Russian River Estuary Outlet Channel Adaptive Management Plan 2013

Prepared for

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with

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1. INTRODUCTION

Sonoma County Water Agency (the Agency) is required to develop a management plan for the Russian River Estuary mouth in response to a 2008 Biological Opinion (BO) from the National Marine Fisheries Service (NMFS) designed to improve salmonid rearing habitat in the estuary (NMFS, 2008). Prior to the BO, the existing Russian River Estuary management plan focused on artificial breaching to prevent flooding. The Agency retained ESA PWA[1] to assist in developing the revised plan to address the objectives of the BO.

The BO stipulates several phases of outlet channel management over fifteen years with additional management options specified for each phase. The phases are part of an adaptive process for management actions to enhance salmonid habitat. If earlier phases are successful in meeting the performance criteria, subsequent phases will not be needed. The existing plan was first developed in 2009 to address the Phase 1 objectives in the BO and then updated in 2010, 2011, and 2012. This document, the management plan for 2013, is largely based on the plan drafted in 2012. The changes between the 2012 and 2013 plan include: documented 2012 inlet conditions (Attachment G), and updated permitting requirements (Sections 3.2 and Attachment C).

Because of permitting issues, the outlet channel was not implemented in 2009. In 2010, the outlet channel naturally established itself for about one a week at the end of June, and was then closed by ocean waves. After this closure, the Agency mechanically re-created the outlet channel. However, waves closed the outlet channel less than a day after implementation. Before the outlet channel could be re-established by the Agency, the lagoon breached, returning the estuary to tidal conditions for the remainder of the summer. Additional closures occurred in September and October, but large wave conditions and imminent flooding prevented efforts to create an outlet channel. In 2011, the inlet never closed long enough to warrant management action. Wave events caused a series of closures between the end of September and into November. However, the closures lasted a week or less, ending when rising lagoon water levels overtopped the beach berm and naturally scoured a new tidal channel. 2012 was similar to 2011, with the June, July, and October closures ending when overtopping naturally scoured a new channel.

The approach of the 2013 plan is to meet the objective of the Reasonable and Prudent Alternative (RPA), Alterations to Estuary Management, to the greatest extent feasible while staying within the constraints of existing regulatory permits and minimizing the impact to aesthetic, biological, and recreational resources of the site. It is recognized that the measures developed in the 2013 management plan, when implemented, may not fully meet the objective established by the RPA. The concept of this approach was developed in coordination with NMFS, California Department of Fish and Game (CDFG)[2], and California State Parks (CSP). This draft plan was provided to these agencies and discussed at a meeting on March 7, 2013 that included representatives from these three

[1] Previously Philip Williams & Associates
[2] CDFG's CESA tracking number is 2080-2009-016-03 and 1600 Notification number is III-1176-96

agencies, as well as the Sonoma County Water Agency and ESA PWA. Comments on the draft plan from these representatives informed the revision of the draft plan to create the final plan.

The goal of the management plan is to reduce marine influence on the Russian River Estuary (Figure 1) during the management period, May 15th to October 15th. The management actions are intended to limit tidal exchange between the ocean and the estuary. Instead of the existing tidal estuary, the BO proposes a perched lagoon with water levels above tidal elevations. With tidal inflows limited, river inflow to the lagoon may enhance the extent of freshwater habitat for the benefit of juvenile salmonid rearing. Maintaining the lagoon water levels in a perched state that is also below flood stage requires an outlet channel to convey water from the estuary to the ocean over the beach berm.

The outlet channel adaptive management plan is organized as follows. Conclusions and recommendations of this plan are described in Section 2. Sections 3-6 describe the planning and analysis steps: (1) defining project performance criteria (Section 3), (2) developing a conceptual model of relevant physical processes (Section 4), and (3) conducting technical analysis to quantify target outlet channel conditions (Sections 5 and 6). The resulting operations and management plan derived from these planning steps is also documented in this report (Section 7). The adaptive management strategy will continue by actual implementation of this plan, then monitoring and evaluating the outlet channel response to refine the plan for subsequent years.

2. CONCLUSIONS AND RECOMMENDATIONS

Conclusions about the physical processes affecting outlet channel behavior and recommendations for 2013 management are summarized below.

2.1 CONCLUSIONS: PHYSICAL PROCESSES AFFECTING OUTLET CHANNEL BEHAVIOR

1. The location of the outlet channel, at the interface of the Russian River estuary and the surf zone of the Pacific Ocean, is a dynamic system influenced by river discharge, ocean waves, and sand transport. As such, the outlet channel will be subject to variable forcing at hourly, tidal, and monthly timescales. In order for the outlet channel mouth to preserve its function in this active transport zone, the net sediment transport must be small, even though the gross sediment transport is large. To sustainably meet its performance criteria, the outlet channel must be resilient in the face of this variable forcing. This resiliency is difficult to predict.
2. Under current management of the Russian River watershed and estuary, there has been one documented occurrence of target outlet channel conditions occurring during the proposed management season of May 15 to October 15 for the twelve year period of record (1999 to 2010). Outlet channel conditions occurred in June 2010 and persisted for about one week before closing. More typically, as a result of natural processes and existing artificial breaching practice, the connection between the estuary and the ocean has been observed in one of two states: bi-directional tidal exchange (88% of the time during the management period) or fully closed with no exchange (12% of the time).
3. Conditions similar to target outlet channel performance criteria were observed outside the management period five times between 1999 and 2012. These events appeared to be extended transitions to fully tidal conditions rather than stable conditions. Estuary water levels steadily declined throughout all events and the estuary typically returned to tidal exchange within 48 hours.
4. To meet the performance criteria, the outlet channel geometry must simultaneously meet two key constraints: convey sufficient discharge from the estuary to the ocean to preserve constant water levels in the estuary and preserve channel function by avoiding closure or breaching. These two constraints can be in conflict, since both conveyance capacity to preserve estuary water levels and the potential for breaching increase with flow rates but closure is more likely for lower flow rates.
5. The target outlet channel is subject to two failure modes: (1) closure caused by deposition, leading to estuary water levels to rise and possibly cause flooding, and (2) breaching caused by scour, leading to tidal exchange and marine conditions in the estuary. Of the two failure modes, breaching is more detrimental to NMFS's goal of reducing or eliminating exposure of the estuary to tidal water levels and saline inflow. Once breaching occurs, the estuary may persist in a breached state for weeks or months before the target outlet channel can re-form. The immediate impact of closure is only increasing estuary water levels, which allows time for management action to prevent habitat loss.

