Parameter Modification		Gross Transport Rate		Net Transport Rate	
				(yd^3/yr)	Difference	(yd^3/yr)	Difference
Calibration	N/A	N/A		1,288,702	N/A	249,637	N/A
K_1	0.55	-	0.15	977,636	- 24%	213,041	- 15%
		+	0.15	1,597,154	+ 24%	288,847	+ 16%
K_2	0.4	-	0.20	1,248,185	- 3%	215,655	- 14%
		+	0.20	1,326,605	+ 3%	281,005	+ 13%
D_{50} (mm)	0.34	-	0.16	1,346,210	+ 4%	333,285	+ 34%
		+	0.16	1,270,404	- 1%	233,953	- 6%
Wave Height (ft)	Varies	x	0.90	978,943	- 24%	198,664	- 20%
		x	1.10	1,603,689	+ 24%	304,531	+ 22%
Wave Dir (deg)	varies	-	5	1,275,632	- 1%	474,441	+ 90%
		+	5	1,235,115	- 4%	-10,456	- 96%

Both the gross and net transport rates are strongly sensitive to the parameter K_1 and wave height. An increase (or decrease) in K_1 by 27 percent results in an increase (or decrease) by 24 percent in the gross transport rate and more than 15 percent in the net transport rate. A change to the wave height by 10 percent induces a change of 24 percent to the gross transport rate and more than 20 percent to net transport rate, as the sediment transport rate is proportional to breaking wave energy or the wave height squared. While the gross transport rate is less sensitive to the K_2 parameter, D_{50} and nearshore wave direction, the net transport rate appears to be sensitive to those model parameters and inputs, particularly the nearshore wave direction. A 5-degree shift in wave direction resulted in a greater than 90 percent alteration of the net transport rate.

7.6.2 Shoreline Position Sensitivity

The sensitivity analysis for shoreline behavior was performed for a 300 foot beach nourishment option in Encinitas-Segment 1 under the 1993 to 2000 wave simulation group, with results provided in Table 7.6-2**Error! Reference source not found.**. Listed are the impacts of varying model parameters on the predicted shoreline change 5 years after the beach nourishment. Additional parameters such as the depth of closure, berm height, and the permeability coefficient of the reef at Swamis were also modified in this sensitivity analysis.

This sensitivity analysis indicates that the predicted shoreline evolution is sensitive to the K_1 parameter, wave height and wave approach angle, but less sensitive to the K_2 parameter, D_{50} and reef permeability. The average change to the simulated shoreline position in the 5^{th} year is more than 12 percent for a 27 percent change of K1, more than 11 percent for a 10 percent change of wave height, and more than 12 percent for a five degree change of wave angle. The many difficulties in shoreline modeling, particularly along shorelines where the updrift and downdrift rates are near equal, which Is thought to be the case in the Oceanside littoral cell. In addition, an increased distance between the berm height and depth of closure results in greater shoreline accretion due to variation of sand volume across the beach profile. It is noted that the predicted shoreline position is relatively insensitive to reef permeability even when the reef is located adjacent to the beach nourishment. This is attributed to the length of the reefs being relatively short compared to the extent of the surf zone where the majority of longshore sediment transport occurs (i.e., the sand entrapment is limited due to the short length of the reefs).

Table 7.6-2 Shoreline Sensitivity After Beach Nourishment for Encinitas-Segment 1

Parameter	Model Value	Parameter Modification	After 5 Years	
			Shoreline Change (ft)	Difference
Calibration	N/A	N/A	189	N/A
K_1	0.55	- 0.15	214	+ 13.0%
		+ 0.15	166	- 12.3%
K_2	0.4	- 0.20	189	+ 0.2%
		+ 0.20	188	- 0.3%
D_{50} (mm)	0.34	- 0.16	188	- 0.7%
		+ 0.16	189	+ 0.2%
Wave Height (m)	varies	× 0.90	210	+ 11.3%
		× 1.10	166	- 12.2%
Wave Dir (deg)	varies	- 5	209	+ 10.4%
		+ 5	144	- 23.8%
Depth of Closure (ft, MLLW)	-23.5	+ 3.3	180	- 4.7%
		- 3.3	196	+ 3.8%
Berm Height (ft, MLLW)	12.5	- 3.3	180	- 4.7%
		+ 3.3	196	+ 3.8%
Permeability	0.5	- 0.5	189	0.0%
		+ 0.5	189	0.0%
Permeability **	0.5	- 0.5	194	+ 2.8%
		+ 0.5	189	0.0%

** Groin is moved to the immediate down-coast end of the beach nourishment

7.6.3 Sensitivity of Segments to One Another

The goal of this third sensitivity analysis was to see how sensitive or independent the shoreline positions of each segment were to one another. This was done to determine if combining shoreline modeling results from configurations with one beach nourishment option at Encinitas-Segment 1 and a different beach nourishment option at Solana-Segment 2 is meaningful.

This sensitivity analysis was carried out by running various combinations of beach nourishment options at the two segments through GENESIS and comparing resulting shoreline positions. The model simulations performed were: 50 foot beach nourishment option in Encinitas-Segment 1 and 50 foot beach nourishment in Solana-Segment 2 (50’/50’ Option), 200 foot in Encinitas-Segment 1 and 200 foot in Solana-Segment 2 (200’/200’ Option), 50 foot in Encinitas-Segment 1 and 200 foot in Solana-Segment 2 (50’/200’ Option), and 200 foot in Encinitas-Segment 1 and 50 foot in Solana-Segment 2 (200’/50’ Option). These configurations were chosen to capture a wide range of possible beach nourishment option combinations. These configurations were modeled for all five wave simulation groups and average shoreline positions were developed.

The 50'/50' Option and the 50'/200' Option both have 50 foot beach nourishment options at Encinitas-Segment 1. Subtracting the 50'/50' Option from the 50'/200' Option shoreline positions results in a maximum shoreline position difference of 0.0004 feet within Encinitas-Segment 1 as illustrated in **Figure 7.6-1**. This difference is very small relative to other shoreline results as would be expected if the segments behaved independently. If the segments were sensitive (i.e., dependent) to one another, one would expect the 200 foot beach nourishment option from Solana-Segment 2 to bleed over to the Encinitas-Segment 1 differently than the 50 foot beach nourishment option. A significant difference between these two amounts of bleed into Encinitas-Segment 1 would show up in the shoreline difference calculation and would indicate sensitivity, or co-dependence. Since the difference between the two options is negligible, the two segments are independent for this test.

Similar comparisons were developed for the other comparisons of beach nourishment options as shown in **Figures 7.6-2** through **Figure 7.6-4** and summarized in **Table 7.6-3**.

Table 7.6-3 Segment Sensitivity or Independence

Compared Options	Segment with Shared Beach nourishment option	Absolute Maximum Difference (feet)	Independent or Sensitive
50'/200' - 50'/50'	Encinitas-Segment 1	0.0004	Independent
50'/200' - 200'/200'	Solana-Segment 2	0.5	Independent
200'/50' - 50'/50'	Solana-Segment 2	0.5	Independent
200'/50' - 200'/200'	Encinitas-Segment 1	0.06	Independent

From the above comparison it was concluded that, for the shoreline modeling performed for this Project, both the economic and environmental results of modeling different beach nourishments at each segment are independent from one another. Therefore, combining economic or environmental results from differing modeled beach nourishments at each segment into one combined alternative is justifiable.

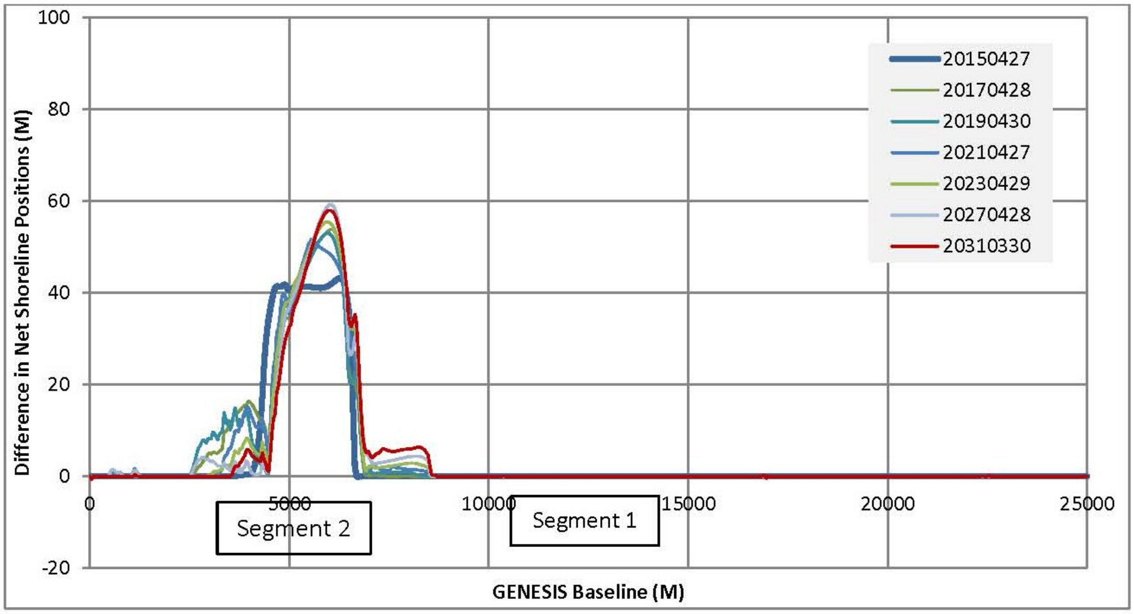


Figure 7.6-1 50'/200' Option Minus 50'/50' Option Shoreline Positions

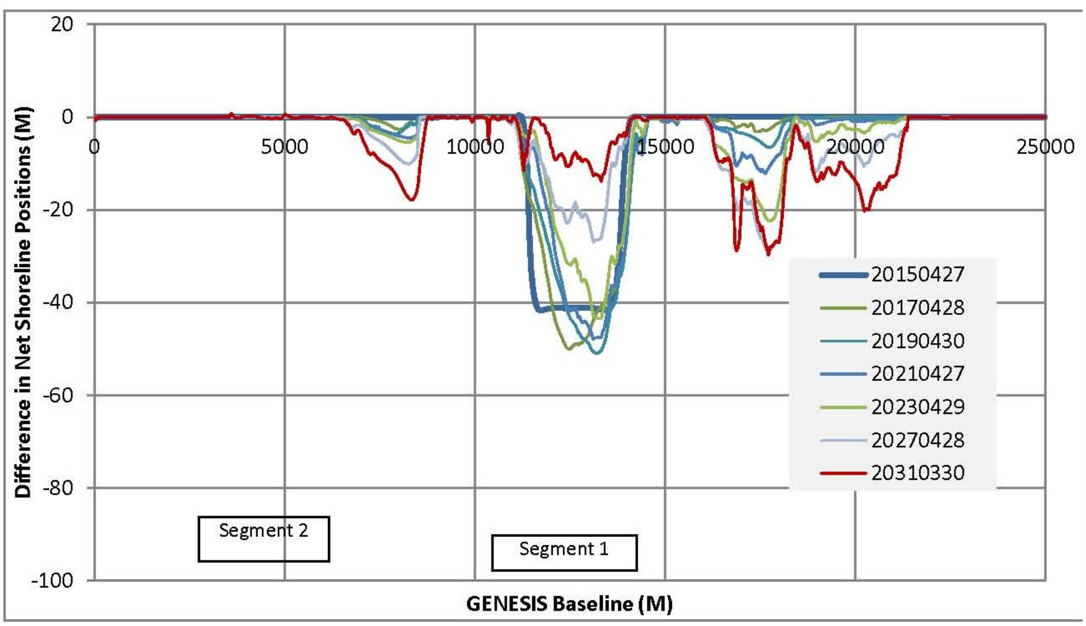


Figure 7.6-2 50'/200' Option Minus 200'/200' Option Shoreline Positions

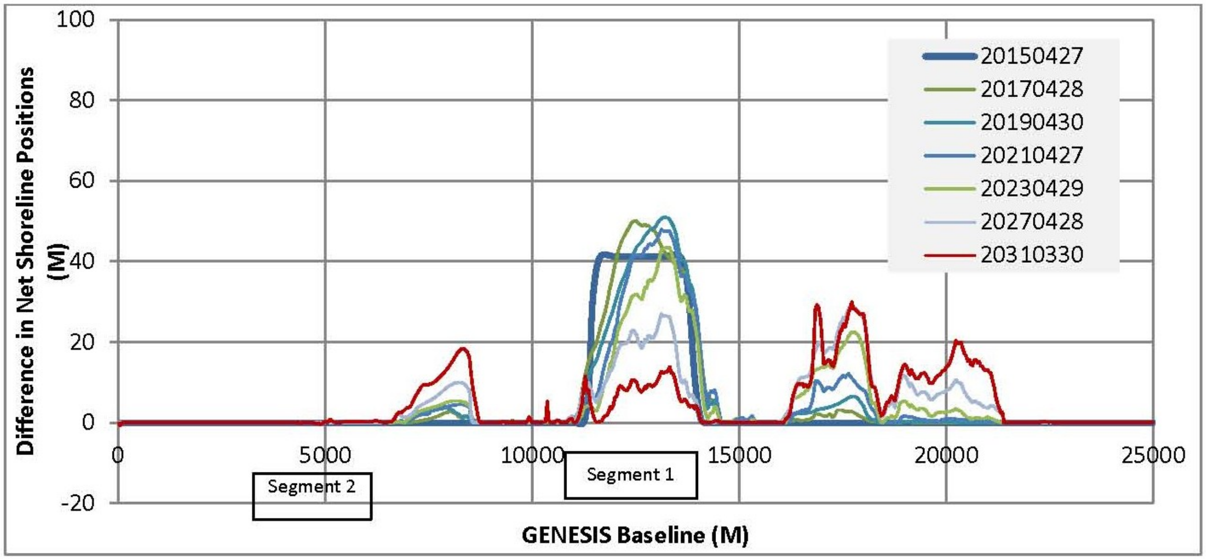


Figure 7.6-3 200'/50' Option Minus 50'/50' Option Shoreline Positions

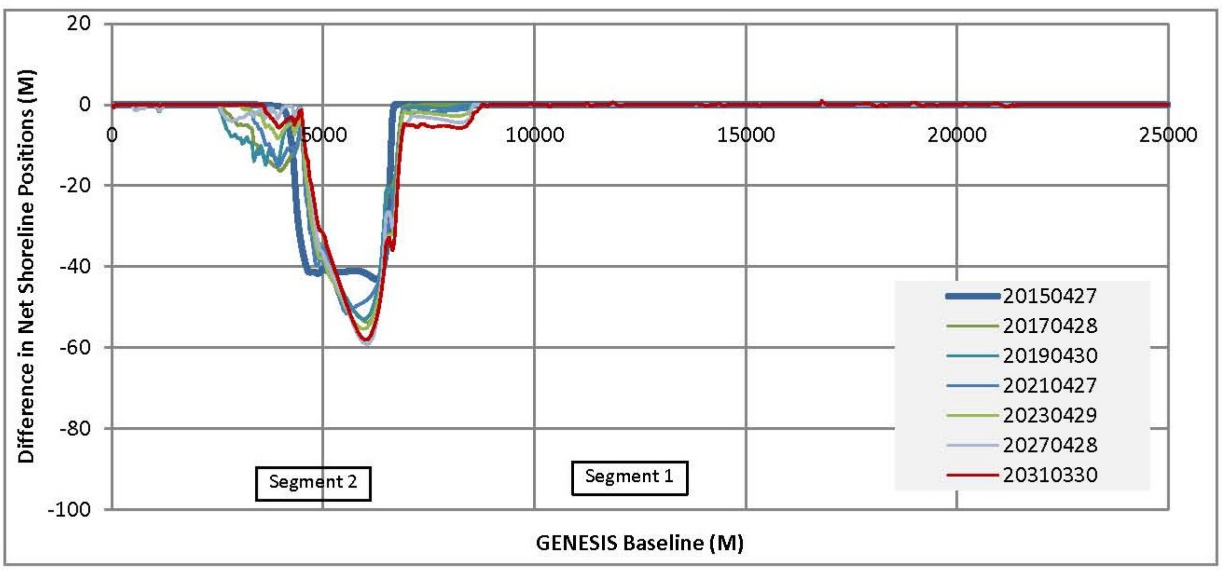


Figure 7.6-4 200'/50' Option Minus 200'/200' Option Shoreline Positions

7.7 Modeled Results

To provide an adequate data base from which impacts could be calculated and an optimization could be performed, various combinations of parameters were input and simulated within the GENESIS model, including:

- Beach nourishment options consisting of 50, 100, 150, 200, 250, 300, 350, and 400 feet; at Encinitas-Segment 1 and Solana-Segment 2;
- Five wave simulation groups; and
- A model duration of 16 years after the base-year.

Cycling the 8 year record of each wave simulation group allowed shoreline modeling to simulate 16 years. The beach widths associated with the beach nourishment options are equilibrium MSL beach widths. These are narrower than as-built or constructed beach widths and typically occur a few months after construction as the constructed beach comes to an equilibrium in the cross-shore direction and some of the nourished material moves offshore. For the remainder of this report, the widths associated with beach nourishment options refer to the equilibrium beach width at the MSL elevation, not the constructed width.

7.7.1 Simulated Average Shoreline Changes in Encinitas-Segment 1

A preliminary GENESIS simulation was first conducted for Reaches 3 to 5 extending from the 700 block of Neptune Avenue to Swamis. However, the preliminary model results indicated that a small portion of the filled beach immediately adjacent to Swamis was subjected to rapid depletion due primarily to the existing shoreline configuration. Therefore, the total alongshore length of the beach nourishment was scaled back to 1.5 miles, extending from the 700 block of Neptune Avenue only to approximately 2,000 feet upcoast of Swamis. This also reduced predicted impacts on the existing rock habitats and surf site at Swamis.

Figure 7.7-1 shows the predicted shoreline change for each modeled year, under each wave simulation group, for the 50 foot beach nourishment option, obtained by subtracting the Project shoreline from the without Project shoreline and spatially averaging these net shorelines over all GENESIS cells within Segment 1. Net shorelines are shown to decrease with time after the initial beach nourishment as sand is laterally dispersed moving along the shoreline in both upcoast and downcoast directions. For example, the average net shoreline under the 1979-1986 wave simulation group retreats from the base-year width of 50 feet to 48 feet in 2 years, 32 feet in 4 years, 31 feet in 6 years and 0 feet in years 12 through 16 as indicated in the table. The temporal shoreline retreat depends strongly on wave simulation group. Shoreline change tables for the other beach nourishment options are provided in **Appendix B7**. Like the 50 foot beach nourishment option, the other beach nourishment options all show shoreline retreat over the 16 year model duration and variation between wave simulation groups.

Table 7.7-1 Encinitas-Segment 1 Average Net Shoreline Change After a 50 foot Beach Nourishment Option

Wave Simulation Group		1979-1986	1983-1990	1987-1994	1991-1998	1993-2000
Year	Date	Average Net Shoreline Change (feet)				
0	20150427	44	43	44	44	43
1	20160429	49	48	47	49	45
2	20170428	48	48	23	48	37
3	20180427	47	36	9	33	37
4	20190430	32	29	0	25	34
5	20200428	34	13	0	25	26
6	20210427	31	0	0	23	12
7	20220430	16	0	0	15	0
8	20230429	9	6	0	4	0
9	20240427	2	6	0	2	0
10	20250430	2	3	0	0	3
11	20260429	1	2	0	0	2
12	20270428	0	1	0	3	2
13	20280430	0	0	0	1	0
14	20290429	0	0	0	0	0
15	20300428	0	0	0	0	0
16	20310330	0	0	0	0	0

Table 7.7-2 summarizes the temporal variation of scenario-mean shoreline change for various beach nourishment options. The scenario-mean shoreline change in each year is obtained by averaging the predicted shoreline changes under the five wave simulation groups.

Table 7.7-2 Summary of Scenario-Mean Net Shoreline Change for Encinitas-Segment 1

Beach nourishment Option (feet)		50	100	150	200	250	300	350	400
Year	Date	Scenario-Mean, Net Shoreline Change (feet)							
0	20150427	43	87	131	174	218	261	305	349
1	20160429	48	95	140	183	225	267	309	351
2	20170428	41	87	130	172	212	251	289	327
3	20180427	32	76	118	159	197	233	269	305
4	20190430	24	68	110	150	186	221	255	288
5	20200428	20	61	102	140	176	208	240	271
6	20210427	13	49	88	125	159	191	223	253
7	20220430	6	32	69	105	138	169	200	229
8	20230429	4	24	60	95	127	157	186	214
9	20240427	2	18	50	84	115	144	172	199
10	20250430	2	16	42	74	105	133	160	187
11	20260429	1	11	35	63	92	119	146	172
12	20270428	1	5	29	57	87	114	140	166
13	20280430	0	6	24	51	79	106	132	156
14	20290429	0	4	20	41	66	92	117	141
15	20300428	0	1	14	29	50	74	98	122
16	20310330	0	1	11	24	42	66	90	113

7.7.2 Simulated Shoreline Evolution in Encinitas-Segment 1

It is intuitive that the shoreline within a segment generally retreats back with time, while the adjacent areas outside the segment accrete. Due to the spatial variation of longshore sediment transport resulting from shoreline orientation and approaching wave characteristics, the dispersal and evolution of a nourished shoreline vary spatially within the segment. Depending on the shoreline configuration, the spatial variation of shoreline change after the beach nourishment can be significant. **Figure 7.7-1** shows an example of 15 years of shoreline evolution at Encinitas-Segment 1 based on a 50 foot beach nourishment option. The x-axis is the distance along the GENESIS model baseline and the y-axis indicates the scenario-mean, net shoreline position. Where the net equals zero, the Project shoreline matches the without Project shoreline. Where there is a positive net value, the Project shoreline is wider than the without Project shoreline. In this figure it can be seen that the shoreline generally recedes within Encinitas-Segment 1 over time. By year 10, the net shoreline is estimated to be reduced to less than 10 feet across the entire segment, while material migrates laterally upcoast and downcoast, widening the adjacent beaches. By year 15, the Project shoreline equals the without Project shoreline and there is no net Project impact within the segment.

Shoreline evolution graphs for both segments, bracketing the entire range of beach nourishment options from 50 to 400 feet are provided in **Appendix B7**. Similar shoreline recession occurs for the wider beach nourishment options, but with less netting out at 0 feet.

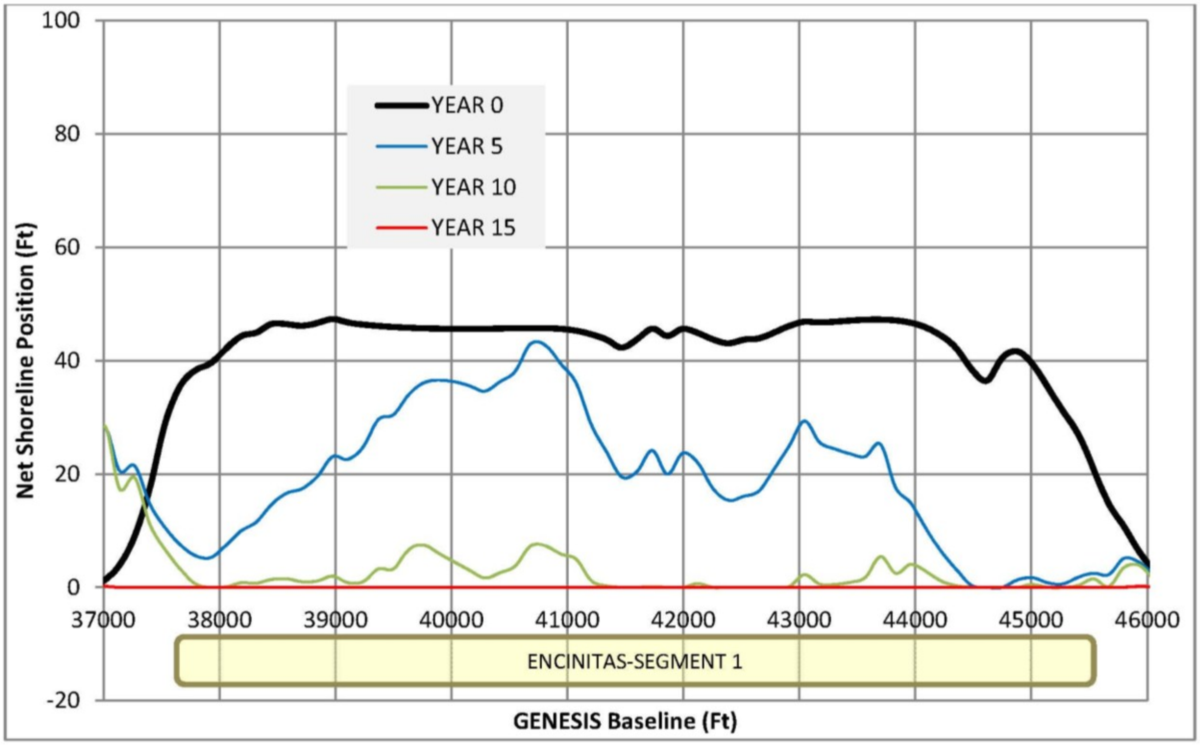


Figure 7.7-1 Shoreline Evolution for 50 Foot Beach Nourishment in Encinitas-Segment 1

7.7.3 Simulated Average Shoreline Change in Solana-Segment 2

As described above, the beach nourishment in Solana-Segment 2 extends from Table Tops to the southern limit of Solana Beach with the same beach nourishment options used for Encinitas-Segment 1. For the 50 foot beach nourishment option, the segment wide average net shoreline changes for every modeled year under the five wave simulation groups are presented in

Table 7.7-3. The trend of temporal variation in shoreline change is similar to that modeled for Encinitas-Segment 1. Since shoreline evolution depends strongly on the impinging waves, it is expected that the resultant average shoreline change would vary under different wave simulation groups. For example, the modeled shoreline change in year 5 year ranges from 29 to 32 feet, depending on wave simulation group.

Table 7.7-3 Solana-Segment 2 Average Net Shoreline Change After a 50 Foot Beach Nourishment Option

Wave Simulation Group		1979-1986	1983-1990	1987-1994	1991-1998	1993-2000
Year	Date	Average Net Shoreline Change (feet)				
0	20150427	44	44	44	44	43
1	20160429	40	41	41	41	41
2	20170428	37	37	38	40	43
3	20180427	37	40	33	37	40
4	20190430	37	37	32	35	37
5	20200428	30	31	29	32	32
6	20210427	29	22	24	28	27
7	20220430	25	18	18	23	21
8	20230429	21	17	16	19	17
9	20240427	17	15	11	16	11
10	20250430	14	12	2	10	10
11	20260429	11	7	0	4	7
12	20270428	7	5	0	2	4
13	20280430	3	0	0	0	0
14	20290429	1	0	0	0	0
15	20300428	0	0	0	0	0
16	20310330	0	0	0	0	0

Table 7.7-4 summarizes the temporal variation of scenario-mean net shoreline change for various beach nourishment options. These shoreline changes for Solana-Segment 2 are expected to decrease from the beach nourishment option width of 50 feet to 41 feet in year 2, 37 feet in year 4, 26 feet in year 6 and continue receding to 0 feet from years 13 to 16. Shoreline change tables for the other beach nourishment options are provided in **Appendix B7**. Segment wide shoreline changes for the other beach nourishment options also recede, but do not reach 0 feet.

Table 7.7-4 Summary of Scenario-Mean Net Shoreline Change for Solana-Segment 2

Beach Nourishment Option (feet)		50	100	150	200	250	300	350	400
Year	Date	Scenario-Mean, Net Shoreline Change (feet)							
0	20150427	44	87	131	175	218	262	306	349
1	20160429	41	84	127	170	214	258	302	346
2	20170428	39	82	123	164	205	247	288	330
3	20180427	37	82	122	161	201	240	280	321
4	20190430	35	81	121	158	194	231	268	306
5	20200428	31	77	119	156	191	226	262	299
6	20210427	26	71	115	154	192	227	264	301
7	20220430	21	67	110	150	186	221	257	294
8	20230429	18	64	107	149	186	220	254	288
9	20240427	14	59	103	143	179	213	247	282
10	20250430	10	56	99	139	174	209	244	278
11	20260429	6	51	94	135	172	206	239	273
12	20270428	3	49	92	132	166	198	230	261
13	20280430	0	44	87	126	162	193	226	257
14	20290429	0	38	80	120	156	191	226	258
15	20300428	0	33	75	114	151	184	218	251
16	20310330	0	30	73	112	149	184	216	248

7.7.4 Simulated Shoreline Evolution Solana-Segment 2

Figure 7.7-2 shows the scenario-mean net shoreline change from years 0 through 15 for the 50 foot beach nourishment option. In this figure, it can be seen that shorelines within the upcoast areas (e.g., x= 19,000 feet) erode slowly and downcoast areas (e.g., x=14,000 feet) are quickly depleted. As a consequence of wave transformation and shoreline orientation in this segment and the net upcoast transport direction, beach nourishment in the downcoast portion is expected to be partially transported and deposited in the upcoast areas of the segment.

Net shoreline evolution graphs for both segments, bracketing the entire range of beach nourishment options from 50 to 400 feet are provided in **Appendix B7**. Similar shoreline recession occurs for the wider beach nourishment options, but with more accretion in the upcoast area and less erosion in the downcoast area.

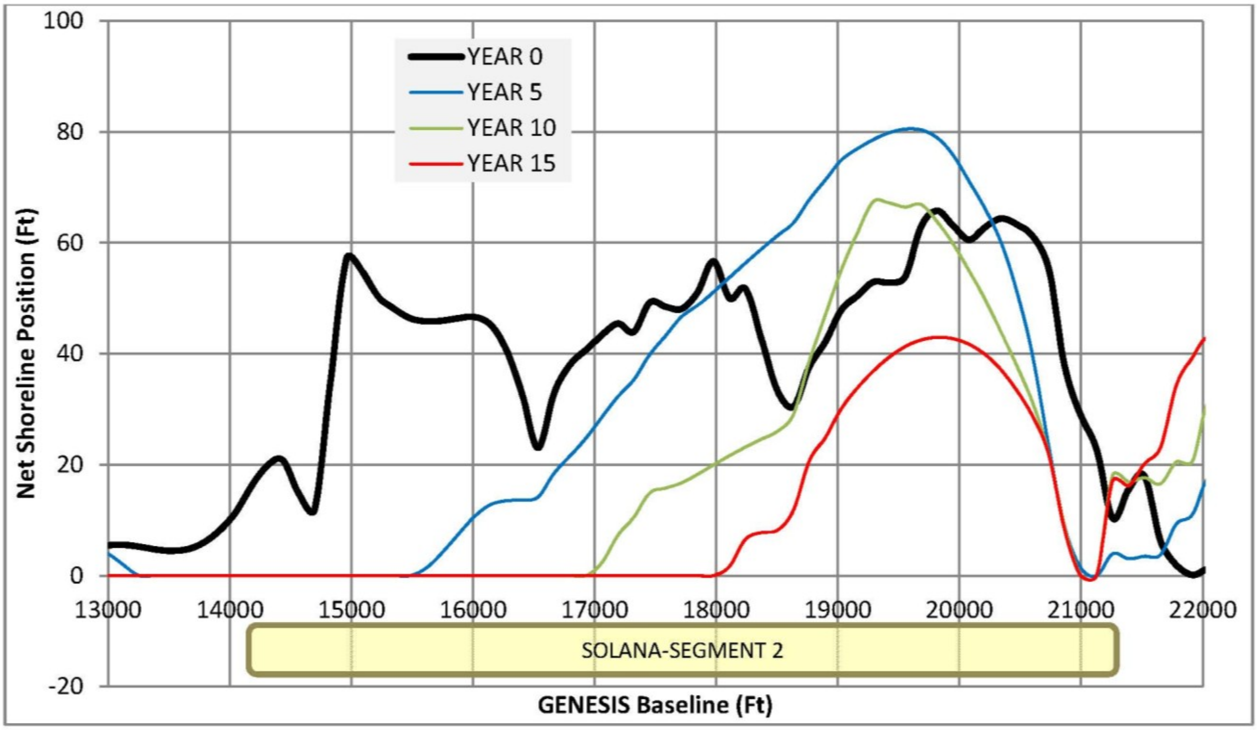


Figure 7.7-2 Shoreline Evolution for 50 Foot Beach Nourishment in Solana - Segment 2

7.8 Beach Widths and Volumes for Economic Analysis

The economic optimization of the various beach nourishment options and replenishment intervals required conversion of the GENESIS predicted net shoreline changes into beach widths and beach nourishment volumes. The beach widths were used to determine how much benefit could be captured at any year and segment. The volumes were required to estimate the construction costs.

7.8.1 Beach Widths for Calculation of Benefits

Estimation of the beach widths required addition of the scenario-mean net shoreline change to the base-year beach width. The scenario-mean net shoreline changes are as reported above in **Section 0** of this report. The base-year beach width was calculated as follows.

Beach profile conditions that existed prior to the RBSPI were taken to represent the without Project condition. Profile conditions that existed between the period of 1997 to 2000, at the two data rich profiles, SD-670 and SD-600, were used to characterize the active littoral volume. SD-670 is representative of Encinitas-Segment 1 and SD-600 is representative of Solana-Segment 2. The without Project active profile volumes were 100 yd^3/ft for Encinitas-Segment 1 and 75 yd^3/ft for Solana-Segment 2, respectively. Extended over the alongshore extent of each segment indicates a without Project active sand volume of about 800,000 yd^3 for Encinitas-Segment 1 and 600,000 yd^3 for Solana-Segment 2.

From **Section 4.3.3**, it can be seen that RBSPI added approximately 237,000 yd^3 in the general vicinity of Segment I in the fall of 2001: 132,000 yd^3 at Leucadia and 105,000 yd^3 at Moonlight Beach. The measured profile response at SD-670 displayed an increase in the active profile volume of 25 yd^3/ft as a result of this fill. The active profile volume at SD-670 over the eight years between 2002 and 2010 decreased from about 200 to 140 yd^3/ft, a loss of 60 yd^3/ft and loss rate of 7.5 yd^3/ft-year.

The lowest historic active profile volume was about 100 cy/ft (1987). This denuded conditions persisted through the 1997-1998 El Nino years when bluff retreat was serious. After 1998 but prior to the RBSP I beach fill, the profile volume increased to about 230 cy/ft and is presumed to be the affect of the Batiquitos beach fill placed in the City of Carlsbad to the north. The RBSP I beach will was about 175 cy/ft. The RBSP I beach fill occurred in the Fall 2001 when the beach fill volume increased to about 230 cy/ft, and by fall 2009 was about 140 cy/ft. An approximate linear fit from 200 to 140 cy/ft between 2002 and 2010 formed the basis for the loss rate in Segment 1. The time series plot of MSL position and active profile volume for SD 670 are show in Figure 7.8-1**.**

From **Section 4.3.3**, it can be seen that RBSPI added approximately 146,000 yd^3 at Fletcher Cove in Solana Beach. The measured profile response at SD-600 also displayed an increase in the active profile volume of 25 yd^3/ft as a result of this fill. The active profile volume at SD-600 over the eight years between 2002 and 2010 decreased from about 85 to 65 yd^3/ft, a loss of 20 yd^3/ft and loss rate of 2.5 yd^3/ft-year.

The RBSPII is expected to be constructed in 2012 and is projected to add 222,000 yd^3 to Encinitas-Segment 1 and 146,000 yd^3 to Solana-Segment 2 (**Section 4.3.3**). Scaling from the measured performance of the RBSPI the affects of the RBSPII on the active profile sand volume in the Project base-year were estimated. This resulted in 9,000 yd^3 of the RBSPII nourishment remaining in the active profile volume during the base year in Encinitas-Segment 1 and 102,200 yd^3 remaining in Solana-Segment 2. These volumes were converted to widths as needed using the previously discussed v/s ratios from **Chapter 8**.

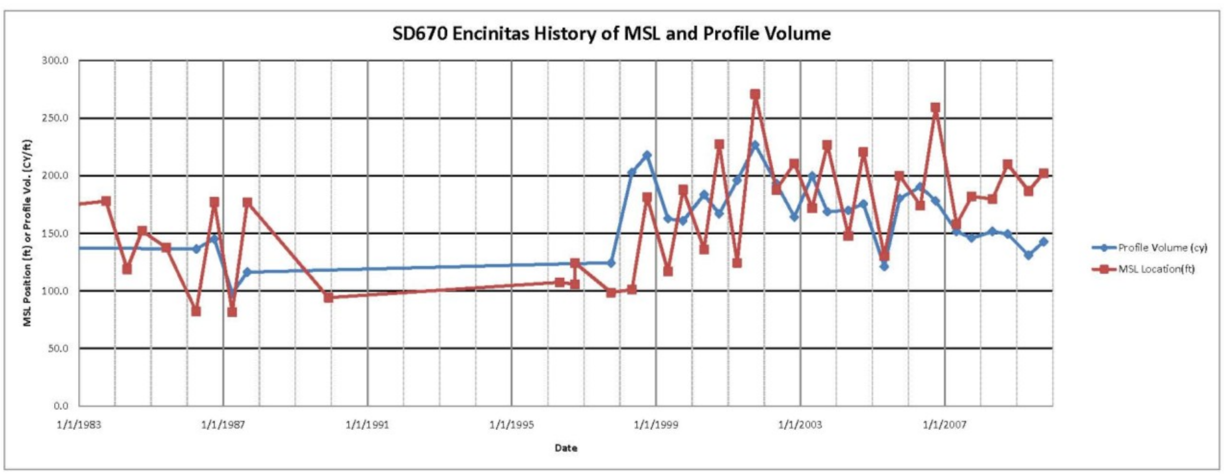


Figure 7.8-1 SD670 Encinitas History of MSL and Profile Volume

7.8.2 Beach Nourishment and Replenishment Volumes

Discreet construction volumes were calculated by segment for every combination of sea level rise scenario (i.e., low, intermediate, and high), replenishment interval (i.e., 2, 3, 4,…16 year), Project year (i.e., 2015, 2016, 2017,…2065), and beach nourishment option (i.e., 50, 100,…400 foot). This resulted in a volume lookup table with over 10,000 values listed for use by the economics optimization. Parameters used in these calculations have already been described. The following provides more detail and describes how they were incorporated into the volume calculations.

Replenishment intervals ranged from 2 to 16 years based on the assumption that annual replenishment would be too frequent and expensive. Once again, v/s ratios from **Chapter 8** were used to convert from shoreline changes to profile volumes. In this case, scenario-mean net shoreline changes from Section **Error! Reference source not found.** of this report were converted to beach nourishment volumes.

The sea level rise quantities were based on the sea level rise scenarios described in **Section 3.2.3** of this report. In year 0, the first sand placement would include the beach nourishment option plus a sea level rise quantity. This sea level rise quantity would be placed before the expected loss from sea level rise, offsetting that loss so that the shoreline modeling performed above would remain valid during the following time period. In other words, the GENESIS shoreline modeling that was performed assuming no shoreline loss from sea level rise would still be valid since any sea level rise shoreline loss would be offset with a pre-filled sea level rise quantity. Subsequent replenishments would include whatever beach replenishment volumes would be required to achieve the original beach nourishment option width plus a pre-filled sea level rise quantity. The volumes were calculated assuming that no replenishment would occur in the final year, year 2065.

Sea level rise quantities were calculated according to the Bruun Rule (Bruun, 1962; USACE, 2002) as shown in **Figure 7.8-2** and the following equation:

$$R=SL/(B+H_*) \qquad \text{(Equation 7-1)}$$

Where R is shoreline retreat, S is increase in sea level, L is cross shore distance to water depth H*, B is berm height of eroded area, and H* is closure depth. This can be interpreted to mean that as the water level rises, the shoreline recedes proportionately. It has been shown (Hands, 1983) that to maintain no shoreline recession (R=0), a volume V must be added according to the following equation:

$$V=LSZ \qquad \text{(Equation 7-2)}$$

Where Z is alongshore distance.

