FEASIBILITY OF ALTERNATIVES TO THE GOAT ROCK STATE BEACH JETTY FOR MANAGING LAGOON WATER SURFACE ELEVATIONS

Prepared for
Sonoma County Water Agency

March 10, 2017

ESA PWA

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1 INTRODUCTION

This feasibility study, developed by ESA PWA at the request of Sonoma County Water Agency (Water Agency), proposes and evaluates alternatives to the jetty on Goat Rock State Beach (GRSB) (Figure 1-1) that may help achieve target water surface elevations in the Russian River Estuary. Before considering jetty alternatives, this study describes the physical processes that influence water surface elevations in the Estuary and that may be influenced by the jetty. As such, this report fulfills a portion of the Water Agency's obligations under the 2008 Biological Opinion[1] issued by the National Marine Fisheries Service (NMFS). The Biological Opinion directs the Water Agency to change its management of the Estuary's water surface elevations with the intent of improving juvenile salmonid habitat while minimizing flood risk.

1.1 Russian River Biological Opinion

In the Biological Opinion, NMFS concluded that historical artificial breaching activities in the spring and summer resulted in a loss of freshwater habitat in the Estuary and that a lack of freshwater estuarine rearing habitat limits recovery of salmonid populations, particularly steelhead (NFMS, 2008). The abundance and growth rates of juvenile steelhead have been positively correlated with the freshwater habitat found in lagoons. A lagoon is intermittently created by the formation of a barrier beach across the Estuary's ocean inlet, thereby blocking or reducing tidal action and limiting salt water transport into the Estuary. NMFS determined that salmonid estuarine habitat may be improved by managing the Estuary to facilitate a perched, freshwater lagoon. Therefore, the Biological Opinion stipulates as a Reasonable and Prudent Alternative (RPA) that the Estuary be managed to facilitate lagoon conditions with an outlet channel between May 15th and October 15th.

Under the RPA's target conditions, the lagoon water surface elevations would be higher than ocean water surface elevations, ideally above 7 feet (ft) NGVD[2]. While it is possible that this condition could be achieved without mechanical intervention, it is expected that the Estuary water levels would rise above 9 ft NGVD at which flooding of development along the Estuary begins to occur. Therefore, the Biological Opinion allows

[1] Biological Opinion for Water Supply, Flood Control Operations, and Channel Maintenance conducted by the U.S. Army Corps of Engineers, the Sonoma County Water Agency, and the Mendocino County Russian River Flood Control and Water Conservation Improvement District in the Russian River watershed

[2] NGVD=National Geodetic Vertical Datum, a fixed reference elevation adopted as a standard reference for the Russian River Estuary

for management (intervention) to regulate the summer-fall Estuary water levels between 7 and 9 ft NGVD. The Biological Opinion suggests that the target conditions can be achieved via groundwater seepage through the barrier beach and a limited, non-tidal outlet channel incised in the beach. Conceptually, these quasi-steady outflows would be sufficient to convey riverine inflow from the Estuary to the ocean at a rate close to the riverine inflows without scouring the outlet channel to a larger size resulting in a tidal inlet. Also, the outflow may be sufficient to counter wave-induced sand transport that can close a small outlet channel, thereby inducing an increase in the Estuary water level. In effect, the outlet channel through the beach must be of a size that is not too big (a too-big outlet results in tidal exchange and impacted habitat) or too small (a too-small outlet results in low outflows, causing water surface elevation to increase and potential flood shore development). One of the challenges is that the ideal channel size may change due to a change in river or ocean conditions, and there is a need to limit the number of mechanical interventions per year.

Recognizing the complexity and uncertainty of managing conditions in the dynamic beach environment, the Biological Opinion stipulates that the estuarine water surface elevation RPA be managed adaptively. This means that it should be planned, implemented, and then iteratively refined based on experience gained from implementation. Part of its adaptive nature is a phased approach which starts with a limited project scope and then expands the scope only if earlier alternatives are not feasible. The first phase, which has been implemented since 2010, is limited to outlet channel management that only involves creating a sand channel in the beach (ESA PWA, 2015). For the second phase, the Biological Opinion expands the project scope to study alternatives to the jetty. The jetty, which is embedded in the barrier beach, may significantly affect some of the physical processes which govern lagoon water surface elevations. This document initiates this second phase by evaluating alternatives to the Goat Rock State Beach jetty that may help achieve target estuarine water surface elevations. The Biological Opinion does not require the implementation of any alternatives. The third stage further expands the project scope to include flood risk reduction measures for properties adjacent to the Estuary.

1.2 Study Goals and Objectives

In context of the Biological Opinion, the goal of this study is to assess if there are alternatives to the GRSB jetty that may improve the likelihood of achieving the Biological Opinion's target lagoon water surface elevations and the feasibility of such alternatives. Alternatives to the jetty may help achieve target water surface elevations by:

- Encouraging more frequent and longer tidal inlet closures during the lagoon management period

- Facilitating water level management during the lagoon management period closures by supporting outlet channel resilience and/or increasing groundwater seepage
- Maintaining existing level of year-round flood risk for low-lying properties adjacent to the Estuary
- Anticipating future changes that may result from sea level rise

Opportunities to modify the jetty exist because:

- The jetty is not in active use, with the objectives which initiated its construction no longer applicable for the landowner (California Department of Parks and Recreation) or Estuary resource managers.
- A collaborative team of local, state, and federal management agencies is working on improving lagoon management, in a multi-year, adaptive management process.
- Substantial data (historic inlet state, Estuary water surface elevations, nearshore waves, beach conditions) are available to evaluate the linkages between physical processes and the jetty.

However, modifications to the jetty should consider the following constraints and considerations:

- Preservation of public access and safety for GRSB beach-based recreation, including the Beach Parking Lot at the south end of the beach.
- Minimizing potential impacts to other species' habitats, e.g. marine mammals, endangered Tidestrom's lupine.
- The complex and dynamic beach environment makes it difficult to quantify linkages between management actions and project objectives, so predicting the effects of alternatives likely includes uncertainty.

To assess feasibility of jetty alternatives with the intent of improving the likelihood of achieving the target lagoon water surface elevations (ESA PWA, 2011), this study does the following:

- Describes the extent and composition of the jetty by researching historic documents, conducting geophysical investigations, and surveying the exposed jetty
- Characterizes the jetty's existing effects on the physical processes which affect Estuary water surface elevations, including beach permeability, sand storage, and sand transport. Analyses to characterize the jetty's existing effects included groundwater monitoring, geophysical analysis of seepage, ocean wave data analysis, ocean wave modeling, and geomorphic assessments. These analyses were used to develop and apply a water balance model of the Estuary water surface elevations.

- Evaluates the jetty's influence on flood risk to properties adjacent to the Estuary by considering three different potential flood scenarios: inlet closure, fluvial discharge, and ocean wave transmission
- Develops and analyzes jetty alternatives, such as jetty removal, partial removal, and notching for potential benefits to Estuary water surface elevation management, potential short-term and long-term environmental impacts and constraints, and probable costs.

Beach permeability, sand storage, and sand transport are physical processes which affect the lagoon water surface elevations and which may be significantly affected by the jetty. Evaluating and quantifying these linkages informs the development and evaluation of management alternatives for the jetty. Flood risk is not a part of the linkages between the jetty and water surface elevation management objective. Instead, increased flood risk is a potential negative impact of modifying the jetty that needs to be considered as part of alternatives' overall feasibility.

Sections 2 through 7 of this report provide an understanding of the existing physical processes which affect the estuarine water surface elevations and which may be significantly affected by the jetty. The findings of existing conditions sections then inform the development (Section 8) and evaluation (Section 9) of alternatives that modify the jetty and the feasibility assessment of implementing these alternatives. Additional technical work may be required to inform potential jetty removal impacts and design features if future proposals are made to modify the jetty.

The Water Agency intends to meet the objectives of the Estuary RPA while staying within the constraints of existing regulatory permits and minimizing the impact to aesthetic, biological, and recreational resources of the Estuary. The Water Agency's management approach is being developed in coordination with NMFS, the California Department of Fish and Wildlife (CDFW), and the California Department of Parks and Recreation (CDPR). The Biological Opinion does not require the Water Agency to implement any recommendations of the jetty study and implementation of any jetty modifications is currently without a funding or implementing agency.

The remainder of this introduction provides an overview of the physical setting of Goat Rock State Beach and the jetty and then summarizes the findings of the analyses conducted for this study. The following sections describe these analyses in more detail, which includes assessments of the jetty structure, ocean waves, beach morphology, and flooding.

1.3 Overview of Physical Setting

The focus of the existing conditions assessments is to understand the role that the existing jetty may play in formation of a barrier beach in the spring and fall and in

determining lagoon water surface elevations. A barrier beach is wave-induced build-up of sand in the inlet connecting the Russian River Estuary and the Pacific Ocean (Figure 1-1) that closes off the inlet. These connections between the jetty, barrier beach formation, and Estuary water surface elevations were quantified to inform the development and evaluation of alternatives in subsequent stages of the feasibility study. The first assessment, which informs subsequent assessments, is the characterization of the jetty's extent and composition, as much of the jetty is hidden within the beach and there is little historical documentation. Next, the study looks at three key processes by which the jetty may influence barrier beach formation and Estuary water surface elevations. These processes are groundwater permeability, beach morphology, and inlet morphology. A section on ocean wave conditions, a dominant determinant of both beach and inlet morphology, precedes the morphology sections. In addition, the study considers the potential impact of the jetty on flood risk.

Note that morphology, or the shape of the barrier beach, is the result of and therefore includes the sand storage and transport elements called for in the Biological Opinion. This study focuses on the morphology since the beach shape is the primary influence on lagoon water surface elevations. For example, to achieve the target water surface elevations defined in the Biological Opinion, the barrier beach morphology needs to close the inlet which connects the Estuary to the ocean. Morphology is examined at two scales: the entire beach and the inlet. Although they are related, these two geomorphic units are shaped by a different balance of physical forcing and are best evaluated by different analyses.

The Russian River Estuary is classified as a bar-built estuary in that its interface with the ocean is strongly defined by the barrier beach or sand berm at its mouth. Historic maps indicate that the barrier beach was a feature of the Estuary before the jetty was constructed. The beach is subject to hydraulic forcing by both the Russian River and the Pacific Ocean. Since it is comprised of mobile, well-sorted sand grains averaging 1 millimeter in diameter (EDS 2009), the beach is in constant state of motion. Much of this motion is oscillatory in nature, e.g. seasonal changes in waves, tides, and river flows, and yields little net change in morphology. However, when the balance between forcings tilts, change can be rapid, on the order of hours, and completely change the state of hydraulic connectivity between the Estuary and the ocean. For example, the inlet has been observed to go from completely closed to more than 200 ft wide in less than three hours (Goodwin and Cuffe, 1994).

The Russian River contributes sediment to the coast within the Russian River Littoral Cell, which is generally between the headland just north of the mouth and the Bodega Head headland, with net alongshore sand transport toward the south (Hapke et al., 2006; Griggs, Patsch and Savoy, 2005; Patsch and Griggs, 2007; all referencing Habel and Armstrong, 1978). The estimated yield of sand and gravel to the shore is about 140,000 cubic meters per year, which is reduced about 17% from the natural supply rate due to

the effects of upstream dams (Willis and Griggs, 2003). Watershed intervention also reduces peak flows during the winter and steadied summer low flows (Florsheim and Goodwin, 1993). Rates of shore change based on historic maps and aerial photographs show accretion in the vicinity of the Russian River mouth, and erosion just south of Goat Rock (Hapke et al., 2006). These changes may be related to the construction of the early 20th-century construction of an elevated road (now the Goat Rock Parking Lot, Figure 1-1) between the shoreline and the coastal headland Goat Rock that filled in a natural tombolo or sand spit. This constructed fill blocks the southward sand transport except for sand which bypasses around the seaward end of Goat Rock. The implication is that the beach at the Russian River mouth may be wider now than before the road to Goat Rock was built, and the effect of the jetty needs to be distinguished from the effects of other actions. It should be noted that the littoral cell has not been studied in detail, and refinements of the Habel and Armstrong (1978) cell descriptions have been made elsewhere. Behrens et al (2009) hypothesizes that alongshore sand transport affects the inlet morphology.

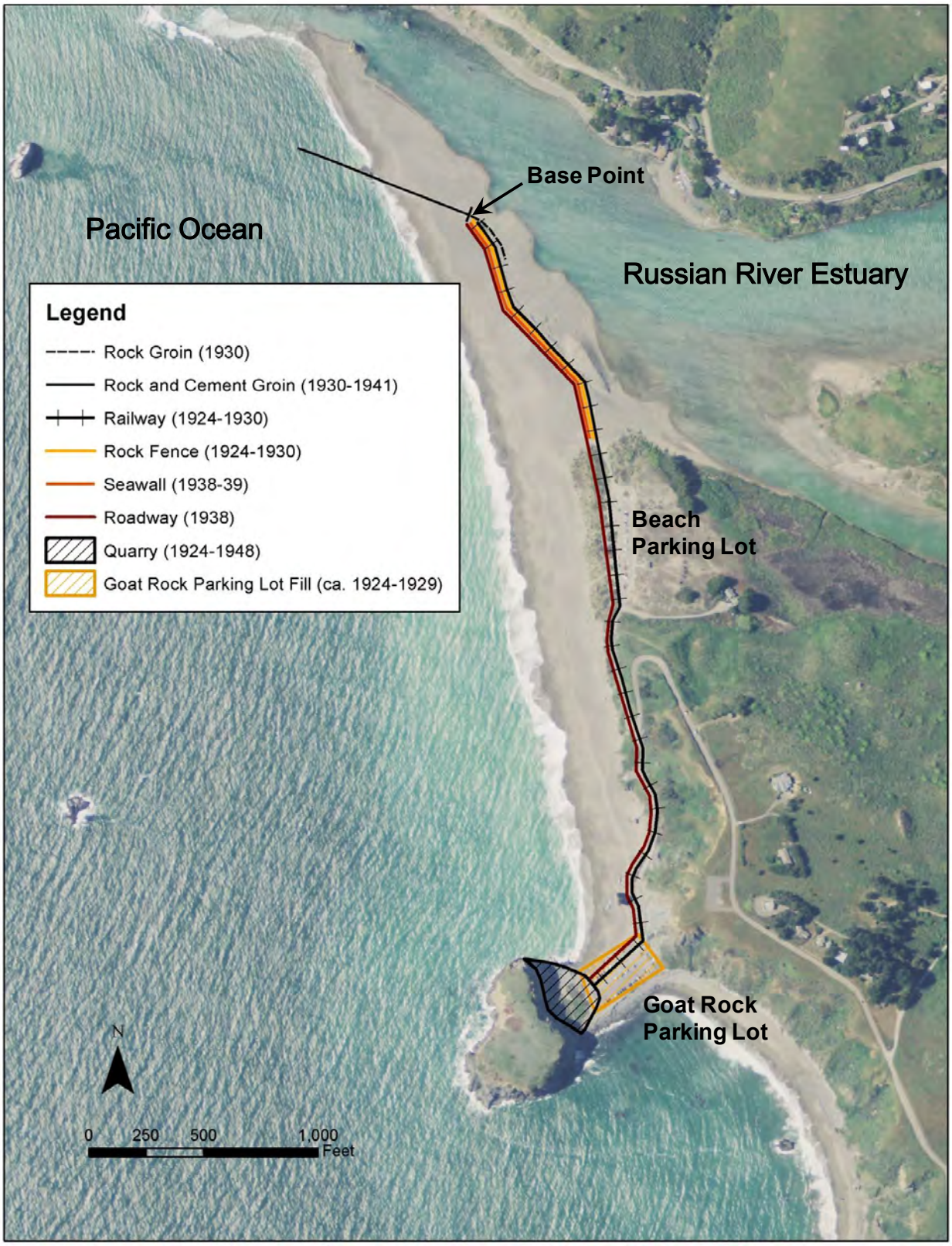
The inlet, the connection between the Estuary and ocean across the barrier beach, shifts between three states – tidal, closed, or outlet. When ***tidal***, the inlet allows the water surface oscillations associated with the tides into the Estuary and allows the river inflow to drain out. The fresh river inflow and ocean seawater mix within the Estuary, creating fresher conditions during the wet season's high river flow and more saline conditions during dry season's low river inflow. When the inlet becomes longer and/or constricted in cross section, muted tidal conditions can occur. Because muting limits saline ocean inflow, this state may promote fresher conditions in the Estuary that could improve salmonid habitat (NMFS, 2008). When muted, the inlet is susceptible to closure by the combination of ocean waves and tides creating elevated water surface elevations that deposit more sand in the inlet than flow through the inlet can remove. When the inlet is ***closed***, thereby forming a lagoon in the Estuary, river inflows cause the lagoon water surface elevations to rise. If the barrier beach is high enough, the water surface elevation may rise to the point of creating a flood risk to properties along the shore of the Estuary. To avoid flooding, the Water Agency breaches the beach with earth-moving equipment, returning to open tidal conditions. Observations in recent decades indicate that the Estuary is typically tidal, with an average of six closures per year (ESA, 2010). Closures typically last one or two weeks, and may end with the Estuary breaching by a management action to reduce flood risk or natural self-breaching. In response to the Biological Opinion, the Water Agency now strives to create the third state, an ***outlet*** channel, between May 15th and October 15th. The outlet channel is intended to convey flow over the barrier beach, while minimizing tidal and wave-induced inflow of saline ocean water into the Estuary. By doing so, the outlet channel may create a deeper freshwater surface layer in the lagoon while minimizing flood risk. Under target conditions, the outlet channel and groundwater seepage through the beach may convey water to the ocean at a rate matching river inflow. This balance of inflow and outflow would sustain roughly constant water surface elevations in the lagoon.

For the purposes of this study plan, the term 'jetty' is assumed to refer to the entire set of manmade structures that are north of the northernmost parking lot (Beach Parking Lot) at the Goat Rock State Beach, as shown in Figure 1-1. As described below, all of these components of the jetty may affect lagoon water surface elevations. The Goat Rock Parking Lot and roadway to south of the Beach Parking Lot are not considered part of the jetty because they do not lie between the Estuary and the ocean and are in active use by State Park visitors and staff. In this document, the components of the jetty are referred to with the following terminology:

- ***Jetty, jetty complex, or complex*** – All manmade structures on the Goat Rock State Beach north of the parking lot that were constructed to stabilize the inlet at the mouth of the Russian River and the sand spit between the Estuary and ocean.
- ***Groin*** – The northern portion of the jetty, which is constructed of rocks, several feet in diameter or larger, and capped with concrete.
- ***Access elements*** – Extending between the southern end of the groin to the parking lot, the access elements collectively consist of the roadway, seawall, railway, and rock fence, which were built to transport equipment and rock to the groin for construction and to prevent the inlet from opening south of the groin.

These components of the jetty are described in more detail in Section 2.

1.4 Figures



SOURCE:NAIP 2007 aerial image of Sonoma County. Construction diagrams are based on a DWR map provided by Johnson (1959) and historical information from Schulz (1942).
NOTE: The indicated positions of jetty elements are approximate, and are based on map and historical document interpretation.

Figure 1.1
Plan view of jetty complex elements constructed between 1930 and 1941.

2 JETTY STRUCTURE

Because much of the jetty is encased in the beach and documentation is minimal, the extent and composition of the much of the jetty's structure is uncertain. Therefore, a necessary first assessment of existing conditions is to describe the jetty's extent and composition in more detail. This description informs subsequent assessments of the jetty's role in the physical processes which determine barrier beach formation and lagoon water surface elevations. An assessment of the geometry and material properties of the structures is also needed for engineering evaluations such as construction quantities and costs.

2.1 Historical Information

The existing jetty on Goat Rock State Beach is the product of several construction phases that occurred between 1929 and 1948. The purpose of the jetty was to maintain a permanently open passageway to facilitate transport of mined gravel from the river; fish passage became an additional driver after the initial phase of construction. Construction was first performed by the Russian River Improvement Company (RRIC) with funds from the RRIC, private sources, the Fish and Game Preservation Fund, and the State of California, and later by the California Division of Water Resources with funding from the Fish and Game Commission and Sonoma and Mendocino Counties (Schulz, 1942). For the purposes of this study, the term 'jetty' is assumed to refer to the entire set of man-made structures that are north of the northernmost parking lot (Beach Parking Lot) at GRSB. The roadway and railway south of the Beach Parking Lot and the parking lot adjacent to Goat Rock (Goat Rock Parking Lot) are not considered parts of the jetty. We focus on the jetty elements north of the Beach Parking Lot because of their proximity to the inlet. As described above, the jetty north of the Beach Parking Lot is considered to have two main components, the groin and the access elements.

In total, over 100,000 tons of rock were quarried from Goat Rock and the adjacent bluff, then placed in excavated pits in GRSB to build the groin and a protective rock fence for the railway. An extensive amount of lumber was also used in the construction of the groin, seawall, and railway.

This section summarizes the existing information about the jetty complex found from a limited number of historic reports. A topographic survey of the exposed portions of the existing jetty structure and geophysical data (NORCAL, 2015) augmented the historic reports. The most detailed existing source of information is a 1942 report written by Schulz (1942) for the California Department of Public Works (DPW), Division of Water Resources (DWR). A planning document provided by Johnson (1959) details the planned

improvements to the groin drafted in 1938. Additional historical information is given by Rice (1974) and Magoon et al. (2008).

2.1.1 First phase of construction

The first phase of construction lasted from 1924 to 1934, and produced most of the existing jetty complex. While access elements and the quarry were probably initiated prior to 1930 (Magoon et al., 2008), most of the groin structure was constructed in the summer-fall of 1930. Quarrying at Goat Rock started in 1924 and initial construction of the railway between the quarry and the current location of the groin was initiated between 1924 and 1929 (Magoon et al., 2008). Construction was initially performed by the Russian River Improvement Company (RRIC), but funds were exhausted before construction of the groin could begin. The available records are not clear regarding the date when the gap between Goat Rock and the headland was initially filled (the site of the present-day Goat Rock Parking Lot). However, the existing planning document (Johnson, 1959) and written sources (Schultz, 1942) indicate that the railway extended to the base of Goat Rock, so the fill probably occurred no later than 1929, and may have occurred in the period between 1924 and 1929.

The State of California (State) became involved with the project in August of 1929, and construction of the groin began in the summer of 1930 (Schulz, 1942). The groin was originally planned as a 1,000-ft long timber frame filled with rocks, with a landward base point (base point) at approximately 38°27’00” N, 123°07’43” W (Figure 1-1). An open pit was excavated, possibly to an elevation of -12 to -16 ft mean lower-low water (MLLW) (Johnson, 1959) and quarried rocks were filled to a height of 17 ft MLLW (Schulz 1942). While the original top elevation of 17 ft MLLW is supported by Schultz (1942), the minimum elevation is based on a drawn cross-section from a 1938 planning document given by Johnson (1959) and shown in Figure 2-1, and is less certain. Rocks were filled by a crane into a timber frame to as far as 675 ft from the base point by July 8, 1930 (Figure 1-1 and Figure 2-2). This seaward portion is oriented approximately 290° from north. The groin was also built south of the base point for a distance of approximately 200 ft. This section is curved, but is approximately oriented in a shore-parallel direction.

Construction ended before completion due to exhaustion of funds. By April 1931, the entire structure had substantially subsided into the sandy substrate. The rock fill material more than 475 ft from the base station (Figure 2-2) had subsided to elevations below mean tide level[1] and waves destroyed the entire timber frame beyond this point. Additional funding from the State allowed the addition of more rocks to the landward portion of the jetty by October 1931, as far as 475 from the base station, and a steel

[1] Mean tide level is the term used by Schultz (1942), but he does not specify the source of this tidal datum.

frame was extended an additional 225 ft past this point. The steel frame was intended as a more durable replacement of the earlier timber frame. However, in the ensuing months, the segment of the structure more than 400 ft from the base station continued to subside. To account for this, additional rocks were used to reinforce the base on the landward section of the structure and to begin filling of the steel frame in the seaward portion. By October 1932, rock had been added as far as station 540 ft from the base point. For the next year, rock was added sporadically while the jetty continued to subside. Waves during the winter of 1933-1934 destroyed the steel frame, and continued subsidence in the subsequent year fully submerged the seaward portion of the jetty.

In total, approximately 90,000 tons of rock quarried from Goat Rock were used during the first phase of construction, of which about two thirds were used on the groin and one third to build a protective fence for the railway, which spanned a distance of 3,500 feet between eastern edge of Goat Rock and the eastern edge of the groin (see Figure 1-1). Schulz (1942) suggests that the wood and steel railroad structure and rock fence had the effect of partially stabilizing the beach, although this may have been a result of these elements preventing southward inlet migration. This stabilization may also have resulted from blocked littoral sand transport between Goat Rock and the adjacent headland. Since construction of the jetty, the southward littoral sand transport is blocked by the Goat Rock Parking Lot (Figure 1-1), where engineered fill overlies what had previously been a low sand spit or tombolo. By blocking southward sand transport, this parking lot fill may cause more sand to accumulate on Goat Rock State Beach, a potentially stabilizing factor. Historic changes to beach morphology are further discussed in Section 5.

2.1.2 Second phase of construction

The second phase of construction began in 1938 in response to a report from the Deputy State Engineer, which recommended further construction by the State to “conserve the existing structure and to improve conditions at the bar for the passage of fish.” The main tasks for meeting these ends were (Schulz, 1942):

1. To create a seawall along the spit south of the groin, in order to prevent the inlet from opening south of the groin,
2. To add more rock to the existing jetty as far as 600 ft from the base station (a scaled back version of the original 1,000 ft-long jetty), as well as a concrete cap and a concrete endpiece structure at the seaward end of the groin, and
3. To provide better conditions for a channel immediately north of the jetty by dredging rock displaced into this region by settlement of the original structure.

The majority of funding was provided by the Division of Fish and Game with smaller amounts made available from Sonoma and Mendocino Counties. The State entered

negotiations with the Federal Public Works Administration (PWA) to provide additional funding, but delayed response and rapidly rising costs of construction materials forced DWR to oversee construction without federal help in 1938. The contractor for the work was Basalt Rock Company (now SYAR Industries), based in Napa, California.

The seawall was constructed in the winter of 1938-1939 (Figure 1-1). It was designed to raise the crest of the beach berm south of the jetty by preventing sediment overwash into the lagoon and encouraging deposition of sand on the ocean side of the structure. The expected result of these actions was the prevention of inlet breaching south of the jetty, which was considered a less favorable location than adjacent to the jetty on its north side (Schultz 1942).

The seawall was constructed entirely from timber and consists of vertical posts connected with horizontal beams, supporting vertical layers of thin redwood sheeting (facing the ocean; Figure 2-1). The wall was built 25 feet oceanward of the railway. Construction on the wall was completed on February 4, 1939, and included the placement of 150 vertical 24 ft-long timber posts, which were driven into the sand until they penetrated to depths of 18-22 ft. The depth of placement varied along the beach. At the northern end of the seawall (near the groin base point) the top of the posts were at an elevation of 22.5 ft MLLW and increased southward to an elevation of 23.5 ft MLLW near the present-day Beach Parking Lot. These posts were placed 8 ft apart (Figure 2-1) and connected by the horizontal wood beams attached near the top and middle of the vertical posts. A double-layer of 2-inch thick redwood sheeting (Figure 2-4c) was fixed to the timber frame and placed in the beach by first digging a trench along the Estuary side of the seawall that was deep enough to fit the sheets. This method was needed because they were too fragile to be driven into the sand (Schulz, 1942). The redwood sheets were 16 ft long, and were affixed to the top of the posts (Figure 2-1). This suggests that at the time of construction they penetrated to depths of 6.5 ft MLLW at the north end of the seawall and 7.5 ft MLLW near the Beach parking lot (Schulz 1942).

During the second phase of construction, trucks were used to transport quarried rocks to the groin rather than the existing railway. To make this possible, a roadway embankment topped with compacted rock fill was built from the base of the quarry to the groin (Schulz, 1942). On the sand spit between the present-day Beach Parking Lot and the groin, the roadway was built parallel to the seawall on its ocean side (Figure 2-1). The roadway is visible in aerial photographs in 1945 (see Section 5), but not afterward, suggesting that major parts of it may have either been buried by sand or destroyed by waves. Schultz notes that most of the roadway was destroyed by a storm in the winter of 1940. Five thousand tons of quarry rock were used in the construction of the roadway (Schulz, 1942), but it is unclear whether the rock was crushed and used solely for the road surface at the top of the embankment or whether larger rocks were also used to underlie the roadway surface. Because of this, it is unclear how deep the

road embankment penetrates into the beach at present. Figure 2-3 suggests that near the parking lot, the road surface is only 1-2 feet thick and composed solely of compacted rock, but it is unclear how deep the embankment penetrates into the beach in other locations.

By 1940, the surface of the original jetty structure had settled from 17 ft MLLW to 15 ft MLLW at the base point and from 17 ft MLLW to -5 ft MLLW 600 ft seaward of the base point (Figure 2-2). In addition, rocks used in the original construction had also spread laterally due to wave action (Schulz 1942). Before reinforcing the groin structure, crews excavated 32,500 cubic yards (yd^3) of sand from the base station to as far as 400 ft seaward, which had buried most of the original rock used in its construction. Between October 1940 and June 1941, 10,700 tons of newly-quarried rock from Goat Rock was placed on top of the excavated structure and also used to stabilize the toe of the groin at its seaward endpoint. During this time, 500 tons of rock were removed from the area just north of the groin, so that the base of the groin no longer encroached on the desired inlet channel area.

The groin was capped with concrete after rocks had been placed in each segment. In total, 1,650 yd^3 of concrete were used in the construction of the new groin and cap. The structure used both quick-drying and seawater-rated concrete. The quick-drying mix was used in areas of the jetty closest the ocean, where tides and waves made it impossible for the concrete to fully set. The cap was made in 20 ft segments, and the bottom edge of the concrete extended to 8 ft MLLW on the ocean side and 11 ft MLLW on the river side (Schulz 1942). To reach the ocean end of the jetty as quickly as possible, alternate 20 ft segments were built, with the gap sections filled later. By November 14, 1940, the cap was mostly complete to as far as 372 ft from the base point, but a severe storm from November 14-18 severely damaged the structure, fracturing the section between 313 and 333 ft from the base point. It also caused a crack to form along the entire structure, which was repaired when the gap sections were filled. The cap was completed to as far as 485 ft from the base point by June 12, 1941. The end piece was completed on June 16, 1941, and consisted of a 20-ft long, 3-sided steel truss structure filled with concrete. This extended approximately from 485 to 505 feet from the base point (Figure 2-2). Rock was placed an additional 45 ft past this point to create a slope of 2 to 1 between the cap of the structure and the ocean floor (Schulz 1942).

With the exception of the eastern end, the capped groin was built to a height of 17 ft MLLW. The original plans called for the top of the groin to be level, but these were modified by raising the section closest to the base point. Under the new design, the jetty sloped down from an elevation of 21 ft MLLW at the base point to 17 ft MLLW at a distance 120 feet seaward of the base point. The purpose of this change was to funnel wave overwash over the jetty and into the inlet channel. Overwash had previously been observed to run parallel to the groin on its south side (Schulz, 1942). It was presumed

that sand transported by the funneled overwash onto the river (northern) side of the jetty would allow the sand to be removed by currents (Schulz 1942). During this time, rocks were also added to the curved portion of the groin south of the base point. These were mostly placed on the river side of the groin, and the entire mass was grouted together (Schulz, 1942).

2.1.3 Maintenance

The jetty continued to subside after 1941, spurring further construction in 1948 (Magoon et al., 2008). This was more limited than the previous actions. An estimated 4,280 tons of quarry stone were used to armor the seaward portion of the groin. On the ocean (south) side of the groin, rock placement was designed to create a 2H:1V slope. A sketch given in Magoon et al. (2008; Figure 2-1) suggests that these were placed from the surface of the groin (17 ft MLLW) to a maximum depth of 0 ft MLLW, and to a lateral distance of about 35 ft from the groin edge to the ocean. An unspecified amount of rocks were also placed seaward of the jetty end piece and a berm of rocks was also laid on the river (north) side of the jetty. In contrast to the ocean (south) side rock placement, on the river side these were only laid to a peak elevation of 4 ft MLLW and to lateral distance of 15 ft from the jetty to the river. After placement, the rocks on both sides of the jetty were filled with a total of 651 yd^3 of Portland cement.

2.2 Extent of Existing Structures

As shown in Figure 2-3 and Figure 2-4, much of the remaining jetty complex is still in place. The concrete-capped part of the groin still occupies a length of approximately 505 ft (Figure 2-4). Remnants of the seawall and the rock fence intended for protecting the railway are visible for most of the distance between the base point of the present-day groin and the Beach Parking Lot. Remnants of the roadway and railway are also visible in an overwash ravine immediately north of the Beach Parking Lot (Figure 2-3), but it is unclear how much farther these extend north past this point. The present-day Goat Rock Parking Lot occupies most of the former quarry and fill area between Goat Rock and the headland. It is unclear at present whether access elements such as the railway or early access roadway were eventually removed within and south of the Beach Parking Lot or filled over by the present-day parking lot and access road.

On March 20, 2014, ESA PWA surveyed the exposed portions of the jetty elements north of the southern Beach Parking Lot. Estimates of potential subsurface alignments were inferred from exposed elements. The horizontal extents of the existing jetty elements based on this survey are shown in Figure 2-5 and the elements' approximate elevations are summarized in Table 2-1.

Survey data from March 2014 indicates that the top of the concrete cap of the groin may have subsided several feet since the last phase of construction in 1948. The 2014 survey data indicate that the top of the groin slopes from an approximate elevation of 16-17 ft NGVD at its landward base point to approximate elevation of 12 ft NGVD at a distance 175 ft seaward of the base point (almost halfway along the length of the groin). The groin's original reported elevation was approximately 15 ft NGVD[1] (Schulz, 1942). This suggests that settlement of 2-3 feet may have occurred for the seaward portions of the groin, assuming the 1938 planning detail of the groin (Figure 2-1) is accurate along the entire length of the groin. This further suggests that the base of the groin may be roughly -16 to -21 ft NGVD at present and the legs of the concrete cap on the groin may have also subsided to a bottom elevation of about 3-4 ft NGVD on the ocean side and 6-7 ft NGVD on the river side of the groin, with the rock and cement armoring placed in 1948 possibly lying beneath these elevations.

Two surveyed cross sections, one with access elements protruding from the berm surface, one with no visible access elements, exhibits different beach dimensions. As seen in Figure 2-6, the cross section with the access elements, XS 1, is approximately two feet higher than the cross section without access elements, XS 2. Where the berm drops into the Estuary, the elevation of XS 2 exceeds that of XS 1 where the wave-transported sand has splayed out into the Estuary.

The original seawall and rock fence (Figure 1-1) are exposed in some areas, with some of the vertical posts shown in Figure 2-4d and the redwood sheeting shown in Figure 2-4c. The groin and remaining access elements have been degrading since their original construction. The ongoing degradation of the groin is particularly visible in ground-based photographs taken in different years (Figure 2-7). Degradation of the access elements is visible in the available series of aerial photographs used to assess the historical changes in beach morphology (see Section 5). The roadway is no longer visible in aerial photographs taken after 1945. Parts of the rock fence and the portion of the groin south of the base point show signs of degradation starting in 1965.

Assessment of the subsurface with ground-penetrating radar (NORCAL, 2015) indicated the jetty is in various states of degradation along its length (Figure 2-5). In addition to the places where jetty elements are visible at the surface and persist with depth, the ground-penetrating radar detected jetty elements under several feet of sand even where there was nothing visible on the surface. In one instance, the subsurface jetty material was displaced more than thirty feet inland from the jetty's original alignment, probably due to wave action. At two transects, the ground-penetrating radar did not

[1] The construction documentation uses mean lower low water (MLLW) as a vertical datum. To convert from MLLW in the old epoch (1960-1978) to NGVD, the following conversion was used: 0 NGVD = 1.55 ft MLLW.

detect any returns from possible jetty material down to depths of approximately 15 ft, the radar's detection limit.

The last aerial image to show what appears to be an unbroken groin and rock fence structure was taken in 1986. After this time, parts of both of these elements disappeared from the aerial images, possibly a result of either removal or sand burial induced by wave overwash. At present, a 30-40 ft gap exists in the groin immediately south of the base point and several gaps exist in the rock fence (Figure 2-3, Figure 2-4d), probably due to wave overwash.

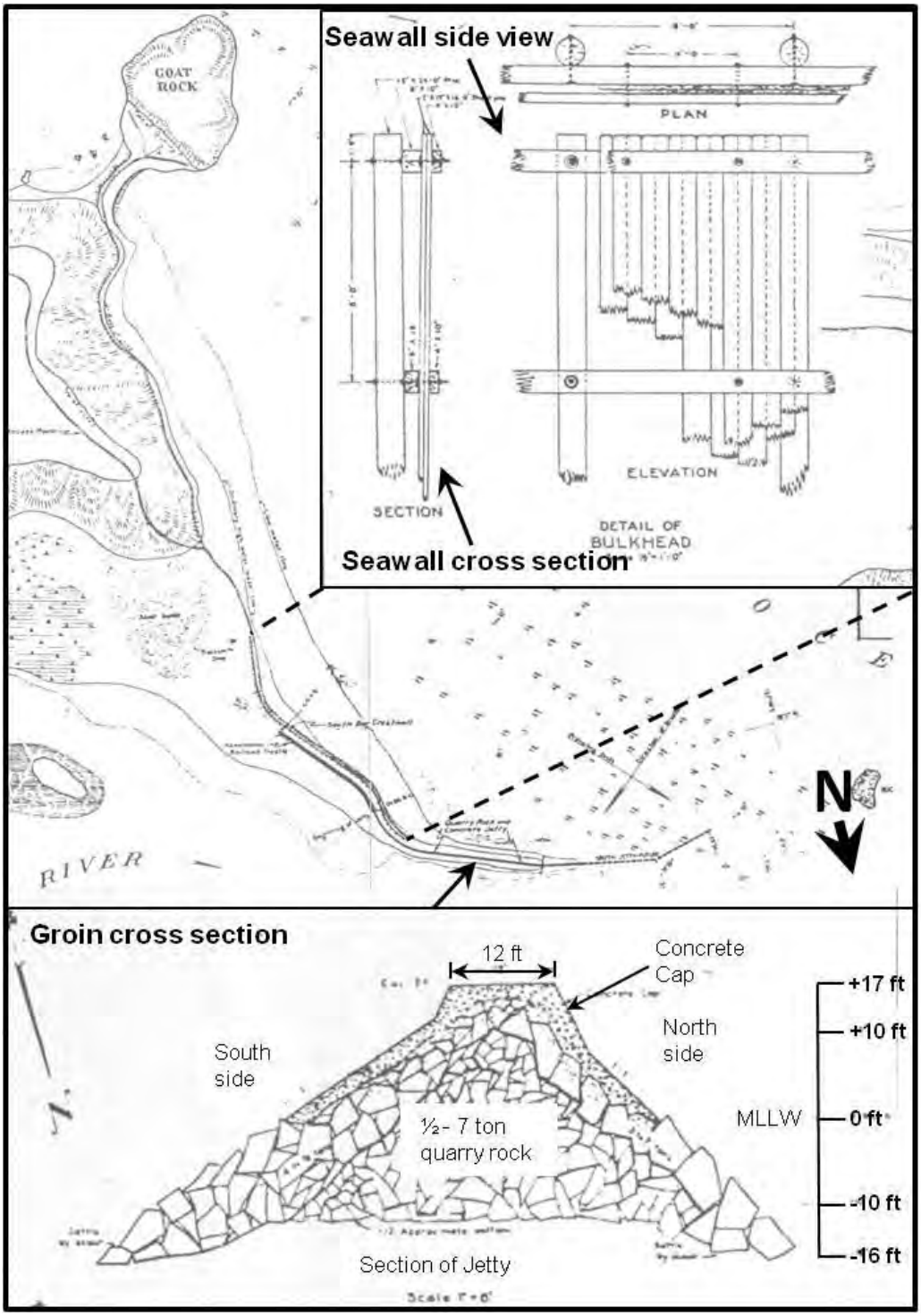
Although the original dimensions of the seawall are known, the depth of rock placement in the rock fence is not. The amount of quarried rock used in the rock fence construction is known to be approximately 30,000 tons, but it is not clear from Schulz (1942) whether this was distributed across the entire 3,500 ft length between Goat Rock and the groin or only along the section between the Beach Parking Lot and the groin. Using the same datum shift described above, the seawall can be assumed to penetrate to elevations of roughly 7-8 ft above MLLW. However, since no survey data exists from the construction period, it is unclear if the beach has shifted relative to MLLW as a result of the access elements altering wave overwash. The redwood planks used to build the seawall are known to have been fixed to the top of the posts (which are still present, but may be somewhat degraded), so the vertical extent of the sheets within the beach was estimated from surveyed elevations on the post tops.

Table 2-1. Extent and material of the elements of the jetty complex on GRSB north of the Beach Parking Lot.

Constructed Element (Time of Completion)	Approx. As-Built Endpoints[1] and (Length)	2014 Survey of Top Elevation (ft NGVD)	Approximate Bottom Elevation[2] (ft NGVD)	Material[3]
Railway(1930)	38°27'00" N, 123°07'42" W; 38°26'52" N, 123°07'35" W; (~1000 ft)	14-15	unknown	Timber pilings, steel railings
Rock Fence (1930)	38°27'00" N, 123°07'42" W; 38°26'52" N, 123°07'35" W;	13 to 20	unknown	Quarried rock (~ 30,000 tons),
Rock and Concrete Groin (1934, 1941, 1948)	38°27'00" N, 123°07'43" W; 38°27'02" N, 123°07'49" W; (505 ft)[1,3]	12 to 17	-21 to -16	Quarried rock (~75,000 tons)[4], concrete cap
Seawall (1939)	38°27'00" N, 123°07'42" W; 38°26'52" N, 123°07'35" W; (1000 ft)[3]	15 to 19	-9 to -5 (24-ft posts) -1-3 (16-ft planks)	Timber posts and 2 in-thick redwood sheets
Roadway (1940)	38°27'0.0" N, 123°07'42.6" W; 38°26'51.6" N, 123°07'35.1" W; (~1000 ft)[3]	19	Lowest observed: 14 (likely extends lower)	Compacted rock, possibly additional rock base in some areas

[1] Based on aerial photographs and map provided by Johnson (1959).
[2] Based on information in Schulz (1942) and map provided by Johnson (1959). Because of the lack of additional sources, these estimates should be considered approximate.
[3] Based on information in Schulz (1942).
[4] Roughly 90,000 tons in the first phase and 10,000 tons in the second phase (Schulz, 1942). Rocks placed loosely together without a concrete fill (Schulz, 1942).
[5] Estimated near parking lot based on observed height of roadway in Figure 2-3 relative to adjacent seawall. The roadway is not visible near the groin. It may be submerged in the beach or destroyed.
[6] Engineering coordinates given in Magoon et al. (2008), but it was not clear how these related to earlier coordinates given by Schulz (1942).

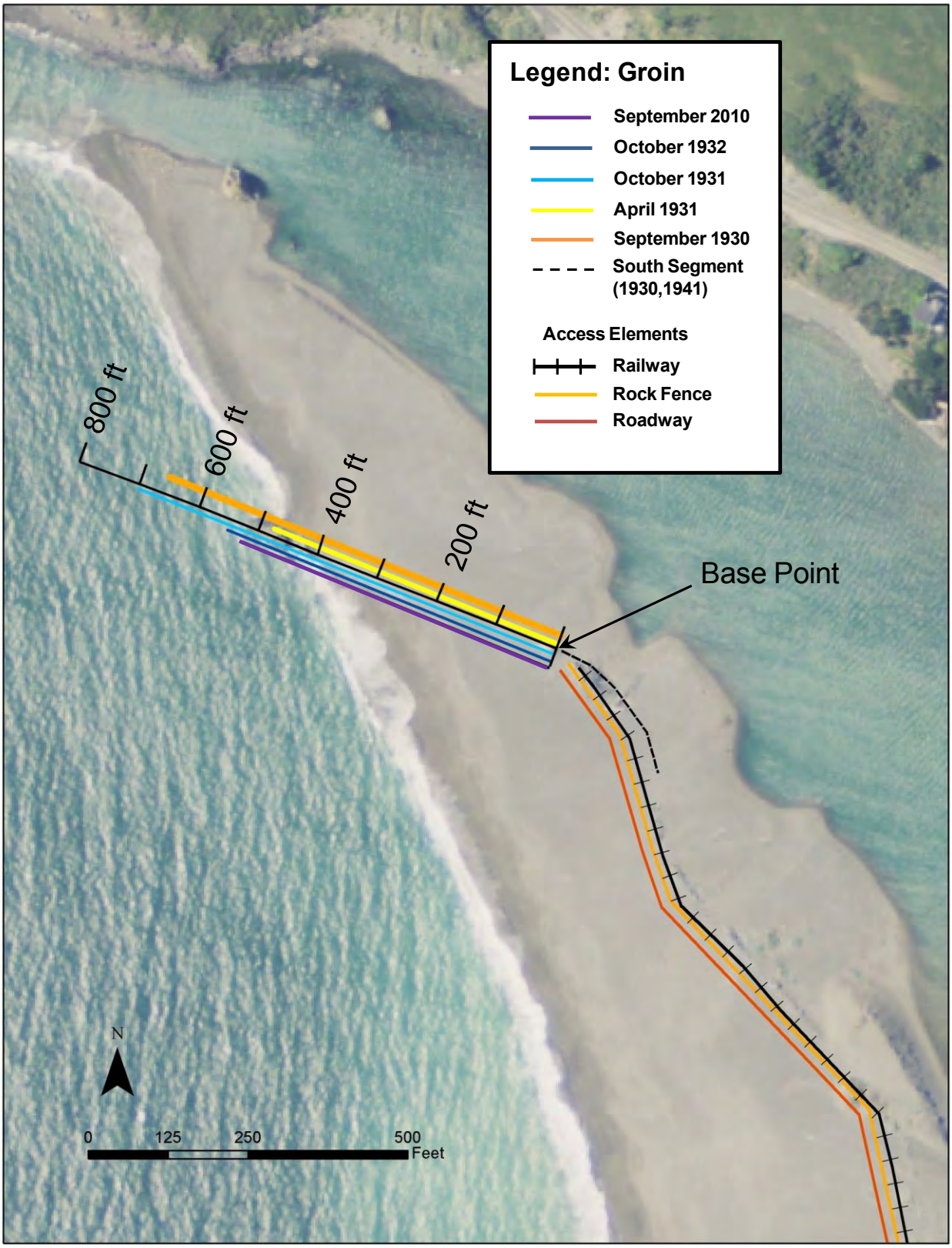
2.3 Figures



SOURCE: DWR (1938), as reproduced in Johnson (1959)

Figure 2-1

1938 planning document with detail of the seawall and groin cross section.



SOURCE:NAIP 2007 aerial image of Sonoma County. Construction diagrams are based on a DWR map provided by Johnson (1959) and historical information from Schulz (1942).

Figure 2-2

Plan view of groin construction: 1930-32

SOURCE: M. Brennan, May 19, 2010.

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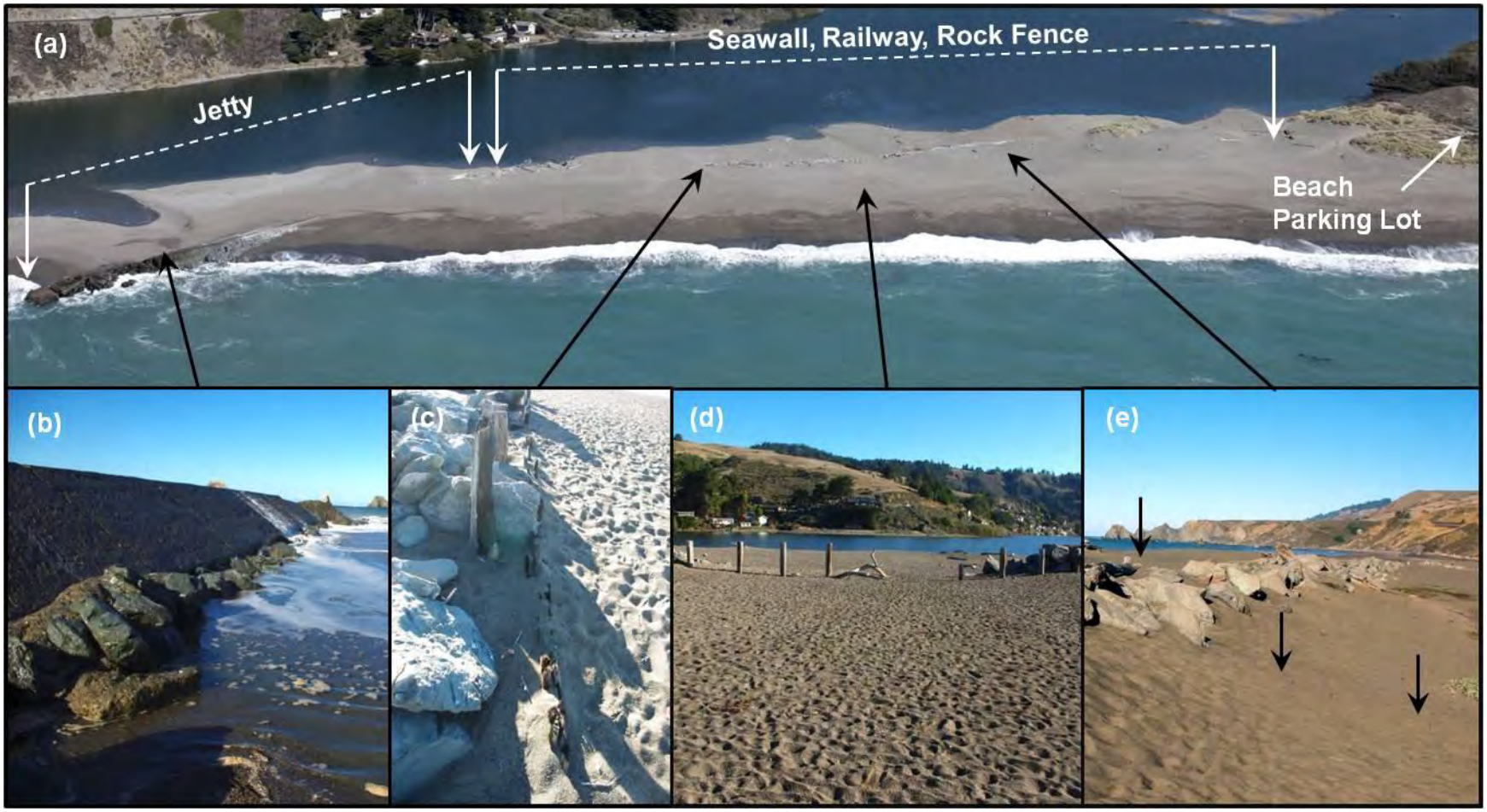
Figure 2-3

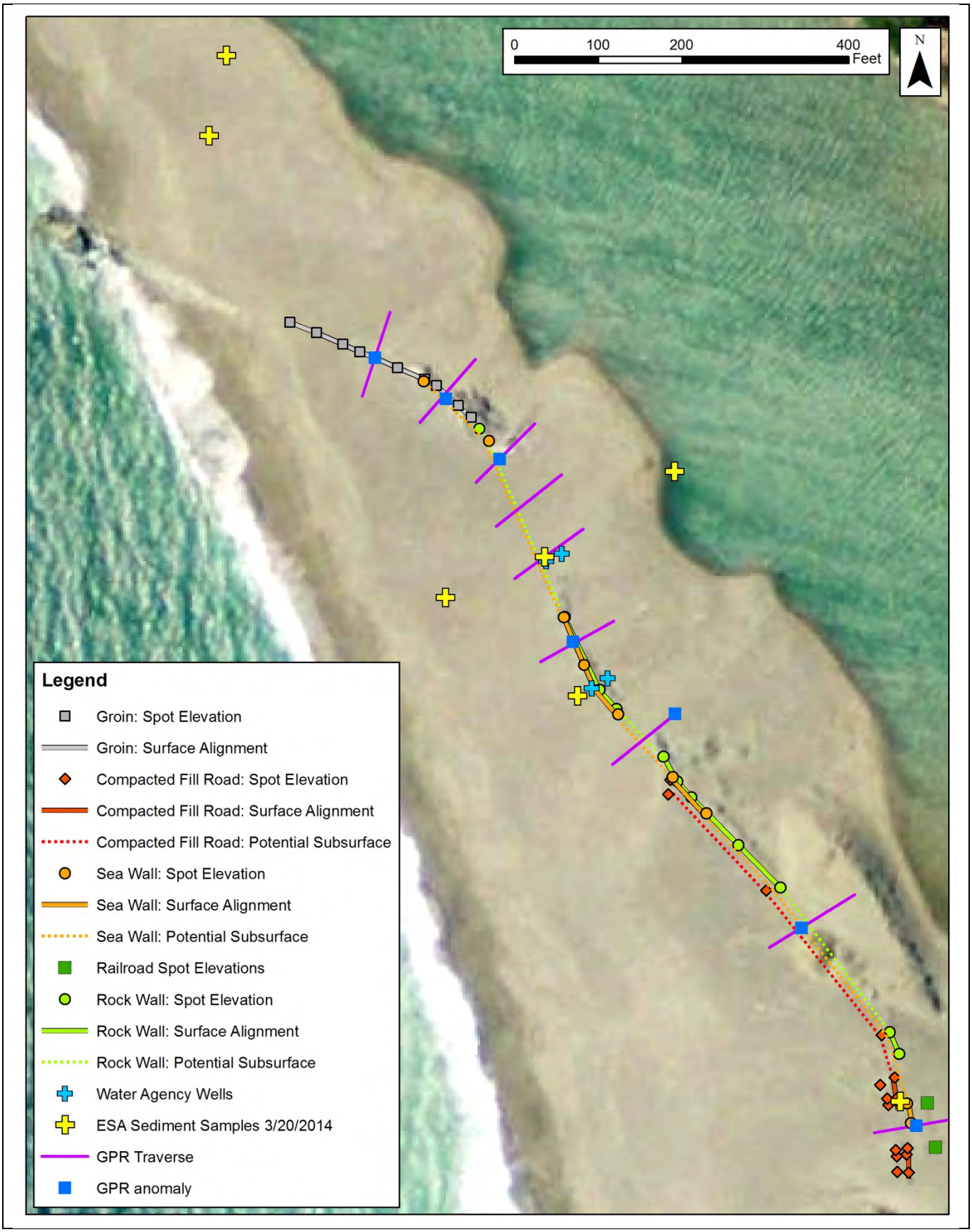
Site photographs of jetty access elements.



NOTE: Beach visitor parking lot located at far right edge of (a). (b) is a detail of the northern edge of the groin, showing the top of the armoring added in 1948. (c) shows the double-layer redwood sheets protruding through the beach. (d) is a wave washover fan where roadway, seawall and railway elements have been destroyed. In (e), note the difference in beach elevation at either end of remaining rock fence, indicating deposition of wave overwash on the ocean side.

Figure 2-4

Detail of the present-day jetty complex (a), with insets of (b) the groin structure, (c,d) seawall elements, and (e) the rock fence.



SOURCE: NAIP 2007 aerial image of Sonoma County.
GPR traverses and anomalies are based on Plate 3 of NORCAL (2015)

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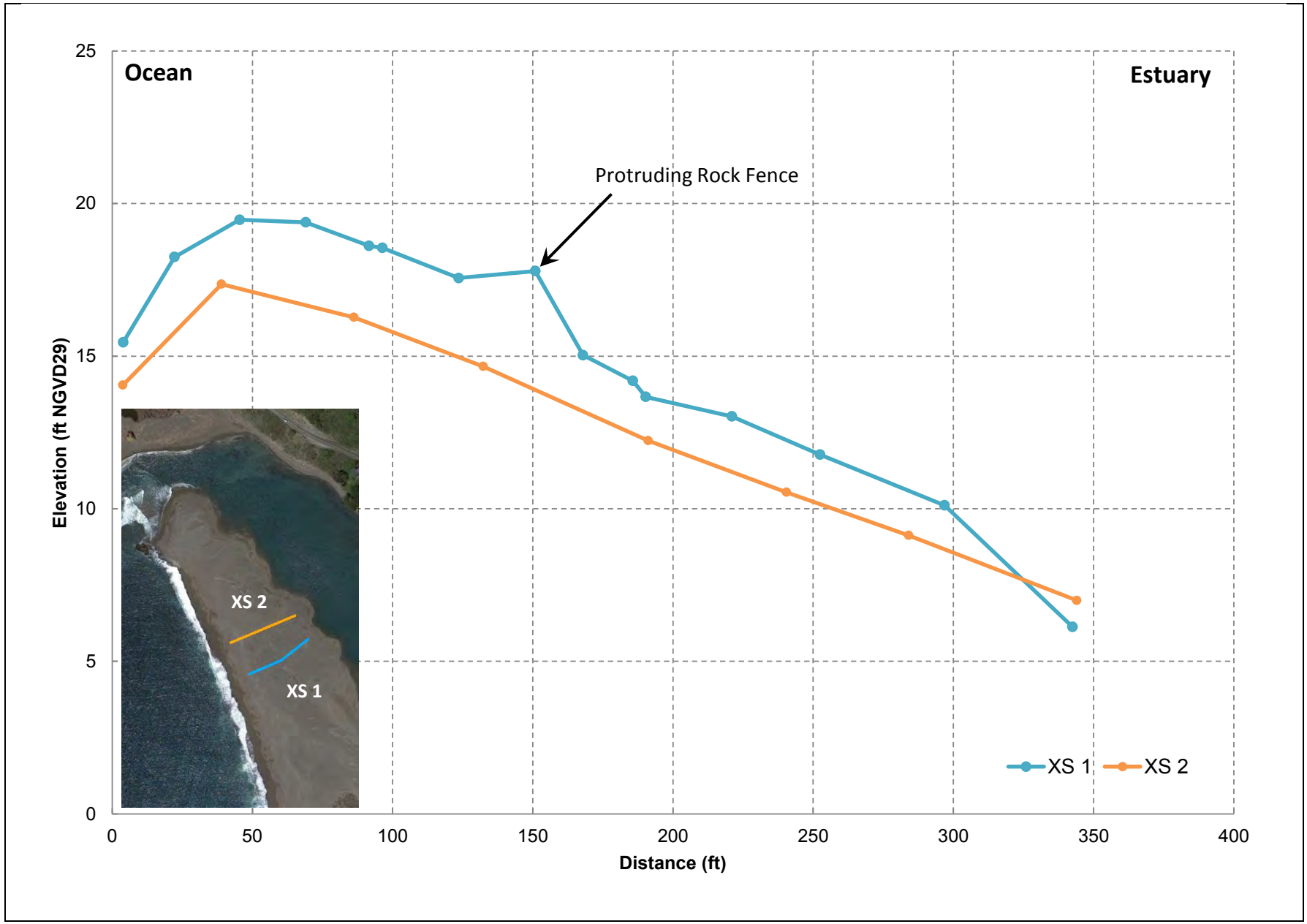
Figure 2-5

Plan view of jetty elements

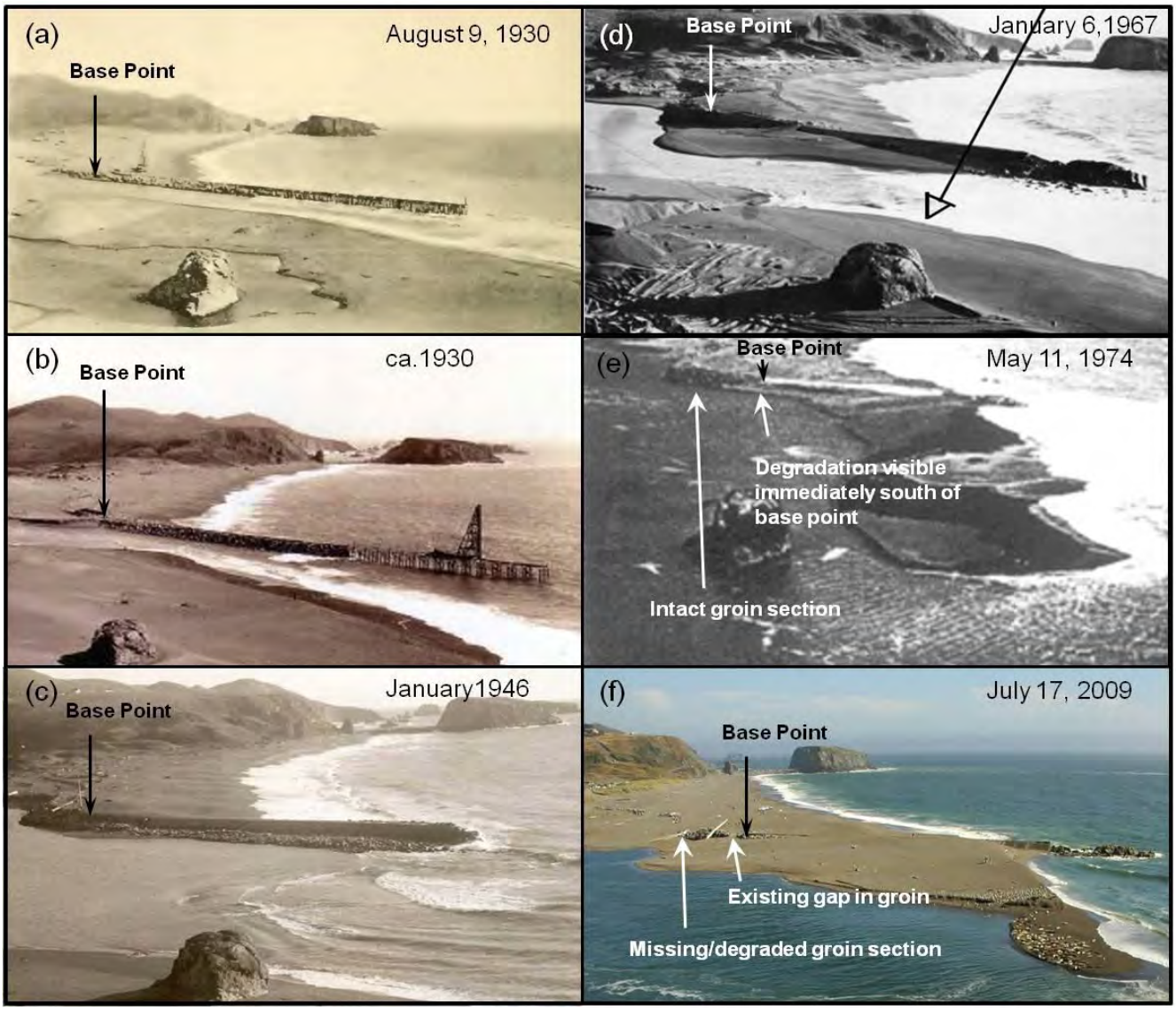


SOURCE: ESA survey

Goat Rock Jetty Feasibility Study . D211669.00

Figure 2-6

Cross shore beach profiles.



SOURCE: (a) DFG (1930), (b) Sonoma County Library (c) Magoon et al. (2008), (d) J. Dyer, (e) Rice (1974), (f) C. Delaney.

NOTE: Arrow in (d) is an unrelated drawing on the photograph. Base point in (e) visible as the last farthest left (east) extent of the concrete cap.

Figure 2-7

Site photographs of jetty from 1930 to 2009.

3 GROUNDWATER PERMEABILITY

In the context of this report, groundwater permeability[1] is the property of the barrier beach which determines the rate of groundwater flow through the beach. Groundwater flow results from water surface elevation differences between the Estuary and the ocean. Permeability through beach sand is largely determined by the sand particles' size, type, and arrangement. Jetty materials embedded in the beach, such as large rock, concrete, and redwood planks, may modify the permeability. Groundwater permeability affects lagoon water surface elevations because permeability determines the seepage rate at which water from the lagoon exits to the ocean through the barrier beach. When the Estuary is either closed or perched due to the establishment of an outlet channel, it is estimated that one of the major sources of outflow from the lagoon is seepage flows through the barrier beach (Largier and Behrens, 2010; Behrens 2012).

Previous efforts at estimating the beach seepage rate have relied on water balance approaches. Largier and Behrens (2010) estimate the total seepage rates from the lagoon to range between 30-80 cubic feet per second (ft^3/s) and to average 60 ft^3/s. Their estimates suggest that seepage rates increase by approximately 20 ft^3/s per foot of water surface elevation difference between the Estuary and the ocean. Although this analysis demonstrates a relationship between water surface elevation difference and seepage, there is substantial, unexplained scatter in the relationship, e.g. ±20 ft^3/s for a specific water surface elevation difference. Modeling of the salt field suggests that most of the seepage occurs in the upper horizontal layer of the stratified Estuary (Largier and Behrens, 2010; Behrens 2012), which overlaps with the elevation range of the jetty. Some of the seepage could also be directed laterally into the Estuary's inland aquifer. While these analyses estimate the order of magnitude of beach seepage, they are not specific about the spatial variability of seepage throughout the beach resulting from the presence of the jetty.

To provide a clearer picture of the jetty's subsurface structure and impact on seepage, Lawrence Berkeley National Laboratory (LBNL) and NORCAL Geophysical Consultants, Inc. (NORCAL) conducted a series of field studies in 2014 at GRSB. Concurrently, the Water Agency installed a series of monitoring wells in 2014 and plans to continue monitoring the wells for up to three years. Findings from these studies are described below.

[1] Permeability is a general term that is independent of the type of fluid passing through the porous medium. When dealing exclusively with water, the related proportionality constant 'hydraulic conductivity' is often used. We maintain the term 'permeability' for consistency with the Biological Opinion.

3.1 Summary of Field Studies

On July 21-24 and 29, 2014, NORCAL performed surveys to characterize the physical properties of the beach berm and jetty elements (NORCAL, 2015). On March 4, September 30, and October 16 - 22, 2014, LBNL used electrical sensors to assess the groundwater flow (LBNL, 2015). The latter period coincided with inlet closure, allowing comparison of seepage conditions during tidal inlet and closed conditions. Because of permitting restrictions and concerns about disrupting marine mammals, NORCAL's and LBNL's surveys were limited to the beach berm between the Beach Parking Lot and the groin. The field work included:

- Seismic refraction: Used to characterize the geometry of bedrock beneath the beach (Figure 3-1)
- Ground penetrating radar (GPR): Used to determine the presence or absence of jetty elements below the beach surface (Figure 2-5)
- Electromagnetics (EM): Used to determine whether certain layers of beach sediments channeled more flow than others (Figure 3-2)
- Electrical resistivity tomography (ERT): Used to determine the subsurface layering of different soil and rock types and also used in concert with electromagnetics to understand flow paths in beach sediments

The Water Agency also installed and monitored groundwater wells within the beach berm. These wells have sensors to monitor water surface elevation as well as the water conductivity (used to estimate salinity) and temperature. The Water Agency's monitoring complements the study by providing longer-term groundwater observations. Five long-term monitoring wells were installed that were drilled roughly 40 feet deep. These straddled the buried seawall in two transects: one with the seawall visible at the surface and one where the seawall was absent. Three additional shallow (~ 7 feet deep) monitoring wells were installed downslope (toward the Estuary) of the deeper wells. The five long-term monitoring wells (Figure 2-5) were equipped with temperature, conductivity, and pressure sensors, as well as a set of ERT cables for a brief period. Data collected from the well monitoring were analyzed and interpreted by LBNL (2015).

This set of beach berm and groundwater monitoring provides a better understanding of the potential flow paths, obstructions, and local flow rates through the beach berm, including the influence of the jetty on groundwater flow. The studies' key findings include:

- **Buried jetty elements:**
    - GPR surveys indicate the seawall may have deteriorated and is absent in the beach in areas where it is not visible at the surface (Figure 2-5).
    - GPR surveys suggest the concrete cap is degraded but still in place in buried portions of the groin

- **Bedrock** (Figure 3-1):
    - The surface of the bedrock below the beach is highest immediately north of the Beach Parking Lot (approx. -15 to -30 feet NGVD).
    - A depression in the bedrock elevation (approx. -60 to -90 feet NGVD) occurs about 500 feet south of the tip of the groin.
    - The bedrock elevation dips below -90 feet NGVD at the groin.
- **Groundwater flow**:
    - EM and ERT results both indicate stronger subsurface flows where the seawall is absent in the beach (Figure 3-2).
    - Seepage was estimated from monitoring well data to be 3.7 times faster (36-50 feet/hr) in sections where the seawall is absent than in sections where it was present (10-14 feet/hr).
    - Freshwater flows through the beach in a contiguous layer above a deeper layer of saltwater in the beach. The interface between the fresh and salt layers is deeper where the seawall is absent.
    - Monitoring well data suggest that freshwater flow through the beach from the Estuary to the ocean increases during low tides, as conductivity records show variations in the groundwater's conductivity.
    - Where the seawall is absent, the conductivity responds rapidly to low tides, quickly becoming fresh. Where the seawall is present, the conductivity signal is muted and slower to respond to changes in tide.
    - During inlet closure, the groundwater within the beach berm gradually converts to almost completely freshwater above about -8 feet NGVD. Deeper, at approximately -18 feet NGVD, salty and fresh conditions alternate. At the bottom of the wells (about -25 feet NGVD), the groundwater is persistently salty.

3.2 Implications

The results suggest that the jetty access elements reduce seepage by slowing flows where the seawall is still present in the beach (Figure 3-2). The NORCAL and LBNL studies did not detect a change in seepage rate in the immediate vicinity of the groin, both because the study area did not extend north of the groin and because the groin itself limited the effectiveness of the monitoring methods for detecting subsurface characteristics of the groin and associated flow. Since the groin's cross-sectional area relative to the groundwater flow direction is small relative to the entire beach berm subject to seepage and since any flow pathways through gaps in the groin's rocks are would likely be packed with sand, the groin is not likely to influence the overall seepage rate.

Depressions in the bedrock south of the groin and directly underneath the groin (Figure 3-1) may suggest that different river mouth configurations occurred in the past, with the inlet sometimes occupying positions farther south along the beach, as has been shown in photographs. The southern extent of the past depressions is about 750 ft south of the groin and about 850 ft north of the Beach Parking Lot.

The seismic refraction survey conducted by NORCAL detected a zone of low seismic velocity in a 300-ft stretch of the beach berm just north of the Beach Parking Lot (Figure 3-1). They attribute the low seismic velocity to dissolved gases released by decaying organic material in the layer, which spans elevations from about 0 to -15 ft NGVD. We interpret this buried organic layer as consisting of former marsh. Currently, vegetated marsh is present just to the east and southwest of the buried organic layer. When sea levels were lower due to the last ice age, it is likely that the buried organic layer was contiguous with the predecessors of the current marsh and extended underneath the current location of the beach berm. In response to sea level rise caused by retreating glaciers, the beach berm transgressed landward and upward, burying the former marsh. Historic maps and photographs (Figure 5-2, top row) show beach sands transgressing landward in this portion of beach before being anchored by beach grasses. To form an organic layer this thick probably required persistent and undisturbed marsh vegetation that gradually accreted upward in response to sea level rise. The seismic survey detected bedrock directly below the organic layer and at considerably higher elevation than the bedrock surface to the north. This stratigraphy of elevated bed rock, organic layer extending from east to west, and beach sands transgressing from west to east leaves little room for the river mouth previously occupying this area. Coring and dating might further clarify the contents and geomorphic history of these layers.

The results also strengthen the theory that freshwater flow from the Estuary to the ocean dominates the seepage flow field in the upper beach during mouth closure, causing one-way flow that is modulated by high tides. This is likely a result of the persistent head difference between the Estuary and ocean at these times. Alternating salty and fresh signals in the beach during tidal conditions suggest that bi-directional seepage may play a role during tidal inlet conditions as well.

Besides reducing lagoon water surface elevations directly, beach seepage indirectly influences inlet and beach morphology. For instance, seepage reduces the amount of water that needs to be conveyed by the outlet channel to maintain constant lagoon water surface elevations. If the outlet channel does not need to convey as much flow, it would be less susceptible to scour that can convert the outlet channel to a tidal channel. Permeability can also affect inlet morphology because more seepage results in slower rise of lagoon water surface elevations after closure, potentially resulting in longer lagoon conditions and reduced flood risk. Anecdotal evidence from the 19th century (Behrens et al., 2013) and severe drought conditions (pre-dam:1977 and post-dam: 2009, 2015), provide examples when the Russian River lagoon stayed closed for long

periods because, in part, seepage may have comprised a larger fraction of outflow to the ocean. Removing the jetty would also afford the inlet and the ocean waves a larger role in beach morphology. Beach morphology, which is discussed in more detail in Section 5, may affect permeability via beach width and possibly the sand deposition conditions.

3.3 Figures

SOURCE: LBNL (2015). Data collected by NorCal Geophysics

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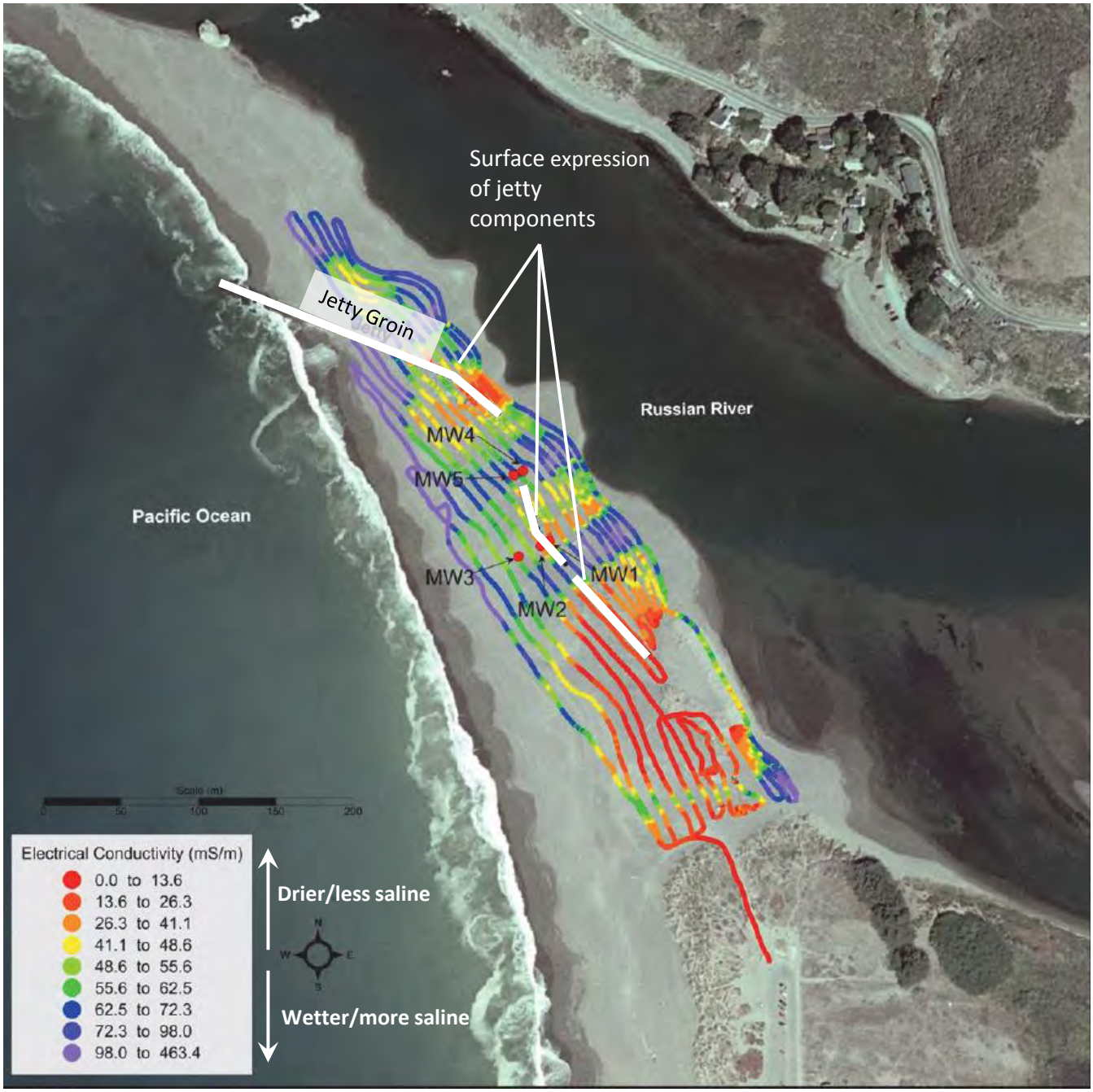
Figure 3-1

Seismic refraction data collected south of the jetty groin.



SOURCE: LBNL (2015)

Figure 3-2

Electrical conductivity data collected during mouth closure on October 15, 2014.

4 OCEAN WAVE CONDITIONS

Quantifying wave-driven processes is a key step to assessing the influence of the jetty on beach and inlet morphology, and also flood risk within the Estuary. Wave-driven sediment transport forms the beach barrier at GRSB and causes inlet closure. Wave overtopping contributes water to the Estuary, influencing flood risks by raising Estuary water surface elevations and influencing beach and inlet morphology by inducing breach events that reopen the inlet to the ocean. This section summarizes the offshore and nearshore wave climates, quantifies wave-driven surf zone processes, and discusses the potential influences of the jetty on GRSB based on its interaction with these processes.

4.1 Offshore Wave Conditions

Offshore waves are the main determinant of wave heights in the nearshore zone. To study offshore wave conditions in detail, wave data from 1997 through 2014 were collected from the Pt. Reyes buoy (NDBC buoy # 46214). The buoy is located in 1700-ft deep water at the edge of the continental shelf, approximately 37 miles southwest of the inlet and 25 miles west of Pt. Reyes. Waves measured at this depth and location can be assumed to be unaffected by the ocean floor and local shoreline, and are representative of waves offshore of the study area (O'Reilly and Guza 1993). We characterize waves by their power (proportional to the product of the wave height squared and the wave period), since this is correlated to sediment transport in the nearshore zone.

To summarize the wave climate, we separate offshore waves by period and direction:

- **Swells**: wave period greater than 10 seconds
- **Seas**: wave period less than 10 seconds
- **Northerly**: direction greater than 270 degrees from north
- **Southerly**: direction less than 270 degrees from north

Overall, wave power is lowest in summer. For most of the management season from May to October, northerly swells and seas dominate the offshore wave spectrum (Figure 4-1, upper panels). Although northerly seas are more common, swells provide more wave power to GRSB because of their longer wave period. Northerly swells are moderate in May and June, reach a minimum in July and August, and begin increasing in September. Southerly seas take up a small fraction of the total wave power spectrum. Southerly swells were weaker at the Pt. Reyes buoy than their northern counterparts during the management period, but are present in the wave spectra and sometimes contribute a significant amount of wave power.

Although offshore waves measured at the Pt. Reyes buoy are dominated by northerly swells for much of the year, the picture is more complex in the nearshore zone at GRSB because of wave refraction. Figure 4-2 compares the offshore (Figure 4-2a) and nearshore (Figure 4-2b) wave roses, showing how refraction causes nearshore waves to have a greater directional spread and to be aligned in a more direct angle to the beach. Wave roses are shown here for both height and wave power, the latter of which is a better indicator of sediment transport, as it reflects both the height and the period of waves. The offshore roses (Figure 4-2a) are based on summary data and not spectra, and thus hide secondary influences such as southerly swells. The nearshore conditions, which are based on wave data provided by BML adjacent and wave modeling by ESA, are discussed in more detail in the next section.

4.2 Nearshore Wave Conditions

Although offshore waves propagate into the nearshore coastal zone, they are transformed along their path to the nearshore by variations in the shelf bathymetry and the shape of the shoreline cause waves to focus and spread locally (O'Reilly and Guza 1993). This is apparent when comparing offshore and nearshore wave roses in Figure 4-2. Reliable estimates of nearshore waves better represent surf zone processes. We approximated nearshore waves using three sources of information:

- Offshore waves measured at the Pt. Reyes Buoy (discussed above);
- Wave measurements immediately offshore of GRSB collected by the Bodega Marine Laboratory (BML);
- A transformation matrix calibrated to the BML observations.

We use a unit-wave approach with the SWAN model (Deltares 2011) to develop separate transformation matrices for 9 locations along GRSB, spanning roughly 850 feet north of the jetty groin to 2,200 feet south of the groin, and including a location at the site of the BML measurements (see Figure 4-3). Our nearshore wave estimates are based solely on transformation of offshore Pt. Reyes waves, and exclude any contribution from wind-waves generated between the Pt. Reyes buoy and GRSB. Our transformation matrix approach is described in more detail in Appendix A. Transformation matrices were shown to be valuable tools in previous inlet closure studies at Bolinas Lagoon (PWA, 1999) and Crissy Field (PWA, 2007).

Formerly (e.g. ESA PWA, 2012), we applied a transformation matrix provided by the Coastal Data and Information Program (CDIP) (pers. comm. Bill O'Reilly) based on a numerical refraction/shoaling model. This was compared to 2009 measurements from BML, and also compared qualitatively to wave transformation information provided by Johnson (1959), based on a wave-tracing method. Since 2009 BML data included mostly short-period waves, this earlier transformation matrix had gaps in important wave

frequency bands associated with swell waves. The current set of transformation matrix represent an improvement in nearshore wave predictions at GRSB.

4.2.1 Comparison of Nearshore Predictions and Observations

We tested the accuracy of nearshore wave estimates against BML measurements at the site from August 30 to November 14, 2012. Measurements were taken with an upward-facing Nortek© acoustic wave and current (AWAC) sensor mounted on the seafloor at 10 m (36 ft) depth offshore of the inlet (approximate location: 38°26’35” N; 123°07’50” W). This instrument measured pressure in the water column at one sample per second (1 Hz), which was used to infer water surface elevation. The time series of the water surface elevation was used with a Fourier analysis to provide significant wave height (Hs) and period (Tp) at the measurement station.

The transformed waves were found to be a good approximation of nearshore observations (Figure 4-4): the root mean square error (RMSE) between observations and estimates was 25 cm for the 2012 data, or about 20 percent of the mean value of Hs. In some cases the model under-predicted wave heights by 0-2 feet when they were observed above 8 feet, but this bias was not systematic. These predictions are a superior alternative to simply applying offshore measurements from the Pt. Reyes buoy at GRSB; offshore waves alone were found to be a poor predictor of wave conditions at GRSB, and were systematically biased 2-4 feet higher and had a broader directional spread than nearshore waves.

4.2.2 Predicted Nearshore Waves During the Management Period

An hourly time series of nearshore wave conditions at GRSB was constructed for the period from January 1997 to December 2014 using the transformed waves. Figure 4-5 and Figure 4-6 summarize the predicted nearshore wave conditions near the jetty groin during the management period for these years. Waves are again categorized by period and direction.

Similar to the results for offshore waves shown in Figure 4-1, Figure 4-5 shows that wave power is larger for northerly swells than for any other wave category. However, there are several key differences between offshore and nearshore wave conditions:

- Swell waves are responsible for more of the total wave power in the nearshore zone than at the Pt. Reyes buoy.
- Southerly swells account for more wave power than any other waves in July and August,

- The daily-average wave power from northerly seas was typically less than the overall average from all waves.

The first point is evident from comparison of the left panels of Figure 4-1 and Figure 4-5: At the offshore buoy location (Figure 4-1), southerly swells provided less power than the overall average from May 1 to September 30. In contrast, near the jetty groin (Figure 4-5), wave power from southerly swells was much closer to the average throughout the same period, and greater than the average in mid-summer. This indicates that refraction and the orientation of the beach amplify the role of southern swells in summer.

Overall, within the management season, wave power reaches a minimum in August for both offshore and nearshore waves, and is largest in May and October. Wave power varies along the beach, which Johnson (1959) first noted, and our modeling confirms. Figure 4-3 shows the significant wave height for 8 shoreline locations relative to wave height at the jetty groin location. Waves at the jetty groin are the weakest, as Johnson (1959) found with his wave ray analysis. This local wave minimum may contribute to the inlet often residing next to the jetty. Mean wave height increases with distance from the groin – 850 feet north of the groin waves are roughly 20 percent larger, and 2,200 feet south of the groin they are more than 50 percent larger.