6. Based on engineering calculations, the channel bed slope must be essentially flat (slope on the order of 0.0001) and water depths less than 2 ft, preferably 0.5 to 1 ft, to reduce the likelihood of channel scour at likely May to October flows.
7. Based on the results of hydrologic modeling, it may be difficult to convey sufficient discharge to maintain estuary water levels while simultaneously keeping the bed shear stress in the outlet channel below the threshold for scour. Even with the anticipated reduced 2013 instream flows, the predicted local bed shear stress during the management period is almost always greater than the critical bed shear stress threshold for erosion.
8. Discharge conditions are a significant source of hydraulic uncertainty for assessing the outlet channel. Discharge measurements are made at the USGS Guerneville gaging station[3], 21 miles upstream from the Russian River's mouth, and changes in flow (losses/gains) are known to occur between the Guerneville station and the mouth. A water balance model for the estuary indicates that net losses between the Guerneville gaging station and the mouth vary from 10% to 53% and average 37%. Limited USGS and Agency discharge measurements at other locations suggest that most losses occur in the lower 6 miles of the river; perhaps in large part due to seepage through the beach berm.

2.2 RECOMMENDATIONS: 2013 MANAGEMENT ACTIONS

1. Two channel configurations will be initially considered for implementation.
    - a wide and short channel that seeks to minimize scour potential; or
    - a narrow and long channel aligned to the north that seeks minimize closure potential.

   The channel selected for implementation will be based on site conditions at the time of closure and discussion with the resource agency management team. Monitoring of the outlet channel and estuary response will be used to inform adaptive management during the management period.
2. Initial management actions may be more frequent, and include maintenance actions that are corrections to the existing channel configuration. Based on experience from these initial efforts, larger and less frequent actions may be undertaken.
3. Once the estuary closes, implement the channel so that when reconnecting the channel, the estuary water levels are no more than 0.5 to 1 ft above the constructed channel bed elevation. This approach reduces the potential for scour.
4. Channel excavation activities should be completed (i.e. the temporary sand barrier removed) coincident with high tides in the ocean. This will reduce the scour potential associated with the initial outflow at the time of breaching.
5. A communication protocol will provide guidance between the Agency and identified points of contact representing key resource management agencies in the estuary.
6. Because of uncertainty about the system and its response to outlet channel management, the adaptive management approach specified in the BO and being pursued by the Agency is

[3] Located just downstream of Hacienda Bridge, USGS station ID 11467000.

critical. A year-end evaluation to assess actual channel performance and revised management for subsequent years is also recommended.

3. PERFORMANCE CRITERIA

The principal estuarine habitat goal stipulated in the Reasonable and Prudent Alternative (RPA), Alterations to Estuary Management, in the BO is to reduce marine influence in the estuary from May 15 to October 15. According to the BO, marine influence includes tidal water level oscillations and saline water. NMFS believes that marine conditions diminish habitat quality for salmonid rearing by reducing the habitat extent, elevating salinity above optimal levels for salmonid juveniles and their invertebrate prey, and flushing juveniles into the ocean.

The performance criteria for outlet channel management are intended to assist in meeting the estuarine habitat objective of the RPA specified in the BO. This section presents performance criteria for Phase 1 of outlet channel management, and minor modifications to these criteria for 2013 management.

Performance criteria for water quality and ecological values in the lagoon are addressed separately and are not included in this document.

3.1 PHASE 1

Phase 1 of outlet channel management has the following performance criteria for the May 15 to October 15 management period:

1. **Estuary water levels**. The estuary water level management target is "[a]n average daily water surface elevation of at least 7 feet [NGVD] from May 15 to October 15" (BO, p. 249). Higher estuary water levels, but not exceeding flood stage of 9 ft NGVD, would be preferred by NMFS. However, water levels greater than 4 ft NGVD are expected to accompany reduced marine influence and would be likely to improve habitat.
2. **Sand channel**. The outlet channel will be a temporary feature, created only by excavating and placing beach sand. No new structures or mechanical devices, temporary or permanent, will be a part of the outlet channel implementation.
3. **Minimize artificial breaching**. Though the overall goal is to create a freshwater estuary, and therefore avoid artificial breaching, in light of natural variability of river discharge and nearshore wave conditions, several years of experience managing the estuary may be required to develop operational procedures which minimize the need for artificial breaching. As such, NMFS estimates "that SCWA will need to artificially breach the lagoon using methods that do not create a perched lagoon twice per year between May 15 and October 15 during the first three years covered by this opinion, and once per year between May 15 and October 15 during years 4-15 covered by this opinion" (BO, p. 302).
4. **Economic feasibility**. Operations and maintenance requirements will not place undue burden on the Agency in terms of cost, particularly as it relates to frequency or duration of maintenance activities.

5. **Public Safety**. The outlet channel management plan will not diminish public safety as it pertains to floodplain property owners, visitors and employees of the State Beach, and the Agency maintenance staff.

To meet the criterion for estuary water level (#1 above), the estuary will function as a perched lagoon with "water surface elevation above mean high tide … where freshwater flows out to the ocean over the sandbar at the lagoon's mouth" (BO, p. 92). This implies uni-directional flow in the outlet channel, from the estuary to the ocean, to minimize marine influence, and minimal sediment transport within the outlet channel to prevent the channel bed from scouring and transforming into a tidal channel.

Note that each time the lagoon breaches, NMFS believes the lagoon is subject to undesirable water quality conditions not just during the breached period, but also for some period of time following the subsequent closure. "NMFS anticipates 3-4 weeks of adverse water quality conditions after the sandbar closes at the mouth of the estuary" (BO p. 302). Thus the management plan seeks to minimize natural, as well as artificial breaching events.

The BO requires the Agency to petition the State Water Resources Control Board (SWRCB) to change minimum instream flow requirements to improve rearing habitat for steelhead. Permanent changes in instream flow requirements will take years to accomplish, therefore, the BO also requires the Agency to petition the SWRCB to change minimum instream flow requirements on an interim (temporary) basis to facilitate management of the Estuary as a summer lagoon. The management plan anticipates an interim reduction in instream minimum flow requirements between the Dry Creek confluence and the mouth starting in 2010. Minimum flows would be reduced from current SWRCB Water Right Decision 1610 levels of 125 ft^3/s to 80-85 ft^3/s [4]. The expected reduction in minimum instream flow will provide more favorable conditions for outlet channel management by reducing the potential for scour-induced breaching.