Table 7.8-1 shows the cumulative sea level rise quantities required to offset volume loss resulting from application of the Bruun Rule to each of the sea level rise scenarios for every Project year for both segments. Between the Project base-year and year 2016, an estimated 12,134 yd^3 of sand would be required to offset the high sea level rise scenario in Encinitas-Segment 1. By year 2065, if the high sea level rise scenario occurs, over 1.8 million yd^3 of sand would be required to offset sea level rise for both segments. The low sea level rise scenario has a constant rate sea level rise, hence a constant addition of beach fill is assumed to counteract that rise. For the low sea level rise condition, the re-nourishment rate consists of two constant parts: one for the sediment loss and one for the sea level rise.

1 **Table 7.8-1 Cumulative Sea Level Rise Quantities**

Scenario	High	Intermediate	Low	High	Intermediate	Low
Segment	Encinitas-Segment 1			Solana-Segment 2		
Year	Cumulative Sea Level Rise Quantity (yd^3)					
2015	0	0	0	0	0	0
2016	12,134	5,252	3,140	11,215	4,854	2,902
2017	24,573	10,575	6,280	22,712	9,774	5,804
2018	37,317	15,971	9,420	34,490	14,761	8,706
2019	50,365	21,437	12,559	46,550	19,813	11,608
2020	63,719	26,976	15,699	58,892	24,932	14,510
2021	77,377	32,586	18,839	71,516	30,117	17,412
2022	91,341	38,267	21,979	84,422	35,368	20,314
2023	105,609	44,020	25,119	97,609	40,686	23,216
2024	120,182	49,845	28,259	111,078	46,069	26,118
2025	135,060	55,741	31,399	124,829	51,519	29,020
2026	150,243	61,709	34,539	138,862	57,035	31,922
2027	165,731	67,748	37,678	153,177	62,617	34,824
2028	181,523	73,860	40,818	167,773	68,265	37,726
2029	197,621	80,042	43,958	182,651	73,979	40,628
2030	214,023	86,296	47,098	197,811	79,760	43,530
2031	230,731	92,622	50,238	213,253	85,606	46,432
2032	247,743	99,020	53,378	228,976	91,519	49,335
2033	265,060	105,489	56,518	244,982	97,498	52,237
2034	282,682	112,030	59,658	261,269	103,543	55,139
2035	300,609	118,642	62,797	277,838	109,655	58,041
2036	318,841	125,326	65,937	294,688	115,832	60,943
2037	337,377	132,081	69,077	311,821	122,076	63,845
2038	356,219	138,908	72,217	329,235	128,386	66,747
2039	375,365	145,807	75,357	346,931	134,762	69,649
2040	394,816	152,777	78,497	364,909	141,204	72,551
2041	414,573	159,819	81,637	383,169	147,712	75,453
2042	434,634	166,932	84,777	401,710	154,287	78,355
2043	455,000	174,117	87,916	420,534	160,928	81,257
2044	475,670	181,374	91,056	439,639	167,635	84,159
2045	496,646	188,702	94,196	459,025	174,408	87,061
2046	517,927	196,102	97,336	478,694	181,247	89,963
2047	539,512	203,573	100,476	498,644	188,152	92,865
2048	561,403	211,116	103,616	518,877	195,124	95,767
2049	583,598	218,731	106,756	539,391	202,162	98,669
2050	606,098	226,417	109,896	560,186	209,266	101,571
2051	628,903	234,175	113,035	581,264	216,436	104,473
2052	652,013	242,004	116,175	602,623	223,672	107,375
2053	675,428	249,905	119,315	624,264	230,975	110,277
2054	699,148	257,877	122,455	646,187	238,343	113,179

Scenario	High	Intermediate	Low	High	Intermediate	Low
2055	723,172	265,922	125,595	668,392	245,778	116,081
2056	747,502	274,037	128,735	690,879	253,279	118,983
2057	772,136	282,225	131,875	713,647	260,846	121,885
2058	797,075	290,484	135,015	736,697	268,480	124,787
2059	822,319	298,814	138,154	760,029	276,179	127,689
2060	847,868	307,216	141,294	783,643	283,945	130,591
2061	873,722	315,690	144,434	807,538	291,776	133,493
2062	899,881	324,235	147,574	831,715	299,674	136,395
2063	926,345	332,852	150,714	856,174	307,639	139,297
2064	953,113	341,541	153,854	880,915	315,669	142,199
2065	980,187	350,301	156,994	905,938	323,766	145,101

One example of the over 10,000 volume calculations is provided here assuming a 50 foot beach nourishment option with a 5 year replenishment interval with a high sea level rise scenario at Encinitas-Segment 1.

The volume of beach sand is calculated as the 50 foot beach nourishment option width times the Encinitas-Segment 1 v/s ratio (i.e., 0.864 yd^3/ft^2) times the segment length (i.e., 7802 feet) yielding 337,046 yd^3. From **Table 7.8-1** it can be seen that in Encinitas-Segment 1, under the high sea level rise scenario at year 2020, the sea level rise quantity is 63,719 yd^3. Adding these yields a construction volume of 400,765 yd^3.

The first replenishment occurs at year 2020. From **Table 7.7-2**, at year 2020, the Encinitas-Segment 1 shoreline would have eroded from 50 feet to 19.7 feet. To restore the shoreline to the original beach nourishment option width requires the addition of 204,293 yd^3 of replenishment sand [i.e., (50 ft -19.7 ft) x 7802 ft x 0.864 yd^3/ft^2]. The next sea level rise quantity is the volume expected to be lost over the next five years from 2020 to 2025 (i.e., 135,060 yd^3 – 63,719 yd^3 = 71,341yd^3). Adding the replenishment volume to the sea level rise quantity yields a total replenishment volume of 275,634 yd^3. These calculations were carried out for the remaining Project years.

7.8.3 Overfill Factor

An overfill factor was applied to the above calculated beach nourishment volumes and sea level rise quantities, increasing these volumes to account for the loss of fine sediment during and immediately after construction. The volumes analyzed within the economic optimization utilized the larger volumes as modified by an overfill factor.

The sand borrow source is expected to be from the near shore areas in the vicinity of SO-5 and SO-6 for initial construction, and possibly off of Mission Bay or Oceanside for future replenishment. An overfill factor is the ratio of the volume removed from the borrow site and the volume added to the active or equilibrium beach profile. This overfill factor is dependent on the geotechnical properties of both the borrow site and receiving beaches. Principal factors are bulk densities and grain size distribution, and to some extent the method of construction. For this study, an overfill factor of 1.20 was applied based on the long term experience of the recurring beach nourishment project at Surfside-Sunset Beach in southern California's Orange County (USACE-LAD, 2002b) where 30 years of beach fills and monitoring showed the nourished profile volume to be approximately 80 percent of the borrow site volume. The material is presumed to be lost offshore during construction. Construction fill volumes can be updated during Project design based on detailed surveys of the segments and detailed geotechnical evaluation of the borrow sites.

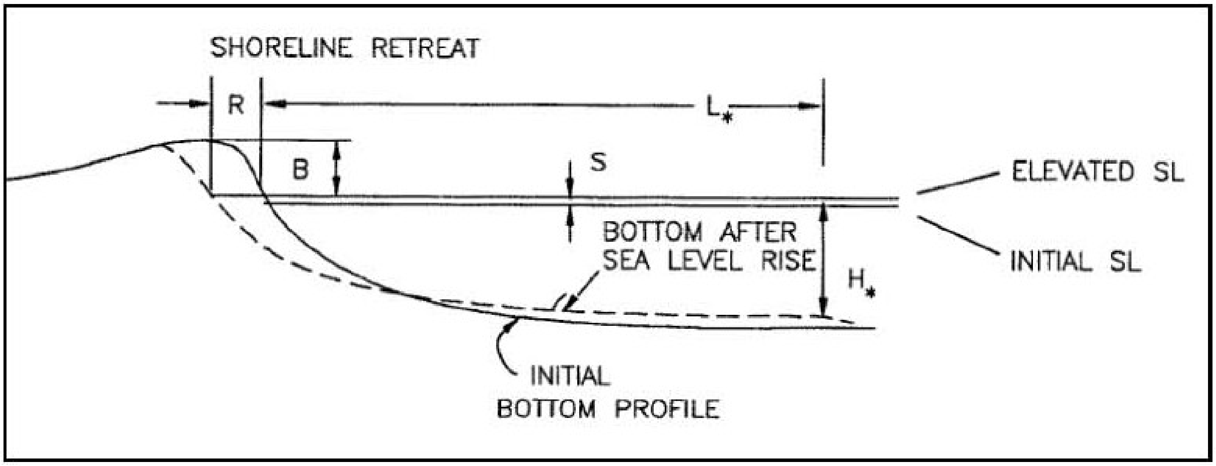


Figure 7.8-2 Shoreline Responses to Sea Level Rise per the Bruun Rule (USACE, 2002)

8 PROFILE ANALYSIS

This chapter documents the cross shore profile analysis used to support the Project. This profile analysis method was applied to the shoreline morphology results to develop intermediate values used in the habitat impact analysis, surfing impact analysis, and lagoon sedimentation analysis. A critical assumption for this analysis is that the distribution of sand levels in the cross-shore dimension will behave as they have historically as shown in the profile behavior from long-term monitoring records.

8.1 Available Profile Data

Surveyed profile data has been collected by various entities covering San Diego County dating back to 1983 as described in **Chapter 4** of this report. The locations of the profiles used in this profile analysis are shown in **Figure 8.1-1**. The profile surveys started at fixed origins extending offshore along a set alignment to past the depth of closure. Elevations were given in feet relative to the MLLW vertical datum based on either the tidal epoch ending in 1978 or the tidal epoch ending in 2001. All data was corrected to the 2001 tidal epoch, in feet, MLLW before further calculations were carried out. The abundance of data is exemplified in **Figure 8.1-2**, which shows all the profiles collected at Fletcher Cove (profile SD-600) up to the time the profile analysis was carried out. In this figure, the horizontal axis is the range from the profile origin. Profile data from before 1996 were provided by the Los Angeles District of the USACE. Profiles from 1996 onward were collected by the Coastal Frontiers Corporation and provided with permission from SANDAG (Coastal Frontiers Corporation, 2010). This profile analysis includes data from 1983 through the fall of 2008, as detailed in **Table 8.1-1**.

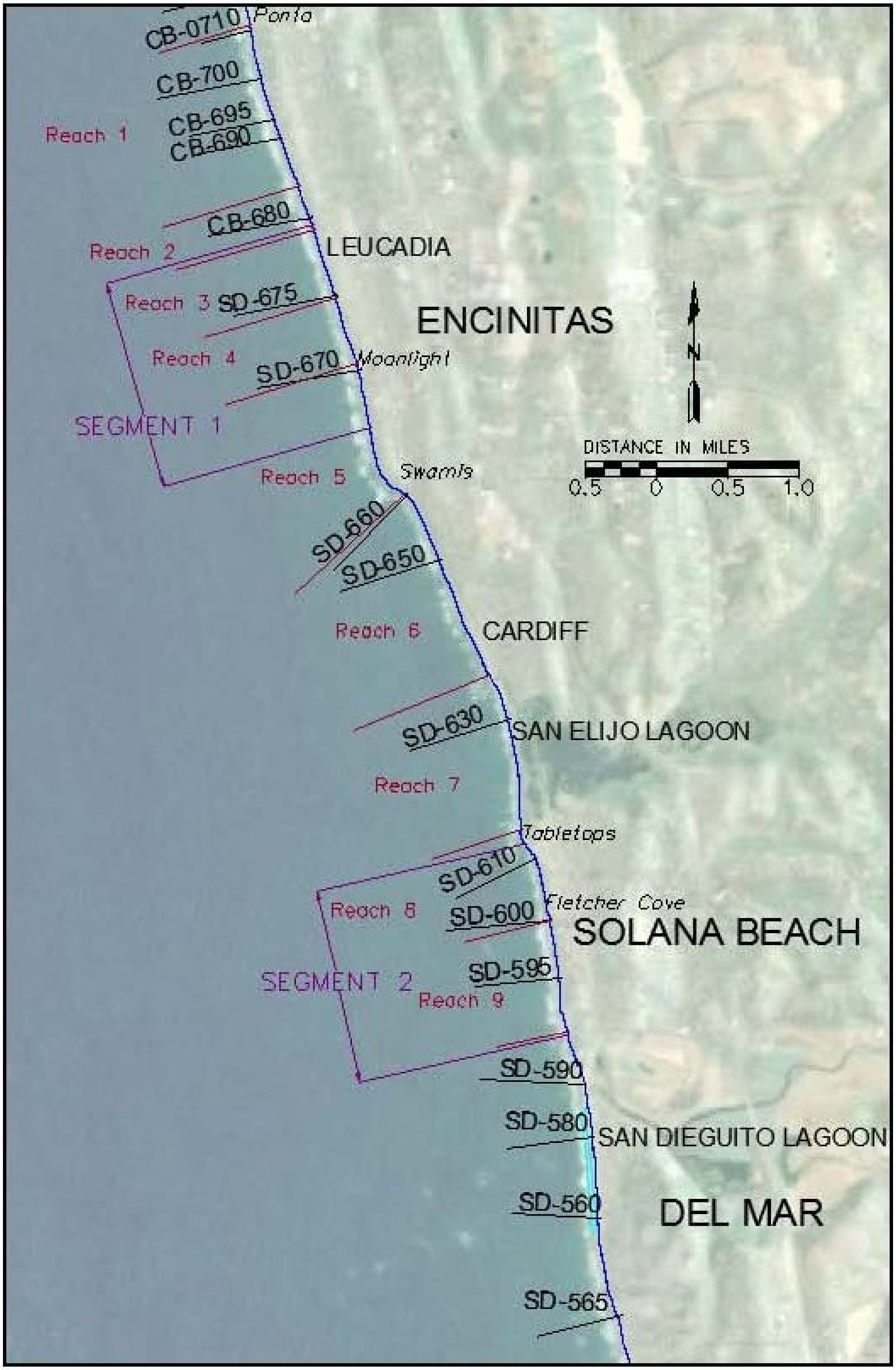


Figure 8.1-1 Profile Used in Profile Analysis

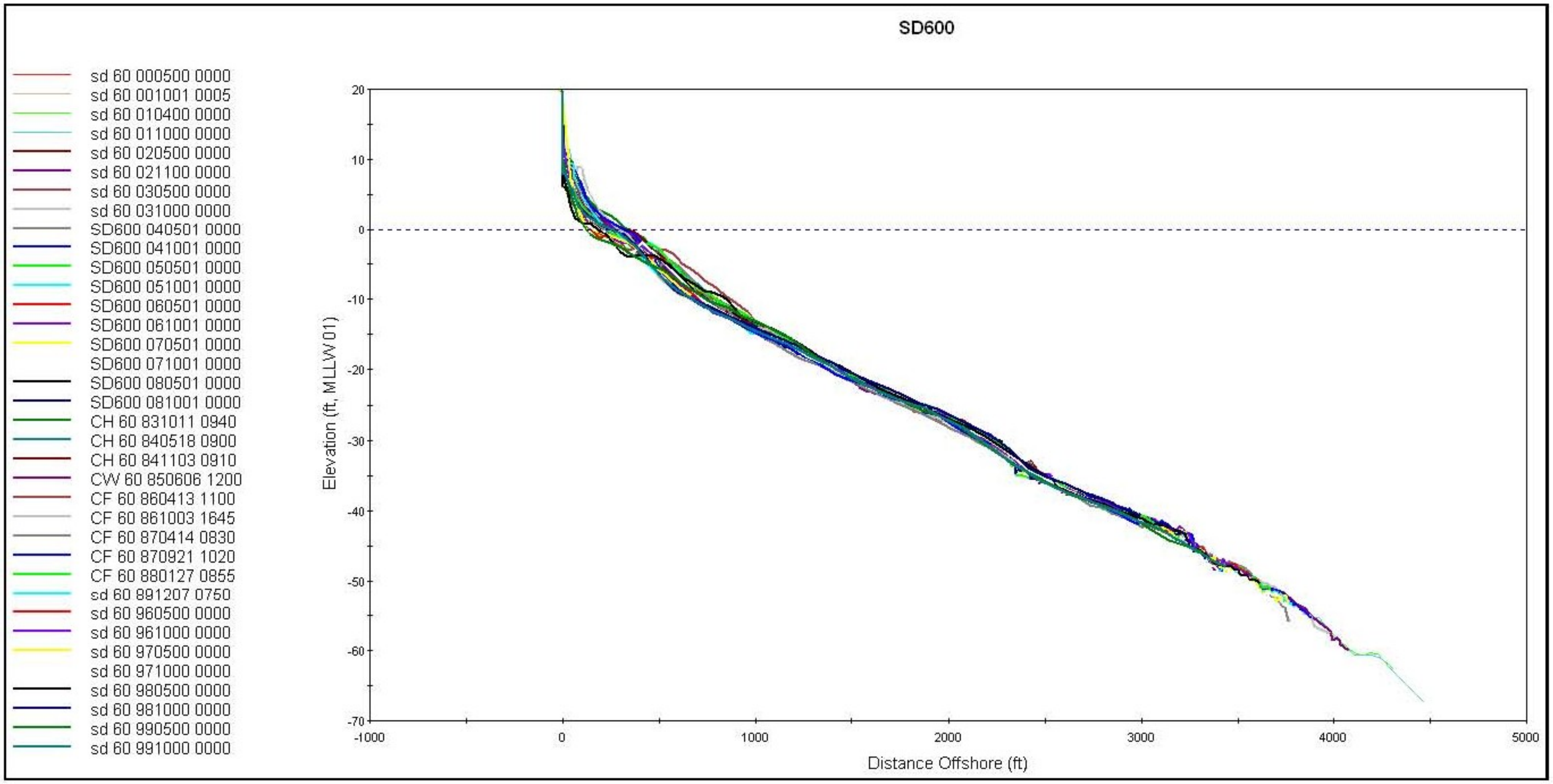


Figure 8.1-2 All Available Profiles at SD0600

Table 8.1-1 Profile Data Used

Profile	DM590	SD600	SD620	SD625	SD630	SD650	SD660	SD670	SD675	SD680	SD690	SD695	SD700
M-YY													
O-83	nd	u	nd	nd	nd	nd	nd	u	nd	nd	nd	nd	nd
M-84	nd	u	nd	nd	nd	nd	nd	u	nd	nd	nd	nd	nd
N-84	u	u	nd	nd	nd	nd	nd	u	nd	nd	nd	nd	nd
J-85	u	u	nd	nd	nd	nd	nd	u	nd	nd	nd	nd	nd
A-86	u	u	nd	nd	nd	nd	nd	u	nd	nd	nd	nd	nd
O-86	u	u	nd	nd	nd	nd	nd	u	nd	nd	nd	nd	nd
A-87	u	u	nd	nd	nd	nd	nd	u	nd	nd	nd	nd	nd
S-87	u	u	nd	nd	nd	nd	nd	u	nd	nd	nd	nd	nd
J-88	nd	u	nd	nd	nd	nd	nd	nd	nd	nd	nd	nd	nd
D-89	u	u	nd	nd	nd	nd	nd	nd	nd	nd	nd	nd	nd
M-96	nd	u	nd	nd	u	nd	nd	u	nd	nd	nd	nd	nd
O-96	nd	u	nd	nd	u	nd	nd	u	nd	nd	nd	nd	nd
M-97	u	u	nd	nd	u	nd	nd	u	nd	nd	nd	nd	nd
O-97	u	u	nd	nd	u	nd	nd	u	nd	nd	nd	nd	nd
M-98	u	u	nd	nd	u	nd	nd	u	nd	nd	nd	nd	nd
O-98	nd	u	nd	nd	u	nd	nd	u	nd	nd	nd	nd	nd
M-99	u	u	nd	nd	u	nd	nd	u	nd	u	nd	nd	nd
O-99	u	u	nd	nd	u	nd	nd	u	nd	u	nd	nd	nd
M-00	u	u	nd	nd	u	nd	nd	u	nd	u	nd	nd	nd
O-00	u	u	u	u	u	u	u	u	nd	u	nd	nd	u
A-01	u	u	u	u	u	u	u	u	u	u	u	u	u
O-01	u	u	u	u	u	u	u	u	u	u	u	u	u
M-02	u	u	u	u	u	u	u	u	u	u	u	u	u
N-02	u	u	u	u	u	u	u	u	u	u	u	u	u
M-03	u	u	u	u	u	u	u	u	u	u	u	u	u
O-03	u	u	u	u	u	u	u	u	u	u	u	u	u
M-04	u	u	u	u	u	u	u	u	u	u	u	u	u
O-04	u	u	u	u	u	u	u	u	u	u	u	u	u
M-05	u	u	u	u	u	u	u	u	u	u	u	u	u
O-05	u	u	u	u	u	u	u	u	u	u	u	u	u
M-06	u	u	u	u	u	u	u	u	u	u	u	u	u
O-06	u	u	u	u	u	u	u	u	u	u	u	u	u
M-07	u	u	u	u	u	u	u	u	u	u	u	u	u
O-07	u	u	u	u	u	u	u	u	u	u	u	u	u
M-08	u	u	u	u	u	u	u	u	u	u	u	u	u
O-08	u	u	u	u	u	u	u	u	u	u	u	u	u

u= data used in profile analysis, nd=no data available

8.2 Method Overview

This profile analysis was used to convert shoreline morphology results into cross shore sand thickness distributions for spring and fall of each year. All the variables used in the profile analysis are summarized in **Table 8.1-1**. The method is generally described below:

1. A change in sand volume at a given profile location was calculated by multiplying the net change in shoreline position by a v/s ratio. The v/s ratio was defined as the relationship between MSL beach width and profile sand volume per alongshore unit-width as described below.
    a. For the habitat impact analysis, the net changes in shoreline position were averaged over the longshore range extending one-half the distance between profiles.
    b. For the lagoon sedimentation analysis, the net changes in shoreline position were averaged over the longshore range extending one-half the distance between profiles.
    c. For the surfing impact analysis, the changes in shoreline position were extracted from the shoreline modeling cell or cells closest to each surfing site.
2. The change in sand volume was then multiplied by a dimensionless sand distribution (described below) resulting in a cross shore sand thicknesses at each 33 foot increment along the profile.
3. For the habitat impact analysis, the sand thicknesses were then added to an assumed static baseline (without Project) bathymetry to estimate the Project induced changes in sand thickness at spring and fall of each year. The assumed baseline bathymetry was based on the LiDAR survey of April 2004.

Table 8.2-1 Profile Analysis Variables

Variable	Description
Average Range of Closure	Range of closure is distance from profile origin where the depth of closure occurs, as calculated for each profile by Coastal Frontiers Corporation (2010). The average of all ranges of closures for the profiles used in the profile analysis is 1607 feet from the profile origins.
Dimensionless sand distribution	Measured sand thicknesses at each 33 foot increment divided by the sum of all the sand thicknesses out to the average range of closure. There are two dimensionless sand distributions for each profile location, one for spring and one for fall.
Hardpan	A profile consisting of the composite of all surveyed minimum elevations along that profile location extending out to the average depth of closure. This hardpan is not an observed feature, but is instead a composite of the lowest elevations of many profiles. There is one hardpan for each profile location.
Measured sand thickness	Vertical distances, at each 33 foot increment along the profile, between the average spring or average fall and the hardpan profiles. There are two measured sand thickness sets for each profile location, one for spring and one for fall.
Post-RBSPI Profile	Any surveyed profile occurring after construction of RBSPI, after spring 2001.
Static Baseline	2004 LiDAR surveyed bathymetry offshore of the study area.
v/s	Volume of sand in the profile per square foot of beach area (yd^3/ft^2). There are two v/s ratios used in for this Project, one for Encinitas-Segment 1 and one for Solana-Segment 2.

8.3 Dimensionless Sand Distribution

A dimensionless sand distribution was calculated for each profile location based on measured profile data. A measured sand distribution for each of the two seasons (i.e., spring and fall) was calculated as the difference between the average of the profiles occurring since implementation of RBSPI (post-RBSPI) for each season minus the hardpan. The variables and their definitions are summarized in **Table 8.2-1**. These calculations of the measured sand distributions were performed within the BMAP computer program, which is part of the CEDAS package developed by the Corps (Veri-Tech, 2011).

Post-RBSPI profiles were used since they best represent (of the data available) the Project conditions, expected after each replenishment interval. In contrast, the Pre-RBSPI profiles represent a more sand-starved condition, which would be less representative of the nourished beach profiles.

The hardpan is not an observed feature, but is instead a composite of the lowest elevations of many profiles representing an assumed feature. The hardpan substrate underlying the beach sand is comparatively non erosive and the elevation of the hardpan fronting the bluffs is assumed to remain constant over the 50 year Project evaluation period. The average range of closure for the profiles considered in this analysis was found to occur approximately 1600 feet from profile origins.

The vertical differences, at each 33 foot increment along the profile, between the average spring or average fall and the hardpan profiles represents the measured sand thicknesses.

The dimensionless sand distribution is the average measured sand thicknesses at each 33 foot increment divided by the sum of all the sand thicknesses out to the average range of closure.

The following is an example of the calculation method for the dimensionless sand distribution for profile SD-600 and an example intermediate result of the sand thickness estimate for that profile. Similar methods were used at the other profile locations in the study area.

1. All the profiles for location SD-600 are shown in **Figure 8.1-2**. The minimum elevation from all profiles was recorded into the hardpan.
2. All the post–RBSPI fall profiles and the hardpan at location SD-600 out to the average range of closure are shown in **Figure 8.3-1**. The average of all the fall post-RBSPI profiles is shown in **Figure 8.3-2** along with the average spring post-RBSPI profile, and the hardpan.
3. The differences between the average fall, post-RBSPI profile and the hardpan were calculated. This was also done for the average spring, post-RBSPI profile. These differences are also shown in **Figure 8.3-2** and labeled “Diff_PostFall” and “Diff_PostSpring.”
4. The sum of differences out to the average range of closure was calculated for both fall and spring conditions.

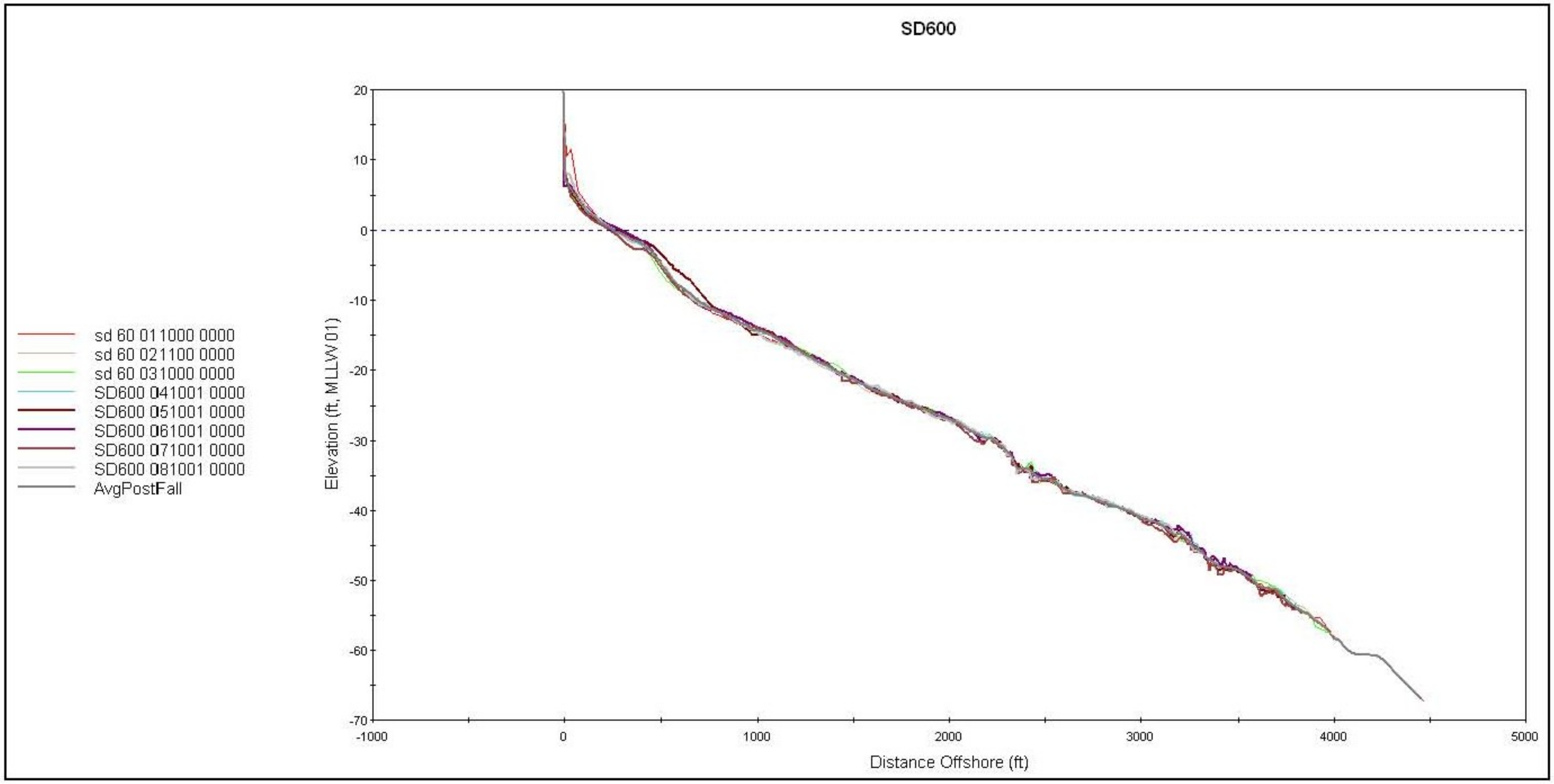


Figure 8.3-1 Fall Post-RBSPI Profiles at SD 600

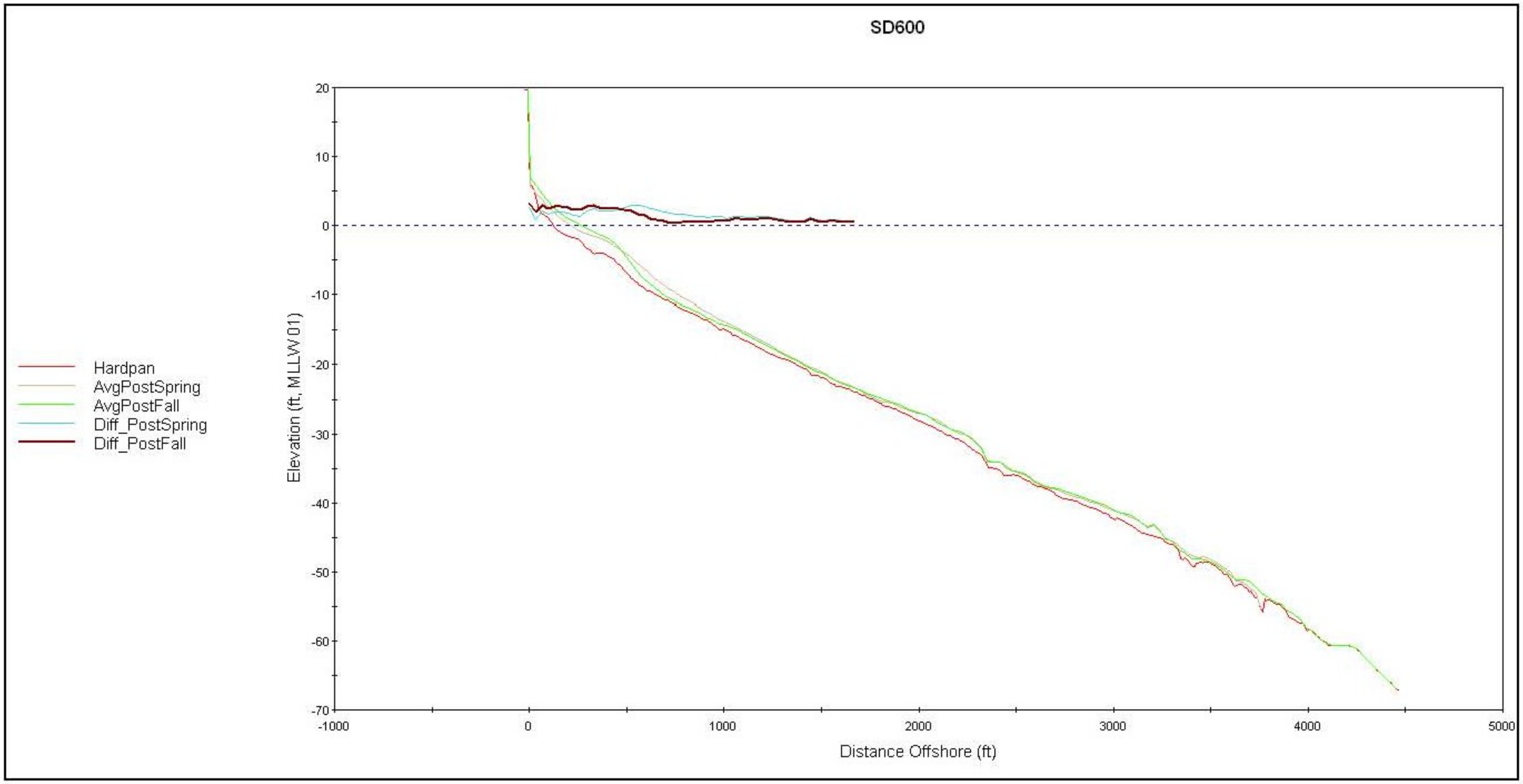


Figure 8.3-2 Average Fall Post-RBSPI Profile, Average Spring Post-RBSPI Profile, Hardpan, and Differences at SD 600

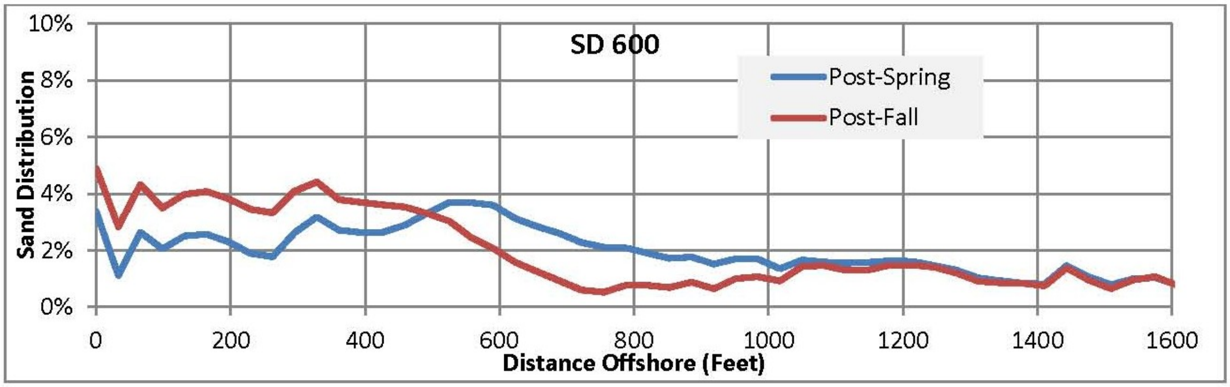


Figure 8.3-3 Spring and Fall Dimensionless Sand Distribution for Profile SD600

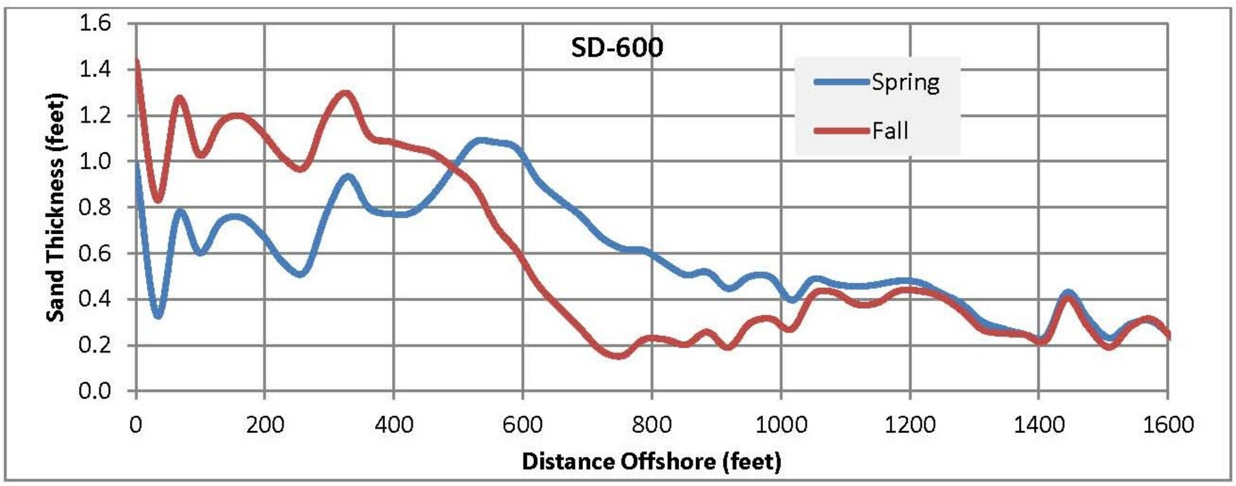


Figure 8.3-4 Example Cross Shore Sand Thickness Distribution for 50 Foot Shoreline Change

5. The difference at each 33 foot increment along the profile was normalized (e.g., divided by the sum of these differences) to find the percent difference at each increment location. The dimensionless sand distribution was composed of these percent differences. This was done for both fall and spring distributions as shown for profile SD-600 in **Figure 8.3-3** and for all the other profile locations in **Appendix B6**. As expected, the fall distribution typically has a greater percentage of material nearshore and the spring distribution has more material in the offshore bar.

6. An example result is provided for an assumed 50 foot scenario-mean net shoreline change for the beach near profile SD-600. This shoreline change multiplied by the v/s ratio for this segment (i.e., 0.713 yd^3/ft^2) yields a sand volume of 36 yd^3/ft alongshore. Distributing this volume in the cross shore using the dimensionless sand distribution calculated for this profile yields a cross shore sand thickness distribution as shown in **Figure 8.3-4**.

8.4 v/s Ratio

This task determined the v/s ratios used within this study. One v/s ratio was developed for the Encinitas-Segment 1 and one v/s ratio was developed for the Solana-Segment 2. Measured profile data were used to calculate these v/s ratios. It was assumed that one profile location within each segment was representative of that segment. The profiles within and extent of Encinitas-Segment 1 are shown in **Figure 8.4-1**. **Figure 8.4-2** shows the same for Solana-Segment 2. Profile SD-670 was chosen to represent Encinitas-Segment 1 since it has much more data (36 profiles over 26 years) than the other profiles within that segment (16 profiles over 8 years). Profile SD-600 was chosen to represent Solana-Segment 2 since it has much more data (36 profiles over 26 years) than the other two profile locations within that segment (14 profiles over 7 years).

All the measured data for profile SD-670 are shown in **Figure 8.4-3**. The profile elevations are given in feet, MLLW based on the 1983 to 2001 tidal epoch. The MSL elevation is shown with a blue line. A similar figure for profile SD-600 was provided earlier in **Figure 8.4-2**. The lowest elevation the sand achieved at the bluff toe was +1.7 feet, MLLW at both profile locations. This was assumed to be the hardpan elevation at the bluff toes. A hardpan was developed for each of the two profile locations. The hardpan profile for SD-670 is shown in **Figure 8.4-4** along with one example profile and the standard deviation of all the profiles for that location.

The profile volume is the cross sectional area between a given profile and the hardpan multiplied by one foot alongshore. This value was divided by 27 to convert from cubic feet to cubic yards. The area covers the entire profile from the bluff face to the range of closure for that profile location as determined by Coastal Frontiers Corporation (2010). The range of closure is the location at which the standard deviation of all the profile data is less than the assumed measurement error of 0.5 feet. The range of closure in **Figure 8.4-4** (SD-670) occurs at a distance of 1600 feet from the profile origin. A similar graph is shown for SD-600 in **Figure 8.4-5** with the range of closure being 1000 feet from the profile origin.

The shoreline position (ΔS) is the distance from the hardpan MSL shoreline position to that of a given profile. Examples are shown in **Figure 8.4-4** and **Figure 8.4-5**.

The profile volume and shoreline positions for all the spring and fall data were graphed in **Figure 8.4-6** and **Figure 8.4-7** for SD-670 and SD-600 respectively. The least-squares straight line fit of these data results in v/s ratio of 0.864 yd^3/ft^2 for SD-670 and 0.713 yd^3/ft^2 for SD-600. As the shoreline position approaches the hardpan and decreases, in these figures, profile volume also decreases, until a point is reached where there is no change in shoreline ($\Delta S=0$) and no profile volume. This relationship allows forcing of the y-intercept through the origin. These v/s ratios are similar to those previously developed (USACE-LAD, 1991).