The dominant swells from the north during the management season cause a net southward-directed wave power vector along the shoreline (Figure 4-6). This was true of all seasons, and suggests that sand transport along the beach is typically southward (see Chapter 5.1; Hapke et al. 2006). However, swells from the south were also an important component of the nearshore wave spectrum (Figure 4-5), and at times cause a northward-directed wave power vector. This is visible in Figure 4-6 as a smaller peak north of the shore-perpendicular line. This influence of southerly swells would reverse the sediment transport, and is strongest in summer and weakest in winter, consistent with observations in Figure 4-5.

4.3 Surf Zone Assessment

We assess surf zone processes including wave setup, runup, and overtopping by combining predicted nearshore wave heights from 1997 to 2014 (discussed above) with information about the beach morphology. Seasonal and longer-term changes to the beach morphology are discussed in more detail in Section 5.

4.3.1 Beach Characteristics

Surf zone processes such as wave runup and overtopping are strongly dependent on the shape of the beach. We characterize the beach using topographic data obtained from three sources:

- Digital elevation models (DEMs) created from airborne light detection and ranging (LiDAR);
- Monthly Water Agency beach surveys north of the groin; and
- Survey data collected by Behrens (2012).

These sources are discussed in more detail in Section 5 addressing beach morphology at the site. The existing LiDAR data includes two DEMs created in September 2002 and September 2010. Both were flown during periods when the inlet was open, and provide a spatial map of the entire beach with accuracy on the order of 20 cm in the vertical (CCC, 2015). While more frequent, Water Agency topography covered a more limited extent of the beach, from the jetty groin structure to as far as the northernmost beach access point, approximately 1,000 feet north of the groin. These were collected monthly starting in 2009. Lastly, survey data were collected independently by Behrens (2012) in August and December of 2009, and January and March of 2010.

Although the beach topography varies throughout the year in response to the changing wave climate, several aspects are consistent: (1) The beach crest slopes downward from a stable elevation of 17-24 ft NGVD near the Beach Parking Lot to a low at the jetty groin of 3-10ft NGVD (Figure 4-7). The lowest point in the beach crest during the management period is typically a notch immediately north of the jetty groin that has an elevation of 5-8 feet NGVD during summer months (ESA 2015). North of the jetty groin, the beach crest height typically slopes up again, but the presence of the inlet can locally lower the beach due to erosion. The foreshore slope (Figure 4-7b), measured on the beach face between mean sea level (MSL) and mean higher-high water (MHHW) level, typically varies from 0.06 to 0.10 (0.10 slope is a vertical change of 1 ft over a horizontal distance of 10 ft).

4.3.2 Wave Runup

Wave runup is the maximum vertical extent that the leading edge of a breaking wave reaches on the shore (Figure 4-8b), assuming that the beach face continues to slope up indefinitely (Komar, 1998). Wave runup on the beach face can be combined with hourly tide levels and wave setup to estimate peak hourly total water level (TWL). Under actual conditions when the runup exceeds the beach crest elevation, the runup becomes overwash before it reaches the TWL. Since individual waves vary in height and period, runup is usually taken as a representative hourly value, such as the value that is exceeded by two percent of the waves.

Figure 4-9 summarizes wave heights, tide level and predicted TWL at the groin from 1999 to 2014. TWL is highest in winter and spring, leading to events where overtopping into the Estuary is often extensive (Figure 4-8a). Since tide levels rarely exceed 4 feet NGVD, wave runup is a large component of the TWL, often adding at least 4 feet to the

tides at the groin (Figure 4-9 lower panel). As Figure 4-9 indicates, runup and TWL are lower during the management season, as waves tend to be weaker during the months of May-October than for the rest of the year. For example, the TWL that is exceeded only 2 percent of the time is roughly 10 feet NGVD for all seasons and roughly 8 feet NGVD for the management period. TWL during the management season tends to build the beach crest north of the jetty groin higher (ESA 2015).

4.3.3 Wave Overwash

When related to the beach berm height, TWL can be used to estimate wave overwash, a measure of the amount of water contributed to the Estuary from waves spilling over the beach crest (Figure 4-8b). We estimated monthly wave overwash volumes under natural conditions and potential future conditions without the jetty present in the beach.

To determine how sensitive the predicted overwash is to seasonal variations in beach morphology, we estimated overwash using the survey data taken from August 2009 and March 2010 to characterize the beach shape. These are intended to represent seasonal beach differences between conditions at or near the beginning of the management period and at the end of the management period. The results are given in Table 4-1.

To assess the influence of the jetty, we considered the morphology of nearby beaches without jetties. We use two sites: the Gualala and Navarro Rivers. These sites are approximately 30 and 60 miles north of the Russian River, respectively. We chose these sites because of their proximity and their similarity to GRSB, and we assume that their beaches were shaped by similar conditions to those at GRSB. Each of these sites have similar wave climates, tide ranges, shore orientation and beach morphologies as the GRSB: the beach at all three sites is a spit separating the ocean from the Estuary, and in all three cases the crest height increases from a minimum at the north end of the spit to a maximum at the south end. In addition, the September 2010 LiDAR flight (CCC 2012) collected elevation data from all three sites within a two-week period, so the beaches likely experienced similar ocean conditions antecedent to the LiDAR data collection. A more extensive comparison of these and other sites is provided in Section 5.

Overwash rates are sensitive to beach crest height, which in turn is sensitive to inlet morphology. If the jetty were not present at GRSB, inlet migration south of the present-day groin could periodically lower the beach, influencing the potential for wave overwash. The inlet migrates extensively, but is limited to the region north of the jetty groin structure (Behrens et al., 2009) and has not moved south of the groin since it was constructed in 1930. As shown in Figure 4-10a, the alongshore slope of the GRSB beach crest is the steepest of the three sites, and is well-approximated by a linear fit to the 2002 and 2010 LiDAR data with a slope (elevation change divided by alongshore distance) of 0.008 (R^2 = 0.80). The Gualala River inlet opens at the north end of its beach

during floods and only migrates to the south end of its beach every 5-10 years (pers. comm. J. Hall). The slope of the Gualala beach crest is intermediate, and is also well-approximated by a linear fit, but with a lower slope of 0.0046 (R^2 = 0.92). Lastly, the Navarro River also opens at the north end of its beach during floods but is known to migrate to the southern end of its beach on a yearly basis. Its slope was the lowest, at 0.0025 (R^2 = 0.93).

We used each of the above beach crests to estimate monthly wave overwash into the Russian River Estuary with and without the jetty present, to simulate the effect of southward inlet migration on overwash volume. Figure 4-10b-d shows the expected overwash volume for each of the months in the management period, which are also summarized in Table 4-1. By shifting the beach profile from one with limited yearly inlet migration (Russian River) to one with 5-10 year (yr) inlet migration recurrence (Gualala River), the overwash rates increase by a factor approximately 5. By shifting to a beach profile representative of extensive inlet migration (Navarro River), the overwash is increased by approximately a factor of 25.

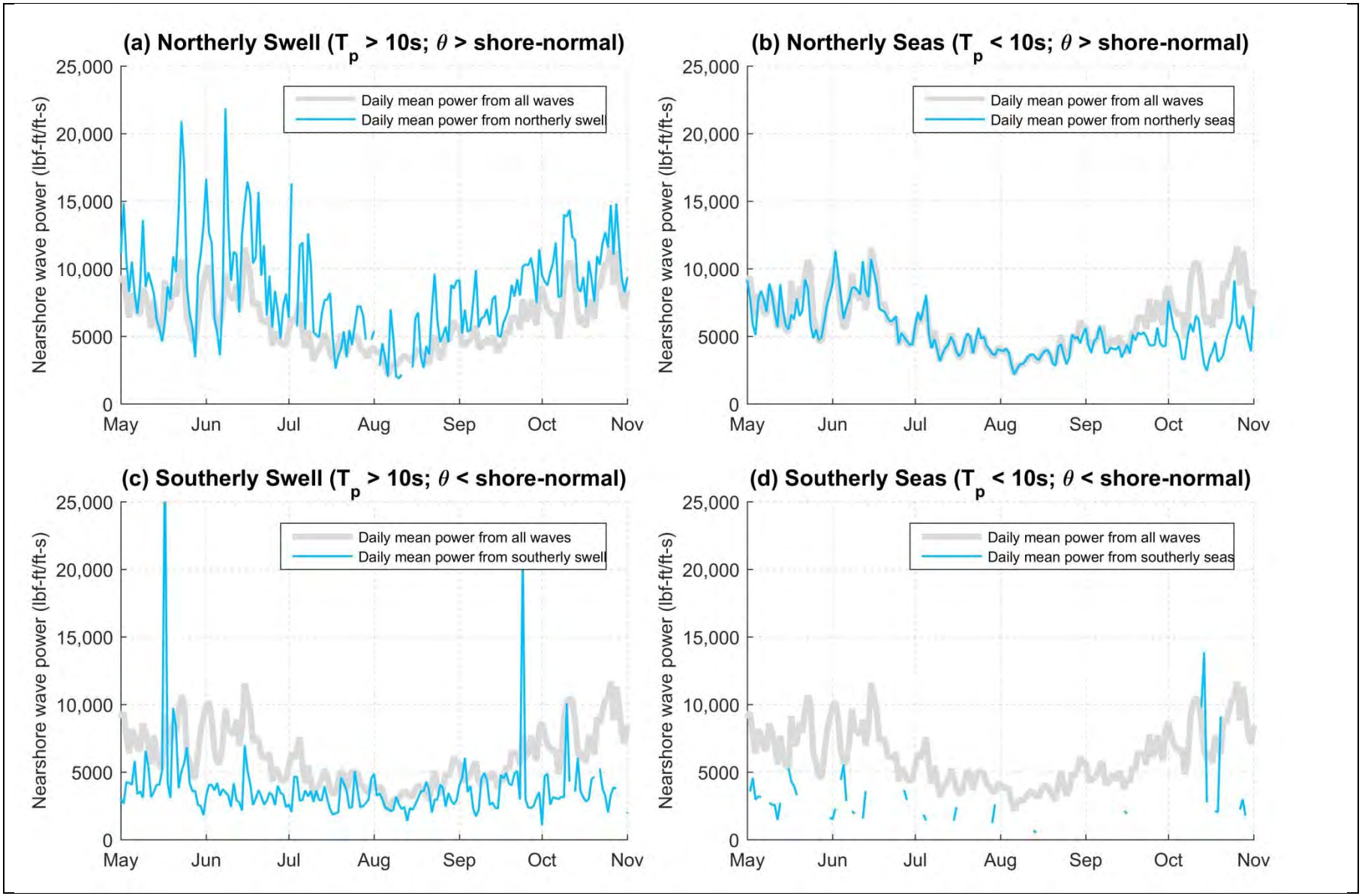
Table 4-1. Expected monthly wave overwash volumes (acre-ft/month).

	May	June	July	August	September	October	Total
Beach Topography:	**Sensitivity to Beach Morphology**						
August 9, 2009 survey data[1]	0.1 ± 0.3	0.01 ±0.03	0.0 ± 0.0	0.0 ± 0.0	0.1 ± 0.1	16.4 ± 45.1	16.6 ± 74.3
May 22, 2010 survey data[1]	0.6 ± 1.4	0.1 ± 0.2	0.001 ± 0.002	0.007 ± 0.01	0.6 ± 0.8	50.1 ± 119.2	51.4 ± 121.6
Beach Crest Slope	**Sensitivity to the Jetty**						
Steep (GRSB)	0.3 ± 0.6	0.03 0.09	0.0 ± 0.0	0.0 ± 0.0	0.2 ± 0.3	28.5 ± 73.3	29.0 ± 74.3
Moderate (Gualala R. Beach)	2.5 ± 5.0	0.5 ± 1.1	0.02 ± 0.02	0.06 ± 0.1	2.6 ± 3.2	124.1 ± 255.9	129.7 ± 265.3
Low (Navarro R. Beach)	26.0 ± 45.0	9.8 ± 14.1	0.9 ± 0.9	2.0 ± 2.8	33.0 ± 35.9	638.1 ± 919.1	713.9 ± 1017.8

Note: Each monthly value is estimated using nearshore wave data from 1997 to 2014. Reported monthly averages are followed by ± the standard deviation of the sample.

[1]Behrens (2012)

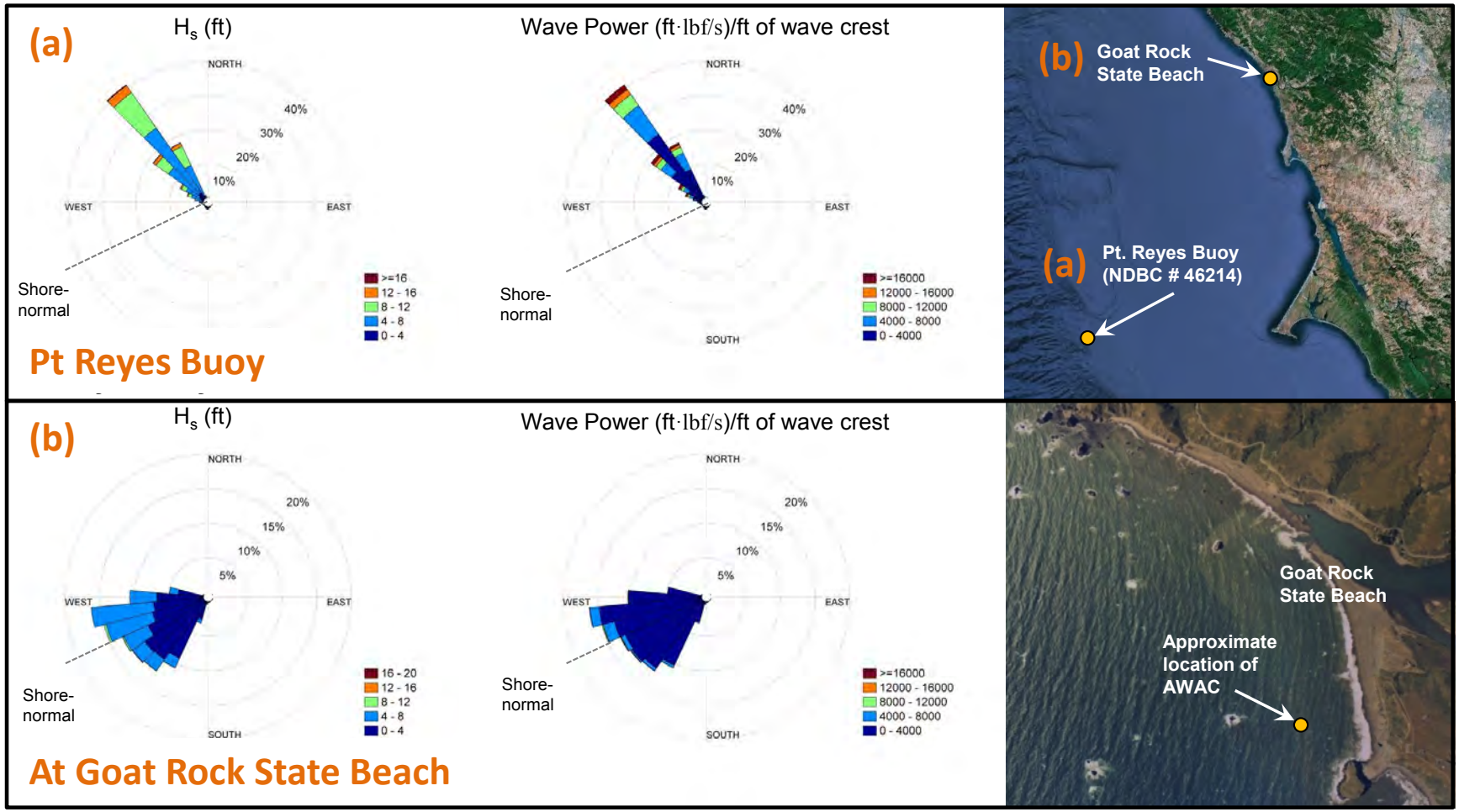
4.4 Figures



SOURCE: NDBC Buoy # 46214

Figure 4-1

Daily aggregated measurements of offshore (Pt. Reyes) wave power grouped by northerly and southerly swells and seas.

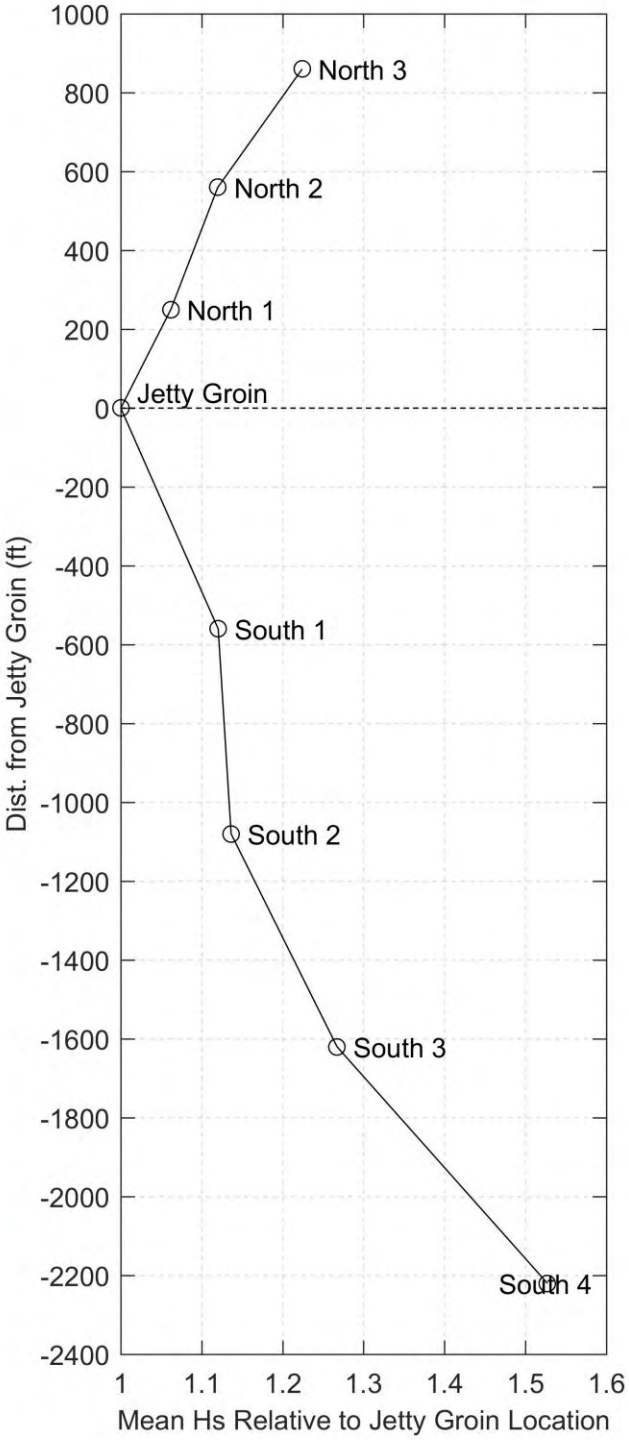


SOURCE: NDBC Buoy # 46214, BML (2012)

NOTE: Shore-normal direction is approximately 255° measured from north. "AWAC" stands for Acoustic Waves And Currents, referring to a wave and current measuring device produced by Nortek©.

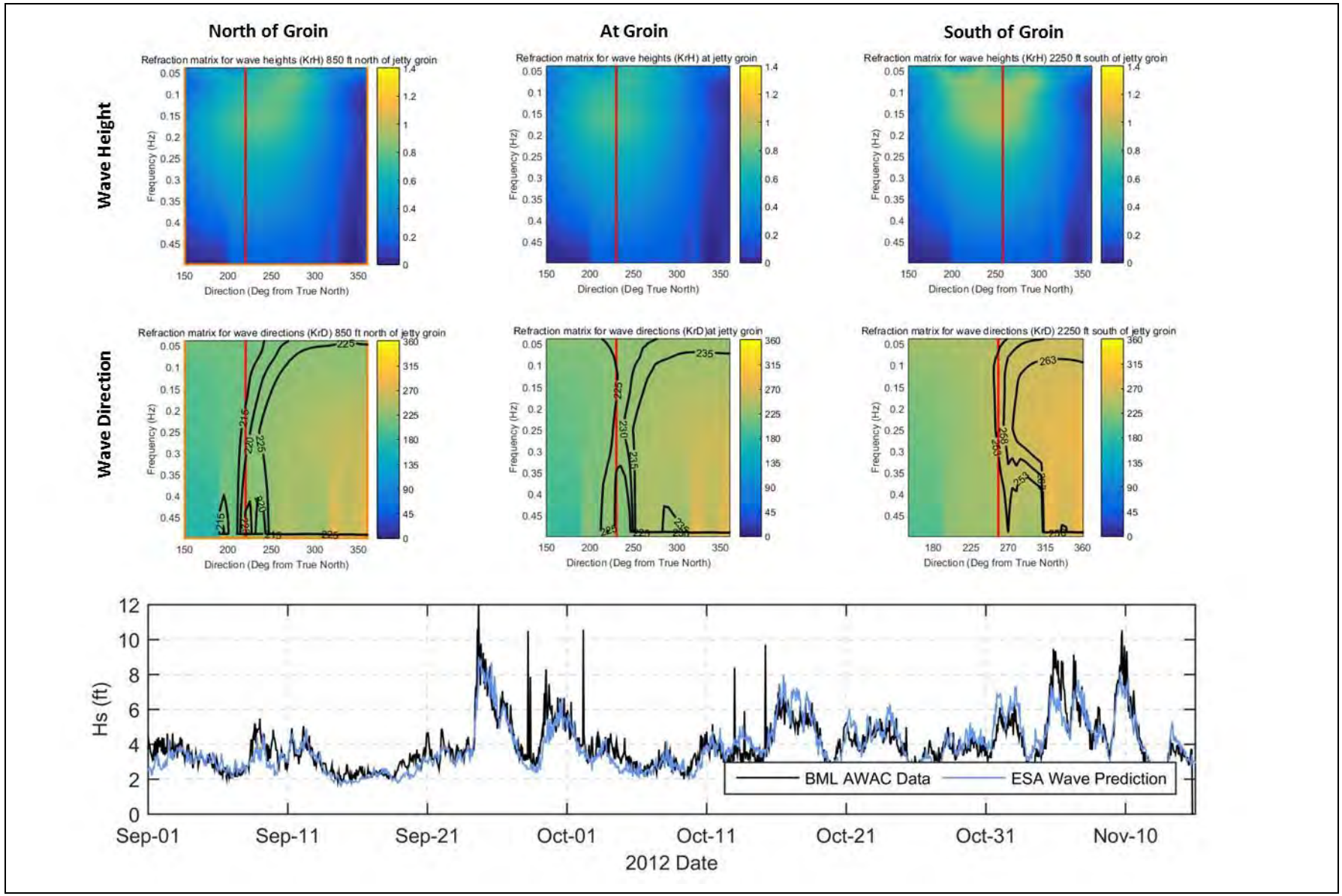
Figure 4-2

Comparison between dominant wave conditions at **(a)** the Pt. Reyes Buoy and **(b)** near Goat Rock State Beach.



SOURCE: ESA SWAN model

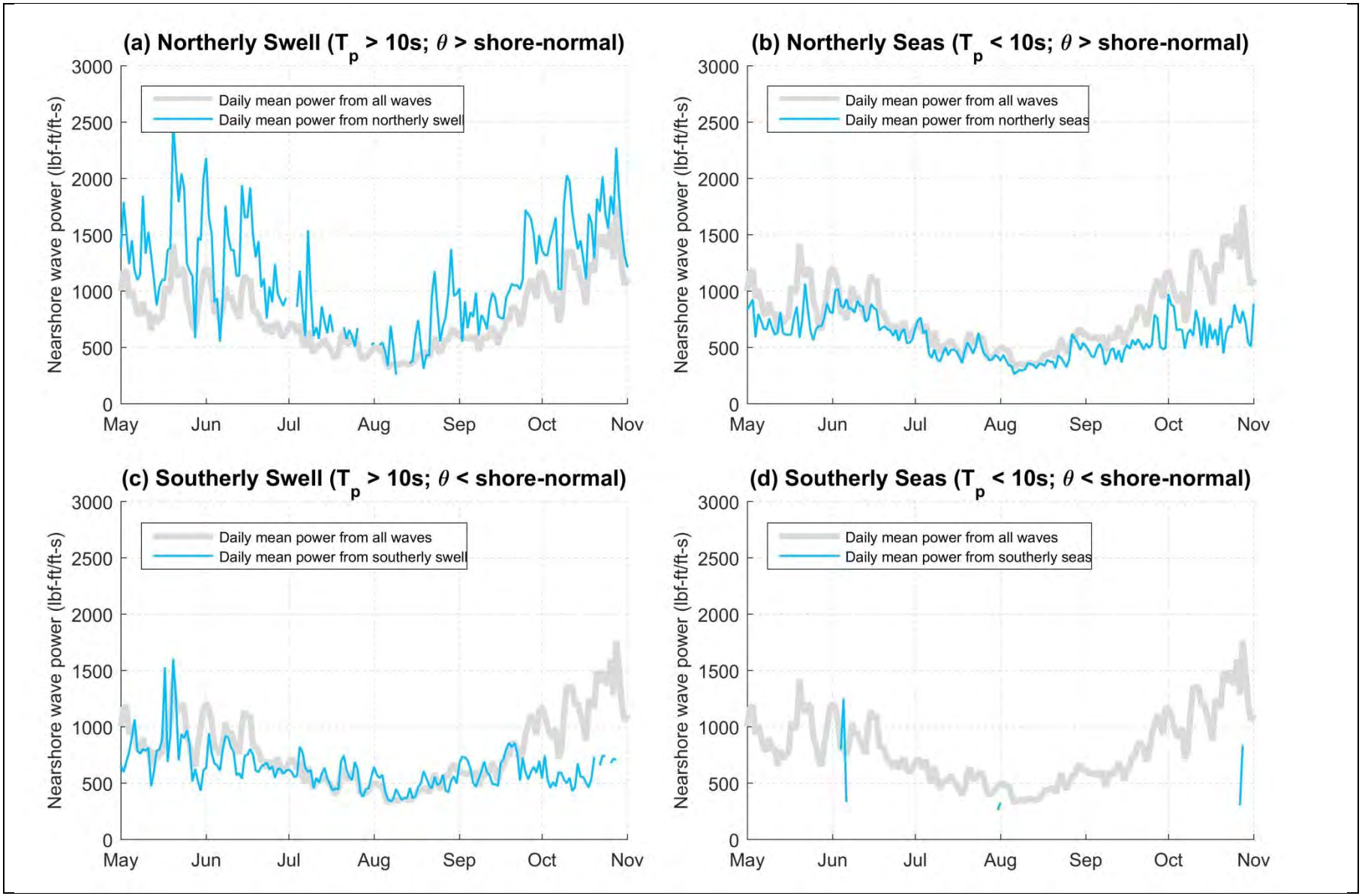
Figure 4-3
Variation in mean significant wave height (Hs) along GRSB.



SOURCE: ESA SWAN model, BML (2012)

Figure 4-4

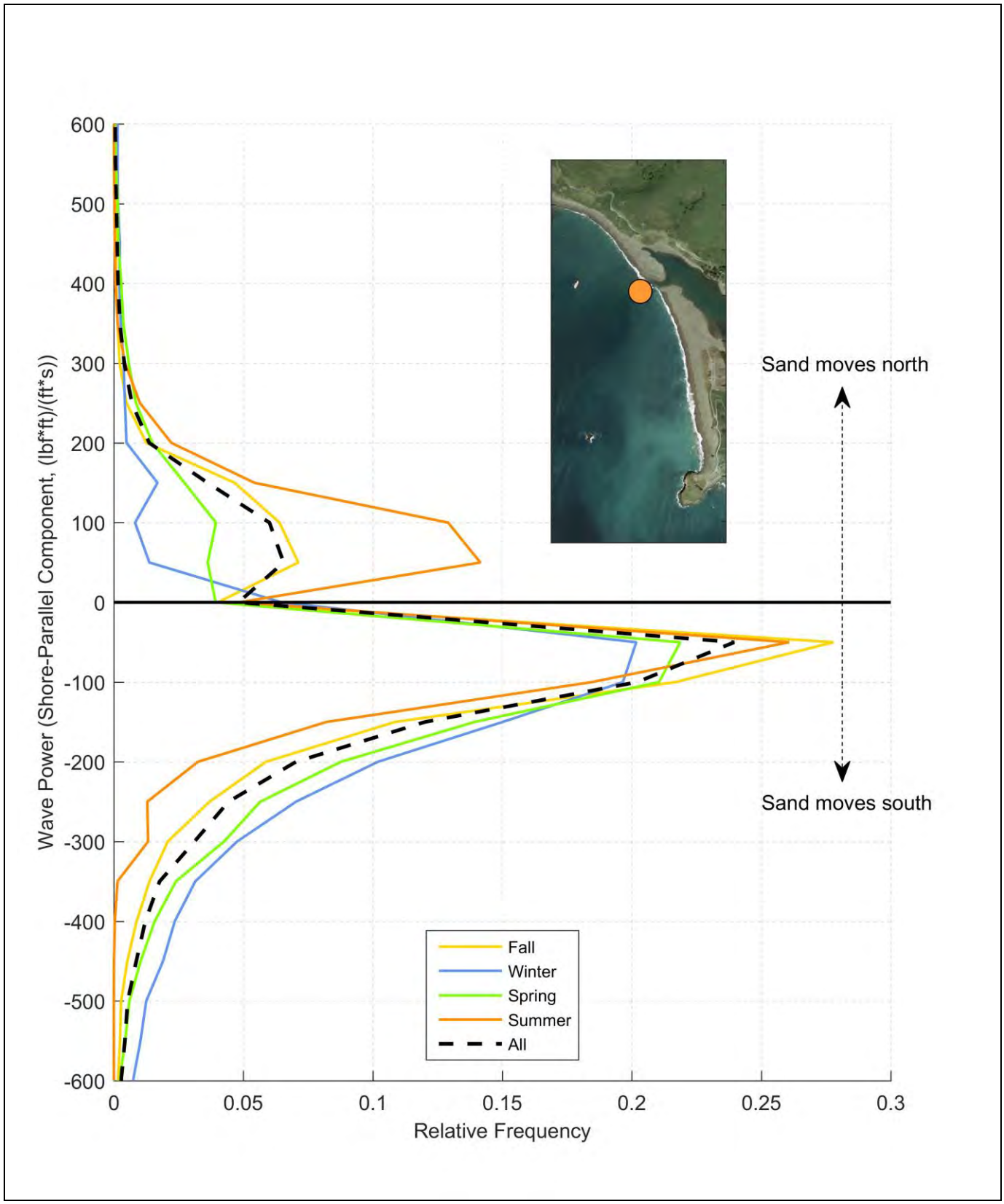
(top) Wave transformation matrices used to obtain nearshore wave estimates and **(bottom)** 2012 wave conditions.



SOURCE: ESA SWAN model

Figure 4-5

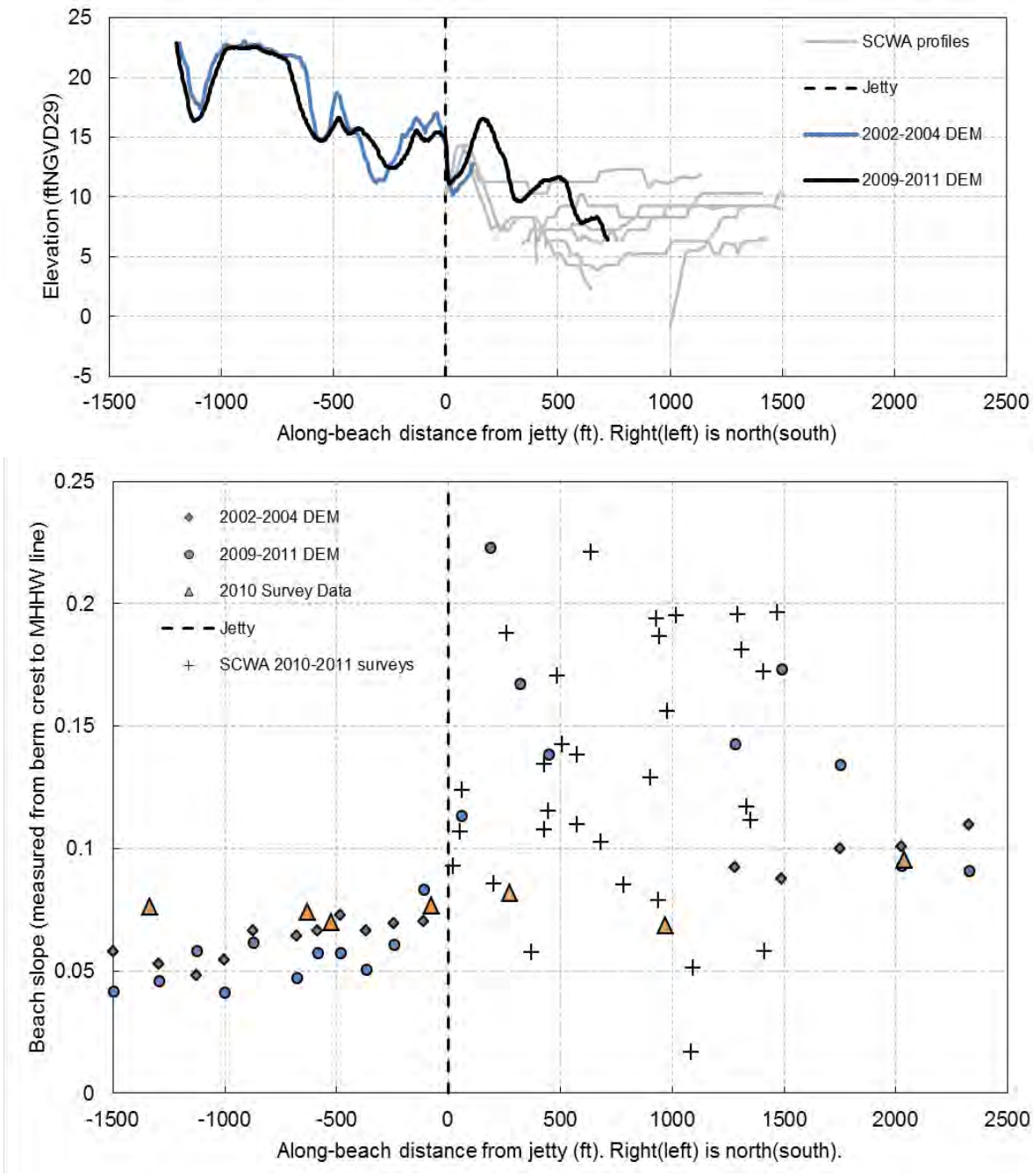
Daily aggregated measurements of nearshore predicted wave power grouped by northerly and southerly swells and seas.



SOURCE: ESA SWAN model

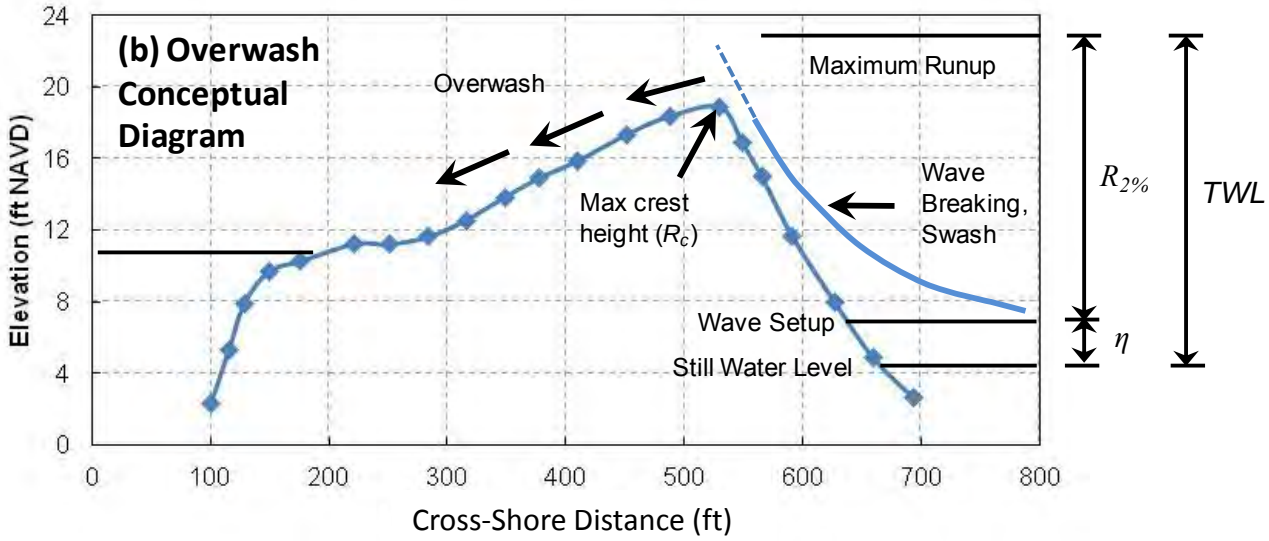
Figure 4-6

Distribution of alongshore component of wave power vector at the jetty groin location.



NOTE: Dashed line represents the point on GRSB where the seaward endpiece of the jetty groin is located. Beach slope was measured between MHHW and MSL.

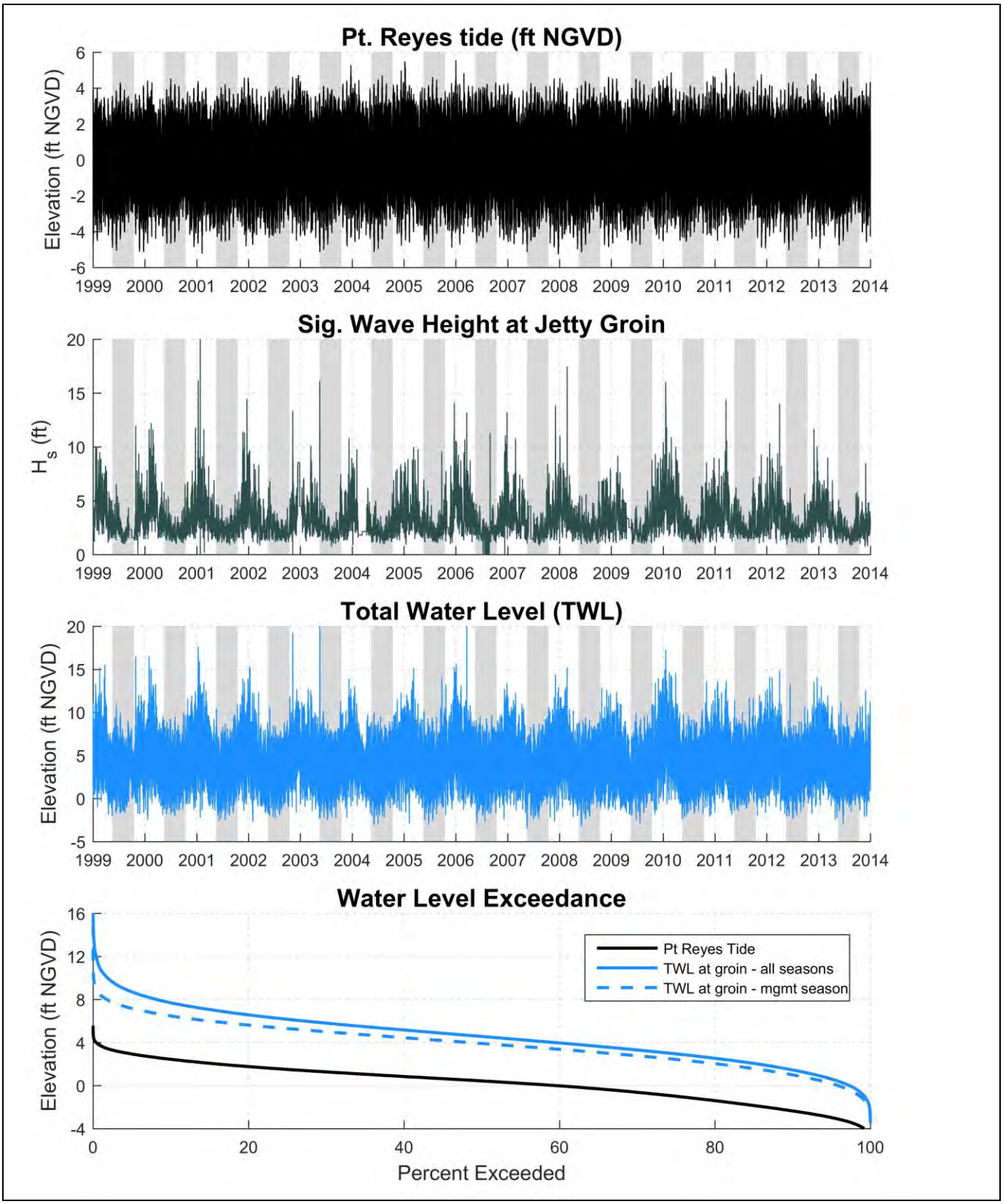
Figure 4-7
(a) Along-beach crest profile of GRSB and **(b)** foreshore beach slope



SOURCE: Photo credit in (a): Peter Baye. Beach profile in (b) taken from survey data from August 9, 2009 by Behrens (2012).

Figure 4-8

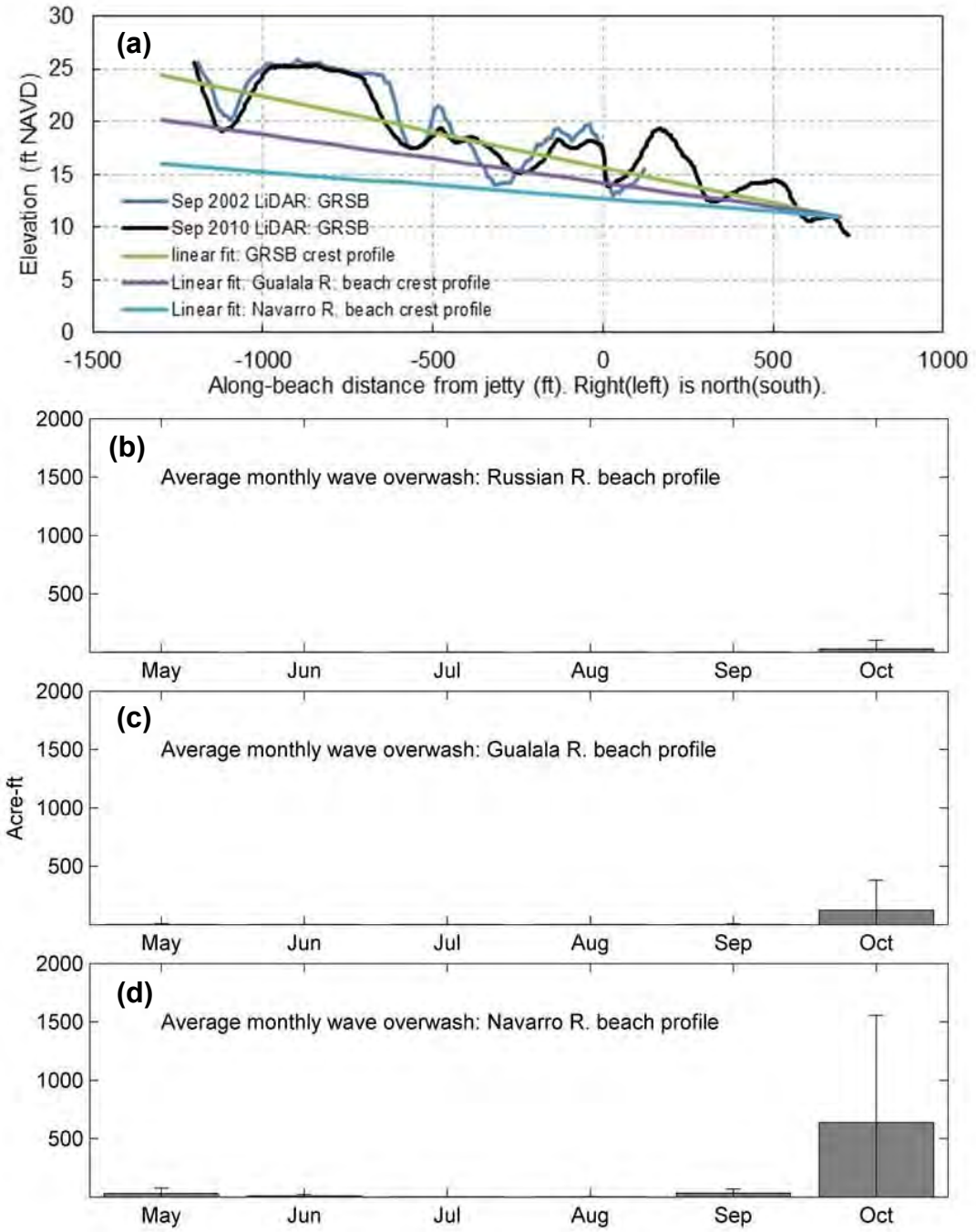
(a) Example of wave overwash at GRSB and **(b)** conceptual drawing of wave overwash.



SOURCE: NOAA Pt. Reyes tide gage, ESA SWAN model.

Figure 4-9

Summary of tide, wave, and total water level conditions at the jetty groin, for 2000-2014.



NOTE: Average monthly wave overwash volumes calculated using nearshore wave estimates from January 1997 to December 2011.
Linear fit of GRSB beach profile in upper panel is a fit of 2010 LiDAR.

Figure 4-10

(a) Reference beach crest profiles and **(b)** resultant expected overwash volume for target months,

5 BEACH MORPHOLOGY

As the barrier that creates the lagoon and hosts the inlet, the beach is a dynamic feature whose morphology and formation exerts control over the water surface elevations inside the lagoon. The morphology, or shape, of Goat Rock State Beach undergoes seasonal variation in response to changes in wave conditions, multi-year changes due to episodic climate events, and long-term changes resulting from alterations to the sediment budget. The beach morphology is closely related to the inlet morphology addressed in Section 6.

Long-term and current trends in the morphology of GRSB can partly be explained within the context of its littoral cell. GRSB is located in the middle of the Russian River Littoral Cell (Figure 5-1), which is bounded on its north end by a headland near Fort Ross and by Bodega Head at its south end (Habel and Armstrong, 1978). The littoral cell is populated with a series of pocket beaches separated by rocky headlands. Goat Rock State Beach is a pocket beach set among the cliffs and bluffs abutting the ocean north and south of the river mouth. As such, the beach is geographically disconnected from regional alongshore sediment transport. The net lateral ("alongshore") transport of sediment along the shoreline is southward, but regions of reversal occur at GRSB and Wright's Beach (Figure 5-1). The net southward transport is indicated by (1) wave refraction diagrams (de Graca, 1976), (2) petrographic analysis of Russian River sediments (USACE, 1962), and (3) examination of the abundance of beaches throughout the cell (Minard, 1971). The mouth of the Russian River is thought to be a divergence zone, with net northerly transport on the northern third of the beach and net southerly transport on the southern two thirds, as suggested by prior studies (Johnson, 1959; de Graca, 1976) and this study's wave modeling (Figure 4-6). The Russian River is the major source of beach material to GRSB and the rest of the littoral cell (Hapke et al., 2006), but small creeks in the area such as Russian Gulch, Wright's Creek and Salmon Creek also contribute sediment to the littoral cell. This is typical of California, where 70-95 percent of the beach sediment is supplied by coastal streams (Runyan and Griggs, 2002).

Because of the dominant southerly transport of sediment in the littoral cell, most of the beaches between the Russian River and Fort Ross, to the north, are small inset pocket beaches, while beaches to the south of the Russian River are wider, although they are also bounded by rocky headlands. Blind Beach is located immediately south of GRSB, and Wright's Beach is located three miles farther south. Salmon Creek Beach, immediately north of Bodega Head, is the largest beach (2.6 mi.) in the littoral cell, and fronts a large dune complex which is thought to be a recent feature (Minard, 1971). Much of the beach sediment between GRSB and Bodega Head can be traced to the Russian River (USACE, 1962).

The construction of the Goat Rock quarry between 1924 and 1929 is likely the event that was first responsible for filling the gap between Goat Rock and the adjacent shoreline (see Section 2 regarding jetty structure). Prior to this time, the limited existing information suggests that this gap was a low-elevation tombolo which may have bypassed sediment southward to Blind Beach from GRSB during periods when coastal water surface elevations were high. This event has likely blocked the natural transport pathways in the littoral cell and has partially isolated GRSB. The existing jetty complex on the GRSB spit (both the groin and the access elements) may also be influencing the long-term beach evolution.

In addition to the littoral processes, GRSB morphology is affected by the riverine sediment supply, which has been altered by land use and water management practices in the watershed (PWA, 1995). For example, sediment loads are thought to have increased due to logging in the watershed and decreased due to gravel mining and dam construction. Recent analysis of sediment yields in the lower Estuary suggests sediment yields on the order of 100,000 tons/yr (PWA, 1995). A comparison of bathymetry surveys from 1992 and 2009 indicates that the lower river and Estuary have not exhibited significant bathymetric change, suggesting the Estuary is at or near equilibrium with respect to its sediment budget.

We use this available historical information to help frame our approach. Past management practices on the beach and in the watershed have probably altered the width and elevation of the barrier beach. An understanding of past beach evolution and current trends can be developed with geo-referenced, aerial photographs and maps to provide landform data characterizing both historic and current trends. We examine the evolution of both GRSB and its south neighbor Blind Beach to allow quantification of the effect of the littoral barrier at Goat Rock. We place these changes in context with the changes to watershed sediment supply. We also examine the evolution of Wright's Beach, which we use as a control site, to determine whether changes at GRSB and Blind Beach are related to changes elsewhere in the littoral cell.

In response to future predicted increases in mean sea level, the beach is expected to transgress (move up and landward) (PWA, 2009). As the rate of sea level rise increases, the landward transgression rate will similarly increase. This process may be augmented by climate-change increases in winter storm frequency and intensity. We predict future morphology changes that may occur in response to sea level rise.

5.1 Historic Beach Morphology

To create an understanding of past beach evolution and current trends, ESA PWA geo-referenced aerial photographs and maps to provide landform data characterizing both

historic and current trends. A later section provides a timeline of management actions on the beach and actions in the watershed affecting beach sediment supply.

Data used to assess beach evolution are listed in Table 5-1. Information was gathered from three sources: (1) historic topographic sheets ("T-sheets") developed by the US Coast and Geodetic Survey (USC&GS), (2) aerial photographs, and (3) shorelines drawn by the US Geological Survey (Hapke et al. 2006) from historical data. The location of the historic shoreline can be defined in several ways. This is because typical beach profiles include a number of common features, including an offshore beach toe, a swash zone, a point of maximum wave runup, a wetted boundary, and often the toe of a bluff or cliff and/or a vegetation line (Boak and Turner 2005). Since historical data are often limited to a set of aerial images, the shoreline is often characterized opportunistically using a wetted boundary line in the photographs. This line represents the uphill limit of saturated sand on the beach. This is related to tide levels and wave runup, but can also be altered by seepage from the lagoon to the ocean. When beach topographic data are available, the shoreline is sometimes characterized using a high-water line (HWL). Historic T-sheets delineate the shoreline in this way. More recently, LiDAR data are used to obtain shoreline positions by intersecting a tidal datum such as mean higher high water (MHHW) with a surface derived from beach surveys.

For the present study, the majority of shorelines are derived from a wetted boundary line in the aerial images listed in Table 5-1. Photographs were first translated into a common geographical reference frame using the Georeferencing Toolbox in ArcGIS v.10.0. Shorelines were then drawn for each image, approximating the wet-dry line. Time series of shoreline position at several transect locations (Figure 5-1) were then created using the Digital Shorelines Assessment System (DSAS) provided by the U.S. Geological Survey (USGS, 2012). Representative maps and aerials from this historic record are shown in Figure 5-2. The methodology and the uncertainties involved with these analyses are described in further detail in Appendix B.

The raw time series of shoreline position and beach width at each of the three beach sites (Figure 5-1 and Figure 5-3) reflects long-term, seasonal and periodic climate-related (e.g. El Niño) changes. The beach undergoes seasonal changes in response to the seasonal changes in the coastal climate (e.g. Komar 1998). El Niño events and large-scale floods also have profound impacts on the morphology (Storlazzi and Griggs 2000), usually leading to erosion of much of the beach. Long-term trends of shoreline position change and beach width change were obtained by fitting a linear curve to the data. To isolate the underlying long-term trend in beach morphology from the seasonal and climate variability, we only obtained the trends using the data obtained during summer-fall months (June through November) and during years when no major floods or El Niño events occurred. The earliest known shoreline given by the 1875 T-sheet was not used in this analysis, since the position of the drawn shoreline was ambiguous.

Time series of the shoreline position at GRSB and its southern neighbor Blind Beach (Figure 5-3) have two important features: (1) at each beach, the long-term trend is remarkably consistent, and (2) the trends are in the opposite direction between the two beaches. At GRSB, the shoreline has been moving toward the ocean at an average rate of 1.5 ft/yr. The rate varies by transect, from about 1.2 to 2.1 ft/yr. South of the Goat Rock Parking Lot, the shoreline at Blind Beach has been eroding at a rate of 0.8 ft/yr, with individual transects showing erosion rates varying from 0.6 to 1.2 ft/yr. For most transects on both GRSB and Blind Beach, the majority of the change occurred between 1930 and 1990, as shown in the top panel of Figure 5-4. After 1990, the rate of change appears to decrease, but with a higher amount of scatter than observed for the prior period.

Aside from shoreline position change, beach width can vary in response to eroding backbarrier bluffs (in areas of the beach backed by bluffs) or in response to changes in the size of the spit (in areas backed by the lagoon). The long-term trends of beach width are mostly similar to the shoreline position trends (Figure 5-3), but with a few differences. GRSB became wider in most transects, in response to the background trend of shoreline accretion. However, in the vicinity of the Beach Parking Lot (Transects 10-12; Figure 5-3), the beach width has been decreasing since 1930. This may be due to the construction of the jetty access elements in 1930 or alternatively due to dune growth and vegetation. Unfortunately, no data exist between 1875 and 1930, which would allow for comparison with the pre-jetty conditions. The width-reduction trend is strongest at Transect 12, and the change was accentuated in the 1960s, at the time when the Beach Parking Lot was constructed. The change in width after 1960 appears to be entirely accounted for by the appearance of the dune complex on the ocean side of the Beach Parking Lot, a difference apparent between the aerial images from the years 1950 and 1970 (Figure 5-2).

Another departure between the width and shoreline position trends is the growth of the spit width between the groin and the Beach Parking Lot (Transects 7-9; Figure 5-3). The spit is growing in width at an average rate of 2.3 ft/yr, outpacing the shoreline accretion rate. This part of the beach contains the remaining jetty access elements, which protrude above the beach crest height under most conditions, and likely alter the morphological influence of extreme storm waves which would normally wash over the beach. As discussed in Section 2, the jetty access elements are also preventing the inlet from migrating south of the groin, which may be contributing to the observed widening trend. In the area north of the groin, the inlet migrates frequently (Behrens et al. 2009), and the increase in width is equal to or less than the local shoreline accretion rate, but the linear trends have less confidence ($R^2 < 0.2$).

South of Goat Rock, the beach has been narrowing at a rate that is commensurate with the observed shoreline retreat. This result suggests that the construction of the quarry and access elements (and the eventual Goat Rock Parking Lot) between 1924 and 1929

has disrupted a primary sediment source to Blind Beach. Prior to this time the sole data point, the 1875 T-sheet, suggests that the site was likely a low-elevation tombolo (Figure 5-2), which may have allowed southward bypassing of Russian River sediments behind Goat Rock when coastal water surface elevations were highest.

Table 5-1. Sources of beach planform data at GRSB used in the present study.

Date	Data Type	Source
Aug 1875 – Jan 1876	T-Sheet	US Coast and Geodetic Survey
Jun - Oct 1930	T-Sheet	US Coast and Geodetic Survey
1942	Aerial Photograph	US Dept. of the Interior
9/23/1945	Aerial Photograph	US Army Corps of Engineers
9/15/1950	Aerial Photograph	US Army Corps of Engineers
7/25/1953	Aerial Photograph	US Dept. of the Interior
2/3/1956	Aerial Photograph	US Army Corps of Engineers
4/21/1958	Aerial Photograph	US Army Corps of Engineers
6/10/1961	Aerial Photograph	Nat. Res. Conservation Service
6/28/1965	Aerial Photograph	Delta Geomatics
5/22/1967	Aerial Photograph	Sonoma County. Water Agency
7/9/1970	Aerial Photograph	Delta Geomatics
6/6/1974	Aerial Photograph	Aerial Archives
6/12/1979	Aerial Photograph	Sonoma County. Water Agency
10/17/1981	Aerial Photograph	US Army Corps of Engineers
4/23/1985	Aerial Photograph	Delta Geomatics
3/25/1986	Aerial Photograph	CA Dept. Boating and Waterways
6/30/1990	Aerial Photograph	Sonoma County Dept. of Planning
7/5/1990	Aerial Photograph	Unknown
9/15/1992	Aerial Photograph	Nat. Res. Conservation Service
6/9/1993	Aerial Photograph	CA Dept. Boating and Waterways
7/12/1993	Orthophoto	Google Earth[2]
9/11/1998	Aerial Photograph	Aerial Archives
4/29/1999	Aerial Photograph	Nat. Res. Conservation Service
7/15/2004	Aerial Photograph	Google Earth[2]
11/2/2004	Aerial Photograph	Google Earth[1]
12/31/2004	Aerial Photograph	Google Earth[2]
6/11/2005	Aerial Photograph	Google Earth[2]
5/8/2006	Aerial Photograph	Google Earth[1]
6/7/2009	Orthophoto	US Dept. of Agriculture
10/24/2009	Aerial Photograph	Google Earth[2]
6/11/2010	Aerial Photograph	Google Earth[1]
5/6/2012	Aerial Photograph	Google Earth[1]

[1]Original image from the US Dept. of Agriculture.

[2]Original image from the US Geological Survey.

The control site, Wright's Beach, showed no consistent trend in shoreline position or beach width change. Further, the different rates of change among the three transects at the site were less than half as large as those observed at Blind Beach and GRSB, indicating a disconnect between long-term geomorphic changes near GRSB and elsewhere in the littoral cell.

5.2 Sediment Budget

In this section, we summarize the existing information about the sediment budget in the Russian River Littoral Cell to give context for the long-term beach morphology trends discussed above. This involves quantifying the sources and sinks of sediment in GRSB. Sources of sediment to GRSB include the following (Florsheim and Goodwin 1993; Patsch and Griggs, 2006):

- Bed load and suspended load contributed from tributaries, upstream reaches, and bank erosion in the watershed,
- Onshore movement of sand in the coastal zone,
- Alongshore transport into GRSB from adjacent beach compartments,
- Cliff erosion,
- Dune erosion

Sinks of sediment include:

- Offshore movement of sand,
- Alongshore transport out of GRSB to adjacent beach compartments,
- Dune growth

For this study, we assume that the Russian River is the dominant source of sediment, and that alongshore transport is the dominant sink. Prior studies of sediment on GRSB have included the assumption that onshore and offshore transport of sediment balanced (de Graca, 1976). Dune growth/erosion and cliff erosion are not included in this analysis for simplicity.

5.2.1 Baseline Sediment Supply from the Russian River

The majority of beach sediment supplied to GRSB by the Russian River is delivered during floods, and the amount supplied during the months from May to October, when flows are often below 200 ft^3/s, accounts for less than one percent of the total load. Between 150,000 yd^3/yr (Slagel and Griggs 2008) and 181,000 yd^3/yr (Willis and Griggs, 2003) of beach sediment reaches the mouth. These values are based on suspended sediment measurements at the USGS Hacienda Bridge gage (gage #11467000) collected from 1965-1986 and flow measurements at the same site, which began in 1939 and continue today (USGS, 2012).

Year-to-year variation in sediment supply to the beach is substantial due to the erratic nature of rainfall in California. As an example, the total sediment supply during the drought of 1976-1977 was less than 0.1% of the supply in the subsequent winter of 1977-1978. As shown in Figure 5-4, total annual discharge varied from 2.74×10^{9} ft^3/yr (WY 1977) to 1.82×10^{11} ft^3/yr (WY 1983) during the time when the sediment gage was active at Hacienda Bridge. Within the same period, the total load of suspended sediment load varied from 45,000 yd^3/yr (WY 1977) to 83,000,000 yd^3/yr (WY 1983).

The sediment supply estimate of Slagel and Griggs (2008) accounts for losses behind Coyote and Warm Springs dams, and can be used to approximate a timeline of beach-size sediment supply to the beach:

- pre-1958: 218,200 ± 87,300 yd^3/yr
- 1958-1982: 187,100 ± 74,900 yd^3/yr
- 1982-present: 150,200 ± 59,800 yd^3/yr

Error ranges are based on sampling error and errors in the methods (Slagel and Griggs, 2008) and amount to 40% of the sediment load. Flow diversions from the Eel River to the Russian River beginning in 1908 are assumed to have a small impact on the sediment load, since these mostly alter the summer baseflow, when a minimal amount of sediment is transported (SEC, 1996). The combined 69,000 yd^3/yr of beach-size sediment trapped behind Coyote and Warm Springs dams (Willis and Griggs, 2003) after 1982 amounts to a 23-53 percent reduction of the total sediment delivery to the mouth (considering the error bounds of the sediment delivery estimate). It is important to note that some of this volume may have been replaced by erosion downstream dam, and these losses may be higher than the actual values.

5.2.2 Sediment Export

An early sediment provenance study by the USACE (1962) indicated that Russian River sediments are abundant on most beaches south of Goat Rock. This suggests that a significant amount of the sediment supply from the river still bypasses Goat Rock, despite the barrier formed by the present-day Goat Rock Parking Lot. Sediment supplied to the beach by the river can by exported by alongshore transport, the movement of sediment parallel to the shoreline, caused by waves breaking at an oblique angle to the shoreline. Although it can be either a source or sink, the net southerly transport throughout the Russian River Littoral Cell (de Graca, 1976) and the scarcity of beaches north of river mouth suggest that it is mainly an exporter of sediment from GRSB (Minard, 1971). The USACE (1969) and de Graca (1976) provide a range for the net transport of 50,000-150,000 yd^3/yr for GRSB using field observations and refraction diagrams in conjunction with early technical publications. Both studies relied on a limited amount of wave data given by hindcasts for the years 1956-1958 (NMC, 1960).

Estimates of alongshore transport have high errors (on the order of ± 50%), so the usefulness of these studies is limited considering the variability in the local wave climate. It is likely that the transport away from GRSB increases after deposition of sand resulting from a large fluvial discharge. The protrusion of deposited sand into the surf zone can increase the rate of offshore and alongshore transport as wave power re-establishes the shore profile.

The shoreline trends on GRSB (Figure 5-3) can be used to approximate the amount of sediment trapped by Goat Rock on a yearly basis. The long-term shoreline accretion suggests that more sediment is entering the system each year than leaving. Multiplying the average accretion rate of 1.5 ft/yr by the length of the beach (~6000 ft) and also by the vertical distance between a representative beach crest height (10-20 ft) and the reported depth of closure of -30 ft (USACE, 1965) gives a volume of 13,000 – 17,000 yd^3/yr. This is an approximation of the net change in beach volume per year, and represents the difference between the amount of sediment supplied to the beach and the amount exported from the beach on a yearly basis. Although this is an approximate value, it suggests that a small amount (~ 10 percent) of the sediment supply to the beach from the river remains on the beach each year, adding to the total volume.

5.2.3 Changes to Sediment Supply from the Watershed

Within the watershed, human impacts have altered the supply of sediment to the beach with the construction of dams, water diversions, gravel extraction, bank stabilization and channelization practices, and agricultural activities (Florsheim and Goodwin, 1993). Natural events such as fires also influence the sediment supply by removing vegetation within the watershed. For the present section we only discuss the influence of dams, timber harvest, and land-use changes on the sediment budget, as these are known to be dominant factors in altering the sediment supply in the Russian River watershed (Opperman et al. 2005). A timeline of relevant supply events is given in Figure 5-4.

Timber harvest alters the sediment budget by removing vegetative cover, which has the effect of increasing erosion, and thus supplying more sediment to the Russian River, some of which is delivered to GRSB. Timber harvest has taken place in the watershed since at least 1860. However, several periods of heightened activity occurred in the twentieth century. Logging prior to 1860 only affected local areas adjacent to Bridgehaven and Duncans Mills, but were responsible for removing most of the original redwood forests adjacent to the Estuary (Marcus and Associates, 2005; Noss, 2000). A boom in logging occurred after the 1906 earthquake, in response to the need for building materials in San Francisco. This is considered the peak period of timber harvest in the region (Florsheim and Goodwin, 1993). A population boom in Sonoma County from 1945 to 1970 resulted in another period of heightened logging activity. Logging

approaches were more destructive during this time, relying on clear-cutting practices using motorized equipment which was not available before. Clear-cutting in the watershed between 1942 and 1961 was largely unrestricted by regulations and resulted in excess sediment influx to the Russian River (Arvola, 1976; Ziemer, 1991). In 1976, the California Forest Practice Rules were amended and timber harvest became more strictly regulated (Arvola, 1976). Since this time, timber harvest has declined heavily, although forest cover is still removed for conversion to residential use and vineyards (Merenlender, 2000; Shih, 2002).

Land-use activities that replace native vegetation increase sediment loading to streams by causing rill and gully erosion, and by enhancing peak flood discharges by reducing percolation of runoff (Florsheim and Goodwin, 1993; Waters, 1995; Opperman et al., 2005). Aside from the logging activities from 1860 to 1971, much of the land-use change in the Russian River watershed is related to population increases after 1945 (DWR, 1964). In 1940, the total population of Sonoma County was 69,052. The population doubled between 1940 and 1960, and again between 1960 and 1980, after which time the growth began to slow to its present rate of 5.2 percent per decade. Between 1950 and 1990, much of the valley floor became dominated by vineyards, urban areas, and suburban developments (Opperman, 2005). Between 1959 and 1990, the amount of land developed for either urban or agricultural use increased from 138,000 to 676,000 acres (CDC, 2010). Although this amount has decreased since 1990, recent development of vineyards and low-density residential housing has shifted to hillslopes within the watershed (Merenlender, 2000), which can have a larger effect on sediment supply to the river than development within the flat areas (Battany and Grismer, 2000).

5.2.4 Relation to Beach Changes

Most of the individual beach transects shown in Figure 5-3 show periods of rapid beach change prior to 1990 but a high amount of scatter afterward (e.g. Figure 5-4). Although Figure 5-3 suggests that the addition of the littoral barrier at Goat Rock is important to this trend, the sediment supply has also likely played a significant role in the long-term beach evolution. The combined influence of logging practices and land conversion for urban or agricultural uses within the watershed increased the supply of sediment to the Russian River from the 1940s to the 1980s, which probably increased the supply of sediment to the beach during this time. These activities peaked at the same time that shoreline evolution on GRSB was fastest and had the least scatter about the linear regression at most transects. Both logging and land conversion have declined substantially since that time, as regulations have been enacted and population growth in Sonoma County has slowed. Other actions, such as the construction of Warm Springs Dam in 1982 have also probably reduced the supply of sediment in recent decades (Willis and Griggs, 2003). The scatter in the shoreline position trends after 1990 may not necessarily indicate a slowdown in beach accretion, but a slowdown is expected given the recent decreases in sediment supply from watershed activities.

5.3 Reference Sites

To provide an understanding of how the morphology of GRSB might change without the jetty, we examined several nearby estuaries without jetties that had similar morphology (Table 5-2). These include the Mattole River Estuary in Humboldt County, the Navarro and Gualala River estuaries in Mendocino County, and both Estero Americano and Estero San Antonio in Sonoma County. These sites are classified as bar-built estuaries, with the beach forming a barrier separating the lagoon from the ocean. Each beach is bounded by headlands at the north and south end. Since all sites are within 150 miles of GRSB, they can be assumed to have similar tides and offshore wave conditions. As with the Russian River watershed, rainfall is seasonal, and the tidal prism at each site is not capable of maintaining an open inlet for the entire year.

The reference sites were compared by the shape of the beach, which was assessed using the available 2002 and 2010 LiDAR maps of the northern California coastline (CCC, 2012). Cross-shore transects were drawn at the center and at the north and south ends of each beach in ArcGIS, and the LiDAR topography was used to characterize the beach shape along each transect. At GRSB, the north transect is located immediately north of the groin, and the center and south transects between the groin and the Beach Parking Lot.

Table 5-2. Beach characteristics at Russian River inlet and reference lagoons.

Site	Location	Distance from Russian River (mi.)	Beach Length (mi.)	Width at south end (ft)[1]	Height at south end (ft NAVD88)[2]
Mattole R.	40º17'43" N 124º21'17"W	143 north	1.7	550	20.2
Navarro R.	39º11'29"N 123º4'36"W	63north	0.5	235	11.6
Gualala R.	38º45'58"N 123º31'54"W	32 north	0.65	586	21.6
Russian R.	38º27'00"N 123º07'47"W	--	1.2	685	24.8
Estero Americano	38º17'46"N 123º00'09"W	12 south	0.2	492	21.8
Estero San Antonio	38º16'11"N 122º58'41"W	15 south	0.2	614	19.9

[1]Beach width measured at mean higher-high water level using 2010 LiDAR data.

[2]Beach height measured from 2010 LiDAR.

The sites all had a similar overall beach shape, but with variations in the height and width of the berm (Figure 5-5). The key traits can be summarized as follows:

- The beach crest slopes upward from a minimum height at the north end to a maximum at the south end of each beach.
- The south end of the GRSB spit is significantly wider and taller than the south ends of most of the other reference sites.
- Sites with inlets that are known to migrate along the beach every year (Navarro and Mattole rivers) form migration terraces, where the beach is relatively narrow and flat.

The last point is important because of its potential implications for GRSB. Prior to the construction of the jetty, the Russian River inlet was known to open south of the present-day groin. Figure 5-6 gives an example of the influence of inlet migration, comparing the beach shape at the mouth of the Navarro River between 2002 and 2010. The 2002 LiDAR data captured the beach topography during a period of inlet closure, at which time the beach crest developed a north-to-south slope, similar to the other sites (Figure 5-5). In contrast, the 2010 LiDAR data were collected when the inlet was open. Prior to the 2010 flight, the inlet had opened at its northern end and migrated southward along the entire extent of the beach, creating a lower beach crest profile (Figure 5-6).

The shoreline orientations are similar at the Russian and Navarro rivers, and the offshore wave conditions are also likely similar. For these reasons, the inlet on GRSB could be expected to periodically migrate south of the present-day groin if the groin were removed. At present, the inlet undergoes a seasonal migration pattern north of the groin (Behrens et al. 2009), typically migrating north during winter floods and returning south to the jetty during the spring or summer. A smaller (lower and narrower) beach berm associated with this change could increase barrier seepage but also limit the height that water surface elevations can attain in the lagoon, by limiting the crest height of the berm.

5.4 Future Morphology with Sea Level Rise

Sea level rise is a well-established and expected consequence of ongoing global warming. Sea levels are rising because global warming increases both thermal expansion of water (e.g. warmer water occupies more volume) and ice melt from land-based glaciers. This sea level rise is expected to contribute to an increase in the severity and duration of flooding and an acceleration of coastal erosion, as well as an overall retreat of GRSB landward. In this section we assess the shift in beach position as a response to the range of sea level rise.

Global, or eustatic, sea level rise is the combination of two factors: (1) thermal expansion of the ocean and (2) melting of global ice (Cayan et al. 2006). Local sea level rise is a combination of global sea level rise together with local factors such as local atmospheric circulation and vertical land movement. Vertical land movement is important because it can either enhance or mitigate relative sea level rise locally (BCDC, 1987). This vertical movement can occur in response to tectonics (earthquakes, regional subsidence or uplift), sediment compaction, isostatic readjustment and groundwater depletion (USACE, 2009). Predictions of local sea level rise for the Russian River region vary based on the type of climate change scenario used in the prediction. NRC (2012), which has been adopted by California as official sea level rise guidance, gives a

projection of 36” of sea level rise by 2100 and a possible range of approximately 17 – 66 inches of sea level rise.

Sea level rise causes shoreline retreat by raising the elevation of tides and waves relative to the beach. The beach, having been formed at the shoreline by the tides and waves, will adjust as sea level rise moves this shoreline upward and landward. As sea level rises, so too do total water levels, the combined mean sea level, tides, and wave runup. The beach crest, having been formed by total water level’s wave overwash (Donnelly, 2007), is expected to shift upwards at the same rate as sea level rise. In addition, sea level rise’s higher total water levels can threaten dune fields and cliffs behind the beach to greater rates of erosion (Sallenger et al. 2006).

The change in the beach position in response to sea level rise and the sediment supply to the beach (see above discussion on sediment budget) can be estimated using the Bruun Rule (Dean, 1990). This relates the change in beach position to three components:

- The length from the crest of the berm to the farthest offshore depth where seasonal erosion or accretion occur
- The vertical height from the crest of the berm to the location where this offshore point exists
- The long-term change in water surface elevation associated with sea level rise

For a given cross-beach profile (e.g. taken from survey data), this analysis predicts the vertical and horizontal (landward) shift of the profile with time. The background accretion rate of the beach (e.g. Figure 5-3) is combined with the Bruun Rule estimate to give the net long-term change in the beach position. The details of this approach are described further in Appendix B.

We applied this analysis to two representative beach profiles at GRSB, using sea level rise of 1, 3, and 5 ft as representative values within the expected range of sea level rise by 2100 (NRC, 2012). To examine the potential effects in relation to the jetty elements, we looked at survey transects with and without jetty elements present. Figure 5-7 shows the projected response of the beach to the range of sea level rise south of the groin at Transects 6 (north of the groin) and 11 (south of the groin), as shown in Figure 5-1. If no background beach accretion is present, the beach is predicted to move landward by amounts of 30, 60, and 150 ft for the cases of 1, 3, and 5 ft of sea level rise, respectively (Figure 5-7). The beach crest is assumed to shift upwards to pace sea level rise and the beach width is assumed to remain relatively unchanged.

For jetty access elements, such as the seawall, which are embedded in the beach landward of the crest, this change would bury the elements over time (Figure 5-7). This could alter berm seepage depending on the position of the seawall relative to the regions with fastest seepage flows. However, the groundwater permeability assessment

(Section 3) has not sufficiently resolved the vertical structure of seepage to make this determination. The seaward tip of the groin structure would become more exposed as a result of the shift in the beach profile, while the rest of the structure would also be buried by the rising beach. Since the top of the groin is well above mean sea level at approximately 12-17 ft NGVD29 (as described in Section 2), the more exposed seaward end of the groin would still interact with surf zone and the inlet.

For the part of GRSB north of the groin and beyond the extent of typical inlet migration (Transects 1-4 in Figure 5-1), sea level rise would result in a retreat of the beach into the backbarrier cliffs, which would also undergo erosion from extreme wave events (Sallenger et al. 2006). This would also happen where dunes back the beach between the Beach Parking Lot and Goat Rock (Transects 8-16 in Figure 5-1).

Since the future rates of shoreline position change are uncertain (see Figure 5-4 or above discussion of sediment budget), it is difficult to predict when and how far the beach would move inland versus the historic trend of adequate sediment supply resulting in shore accretion (seaward and upward building). If we consider the long-term (1930-2012) background accretion rates (Figure 5-3) in the Bruun Rule analysis, the movement caused by sea level rise would be slower, causing the beach to increase in height but adjust more slowly in the horizontal. The long-term trend may be changing, as discussed in Section 5.2.4 for post-1990 conditions, but a longer record is needed to see if the recent trend will diverge from the long-term trend. Even with the addition of background accretion, the jetty elements are still likely to be buried over time.

5.5 Synthesis

The beach berm at GRSB has changed significantly since the initial construction of the jetty complex between 1924 and 1934. This has implications for the lagoon water surface elevation, because the beach acts as the barrier separating the lagoon from the ocean, enabling the lagoon water surface elevation to perch above the ocean. The most important changes to the beach morphology at GRSB can be summarized as:

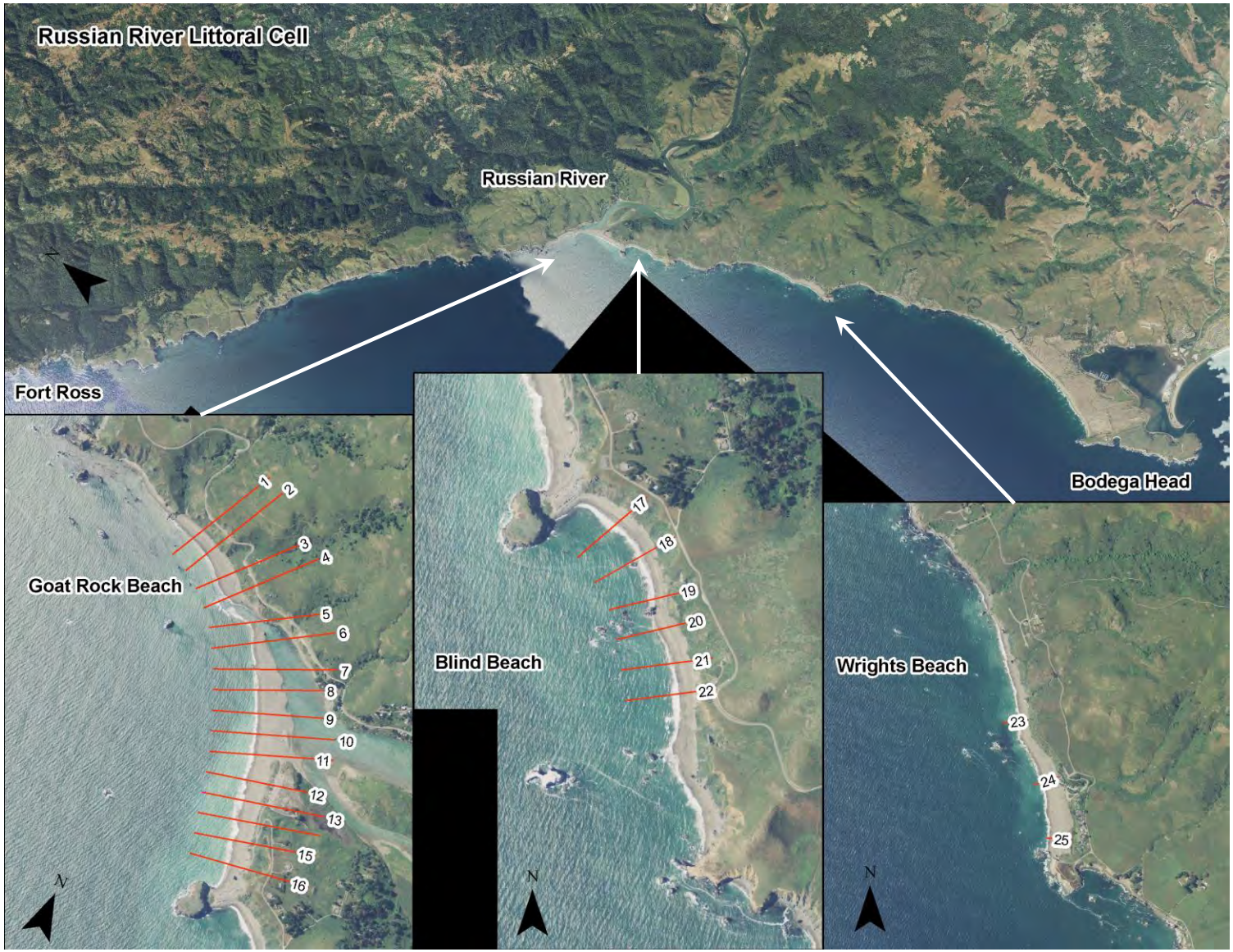
- A nearly spatially-uniform shoreline accretion toward the ocean of 1.5 ft/yr,
- An increase in the width of the beach, with maximum rates of 2.2 ft/yr on the beach spit separating the lagoon from the ocean, and
- Vertical stabilization of the beach crest south of the groin.

The first point is likely a result of the Goat Rock Parking Lot blocking littoral transport across the historic tombolo between Goat Rock and the main land. The beach accretion may also be influenced by sediment supply from the watershed, which may have reached a peak between 1940 and 1980. The second point has probably led to a decrease in seepage compared with pre-jetty conditions. Beach berm seepage from the Estuary to the ocean is strongest for high water surface slopes between the lagoon and the ocean, and is expected to have reduced with a wider beach. Although the segment

of the beach spit north of the groin is still relatively narrow, this accounts for about a third of the total spit length fronting the Estuary. Lastly, the existing seawall and other access elements south of the groin appear to have maintained an artificially high berm by preventing inlet opening or migration south of the groin. This has probably contributed to the observed widening of the spit.

In response to sea level rise, GRSB is expected to migrate landward and upward. The extent of the landward adjustment will vary depending on future rates of sediment supply to the beach and the continued adjustment of the beach to the disruption of the littoral transport at Goat Rock. The vertical adjustment of the beach will likely bury the existing jetty access elements and most of the groin. To the extent that the beach moves landward, the seaward end of the groin could become more exposed, influencing wave breaking patterns and the alongshore transport. Burial of jetty elements could lead to a flatter beach crest profile, since the additional sand above the jetty elements would be more readily transported and the waves may more frequently overwash along the entire length of the spit. To the extent that sea level rise mitigates the effectiveness of the jetty, either due to burial or because of further degradation by wave action, the inlet morphology may change. Changes potentially include greater range of inlet location due to migration south of the existing groin, and increased water outflow through the sand berm to the ocean.

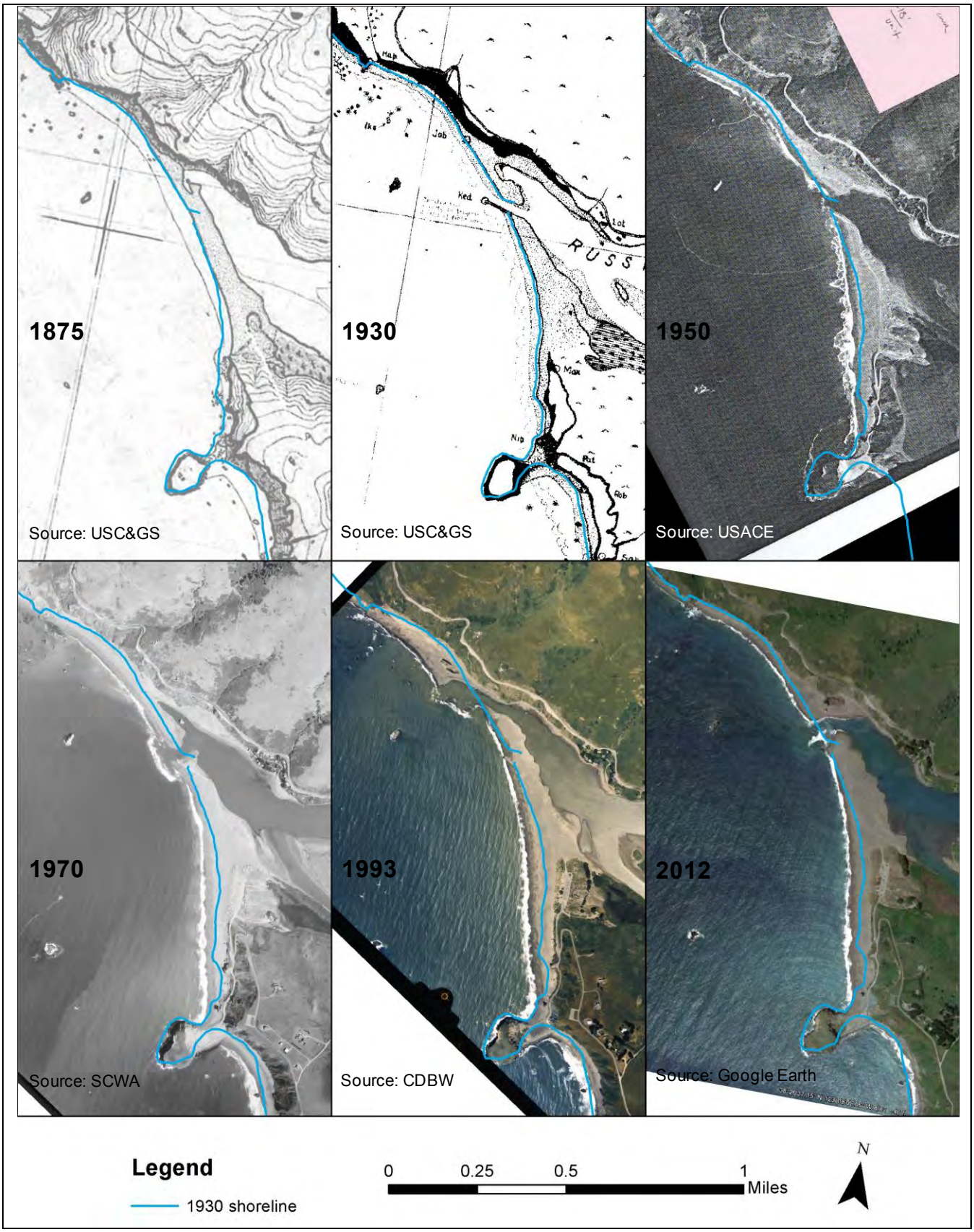
5.6 Figures



NOTE: Bars shown in insets represent transects used in the analysis of historical images. Background image in all panels is an orthophoto taken in 2009 by the USDA NAIP.