For channel location, the BO suggests the use of "a lagoon outlet channel cut diagonally to the northwest. … Alternative methods may include … use of a channel cut to the south if prolonged south west swells occur" (BO p. 250).

3.2 2013 MODIFICATIONS

As discussed above (Section 1), the approach of the 2013 plan is to meet the objective of the RPA to the greatest extent feasible while staying within the constraints of existing regulatory permits. It is recognized that the measures developed in the 2013 management plan, when implemented, may not fully meet the objective established by the RPA as summarized in Section 3.1 above. The concept of this approach was developed in coordination with NMFS, CDFG, and CSP.

[4] The proposed instream flow requirement is 70 ft^3/s, but "SCWA maintains a 10 to 15 ft^3/s buffer to avoid non-compliance of the minimum standard" (BO, p. 245).

Because of the estuary's coastal location and hydrologic significance, the Agency must manage the estuary's mouth in accordance with multiple land use permits from various state and federal agencies. A table summarizing all these permits is provided in Attachment C. Key aspects of these permits which directly affect 2013 outlet channel management include:

- Excavation is limited to 1,000 cubic yards of sand per event to create a channel 25 to 100 ft wide. The channel width range is consistent with historic widths observed within the management covered by existing permits (Behrens, 2008).
- Management actions are permitted only on Monday-Thursday to minimize interference with public use.
- Management actions cannot be longer than two consecutive days (unless flooding is threatened).
- Access is constrained during marine mammal pupping season (March 15 – June 30) to reduce incidental harassment of habor seals, sea lions, and elephant seals.

Artificial breaching may be required during 2013. With this management plan, the Agency seeks to minimize or avoid such breaches during the management period, but recognizes that they may be needed to avoid flooding of adjacent properties.

4. CONCEPTUAL MODEL

The conceptual model of the outlet channel articulates the project's working assumptions about process linkages between channel features, external conditions (e.g. river flow and ocean processes), and channel performance. These working assumptions are uncertain, and may not capture all relevant processes. However, by making these assumptions explicit, they can be documented, discussed, and tested, all of which are necessary steps in the adaptive management process. Observations of the actual outlet channel response will then enable refinement of the conceptual model. In addition, because the conceptual model is expressed in a relatively non-technical manner, it provides an avenue for public outreach and education about the outlet channel. The conceptual model is not a hydrodynamic, sediment transport model but rather uses empirical observations and geomorphic interpretations to identify likely responses to key forcing parameters, given antecedent conditions and management actions.

Development of a conceptual model for the outlet channel focuses on the essential physical processes and linkages, as well as the management parameters of the channel. Although this approach leaves out some processes which may slightly alter the channel's performance, it prevents the conceptual model from becoming so complex that it becomes unwieldy. In addition to limiting the conceptual model's scope to only the essential processes, the model also excludes impacts of the outlet channel on water quality and ecological aspects of the estuary. To further enhance model clarity, the conceptual model is presented graphically with a schematic that reflects the layout of the physical system. One caveat to simplification is that the static, schematic diagrams clearly do not encapsulate the full complexity of this dynamic system.

The conceptual model first describes target conditions for the outlet channel, in accordance with the performance criteria in Section 3. Then the model identifies the morphological processes which may lead to the two failure modes for the outlet channel: closure and breaching. Closure refers to sand transport induced by ocean waves that deposits sufficient volume of sand in the outlet channel mouth that it blocks the outlet channel. Closure prevents discharge through the outlet channel, leading to increasing estuary water levels and the threat of flooding. Breaching refers to the flows enlarging the outlet channel to the point that it becomes a tidal inlet subject to bi-directional flow. It is important to note that these "failure modes" are conditions associated with natural tidal inlets and river mouths, but are considered problems at the Russian River Mouth because modified forcing parameters have affected the timing and frequency such that native species may be adversely affected (see the BO), as well as conflicts with other man-made constraints. One of the key questions in this management plan is whether the inherently dynamic system can be "trained" to drain gradually without breaching and then closing repeatedly.

There are additional aspects of the site which may impact the outlet channel, but whose impacts are thought to be secondary or not well defined. Therefore, they are not included in the conceptual model at this time. If implementation of the outlet channel suggests these aspects are important, they will be incorporated into a revised conceptual model. These aspects include large rocks and/or bed rock within the beach berm, jetty impacts on seepage, and decadal changes to beach width.

Specifically, the jetty at the river mouth and the fill across the tombolo to the south of the site may have affected littoral processes and mouth dynamics, but are not addressed in this study.

This conceptual model is based on existing literature, knowledge of similar estuaries, professional judgment, and ongoing discussion with the Agency, NMFS, CDFG, and CSP. New data and experience adaptively managing the outlet channel will be used to revise the conceptual model in subsequent management plans.

4.1 TARGET OUTLET CHANNEL CONDITIONS

The conceptual model for target outlet conditions is shown in Figure 2. Ideally, the outlet channel conveys water from the estuary to the ocean so that estuary can be maintained in a non-tidal state during the management period. A key performance criterion of this non-tidal state is that the water levels in the estuary (h_l) fall within the range of 4 to 9 ft NGVD, with elevations above 7 ft NGVD preferred. The estuary water level will not be managed directly, e.g. by pumping. Instead, it will be managed indirectly by management actions dictated by the BO, the operation and maintenance of the outlet channel and the reduction of instream flow requirement.

The estuary water level is determined by the balance between inflowing river discharge (Q_r) and three outflows: outlet channel discharge (Q_c), evaporation (Q_e), and seepage through beach berm (Q_s). For estuary water levels to remain within the target range, the inflow and outflows must sum to zero when averaged over a period of several days. As indicated by the width of the arrows depicting these flows in Figure 2, the river inflow, seepage and the outlet channel discharge are the three largest flows; evaporation is a minor factor in the water balance. As such, the sum of the seepage and outlet channel discharge capacity needs to nearly match the river discharge. If the combined outflows are too low, the estuary water level will rise to flood stage and artificial breaching will be necessary. If the outlet channel discharge is too high, the channel will scour and deepen, allowing tidal flows to enter through the channel. The outlet channel discharge is determined in part by its width, bed elevation, slope, and planform alignment. These parameters can be managed to a certain degree, but are likely to evolve in response to the natural variability of the discharge and wave forcing, and the effects of tide range. Seepage is determined by the beach berm's permeability, the water level difference between the estuary and the ocean, and the ambient conditions of the regional water table (Largier and Behrens, 2010). Presently, only the water level difference is subject to management influence. In the future, modification of the jetty to increase the beach berm's hydraulic conductivity will be studied (NMFS, 2008). The river inflow is another management parameter, however, since its value is determined as part of a separate water supply determination and permitting process, its manipulation is not considered here.