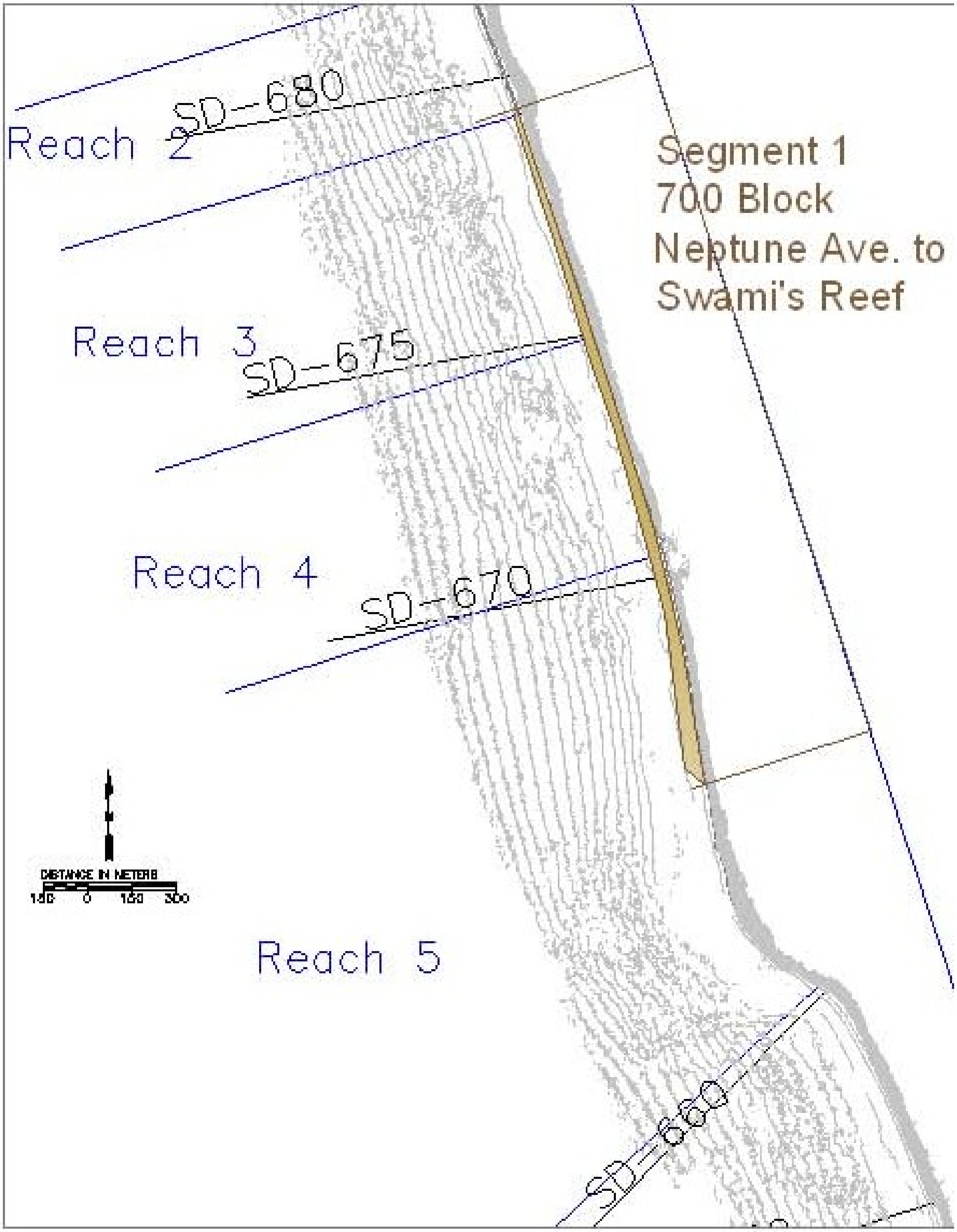


Figure 8.4-1 Profiles Within Encinitas - Segment 1

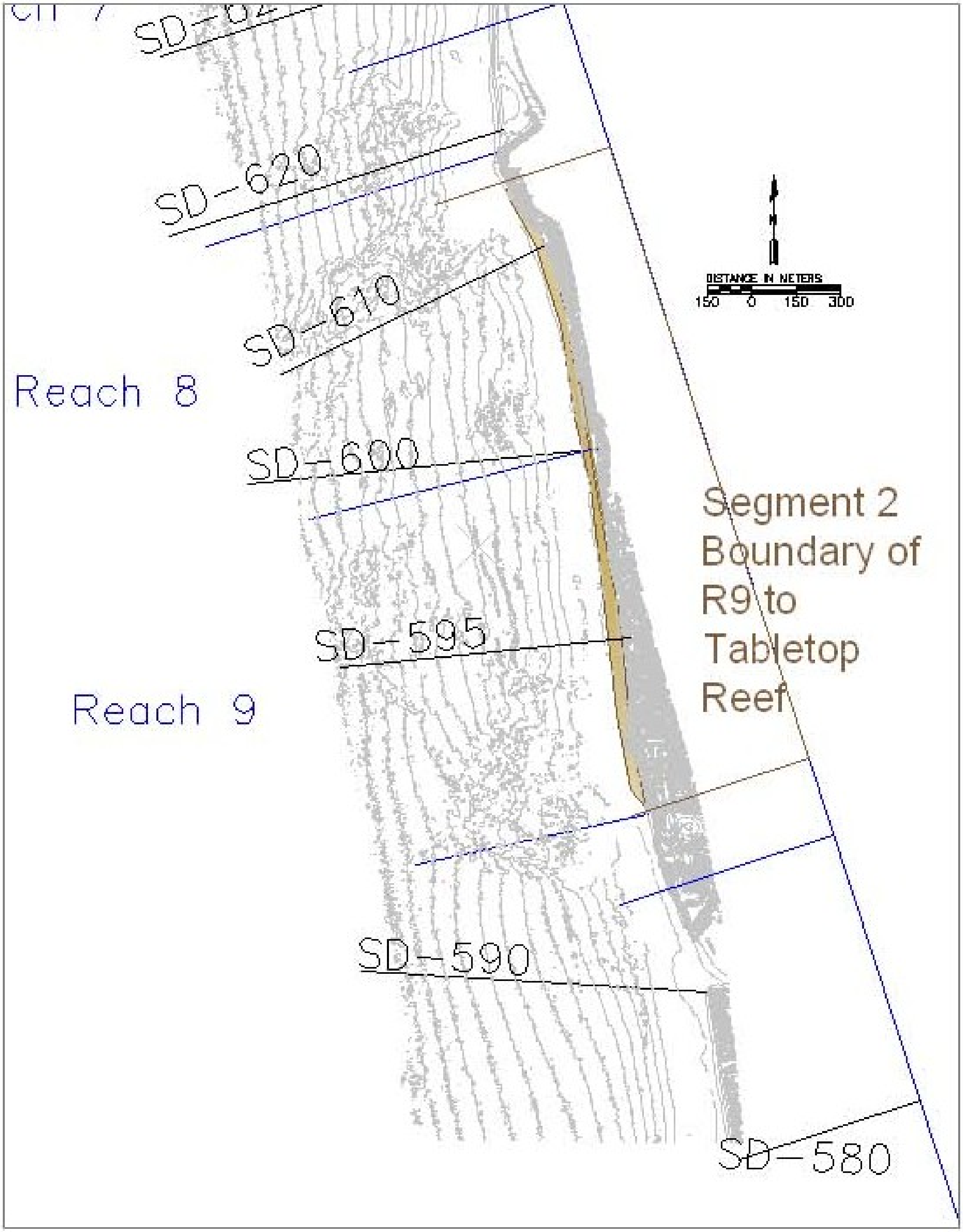


Figure 8.4-2 Profiles Within Solana - Segment 2

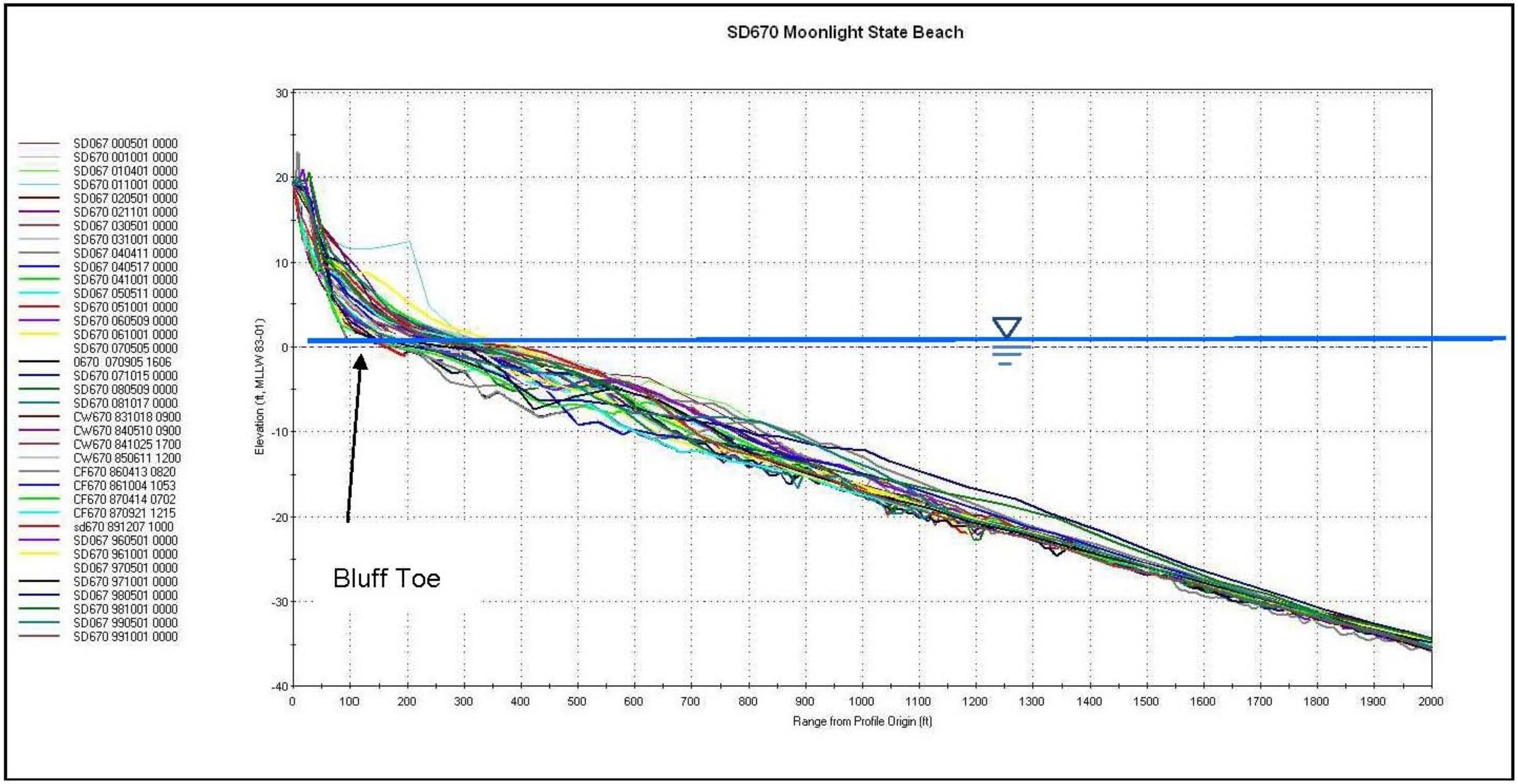
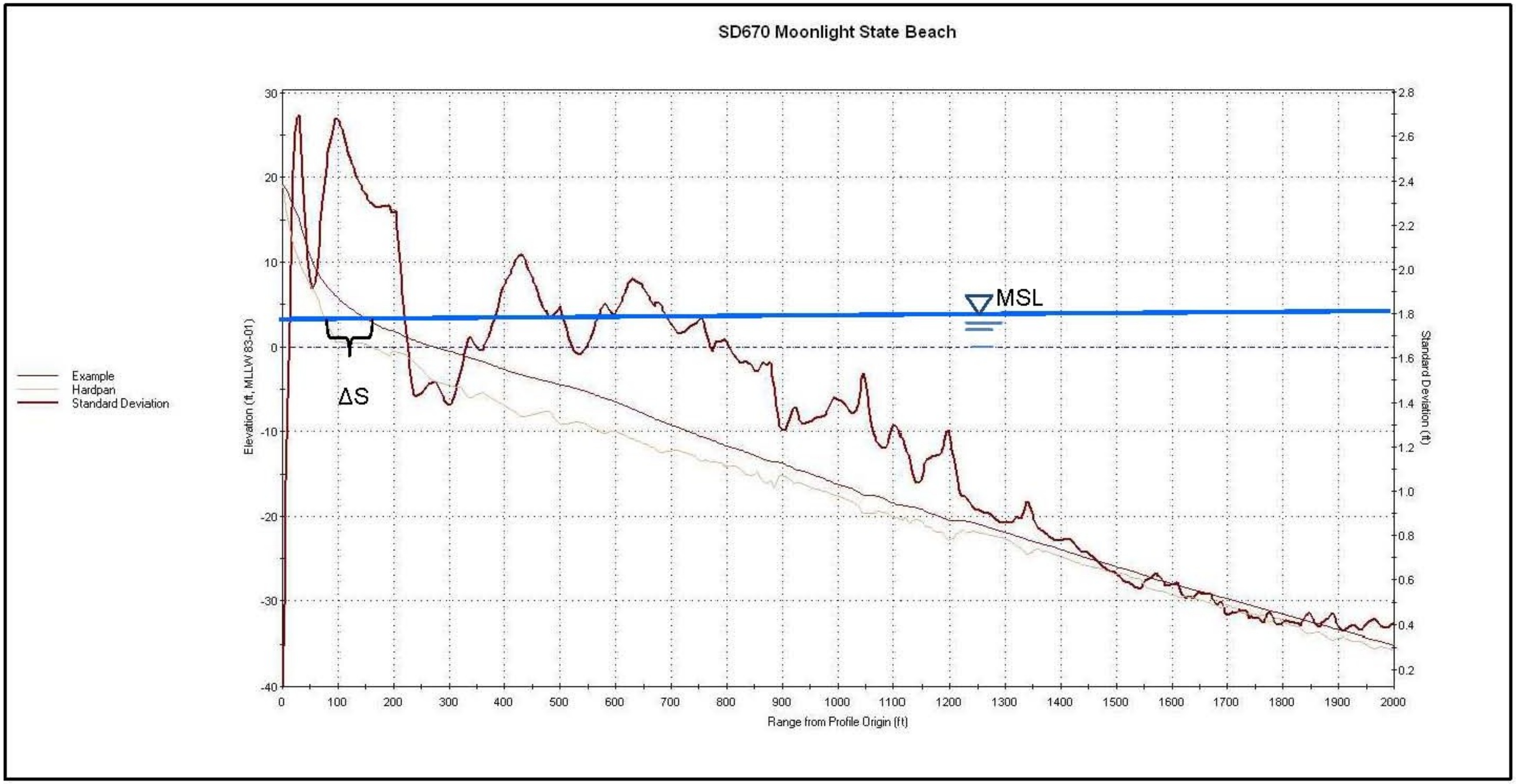


Figure 8.4-3 Profile Data for SD670



1
2 **Figure 8.4-4 SD670 Hardpan Profile, Example Profile, and Standard Deviation**

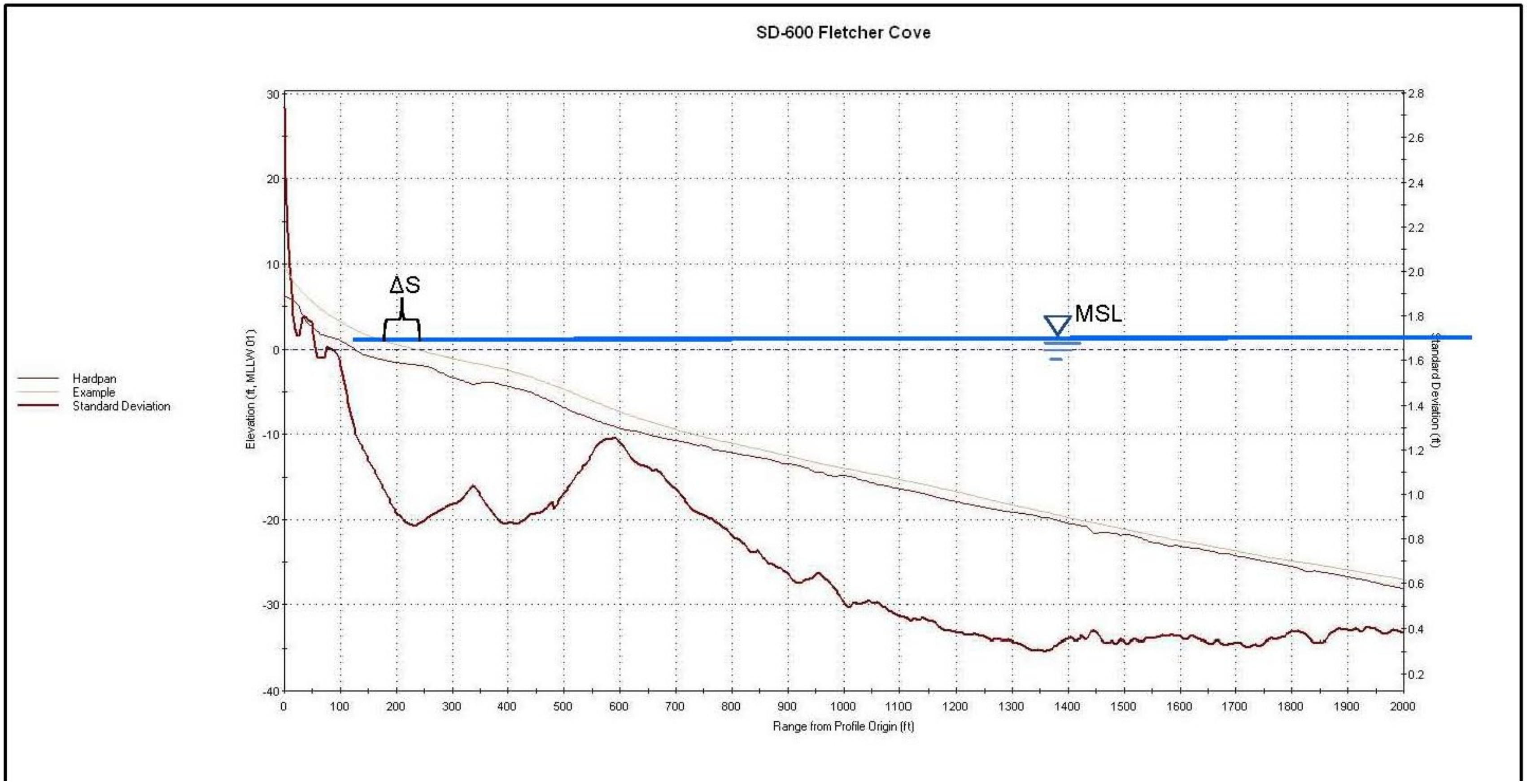


Figure 8.4-5 SD600 Hardpan Profile, Example Profile, and Standard Deviation

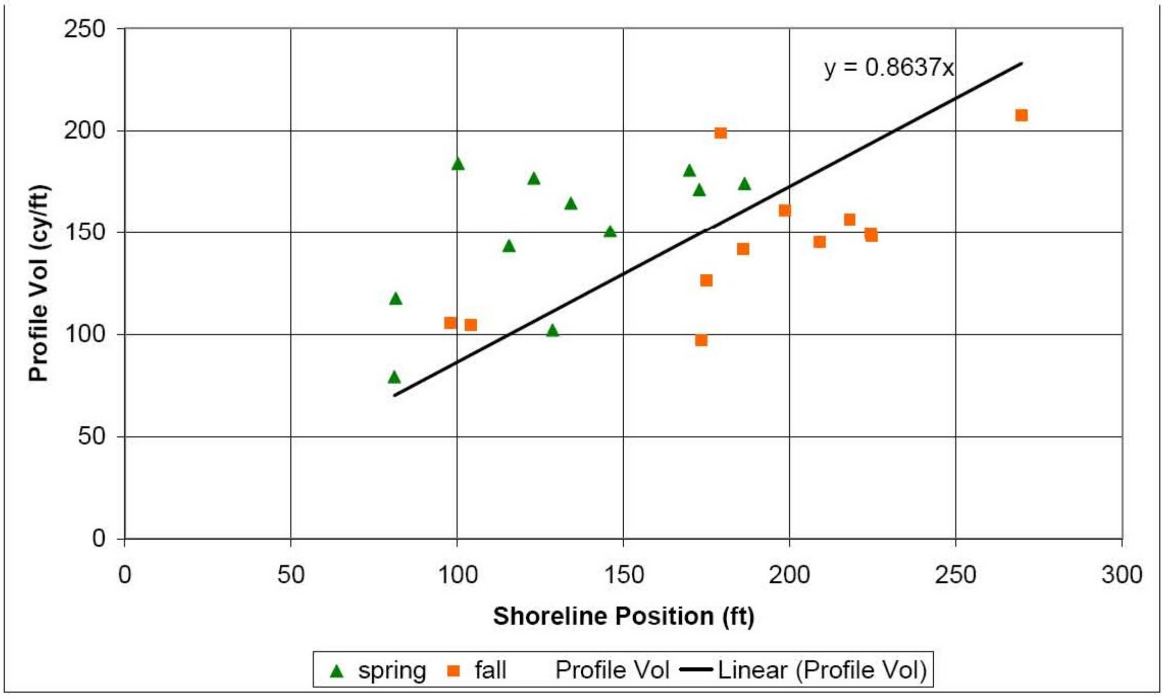


Figure 8.4-6 SD670 Change in Volume vs. Change in Shoreline Position

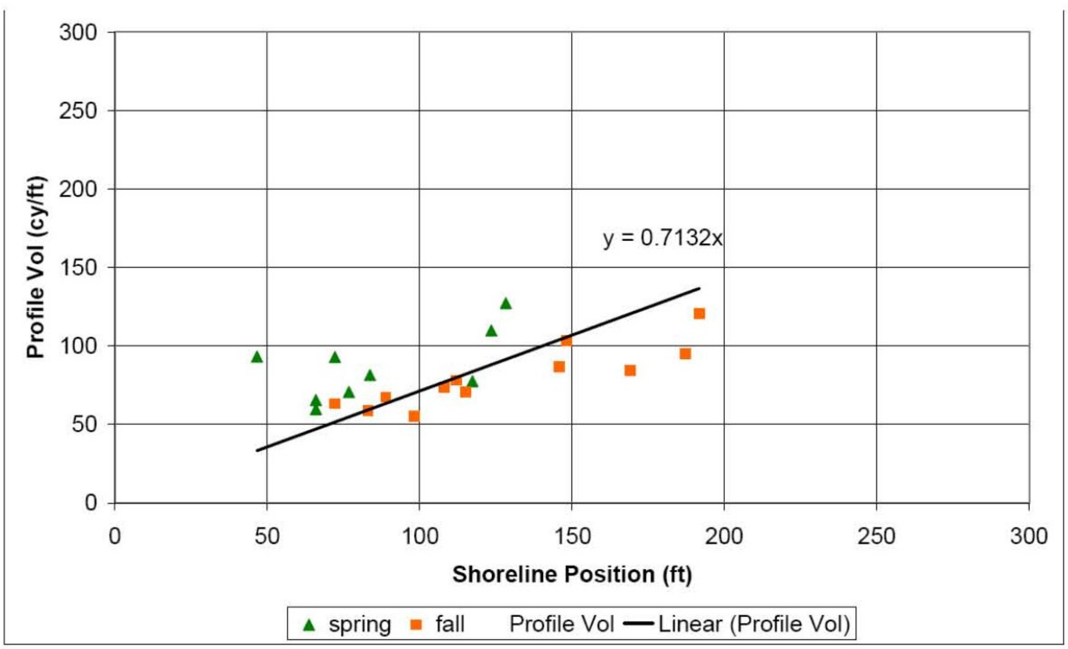


Figure 8.4-7 SD600 Change in Volume vs. Change in Shoreline Position

9 NEARSHORE HABITAT IMPACT ANALYSIS

Project induced impacts to nearshore habitats were estimated for inclusion in the environmental review document and for estimating mitigation costs. This section of this report outlines how shoreline morphology results were modified and made available for the habitat impact analysis. A detailed report of the habitat impact analysis is available in **Appendix H**.

Shoreline modeling positions were output at each model cell within the GENESIS model domain. For each profile, the average shoreline position was calculated including data from one half the distance to the next downcoast profile up through one half the distance to the next upcoast profile. These averages were calculated for the spring and fall of year 2, for each profile in the study area, and each beach nourishment option including the without Project condition. Profiles from DM-590 through SD-700 were utilized.

Net differences between each beach nourishment option and the without Project condition were calculated. These net shoreline changes at each profile location were then converted into sand volumes using v/s ratios from **Chapter 8** of this report. These sand volumes were distributed across the profiles using the cross shore sand thickness distributions as described in **Chapter 8** of this report. Sand thicknesses were interpolated between the profiles where data were non-existent. The cross shore impacts and depth of coverages are show in **Appendix BB.**

In addition to sand thickness from beach nourishment, sand thickness was also added to each segment to keep pace with the low and high sea level rise scenarios as calculated with the Bruun Rule and described in **Section 0**.

A theoretical sand surface running through existing reefs was developed upon which the combined (beach nourishment and sea level rise) sand thickness were added. Development of the theoretical sand surface is described in **Appendix H**. The April 2004 LiDAR survey bathymetry was used as baseline bathymetry from which the theoretical sand surface was created. This baseline bathymetry was also used for the without Project condition. It is conceded that this data was collected at a snapshot in time, and it may be similar to a spring profile thereby not representing any long-term or average bathymetry. Also, it may not represent the actual bathymetry that will be present in the Project base-year, but it was the most detailed bathymetry set available at the time of analysis.

The combined sand thickness was added to the theoretical sand surface resulting in a suite of new surfaces. These new surfaces were compared to the baseline bathymetry to determine changes in reef height, amounts of coverage, persistence, and impacts to habitats.

10 LAGOON SEDIMENTATION ANALYSIS

10.1 Overview and Summary

Three tidal lagoons with ocean inlets are potentially affected by a beach fill project where increased littoral drift could accumulate within inlets that are currently maintained by dredging. The average annual Project induced changes in dredging costs were estimated for lagoons within the study area. This was done by first determining whether and by how much the pre-Project baseline profile was sand starved. For beaches near lagoons that are expected to have a sand surplus prior to Project construction, no increase in lagoon sedimentation and dredging is expected as a result of the Project. For sand starved beaches, the fraction of time the beach was sand starved in the past was determined through review of historical profile data. The Project induced increase in profile volume was used as a proxy for Project induced increases in gross transport rate which is assumed to be directly proportional to changes in lagoon sedimentation. These two factors (fraction of time sand starved, and increase in profile volume) in combination with historical lagoon sedimentation and dredging rates were used to calculate future, Project induced changes in lagoon sedimentation. The changes in lagoon sedimentation, multiplied by lagoon specific dredging unit costs provided Project induced annual increases in dredging costs for each lagoon as summarized in **Table 10.1-1**. The GENESIS model wants to straighten out the concave shoreline, therefore this one-line model is limited in its prediction of shoreline change along complex shorelines. The absolute model outputs from GENESIS were interpreted broadly only as an indication of the relative behaviors between beach fill alternatives. If the post-construction monitoring shows the inlets to be affected by the widened beaches, an adjustment would be made in the re-nourishment plans. Detailed calculations and intermediate results are provided in **Appendix B8**.

Table 10.1-1 Annual Project Induced Increase in Lagoon Dredging Costs

Beach nourishment option	Batiquitos Lagoon	San Elijo Lagoon	San Dieguito Lagoon
50'	$23,000	$2,000	$18,000
100'	$55,000	$2,000	$48,000
150'	$79,000	$2,000	$77,000
200'	$99,000	$2,000	$104,000
250'	$112,000	$2,000	$110,000
300'	$121,000	$3,000	$117,000
350'	$128,000	$3,000	$124,000
400'	$133,000	$3,000	$132,000

10.2 Method and Results

This section describes the methods used in the lagoon sedimentation analysis as well as providing intermediate and final results. Results are provided in tables as they are developed and combined into one summary table in **Appendix B8**.

The basic equation for predicting the Project induced change in lagoon dredging costs is:

$$\Delta C = \frac{\mathrm{U\ S\ \Delta BW\ 3.2808\ ^{v}/_{s}}}{\mathrm{V}} \qquad \text{(Equation 10-1)}$$

where U is the lagoon dredging unit cost ($/yd^3), S is the annual average lagoon sedimentation or dredging requirement (yd^3/yr), ΔBW is the Project induced increase in beach width in meters, 3.2808 is a conversion factor from meters to feet, v/s is the profile volume of per square foot of beach, and V is the profile volume per foot alongshore (yd^3/ft).

10.2.1 Definitions

Supplementary definitions of variables used in this lagoon sedimentation analysis are alphabetically listed below:

- ΔBW = Project induced increase in beach width as estimated from GENESIS shoreline predictions for specific profile locations, beach nourishment options, and time periods (meters).
- ΔC = Project induced increase in the annual lagoon dredging cost in (2010, $U.S.).
- D_c = depth of closure (ft, MLLW).
- G = gross transport rate (yd^3/yr).
- G_p = gross potential transport rate as estimated in the literature (yd^3/yr). See **Chapter 7** of this report for a summary of G_p estimates. A sand surplus occurs when G=G_p.
- N = fraction of time a profile was sand starved, calculated as the ratio of profiles which do not achieve G_p over the total number of profiles within a given time period.
- P = fraction of time that potential transport was achieved.
- S = annual average lagoon sedimentation or dredging rate (yd^3/yr), ΔS = Project induced increase in S (yd^3/yr).
- U is the lagoon dredging unit cost ($/yd^3).
- v/s = volume of sand in a profile per square foot of beach as determined in **Chapter 8** of this report:
    - v/s Solana-Segment 2 (from DM-565 to SD-660)
    - v/s Encinitas-Segment 1 (from SD-670 to CB-740)
- V = profile volume per foot alongshore calculated as difference between an average profile and the hardpan profile as determined in BMAP (yd^3/ft).
- ΔV = Project induced increase in profile volume per foot alongshore as calculated from ΔBW (yd^3/ft).
- V_p = Potential volume, which is the profile volume required to achieve a sand surplus (yd^3/ft).
- ΔV_{max} = maximum allowable volume increase to bring a profile volume up to a sand surplus.

10.2.2 Assumptions

The lagoon sedimentation analysis was based on the following assumptions:

- Assume future tidal prisms, future wave conditions, future fluvial flow and future fluvial sedimentation at lagoons of interest will be similar to those of the past and are not dependent on Project alternatives.
- Assume Project impacts are restricted to San Elijo Lagoon, San Dieguito Lagoon, and Batiquitos Lagoon.
- Assume historical measurements, estimates, and records of lagoon sedimentation and dredging are sufficiently representative of historical conditions for extrapolation to future conditions. Also, assume that dredging rate and sedimentation rates are approximately equal.
- Assume the following proportionalities: VαBW, GαV, SαG, and CαS., Therefore CαBW .
    - Note: S is proportional to changes in waves, fluvial sedimentation, tidal prism and G. Since the first three parameters are assumed to remain constant, only S being proportional to G useful here.
- Assume G is driven by interaction between waves and sand in the active littoral profile. Once the active profile is completely covered with sand, G_p is achieved and the beach is considered to be in a sand surplus.
    - Under a sand surplus condition, when all of the profile is covered, addition of more sand will not increase G beyond G_p
    - Under a sand starved condition, some of the profile is not covered with sand and G is less than G_p. G is reduced when reef, cobble, immovable bluff face, or other hard bottom become exposed
    - Historical longshore sediment transport rates are discussed and quantified in **Section 4** and **Section 7** of the current report.
- Assume lagoon dredging unit costs from the SANDAG RBSPII apply to the Project.
- Assume the baseline is that condition which exists prior to construction. This baseline was represented by an average of the post RBSPI littoral conditions as a surrogate for post RBSPII conditions which are expected prior the Project construction. This implies that other time periods, such as the Pre-RBSPI, conditions are less representative of the baseline.
- Assume the baseline represents the future without Project condition and remains constant in the future. This same assumption was used to drive the Habitat Impact Analyses based on an EIR condition of a static baseline. Attempting to estimate the future without Project profile changes resulting from sea level rise would be too speculative to be useful. This means that the without Project shoreline does not recede with sea level rise through application of the Bruun rule.
- Assume surveyed profiles near lagoon mouths can be used to determine whether or not a base condition was sand starved. While this is the best available data for this purpose, it is unknown whether this type of data has been used for this purpose in previous studies.
    - Also, assume elevations below the hardpan consist of immovable material that does not contribute to G
    - Also, assume profiles that are above the hardpan consist of sand and are not measurements of movable cobble
- Assume shoreline morphology estimates are accurate.

10.2.3 Representative Profiles

The first step of this method was to select profiles for estimating V and ΔV_{max} near the lagoons. Profiles nearest to and on either side of the lagoon entrances were selected as shown in **Figure 10.2-1** through **Figure 10.2-3** and in **Table** 10.2-1. Profiles that have data both before and after RBSPI were preferred. With longer data records, these profiles tend to capture a lower, more representative hard bottom. Profiles DM-560, SD-595, SD-710, SD-610, SD-650, and SD-660 were initiated in 2000 or later thus are only useful for characterizing Post-RBSPI conditions. Near the lagoons, RBSPI was constructed from April 6 through August 23, 2001. First Year in **Table** 10.2-1 is the first year that a profile location was measured. SD-670 & SD-610 are separated from lagoons by reef and less representative of their respective lagoons so were not used. The D_c for each profile was noted as published by Coastal Frontiers Corporation (2010).

Table 10.2-1 10-2 Representative Profiles for Project Lagoons

Lagoon	**Batiquitos**		**San Elijo**		**San Dieguito**		
Profile	CB-740	CB-720	SD-650	SD-630	SD-600	DM-590	DM-580
D_c (ft, MLLW)	-18	-27	-30	-30	-13	-16	-29
First Year	1987	1983	2000	1983	1983	1984	1983

Figure 10.2-1 Profiles Near Batiquitos Lagoon

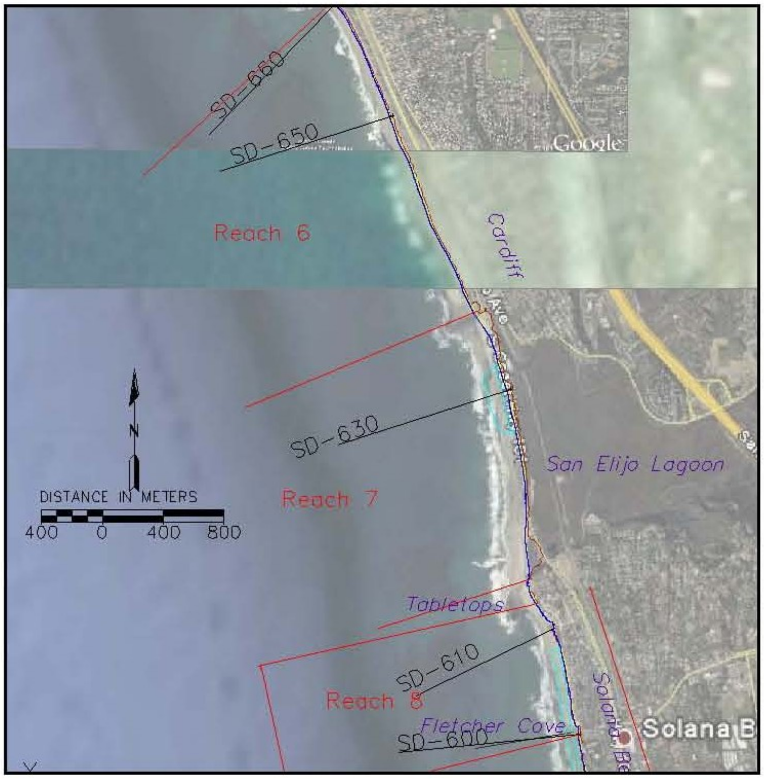


Figure 10.2-2 Profiles Near San Elijo Lagoon

Figure 10.2-3 Profiles Near San Dieguito Lagoon

10.2.4 Estimating N & P

The fraction of time profiles were sand starved (N) and the fraction of time potential transport was achieved (P) were estimated for the various profiles and time periods.

The hardpan is a theoretical profile line consisting of a composite of all the lowest recorded elevations at each offshore position for all the dated profiles at a given location. Since the estimated accuracy of each bathymetric measurement for the profiles was ± 0.5 feet (Coastal Frontiers Corporation, 2010), this buffer was added to the hardpan to create an envelope within which relative confidence of a hardpan can be had. Only the top of this envelope was of interest as a threshold. Each dated profile was compared to the translated hardpan. Graphs of all dated profiles within each time period and profile location are available in **Appendix B8**. When a dated profile dropped below the translated hardpan at any offshore point in the profile, then that dated profile was considered sand starved. The assumption was that elevations below the translated hardpan consist of immovable material that does not contribute to G, and G< G_p . At a few locations along the profile out near D_c, it was not obvious whether the hardpan was a stable sandy bottom or a non-erodible material. Hence, positions that satisfied the following conditions were not considered sand starved at those locations: 1), located near D_c, 2) profiles had low variability, and 3) profile dropped below the translated hardpan. While this condition was rare, it did reduce the number of sand starved profiles thus reducing ΔC.

An example is provided in **Figure 10.2-4** for Profile CB-720. The translated hardpan is shown as a black line running along the bottom of the other dated profiles. Great variability can be seen high in the profile where it is assumed that a non-erodible substrate exists. Where a dated profile drops below the translated hardpan, that date is noted as being sand starved. Farther down the profile, near D_c (-27 ft, MLLW), the profile is smoother and it is assumed that any changes in elevation mainly result from changes in wave activity and measurement uncertainty, and are not the result of rocky substrate becoming exposed.

The Pre-RBSPI time period includes all profiles dated before May of 2001 and the Post-RBSPI baseline includes all profiles from May of 2001 through 2009. The Post-RBSPI time period is expected to be most similar to the condition occurring prior to the Project base-year, especially since the RBSPII is expected to nourish the beaches again in the summer of 2012. The Post-RBSPI time period is the baseline time period.

The number of sand starved dated profiles during the baseline were divided by the total number of dated profiles within the baseline, representing the fraction of time the baseline was sand starved (N). A sand surplus exists when N=0 and total sand starvation exists when N=1. For the example shown in **Figure 10.2-4**, dated profiles dropped below the translated hardpan 4 out of the 17 dates within the baseline period (N = 0.24). The nine year baseline was sand starved 24% of the time.

The fraction of time that potential transport was achieved (P) was calculated to simplify line fitting of measured data as discussed in the next section. The equation for this is:

$$P = 1-N \qquad \text{(Equation 10-2)}$$

These values are summarized in

Table 10.2-2.