Figure 5-1

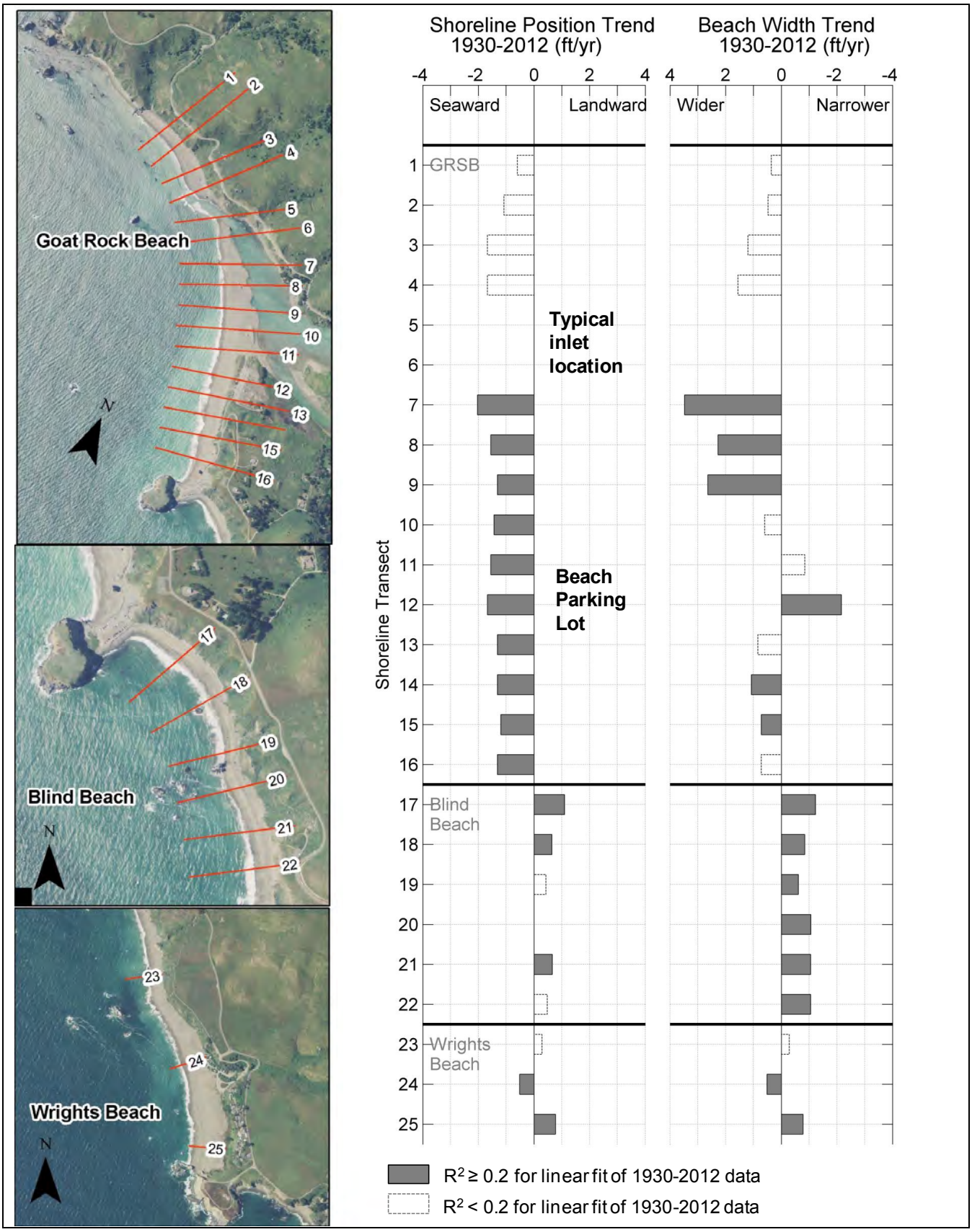
Aerial view of the Russian River Littoral Cell, with insets of beaches analyzed with historical aerial images.



SOURCE: Image sources given in Table 5.1

Figure 5-2

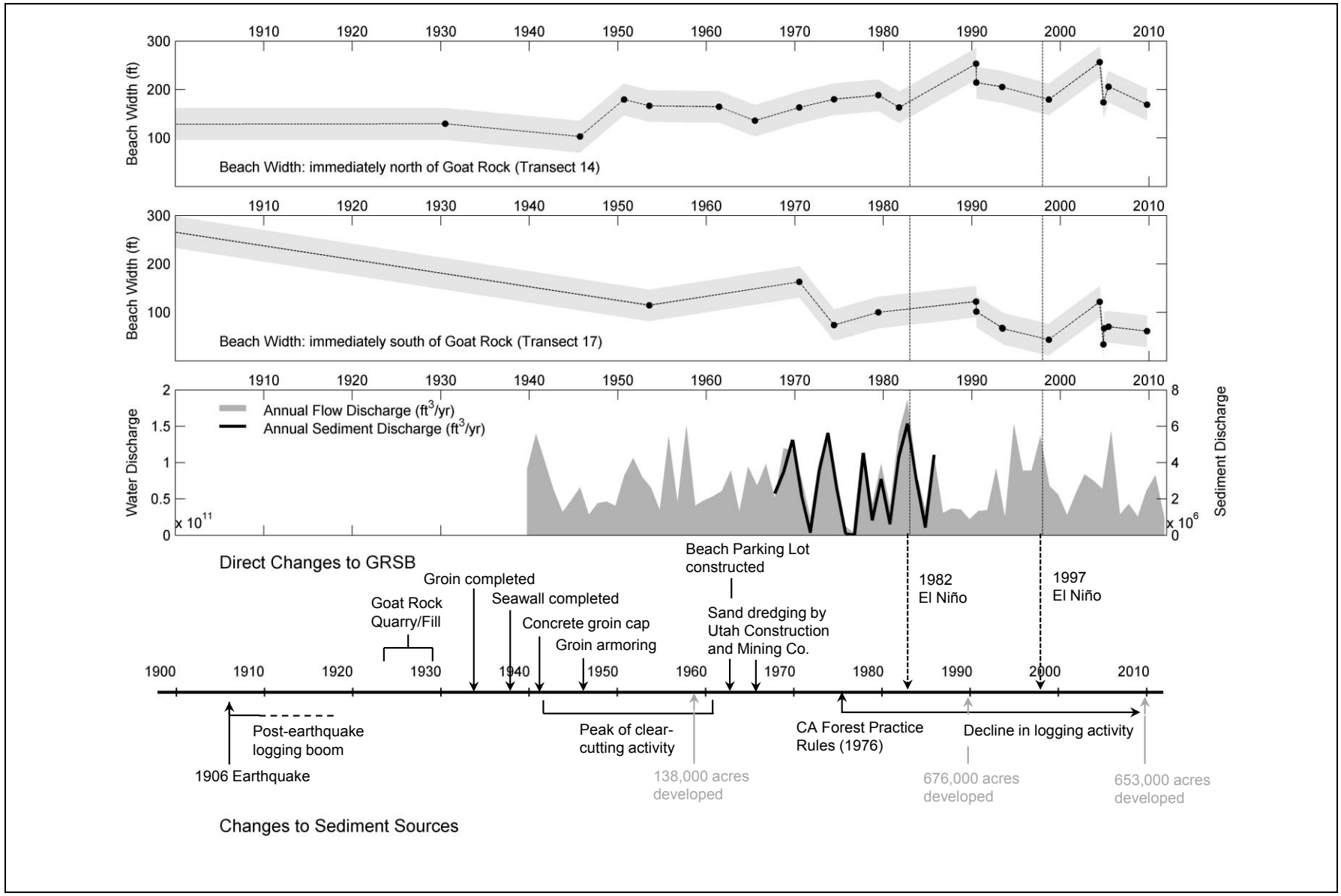
Aerial views of GRSB and the northern extent of Blind Beach taken from T-sheets and aerial imagery.



NOTE: All linear trends were obtained using DSAS with a linear fit of the data. Data and trend lines are given in a separate appendix

Figure 5-3

(left) Beach transects and **(right)** corresponding trends in shoreline position and beach width

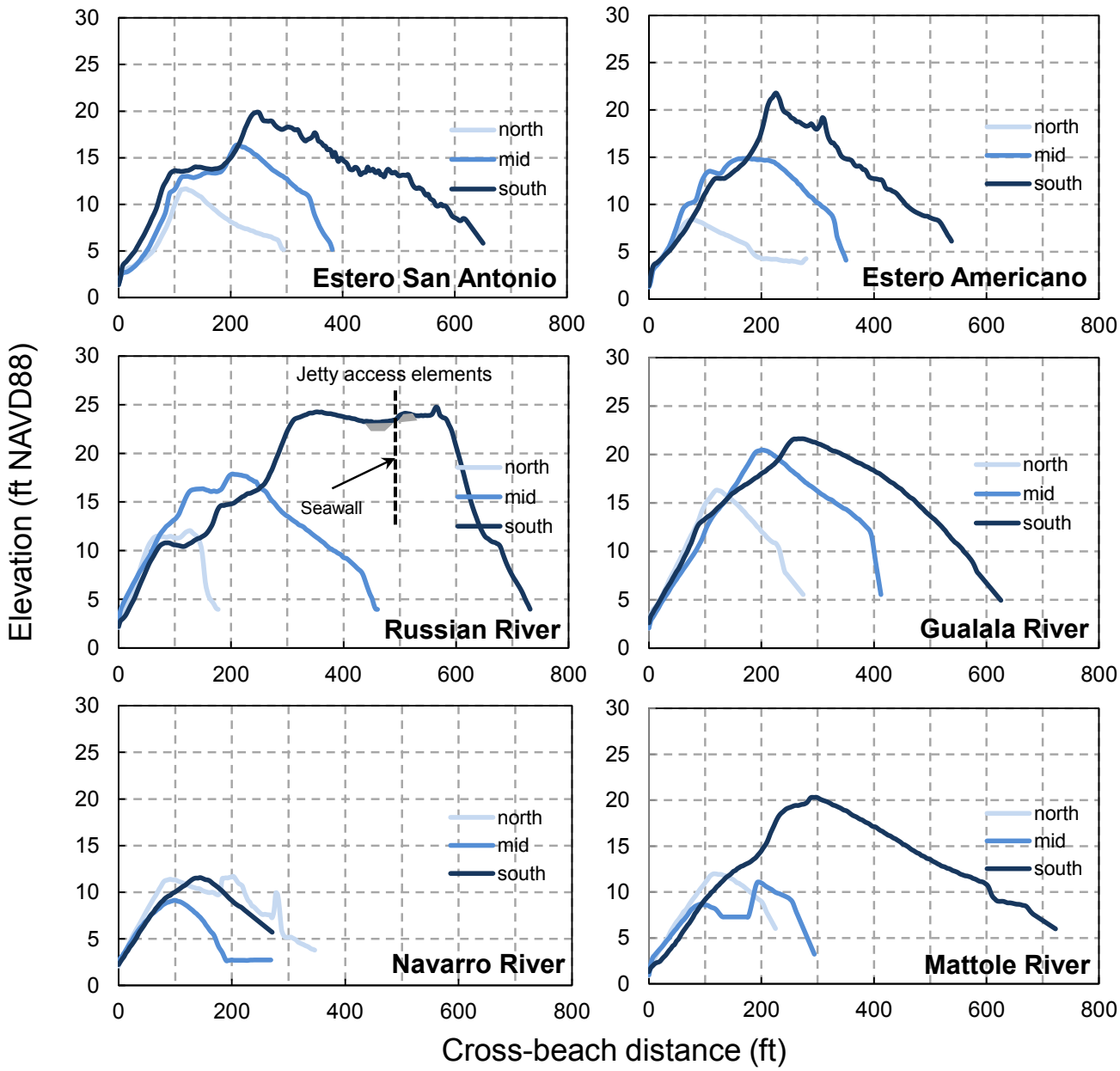


SOURCES: References for direct changes to GRSB: Behrens (2012), Johnson (1959), Magoon et al. (2008), Rice (1974), Schulz (1942)

References for Changes to Sediment Sources: Arvola (1976), CDC (2010), DWR (1964), Florsheim and Goodwin (1993), Merenlender et al. (1998), Opperman et al. (2005), Runyan and Griggs (2002), Slagel and Griggs (2008), SEC (1996), Waters (1995), Willis and Griggs (2003)

Figure 5-4

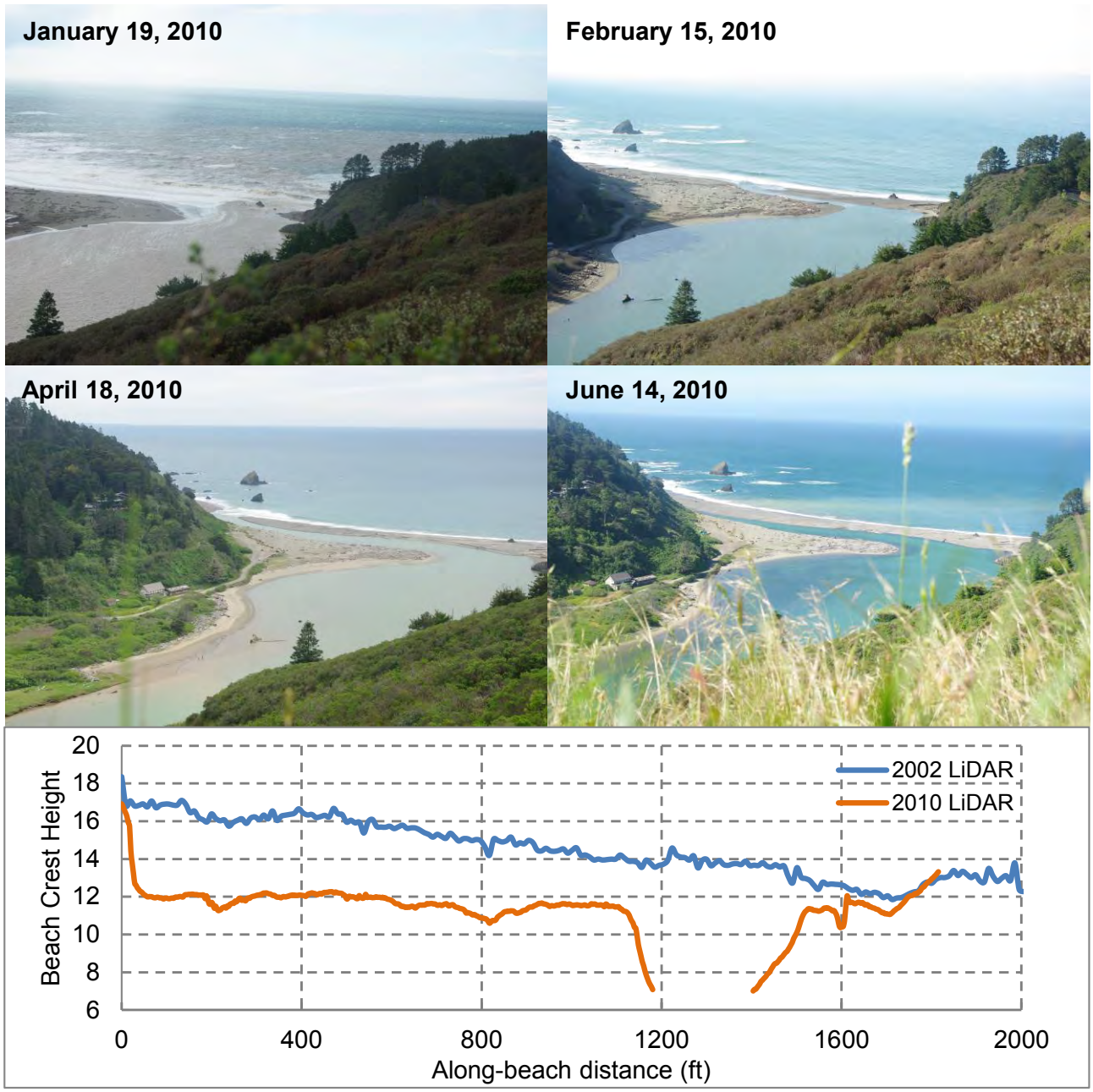
(upper panels) Time series of beach width north and south of Goat Rock, compared against **(mid panel)** river and sediment discharge and **(bottom panel)** changes to beach and sediment supply



NOTE: Transects were only taken on the spit between the estuary and ocean at each site. Sites were chosen based on similarity to the Russian River Estuary. All sites are located within 200 mi. of GRSB.

Figure 5-5

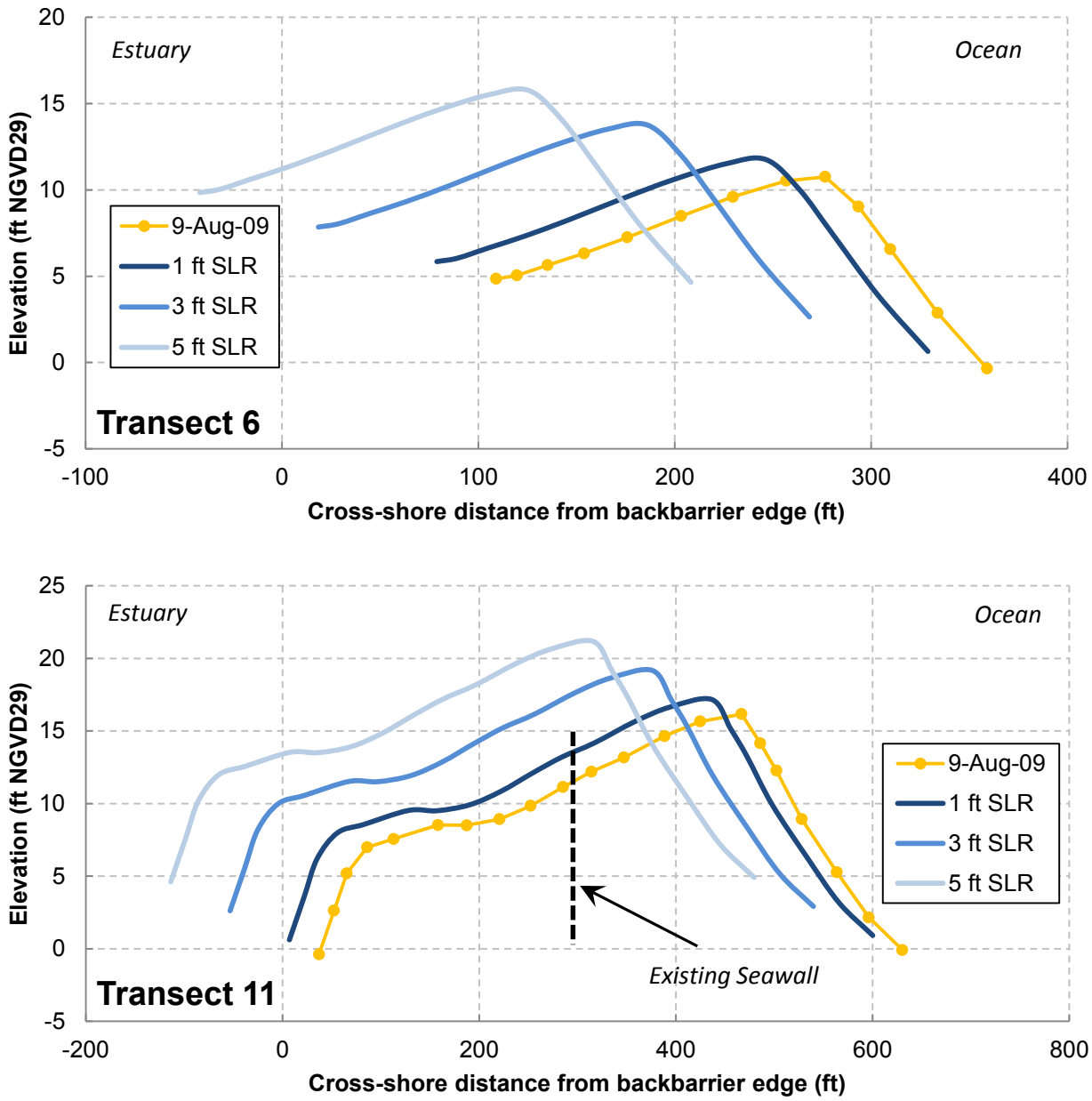
Beach transects from the GRSB and five reference sites, using September 2010 LiDAR data.



SOURCE: Images provided by D. Behrens. Beach crest profiles extracted from LiDAR data collected by CCC (2012)

Figure 5-6

Navarro River inlet migration in 2010, with comparison of crest profiles in 2010 (open inlet) and 2002 (closed inlet).



SOURCE: Aug 9, 2009 survey data provided by Behrens (2012)

Figure 5-7

Examples of potential beach adjustment due to sea level rise at transects with **(upper panel)** and without **(lower panel)** jetty elements present.

6 INLET MORPHOLOGY

Inlet morphology refers to the changes in inlet dimensions and alignment that occur in response to river discharge, tidal exchange, and ocean waves and cause the formation of a barrier beach. Like most other inlets in California, the Russian River mouth is a dynamic system that can rapidly change state at the tidal time scale of hours or be quasi-stable for months. Because its shape determines the magnitude and direction of the flow between the Estuary and the ocean, the morphology of the inlet is a key determinant of the Estuary water surface elevation.

To summarize the inlet morphology, the mouth of the Russian River can be categorized into a small number of states:

- **Closed-mouth:** When waves tend to dominate over tidal and river flows, tidal and fluvial currents are insufficient to erode wave-delivered sand, and the mouth closes, providing a high amount of freshwater habitat in the Estuary, trapped behind the beach.
- **Tidal Inlet:** When waves are weak or when river floods or spring tides dominate, the mouth becomes a deep tidal inlet, creating a smaller space of colder, brackish-saline habitat that is more similar to marine conditions.
- **Muted Inlet:** When the balance of sand delivery from ocean waves and erosion from inlet flows does not cause either of the above cases, a shallow sill usually forms in the mouth and partially mutes tides in the Estuary, by cutting off lower tides.
- **Perched overflow:** When the sill in the mouth builds up and the mouth transitions from open to closed, it sometimes temporarily operates as an overflow channel, spilling water to the ocean over a beach that is above high tides.

Due to the complex interplay of coastal and fluvial processes in the Estuary, the mouth of the Russian River frequently transitions from state to state, transitioning from tidal inlet, to muted inlet, to perched overflow, to closed. When the lagoon fills to the beach crest, it spills over the beach and erodes a tidal inlet again. The jetty may have a direct effect on Estuary water surface elevations if it changes the frequency at which the inlet changes between these morphologic states. Whether or not the jetty's effect is favorable to the target Estuary water surface elevation outlined in the Biological Opinion (NMFS 2008) depends on which state changes the jetty affects the most.

The purpose of this section is to outline a model of the mouth state and the resulting lagoon water surface elevations. We validate this model with observations from 1999 to 2014. In a later section we use the model to examine the potential effects of the jetty on lagoon water surface elevations.

6.1 Lagoon Quantified Conceptual Model

ESA have developed a quantified conceptual model (QCM) of the Russian River Estuary. At its core, the QCM is a water balance model, accounting for the different sources of inflows and outflows to the Estuary. This water balance is coupled with a sand balance that makes up a dynamically-varying beach and inlet system, accounting for the fact that bar-built estuaries are often defined by a morphologically unstable mouth (inlet) that influences the lagoon water surface elevation, volume, and flows.

The model dynamically simulates time series of inlet, beach, and lagoon state based on external forcing from waves, tides, and stream input. The QCM approach was originally developed for Crissy Field Lagoon, in San Francisco Bay (Battalio et al. 2006) and has since been refined using approaches developed by ESA PWA for Scott Creek, Devereux Slough, and Mission Creek lagoons. It benefits from lessons learned in similar approaches from Shuttleworth et al. (2005) in Australia and more recently from Rich and Keller (2013) for Carmel River Lagoon. Peer-reviewed application of the QCM to the Russian River Estuary was published in Behrens et al. (2015). The model is based on two core concepts:

- All water flows entering and leaving the system should balance.
- The net erosion/sedimentation of the inlet channel results from a balance of erosive (fluvial and tidal) and constructive (wave) processes.

6.1.1 Lagoon Water Balance

The lagoon water balance is illustrated in Figure 6-1 and can be expressed as:

$$\Delta V_{lagoon} = (Q_{stream} + Q_{mouth} + Q_{overwash} - Q_{seep} - Q_{evaporative} + Q_{error})\Delta t \qquad (1)$$

Where ΔV_{lagoon} is the change in lagoon volume, Δt is the time step, Q_{creek} is the freshwater input to the system, Q_{mouth} is the mouth flow rate (may be positive or negative), $Q_{overwash}$ is the flow rate into the lagoon of waves overtopping the beach crest, Q_{seep} is the seepage flow through the beach berm, $Q_{evaporative}$ represents losses from evapotranspiration, and Q_{error} is an error term. For each time step, the sum of all inflow and outflow terms is multiplied by the length of the time step to give the change in lagoon volume, which is used in conjunction with a known stage-storage curve to arrive at the new lagoon stage.

Stream inputs are taken from the USGS Guerneville Gage. Wave overwash is estimated using the empirical method of Laudier et al. (2011) with a constant beach slope of ten percent, as discussed in Chapter 4. Evapotranspiration is estimated using data from BML, 10 miles away. Seepage losses are estimated with a D'Arcy approach (Bear, 1988) using Water Agency beach surveys to characterize beach width (Behrens et al. 2015). Tidal flows through the mouth are resolved using the solution to a one-dimensional momentum equation accounting for water surface slope and channel friction. This is described in further detail by Behrens et al. (2015).

6.1.2 Lagoon Mouth

The size and shape of the lagoon mouth are influenced by the estimated inlet flows, via a set of empirical relations that include data from small inlets throughout the US Pacific and Atlantic coasts, and parts of Australia and New Zealand (Behrens et al. 2015). Inlet hydraulics are estimated with the Van de Kreeke (1967) model. Flow velocities through the channel are powered by the head difference between the lagoon and ocean tides, and are slowed by channel friction, which scales with inlet length, the sediment type (taken here as coarse beach sand), and inversely with depth. Inlet shape and flow rate are interrelated, and flow velocity is used to assess the total erosion rate in the inlet bed for each time step. Deposition from waves is also assessed based on nearshore wave power and TWL. The deposition rate in the inlet is adjusted via an "inlet trapping efficiency" (Rosati 1999), which is the main way the model is adjusted to match observed inlet closure events. Total erosion and deposition are summed with each time step to give a net deposition/erosion rate, causing the mouth thalweg in the model to either erode (i.e. during breach or flooding events) or accrete (during mouth closure).

As the QCM advances in time, the inlet state ("closed" or "open") is determined based on the elevation of the mouth thalweg relative to the ocean and lagoon levels. When closed, mouth flow terms are zero, and the seepage and evaporative terms become the only pathway for flows to leave the lagoon. Breaches are induced in the model if lagoon levels overtop the barrier beach elevation. When this happens, the model reintroduces a small channel on the beach, which either leads to non-breaching perched overflow conditions or a full inlet breach depending on hydraulic conditions (primarily driven by hydraulic head between the lagoon and ocean water surface elevations). Figure 6-2 illustrates a flow chart for the model.

6.1.3 Beach Dynamics

The beach berm influences the lagoon hydrology by blocking runoff from leaving the lagoon when the lagoon mouth is closed, by moderating the rate that waves spill into

the lagoon, and by setting the potential flood level. Since the lagoon water surface elevation cannot rise above the minimum beach crest elevation without spilling over the beach, the crest elevation provides a good surrogate for the peak inundation level (Behrens et al. 2015).

In the QCM, the beach is modeled separately from the lagoon and the lagoon mouth. A representative beach width (cross-shore direction), length (alongshore direction) and beach slope are assumed based on the Water Agency monitoring surveys of the beach. The beach crest is modeled in more detail because of its direct influence on lagoon hydrology. The beach crest in the QCM directly influences the hydrology in two ways: it sets the level at which the lagoon self-breaches, and it influences the amount of wave overwash that spills into the lagoon.

Since the beach crest height at GRSB varies along its length, we subdivide the beach into two groups: three “non-mouth” 800-foot segments (two segments south of the groin, and one north) and a variable-width “mouth” segment occupied by the river mouth. The crest height of the non-mouth sections of the beach is constant and is only used to estimate their contribution of the total wave overwash into the lagoon. The crest height within the mouth segment is variable. This part of the beach where inlet closure occurs is crucial, because it tends to be much lower than the other segments as waves have had less time to build it. In the model it also contributes overwash into the lagoon, but also sets the elevation for lagoon self-breaching after closure.

For existing conditions, within each non-mouth beach segment, a representative beach crest elevation is assumed based either LiDAR or Water Agency survey data. For proposed alternatives, as evaluated in Section 9.1.3, a range of crest elevations are applied to represent the effects of modifying the jetty, which could allow the beach to be lower due to expanded inlet migration, or more erosion from wave action. Within the river mouth segment of the beach, the beach crest is assumed to vary in height. When the river mouth is open (i.e. either perched, muted, or tidal inlet phases), the crest is taken to be the same as the inlet thalweg elevation and overwash is not allowed. When the mouth is closed, the beach crest is allowed to grow vertically, but only when TWL exceeds it. The deposition rate is taken to be proportional to the wave power, so long-period swell waves contribute more to the beach growth than shorter-period waves. This growth is capped at the 99th percentile of TWL for the year, which is usually in the range of 12-13 feet NGVD. Prior comparisons of beach crest elevation and the highest percentiles of TWL have found a good relationship between these two variables (Battalio et al. 2006).

Since the river mouth is breached by the Water Agency when water surface elevations approach 7 feet NGVD, the QCM assumes breaches are induced whenever the lagoon water surface elevation reaches this height (i.e. it assumes that flooding is not permitted), even if the beach crest is estimated to be higher. If waves close the mouth of the river but are then too weak to build the beach crest quickly, it is possible in the

model for the lagoon to breach at lower elevations if lagoon water surface elevations overtake the crest height.

6.1.4 Migration

Inlet migration affects the length of the inlet channel in the QCM, which alters the likelihood for closure (longer channels are more frictional and have a higher likelihood of closing in the model). An inlet migration sub-model in the QCM was developed based on comparison of inlet position (from the BML camera and prior observations) against the alongshore vector of wave power generated in Chapter 2. Cumulative migration distance was also compared to a measure of inlet length estimated from the BML camera (ESA 2015b). Migration rate per model time step is related to inlet width and the alongshore power vector with an empirical coefficient, which is intended to characterize the rate of sediment accumulation on one side of the inlet channel. Since artificial breaches and river floods influence the migration at the site, the migration sub-model includes a number of rules:

- Assume artificial breaches take place when the water surface elevation reaches 7.5 ft NGVD.
    - If the inlet breaches naturally below 7.5 ft NGVD, do not relocate the inlet to Haystack Rock.
    - During manual breach events, assume the inlet is relocated to Haystack Rock.
- Assume the inlet only migrates if the inlet flows are less than a threshold value (lower than peak spring tide flows or river flood flows below ~10,000 ft^3/s).
- The inlet length resets to a minimum (100 ft) during breach events and floods above a threshold value (>40,000 cfs).

6.2 Model Results

We use the QCM to simulate lagoon water surface elevations from 1999 to 2014 and test the model against water surface elevations measured by the Water Agency at Jenner during that period. Water Agency records of closure events were also used to test the model predictions of the number of closure events and number of days closed. The water surface elevation record is discontinuous because of occasional instrument malfunction and did not record levels below 0 feet NGVD, so water surface elevations are only compared qualitatively. Also, since the mouth rarely experienced perched, one-way flow from 1999 to 2014, model representation of perched overflow conditions had little data to compare against. This is discussed in more detail in Chapter 8.

The model compares well against the observations, especially with regard to capturing the seasonality of closure events. At times, the predicted lagoon water surface elevations closely reflect the observations as well. This is a reflection that the water balance approach is a reasonable method for predicting the lagoon water surface elevations.

Predicting the timing of closure and breach events was difficult, especially since some of the observed closure events ended in managed breaches and some ended in the lagoon self-breaching. In general, when several closure and breach events were observed to occur in succession, the model did not always match the correct timing of the events. This is an expected shortcoming of the model, given the complexity of the system, the relative simplicity of the approach, and the sensitivity of closure events to previous conditions. As an example, if the mouth self-breaches at 8 feet NGVD due to a large rainstorm flooding the lagoon, the subsequent breach event may scour a deep inlet thalweg, allowing the inlet to remain open for weeks or months afterward. In contrast, if the inlet breaches at a lower elevation (e.g. a managed breach at 7 feet NGVD) a few days before the rainfall event, scouring may be weaker, even if the subsequent rainfall raises discharge into the lagoon, potentially allowing the inlet to close again much sooner. This sensitivity is apparent throughout the data, and thus this model is only anticipated to represent the seasonality of closure events and water surface elevations, rather than specific timing at the time scale of days.

Figure 6-3 illustrates the modeled lagoon water surface elevation from September 2008 through December 2009. River flow and wave power are also shown for context. This figure summarizes a number of expected lagoon behaviors that the model successfully captured:

- Closures are most frequent in fall and spring, when wave power is higher than in summer.
- Closure events are brief when river flows are above 200 ft^3/s and mostly prevented for flows above 1,000 ft^3/s.
- Muted and perched mouth conditions are brief transitions between tidal inlet and mouth closure conditions.
- Water surface elevations during mouth closure cannot be explained only by river flows. Wave overwash contributes strongly to lagoon water surface elevation, especially within the first week of closure when the beach berm is not fully built up.

Although the QCM results shown in Figure 6-3 sometimes deviate from observations, a major advantage of this approach is apparent: these processes that the model reproduces would be hard to predict from river flow or waves alone, or from models that only take into account one or the other. A modeling approach that combines processes in the way the QCM does is a necessary approach to properly model the hydrology of the system.

Figure 6-4 gives another comparison of the modeled water surface elevation against observations, from July to December 2007. In this case, the model predicts a closure event in September that was not observed, and does not predict an observed closure events in early November. However the overall number of days closed is similar between the model and observations, and the timing of closure and breach events is otherwise relatively close.

Figure 6-5 shows monthly summary statistics of the QCM. The model predicts 59 days of closure per year from 1999-2014 compared with 54 observed days of closure. The average number of closure days per month (averaged from 1999-2014) closely follows observations, deviating at most by two days per month in January. The lower panel of Figure 6-5 summarizes the number of closure events. The model predicts nine closures per year, which is close to the eight observed per year. The model under-predicts the average number of events in October even though the number of days closed in this month is more reliable. We attribute this to closure events in the model that begin in September and carry over into October.

To assess the model predictions of tidal muting, the model predictions of tide range were compared with the observed tide range. Tide range was defined as the difference between a day's highest and lowest water level. The estuary tide range is muted relative to the ocean's average tide range of 5.8 ft, with the amount of muting depending on inlet states, river flow, waves, and the spring-neap tidal cycle. The daily tide range during observations made from 1999-2014 were collated into one-foot intervals and then tallied for their frequency, as shown in Table 6-1. Also shown in Table 6-1 are the tide range frequencies for the model predictions of estuary water levels.

Consistent with closure statistics discussed above, the QCM predictions for closure frequency agree closely with observed water levels. When the inlet is open, tide ranges greater than 2 ft are considerably more frequent than tide ranges less than 2 ft. For instance, aggregating across columns in Table 6-1, the observed tide range was greater than 2 ft only 62% of the time and less than 2 ft 17% of the time. The predicted tidal range is biased somewhat higher than observations. The model predicts the tide range to greater than 2 ft for 73% of the time and below 2 ft 8% of the time. An initial review of the observed water level time series indicates that muted tide ranges less than 2 ft typically occur for only a few days at a time, either as the inlet transitions from open to closed or during the weakest neap tides. Some of the difference between observed and predicted tide range, particularly for tide ranges above 4 ft, is probably due to the elevation of the Jenner water level gage. When the estuary drops to its lowest water levels, the water level falls below the Jenner gauge. Hence, the observations do not fully record the lowest water levels, and therefore under-predict the tide range when it is largest.

Table 6-1. Observed and predicted estuary tide range frequency.

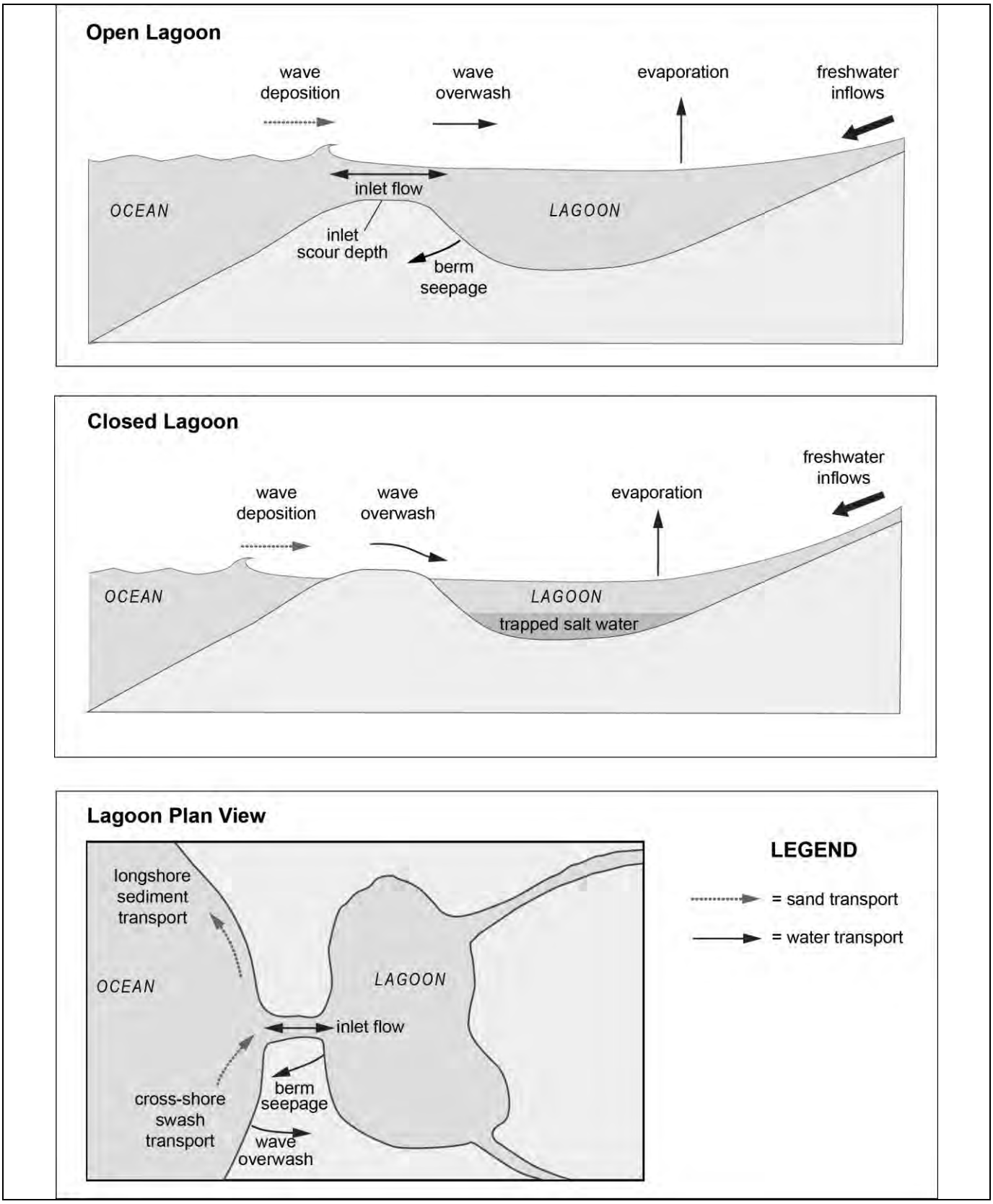
Estuary Water Level	Closed	Tide Range					
		0-1 ft	1-2 ft	2-3 ft	3-4 ft	4-5 ft	>5 ft
Observed	21%	6%	11%	24%	28%	9%	1%
QCM Predictions	20%	4%	4%	13%	27%	22%	11%

Inlet migration results are also promising, although this aspect of the QCM can be refined further as more BML camera photographs become available in time. Figure 6-6 shows that the seasonal pattern of northward migration in winter and return migration in most years in spring or summer is generally reproduced by the model. Most importantly, we found that without the migration sub-model, the QCM under-predicted mouth closures in spring. We suspect from this that migration (and its associated lengthening of the inlet) play an important role in influence closure events at the RRE.

While the QCM includes key processes affecting the inlet and Estuary water surface elevations and replicates many of the characteristics of the observed water surface elevations, the QCM does not reflect all of the system's processes. In particular, the complex dynamics of the surf zone, where breaking waves, inlet flows, and sand transport interact with one another and are locally modulated by the jetty, are not included in the QCM. Even the most detailed hydrodynamic, wave, and sediment transport models available would not fully resolve all processes and would require extensive computing resources to simulate just a few hours or days. One process that is not represented in the QCM is turbulence, and particularly its coupled role in sediment transport. Breaking waves, tidal currents, and river discharge all create turbulence around the jetty that affect the local erosion and deposition of sand, and hence the geomorphology of the inlet channel when it is adjacent to the jetty. Turbulence generated when waves and currents interact with the jetty may cause the channel to have a lower elevation and thereby reduce tidal muting and closures. There is no data to estimate how much deeper the channel might be due to its interaction with the jetty since the highly energetic turbulence through the channel make data collection difficult and dangerous.

Even with these limitations, the QCM was calibrated to match historic closure and breaching conditions, indicating that the model does capture, to first order, the net effect of the hydrodynamic and wave forces and sediment transport on the channel's geomorphology. In addition, the model does account for wave energy decreasing at the jetty, so when channel is adjacent to the jetty, the model predicts less deposition than when the channel is not at the jetty. While this is not the same process as scour, it does result in a similar tendency for the model to predict a deeper channel when the channel is near the jetty. In the alternatives, this influence is reduced since the channel is not fixed at the jetty, but may migrate further south.

6.3 Figures

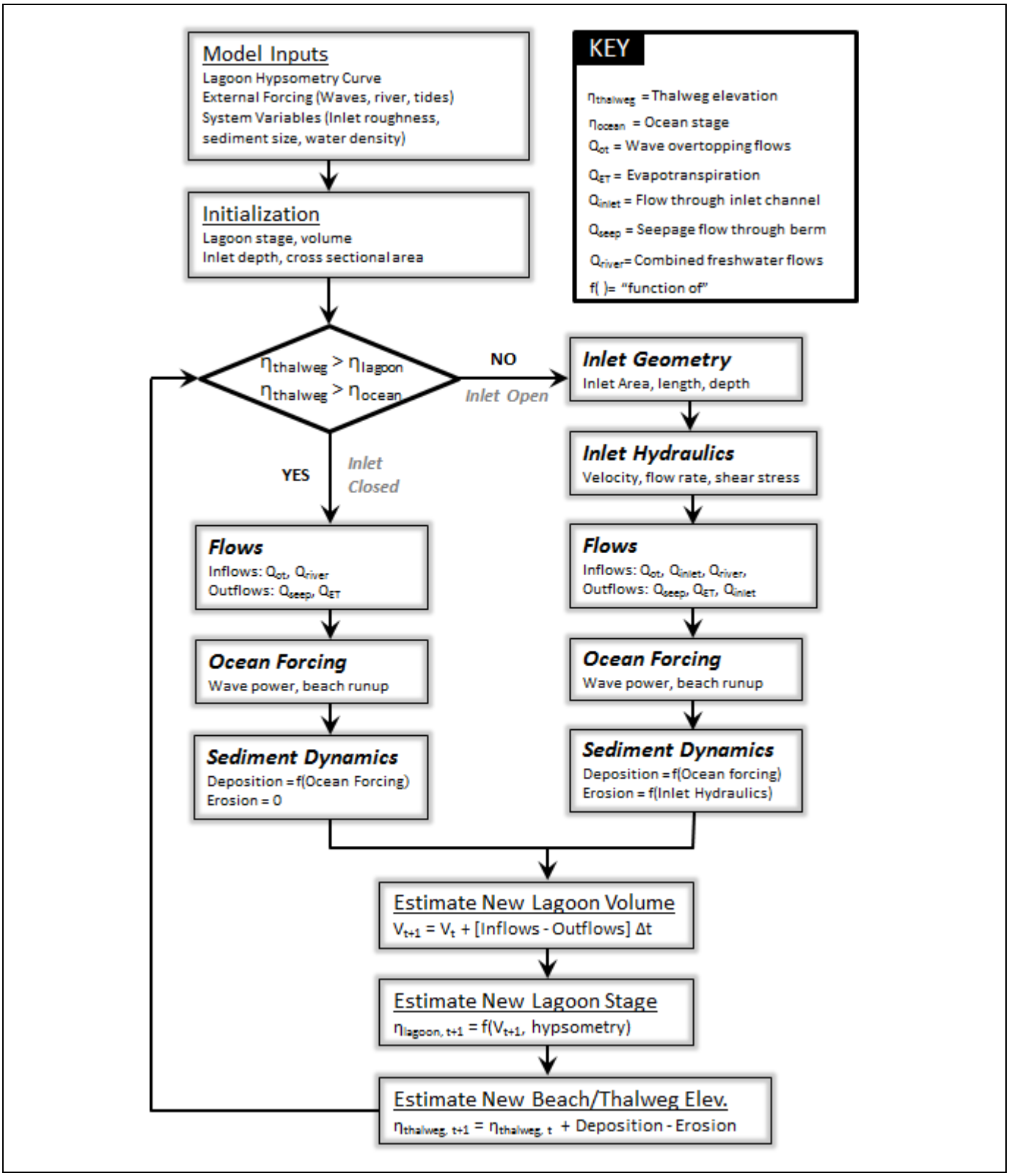


SOURCE: B. Evans illustration

Goat Rock Jetty Feasibility Study . D211669.00

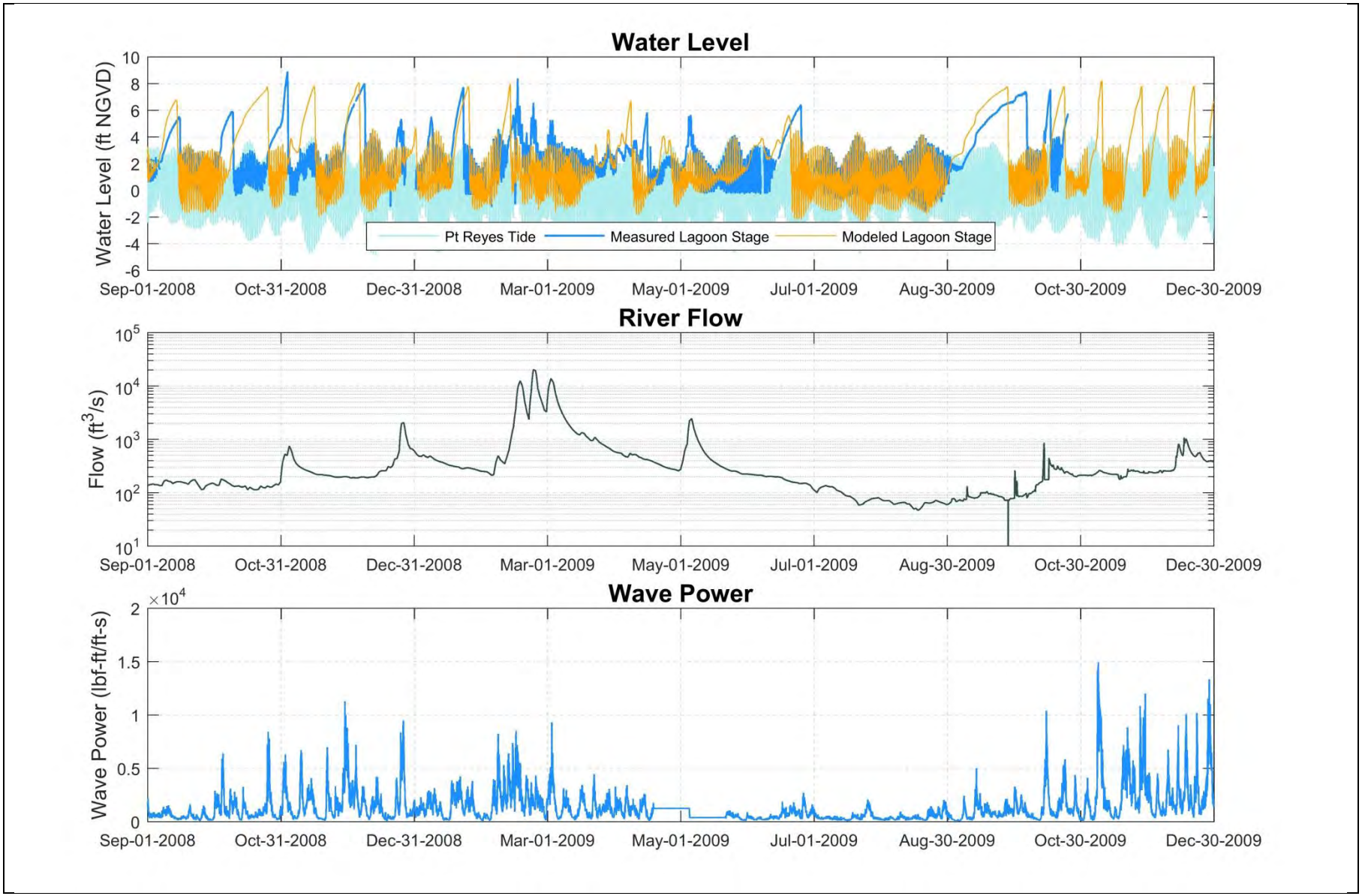
Figure 6-1

Schematic of the dominant processes affecting lagoon hydrology.



SOURCE: Behrens et al. 2015; modified from an earlier figure by Rich and Keller (2013)

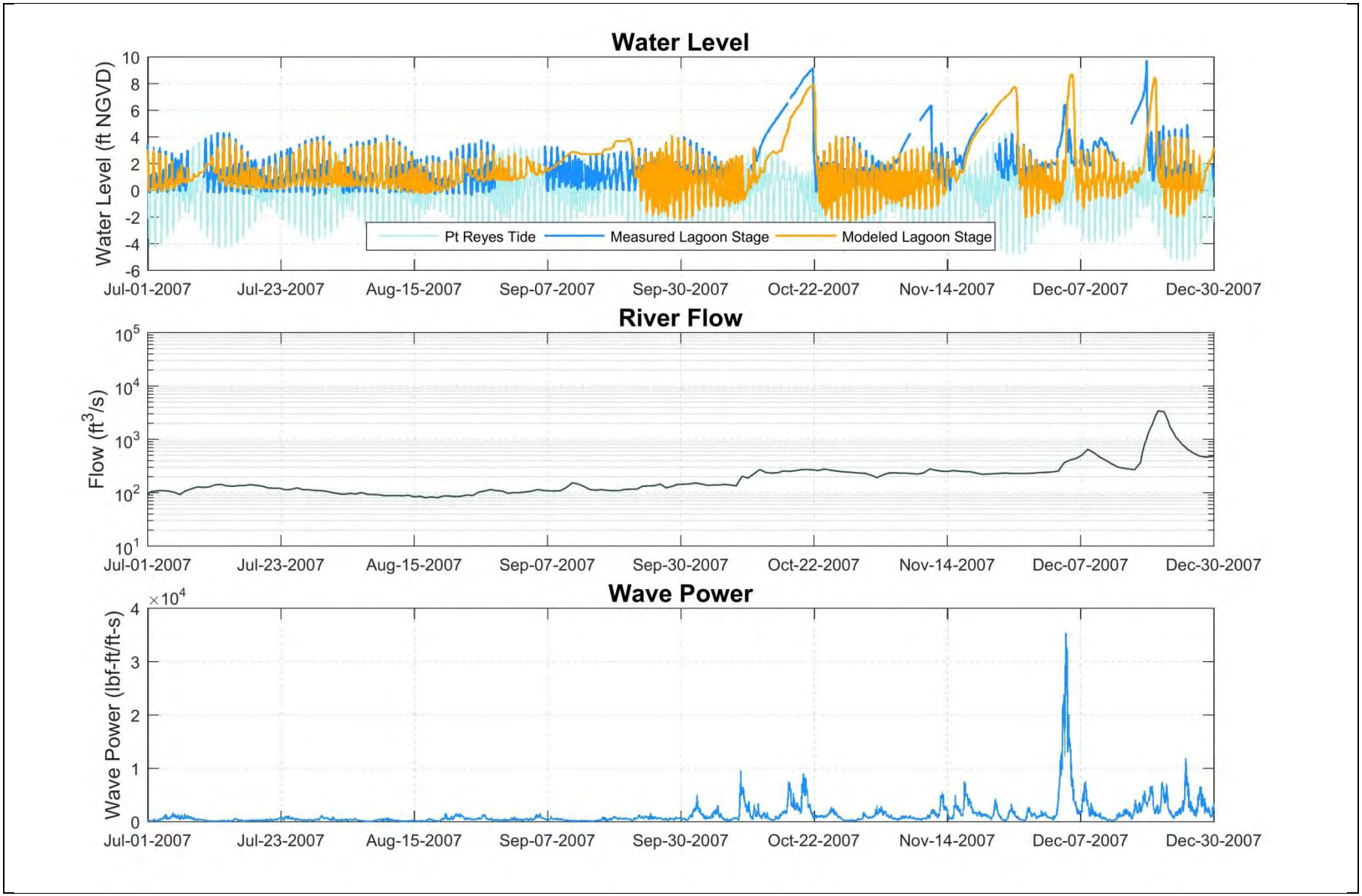
Figure 6-2
Flow chart of the lagoon quantified conceptual model.



SOURCE: River flow from USGS Guerneville station, wave power from ESA SWAN model, tides provided by SCWA at Jenner, model water levels from ESA QCM model.

Figure 6-3

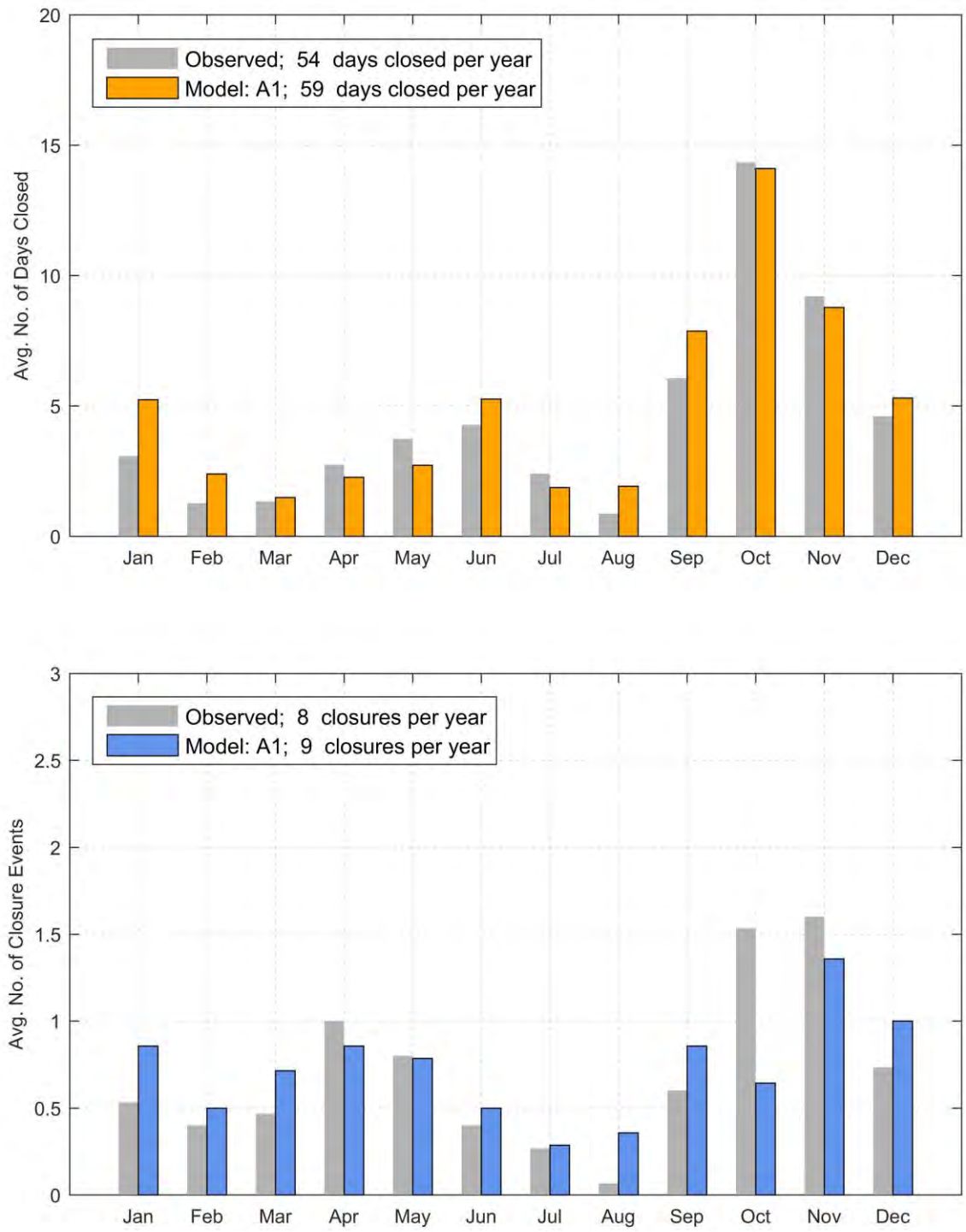
Test of QCM model accuracy in **(top)** predicting Russian River Estuary water levels, compared against **(middle)** river flow and **(bottom)** nearshore wave power for 2008-2009.



SOURCE: River flow from USGS Guerneville station, wave power from ESA SWAN model, tides provided by SCWA at Jenner, model water levels from ESA QCM model.

Figure 6-4

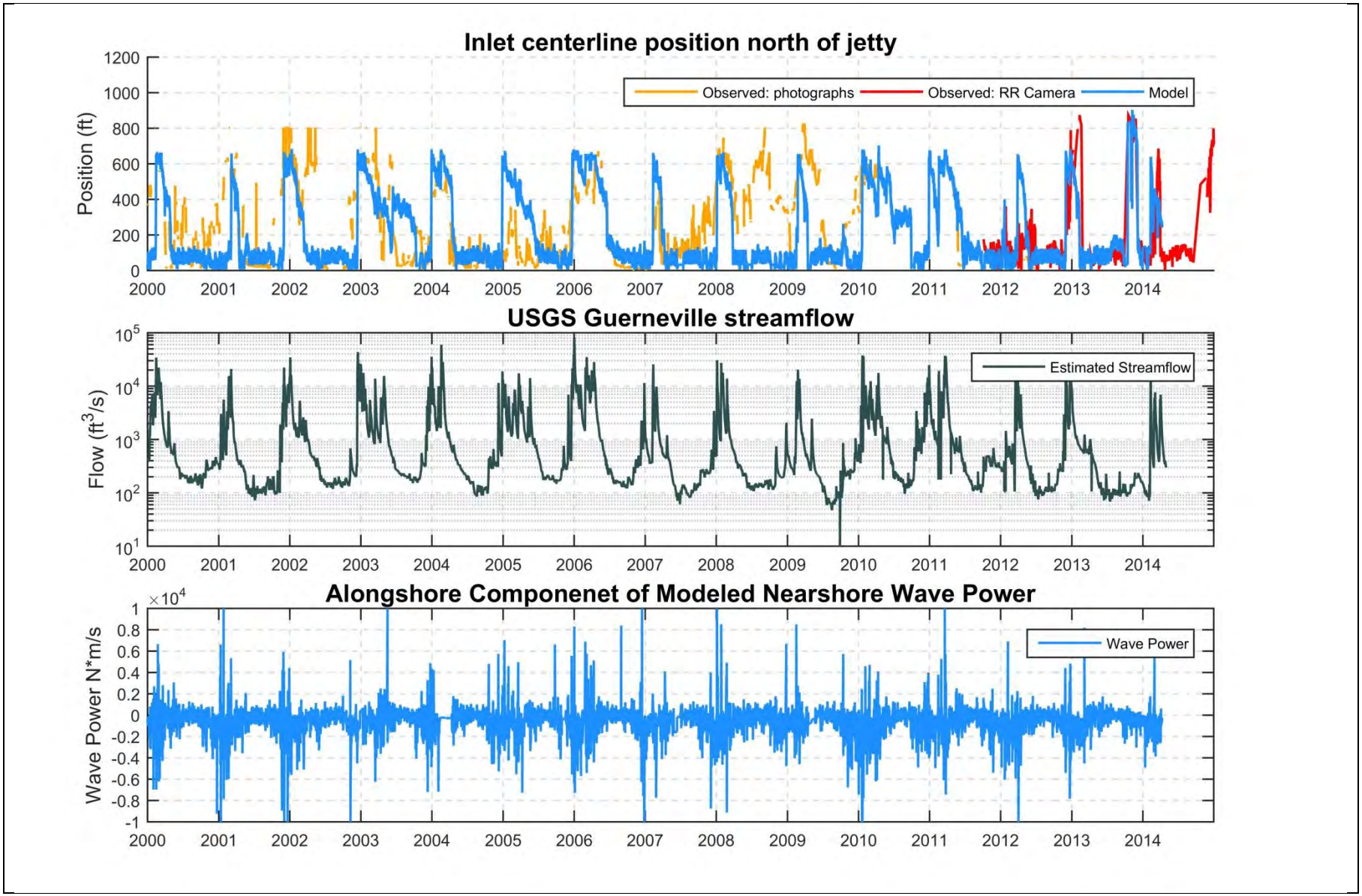
Test of QCM model accuracy in **(top)** predicting Russian River Estuary water levels, compared against **(middle)** river flow and **(bottom)** nearshore wave power for part of 2007.



SOURCE: ESA lagoon QCM model, SCWA closure data

Figure 6-5

Test of lagoon quantified conceptual model accuracy: comparison of **(top)** predicted number of days closed per month and **(bottom)** number of closure events.



SOURCE: River flow from USGS Guerneville station, wave power from ESA SWAN model, migration data provided by Behrens (2012).

Figure 6-6

Test of QCM migration model accuracy from 2000 to 2014, compared against **(middle)** freshwater runoff and **(bottom)** alongshore component of wave power vector.

7 FLOOD RISK

In this section, we assess the flood risk of low-lying areas adjacent to the Estuary by predicting extreme values of water surface elevations resulting from coastal and fluvial processes. This section builds on earlier sections documenting the jetty structure, wave assessments, and beach morphology, and uses historic data to estimate a range of possible flood scenarios.

In contrast to the previous topics, flood risk is not a process which affects lagoon water surface elevations or barrier beach formation. Instead, flood risk is affected by the jetty and the jetty's effect on lagoon water surface elevation and beach morphology. An assessment of flood risk is included in this study because it is a potential significant impact of jetty modification. Jetty modification designs would strive to not increase the existing level of flood risk. Jetty modifications may affect these conditions by changing the frequency and duration of elevated lagoon water surface elevations or by altering the beach morphology, and therefore, the propagation of ocean waves into the lagoon. However, if jetty modifications "appreciably increase flood risk" (Biological Opinion, p. 251), they may not be implemented. The effect on wave propagation and coastal flood risk is pertinent to assessing potential impacts of jetty removal.

Properties along the Russian River Estuary shoreline are at risk from flooding events within the Estuary. Flood events are associated with heightened water surface elevations along the inner Estuary shoreline, potentially causing inundation of low-lying properties or exposing them to wave action. Once lagoon water surface elevations exceed 9 ft NGVD, structures on adjacent properties can be impacted (ESA, 2010). Water surface elevations inside the Estuary are controlled by a balance between water inflows and outflows, and by the breaking and runup of waves inside the Estuary. During the wet-season (December-April) the inlet is normally open, and inflows are dominated by the river during flood events. Since the Estuary occupies a canyon-like channel, flood flow conveyance is constrained by the bathymetry and leads to higher water surface elevations in the Estuary.

During the dry-season (May-November), flooding is typically caused by inlet closures of the beach berm, rather than fluvial inputs. The Russian River inlet closes periodically in response to ocean beach-building processes overwhelming the erosive capacity of tidal or freshwater flows. When the inlet is closed, the GRSB forms a complete barrier between the Estuary and the ocean. During these periods, inflows to the Estuary, now a closed lagoon, are comprised of low, dry-season inflows from the river, and from occasional overwash of ocean waves over the beach and into the lagoon. Although these inflows are partly balanced by seepage losses through the barrier beach (Behrens, 2012), they tend to raise Estuary water surface elevations over time.

When the inlet is open, ocean waves can transmit through the inlet and into the lagoon, propagating up the Estuary channel and breaking on the shoreline. Ocean waves typically have long periods (greater than 8 seconds) and can break over the inlet channel when the depth becomes comparable to the wave height. Because of this, the depth of the inlet determines the size of waves that are transmitted into the Estuary. An outflowing current can also cause waves to shoal and break, thereby limiting wave penetration into the Estuary. When the inlet is closed, ocean waves are not able to transmit into the Estuary without first breaking on the beach shoreface and losing much of their energy. However, wind-generated waves can form inside the Estuary and break on the inner shoreline. Wind-generated waves in small water bodies such as the Russian River Estuary are limited by the length of the fetch, and are much smaller and have lower periods than ocean waves.

The Federal Emergency Management Agency (FEMA) has an existing flood assessment for the Estuary that maps the 100-year fluvial flood levels to within 1,500 feet east of GRSB. This assessment was performed in 1991 using topographic and hydrologic data available at that time. This data was used to construct a HEC-2 one-dimensional model of the Russian River. Starting from 1,500 feet east of GRSB and moving upstream, the model predicts that the 100-year water surface elevations are set by fluvial discharge alone. The Base Flood Elevations (BFEs) are between 12.5 ft and 13.4 ft NGVD near Jenner and increases for locations upstream (FEMA, 2008)[1]. Some of the low-lying property in Jenner and the lower Estuary is below the BFEs and probably should be within the 100-year flood plain, but is not mapped in the flood plain (Figure 7-1). The reason for this discrepancy is not known, but may result from old analysis and/or poor mapping resolution. In the last 1,500 ft of the Russian River across GRSB to the ocean, FEMA's BFEs are simply a linear interpolation between the BFE from the last reported model cross section (12.5 ft NGVD) and the estimated 1% annual chance still water level for the ocean (6.6 ft NGVD). Provisionally, we believe that these prior studies do not adequately represent estuarine hydraulics or shore morphology, and therefore may be low and should not be used as the sole measure of estuarine flood risk. Note that the still water level at the ocean shoreline does not consider the influence of waves. Nor does FEMA's analysis consider waves, either propagating ocean waves or locally-generated wind waves, within the Estuary.

Fluvial flood events are managed to some extent by impoundment behind Warm Springs and Coyote Valley dams (Florsheim and Goodwin, 1993), which tend to reduce peak flood flows. However, severe fluvial flood events still occur. Some of the most

[1] The 2008 FEMA FIS appears to only update the 1991 hydraulic analysis by converting from NGVD to NAVD. NGVD is used in this report to be consistent with estuary management convention.

severe floods have occurred during 1982, 1986, 1995, and 2006. These flood events tend to increase the water surface elevation in the lower Estuary (Figure 7-2 and Figure 7-3). Since the Estuary is wider than the channel at the USGS Hacienda Bridge flow gage station (USGS, 2012), approximately 17 miles upstream, the increase in depth with increased flows is smaller for the Estuary than observed upstream at Hacienda Bridge.

During the lagoon management period (May 15 – October 15), flooding is not normally driven by extreme river flow. Instead, flooding during this period is influenced by inlet closure and the subsequent filling of the lagoon behind the closed beach berm (Figure 7-2 and Figure 7-4). Once lagoon water surface elevations exceed 9 ft NGVD, structures or adjacent properties can be impacted (Goodwin and Cuffe, 1994). During the management period and any time the inlet closes, the Water Agency often artificially breaches the beach berm to prevent flooding when water surface elevations threaten to exceed 9 ft NGVD. There have been closures when the Water Agency could not artificially breach because large ocean waves caused hazardous beach conditions or because construction equipment could not access north of the groin. Approximately 70 percent of inlet breaches after 1973 have been manually induced with construction equipment (Behrens, 2012), performed by various public agencies. When coastal conditions are too hazardous for the use of construction equipment on the beach, the inlet typically breaches naturally (albeit later than desired and with a higher water surface elevation) because of erosive flows overtopping the beach, or failure of the beach to withstand the pressure gradient imposed by the super-elevated trapped water in the lagoon (Kraus et al., 2008).

7.1 Description of Flood Scenarios

Although the prior sections regarding the GRSB and the jetty have focused on the lagoon water surface elevation management period (May-October), potential impacts of jetty modification or removal may influence flood risks throughout the entire year. To reflect this, we assess flood risk using scenarios that represent both dry-season and wet-season conditions. We focus on three flooding scenarios:

- Scenario one (closed inlet scenario): A late-fall inlet closure event with beach-building processes causing the beach to impound high water surface elevations. Wind waves generated within the lagoon add to the flood stage.
- Scenario two (wet-season scenario): A winter fluvial flood event increases Estuary water surface elevations.
- Scenario three (wave-transmission scenario): Oceanic waves pass through the inlet and run-up along the Estuary shoreline.

For the closed inlet flooding scenario, the elevated beach crest would create a lagoon that can support flood water surface elevations. Inside the Estuary, the water surface elevation is assumed to reach the height of the beach crest; any higher and the water would overtop the beach, limiting further increases in water surface elevations and

scouring a new, lower channel. In addition to the lagoon water surface elevation, waves generated within the Estuary from storm winds can add to the total water level by running up the Estuary shoreline. Wind waves were predicted by extrapolating extreme events from a time series of wind data at the Bodega Marine Laboratory (BML, 2012), as discussed below. The beach crest height is the main determinant of flooding conditions for this scenario, and a range of heights were estimated from the existing sources of data. We assume that breaching (inlet re-opening by erosion of the beach) would only occur if the Estuary water level were higher than the beach crest, although Kraus et al. (2008) have shown that breaching may occur when the water surface elevation is below the crest as a result of the pressure gradient destabilizing the beach.

For the wet-season (fluvial) flood scenario, the flood stage in the lagoon is estimated using the reported 10-, 50-, and 100-year recurrence interval floods reported by FEMA (2008) and comparing these against representative flood stages measured after 1999 by the Water Agency at Jenner. Although the inlet is open in this scenario, wave transmission through the inlet into the lagoon is not considered. This is because extreme floods are associated with rapid currents, which would have the effect of forcing waves to break before entering the Estuary or would negate their upstream propagation.

The third scenario (wave transmission) considers flood elevations that would result from oceanic waves breaking along the inner Estuary shoreline. Waves can propagate through channels and for long distances if the currents are slack or are traveling in the same direction (e.g. during flood tide). Hence, when we treat the inlet as being open and consider wave propagation, we assume that the Russian River is not flowing with flood discharges. The stage in the Estuary is taken as the sum of: (1) a typical river water surface elevation during the winter and (2) the runup of depth-limited ocean waves that transmit into the Estuary through the open inlet.

For all of the above scenarios, we approximate the total flood elevation by summing the individual components. This is a conservative approach, in that estimated water surface elevations may be higher than actual water surface elevations, because this approach does not consider the joint probability of the contributing events. While the joint probabilities of some events (e.g. extreme winds and extreme waves) have been studied in the past, others (e.g. beach crest elevation and extreme winds) are less clear. Joint probabilities may be assessed in the future as part of a continuation of this analysis.

7.2 Estimating Flood Components

This section explains the analyses used to quantify the components of the flood scenarios described above.

7.2.1 Flood Stage During Closed Inlet Conditions

When the inlet is closed by a continuous spit along GRSB, peak water surface elevations in the Estuary (Figure 7-4) are likely controlled by the low point in crest elevation of the beach. Considering this, we use two sources to estimate flood stages at the Jenner Gage:

- Beach topography measured from ground surveys and coastal LiDAR, and
- An extreme value analysis of stage time series at the Jenner Gage from 1996 to 2015.

Most of the monthly Water Agency topographic surveys, started in 2009, focus on the region of the spit between the groin and the northern headland. This is where the inlet typically resides, and is normally where the beach crest is lowest. Since the lowest elevation along the beach crest is the controlling elevation for Estuary water surface elevations (i.e. Estuary water surface elevation cannot surpass this level without breaching), we used this to estimate potential flood stages during closure in the Estuary. Crest profiles from all Water Agency surveys were collated, and the highest measured elevation was traced along the beach crest from all surveys. The lowest point on this aggregate highest beach profile was then taken as a reference point for the other data. This was approximately 14.5 ft NGVD.

The LiDAR topographic data collected in September 2002 and September 2010 (CCC, 2012) were used to find the elevation of the beach at points along the spit where overwash into the lagoon occurs, and also at the points where the beach intersects with the toe of the back barrier cliffs or dunes. These beach elevations are related to the total water level determined by tides and wave action (Revell et al., 2011), and are indicative of the height that beach berm could reach if the inlet were closed for a long period of time.

Topographic data from the September 2002 LiDAR indicate that the elevations of the beach where it intersects the toe of the back barrier cliffs and dunes was lowest north of the groin (~10-17 ft NGVD) and highest (21-24 ft NGVD) between the Beach Parking Lot (1,200 ft south of the groin) and Goat Rock. Elevations on the spit at areas experiencing overwash ranged from a minimum of 12.5 ft (adjacent to the groin) to a maximum of 17.5 ft near the Beach Parking Lot. September 2010 LiDAR showed a much more homogeneous beach, with an average back barrier toe elevation of 15.5 ft NGVD, and a standard deviation of roughly 1 ft. The elevations at the overwash points along the spit were relatively unchanged from the 2002 LiDAR survey.

As another means of approximating flood levels during the dry-season scenario, peak annual Estuary water surface elevations measured at the Jenner Gage (Figure 7-2) were taken from closure events between 1996 and 2015. Peak water surface elevation occurred at the end of closure events (i.e. immediately prior to breaching). We only

included water surface elevations taken from closure events that resulted in a natural breach, since water surface elevations during manual breach events may not be reflective of the actual beach crest height. We used statistical methods to obtain expected water surface elevations during peak events at certain recurrence intervals. Peak annual water surface elevations from closures ending in natural breaches were ranked by magnitude, and the sample mean and standard deviation were obtained. Extreme value analyses assume that the set of peak annual values are distributed according to a well-known shape. Here, we chose the Gumbel distribution, a widely-used approach for extreme value analyses. These methods provide more accurate results when many years of data are available. The limited size (20 years) of the Estuary water surface elevation data means that predicted values for less frequent recurrence intervals are associated with some uncertainty.

Dry-season flood stage estimates from the topographic and water surface elevation methods described above are summarized in Table 7-1. The two methods result in an overlapping range of beach crest heights and hence, overlapping estimates of flood stages due to closed beach berm conditions.

On December 12, 2015, when the inlet was closed, the Estuary water surface elevation reached its highest observed level of approximately 12.2 ft NGVD. Because of heavy surf overtopping the beach, the Water Agency could not implement artificial breaching in the days preceding this peak. Photographs indicate that the Estuary self-breached on December 12th, flowing over the low point in the berm crest just north of the jetty.

Table 7-1. Potential flood stages due to closed beach berm conditions during the dry-season.

	Beach crest height[1] (ft NGVD)
From LiDAR and Water Agency topographic surveys	
Aggregate of Water Agency beach crest surveys	14.5
Back barrier toe heights[2]	15.5-17.5
From Jenner water surface elevations	
Maximum observed stage[3]	12.2
10-year stage (Gumbel Distribution)[4]	11.2
50-year stage (Gumbel Distribution)[4]	13.1
100-year stage (Gumbel Distribution)[4]	13.9

[1]Impounded water in Estuary assumed to be at this height.

[2]Range reflects average back barrier toe heights from 2002 and 2010 LiDAR surveys (CCC, 2012).

[3]Preliminary estimate from December 12, 2015 derived from observed high water, topographic surveys, and LiDAR.

[4]Stage during closure conditions, estimated from 1996-2015 stage data in the Estuary. Pre-breach water surface elevations are available prior to 1999, but not continuous water surface elevation time series.

7.2.2 Flood Stage During Fluvial Flood Conditions

The 100-year flood stage in the Estuary resulting from river flow was obtained from two sources: (1) existing information from FEMA (2008) which relied on a numerical flood model of the Russian River, and (2) existing flood stage data from the Water Agency gage at the Jenner Visitors Center (Table 7-2).

Table 7-2. Predicted and observed flood stages near Jenner Visitors Center.

	River Flow Rate (ft^3/s)	Estuary Stage (ft NGVD)
FEMA (2008) Predictions		
10-year flood	76,000	9.25 ± 0.25[1]
50-year flood	102,000	11.25 ± 0.25[1]
100-year flood	114,000	13.4
USGS (Flow Rate) and Water Agency (Stage) Observations (after 1999)		
February 18, 2004	63,000	8.5
January 1, 2006	84,000	9.8

[1]Uncertainty in estimate due to interpretation from a graph

The FEMA model obtained flood stages (Figure 7-1) by conveying the expected 100-year discharge through a one-dimensional numerical domain meant to approximate the system. It is unclear to what extent the model accounted for the presence of GRSB and ocean water surface elevation. However, the reported model output appears to end 1,500 ft upstream of the beach, which suggests that the beach may not have been included or that it was included, but the riverine flood conditions were not thought to govern 1% water surface elevations in the coastal region.

For comparison, the Water Agency's water surface elevation gage at the Jenner Visitors Center was used as a reference for fluvial flood elevations (Figure 7-3). An extreme value analysis was not used with the gage data for this scenario because sensor malfunctions occurred during several years. This reduced the sample size to the point that extrapolation to 50- or 100-year events would be associated with high uncertainty. Despite this, flood events in 2004 and 2006 provide a useful reference. In particular, the January 1, 2006, flood (Table 7-1) was between a 10- and 50-year event, and provided a flood stage (9.8 ft NGVD) that fell between the predicted FEMA (2008) values for the 10- and 50-year floods.

Neither data source provides a definitive flood stage in the Estuary. The FEMA (2008) flood stages use a simplified bathymetry to model flood flows and do not account for coastal influences (storm surge, tidal variations). The time series of measured water surface elevations at the Jenner Visitors Center includes the effects of tidal variability and the real bathymetry of the Estuary, but gaps in the data and its short duration limit the extent of information that can be used from it.

7.2.3 Wave Transmission and Runup

The largest waves that reach GRSB are long-period swell waves generated in the open ocean, hundreds of miles from the Russian River. At GRSB, most of these break directly on the shore, but some may pass through the inlet and travel into the Estuary, breaking and running up on the Estuary shoreline. Waves break during the transition over shallow zones, a process that can be approximated by comparing the wave height to the local depth (SPM, 1984). This is important because the largest waves that pass through the inlet are those that do not break or that are partially broken to their depth-limited height when passing over the inlet thalweg. Although waves are known to propagate into estuaries at times, it is unlikely for this to occur during peak flood events, as discussed above.

The height of waves transmitted through the inlet was assumed to be 78% of the water depth, since waves will reduce in height to this fraction of the water depth by breaking (USACE, 2005). The sand bars offshore are typically not lower than -2 ft MLLW (about -5 ft NGVD). Hence, wave heights are likely to be no more than 6 feet when propagating through the inlet and into the Estuary. This estimate is based on limited on-site observations, given the lack of bathymetry data seaward of the inlet. The height of penetrating waves could be reduced by approximately 50 percent as a result of dissipation and lateral energy spreading inside the Estuary (USACE, 2005). The worst case condition for wave penetration is expected to be a set of large waves riding on a surge of dynamic wave setup during flooding, high tide and low river flow. Once inside the inlet, wave refraction, diffraction, and interactions with the flood currents would tend to either reduce wave heights or cause them to break before propagating upstream (USACE, 2005). Resolving these influences requires a numerical modeling approach that combines wave propagation and fluvial flooding, which is beyond the scope of the present analysis. By neglecting these processes, the analysis in this study probably overstates the wave heights of ocean waves transmitted into the Estuary.

Wave runup along the Estuary shorelines was approximated using Hunt's method (Hunt, 1959), which relates the wave runup height to the shore slope, and the wave height and length. The shoreline slope inside the Estuary is variable, with much of the shoreline adjacent to Jenner defined by steep slopes where riprap armoring and development are present. For the present analysis, we assumed that the runup was no greater than three times the height of the waves (USACE, 2005). Our methods are described in more detail in Appendix B.

We examine the potential runup elevations for a 6-ft wave propagating into the lagoon, based on the approximate limiting depth described above. We assume the wave has a period of 12 s, which is typical for GRSB (Behrens, 2012). We estimate the wave length in the Estuary using a representative depth of 15 ft (EDS, 2009) and the wave length

approximation provided by Fenton and McKee (1990). We found approximate wave runup values after Hunt (1959) using a range of shore slopes (Table 7-3).

Table 7-3. Wave runup for a 6-ft wave with a period of 12 s entering the Estuary through the inlet. Refraction and diffraction are not included.

Shoreline Slope[1]	Runup/Wave Height[2]	Max Runup Height (ft)	Estuary Stage at Shoreline (ft NGVD)[3]
1:2	3.0	18.0	25.1
1:3	2.2	13.0	20.1
1:5	1.3	7.8	14.9
1:10	0.7	3.9	11.0

1Units are given as (vertical length):(horizontal length).

[2]Estimated from Hunt (1959).

[3]Estimated as MHHW + Max runup height. This is the elevation that runup would reach at the shoreline, not the average elevation of the Estuary.

These runup values and total water levels are expected to be conservative (i.e. higher than actual values), since we have not considered diffraction and refraction of the wave field after it passes through the inlet. These processes would have the effect of reducing the wave energy (and thus the height). These processes could be resolved with a numerical model, which is beyond the scope of the present analysis. Also, Hunt's method does not explicitly account for different shoreline surfaces, such as riprap armoring, which would have the effect of limiting wave runup.

7.2.4 Wind Waves

Wind waves are generated by winds blowing across exposed, open water fetches of the Estuary. Wind wave growth is proportional to several parameters: the speed and direction the wind is blowing; the duration of time the wind of a given velocity is blowing; the length of area over which the wind is blowing (or the fetch); and the depth of water across the fetch. Wind waves in the Estuary are "fetch-limited" because the twisting shape of the Estuary limits the length of open water in any given direction.