Although sediment transport will be minimal within the outlet channel under target conditions, the channel's mouth will perpetually be an active transport zone. This portion of the channel, at its interface with the ocean, will be an active transport zone for two reasons. First, it lies within the surf zone and breaking waves move up and down its face in response to the tides and variations in wave direction, magnitude, and period. Second, this wave action creates a slope on the order of 10:1, which is sufficiently steep that flows of nearly any magnitude from the outlet channel will accelerate to above the scour velocity threshold. In order for the outlet channel to persist with this active transport zone at its mouth, this zone will have to experience minimal net sediment transport. In other words, tidal fluctuations in water level and variability in wave intensity will cause the locations of scour and deposition to shift at hourly timescales, but averaging across several tidal cycles, any sand lost by scour will be balanced by an equivalent amount of deposition. This active transport zone also plays a significant role in lateral migration of the existing channel mouth. This process is discussed in Section 4.4 on planform alignment.

Preserving these target conditions, particularly the discharge conveyance capacity, requires that the outlet channel maintain its cross-sectional flow area. This flow area can decrease or increase, leading to the two failure modes of the outlet channel, closure and breaching. These two failure modes are discussed in the sections below.

4.2 CHANNEL FAILURE: CLOSURE

The processes which lead to outlet channel closure are likely to originate from elevated total water levels in the ocean (z_{wave}), as shown on the right side of Figure 3. Elevated ocean water levels will move the active transport zone into the outlet channel, increasing deposition at elevations above that of the outlet channel's bed, z_{out}. Once deposition rates exceed any capacity of the outlet channel discharge to scour sediment, a berm will build at the mouth of the outlet channel, causing it to close. This process is thought to occur over one to several high tides, corresponding to one to several days. During the management season, total ocean water level is the combination of two ocean processes, the tides and ocean waves. As offshore waves interact with the coastline and nearshore, they are transformed such that the significant elevation on the beach is a function of the wave direction, magnitude, period and runup. While the tides fluctuate with a predictable schedule, ocean waves vary according to the unpredictable weather and wind patterns over the ocean. Therefore, the total water level can be best characterized as frequency distribution that is based on observed tide and wave data.

If the outlet channel closes and flow through the channel stops, the estuary water level will increase since the continuing river inflow cannot be exported through evaporation and seepage alone. Although seepage rates are likely to increase as a result of increasing water levels, it is assumed that seepage rates will remain below river inflow. As the water level rises, it will again overflow the beach berm when it reaches the minimum elevation of the berm crest. Early in the management season, the flow may overtop the berm below flood stage of 9 ft NGVD. However, as the berm crest elevation rises over the course of the management period, the water levels can rise above flood stage. If more moderate management actions do not stop this rising water level, a full artificial breach, as is currently practiced, will be necessary to prevent flooding.

4.3 CHANNEL FAILURE: BREACHING

The breach failure considered as part of the conceptual model and shown in Figure 4 is breaching that occurs when the outlet channel is operating according to the target conditions described above. Breaching is likely to result from two processes, high discharge which scours the channel bed or seepage-induced bed mobilization. Natural or artificial breaching after a closure event are not discussed in this section because it is assumed that management actions would be enacted to return the outlet channel to target conditions prior to either of these breach mechanisms occurring. Additionally, breaching by wave overtopping or strong river discharge are not considered because these processes are associated with winter storm events, which are rare during the management period.

Because the outlet channel is an unconsolidated bed composed of relatively small particles, it is susceptible to scour by the discharge flowing through the outlet channel. Sand scoured from the channel will be lost to the ocean and there is not a significant upstream source to replace scoured sand. Extensive scour will enlarge the channel to the point of breaching and tidal inflows. To prevent scour, flow conditions within the outlet channel (u_c) must be below the threshold for scouring sand (u_{crit}). This threshold is a function of the sand grain size, which has been observed to be coarse sand, narrowly distributed around 1 mm at the Russian River mouth (EDS, 2009a). Further north on the beach, large rocks imbedded in the beach berm may provide grade control and limit scour. Whether the flow velocity is below the threshold depends on the type of bed material and hydraulic conveyance through the management parameters of the outlet channel’s width, length, and bed slope.

As noted in the description of target channel conditions, the beach face slope is set by wave action in the surf zone and is sufficiently steep that flow velocity exceeds threshold for sand movement for all expected discharge rates. Under target conditions, the sand scoured by this process will be replaced by wave action on high tides, yielding no net change in the channel mouth morphology. However, if the scour is larger than deposition on the beach face, the active scour zone may move landward, into the outlet channel. This upstream movement is similar to nick point migration or head-cutting observed in streams and rivers. It is also the process observed by the Agency’s maintenance staff when the beach berm is artificially breached under current practice. The breaching typically happens very quickly, before wave-induced sand transport can close off the breach in subsequent higher tides.

A second possible mechanism of breaching is seepage-induced sand mobilization, represented in Figure 4 as an arrow associated with Q_s. If seepage rates are sufficiently large, the movement of water through the sand can mobilize sand particles where the seepage flow daylights at the ground surface. Piping of groundwater along preferred pathways, which may exist within or adjacent to the jetty, might encourage this process by increasing flow rates through portions of the beach. Although seepage failure has not been observed at the Russian River estuary, it has been observed at other estuaries including Crissy Field (Battalio et al 2006) and others (Kraus et al 2002). Seepage failure may simultaneously accompany other breach mechanisms and hence be difficult to identify on its own. Or, seepage failure may require a larger head difference between the estuary and the ocean than what occurs at the Russian River mouth because of artificial breaching to prevent flooding.

In contrast to closure which can be managed with further intervention, breaching can immediately and negatively impact NMFS's habitat objectives by allowing the marine influences of tidal water levels and saline water to enter the estuary. For this reason, breaching is more detrimental to NMFS's habitat goals than closure.

4.4 PLANFORM ALIGNMENT

Because of the presence of hard barriers in the form of the southern jetty and the northern cliffs, the outlet channel is expected to occupy an alignment within the same region that the current tidal inlet occupies, as show in Figure 1. At this initial stage in the adaptive management process, the conceptual model for the outlet channel's planform alignment is indeterminate as to a target alignment most likely to facilitate outlet channel sustainability. Therefore, observations and interpretations of the existing channel are presented in this section to provide an indication of factors acting on the proposed outlet channel. Once the outlet channel is implemented and monitored, a more definitive conceptual model for target alignment will be developed.