Table 10.2-2 Results for V, N, P, Vp, and ΔVmax

Lagoon	**Batiquitos**		**San Elijo**		**San Dieguito**		
Profile	CB-740	CB-720	SD-650	SD-630	SD-600	DM-590	DM-580
V (yd^3/ft)	92.3	189.1	73.3	188.4	61.9	92.3	157.5
N	0.53	0.24	0.63	0.00	0.29	0.47	0.33
P	0.47	0.76	0.38	1.00	0.71	0.53	0.67
V_p (yd^3/ft)	200.4						
ΔV_{max} (yd^3/ft)	108.1	11.3	127.1	12.0	138.5	108.1	42.9

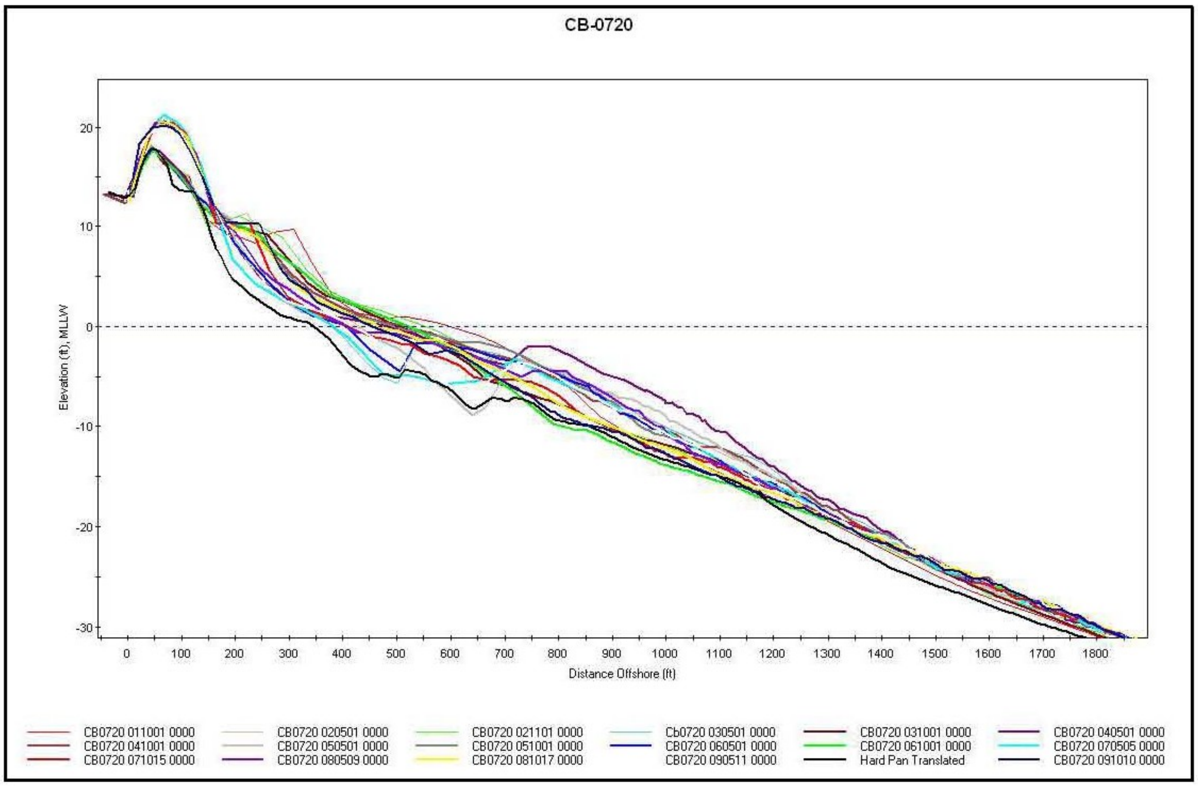


Figure 10.2-4 Post RBSPI Profiles at CB-720

10.2.5 Estimating ΔVmax

ΔV is limited to be less than or equal to the increase in sand volume required to achieve a sand surplus (ΔV_{max}).

V is the volume of sand between an average profile and the hardpan for a given profile location over a given time period. The BMAP software was used to calculate this value, providing volumes in yd^3/ft of alongshore beach. This was calculated from the profile origin out to D_c. **Figure 10.2-5** shows an example for profile location SD-630 with the average of the Pre-RBSPI profiles (Pre-Avg) in gray, Post-RBSPI profiles (Post-Avg) in green, and hardpan in red. Graphs for the other profile locations are provided in **Appendix B8** and V results for each profile are listed in

Table 10.2-2.

To find the relationship between N, P, and V, a scatter plot of all measured V and P was prepared in **Figure 10.2-5**. Data for this figure are listed in

Table 10.2-2. Where there is no sand in the profile (V=0), none of the potential transport is achieved (P=0), thus the line was forced through the zero intercept. The least squares line fit to the data results in the following equation:

$$V = 200.4\ P \qquad \text{(Equation 10-3)}$$

The minimum volume in the profile (V_p) required for potential transport to be achieved (P=1) is V_p=200.4 yd^3/ft. There is a high level of uncertainty in this estimate of V_p, and therefore this one generalized value was used for all profile locations. Another option would be to calculate a V_p for each profile location, but these would have even greater uncertainty.

The maximum increase for any V is the difference between V_p and V as expressed by:

$$\Delta V_{max} = V_p - V \qquad \text{(Equation 10-4).}$$

Where the units are yd^3/ft and ΔV_{max} results for each profile are listed in

Table 10.2-2.

Setting V_p=200.4 yd^3/ft uniformly results in a minor conflict at Profile SD-630. At this profile, during the Post-RBSPI time period, N was equal to 0 indicating a sand surplus. By moving V_p to a higher uniform value, SD-630 is then forced to accept a nominal increase in volume to achieve a sand surplus. This is a conservative assumption at this location, slightly increasing ΔC over other methods.

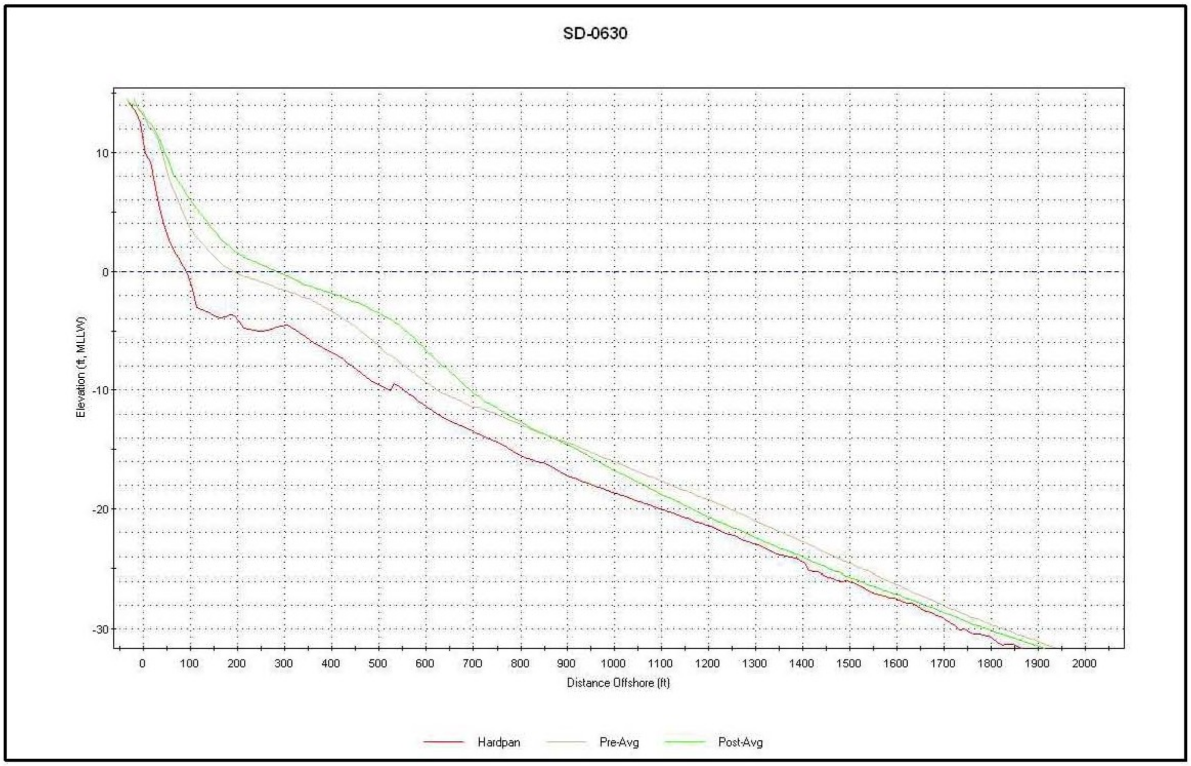


Figure 10.2-5 Average Pre-RBSPI, Average Post RBSPI and Hardpan Profile for SD-630

10.2.6 Estimating ΔV

ΔV is the Project induced increase in profile volume for a location, beach nourishment option, and time condition (e.g., SD-630, 100 foot, annual average). This was calculated from estimates of net change in beach width (ΔBW), then converted to a volume using the v/s ratios for each segment with the following equation:

$$\Delta V = 3.2808\ \Delta BW\ v/s \qquad \text{(Equation 10-5)}$$

where 3.2808 was used to convert ΔBW from meters to feet, and ΔV is given in yd^3/ft . The v/s ratios were developed in **Chapter 8** of this report, with one value for Encinitas-Segment 1 and another for Solana- Segment 2. ΔV was calculated for each profile location and each of the eight possible beach nourishment options. ΔBW averages were calculated across space (all GENESIS cells ranging from ½ the distance from a lower numbered profile to ½ the distance to the next higher numbered profile) and across time (all the 16 year GENESIS model results). These values are summarized in **Table 10.2-3**.

Table 10.2-3 Results for ΔBW and ΔV

Lagoon		**Batiquitos**		**San Elijo**		**San Dieguito**		
Profile		CB-740	CB-720	SD-650	SD-630	SD-600	DM-590	DM-580
v/s (yd^3/ft^2)		0.8637		0.7132				
	Beach Nourishment Option							
ΔBW (m)	50'	1.3	2.8	0.1	8.7	11.1	0.5	0.1
	100'	4.3	7.0	0.1	11.4	29.1	1.6	0.4
	150'	7.0	10.1	0.1	13.1	45.5	3.7	1.1
	200'	9.3	13.2	0.1	14.9	60.1	7.2	2.1
	250'	10.8	16.0	0.1	17.2	73.3	12.2	3.2
	300'	11.8	18.5	0.2	19.9	85.6	18.4	4.6
	350'	12.6	20.4	0.2	22.8	97.6	24.5	5.8
	400'	13.2	22.0	0.3	25.9	109.6	30.5	7.1
ΔV (yd^3/ft)	50'	3.7	7.9	0.2	20.4	26.0	1.2	0.2
	100'	12.2	19.8	0.2	26.7	68.1	3.7	0.9
	150'	19.8	28.6	0.2	30.7	106.5	8.7	2.6
	200'	26.4	37.4	0.2	34.9	140.6	16.8	4.9
	250'	30.6	45.3	0.2	40.2	171.5	28.5	7.5
	300'	33.4	52.4	0.5	46.6	200.3	43.1	10.8
	350'	35.7	57.8	0.5	53.3	228.4	57.3	13.6
	400'	37.4	62.3	0.7	60.6	256.4	71.4	16.6

10.2.7 Estimating S

The sedimentation rates for different lagoons and time periods (S) were estimated from surveys or dredging records as described below and summarized in **Table 10.2-4** and **Figure 10.2-6**.

Table 10.2-4 Southern California Lagoon Sedimentation and Dredging Rates

Lagoon			Batiquitos	San Elijo	San Dieguito
Time Period	Estimated From	Unit			
Pre-RBSPI	Sedimentation	yd^3/yr	-	-	-
	Dredging	yd^3/yr	16,721	14,000	-
Post-RBSPI	Sedimentation	yd^3/yr	59,818	-	26,500
	Dredging	yd^3/yr	31,343	22,000	26,500

- = unknown

<u>San Elijo Lagoon</u>

From 1995 through 2009 a total of 295,800 yd^3 was dredged from the San Elijo Lagoon (Coastal Frontiers Corporation, 2010). The average Pre-RBSPI dredging rate was 14,000 yd^3/yr and the average Post-RBSPI dredging rate (S) was 22,000 yd^3/yr (Coastal Frontiers Corporation, 2010). The increased dredging rate is somewhat attributable to increased funding availability.

<u>Batiquitos Lagoon</u>

From 1999 through 2010 a total of 363,600 yd^3 was dredged from Batiquitos Lagoon, averaging 30,300 yd^3/yr (Coastal Frontiers Corporation, 2010: Merkel & Associates, 2009) with 16,800 yd^3/yr dredged from 1999 through 2001 (Coastal Frontiers Corporation, 2010) and 31,300 yd^3/yr expected to be dredged from 2002 through 2011 (Webb, 2010). There were substantial funding and contractual issues that limited dredging work, so this value is believed to under estimate actual dredging needs and sedimentation rates. Merkel & Associates (2009) estimated a post restoration sedimentation rate of between 50,420 yd^3/yr and 69,216 yd^3/yr. An average value of 59,818 yd^3/yr was used for this study representing S for this lagoon.

<u>San Dieguito Lagoon</u>

The San Dieguito Lagoon restoration maintenance plan estimated removal of 4,000 yd^3 of sand from the inlet between the ocean and Highway 101 Bridge, and about 12,000 yd^3 from the channel west of the railroad bridge every eight months. In addition approximately 5,000 yd^3 of sand from the channel east of the railroad bridge is planned to be dredged every two years or as needed (Coastal Environments, 2010). In addition to prescribed dredging, annual monitoring of channels east of Jimmy Durante Bridge is recommended. Coastal Environments assumed that maintenance dredging would equal sedimentation. The planned maintenance dredging without the Project (S) was calculated as [(4,000yd^3 +12,000yd^3)/8 months] x12 months/yr +5,000 yd^3/2yr =26,500 yd^3/yr.

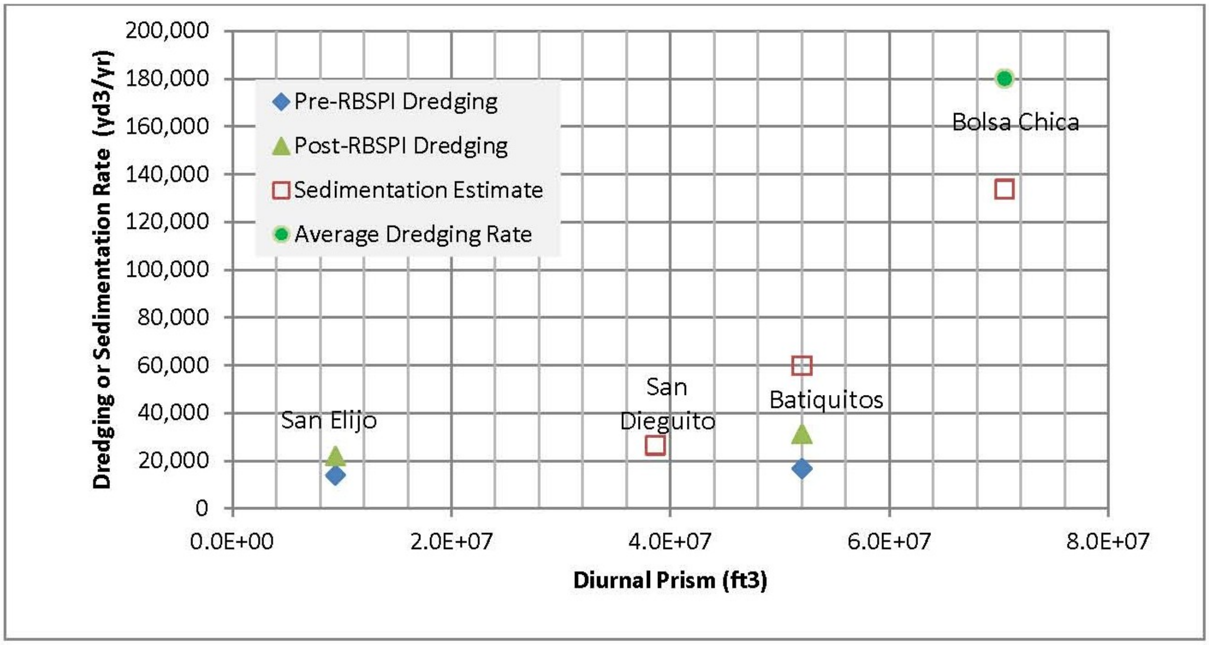


Figure 10.2-6 Southern California Lagoon Sedimentation and Dredging Rates

10.2.8 Estimating ΔS

The lagoon sedimentation rate (S) should ideally equal the lagoon dredging rate if all the deposited material was dredged. It was assumed that S is proportional to G which is proportional to the V. The relationship between S and V is plotted in **Figure 10.2-7**. The line was forced through the zero intercept since no sand in the profile (V=0) results in no gross transport and no littoral sedimentation in the lagoon (S=0). The resulting linear equation is:

$$\Delta S = S\ \Delta V/V \qquad \text{(Equation 10-6)}$$

Where ΔS is given in yd^3/yr. This value is limited by the equation:

$$\Delta S \leq S\ \Delta V_{max}/V \qquad \text{(Equation 10-7)}$$

Results are provided in **Table 10.2-5**. Where ΔV was less than ΔV_{max}, Equation 10-6 was used, otherwise Equation 10-7 was used. The ΔV/V values associated with each profile surrounding a lagoon were averaged to provide one value for each lagoon as listed in **Table 10.2-5**
.

Table 10.2-5 Results for Average ΔV/V and ΔS

	Beach Nourishment Option	Batiquitos Lagoon	San Elijo Lagoon	San Dieguito Lagoon
Average $\Delta V/V$ or $\Delta V_{max}/V$	50'	0.04	0.03	0.14
	100'	0.10	0.03	0.38
	150'	0.14	0.03	0.61
	200'	0.17	0.03	0.82
	250'	0.20	0.03	0.87
	300'	0.21	0.03	0.92
	350'	0.22	0.03	0.98
	400'	0.23	0.04	1.04
ΔS (yd^3/yr)	50'	2,448	735	3,834
	100'	5,733	735	10,134
	150'	8,212	735	16,176
	200'	10,324	735	21,671
	250'	11,701	735	22,936
	300'	12,619	770	24,508
	350'	13,354	770	26,032
	400'	13,905	805	27,547

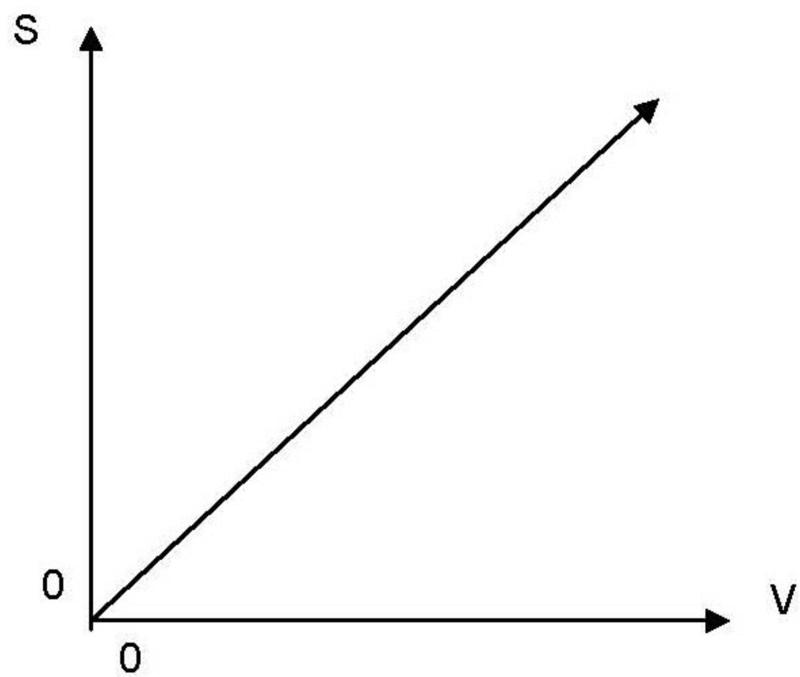


Figure 10.2-7 Graph of Relationship Between S and V

10.2.9 Estimating ΔC

The last step was estimating the change in annual lagoon dredging costs (ΔC). Annual dredging unit costs (U) were available for each lagoon from SANDAG (AECOM et. al., 2011). The equation is:

$$\Delta C = U\ \Delta S \qquad \text{(Equation 10-8)}$$

Where U are in United States $\$/yd^3$, valid for year 2010. Results are listed in **Table 10.2-6** and ΔC estimates were rounded to the nearest $1000 in **Table 10.1-1**.

Table 10.2-6 Results for U and ΔC

	Lagoon		**Batiquitos**	**San Elijo**	**San Dieguito**
	U	$\$/yd^3$	$9.56	$3.26	$4.78
	Beach Nourishment Option	Unit			
ΔC	50'	$/yr	$23,407	$2,395	$18,326
	100'	$/yr	$54,812	$2,395	$48,440
	150'	$/yr	$78,510	$2,395	$77,320
	200'	$/yr	$98,698	$2,395	$103,589
	250'	$/yr	$111,863	$2,395	$109,633
	300'	$/yr	$120,641	$2,509	$117,150
	350'	$/yr	$127,662	$2,509	$124,434
	400'	$/yr	$132,929	$2,623	$131,674

11 SURFING CHANGE ANALYSIS

Surfing is an important recreational activity for beaches in north San Diego County. A set of analyses were performed to ascertain the likely changes to surfing resulting from the Project. For the surf sites within the study area each of the following topics were addressed:

- Waves that reflect off the shore back to sea are known to surfers as backwash. The effect is most commonly known for making catching and riding waves more difficult. Changes in backwash were estimated from three different possible sources: 1) increased beach slopes from constructed beach fills, 2) increased surf zone slope from increased D_{50}, and 3) bluff reflection with sea level rise. The Project is expected to result in an overall improvement (decrease) in the amount of backwash.

- Wave breaking intensity is an indicator of how hollow the breaking wave is, with mushy waves having low intensity and hollow waves having high intensity. The breaking intensity is primarily determined by the seabed slope, which for beach breaks can change with D_{50}. If the nourishments result in no change to D_{50}, no change in wave breaking intensity is expected. However, if an increase in D_{50} is expected within the littoral zone, the breaking intensity is expected to increase slightly throughout the study area.

- Each reef break within the study area was analyzed with respect to Project induced changes in sedimentation. If a beach fill alternative fills in the low areas around a naturally high relief reef, this can change the way the wave breaks over the reef. A silted in reef can make a reef break behave more like a beach break, with lower breaking intensities, shorter ride lengths, lower peel angles, and more closed out conditions. For the beach nourishment options and sea level rise scenarios, changes are likely at some of the reefs.

- Nearshore currents in and around surf sites change the way surfers access the sites and change the way the waves break. Nearshore currents in the study area generally tend to be amorphous, constantly changing with wave, wind, and tide conditions, except near lagoon mouths where they are slightly more predictable. The beach fills are not expected to change these nearshore currents in any detectable amount.

- In addition to changes in wave quality, the location and frequency of these breaking waves is also important. The beach fill alternatives are expected to move the entire surf zone sea bed profile seaward, thus shifting the location of breaking waves seaward an associated distance. The beach fills are not expected to change the wave breaking frequency in any detectable amount.

11.1 Key Surf Site Characteristics

Parameters used in the surfing change analysis are briefly described below. More detailed descriptions are available in **Appendix B9**.

11.1.1 Basic Surfing Terms

The basic terminology of a surfing wave is shown in **Figure 11.1-1** and **Figure 11.1-2**. The breaking wave height (H_b) is the vertical distance between the wave trough and crest. Surfers ride all parts of the wave from the foam through the shoulder, and ideally attempt to ride inside the tube or curl (i.e., pocket) of the wave. A good surfing wave will peel either right or left at a rate that allows the surfer to stay ahead of the break and maximize the length and speed of the ride. Directionality is based on the surfer's perspective while facing shore. The rate at which the wave peels is primarily determined by the characteristics of the wave and shape of the seabed. Seabed shape in combination with wave height, period and direction are the primary factors in determining a good surfing wave.

Along with H_b, the wave peel angle (α) is a critical parameter for determining whether a wave can be surfed. Generally waves with peel angles between 30 and 60 degrees are sought most by surfers. Peel angles less than 30 degrees are unsurfable approaching closeout conditions. Peel angles approaching 90 degrees result in the rider headed straight to shore and are less preferable. The peel angle was first defined as the included angle between the peel-line and a line tangent to the wave crest at the breaking point (Walker et. al, 1972). In this context the peel line is the path of broken white water left after the wave breaks. **Figure 11.1-2** shows the parameters defining peel angle (Walker, 1974). In this figure, the wave breaks along a line from point A to point B. At position A the wave has a velocity of propagation (V_w) which is perpendicular to the wave crest. The peel velocity (V_p), is the velocity the wave breaks, or peels, along the wave crest. Summing the vectors gives the resultant velocity vector (V_s), which approximates the surfers speed if the surfer remains close to the wave break point.

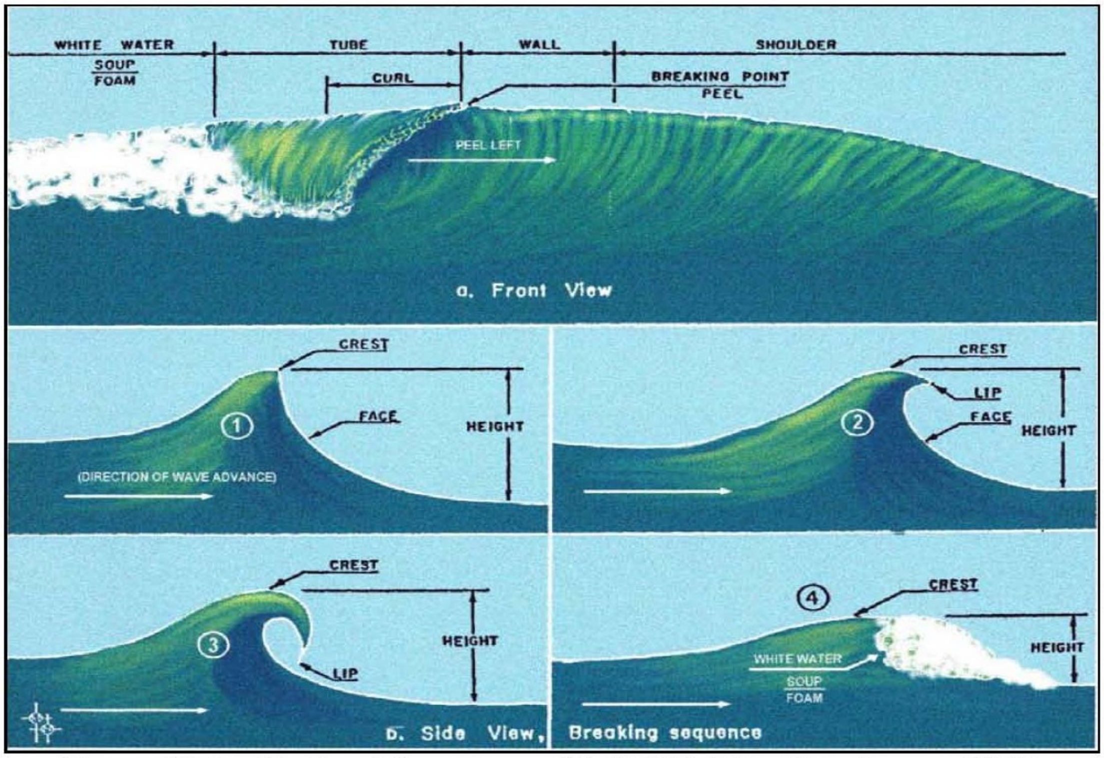


Figure 11.1-1 Surfing Wave Terms (Source Moffatt & Nichol Engineers, 2000)

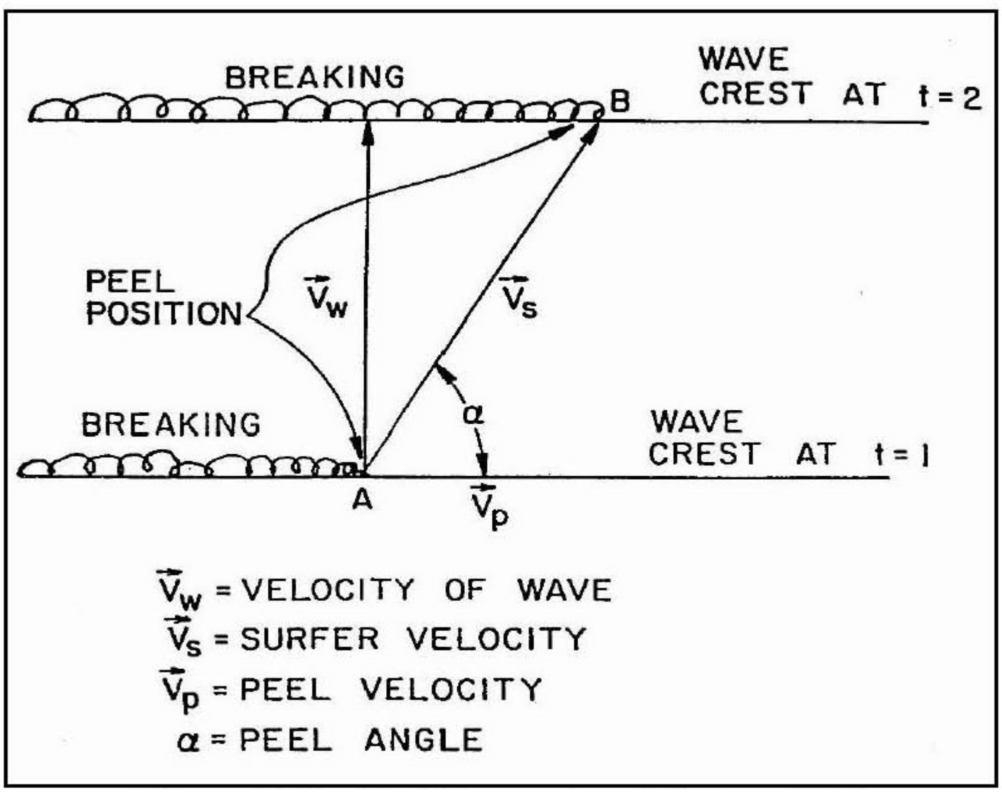


Figure 11.1-2 Peel Parameters (Walker, 1974)

11.1.2 Breaking Intensity

Measures for estimating wave breaking intensity have been developed by various researchers as discussed below.

Iribarren derived a parameter correlating the breaker type to the bed slope, breaking wave height, and wave length (Iribarren and Nogales, 1949; reprinted in USACE, 2002). This value (Iribarren number, surf similarity parameter, or breaker intensity) is calculated as:

$$\xi_o = \tan\beta/(H_o/L_0)^{½} \qquad \text{(Equation 11-1)}$$

Where β is the angle of the seabed slope (tan β=rise/run), H_o is the deep water wave height, and L_0 is the deep water wave length as described by $L_0 = gT^2/2\pi$, g is the acceleration due to gravity and T is the wave period.

The surf similarity parameter indicates under what circumstances and how waves will break. Waves will break on the seabed slope when the surf similarity parameter is less than 2.3 (Battjes 1974). A wave will not break on a very steep seabed slope but instead be reflected back to sea. The surf similarity parameter increases with increasing seabed slope, increasing wavelength and decreasing wave height. Therefore smaller waves will break with higher surf similarity parameter (higher breaking intensity) than larger waves over the same seabed slope.

Ranges of surf similarity parameters are described by the breaker type as summarized in **Table 11.1-1** and **Figure 11.2-1**. Breaker type is used to classify wave shape during breaking, which is of great importance for surfing. The higher the surf similarity parameter, the more intensely the wave breaks.

The waves in San Diego County are generally spilling to plunging breaker types, due to the presence of mildly sloping sandy beaches interspersed with steep bottomed reefs. Spilling and plunging breakers are preferred for general surfing (Walker 1972). Surging and collapsing breakers are unsurfable.

Table 11.1-1 Summary of Breaker Types

Breaker Type	Surfing Terminology	Surf Similarity[1], ξ_o	Vortex Ratio[2], L/W
Surging and collapsing	Not surfable	$\xi_o > 3.3$	Not available
Plunging	Tubing, hollow	$0.5 \leq \xi_o \leq 3.3$	$1.4 \leq L/W \leq 3.4$
Spilling	Mushy, fat	$\xi < 0.5$	Not available

Sources: 1= USACE, 2002; 2=Mead and Black, 2001

The vortex ratio was developed to better estimate subtle wave differences within the plunging breaker type. The vortex ratio is defined as the ratio of the wave's vortex length to its vortex width when viewed parallel to the wave crest (Mead and Black, 2001). This method of grading wave intensity eliminates wave characteristics focusing solely on the seabed slope as the forcing variable. A linear relationship was found for the vortex ratio:

$$L/W = 0.065m + 0.821 \qquad \text{(Equation 11-2)}$$

where L is the length of the breaking vortex, W is the width of the breaking vortex, and m is the seabed slope (horizontal/vertical).

The lower the vortex ratio, the greater the area of the vortex and more intensely the wave breaks. The measured prototype data for the vortex ratio ranged from 1.42 to 3.43 and this range is reflected in **Table 11.1-1**.

11.1.3 Wave Section Length

By the time a wave crest reaches a surf site it is sometimes bent or broken when the crest is viewed from an aerial perspective, with variation in height and angle along its length. The variations can be caused by a mixed swell spectrum, bathymetric effects, non-linear wave-wave interactions, and island sheltering. Although generally surfers desire waves that peel cleanly along the wave crest at a surfable speed, often waves break in sections with a length S_L. The total ride length is equal to all the section lengths combined. Small sections that break at once, with a peel angle near 0 degrees, are not a problem for a surfer provided the surfer can generate enough speed to make it past the section to the unbroken wave crest. The ability to negotiate a section is related to the surfer's ability to generate enough speed to make it past the section to the unbroken wave crest.

11.1.4 Backwash

Waves that reflect off the shore back to sea are known to surfers as backwash. The effect can make paddling out to sea somewhat easier, but is most commonly known for making catching and riding waves more difficult. This is investigated in more detail in **Appendix B9**. No guidance on acceptable ranges of backwash was found in the literature. Backwash is frequently developed as waves reflect off a steep beach, bluff face, or seawall. The degree of wave reflection is defined by the reflection coefficient, $C_r=H_r/H_i$, where H_r and H_i are the reflected and incident wave heights, respectively. Changes in backwash intensity can be estimated by changes in the reflection coefficient as defined by the USACE (2002):

$$C_r= a\xi_o^2/(b+ \xi_o^2) \qquad \text{(Equation 11-3)}$$

Where a=0.5, b=5.5, and ξ_o is the surf similarity parameter at the structure face. Combining terms results in: $C_r = 0.5L_o/[H_o\ m^2(5.5+L_o/(H_o m^2))]$. The reflection coefficient was calculated for post-construction and long-term changes to the profiles resulting from the Project.

11.2 <u>Surf Site Categorization</u>

Surf sites are locations with the right wave, wind, and bottom conditions where waves break regularly in a form desirable for surfing. To supply surfable waves, a surf site must be open to ocean swell, be oriented in the right direction, and have the right bottom conditions. Types of surf sites that exist in the study area defined below, organized by substrate type.

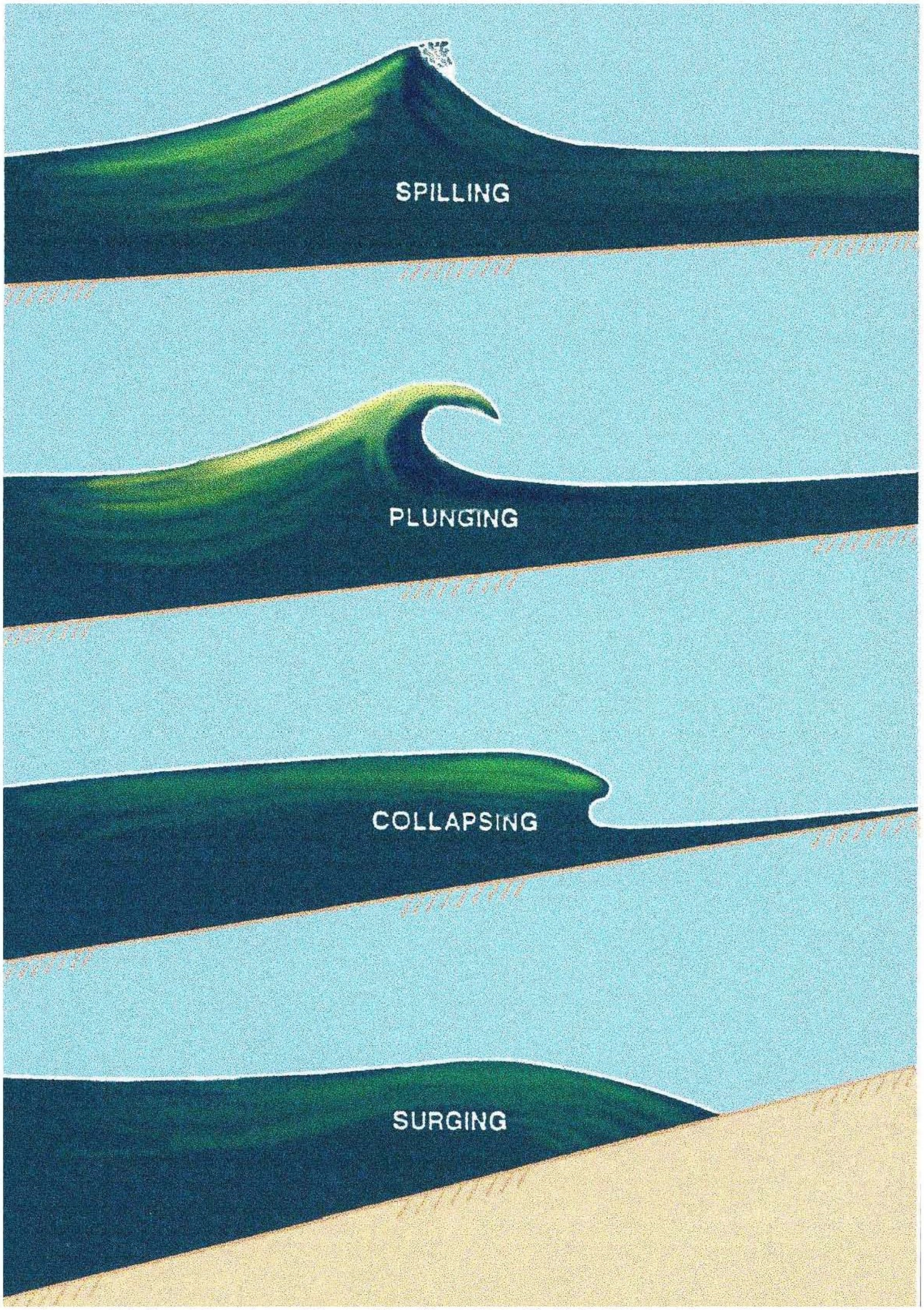


Figure 11.2-1 Breaker Classification (Source: Moffatt & Nichol Engineers, 2000)

Beach breaks are characterized by generally sandy bottoms with straight and parallel bathymetric contours. At a beach break, waves break in walls or peaks along the beach caused by offshore wave focusing and nearshore sand bars and rip currents. Examples of open, unmodified beach breaks can be found in Moonlight Beach in Encinitas and Georges near San Elijo Lagoon.

Bedrock reefs can be found where a softer material has eroded, leaving behind the harder substrate. They occur on open coasts typically in the vicinity of bluff or cliff backed shorelines and are often found near points or headlands. These reefs can range from mildly sloping longboard waves to steep ledges such as Maverick's in San Mateo County. Bedrock reefs in San Diego County include both Swamis and Black's beach, and headland type reefs are represented by Pacific Beach Point and Point Loma.

River deltas are surfing features found seaward of large river mouths. During extreme rainfall events, fast moving water carries sand, cobble and boulders into the surf zone where it is deposited into a delta shape. River mouth surf sites often benefit from offshore wave focusing resulting in larger wave heights than surrounding areas. Trestles in northern San Diego County has two cobble river deltas at the current and relic mouths of the San Mateo River.

Ebb bars are formed at the mouths of tidal lagoons and rivers. They are mobile sand features dependant mainly on sand carried out of the lagoon or river during ebb tidal flow. The deposited sand forms a bar which can improve wave refraction and focusing and steepen the bed profile. Where a river runs through a lagoon, a river delta and ebb bar can form at the same location. Example ebb bars within the study area can be found at the mouths of Batiquitos and San Dieguito Lagoons (i.e., Del Mar Rivermouth).

Man-made structures such as jetties, groins, piers, pipelines, and artificial reefs can modify the wave and or bottom characteristics to improve the wave breaking for surfing. These commonly occur near sandy bottom beach breaks. Southern California examples are near the south jetty of Oceanside Harbor and near the jetties of Batiquitos Lagoon (Ponto Surf Site).

While these definitions are useful generalizations, surf sites often blend the various categories. For example, beach breaks often have features such as offshore reefs, which control the sand bar development and wave focusing and some river deltas often behave like a point break.