Hourly-averaged wind direction and speed measurements were collected from the meteorological station at the Bodega Marine Laboratory (BML), which is located 12 miles south of the Estuary (Longitude 123°04'18", Latitude 38°19'05"). The time series spans 24 years from January 1, 1988, to December 31, 2011. Wave prediction was performed using the wind time series, a representative fetch for the outer Estuary, and wind wave generation equations from the Coastal Engineering Manual (Resio et al., 2006). The prediction assumes that the wind blows steadily in a constant direction for a sufficient amount of time to achieve steady-state fetch limited values and wave dissipation process like wave breaking and bottom friction do not occur or are considered negligible for this case.

An extreme value analysis was performed for the 24 years of the predicted wind wave and the estimated run up from 1988 to 2011 was conducted. As with the earlier stage estimates, maximum values for each water year (October 1 – September 30) were fit to the Gumbell Distribution to provide information about the expected wave heights at different recurrence intervals (Table 7-4). These were then used to calculate runup on the shoreline using the same range of shoreline slopes as in Table 7-4. The runup is smaller for wind waves compared to ocean waves that transmit through the inlet, ranging from 1.52 to 3.73 ft for 100-year wind conditions.

Table 7-4. Wave height and runup for wind waves generated during 10-, 50-, and 100-year wind events in the Estuary.

Return Period (yrs)[1]	Max Wind Wave Height (ft)	Runup (ft): slope = 1:2	Runup (ft): slope = 1:3	Runup (ft): slope = 1:5	Runup (ft): slope = 1:10
10	2.6	2.9	2.2	1.6	1.2
50	3.1	3.5	2.6	1.9	1.4
100	3.4	3.7	2.8	2.1	1.5

[1]Estimated from Gumbel distribution.

7.3 Results

Flood stage estimates vary considerably between each of the three scenarios described above (Table 7-5). Due to the data limitations, values reported here should be considered approximate, and are intended (when possible) to give a range of expected values under present conditions, with the jetty present on GRSB.

The first and second scenarios examined the difference in expected flood stages during periods of inlet closure and periods of fluvial flooding, respectively. Although the information available for assessing the fluvial case was limited, our analysis suggests that flood stages may be higher for the former. This is contingent upon wave conditions preventing Water Agency staff from performing a breach during a period when beach building from waves is rapid and the Estuary fills in rapidly from trapped river inflows. The flood stage during large fluvial floods is likely limited because the lateral width of the Estuary becomes large near the mouth. Upstream, flood flows result in a rapid rise owing to a more confined channel (Florsheim and Goodwin, 1993). At the mouth, the inlet width usually spans the distance between the groin and the northern headland (width ~ 700 ft), so flood flows are conveyed rapidly to the ocean. Although inflows are typically low during inlet closure, the height of the beach allows for a higher stage to be achieved simply because trapped inflows can only leave via evaporation and porous exchange flows between the Estuary and the ocean or local aquifer.

Table 7-5. Summary of potential flood stages in the Estuary resulting from the three flood scenarios described above.

Scenario	Estimation Method	Estuary Flood Stage at Shoreline (ft NGVD)
Scenario 1 (Closed inlet flood)	• Gumbel Dist. of Jenner peak stages during closures[1]	13.9
	• Beach topographic data used for crest height reference	14.5-17.5
	• Runup estimated for extreme wind waves	1.5-3.7
	Total:	16.0-21.2
Scenario 2 (Wet-season flood)	FEMA (2008) BFE	13.4
Scenario 3 (Wave transmission and runup)	• Assumed MHHW tidal level in the Estuary	7.1
	• Estimated depth-limited wave transmission cutoff height	6.0
	• Runup on Estuary shorelines using Hunt (1959)	3.9-18.0
	Total[2]:	11.0-25.1

[1]Only peak stages prior to natural breach events were used.

[2]Estimated as MHHW + runup.

Flooding due to wave transmission through the inlet and runup on the shoreline is associated with a high amount of uncertainty (Table 7-5), owing to the fact that the shoreline slope varies widely throughout the Estuary, and the wave exposure is greatest near the mouth. This is reflected in a range of expected runup heights that is larger than the ranges of expected flood stage approximated for the first two scenarios.

Since the bathymetry has not been measured during flooding, the limiting depth was approximated using the understanding that river floods on coasts with high wave energy often lead to the formation of a subtidal ebb shoal which provides shallower depths then in the inlet channel (see wave breaking in upper panel of Figure 7-2). This approximation could be improved with bathymetry data offshore of the inlet. Additionally, numerical modeling could provide a better assessment of wave runup inside the Estuary by making it possible to assess wave diffraction, energy dissipation, and spreading over the complex bathymetry. Despite these shortcomings, wave runup can be assumed to be highest for parts of the shoreline closest to the current inlet location (north of the groin), and is expected to decrease with distance upstream.

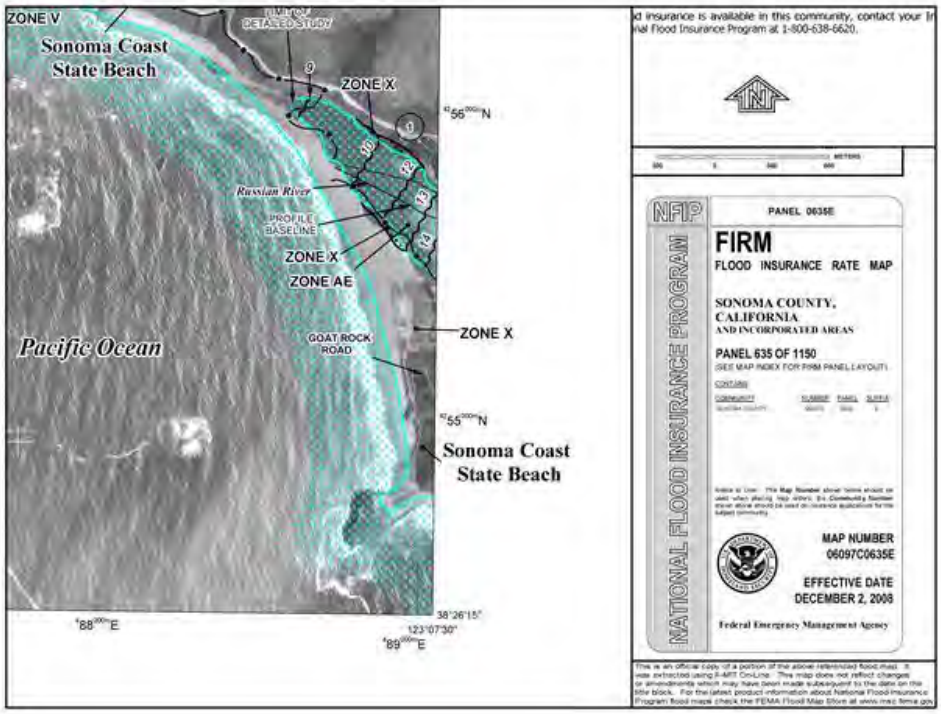
The presence of the jetty in GRSB may influence the potential flood elevations in the Estuary. Based on information provided in previous sections and related literature, GRSB is influenced in the following ways by the jetty:

- The inlet position is limited to the northern third of the sand spit,
- The inlet width is constrained to approximately 700 ft,
- The sand spit south of the groin is likely higher and wider than it would be under natural conditions (owing to the presence of the embedded jetty access elements).

The first two points have implications for the second and third flooding scenarios. Since the inlet is constrained to the northern end of the spit, any waves that transmit into the Estuary must undergo a high amount of refraction before arriving at most of the Estuary shoreline (see upper panel of Figure 7-2), which is expected to have the effect of reducing wave heights before they reach much of the shoreline. In addition to this, the inlet may have been wider than 700 ft during some peak flood events prior to jetty construction, which would have allowed for greater conveyance of flood flows to the ocean, and may have limited the fluvial flood stage in the Estuary as a result. Lastly, the final point has implications for the first flooding scenario (flooding during inlet closure). The size of the beach berm is an important factor for both overtopping and seepage losses. Independent from the jetty's permeability properties, the seepage would be greater for a narrower, more uniform beach berm (see Section 3). To some extent, seepage flows export trapped inflows (Behrens, 2012), either increasing the time for flood levels to be reached, or possibly balancing inflows, leading to a steady water surface elevation in the lagoon.

Based on this study, the existing flood risk may not be well defined by the effective FEMA maps and studies. The analysis presented herein indicates flood elevations and extents are greater than indicated by the FEMA documents. However, the analysis herein is not sufficient to replace or appeal the existing FEMA estimates, and is not intended for these purposes.

7.4 Figures



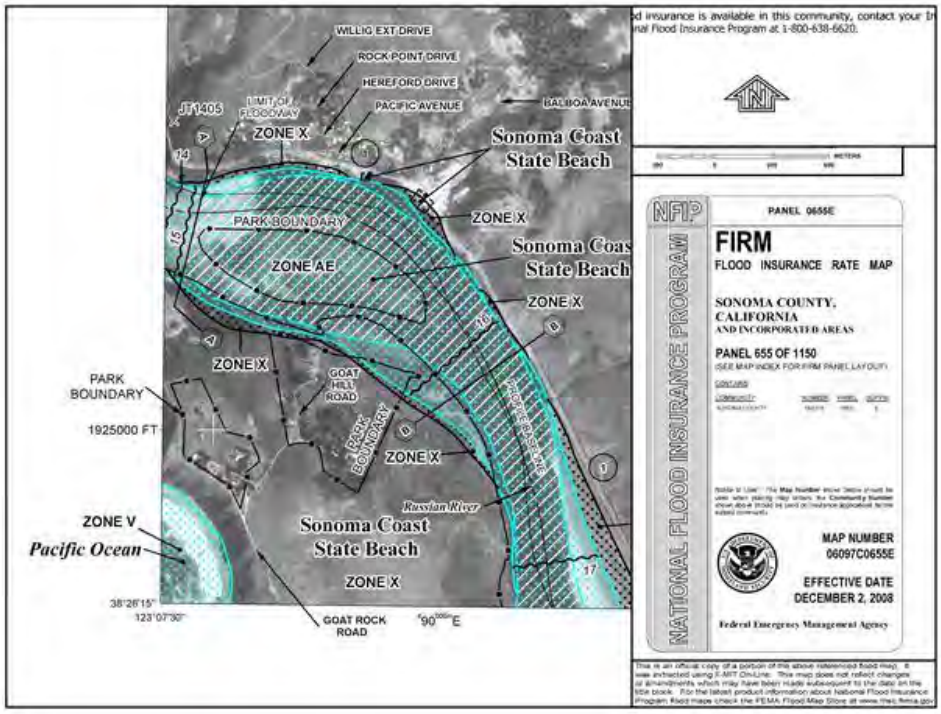


Figure 7-1

FEMA Flood Insurance Rate Map (FIRM) of Jenner, CA.

SOURCE: (top): California Dept. of Boating and Waterways; (bottom) Google Earth

NOTE: The available aerial images used in this study do not show a representative wet-season flood event.

Figure 7-2

Aerial views from representative (top) wave penetration and (bottom) dry-season events associated with flood risk.

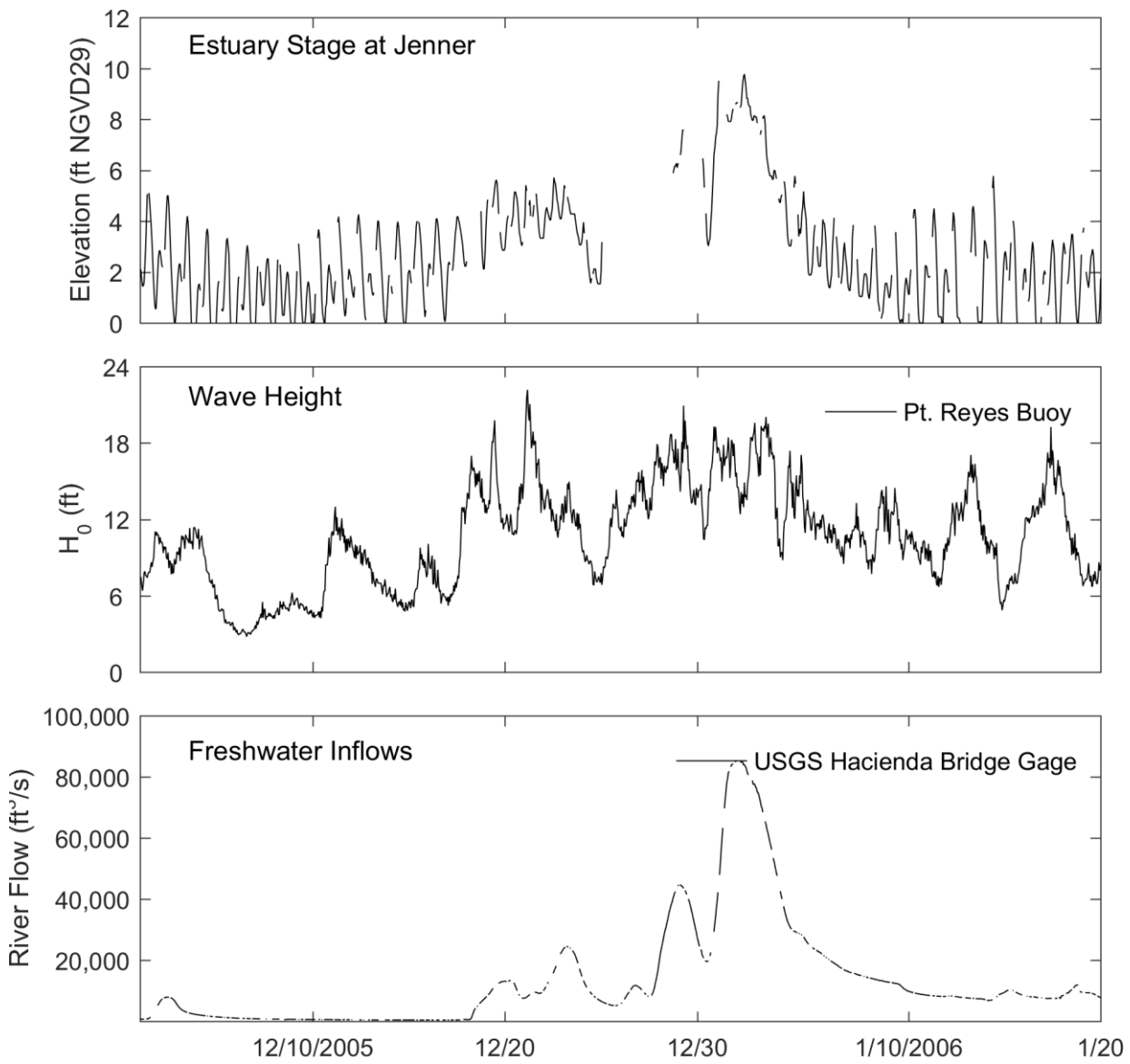


Figure 7-3

Example of a wet-season flooding event, driven by fluvial conditions.

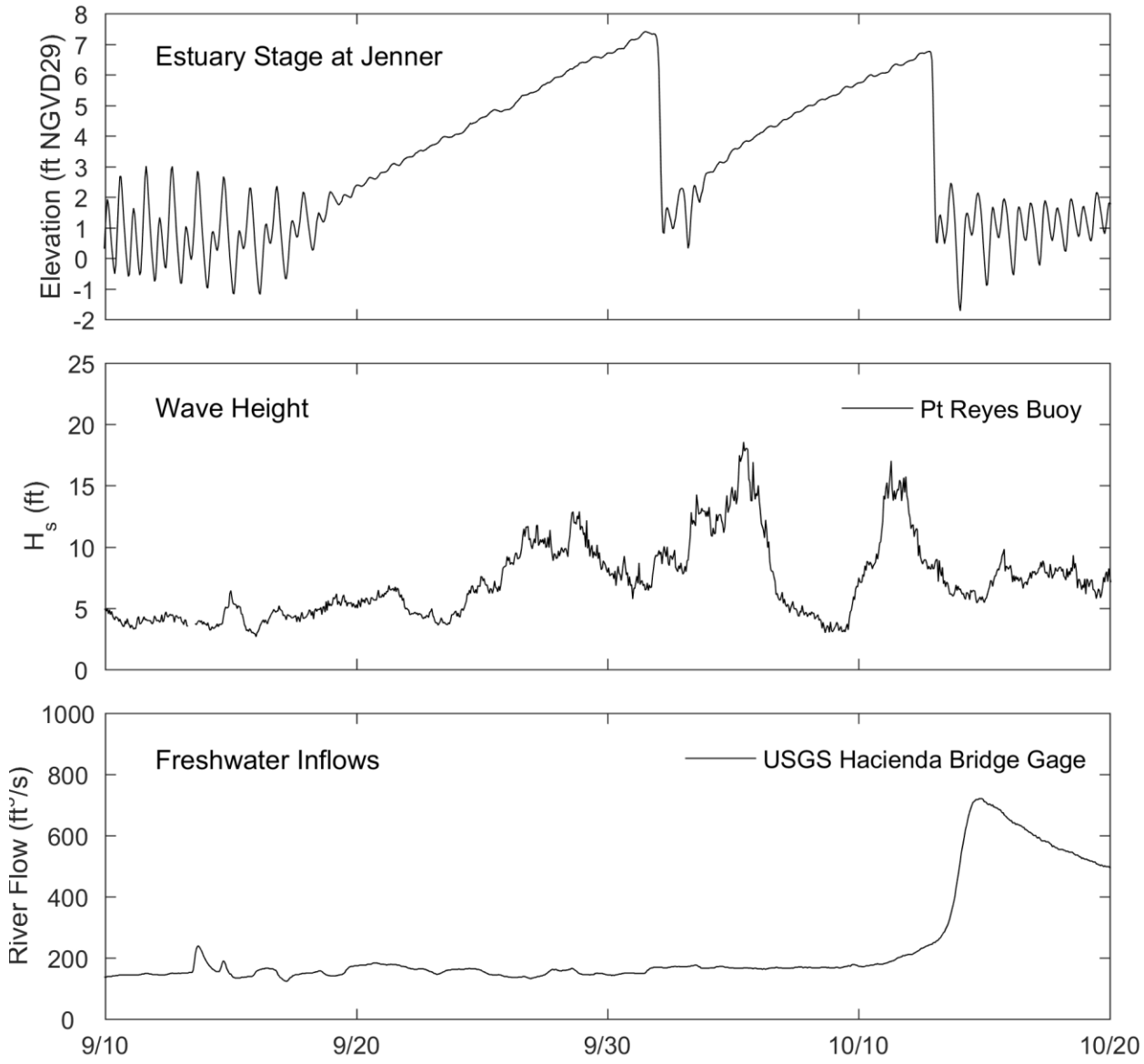


Figure 7-4
Example of dry-season flooding events in 2010, driven by beach conditions.

8 DESCRIPTION OF ALTERNATIVES

A range of alternatives to the existing jetty were developed for comparison with the no-action alternative of leaving the existing jetty unchanged. Removal or modification of parts of the existing jetty would change the way in which physical processes interact with the beach, inlet, and/or groundwater seepage. The objective of these potential alternatives is to increase the likelihood of achieving the target lagoon water surface elevations described in the Biological Opinion. The alternatives are numbered in order of increasing change to existing conditions. Figure 8-1 provides a schematic representation of the alternatives. The alternatives developed for this study include:

- Alternative 1: No action – The existing jetty would remain in its current configuration, and degrade slowly due to natural processes.
- Alternative 2: Notch groin – A notch 40 to 80 ft wide in the groin to facilitate an erosion-resistant outlet channel over the groin.
- Alternative 3: Remove groin – The concrete-capped rock groin would be demolished and removed to eliminate the portion of the jetty's influence on the surf zone. An option for implementing this alternative, Alternative 3a, would be to structurally degrade and disperse the groin, but leave the material on the beach.
- Alternative 4: Remove access elements – The construction access elements (roadway, railway, seawall, and rock fence) embedded in the beach berm would be removed to improve groundwater seepage and natural beach processes
- Alternative 5: Remove groin and access elements – The previous two alternatives would be combined to completely remove the entire jetty from the beach, eliminating its influence on the surf zone, groundwater seepage, and beach processes.

These alternatives were developed to be consistent with the Biological Opinion (NMFS, 2008) and span a range of jetty modification options. These alternatives were presented to the project's key resource management groups (NMFS, CDFW, CDPR, and the Water Agency) for discussion at a meeting held at the Water Agency on March 7, 2013. The resource managers had previously been provided with a draft of this study's existing conditions sections (ESA PWA, 2012). The resource managers were also provided a memorandum describing the alternatives (ESA PWA, 2013) and a memorandum describing the proposed alternatives evaluation (ESA PWA, 2013b). NMFS provided comments on these documents in its letter to the Water Agency dated April 24, 2014. A public presentation about the existing conditions sections was hosted by the Water Agency in Monte Rio on May 15, 2013. The Water Agency hosted a second public

meeting in Monte Rio to discuss the jetty alternatives and alternatives evaluation on June 11, 2015.

For this study, the alternatives have only been described at a conceptual level sufficient for feasibility evaluation. Additional development of alternative designs, construction methods, environmental review, and cost estimates would be required before implementation. These alternatives are evaluated for their potential to meet project objectives and possible impacts in Section 9.

8.1 Alternative 1 - No Action

Under this alternative, the jetty would be left in its current condition (Figure 8-1a). Natural processes would continue to degrade the structure over time. The jetty would continue to influence Estuary water surface elevations to the same extent as in recent years, as described above in the existing conditions sections. Since the groin would be left in place, the inlet would continue to be constrained to the northern end of the beach, and would likely continue residing in the northern lee of the jetty during dry conditions. Beach management activities, as described in the Russian River Estuary Adaptive Management Plan (ESA PWA, 2015), would continue to manage Estuary water surface elevations for salmonid habitat and flooding.

8.2 Alternative 2 - Notch Groin

Under this alternative, a portion of the groin would be removed to create a notch. The notch would serve as a water control structure that facilitates an erosion-resistant outlet channel. A likely location for the notch would be near the landward endpoint of the groin, as shown in Figure 8-1b. The notch would be 40-80 ft wide and at an elevation within the range of target water surface elevations that is above oceanic tides and below Estuary flood stage. As such, the notch is assumed to limit outlet channel scour and the associated lagoon breaching. By limiting the risk of breaching while still providing Estuary outflow, the notch could increase the likelihood of extended periods of perched lagoon water surface elevations.

An outlet channel through the notch would be implemented after a natural closure during the lagoon management period. Construction equipment would be used to remove sand in the notch, down to the remaining groin. Design of the notch would need to consider construction equipment access, and even taking this need into account, the groin may not always be accessible because of changing beach morphology, possible interference from buried rocks and groin material, and areas with saturated sand that cannot support construction equipment. Because the outlet channel would need to include the notch in its alignment, there would be less flexibility to select the channel

alignment as compared to Alternative 1. With reduced alignment flexibility, Alternative 2 would be less able to accommodate changes to beach shape, location of seals, and operator safety.

Since the notch would be underlain by the remaining large rock and concrete of the groin, flow would presumably not be able to erode the notch section of the outlet channel. The notch would have diminishing effects outside of the notch itself, and in the majority of the outlet channel that is outside of the notch, scour could still be an issue. In cases where the upstream outlet channel would enter the notch, a westward flow path down the face of the groin could form, capturing the outlet channel and diverting it away from the notch. This flow path could be blocked with a wing wall similar to the entrance of culverts, but this would entail adding a protruding structure to the jetty that would likely raise concerns about public safety, aesthetics, and beach management. Where a channel would exit the notch and flow onto the beach could be susceptible to scour that undercuts and potentially de-stabilizes the groin. Therefore, this alternative may include an apron of buried rock just south of the groin. Rock for this apron could come from material excavated to create the notch, material salvaged from other locations on the beach, or be imported.

This description of the notch alternative assumes that the groin has no significant structure deficiencies that complicate the construction of a notch. However, the structural condition of the groin to serve as an erosion-resistant grade control has not been fully assessed, either for the groin's current condition or its condition with degradation over time.

Although the landward portion of outlet channel would be fixed at the notch, the seaward portion of the outlet channel would occupy a portion of GRSB south of the groin. The channel could migrate southward for a limited distance. South of the groin, the outlet channel would be exposed to more wave energy than it currently faces north of the groin. This wave energy could push sand into the outlet channel in excess of the outlet channel's scour potential, and thereby close the channel.

While routing the outlet channel over the notch may provide benefits to management period objectives, the size of the notch would not be sufficient to convey riverine discharge during the wet season. The notch also may not be favorable for fish passage. Therefore, if the outlet channel is flowing through the notch at the end of the management period, additional intervention by the Water Agency would likely be required. The outlet channel would need to be filled with beach sand and a new inlet excavated on the north of the groin.

8.3 Alternative 3 - Remove Groin

Under this alternative, the rock and concrete groin would be removed from the beach (Figure 8-1c). This alternative would eliminate the groin's attenuation of wave energy that reaches the beach just north of the groin. The inlet could migrate south of the groin, but would be limited in its migration extent by the remaining access elements.

Beach surveys and photographs of self-breaching indicate that the low point in the beach berm is often just north of the groin. This low point limits the elevation of lagoon water surface elevations because overtopping of this low point initiates breaching. The low point is apparent in topographic data from LiDAR and from Water Agency surveys. When the inlet closes just north of the groin, the beach in this location often accretes more slowly than elsewhere, probably due to the jetty reducing the beach-building wave energy that reaches the beach. This leaves a low point along the beach. Removing the groin would reduce the wave blocking, and would likely allow the beach to reach a higher, more uniform height. This could increase the duration of closure events by raising the water surface elevation needed for beach overtopping during closure events. It could also encourage closure events by reducing wave sheltering that the groin may provide to the inlet. Without wave sheltering, more sand would be transported into the inlet, increasing the chance of closure.

Removing the groin would also allow the inlet to migrate farther south along GRSB. At present, the groin provides a physical barrier to migration. The groin's removal would likely allow the seaward end of the inlet to migrate along the beach face. However, the degree to which the remaining access elements might impede the migration of the landward end of the inlet is unknown. The wave-driven processes that encourage migration (sediment deposition at the lateral edges of the inlet) and fluvial driven processes that encourage migration (erosion of the downdrift lateral edge of the inlet) tend to act near the ocean boundary of the inlet, so the access elements within the beach core may have limited influence on migration along the beach face.

Since the access elements were constructed from less material than the groin, and were not reinforced with concrete, they could be susceptible to erosion caused by inlet migration if the groin is removed. The access elements are thought to be composed primarily of compacted fill, timber, and boulders smaller than those used in the groin (see Section 2 above). The inlet could degrade these elements by erosion during fluvial flood events, or by exposing them to wave attack, which was known to disperse the larger boulders used in the original construction of the groin.

For purposes of this feasibility study, removal of the all the groin material is assumed. The actual subsurface extent of the groin is not well known since limited design documents remain and most of the groin is below the surface of the beach. Removing the entire groin, particularly the lower portion which may have subsided and been

dispersed over time, could be challenging, particularly since the work area is the surf zone. The extent to which lower portions of the groin may influence Estuary water surface elevations is not known. Partial removal of the groin may be considered as an option to full removal. For example, removing the groin to just below the surface of the beach would limit the groin's reduction of wave power reaching the beach and the associated beach building. However, the remaining subsurface groin would likely impede southward inlet migration. The remaining groin could also be an impediment to construction equipment access during beach management activities. Removal of more groin from the subsurface could enable the shallower, management season inlet to flow southward over the remaining lower portion of the groin. Leaving the groin's lower portion in place could impede the southward migration of the inlet during large, winter-time fluvial discharge events when the river flows probably scour a deeper channel in unconsolidated sand. The benefits and trade-offs of partial removal have not been considered in this study and, if this alternative is considered further, would require additional consideration in later stage of planning and design.

8.3.1 Degrade In Place

Under this option for Alternative 3, construction equipment and other methods would be used to degrade the groin by demolishing and/or weakening and leaving the groin material in place on the beach (Figure 8-1c). Since construction of the jetty was terminated in the 1940s, the groin has experienced significant degradation due ocean and riverine processes. This alternative would weaken the groin to accelerate its natural degradation. This alternative may be implemented in yearly phases. Each year's additional work would build off of work and natural degradation in previous years.

Demolition and dispersal could be implemented with varying intensities. A more intensive variation might reduce the majority of the groin to some nominal size fragments and then use construction equipment to disperse these fragments on the beach. A less intensive variation might just weaken the upper portion of the groin with large fractures and then leave further degradation and dispersal to natural processes. Such weakening could be achieved by fracturing the concrete cap of the groin with non-explosive rock or concrete breaking techniques commonly used in the quarrying and construction industries, such as expansive grout. In any case, the demolished groin materials would be left in place on the beach.

This alternative would have the potential impact of reducing the groin's effect on natural processes that shape the beach and on berm seepage from the lagoon to the ocean. However, the dispersed groin material would likely continue to influence the beach, inlet, and Estuary. Wave energy and large fluvial events may either bury the demolished materials in place or transport the materials off-shore. The remaining groin could also be an impediment to construction equipment access during beach management activities and to public access. Since most of the degradation would be

focused on the exposed portions of the groin, the remaining buried portions of the groin and the dispersed materials may provide some grade control for the inlet. However, unlike Alternative 2, the elevation of the grade control is presumed to be within the ocean tide range, and so would not necessarily facilitate perched Estuary water surface elevations. The degraded groin would also not restrict lateral inlet migration like the incised notch of Alternative 2. Eventually, beach transgression in response to sea level rise would also facilitate the dispersal of materials.

8.4 Alternative 4 - Remove Access Elements

Under this alternative, the access elements within the beach berm would be demolished and removed. This would return the berm to a more natural state. If the inlet remains north of the groin, removing the access elements would probably increase groundwater seepage, since the access elements appear to impede seepage in some sections of the berm (Section 3). Increased seepage would slow the rate at which estuarine water surface elevations rise during a closure, increasing the length of closure. However, removing the access elements would probably allow more wave overwash. Since wave overwash adds water to the Estuary, removing the access elements may cause an increase in estuarine water surface elevation via this mechanism. In addition, increased wave overwash may reduce the elevation of the beach berm. A lower berm elevation may increase the risk of the inlet forming south of the groin. If inlet formed south of the groin, the groin might inhibit, but not prevent, the inlet from returning to north of the groin. South of the groin, the inlet could increase flood risk.

8.5 Alternative 5 - Remove Full Jetty

Under this alternative, the entire jetty's groin and access elements would be removed from the beach. This alternative may alter the Estuary water surface elevations in the same ways as described for both the groin removal and access element removal alternatives. Since both the groin and access elements would be removed, southward migration of the inlet past the present-day location of the groin is more likely to occur. Extreme fluvial flood events would also be less constrained to the northern end of the beach, and could periodically erode the southern end of the berm.

As discussed for Alternative 3 (Remove Groin), this alternative presumes the entire groin is removed. Instead, this alternative could be modified to limit the extent of groin removal. The benefits and trade-offs of partial removal have not been considered in this study and, if this alternative is considered further, would require additional consideration in later stage of planning and design.

8.6 Probable Methods for Jetty Removal

This section considers the probable methods that would be used to remove part or all of the jetty (Alternatives 2 through 5). Descriptions of removal methods are provided to inform the feasibility analysis in regards to potential impacts, constraints, and probable costs. As noted above, including this description does not imply that any of these alternatives are going to be implemented. This section is organized according to the removal planning process, including the estimated quantities, construction methods, access and traffic, and schedule. Specific information about each alternative is included within each of these sections.

8.6.1 Estimated Quantities

The quantities of removed materials for each alternative would be the primary determinant of each alternative's level of effort. Other aspects of jetty removal, such as construction methods, access paths, and schedule would depend on the removal quantities. These other aspects are described in the following sections.

Since most of the jetty elements are hidden below the ground surface, only indirect estimates of the quantities of material that would be removed under each alternative can be made (Table 8-1). The limited data used for the estimates in Table 8-1 include minimal as-built records, historical photographs, and remote sensing data (NORCAL, 2015; LBNL, 2015). This data was used to approximate the amount of material in representative cross-sections and then these cross-sections were assumed homogeneous along jetty alignments taken from aerial photographs and ground surface surveys. Since the jetty is mostly underground, the actual extent of the jetty elements would likely only be known after excavation enables direct inspection. Sufficient excavation to fully inspect the jetty is probably cost effective only if the alternative was going to be removed.

Table 8-1. Estimated quantities of jetty material for removal and overburden for excavation and re-grading.

Alternative	Jetty materials, yd^3	Overburden, yd^3
Alternative 1 – No Action	0	0
Alternative 2 – Notch Jetty	510	0
Alternative 3 – Remove Groin*	47,000	30,000
Alternative 4 – Remove Access Elements	14,000	40,000
Alternative 5 – Remove Full Jetty	61,000	70,000

* Alternative 3's Demolish in Place option would not remove any jetty material and at most, only remove a nominal amount of overburden.

Because much of the jetty is located below the ground surface, a significant amount of beach sand would have to be excavated to access and remove the jetty elements. This

sand, known as overburden, would be placed in temporary stockpiles adjacent to the excavation area. After the jetty materials are excavated and removed, the overburden would be used to backfill the excavation. When overburden is excavated immediately adjacent to and within the jetty material, the combined excavated material would be sorted to remove jetty material from sand. This sorting would minimize the amount of sand that is removed from the beach along with the jetty materials. The estimated overburden for the alternatives is presented in Table 8-1. These estimates are based on the assumptions described in the next two paragraphs.

Because the groin is approximately 30 feet tall, with much of its volume below the existing beach surface (Table 2-1), and because most of the groin is located in the surf zone, its excavation would almost certainly require an extensive excavation support structure, such as an encircling sheet pile wall. The sheet pile wall would both reduce the extent of excavation needed to expose the lowest parts of the groin and prevent waves from quickly filling in the excavation with sand. Within the surf zone, even smaller waves common during the expected summer work period are powerful enough to fill an excavation.

The wooden seawall's piles and planks could possibly be extracted using vibratory methods. This method, which is usually viable in sand, only requires excavation of the top few feet of overburden even though the piles and planks may penetrate up to 24 ft deep (Table 2-1). Information about the depth of the other access elements (the roadway, the railway, and the rock fence) is limited. For these elements, the overburden could consist of a trapezoidal trench approximately 15 ft deep and with side slopes of 3H:1V. This trench would likely expose most of the access elements and may be stable enough to not require additional excavation support. The preferred excavation method would need to be refined with more information about the access elements and additional geotechnical analysis. The excavated sand could be piled on the ocean side of the trench to serve as a temporary protective berm to reduce the risk of wave overtopping to the excavation.

8.6.2 Construction Methods

Removing jetty elements would require heavy construction equipment to execute a series of demolition processes. The project would begin with mobilization to deliver equipment to the project site and to prepare the temporary site infrastructure. Most equipment would probably be delivered to the site by truck, unless barges were used to transport the jetty material. Temporary site infrastructure may include staging area improvements, access way improvements, excavation supports (such as sheet pile), water control equipment, and electric power systems.

Once the site was prepared, excavation of jetty materials would begin. To enable excavation of the jetty materials, sand overburden would first be removed to expose the

jetty materials. Depending on the excavation method, this may require additional excavation support as excavation deepens. Since the groin is capped with concrete, the concrete would need to be broken apart with jackhammers or similar equipment to reduce the groin to manageable sizes for excavation and hauling. The excavated material would be sorted to separate sand, which would remain on the beach, from the jetty materials to be removed.

After excavation, jetty materials would need to be hauled off the beach to a disposal site. Truck transport of jetty material may use a combination of off-road dump trucks on the beach and tractor-trailer dump trucks for surface roads. To avoid the need to transfer from off-road to tractor-trailer dump trucks, a temporary road could be built on the beach. Barges may be another transport option. While barges would bypass some or all of the road transport, they would require additional temporary infrastructure along either the beach's ocean or Estuary shoreline to enable loading and unloading.

Once excavation and transport were complete, overburden sand would be placed back into excavated areas. De-mobilization would follow and would include removing temporary infrastructure, cleaning up any project materials, and transporting all construction equipment off the beach. No import of sand to replace the removed jetty material is assumed. The volume of removed materials is a relatively small fraction compared to the beach berm's volume, the annual supply from the river (Section 5.2.1), and annual littoral sediment transport. The waves that intermittently overwash the beach berm would likely re-distribute sand to fill any depressions that might remain after removal was completed. To confirm this assumption that no sand needs to be imported to replace the removed jetty materials, a more detailed assessment of potential beach impacts may be needed, particularly if the groin, which constitutes the largest volume of jetty materials, was further considered for removal.

8.6.3 Site Access

Jetty modification would involve the delivery of construction equipment to the project site to demolish, excavate, sort, and load the jetty material. In addition, dump trucks and/or barges would be used to transport jetty materials from the beach to a disposal site. Removal of the jetty material from the beach would require transport of between approximately 500 and 61,000 cubic yards of material from the beach, depending on the alternative (Table 8-1). Only the degrade in place option, Alternative 3a, would not include any off-site material transport.

8.6.3.1 Existing access

Goat Rock State Beach connects to the regional road network via State Highway 1. Starting from the park's Highway 1 entrance, the access pathway to the jetty continues

along Goat Rock Road for about two miles, past several private residences before branching to State Park Road for several hundred yards to the Beach Parking Lot (Figure 1-1). Goat Rock Road is approximately 20 feet wide and State Park Road is approximately 16 feet wide. The jetty access elements begin just north of the Beach Parking Lot. The groin is another third of a mile across the beach from the Beach Parking Lot.

The jetty access pathway described in the paragraph above is currently used by Park staff to monitor the beach and Water Agency work crews to perform artificial breaching of the beach berm with one or two pieces of construction equipment. While this current use demonstrates that construction equipment can already access the project site, the roads' suitability for increased use by construction equipment would need to be assessed, particularly for dump truck traffic used to haul jetty materials. Improvements may be needed for the access roads, as well for the Beach Parking Lot and possibly the Goat Rock Parking Lot for equipment staging. The narrow widths of the roads would require traffic management, particularly where wide construction vehicles preclude two-way traffic on certain road stretches.

8.6.3.2 Hauling options

After demolition, excavation, and sorting, jetty materials would need to be hauled off the beach to for disposal. Because GRSB is comprised of loose sand that may pose challenges for heavy equipment access, hauling jetty materials off the beach would likely require either a temporary improved road across the beach so that tractor-trailer dump trucks or off-road dump trucks can reach the excavation site. If off-road dump trucks were used haul material off the beach, the jetty materials would be transferred into tractor-trailer dump trucks for long-range transport.

Another hauling option for some or all of the jetty material may be barges. Traversing the inlet with barges is probably not feasible, so the barges would either be exclusively ocean-based or Estuary-based. Ocean-based barges would require substantial temporary infrastructure, such as a temporary pier, and/or a barge-based crane to load material in the surf zone. Currently, ocean dumping is limited to dredged sediments, not large rock. So the barges would need to be towed to a nearby port. Like ocean-based barges, Estuary-based barges would need similar temporary infrastructure (e.g. pier and/or barge-based crane) to facilitate loading. Because of the calmer waters of the Estuary, this infrastructure would be less extensive than the needs for ocean-based barges. Estuary-based barges would be towed a short distance to an unloading area (e.g. the Jenner boat ramp), transferred to tractor-trailer dump trucks and then hauled by road to the disposal site. Infrastructure needs for transferring jetty materials to dump trucks at the unloading area would also need to be assessed and may require improvements to supplement existing infrastructure.

8.6.3.3 Disposal sites

Off-site disposal areas would have to be identified. Re-use of materials at nearby construction projects would reduce disposal costs. The rock in the jetty from Goat Rock is probably greywacke, durable sandstone that may be used in construction. If the redwood posts and piles are in suitable condition, they may be accepted into a wood recycling program. However, since some of the jetty is comprised of less in-demand construction materials (e.g. concrete), these materials may not be needed in the vicinity or may need to be crushed into smaller aggregate. If construction re-use cannot be arranged, the remaining material would need to be transported to an appropriate disposal site. For purposes of this feasibility study, we assume a one-way hauling distance of 32 miles. This is the approximate distance in road miles to Petaluma, where the County landfill is located, and also more than the distance to Santa Rosa, Guerneville, and Bodega Bay, possible developed areas which may need construction materials or have other disposal options.

8.6.4 Schedule

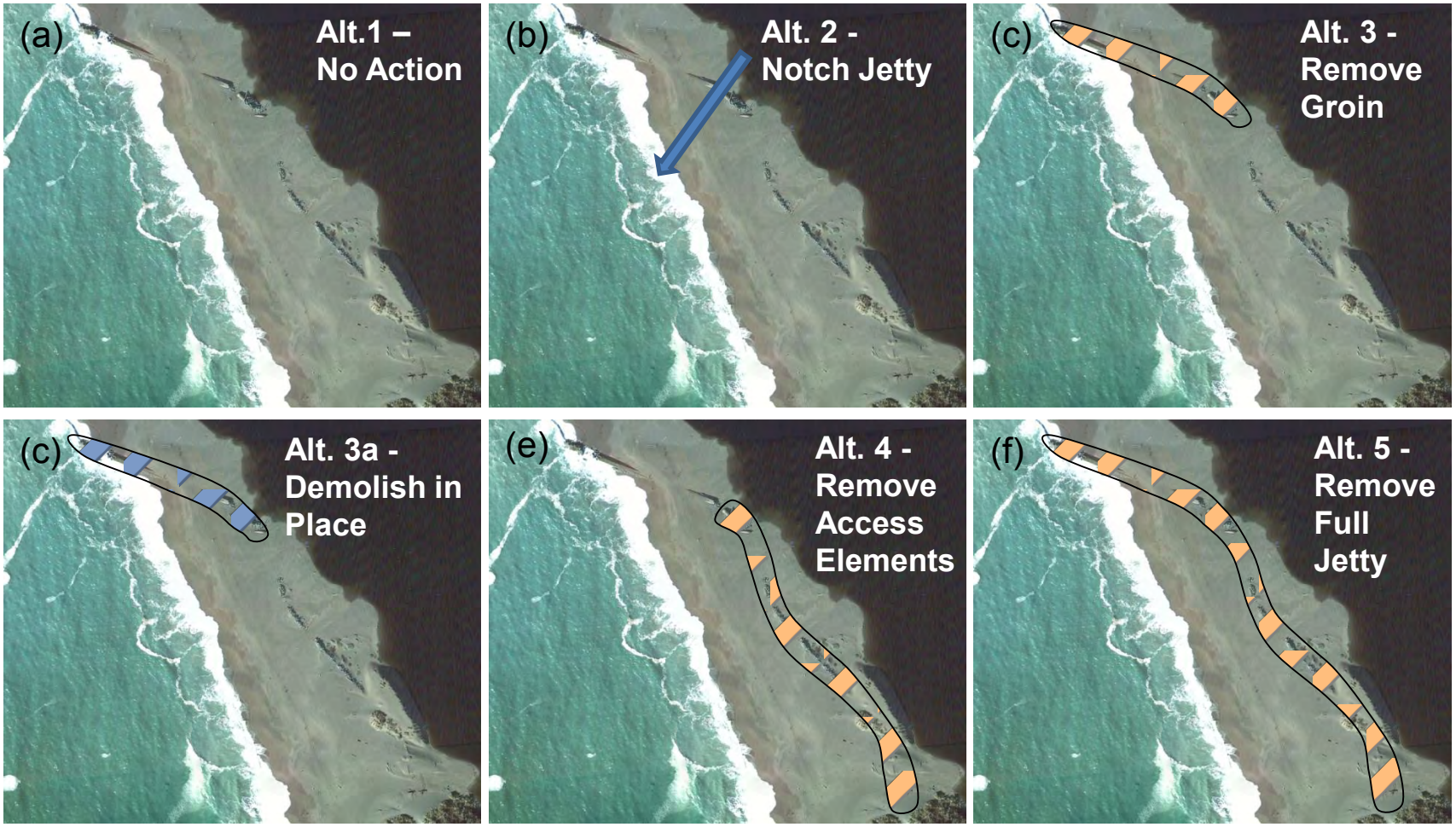
Each alternative would be constrained by two key scheduling issues: winter storms and pinniped pupping season. During the months of November through March, large ocean waves arrive with winter storms. Since large waves can overtop the project site, potentially damaging equipment, excavation supports, and filling in excavations, alternatives would probably not be executed during this period. Starting around March 15, pupping season begins when pregnant females give birth and nurse newborn pups on the beach. The pupping season lasts through the end of June. To avoid impacts to female harbor seals and their pups, jetty removal would likely require avoiding the beach during pupping season. The specific requirements for minimizing impacts to harbor seals would need to be coordinated with NMFS via an Incidental Harassment Authorization.

Accounting for these constraints, the remaining four months of July through October would likely be the period available for working to remove some or all of the jetty. Extensions on either end of this period may be possible for some limited work. For example, site mobilization, such as roadway access improvements and staging in the parking lots.

As indicated by the amount of material to be removed (Table 8-1), the alternatives vary considerably in level of construction intensity and duration. The least intense alternative, Alternative 2 (Notch Groin), would likely be completed within the four-month work window. However, as the effort intensity increases, up through Alternative 5 (Full Jetty Removal), it may be more difficult to complete the project in a single year's July-October work window.

More detail about the extent of jetty to be removed and the methods for removal is needed to estimate the project duration. These details would be developed if one or more alternatives were selected for additional planning and design. While it may be possible to complete Alternative 5 (Full Jetty Removal) in a single year, the schedule would be more tightly constrained and have less contingency. Trade-offs, such as the extent that an alternative disrupts visitor and recreation access, would need to be discussed with resource management agencies and stakeholders. For example, what areas of the access roads, parking lots, and beach would be closed to the public and when these closures would be in effect would need to be considered. Alternatives may need to be split over multiple years. Another option might be phasing the alternative so the response of the beach and Estuary to earlier phases can be monitored and used to inform subsequent phases. This phasing could be for a single alternative: Alternative 3a (Degrade In Place) could be carried out over several years to adapt the mechanical demolition effort to the natural dispersal by winter waves and river discharge. Or, the phasing could consist of escalating jetty removal. Alternative 2 (Notch Jetty), Alternative 3 (Remove Groin), and Alternative 4 (Remove Access Elements) could be done in phases that combine to comprise Alternative 5 (Full Jetty Removal). While this phasing may help address some of the uncertainty about the modifications' effects, the monitoring and repeated mobilizations would increase overall project cost.

8.7 Figures



SOURCE: Background image taken on 10/24/2009 and provided by Google Earth

NOTE: Blue coloration implies that modification does not include removal. Orange coloration implies that modifications include removal.

Goat Rock Jetty Feasibility Study. D211669.0

Figure 8-1

Illustration of jetty modification alternatives

9 EVALUATION OF ALTERNATIVES

This section evaluates the jetty alternatives described in Section 8 from multiple perspectives to inform the overall feasibility of the alternatives. Each of the alternatives examined would have the potential for short and long-term effects to the environment. The options for jetty removal, Alternatives 2 through 5, are compared relative to Alternative 1 (No Action).

First, the alternatives' potential benefits towards improving estuarine water surface elevation management are evaluated, as per the Biological Opinion's RPA 2.1. This includes the likely effects of the alternatives on beach permeability, beach and inlet morphology, estuarine water surface elevations, and water quality. Then, the evaluation considers the environmental impacts and constraints that may restrict jetty alternatives and also incur costs and require additional time for planning. These potential impacts and constraints would need to be described and possibly mitigated for under environmental compliance documentation (at state and possibly federal level) and regulatory permits. The consideration of impacts and constraints is divided into the short-term period during jetty removal and long-term after removal. In the next subsection, an estimate of probable cost is provided, both for jetty removal and long-term operations and maintenance. Finally, these evaluations are summarized in a series of tables to facilitate consideration of the tradeoffs between potential benefits, impacts, constraints, and costs.

This study's assessment of existing conditions identified the Goat Rock Parking Lot fill as impeding littoral sand transport and causing a considerably wider beach between Goat Rock and the groin (Section 5.1). When evaluating the alternatives, this condition is assumed to continue. If the removal of the Goat Rock Parking Lot fill were to be considered, perhaps as part of a regional beach restoration effort, this study's assessment of existing conditions, opportunities and constraints, and alternatives would potentially change.

9.1 Potential Benefits to Estuarine Water Surface Elevation Management

As per Biological Opinion, this study assesses the potential benefits of jetty removal for achieving target Estuary water surface elevations. As described in Section 1.3, Estuary water surface elevations are primarily a function of inlet morphology, with inlet closure causing the water surface to perch above the tidal elevation range that occurs when the

inlet is open. The inlet morphology is a function of the beach that hosts the inlet, ocean waves, tidal exchange, and river flow. Inlet morphology is also affected by antecedent artificial and self-breaching, for a period on the order of one year.

The Biological Opinion specifically calls for analyzing the effect of the jetty on 'sand storage and transport'. While these processes are accounted for both directly and indirectly in this study, this study focuses on beach and inlet morphology, which are the manifestation of sand storage and transport.

9.1.1 Beach Permeability

Beach permeability, the property of the beach berm which determines the rate of groundwater seepage, depends on the materials within the berm. As summarized in Section 3, LBNL (2015) observations indicate that seepage rates are 3.7 times faster (36-50 feet/hr) in sections where the seawall is absent than in sections where it was present (10-14 feet/hr).

The jetty removal alternatives, Alternatives 2-5, would remove different amounts of jetty material from the beach berm. In areas where jetty material is removed, the beach would presumably become more permeable, with an increase similar to the LBNL's estimates for sections with and without the seawall. The amount of jetty removed for Alternative 2 (Notch Groin) is presumed to cause negligible change because of its small volume removed from relatively high in the beach berm.

Seepage through the entire beach berm was estimated with a bulk D'Arcy approach. Estimates in Table 9-1 assume the ocean water surface is at mean sea level and assume a typical mean beach width of 350 ft (based on Water Agency surveys and LiDAR). These modeled seepage rates are consistent with several data sources. Beach permeability estimates are based on testing of sediment samples collected on the beach (see Appendix D). The modeled rates for Alternative 1 (No Action) are within the range of inferred flow rates based on the Estuary water balance during closures (Largier and Behrens, 2010). When these seepage rates are used in the QCM to simulate existing conditions (Section 6), the predicted rates of increase in water surface elevation closely match the observed water surface elevation time series in the closed lagoon.

Estimated seepage increases through the south beach berm as a result of Alternatives 3, 4, and 5 are approximately 50%, 100%, and 150%, respectively. When the seepage north of the groin is also considered, the relative impact of Alternatives 3, 4, and 5 is smaller: about 20%, 40%, and 60%, respectively. Removing just the access elements (Alternative 4) results in a greater seepage increase than the increase due to removing just the groin (Alternative 3) because the access elements are assumed to comprise a longer extent of the jetty along the beach berm. The LBNL permeability observations and seepage rate estimates were limited to the access elements and did not include the groin. Hence,

using the LBNL seepage rates from just two cross sections for the entire berm is an extrapolation; additional geophysics data could verify this extrapolation for a larger portion of the beach berm. For purposes of these estimates, the groin is assumed to reduce permeability at the same ratio as the access elements, i.e. by a factor of 3.7. The groin is assumed to block seepage because seepage through coherent pieces of rock would be negligible and the pores space between the pieces of rock is likely filled with either concrete or decades of tightly packed sand and finer sediments (C. Ulrich, pers. comm.).

Table 9-1. Jetty alternatives' estimated seepage through beach berm with closed inlet and Estuary water surface elevation at 5 ft, 7 ft, and 9 ft NGVD

Alternative	Alongshore jetty extent (linear ft)	Estimated seepage (ft^3/s)		
		5 ft	7 ft	9 ft
North of groin				
All alternatives	1,000 ft	7.2	15	25
South of groin (unobstructed / obstructed)				
1 (No Action)	200 ft / 1,000 ft	4.8	10	16
2 (Notch Groin)	200 ft / 1,000 ft	4.8	10	16
3 (Remove Groin)	550 ft / 650 ft	7.2	15	25
4 (Remove Access Elements)	850 ft / 350 ft	10	19	33
5 (Remove Full Jetty)	1,200 ft / 0 ft	12	24	41

9.1.2 Beach and Inlet Morphology

This section describes the potential changes in beach and inlet morphology as a result of jetty modifications. The jetty was designed with the primary objective of stabilizing the beach berm and reducing the morphologic response of the beach and inlet to waves, tides, and discharge. The groin was intended to reduce inlet closure and migration while the access elements were intended to stabilize the beach for construction equipment access. The proposed alternatives to remove some or all the jetty are therefore hypothesized to allow increased sand transport and thereby alter the current morphology. Increased transport would be consistent with more natural conditions, when beach sands are no longer stabilized by the groin and access elements. Changes to beach morphology would generally increase as more jetty material is removed.

The potential impacts of the jetty alternatives on beach and inlet morphology are discussed below. Anticipated conditions for each alternative are described first and then, after a discussion of maximum migration extent under Alternative 5, the risk of unanticipated migration extent for Alternatives 1-4 is discussed.

9.1.2.1 Alternative 1 – No Action

The jetty currently affects natural beach morphology in two primary ways: the groin constrains inlet migration to north of the groin and the access elements stabilize the beach berm south of the groin.

North of the groin, the inlet migrates between the groin and the northern bluffs, a distance of approximately 1,000 feet. As such, the beach north of the groin is intermittently eroded down to sub-tidal elevations. As the inlet migrates, the berm reforms in its wake. As such, most sections of beach north of the groin are, at most, only a few years old. Since the berm is regularly eroded, it is narrower and lower than south of the jetty (Section 5). Also, the north berm is likely less consolidated and has accumulated less fine material within pore spaces. As such, the north berm probably conducts groundwater seepage at a greater rate than the south berm.

Maps prior to jetty construction (Figure 5-2) indicate that the berm naturally widened towards the south, but was narrower overall because of the absence of the Goat Rock Parking Lot (Section 5.1). Without erosion by the inlet and with stabilization by the access elements, the south berm has grown higher, and probably less conducive to seepage, than would occur naturally. Historic photographs of GRSB, as well reference sites lacking a groin (Section 5.3), suggest migration and breaching south of the groin did occur, even if not necessarily typical. However, there is no record of the inlet crossing the beach berm south of the groin since construction of the jetty began more than eight decades ago. Although the core of the south berm remains largely undisturbed since jetty construction, waves do overwash and shape the beach face and crest (Figure 4-8). In some places, the wave erosion has been substantial enough to remove significant sections of the access elements and increase seepage (Section 3). In addition to stabilization provided by the jetty, other factors have contributed to growth of the south berm (Section 5.1). The construction of Goat Rock Parking Lot blocks north-to-south littoral sediment transport, contributing to beach widening and progradation. Logging and removal of other vegetation probably augmented the delivery of sediment from the watershed to the beach (Section 5.2). Overall, these two factors have resulted in net growth of the beach berm, overcoming whatever potential influence that sand mining and inlet breaching may have on decreasing berm size.

Although the inlet has not been south of the groin since the groin's construction, there is some small chance of the inlet eroding through the berm south of the groin. This potential exists because the south berm is already low enough to be overtopped by waves (Figure 4-8) and because low points on the south berm (Figure 4-7) are below extreme water surface elevations predicted for the Estuary (Table 7-5). So, under extreme and rare conditions, an inlet could be eroded through the south berm either by ocean waves or Estuary waters.

Waves and riverine currents have slowly degraded the jetty, and with no maintenance anticipated in the future, will continue to degrade the jetty. As jetty degrades, the risk of the inlet eroding through the south berm increases.

9.1.2.2 Alternative 2 – Notch Groin

During portions of the management period, notching the groin would connect the inlet, in the form of an outlet channel, to the south side of the jetty. When the outlet channel is flowing over the notch, it would erode the beach south of the groin more deeply than just the wave-induced erosion that currently occurs. The notch would fix the landward end of the outlet channel, so southward erosion would be limited to the seaward portion of the berm and extend for a similar range of lengths and widths as currently occurs north of the groin (Behrens, 2008). Using typical management period conditions and accounting for the portion of the channel north of the groin, the estimated southward extent of the seaward end of the outlet channel is 250 ft south of the groin. During periods when the inlet does not pass through the notch, the inlet would be north of the groin, similar to existing conditions.

This concept of a lagoon outlet channel draining over a groin that would serve as a weir does not have a known predecessor. While certainly weirs have been used to control outflow, their typical configuration does not entail flow onto a beach face in the active surf zone. These dynamic conditions may limit the outlet channel's capacity to convey stable outflow. The operations and maintenance of this channel would need to be considered during notch design and adjusted adaptively if the notch was constructed.

9.1.2.3 Alternative 3 – Remove Groin

Similar to Alternative 2's notch, removing the groin is anticipated to hold the landward end of the inlet fixed at the north end of the access elements and enable the seaward end of the inlet to migrate further south than its current range. Since the inlet could always migrate south of the groin, as compared to Alternative 2's outlet channel which is presumed to occur only during the management period, the Alternative 3 inlet may stretch further, up to approximately 400 ft south of the groin.

If Alternative 3a (Degrade In-Place) were implemented and the groin material submerged into the natural slope of the beach, the groin would have very limited role in attenuating waves before they break on the beach and runup. As a result, this option would reduce the groin's contribution to the formation of a low point at the beach crest during closures. However, the remaining groin material would continue to affect inlet migration. Unless the mechanical dispersion and natural wave processes was extensive, a large amount of groin material would exist just under the beach face. Once flush or just below the beach face, these buried materials would be sheltered from much of the wave energy and so would slow in their rate of dispersal. In this position just below the beach face, the groin material would be exposed by inlet migration. Given the size of

groin material, and its volume, even larger riverine flows would be slow to erode the groin material from its current alignment. As such, this option is anticipated to allow fairly limited southward migration of the inlet beyond its current alignment. The extent of the migration would depend on the amount of degradation and may be further for times when the inlet channel is relatively shallow and may be able to cross over the submerged groin materials.

9.1.2.4 Alternative 4 – Remove Access Elements

Erosion from powerful and steep winter waves can cause beach erosion (e.g. ESA 2014), which the jetty access elements may limit (Section 5). As a result, the current beach berm south of the groin is believed to be higher than if the access elements were not present (Figure 2-6). Removing the stabilization provided by the access elements would likely result in waves eroding the top of the beach berm and re-distributing the sand from the top into more extended splays on the backside of the berm, into the Estuary.

While removing the access elements is anticipated to result in a lower crest elevation south of groin, the trend of crest elevation increasing to the south (Figure 4-7) is expected to persist. This trend is a function of the overall geomorphic setting of the beach; the access elements only change the slope. While the overall trend of the crest is increasing to south, there are local low points south of the groin. These local low points correlate with sections of the berm without jetty access elements protrusions (Figure 2-6). So removing the access elements is likely to result in the higher sections of the crest decreasing toward the elevations of the nearby low points and less vertical variation of the alongshore crest profile.

9.1.2.5 Alternative 5 – Remove Full Jetty

Removing the entire jetty could restore the natural beach and inlet morphologic conditions that existed prior to the jetty. The inlet would likely migrate up to 600 feet south of the groin, as explained below. Physical process constraints and historic geomorphic markers indicate that migration further south than this is unlikely. The inlet's first migration south of the groin would be slower than subsequent southward migrations since the south beach berm has not been eroded by the inlet for more than eighty years. This lack of inlet erosion, as well as the effects of the Goat Rock Parking Lot fill, has caused the berm to be larger and more consolidated than it would be when undergoing subsequent inlet migrations.

9.1.2.5.1 Inlet Migration Extent

The inlet's frequent presence up against the north side of the jetty indicates that the jetty limits the southward migration of the inlet. So removing the jetty would enable the inlet to migrate further south. Southward migration past the groin would erode the south berm, a section of the beach that has probably not hosted the inlet since before jetty construction.

If the jetty was removed, how far south is the inlet likely to migrate? Physical process gradients and constraints suggest that the inlet would likely be limited in its southward migration. Although no hard limit exists, the anticipated maximum southward migration is anticipated to be up to half the distance between and groin and the Beach Parking Lot, or approximately 600 feet south of the groin.

Although migration further south than this cannot be definitively ruled out for such a dynamic and natural system, for reasons provided in the paragraphs below, further migration is thought to be unlikely and infrequent. A channel shift of this scale could result from avulsion, whereby a rare and extreme flood and/or wave event causes a river channel to abruptly change course. Overall, this assessment of inlet migration extent is tentative and if jetty modification is pursued, should be assessed in greater detail.

The general topographic gradient counters southward inlet migration, since southward migration would entail cutting through a higher and wider beach berm. While the larger berm is partially caused by the jetty, the increase in berm size is also the result of the mouth's watershed and regional littoral sediment setting, as evidenced by pre-jetty conditions (Figure 5-2). Spatial gradients of average alongshore wave energy (Section 4.2) reveals minimum wave energy to occur just offshore of the groin and increase to the north and south (Figure 4-3). At this minimum, the inlet would be more resistant to closure than at points further south. The increasing wave energy to the south likely corresponds to a decreasing likelihood of the inlet occurring further south.

The alignment of the river channel provides one constraint on southward erosion. The main channel of the Russian River flows along the north side of Penny Island and then turns northward just behind the beach berm. While removal of the jetty may enable the main channel to head west rather than make its northward turn, for the channel to align further south would require both a sharper radius turn and also the erosion of Penny Island and the shallow area between the island and the beach berm.

The subsurface remote sensing data also reveals factors which would limit southward inlet migration. Bedrock occurs substantially higher in the area just north of the beach parking lot (Section 3.1), suggesting that the inlet was less likely to have shifted this far south in the past. If the channel did migrate over the bedrock sill, it would be shallower. In addition, underlying the berm just north of the Beach Parking Lot and extending for approximately 300 ft north, the subsurface data detect the likely presence of a former marsh layer approximately 15 feet thick (Section 3.2). A marsh in this location would not co-exist with an inlet, indicating little historic precedent for the inlet to approach the current location of the Beach Parking Lot.

Further hindering the southward migration of the inlet is artificial fill located approximately five hundred feet north of the Beach Parking Lot, to the east of the jetty access elements. The source of this fill is not clear. Historic photographs indicate that this fill was placed between 1950 and 1972, several decades after jetty construction was completed. The fill is more than two hundred feet long, seventy feet wide, and fifteen feet high. The composition of the fill is uncertain, but its relatively stable shape and steep side slopes suggest material that can resist erosion by wind and precipitation runoff. This fill probably has relatively little impact on existing beach processes and the Estuary's water surface elevation, given its location on the backside of the beach berm, well south of the inlet. It may alter seepage through the beach berm, but this has not been assessed. Although this fill is artificial, the special status plant, Tidestrom's lupine, may be on or nearby this fill. If jetty removal enabled the inlet to migrate further south and this fill remained on the beach, this southward migration would eventually intersect with the north end of the artificial fill. Erosion of the fill by the inlet would cause fill material to spill down into the inlet, much like riprap placed to launching into a scour zone. Depending on the size of the material in the artificial fill, this influx of material into the inlet could slow or halt southward inlet migration. The future presence of this fill has not been evaluated; the fill could be removed as part of beach restoration or it could be eroded by natural processes, such as a channel avulsion.

9.1.2.5.2 Inlet Migration Timing

Southward migration of the inlet and its alteration of beach morphology would not be part of removing jetty materials. Rather, this change would be a natural geomorphic response to the jetty's removal. As such, prediction of time frame and likelihood are imprecise, with rates potentially influenced by timing of large, but infrequent extreme riverine discharge and coastal erosion events.

In the absence of an extreme event (such as riverine flooding or coastal erosion on the order of the 10-year event or larger), as channel migrates southward for the first time in decades, it would scour through a beach berm that increases in size (Figure 5-5, panel in left column and middle row). This migration would release sand into the inlet, which the inlet would then need to scour to migrate further. Because of the berm increases in size to the south and since the sands are likely to be more imbricated and erosion-resistant than the sands north of the groin which have been eroded more frequently, it may take typical river flows and ocean waves several years or even as long as a decade or two to erode southward the first time.

Sand eroded from the beach berm due to jetty removal would be transported southward, as this is the net littoral sediment transport direction along this portion of the beach (Section 5.1). The transported sand would be distributed and stored on the beach berm and nearshore bed between the groin and the Goat Rock Parking Lot. Sand transport and the potential for bypassing Goat Rock has not been evaluated in this study.

Once a section of the beach berm south of the groin has been eroded, then this section would be more erodible in subsequent years, even if the beach berm builds back up at that location. The re-built berm would likely to be lower and narrower than with the jetty. Before the re-built berm experiences decades-long deposition and stasis, the berm would likely be eroded again by the next southward migration of the inlet.

Although typical conditions may cause the initial southward migration to be slower that migration rates observed north of the jetty, larger, but less-frequent river discharge and waves could rapidly accelerate southward migration. Erosion is generally a non-linear function of discharge and wave height, such that increases in one or both of these hydrologic forcing parameters could result in an exponential increase in erosion rates. So if a less frequent, but considerably larger than typical, event occurs before the first gradual southward migration, this single event (which could include erosion from river discharge, waves or both) could cause relatively rapid southward migration or possibly an abrupt channel avulsion.

Regardless of the rate of the inlet's first south migration, the conditions within a decade or two after full removal would likely to converge to a case where the inlet has been as far south as possible without scouring Penny Island, i.e. about six hundred feet south of groin. Once the first southward migration has occurred, when the beach re-forms in a section due to constructive wave-building, the re-built beach berm would be more susceptible to erosion via lateral migration of the inlet. Once the inlet has reached that location, it is more likely to re-visit that location in subsequent years.

9.1.2.6 Inlet Migration Extent: Alternatives 1-4

The assumed maximum southern extent for full jetty removal is approximately 600 feet south of the groin, about halfway between the groin and the Beach Parking Lot. The rationale for this extent is described above for Alternative 5, which would completely remove the jetty from the beach.

For Alternatives 1-4, the jetty would be anticipated to constrain the southern extent to less than the maximum extent predicted for Alternative 5. For Alternative 1 (No Action) and Alternative 4 (Remove Access Elements), the inlet would be anticipated to remain north of the groin. For Alternative 2 (Notch Jetty), the tidal inlet would be anticipated to remain north of the groin, with the outlet channel crossing over the groin and onto the seaward beach face during portions of the management period. Because the landward end of the outlet channel would be fixed by the notch, the channel’s southern extent would be limited. For Alternative 3, the landward end of the channel would be prevented from further south migration by the north end of the access elements; the seaward end would migrate southward to some extent.

Although the extents described above are the anticipated conditions, there is some potential for further southward migration of Alternatives 1-4. Probably due to an extreme coastal or fluvial flood, but possibly due to accumulated degradation of the jetty and typical conditions, the inlet under Alternatives 1-4 could migrate as far south as the potential maximum extent described for Alternative 5, approximately halfway between the groin and the Beach Parking Lot. The potential for additional migration as a function of the alternatives is difficult to quantify, but probably scales with the amount of jetty material removed. So Alternative 1, with no material removed, would probably be the least likely to depart from its anticipated extent and Alternative 3, with the largest amount of material removed (besides Alternative 5), would be most likely to depart from its anticipated extent. Overall, this assessment of inlet migration extent is tentative and if jetty modification is pursued, should be assessed in greater detail.

9.1.3 Estuary Water Surface Elevations

As described in the preceding two sections, jetty removal is anticipated to change beach permeability as well as beach and inlet morphology. These changes would then influence Estuary water surface elevations. Water surface elevations would be affected by changes in these processes as follows:

- Increased beach permeability that increases groundwater seepage from the Estuary to the Ocean
- Decreased beach crest elevation that increases wave overtopping
- Increased inlet migration south of the groin that increases wave energy exposure and inlet closure

To assess how these physical process changes may combine to affect the Estuary's water surface elevations, the alternatives were evaluated with the QCM (Section 6.1) which tracks the Estuary water balance and the inlet's sand balance.

Table 9-2 lists the potential changes to physical processes for the five alternatives that were assumed for the QCM. Note that these conditions only include the anticipated inlet morphology and extents described above and do not consider the potential for channel avulsion. Alternative 1 assumes that no jetty removal action is taken. Alternative 2 (Notch Groin) would rely on the inlet closing naturally at the beginning of the management season in late spring or early summer. Once water surface elevations in the Estuary subsequently fill to the elevation of a notch cut in the groin at 5.5 feet NGVD, the notch would be cleared of sediment with construction equipment, allowing the Estuary to spill through the notch and onto the beach south of the groin. If this perched overflow persists to the end of the management period, the notch and inlet would again need to be actively managed to close the notch in mid-October and then to artificially breach north of the groin. Alternative 3 (Remove Groin) would slightly increase seepage and expand the inlet migration range. Alternative 4 (Remove Access

Elements) would increase seepage by a greater amount than Alternative 3 and lower the beach crest due to erosion from winter waves, but is assumed to not allow inlet migration south of the groin. Alternative 5 (Remove Full Jetty) would result in the highest seepage, the lowest beach crest, and the greatest migration range. Note that Alternative 3a (Degrade In Place) was not analyzed for its potential effect on estuarine water surface elevation, but is assumed to progress towards the same effects as Alternative 3 (Remove Groin).

Table 9-2. Changes to physical processes for each jetty alternative

Alternative	Permeability	Beach Crest	Inlet Migration Range
1 (No Action)	• No change	• No change	• North of groin
2 (Notch Groin)	• No change	• No change	• South of groin • Migration does not affect beach crest south of groin
3 (Remove Groin)	• Higher seepage	• No change	• South of groin • East end of channel limited by remaining access elements
4 (Remove Access Elements)	• Higher seepage	• Lower beach crest	• North of groin
5 (Remove Full Jetty)	• Highest seepage	• Lowest beach crest	• South of groin • East end of channel may migrate about 600 ft south

The changes to physical processes listed in Table 9-2 were represented in the QCM as described in the next section. By comparing the results of Alternatives 2-5 with Alternative 1 (No Action) over this period, the potential changes in Estuary water surface elevations are assessed.

9.1.3.1 Methods

The methodology for representing changes to the beach permeability, beach crest elevation, and inlet migration are described in the paragraphs below. Before combining these changes as indicated by Alternatives 2-5, the sensitivity of model predictions to varying only one process at a time was evaluated.

Since LBNL (2015) has shown that seepage rates are lower in GRSB where access elements are present (Section 3), removing all or parts of the jetty was assumed to increase beach permeability and hence, seepage from the Estuary to the Ocean during

inlet closure (Section 9.1.1). For the jetty removal alternatives, the permeability is assumed to increase according to ratio of LBNL's estimated permeability for the beach berm with and without the access elements. This increase in permeability is scaled by the length of jetty removed to predict the overall increase in seepage (Table 9-1).

The beach crest elevation influences the wave overwash into the Estuary during mouth closure, and we assume that alternatives may change it due to a few factors: First, comparable estuaries that have varying degrees of migration have different beach crest profiles. This was discussed in Section 4 for the Gualala River and Navarro River. Beaches without inlets tend to have high crests that can be related to infrequent (e.g. 90th percentile or higher) wave runup events (e.g. Battalio et al. 2006). In contrast, beaches with inlets have crests that are lower than would be predicted by wave runup alone, since migration erodes the beach, sometimes reducing its elevation. Another factor is that erosion from powerful and steep winter waves can cause beach erosion (ESA 2014), which Section 5 has suggested has been prevented by the jetty access elements.

For alternatives which remove the access elements from the beach berm (Alternatives 4 and 5), the berm crest elevation was lowered in the QCM using the relationship between total water levels (ocean tide levels plus wave runup) and beach berm crest elevation north of the groin. The total water levels were derived from the wave modeling results and the beach berm crest elevations were from Water Agency surveys. After establishing this relationship for north of the groin, the total water levels for south of the groin were used to predict the berm crest elevation south of the groin. For Alternative 4 (Remove Access Elements), the reference berm crest elevations were selected from well north of the groin, where the inlet does not play as significant role in crest elevation. For Alternative 5 (Remove Full Jetty), the reference berm elevations were selected from just north of the groin so that the relationship also included the influence of inlet migration on the crest elevation. The resulting south berm's crest elevation for Alternative 4 was higher than the crest elevation for Alternative 5.

Also, since the jetty groin poses a direct barrier to southward inlet migration on a yearly basis (ESA 2012-2015), jetty removal is assumed to allow more extensive inlet migration (Sections 9.1.2.5.1 and 9.1.2.6). For Alternative 3 (Remove Groin), the west side of the inlet can migrate south along the beach face, but the jetty access elements act as a hard boundary, preventing the east side of the inlet from also moving south, meaning that while the inlet mouth may close south of the groin, it can only breach north of it. For Alternative 5 (Remove Full Jetty), since there would no longer be access elements in the beach south of the groin, the inlet mouth could both migrate freely south of the groin's present location and also breach south of the groin's present location after closures. This could lead to the inlet remaining south for longer periods of time. In both alternatives, as the inlet migrates further south, it faces higher wave energy and hence has greater potential for closure.

Before evaluating the alternatives, the model's sensitivity was tested by independently varying seepage rates, beach crest height, and migration range. The jetty alternatives use combinations of these changes which can change estuarine water surface elevations in opposite directions and hence result in less net change. This sensitivity analysis helps isolate the effect of each change in physical process on the Estuary's water surface elevation.

The results of the sensitivity analysis are provided in Table 9-3. This table provides the simulated number of days closed for each year of the simulation, as well as the average annual number of days closed. Closure days are a key, yet simple indicator of Estuary water surface elevation since closure results in the water surface elevation shifting from the tidal range (< 5 ft NGVD) to the perched range (> 5 ft NGVD).

Increasing seepage generally increases the number of closure days, as it lengthens closure events by slowing the rate that the lagoon would fill to breaching elevations. This relationship between seepage and closure days was identified in similar modeling of the Carmel River lagoon (Rich and Keller, 2013). More change occurs between the 'existing' and 'higher' (Alternative 4) seepage cases than between 'higher' and 'highest' (Alternative 5) cases because the 'higher' case involves removing all access elements (a longer section of the beach), while the 'highest' case only incrementally increases seepage since the groin occupies a shorter section of the beach than the access elements.

Lowering the beach crest reduces the number of days with mouth closure. Reduced closures are predicted because larger amounts of wave overwash fill the lagoon and induce mouth breaching more quickly, a relationship also identified by Rich and Keller (2013). The amount of overwash and the extent of the effect on reducing closure days per year increased as the beach crest was lowered from the 'lower' (Alternative 4) to the 'lowest' (Alternative 5) cases.

In contrast, increasing the migration range is predicted to increase the number days closed. This is because southward migration exposes the mouth to higher-energy waves (Figure 4-3). For the 'greater' migration range (Alternative 3), the mouth was allowed to move south of the groin, but could only breach north of it, whereas for the 'greatest' migration range (Alternative 5), the mouth was allowed to both migrate and breach south of the groin, so that it could potentially stay south of the groin for more of the year. The latter case exposed the mouth to higher wave energy and led to more closure days.

Table 9-3. Modeled number of mouth closure days from 2001 to 2013 as a result of modifying beach seepage, beach crest height, and inlet migration range.

	Beach Seepage			Beach Crest Height			Inlet Migration Range		
	Exist.	Higher S1	Highest S2	Exist.	Lower C1	Lowest C2	Exist.	Greater M1	Greatest M2
2001	72	76	75	72	53	47	72	73	71
2002	53	58	56	53	50	48	53	56	61
2003	25	31	28	25	24	18	25	22	25
2004	49	50	51	49	48	44	49	49	50
2005	33	36	37	33	28	31	33	33	33
2006	32	33	33	32	32	29	32	33	34
2007	103	107	108	103	104	90	103	105	103
2008	78	89	83	78	74	63	78	69	65
2009	99	110	112	99	83	86	99	98	106
2010	27	27	26	27	24	18	27	35	35
2011	43	49	47	43	41	39	43	49	50
2012	62	68	72	62	58	52	62	67	80
2013	85	87	100	85	79	70	85	86	85
Avg	58	63	64	58	54	49	58	60	61

[S1] Seepage increase due to removing access elements (Alternative 4)
[S2] Seepage increase due to removing access elements and groin (Alternative 5)
[C1] Crest elevation lowered due to removing access elements (Alternative 4)
[C2] Crest elevation lowered due to removing access elements and groin (Alternative 5)
[M1] Inlet migration south of groin, breaching after closure north of groin (Alternative 3).
[M2] Inlet migration south of groin, and breaches occur wherever closure occurs (Alternative 5).

In addition, the model's sensitivity to the management parameter of artificial breaching threshold was also assessed. For existing conditions (Alternative 1), the model's artificial breaching threshold was raised to 8.0 ft and 8.5 ft NGVD. When compared to the artificial breaching threshold of 7.5 ft, the model predictions exhibit a corresponding increase of about a half foot at the top end of the water level range (Figure 9-1). These higher water levels also occur for slightly larger fraction of time. During a closure in the management season, the rate of rise of the estuary water level is highly variable depending on the freshwater inflows. For freshwater inflows above 100 ft^3/s the rate tends to be 0.25-0.5 ft/day, but can be higher following a rare rainfall event during the lagoon management season or when wave overwash into the estuary is extensive. This rate of increase would mean that closures only last a few days longer for each half foot increase in the artificial breaching threshold. When inflows drop below 100 ft^3/s, rates of increase tend to be less than 0.25 ft/day, and raising the breach threshold would have a slightly larger impact. For the full year conditions, which include higher riverine inflows, the rate of rise of the estuary can be even faster, so the increase in artificial breaching threshold has less effect on the fraction of time.

9.1.3.2 QCM Results For Jetty Alternatives

The existing conditions QCM model (Section 6.1) was assumed to represent Alternative 1 (No Action) and this model was then modified to represent Alternatives 2 through 5 by combining the changes to physical processes described in Table 9-2. The QCM model simulations suggest that some of the jetty removal alternatives would raise the water surface elevations in the lagoon, while others would lower it. Figure 9-2 summarizes the results for each year and for each management season from 2001 to 2013. The total annual flow at Guerneville and average September flow are also shown in this figure to identify the sensitivity of the results to year-to-year variability in river flows.

To achieve the Biologic Opinion's target of perched water surface elevations 7-9 ft NGVD requires the Estuary to be closed. Therefore, to evaluate the alternatives' likelihood of achieving these water surface elevations, the full 15-year time series of QCM outputs were distilled to the average number of days closed. The average number of days closed per year predicted by the QCM for Alternative 1 (No Action) is 59 days. For most years and alternatives, the number of closure days per year was more sensitive to the river flows at Guerneville than to all of the jetty removal alternatives except one. The exception is the predicted increase in closure days for Alternative 2 in 2008 and 2009 (Figure 9-2). In these two years with less-than-average river flow, a closure was predicted for early in the management period because of coincident decline in river flow and increase in wave energy. Alternative 2's notch then provided the mechanism for a sustained outlet channel that enabled the closure and associated perched lagoon water surface elevations to persist through June, July, and August (Figure 9-2, top left panel). Smaller increases in number of days closed were predicted for Alternative 2 in 2001 and 2013 (Figure 9-2), for an average of approximately one in three years with at least a 20% increase in number of days closed as compared to Alternative 1 (No Action). When these early management season closures occur under Alternative 1, the river flows fill the Estuary rapidly enough that either self or mechanical breaching occurs. New closures in July and August are less frequent because of low wave energy in these months (Figure 4-1). With the fall increase in wave energy, closures are more likely and river inflows have declined further. Therefore, Alternative 2 does not result in as much difference in closure days in September and October.

In general, alternatives that minimize the beach crest (Alternative 4, Alternative 5) have slightly lower water surface elevations for the calendar year than Alternative 1 (No Action), as shown in the left panel of Figure 9-4. This was predicted because overwash had a larger impact than the other factors. Alternative 4 has the lowest water surface elevations of all alternatives because it is predicted to increase overwash, but without also increasing the inlet migration range. Since Alternative 5 allows for full migration (encouraging perched conditions and exposing the inlet to more powerful waves south of the groin), predictions are for slightly higher water levels than Alternative 4. Even though Alternative 5 is predicted to have slightly higher water levels than Alternative 4,

Alternative 5's higher rates of overwash end some closures sooner, resulting in fewer days closed. In contrast to Alternatives 4 and 5, Alternative 3 (Remove Groin) encourages migration but is not expected to lower the beach crest elevation, so it is predicted to raise water levels relative to Alternative 1 (No Action). As shown in Figure 9-2's upper right panel, the increase in closures for Alternative 3 occur primarily in summer and early fall because the simulated inlet migrated south, and was exposed to higher wave energy during this time.

Alternative 2 (Notch Groin) had the highest water levels of all alternatives, since the notch in the groin is assumed to preserve outlet channel conditions for early management period closures. While the QCM assumes that these management actions can be executed when needed, several factors are likely to decrease the reliability of creating and sustaining flow through the notch. As discussed in Section 8.2, current beach management activities indicates that beach and surf conditions can already impede actual execution. Conducting beach management around the groin further complicates beach management because it requires excavation right next to buried rock and reduces flexibility in selecting channel alignment. Also, as discussed in more detail in Section 9.2.3.2.2 below, the early part of the management period overlaps with harbor seal pupping season constraints, when an outlet channel over the notch is predicted to yield the biggest increase in number of days closed and Estuary water surface elevations. Section 8.2 also notes that the outlet channel on both the upstream and downstream portions of the notch may be susceptible to scour that could be partially addressed by adding additional armoring to the notch structure, but that this armoring would need to consider public safety, aesthetics, and beach management.

Overall, compared to 59 days of closure per year for Alternative 1 (No Action), Alternative 2 is predicted to increase closure to 71 days per year, compared with 63, 57, and 56 days per year for Alternative 3, Alternative 4, and Alternative 5, respectively (Figure 9-3). None of the alternatives are predicted to change the water level enough to meet the goal of the Biological Opinion to attain 3.2 ft NGVD or higher water levels for 70 percent of the year (Figure 9-4). The alternatives' effect on days closed during the management period follows a very similar inter-annual pattern as the annual number of days closed (Figure 9-2).

The predicted changes to the Estuary tide range frequencies are provided in Table 9-4. Like other water level metrics, Alternatives 3-5 are predicted to cause only relatively minor shifts from Alternative 1's tide range frequencies. Also like the other metrics, Alternative 2 (Notch Groin) results in the largest change in the tide range frequencies. In addition to the increase in number of days closed, Alternative 2 also results in increased frequency of tidal range between 0-1 ft. The periods when Alternative 2's tide range is between 0-1 ft occur during closures and should probably be viewed as an extension of closed conditions rather than an alternate state.

The 0-1 ft tide ranges may be occurring as the model re-equilibrates to the rapid initial discharge when flow over the notch begins or because of backwater effects from higher ocean tides.

Table 9-4. Predicted tide range frequency for jetty alternatives.

		Tide Range					
Alternative	**Closed**	**0-1 ft**	**1-2 ft**	**2-3 ft**	**3-4 ft**	**4-5 ft**	**>5 ft**
1 (No Action)	20%	4%	4%	13%	27%	22%	11%
2 (Notch Jetty)	25%	14%	4%	12%	22%	16%	7%
3 (Remove Groin)	21%	3%	4%	13%	27%	21%	11%
4 (Remove Access Elements)	19%	4%	4%	14%	27%	22%	11%
5 (Remove Full Jetty)	19%	4%	5%	13%	27%	21%	11%

These predictions suggest that the biggest change in beach management activities would occur for Alternative 2 (Notch Groin). The notch would need to be opened for all closures during the management period, which could include the initial opening and subsequent notch maintenance if wave events deposit sand in the notch. If the notch is conveying outflow at the end of the management period, beach management would be required to fill in the notch and artificially breach north of the groin. For Alternatives 3-5, the number of closures was not predicted to change much due to jetty removal. Rather, jetty removal affected the duration of closures. So beach management would likely be required with similar frequency as Alternative 1, with several management actions needed throughout the year for artificial breaching and outlet channel facilitation. The frequency of management would probably vary more inter-annually as a function of river flows, ocean waves, and beach morphology than between Alternatives 1, 3, 4, and 5.

9.1.4 Water Quality

The Biological Opinion sets a target for higher estuarine water surface elevations partly because these higher elevations are hypothesized to positively correlate with water quality conditions supporting the Estuary's ecological function as salmonid habitat (NMFS, 2008). Two key water quality benefits associated with higher water surface elevations are freshening of the Estuary's salinity and increasing the area of shallow inundation.