The exiting channel's initial alignment after a closure is typically straight and set by one of three factors, depending on the breaching mechanisms. When breached by high river discharge, the channel aligns itself to the northwest, primarily in response to the direction of the river flow during these events. When the channel naturally breaches itself at water levels below flood stage, it will overflow the berm at the minimum elevation in the berm crest. For example, in April 2009, this low point was toward the north since this was where the antecedent inlet had lowered the berm crest elevation. The Agency has attempted artificial breaching in several locations; under current practice, the initial alignment is perpendicular to the beach and just to the north of the large rock ("Haystack Rock") at the northwest corner of the estuary (Agency staff, personal communication).

Once breached, the existing channel typically changes alignment because the mouth migrates laterally in response to wave and littoral transport processes (Behrens et al., 2009). Lateral migration by the mouth while the upstream channel lags behind creates a sinuous channel. The direction and magnitude of wave energy and the resultant littoral sand transport are thought to determine the migration direction and extent. For the case of a tidal inlet, the mouth typically moves in the direction of the littoral transport (Dean and Dalrymple, 2002). However, several mechanisms have been identified that enable an inlet to move updrift, opposite to the direction of the littoral transport. Aubrey and Speer (1984) demonstrate that sand bars associated with the inlet's ebb tide delta can attach to the downdrift beach, displacing the inlet in the updrift direction. Pranzini (2001) documents a mechanism whereby riverine sediments discharged to a prograding delta preferentially deposit on the downdrift side side, which translate and rotate the inlet mouth towards incoming wave energy. Aubrey and Speer (1984) also propose that flow patterns created by inlet channel bends can create erosion on the outside of the bend and deposition on the inside, much like the development of river meanders, with a net result of the inlet migrating updrift. Mechanisms similar to these may explain observations by NMFS that suggest that the direction of migration of the outlet channel may be against the direction of littoral transport (J. McKeon, personal communication).

Observations by Behrens et al. (2009) show that the existing tidal mouth typically moves both northward and southward during the management period. Their analysis correlates large changes in mouth location with rapid changes in significant wave height, indicating that the wave processes control the migration process. The bi-directional migration of the mouth suggests that wave energy also changes directions. This is further supported by the resulting shape of the channel, which can develop multiple channel bends in response to the mouth reversing directions. The temporal and spatial distribution of wave energy along the mouth is not well documented since wave observations have only been made offshore and estimates of how the offshore waves are transformed by local bathymetry have not been verified. Studies using trace elements and sand budgets along this stretch of coast indicate reversing directions of littoral transport because of varying periods of convergence and divergence of wave energy (DeGraca, 1976). The predominant direction may be sensitive to the relative contributions of northwest wind waves versus southerly swell. For instance, Behrens et al. (2009) show that mouth migration patterns are significantly different during El Niño years with the channel remaining in at the northern end of its range for the entire summer. They speculate that the decrease in northerly wind waves during El Niño events may explain this phenomenon. Another potential cause for this pattern is the more southerly approach angle of incident swell waves during El Nino years, as suggested by Allen and Komar (2006).

An additional factor which may affect the mouth location is the landward migration of the offshore bar. This bar, which is created by sand eroded off the beach during winter storms, moves landward with the low steepness summer waves. If this bar, which runs parallel to the shore, moves sufficiently close to the channel mouth, it may force the mouth to either side.

5. EMPIRICAL ASSESSMENT OF HISTORIC INLET CONDITIONS

The Russian River inlet is highly variable in form, position, and capacity for tidal conveyance. Analyses of field data and an extensive photographic record of daily conditions show that this variability is largely influenced by tides as well as seasonal changes in wave and river conditions (Rice, 1974; Behrens, 2008). Management actions also influence the timing and duration of closure events (Goodwin and Cuffe, 1994).

When the estuary is open to the ocean, the inlet can take one of the following forms:

- A river-dominated channel with minimal influence from tides and waves. This occurs during short-lived river flood events between December and April.
- A channel controlled by a mix of river flow, tides, and wave action. This is the most common inlet state, with waves tending to deposit sand in the inlet and estuary-to-ocean flows due to tide and river being active in removing sand from the inlet. Estuary tidal range is a fraction of the ocean tidal range, ranging from zero to over 70%, varying in response to sediment infilling and scouring of the inlet channel. Here we give special attention to "marginally tidal inlets", where tidal conveyance is less than 10%.
- A one-way overflow channel with water draining from a perched estuary, i.e., the sand barrier is built across the mouth of the estuary, but the estuary water level is high enough to overflow. Waves have limited control over such an "overflow inlet", and tidal influence is nonexistent. River flow rate controls estuary water level and overflow volume, which determines the susceptibility to breaching.

This section provides an overview of inlet states observed during the years 1999 to 2008, the time period for which the photographic record has been analyzed in detail. The analysis emphasizes the dates corresponding to the proposed management period of May 15 to October 15. The purpose of this assessment is to use existing data to identify relationships between forcing due to river, tides and waves and the response of the estuary mouth ("inlet") – and to explore the frequency of the latter two conditions described above.

5.1 FREQUENCY AND FATE OF RUSSIAN RIVER INLET STATES

The possible occurrence of an "overflow" channel at the mouth of the Russian River estuary was investigated by comparing water level records from the Jenner gage with tidal data from the NOAA Point Reyes station. The focus was to analyze events when the inlet was open for at least 24 hours with water levels remaining above tidal influence and slowly varying. Attention was also given to events when the inlet allowed minimal amounts of tidal interaction. Dates for which the inlet was at least partially open were disaggregated into a series of categories based on the ratio of the estuary tide range observed at the Jenner gage to ocean tide range (defined here as "tidal conveyance") – see Table 1. Estuary tide is driven by ocean tide, but estuary tide range is reduced either due to the elevation of the channel base that precludes complete draining of the estuary to low tide levels or due to the channel size being too small for enough water to be transported between estuary and ocean. The estuary-ocean tidal ratio is thus an indicator of mouth state, with smaller values representing an increasingly choked mouth (near to closure or overflow state).

Table 1 Frequency of observed inlet states from May 15 to October 15 for years 1999-2008.