11.3 Existing Conditions

The surf sites within the study area are listed in **Table 11.3-1** and shown in **Figure 11.3-1** through **Figure 11.3-3**. Information in this table was collected from various sources (City of Encinitas; Surfer Magazine, 2006; Cleary and Stern, 1998; Guisado and Klaas, 2005; Wright, 1985; Nielsen, 2007; surf-forecast.com; Wannasurf.com). Detailed descriptions of individual surf sites are provided in **Appendix B9**. Within this table, the Encinitas-Segment 1 is highlighted in green and Solana -Segment 2 is highlighted in purple.

In addition to the locations of the surf sites shown in **Figure 11.3-1** through **Figure 11.3-3,** the profile locations, Project reaches, and Project segments are also shown. In these figures, reefs are indicated with a circle and beach breaks are indicated with a square.

Table 11.3-1 Surf Sites in the Study Area

Name	Type	Note
Ponto, Batiquitos	Ebb Shoal, beach	Right & left, near jetties
Grandview	Reef-beach break	Right & left
Avocados	Beach break	Right & left
White Fence	Beach break	Right & left
Log Cabins	Beach break	Right & left
North Beacons	Reef-beach break	-
Bamboos	Reef-beach break	-
South Beacons	Reef-beach break	-
North El Portal	Beach break	Right & left
Stone Steps	Reef-beach break	Right & left
Rosetas	beach break	Right & left
Moonlight	Beach break	Right & left
D Street	Beach break	Hollow left
Trees	Reef	-
Boneyards, outside Swamis	Reef	right
Swamis	Reef/pointbreak	Hollow to mushy, advanced, right
Dabbers	beach break	Right & left for beginners
Brown House		-
Pipes	Reef	Left (some rights), hollow to mushy, all surfers
Traps	Reef-beach break	-
Turtles	Reef-beach break	Mushy longboard Right & left
Barneys	-	-
85/60s	Reef	-
Tippers	Reef	Mushy longboard Right & left
Campgrounds	Reef	-
Suckouts, Lagoon Mouth	Reef	Hollow, advanced, Right & Left
Cardiff Reef, South Peak	Reef	Right (some lefts), medium, all surfers
Evans	Beach break	Right & left, intermediate
Georges, Cardiff Beach	Beach break	Right & left, medium all surfers
Parking Lots	beach break	Right & left
Seaside Reef	Reef	Left (some rights), hollow, intermediate to
Pallies	Reef	Left
Table Tops, Tide Beach Park	Reef	Hollow Right
Pillbox, Fletcher Cove	Reef-beach break	right
South Side, Fletcher Cove	Reef-beach break	Left
Cherry Hill, Seascape Surf Beach	Beach break	Right & left
Del Mar, 17th – 20th Street	Beach break	Right & left, intermediate
15th Street	Reef	Right & left, all surfers

- = unknown information

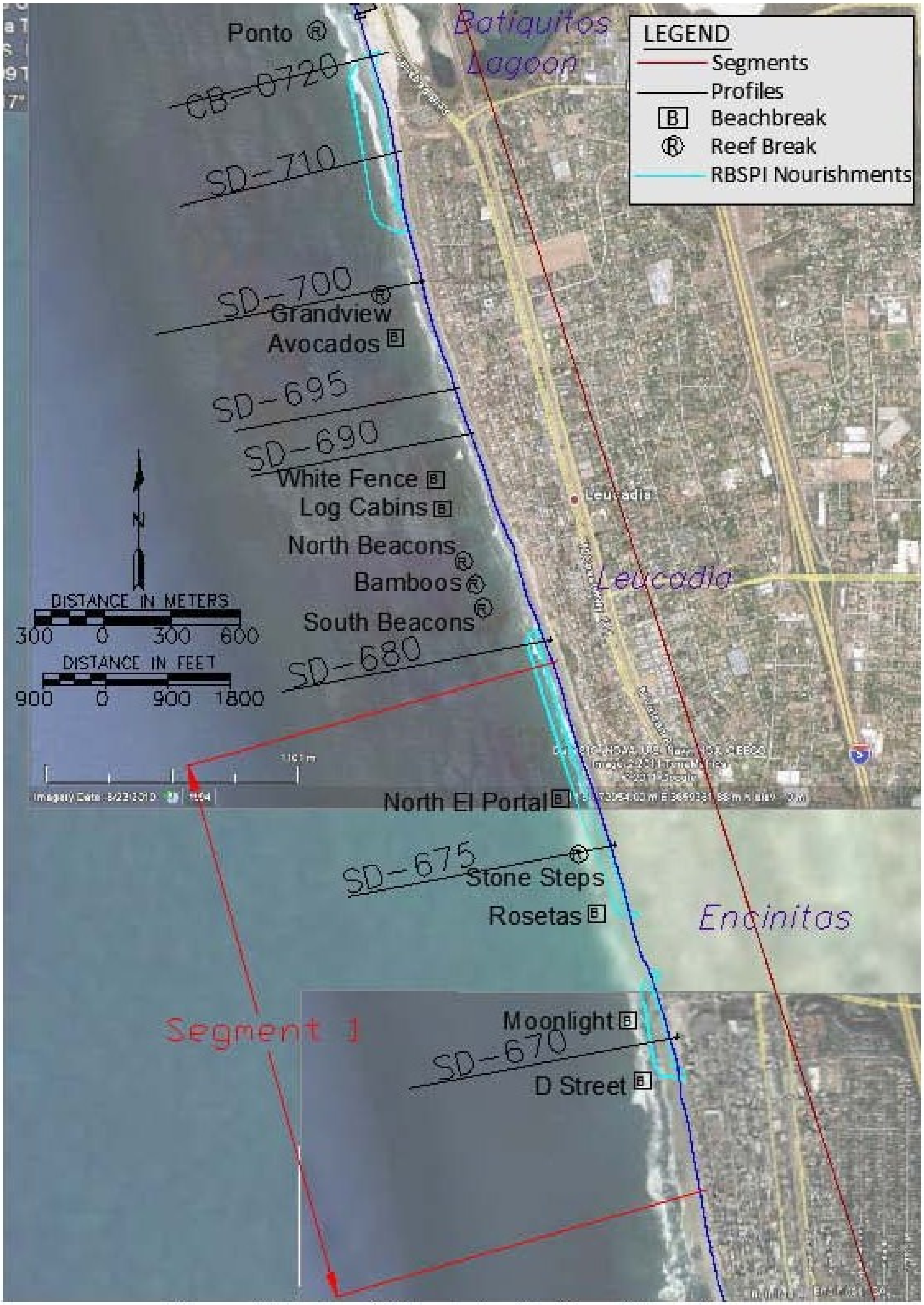


Figure 11.3-1 Surf Sites in the Northern Study Area

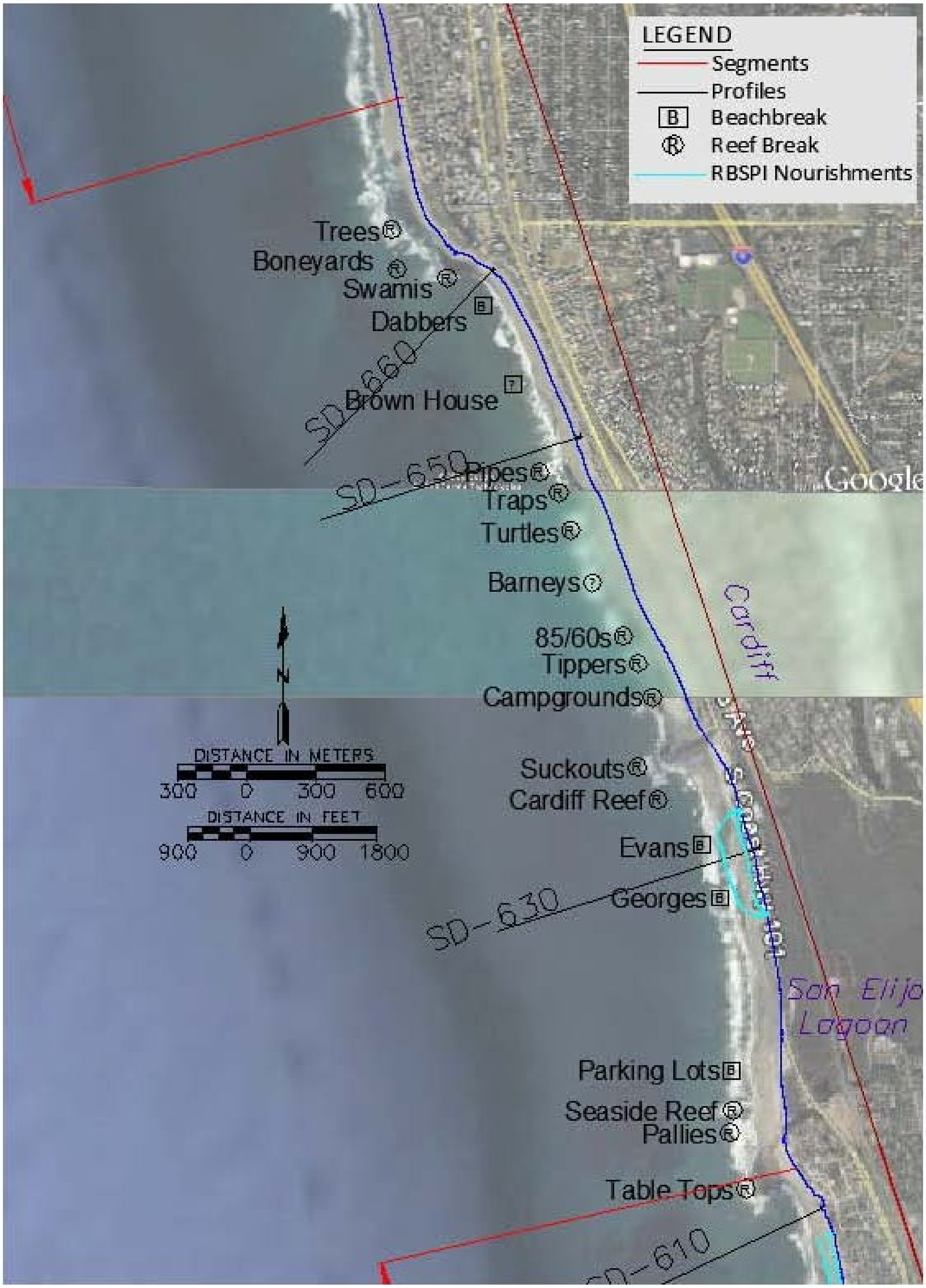


Figure 11.3-2 Surf Sites in the Middle Study Area

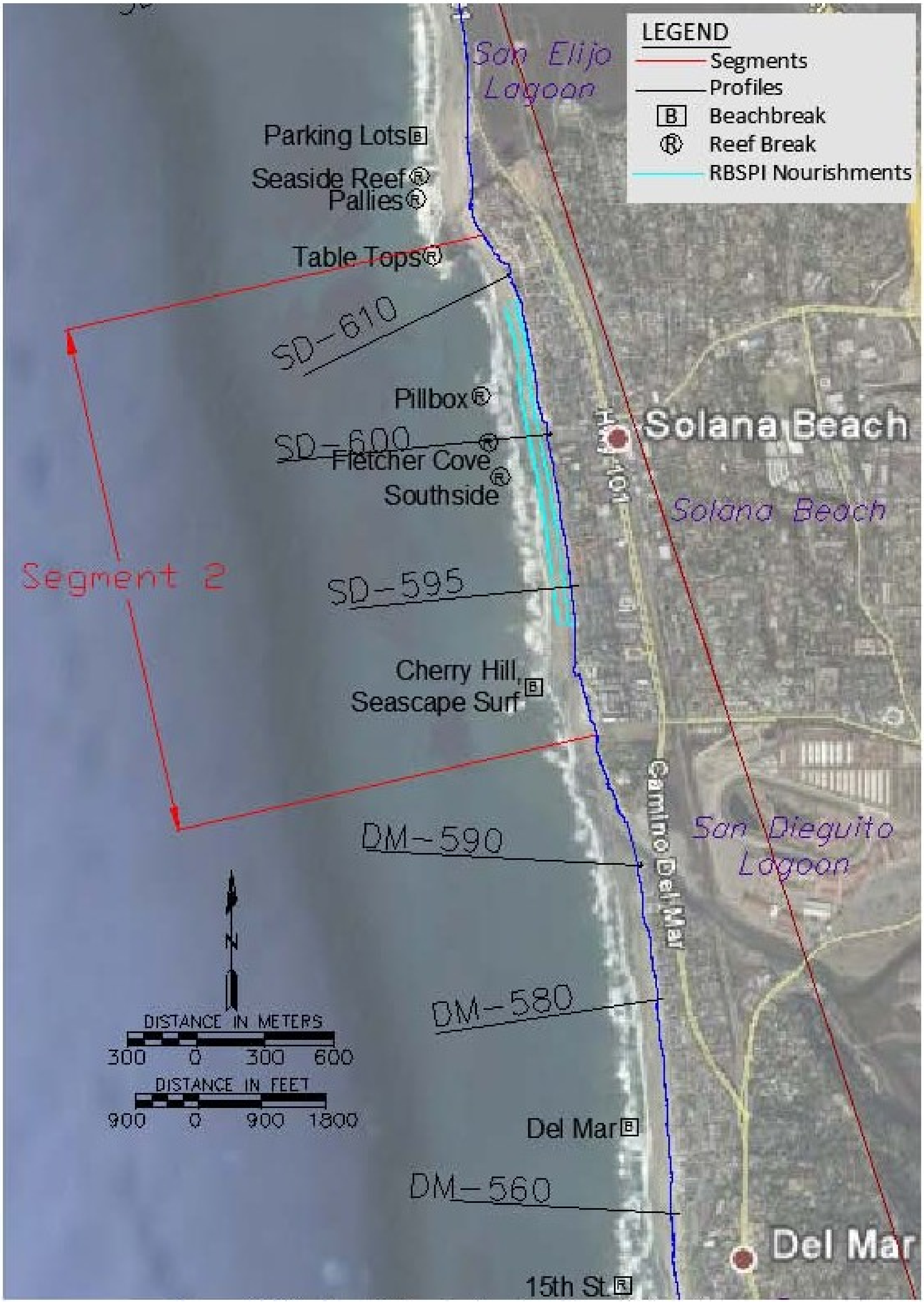


Figure 11.3-3 Surf Sites in the Southern Study Area

11.4 Analysis and Results

The method for each type of analysis and results of that analysis are provided below. Analyses and discussion were performed for:

- Backwash changes,
- Breaking intensity for beach breaks,
- Sedimentation changes to reef breaks,
- Currents at surf sites, and
- Changes to surf break location and surfing frequency.

11.4.1 Backwash Changes

Three types of backwash changes were analyzed: post-construction backwash, year two backwash from increased D_{50}, and long-term backwash from sea level rise.

Post-Construction Backwash

The beach profile can be expected to differ from a natural shape immediately after construction of the beach nourishments each segment. These post-construction profiles are expected to be short lived, evolving to equilibrium profiles within a month or two after construction. Post-construction beach slopes are mild, low in the profile (during low tides), due to a nearshore bar and they are steep, high in the profile (during high tides) as compared to the long-term average fall beach slope. Fall slopes were used since RBSPI was completed in the summer and significant profile data is available for the post construction fall conditions. During low tides, the post construction backwash was found to be either the same or less than the long-term average. During high tides, the post construction backwash was found to be either the same or higher than the long-term average. Measuring beach slopes across the entire beach averages out these differences, resulting in negligible changes in beach slopes and backwash. To estimate the worst case changes, the design and post-construction, high tide, change in backwash from the long-term condition is quantified below.

Beach slopes were measured from profile survey data from 10 ft MLLW down to 0 ft MLLW. A uniform elevation was chosen for the top of the beach berm at 10 ft, MLLW for consistency of method. This is below the plateau of most beach berms, but high enough to capture most wave runup and backwash. The bottom of the range was chosen as 0 ft, MLLW since this is a common location for the bottom of the dry beach and the lower limit of the swash zone. Reflection coefficients were calculated from these beach slopes using Equation 11-3. Goda (2000) reports reflection coefficients for natural beaches ranging from 0.05 to 0.2 and the *Shore Protection Manual* (USACE, 1984) report reflection coefficients for beaches ranging from 0.01 to 0.45. Design beach slopes, measured beach slopes and calculated reflection coefficients during and after construction of the RBSPI were assumed to be similar to what will be expected during (design) and a few months after construction of the Project (post-construction). An example calculation of this backwash is provided for one location, followed by a summary table for other locations within the study area.

To solve the surf similarity parameter in Equation 11-3, the long-term average wave conditions were developed as follows. The Del Mar wave gage (#051) was assumed to be indicative of wave conditions along the study area. This gage is located in 30 feet of water (CDIP, 2011). The average long-term conditions were calculated by averaging the annual average wave conditions for this gage with the significant wave height being 3.0 feet and the peak wave period being 11.8 seconds. The following parameters were calculated using the ACES/CEDAS (Veri-Tech, 2011) software assuming straight and parallel bottom contours: deep water significant wave height is 2.87 feet, deep water wave length is 707 feet, breaking wave height is 5.9 ft (assuming 40:1 slope), breaking wave depth is 6.3 feet, and wavelength at breaking is 166 feet.

The long-term average fall beach profile (Average Fall) and slope are shown in **Figure 11.4-1** for profile location SD-600, which runs through the RBSPI Solana Beach nourishment site and Solana-Segment 2. The average fall profile contains all measured fall profiles, except October 2001. At this location, the average fall beach slope was 27:1 (horizontal:vertical) as shown with a grey line. Also shown in **Figure 11.4-1** are the post-construction beach profile (October 2001) and slope measured after the RBSPI (red line). The RBSPI nourishment at this site ended on June 24, 2001 and the post-construction profile occurred in October of that year, thus there was a four month interval between construction and the post-construction profile measurement. The design beach slope was 10:1 (SANDAG, 2000) and the post-construction beach slope, from **Figure 11.4-1**, was 23:1. The calculated reflection coefficient changed from an average fall value of 0.03, to a design value of 0.15, and a post-construction value of 0.04. In other words the long-term average fall backwash during high tides was approximately 3 percent. This increased to 15 percent during and immediately after construction, and dropped back to 4 percent by the October after construction. These values are summarized in

Table 11.4-1.

As mentioned before the backwash during low tides was expected to be less than normal. As shown in **Figure 11.4-1**, this is evidenced by a milder October 2001 slope extending seaward from MLLW than for the average fall profile. This milder post-construction nearshore slope was also found in other profile locations.

Table 11.4-1 shows beach slopes and reflection coefficients for average fall, design, and post-construction conditions for profiles that occurred at RBSPI nourishment sites and at Encinitas-Segment 1 or Solana-Segment 2. Some profiles did not have measurements for October, 2001 so were not included. Only RBSPI nourish sites showed steep beach slopes after construction. Beach slopes upcoast and downcoast from RBSPI nourish sites remained relatively unchanged by the beach nourishment construction. This is assumed to be the case for the Project as well, so surf sites upcoast and downcoast of the segments are assumed to not be changed in this way. Also listed in

Table 11.4-1 shows the segments, RBSPI receiver sites, and surf sites associated with each profile. RBSPI nourishments at Leucadia and Moonlight occurred in June and August of 2001, respectively and the post-construction slopes were measured in October of that year.

Table 11.4-1 Beach Slopes and Reflection Coefficients

Surf Sites	Profile	RBSPI Site	Segment	Beach Slope			Reflection Coefficient, C_r		
				Avg Fall	Design	Post Const	Avg Fall	Design	Post Const
Ponto to South Beacons	-	Batiquitos	-	-	-	-	-	-	-
North El Portal to Rosetas	SD-675	Leucadia	1	26	10	25	0.03	0.15	0.03
Moonlight, D Street	SD-670	Moonlight	1	26	20	15	0.03	0.05	0.09
Trees to Palies	-	Cardiff	-	-	-	-	-	-	-
Table Tops to Cherry Hill	SD-600	Solana	2	27	10	23	0.03	0.15	0.04
Del Mar, 15th Street	-	-	-	-	-	-	-	-	-

- = not applicable

Since Project beach nourishments are to only occur within the nourishment segments, no change to post-construction backwash is expected at surf sites from Ponto through South Beacons, Trees through Palies, and Del Mar through 15th Street. All the surf sites within Encinitas-Segment 1 and Solana-Segment 2 can expect to have increased backwash during high tide immediately during and after construction due to the increased steepness of the design berm. Changes in high tide, post-construction backwash are expected to be negligible at surf sites from North El Portal to Rosetas. Surf sites near SD-670 such as Moonlight and D Street can expect to have a post-construction, high tide, increase in backwash of approximately 6 percent after each nourishment interval (i.e., the backwash would increase from 3 to 9 percent). Surf sites between Table Tops and Cherry Hill can expect a similar increase in backwash of approximately 1 percent. These post-construction changes are expected to be short lived, lasting one to two months and are expected after each nourishment interval.

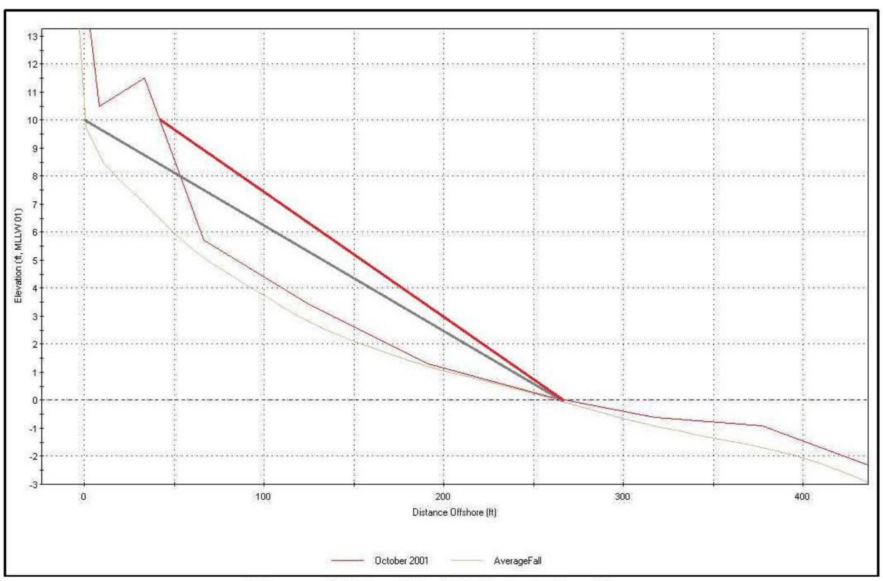


Figure 11-7 SD-600 Beach Profiles

Figure 11.4-1 SD-600 Beach Profiles

Year Two Backwash from Increased D50

This section estimates the backwash from increased D_{50} in the surf zone at year two. It was assumed that changes at year two represent the worst case condition. Changes to surf zone slopes resulting from the Project nourishment were estimated based on changes in D_{50}. Changes to reflection and backwash were then calculated based on these changes to bottom slopes.

Increases in D_{50} within the littoral zone have been documented to steepen equilibrium beach profiles according to the following equation:

$$h=Ay^{2/3} \qquad \text{(Equation 11-4)}$$

Where h is water depth, A is a sediment scale parameter dependent on D_{50}, and y is distance offshore (USACE, 2002). From here, a relation was developed between existing and Project surf zone slopes based on Equations 11-4 and 11-5. The existing bottom slope can be expressed as:

$$m_1 = y/h_1 \qquad \text{(Equation 11-5)}$$

where the subscript 1 indicates existing. The ratio between Project (subscript of 2) and existing bottom slopes can be expressed as:

$$m_2/m_1 = (y/h_2)/(y/h_1) \qquad \text{(Equation 11-6)}$$

And substituting Equation 11-4 into Equation 11-6 yields a slope ratio which is dependent solely on the value of A:

$$m_2/m_1 = A_1/A_2 \qquad \text{(Equation 11-7)}$$

From a review of sediment sampling performed in 2009 for the RBSPII, it was concluded that the existing D_{50} is 0.19 mm. While it is possible that D_{50} will not change appreciably from existing conditions, the most conservative approach is to assume that under Project conditions, the entire study area will have the same large D_{50} as that of the borrow sources. **Table** 11.4-2 contains D_{50} for each borrow site (USACE, 2011), each segment receiving sediment from that borrow site during the beach nourishment, and the surf sites associated with that segment. The following analysis, conservatively assumes that D_{50} will increase to values listed in this table.

The existing long-term average surf zone slope (m_1) was measured from profile data for all profile locations within the study area. This existing surf zone slope is the ratio of horizontal to vertical distances covering the vertical range of 10 ft, MLLW down to the long-term average breaking depth, -5 feet MLLW. The existing reflection coefficient was calculated for these beach slopes with Equation 11-3. Values for A, were read from the *Coastal Engineering Manual* (USACE, 2002). The D_{50}, A, and slope ratios for the existing and Project conditions are summarized in **Table** 11.4-2.

Table 11.4-2 Existing and Project Grain Sizes and Slope Ratios

condition	condition	Location	Segment	Borrow Site	D_{50} (mm)	A ($ft^{1/3}$)	$m_2/m_1=A_1/A_2$
Existing	1	Encinitas & Solana	1 & 2		0.19	0.144	-
Project	2-1	Ponto to Campgrounds	1	SO-6	0.35	0.201	0.717
Project	2-2	Suckouts to 15th Street	2	SO-5	0.59	0.255	0.563

- = not applicable

The idealized existing nearshore profile based solely on D_{50} is shown in **Figure 11.4-1**. With the slope ratios, the Project nearshore profiles can also be calculated as shown in the figure. With the increased D_{50}, the Project slope becomes steeper than the existing slope for both segments. These slope ratios in combination with Equation 11-3 were also used to calculate the Project reflection coefficients for each segment. More detailed estimates are available if the measured surf zone slope is used instead of a slope based solely on grain size. Detailed results of existing slope, existing reflection coefficient, Project reflection coefficient, and change in reflection coefficient (Δ) are shown in **Table 11.4-3**.

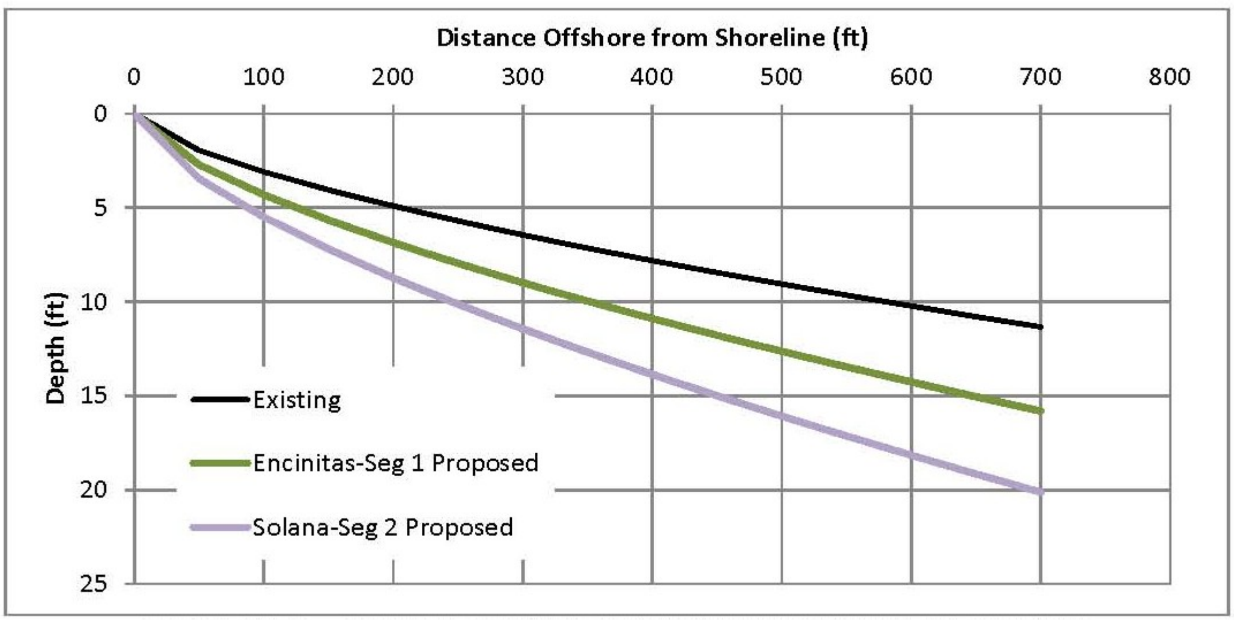


Figure 11.4-2 Idealized Existing and Proposed Nearshore Profiles

Table 11.4-3 Existing and Proposed Reflection Coefficient

Surf Sites	Profile	Segment	Slope, m1	Reflection Coefficient, C_r		
			Exist	Existing	Proposed	Δ
Ponto	CB720		39	0.01	0.03	0.01
Grandview, Avocados	SD695		33	0.02	0.04	0.02
White Fence, Log Cabins	SD690		31	0.02	0.04	0.02
North Beacons to South Beacons	SD680		31	0.02	0.04	0.02
North El Portal to Rosetas	SD675	1	31	0.02	0.04	0.02
Moonlight, D Street	SD670	1	33	0.02	0.04	0.02
Trees to Dabbers	SD660		25	0.03	0.06	0.03
Brown House to Campgrounds	SD650		31	0.02	0.04	0.02
Suckouts to Pallies	SD630		32	0.02	0.06	0.04
Table Tops	SD610	2	28	0.03	0.08	0.05
Pillbox to South Side	SD600	2	33	0.02	0.06	0.04
Cherry Hill	SD595	2	34	0.02	0.06	0.04
Del Mar	DM580		36	0.02	0.05	0.03
15th Street	DM560		32	0.02	0.06	0.04
Mean			32	0.02	0.05	0.03
Median			32	0.02	0.05	0.02
Maximum			39	0.03	0.08	0.05
Minimum			25	0.01	0.03	0.01

As explained earlier, a reflection coefficient, in surfing terms, is used as a proxy for the amount of backwash expected. The change in reflection coefficients in **Table 11.4-3** ranged from 0.01 to 0.05. Stated differently, assuming grain sizes increase, from 1 to 5 percent more of the incident wave is expected to backwash under the Project condition. Changes in backwash at each surf site are expected to be between zero (assuming no change in grain size) and the conservatively calculated changes at the nearest profile shown in **Table 11.4-3**. For example, the year two backwash at Pallies as a result of increasing D_{50} is expected to change from an existing 2 percent to somewhere between 2 and 6 percent, with an associated increase of up to 4 percent over existing conditions.

<u>Long-Term Backwash from Sea Level Rise</u>

The approach to addressing sea level rise within the Project was to quantitatively include sea level rise changes on Project conditions and qualitatively address sea level rise changes on without Project conditions. It is believed that quantifying future without Project beach profiles is too speculative to be useful. So, long-term backwash from sea level rise is addressed qualitatively here.

The without Project beach profiles can be expected to adjust for sea level rise according to the Bruun Rule as discussed in **Chapter 7.8.3** of this report. Based on Equation 7-2, if shoreline recession is impeded with a bluff or seawall and no volume is added, the effective result is that the profile lowers, relative to the water level, a distance equal to the sea level rise.

For the bluff backed beaches within the study area, substantial shoreline recession is not possible beyond the bluff toe and there are currently many locations within the study area that have no beach during high tide. For locations within the study area, as the sea level rises, the amount of time without a beach between the bluff and water will increase and the amount of time water is in contact with the highly reflective bluff will increase. Reflection coefficients for vertical walls similar to the bluff range from 0.7 to 1.0 (Goda, 2000). Eventually, for the without Project condition, with sea level rise, reflection and backwash are expected to increase significantly. A good example of what to expect can be found at the nearby Sunset Cliffs, as shown in **Figure 11.4-3**, where there is typically no beach and waves reflect off the cliffs regularly during high tide. As stated by one of the locals on Wannasurf.com, “Getting in and out at a low tide is not hard. Higher tide, big day? Better not surf here unless you are a really strong swimmer. Getting out of the water is challenging.”

Figure 11.4-3 Sunset Cliffs (Source:californiabeachhike.com)

As explained in **Chapter 7** of this report, the beach nourishment options were modeled assuming sufficient sand would be added to the segments to keep pace with sea level rise. Beaches would be maintained in those segments with the Project and reflection would be as calculated in **Table 11.4-3**. These reflection coefficients which range from 0.03 to 0.08 are significantly less than the eventual without Project coefficients of 0.7 to 1.0. Thus under the Project conditions, bluff induced backwash would be significantly less than the without Project condition.

<u>Backwash Summary</u>

Currently, approximately 3 percent of the fall high tide waves backwash. At high tides during construction, backwash resulting from the steep slope of the constructed beach berm can be expected to increase to 15 percent. These changes are expected to be short lived, on the order of a few weeks. Post-construction at high tides, backwash can be expected to increase to up to 9 percent at some of the surf sites within the segments. This increased reflection is expected to last less than a month or two. Low tide backwash during construction and post-construction is expected to be less than currently exist.

Long-term backwash resulting from a potential increase in D_{50} is expected to stay the same or increase. These increases are generally expected to be on the order of 3 percent over existing conditions, with the maximum Project backwash of up to 8 percent at Table Tops.

Sea level rise induced backwash for the without Project condition is expected to increase significantly as a result of wave reflection off vertical bluffs and seawalls. The Project with sea level rise is expected to decrease backwash as compared to the without Project condition since the nourishment is expected to maintain mildly sloping beaches.
Overall, assuming sea level rise does occur, the Project is expected to eventually reduce backwash as compared to the without Project condition.

11.4.2 Breaking Intensity for Beach Breaks

Changes to wave breaking intensity at beach breaks are analyzed below using the surf similarity parameter and the vortex ratio. Some basic assumptions for these beach break analyses include:

- Peel angles and section lengths for beach breaks are variable, primarily depending on wave conditions and are not expected to change between existing and Project conditions.
- The historical average significant wave height and average peak period are representative of typical conditions expected during the Project duration.
- Wave conditions from the Del Mar wave gage are sufficiently representative of the entire study area. Wave conditions from this gage were described previously, under the Post-Construction Backwash section of this report.
- Historically surveyed profiles can sufficiently represent nearby beach break profiles. For example, it was assumed that Avocados is represented by Profile SD-700 which is located to the north of the surf site.
- Surf sites that are classified as “reef-beach break” typically have reefs located farther offshore that break during larger swells and the remaining time the surf site breaks like a beach break. Thus, the following analysis is valid for the beach break portion of reef-beach break surf sites.

For both the surf similarity parameter approach and the vortex ratio, seabed slope was calculated for the average spring condition ($m_{bspring}$) and average fall condition (m_{bfall}) at the point of wave breaking, extending one wavelength offshore. As discussed above, the Project grain sizes are expected to stay the same or increase over existing conditions. The following analyses is for the conservative assumption that grain sizes increase, thus steepening the seabed slopes per Equation 11-7.

Surf Similarity

The existing and Project seabed slopes were used to calculate the existing (1) and Project (2), spring and fall surf similarity parameters ($\xi_{o1spring}$, ξ_{o1fall} $\xi_{o2spring}$, ξ_{o2fall}) using Equation 11-1. As described earlier, ξ_o<0.5 indicates spilling waves and 0.5≤ ξ_o <3.3 indicates plunging waves. Measured existing spring and fall slopes and calculated spring and fall, existing and Project surf similarity parameters are summarized in **Table 11.4-4**.

Table 11.4-4 Surf Similarity Parameters for Profiles and Nearby Beach Breaks

Surf Site	Profile	Existing				Proposed	
		$m_{bspring}$	m_{bfall}	$\xi_{o1spring}$	ξ_{o1fall}	$\xi_{o2spring}$	ξ_{o2fall}
Ponto	CB720	43	40	0.37	0.40	0.52	0.55
Grandview, Avocados	SD700	40	26	0.40	0.60	0.55	0.83
White Fence, Log Cabins	SD690	54	64	0.29	0.25	0.41	0.34
North Beacons to South Beacons	SD680	57	75	0.27	0.21	0.38	0.29
North El Portal to Rosetas	SD675	34	24	0.46	0.66	0.65	0.92
Moonlight, D Street	SD670	98	28	0.16	0.57	0.22	0.79
Dabbers	SD660	32	40	0.49	0.39	0.69	0.54
Brown House to Campgrounds	SD650	59	61	0.26	0.26	0.37	0.36
Evans to Pallies	SD630	54	28	0.29	0.56	0.52	0.99
Pillbox to South Side	SD600	54	33	0.29	0.48	0.52	0.86
Cherry Hill	SD595	75	33	0.21	0.48	0.37	0.86
Del Mar	DM580	111	33	0.14	0.47	0.25	0.84
15th Street	DM560	79	40	0.20	0.40	0.35	0.70
Minimum				0.14	0.21	0.22	0.29
Maximum				0.49	0.66	0.69	0.99
Percent Spilling				100%	69%	54%	23%
Percent Plunging				0	31%	46%	77%

All of the existing spring beach breaks have spilling waves (0 percent plunging) and 69 percent of the fall beach breaks have spilling waves (31 percent plunging). Under Project conditions the amount of plunging beach breaks increases to 46 percent during the spring and 77 percent during the fall. In all cases the intensity of the plunging is on the low end of the plunging scale, with the maximum surf similarity parameter being 0.99 at Profile SD-630, near the Georges beach break. None of the beach breaks are expected to become surging under Project conditions. Under Project conditions, the breaking waves are expected to either not change (assuming no change in grain size) or become more hollow at locations where there is a Project influence.

Whether or not this is an improvement over existing conditions is a matter of perspective, with short boarders likely appreciating the change and longboarders disliking it.

<u>Vortex Ratio</u>

A similar exercise was performed for the vortex ratio (Equation 11-2) with results presented in **Many of** the reefs in the study area neither break like pure reefs nor pure beach breaks, but rather somewhere on a graded scale between the two. Where on that scale depends on the time of year, breaking wave height, swell combination, swell direction, sand coverage, tide, and surfer perception. For example, Bamboos is mostly a beach break, but during large winter swells can break more like a reef break either due to waves refracting and breaking over the reef or from waves refracting over the reef and breaking over the sandy beach. Changes in sand elevation can change the extent to which any reef behaves like a reef break, whether or not the reef is entirely covered, partially covered, or just lowered in contrast with the surrounding sandy seafloor. Raising the sandy seafloor surrounding a reef reduces the elevation contrast (relief) between the reef and sandy seafloor. This results in less refraction at the reef and less definition to the surf site. So any change in the sand thickness surrounding a reef could potentially change how that surf site breaks.

Table 11.4-5. This method is less applicable here since the upper limit on vortex ratio is 3.4 and many surf sites within the study area have vortex ratios higher than that, meaning the surf sites break less intensely than the valid range of the vortex ratio. Where vortex ratios are above 3.4 the method is not supported, and the wave is assumed to be spilling.

Where the method is applicable, the Project vortex ratios are uniformly lower than the existing vortex ratios. Where valid, the breaking intensities associated with vortex ratios have gone from medium under existing conditions to high and even very high under Project conditions. The lowest vortex ratio (highest breaking intensity) is expected to occur at beach breaks near SD-675 and SD-630. This means that the waves are expected to either not change or become hollow at locations where there is a Project influence.

As with the surf similarity parameter, whether or not these changes are an improvement is a matter of perspective.

11.4.3 Sedimentation Changes to Reef Breaks

Adding sand to reef breaks has the potential to make them behave more like beach breaks so reef breaks are analyzed in a different way than above. Beach breaks are not included in this analysis since adding more sand on top of beach breaks does not change them from beach breaks. The most common surfing change expected as a result of changing a reef breaks in the study area to more beach break like conditions would be reduced peel angles, section lengths, and surfability, especially during larger swells.

Many of the reefs in the study area neither break like pure reefs nor pure beach breaks, but rather somewhere on a graded scale between the two. Where on that scale depends on the time of year, breaking wave height, swell combination, swell direction, sand coverage, tide, and surfer perception. For example, Bamboos is mostly a beach break, but during large winter swells can break more like a reef break either due to waves refracting and breaking over the reef or from waves refracting over the reef and breaking over the sandy beach. Changes in sand elevation can change the extent to which any reef behaves like a reef break, whether or not the reef is entirely covered, partially covered, or just lowered in contrast with the surrounding sandy seafloor. Raising the sandy seafloor surrounding a reef reduces the elevation contrast (relief) between the reef and sandy seafloor. This results in less refraction at the reef and less definition to the surf site. So any change in the sand thickness surrounding a reef could potentially change how that surf site breaks.