9.1.4.1 Freshening

Fresher, less saline water is thought to improve ecological function of the Estuary by providing habitat for juvenile salmonids that have not yet acclimated to higher salinity levels and by increasing the availability of aquatic invertebrate prey (NMFS, 2008). Fresher conditions are associated with higher estuarine water surface elevations, which occur due to inlet closure. When the inlet closes, saline ocean water no longer enters the Estuary with each incoming tide. With the blocking of tidal flows, the less-dense

fresh water can layer upon the denser saline water, reducing mixing between fresh and saline, thereby preserving the fresh water. The saline water remaining in the Estuary tends to seep out through the beach berm as riverine inflows continue to add fresh water (Largier and Behrens, 2010). Therefore, as inlet closure persists, the volume and spatial extent of the upper freshwater layer tends to increase.

Although other factors such as the amount of river discharge and the strength of the tides contribute to the balance between fresh and saline water in the Estuary, the strongest shift to fresher conditions occurs when Estuary water surface elevations perch above oceanic tides. So the water surface elevations can be used as an indicator of the potential for freshening. When water surface elevations are 5 ft NGVD or higher, they are just spring high tide elevations in the ocean. As shown in Figure 9-4, Alternatives 3-5 are not anticipated to be substantially change the fraction of time that water surface elevations exceed 5 ft NGVD as compared to Alternative 1 (No Action). Therefore, these alternatives are not expected to cause much change in the frequency of freshwater conditions as compared to existing conditions. The biggest change in 5 ft NGVD exceedance is predicted for Alternative 2 (Notch Groin). Over the entire year, this alternative is predicted to increase the time above 5 ft NGVD by approximately 18%. During the management period, which is when the notch could be used to manage Estuary water surface elevations, the increase is larger, approximately 37%. Higher water surface elevations create a deeper fresh water surface layer, both by raising the surface and increasing the seepage rate for saline water from the lower layer. However, at higher elevations, the increase in percent exceedance (Figure 9-4) is less between Alternative 1 and Alternative 2. For example, the increase at 7 ft NGVD during the management period is only about 4%.

During periods when the inlet is not closed, but constricted enough to substantially mute the estuarine tide range, the Estuary probably becomes fresher. However, initial assessments from an ongoing study have not identified a clear relationship between tidal muting and decreased estuarine salinity (Largier and Koohafkan, 2016). Other factors, such as the spring-neap cycle, tidal phase, river discharge, and wave overtopping also affect salinity, and obscure the relationship between tidal muting and salinity. As indicated by Table 9-4, the predicted change in tidal muting for Alternatives 3-5, as compared to Alternative 1 (No Action), is minimal. Alternative 2 (Notch Groin) is predicted to increase muting, which may result in fresher estuarine conditions, presuming that the other potential constraints of Alternative 2 can be addressed.

Although during perched conditions, ocean tides are blocked from the Estuary, saline water can enter via wave overwash. Increases in salinity during an overwash event can be on the order 5-10 ppt once the overwash mixes with freshwater in the Estuary (Largier and Behrens, 2010). The salinity increase typically extends 2 to 3 miles upstream from the mouth, where a shallow sill tends to block salinity from advancing farther. The time to purge the influx of wave overwash salinity could take up to a month. Because

Alternative 4 and 5 would reduce the berm crest elevation, thereby increasing overwash, they would also increase amount of salinity entering the Estuary from overwash.

9.1.4.2 Shallow inundation area

Increased water surface elevations also directly cause shallow inundation of the land fringing the Estuary's tidal shoreline. This additional shallow inundation areas expand the volume of available habitat. This expanded habitat is not only available for salmonids, but also, because of the shallow depths and proximity with vegetation, well-suited for the populations of aquatic invertebrates upon which salmonids feed.

As shown in Figure 9-4, Alternatives 3 through 5 are not anticipated to substantially change the fraction of time that water surface elevations exceed 5 ft NGVD as compared to Alternative 1 (No Action). Therefore, these alternatives are not expected to cause much increase in the Estuary's shallow inundation area as compared to existing conditions. The biggest change in 5 ft NGVD exceedance is predicted for Alternative 2 (Notch Groin). Over the entire year, Alternative 2 is predicted to increase the time above 5 ft NGVD by approximately 18%. This elevation increase from the daily high tide to perched lagoon conditions inundates an additional 41 acres within the Estuary. During the management period, which is when the notch could be used to manage Estuary water surface elevations, the increase is larger, approximately 37%. Higher water levels could further increase shallow inundated area by about 30 acres per foot for water surface elevations above 5 ft NGVD. However, at higher elevations, the increase in percent exceedance (Figure 9-4) is less between Alternative 1 and Alternative 2. For example, the increase at 7 ft NGVD during the management period is only about 4%.

9.2 Environmental Impacts and Constraints

In addition to the potential benefits of jetty removal, the alternatives would likely cause some environmental impacts and be subject to constraints. To help inform the feasibility of jetty removal, this section describes the regulatory setting under which jetty removal would occur. Then the potential short term impacts and constraints during the process of jetty removal are presented. In addition to short term conditions, the potential impacts and constraints over the long term are also considered.

The assessments in this section are intended as preliminary information at this feasibility planning stage. They are not sufficiently detailed to meet environmental documentation or permitting standards. If any of the alternatives are selected for additional analysis or design, these environmental compliance assessments should be re-evaluated and the potential for other impacts considered.

9.2.1 Regulatory Setting

Modification of the jetty would require environmental review under the California Environmental Quality Act (CEQA). If a federal agency was the lead agency or provided funding, environmental review under the National Environmental Policy Act (NEPA) would also be required. In addition to these environmental reviews, a project to modify the jetty may need consultations, permits, and approvals in accordance with the following federal, state, and local laws and regulations. This list is not intended to be exclusive and exhaustive; other permits and approvals may be required.

- Federal
    - Endangered Species Act
    - Marine Mammal Protection Act
    - Migratory Bird Treaty Act
    - Rivers and Harbors Act
    - Clean Water Act
    - Magnuson Stevens Act
    - Coastal Zone Management Act
    - Greater Farallones National Marine Sanctuary
- State
    - California Endangered Species Act
    - Fully protected and species of special concern
    - California Fish and Game Code
    - Marine Life Protection Act
    - Marine Recreation Area and Marine Conservation Area
    - Native Plant Protection Act
    - Regional Water Resources Control Board
    - State Lands Commission
    - Coastal Act
- Local
    - Sonoma County General Plan
    - Sonoma County Local Coastal Plan
    - Sonoma Coast State Park General Plan

Many of the environmental compliance issues and permits raised by jetty removal would be similar to those identified in the Environmental Impact Report (EIR) for the Russian River Estuary Management Project (ESA 2010; ESA, 2011). Jetty modification was considered in the EIR's alternatives analysis. The EIR concluded that jetty modification was not likely to achieve the larger Estuary Management Project objectives on its own. However, the EIR acknowledged that jetty modification could enhance salmonid habitat in conjunction or combination with other Estuary management actions, pending further feasibility assessment. Also note that while jetty modification

was considered under the prior EIR, it was not considered as the preferred project. Therefore, jetty removal should be considered a separate project in need of its own environmental compliance review, led by the appropriate, but as-yet not determined, agency.

9.2.2 Potential Short Term Impacts and Constraints During Jetty Removal

To provide an integrated review of each alternative, key issue areas were reviewed to characterize anticipated impact related to construction. This review focuses on issue areas that would be perceived by the public and regulatory agencies as being of greatest concern and may require some level of mitigation. Key issue areas included in this analysis include: transportation and traffic, noise, water quality, biological resources, recreation and access, and cultural resources. While not exhaustive, these key environmental issue areas have the potential to affect the feasibility or level of impact related to each alternative. Comparisons of jetty removal alternatives' impacts relative to Alternative 1 (No Action) are provided. Additional detail and reasoning for these assessments are provided in the following subsections. These impacts are likely only during and shortly after the period when jetty materials would be removed from the beach. Potential long-term benefits and impacts are described in the following section.

9.2.2.1 Transportation and Traffic

As described in Section 8.6, implementing a jetty alternative would require access to the beach with construction vehicles and hauling jetty materials either by truck or barge. Meeting these transportation needs of the project has the potential to impact GRSB staff and recreational users, private homeowners along the Park access road, and users of the regional roadways. In addition to these users, Estuary-based barges may also impact recreational boaters on the Estuary. The potential impacts of these aspects of jetty removal on transportation and traffic are described in this section.

Regardless of the method for hauling removed jetty material to the disposal site, Alternatives 2 through 5 would likely require between three and ten heavy construction vehicles. These construction vehicles would be transported to the site on wide-load trucks, and offloaded and staged in one of the GRSB's parking lots, with the potential for parking lot closure. The Beach Parking Lot, as the closest to the project site, would be a likely staging area. The Goat Rock Parking Lot or upper overlook parking lot could also be needed. As the amount of material to be removed (Table 8-1) increases, so would the number of construction vehicles and use of parking areas increase. Of the alternatives which remove material from the beach, Alternative 2 (Notch Jetty) would have the least impact and Alternative 5 (Remove Full Jetty) would have the largest impact. Alternative 4 (Remove Access Elements) and Alternative 3 (Remove Groin) would have intermediate

impacts. Alternative 3a (Degrade In Place) would have the least impact on transportation and traffic since it would only require construction equipment access, but no hauling of jetty material from the beach.

If truck-based removal was used to haul jetty material to its disposal site, this material would probably be transported via tractor-trailer dump trucks once the material was off the beach. The dump trucks would travel along GRSB access roads to Highway 1, and then use Highway 1 and other regional roadways to reach disposal sites. A disposal site within 32 miles of GRSB is assumed, a travel distance which includes the municipalities of Guerneville, Bodega Bay, Petaluma, and Santa Rosa, as well as other smaller towns and unincorporated areas. The number of truck trips that would be required for each alternative, based on initial estimate of the amount of jetty material to be hauled away and 12 cubic yards (CY) per truck trip, are shown in Table 9-5. In addition, the number of 5-day work weeks during which hauling would occur is estimated in order to approximate the average daily frequency of truck trips. These transportation needs are provided as an initial assessment of additional inputs to regional traffic flow. A more complete traffic analysis, with revised estimates of the project's transportation needs, would need to be completed. The need for roadway improvements and repairs would likely scale with the number of truck trips.

Table 9-5. Estimated number and frequency of truck and barge trips to haul jetty material to unidentified disposal sites.

Alternative	Haul volume (CY)	Total # haul weeks	Total # truck trips	Avg. # truck trips per day	Total # barge trips	Avg. # barge trips per week
2 (Notch Jetty)	610	2	51	5	-*	-*
3 (Remove Groin)	56,000	13	4,700	72	28	3
4 (Remove Access Elements)	17,000	8	1,400	36	9	1
5 (Remove Full Jetty)	73,000	13	6,100	94	37	4

* Haul volume for Alternative 2 is assumed to be too small to justify temporary barge loading infrastructure.

Barge-based removal may be an option for jetty material removal from GRSB. Estuary-based barges would transit the Estuary and then offload material at a temporary landing, probably in Jenner. Ocean-based barges may be able to offload at a port with existing infrastructure to handle barge mooring and unloading. Assuming a barge capacity of 2,000 CY, the total number of barge trips and averaged number of weekly barge trips are summarized in Table 9-5. Unless the material can be disposed of at the offload site, it would need to be loaded onto dump trucks and transported from the barge landing to the final disposal site. So, while Estuary-based barging would reduce

the traffic through GRSB, the material may need to be loaded on trucks in Jenner at the same use rates as summarized in Table 9-5.

9.2.2.2 Noise

Modifying the jetty would result in construction noise during demolition and removal. Construction equipment noise could include sounds such as vehicle engines, jackhammers, and generators. Because of its concrete cap and larger rock size, demolishing the groin (Alternatives 3 and 5) would likely entail more noise than the other project alternatives. One possible option for reducing noise during groin demolition may be the use of expansive grout, which requires drilling instead of jackhammering. The duration of the construction activity noise would roughly scale with the volume of demolition and jetty material removal.

Construction noise would primarily affect wildlife and recreational users of the beach. The noise would be heard beyond the physical footprint of the construction activity. Potential impacts to these biologic resources and recreational users are discussed in more detail in Section 9.2.2.4 and Section 9.2.2.5, respectively. In addition, construction noise may be heard by residents along the Estuary, particularly its western end.

9.2.2.3 Water Quality

Alternatives 2 through 5 would need to include construction best management practices (BMPs) for reducing impacts due to sedimentation and potential contamination due to accidental fuel or other spills. Construction activities would occur in the highly erosive beach environment, and control of sand and sediment within the construction area would require special attention to BMPs and their relationship to the tidal environment.

The alternatives vary in amount of material to remove from the beach (Table 8-1). While removing larger amount of material does increase the potential for water quality impacts, even the most extensive alternative, Alternative 5 (Remove Full Jetty), is anticipated to be within the range of normal construction activities whose water quality effects can be minimized with typical and appropriate BMPs. Potential BMPs would be identified as part of project design to control sand and sediment movement, isolate the work area, and minimize sedimentation. For example, installation of sheet pile may be necessary to isolate the work area; similarly, dewatering wells or pumps may be necessary to reduce groundwater levels. The level and complexity of BMPs would be greatest for Alternatives 3 and 5, which result in greater material removal.

9.2.2.4 Biological Resources

9.2.2.4.1 Fisheries

As discussed with respect to water quality, jetty removal alternatives would include construction BMPs for reducing impacts due to sedimentation and potential contamination due to accidental fuel or other spills that could have adverse effects on fisheries habitat either within the Estuary or within the surf zone. The potential for vibration caused by construction equipment to propagate into waters surrounding the site should be assessed and may induce schedule constraints to avoid important life cycle phases, such as migration. The alternatives vary in amount of material to remove from the beach (Table 8-1). While removing larger amount of material does increase the potential for water quality impacts, even the most extensive alternative, Alternative 5 (Remove Full Jetty), is anticipated to be within the range of normal construction activities whose water quality effects can be minimized with typical and appropriate BMPs and isolation of the work area from fisheries habitat.

9.2.2.4.2 Plants, Butterflies, and Birds

Tidestrom's lupine, a federal and state-listed endangered plant, grows on the dunes of GRSB. Other special-status plant, butterfly, and bird species may also use the habitat adjacent to the jetty project area and the construction access route (ESA, 2010). Alternatives 2 through 5 would require biological surveying to assess if special-status species are located in or near the construction area. In the event of finding special-status species, the location would be documented and jetty removal altered as necessary.

Tidestrom's lupine has been found in the vicinity and possibly on or immediately adjacent to the south end of the jetty access elements. Therefore both Alternatives 4 and 5, which propose removing the access elements, may impinge upon existing Tidestrom's lupine plants and possibly degrade their potential dune habitat. The potential for the access elements' removal to disrupt the plants' habitat has not been assessed. If a biological assessment identifies plant or habitat impacts from access element removal, then Alternatives 4 and 5 would probably need to be revised to reduce extent of access element removal. Reduced extent of access element removal would lessen Alternatives 4 and 5's effect on beach seepage and beach crest lowering. However, the decrease in Alternatives 4 and 5's performance relative to beach seepage and crest lowering is not likely to be a major detriment to their overall performance if their extent was reduced in just the southern portion of the access elements. As such, reducing their extent would probably be preferable to attempting to re-locate the lupine.

All the jetty removal alternatives, Alternatives 2 through 5, would include a significant construction equipment presence on the State Park roadways, parking lots, and out to the jetty. These unvegetated areas are already used by State Park and Water Agency

vehicles. However, all the modification alternatives have the potential for impacting terrestrial biological resources more than Alternative 1 (No Action). The alternatives vary in amount of material to remove from the beach (Table 8-1). While removing larger amount of material does increase the potential for terrestrial biological impacts, even the most extensive alternative, Alternative 5 (Remove Full Jetty), can likely be modified to limit impacts with typical and appropriate BMPs and isolation of the work area.

9.2.2.4.3 Pinnipeds

Harbor seals (and occasionally California sea lions and northern elephant seals) haul out year-round on the beach and within the Estuary. The daily pinniped visits are typically higher between December through July and peak in July during the molting season. The pupping season occurs between mid-March and the end of June, with birthing and care of seal pups occurring on the beach.

Construction activity on the beach would likely be prohibited during the pupping season, March 15th – June 30th. At other times of the year, the construction may disrupt pinniped haul out, particularly on the beach itself, since pinnipeds generally avoid close proximity with construction equipment and people. In addition to spatial proximity, vibrations from construction equipment may disturb the pinnipeds. Jetty modification would likely require a NMFS Incidental Harassment Authorization that describes specific conditions and operating procedures to minimize pinniped disturbance. The alternatives with larger volumes of material to remove from the beach would likely last longer and incur greater pinniped disturbance.

9.2.2.5 Aesthetics

As a Pacific Ocean beach included in the State Park system and visible from Highway 1, the jetty is situated within a high-value aesthetic setting. Parts of the jetty are visible to varying degrees both on the beach and from nearby overlooks. Construction equipment and activities to execute Alternatives 2 through 5 would be visible from several overlooks. The degree of aesthetic impact would increase with the duration and construction intensity of the alternatives. Based on the amount of material to be removed (Table 8-1), Alternative 2 would have the least aesthetic impact and Alternative 5 would have the greatest aesthetic impact, with Alternatives 3, 3a, and 4 having intermediate aesthetic impacts.

9.2.2.6 Recreation and Public Access

Goat Rock State Beach and the Russian River Estuary host the Russian River State Marine Recreational Management Area and part of the Russian River State Marine Conservation Area. In addition, the Greater Farallones National Marine Sanctuary overlaps with parts of GRSB and is approximately 100 ft offshore of the river mouth. Many of the same features that earn this area these designations attract recreation

visitors seeking activities such as beach access, boating, wildlife viewing, surfing, and camping.

Most of the recreational visitors to Goat Rock State Beach use the same access road and parking lots that would be used by construction equipment for Alternatives 2 through 5; this is the only beach access point that is maintained by CDPR. In addition, the construction area, to which recreation access would be restricted or prohibited, occupies about one third of the total length of the beach. Therefore, the jetty alternatives would be substantial reduction in recreational access to Goat Rock State Beach.

The duration of jetty demolition and removal, and therefore impacts to recreation, would roughly scale with the volume of jetty to be removed, which varies by alternative (Table 8-1). Notching the groin (Alternative 2) or demolishing the groin and leaving the resulting debris in place (an option of Alternative 3) would likely require lower construction effort. Removing the groin (Alternative 3) or the access elements (Alternative 4) would likely require moderate construction effort. Full removal of the jetty (Alternative 5) would likely require the largest construction effort. Closure of the Beach Parking Lot and restriction of beach access in the vicinity of the jetty removal area would be required during construction, resulting in substantial limitations to recreational access during construction.

The construction schedule and access could be developed with recreational use in mind to balance between measures to facilitate construction (e.g. leave equipment in staging areas closest to the beach) and measures to facilitate recreation (e.g. schedule construction for the off-season and stage equipment further from recreation access during weekends). Alternatives would need to consider tradeoffs between impacts. For example, the peak season for recreational use – summer and early fall – is the likely construction window since it avoids pinniped pupping season and the higher risk of winter storm waves overtopping the beach (Section 8.6.4). For alternatives with a considerably larger amount of material to remove, e.g. Alternative 3 (Remove Groin) and Alternative 5 (Remove Full Jetty), it may be harder to accommodate recreation and public access. Measures to reduce impacts on public access and safety might include signage, temporary fencing to secure unsafe areas, and limiting the size of active excavation by completing jetty removal and re-grading in a section before moving to the next section.

If Estuary-based barges were chosen for hauling jetty material, they would intermittently traverse the Estuary, potentially impacting recreational boating. If the boat launch at the CDPR Visitor Center was used for unloading the barges, this process would also likely interfere with recreation boating and possibly public access to the Visitor Center.

9.2.2.7 Cultural Resources

The jetty is more than 75 years old and, based on CDPR guidance, would likely require a historic resource assessment. It is not believed to currently be considered a historical resource at the national, state, or local level. Historic assessments typically consider a resource's association with historic events or association with important persons, either in terms of a person's importance within history generally or as the architecture or designer of the resource.

Excavation associated with the Alternatives 2 through 5 would occur in recently deposited and annually disturbed beach sands that have a very low potential to contain cultural materials. Changes in the annual water surface elevation on the Russian River would remain within previously recorded levels following any of the alternatives. There is a low potential for archaeological materials to be uncovered from any of the jetty modification alternatives.

9.2.3 Long-Term Impacts and Constraints After Jetty Removal

9.2.3.1 Flood Risk

As described in Section 7, properties adjacent to the Estuary already face flood risk from three possible scenarios. As such, Alternative 1 (No Action) consists of the baseline flood risk conditions. The study objectives (Section 1.2) are to assess the potential for Alternatives 2 through 5 to change the flood risk relative to the exiting flood risk under Alternative 1. The increase in flood risk due to sea level rise is discussed in Section 9.2.3.5.1.

9.2.3.1.1 Alternative 1 – No Action

For Alternative 1 (No Action), the three flooding scenarios and extreme water levels identified in Section 7.3 would continue to drive flood risk in the Estuary.

The first flood scenario (Section 7.2.1) occurs during closed inlet conditions when the beach berm blocks riverine outflow and water surface elevation rises in the Estuary. Seepage through the berm can reduce the rate of rise, but is unlikely to prevent continually increasing water levels except when inflows are limited to dry-year riverine discharge and no wave overtopping occurs. Heavy surf and berm overtopping can prevent construction equipment from safely accessing the beach and implementing an artificial breach. When an artificial breach cannot be implemented, the Estuary water can rise as high as the low point in the beach berm. In recent years, many of the self-breaches ending closed conditions have occurred at a low point in the berm just north of the groin (ESA PWA, 2015). This low point probably occurs because the minimum in nearshore wave energy is coincident with the groin (Section 4.2), the inlet is likely to

have recently scoured through the berm just north of the groin, and the groin's oblique angle to the beach reduces beach-building wave runup in its lee.

During the second flood scenario (Section 7.2.2), high fluvial discharge, the inlet can expand to include the entire beach between the groin and the bluffs. For this flood scenario, the flow conveyance area across the beach berm probably influences the water surface elevations in the Estuary.

The third flood scenario (Section 7.2.3) is caused by ocean waves passing through the open inlet and then breaking and running up on the Estuary shoreline. The size and location of the inlet are important factors in determining the amount of oceanic wave energy that could be transmitted into the Estuary and what portion of the Estuary shoreline would be exposed to the transmitted waves.

9.2.3.1.2 Alternative 2 – Notch Groin

Alternative 2 would probably not affect the closed inlet flood scenario. For these conditions, the notch in the groin would probably be filled with sand and not affect the formation of a low point elsewhere. We assume the notch would be filled with sand because of its likely location at the landward end of the groin, where the beach crest often forms, and because the notch would be directly exposed to wave runup from the south.

For the fluvial discharge flood scenario, the notch may provide a slight increase in conveyance area for discharge exiting the Estuary across the beach berm. For the reasons described in the preceding paragraph, the notch could be filled with sand during fluvial discharge and not provide any increased conveyance. Even if the notch is open during the fluvial discharge, the conveyance area of the notch would only be a small fraction of the total conveyance area across the beach and have minimal, if any effect, on Estuary water surface elevations. Anticipated notch management would probably require using construction equipment to close the notch and re-open the inlet north of the groin at the end of the management period. Locating the inlet north of the groin provides conveyance for high fluvial discharge. If construction equipment cannot access the beach due to wave overwash or impassable beach terrain, the inlet may not be re-located north of the groin, potentially increasing flood risk from fluvial discharge.

Because the notch's invert elevation would be above the tides, its capacity to transmit ocean wave energy into the Estuary is negligible. Therefore, the notch would not change the existing flood risk from this scenario.

9.2.3.1.3 Alternative 3 – Remove Groin

Groin removal would eliminate wave runup attenuation for the section of beach berm just to the north of the groin, which is often the berm's low point. As such, this alternative may cause an increase in flood level for the closed inlet flood scenario. The

low point in the berm crest may still occur just north of the groin's location because removal of the groin would not affect the minimum in nearshore wave energy and the remaining jetty access elements would probably still encourage the inlet to reside at this location. However, to increase flood levels for this scenario, the groin removal only needs to result in an increase in the low point's elevation, if not its location. Assuming that Alternative 3a (Degrade In Place) results in the groin materials submerged in the beach face, then Alternative 3a would have a similar effect on this flood scenario.

Because the groin is oblique to the beach and likely deflects fluvial discharge northward, removing the groin would likely enable the inlet to increase in width by several hundred feet during the second flood scenario, high fluvial discharge. A wider inlet during peak riverine discharge means that this alternative could lower Estuary water surface elevations for this flood scenario. For Alternative 3a (Degrade in Place), the groin materials are anticipated to remain just below the surface of the beach face. In this position, they would become exposed when the inlet migrated further south. Unless the degradation and dispersal was very extensive, considering the size and volume of groin materials, Alternative 3a might not enable a significant increase in the inlet’s width for increasing fluvial flood conveyance.

While a wider inlet would convey fluvial discharge more efficiently from the Estuary, it would also transmit ocean wave energy to the Estuary more efficiently. As a result, groin removal may increase the flood potential for the flood scenario. In addition to groin removal allowing the inlet to expand further south, the groin, which is angled across the inlet, would no longer block wave energy from propagating into the Estuary.

Since the flood assessment suggests that the closed inlet and ocean wave scenarios may result in greater flood stages (Table 7-5), this alternative’s potential benefit to fluvial flooding is likely offset by the potential increased flood risk via the other two scenarios. As such, the alternative may increase overall flood risk in the Estuary.

9.2.3.1.4 Alternative 4 – Remove Access Elements

As described in Section 9.1.2.4, removing the access elements is likely to result both a lower beach berm crest south of the groin and increased seepage through the beach berm.

Since the low points on the south berm are not likely to change with access element removal, the closed inlet flooding would still be controlled by the low point north of the groin. Therefore, Alternative 4 would not significantly change the peak water surface elevation during the closed inlet flooding scenario as compared to Alternative 1 (No Action).

Removing the access elements would likely increase seepage through the beach berm. Increased seepage would slow the rate of rise of Estuary water surface elevations during a closure, potentially providing more time to artificially breach the berm and minimize flooding. Assessments of the effect of increased seepage on number of days closed (Section 9.1.3.2) predicts an increase in closure duration of about 10% or about 1-3 days per closure. Periods of high waves which preclude beach access may last as long as a week. In some instances the area north of the groin has been inaccessible for several weeks because of steep banks created by the inlet and the groin (ESA PWA, 2015). Since the increase in closure duration with increased seepage is less than the period which the berm may be inaccessible, the increased seepage is not likely to affect the flood risk from this scenario.

Because removing the access elements affects only the area south of the groin and the fluvial and ocean wave flood scenarios are directly related to the inlet, so long as the inlet stays north of the groin, then neither the fluvial nor the ocean wave flood scenario would change due to Alternative 4.

Since this alternative is not anticipated to change any of the flood scenarios, it is presumed to preserve the existing level of flood risk. The only exception would be the unlikely case in which access element removal facilitated the inlet to breach south of the groin. This breaching could create conditions more favorable for transmission of ocean waves into the Estuary.

9.2.3.1.5 Alternative 5 – Remove Full Jetty

Since Alternative 5 combines Alternative 3 and Alternative 4, it would likely have similar impact on flooding as merging those two Alternatives. Namely:

- **Closed inlet flood scenario** – without the groin, the low point on the beach crest may form higher, which could mean that the peak water level that could occur in the Estuary before breaching would be higher
- **Fluvial flood scenario** – Removing the groin and the access elements would give the inlet unfettered capacity to scour a larger opening during fluvial flooding. This larger opening could improve conveyance from the Estuary to the ocean, potentially reducing Estuary water levels.
- **Ocean wave scenario** - With a greater range of mobility, particularly of the east end of the inlet channel, which would be limited by just Alternative 3, there is a greater likelihood that the inlet would shift to a location and dimensions which permits ocean waves to transit the inlet and runup on the Estuary shoreline in areas with infrastructure. In addition, the groin would no longer protrude at an oblique angle from the beach, providing wave attenuation as waves approach the inlet.

Since the flood assessment suggests that the closed inlet and ocean wave scenarios may result in greater flood stages (Table 7-5), this alternative's potential benefit to fluvial

flooding is likely offset by the potential increased flood risk via the other two scenarios. As such, the alternative may increase overall flood risk in the Estuary.

If unanticipated breaching occurred south of groin for any of the other alternatives, as described in Section 9.1.2.6, then the change in flood risk for these conditions would start to approach the flood conditions described for Alternative 5, particularly as the remaining jetty elements are eroded.

9.2.3.2 Biological Resources

Removal of some or all the jetty may increase the duration of perched and freshwater lagoon conditions, as proposed by the Biological Opinion (NFMS, 2008) and described in Section 9.1.3. It may result in wetland habitat shifts related to changing inundation durations, but the net change is unlikely to be significant to the extent of vegetation communities or special-status species that may use these communities (ESA, 2010). As such, overall effects related to increased perched and freshwater lagoon conditions are not anticipated to be substantially different than those identified for Alternative 1. A discussion of potential variations in these effects related to Alternatives 2 through 5 for specific biological resources is provided below.

9.2.3.2.1 Fisheries

Inlet closures, and the associated freshwater conditions, are hypothesized to improve ecological function for juvenile salmonids, particularly steelhead, by increasing prey availability and refuge (NMFS, 2008). As described above in Section 9.1.3, the changes in water quality toward the fresher lagoon conditions are anticipated to be relatively minor for Alternatives 3 through 5 as compared to Alternative 1 (No Action). For Alternative 2 (Notch Jetty), a more substantial increase in the number of closure days is anticipated. Based on this analysis, Alternative 2 would provide the greatest benefit to salmonid habitat in the Estuary.

The end of the lagoon management period, October 15th, is dictated in part by the onset of peak salmonid migration and the need for an open inlet to enable this migration. All but Alternative 2 (Notch Groin) are not expected to alter fall fish passage as currently implemented. Alternative 2 may alter existing fish passage if the Estuary has an outlet channel flowing over the notch after October 15th. The notch is presumed to act as a grade control, the lower limit for target lagoon water surface elevations. Just downstream of the notch, the outlet channel would traverse the beach sand. Particularly if the outlet channel has been in place for a month or more, the channel in the beach sand may be scoured below the elevation of the notch. This rapid drop in the channel bed elevation from the notch to the sand channel could create a fish passage obstruction, particularly for ocean-to-river migrants. To eliminate this obstruction could require closing the outlet channel using construction equipment to move sand into the

notch. Once the outlet channel was closed, a new tidal inlet would need to be opened north of the groin.

9.2.3.2.2 Pinnipeds

Alternative 2's success with regard to increasing Estuary water surface elevations would rely upon the notch and outlet channel helping to sustain closures that starting in the first months of the management period (Figure 9-3). Beach management, in the form of excavating sand from the notch and a channel leading to the notch, would probably be required to achieve these sustained closures. The Estuary water surface elevation modeling assumes that this beach management could occur as soon as hydraulic conditions were appropriate. However, from May 15th to June 30th, seal pupping on the beach would likely constrain beach access for such an excavation. This beach access constraint would decrease the chance of activating an outlet channel through the notch before the Estuary self-breaches. Since early management period conditions to activate flow through the notch were predicted to occur only once every three years (Figure 9-2), missing one year's opportunity could diminish the predicted increase in perched water levels predicted for Alternative 2 (Figure 9-4).

Alternatives 3 through 5 would likely result in only small changes to the duration of lagoon conditions under Alternative 1 (No Action). Alternative 2 (Notch Groin) is predicted to increase annual closure time by approximately 18% and during the management period, increase closure time by approximately 37%. An increase in the duration of perched lagoon conditions could inundate river haul outs more often, seasonally reducing their availability. Alternative 2 would create a discharge channel during overflow conditions, and could alter pinniped haul out opportunities, either positively or negatively, by providing haul out access along the channel on the ocean side of the jetty structure. However, it is unlikely that notching would afford access to the Estuary (see discussion of fish passage below). Alternatives 3 and 3a would alter inlet geometry, resulting in changes to both the eastern and western ends of the inlet channel location; but in general these alternatives would continue to maintain the berm dynamics associated with current conditions.

Alternative 4 would be anticipated to reduce the height of the beach berm, and redistribute sand from the top into more extended splays on the backside of the berm, into the Estuary. This could provide increased haul out opportunity within the Estuary, but access would be more difficult since harbor seals would need to move further overland, as compared to water access during open inlet conditions. Alternative 5 would restore the beach and inlet morphologic conditions that existed before the levee, and southward migration of the inlet up to 600 feet south of the groin would be anticipated. The southward migration of the inlet would alter, and could potentially reduce, the extent of haul out opportunities currently provided on the Estuary side of the berm. These changes in haul out opportunities may not result in a net loss, and could be made

up by sand deposition on the northern side of the channel, depending upon channel dynamics.

9.2.3.3 Aesthetics

As a Pacific Ocean beach included in the State Park system and visible from Highway 1, the jetty is situated within a high-value aesthetic setting. Parts of the jetty are visible to varying degrees both on the beach and from nearby overlooks. The jetty's visibility varies, depending on the coverage provided by shifting beach sands. Since Alternatives 3 through 5 involve removing some or all of the jetty, an unnatural feature in a striking natural setting, they would probably all be considered as improving the site aesthetics to varying degrees as compared to no action (Alternative 1). The groin is the most notable visual feature, because of its exposed surface area, proximity to the surf zone, and incongruous linearity in a natural setting. So its removal (Alternatives 3 and 5) would provide the biggest aesthetic improvement. The exposed portions of the access are more disperse and have a sinuous alignment that blends more with the beach morphology, so have less visual impact on the beach. For visitors walking on the beach, the access elements are readily apparent, so their removal (Alternatives 4 and 5) would provide some benefit. Notching the groin (Alternative 2) might remove some of the groin that is visible at times, so would have some smaller, local improvements on visual aesthetics when the inlet is not flowing through the notch and the notch is filled with sand. However, if the inlet was flowing through the notch or had recently been flowing through the notch, leaving the notch exposed, the notch would expose a portion of the groin typically buried by sand and may appear as a discontinuity in the beach that highlights both its presence and the presence of the groin.

9.2.3.4 Recreation and Public Access

The jetty alternatives would likely impact recreation. Namely, increased inlet closure could reduce the occurrence of an offshore sandbar which yields favorable surfing conditions. The higher Estuary water surface elevations would also increase inundation of recreational beach area within the Estuary.

Alternatives 2 through 5 may alter public access. When the open inlet intersects the beach berm as either a tidal channel or an overflow channel, the flowing water inhibits most visitors from crossing from the beach on one side of the channel to the beach on the other side of the channel.

Removing some or all of the jetty would enable the inlet to migrate further south, which would reduce the extent of beach that is accessible from the south when the inlet is open. Because of road access and parking, most visitors access the beach from the south. For Alternative 2 and Alternative 3, it is anticipated that the remaining jetty elements prevent the east end of the channel from migrating further south. However,

the western end of the channel may migrate up to approximately 250 feet (Alternative 2) or 400 feet (Alternative 3) further south than the groin. Because Alternative 4 preserves the groin, we anticipate that the inlet stays north of the groin in its existing range, i.e. Alternative 1. Removing the entire jetty (Alternative 5) would probably allow the inlet to migrate further south. As compared to the southern limit of the current inlet at the groin, Alternative 5 may permit the east end of the inlet to migrate as much as 600 feet further south and the west end could possibly migrate about 1,000 feet further south (Section 9.1.2.5.1). This extent of southern migration would cut through the sandy beach berm and stay north of the Beach Parking Lot. While beach access from the south would potentially be reduced by an inlet further south under Alternatives 2, 3, and 5, there would be a commensurate increase in accessible beach north of the inlet. However, no formal access to the beach on the north side is provided or maintained by CDPR, and access is more difficult since it requires descent down a narrow footpath.

Waves in the beach's surf zone pose an inherent risk of drowning to beach visitors. The existing groin may exacerbate this risk by providing a seemingly safe pathway for approaching the surf zone that can be suddenly overtopped by large waves capable of knocking visitors down and sweeping them into the surf. Groin removal (Alternatives 3 and 5) would eliminate this potentially dangerous pathway for recreational visitors approaching the surf zone. Alternative 3a (Degrade in Place) would leave rocks that could be exposed and present a potential risk, depending on beach erosion and deposition around the rocks.

Notching the groin (Alternative 2) could create a hazard of swift water as outflow spills over the notch. Although the existing inlet already creates a swift water hazard, the hazard potentially created by the notch could be different for several reasons. Flow of the notch may be faster since the groin can resist higher water velocities, it may give the appearance of a safer approach since it would be bracketed by the groin, and the notch would be a constructed management feature instead of a naturally-formed channel.

9.2.3.5 Potential Implications of Sea Level Rise

The analyses in the preceding sections assumed one of Alternatives 2 through 5 was in place and had a chance to re-equilibrate before sea level rise reaches substantial enough levels to cause changes to GRSB water levels and morphology. The sections below consider the potential for sea level rise to modify flood risk, beach and inlet morphology, beach permeability, and Estuary water surface elevation.

Jetty degradation is not considered in the sections below. While further degradation of the jetty is likely over the multiple decades of sea level rise's time frame, predicting degradation rate would be challenging since the structural state of the jetty is not well characterized and degradation would likely be in response to rare extreme fluvial flooding and ocean waves. Degradation would tend to shift conditions toward jetty

removal. By assuming no degradation, the assessment considers the greatest possible role for the jetty component(s) that would remain after construction of an alternative.

As discussed in Section 5.4, the scenarios considered are 1 ft, 3 ft, and 5 ft of sea level rise. According to a federal study adopted by California (NRC, 2012), projected decadal ranges for these scenarios to occur are: 1 ft of sea level rise between 2040 and 2060, 3 ft of sea level rise between 2070 and 2110, and 5 ft of sea level rise as early as 2090.

9.2.3.5.1 Flood Risk

Sea level rise would increase the flood risk in the Estuary. The current operational range for lagoon water surface elevations is between 5 ft and 9 ft NGVD, with the low end of the range being just above the ocean tide range and the high end of the range being just below inundation of low-lying properties around the Estuary. By shifting the ocean tide range upward, sea level rise would reduce this range from the bottom.

For this study, the existing flood stage is assumed to stay at 9 ft NGVD. As another part of fulfilling the Biological Opinion's Estuary Management RPA, the feasibility of reducing flood risk at 9 ft is being assessed. Also, the County of Sonoma Permit and Resource Management Department is planning to develop a Community Adaptation Plan for Jenner under the County's Local Coastal Program. If these efforts identify and help implement options for reducing flood risk for water surface elevations of 9 ft NGVD, then the change of flood risk with sea level rise as it relates to the jetty could be re-considered.

For the most common conditions in the Estuary, when the inlet is open and water surface elevations are set primarily by the fluctuating tides, high tides would increase with sea level rise. So, with 3 ft of sea level rise, the high tides would reach to over 6 ft NGVD every day, which currently is a water surface elevation that occurs only after inlet closure. The higher tide range for open conditions means that when inlet closures occur, the initial water surface elevation at the start of the closure would be higher. This higher initial water surface elevation would increase flood risk since the time for the water surface to rise from its initial closure elevation to 9 ft NGVD would be less, reducing the opportunity for artificial breaching when it is needed.

Since the beach berm is anticipated to shift upwards to pace sea level rise (Section 5.4), and since the low point of the beach berm sets the Estuary water surface elevation for the closed inlet flooding scenario (Section 7.2.1), sea level rise would also increase flood risk from this scenario. The other two extreme flood scenarios, fluvial flooding (Section 7.2.2) and ocean wave transmission (Section 7.2.2), would also worsen with sea level rise. Both of these flood scenarios add additional height to Estuary water surface elevations; by increasing Estuary water surface elevations, sea level rise would cause a comparable increase in peak flood levels.

As a first order assessment appropriate to this feasibility study, the overall increase in flood risk from all three flood scenarios is expected to augment Alternatives 2 through 5 in a similar manner. The increased risk occurs because of higher ambient water levels and therefore is not that sensitive to the presence or absence of the jetty elements. The variations in flood risk as a result of the alternatives that is described in Section 9.2.3.1 would persist.

9.2.3.5.2 Beach and Inlet Morphology

As described in Section 5.4, the beach is anticipated to shift upward and landward in response to sea level rise. Much of the jetty access elements protrude approximate three feet above beach. So with the onset of 3 ft of sea level rise and the corresponding upward shift of the berm crest, the access elements would become almost completely buried within the beach berm. Even with 3 ft of sea level rise, there would still be a limited chance that episodic erosion reveals the access elements. However, they would play a decreasing role in beach morphology, and the access elements' contribution to stabilizing and elevating the beach crest elevation would be substantially diminished. Therefore, at about 3 ft of sea level rise, the difference that Alternative 4 (Remove Access Elements) would have on the beach crest elevation relative to Alternative 1 (No Action) would be largely diminished. Alternative 5 (Remove Full Jetty) would still have a lower beach crest elevation as compared to Alternative 1 since the removal of the groin would enable greater southward migration of the inlet, which would periodically reset the beach crest elevation.

Similar berm crest elevations between Alternative 1 and Alternative 4 would mean that these alternatives experience a similar amount of wave overwash, which is currently thought to be impeded by the protruding access elements. As demonstrated by the QCM sensitivity analysis (Section 9.1.3.), increased overwash is anticipated to decrease closure duration by raising water surface elevations more rapidly. Wave overwash also brings saline ocean water into the Estuary, which counters the Estuary freshening that is preferred ecological conditions during management period closures.

In concert with this sea level rise shift in the beach berm, the inlet would also be anticipated to shift upwards and landwards with the beach berm. The exception would be the outlet channel created by flow through the notch in Alternative 2, which is discussed in more detail below. In all other cases besides notch outflow, the anticipated relative differences in inlet closure between alternatives would be similar to what was previously described in Section 9.1.2. As described in Section 9.1.2, the relative difference in number of days closed and the percent exceedance of Estuary water surface are comparable for Alternatives 1, 3, 4, and 5. All of these alternatives would probably experience some changes in inlet closure with sea level rise, but these changes are anticipated to affect these alternatives similarly. For example, the tide range will occur in a higher portion of the Estuary's topography and this portion will have a greater tidal prism. Therefore, in some cases where deposition would have just overcome tidal

scour to cause a closure, the slightly larger tidal prism may be able to keep the inlet open whereas before sea level rise, the inlet would have closed. Since all the alternatives would experience nearly the same increase in tidal prism, the differences between them would likely stay relatively minor and unchanged.

Since Alternative 2's notch would be excavated from the armored groin, sea level rise would cause the notch's elevation to decrease relative to the tide frame and perched lagoon conditions. For this study, the notch's elevation was assumed to be 5.5 ft NGVD. This elevation allows the notch to support an outlet channel before the Estuary water surface elevations would reach the low point of the beach berm during some closures.

With one foot of sea level rise, the notch could just support water surface elevations perched above the tides. However, since the notch would be lower in the tide range, it would be more susceptible to closure from ocean waves and be in need of more beach management. Presumably, if construction equipment access was available, the notch would be cleared of sand after a closure and continue to support lagoon conditions. Sand would tend to fill the notch to greater depths in response to sea level rise, thereby increasing the needed excavation. With three feet or more of sea level rise, the notch elevation would be below the ocean's high tides, so at most it could facilitate muted tidal conditions in the Estuary. It would no longer be high enough to support an outlet channel that perches above tides and prevents tides and salinity from entering the Estuary.

To preserve its purpose of perching Estuary water surface elevations above the tides, the notch would need to be raised to pace sea level rise. While 5.5 ft NGVD was selected as the notch elevation for purposes of this feasibility study, variations on the notch's elevation could be studied to inform any subsequent planning. These assessments could look at the trade-offs between:

- Constructing the notch for current sea level, with accommodations to adapt to sea level rise by raising the notch, probably by adding rock and concrete back into the notch to raise its elevation.
- Initially constructing the notch at a higher elevation than 5.5 ft NGVD. This would potentially cause the notch to miss some closures in the short term because the Estuary self-breaches over a lower portion of the beach berm, but extend the notch's initial utility for a greater increase in sea level rise.

9.2.3.5.3 Beach Permeability

As discussed in Section 9.1.1, removing jetty elements would likely increase beach permeability. The biggest permeability gain would come from removing the access elements and a smaller gain from removing the groin. Beach permeability is a property of the subsurface beach berm and any embedded structures, so permeability itself would not be affected by sea level rise. However, assuming that water surface elevations in the Estuary continue to be managed for a flood stage of 9 ft NGVD, then

seepage rates through beach berm during closed conditions would decrease with sea level rise. The seepage rates would decrease because while the Estuary water levels would be managed within a similar range, the ocean water levels would increase. Therefore, the hydraulic gradient which causes seepage between the Estuary and the ocean would be less, resulting in less seepage.

9.2.3.5.4 Estuary Water Surface Elevations

As described above when considering flood risk, sea level rise would result in higher initial water surface elevations at the start of closures. Since the duration of closures is generally set by the time that it takes for water surface elevations to reach flood stage, the higher initial water surface elevation would mean less time before either a self-breach occurs or an artificial breach would be required. As such, all the alternatives would likely see a decrease in the average annual number of closure days.

If the notch elevation of Alternative 2 was raised to pace sea level rise, then this alternative may be less sensitive to sea level rise decreasing the number of closure days, at least during the management period when the notch is used to support an outlet channel. The QCM indicates that during the dry season, the outlet channel across the notch is relatively successful at maintaining water levels below flood stage for extended periods.

9.3 Estimate of Probable Costs

To enable cost comparison of alternatives for planning purposes, the cost estimates presented in Table 9-6 were developed. These cost estimates are intended to provide an approximation of total project costs appropriate for the conceptual level of design, primarily for comparing relative costs between the alternatives. These cost estimates are considered to be approximately -50% to +100% accurate, and include a 40% contingency to account for project uncertainties (such as final design, permitting restrictions and bidding climate). These estimates are subject to refinement and revisions as the design is developed in future stages of the project. Note that the accuracy of the cost estimate (-50% to +100%) is for the extent of construction described in Section 8. According to the AACE International cost estimate classification system (AACE, 2005), this is a Class 4 cost estimate, which is characteristic of a 1% to 15% level of project definition and suitable for a feasibility study such as this one.

The calculated cost estimate for groin removal (in Alternatives 3 and 5) is a similar order of magnitude for other, more detailed cost estimates and bids for groin construction and removal projects at other locations.

9.3.1 Quantities

Quantities of jetty material were derived from estimates of the jetty's dimensions from surface observations, historic accounts and plans of jetty construction (Section 2), and geophysical remote sensing (Section 3). However, since much of the jetty remains buried, additional subsurface exploration should be considered if further refinements to cost estimates are planned.

Once jetty materials were excavated, their volume was assumed to bulk up by 40% as the materials are disturbed from in-place volume to unorganized piles for hauling.

Estimates of the volume of overburden, the volume of beach sand excavated and used to safely access buried jetty, were made separately for the access elements and the groin.

The access elements were assumed to need removal of overburden to create an open trench with side slopes that do not need additional excavation support. Depth of the rock wall is not known, but was assumed to be 15 ft below the ground surface. For most of the beach berm, a trench of this depth would be mostly above the water table (presumed to be approximately mean sea level), so that de-watering would only be required for the lowest portion of the excavation. De-watering and other water management is estimated at 2% of the construction cost. Assuming construction occurs during July through October, the risk of wave overtopping the beach and entering the trench is low, but increasing in September and October. To further reduce the trench exposure and to minimize the safety hazard of the trench may present to the public, access elements removal would likely be completed in sections so the extent of open excavation is limited to several hundred feet at a time. This approach may also facilitate material handling as newly excavated sand can be used to fill areas from which access elements have been removed.

Because of the groin's depth and location within or proximal to the surf zone, its removal is assumed to require an encircling sheet pile structure. The sheet piles would provide excavation support, sheltering from waves, and enable de-watering. Much of the excavation would be below MLLW and require pumping to de-water.

9.3.2 Unit costs

Unit cost estimates were made for demolition, excavation, material sorting, beach hauling, road hauling, disposal fees, and re-grading. The estimated unit costs were derived from ESA's experience with similar projects. Higher excavation unit costs were assumed for the seaward half of the groin since access is limited by the surf zone for this part of the groin.

The disposal costs of removed jetty material are a function of the hauling distance and the disposal fee (or 'tipping fee'). All material was assumed to be hauled a one-way distance of 32 miles, the distance from the project site to the County landfill outside of Petaluma. This landfill would be the disposal site if no other options are available. It also means the cost includes hauling as far as Santa Rosa for re-use. If the bulk refuse disposal fee at the county landfill was used as for cost estimating, disposal costs could be an order of magnitude higher. However, this high cost of landfill disposal, as well as the reuse potential of much of the jetty material, are assumed to motivate the identification of a disposal site that charges the lower disposal fee of $12/cy used in this cost estimate. Disposal of rock, the largest fraction of the jetty, dominates the disposal costs. The rock is thought to come from Goat Rock and other nearby locations. Goat Rock is comprised of greywacke, durable sandstone that is typically suitable for construction.

9.3.3 Additional assumptions

In addition to the quantity and unit cost assumptions described above, the following assumptions were also made for cost estimating:

- Mobilization and de-mobilization is assumed to be 3% of other construction costs.
- Site preparation such as temporary access and staging, is assumed to be 3% of other construction costs.
- Traffic control and site public safety is assumed to be 2% of other construction costs.
- All hauling is assumed to be by truck; the option for barge-based hauling was not considered in the cost estimates.
- Alternative 2's notch was assumed to be 100 ft long through the entire cross section of the groin. Since it would be excavated primarily from the top portion of the groin, it was assumed that 80% of the volume removed would need to be demolished to reduce the material to sizes suitable for excavation and hauling.
- For Alternatives 3 and 5, 20% of the groin's volume was assumed to require demolition prior to excavation
- Alternative 3a assumed demolition and dispersal with exaction equipment for the portion of the groin above MLLW.
- For all alternatives besides Alternative 3a (Degrade In Place), a construction contingency of 20% of construction cost was used. Since the extent and methods of dispersal for Alternative 3a are still uncertain, a higher contingency of 40% was used for this alternative.
- The environmental compliance and permitting costs were based on cost estimates for the Russian River Estuary Management Project provided by Water Agency staff.

- Mitigation costs, such as those due to environmental, recreation, or public access impacts, are not included.
- Annual operations and maintenance costs for Alternative 1 are based on costs estimates for recent years provided by Water Agency staff. Since Alternative 2 is anticipated to require more beach management activity to open, maintain, and close the notch, the operations and maintenance costs for this alternative were assumed to be twice those of Alternative 1. For Alternatives 3 through 5, which see little change in number of days closed and are presumed to continue beach management, the operations and maintenance is assumed to be the same as Alternative 1.

Table 9-6. Estimate of probable costs. These cost estimates are intended to provide an approximation of total project costs appropriate for the conceptual level of design, primarily for comparing relative costs between the alternatives.

Project component	Alt 1 No Action	Alt 2 Notch Groin	Alt 3 Remove Groin	Alt 3a Degrade In Place	Alt 4 Remove Access Elements	Alt 5 Remove Full Jetty
Construction	$0	$300,000	$10,164,000	$524,000	$1,922,000	$12,110,000
Construction Contingency	$0	$60,000	$2,033,000	$209,000	$384,000	$2,422,000
Subtotal Construction	$0	$360,000	$12,197,000	$733,000	$2,306,000	$14,532,000
Design	$0	$200,000	$400,000	$200,000	$300,000	$500,000
Environmental Compliance & Permitting*	$0	$1,000,000	$1,000,000	$1,000,000	$1,000,000	$1,000,000
Project Management	$0	$50,000	$245,000	$50,000	$50,000	$290,000
Construction Admin./Inspection	$0	$25,000	$245,000	$25,000	$45,000	$290,000
Project Contingency	$0	$25,000	$1,220,000	$75,000	$230,000	$1,455,000
ESTIMATED PROJECT COST	**$0**	**$1,670,000**	**$15,300,000**	**$2,100,000**	**$3,900,000**	**$18,100,000**
Annual operations and maintenance	**$300,000**	**$600,000**	**$300,000**	**$300,000**	**$300,000**	**$300,000**

* Does not include mitigation or environmental monitoring costs.

9.4 Summary of Alternatives Evaluation

Below are summaries of key findings for each of the alternatives. These key findings, as well as additional other findings and consideration of uncertainty, are discussed in more detail in the preceding sections and summarized in more detail in the following sections and tables.

Alternative 1- No action

- Under this alternative, the jetty's effects on beach permeability, beach morphology, and inlet morphology would remain unchanged. Therefore, Estuary water surface elevations and water quality would continue in similar manner to existing conditions. Beach management to artificially breach (for flood risk minimization) and to facilitate lagoon conditions (for salmonid rearing habitat, when feasible) would also continue at similar frequency and intensity as existing conditions.
- Although the artificial fill that forms the Goat Rock Parking Lot appears to have impeded littoral sediment transport and caused GRSB to widen considerably, the present study does not consider removal of this feature. If the removal of this fill were to be considered, perhaps as part of a regional beach restoration effort, this study's assessment of existing conditions, opportunities and constraints, and alternatives would potentially change.
- With sea level rise, flood risk will increase since tidal water surface elevations will be closer to structures on low-lying properties adjacent to the Estuary. As such, water surface elevations at the time of closure will be closer to flood stage, leaving less time for beach management that may be needed to artificially breach. Increase in sea level rise will also increase flood risk from fluvial and ocean wave transmission scenarios.
- The beach and inlet will shift upwards and landwards in response to sea level rise. This would bury the jetty access elements, thereby reducing their stabilizing influence on the beach berm. If Estuary water levels are managed within the current range, then the hydraulic gradient between the Estuary and the rising ocean would be reduced, potentially reducing seepage rates. Otherwise, other processes by which the jetty affects Estuary water surface elevations are likely to be similar to existing conditions.
- This alternative would incur no additional costs. Annual operation and maintenance costs are estimated to be $300,000, based on current expenditures.

Alternative 2 - Notch groin

- Notching the groin would entail the lowest level of effort to implement as compared to all of the alternatives except Alternative 1. This lowest level of effort implies reduced impacts and constraints on traffic due to hauling, noise and vibration due to construction equipment, and limited public access due to equipment staging and safety measures.
- In addition to removing material from the jetty, this alternative may need to add additional armor to the beach to reduce chance of scour where the channel transitions from sand to the notch.
- During inlet closure or outlet channel conditions, this alternative would not change the seepage through the beach berm as compared to Alternative 1 (No Action).
- Alternative 2 would probably enable the inlet to migrate up to 250 ft south of the groin on the inlet's west end. The remaining jetty would probably hold the inlet's east end to its current extents.
- The notch may facilitate formation of an outlet channel, which would increase outflow from the Estuary during lagoon conditions. To create and sustain the notch would likely require additional beach management activity as compared to Alternative 1 (No Action).
- If the notch could be successfully created whenever conditions were appropriate, then this alternative is predicted to extend lagoon conditions. Extended lagoon conditions are predicted for this alternative particularly in the early part of the management period (June through August) and at a frequency of approximately one out of every three years. Over the long-term, the average duration of perched lagoon conditions is predicted to increase by 18% annually and by 37% during the management period. This increased duration of perched conditions would also increase fresh water conditions and inundation extent.
- The predictions of increased lagoon conditions assume as-needed beach management to create and sustain an outlet through the notch. However, beach and surf conditions already impede existing beach management and constraining the outlet channel alignment to include the notch would reduce operational flexibility to adapt to these conditions. Beach management for this alternative would likely face additional constraints because of the potential hazard of operating construction equipment adjacent to the hard, buried structure of the groin. In addition, during the early part of management period, when this alternative offers the largest potential to increase water surface elevations for

the remainder of the salmonid rearing season, there would likely be beach management constraints due to the harbor seal pupping season.

- The outlet channel through the notch may impair fish passage and increase fluvial flood risk. As such, it would require beach management to close the outlet channel at the end of the management period and to re-open the inlet north of the groin. Closing estuary outflow using excavation equipment could pose new management challenges due to the instability of saturated beach sands. This alternative could increase the risk due to fluvial flooding if management crews are unable to access the beach to close the notch channel and breach the estuary north of the groin due to hazardous beach conditions or permitting constraints.
- Natural beach aesthetics would be degraded because the notch would create a water control structure on the beach. Outlet channel flow would expose a portion of the groin typically buried by sand.
- The increased southward migration would reduce the length of beach that could be accessed by the public from the south by up to 250 ft. The notch could also create a swift water flow hazard accessible by the public.
- As a hard structure, the notch would need to be raised to adapt to sea level rise.
- This alternative would have an estimated probable project cost of $1,670,000 with possible range of -50% to +100% of this estimate. Annual operation and maintenance costs are estimated to be $600,000, based on anticipated increases to beach management.

Alternative 3 - Remove groin

- Removing the groin would entail a higher level of effort as compared to the all of the alternatives except Alternative 5. This higher level of effort implies associated impacts and constraints on traffic due to hauling, noise and vibration due to construction equipment, and limited public access due to equipment staging and safety measures.
- An option for Alternative 3, Alternative 3a (Degrade in Place) would demolish and disperse the groin, but not haul any of the groin materials off the beach. This would lessen the construction impacts for noise, vibration, and public access and eliminate the hauling impacts on traffic. Alternative 3a would progress towards Alternative 3's conditions, with the rate of response dependent on the rate of natural and mechanical dispersal of the groin materials.

- During inlet closure or outlet channel conditions, removing the groin would likely increase total seepage through the beach berm by about 20% as compared to Alternative 1 (No Action).
- Alternative 3 would probably enable the inlet to migrate up to 400 ft south of the groin on the inlet's west end. The remaining access elements would probably hold the inlet's east end to its current extents.
- The increase in seepage outflow and increase in inlet closure due to additional southward migration are predicted to cause relatively small changes in the Estuary's number of days closed and frequency of perched Estuary water surface elevations as compared to Alternative 1.
- Alternative 3 would result in beach management needs that are similar to Alternative 1.
- In the long-term, Alternative 3 may increase potential flood risk for the closed inlet and ocean wave flood scenarios and may decrease potential flood risk for the fluvial flood scenario.
- Natural beach aesthetics would improve because of the removal of a portion the jetty structure, the groin.
- The increased southward migration would reduce the length of beach that could be accessed by the public from the south by up to 400 ft. Removing the groin would eliminate the public safety hazard posed by the groin encouraging the public to walk close to the surf zone.
- Changes due to sea level rise would be similar as compared to Alternative 1 (No Action), with the beach and inlet anticipated to shift upward and landward.
- This alternative would have an estimated probable project cost of $15,300,000 with possible range of -50% to +100% of this estimate. Alternative 3a (Degrade In Place) would have an estimated probable project cost of $2,100,000 with possible range of -50% to +100% of this estimate. Annual operation and maintenance costs are estimated to be $300,000, based on anticipated similarity to existing conditions.

Alternative 4 - Remove access elements

- Removing the access elements would entail a higher level of effort as compared to the less intensive alternatives, e.g. Alternatives 2 and 3a. This higher level of effort implies associated impacts and constraints on traffic due to hauling, noise and vibration due to construction equipment, and limited public access due to equipment staging and safety measures. Removal of a portion of the south end

of the access elements may be constrained by the special status plant, Tidestrom's lupine.

- During inlet closure or outlet channel conditions, removing the access elements would likely increase total seepage through the beach berm by about 40% as compared to Alternative 1 (No Action).
- Removing the access elements, which stabilize the beach berm south of the groin, would result in lower beach crest elevations and a corresponding increase in wave overwash of salty ocean water into Estuary.
- The increase in seepage outflow and increase in wave overwash inflow offset each other, limiting the change in net inflow to the Estuary. Overall, the Estuary is predicted to have slightly fewer days closed and similar frequency of perched Estuary water surface elevations as Alternative 1.
- Alternative 4 would result in beach management needs that are similar to Alternative 1.
- Natural beach aesthetics would improve because of the removal of a portion the jetty structure, the access elements.
- Changes due to sea level rise would be similar to Alternative 1 (No Action), with the beach and inlet anticipated to shift upward and landward. This alternative would continue to provide improved seepage, which may partially offset the decrease in hydraulic gradient under Alternative 1.
- This alternative would have an estimated probable project cost of $3,900,000 with possible range of -50% to +100% of this estimate. Annual operation and maintenance costs are estimated to be $300,000, based on anticipated similarity to existing conditions.

Alternative 5 - Remove full jetty

- Removing the full jetty would entail the highest level of effort as compared to all the other alternatives. This highest level of effort implies associated impacts and constraints on traffic due to hauling, noise and vibration due to construction equipment, and limited public access due to equipment staging and safety measures. Removal of a portion of the south end of the access elements may be constrained by the special status plant, Tidestrom's lupine.
- During inlet closure or outlet channel conditions, removing the access elements would likely increase total seepage through the beach berm by about 60% as compared to Alternative 1 (No Action).

- Alternative 5 would probably enable the inlet to migrate up to 1,000 ft south of the groin on the inlet's west end and up to 600 ft south of the groin on the inlet's east end.
- Removing the access elements, which stabilize the beach berm south of the groin, and enabling the inlet to migrate further south, which would intermittently erode the south beach berm, would result in the lowest beach crest elevations of any alternative. A corresponding increase in wave overwash of salty ocean water into Estuary would be anticipated.
- The increase in seepage outflow and increase in wave overwash inflow offset each other, limiting the change in net inflow to the Estuary. In addition, when the modeled inlet migrates further south, it faces more wave power and hence greater chance of closure. Overall, the Estuary is predicted to have a slightly fewer days closed and similar frequency of perched Estuary water surface elevations as Alternative 1.
- Alternative 5 would result in beach management needs that are similar to Alternative 1.
- In the long-term, Alternative 5 may increase potential flood risk for the closed inlet and ocean wave flood scenarios and may decrease potential flood risk for the fluvial flood scenario.
- Natural beach aesthetics would improve the most of all alternatives because of the full removal of the jetty structure
- The increased southward migration would reduce the length of beach that could be accessed by the public from the south by up to 1,000 ft. Removing the groin would eliminate the public safety hazard posed by the groin encouraging the public to walk close to the surf zone.
- Changes due to sea level rise would be similar as compared to Alternative 1 (No Action), with the beach and inlet anticipated to shift upward and landward. This alternative would continue to provide improved seepage, which may partially offset the decrease in hydraulic gradient under Alternative 1.
- This alternative would have an estimated probable project cost of $18,100,000 with possible range of -50% to +100% of this estimate. Annual operation and maintenance costs are estimated to be $300,000, based on anticipated similarity to existing conditions.

9.4.1 Potential Benefits to Estuary Water Surface Elevation Management

The potential benefits of the alternatives to Estuary water surface elevation management and other objectives of the Biological Opinion's Estuary Management RPA are summarized in Table 9-7. The table also includes potential constraints on achieving these benefits. Additional detail and reasoning for these assessments were provided in Section 9.1.

Table 9-7. Summary of jetty alternatives' potential benefits and constraints on achieving those benefits to Estuary water surface elevation management

Alternative	Beach Permeability	Beach and Inlet Morphology	Estuary Water Surface Elevations	Water Quality
Alternative 1: No action	• No change	• Maintains current management conditions	• Maintains current management conditions*, which include artificial breaching (for flood management) and outlet channel management (when feasible)	• Maintains current management conditions
Alternative 2: Notch groin	• No change	• West end of inlet may migrate additional 250 ft further south • May need to add additional armor to the beach to reduce chance of scour on beach adjacent to notch • May facilitate formation of outlet channel, but likely to require additional beach management activities to maintain open inlet • Alternative is reliant on beach morphology conditions, which are dynamic and not always conducive for an outlet channel through the notch	• Increases duration of perched lagoon conditions by 18% annual and by 37% during management period • Increased water surface elevations may be limited to approximately 1 in 3 years • Analysis assumes as-needed beach management; beach and surf conditions can impede beach management and constraining alignment to include the notch would reduce operational flexibility	• Increased duration of perched conditions would increase fresh water conditions and inundation extent
Alternative 3: Remove groin	• Increase total seepage by 20%	• West end of inlet may migrate additional 400 ft further south, east end of inlet migration is limited by remaining access elements	• Slight increase in annual number of days inlet is closed • Minimal change in time water surface elevation is above 3.2 ft NGVD	• Similar to current management conditions
Alternative 3a: Degrade In Place	• Increase total seepage by less than 20%	• Potential for limited additional southward migration, depending on dispersal of groin materials • Leaves artificial fill embedded in the beach	• Anticipated to approach Alternative 3 conditions at time scale which depends on mechanical and natural dispersal rate	• Similar to current management conditions
Alternative 4: Remove access elements	• Increase total seepage by 40%	• Maintains current management conditions (slight increase in risk of inlet breaching south of groin)	• Slight decrease in annual number of days inlet is closed • Minimal change in time water surface elevation is above 3.2 ft NGVD	• Lower berm allows increased overwash of salty ocean water into lagoon
Alternative 5: Remove groin and access elements	• Increase total seepage by 60%	• West end of inlet may migrate additional 1,000 ft further south, east end of inlet may migrate additional 600 ft further south;	• Slight decrease in annual number of days inlet is closed • Minimal change in time water surface elevation is above 3.2 ft NGVD	• Lower berm allows increased overwash of salty ocean water into lagoon

* Russian River Estuary Management Plan DEIR characterizes the number of breaching events and historical water surface elevations at those events. Figures 3-2, page 3-6 characterizes water surface elevation at all breaching events 1996-2009. Figure 3-3, page 3-7 characterizes the frequency of water surface elevations at all breaching events, 1996-2009.

9.4.2 Environmental Impacts and Constraints

9.4.2.1 Short Term Impacts and Constraints During Removal

During the process of removing all or part of the jetty in Alternatives 2 through 5, as described in Section 8.6, there would be the potential environmental impacts of greatest concern are transportation and traffic, noise, water quality, biological resources, recreation and access, and cultural resources. Other potential impacts may exist to a lesser extent or become apparent if the alternatives were analyzed at later stages of planning and design. Relative comparisons of these impacts relative to Alternative 1 (No Action) are summarized in Table 9-8. Additional detail and reasoning for these assessments were provided in Section 9.2.2.

Table 9-8. Potential short-term impacts of jetty alternatives

Alternative	Transportation & Traffic	Noise	Water Quality	Biological Resources	Aesthetics	Recreation & Access	Cultural Resources
Alternative 1: No action	None	None	None	None	None	None	None
Alternative 2: Notch groin	Low hauling effort: 50 dump truck trips, 5 trips per day	Shorter period of construction noise	Limited BMPs to protect water quality	Sensitive plant species likely avoided Low vibration & noise effects on pinnipeds	Shorter duration and lower construction intensity	Parking lot used for staging, shorter period of limited beach access during construction	Low potential for on-site cultural resources
Alternative 3: Remove groin	High hauling effort: 4,700 dump truck trips, 72 trips per day	Longer period of construction noise	Increased BMPs to protect water quality	Sensitive plant species likely avoided High vibration & noise effects on pinnipeds	Longer duration and greater construction intensity	Parking lot used for staging, longer period of limited access during construction	Low potential for on-site cultural resources
Alternative 3a: Degrade groin	No hauling	Moderate period of construction noise	May have less BMPs compared to Alt 3.	Sensitive plant species likely avoided Possible that vibration & noise effects on pinnipeds could be avoided with expansive grout	Moderate duration and construction intensity	Parking lot used for staging, moderate period of limited access during construction	Low potential for on-site cultural resources
Alternative 4: Remove access elements	High hauling effort: 1,400 dump truck trips, 36 trips per day	Longer period of construction noise	Increased BMPs to protect water quality	May overlap with sensitive plant species habitat High vibration & noise effects on pinnipeds	Longer duration and greater construction intensity	Parking lot used for staging, , longer period of limited access during construction	Low potential for on-site cultural resources
Alternative 5: Remove groin and access elements	Highest hauling effort: 6,100 dump truck trips, 94 trips per day	Longest period of construction noise	Intensive BMPs to be protective water quality	May overlap with sensitive plant species habitat Highest vibration & noise effects on pinnipeds	Longest duration and greatest construction intensity	Parking lot used for staging, longest period of limited access during construction.	Low potential for on-site cultural resources

9.4.2.2 Long Term Impacts and Constraints After Removal

Following completion of partial or full jetty removal for Alternatives 2 through 5, long-term effects related to the issue areas of transportation and traffic, noise, air quality, water quality and cultural resources would be expected to return to baseline conditions similar to Alternative 1 (No Action). No long-term direct effects within these issue areas would be expected following completion of construction activities, de-mobilization and restoration of the construction area.

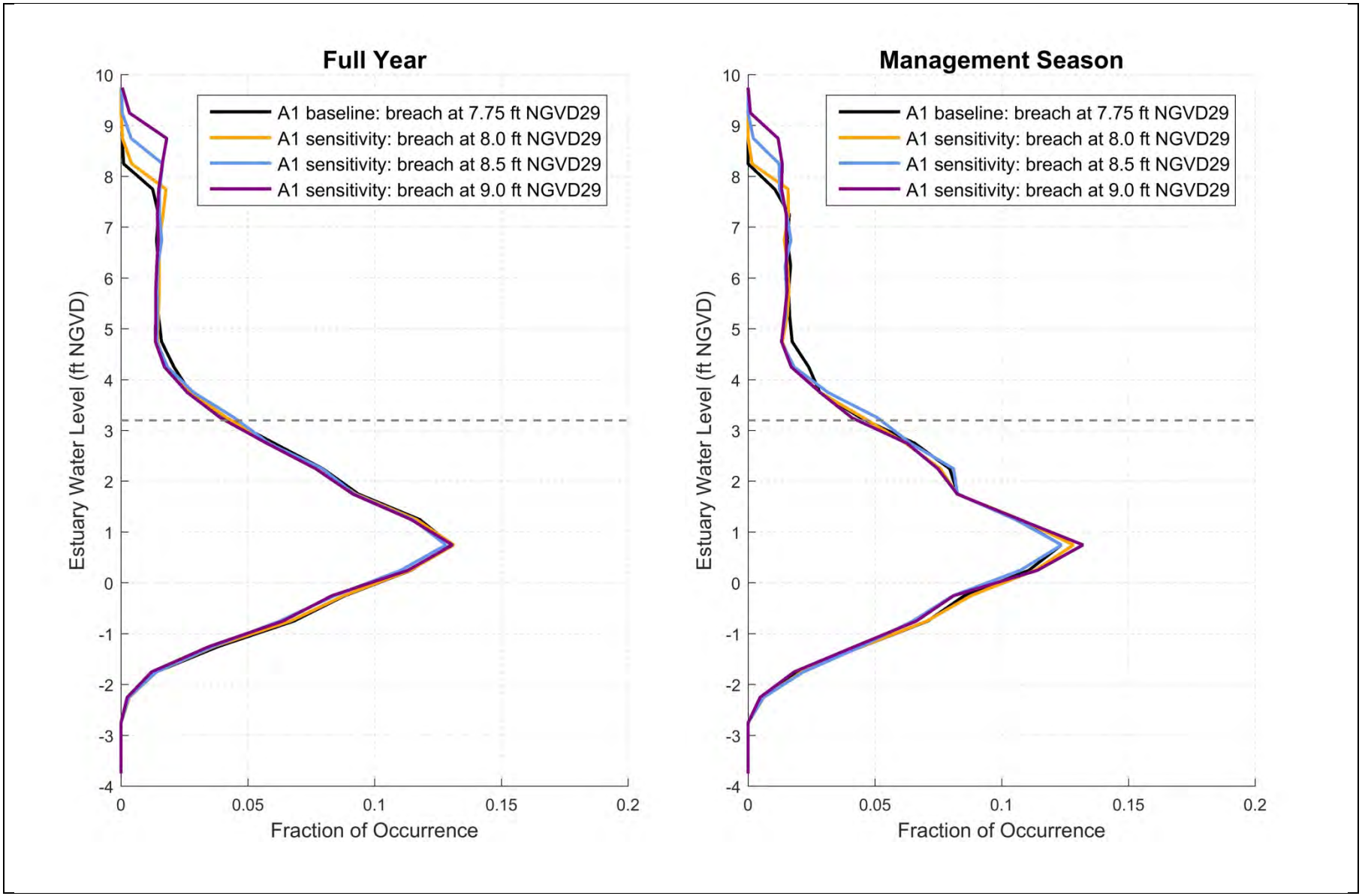
Alternatives 2 through 5 would be expected to affect six key issue areas: flooding, biological resources, aesthetics, recreation and public access and sea level rise implications. A summary of the alternatives' potential impacts and constraints on these issue areas relative to Alternative 1 (No Action) is provided in Table 9-9 and discussed in more detail in the Sections 9.2.3.

Table 9-9. Summary of jetty alternatives' potential long-term impacts and constraints

Alternative	Flooding	Biological Resources	Aesthetics	Recreation & Access	Sea Level Rise Implications
Alternative 1: No action	• Maintains current level of artificial breaching for flood management	• Maintains current level of management effort for salmonid habitat	• Maintains current management conditions	• Maintains current management conditions	• Increasing flood risk • Beach shifts upwards and landwards • Access elements buried
Alternative 2: Notch groin	• Increases potential flood risk for fluvial flood scenario if inlet cannot be re-located north of the jetty in advance of discharge event	• Outlet channel through notch may impair fish passage and would require active management to close • Early part of management season, when this alternative offers largest potential benefits to water surface elevations, would face beach management constraints due to seal pupping season	• Creates water control structure beach • Notch inconsistent with natural beach morphology	• Inlet channel further south, restricting access to as much as 250 ft beach • Hazard of swift water outflow over notch	• Notch would need to be raised to adapt to sea level rise
Alternative 3: Remove groin	• Increase in potential flood risk for closed inlet and ocean wave scenarios; may decrease fluvial flood risk	• Channel may migrate • Berm/haul out conditions largely unchanged	• Improved due to partial removal	• Inlet channel further south, restricting access to as much as 400 ft of beach • Benefit of reducing access to unsafe surf zone conditions.	• Similar to current management conditions
Alternative 3a: Degrade groin	• Increase in potential flood risk for closed inlet scenarios	• Channel may migrate • Berm/haul out conditions largely unchanged	• Improved due to partial removal (but remaining materials may occasionally be exposed)	• Inlet channel further south, restricting access to as much as 400 ft of beach • Possible long-term benefit of reducing access to unsafe surf zone conditions.	• Similar to current management conditions
Alternative 4: Remove access elements	• Maintains current management conditions (slight increase in risk of inlet breaching south of groin)	• Haul out area on estuary side could be increased by sand from berm	• Improved due to partial removal	• Maintains current management conditions	• Similar to current management conditions

Alternative	Flooding	Biological Resources	Aesthetics	Recreation & Access	Sea Level Rise Implications
Alternative 5: Remove full jetty	• Increase in potential flood risk for closed inlet and ocean wave scenarios; may decrease fluvial flood risk	• Channel may migrate • Haul out area on estuary side could be increased by sand from berm	• Largest improvement due to full removal	• Inlet channel furthest south, restricting access to as much as 1,000 ft of beach. • Benefit of reducing access to unsafe surf zone conditions.	• Similar to current management conditions

9.5 Figures

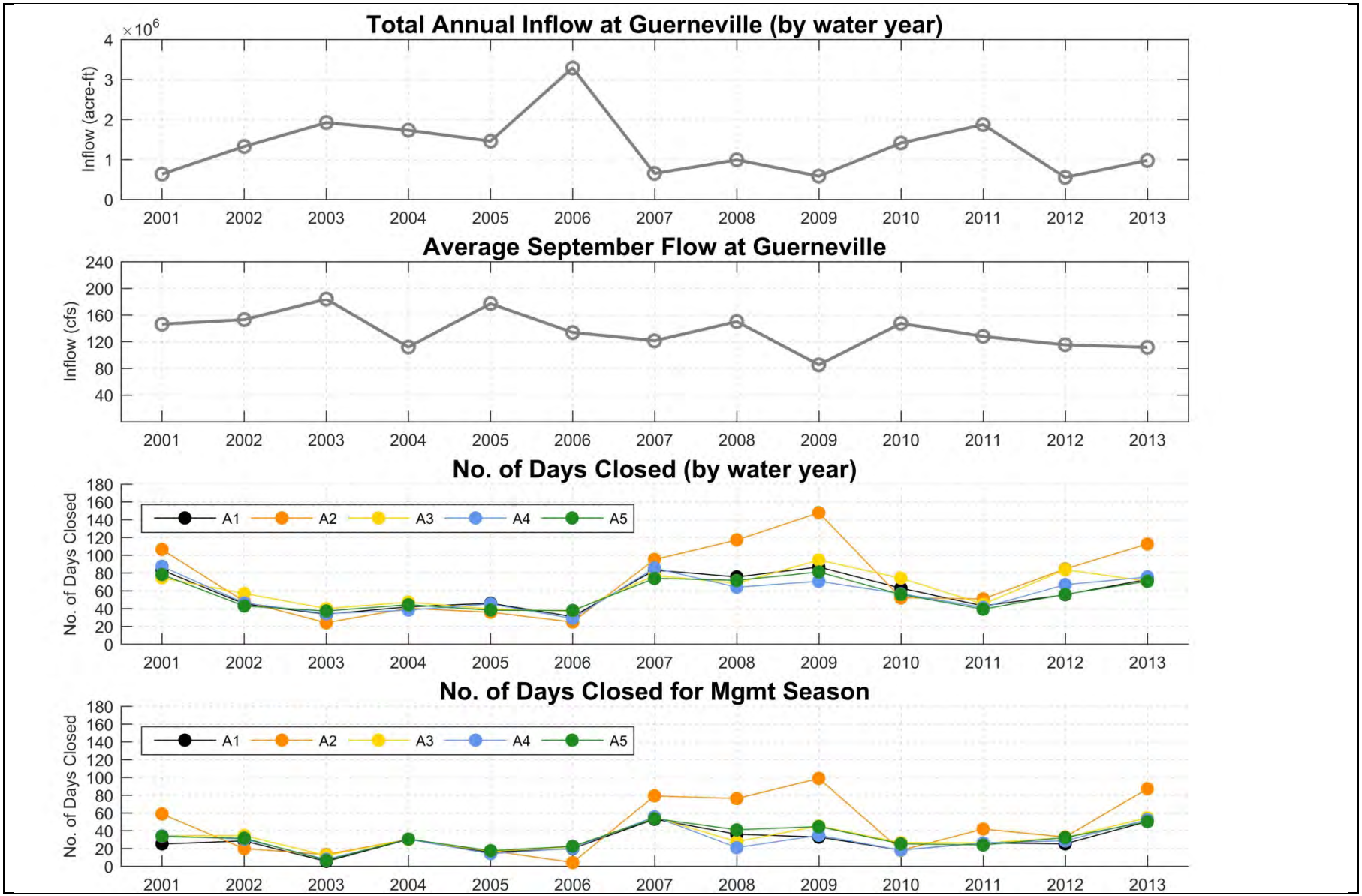


SOURCE: ESA model

Goat Rock Jetty Feasibility Study D211669.0.0

Figure 9-1

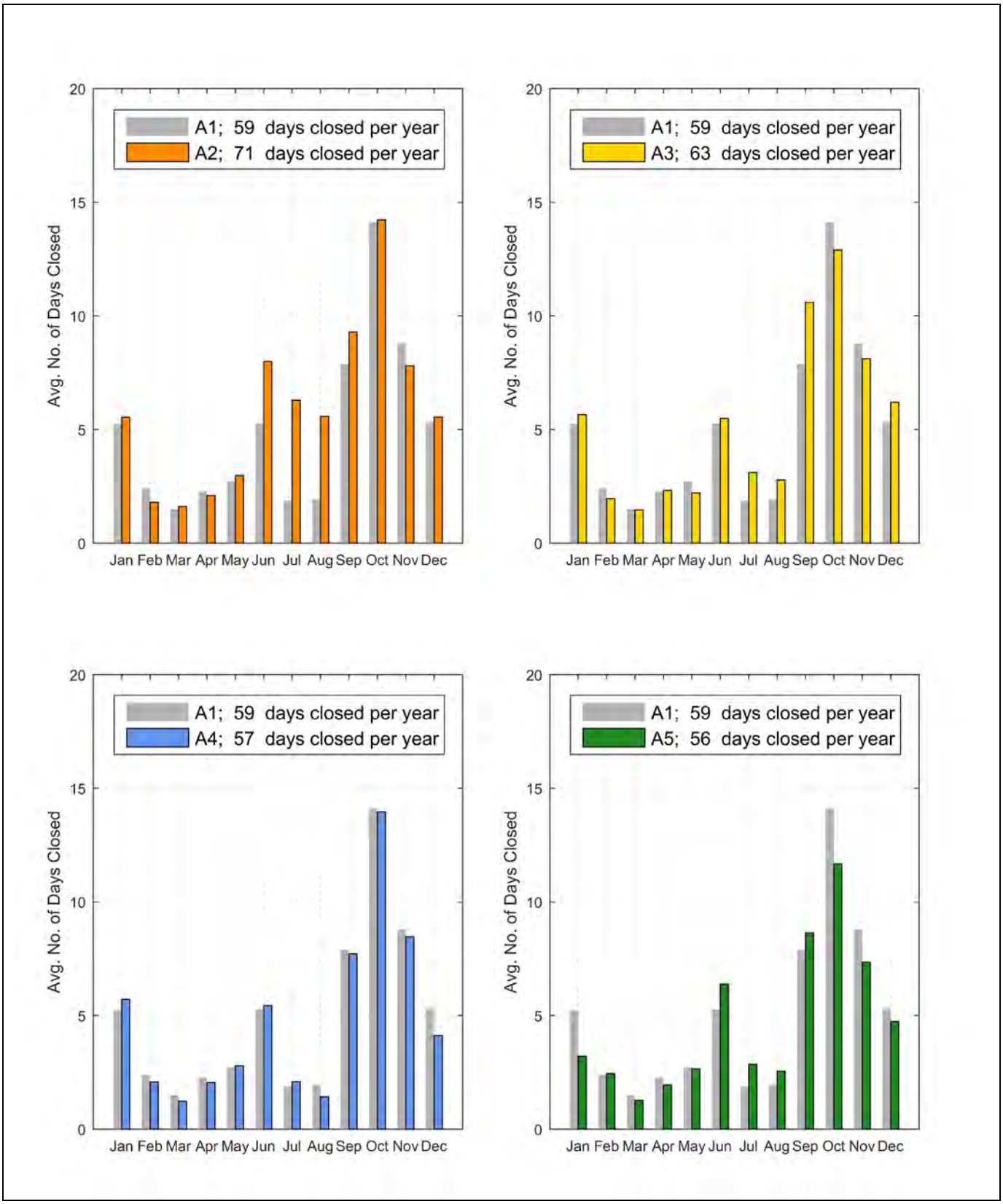
Occurrence distribution of estuary water levels as a function of artificial breaching threshold, 2000-2014.



SOURCE: USGS flow data at Guerneville. ESA lagoon QCM model.