Inlet state		**Number of days observed**	**Proportion of period**
Tidal conveyance[1]	0-5%	10	0.8%
	6-10%	4	0.3%
	10-29%	82	5.4%
	30-49%	315	20.9%
	50-69%	590	39.2%
	≥ 70%	142	9.4%
Full inlet closure		161	10.7%
Overflow channel, stable or decreasing water level(≥ 24 hours)		0	0.0%
Device error		199	13.2%

[1]Defined as the ratio of estuary tide range to ocean tide range.

The 161 days when the estuary was closed consisted of 26 separate closure events. Of these, 19 were artificially breached and the remaining 7 were natural breaches. Although the low number of natural breach events prevents any statistically significant comparisons with river or wave data, it is worth noting that flows over 400 ft^3/s resulted in natural breaches within 1-2 days of closure. Including all closures, there was a correlation between Guerneville flow and closure duration, with lower flows leading to longer closure periods.

During the years 1999-2008, there were no instances of overflow conditions during the proposed management period, but there were five relevant events that occurred just outside of the management period. All events had decreasing water levels, reflecting down-cutting of the barrier, although the rate of down-cutting was slow enough to prevent tidal interaction for at least 24 hours. Two of these events occurred during October, one in November, and two in May. Three of the events were associated with closure events and most lasted for less than 48 hours. An exception was a five-day event that occurred 6-11 May 2008. In this case, the inlet was breached artificially, and the Agency immediately noted that the channel had become elongated, beginning near "Haystack Rock", nearly 450 feet north of the jetty, and terminating at the jetty. This is uncommon, as post-breach channels are almost always short and wide (Behrens, 2008). The sudden elongation of the channel is likely associated with onshore bar migration.

During tidal periods, tidal conveyance was less than 10% on only 14 days during the management period from 1999-2008. These states were generally a precursor to closure events – all dates for which tidal conveyance was below 10% resulted in closure and the muted tidal state typically lasted for only one or two days. They were most commonly observed during short periods when an artificial breach failed to keep the inlet open for more than 1 or 2 days, or during periods of low flow when the inlet was narrow and elongated. Note that there is a diminishing propensity for the inlet to be in a muted tidal state when it is close less than 30% of the full tide range. This indicates that being in between fully open or fully closed is not a condition supported by natural processes at this site.

5.2 WAVE AND RIVER CHARACTERISTICS

Wind waves and river outflow characteristics strongly influence the behavior of the inlet. These forcings exhibit seasonal patterns and other trends that correlate with different inlet states. Details of these relationships are presented below.

5.2.1 Seasonal patterns

Wave data were obtained from the CDIP Point Reyes buoy and a transformation matrix accounting for shoaling and refraction (e.g. http://cdip.ucsd.edu/) was used to transfer deepwater conditions to conditions at a location at 10-meter depth near the inlet. This method provides a first-order estimate of nearshore wave conditions that is necessary as there is a significant difference between deepwater/offshore waves and those nearshore. Wave energy is greatest in winter, declining through spring, to a minimum in July-August. However, late spring storms and/or early fall storms can occasionally produce waves exceeding 10 feet in the vicinity of the inlet during the management period. As discussed in Rice (1974) and Behrens et al. (2009), predominant swell waves from the northwest are often the cause of prolonged inlet migration or closure during late spring.

Data on river flow at Guerneville[5] show a rapid decline from a maximum at the beginning of the management period (mid-May) to a minimum in August (Table 2). Flows in July through September are low, between 80 and 225 ft^3/s for the years 1999 to 2008.

5.2.2 Conditions during different inlet states

Wave and flow conditions were compared with specific inlet states, as shown in Table 2.

Marginally tidal inlet: There is a relation between tidal conveyance and nearshore waves (H_s is significant wave height). Marginal tidal conveyance (< 10%) occurs during larger waves (H_s of 2.5 to 3.25 feet), consistent with the idea that these are transitory states associated with inlet closure and one needs waves big enough to overcome tidal (plus river) flows. These wave conditions may be lower during periods of weaker river flow. Further, if this marginally tidal mouth condition persisted, it could do so for any weaker wave conditions (which would not close the mouth).

Closed inlet: Estuary water level increase during closure events was analyzed to understand how close these conditions were to a steady-state overflow scenario. In all cases, water levels rose at rates of 0.1 ft/day or faster (Table 2). However, accounting for estuary area, the slower water level rise suggests that it may be possible to achieve a steady state with limited flow over the berm if river

[5] USGS gaging station located just downstream of Hacienda Bridge, station ID 11467000.

flows are of order 100 ft^3/s or weaker. Flows marginally over 100 ft^3/s may be possible, depending on the limit on overflow rate without eroding the sand barrier.

Overflow inlet: All of the five observed overflow events had flows higher than 100 ft^3/s, but only one persisted for more than a couple of days. Further, all of these events exhibited unusual conditions. The October 1999, November 1999 and first May 2008 event occurred during a sequence in which high waves began to induce closure, but a sudden increase in river flow prevented full closure and eroded the channel down to its original state. It appears that overflow conditions only occurred because the initial transition towards closure allowed estuary water levels to temporarily exceed high tide levels. The event in October 2006 occurred after a natural breach of a four-day closure, so the lower flows observed in this case are expected. Finally, the most persistent event in May 2008 was associated with an unusually long channel, which is important in that frictional losses may have encouraged the prolonged high water elevation in the estuary. As noted above, this event was likely due to seasonal onshore bar migration.

Table 2 Comparison of average wave and average river conditions for various ranges of tidal conveyance and water level increase in the estuary. Overflow conditions are analyzed for five events observed outside of the proposed management period.

Inlet state		Guerneville flow, ft^3/s	Nearshore H_s, ft
Open inlet with given tidal conveyance:	<10%	323	3.2
	10-29%	261	2.5
	30-49%	219	2.1
	50-69%	276	2.0
	≥70%	328	1.8
Closed inlet; estuary stage rising at given rates:	0.1-0.29 ft/day	146	2.7
	0.3-0.49 ft/day	175	2.6
	0.5-0.7 ft/day	185	3.4
	≥0.7 ft/day	211	4.1
Overflow channel (outside management period)	Oct 28, 1999	291	15.7
	Nov 4-5, 1999	247	5.9
	Oct 26, 2006	155	2.2
	May 1-2, 2008	323	6.6
	May 6-11, 2008	283	1.3

5.2.3 Analysis of wave runup

The mouth of the estuary is typically closed by waves depositing sediment in the inlet channel during slack high tides, but waves can only do so if wave runup can reach the height of the inlet channel base. Thus, wave runup exceedance curves were generated for each of the management months to assess the likelihood of the (overflow) channel being closed by wave action. De-shoaled deepwater equivalent wave heights were combined with daily higher-high tide water levels to estimate runup height following Stockdon et al. (2006), and assuming a constant beach-face slope.