Table 11.4-5 Vortex Ratios for Profiles and Nearby Beach breaks

Surf Site	Profile	Existing				Proposed	
		$m_{bspring}$	m_{bfall}	$L/W_{1spring}$	L/W_{1fall}	$L/W_{2spring}$	L/W_{2fall}
Ponto	CB720	43	40	3.6	3.4	2.8	2.7
Grandview, Avocados	SD700	40	26	3.4	2.5	2.7	2.0
White Fence, Log Cabins	SD690	54	64	4.3	5.0	3.3	3.8
North Beacons to South Beacons	SD680	57.2	75.5	4.5	5.7	3.5	4.3
North El Portal to Rosetas	SD675	33.9	23.7	3.0	2.4	2.4	1.9
Moonlight, D Street	SD670	97.6	27.7	7.2	2.6	5.4	2.1
Dabbers	SD660	31.9	40.5	2.9	3.5	2.3	2.7
Brown House to Cardiff Reef	SD650	59.3	61.5	4.7	4.8	3.6	3.7
Evans to Pallies	SD630	53.5	28.1	4.3	2.6	2.8	1.9
Pillbox to South Side	SD600	53.5	32.5	4.3	2.9	2.8	2.0
Cherry Hill	SD595	75.5	32.5	5.7	2.9	3.6	2.0
Del Mar	DM580	110.7	33.2	8.0	3.0	4.9	2.0
15th Street	DM560	79.0	39.5	6.0	3.4	3.7	2.3
Minimum				2.9	2.4	2.3	1.9
Maximum				8.0	5.7	5.4	4.3
Not Valid (% Spilling)				77%	31%	46%	23%
Percent Plunging				23%	69%	54%	77%

There are at least three ways to analyze Project induced changes to these reef surf sites described as follows:

1. Detailed wave modeling would require multiple sets of bathymetric data, wave data, and surf observations, ideally measured while the surf sites were behaving like beach breaks and while they were behaving like reef breaks. This would allow for development of a graded scale upon which the sand thickness changes could be applied to determine extent of change. However, this level of data does not exist.
2. Lacking this data, numerical modeling could be performed driven by one bathymetric data set and a broad group of assumptions about how and when the surf site behaves in different ways and what bathymetric and wave conditions drive those breaks. Due to the assumptions, the level of confidence for this type of analysis would be low.
3. A conservative, subjective scale based on quantitative data could be developed to compare Project induced changes in profile volumes to the natural variability of the profile volumes. Profile volumes are used as a simple proxy for more detailed analysis of variable cross shore sand thickness (for which there is no quantitative guidance either). This approach was chosen for the current analysis.

Three key variables were developed to carry out this third approach: 1) the year 2 increase in profile volume resulting from beach nourishment, 2) the increased profile volume resulting from offsetting the sea level rise quantity, and 3) the standard deviation of the historical profile volume changes. These variables and their comparison are described in detail below.

The GENESIS predicted changes in beach widths (ΔBW) at the model cell nearest to each reef break were converted to changes in profile volumes (V_{BW}) using the v/s ratios described in **Chapter 8** of this report. As previously defined, the separation between Encinitas and Solana Beach occurs at San Elijo Lagoon. Thus, the changes from the Encinitas-Segment 1 beach nourishment was assumed to extend from Ponto through Campgrounds and the Solana-Segment 2 change extends from Suckouts to 15th Street. Values were calculated for various combinations of segment, beach nourishment option, and sea level rise scenario, as detailed in **Table** 11.4-6.

Table 11.4-6 Matrix of Reef Change Variable Combinations

Segment	BNO = Beach Nourishment Option (feet)	SLR = Sea Level Rise Scenario	RI = Replenishment Interval (years)	Plan
Encinitas-Segment 1	100	Low	5	NED & LPP
Encinitas-Segment 1	100	High	5	NED & LPP
Encinitas-Segment 1	100	Low	10	Hybrid
Encinitas-Segment 1	100	Low	10	Hybrid
Solana-Segment 2	200	Low	13	NED
Solana-Segment 2	300	Intermediate	15	NED
Solana-Segment 2	300	High	14	NED
Solana-Segment 2	200	Low	10	LPP

Solana-Segment 2	200	High	10	LPP
Solana-Segment 2	150	Low	10	Hybrid
Solana-Segment 2	150	High	10	Hybrid

NED=National Economic Development Plan, LPP=Locally Preferred Plan, Hybrid = Hybrid Plan

As described in **Section 0** of this report, sea level rise quantities were assumed to be placed at the two segments to offset various sea level rise scenarios. Sea level rise quantities can be read from **Table 7.8-1** for various replenishment intervals. For example, at Encinitas-Segment 1 with a low sea level rise scenario and a 5 year replenishment interval, the sea level rise quantity from **Table 7.8-1** would be 15,699 yd^3. Dividing this quantity by the segment length yields the sea level rise profile volume (V_{SLR}). The sea level rise quantities would be added to each segment during the initial beach nourishment and are assumed to remain within their respective segment through year 2. The sea level rise quantity is assumed to only change at the nourishment segments and not change reefs outside the nourishment segments.

These Project induced profile volumes were added to create a total profile volume according to the following equation:

$$V_T = V_{BW} + V_{SLR} \qquad \text{(Equation 11-8).}$$

The total profile volume was compared to the standard deviation of measured profile volumes nearest to each reef beak (STDEV). The average historical profile volumes nearest to each reef break (V_H) are also shown in **Table 11.4-7** for additional comparison.

For the current study, the assumed threshold for measurable reef change is an increase in profile volume over the standard deviation expressed as:

$$\underline{V_T} = \begin{cases} < STDEV, & \text{Measurable Reef Change} = \text{not likely} \\ \geq \text{STDEV}, & \text{Measurable Reef Change} = \text{likely} \end{cases} \qquad \text{(Equation 11-9)}$$

Table 11.4-7 shows results for all the alternatives.

Table 11.4-7 Changes to Reef Breaks

National Economic Development (NED) Plan												
Surf Site	**Profile**	**V_H yd^3/ft**	**STDEV yd^3/ft**	**BNO feet**	**ΔBW feet**	**V_{BW} yd^3/ft**	**RI year**	**SLR**	**V_{SLR} yd^3/ft**	**V_T yd^3/ft**	**Measurable Reef Change**
Grandview	SD700	68	24	100	0	0.0	N/A	N/A	0	0.0	not likely
North Beacons	SD680	108	36	100	9	7.7	N/A	N/A	0	7.7	not likely
Bamboos	SD680	108	36	100	7	6.0	N/A	N/A	0	6.0	not likely
South Beacons	SD680	108	36	100	5	4.1	N/A	N/A	0	4.1	not likely
Stone Steps	SD675	60	19	100	126	108.9	5	low	2.01	110.9	likely
Trees	SD660	67	15	100	0	0.1	N/A	N/A	0	0.1	not likely
Boneyards	SD660	67	15	100	0	0.0	N/A	N/A	0	0.0	not likely
Swamis	SD660	67	15	100	0	0.0	N/A	N/A	0	0.0	not likely
Pipes	SD650	73	15	100	0	0.0	N/A	N/A	0	0.0	not likely

Traps	SD650	73	15	100	0	0.0	N/A	N/A	0	0.0	not likely
Turtles	SD650	73	15	100	0	0.0	N/A	N/A	0	0.0	not likely
85/60s	SD630	149	64	100	0	0.0	N/A	N/A	0	0.0	not likely
Tippers	SD630	149	64	100	0	0.0	N/A	N/A	0	0.0	not likely
Campgrounds	SD630	149	64	100	0	0.0	N/A	N/A	0	0.0	not likely
Suckouts	SD630	149	64	300	14	9.7	N/A	N/A	0	9.7	not likely
Cardiff Reef	SD630	149	64	300	14	10.2	N/A	N/A	0	10.2	not likely
Seaside Reef	SD630	149	64	300	21	14.9	N/A	N/A	0	14.9	not likely
Pallies	SD630	149	64	300	5	3.7	N/A	N/A	0	3.7	not likely
Table Tops	SD610	50	24	300	168	119.6	14	low	5.63	125.2	likely
Pillbox	SD600	65	16	300	358	255.1	14	low	5.63	260.7	likely
South Side	SD600	65	16	300	311	221.4	14	low	5.63	227.1	likely
15th Street	DM560	90	30	300	0	0.0	N/A	N/A	0	0.0	not likely

National Economic Development (NED) Plan

Surf Site	Profile	V_H yd^3/ft	STDEV yd^3/ft	BNO feet	ΔBW feet	V_{BW} yd^3/ft	RI year	SLR	V_{SLR} yd^3/ft	V_T yd^3/ft	Measurable Reef Change
Grandview	SD700	68	24	100	0	0.0	N/A	N/A	0	0.0	not likely
North Beacons	SD680	108	36	100	9	7.7	N/A	N/A	0	7.7	not likely
Bamboos	SD680	108	36	100	7	6.0	N/A	N/A	0	6.0	not likely
South Beacons	SD680	108	36	100	5	4.1	N/A	N/A	0	4.1	not likely
Stone Steps	SD675	60	19	100	126	108.9	5	high	8.17	117.1	likely
Trees	SD660	67	15	100	0	0.1	N/A	N/A	0	0.1	not likely
Boneyards	SD660	67	15	100	0	0.0	N/A	N/A	0	0.0	not likely
Swamis	SD660	67	15	100	0	0.0	N/A	N/A	0	0.0	not likely
Pipes	SD650	73	15	100	0	0.0	N/A	N/A	0	0.0	not likely
Traps	SD650	73	15	100	0	0.0	N/A	N/A	0	0.0	not likely
Turtles	SD650	73	15	100	0	0.0	N/A	N/A	0	0.0	not likely
85/60s	SD630	149	64	100	0	0.0	N/A	N/A	0	0.0	not likely
Tippers	SD630	149	64	100	0	0.0	N/A	N/A	0	0.0	not likely
Campgrounds	SD630	149	64	100	0	0.0	N/A	N/A	0	0.0	not likely
Suckouts	SD630	149	64	300	14	9.7	N/A	N/A	0	9.7	not likely
Cardiff Reef	SD630	149	64	300	14	10.2	N/A	N/A	0	10.2	not likely
Seaside Reef	SD630	149	64	300	21	14.9	N/A	N/A	0	14.9	not likely
Pallies	SD630	149	64	300	5	3.7	N/A	N/A	0	3.7	not likely
Table Tops	SD610	50	24	300	168	119.6	16	high	29.57	149.2	likely
Pillbox	SD600	65	16	300	358	255.1	16	high	29.57	284.6	likely
South Side	SD600	65	16	300	311	221.4	16	high	29.57	251.0	likely
15th Street	DM560	90	30	300	0	0.0	N/A	N/A	0	0.0	not likely

National Economic Development (NED) Plan

Surf Site	Profile	V_H yd^3/ft	STDEV yd^3/ft	BNO feet	ΔBW feet	V_{BW} yd^3/ft	RI year	SLR	V_{SLR} yd^3/ft	V_T yd^3/ft	Measurable Reef Change
Grandview	SD700	68	24	100	0	0.0	N/A	N/A	0	0.0	not likely
North Beacons	SD680	108	36	100	9	7.7	N/A	N/A	0	7.7	not likely
Bamboos	SD680	108	36	100	7	6.0	N/A	N/A	0	6.0	not likely
South Beacons	SD680	108	36	100	5	4.1	N/A	N/A	0	4.1	not likely
Stone Steps	SD675	60	19	100	126	108.9	5	high	8.17	117.1	likely
Trees	SD660	67	15	100	0	0.1	N/A	N/A	0	0.1	not likely
Boneyards	SD660	67	15	100	0	0.0	N/A	N/A	0	0.0	not likely
Swamis	SD660	67	15	100	0	0.0	N/A	N/A	0	0.0	not likely
Pipes	SD650	73	15	100	0	0.0	N/A	N/A	0	0.0	not likely
Traps	SD650	73	15	100	0	0.0	N/A	N/A	0	0.0	not likely
Turtles	SD650	73	15	100	0	0.0	N/A	N/A	0	0.0	not likely
85/60s	SD630	149	64	100	0	0.0	N/A	N/A	0	0.0	not likely
Tippers	SD630	149	64	100	0	0.0	N/A	N/A	0	0.0	not likely
Campgrounds	SD630	149	64	100	0	0.0	N/A	N/A	0	0.0	not likely
Suckouts	SD630	149	64	300	14	9.7	N/A	N/A	0	9.7	not likely
Cardiff Reef	SD630	149	64	300	14	10.2	N/A	N/A	0	10.2	not likely
Seaside Reef	SD630	149	64	300	21	14.9	N/A	N/A	0	14.9	not likely
Pallies	SD630	149	64	300	5	3.7	N/A	N/A	0	3.7	not likely
Table Tops	SD610	50	24	300	168	119.6	15	inter	11.06	130.7	likely
Pillbox	SD600	65	16	300	358	255.1	15	inter	11.06	266.1	likely
South Side	SD600	65	16	300	311	221.4	15	inter	11.06	232.5	likely
15th Street	DM560	90	30	300	0	0.0	N/A	N/A	0	0.0	not likely

Hybrid Plan

Surf Site	Profile	V_H yd^3/ft	STDEV yd^3/ft	BNO feet	ΔBW feet	V_{BW} yd^3/ft	RI year	SLR	V_{SLR} yd^3/ft	V_T yd^3/ft	Measurable Reef Change
Grandview	SD700	68	24	100	0	0.0	N/A	N/A	0	0.0	not likely
North Beacons	SD680	108	36	100	9	7.7	N/A	N/A	0	7.7	not likely
Bamboos	SD680	108	36	100	7	6.0	N/A	N/A	0	6.0	not likely
South Beacons	SD680	108	36	100	5	4.1	N/A	N/A	0	4.1	not likely
Stone Steps	SD675	60	19	100	126	108.9	10	low	4.02	112.9	likely
Trees	SD660	67	15	100	0	0.1	N/A	N/A	0	0.1	not likely
Boneyards	SD660	67	15	100	0	0.0	N/A	N/A	0	0.0	not likely
Swamis	SD660	67	15	100	0	0.0	N/A	N/A	0	0.0	not likely
Pipes	SD650	73	15	100	0	0.0	N/A	N/A	0	0.0	not likely
Traps	SD650	73	15	100	0	0.0	N/A	N/A	0	0.0	not likely
Turtles	SD650	73	15	100	0	0.0	N/A	N/A	0	0.0	not likely
85/60s	SD630	149	64	100	0	0.0	N/A	N/A	0	0.0	not likely

Tippers	SD630	149	64	100	0	0.0	N/A	N/A	0	0.0	not likely
Campgrounds	SD630	149	64	100	0	0.0	N/A	N/A	0	0.0	not likely
Suckouts	SD630	149	64	150	4	2.9	N/A	N/A	0	2.9	not likely
Cardiff Reef	SD630	149	64	150	4	2.6	N/A	N/A	0	2.6	not likely
Seaside Reef	SD630	149	64	150	9	6.6	N/A	N/A	0	6.6	not likely
Pallies	SD630	149	64	150	1	0.9	N/A	N/A	0	0.9	not likely
Table Tops	SD610	50	24	150	50	35.6	10	low	4.02	39.6	likely
Pillbox	SD600	65	16	150	213	151.8	10	low	4.02	155.8	likely
South Side	SD600	65	16	150	166	118.7	10	low	4.02	122.7	likely
15th Street	DM560	90	30	150	0	0.0	N/A	N/A	0	0.0	not likely

Hybrid Plan

Surf Site	Profile	V_H yd^3/ft	STDEV yd^3/ft	BNO feet	ΔBW feet	V_{BW} yd^3/ft	RI year	SLR	V_{SLR} yd^3/ft	V_T yd^3/ft	Measurable Reef Change
Grandview	SD700	68	24	100	0	0.0	N/A	N/A	0	0.0	not likely
North Beacons	SD680	108	36	100	9	7.7	N/A	N/A	0	7.7	not likely
Bamboos	SD680	108	36	100	7	6.0	N/A	N/A	0	6.0	not likely
South Beacons	SD680	108	36	100	5	4.1	N/A	N/A	0	4.1	not likely
Stone Steps	SD675	60	19	100	126	108.9	10	high	17.3	126.2	likely
Trees	SD660	67	15	100	0	0.1	N/A	N/A	0	0.1	not likely
Boneyards	SD660	67	15	100	0	0.0	N/A	N/A	0	0.0	not likely
Swamis	SD660	67	15	100	0	0.0	N/A	N/A	0	0.0	not likely
Pipes	SD650	73	15	100	0	0.0	N/A	N/A	0	0.0	not likely
Traps	SD650	73	15	100	0	0.0	N/A	N/A	0	0.0	not likely
Turtles	SD650	73	15	100	0	0.0	N/A	N/A	0	0.0	not likely
85/60s	SD630	149	64	100	0	0.0	N/A	N/A	0	0.0	not likely
Tippers	SD630	149	64	100	0	0.0	N/A	N/A	0	0.0	not likely
Campgrounds	SD630	149	64	100	0	0.0	N/A	N/A	0	0.0	not likely
Suckouts	SD630	149	64	150	4	2.9	N/A	N/A	0	2.9	not likely
Cardiff Reef	SD630	149	64	150	4	2.6	N/A	N/A	0	2.6	not likely
Seaside Reef	SD630	149	64	150	9	6.6	N/A	N/A	0	6.6	not likely
Pallies	SD630	149	64	150	1	0.9	N/A	N/A	0	0.9	not likely
Table Tops	SD610	50	24	150	50	35.6	10	high	17.31	52.9	likely
Pillbox	SD600	65	16	150	213	151.8	10	high	17.31	169.1	likely
South Side	SD600	65	16	150	166	118.7	10	high	17.31	136.0	likely
15th Street	DM560	90	30	150	0	0.0	N/A	N/A	0	0.0	not likely

In general, the wider the beach nourishment option, and the greater the assumed sea level rise scenario, the more likely the Project will have a measurable change on the reef break. Through this analysis, it was found that reef changes are equal between alternatives. Thus, the narrative descriptions below are applicable to reef changes for all Project alternative listed in **Table 11.4-6.**

Grandview

Grandview is a typical reef-beach break in which the surf site is a nearshore beach break most of the time, and either breaks over the reef or focuses waves over an offshore reef during larger swell. Reef features are shown in the aerial image of **Figure B9-4-1**. Most of the beach break surfing at Grandview takes place from 300 to 800 feet from shore, in water depth shallower than 10 feet below MLLW. For example, **Figure B9-4-2** shows surfers in the lineup about 700 feet from shore. Profile SD-700 runs directly through Grandview. The year two, Project induced net change in profile volume under all alternatives analyzed are less than the profile volume standard deviation, so Project induced changes to surfing at this reef are not likely.

Beacons

North Beacons, Bamboos, and South Beacons have reefs that break on larger swells. The surf sites are not as clearly defined as a pure reef breaks since they are generally low relief reefs. Peaks are shifty, similar to a beach breaks, but there may be some reef focusing effect from the subtle variation in bottom contours. Therefore, these are characterized as reef-beach breaks. Bottom contours are generally parallel to shore as shown in **Figure B9-4-3**, but a reef can seen beginning approximately 600 feet from shore and extending to deeper water in **Figure B9-4-4.** Most of the surfing takes place at Beacons from 300 to 700 feet from the profile origin. An example is shown in the aerial photograph of **Figure B9-4-5**. Larger swell can break in 15 feet of water, 1000 feet from shore. The nearest profile to North Beacons is SD-680. The year two, Project induced net change in profile volume under all alternatives analyzed are less than the profile volume standard deviation, so Project induced changes to surfing at this reef are not likely.

Stone Steps

There are conflicting reports on whether Stone Steps is a reef or beach break. WannaSurf.com and Surf-Forecast.com state that it is beach break, but with specific break locations during large swells. It is likely that this is a typical reef-beach break with rights and lefts. From the bathymetric contours, shown in **Figures B9-4-6** it seems that whatever reef does exist is low relief. The surf site is not as clearly defined as a classical reef break since it is generally low relief. Peaks are more shifty, similar to a beach break, but there may be some reef focusing effect from the subtle variation in bottom contours. Bottom contours are mostly straight and parallel. The nearest profile is SD-675.

The total profile volume is greater than the profile volume standard deviation, so measurable Project induced changes to surfing at this reef are likely. Thus, this surf site would be expected to behave more like a beach break under the alternatives analyzed. As reefs change to more like beach breaks, the reef effect is expected to be reduced as it becomes buried by sand. For beginning surfers, who generally go straight towards shore and do not take advantage of the peeling breakers along reefs, there would be very little change to their surfing experience at Stone Steps. For other surfers, the change would likely result in reduced peel angles, more closeouts, reduced section lengths, shorter rides, and reduced surfability.

Trees

Trees is generally described as a reef break. The year two, Project induced net change in profile volume under all alternatives analyzed are less than the profile volume standard deviation, so Project induced changes to surfing at this reef are not likely.

Swamis and Boneyards

Swamis is the premier surf site within the project domain. The wave peels right over a bedrock reef for up to ¼ mile during large swell. The outside reef is known as Boneyards and only breaks during the largest west swells. During smaller days, a few lefts can be found. The breaking intensity is normally semi-hollow but can be mushy during south swells and during higher tides (Cleary and Stern, 1998). Since this is a well defined reef break, with waves breaking near the same location with regularity, it is possible to determine the peel angle and ride length. An analysis of four aerial photographs spanning 2003 through 2009 revealed peel angles ranging from 52 to 65 degrees with the median being 53 degrees and ride lengths from 170 to 980 feet. The peel line and wave crests are shown in **Figure B9-4-7** for a long period west swell occurring on January 3, 2006. Surfers can be seen floating just to the south and west of the whitewash. Typical of shallow areas with broken waves, the LiDAR measured elevation contours (blue lines in **Figure B9-4-7**) reveal no data over the reef and in the surf zone, so detailed wave transformation is not possible here. The deep water wave energy polar spectral plot is provided by CDIP (2011) at the 100 Torrey Pines gage for the condition shown in the figure. The year two, Project induced net change in profile volume under all alternatives analyzed are less than the profile volume standard deviation, so Project induced changes to surfing at this reef are not likely.

Pipes, Traps, and Turtles

Pipes is mostly a reef break while Traps and Turtles are more reef-beach breaks. The bathymetric contours shown in **Figure B9-4-8** show some reef like features at these sites. The year two, Project induced net change in profile volume under all alternatives analyzed are less than the profile volume standard deviation, so Project induced changes to surfing at these reefs are not likely.

85/60s, Tippers, and Campgrounds

85/60s, Tippers, and Campgrounds are typical North County reef-beach breaks and are best represented by profile SD-630. The bathymetric contours for these surf sites, shown in **Figure B9-4-9**, shows mainly low relief reefs. The year two, Project induced net change in profile volume under all alternatives analyzed are less than the profile volume standard deviation, so Project induced changes to surfing at these reefs are not likely.

Suckouts through Pallies

Suckouts, Seaside Reef, Cardiff Reef, and Pallies are all reef breaks and are best represented by profil**e SD-630.** Bottom contours for these reefs are relatively prominent as shown in **Figure B9-4-10**. The reefs extend approximately 300 to 1000 feet from the back beach and surfing takes place approximately in this range as well. The year two, Project induced net change in profile volume under all alternatives analyzed are less than the profile volume standard deviation, so Project induced changes to surfing at these reefs are not likely.

Table Tops

Table Tops is a hollow right reef break and is best represented by profile SD-610. Bottom contours for this reef are relatively prominent as shown in **Figure B9-4-11**. The total profile volume is greater than the profile volume standard deviation, so measurable reef changes are likely. If this surf site were measurably changed to more like a reef-beach break, it is expected that the reef exposure above the sandy bottom would become less pronounced and the break would become somewhat less hollow, with lower breaker intensities. This could be considered an improvement for intermediate surfers, but would likely be a detriment to more advanced surfers. If the sand thickness were further increased, the reef could become completely buried, changing the surf site to a beach break. If this were to occur, the rather unique albeit fickle nature of this surf site would be lost, changing it to yet another beach break. Since this is currently an advanced surf site and it is far from shore, beginning surfers are not likely to attempt this surf site and would not experience any change to their surfing experience. For other surfers however this would likely result in more closeouts, shorter rides, and reduced surfability.

<u>Pillbox & Southside</u>

Pillbox is a right-peeling reef-beach break and the surf spot called Southside is a left-peeling reef-beach break. These surf sites are best represented by profile SD-600. Bottom contours for these surf sites are relatively smooth and parallel profile as shown in **Figure B9-4-11**. The total profile volume is greater than the profile volume standard deviation, so measurable reef changes are likely. With the added sand these two surf sites would become more like beach breaks, reducing their reef tendencies. Beginning surfers would not likely experience any change to their surfing experience, but for other surfers this would result in more closeouts, shorter rides, and less surfability.

<u>15th Street</u>

The surf site at 15th Street is a combination reef-beach break best represented by profile DM-560. The year two, Project induced net change in profile volume under all alternatives analyzed are less than the profile volume standard deviation, so Project induced changes to surfing at this reef are not likely.

11.4.4 Currents at Surf Sites

Ocean currents can change surfing by changing a surfer's ability to line up for and catch a wave and by changing the way waves break. The most frequent currents around these North County surf sites are rip currents and ebb and flood tidal currents associated with the various lagoon mouths. Some currents can also be expected near high relief reefs. All of these currents are expected to be highly variable, changing with swell, tide, and wind conditions.

As beaches widen with the Project alternatives, the break point of the surf sites are expected to move proportional distances seaward, bringing with them the various currents that exist under normal without Project conditions. These currents are not expected to change in magnitude or direction, but only relocate seaward. Therefore, the Project is not expected to measurably change currents or change surfing in any discernible way through changes to currents.

11.4.5 Changes to Surf Break Location and Surfing Frequency

As with ocean currents, the location of the break point of surf sites are expected to move seaward distances that are proportional to the amount of beach widening. For example, if a beach is expected to widen by 100 feet, it can be expected that the beach break fronting that shoreline would move a similar distance seaward, maintaining an unchanged distance between the break point and the shoreline. The primary change to surfing locations is that they would move seaward relative to geographic coordinates, but not change perceptibly relative to the shoreline.

With only minor changes to the surf zone seabed slope, most waves at beach breaks that would have been surfable prior to Project implementation would still likely be surfable under the Project condition. The above described changes to surfing quality can change the frequency of surfability as detailed in

Table 11.4-8.

Table 11.4-8 Project Induced Changes to Surfing Frequency

Phenomenon	Project Induced Change	Change to Frequency of Surfability
Backwash	Decreased backwash	More frequent
Beach break breaking intensity	Spilling to plunging	Negligible
Sedimentation of Reef breaks	Reef break to beach break	Less frequent

An overall reduction in the amount of backwash (as a result of beach nourishment combined with sea level rise) would likely result in an increase in the frequency in which a site would be surfable over without Project conditions. Changing a surf site from spilling to more plunging is not expected to change the surfing frequency, only the ride and board type. Changing a surf site from a reef break to more of a beach break could reduce the surfing frequency, especially during walled conditions or windy conditions where the only surfable places tend to be reef breaks. Assuming the phenomena listed in

Table 11.4-8 are equally weighted, the overall frequency of surfable waves within the study area are not expected to change significantly as a result of the Project alternatives.

12 OPTIMIZATION OF BEACHFILLS

For the beach fill or hybrid plan alternative an optimization analysis is performed to determine the combination of initial design beach fill volume and replenishment volume and cycle that results in the highest net NED benefits. This analysis is based on a project life of 50 years. **Appendix E** details the evaluation of storm damage reduction benefits, the recreation benefits, and the economic discounting of first and future cost. This section presents the engineering parameters that form the basis of unit-cost, beach fill quantities, and environmental cost. The expected performance of the beach fill is discussed in **Section 7**and its impact to reducing bluff erosion and associated storm damages is discussed in **Section 6.6**.

12.1 Offshore Sand Sources

Prior marine geology studies in the project area conducted by the Corps of Engineers and other agencies as well as the offshore sand source exploitation carried out in the RBSPI (SANDAG, 2000 & Noble Consultants, 2000) and more recently the RBSPII (URS, 2009) have identified potential offshore borrow sites (SO-5, SO-6, MB-1 and SM-1) within which the median sand grain size (d_{50}) is greater than 0.3 mm**.** SO-5 and SO-6 are near the proposed beach fill sites located off shore of San Dieguito Lagoon in Del Mar and offshore of San Elijo Lagoon in Encinitas. MB-1 is located offshore of Mission Bay, and SM-1 is located offshore of the Santa Margarita River mouth in Oceanside. Figure 12.1-1 illustrates the locations of the four potential offshore borrow sites in relation to the Cities of Encinitas and Solana Beach.

Table 12.1-1 presents the site characteristics of these borrow sites as well as the distances to Moonlight Beach in Encinitas and Fletcher Cove in Solana Beach, respectively.

Table 12.1-1 Site Characteristics in Offshore Borrow Areas

Site Location	Water Depth (ft, MLLW)	Ave. D_{50} (mm)	Potential Volume (cy)	Approx. Distance to Receiver Site (miles)	
				Moonlight Beach	Fletcher Cove
SO-5	-35 to -60	0.51	~7,800,000	8.5	1.5
SO-6	-19 to -27	0.35	~1,300,000	5	2
MB-1	-18 to -24	0.51	~5,800,000	19.5	15
SM-1	-21 to -24	0.38	~23,000,000	14	18.5

Another offshore site (SO-7) that was previously evaluated was used during the construction of RBSPI and no longer has available volume.

Based on previous offshore mapping, various vibra-core logs taken from marine geophysical surveys, and sand grain size analyses, the potential sand volumes within these four offshore borrow sites would provide adequate sand sources for the proposed beach fill or hybrid plan alternative. Detailed descriptions of offshore sand sources at these three sites can be found in the **Appendix C**, the EIR/EA documents of the RBSPI (SANDAG, 2000) and RBSPII (SANDAG, 2011). The estimated volumes listed in

Table 12.1-1 include an adjustment for the anticipated RBSPII project, as discussed in **Appendix C**. The following optimization analysis is thus based on sands dredged initially from the two nearby sites, SO-5 and SO-6, and then once those sand sources are exhausted from the further sites MB-1 and/or SM-1.

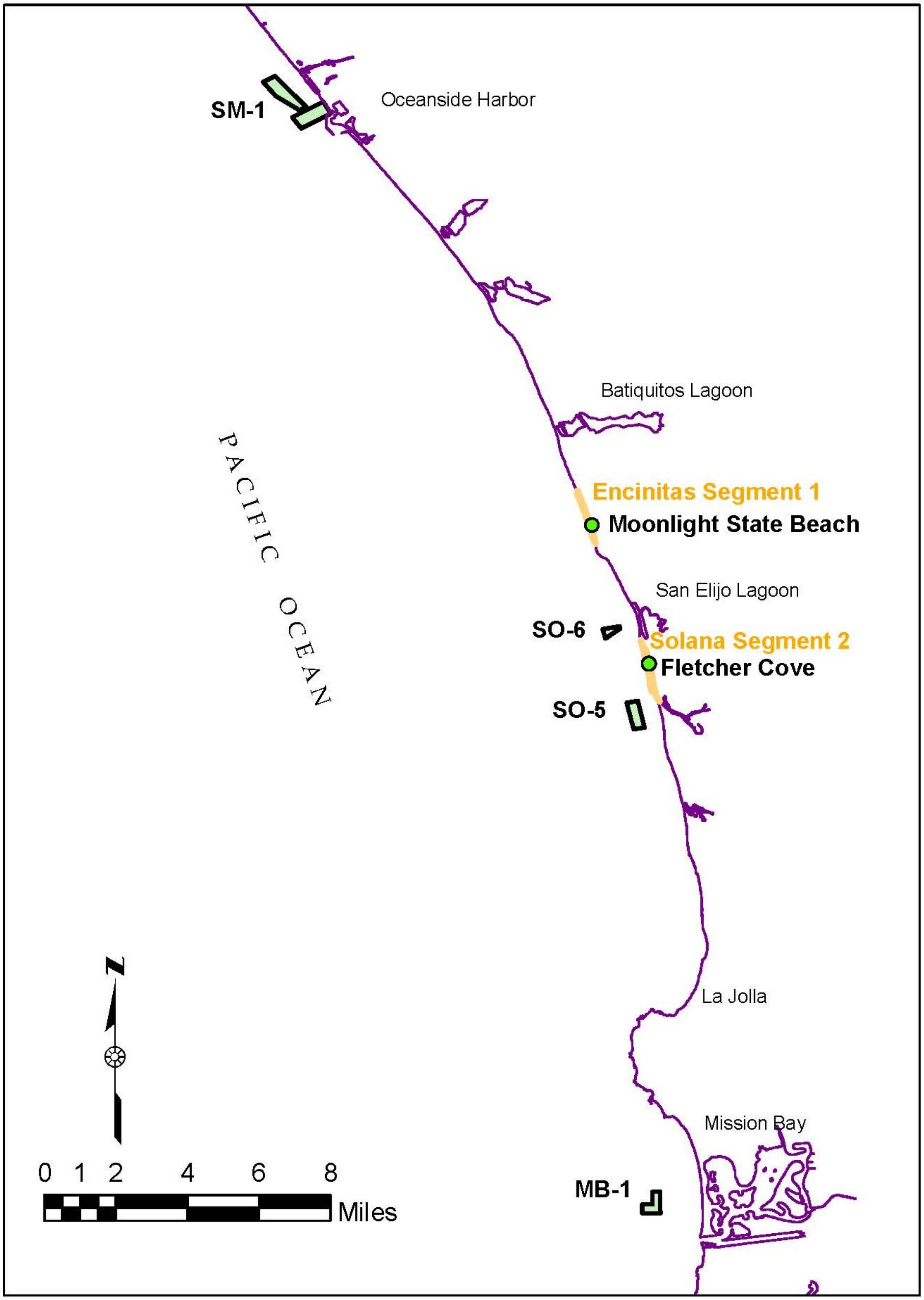


Figure 12.1-1 Potential Offshore Borrow Sites

12.2 Dredge Mobilization and Unit-Cost

The project costs for different initial beach fills and subsequent beach replenishment programs is dependent on initial and total sand quantities required, and the number of replenishment cycles. It is assumed that beach fills will be constructed using a hopper dredge to scrap and transport sand from borrow sites to be pumped ashore from its hopper to the beach fill receiver site. Because the available nearby sand borrow resource is finite, SO-5 or SO-6 is used for the initial construction and early replenishment cycles until a total volume of 6 MCY is borrowed, at which point the hopper dredge would use the MB-1 or SM-1 borrow sites. The costs for the beach fill operation including the lump sum of mobilization/ demobilization and the unit price of dredged, transported and placed sands for the initial fill and subsequent sand replenishments are presented in **Table 12.2-1**. The unit costs for each identified offshore sites are estimated using a Corps of Engineers Dredge Estimating Program (CEDEP) with the assumption of a hopper dredge with pump-out to the beach. The unit-prices are also compared to contract prices from the RBSPI (Noble Consultants, 2001). The initial construction assumes both Segment 1 and Segment 2 beach fills to be constructed together, hence, mobilization and demobilization cost is shared. Subsequent replenishment cycles between the two segments are assumed independent where the mob/demob is not shared.

The unit costs used in this optimization analysis start at $7.62 and $7.15 per cubic yard (October 2011 price-level) for Segments 1 and 2, respectively, and then increase by 50% once a total borrow volume of 6 million cubic yards is reached. A cost risk analysis to quantify risk and uncertainties will be computed for the Public Draft Report.

Table 12.2-1 Dredging Construction Costs

Mob/Demobilization Initial Fill	to Encinitas (Segment 1) and Solana (Segment 2)	$3,070,000
Mob/Demobilization Per Replenishment Cycle	to Encinitas (Segment 1)	$2,482,092
	to Solana (Segment 2)	$2,657,864
Unit Cost from SO-5 and SO-6 for first 6MCY	to Encinitas (Segment 1)	$7.62 / cubic yard
	to Solana (Segment 2)	$7.15 / cubic yard
Unit Cost from MB-1 and/or SM-1 over 6MCY (assumed 50% increase)	to Encinitas (Segment 1)	$11.43 / cubic yard
	to Solana (Segment 2)	$10.73 / cubic yard

12.3 Environmental Mitigation

The NED analysis considers the cost of environmental mitigation that would be required to offset adverse environmental impacts resulting from potential sand burial that is discussed in **Section 9** and in the Environmental Impact Statement. These impacts vary by beach fill size and include the following categories: Biological monitoring of construction, Surf Grass transplanting, Reef Mitigation, Kelp Transplanting, and monitoring of the mitigation. All constructed beach fill alternates would require 2 years of post construction biological surveys of the near shore benthic habitats. This would be in addition to the physical monitoring of beach profiles and bathymetry that tracks project performance and is cost accounted for elsewhere. Loss of surf grass and high relief, high value reef habitat occur for increased beach widths of 150 feet and greater in the Encinitas segment, resulting in a one-time mitigation cost to create new kelp reefs and to restore loss surf grass. This mitigation would be implemented at the completion of the initial biological monitoring, that is 3 years after the first construction. Once, mitigation is in-place, biological monitoring of its performance is continued for 6 years after its construction. **Table 12.3-1** and **Table 12.3-2** list the environmental mitigation cost by beach fill alternative, based on a preliminary mitigation ratio of 2:1.

Table 12.3-1 Environmental Mitigation Costs Encinitas (Segment 1)

Alternative Width (ft)	Post Construction Monitoring	Surf Grass Transplanting	Reef Mitigation	Kelp Transplanting	Mitigation Monitoring
50	$37,500 / yr	-0-	-0-	-0-	-0-
100	$37,500 / yr	-0-	-0-	-0-	-0-
150	$37,500 / yr	$1,012,000	$17,624,000	$68,000	$12,500
200	$37,500 / yr	$1,700,000	$37,006,000	$82,000	$12,500

Table 12.3-2 Environmental Mitigation Costs – Solana Beach (Segment 2)

Alternative Width (ft)	Construction Monitoring	Surf Grass Transplanting	Reef Mitigation	Kelp Transplanting	Mitigation Monitoring
50	$37,500 / yr	-0-	-0-	-0-	-0-
100	$37,500 / yr	-0-	$1,487,000	$14,900	$12,500
150	$37,500 / yr	-0-	$6,530,000	$65,300	$12,500
200	$37,500 / yr	-0-	$7,971,000	$100,000	$12,500
250	$37,500 / yr	-0-	$10,646,000	$123,900	$12,500
300	$37,500 / yr	-0-	$12,797,000	$148,100	$12,500
350	$37,500 / yr	-0-	$12,797,000	$148,100	$12,500
400	$37,500 / yr	-0-	$12,797,000	$148,100	$12,500

12.4 Lagoon Sedimentation/Inlet Maintenance Cost

Another adverse impact of introducing a larger volume of sand into the littoral zone is an increased dredging requirement at the lagoon entrances for the lagoon managers. The three lagoons that are affected are Batiquitos Lagoon, San Elijo Lagoon and San Dieguito Lagoon, which all have on-going inlet maintenance dredging programs to maintain their tidal ecosystems. **Section 10** of this Appendix details the analysis with the resulting annual increased cost presented in **Table** 12.4-1.

Table 12.4-1 Annual Lagoon Maintenance Mitigation Cost

Alternative Width (ft)	**Encinitas (Segment 1)**		**Solana (Segment 2)**	
	Batiquitos Lagoon	**San Elijo Lagoon**	**San Elijo Lagoon**	**San Dieguito Lagoon**
50	$23,000	$1,000	$1,000	$18,000
100	$55,000	$1,000	$1,000	$48,000
150	$79,000	$1,000	$1,000	$77,000
200	$99,000	$1,000	$1,000	$104,000
250	$112,000	$1,000	$1,000	$110,000
300	$121,000	$1,500	$1,500	$117,000
350	$128,000	$1,500	$1,500	$124,000
400	$133,000	$1,500	$1,500	$132,000

12.5 <u>Optimization of Beach Fill Volume</u>

Based on the Corps' planning guidance and evaluation of beach fill performance described previously in this Appendix (**Section 6.6** and **Section 7**), the procedural steps used for the optimization are described in the following:

a) Alternate beach fill sand volumes to widen the beach and push the MSL contour seaward in increments of 50-feet beach from its without-project condition is initially determined. This creates the initial beach fill alternatives, which were as wide as 200-feet in Segment 1 and 400-feet in Segment 2.

b) Replenishment cycles were evaluated from a two-year to 15-year cycle. Replenishment beach fill volumes were selected to re-establish the initial beach fill based on the beach fill erosion rates predicted by the analysis in **Section 7**.

c) The project net benefit is defined as the difference between the implementation cost and the project benefit, which includes both the storm damage reduction and associated recreational benefit. The project cost includes construction; planning, engineering and construction management; physical and biological monitoring; and mitigation. The cash flow of benefits and cost over the entire 50-year project life is discounted, as detailed in **Appendix E**.

d) The NED plan is the alternate that maximize net benefits.