Figure 9-2

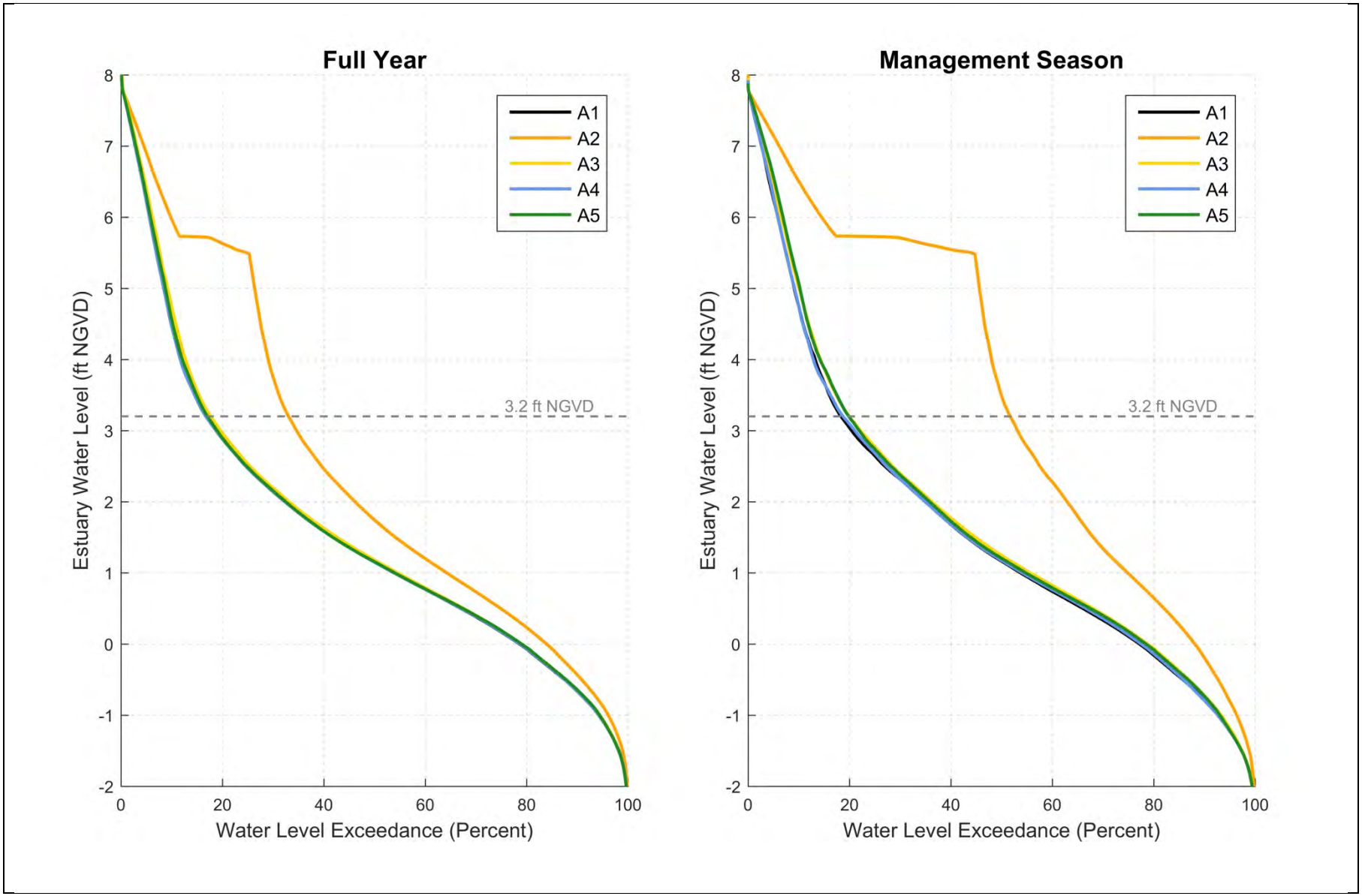
Comparison of the number of mouth closure days among jetty alternatives for years with different amounts of freshwater runoff.



SOURCE: ESA QCM model

Figure 9-3

Predicted number of days of mouth closure by month for the period from 2000 to 2014 for different jetty alternatives.



SOURCE: USGS flow data at Guerneville. ESA lagoon QCM model.

Figure 9-4

Comparison of the lagoon water level exceedance among jetty alternatives for **(left)** the full year and **(right)** the management season for the years 2000-2014.

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APPENDIX A. OCEAN WAVE ANALYSES

This appendix details the analysis used to predict nearshore wave conditions at GRSB, including predictions of runup and overwash on the beach face. Wave conditions are summarized in Chapter 4.

A.1. Obtaining Nearshore Wave Estimates

To estimate nearshore waves, we formally applied a transformation matrix provided by the Coastal Data Information Program (CDIP), which had been tested against BML-measured waves from 2009. The transformation matrix presented an improvement over using offshore waves at the Pt. Reyes Buoy, but there were gaps in important bands of the wave spectrum associated with swell waves. The present effort summarized in Chapter 4 uses data collected in 2012, and involves a transformation matrix that we developed using these data. Our methods are described below.

A.1.1 Wave Transformation

Wave transformation methods account for changes in the wave field due to wave refraction, shoaling, and in some cases, secondary processes such as diffraction and wave-current interactions. Wave refraction is the change in wave direction caused by changes in bathymetry. At headlands, refraction causes waves to focus and wave height to increase, while at bays (concave shorelines) it causes waves to spread out (distribute their energy over a greater lateral length and decrease in height). The orientation of the shoreline relative to the approaching waves is also important: waves approaching the shoreline from a shore-normal angle generally undergo the least amount of refraction, whereas waves approaching from oblique angles experience larger amounts of refraction and lose more energy (Komar, 1998). Shoaling is the change in waves in response to the transition from deepwater to shallow water conditions. In the transition to shallower water, group velocity becomes increasingly controlled by the local depth. The above processes can be represented as follows:

$$K_r = \sqrt{\frac{b_0}{b}} \tag{A.1}$$

$$K_s = \sqrt{\frac{C_{g0}}{C_g}} \tag{A.2}$$

where K_r is the refraction coefficient, K_s is the shoaling coefficient, b is the lateral distance between wave rays, the subscript "*0*" refers to offshore (deep-water) conditions, and

$$C_g = \frac{1}{2}\left(1 + \frac{4\pi d / L}{\sinh(4\pi d / L)}\right)\frac{gT}{2\pi}\tanh\left(\frac{2\pi d}{L}\right) \tag{A.3}$$

is the group velocity, where L is the nearshore wave length, d is the local depth and T is the wave period. L is approximated after Fenton and McKee (1990). Shoaling is only a function of frequency and the change in depth, whereas the effects of refraction are a function of frequency, wave direction, and depth.

To summarize how a broad wave spectrum representative of offshore conditions would translate to the nearshore environment at GRSB, we apply the unit-wave approach, which translates simulated one-meter high waves of varying direction and period from the Pt. Reyes buoy to GRSB, accounting for refraction and shoaling along the way. Nearshore wave estimates are then de-shoaled (SPM 1984). Since shoaling changes with depth in the nearshore, choosing a depth to represent a shoaling location would be an arbitrary procedure, and estimates for wave setup, runup, and overwash require de-shoaled wave heights. We use the SWAN model to simulate wave conditions, using a series of nested grids to define the bathymetry offshore of the site (Figure A-1). By starting with unit waves with a range of wave periods and directions offshore, and determining the final wave energy at the nearshore locations, a matrix can be populated for each nearshore site that describes the expected changes in wave energy. These location-specific matrices are then used with real data at the Pt. Reyes buoy to create time series wave conditions along GRSB (see Figure 4-3).

The Pt. Reyes buoy[1] supplies spectral information about the wave field at hourly intervals, with the total wave energy separated by wave frequency (f) and direction (ϑ). The shallow water energy density spectrum is related to the deep water spectrum by:

$$S(f,\theta) = K_r^2(f,\theta)K_s^2(f)S_0(f_0,\theta_0)J \tag{A.4}$$

where S is the wave energy density and J is the transformation Jacobian. Since we did not consider shoaling, we used a form of Equation A.4 absent of K_s. To get hourly nearshore wave heights from the predicted nearshore spectrum $S(f, \vartheta)$, we assume the wave heights are Rayleigh distributed, and use the following:

[1] CDIP station 029, http://cdip.ucsd.edu/?ximg=search&xsearch=029&xsearch_type=Station_ID

$$H_s = 4\left[\sum_f \sum_\theta S(f,\theta)\Delta f \Delta\theta\right]^{1/2} \tag{A.5}$$

where H_s represents the significant wave height adjacent to GRSB. This is also referred to as the equivalent unrefracted deepwater wave height, $H_0^{'}$. It represents the equivalent offshore waves after they have undergone refraction, but not shoaling.

A.1.2 Validation of Wave Predictions

As shown by Figure 4-4, the new transformation matrix is a good predictor of nearshore waves at the location of BML measurements. This represents and improvement over using Pt. Reyes waves without transformation, and over using the previous CDIP matrix.

As discussed in Chapter4, wave power is lowest at the groin, and increases north and south of the groin. This matches what Johnson (1959) originally found using hand-drawn wave refraction maps. He used offshore wave information from 1954-1956 (NMC, 1960) and a published bathymetry chart from 1930-1931 (US Coast and Geodetic Survey Chart H-5098) to trace individual wave rays from offshore to nearshore. An offshore set of parallel wave rays were propagated landward, with the rays' paths altered due to changes in bathymetry. Johnson (1959) only applied this method for waves approaching from the southwest, west, and west-northwest angles (225°-292°) and having periods of 8, 12, and 16 seconds. However, his analysis gives a useful check of the influence of refraction on nearshore wave energy at GRSB (Table 4-1).

Table A-1. Height ratio (H_1/H_2) of nearshore (H_1) to offshore waves (H_2).

Direction[1] \ T_p	8 seconds	12 seconds	16 seconds
200 ft North of Jetty			
WNW	--	0.65	--
W	--	0.84	0.87
SW	0.99	0.92	0.87
200 ft South of Jetty			
WNW	--	0.57	0.74
W	0.89	0.84	0.81
SW	0.96	1.0	0.83
1200 ft South of Jetty			
WNW	--	0.62	--
W	0.96	0.84	0.87
SW	--	--	--

Note: As estimated by Johnson (1959). Values account for wave refraction, but not shoaling.
[1]WNW=west northwest; W=west; SW=southwest

A.2. Surf Zone Processes

The predicted nearshore waves were used to provide hourly time series of wave setup, runup and overtopping from 1999 to 2014, as described in Chapter 4. The methods used to estimate these are described here.

A.2.1 Estimating Wave Runup and Overtopping

Wave setup (η) is an increase in the time-averaged water level due to wave breaking. It is an important feature in the coastal zone because it provides a platform for breaking waves. The static component of wave setup was estimated using the methodology of Stockdon et al. (2006):

$$\langle \eta \rangle = 0.35\beta(H_0 L_0)^{1/2} \tag{4.1}$$

Where β is the foreshore slope, and H_0 and L_0 are representative deepwater (offshore) significant wave height and length, respectively. L_0 is estimated from the wave period alone (Komar, 1998), and H_0 is estimated by de-shoaling nearshore wave estimates using the methods described in the Shore Protection Manual (USACE, 1984). We used the nearshore predictions of H_s to characterize H_0, since these represent equivalent deepwater wave heights that account for refraction but not shoaling. Setup due to coastal winds was not considered here, as Rice (1974) noted that this is probably less than 1.5 feet, owing to the small shelf and absence of hurricane-force winds. The dynamic wave setup is combined with the dynamic wave runup in the right-hand term of Eq. 4.2 below.

Wave runup is the maximum vertical extent that the leading edge of a breaking wave reaches on the shore (Figure 4-8b), assuming that the beach face continues to slope up indefinitely (Komar, 1998). Under actual conditions when the runup exceeds the beach crest elevation, the runup is transformed to overwash before it reaches the total water level (TWL). Since individual waves have variable characteristics, this is usually taken as a representative hourly value, such as that value that is exceeded by two percent of the waves. This parameterization (*R2%*) uses the same input parameters as the setup (Stockdon et al., 2006):

$$R_{2\%} = 1.1\left((0.35\beta(H_0 L_0)^{1/2} + \frac{\left[H_0 L_0 (0.563\beta^2 + 0.004)\right]^{1/2}}{2} \right) \tag{4.2}$$

Wave runup on the beach face can be combined with hourly tide levels and wave setup to estimate peak hourly *TWL*. When related to the beach berm height, *TWL* can be used to estimate wave overwash, a measure of the amount of water contributed to the Estuary from waves spilling over the beach (Figure 4-8b). To estimate overwash, we

must first have an understanding of the breaker type, which is characterized with the Irribarren Number (Komar, 1998):

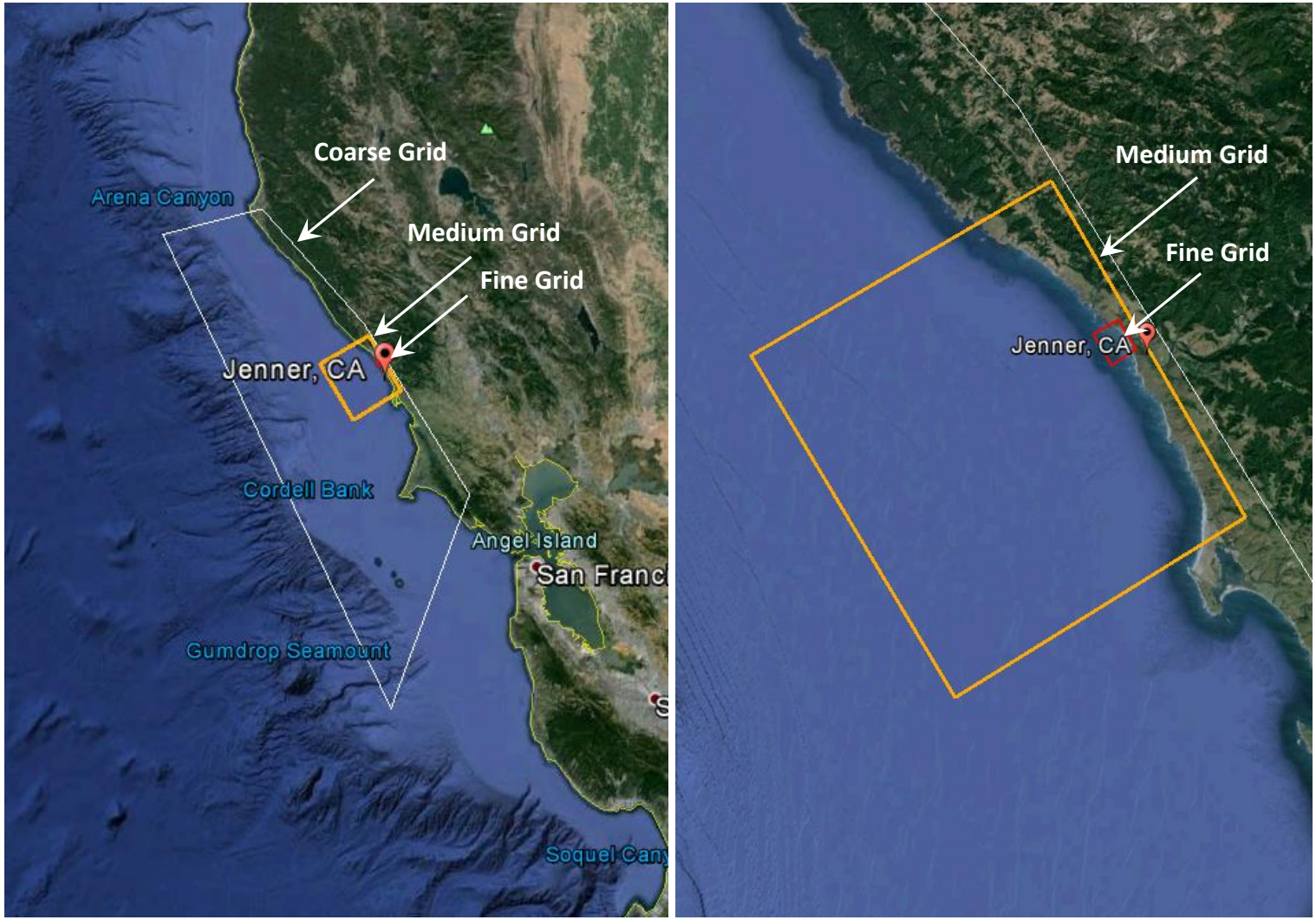
$$\xi = \frac{\beta}{\sqrt{H_0 / L_0}} \tag{4.3}$$

The overwash is then characterized with the following relation from Van der Meer and Janssen (1995):

$$\frac{Q}{\sqrt{gH_S^3}} = \frac{A}{\sqrt{B}} \xi \exp\left(-B \frac{R_C}{\gamma_r \xi H_S} \right) \qquad \xi \le 2$$

$$\frac{Q}{\sqrt{gH_S^3}} = C \exp\left(-D \frac{R_C}{\gamma_r H_S} \right) \qquad \xi \ge 2 \tag{4.4}$$

where Q is a representative overwash flow rate per unit width of the beach, R_c is the freeboard height of the *TWL* over the beach crest height, and *A*, *B*, *C*, and *D* are empirical coefficients. We use the transformed nearshore wave height predictions for H_s. Van der Meer and Janssen (1995) provide values of the coefficients based on laboratory tests. We estimate the total wave overwash volume by solving Equation 4.4 for 100-ft segments of GRSB between the south end of the spit (approximately 1200 ft south of the groin) and the north end of the spit, where it connects with a rocky headland (approximately 800 ft north of the groin). Laudier et al. (2011) found that this model performed well in predicting overwash into Carmel Lagoon, in central California.



SOURCE: ESA SWAN model

Figure A-1

Nested grids used in SWAN model of waves.

APPENDIX B. SHORELINE CHANGE ANALYSES

This appendix describes in more detail the shoreline and beach width change analyses of Goat Rock State Beach (GRSB) that are presented in the main report's Section 5: Beach Morphology.

As discussed in the main report, shoreline positions were identified using available aerial images (Table 5-1). Since these were obtained from dozens of separate flights between 1940 and 2012, they were taken at variable heights and resolution. To be able to compare the images directly, each image was first georeferenced using the Digital Shoreline Analysis System (DSAS) provided by the U.S. Geological Survey (USGS, 2012). A 2009 orthophoto from the U.S. Department of Agriculture (USDA) was used as a baseline for comparing against each image. For each aerial image, ground objects on or near GRSB were identified. Each of these objects was also identified in the 2009 baseline image. DSAS was then used to rotate, translate, and warp the original aerial so that it had a common reference frame as the baseline image.

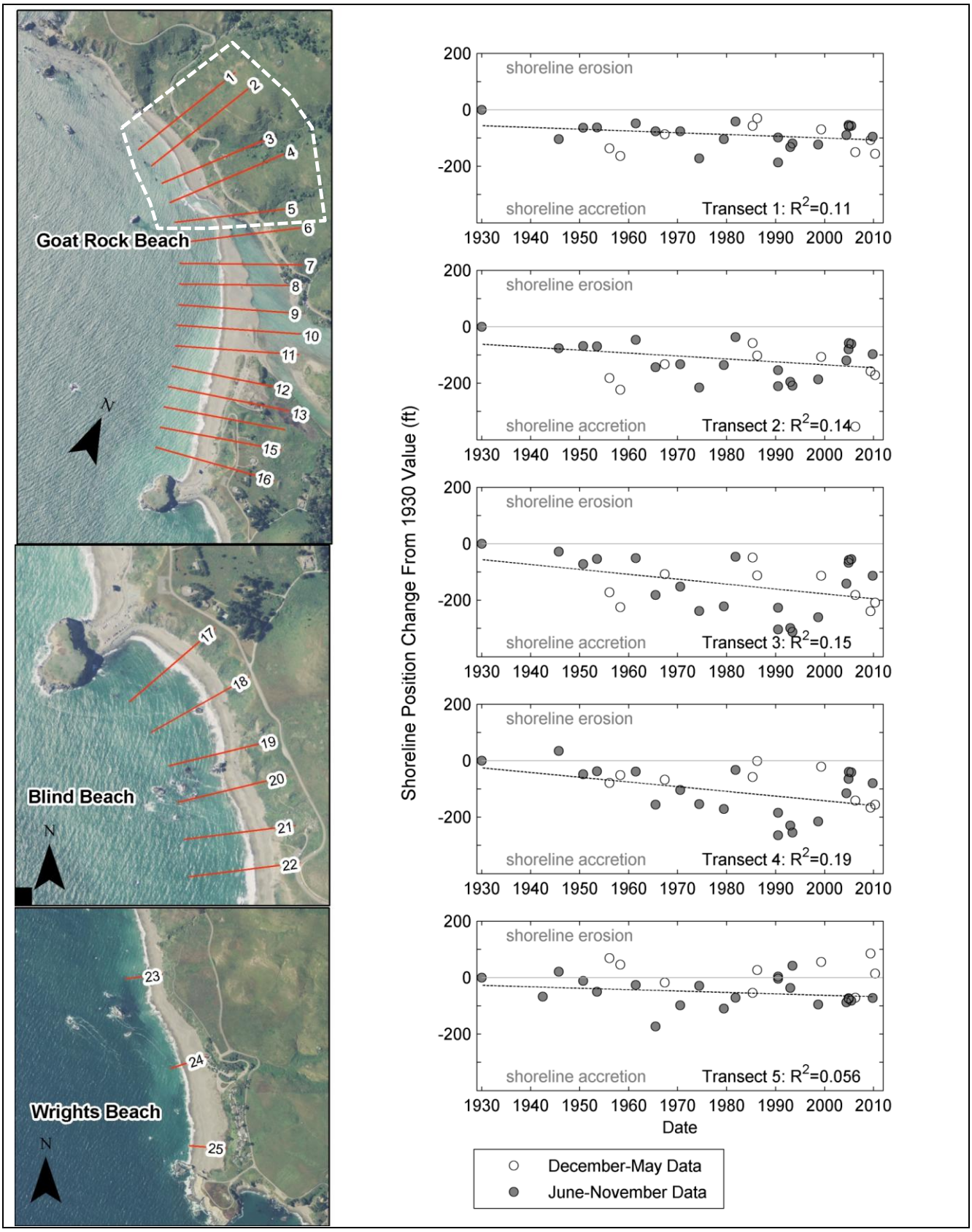
To assess changes in the beach shape over time, two vectors were drawn for each corrected aerial image: (1) a shoreline vector drawn along the ocean side of the beach and (2) a backbarrier vector representing either the estuary side of the beach or the convergence of the beach with the cliff or bluff at its landward boundary. Cross-shore transects (Figure 5-3) were then drawn along GRSB, Blind Beach and Wright's Beach. A time series of shoreline position was generated by tabulating the position that the shoreline vector for each aerial image intersected each transect. A beach width time series was generated by tabulating the difference between the positions of the shoreline and backbarrier vectors for each aerial image. Shoreline vectors drawn by Hapke et al (2006) were to assess the beach shape during the summer of 1930.

Figures B-1 through B-5 show the raw time series of shoreline position at GRSB, Blind Beach, and Wright's Beach for the period from 1930 to the present. Figures B-6 through B-10 give time series of beach width. These estimates have associated uncertainty which arises from the following:

- Limited map or aerial image resolution,
- Errors arising from georeferencing images,
- Errors in visually estimating the shoreline and backbarrier positions and tracing the associated vectors.

Boak and Turner (2005) and Hapke et al. (2006) discuss these and other errors involved with estimating shoreline features. Digitizing maps and aerial images of limited

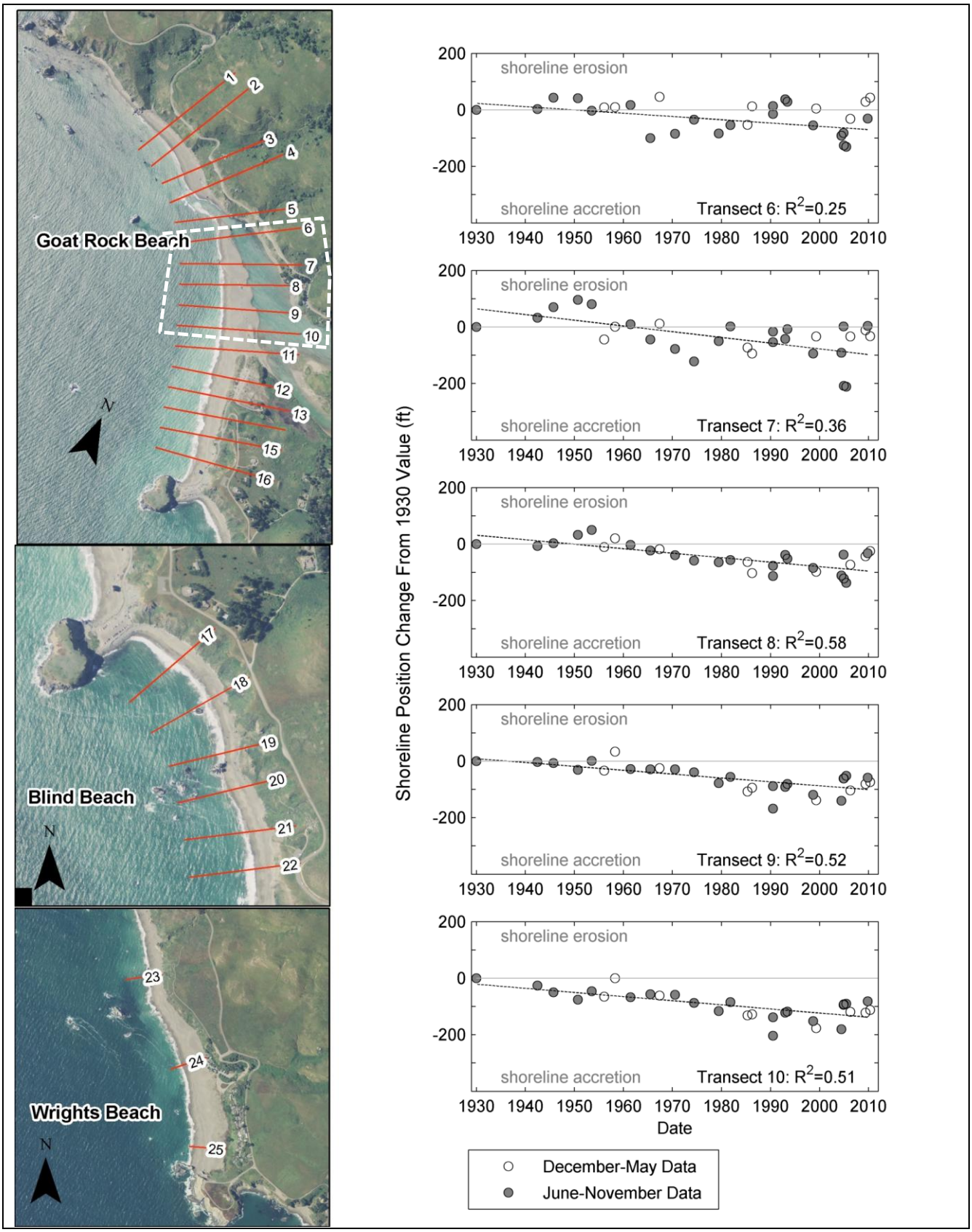
resolution can produce errors of ~3 ft (Hapke et al., 2006). Errors produced by the georeferencing process range from roughly 6-30 ft for the present analysis. This was assessed using the DSAS software, which calculates the root-mean-squared error (RMSE) of each of the ground control points relative to their original position after the georeferencing process alters the original image. Errors in visually estimating the shoreline position are difficult to assess, but are known to be largest for beaches with relatively-low slopes, and can surpass 30 ft (Boak and Turner, 2005). For the present study, the steep slope of GRSB is expected to limit the error of assessing the horizontal position of the shoreline. Since the observed changes in shoreline position and width were often greater than 100 ft between 1930 and 2012 (Figures B-1 through B-10), the results are assumed to be significant, despite the uncertainties present.



NOTE: All linear trends were obtained using summer-fall data.

Figure B-1

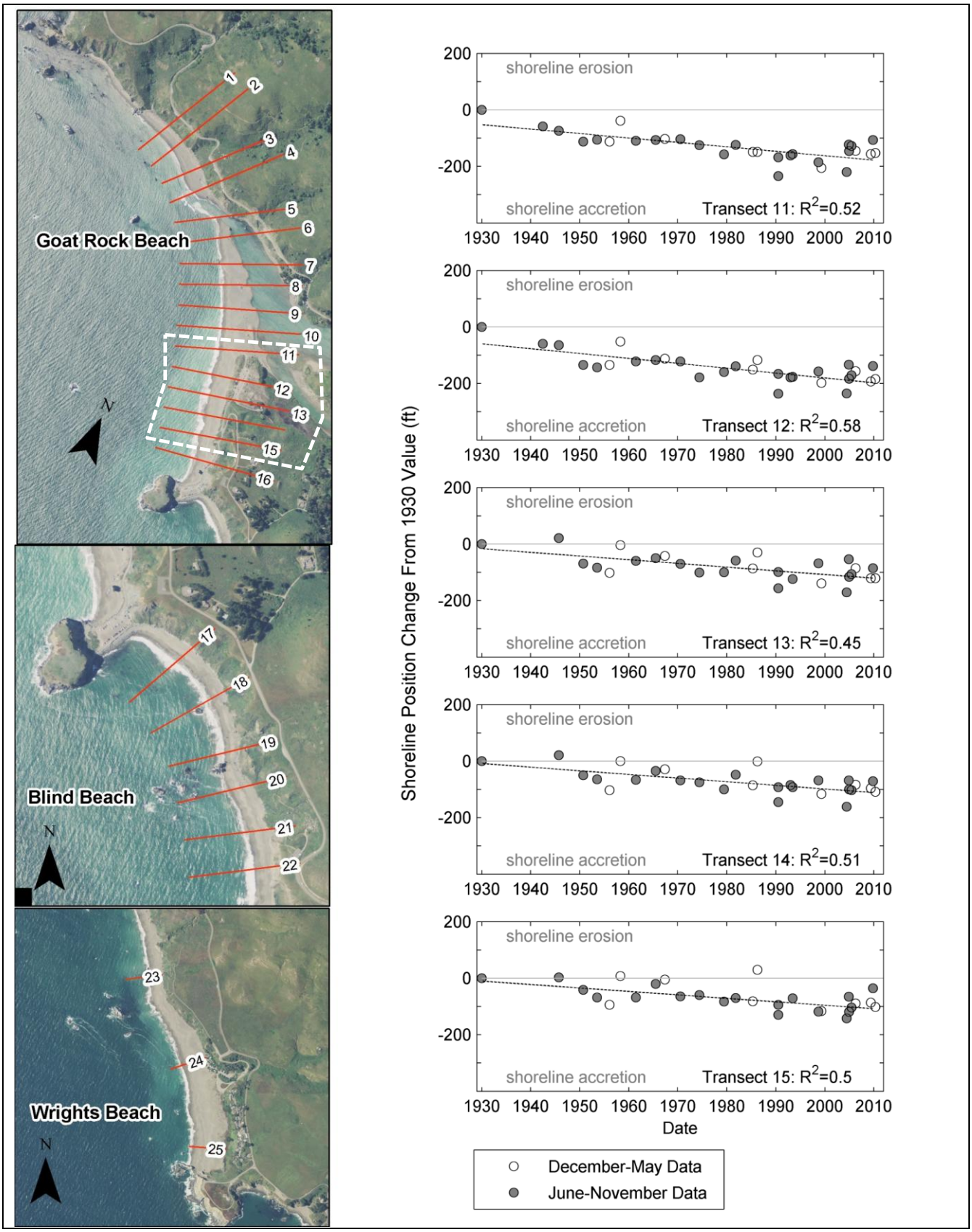
(left) Index of transects and **(right)** corresponding shoreline position data for transects 1-5.



NOTE: All linear trends were obtained using summer-fall data.

Figure B-2

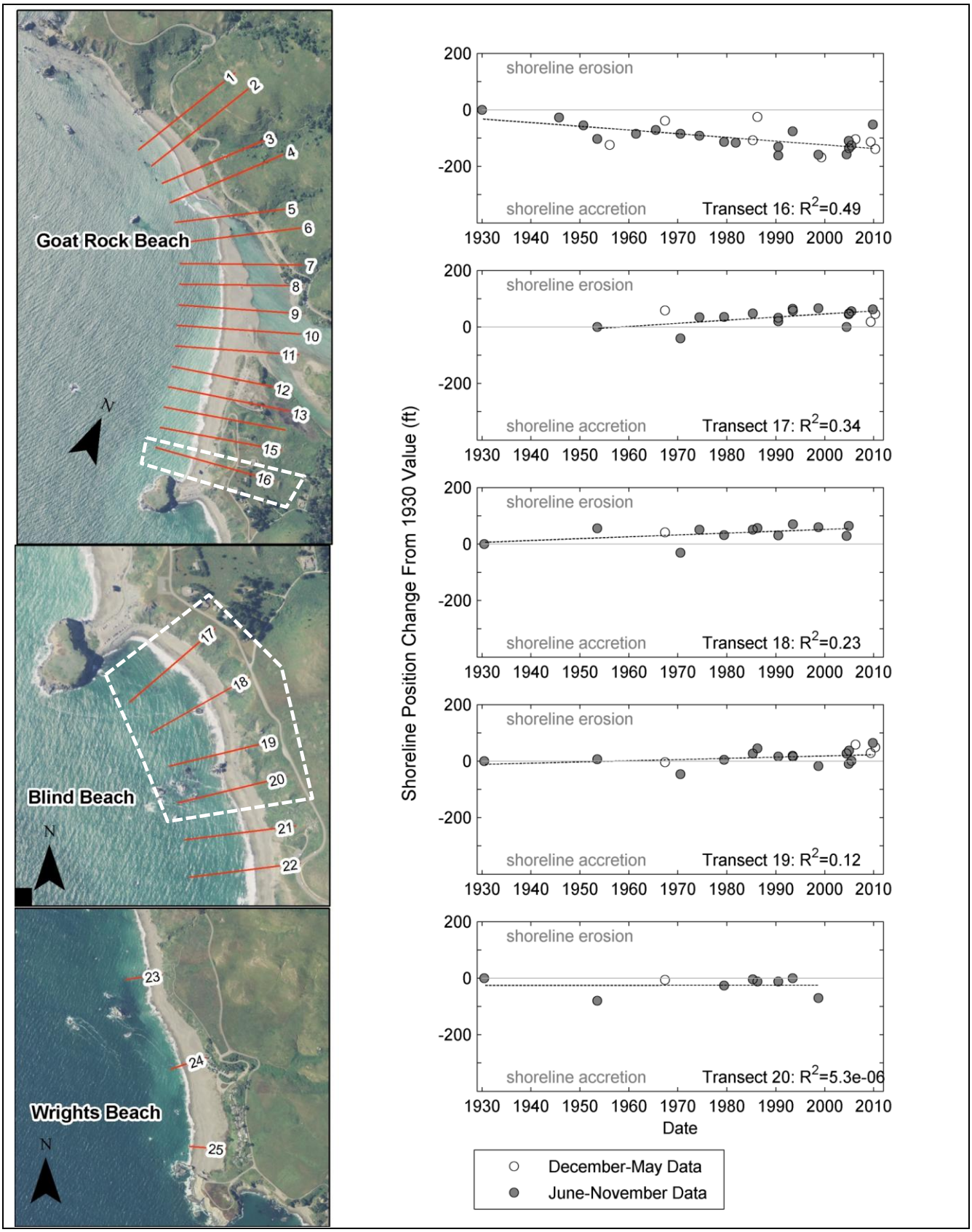
(left) Index of transects and **(right)** corresponding shoreline position data for transects 6-10.



NOTE: All linear trends were obtained using summer-fall data.

Figure B-3

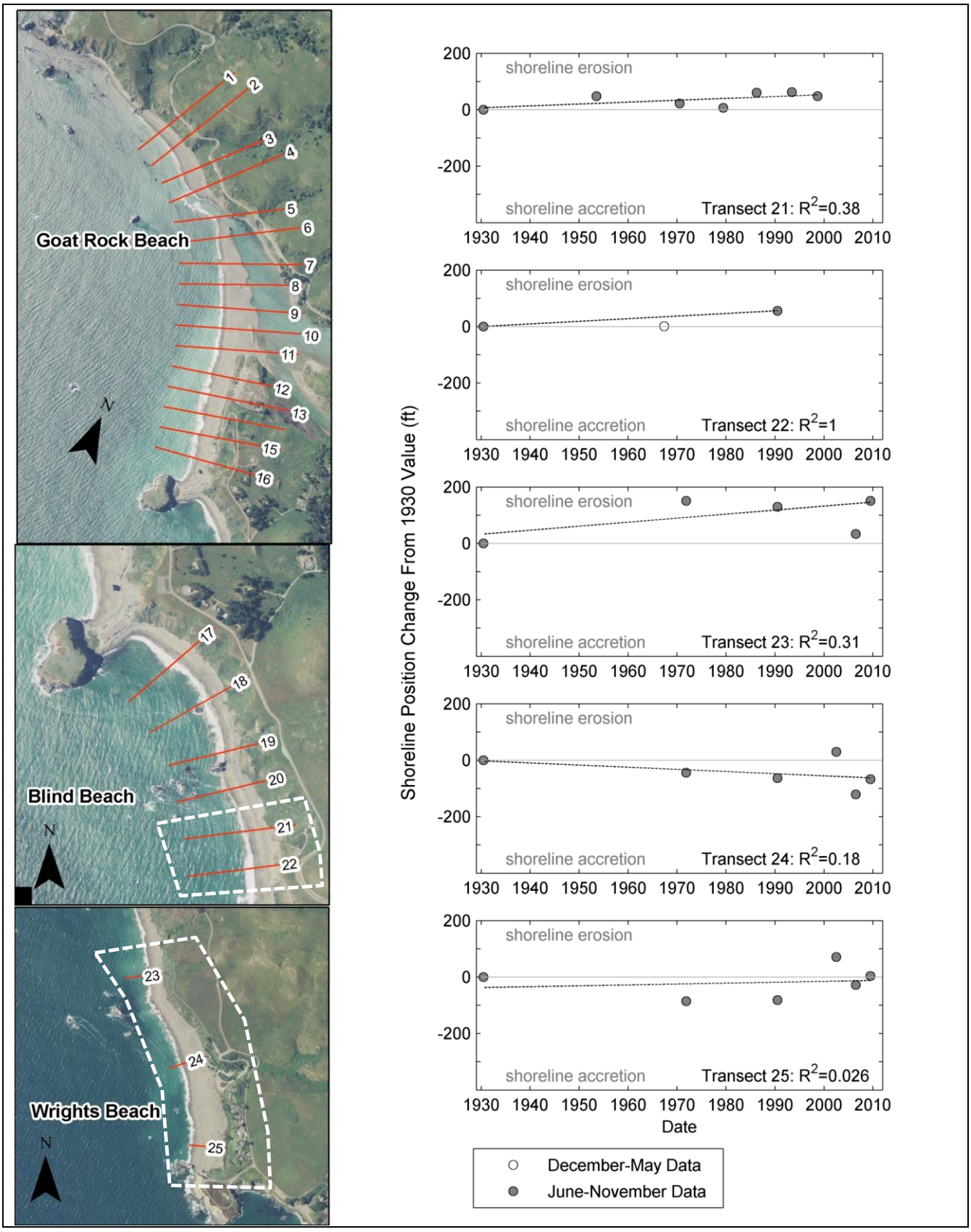
(left) Index of transects and **(right)** corresponding shoreline position data for transects 11-15.



NOTE: All linear trends were obtained using summer-fall data.

Figure B-4

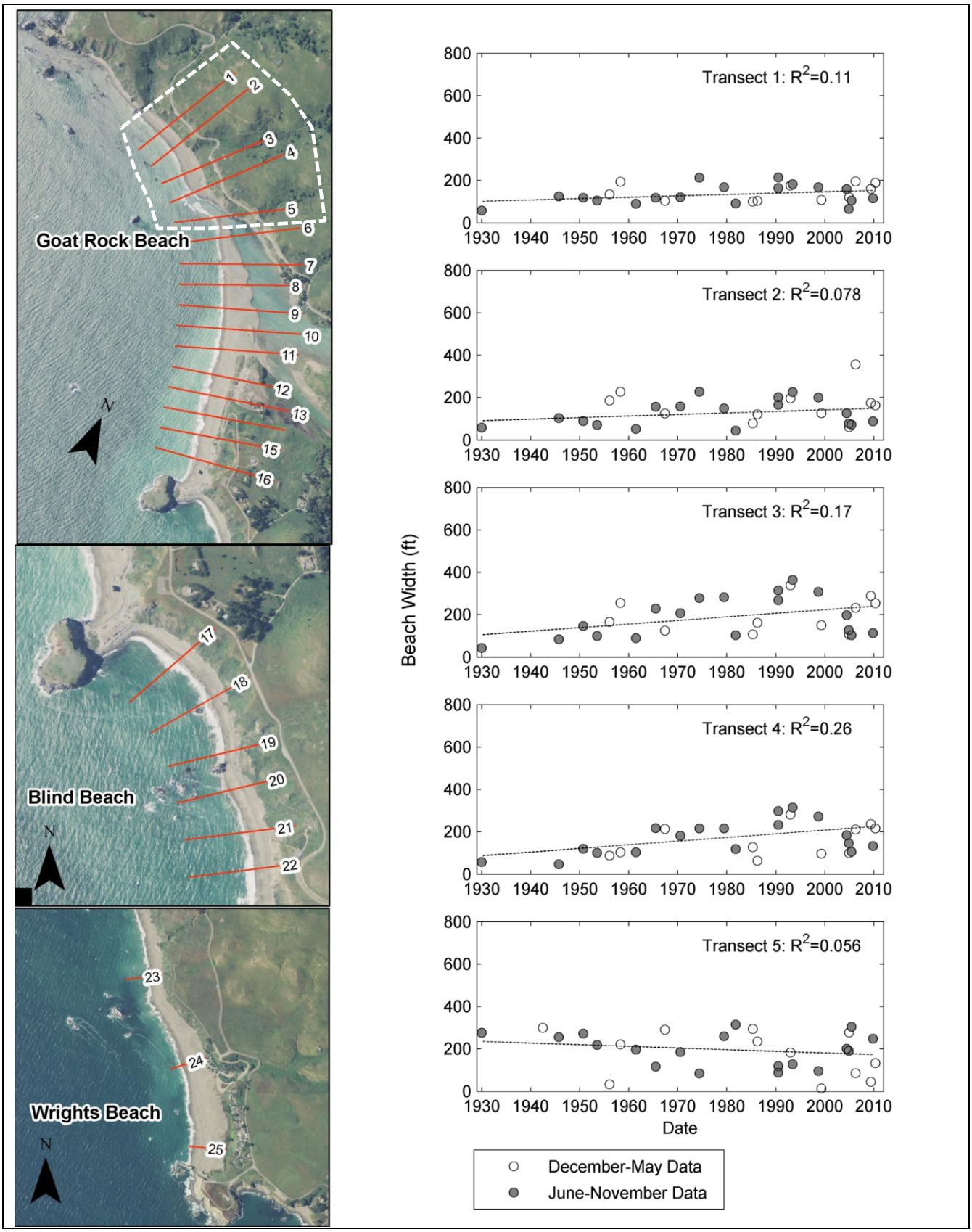
(left) Index of transects and **(right)** corresponding shoreline position data for transects 16-20.



NOTE: All linear trends were obtained using summer-fall data.

Figure B-5

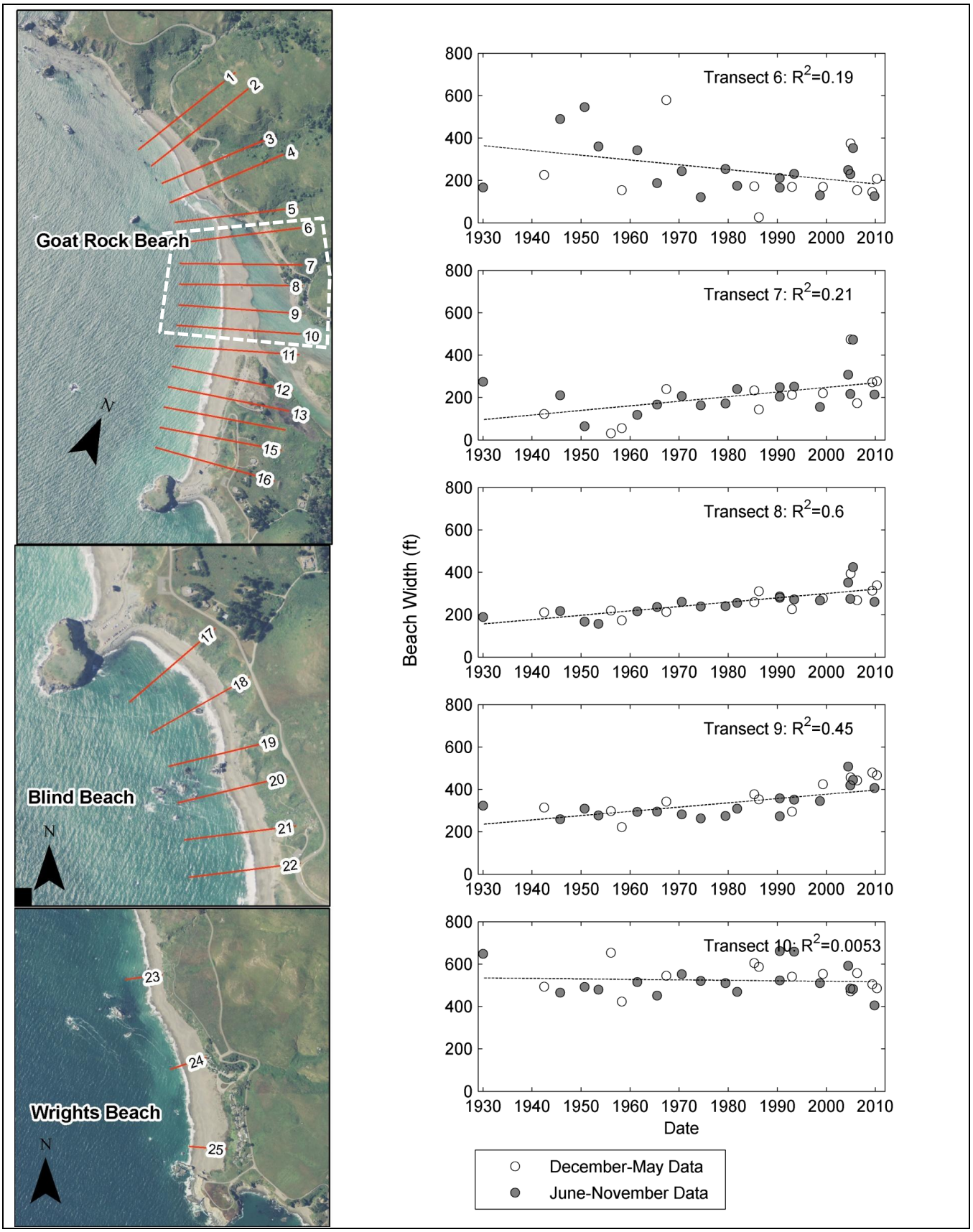
(left) Index of transects and **(right)** corresponding shoreline position data for transects 21-25.



NOTE: All linear trends were obtained using summer-fall data.

Figure B-6

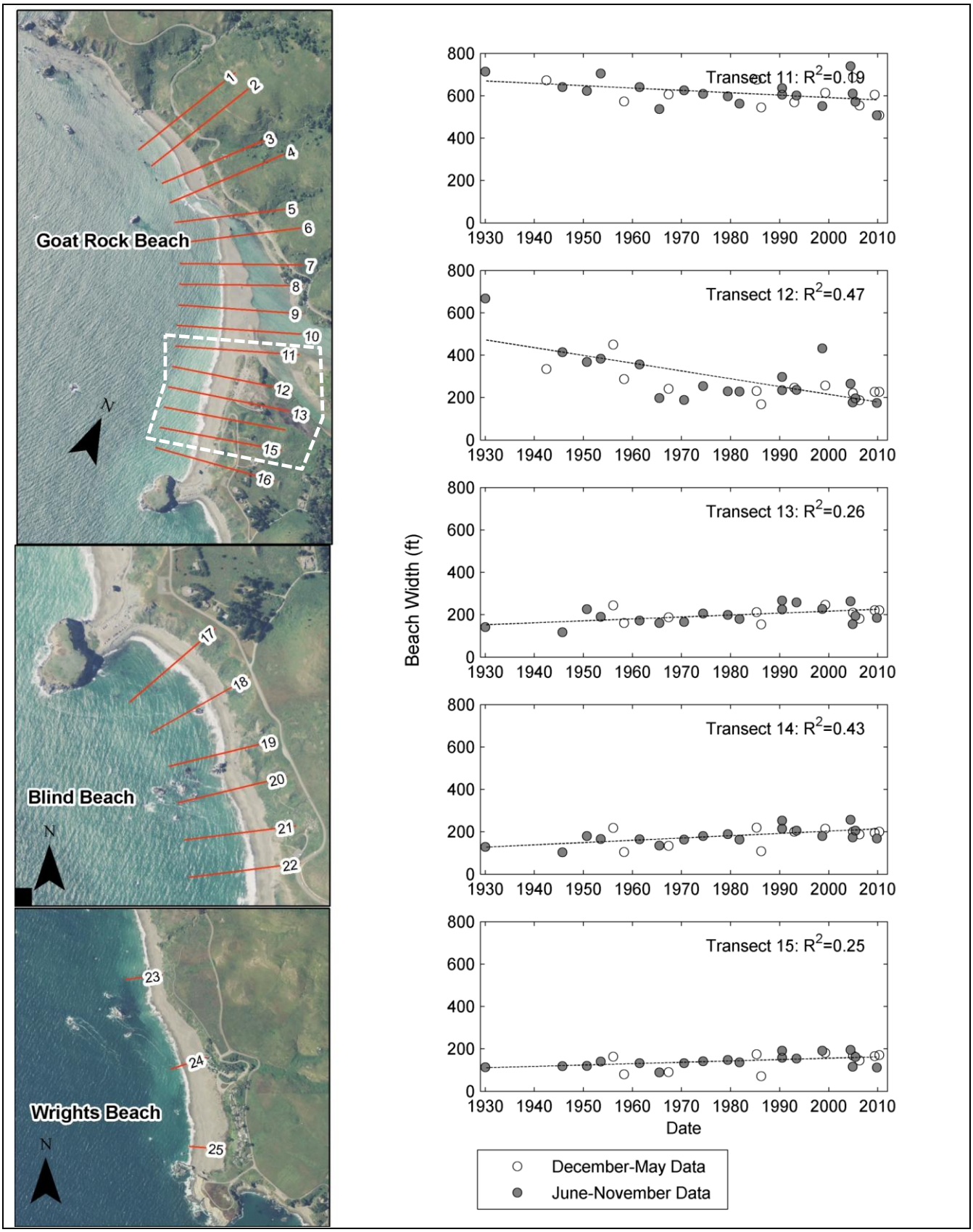
(left) Index of transects and **(right)** corresponding beach width data for transects 1-5.



NOTE: All linear trends were obtained using summer-fall data.

Figure B-7

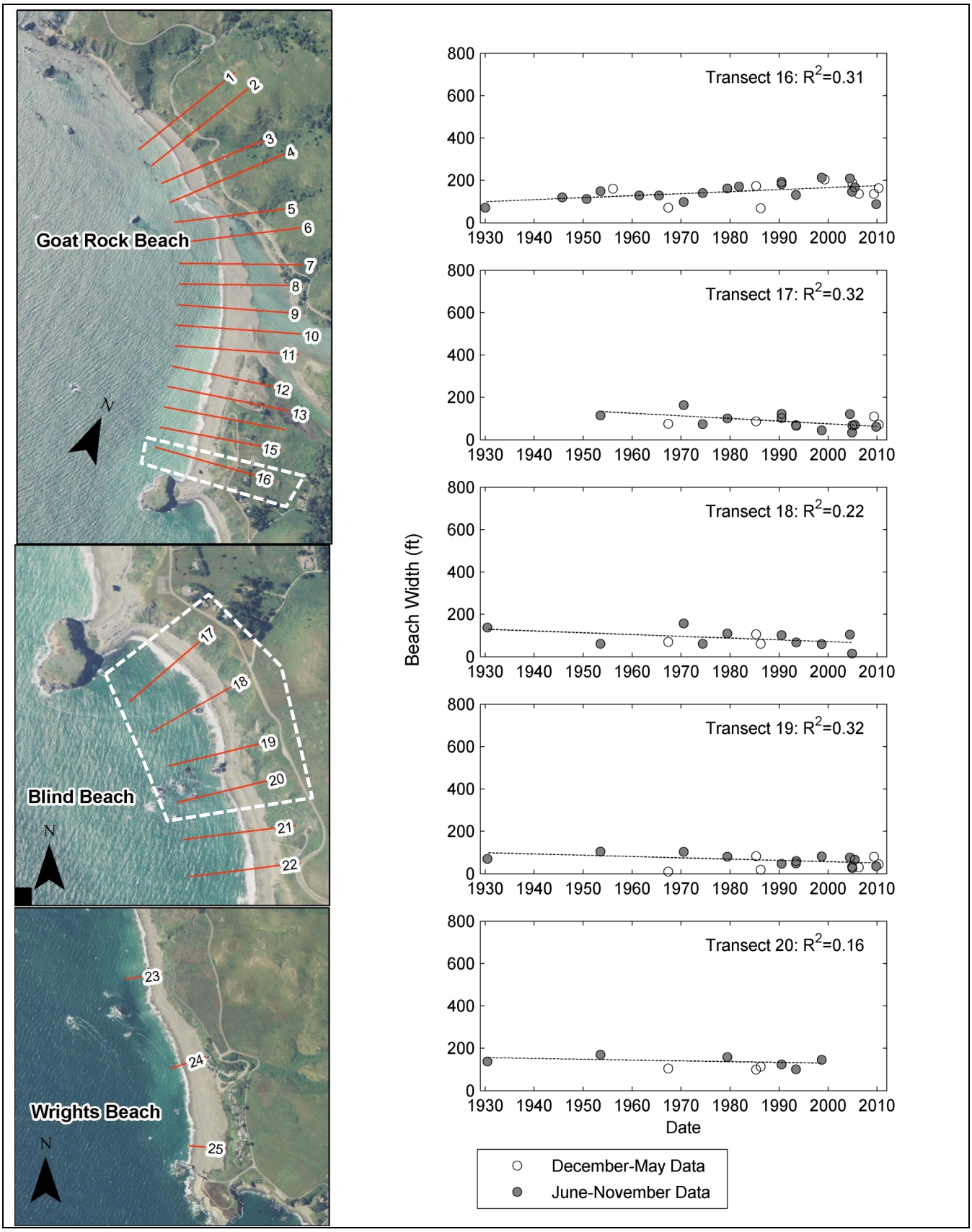
(left) Index of transects and **(right)** corresponding beach width data for transects 6-10.



NOTE: All linear trends were obtained using summer-fall data.

Figure B-8

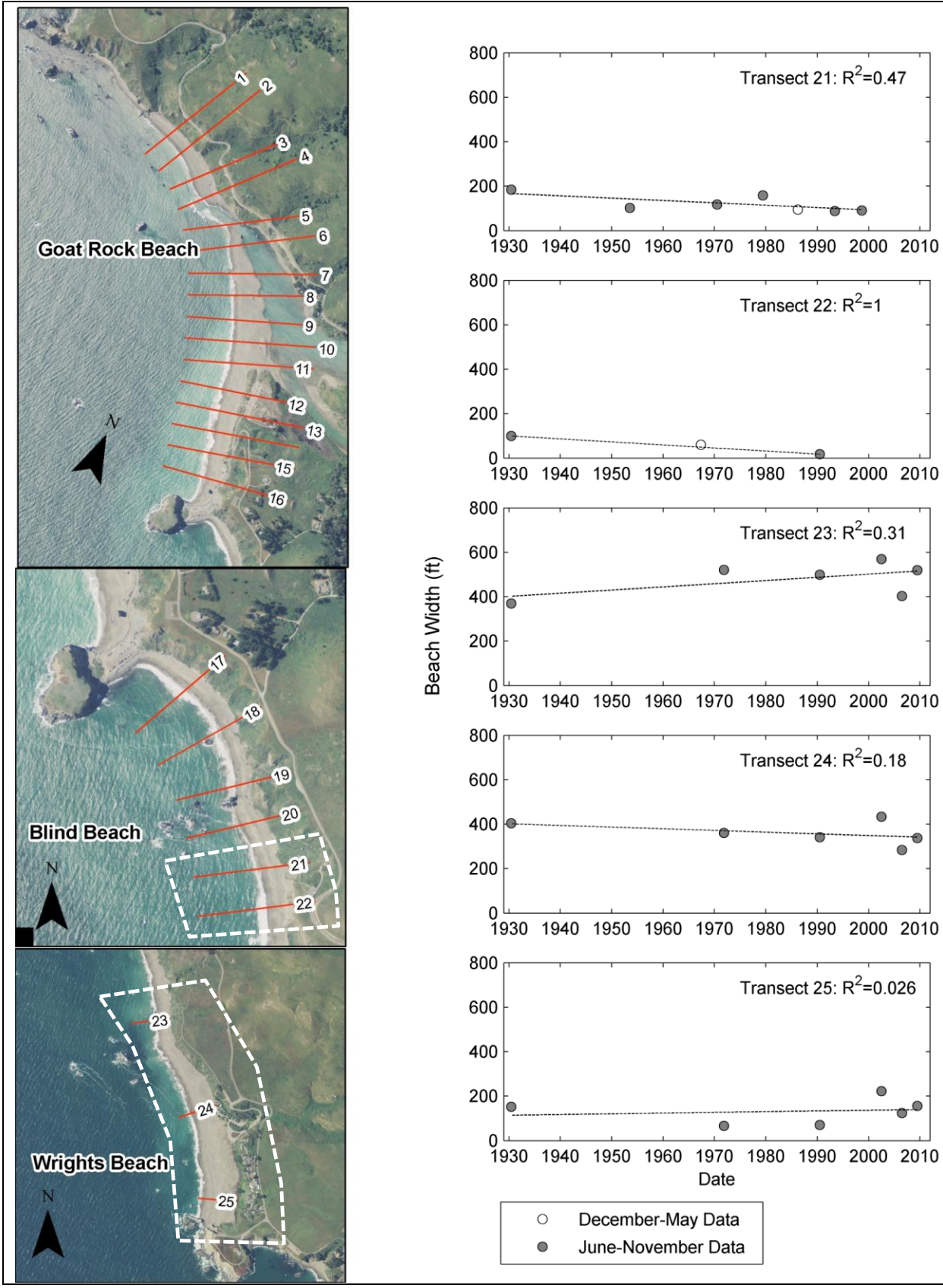
(left) Index of transects and **(right)** corresponding beach width data for transects 11-15.



NOTE: All linear trends were obtained using summer-fall data.

Figure B-9

(left) Index of transects and **(right)** corresponding beach width data for transects 16-20.



NOTE: All linear trends were obtained using summer-fall data.

Figure B-10

(left) Index of transects and **(right)** corresponding beach width data for transects 21-25.

APPENDIX C. ESTUARY WIND WAVE ANALYSIS

This appendix describes the assessment of wind waves included in the flood risk portion of the study, Section 7. It provides information about the methods used to estimate extreme wind waves generated inside the Russian River Estuary (RRE) during an inlet closure event.

This analysis used wind data measured at the Bodega Marine Laboratory (BML), approximately 12 miles south of the estuary. Winds in the estuary are likely influenced by topographic effects owing to the canyon-like shape of the estuary, in addition to sea-breeze augmentation (Behrens, 2012). However, the differences between the winds at the RRE and at BML are probably time-varying and complex. Since the part of the estuary considered in the flood risk analysis (Section 7) is the farthest seaward extent of the estuary (and thus most similar to BML conditions), and since wind data are not available within the estuary, we use the BML data to characterize the estuary winds.

As discussed in Section 7.2.4, wind waves are generated inside the estuary by winds blowing across the exposed water surface. These are probably largest during inlet closure events, since these events are tied to higher water levels, and thus greater expanses of open water. Wave generation was approximated using the methodology of Resio et al. (2006). The wind velocity measured on land (U_L) was transferred to a wind velocity over the estuary water (U_w) using the following relation:

$$U_w = U_L\left(1.2 + \frac{1.85}{U_L}\right)\left\{1 - \frac{\Delta T}{|\Delta T|}\left(\frac{\Delta T}{1920}\right)^{1/3}\right\} \qquad \text{(C-1)}$$

where $\Delta T = T_{air} - T_{sea}$ is the air-water temperature difference, which we assume is close to zero for the estuary. Methods provided by the Coastal Engineering Manual (Resio et al., 2006) were then used to relate U_w to the shear velocity u_*:

$$u_* = \sqrt{0.001(1.1 + 0.035U_w)U_w^2} \qquad \text{(C-2)}$$

Before determining the fetch-limited wave height, dimensionless wave height, fetch length, and peak wave frequency are defined:

$$\hat{H} = \frac{gH_{m0}}{u_*^2} \qquad \text{(C-3)}$$

$$\hat{X} = \frac{gX}{u_*^2} \tag{C-4}$$

$$\hat{f}_p = \frac{u_* f_p}{g} \tag{C-5}$$

where g is the acceleration due to gravity, H_{m0} is the significant wave height or energy based wave height, *X* is the fetch length over which the wind blows, and f_p is the frequency of the spectral peak (the inverse of the peak period $1/T_p$). The idealized, fetch limited wave height and frequency are then expected to follow relationships of the form:

$$\hat{H} = \lambda_1 \hat{X}^{m_1} \tag{C-6}$$

$$\hat{f}_p = \lambda_2 \hat{X}^{m_2} \tag{C-7}$$

Where λ and *m* are dimensionless coefficients provided by the Shore Protection Manual (USACE, 1984). Once the dimensionless wave height and frequency are solved, these are used in Equations C-3 and C-5, respectively, to determine the dimensional wave height and frequency. The latter is inverted to give the wave period.

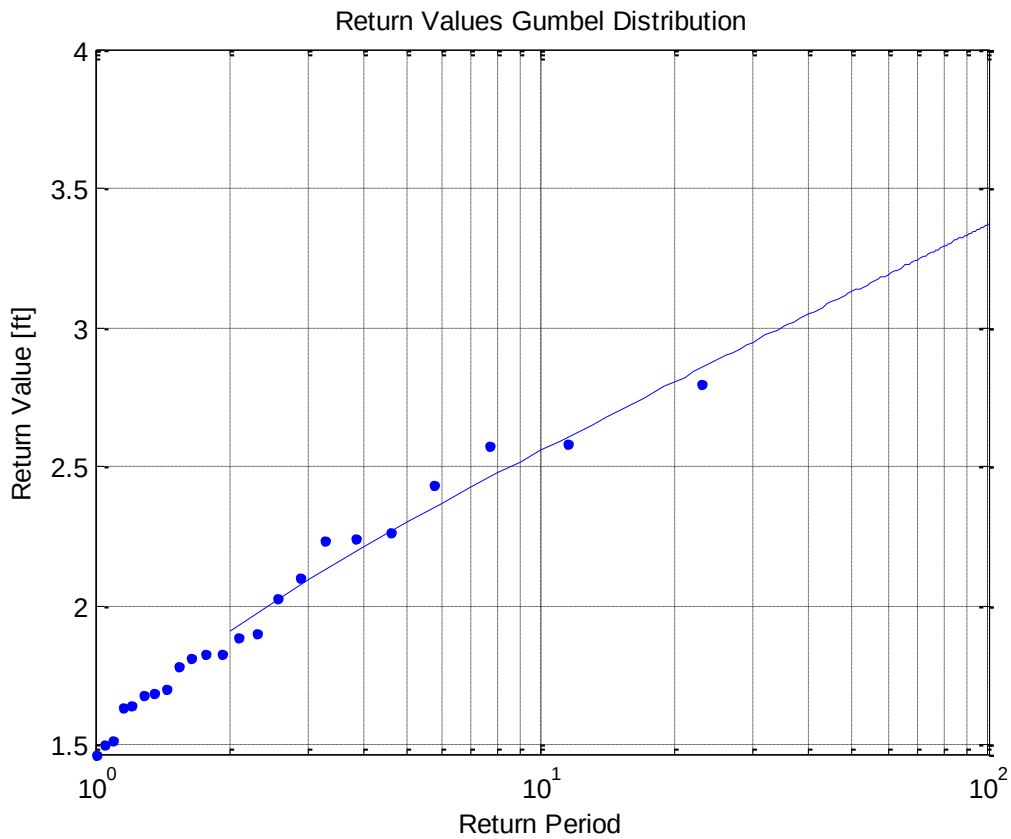
Predicted wind waves were generated using BML wind data from 1988 to 2011, and fit to the Gumbel distribution to estimate wind-wave heights during events with long return periods (Figure C-1). Wave runup was calculated using the same methodology as discussed in Section 4. The estuary side slope influences the potential for runup, and was varied from 1:2 to 1:10 (Figures C-2, C-3). Peak yearly values and predictions for low-recurrence values are given in Tables C-1 and C-2.

Table C - 1. Maximum wave heights values from 1988 to 2011, in descending order.

Year	Month	Day	U_{10} (mph)	Direction	T_p(s)	H_{max} (ft)	Runup (ft)			
							Slope 1:2	Slope 1:3	Slope 1:5	Slope 1:10
2006	12	27	90.94	300	2.08	2.79	3.14	2.35	1.73	1.27
2008	6	24	85.41	290	2.02	2.58	2.93	2.19	1.61	1.18
1993	2	20	85.33	245	2.02	2.58	2.93	2.19	1.61	1.18
1997	11	26	81.64	281	1.99	2.44	2.79	2.08	1.53	1.12
2001	11	24	77.10	267	1.94	2.26	2.62	1.96	1.43	1.05
2002	12	28	76.55	248	1.93	2.24	2.60	1.94	1.42	1.04
2005	12	31	76.28	275	1.93	2.23	2.59	1.93	1.42	1.04
2005	6	5	72.70	249	1.89	2.10	2.46	1.83	1.34	0.98
1996	3	5	70.60	244	1.87	2.03	2.38	1.78	1.30	0.95
1996	12	22	66.92	265	1.83	1.90	2.25	1.68	1.23	0.89
1988	12	24	66.60	300	1.82	1.89	2.24	1.67	1.22	0.89
1998	11	7	64.92	269	1.80	1.83	2.17	1.62	1.18	0.86
1995	1	5	64.87	292	1.80	1.82	2.17	1.62	1.18	0.86
2010	3	10	64.46	268	1.80	1.81	2.16	1.61	1.18	0.86
2011	4	7	63.71	300	1.79	1.78	2.13	1.59	1.16	0.84
1994	2	17	61.31	298	1.76	1.70	2.05	1.52	1.11	0.81
2008	12	25	60.93	247	1.76	1.69	2.03	1.51	1.11	0.80
2003	12	10	60.68	246	1.76	1.68	2.02	1.51	1.10	0.80
1991	5	8	59.65	299	1.74	1.65	1.99	1.48	1.08	0.78
1992	1	5	59.22	251	1.74	1.63	1.97	1.47	1.07	0.78
1990	1	30	55.89	253	1.70	1.52	1.86	1.38	1.01	0.73
2001	2	11	55.36	269	1.69	1.50	1.84	1.37	1.00	0.72
2000	1	16	54.31	250	1.68	1.47	1.80	1.34	0.97	0.71

Table C - 2. Gumbel fit of wave height and run up using data from 1988 to 2011.

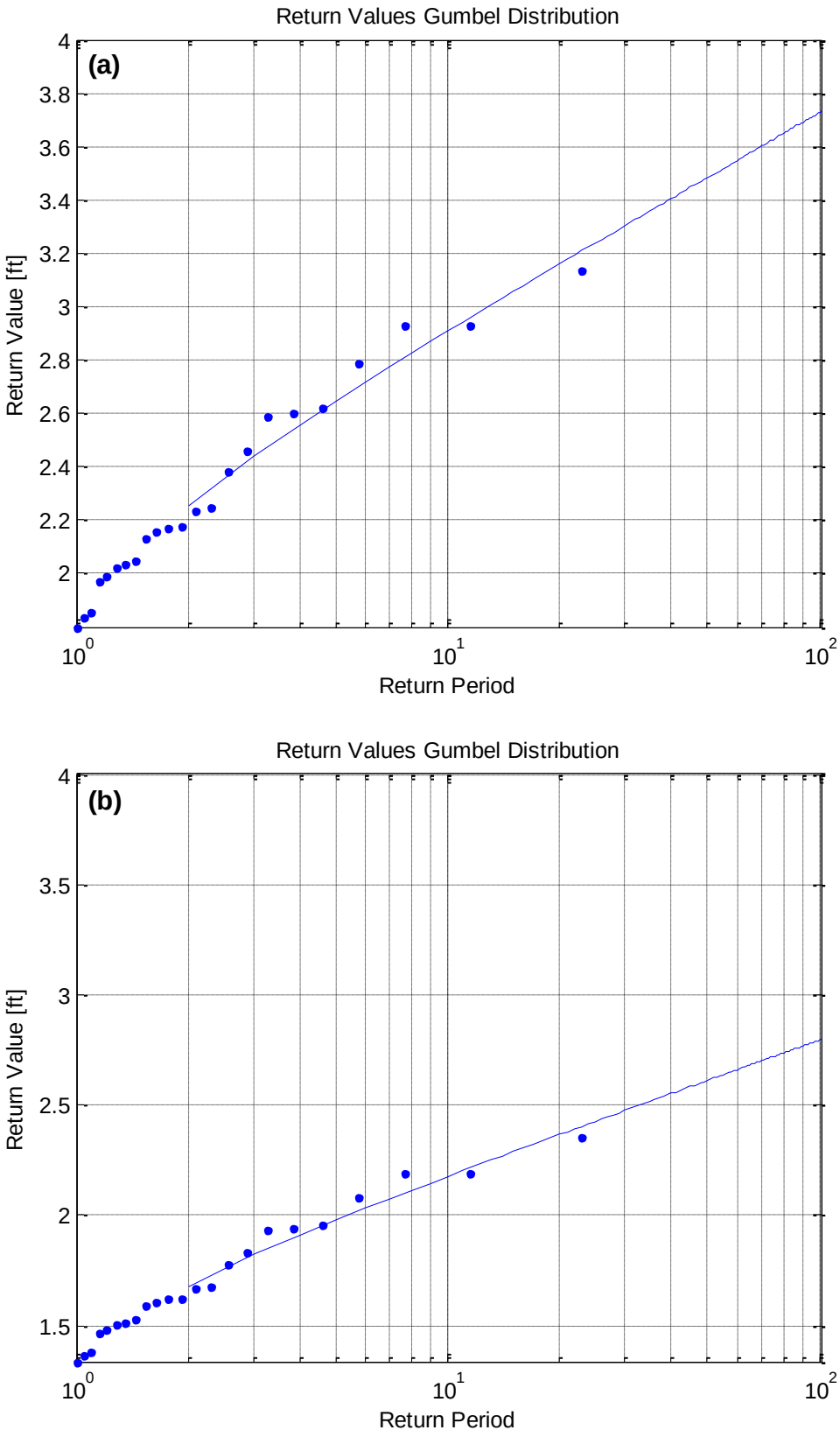
Return Period (Gumbel)	H_{max} (ft)	Runup (ft)			
		Slope 1:2	Slope 1:3	Slope 1:5	Slope 1:10
1	1.91	2.25	1.68	1.23	0.90
5	2.37	2.72	2.03	1.49	1.09
10	2.59	2.94	2.20	1.62	1.19
20	2.82	3.18	2.38	1.75	1.29
50	3.13	3.49	2.62	1.93	1.42
100	3.37	3.73	2.80	2.06	1.52



NOTE: Wind-wave heights estimated after Resio et al. (2006) using wind data from the Bodega Marine Laboratory.

Figure C-1

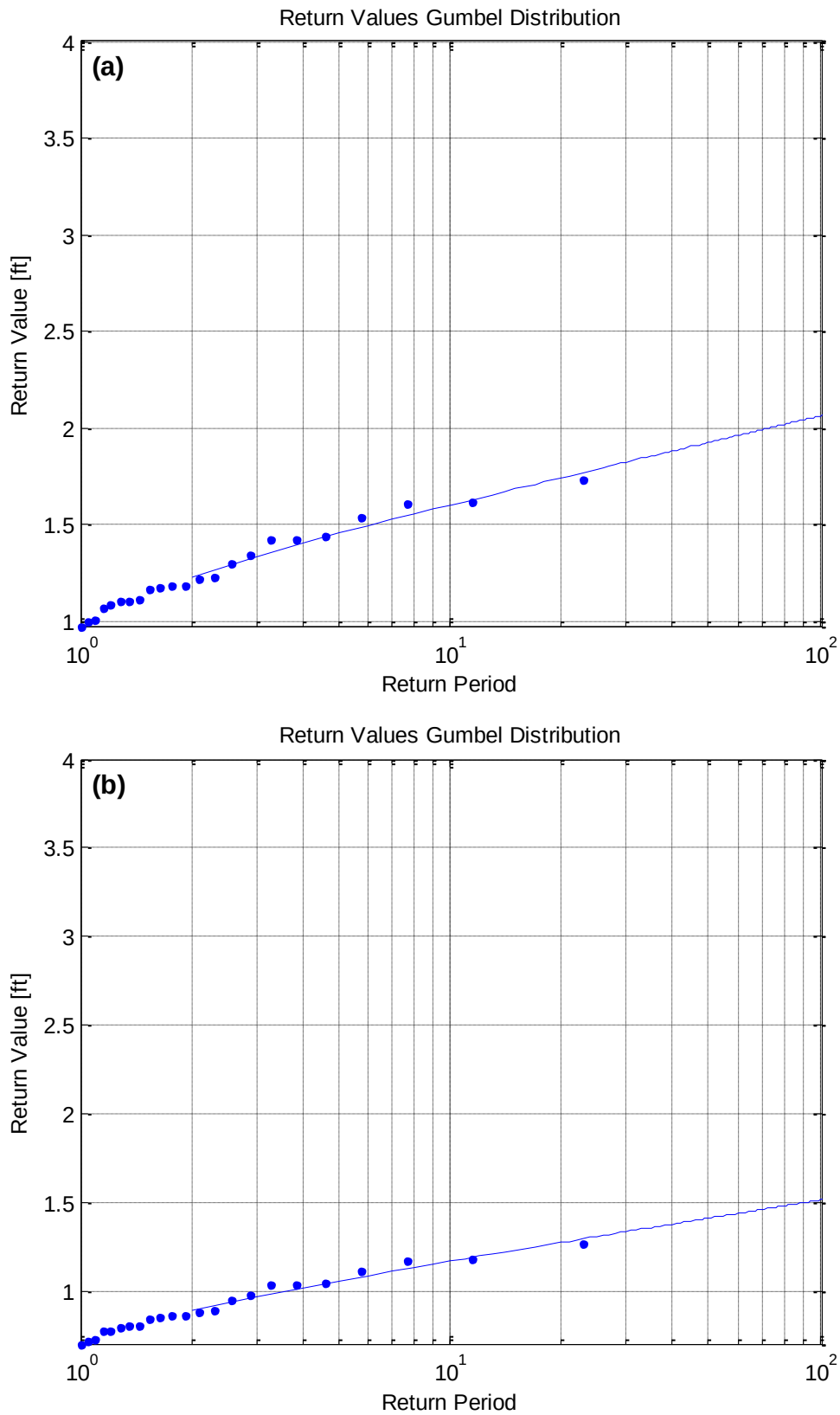
Predicted wind-wave heights and return periods (years).



NOTE: Wind-wave runup calculated using wind-wave heights estimated after Resio et al. (2006) using wind data from the Bodega Marine Laboratory.

Figure C-2

Predicted wind-wave runup and return periods (years) for estuary shoreline slope of (a) 1:2 and (b) 1:3.



NOTE: Wind-wave runup calculated using wind-wave heights estimated after Resio et al. (2006) using wind data from the Bodega Marine Laboratory.

Figure C-3

Predicted wind-wave runup and return periods (years) for estuary shoreline slope of (a) 1:5 and (b) 1:10.

APPENDIX D. SOIL SAMPLE PERMEABILITY

Constant Head Permeability Test
ASTM D 2434

CTL Job No:	381-025	**Boring:**	Core 1	**Date:**	5/23/2014
Client:	ESA-PWA	**Sample:**		**By:**	PJ
rojject Name:	Goat Rock State Beach Jetty Permeability Study	**Depth, ft:**	1		
Project No.:	D211669.00				

Soil Description: Dark Olive Gray Poorly Graded SAND w/ Gavel

Remolding Data:

Constant Head Calculation, K=QL/thA

Test #	Elapsed Time t, (sec)	Volume Q, (cc)	Head Loss h (cm)	Water Temp (°c)	Hydraulic Gradient	Coef. Of Permeabilit K, (cm/sec)
1	18	102	0.3	22.0	0.05	3.93
2	18	103	0.3	22.0	0.05	3.97
3	17	97	0.3	22.0	0.05	3.95
4	15	85	0.3	22.0	0.05	3.93
5	17	93	0.4	22.0	0.06	2.84
6	18	101	0.4	22.0	0.06	2.92

Average Permeability (cm/sec):	**3.6**
Average Permeability (in/hr):	**5087**

Sample Data:		**Initial**		**Final**
Height, (L)	**in.:**	4.87		4.86
Diameter,	**in.:**	2.40		2.40
Area, (A)	**in²:**	4.51		4.51
Volume,	**in³:**	21.96		21.94
Total Volume.	**cc:**	360		360
Vol. of Solids,	**cc:**	234		234
Vol. of Voids,	**cc:**	125		125
Void Ratio	**e:**	0.54		0.53
Porosity,	**%:**	34.9		34.8
Saturation,	**%:**	13.4		100.0
Sp. Gravity:		2.65	assumed	2.65
Wet Weight,	**gm:**	638.1		746.4
Dry Weight	**gm:**	621.3		621.3
Moisture,	**%:**	2.7		20.1
Density,	**pcf:**	107.7		107.8

Remarks: A significant amount of fines migrated out of the sample during the saturation and testing phases. The migration of fines can have a significant impact on the permeability.

Constant Head Permeability Test
ASTM D 2434

CTL Job No:	381-025	**Boring:**	Core 2	**Date:**	5/23/2014
Client:	ESA-PWA	**Sample:**		**By:**	PJ
Project Name:	Goat Rock State Beach Jetty	**Depth, ft:**	1		
Project No.:	D211669.00				

Soil Description: Dark Olive Gray Poorly Graded SAND

Remolding Data:

		Constant Head Calculation, K=QL/thA				
Test #	**Elapsed Time t, (sec)**	**Volume Q, (cc)**	**Head Loss h (cm)**	**Water Temp (°C)**	**Hydraulic Gradient**	**Coef. Of Permeability K, (cm/sec)**
1	60	86	0.3	23.0	0.05	0.97
2	60	86	0.3	23.0	0.05	0.97
3	40	58	0.3	23.0	0.05	0.98
4	35	50	0.3	23.0	0.05	0.97
		Average Permeability (cm/sec):				**0.97**
		Average Permeability (in/hr):				**1378**

Sample Data:		**Initial**		**Final**
Height, (L)	**in.:**	4.51		4.44
Diameter,	**in.:**	2.40		2.40
Area, (A)	**in^2:**	4.51		4.51
Volume,	**in^3:**	20.34		20.01
Total Volume.	**cc:**	333		328
Vol. of Solids,	**cc:**	204		204
Vol. of Voids,	**cc:**	130		124
Void Ratio	**e:**	0.64		0.61
Porosity,	**%:**	38.9		37.9
Saturation,	**%:**	12.5		100.0
Sp. Gravity:		2.65	assumed	2.65
Wet Weight,	**gm:**	555.9		664.0
Dry Weight	**gm:**	539.7		539.7
Moisture,	**%:**	3.0		23.0
Density,	**pcf:**	101.0		102.7

Remarks:

Constant Head Permeability Test
ASTM D 2434

CTL Job No:	381-025	Boring:	Core 4	Date:	5/23/2014
Client:	ESA -PWA	Sample:		By:	PJ
Project Name:	Goat Rock State	Depth, ft:	1		
Project No.:	D211669.00				

Soil Description: Very Dark Brown Poorly Graded SAND

Remolding Data:

		Constant Head Calculation, K=QL/thA				
Test #	Elapsed Time t, (sec)	Volume Q, (cc)	Head Loss h (cm)	Water Temp (°C)	Hydraulic Gradient	Coef. Of Permeability K, (cm/sec)
1	220	40	0.4	19.7	0.06	0.10
2	150	28	0.4	19.7	0.06	0.10
3	150	28	0.4	19.7	0.06	0.10
4	220	41	0.4	19.7	0.06	0.10
5	225	42	0.4	19.7	0.06	0.10
6	240	69	0.5	19.7	0.08	0.12
7	240	69	0.5	19.7	0.08	0.12
		Average Permeability (cm/sec):				0.11
		Average Permeability (in/hr):				152

Sample Data:		Initial		Final
Height, (L)	in.:	5.48		5.13
Diameter,	in.:	2.42		2.42
Area, (A)	in^2:	4.60		4.60
Volume,	in^3:	25.21		23.60
Total Volume.	cc:	413		387
Vol. of Solids,	cc:	253		253
Vol. of Voids,	cc:	160		134
Void Ratio	e:	0.63		0.53
Porosity,	%:	38.7		34.6
Saturation,	%:	9.9		100.0
Sp. Gravity:		2.65	assumed	2.65
Wet Weight,	gm:	686.3		804.1
Dry Weight	gm:	670.4		670.4
Moisture,	%:	2.4		19.9
Density,	pcf:	101.3		108.2

Remarks:

Constant Head Permeability Test
ASTM D 2434

CTL Job No:	381-025	**Boring:**	Core 5	**Date:**	5/23/2014
Client:	ESA-PWA	**Sample:**		**By:**	PJ
Project Name:	Goat Rock State Beach Jetty	**Depth, ft:**	1		
Project No.:	D211669.00				

Soil Description: Very Dark Brown Poorly Graded SAND

Remolding Data:

		Constant Head Calculation, K=QL/thA				
Test #	**Elapsed Time t, (sec)**	**Volume Q, (cc)**	**Head Loss h (cm)**	**Water Temp (°C)**	**Hydraulic Gradient**	**Coef. Of Permeability K, (cm/sec)**
1	120	65	0.7	21.4	0.11	0.163
2	120	65	0.7	21.4	0.11	0.163
3	120	65	0.7	21.5	0.11	0.163
4	205	112	0.7	21.6	0.11	0.164
5	90	65	1.0	21.7	0.16	0.151
6	120	87	1.0	21.7	0.16	0.152
7	70	51	1.0	21.7	0.16	0.153
8	45	33	1.0	21.7	0.16	0.154
9	50	46	1.2	21.7	0.19	0.161
10	100	93	1.2	21.7	0.19	0.162
		Average Permeability (cm/sec):				**0.16**
		Average Permeability (in/hr):				**225**

Sample Data:		**Initial**		**Final**
Height, (L)	**in.:**	4.96		4.78
Diameter,	**in.:**	2.40		2.40
Area, (A)	**in^2:**	4.51		4.51
Volume,	**in^3:**	22.38		21.57
Total Volume.	**cc:**	367		353
Vol. of Solids,	**cc:**	230		230
Vol. of Voids,	**cc:**	137		123
Void Ratio	**e:**	0.59		0.54
Porosity,	**%:**	37.2		34.9
Saturation,	**%:**	11.7		100.0
Sp. Gravity:		2.65	assumed	2.65
Wet Weight,	**gm:**	626.0		733.3
Dry Weight	**gm:**	610.0		610.0
Moisture,	**%:**	2.6		20.2
Density,	**pcf:**	103.8		107.7

Remarks:

Constant Head Permeability Test
ASTM D 2434

CTL Job No:	381-025	**Boring:**	Core 6	**Date:**	5/23/2014
Client:	ESA-PWA	**Sample:**		**By:**	PJ
roject Name:	Goat Rock State Beach	**Depth, ft:**	1		
Project No.:	D211669.00				

Soil Description: Dark Olive Gray Poorly Graded SAND

Remolding Data:

Constant Head Calculation, K=QL/thA

Test #	Elapsed Time t, (sec)	Volume Q, (cc)	Head Loss h (cm)	Water Temp (°c)	Hydraulic Gradient	Coef. Of Permeabilit K, (cm/sec)
1	160	86	0.6	22.7	0.09	0.1832
2	180	98	0.6	22.7	0.09	0.1856
3	130	71	0.6	22.7	0.09	0.1862
4	100	55	0.6	22.7	0.09	0.1875

Average Permeability (cm/sec):	**2.E-01**
Average Permeability (in/hr):	**263**

Sample Data:		**Initial**		**Final**
Height, (L)	**in.:**	4.41		4.40
Diameter,	**in.:**	2.40		2.40
Area, (A)	**in²:**	4.51		4.51
Volume,	**in³:**	19.92		19.86
Total Volume.	**cc:**	326		325
Vol. of Solids,	**cc:**	210		210
Vol. of Voids,	**cc:**	117		116
Void Ratio	**e:**	0.56		0.55
Porosity,	**%:**	35.7		35.5
Saturation,	**%:**	7.0		100.0
Sp. Gravity:		2.65	assumed	2.65
Wet Weight,	**gm:**	564.3		671.6
Dry Weight	**gm:**	556.1		556.1
Moisture,	**%:**	1.5		20.8
Density,	**pcf:**	106.3		106.7

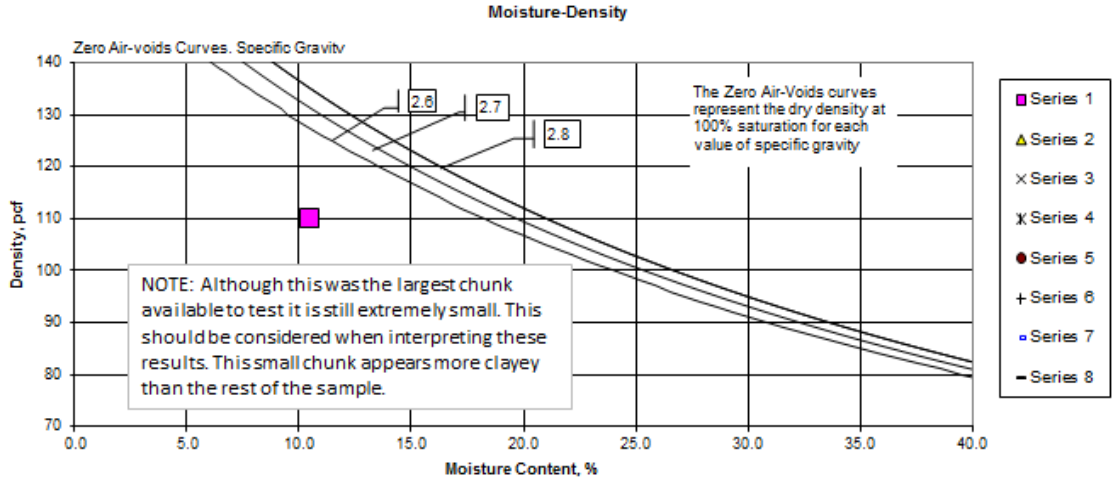
Remarks: The sample was fairly loose in the liner and may have been disturbed

Chunk Density
(ASTM D7263-A)

CTL Job No: 381-025 **Project No.:** D211669.00
Client: ESA-PWA **Date:** 5/29/14
Project Name: Goat Rock State Beach Jetty Feasibility Study **By:** PJ

Boring:	Core 8							
Sample:								
Depth, ft:	7-7.5							
Visual Description:	Dark Brown CLAY w/ Sand							
Actual G_s								
Assumed G_s	2.70							
Total Vol cc	15.48							
Vol Solids,cc	10.10							
Vol Voids,cc	5.38							
Moisture, %	10.5							
Wet Unit wt, pcf	121.5							
Dry Unit wt, pc	109.9							
Saturation, %	53.3							
Porosity, %	34.8							
Void Ratio	0.533							
Series	1	2	3	4	5	6	7	8

Note: If an assumed specific gravity (Gs) was used then the saturation, porosities, and void ratio should be considered approximate.