The height exceeded by 2% of the waves under given monthly wave conditions is shown in Figure 5. Runup is highest in October, with heights of 11ft being exceeded on 1 in 10 days. For May, June and September, runup exceeds 10ft on 1 in 10 days, and this drops to 9ft for July and August. This is consistent with the seasonal cycle of large swell events, due to winter storms in the north Pacific, which may occur in October, and occasional swell events due to storms in the tropical or south Pacific during summer. The locally generated waves due to northerly winds in summer are of shorter period and lower height. These data suggest that wave-induced closure of an overflow channel will be a greater concern at the beginning and end of the May-October management period.

5.3 CHANNEL PLANFORM GEOMETRY

Inlet morphological behavior has been studied by Behrens (2008) for the years 1999-2008 through an analysis of inlet width, length and position estimates derived from photographic records. Data collection methods and error estimates are described in Behrens et al (2009). Inlet planform geometry and closure risk are summarized for different mouth states (Table 3).

Table 3 Inlet planform geometry for overflow conditions and various ranges of tidal muting (May 15 to October 15, 1999-2006). Overflow conditions are analyzed despite the fact that they occurred outside of this timeframe.

Inlet state		**Inlet width[1], ft**	**Inlet length[1], ft**	**Most common configuration**	**Closure risk[2]**
Open inlet with given tidal conveyance:	<10%	25 ± 1.8	530 ± 37.1	≥2 channel bends	81.3%
	10-29%	51 ± 3.6	358 ± 25.1	1-2 channel bends	35.3%
	30-49%	71 ± 5.0	282 ± 19.7	1 channel bend	28.6%
	50-69%	86 ± 6.0	236 ± 16.5	1 channel bend	13.7%
	≥ 70%	92 ± 6.4	221 ± 15.5	Straight	3.5%
Overflow channel (outside management period)	Oct 28, 1999	60 ± 4.2	140 ± 9.8	Straight	--
	Nov 4-5, 1999	20 ± 1.4	360 ± 25.2	Deflected by jetty	--
	Oct 26, 2006	25 ± 1.8	110 ± 7.7	Straight	--
	May 1-2, 2008	65 ± 4.6	100 ± 7.0	Straight	--
	May 6-11, 2008	20 ± 1.4	480 ± 33.6	Deflected by jetty	--

[1] Ranges are based on error estimates from Behrens *et al* (2009).
[2] Defined as the number of observations that were followed by closure within two weeks, divided by the total number of observations.

The data for overflow channel geometry indicate that the limited number of overflow events exhibited a range of shapes. The geometry of the only persistent case (6-11 May 2008) suggests that frictional loss plays an important role in attenuating channel velocity and the resulting downcutting.

However, there is a tradeoff for the frictional losses associated with sinuous channels. For a marginally tidal inlet the channel is long and narrow, with a couple of bends – and there is a very high risk of closure. There is no apparent relation between inlet position (not shown in this table) and tidal conveyance. However, marginally tidal inlets and overflow inlets were observed only at the northern or southern extreme of the inlet's migration range. Inlet width and length are known to vary in concert with river flow during the wetter months of the year and with tidal range during the drier months (Behrens et al., 2009). In general, low-flow conditions (low tides or river flow) appear to encourage inlet elongation and narrowing. Inlet width, length, and the number of channel bends all influence the tidal signal by determining frictional losses in the channel.

5.4 NOTES ON OTHER ESTUARIES

Overflow inlets have been observed in numerous estuaries along the coasts of California, Oregon, Chile and South Africa (and probably other areas with comparable climate and topography) (personal communication, John Largier). These are unpublished observations. Specifically, an overflow inlet is typically observed to persist for 1 to 3 months each year at the mouth of Salmon Creek (10 miles south of the Russian River) and at the mouth of the Gualala River, discussed below. Further, small central coast estuaries exhibit overflow states during spring and summer, e.g., Scott Creek and Waddell Creek. Systems photographed along the Chilean, South African and Oregon coasts are of similar size in terms of river flow and lagoon area. The absence of observations of overflow conditions in larger estuaries, similar to the size of the Russian River, suggests that there is a limit to the flow energy that can be accommodated by flow over a sand barrier of finite width (and thus high slope).

5.4.1 Gualala River

The mouth of the Gualala River is located 31 miles northwest of Jenner. Both its tidal prism and annual river flow are significantly lower than those of the Russian River. Despite this, the sites have several similarities, most notably their similarly sized beaches bordered by headlands. During a typical year, the inlet is closed for the entire summer and is opened by the first major storm of the winter (ECORP, 2005). The inlet requires consistent rainfall to remain open, and it is common for closures to occur within several weeks after each major storm event. As rainfall decreases during the spring, the inlet undergoes repeated cycles involving a closure event, a period of gradual estuary stage increase leading to a natural breach, and finally, several days to several weeks of minimal tidal conveyance and/or overflow conditions culminating in a new closure event. These cycles appear to continue until evaporative and seepage losses counterbalance inflows into the estuary, preventing the stage increase required to cause a natural breach event.

5.4.2 Carmel River

California State Parks adaptively manages the beach berm which creates a lagoon at the mouth of the Carmel River (CA Dept. of Parks and Recreation, 2008). The goal of this management is similar to the goal stated in the Russian River BO (NMFS, 2008): to enhance the freshwater salmonid rearing habitat during summer months. Sometime in April, May, or June, once the Carmel River discharge into the estuary drops below 20-25 ft^3/s, bulldozers are used to increase the height of the beach berm. This elevated berm blocks ocean tides and saline water from entering the estuary, thereby creating a perched lagoon. When forming the elevated beach berm, an outlet channel is also created so that if lagoon water levels exceed 10 feet NGVD, the outlet channel will drain water from the lagoon into the ocean. The outlet channel only conveys water if the discharge to the lagoon does not taper off from 25-20 ft^3/s to 10 ft^3/s as rapidly as expected. Once river discharge falls below approximately 10 ft^3/s, evaporation and seepage export enough water from the lagoon that lagoon water levels no longer increase.

The Carmel Lagoon outlet channel differs from the proposed Russian River outlet channel with respect to several key features, as summarized in Table 4. Overall, the Russian River outlet channel is likely to be more difficult to manage than the Carmel River outlet channel because of its higher required conveyance, longer operational period, and lack of natural grade control.