12.5.1 Beach Fill Alternative

As presented in **Section 7**, a GENESIS modeling effort was performed to estimate shoreline evolution during the subsequent years from Year 1 to Year 16 after an initial sand placement in Year 0. **Section 6.6** provides the rationale for the effectiveness of the beach fill in mitigating bluff erosion and delaying or avoiding the construction of private seawalls.

Following the procedure described above, beach fill alternates that would initially widening the existing beach in increments of 50 feet up to 400 feet, combined with replenishment cycles ranging from two to 16 years to re-establish the initial widening were developed – a matrix of 6X15 alternates for each segment and each SLR scenario for a total of 360 possible beach fill programs. **Table 6.4-1** and **Table 6.4-2** presents the estimated initial sand volumes required for each designated width. Each alternate is evaluated until the incremental increase in benefit is smaller than the incremental increase in total project cost.

The placement density (V/S ratio) or sand volume required to push the MSL seaward for each linear alongshore measure are 0.864 and 0.714 cy/ft/ft for Segments 1 and 2, respectively. These conversion rates are based on the long history of profile behavior, as discussed in **Chapter 8**. **Table 6.4-3** and **Table 6.4-4** show the sand volumes required for individual replenishment cycles in Segments 1 and 2, respectively, based on the GENESIS modeled results as presented in **Chapter 7**.

For a typical storm damage reduction project, the full or partial project benefit is derived from the degree of protection provided by a designated beach width under various discrete storm events of wave and surge with defined probability of occurrence and response. However, the storm damage process of bluff retreat addressed herein vastly differs from the direct storm-induced damage, as the bluff may still be stable even under the 100-year return wave attack as long as the accumulative toe erosion does not extend to the prescribed threshold depth (see **Chapter 5**), and the coastal storm damage accumulates over all of the seasonal storms as the toe notch deepens increasing the vulnerability of bluff top failure. Therefore, a relationship was developed between MSL beach width and the remaining storm damage benefits associated with bluff retreat. This relationship, discussed in **Section 6.6**, is based on the formulation describing the rate of notch depth growth, the seasonal beach profile behavior, and the frequency of wave and tidal water levels. **Figure 6.6-1** shows the relationships for the Encinitas and Solana segments. Storm-damage reduction benefits increase from zero at a MSL width of about 100 to 120 feet to 100 percent with MSL widths from 200 to in excess of 400 feet.

Figure 12.5-1 and **Figure 12.5-2** are sample graphs of the expected value of net benefits versus initial beach width and replenishment cycle for the Encinitas and Solana segments, respectively. The full economic risk and uncertainty analysis is presented in **Appendix C**.

12.5.2 Hybrid Plan Alternative

Similar to the beach fill alternative, the same procedure is applied to determine the optimal hybrid plan alternative. Since the additional notch-fill element for this alternative does not change the alongshore transport mechanism induced by impinging waves, the shoreline evolution for each beach width option under the hybrid plan alternative would be the same as that for the beach fill alternative. Thus, the required sand volumes and construction cost for the same initial beach width and replenishment cycle combination are identical to those computed for the beach fill alternative, as presented in **Table 6.4-1** through **Table 6.4-4.** However, the project cost in each segment is increased slightly to include the notch-fill expense. Unit-cost values for the notch fill are shown on **Table 12.5-1**.

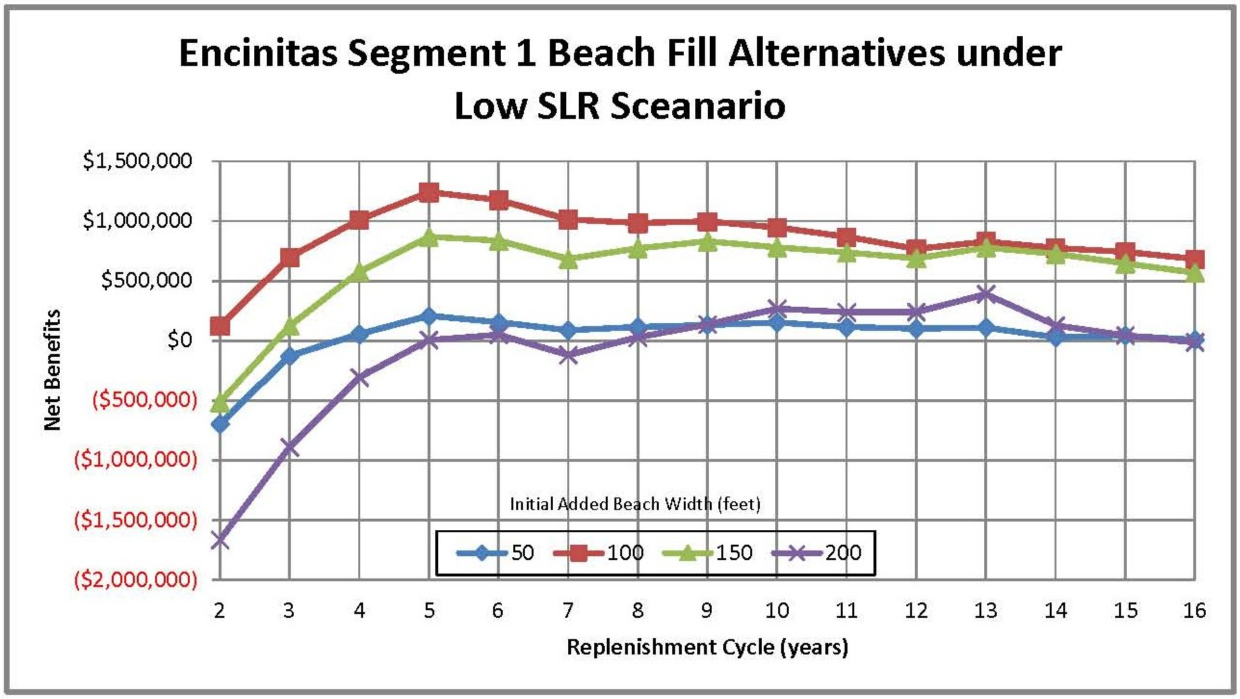
Table 12.5-1 Notch Fill Construction Cost

Segment	Unit of Measure	Quantity	Unit-Cost	Total Cost
Encinitas (Segment 1)	LF	6,365	$285.83	$1,819,308
Solana (Segment 2)	LF	5,336	$281.28	$1,500,910

Source: TRACES MII V4.1 estimate dated 26Oct11.

Similar to the beach fill alternative, a relationship of benefits as a function of the MSL beach width is applied to quantify the residual benefit in each project year, based on the spatial and temporal beach widths that were simulated from GENESIS. The difference in potential benefits from the beach fill only alternatives is obtained by setting all of the existing notch depths to zero for the base year in the bluff retreat model of **Section 5**.

Figure 12.5-4 and **Figure 12.5-5** are sample graphs of the expected value of net benefits versus initial beach width and replenishment cycle with notch fill for the Encinitas and Solana segments, respectively. The full analysis is presented in **Appendix C**.



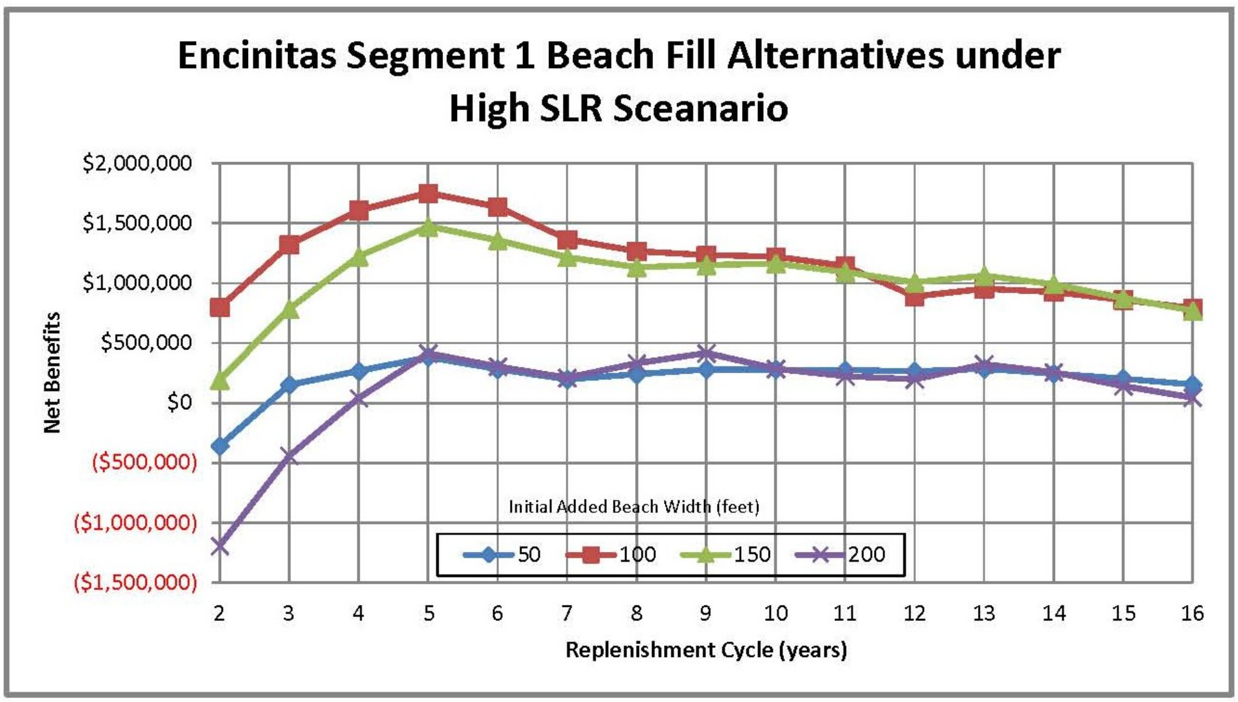
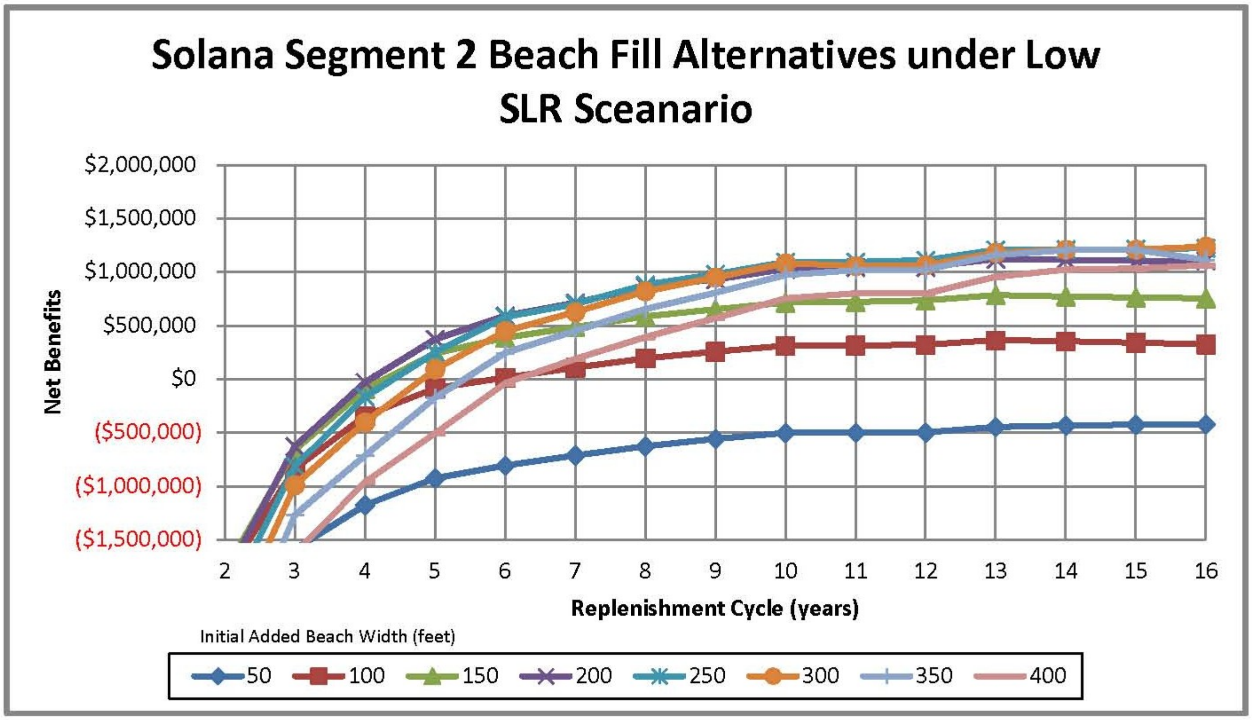


Figure 12.5-1 Encinitas - Segment 1 Beach Fill Alternatives Net Benefits vs Width and Nourishment Interval



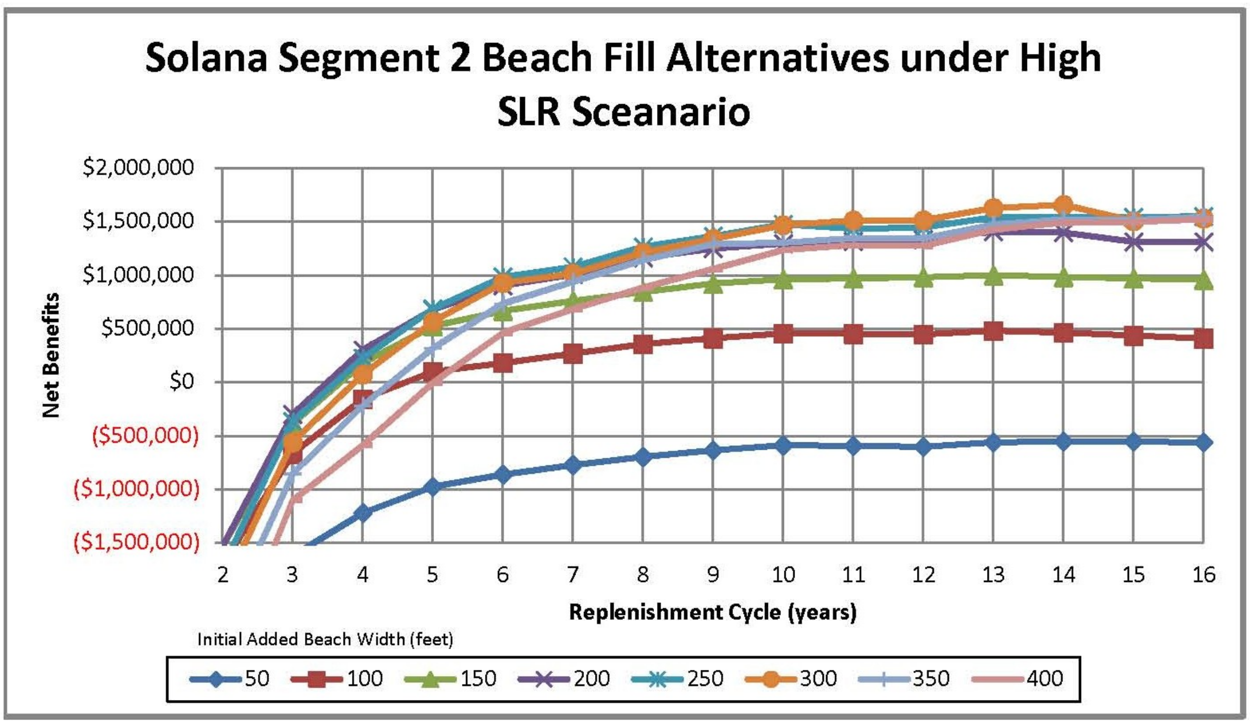
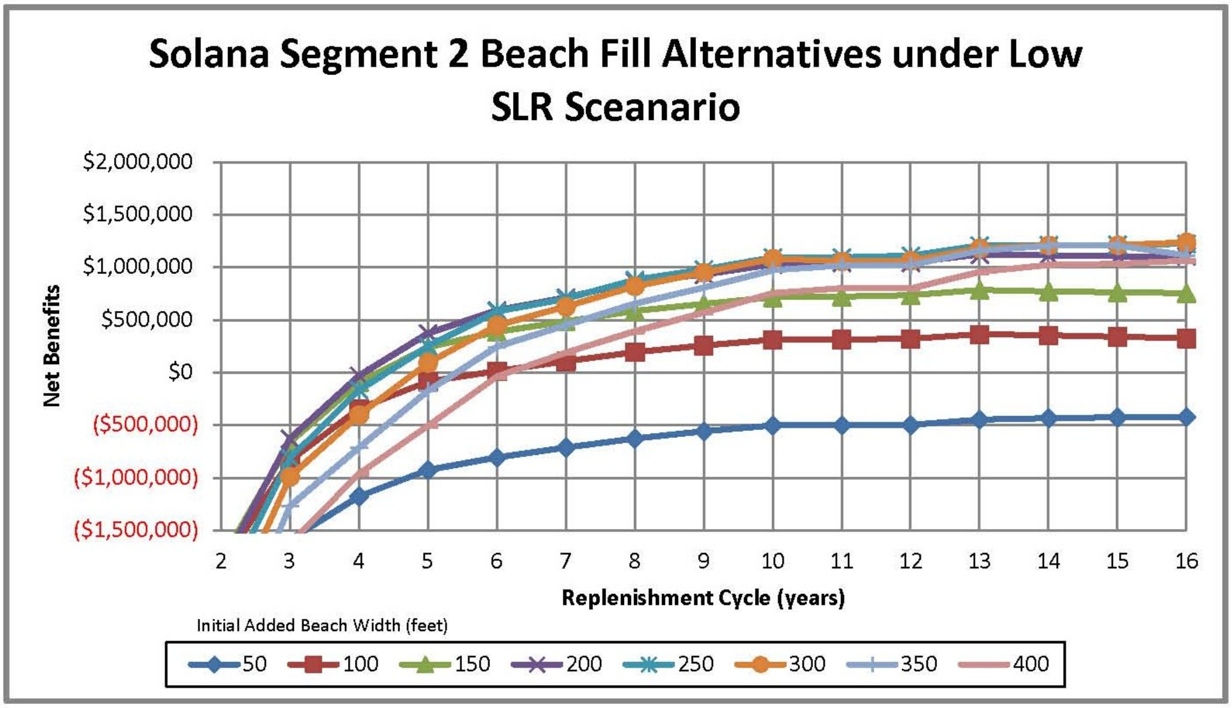


Figure 12.5-2 Solana - Segment 2 Beach Fill Alternatives Net Benefits vs Width and Nourishment Interval



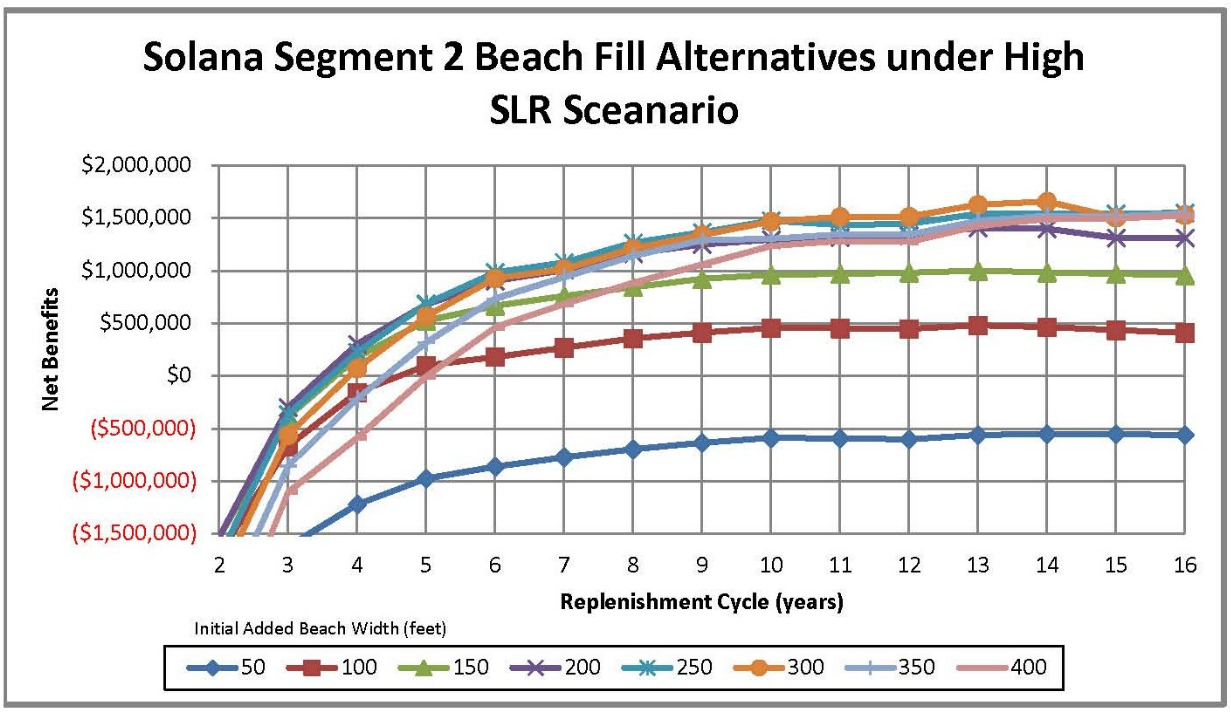
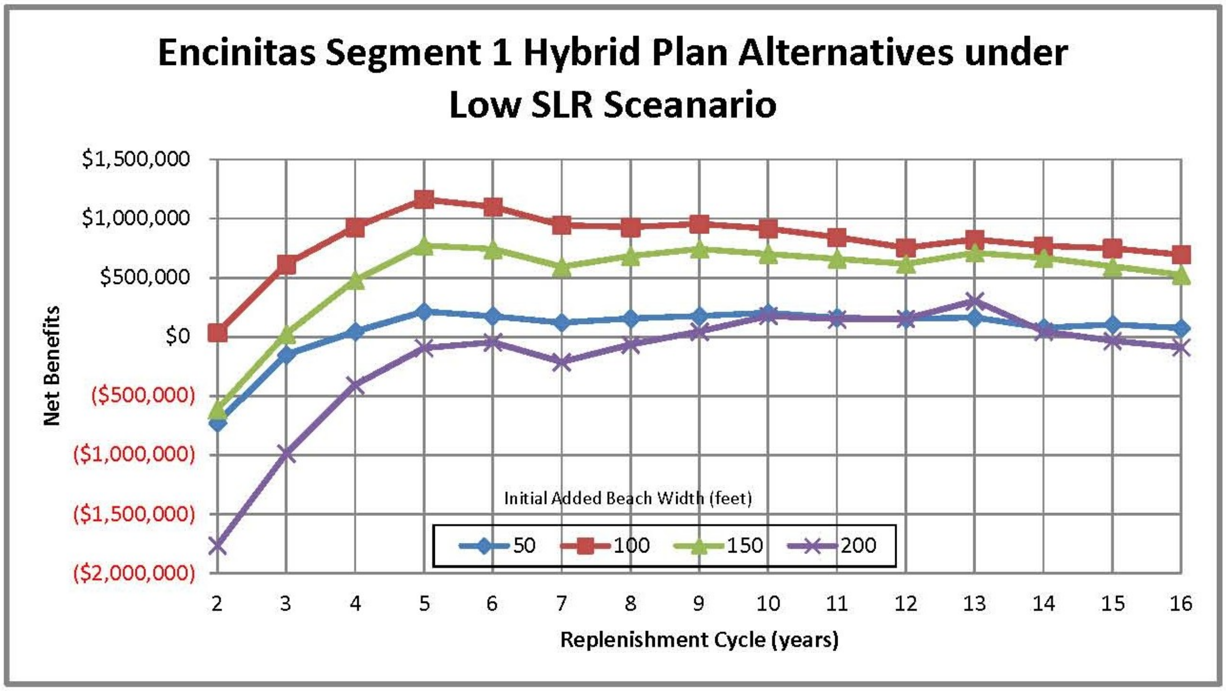


Figure 12.5-3 Solana - Segment 2 Beach Fill Alternatives Net Benefits vs Width and Nourishment Interval



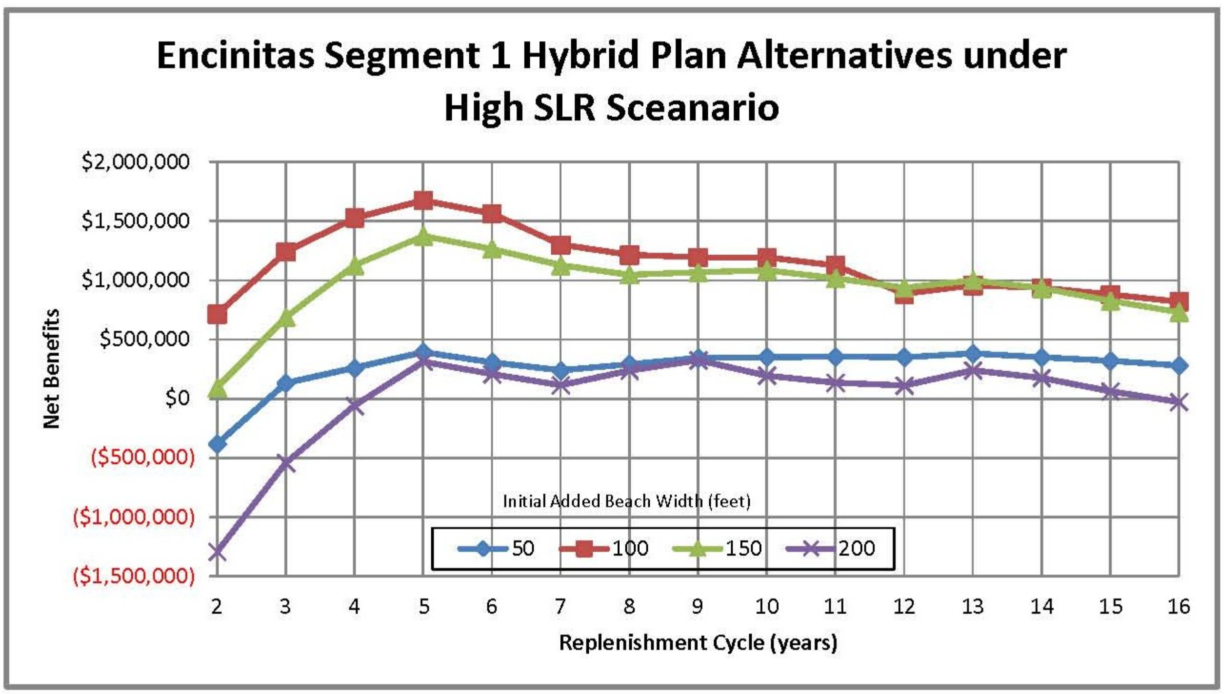
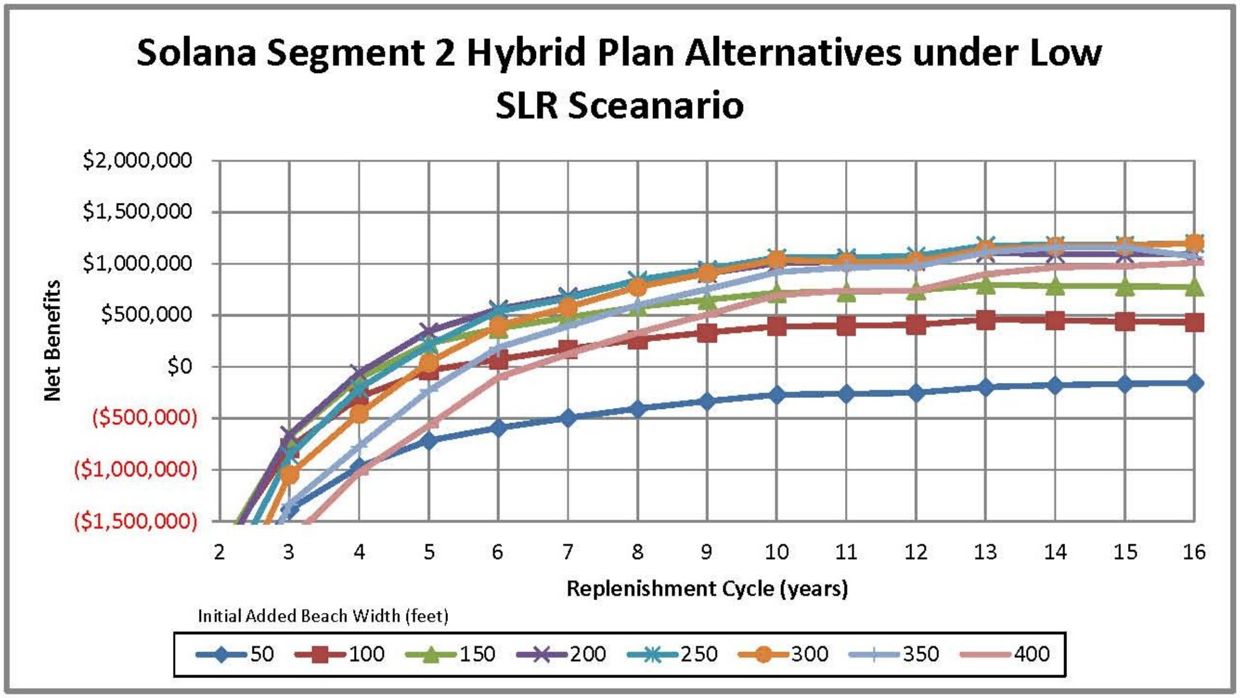


Figure 12.5-4 Encinitas - Segment 1 Hybrid Alternatives Net Benefits vs Width and Nourishment Interval



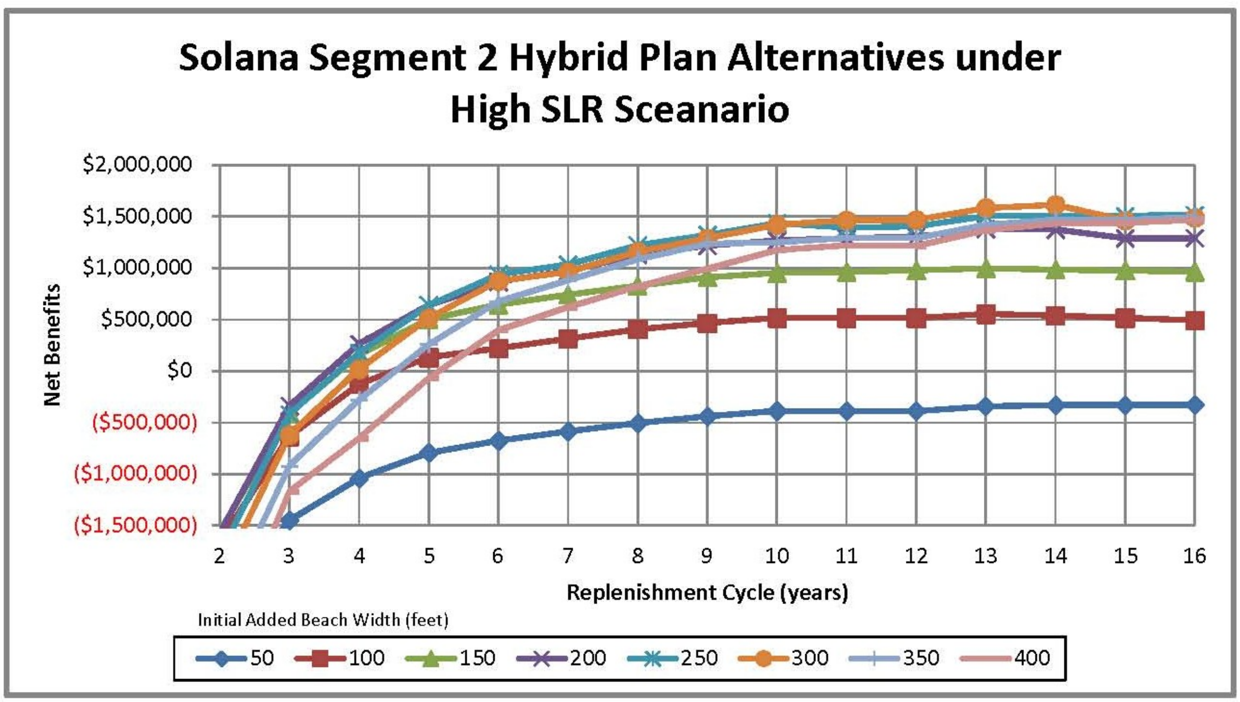


Figure 12.5-5 Solana - Segment 2 Hybrid Alternatives Net Benefits vs Width and Nourishment Interval

13 SENSITIVITY AND UNCERTAINTY OF PLAN OPTIMIZATION ANALYSIS

The NED plan optimization considers variables that have high variability and can only be represented in probabilistic terms, and variables that are not precisely known and are predicted by methods with unquantifiable precision. Uncertainty in the primary factors of the cost and benefit estimates is examined in this Section where the measured statistics of critical parameters are displayed and a sensitivity test on Net Benefits is performed on key predictive values that cannot be forecast in advance. The factors considered include the wave climate, the cross-shore distribution of sand which forms the protective beach, the conversion rate of sand volume for a unit area of shoreline change (V/S ratio), the erosion rate of the beach fill, and the potential cost of mitigation. The uncertainty in future Sea Level Rise is examined in scenarios as discussed in other **Section 5.2.1**.

13.1 <u>Statistical Characteristics of V/S Ratio</u>

The conversion rate between sand volume and a unit area of shoreline change (V/S ratio) is used to estimate the sand volumes for the initial beach fill and subsequent sand replenishment. The V/S ratio depends on the design beach fill profile and the initial beach profile prior to the sand placement. Based on historic surveyed beach profiles in the study area, the V/S ratios calculated in the Coast of California Study (CCSTWS-SD) have ranged from 0.222 to 0.726 cubic yards per foot. The "rule-of-thumb value in coastal engineering has been 0ne cubic yard per square foot of beach and an analogous V/S ratio that is used in the simple parallelepiped prism model of the one-line shoreline model of **Section 7** used a beach berm height of +12.5 feet (MLLW) and a depth-of-closure of -23.5 feet (MLLW), equating to a V/S ratio of 36 cubic feet per foot or 1.33 cubic yards per foot.

Reexamination of the ratio in MSL shoreline position and profile volume for the two most data rich profiles in the study area is displayed as a scatter diagram on **Figure 6.6-4** and **Figure 6.6-5** for an Encinitas profile and Solana Beach profile, respectively. The least-squares linear fit V/S ratio in Segment 1 is 0.86 cubic yards per square foot, and in Segment 2 is 0.71 cubic yards per square foot. As demonstrated by the wide scatter in **Figures 6-13 and 6-14**, the V/S ratio is highly variable, hence the MSL beach widths associated with each alternative beach fill volume is only a seasonal average MSL position. Of greater importance than the MSL width is the total active profile volume and the portion of that profile volume that remains close to shore at the bluff base. The beach fill plans are formulated by the profile volume, or when normalized by alongshore length, placement density in cubic yards per foot. The least-square ratios above are used to equate the fill densities to seasonally averaged MSL widths.

13.2 <u>Variability in the Cross Shore Distribution of Sand</u>

The cross-shore distribution of sand in the active littoral zone and the active profile sand volume is the primary determinant of beach fill effectiveness in reducing storm damages from waves and tides. **Figure 13.2-1** displays the profile record for Segment 2 off Solana Beach and **Figure 13.2-2** shows the time history of key parameters describing the profile. The common feature for this profile is spring sand levels next to the bluff toe being lower than high tide levels. Hence wave runup impacts directly on the bluff face under historic and existing conditions. Fall profiles have higher sand levels next to the bluff which insulates the toe notches and bluff face from the erosive wave action. The average seasonal distribution of sand in the cross-shore direction of the active profile is discussed in **Sections 6.6 and 8.3** and shown on **Figure 6.6-6**, **Figure 6.6-7**, **Figure 8.3-3** and **Appendix BB-6**.

The portion of the active profile volume within 200-feet of the bluff is used as an indicator for the beach fill effectiveness and is used to approximate average beach berm levels and in definition of the Benefit-Capture Curve described in **Section 6.6**. The variability in the nearshore sand distribution is indicated on **Figure 13.2-3**. This histogram of the percent of active profile volume within 200-feet of the bluff toe for spring, fall and all profiles shows the well established seasonal change and variation of the fraction of total active profile volume. For spring conditions, the mode is for 12.8% of the active profile volume to be within 200-feet of the bluff as compared to 20.3% for fall conditions. However, the range of values for spring profiles is from 5.4% to 27.7% with a standard deviation of 6.1%.

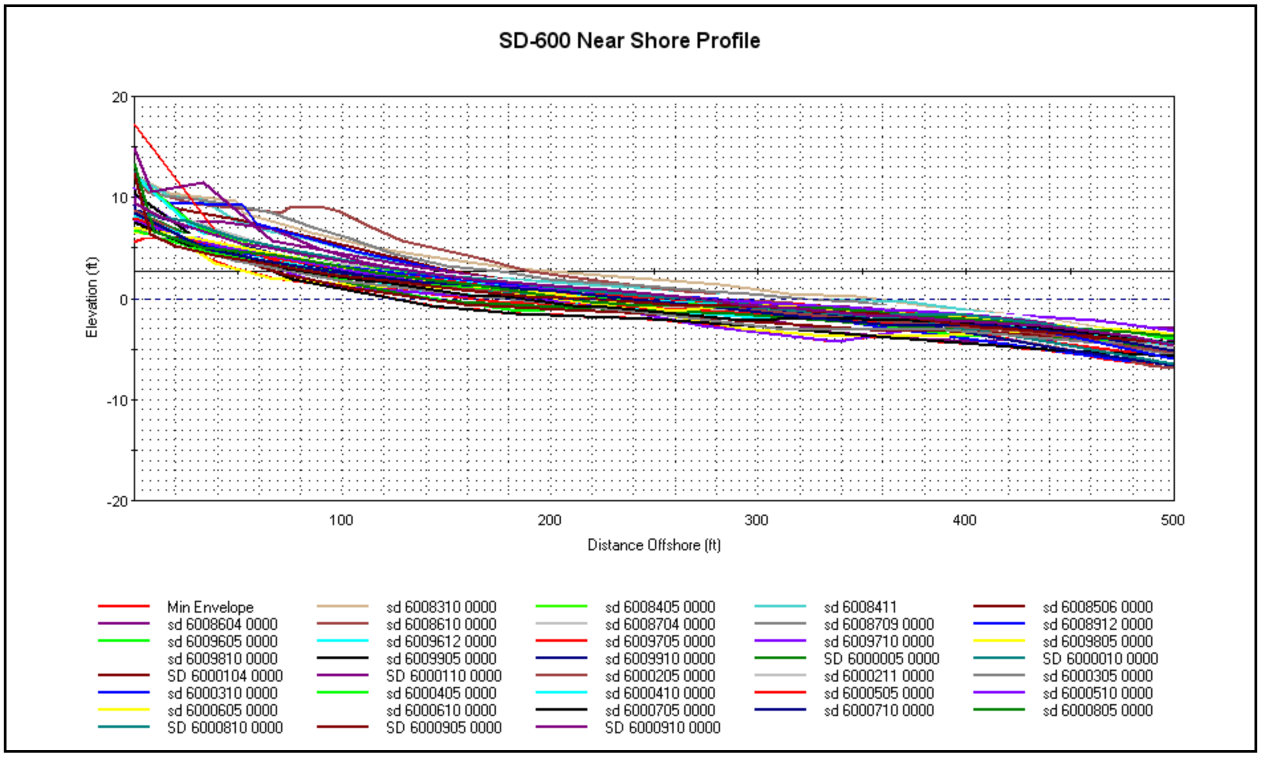


Figure 13.2-1 Nearshore Profile Variability

As discussed in **Section 6.6**, the Benefit Capture Curve (BCC) is derived from the nearshore sand distribution. The optimization analysis in **Section 12** utilized a BCC as shown on the top panel of **Figure 13.2-4**. This BCC used the spring profiles to define a mean bluff base sand volume of 12.7% with a standard deviation of 6.1% of the total active profile volume. The economic analysis applied the BCC as a normally distributed random variable with these mean and standard deviations.