Hydraulic Conductivity
ASTM D 5084
Method C: Falling Head Rising Tailwater

Job No:	381-025	Boring:	Core 8	Date:	06/26/14
Client:	ESA-PWA	Sample:		By:	MD/PJ
Project:	D211669.00	Depth, ft.:		Remolded:	109.9 @ 10.5%

Visual Classification: Dark Brown Clayey SAND

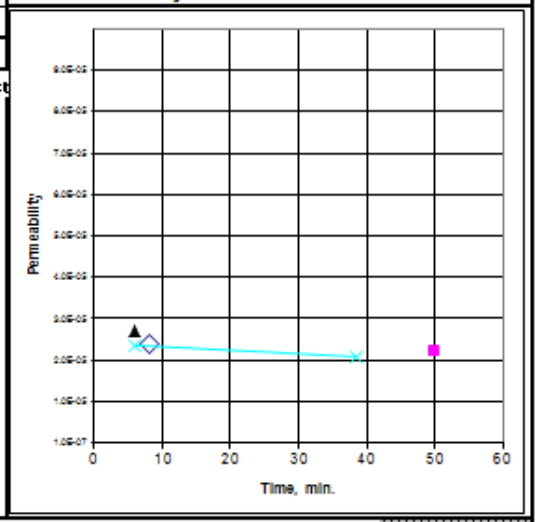
Max Sample Pressures, psi:

Cell:	Bottom	Top	Avg. Sigma3
73.5	68.5	68.5	5

B: = >0.95 "B" is an indication of saturation

Max Hydraulic Gradient: = 5

Date	Minutes	Head, (in)	K,cm/sec
6/9/2014	0.00	15.00	Start of Test
6/9/2014	8.00	13.10	2.4E-05
6/9/2014	50.00	6.90	2.2E-05
6/11/2014	6.00	13.40	2.7E-05
6/11/2014	6.00	13.60	2.4E-05
6/11/2014	38.50	8.60	2.1E-05



Average Hydraulic Conductivity: 2.E-05 cm/sec

Sample Data:	Initial (As-Received)		Final (At-Test)
Height, in	3.00		2.98
Diameter, in	2.38		2.37
Area, in2	4.43		4.40
Volume in3	13.29		13.13
Total Volume, cc	217.8		215.2
Volume Solids, cc	138.3		138.3
Volume Voids, cc	79.5		76.9
Void Ratio	0.6		0.6
Total Porosity, %	36.5		35.7
Air-Filled Porosity (θa),%	18.1		0.6
Water-Filled Porosity (θw),%	18.4		35.1
Saturation, %	50.3		98.2
Specific Gravity	2.75	Assumed	2.75
Wet Weight, gm	420.3		455.8
Dry Weight, gm	380.3		380.3
Tare, gm	0.00		0.00
Moisture, %	10.5		19.9
Wet Bulk Density, pcf	120.4		132.2
Dry Bulk Density, pcf	109.0		110.3
Wet Bulk Dens.ρb, (g/cm^3)	1.93		2.12
Dry Bulk Dens.ρb, (g/cm^3)	1.75		1.77

Remarks:

SOURCE: NAIP 2007 aerial image.

Goat Rock Jetty Feasibility Study . D211669.00

Figure D-1

Sediment core locations

APPENDIX E. GEOPHYSICAL INVESTIGATION: GOAT ROCK STATE BEACH (NORCAL, 2015)

GEOPHYSICAL INVESTIGATION
GOAT ROCK STATE BEACH

Russian River Estuary Management Project

NORCAL Project No. 14-165.12

April, 2015

Prepared for:

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1.0 SUMMARY

NORCAL Geophysical Consultants, Inc. conducted a geophysical investigation at Goat Rock State Beach in Sonoma County, California. The geophysical investigation is part of the Russian River Estuary Management Project being conducted by the Sonoma County Water Agency (SCWA). The purpose of the geophysical investigation was to explore for ancient river channels beneath the barrier beach at the mouth of the Russian River and to evaluate the continuity of the jetty that transects the central portion of the beach and whether the jetty is founded on sand dune materials, terrace deposits or bedrock.

The geophysical investigation consisted of a seismic refraction (SR) survey, an electrical resistivity (SR) survey and a ground penetrating radar (GPR) survey. The SR and ER surveys were used to delineate the configuration of the bedrock surface in order to determine if the barrier beach is underlain by one large channel or by multiple smaller channels. Other objectives were to evaluate the lateral continuity of the bedrock in terms of hardness and weathering characteristics, and to determine the general composition of the materials filling the river channel(s). The GPR survey was used to evaluate the jetty.

The results of the SR and ER surveys indicate that the portion of the barrier beach between the vegetated sand dunes on the south and the outlet channel on the north is underlain by two relatively deep channels. The seismic velocities of the materials filling the two channels suggest that the channels are filled by saturated unconsolidated sediments, possibly river sediments and/or terrace deposits. The deeper portions of the channels, as defined by the seismic refraction and electrical resistivity models, may also consist of deeply weathered rock. The very low resistivities of the material in the channels suggest that they are saturated by relatively saline water.

The results of the ground penetrating radar survey indicate that the jetty is missing or offset in at least two locations. The GPR survey did not penetrate deep enough to define the bottom of the jetty.

2.0 INTRODUCTION

This report presents the findings of a geophysical investigation conducted by NORCAL Geophysical Consultants, Inc. for the Sonoma County Water Agency (SCWA) at Goat Rock State Beach in Sonoma County, California. The geophysical investigation was authorized under Agreement No. TW 11/12-023 between the SCWA and NORCAL dated October 21, 2011, by Amendment No. 1 dated October 25, 2012 and by Amendment No. 2 dated December 13, 2013. The geophysical investigation field work was conducted during the period of July 21-24 and 29, 2014 by California Professional Geophysicist William E. Black (PGp No. 843) with assistance from Geophysical Technicians Christopher J. Bissiri and Erasmo Tapia. Mr. Chris Delaney of the SCWA provided background information, site access, site logistical support and on-site health and safety information. The field operations were observed by various SCWA interns and by personnel from the Lawrence Berkeley National Laboratories (LBNL) in Berkeley, California.

2.1 GOAT ROCK STATE BEACH

Goat Rock State Beach is located in northwestern Sonoma County, California. It is a sub-unit of the Sonoma Coast State Beach that is owned and managed by the State of California. The beach is situated on the west side of Highway 1 about one kilometer (km) west of the town of Jenner. Access to the beach is provided from Highway 1 by Goat Rock Road. Paved parking areas are situated on the Goat Rock isthmus and among the vegetated sand dunes just south of the Russian River.

The crescent shaped beach extends from Goat Rock on the south to the base of the cliffs bordering Highway 1 on the north, as shown in Figure 1. Goat Rock (photo at left) is an iconic outcrop of the Sonoma Coast. The rock is barely attached to the mainland by a narrow isthmus that was created by previous quarrying operations. The northernmost 730 meters (m) of

the beach forms a sand bar that extends across the mouth of the Russian River estuary. The estuary usually closes during the spring, summer and fall when river flows are relatively low and long period waves transport sand landward rebuilding the beach that was removed by winter waves and river outflows. When this occurs the SCWA mechanically opens an outlet channel to alleviate potential flooding of low lying shore-line properties near the town of Jenner. The agency also manages the water level in the estuary in order to minimize tidal influence and to enhance summer rearing habitat for steelhead.

Figure 1: Goat Rock State Beach and vicinity.

2.2 MARINE MAMMALS

Goat Rock State Beach is a haul-out spot for migratory species, including California sea lions, northern elephant seals, northern fur seals and harbor seals. These marine mammals (also known as pinnipeds) use the protected area of the Russian River outlet and sandy barrier beach for resting and refuge (photo at left). The pinnipeds are monitored in order to detect their response to the SCWA's estuary management activities as authorized under a Marine Mammal Protection Permit Incidental Harassment Authorization. One of the conditions for this authorization was that the work could not be initiated until the end of the seal pupping season. Furthermore, the SCWA established and maintained a seal monitoring program while the geophysical investigation was in progress. This involved having a marine biologist observe the seals from the cliffs to the north of the work area. The biologist was in constant radio contact with SCWA personnel stationed with the geophysical field crew. This made it possible to minimize contact with, or harassment of, the seals and their pups as the work progressed.

2.3 SITE GEOLOGY

Over the last geologic epoch the Sonoma County coast has been subject to uplift, a process combined with marine erosion, which has created a marine terrace. In the vicinity of Goat Rock State Beach the terrace ranges in elevation from 10- to 45-m above mean sea level. This has resulted in a steep bluff directly above the littoral zone, and a succession of terrace levels. The rocks underlying the terrace deposits consist of basalt, chert, marine sandstone, mélange and greywacke of the Franciscan assemblage. Goat Rock consists of the latter. The San Andreas fault runs parallel to and near the coastline. Most of the beach sands consist of medium

gray to brown coarse sandy materials reflecting the high rate of erosion of escarpment materials into the ocean.

The SCWA installed five monitoring wells within the Goat Rock State Beach study area. The locations of these wells, labeled MW-1 through MW-5, are shown on Plates 1 and 3. Each well was 12-m deep and was screened over its entire length. The materials encountered by the wells consisted of well graded sands similar to the dune sands at the ground surface. None of the wells encountered terrace deposits or bedrock. All five wells encountered groundwater, at depths ranging from 2.1- to 5.6-m. Given the surface elevations of the wells this equated to groundwater elevations ranging from 0.0- to 1.5-m above mean sea level.

2.4 JETTY STUDY

In the era circa 1920 a large quarrying operation was conducted at Goat Rock. This activity resulted in the apparent separation of Goat Rock from the mainland and the formation of the isthmus that is now occupied by a parking lot. Quarry products were transported from Goat Rock to the mouth of the Russian River via a now abandoned rail line. Some of the materials were used to construct a jetty designed to keep the river mouth open to shipping. This process was maintained over a period of two decades but was eventually abandoned when the jetty failed in its purpose. Today most of the jetty, including fragments of the old rail line, is now buried under sand.

The Russian River Biological Opinion requires the SCWA to study the jetty to determine if and how it impacts the formation of the sand bar (barrier beach) at the mouth of the Russian River estuary and the impact of the jetty on water levels in the estuary. The study consists of three components as follows:

1. Monitoring wells to determine how and where water seeps through the barrier beach and jetty, both from the river and the ocean.
2. Seismic refraction, electrical resistivity and ground penetrating radar surveys to map materials that have been buried by sand.

3. Electromagnetic profiles to determine how the jetty impacts water seeping through the beach.

2.5 OBJECTIVES

NORCAL was contracted by the SCWA to conduct the studies itemized in number 2, above. Our specific objectives in conducting these geophysical surveys were as follows:

1. Determine the general depth and configuration of the bedrock surface in order to determine if the mouth of the Russian River is characterized by one large bedrock channel or multiple smaller channels.
2. Evaluate the lateral continuity of the bedrock in terms of hardness, weathering characteristics, etc. This is because lateral variations in the velocity of the rock, or a decrease in its depth may be indicative of older terrace material overlying rock.
3. Determine the relative thickness of the unsaturated and saturated beach sands and whether they overly terrace deposits or bedrock.
4. Assess the general foundation characteristics of the jetty and whether it may be founded on sand dune or other materials such as terrace deposits or bedrock.
5. Assess the continuity of the jetty and whether portions of the jetty not observable at the surface may be buried at depth.

2.6 SCOPE OF WORK

The following geophysical surveys were conducted in order to meet the objectives listed above:

1. A seismic refraction (SR) survey to meet objectives 1 and 2 listed above.
2. An electrical resistivity profiling (ERP) survey to meet objective number 3.
3. A ground penetrating radar (GPR) survey to meet objectives 4 and 5.

Our scope of work also consisted of analyzing and interpreting the SR, ER and GPR data and presenting our findings in a written report.

2.7 GLOSSARY OF GEOPHYSICAL TERMS

Compressional (P)Waves – elastic waves caused by the compression and dilation of earth materials; these are the fastest travelling seismic waves

Electrical resistivity – the resistance of a volume of earth material to the flow of electricity

Electrical Resistivity Survey – a technique for measuring electrical resistivity along a profile

Electrode – typically a 12" long stainless steel stake used to transmit electrical current

Geophone – a velocity sensitive electromagnetic device used to detect vibratory ground motion

Line – a traverse along which geophysical data are acquired; may consist of one or more spreads

Profile – a cross-section depicting variations in geophysical properties beneath a portion of a line

Seismic Refraction Survey – a geophysical method for measuring the velocity of P-waves

Seismic waves – elastic waves that propagate through the subsurface

Shot point – a point on a geophysical line at which seismic energy is transmitted into the ground

Spread – a collinear array of electrodes, geophones and/or shot points

2.8 STANDARD CARE

The scope of services for this project consisted of using geophysical techniques to delineate variations in the physical properties of the subsurface and buried objects. The accuracy of our findings is subject to specific site conditions and limitations inherent to the geophysical techniques we employed. We performed our services in a manner consistent with the level of skill ordinarily exercised by members of the profession currently employing similar methods. No warranty, with respect to the performance of services or products delivered under this agreement, expressed or implied, is made by NORCAL.

3.0 GEOPHYSICAL INVESTIGATION

3.1 SEISMIC REFRACTION (SR) SURVEY

The SR survey was conducted along a single transect (line) adjacent to the estuary on the east side of the barrier beach, as indicated by the solid red line labeled SR-1 shown on Plate 1. SR-1 extended from the vegetated sand dunes on the south, to the south edge of the outlet channel on the north, a distance of 411-m. A portion of SR-1, looking north, is shown in the photo at left. Descriptions of our SR data acquisition and analysis procedures are provided in Appendix A. The results of the SR survey are provided in Section 4.1.

3.2 ELECTRICAL RESISTIVITY (ER) SURVEY

The ER survey was conducted along a single transect (line), as indicated by the dashed blue line labeled ER-1 shown on Plate 1. The southern half of the line was coincident with SR-1. However, the northern half diverged from SR-1. This was done because of a change in access conditions when the ER survey was conducted. A portion of ER-1, looking south, is shown in the photo at left. Descriptions of our ER data acquisition and analysis procedures are provided in Appendix B. The results of the ER survey are described in Section 4.2.

3.3 GROUND PENETRATING RADAR (GPR) SURVEY

The GPR survey was conducted along nine transects (lines) oriented perpendicular to the jetty, as indicated by the alignment of jetty materials that are visible at the surface. The locations of the GPR lines, labeled GPR-1 through GPR-9, are shown on Plate 1. Most of the GPR lines were about 30-m long. The one exception is GPR-8 which was 37-m long. A portion of one of the GPR lines, looking west, is shown in the photo below. Descriptions of our GPR data acquisition and analysis procedures are provided in Appendix C. The results of the GPR survey are provided in Section 4.3.

4.0 GEOPHYSICAL RESULTS

4.1 SEISMIC REFRACTION SURVEY

The results of the seismic refraction survey are illustrated by the top diagram shown on Plate 2. This is a cross-section (profile) illustrating the variation in compressional (P) wave velocity versus depth and distance beneath the SR line (SR-1, Plate 1). The horizontal axis represents distance (Station) in meters along the line and the vertical axis represents elevation above mean sea level in meters. The P-wave velocities (Vp) are indicated by colored shading according to the scale shown beneath the profile. The colors indicate that Vp ranges from about 200 meters/sec (m/sec) near the surface to as high as 2,500 m/sec near the bottom of the profile. We have subdivided this velocity range into four seismic layers designated V1 through V4 according to increasing depth and velocity. For the sake of clarity we have shown only the velocity contours (solid dark lines) that define the tops and bottoms of these layers. The velocity range, thickness and assumed lithology of the four seismic layers are tabulated in Table 1, below.

Table 1: Seismic layer velocities, thicknesses and assumed lithology

SEISMIC LAYER	VELOCITY RANGE (M/SEC)	THICKNESS RANGE (M)	ASSUMED LITHOLOGY
V1	200 – 1,400	1.0 – 4.5	Dry to moist beach sand
V2	1,400 – 1,800	1.5 – 9.0	Saturated beach sand
V3	1,800 – 2,000	1.5 – 24+	Saturated river sediments and/or terrace deposits
V4	>2,000	undefined	Moderately weathered rock

V1 is the surface layer. Given that most of the layer occurs above sea level, we assume that the sands comprising the layer are unsaturated. Conversely, the V2 layer is at or below sea level. Therefore, we assume that the sands comprising that layer are saturated. Typically, we would expect unconsolidated sediments that are saturated to have velocities in excess of 1,460-m/sec (the velocity of water). However, V2 has velocities as low as 1,400-m/sec (probably in the

upper one meter or so of the layer). The most likely explanation for this is that gas released by organic materials has become entrained in the pores of the sands. These organic vapors tend to displace water thus reducing the seismic velocity of the material. The degree to which the seismic velocity is depressed is directly proportional to the concentration of the gas. This phenomenon is especially evident in a zone of low velocity that occurs at elevations of -6 to 0 m between Stations 15- and 120-m. We interpret this feature, labeled "A" and enclosed by a 1,400-m/sec contour as shown on Plate 2, as a buried channel. Given that the channel is situated within the saturated zone, we assume that its low velocities (460- to 1,400-m/sec) are also caused by the presence of organics. However, the very low bottom end of this velocity range suggests a relatively high concentration of entrained gas.

Seismic layer V3is situated below sea level and, therefore, is saturated. The fact that its velocity range is higher than V2 suggests that it probably consists of materials that are coarser and more consolidated that those of the overlying V2 layer. Consequently, we assume that V3 consists of river sediments and, possibly, terrace deposits. However, in areas where the bottom of V3 is deepest, it may also consist of deeply weathered rock. The V3 layer is very thin (1.5- to 6.0-m) southeast of Station 130-m where the underlying V4 layer is relatively shallow. However, northeast of this point the upper surface of V4 drops off and V3 is up to 24-m thick.

Given its depth and velocity range, we interpret the V4 layer as representing moderately weathered rock. The surface of V4 is only 6- to 14-m deep beneath the southeast 130-m of the profile. However, northwest of Station 130-m it drops off into two apparent buried channels where it reaches depths in excess of 30-m. One channel, labeled "B" on Plate 2, is about 120-m wide and up to 20-m deep. It extends from Stations 130- to about 250-m. The other channel, labeled "C", extends from about Station 275-m to an unknown distance beyond the northwest end of the profile. The bottom of the apparent channel drops below the depth range of the profile at about Station 345-m. Consequently, the northwest end of the channel is not defined.

4.2 ELECTRICAL RESISTIVITY SURVEY

The bottom illustration on Plate 3 is a cross-section (profile) illustrating the variation in electrical resistivity versus depth and distance beneath the ER line (ER-1, Plate 1). The horizontal axis represents distance (Station) in meters along the line and the vertical axis represents elevation above mean sea level in meters. The electrical resistivity values are indicated by colored shading according to the logarithmic scale shown beneath the profile. The measured electrical resistivities cover a very low, very narrow range that extends from 1.0 to 4.5 ohm-m. We attribute these very low resistivities to the proximity of the ER line to the ocean and the estuary, a very electrically conductive environment.

Most of the electrical resistivity (ER) profile shown on Plate 2 is dominated by electrical resistivity values in the lower half of the range, below 2.5 ohm-m (blue to green contours). The only place where higher values occur (yellow to red contours) is in the near surface which probably consists primarily of dry to moist sand. In spite of the low range of resistivity values, it appears that the ER profile does exhibit some interesting geologic features. Most notable is the interface between the dark blue contours (<2 ohm-m) and the underlying light blue to green contours (2.0 to 2.5 ohm-m). The configuration of this interface, which we've denoted by a dashed line, is similar to that of the 2,000-m/sec contour described in Section 4.1. Although the two are not exactly the same, they are close enough that we interpret them both as an approximate representation of the bedrock surface. That being the case, the material underlying the interface represents bedrock and the material overlying the interface represents unconsolidated materials consisting of sand, river sediments and possible terrace deposits. The fact that ER of these materials are so low suggests that the materials are saturated by relatively saline water. This implies that sea water intrusion is occurring within the zone defined by the light blue to dark blue contours. If this is the case, the highest flow rates probably coincide with the darkest blue shading.

The ER interface (dashed line) that we interpret as the top of weathered bedrock delineates two possible buried channels that are roughly similar to those defined by the 2,000-

m/sec velocity contour (Section 4.1). Therefore, we have also labeled these channels as B and C. The main discrepancies between the SR and ER versions of the channels are as follows:

1. ER channel B doesn't extend as far southeast as SR channel B.
2. Both of the ER channels appear to be deeper than their SR counterparts.

4.3 GROUND PENETRATING RADAR (GPR) SURVEY

The results of the GPR survey are illustrated by the GPR profiles shown on plates C-1 through C-9 in Appendix C, and by the GPR Survey Map shown on Plate 3. Each plate in Appendix C contains a processed GPR section obtained with the 270 MHz antenna (top) and the 120 MHz antenna (bottom). On each section, the vertical axis represents elevation and the horizontal axis represents Station (distance along the line) in meters. The elevations along each profile were derived from LIDAR data provided by the SCWA. The GPR depth of investigation is limited by the time windows we used with each antenna. This consisted of a 50 nanosecond (nS) window with the 270 MHz antenna, and a 100 nS window with the 120. Assuming a dielectric constant of 6, this equates to an apparent depth of 3-m for the 270 and 6-m for the 120. In both cases, the bottom of the time window is represented by the solid line that separates the multi-colored GPR record from the monotone peach color. Although we can see what we interpret to be real GPR reflections (as opposed to noise) to the bottom of the time window with both antennas, we do not see a reflecting horizon that we would interpret as evidence of the bottom of the jetty. The dashed lines plotted on each section represent the limits of anomalous reflection patterns that we interpret as evidence of the jetty. These dashed lines extend to the maximum depth at which we can define the jetty anomalies, even though we can see reflections at greater depths.

Plate 3 is an air photo of the survey area showing the locations, lengths and orientations of the GPR lines. The locations of the five monitoring wells installed by the SCWA are also shown for reference. Also shown are the locations and lateral extent of GPR anomalies shown on the plates in Appendix C. The dashed blue lines connecting the GPR anomalies represent the interpreted alignment of the jetty. This alignment is based not only on the GPR anomalies, but also on visual evidence of areas where portions of the jetty are visible at the surface, as seen on

the air photo. Note that we did not detect the jetty on lines GPR-4 and GPR-5. Also, on line GPR-7 the GPR anomalies are displaced about 7.5-m east of the interpreted jetty alignment. This could be by design or it could be the result of wave action during a strong storm.

On line GPR-1 the top of the jetty is visible as a smooth concrete surface that is only partially covered by sand. The reflection patterns associated with the jetty at that location, as shown on Plate C-1, are consistent with that symmetry. The only other location where the top of the GPR anomaly is essentially flat is beneath GPR-8. On all of the other profiles, the GPR anomalies have a rounded or irregular surface that is more consistent with the rip-rap that is exposed at the surface along most of the jetty alignment.

On most of the GPR profiles there is a high amplitude reflector (HAR) at a depth of about 1.5-m that generally parallels the ground surface. Given its elevation above sea level (about 4.5-m), we don't believe that this reflector represents the water table. Also, the geologic logs from the five aforementioned monitoring wells indicate only well sorted sands to depths of at least 12-m. Given these factors, we interpret the HAR as indicating an increase in moisture content and/or the degree to which the sands are compacted.

At the north end of the jetty (GPR-1), the GPR anomaly associated with the jetty extends above the HAR to the ground surface. This is as expected since the GPR line went directly over the exposed concrete surface of the structure. However, proceeding south, the tops of the GPR anomalies that we interpret as being associated with the jetty get deeper and deeper. At GPR-3, the top of the GPR anomaly is almost flush with the top of the HAR. Beneath lines GPR-4 and GPR-5 there are no jetty type GPR anomalies. Further south, beneath lines GPR-6 through GPR-9, the interpreted jetty anomalies are flush with the top of the HAR.

We did not observe any reflection patterns that we would associate with the bottom of the jetty. This is probably because the bottom of the structure lies beneath the depth of investigation of the GPR survey. Consequently, we cannot make any conclusions regarding the type of material that the jetty is founded in. Given the findings of the five monitoring wells (MW-1 through MW-5, Plate 3) the jetty is surrounded by beach sands, at least to the depth of investigation of the GPR survey.

VICINITY MAP

SCALE
0 20 40 80
(1 centimeter = 20 meters)

LEGEND

	SEISMIC REFRACTION LINE
	ELECTRICAL RESISTIVITY LINE
	GPR TRAVERSE
	MONITORING WELL

NORCAL

SITE LOCATION MAP
GEOPHYSICAL INVESTIGATION
GOAT ROCK STATE BEACH

LOCATION: JENNER, CALIFORNIA

CLIENT: SONOMA COUNTY WATER AGENCY

NORCAL GEOPHYSICAL CONSULTANTS INC.

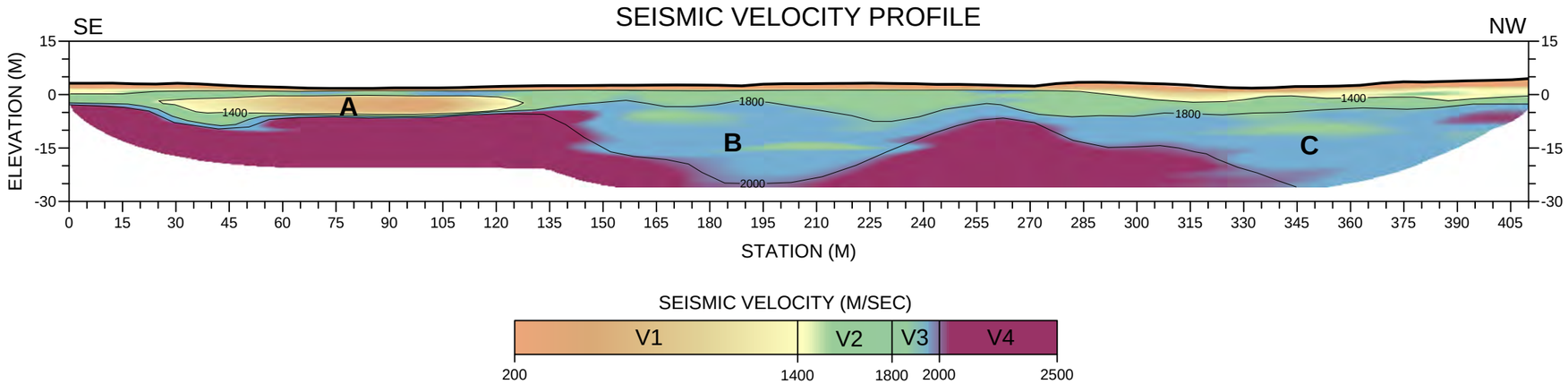
JOB #: 14-165.12

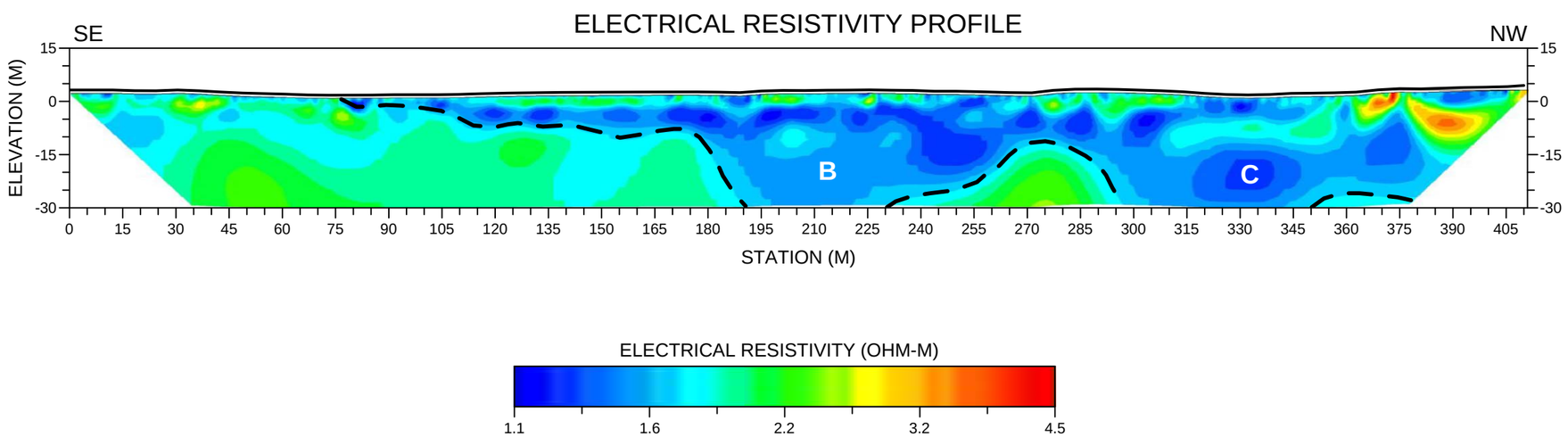
DATE: APR. 2015

DRAWN BY: G.RANDALL

APPROVED BY: WEB

PLATE 1





INTERPRETATION	
— — -	INTERPRETED BEDROCK SURFACE
A,B,C	BURIED RIVER CHANNEL

SCALE

0 15 30 45 60

(1 centimeter = 15 meters)

NORCAL

SEISMIC VELOCITY AND
ELECTRICAL RESISTIVITY PROFILES
GEOPHYSICAL INVESTIGATION
GOAT ROCK STATE BEACH

LOCATION: JENNER, CALIFORNIA

CLIENT: SONOMA COUNTY WATER AGENCY

JOB #: 14-165.12

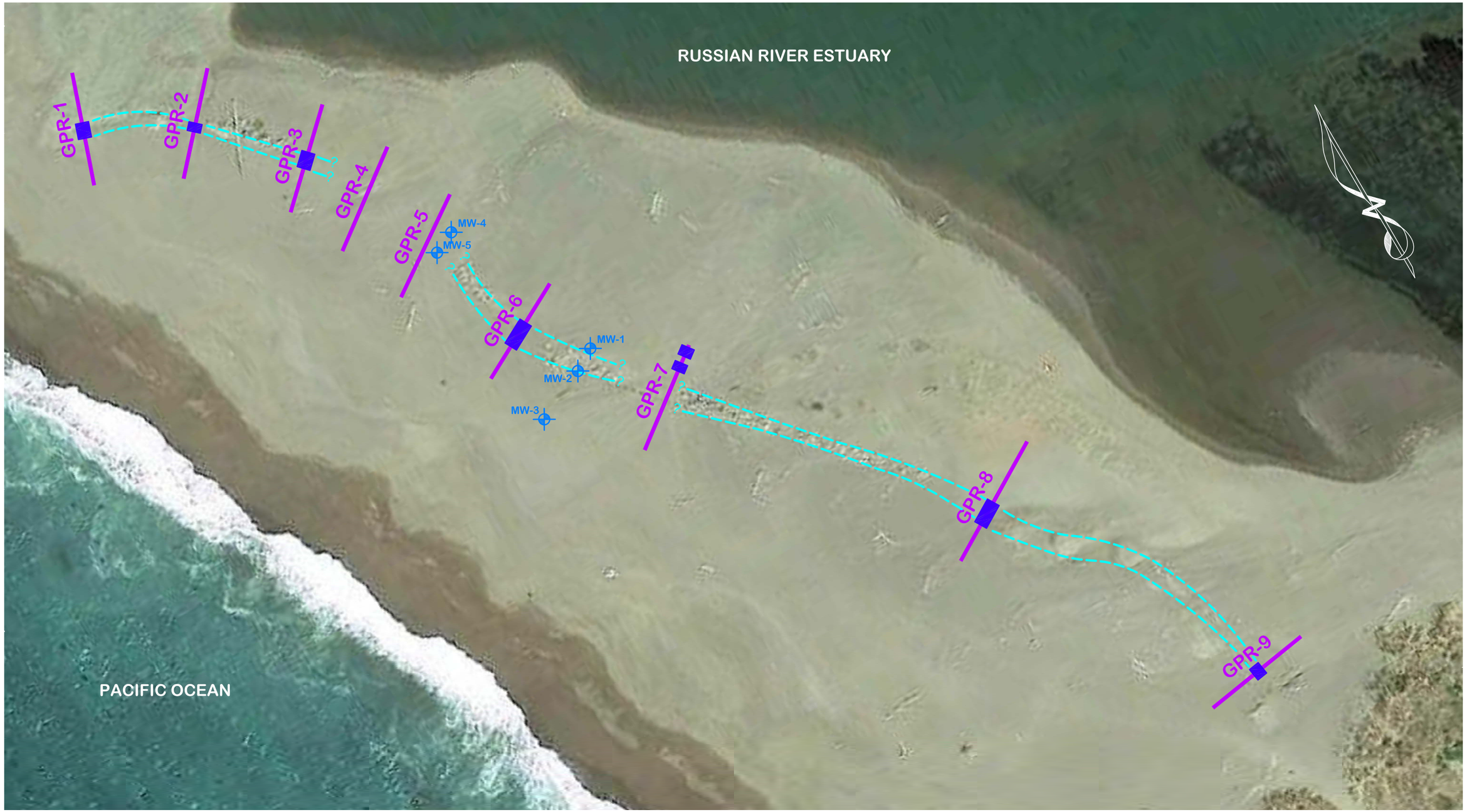
NORCAL GEOPHYSICAL CONSULTANTS INC.

DATE: APR. 2015

DRAWN BY: G.RANDALL

APPROVED BY: WEB

PLATE 2



LEGEND

	GPR TRAVERSE
	GPR ANOMALY
	INTERPRETED JETTY ALIGNMENT
	MONITORING WELL

SCALE

0 10 20 40

(1 centimeter = 10 meters)

NORCAL

GPR SURVEY MAP
GEOPHYSICAL INVESTIGATION
GOAT ROCK STATE BEACH

LOCATION: JENNER, CALIFORNIA

CLIENT: SONOMA COUNTY WATER AGENCY

NORCAL GEOPHYSICAL CONSULTANTS INC.

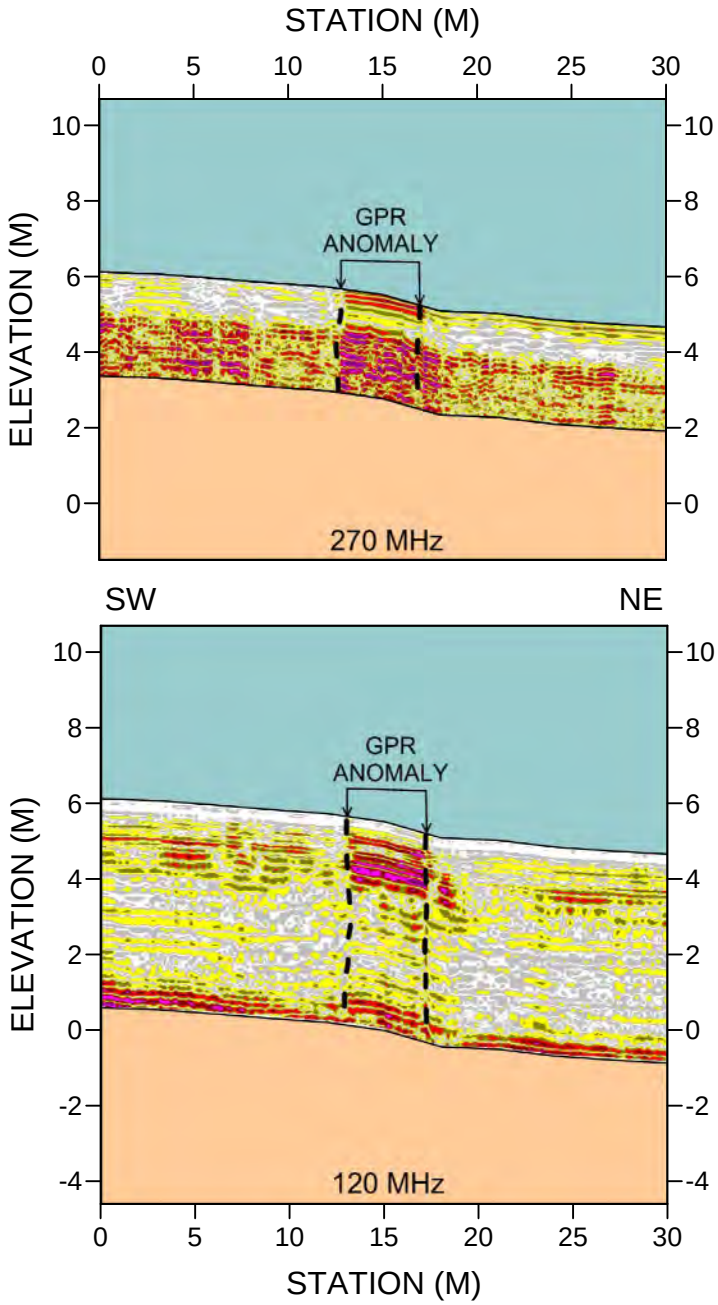
JOB #: 14-165.12

DATE: APR. 2015

DRAWN BY: G.RANDALL

APPROVED BY: WEB

PLATE 3



Interpreted Limits of GPR Anomaly

Note: Vertical Exaggeration = 2:1

NORCAL

GPR-1
GROUND PENETRATING RADAR SURVEY
GOAT ROCK STATE BEACH

LOCATION: JENNER, CALIFORNIA

CLIENT: SONOMA COUNTY WATER AGENCY

JOB #: 14-165.12

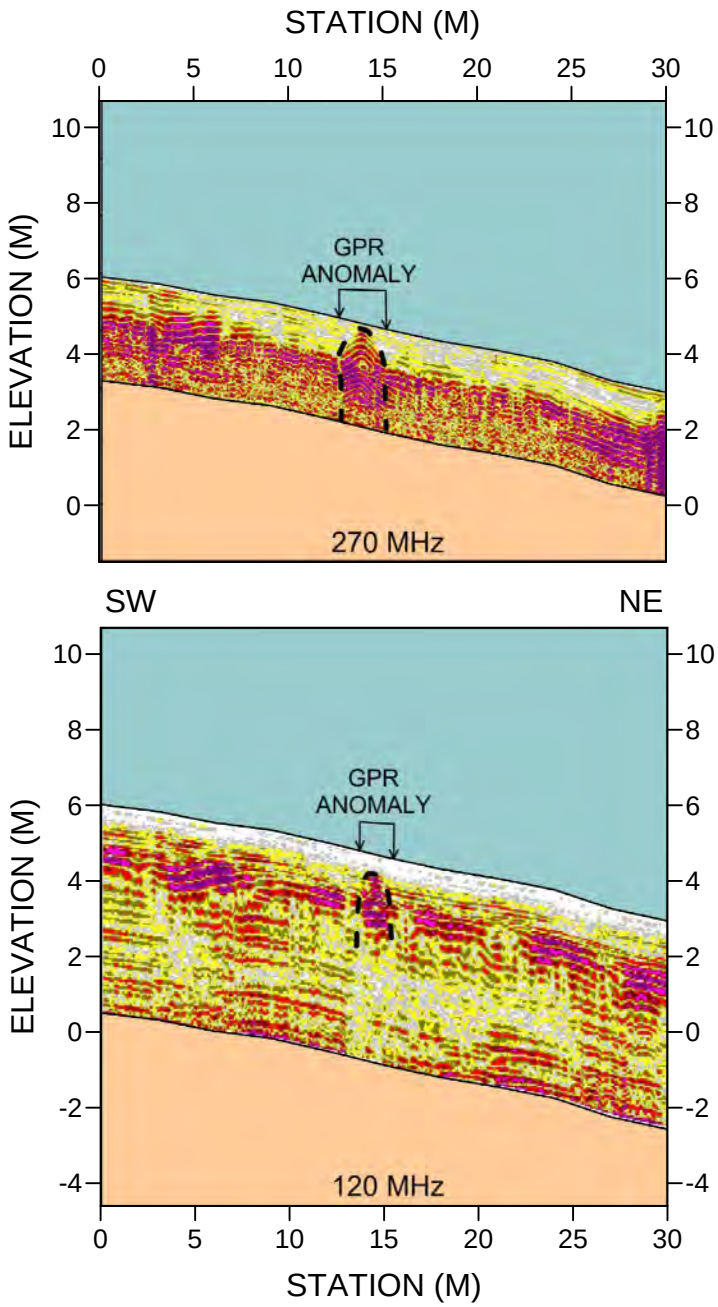
NORCAL GEOPHYSICAL CONSULTANTS INC.

DATE: APR. 2015

DRAWN BY: WEB

APPROVED BY: WEB

PLATE
C1



Interpreted Limits of GPR Anomaly

Note: Vertical Exaggeration = 2:1

NORCAL

GPR-2
GROUND PENETRATING RADAR SURVEY
GOAT ROCK STATE BEACH

LOCATION: JENNER, CALIFORNIA

CLIENT: SONOMA COUNTY WATER AGENCY

NORCAL GEOPHYSICAL CONSULTANTS INC.

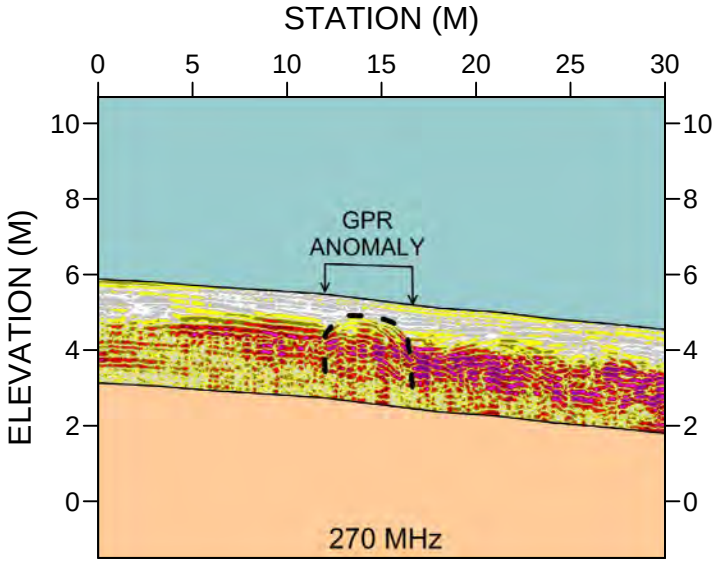
JOB #: 14-165.12

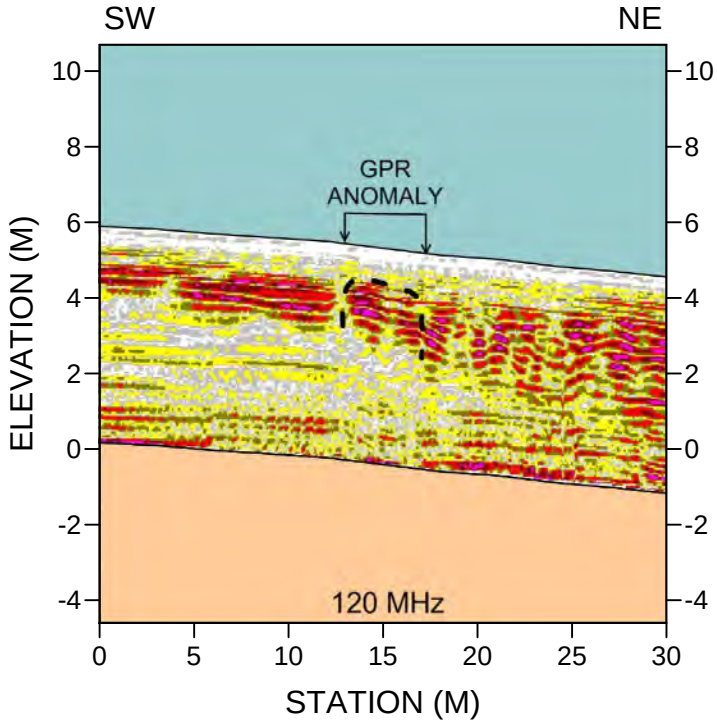
DATE: APR. 2015

DRAWN BY: WEB

APPROVED BY: WEB

PLATE
C2





Interpreted Limits of GPR Anomaly

Note: Vertical Exaggeration = 2:1

NORCAL

GPR-3
GROUND PENETRATING RADAR SURVEY
GOAT ROCK STATE BEACH

LOCATION: JENNER, CALIFORNIA

CLIENT: SONOMA COUNTY WATER AGENCY

JOB #: 14-165.12

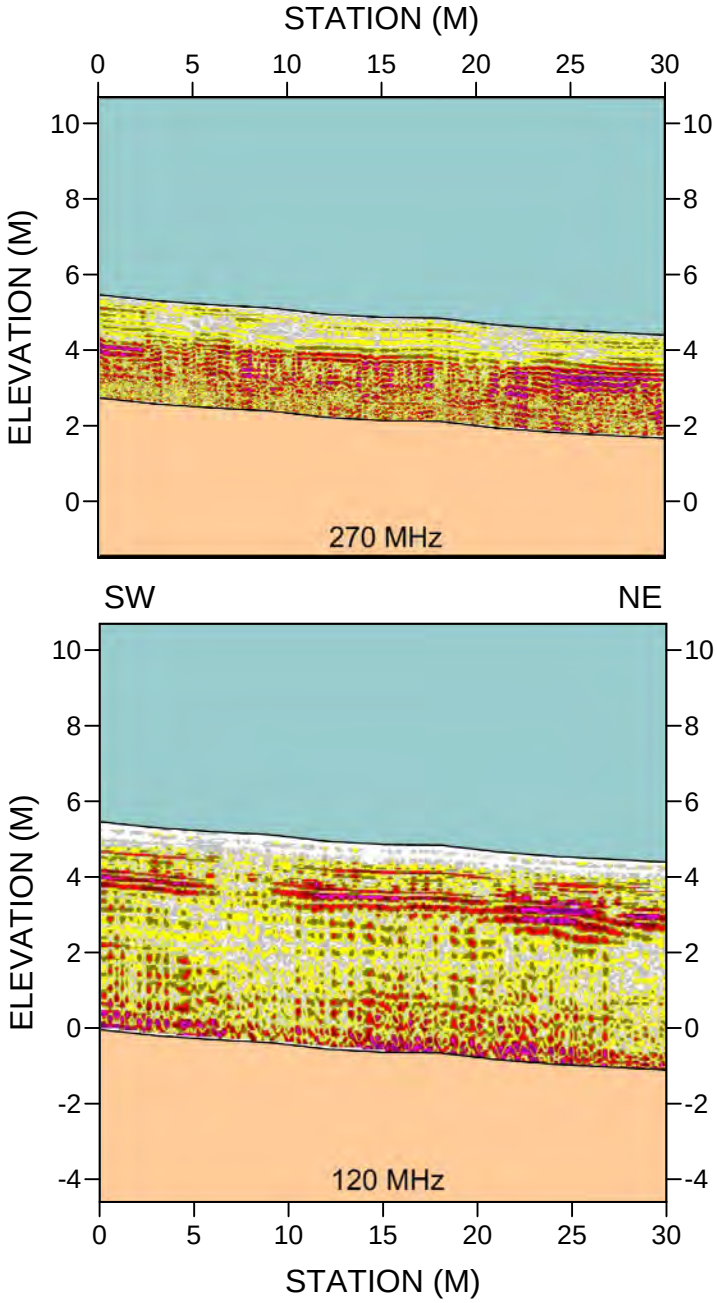
NORCAL GEOPHYSICAL CONSULTANTS INC.

DATE: APR. 2015

DRAWN BY: WEB

APPROVED BY: WEB

PLATE
C3



Interpreted Limits of GPR Anomaly

Note: Vertical Exaggeration = 2:1

NORCAL

GPR-4
GROUND PENETRATING RADAR SURVEY
GOAT ROCK STATE BEACH

LOCATION: JENNER, CALIFORNIA

CLIENT: SONOMA COUNTY WATER AGENCY

JOB #: 14-165.12

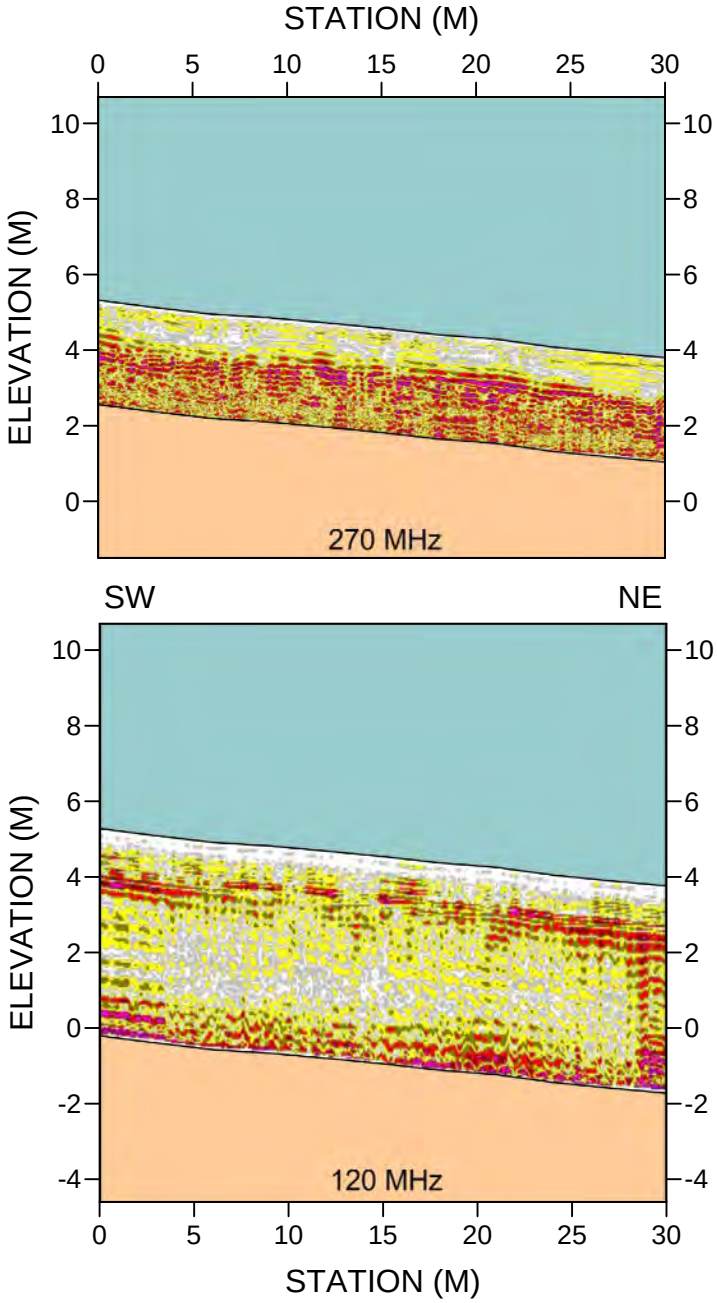
NORCAL GEOPHYSICAL CONSULTANTS INC.

DATE: APR. 2015

DRAWN BY: WEB

APPROVED BY: WEB

PLATE
C4



Interpreted Limits of GPR Anomaly

Note: Vertical Exaggeration = 2:1

NORCAL

GPR-5
GROUND PENETRATING RADAR SURVEY
GOAT ROCK STATE BEACH

LOCATION: JENNER, CALIFORNIA

CLIENT: SONOMA COUNTY WATER AGENCY

JOB #: 14-165.12

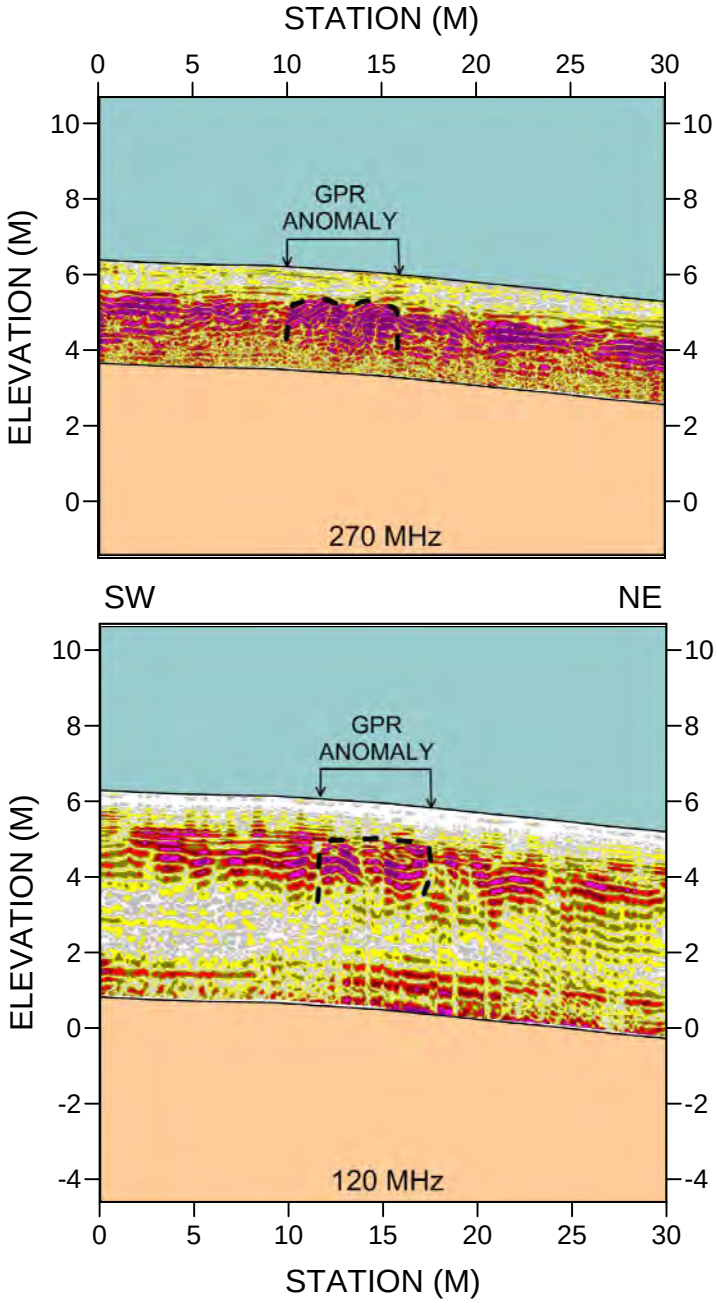
NORCAL GEOPHYSICAL CONSULTANTS INC.

DATE: APR. 2015

DRAWN BY: WEB

APPROVED BY: WEB

PLATE
C5



Interpreted Limits of GPR Anomaly

Note: Vertical Exaggeration = 2:1

NORCAL

GPR-6
GROUND PENETRATING RADAR SURVEY
GOAT ROCK STATE BEACH

LOCATION: JENNER, CALIFORNIA

CLIENT: SONOMA COUNTY WATER AGENCY

NORCAL GEOPHYSICAL CONSULTANTS INC.

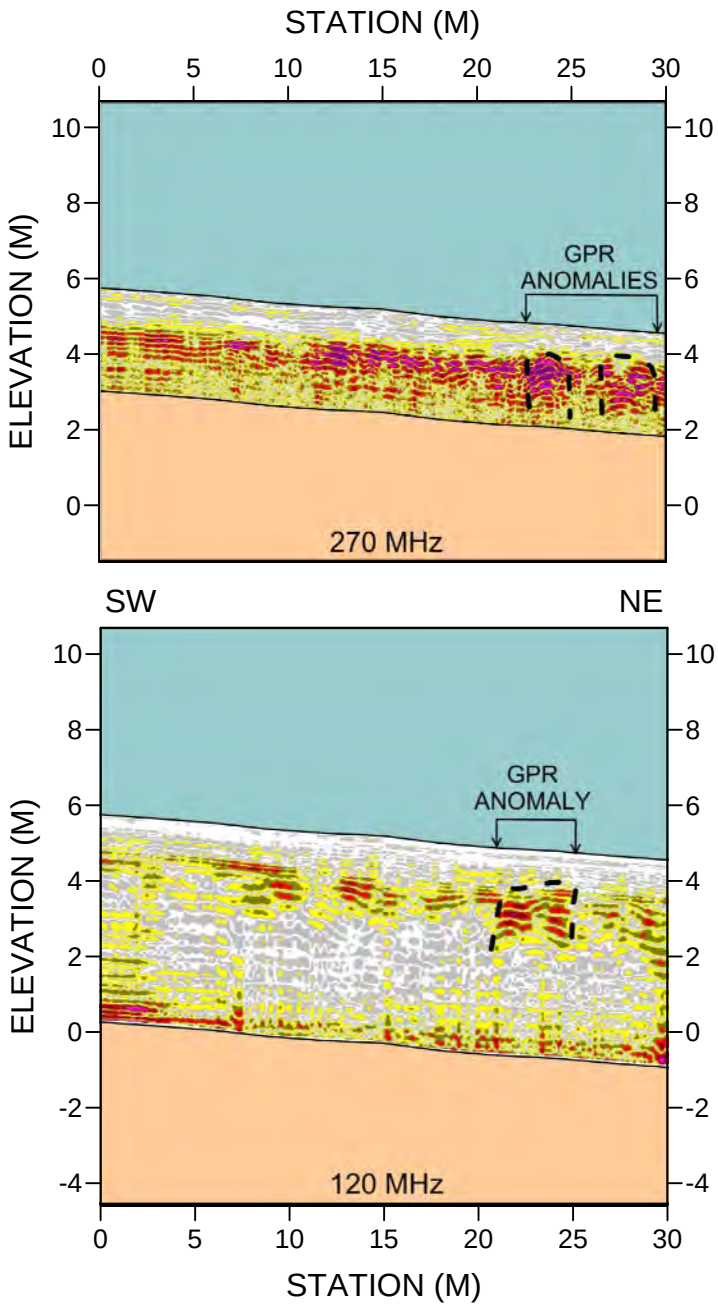
JOB #: 14-165.12

DATE: APR. 2015

DRAWN BY: WEB

APPROVED BY: WEB

PLATE
C6



Interpreted Limits of GPR Anomaly

Note: Vertical Exaggeration = 2:1

NORCAL

GPR-7
GROUND PENETRATING RADAR SURVEY
GOAT ROCK STATE BEACH

LOCATION: JENNER, CALIFORNIA

CLIENT: SONOMA COUNTY WATER AGENCY

JOB #: 14-165.12

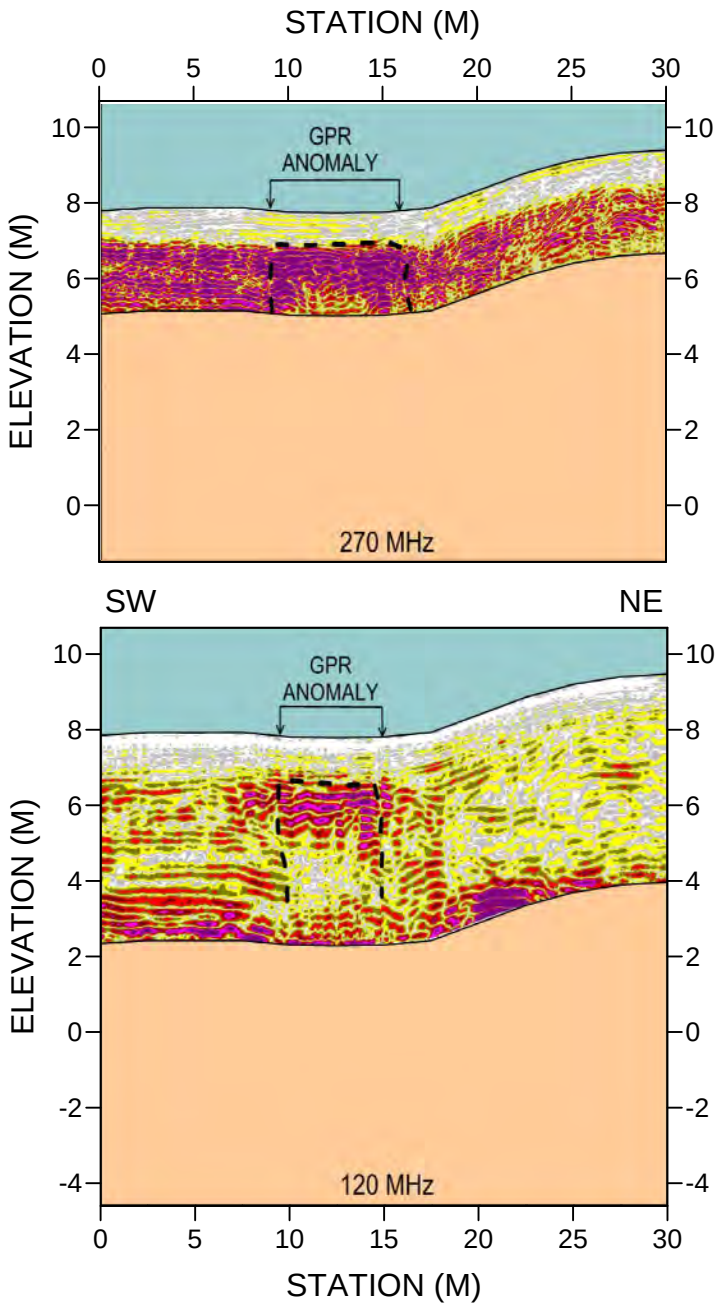
NORCAL GEOPHYSICAL CONSULTANTS INC.

DATE: APR. 2015

DRAWN BY: WEB

APPROVED BY: WEB

PLATE
C7



Interpreted Limits of GPR Anomaly

Note: Vertical Exaggeration = 2:1

NORCAL

GPR-8
GROUND PENETRATING RADAR SURVEY
GOAT ROCK STATE BEACH

LOCATION: JENNER, CALIFORNIA

CLIENT: SONOMA COUNTY WATER AGENCY

JOB #: 14-165.12

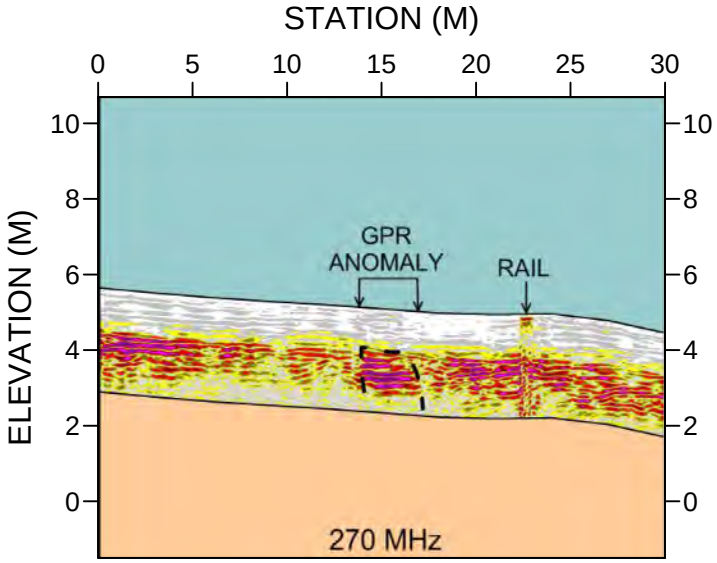
NORCAL GEOPHYSICAL CONSULTANTS INC.

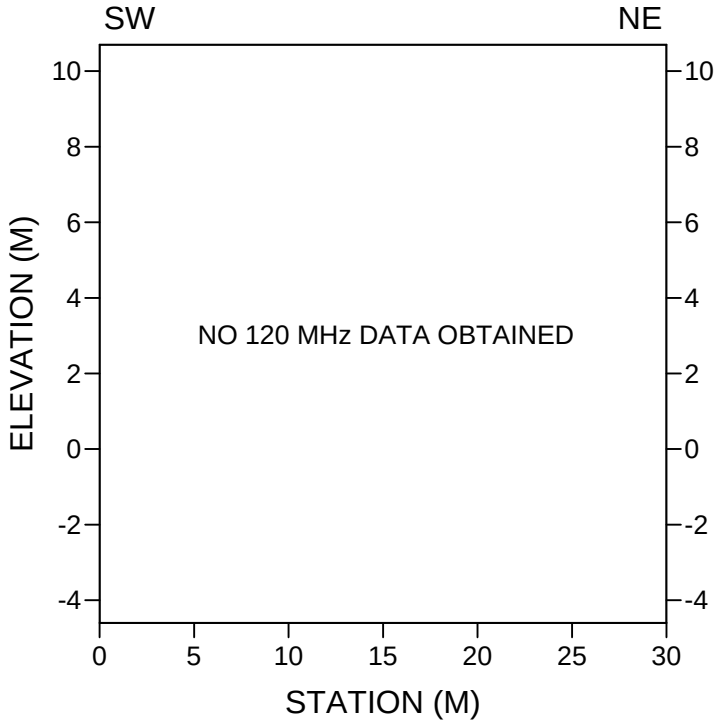
DATE: APR. 2015

DRAWN BY: WEB

APPROVED BY: WEB

PLATE
C8





Interpreted Limits of GPR Anomaly

Note: Vertical Exaggeration = 2:1

NORCAL

GPR-9
GROUND PENETRATING RADAR SURVEY
GOAT ROCK STATE BEACH

LOCATION: JENNER, CALIFORNIA

CLIENT: SONOMA COUNTY WATER AGENCY

JOB #: 14-165.12

NORCAL GEOPHYSICAL CONSULTANTS INC.

DATE: APR. 2015

DRAWN BY: WEB

APPROVED BY: WEB

PLATE
C9

Appendix A

SEISMIC REFRACTION (SR) SURVEY

Appendix A

SEISMIC REFRACTION (SR) SURVEY

1.0 METHODOLOGY

The seismic refraction method provides information regarding the seismic velocity structure of the subsurface. An impulsive (mechanical or explosive) source is used to produce compressional (P) wave seismic energy. The P-waves propagate into the earth and are refracted along interfaces caused by an increase in velocity. A portion of the P-wave energy is refracted back to the surface where it is detected by sensors (geophones) that are coupled to the ground surface in a collinear array (spread). The detected signals are recorded on a multi-channel seismograph and are analyzed to determine the shot point-to-geophone travel times. These data can be used along with the corresponding shot point-to-geophone distances to determine the depth, thickness, and velocity of subsurface seismic layers.

The seismic refraction technique is based on several assumptions. Paramount among these are that seismic velocity:

1) Increases with depth, and,
2) Is uniform within each layer over the length of the given spread.

In cases where these assumptions do not hold, the accuracy of the technique decreases. For example, if a low velocity layer occurs between two layers of higher velocity, the low velocity layer will not be detected and the depth to the underlying high velocity layer will be erroneously large. Also, if the velocity of a seismic layer varies laterally within a spread, those variations will be interpreted as fluctuations in the elevation of the underlying seismic layer.

2.0 DATA ACQUISITION/INSTRUMENTATION

We collected SR data along three overlapping seismic spreads. Each spread consisted of 24 geophones and 7 shot points distributed in a collinear array. The geophones were distributed at 6.1-m intervals. The shot points were distributed at 24.4- to 27.4-m intervals starting 6.1-m from the first (southeastern most) geophone and ending 6.1-m beyond the last (northwestern most) geophone. The seismic spreads overlapped each other by 1 to 6 geophones resulting in a total line length (from first to last shot point) of 411-m. The location of the seismic line, labeled SR-1, is shown on Plate 1.

The geophones in each spread were coupled to the ground surface by metal spikes affixed to the bottom of each geophone case. In areas where the surface sands were too loose to allow good coupling, we excavated the loose sand in order to reach firm, moist sand that would provide better coupling. A portion of the seismic line is shown in Figure 1, below.

Figure 1: Looking northwest along a portion of SR-1. Pin flags mark the geophone locations which, in most cases are buried at depths of 15- to 30-cm.

Seismic energy was produced at each shot point by multiple impacts with a Digipulse AWD-100 accelerated weight drop against an aluminum strike plate placed on the ground surface. The AWD-100 consists of a 1-m long cylindrical weight that is lifted to a height of about 36-cm by a hydraulic ram. The weight is lifted against the restraining force of a pre-tensioned industrial elastic band. When the weight reaches its maximum height it slips off the ram and accelerates downward until impacting the strike plate. The AWD-100 was mounted on the back of a Kawasaki Mule 4 w.d. all-terrain vehicle, as shown in Figure 2.

Figure 2: Digipulse AWD-100 mounted on Kawasaki Mule 4 w.d. ATV.

The seismic waveforms produced at each shot point were recorded using a Geometrics ***Geode*** 24-channel engineering distributed array seismograph and Oyo ***Geospace*** geophones with a natural frequency of 8-Hz. The seismic waveforms were digitized, processed and amplified by the Geode and transmitted via a ruggedized Ethernet cable to a field computer. There the data were archived for subsequent processing and displayed on the computers LCD screen in the form of seismograms. These were subsequently used to determine the time required for P-waves to travel from each shot point to each geophone in a given array (spread).

A Trimble Geo XH global positioning system (GPS) was used to determine the coordinates of the 21 shot points comprising SR-1. The resulting data were incorporated in the processing of the seismic refraction data.

3.0 DATA ANALYSIS

Preliminary seismic refraction models were computed using the software package ***SeisImager*** which was written by Oyo Corporation (Japan) and distributed by Geometrics Inc. The first stage of seismic processing included compilation and identification of first arriving P-wave energy at each geophone from each shot point. This process was conducted using ***Pickwin, Version 3.2.0.1*** (2004), which is part of the ***SeisImager*** package. A second interactive program ***Plotrefa, Version 2.8.0.1*** (2006) was used to assign surface elevations to each geophone and velocity layer assignments to travel time. We then used ***SeisImager's*** time-term routine to compute a 2D seismic velocity model based on these inputs. Actual examples from this survey are presented in the following figures.

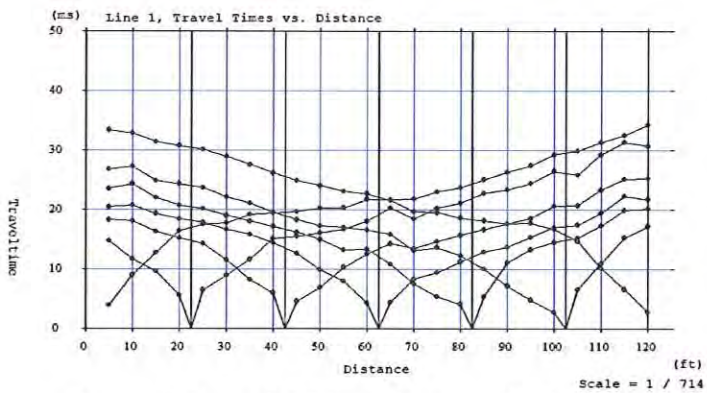


Figure 3: Example SR Time-Distance Graph

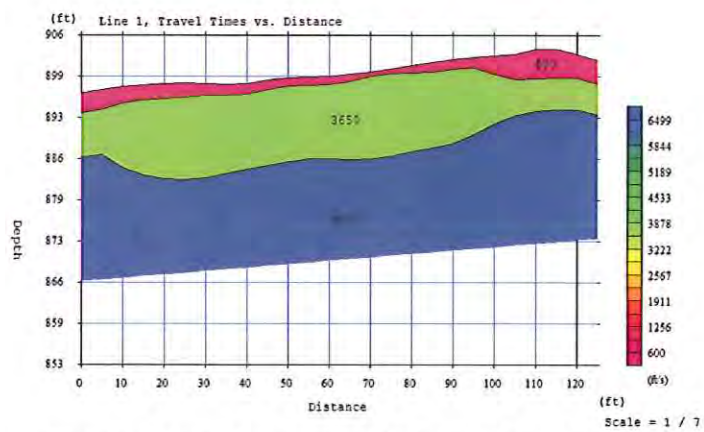


Figure 4: Time-Term Inverted Seismic Velocity Model

We used the resulting 2D seismic velocity model as input to the computer program ***Rayfract*** by Intelligent Resources, Lt. ***Rayfract*** uses wavepath eikonal travel time (WET) tomography to model multiple signal propagation paths contributing to one first break, based on the Fresnel volume approach. Conventional ray tracing tomography is limited to the modeling of just one ray per first break. ***Rayfract*** also uses an Eikonal solver (Lecomte, Gjoystdal et al. Geophysical Prospecting May 2000) for traveltime field computation to explicitly model diffraction besides refraction and transmission of acoustic waves. As a consequence the velocity anomaly imaging capability is enhanced with the WET tomographic inversion compared to conventional ray tomography. ***Rayfract*** also includes a smooth inversion tomographic method that is based on physically realistic modeling of first break propagation, for P-wave and S-wave surveys. It forward models refraction, transmission and diffraction (Lecomte, 2000) and back-projects traveltime residuals along wave paths, also known as Fresnel volumes (Watanabe, 1999) instead of conventional rays. This increases the numerical robustness of the inversion. A smooth minimum-structure and artifact-free 1D starting model is determined automatically, directly from the seismic traveltime data, by horizontally averaging DeltaV (Wiechert-Herglotz) method 1D velocity-depth profiles along the seismic line. The starting model is then refined with 2DWET inversion (Schuster, 1993). ***Rayfract*** uses an adapted SIRT algorithm for velocity update of grid cells, when back-projecting traveltime residuals along wave paths (Schuster, 1993) and (Watanabe, 1999). The result of these features is a seismic velocity model that is geologically reasonable even in cases involving rugged topography and strong lateral velocity variations.

4.0 LIMITATIONS

In general, there are limitations unique to the SR method. These limitations are primarily based on assumptions that are made by the data analysis routine. First, the data analysis routine assumes that the velocities along the length of each spread are uniform. If there are localized zones within each layer where the velocities are higher or lower than indicated, the analysis routine will interpret these zones as changes in the surface topography of the underlying layer. A zone of higher velocity material would be interpreted as a low in the surface of the underlying layer. Zones of lower velocity material would be interpreted as a high in the underlying layer.

Second, the data analysis routine assumes that the velocity of subsurface materials increase with depth. Therefore, if a layer exhibits velocities that are slower than those of the material above it, the slower layer will not be resolved. Also, a velocity layer may simply be too thin to be detected. Due to these and other limitations inherent to the SR method, the results of the SR survey should be considered only as approximations of the subsurface conditions. The actual conditions may vary locally. Other independent data (e.g., surface and borehole geology) should be integrated with SR data to enhance the subsurface interpretation.

Appendix B

ELECTRICAL RESISTIVITY (ER) SURVEY

Appendix B

ELECTRICAL RESISTIVITY (ER) SURVEY

1.0 METHODOLOGY

Electrical resistivity is the resistance of a volume of material to the flow of electrical current. The electrical resistivity method is often used to measure the variation in electrical resistivity with depth beneath a fixed point or along a line. This information can be valuable in determining stratographic and lithologic variations and other subsurface conditions that may relate to the depth and thickness of groundwater aquifers and permeable layers, the thickness of landfills, and in some cases, the depth to bedrock.

There are a number of electrode configurations (arrays) that can be used to measure electrical resistivity beneath a line. For this survey, we used one of the most commonly used configurations, the dipole-dipole array. In its simplest form, this consists of four electrodes distributed at even intervals in a collinear fashion. For each reading, electrical current (***I***) is input to the ground through one pair of electrodes (transmitter dipole) and the resulting potential drop (***V***) is measured across a second pair of electrodes (receiver dipole). The separation between electrodes (***a***) is the same for both dipoles and the separation between dipoles is always a multiple (***n***) of that distance. A simplified diagram of the dipole-dipole array is shown in Figure 1.

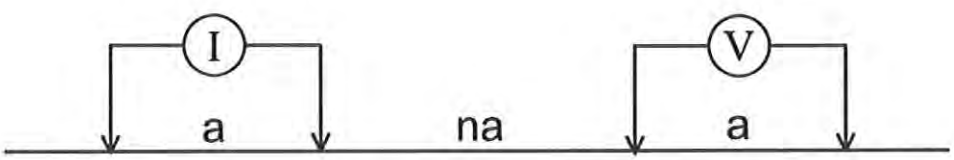


Figure 1: Dipole-Dipole Diagram

The values measured using this configuration are termed "apparent" resistivity (ρa) because they represent the resistance to the flow of electricity of a volume of material rather than a discrete layer. With the dipole-dipole array, apparent resistivity is calculated according to the following equation:

$$\rho a = \frac{V}{I}\pi a n(n+1)(n+2)$$

Moving the array along a line and measuring ρa at even intervals (multiples of ***a***) results in a data set that can be used to construct a cross-section (profile) depicting variations in electrical resistivity versus depth and distance. The depth of investigation of the profile can be increased by increasing both ***a*** and ***n*** by even multiples of ***a.***

Modern geophysical instrumentation makes it possible to collect dipole-dipole data using a large number of closely spaced electrodes and a wide range of ***a*** and ***n*** values. The resulting profiles have such high resolution that they closely correlate with subsurface geology and, therefore, are referred to as images. The process of obtaining those profiles is referred to as Electrical Resistivity Imaging (ERI). An example of how ERI data are obtained is illustrated by the three dipole-dipole diagrams shown in Figure 2. The rectangles at the top of each diagram represent electrode locations (42 in this case). The red and blue rectangles represent the transmitter dipole used for a particular measurement and the green rectangles represent the electrodes used as receiver dipoles for that measurement. The dark blue circles represent the data points obtained when the dipole length (***l***) is equal to the electrode separation (***a***), the red circles represent data points obtained when ***l*** is equal to 2***a*** and the light blue circles represent the data points obtained when ***l*** is equal to 3***a***. In each diagram, the farthest right (diagonal) column of circles represents data points obtained from the active (colored) dipoles shown at the top of the diagram. Note how the matrix of data points is being filled-in from left to right and top to bottom with each successive diagram. Note also that as the dipole length and spacing increases the depth of investigation also increases but with a corresponding decrease in resolution.

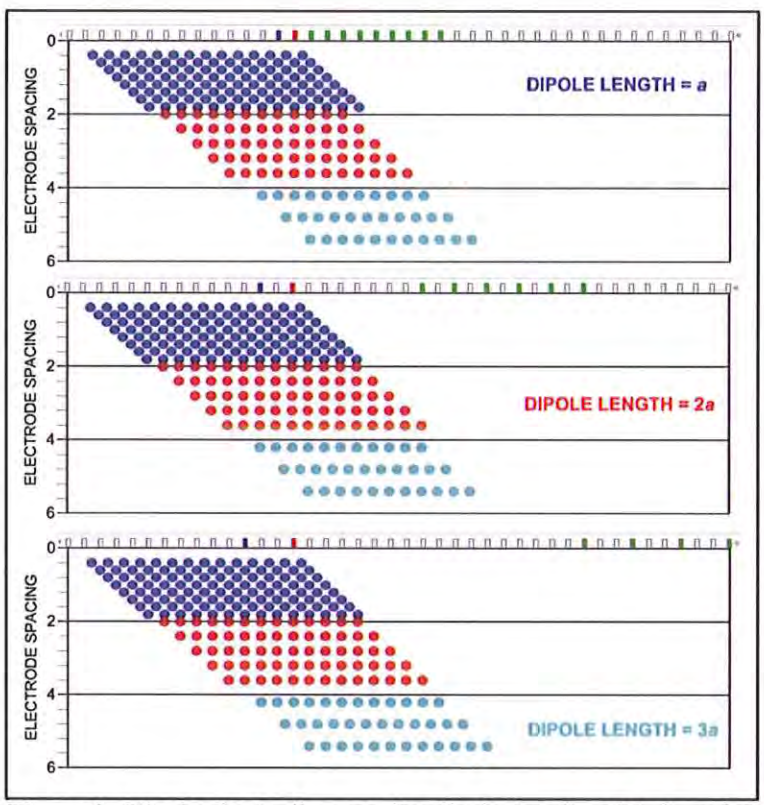


Figure 2: Dipole-dipole diagrams for dipole lengths of 1, 2, and 3 times the electrode separation (a).

2.0 DATA ACQUISITION

We collected ER data using the dipole-dipole array described above. A total of 140 stainless steel stakes (electrodes) were distributed along the resistivity traverse at 3.0-m intervals resulting in a total array length of 417-m. At each electrode station, we excavated a pit deep enough to reach firm, moist sand. This was done in order to improve the electrical contact between the electrode and the ground. The 25-cm long electrode was then driven into the ground and attached to the appropriate connector (take-out) on a multi-conductor cable that was, in turn, connected to the ER instrumentation (Section 3.0). Salt water was then added to the pit in order to further improve the contact resistance. In areas where the moist, firm sand was too deep to reach by digging, we replaced the stainless steel electrodes with 1-m lengths of rebar. The electrical resistivity line, labeled ER-1, is shown on Plate 1. A photo of the electrical resistivity array is shown in Figure 3.

Figure 3: A portion of the electrode array looking northwest. Note rebar in foreground.