An uncertainty in the definition of the active profile volume and its cross-shore distribution is in the delineation of the hardpan and lowest elevation of active sediment movement. A sensitivity test of the net benefits optimization was performed by adjusting the hardpan level in the nearshore in the profile analysis resulting in the mean value of bluff base volume to change from 12.7% to 20.45%, and the standard deviation to change from 6.1 % to 5.0%. The resulting BCC curve is shown on the lower panel of **Figure 13.2-4**. The sensitivity analysis on the NED plan is performed with this alternate BCC.

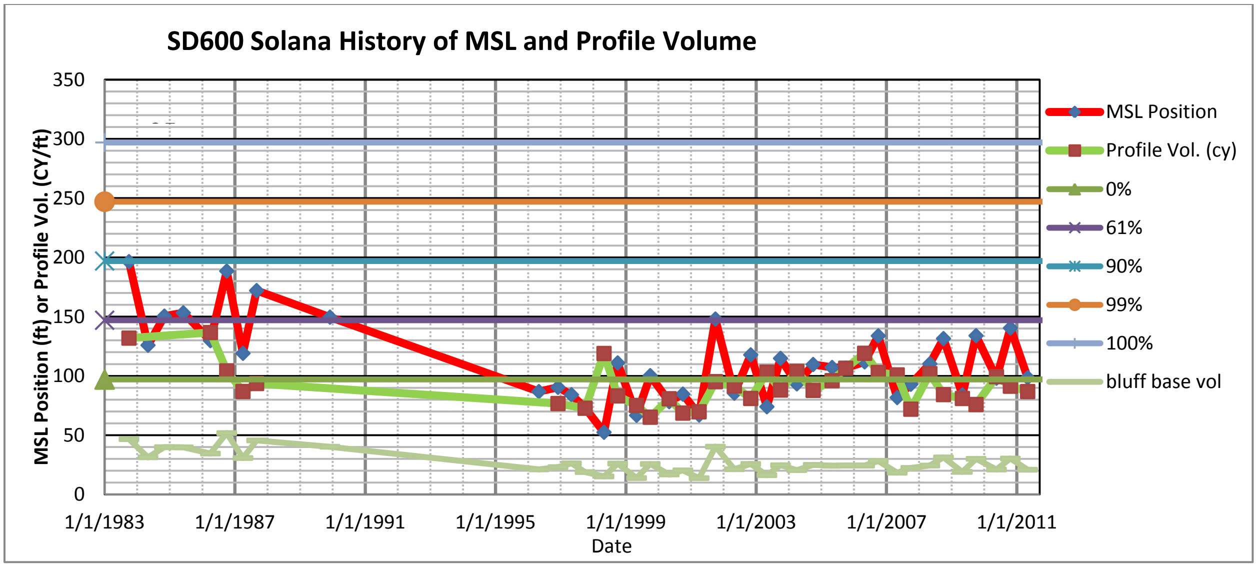


Figure 13.2-2 Time History of Profile Volume and Nearshore Volume

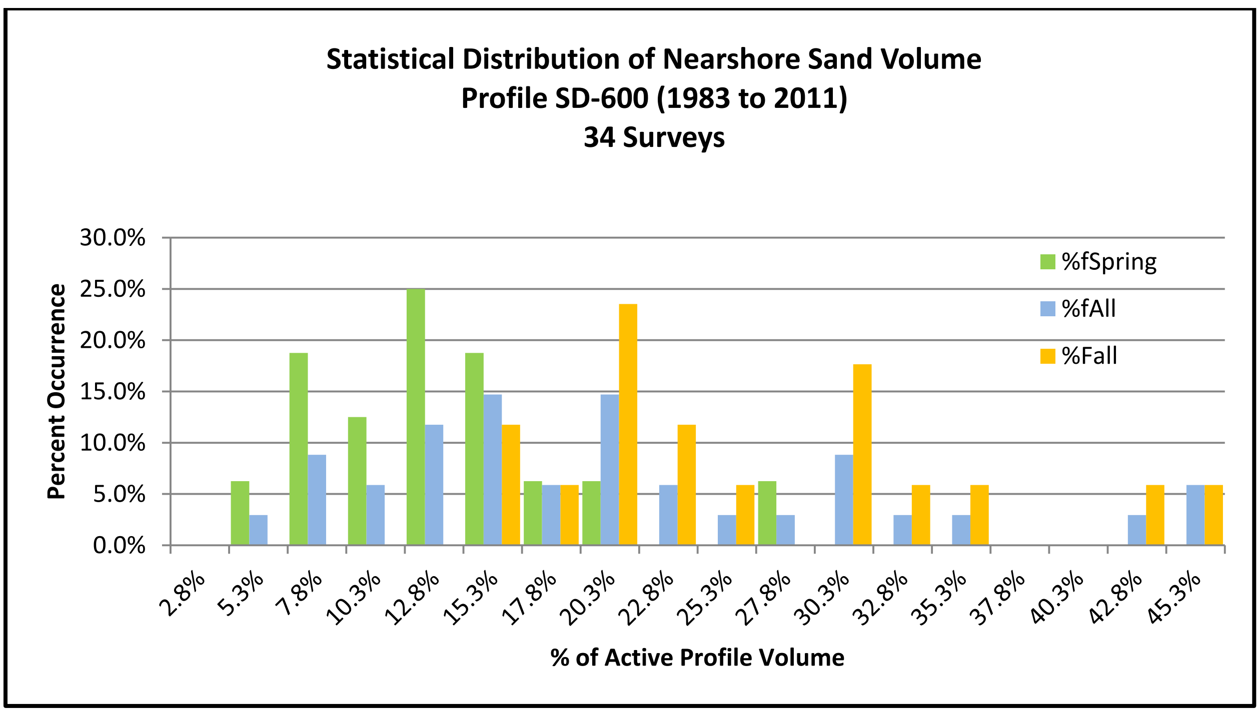
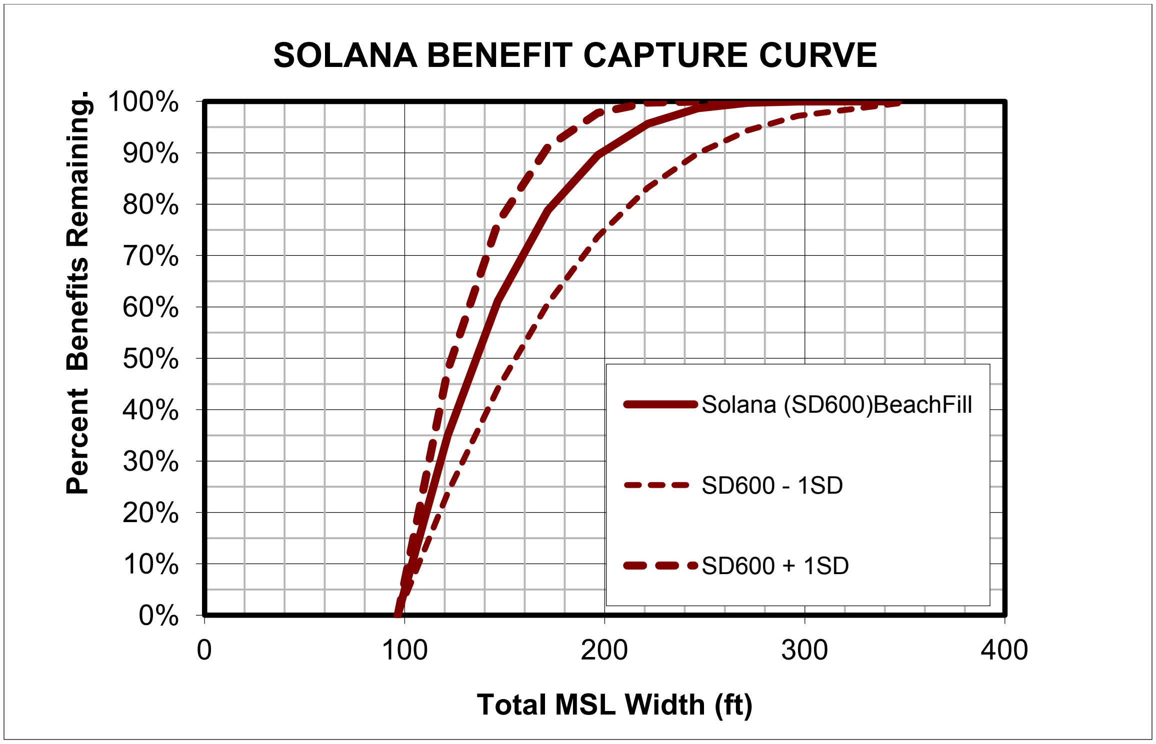


Figure 13.2-3 Distribution of Profile Volume within 200 feet of Bluff



Sensitivity Curve with mean nearshore sand volume of 20.45% with STDEV = 5.01%

Figure 13.2-4 Sensitivity of Benefit Capture Curve to X-Shore Distribution of Sand

13.3 Statistical Characteristics of Wave Climate

While the future wave climate is assumed to be similar to the recent past that is well represented by the hindcast wave data of 22 years from 1979 to 2000, the future sequence of storms and ENSO years are not deterministic, and will vary between years in the number and severity of storm wave events. To capture this variability in this study, the 22-year wave hindcast is parsed into five different sequences of future wave events to represent relatively severe, mild, and average groups of storm waves. Details are described in **Section 7.3.1**. The five wave climates predict five different shoreline responses, resulting in varied beach widths (i.e., sand volumes) that would remain on the beach after an identical initial sand placement. The mean sand volume, which is calculated by averaging sand volumes computed under the five wave-climate groups, was used in the optimization analysis described in **Chapter 12**. The variability in beach evolution predicted by the GENESIS modeling in **Section 7** resulting from each of the five wave-climate groups is displayed on **Tables B6-1** through **B6-8**, and a sample of these data is displayed graphically on **Figure 13.3-1** for the Encinitas Segment and **Figure 13.3-2** for the Solana Beach Segment. On these figures, the mean, maximum and minimum net shoreline change from the initial shoreline is plotted for various sizes of initial beach fill. Each of the five wave-climate groups are assumed to be equally likely.

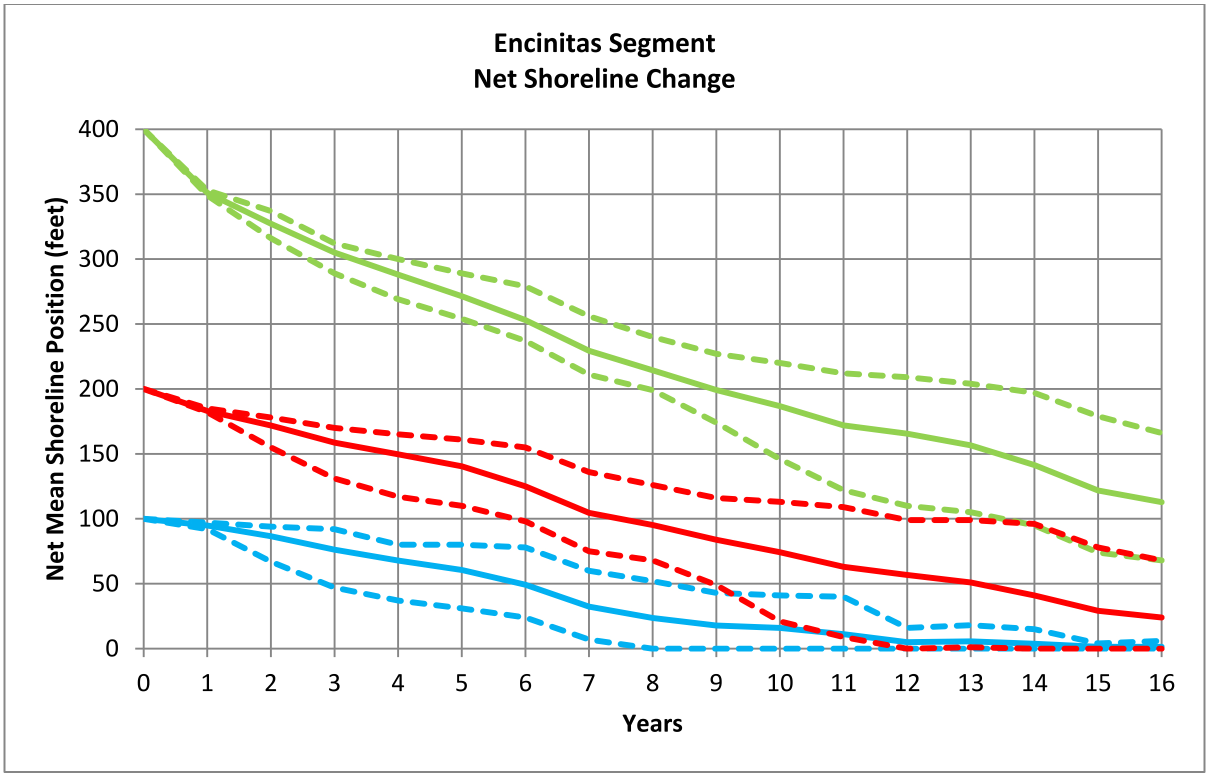


Figure 13.3-1 Range of Predicted Shoreline Response with Five Wave Sequences Encinitas Beach Segment 1

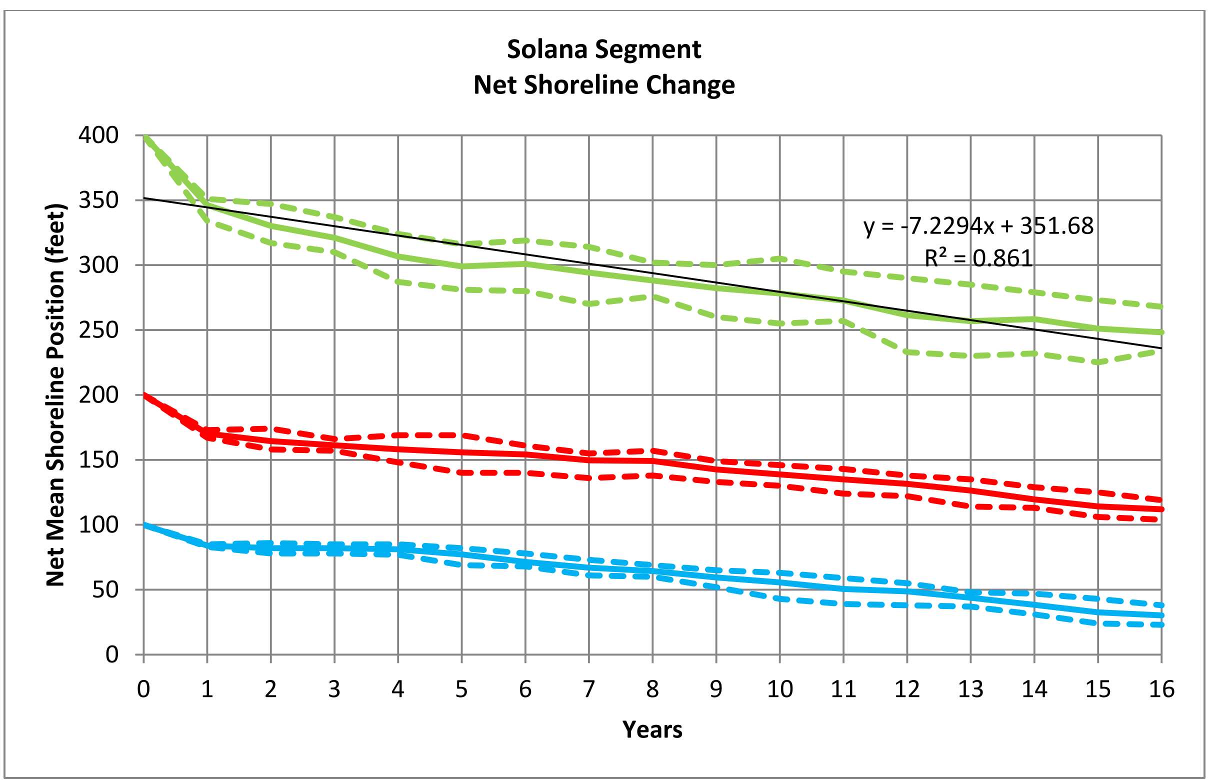


Figure 13.3-2 Range of Predicted Shoreline Response with Five Wave Sequences Solana Beach Segment 2

13.4 <u>Beach Fill Erosion Rates</u>

The performance of the beach fill is also a critical determinant of project cost and effectiveness in reducing coastal storm damages. **Figure 13.4-1** shows the time history from 1983 to present of the MSL position, active profile volume and Bluff base volume (sand volume within 200-feet of bluff) for the profile representative of Segment 2. The only beach fill during this time period was the 146,000 cubic yard RBSPI project in the fall of 2001. The historic and existing profile condition is sediment starved where a slight trend in profile volume loss is observed. The least squares linear trend pre-RBSPI and post-RBSPI profile volume loss is -2.45 cubic yards per year and -0.80 cubic yards per year, respectively (**Figure 13.4-1**, which shows the regression equation in units of cubic yards per day). A linear least-squares trend line to the baseline net shoreline for the 400-foot initial fill alternative in Segment 2 has a slope of -5.16 cubic yards per foot per year (**Figure 13.3-2**, which shows the regression equation with a slope of -7.23 ft/year; converted by the V/S ratio of 0.713 cy/ft/ft equates to -5.16 cy/ft/ft/year).

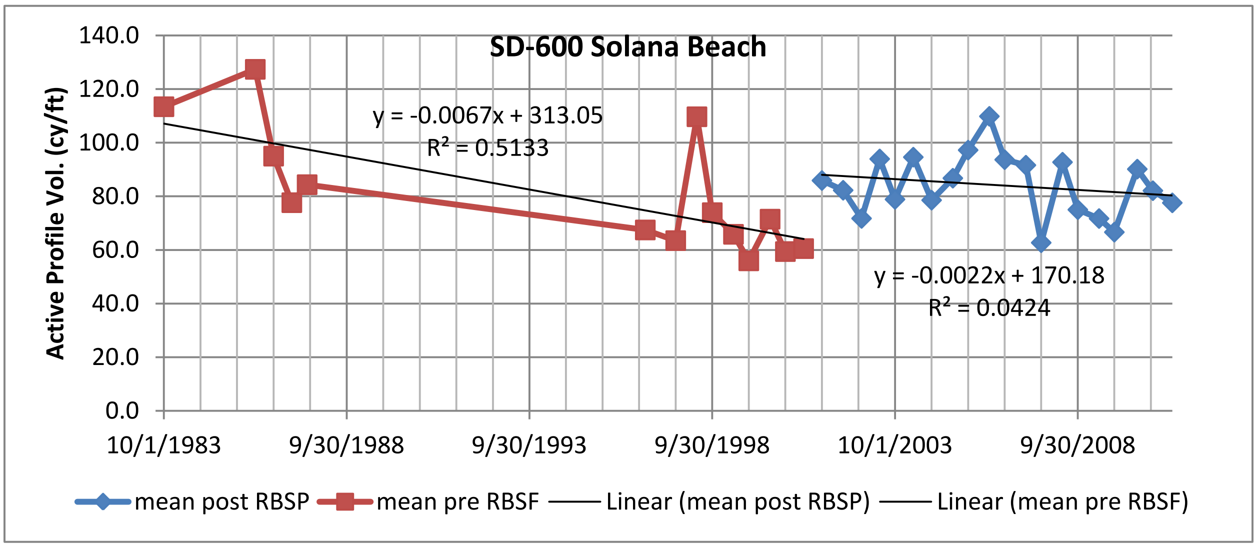


Figure 13.4-1 Segment 2 Profile Volume History

While the historic record and the predicted shoreline response in Segment 2 suggests a relatively low rate of profile volume loss, the stability of a large perturbation of sand fill is unexpected. Limitations in the one-line shoreline model to accurately predict along concave shorelines, and the introduction of numerical breakwaters to mimic the effect of the nearshore reefs may overestimate shoreline stability in Segment 2. Other larger fills in the northern San Diego County RBSPI experienced post-construction retreat rates on the order of 12 feet per year.

Four different erosion rates were used in the sensitivity testing of Segment 2 as graphically depicted on **Figure 13.4-2**. The baseline is the mean shoreline response as shown on **Figure 13.3-2** and on the top left panel of **Figure 13.4-2**. A modified shoreline model that removed the breakwater reef structures and reduced the concavity of the existing shoreline resulted in the Modified GENESIS shoreline change rates shown on the top right panel of **Figure 13.4-2**. In addition, two simple straight line erosion rates of 12.8 feet per year and 25 feet per year were used. The 12.8 feet per year was the erosion rate experience at the Oceanside RBSPI project. The 25 feet per year is approximately double of this value and would be considered an improbable extreme.

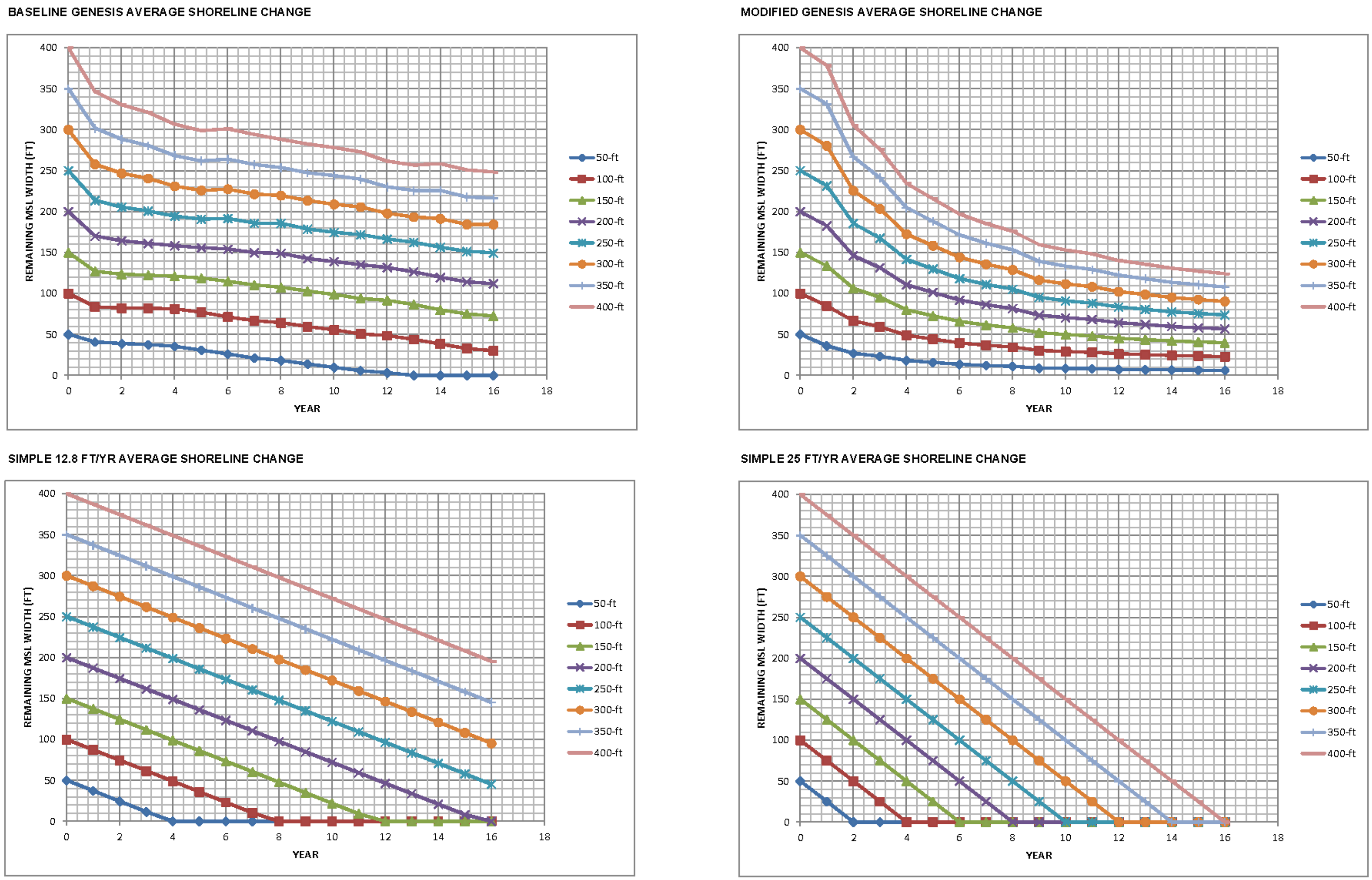


Figure 13.4-2 Solana Beach Erosion Rate used in NED Sensitivity Analysis

13.5 Mitigation Costs for Habitat Loss

Adverse impacts to nearshore benthic habitat and tidal lagoons may result with the introduction of significantly more sand into the active littoral environment. The **Integrated Report** and **Section 12.3** describe the nature of the impacts and the estimated costs for possible mitigation measures. The preliminary mitigation cost used in the NED optimization in **Appendix E** is based on an impact to mitigation area ratio of 2:1, as described in **Section 12.3** and **Section 12.4**.

Unfortunately the prediction of adverse impact and the effectiveness of mitigation are rife with uncertainty. A sensitivity analysis of mitigation cost in determination of the NED plan was performed on Segment 2 by varying the mitigation ratio between a 1:1 and 4:1.

13.6 Sensitivity of Net Benefits

The baseline net NED benefits for beach fill alternatives as a function of initial beach fill width and replenishment interval is displayed on **Figure 12.5-1** and **Figure 12.5-2**. **Figure 13.6-1** shows the sensitivity, under the low SLR scenario, of changing the erosion rates of the beach fill, and **Figure 13.6-2** shows the sensitivity for changing the BCC curve as a result of different interpretation in the cross-shore distribution of sand as described in the previous **Section 13.2**. Finally, **Figure 13.6-3** show the effect of changing the mitigation cost by a factor of 4.

Table 13-1 summarizes the sensitivity in selecting the plan that optimizes net benefits. Net Benefits are usually higher for the High Sea Level Rise scenario in comparison to the Low (Historic) Sea Level Rise scenario. For all of the sensitivity tests, net benefits were positive except for the baseline BCC curve and extreme erosion rate of 25-feet/year in Segment 2. In Segment 1, the selected plan is not sensitive to the parameters that were varied, that is, the optimal plan is consistently a 100-foot initial width with a 5-year replenishment cycle.

The optimal plan for Segment 2 varied from an initial width of 200 feet to 400 feet and a replenishment cycle ranging from 10 to 16 years. The baseline NED Plan has an initial width of 300-feet and replenishment cycle from 14 to 16 years depending on the SLR scenario. Changing the BCC to reflect a larger portion of sand near the bluff toe reduced the NED Plan initial width to 250 feet but the optimal replenishment cycle remained at 14 or 16 years. Increasing the erosion rate tends to increase the initial fill width, decrease the replenishment cycle time, and decrease net benefits and BCR.

The last two rows of **Table 13.6-1** show the results of the 4:1 mitigation cost assumption with the 20.45% BCC. For both SLR scenarios, net benefits optimize at a 200-foot initial width and 13 year replenishment cycle.

Table 13.6-1 Plans that Optimize Net Benefit Estimates, Segment 2

Evaluation Conditions				Optimal Plan		Net Benefits ($)	BCR
Segment	SLR Scenario	BCC	Erosion Rate	Initial Width (ft)	Fill Cycle (yr)		
1	Low	Baseline	Baseline	100	5	$1,240,783	1.55
1	High	Baseline	Baseline	100	5	$1,750,935	1.69
1	Low	20.45%	Baseline	100	5	$1,622,947	1.73
1	High	20.45%	Baseline	100	5	$1,728,640	1.68
2	Low	Baseline	Baseline	300	16	$1,236,494	1.51
2	High	Baseline	Baseline	300	14	$1,655,908	1.60
2	Low	20.45%	Baseline	250	14	$1,969,956	1.90
2	High	20.45%	Baseline	250	16	$2,304,467	1.93
2	Low	Baseline	Modified GENESIS	200	13	$746,436	1.37
2	Low	Baseline	12.8 ft/yr	300	13	$1,230,032	1.45
2	Low	Baseline	25 ft/yr	400	10	($109,558)	0.98
2	Low	20.45%	Modified GENESIS	250	16	$1,637,133	1.69
2	Low	20.45%	12.8 ft/yr	300	13	$1,907,040	1.69
2	Low	20.45%	25 ft/yr	400	13	$531,669	1.13
2 w/ 4:1	Low	20.45%	Baseline	200	13	$1,073,902	1.41
2 w/ 4:1	High	20.45%	Baseline	200	13	$1,271,114	1.41

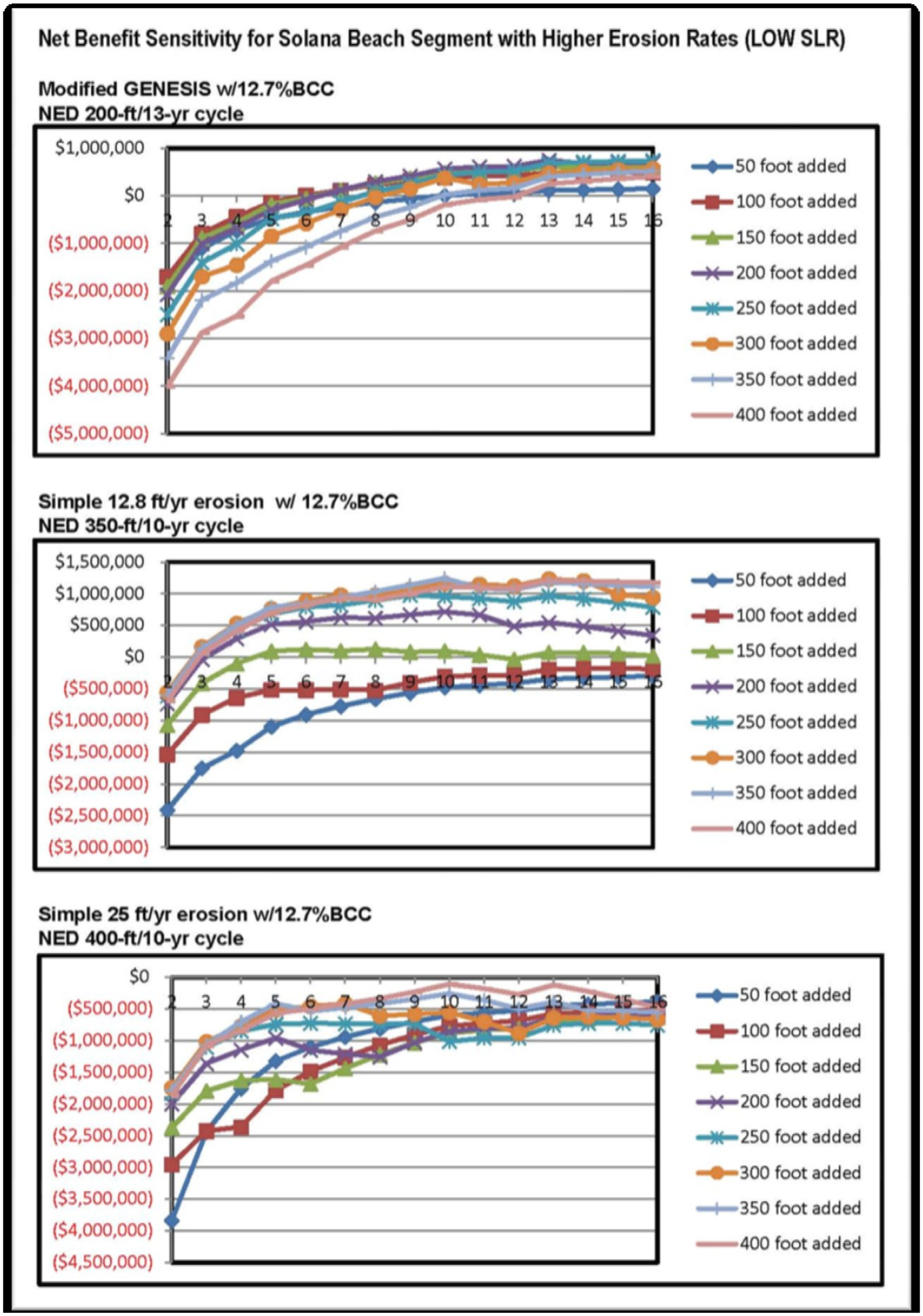


Figure 13.6-1 Net Benefits Sensitivity to Erosion Rate

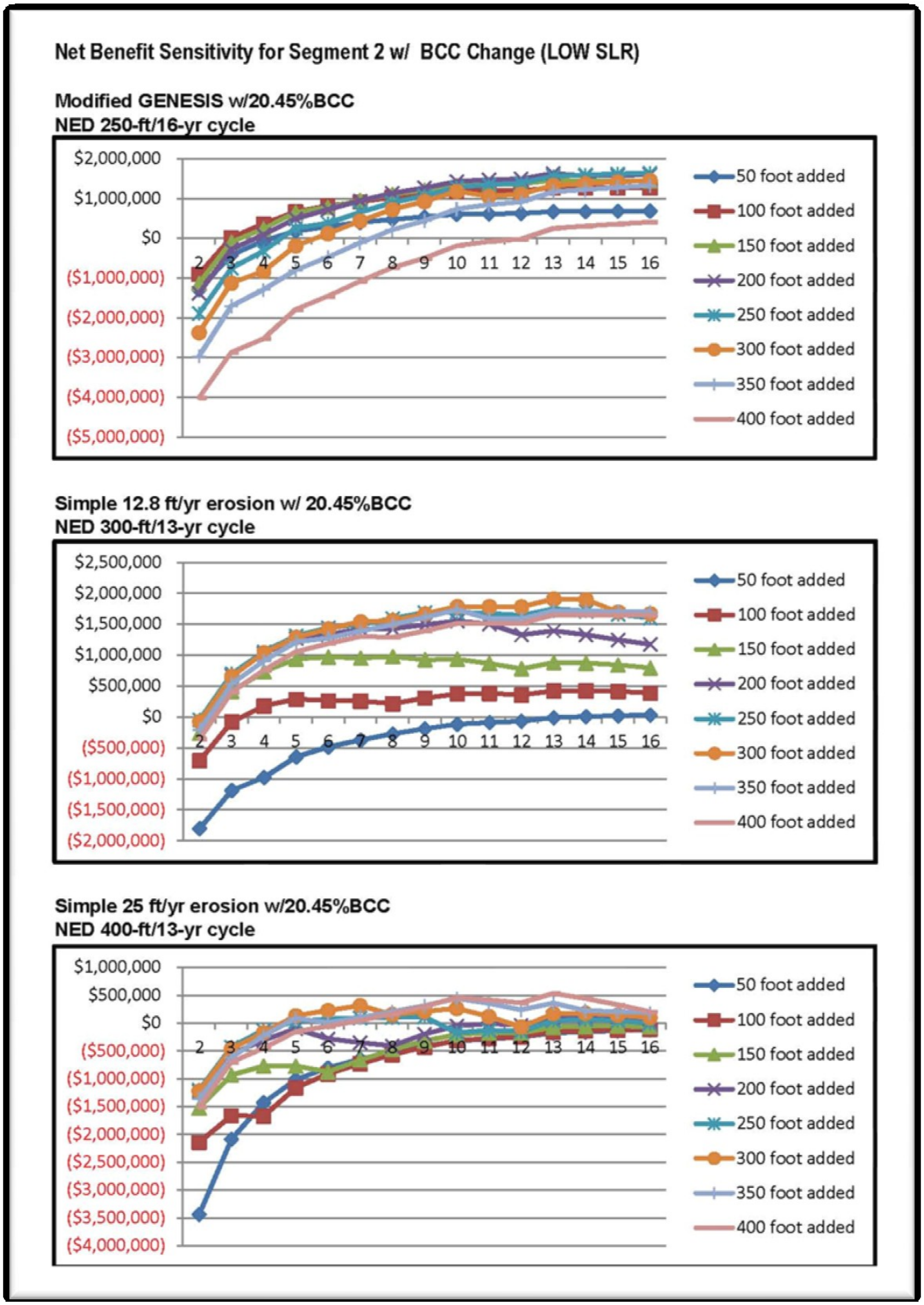


Figure 13.6-2 Net Benefits Sensitivity to Benefit Capture Curve

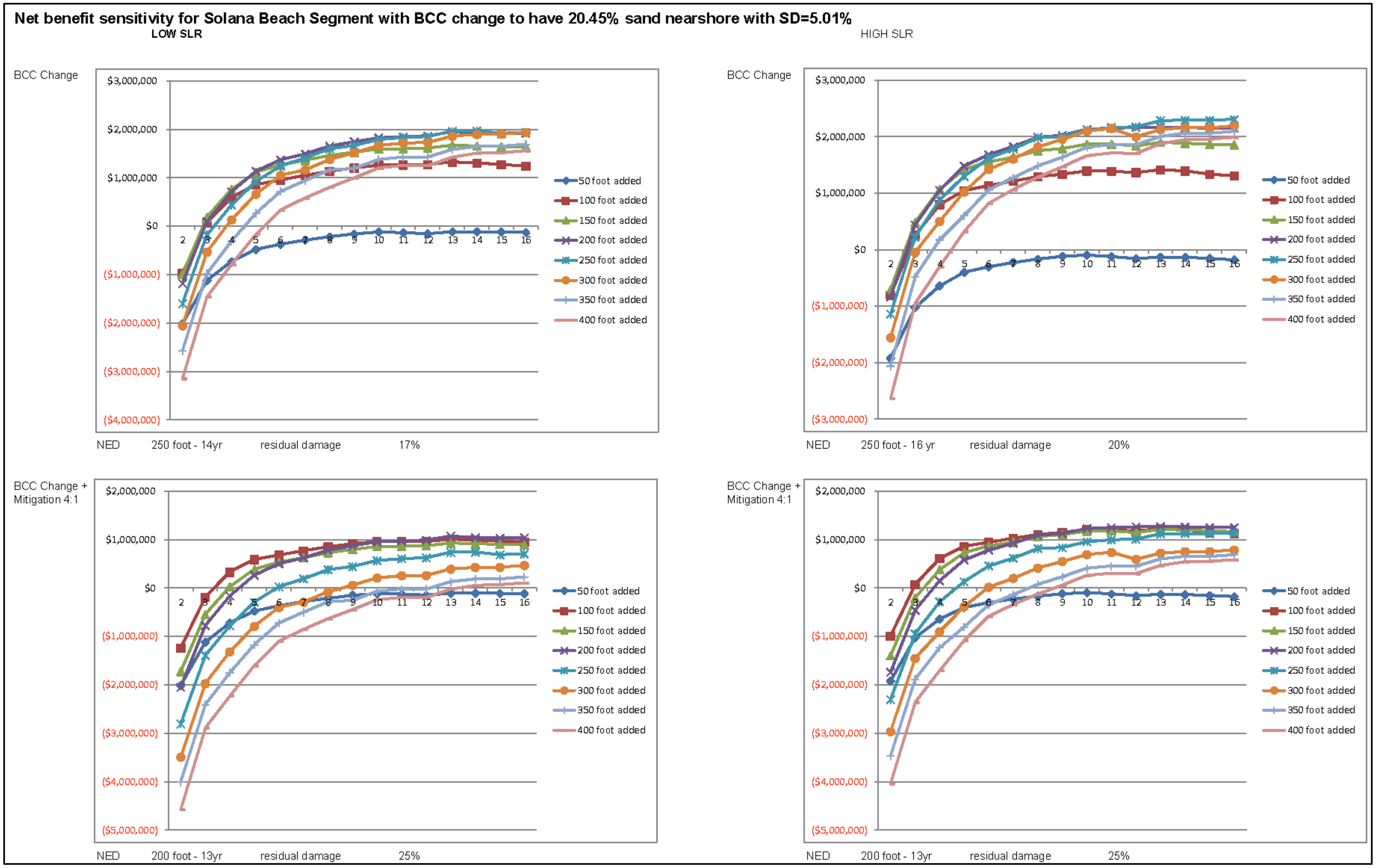


Figure 13.6-3 Sensitivity to Benefit Capture Curve and Mitigation Cost

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