Our procedure was to collect the ER data using only 56-electrodes at a time. Once the readings were completed we would advance (roll) the first 14-electrodes to the end of the array and then continue collecting data. This procedure was repeated until the last electrode was at Station 417-m. Given the instrumentation we were using (Section 3.0) we could have deployed as many as 112-electrodes at a time. However, since we were working on a public beach we wanted to minimize the length of the active array for safety reasons and to prevent tampering. Upon completion of the ER survey we determined the coordinates of strategic points along ER-1using a Trimble Geo XH global positioning system (GPS).

3.0 INSTRUMENTATION

We collected the ER data using a ***SuperSting R8*** resistivity meter. This system was configured with a 56-electrode switch box, four cables with 14 connectors (take-outs) per cable, and 56 stainless-steel electrodes. The meter had ***Wi-Fi*** capability and could be controlled remotely using a tablet computer. All of the instrumentation and cables comprising this system were manufactured by Advanced Geosciences Incorporated (AGI).

The ***SuperSting R8*** is a micro-processor controlled instrument that transmits current at outputs ranging from 1 to 2000 milliamps (mA). The instrument measures the potential drop (voltage) caused by the current influx and converts the data to values of apparent resistivity. A command file programmed into the instrument directs the switch box to activate certain electrodes for each reading according to electrode configuration and a number of other parameters that are selected by the operator. The resulting data are stored in the instrument internal memory and can be downloaded to a computer for subsequent processing and archival.

4.0 DATA PROCESSING

Once data acquisition was complete, the electrical resistivity data were downloaded from the ***SuperSting R8*** to a computer using the computer program ***AGISSADMIN*** by AGI. The data were archived and then inverted using the computer program ***EarthImager 2D,*** also by AGI. We used this program to produce a 2D cross-section (profile) depicting the variation in electrical resistivity versus elevation and distance beneath the line. The surface elevations were determined by correlating the GPS data we obtained along the line (Section 2.0) with LIDAR data provided by the SCWA. The resistivity data inversion proceeded as follows:

1) A starting resistivity model was constructed based on the apparent resistivity (ρa) distribution.

2) A virtual survey (forward modeling) was carried out for a predicted data set over the starting model and the initial root mean squared (RMS) error at the zero-th iteration was calculated.

3) A linearized inverse problem was solved based on the current model and data misfit for a model update ($\Delta \mathbf{m}$).

4) The resistivity model was updated using the formula: $\boldsymbol{m}_{i+1} = \mathbf{m}_i + \Delta\mathbf{m}$, where **m** consists of electrical conductivity of all model blocks in the finite element mesh and i is the iteration number.

5) A forward model (virtual survey) was run based on the updated model for an updated predicted data set.

6) A new RMS was calculated for the error between the predicted data and the measured data.

7) Steps (3) to (6) were repeated until the programmed stop criteria were satisfied, at which point the inversion was stopped.

Following each run of the inversion routine we reviewed the inversion diagrams produced by the software (not shown) to determine the degree of fit. If the RMS was considered to be too high, we would use routines included in ***EarthImager*** to filter the data by removing noisy data points. Typically, we removed no more than 10% at a time. We would then rerun the inversion process. This procedure was continued until an inverted model with acceptably low RMS was produced.

5.0 LIMITATIONS

A common feature of all electrical methods is that the models derived from the electric imaging are not unique. That is, depending on the subsurface geo-electric structure, there may be many models that will produce essentially the same apparent resistivities. This is known as the *principal of equivalence*. To overcome this limitation, computer software programs include routines for evaluating the equivalence of a given model relative to the observed resistivity values, resulting in a model that provides the closest fit to the observed data. Additionally, if the ground surface is too resistive, the system may have problems transmitting current into the subsurface (this situation can be remedied through the application of salt water at the base of each electrode). Conversely, if the ground surface is highly conductive, the potentials measured become negligible, resulting in a very low signal-to-noise ratio and therefore unreliable data.

Appendix C

GROUND PENETRATING RADAR (GPR) SURVEY

Appendix C

GROUND PENETRATING RADAR (GPR) SURVEY

1.0 METHODOLOGY

Ground penetrating radar is a method that provides a continuous, high resolution cross-section depicting variations in the electrical properties of the shallow subsurface. The method is particularly sensitive to variations in electrical conductivity and electrical permittivity (the ability of a material to hold a charge when an electrical field is applied).

GPR operates by repeatedly radiating an electromagnetic pulse into the subsurface from a transducer (antenna) as it is moved along a transect. Since non-metallic materials (e.g., wood, concrete, soil, and rock) are transparent to electromagnetic energy, the radar signal propagates into the subsurface. When the signal encounters a change in electrical permittivity, a portion of the energy is reflected back to the surface where it is detected by the transducer. The stronger the contrast in electrical permittivity, the more electromagnetic energy is reflected. When the signal encounters metal, all of the incident energy is reflected. The reflected signals are printed in cross-section form on a graphical recorder. The resulting records can provide information on the location, depth, and areal extent of buried objects or stratigraphic horizons.

The resolution power and depth of investigation for GPR depends, to a large extent, on the frequency of the transducer. Available frequencies range from as low as 25 megahertz (MHz) to as high as 1200 MHz. Generally, the higher the frequency, the better the resolution. The lower the frequency, the greater the depth of investigation. Electrically conductive materials, such as saturated clay, uncured or partially cured concrete or significant amounts of rebar can reduce the penetration capability and limit radar performance.

2.0 DATA ACQUISITION

We collected continuous GPR data along a total of nine traverses (lines) oriented perpendicular to the jetty which is visible at the surface along much of its alignment. The locations of these lines, labeled GPR-1 through GPR-9, are shown on the Site Location Map, Plate 1 and on the GPR Survey Map, Plate 3. All of these traverses were 30-m long with the exception of GPR-8 which was 36-m long. GPR-1 was centered over the northernmost exposure of the jetty. This consisted of an approximately 5-m wide, smooth concrete surface that was partially covered by sand. South of this location, the exposed portions of the jetty consisted of large boulders (rip-rap) which could not be traversed by the GPR equipment. Consequently, the remaining GPR lines (GPR-2 through GPR-9) were positioned in gaps through the rip-rap.

3.0 INSTRUMENTATION

We collected the GPR data using a Geophysical Survey Systems, Inc. (GSSI) subsurface interface radar (SIR-) 3000 configured with both a 120 MHz and a 270 MHz transducer. The SIR-3000 control console was attached to a harness that allowed it to be chest-mounted for portability and ease of operation. The console was connected to the radar transducer by a 2-m long umbilical cable. We collected continuous GPR data along each traverse using first, the 270 MHz transducer and then the 120 MHz transducer. The exception was line GPR-9 where only the 270 MHz transducer was used. We used the 120 MHz transducer for maximum depth of penetration and the 270 MHz transducer for optimum resolution. For the most part, we based most of our interpretation on the 270 MHz data because of its higher resolution. Both GPR transducers were hand-towed along each GPR traverse, as shown in Figure 1, below.

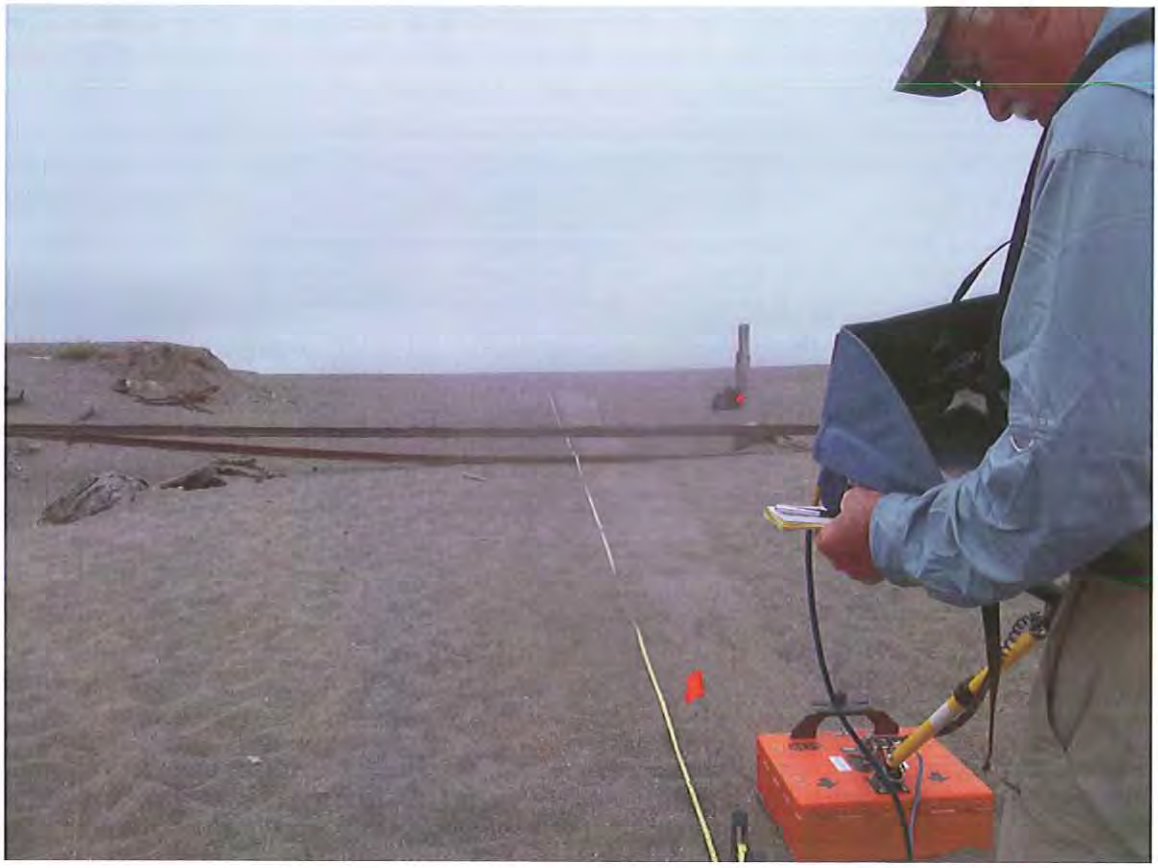


Figure 1: Deployment of 270 MHz transducer (orange box in foreground). Note path of transducer in background. Also shown is a portion of the abandoned railroad tracks that were used to transport quarry materials from Goat Rock during construction of the jetty.

We collected the GPR data on each line using the continuous (time relative) mode rather than the distance mode. This was done because a survey wheel (required for the distance mode) would not be accurate in the loose sand. Instead, we annotated each GPR record with fiducial marks at 10-ft intervals as determined with a fiberglass measuring tape. These marks would later serve as a basis for converting the GPR records continuous mode to distance mode (see Section 4.0).

Upon completion of the data acquisition a Trimble Geo XH global positioning system (GPS) was used to determine the coordinates of the start and end points of each GPR line.

4.0 DATA PROCESSING

We processed the GPR data using the computer program ***Radan 6.6*** by GSSI. For each GPR profile we used the following processing steps:

1) The first positive peak of the direct wave from the ground coupled transducer was positioned at the top edge of the video screen. This had the effect of positioning the ground surface at the top edge of the display window (Time Zero).

2) A background removal filter was used to remove flat-lying ringing system noise.

3) A triangular horizontal stacking (FIR) filter was used to remove high frequency noise.

4) A distance normalization procedure was used to establish a constant horizontal scale between fiducial marks. This "rubber-sheeting" technique adjusted the data so that there was an equal distance, or equal number of scans, between markers.

5) LIDAR data provided by the SCWA was used to assign surface elevations at 10-ft intervals along the GPR line (represented by the fiducial markers). Surface normalization was then used to adjust the GPR profile so that the vertical axis of the GPR section was a function of elevation rather than depth. This function was used because the surface along the GPR lines was not flat.

6) Color table and color transform parameters were selected to enhance subsurface features.

7) The Range gain and display gain parameters were adjusted to further enhance the resolution of the processed GPR section.

The processed 270 MHz (top) and 120 MHz (bottom) sections for lines GPR-1 through GPR-9 are shown on Plates C1 through C9.

5.0 LIMITATIONS

The detection of underground objects is dependent upon their size, depth, composition and construction. Objects buried below the GPR depth of investigation will not be directly detectable with GPR. However, their excavations may be discernible. Even objects within the GPR depth of investigation may not be detected if their signal is masked by overlying objects or reflecting horizons.

APPENDIX F. GEOPHYSICAL ASSESSMENT OF THE JENNER JETTY AND BEACH BERM SEEPAGE (LBNL, 2015)

Geophysical Assessment of the Jenner Jetty and Beach Berm Seepage

Photos courtesy of ESA PWA

Prepared for: Sonoma County Water Agency

Date: August 26, 2015

Prepared by: Lawrence Berkeley National Laboratory, Earth Science Division

Table of Contents

1.0 ABSTRACT

A geophysical study to assess groundwater flow between the Pacific Ocean and Russian River estuary was performed at the Goat Rock State Park, which is located at the mouth of the Russian River near Jenner, CA. The study focused on quantifying the influence of a man-made jetty on the functioning of a barrier beach, including seepage through the beach and implications for estuary fish habitat and flood control. Water level differences between the estuary and the ocean creates flow through the beach berm. When the estuary is closed or perched, outflow from the estuary predominantly occurs as seepage flow through the barrier beach. The location and design of the jetty could influence flow in the vicinity of the jetty, and possibly impede flow where the jetty and support structures (wood seawall and rock wall) are intact. The groin portion of the jetty consists of buried rock riprap with a concrete cap. The support structures consist of a wood seawall with a buried rock riprap supporting the wood seawall. These could influence connectivity between the ocean and the estuary, leading to atypical surface water elevations and possibly salinity imbalance. This aspect of the study focused on using geophysical data to characterize and monitor beach features, including location of buried jetty structures and flow characteristics, as part of the Sonoma County Water Agency's efforts to evaluate the feasibility of alternatives to the existing jetty to achieve target estuarine water surface elevations.

Surface and crosshole geophysical methods were used to delineate the jetty extent, and to monitor seepage through the jetty and through the beach berm. Geophysical campaigns were carried out on March 4th and September 30th to October 22nd. Multiple surface geophysical methods were deployed, including: electrical resistivity, seismic refraction, ground penetrating radar, and electromagnetic methods. In general, surface seismic data were used to characterize the geometry of deeper bedrock that may influence barrier beach processes, such as channeling of estuarine water beneath the barrier beach. Surface electrical and electromagnetic methods were used to characterize the beach sediment layers that could contribute to preferential flow paths during tide cycles, in addition to preferential flow paths created by the jetty structure. Time-lapse crosshole electrical and surface electromagnetic data were used to monitor moisture changes and mixing of saline and fresh water within the beach berm. Ground penetrating radar data were used to delineate the geometry of the (often buried) jetty and support structures. All data were integrated with topography, tidal and hydrological information, and electrical conductivity and temperature data from monitoring wells.

The geophysical data were useful for gaining an understanding of the role of the jetty and seawall on flow through the beach. Electromagnetic and electrical resistivity data suggested that portions of the wood seawall had deteriorated substantially, resulting in zones of fast seepage between the ocean and estuary during tidal cycles. Seismic results identified two bedrock low topography depressions that may influence the seepage through those two fast path zones. Seepage estimated from temperature and salinity sensors installed within the beach berm indicated seepage velocities of 3-4 m/hr where the wood seawall was intact, and 11-15 m/hr in the fast path zones where the seawall had deteriorated. The fast path zone seepage was 3.7 times faster than zones with intact seawall. These results suggest that the jetty structure and associated seawall currently influence flow through the barrier beach. The results from this study fulfilled a portion of the Sonoma County Water Agency's obligations under the 2008 Biological Opinion issued by the National Marine Fisheries Service.

2.0 INTRODUCTION

This document, developed by Lawrence Berkeley National Laboratory, Earth Science Division (LBNL ESD), at the request of Sonoma County Water Agency (Water Agency), describes the subsurface characterization of the beach, jetty, support structures, and subsurface water conveyance through the Goat Rock State Beach (GRSB) to the Russian River Estuary (Estuary). The results from this study fulfilled a portion of the Water Agency's obligations under the 2008 Biological Opinion (Biological Opinion) issued by the National Marine Fisheries Service (NMFS). The Biological Opinion directed the Water Agency to change its management of the Estuary's water surface elevations with the intent of improving juvenile salmonid habitat while minimizing flood risk.

A summary provided in a report by Environmental Science Associates (ESA), who performed the feasibility of alternatives study, stated:

"In the Russian River Biological Opinion (Biological Opinion), NMFS has concluded that historical artificial breaching activities in the spring and summer resulted in a loss of freshwater habitat in the Estuary and that a lack of freshwater estuarine rearing habitat limits recovery of salmonid populations, particularly steelhead (NFMS, 2008). The abundance and growth rates of juvenile steelhead have been positively correlated with the freshwater habitat found in lagoons. A lagoon is created by the barrier beach blocking ocean tides and salt water from entering the Estuary. NMFS determined that salmonid estuarine habitat may be improved by managing the Estuary as a perched, freshwater lagoon. Therefore, the Biological Opinion stipulates as a Reasonable and Prudent Alternative (RPA) to existing conditions that the Estuary be managed to achieve perched lagoon conditions between May 15th and October 15th. Under target conditions, the lagoon water surface elevations would be higher than ocean water surface elevations, ideally above 7 ft NGVD. To accomplish the target conditions, and to prevent lagoon water surface elevations from reaching flood stage (currently 9 ft NGVD), the Biological Opinion suggests that groundwater seepage through the barrier beach and an outlet channel incised in the beach balance riverine inflow.

Recognizing the complexity and uncertainty of managing conditions in the dynamic beach environment, the Biological Opinion stipulates that the estuarine water surface elevation RPA be managed adaptively… For the second phase, the Biological Opinion expands the project scope to consider alternatives to the jetty. The jetty, which is embedded in the barrier beach, may significantly affect some of the physical processes which determine lagoon water surface elevations… The Biological Opinion calls for a study analyzing the effects of the jetty on beach permeability, sand storage and transport, flood risk, and seasonal water surface elevations in the Estuary… Beach permeability, sand storage, and sand transport are physical processes which are affected by the jetty, and, in turn, affect the water surface elevations. Evaluating and quantifying these linkages will inform the development and evaluation of management alternatives for the jetty…the goal of the study proposed in this plan is to evaluate the feasibility of modifying the Goat Rock State Beach jetty to improve the likelihood of achieving the target lagoon water surface elevations. To accomplish this goal, the study objectives include:

- *Describe the extent and composition of the jetty*
- *Understand the jetty's effects on the physical processes which partially determine lagoon water surface elevations, including beach permeability, sand storage, and sand transport."*

2.1 GOAT ROCK STATE BEACH HISTORY

The GRSB is located on the northwest side of Sonoma County, California, near the town of Jenner, at the confluence of the Russian River and the Pacific Ocean. The GRSB Jetty was constructed during multiple construction phases that spanned almost 20 years from 1929 to 1948. The purpose of the jetty was to maintain a permanently open river passageway to transport upstream mined gravel to markets. In later years, fish migration through the mouth of the river became an additional driver for maintaining predictable river mouth openings. According to EWA PSA 2012 report, throughout the construction phases, multiple funding agencies contributed to the development of the jetty and access elements, including but not limited to: Russian River Improvement Company, Fish and Game Preservation Fund, State of California, CA division of Water Resources, Fish and Game Commission, Sonoma and Mendocino Counties, and private funding. The man-made structures that exist within the GRSB include the groin (that portion of the jetty constructed of rocks with a concrete cap), and the access elements (the roadway, wood seawall, railway, and rock fence, built to transport rock and equipment to the groin). Figure 1 shows the jetty and access elements, labeled with years installed, on an aerial image of the GRSB. The rock used in the construction of the groin and access elements was mined at what is now the Goat Rock parking lot in Figure 1. Approximately 90,000 tons of rock was quarried from Goat Rock for construction of the jetty. Of this, two thirds were used in the groin and one third to support the wood seawall installed along the access road across the beach. More information about the construction and historical use of the GRSB Jetty is in the ESA PWA "Feasibility of Alternatives to the Goat Rock State Beach Jetty for Managing Lagoon Water Surface Elevations: Existing Conditions Report."

2.2 SITE GEOLOGY

GRSB is a barrier beach that has been eroded and accreted by long-shore currents that transport sediments parallel to the California coast. The barrier beach runs generally in the north-south direction, with the Pacific Ocean to the west and the Russian River estuary and river to the east. At the confluence of the Russian River, the mouth of the river is not consistently open to the Pacific Ocean. River mouth closures generally occur between May and December during low river flow conditions. During times of low river flow, elevated waves and tides act as bulldozers, which push ocean-lying sand back up on the beach and close the mouth of the Russian River at the GRSB. The beach sand, according to EDS (2009), is comprised of mobile, well-sorted sand grains averaging 1 mm in diameter. The beach is in constant motion. During well installation at GRSB, a minor amount of silt was observed with the sand cuttings from drilling activities. The GRSB is underlain by Franciscan complex mélange that is Jurassic to Cretaceous in age. A mélange is characterized by a lack of internal continuity of contacts or strata by the inclusion of fragments and blocks of all sizes, both exotic and native, embedded in a fragmental matrix of finer-grained material (ESA PWA, 2012).

2.3 SCOPE OF WORK AND OBJECTIVES

The primary objectives of this investigation were to use geophysical methods to explore how the jetty impacts beach seepage relative to the natural beach berm, and to describe the extent and composition of the jetty. The objectives were met through geophysical campaigns and interpretations performed by NORCAL Geophysics, LLC (NORCAL) and Lawrence Berkeley National Laboratory (LBNL).

To delineate the jetty and characterize its influence on flow, NORCAL deployed Ground Penetrating Radar (GPR) methods to characterize the depth and lateral extent of the jetty and access structures and seismic refraction with electrical resistivity tomography (ERT) to assess the beach vertical and lateral lithological variations. Lawrence Berkeley National Laboratory (LBNL) collected time-lapse electromagnetic and electrical resistivity tomography datasets along the sides of the jetty to identify locations of faster seepage associated with tidal changes. Because flux through the jetty is likely controlled by such fast paths, identification of their locations, followed by quantification of their hydraulic properties, is important for parameterizing a numerical hydrological model. Since an increase in both moisture and salinity increases electrical conductivity, analysis of time-lapse data permits the 'removal' of the impact of the jetty on the geophysical signature, and highlights those regions of the jetty that have relatively higher seepage zones.

The same electrical methods described above were also used by LBNL to assess seepage mechanisms through the beach (south side of groin), through the ephemeral berm (north side of jetty), and across the buried access structures. Interpretation of the time-lapse electrical conductivity and crosshole resistivity data, along with downhole temperature and salinity data, were used to: (a) quantify the spatial distribution of the seepage through the beach, and (b) assess the role of the access elements on that seepage. Datasets were collected in rapid succession as the head difference (from the lagoon to the ocean) varied due to changing tides. The time-lapse electrical data provided information useful for determining the timing and spatial distribution of seepage.

3.0 GEOPHYSICS BACKGROUND

This section provides a brief description of the different methods used to characterize the GRSB subsurface structure and permeability.

3.1 ELECTROMAGNETICS

Controlled source inductive electromagnetic (EM) methods consist of injecting a time- or frequency-varying current in a transmitter coil to create a primary EM field that travels to a receiver coil via paths above and below surface. Governed by Maxwell's equations, the created EM field induces eddy currents in any conductors, which creates a secondary magnetic field. Attributes of this secondary magnetic field, such as amplitude, orientation, and/or phase shift, can be measured by a receiver coil. By isolating these attributes from those of the primary field signal, information about the subsurface electrical conductivity distribution can be inferred (e.g., McNeill 1990; Telford et al. 1990). A review of EM methods for shallow subsurface investigations is given by Everett and Meju (2005). Frequency domain EM tools used in this study for shallow subsurface investigations are EM31 and EM38 (e.g., McNeill 1980; Geonics 2009), which are ground conductivity meters that operate at a frequency of 14,500Hz using transmitter receiver coils oriented vertically or horizontally, and with an offset distance of 3 m (EM31) and 1 and 0.5 m (EM38). Because this method does not require contact with the ground, it allows data to be acquired very quickly. EM31/38 data are often displayed as maps of apparent electrical conductivity, which highlight lateral variation over large areas at an averaged depth interval. For this study, the main influences on electrical conductivity were expected to be moisture content, pore fluid salinity, and lithology, where higher moisture content, salinity and

clay content lead to higher electrical conductivity. These tools report an average conductivity value sensed over a depth range from 0 to 4 m below the ground surface.

3.2 ELECTRICAL RESISTIVITY TOMOGRAPHY

Electrical resistivity tomography (ERT) methods are probably more frequently used for shallow subsurface studies than any other geophysical method. Resistivity, indicating its ability to resist electrical current flow, is an intrinsic property of a material: it is the inverse of electrical conductivity. When measuring at low frequencies, energy loss via ionic and electronic conduction dominates. Ionic conduction results from the electrolyte filling the interconnected pore space (Archie 1942), as well as from surface conduction via the formation of an electrical double layer at the grain-fluid interface (e.g., Revil and Glover 1998). Most resistivity surveys use a four-electrode measurement approach, where current is injected between two electrodes, and electrical potential difference is measured between two others, while varying the location of electrodes along the profile and the distance between them (e.g., Binley and Kemna 2005). Modern multi-channel, geoelectrical equipment decrease acquisition time by injecting current through two electrodes and measuring the potential difference (voltage) signal between several pairs of electrodes and using electrodes alternatively as both current and potential electrodes, a method now referred to as electrical resistivity tomography (ERT). A review of this method is provided by Binley and Kemna (2005). Data quality is initially assessed through creating an apparent resistivity (pseudo-section) section, which is developed following Ohm's Law with information about the injected current, the measured potential difference and the geometric factor (which is a function of the electrode configuration), and through assuming uniform subsurface conditions. Further processing involves estimating the spatial distribution of resistivity that reproduces the measured data within a given range of uncertainty.

Inversion of ERT data typically involves iterative minimization of the misfit between measured and calculated data by optimizing two- or three-dimensional electrical resistivity models (e.g., Kemna 2000; Ramirez et al. 2005; Guenther et al. 2006). For the inversion of the ERT data, the discretization includes 0.05 m thick cells for the shallowest 0–0.2 m depth, 0.1 m thick cells for 0.2–0.8 m depth, .25 m thick cells for 0.8–5 m depth, and 0.5 m thick cells for 5 m or deeper. The modeling grid is defined to be much larger than the region of interest to ensure reliable inversions. Minimal smoothing is applied. The highest source of error is associated with the imaging of deepest structures. No corrections for temperature dependency were made in this study, although it is recognized that correcting resistivity to a reference temperature of 20°C leads to lower resistivity values than the values considered here (e.g., Hayley et al. 2007). In contrast to the EM approach, which provides a depth averaged value of electrical conductivity, inverted ERT data provides 2D information about electrical conductivity distribution.

3.3 SEISMIC REFRACTION

Seismic refraction survey was used to investigate the stratigraphy changes, depth to bedrock, and the general structure of GRSB. In a uniform isotropic earth, the shock wave from a hammer blow or explosion at the surface travels outward and downward in a hemispherical wave front, like a three-dimensional ripple from a pebble in a still pond. At any point on the wave front, a straight line from the shock source to the wave front depicts the path of the seismic wave, and is called a ray path. For the purposes of most seismic refraction surveys, only the fastest moving wave front—the P wave—is considered. To perform a refraction survey, a linear array of

ground motion sensors or geophones is spaced out from the seismic source, or shot point, forming a geophone spread. Each geophone is connected to a separate channel in a seismograph. The seismograph records a wiggle trace representing the ground motion resulting from the passage of the various seismic rays. Analysis of the seismographic wiggle traces can determine layered earth structures. At distances close to the seismic source, the first wiggle or ground motion (the first arrival after the hammer strike) is due to passage of the direct wave travelling at the velocity of the upper layer. Reflected waves arrive later, since they have, by definition, traveled a greater distance at the same velocity. For a full description of seismic wave propagation see Redpath (1973), Reynolds (2011) and the NORCAL Geophysics report in Appendix A.

3.4 GROUND PENETRATING RADAR

GPR methods use electromagnetic energy at frequencies of ~10MHz to 1GHz to probe the subsurface. At these frequencies, the separation (polarization) of opposite electric charges within a material that has been subjected to an external electric field dominates the electrical response. GPR systems consist of an impulse generator which repeatedly sends a particular voltage and frequency source to a transmitting antenna. The most common ground surface GPR acquisition mode is surface common-offset reflection, in which one (stacked) trace is collected from a transmitter-receiver antenna pair that is pulled along the ground surface. When the electromagnetic waves in the ground encounter a contrast in relative dielectric permittivity (also known as dielectric constant), part of the energy is reflected, and part is transmitted deeper into the ground. The reflected energy is displayed as 2D profiles that indicate the travel time and amplitude of the reflected arrivals; such profiles can be displayed in real time during data collection, and can be stored digitally for subsequent data processing. The velocity of the GPR signal can be obtained by measuring the travel time of the signal for various known separation distances between the transmitter and the receiver, or by hyperbola matching. Hyperbola matching was used because the GPR antennae had a fixed separation. In the GPR tomogram, a hyperbolic reflector can be fitted with a curve assuming data was collected at a constant speed to estimate the radar propagation velocity. This velocity can be used to convert the GPR profiles, which are recorded as distance versus travel time, into distance versus depth sections. A review of GPR methods used in hydrogeological applications is given by Annan (2005), and further explanation of the GPR type is provided in Appendix A.

3.5 MONITORING WELLS

Eight monitoring wells (MW) were installed at the GRSB: five MWs drilled 12 m deep, straddling the seawall (Figure 2, red dots), and three shallow MWs (~2 m; Figure 2, yellow dots) hand driven just inside the estuary downslope from the other five MWs. The MWs (5.08 cm diameter) were instrumented with temperature/salinity/pressure sensors and a set of removable ERT cables to collect cross-well ERT data. The temperature, salinity and water level sensors were stand-alone sensors: (1) Hobo Pendant (temperature only), and (2) Schlumberger Diver CTD (water level/salinity/temperature combo). The Pendant sensor specifications state the measurement range of these particular data sensors as -20 to 70°C with an accuracy of +/- 0.53°C and a resolution of +/- 0.25°C. The Diver sensor specifications state the temperature measurement range is 0 to 50°C with an accuracy of +/- 0.18°C and a resolution of +/- 0.018°C. The Diver salinity (conductivity) measurement specs state a range of 10 μS/cm with an accuracy

and resolution of +/- 1.0% of reading and +/- 0.1% of reading, respectively. Vertical temperature profiles were installed at each of the five measurement stations to investigate the change in riverbed seepage from May to November 2014. Eighteen stand-alone sensors (three Divers [salinity/temp./water-depth] and fifteen Pendant [temperature]) were installed inside each of the five monitoring wells (MWs 1-5). Seven sensors were installed in each of the shallow monitoring wells (MWs 6-8).

Salinity and temperature data were used to empirically estimate the vertical and horizontal seepage through the GRSB. Salinity data were used to determine the salinity gradients between MWs in the beach and how that changed with tidal cycles. A salinity front was expected as freshwater and saline water push back and forth through the beach during tidal changes. Measuring peak changes in salinity was performed to aid in the estimation of beach sand permeability. It was assumed that the difference in temperature between the ocean water and estuary water could be used as a tracer during tide cycles. Heat as a tracer (temperature flux) method is commonly used to monitor seepage from vertical movement of heat via daily solar radiation, while using salinity peaks has been rarely used. In this study's case we used the horizontal movement of heat during tidal exchange deep below the surface of the beach. Historically, studies using heat as a tracer investigate the near-surface heat exchange from solar radiation along a single vertical string of sensors instead of between wells horizontally. In riverine systems, surface water is heated by daily cyclical radiant heating and cooling (Blasch et al. 2007). The propagation of temperature from river water into the subsurface is assumed to be governed by conduction, advection, and/or dispersion (Battin and Sengschmitt 1999; Constantz 2008). As the oscillating temperature signals propagate, the temperature signal is attenuated from interaction with sediments and, where present, upwelling and/or downwelling water (Battin and Sengschmitt 1999). The degree to which thermal gradients propagate in the subsurface also depends on the thermal properties of water and sediments (Hatch et al. 2006). Highly sampled logs of daily temperature variations at different riverbed depths are used to measure temporal changes. Shifts in the temperature amplitude and peak-lag (phase) between vertical temperature sensors have been used to estimate hydraulic properties (Constantz et al. 2004; Cox et al. 2007; Gordon et al. 2012; Hatch 2007; Hatch et al. 2006; Lautz 2012). Time series distributions of the thermal records are used to estimate seepage (Hatch 2007; Hatch et al. 2006; Shanafield et al. 2011).

4.0 RESULTS

The following describes the results from geophysical surveys conducted on March 4th, 2014, from September 30th to October 2nd, 2014, and October 15th, 16th, 20th, 21st, and 22nd, 2014. The geophysical survey in March 2014 was a preliminary survey to rough characterize the GRSB for the placement of five monitoring wells. The survey in late 2014, during a river closure, was the first monitoring survey to characterize beach seepage. At the end of 2014, the Water Agency was informed that the approved end-date of access to the beach for geophysical study was set as March 15th, 2015, and no further geophysical campaigns were permitted past that date. This only allowed a few months for a river mouth closure to occur to allow further study beach permeability. Unfortunately, no river closures of substantial length occurred between November 2014 and March 2015. All of the geophysical data collection locations are shown in Figure 2.

March 2014 Survey Results

On March 4th, 2014, the LBNL geophysics team performed a reconnaissance survey of the GRSB. This survey was performed to guide effective placement of monitoring wells that would later be used after the seal pupping season for beach permeability monitoring. During the geophysical survey, the river mouth was open to the ocean, and estuary water levels mirrored the ocean tide elevation. The EM method was used to investigate the potential and expected subsurface groundwater flow paths that convey groundwater between the ocean and the estuary.

Figure 3 shows the extent of data coverage (just north of the groin; the river mouth was open to the ocean) and the variation in electrical conductivity. The electrical conductivity variations are caused by changes in moisture content. Dryer/low salinity sediments are shown as red to yellow in the figure, and wetter/higher salinity sediments are shown as green to purple. A dominant preferential flow path is interpreted to extend perpendicular to the wood seawall (beach long axis). This is denoted by the blue-purple zones in Figure 3, and the locations of MW 4 and MW 5 (no exposed wood seawall). North of the groin, a zone of high conductivity was observed. This signature is interpreted to be due to ocean water being pushed through the mouth of the river directly adjacent (dashed zone, Figure 3). Overall, the electrical conductivity data show spatial variations in moisture, salinity and fast groundwater flow paths. The locations of the monitoring wells are shown as red dots (Figure 3).

Following monitoring well installation in March 2014, the Water Agency instrumented all five monitoring wells and all three piezometers with temperature and salinity sensors, and set the sensors to record data every 15 minutes.

October 2014 Survey Results

On September 30th, 2014, the LBNL geophysics team returned to GRSB to investigate the jetty structure, and monitor beach permeability during a river-mouth closure event. At the beginning of the survey, the estuary stage was at 1.82 m (6 ft) above mean sea level (msl). Because of the flow gradient created by the difference in ocean and estuary elevations, the subsurface groundwater fast paths show more pronounced variations in electrical conductivity (Figure 4). This result indicates that the two sections of missing seawall (absence of black dotted line in Figure 4) are potential fast paths during oscillating tide events. What appears to be a third fast path through the groin, is instead the result of a large wave that over-crested the beach and left salt water near the surface at the time of the EM survey. This event also left salt water at the north end of the survey area above the end of the groin.

Between September 30th and October 16th to 22nd, multiple surface and cross-well geophysical surveys were completed. These included ERT, seismic, and GPR. Surface ERT was collected along a transect paralleling the wood seawall on the estuary side (Figure 2, magenta line ‘ERT Profile’). The ERT profile was collected from southeast to northwest towards the groin. A full ERT data set was collected every 2-3 hours from high tide to low tide. The expectation was infiltration of freshwater from the estuary through the beach towards the ocean at low tide.

Two time snapshots were selected for comparison in Figure 5: (1) ERT response at high tide, and (2) ERT response at low tide. The 2D cross-sections are shown in log10 resistivity (Ohm-m), so a value of 1 = 100 Ohm-m. Hot colors (red to yellow) indicate more freshwater, and cool colors (light blue to blue) indicate more saline water. The results in Figure 5 show clear evidence of freshwater perched on top of the saline water to a rough depth of -3 to -4 m below grade surface

(yellow-green transition). From 18 to 58 m along the ERT profile, the freshwater extends deeper where the wood seawall is intact. The black line at -4 m represents the estimated bottom of the seawall. The dashed portion indicates where the seawall is not exposed at the surface, and, conversely, the solid line indicates the seawall is exposed at the surface. The locations of the monitoring wells are also indicated in Figure 5 as black vertical lines. Where the seawall is exposed, freshwater extends deeper. Where the seawall is not exposed at the surface, saline water appears shallower. The change between high and low tide in Figure 5 is not readily apparent.

To further investigate the permeability of the beach in this zone, the two ERT data sets from high and low tides were simply differenced from each other after inversion (Figure 6). In Figure 6, topography was applied. The y-axis represents elevation in NGVD29 m. Red to yellow colors indicate an increase in fresher water (increased resistivity), and blue represents an increase in saline water (decreased resistivity). Between 58 m and the end of the profile up to 25% change in resistivity occurs, caused by the influx of freshwater in the zone of MWs 4 and 5, where the seawall is missing. Little to no change in resistivity was observed where the seawall was intact at MWs 1 and 2, indicating less hydraulic connectivity in this zone.

Cross-well ERT results from September 30th indicate no fresh water moving through the beach when the estuary elevation is 1.82 m above msl as shown in Figure 7A and 7B on 9/30/14 by no vertical and horizontal change in resistivity (all blue). In contrast, once estuary elevations peaked around 2.4 m above msl (Figure 7, 10/15/14), an increase in resistivity was observed between the wells, indicating an influx of fresher water. To further investigate the permeability of the beach, two ERT data sets, collected from low to high tide for these four monitoring wells, were differenced (like the surface ERT data in Figure 6) to capture any changes in salinity that may not be readily apparent. Figure 8, A and B, show the differenced data sets between MWs 1 and 2 and MWs 4 and 5. Again the wood seawall is exposed between MWs 1 and 2, and not between MWs 4 and 5. In Figure 8A, little to no change in resistivity occurred to a depth of about -6 m between MWs 1 and 2. Compare to Figure 8B, where a decrease in resistivity (influx of saline water) is observed by a roughly 15% change between MWs 4 and 5.

During the same time period, NORCAL performed both a seismic refraction and a GPR survey (Figure 2, red and blue lines). Figure 9A shows the seismic velocity tomogram along the roughly 400 m profile. Unconsolidated, weathered low velocity material (<2000 m/s) was observed overlying bedrock (>2000 m/s). At 195 m and 350 m along the seismic profile, two bedrock elevation lows were observed (B and C, Figure 9A). A low velocity zone (A, Figure 9A) is associated with a former, manmade pile of quarried stone suspected to be deposited during jetty construction. For reference, these locations were projected onto the plan view site map, Figure 9B. The two bedrock lows (B and C) appear in line with the zones of missing wood seawall and high water flux observed in the EM and ERT data.

Multiple GPR transects were collected at the same time as the seismic refraction survey. Only four transects are shown in Figure 10. These investigated the structure of the groin in two locations (GPR 1 and 2), and the wood seawall, both where it was not exposed (GPR 5), and was exposed (GPR 6). These locations are shown in the plan view map in Figure 10. GPR 1 (Figure 10) shows a shallow, high-amplitude reflector caused by the concrete cap over the groin, at depth (~5 m) the groundwater table is designated by the yellow-pink boundary. The GPR 2 transect at the end of the groin adjacent to the concrete capped section (i.e. no concrete cap), a parabolic anomaly appears just below the ground surface indicating the concrete cap is degraded

but still somewhat viable. The GPR 5 transect is located in the area of the unexposed wood seawall adjacent to MWs 4 and 5, no subsurface structure is apparent, indicating the seawall has been degraded or is non-existent. On the other hand, where the GPR 6 transect crosses a section of exposed seawall, a wide anomaly ~1 m below the surface is apparent at the groundwater table. The anomaly is roughly 5 m wide and appears to be the result of the seawall and support rock. Deeper information from the GPR survey is limited due to the saline nature of the groundwater and its attenuation of the GPR signal.

From March to November 2014, temperature and salinity sensors collected data at 15-minute intervals, 24 hours a day, in all monitoring wells. A snapshot of the data was used to evaluate the subsurface hydraulic connectivity between the ocean and estuary in conjunction with the geophysical data. Three depths of sensors (-1.0, -4.2, and -6.5 m NGVD29) were evaluated from salinity sensors in MWs 1 and 2 (Figure 11).

In Figure 11, the red dots on the ERT image show the depth of the sensors between the two wells. Prior to September 30th, the river mouth is open, and salinity along the length of the well ranges between 38 and 43 mS/cm (ocean water is about 50 mS/cm). After September 30th (river mouth closure), freshwater begins to infiltrate the beach, and stratification of fresh and saline water begins. At -1.0 m, salinity decreases to 10 mS/cm; at -6.5 m, little change (~40 mS/cm) occurs. At the intermediate depth (-4.2 m), the groundwater appears brackish (~31 mS/cm). Sinusoidal variations are evident at all three depth intervals following daily tidal fluctuations. At the intermediate level, the sinusoid amplitudes from sensor MW-1O are considerably muted compared to sensor MW-2P at the same depth interval (Figure 11).

By comparison, both of these are considerably muted compared to the intermediate (-4.8 m) salinity sensors in MWs 4 and 5 shown in Figure 12. During tidal changes, the intermediate level flips back-and-forth between 10 and 38 mS/cm as the tide rises and falls. This indicates a much faster conveyance of water at the same depth interval compared to MWs 1 and 2. On October 12th (see Figure 12) a wave apparently over-washed the beach, and saline water infiltrated vertically, causing the stratified fresh-saline water to collapse. All sensors increase briefly to higher salinity, then the fresh-saline water promptly re-stratifies. To look at the response time between tidal changes and salinity sensors, the four intermediate levels are plotted along with daily tidal changes. At MWs 4 and 5, the tide influences the well data very quickly, with sensors in both wells (gold and green lines, Figure 13) peaking with peak tide (saline water intrusion) and bottoming at low tide (fresh water intrusion), and then repeating (i.e., minimal lag time between sensor peaks). Comparatively, the responses in MWs 1 and 2 are extremely muted with almost no infiltration of fresh water, even though they are at the same depths at MWs 4 and 5. The salinity trends, along with temperature trends (not shown), are used to estimate horizontal and vertical seepage.

The estimated seepage results are shown in Table 1, and characterize horizontal seepage through the beach at sections where the seawall was exposed and where it is not. Seepage in the beach was estimated using the temperature and salinity curves from the Schlumberger Divers at -4.2 (MWs 1 and 2) and -4.8 (MWs 4 and 5) that were shown in Figures 10 and 11. This seepage estimate is computed simply by taking the salinity and temperature peaks on October 10th around 1 pm, then dividing the distance (7 m) by that time in hours. The same was done for vertical flow in MWs 6 and 8, with a sensor pair spacing of 0.46 m and 0.61 m, respectively. Horizontal seepage estimated from temperature and salinity ranges from 3-4 m/hr between MWs 1 and 2,

and 11-15 m/hr between MWs 4 and 5. Seepage was 3.7 times faster in the zone where the seawall is missing. Vertical seepage from the estuary into the backside of the beach ranged from 0.2 to 0.5 m/hr when estuary levels were approximately 2 m NGVD29.

5.0 DISCUSSION

Surface and cross-well geophysics all independently identified subsurface groundwater preferential flow paths between the estuary and ocean during tidal cycles (EM blue zones, Figure 4). Zones where the wood seawall is missing at the surface coincide with the preferential flow paths, as confirmed by subsurface ERT changes between elevations 4 to -5 m NGVD29. Results from the seismic survey indicate these preferential flow paths (EM blue zone, Figure 4) are likely controlled by the two bedrock lows (B and C, Figure 9) that possibly cut through the beach. Further, in-well temperature and salinity sensors captured the daily tidal cycles and permitted quantified seepage. Seepage estimated from the preferential flow paths was 3.7 times faster than outside these zones. Overall, these results indicate that portions of the wood seawall are degraded and influence natural daily flow between the ocean and estuary.

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Table 1. Horizontal and vertical seepage at GRSB obtained from temperature and salinity curves

Well(s)	Temp. Seepage Est. m/hr	Salinity Seepage Est. m/hr	Flow Direction & Where
MWs 1 & 2	4.0	3.0	Horizontal (btn wells)
MWs 4 & 5	15.0	11.0	Horizontal (btn wells)
MW 6	0.2	NA	Vertical (estuary)
MW8	0.5	NA	Vertical (estuary)

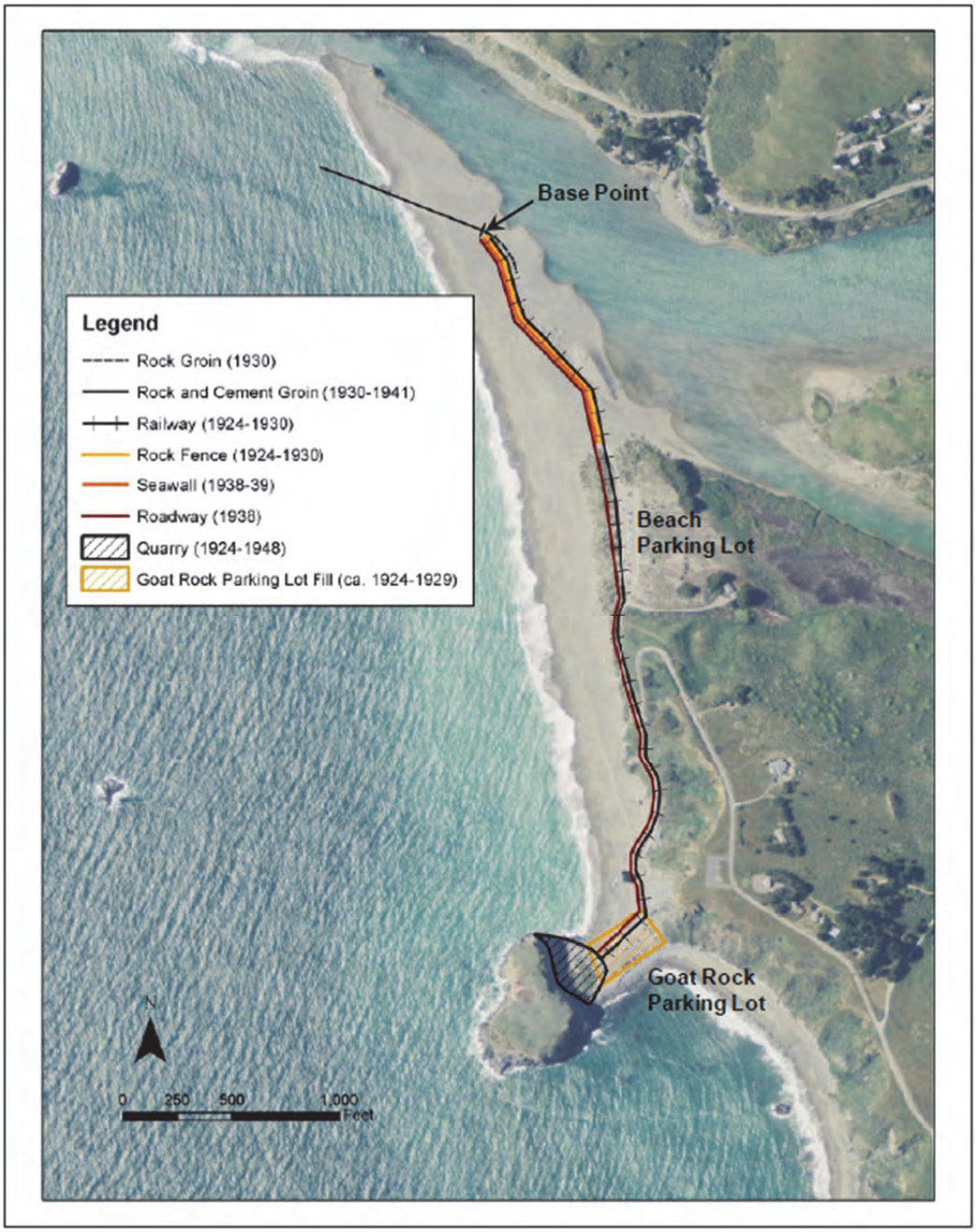

Figure 1. Plan view of jetty complex elements constructed between 1930 and 1941, courtesy of ESA PWA.

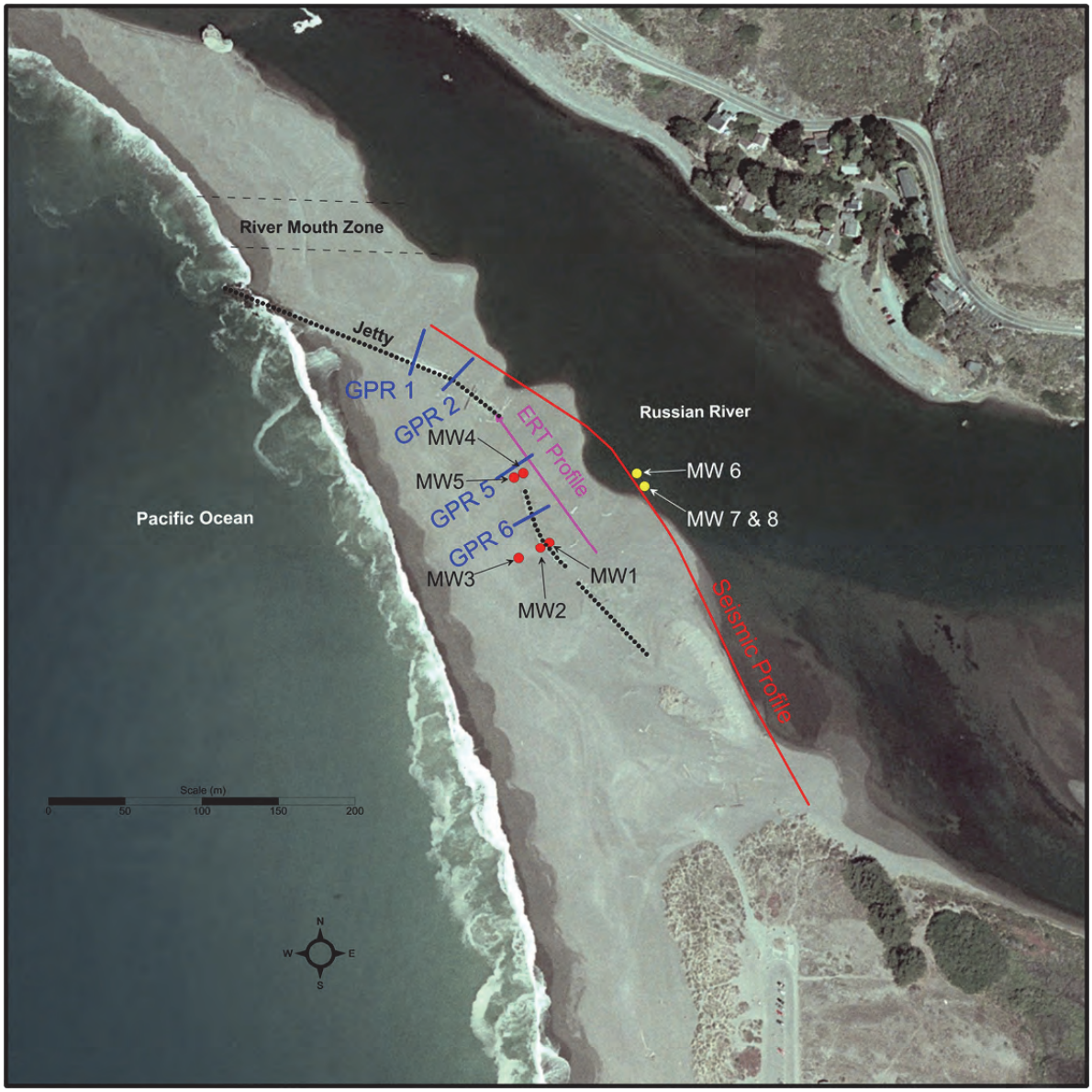

Figure 2. Plan view map of GRSB showing the locations of the Jetty components, ERT profile (magenta), seismic profile (red), and GPR profiles (blue). Deep (12 m) monitoring wells shown as red dots and shallow (2 m) shown as yellow dots.

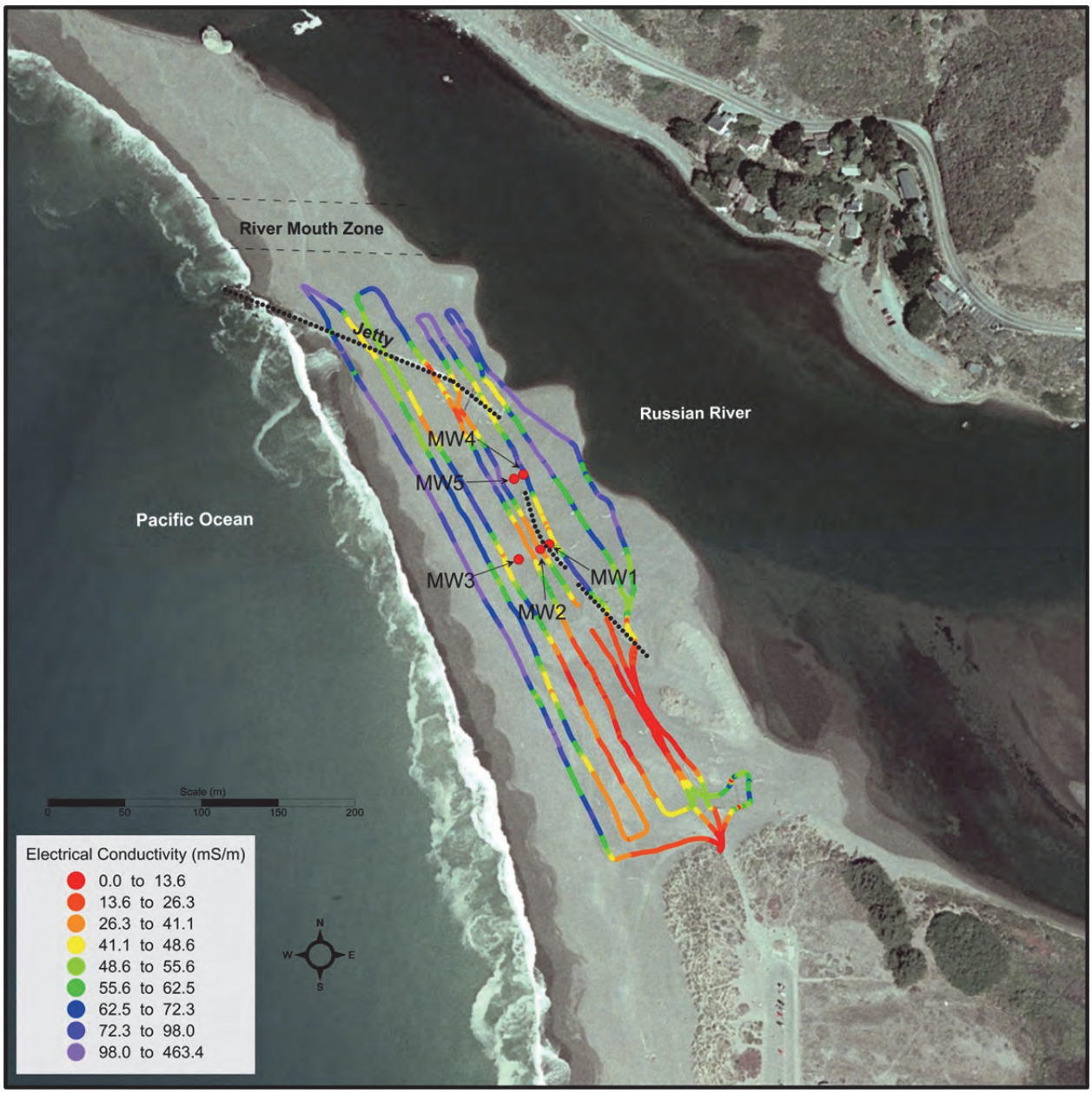

Figure 3. Plan view map showing the Jetty components, open river mouth zone, and electrical conductivity results from March 4th, 2014 survey. Lower salinity/dryer soils are represented by red to yellow colors and higher salinity/wetter soils by green to purple colors. Monitoring wells (red dots labeled by 'MW') installation locations were decided from the electrical conductivity results.

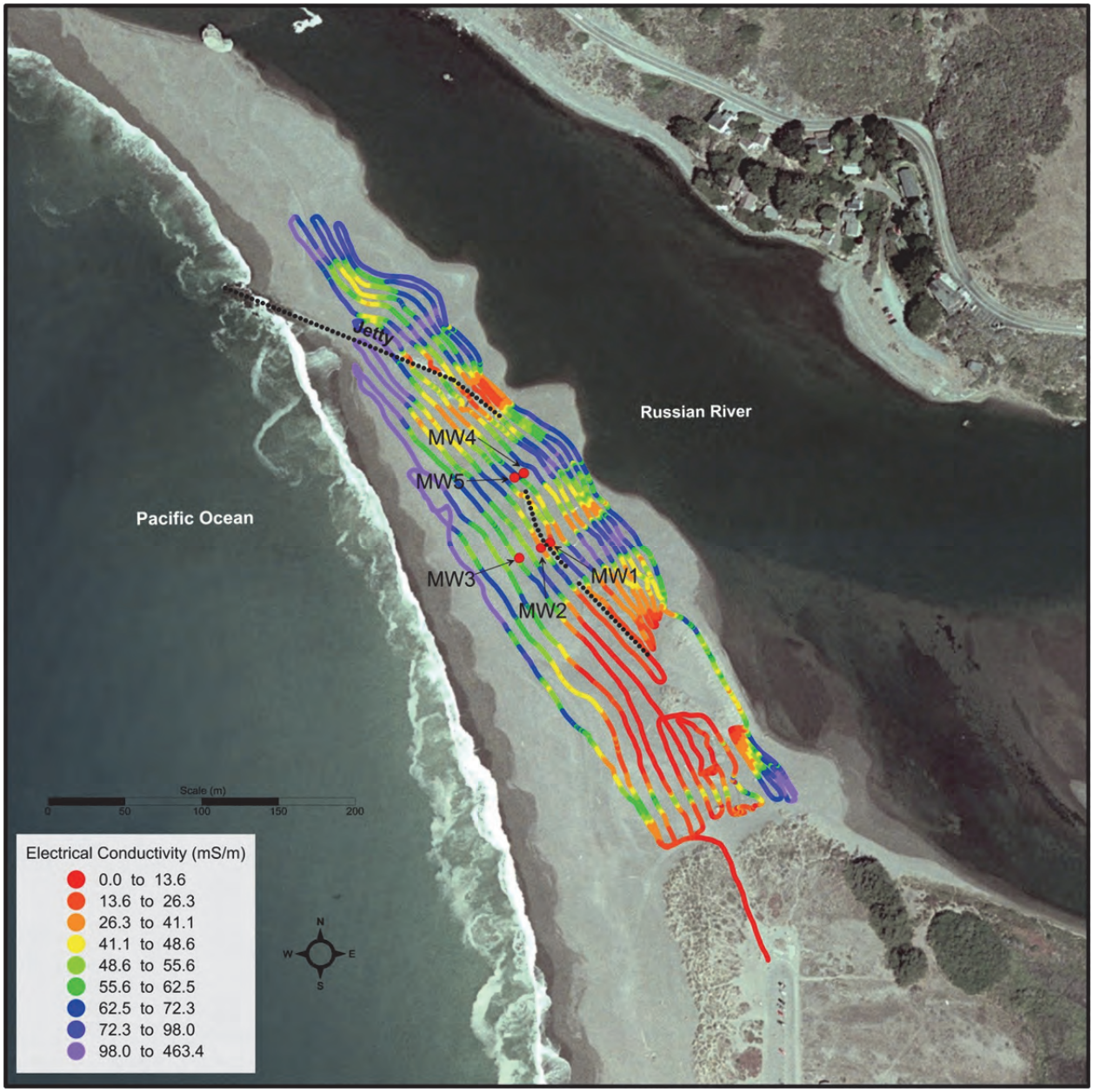

Figure 4. Plan view map showing the Jetty components, open river mouth zone, and electrical conductivity results from September 30th, 2014, survey. Lower salinity/dryer soils are represented by red to yellow colors and higher salinity/wetter soils by green to purple colors. Monitoring wells (red dots labeled by 'MW') installation locations were decided from the electrical conductivity results.

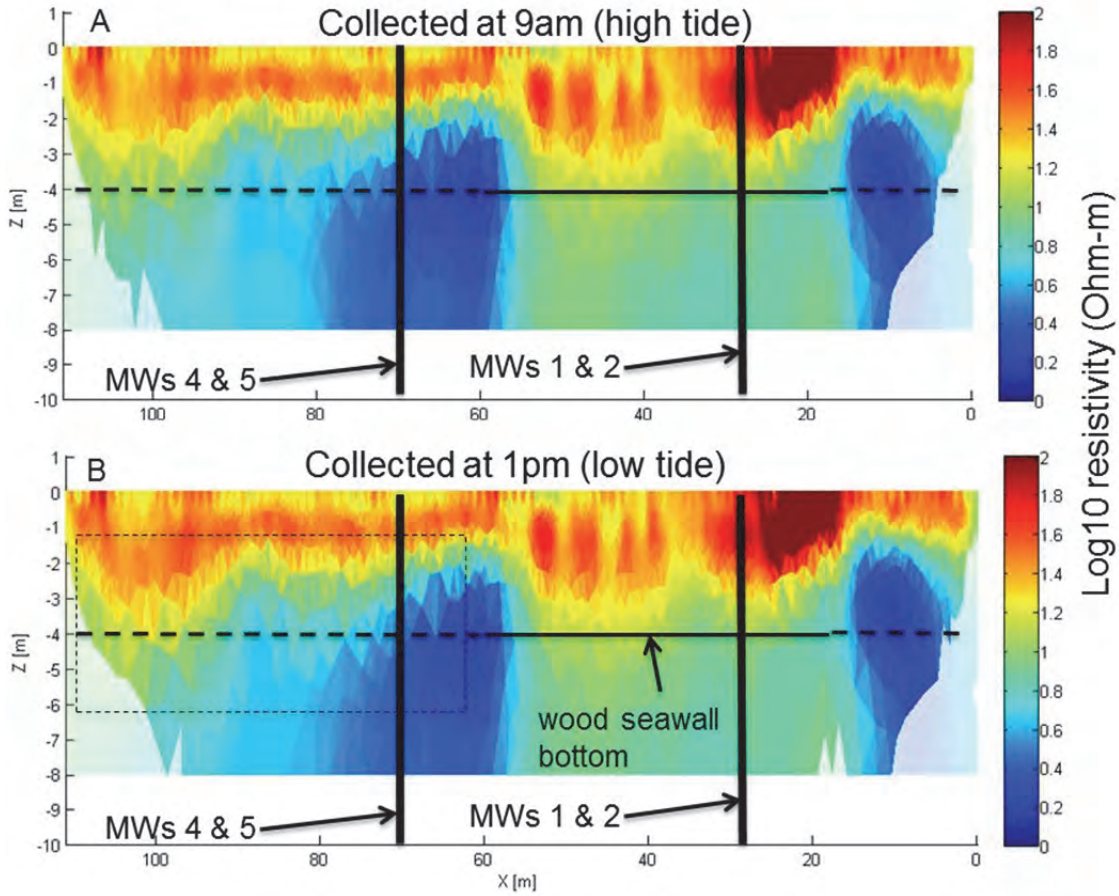

Figure 5. Surface ERT cross-sections collected at (A) high tide and (B) low tide. The horizontal line (-4m) indicates the bottom of the wood seawall, dashed represents no surface exposure and solid exposed at the surface. MWs 1 and 2 are located in the zone of exposed seawall and MWs 4 and 5 in the zone of no exposed seawall. Fresh water (red to yellow) is perched on top of salty/brackish water (light green to dark blue).

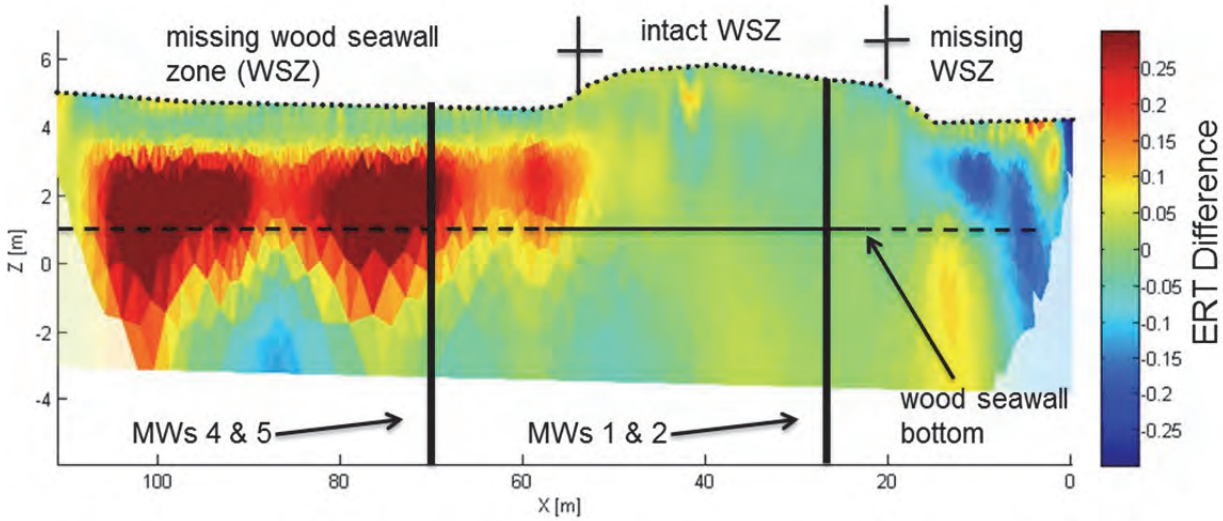

Figure 6. Surface ERT difference (Figure 5 panel A subtracted from panel B) cross-section. The horizontal line (1m msl) indicates the bottom of the wood seawall, dashed represents no surface exposure and solid exposed at the surface. MWs 1 and 2 are located in the zone of exposed seawall and MWs 4 and 5 in the zone of no exposed seawall. ERT difference in decimal of percent (i.e. 0.25 = 25% change), red colors indicate increase of fresh water (more resistive) and blue increase in saline water (more conductive).

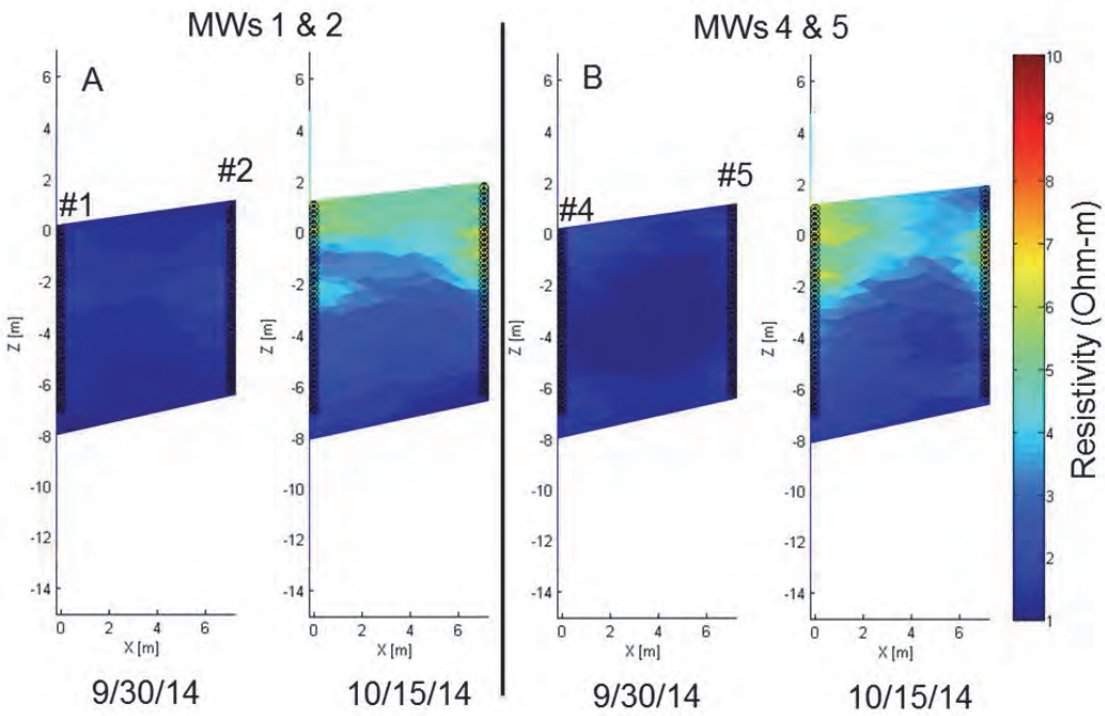

Figure 7. Cross-well ERT cross-sections collected between MWs 1 and 2 and MWs 4 and 5 at two estuary elevations. On September 30th estuary stage is at 1.82 m above msl, and on October 15th 2.4 m above msl. Cool blue colors indicate saline water, and light green to red colors indicate more fresh water infiltration on October 15th.

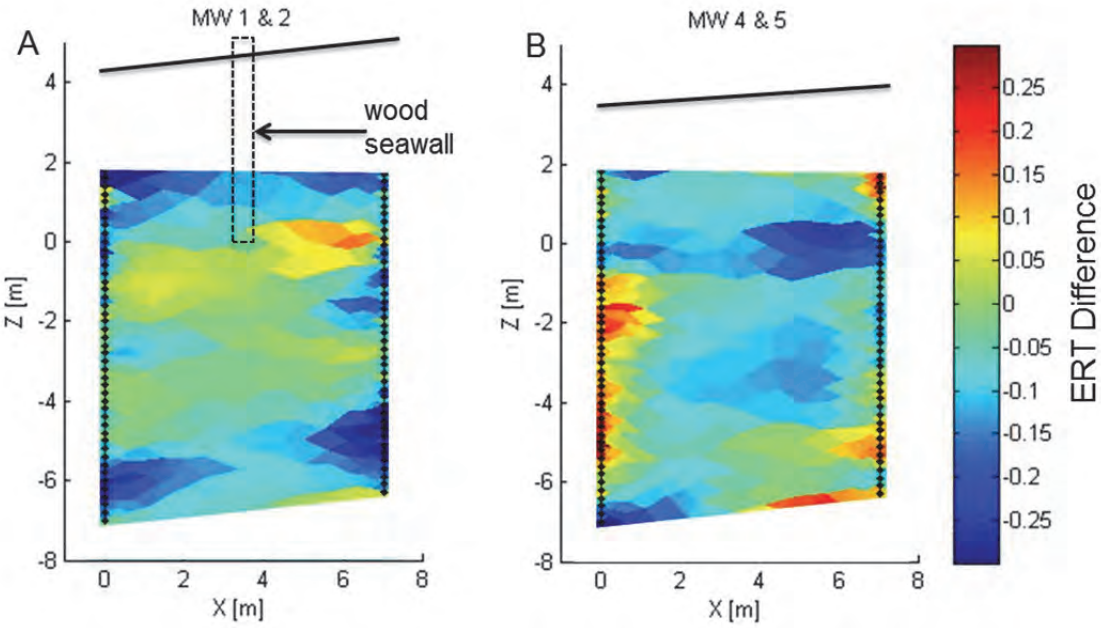

Figure 8. Cross-well ERT difference from low tide to high tide on October 15th, 2014, between MWs 1 and 2 (exposed seawall) and MWs 4 and 5 (no exposed seawall). ERT difference in decimal of percent (i.e. 0.25 = 25% change), red colors indicate increase of fresh water (more resistive) and blue increase in saline water (more conductive). Little to no change is observed between MWs 1 and 2 compared to an average 15% change between MWs 4 and 5.

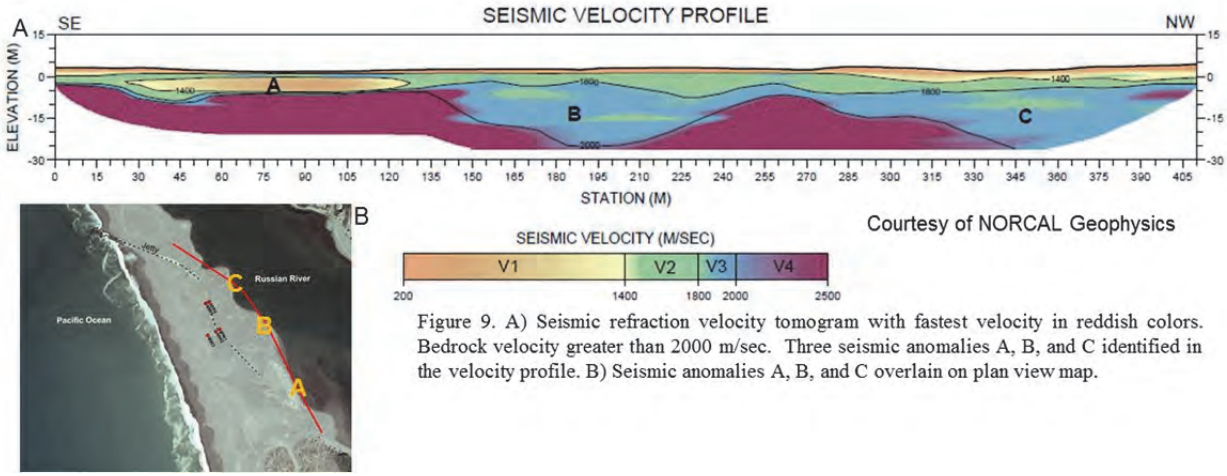

Figure 9. A) Seismic refraction velocity tomogram with fastest velocity in reddish colors. Bedrock velocity greater than 2000 m/sec. Three seismic anomalies A, B, and C identified in the velocity profile. B) Seismic anomalies A, B, and C overlain on plan view map.

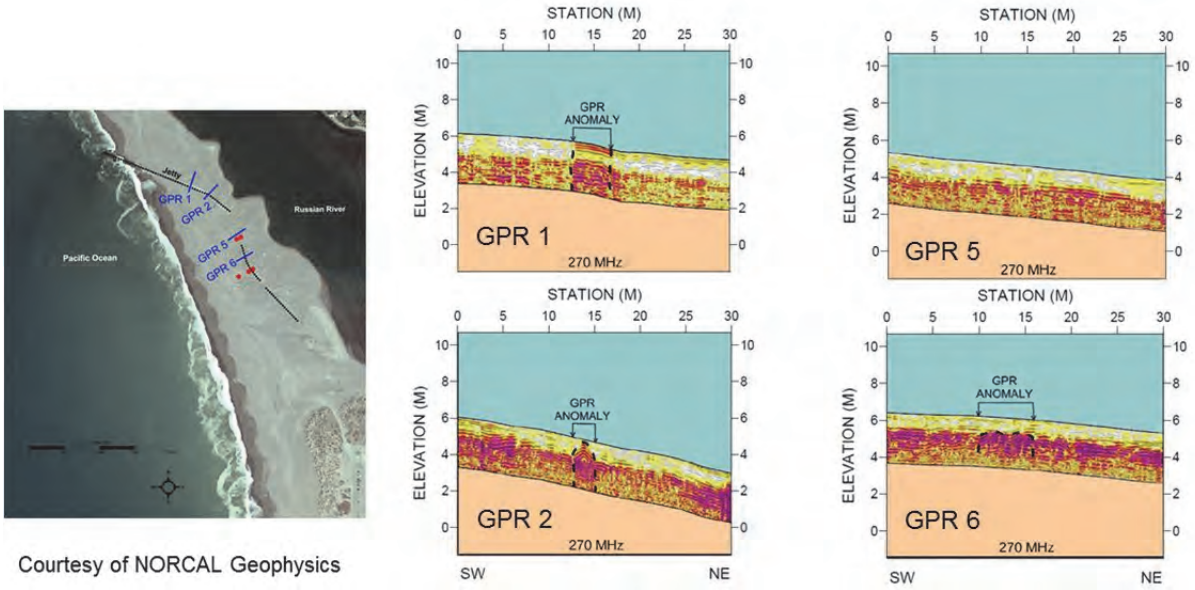

Figure 10: Selected GPR profiles from the areas of interest to investigate the condition of the jetty and support elements. High amplitude reflections are in pink to purple. GPR data collected with 270 MHz antennae.

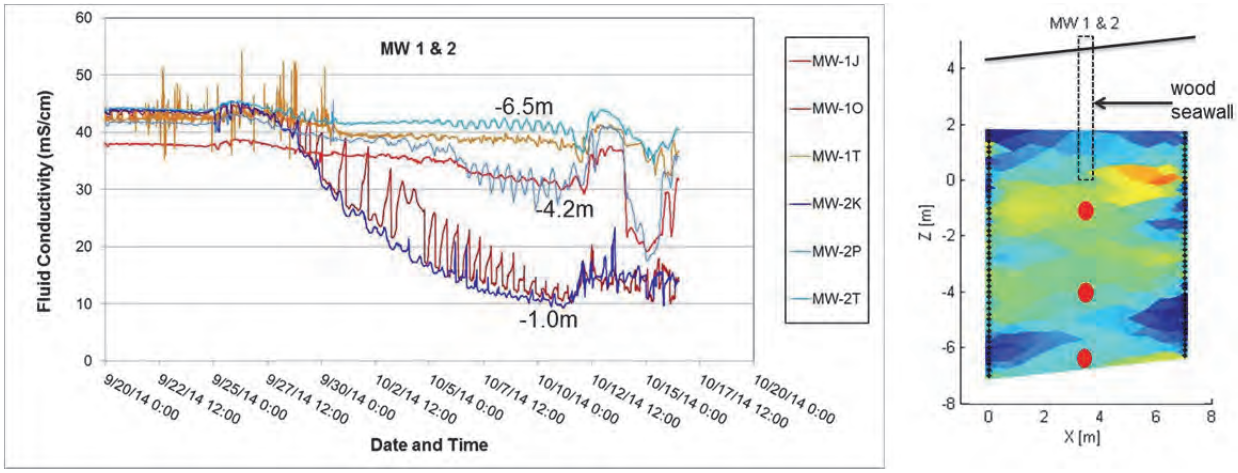

Figure 11: Graph showing raw fluid conductivity from salinity sensors in MWs 1 and 2 at -1.0 m, -4.2 m, and -6.5 m NGVD. The red dots on the ERT cross-well section show the location represented by the salinity sensors. Sinusoidal patterns at each sensor are the result of tidal action rising and falling. River mouth closure occurs on 9/29/15 and ensuing fresh-saline water stratification occurs.

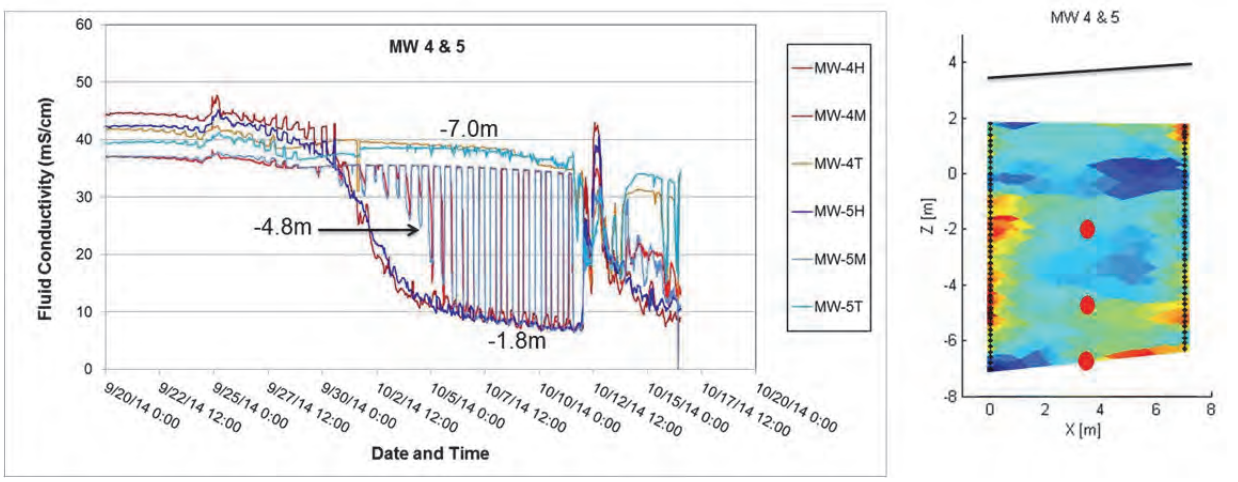

Figure 12: Graph showing raw fluid conductivity from salinity sensors in MWs 4 and 5 at -1.8 m, -4.8 m, and -7.0 m NGVD. The red dots on the ERT cross-well section show the location represented by the salinity sensors. Sinusoidal patterns at each sensor are the result of tidal action rising and falling. River mouth closure occurs on 9/29/15 and ensuing fresh-saline water stratification occurs.

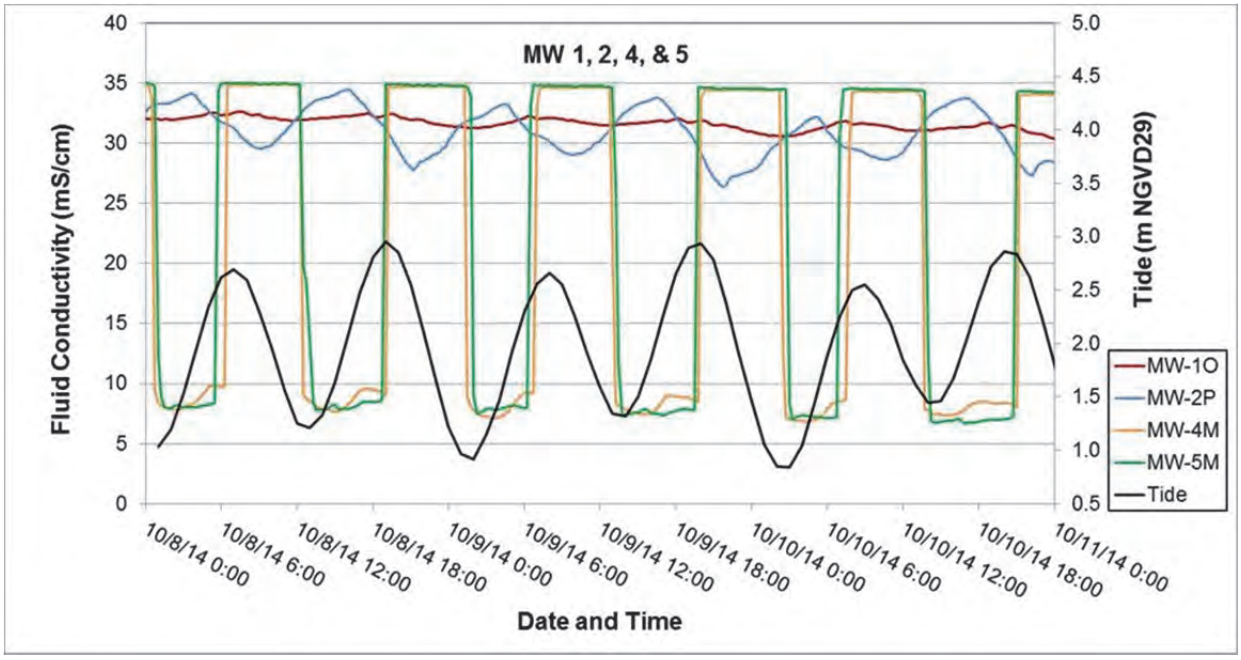

Figure 13: Fluid conductivity in MWs 1, 2, 4, and 5 with daily tides. At MWs 4 and 5 (gold and green lines, respectively) very little lag between peak and trough of salinity signal and matches tidal cycles. MWs 1 and 2 sensors have very little to no amplitude comparatively and a phase shift.