Table 4 Comparison between Russian River and Carmel River outlet channel features

Outlet channel feature	Russian River	Carmel River
Conveyance capacity	50 ft^3/s	10 ft^3/s
Operational period	5 months (May-Oct)	1 month
Grade control	none	natural rock outcrops

6. CHANNEL CONFIGURATION ANALYSIS

As discussed in the conceptual model for target conditions, the outlet channel geometry must simultaneously meet two key constraints: convey sufficient discharge from the estuary to the ocean to preserve constant water levels in the estuary and preserve channel function by avoiding closure or breaching. Note that these two constraints can be in conflict since both conveyance capacity and the potential for breaching increase with flow rates but closure is more likely for lower flow rates. The technical analyses described in this section inform the range of target channel conditions by quantifying the relationship between outlet channel dimensions, bed scour potential, and hydraulic conditions. The ocean-driven processes associated with closure, the wave runup elevation and planform alignment, are discussed above in Section 5. Preventing breaching, a necessary condition for reducing marine influence on the estuary is the focus of this section.

Since the outlet channel will be located within a bed of unconsolidated beach sand, a key management objective is creating a channel which can sustain its cross section geometry instead of scouring. Breaching can occur if the discharge through the outlet channel is sufficiently forceful to scour the channel bed. To reduce the possibility of scour, threshold design principles (NRCS, 2007) are used to examine channel configurations most likely to avoid scour while meeting the other constraints of the system.

Channel design using a threshold methodology consists of the following steps:

- *Estimate the critical shear stress threshold.* This is a function of the site's bed particle composition, which can be characterized by grain size.
- *Predict hydraulic conditions for the proposed channel.* Use engineering calculations of steady flow and a one-dimensional hydraulic model of time-varying flow to estimate the velocity and shear stress for a proposed set of channel geometry, flow, and bed roughness.
- *Compare threshold and predicted bed shear stress.* The estimates from the two previous steps are compared with a factor of safety to account for variations in hydraulic conditions about the mean and uncertainty in parameter estimation.
- *Sensitivity analysis and uncertainty.* Evaluate the sensitivity of threshold and predicted bed shear stress to input parameters as well as the factors contributing to overall uncertainty.

6.1 CRITICAL SHEAR STRESS

The critical shear stress is defined as the applied bed shear stress at which sediment motion occurs. The critical threshold represents a balance between the force exerted by the flow on the bed and the resisting gravitational force of individual sediment particles. Flows above the critical shear stress will transport sediment while flows below the critical shear stress will result in no motion. The critical shear stress is dependent on characteristics of the sediment such as sediment density and particle size.

Sediment samples at the Russian River mouth were collected in March 2009 to inform the assessment of critical shear stress within the outlet channel. Ten sediment samples taken along the proposed outlet channel alignment were analyzed to determine the characteristic grain size distribution. On average, 78% of the sediment had a grain diameter between 0.6-2.0 mm (coarse sand), 18% was greater than 2.0 mm (granular), and 4% was between 0.2-0.6 mm (medium sand) (EDS, 2009a). Visual observations of grain size by ESA PWA near the mouth indicated a typical diameter between 0.8-1.25 mm (coarse sand).

Based on this assessment of typical beach grain size, ESA PWA estimated the critical shear stress using methods outlined in Soulsby (1997) and Fischenich (2001). For the typical range of observed grain size from 0.8-1.25 mm, a critical shear stress of 0.4-0.7 Pa (0.008-0.015 lb/ft^2) was determined for sand particles in the vicinity of the proposed outlet channel (Attachment A-1).

6.2 PREDICTED HYDRAULIC CONDITIONS

6.2.1 Steady mean flow conditions

ESA PWA conducted a preliminary assessment of outlet channel hydraulics under steady typical summer flow conditions as a screening tool to characterize the range of possible channel geometry parameters (bed elevation, channel slope, width, and length). Simple hydraulic equations for open channel flow were used to estimate the in-channel velocity and bed shear stress.

ESA PWA evaluated different combinations of river discharge, bed roughness, channel slope, and flow depth to evaluate channel performance. For a given discharge the hydraulic equations can be solved to determine the values of slope, width, and depth that satisfy the critical shear stress threshold for sediment motion. Once one of these three parameters is selected, the other two are fixed to meet a given shear stress threshold (NRCS, 2007). Multiple combinations of channel slope and width are capable of conveying the design flow at or below the critical shear stress threshold.

Figure 6 shows an example slope-versus-width stability curve for the outlet channel design. A stability curve is a tool used by designers to evaluate channel stability under a range of feasible slope-width combinations. Any combination of slope and width that falls on the stability curve will be stable for the prescribed discharge. Combinations of width and slope that plot above the stability curve will result in erosion and scour of the channel. Combinations of width and slope that plot on or below the stability curve will be stable (or depositional). For a given width, the depth of flow can be determined from the corresponding depth-width curve (Figure 6). For example, a 100-ft wide channel discharging 70 ft^3/s will be stable for channel slopes less than approximately 0.000125 and will flow at a depth of approximately 11 inches. The stability curve shows that as slope increases, channel width must also increase to keep channel velocities below the critical threshold for transport. Channel width and depth are inversely related for points on the stability curve, resulting in either a narrow channel with relatively deep flow or a wide channel with relatively shallow flow.

6.2.2 <u>Calculation of estuary inflows</u>

ESA PWA developed and calibrated a water balance model based on observed lagoon water levels at Jenner, CA. The purpose of the water balance model is to estimate the reduction in river discharge that occurs over the 21 river miles between Guerneville, a USGS continuous discharge gaging station, and the mouth of the estuary. The losses in discharge are attributed primarily to seepage through the beach berm (Largier and Behrens, 2010), with diversions, interaction with the adjacent aquifer, and groundwater pumping as possible contributing factors. No direct observations of these loss terms is available. The reduction factor serves as the calibration variable for the water balance model. For all cases, predicted estuary water levels during closure periods do not match observations unless lagoon inflows are reduced relative to the Guerneville discharge.

Model Setup

During a closure event, the rate of water level increase is a direct function of the net flows into and out of the lagoon (Goodwin and Cuffe 1993):

$$\frac{\Delta V}{\Delta t} = A\frac{\Delta h}{\Delta t} = \alpha Q_R - A\, i_{evap} - Q_s$$

where:	$\triangle V$	=	lagoon inflow during closure (ft^3)
	$\triangle t$	=	duration of closure (days)
	A	=	surface area of the lagoon (ft^2)
	$\triangle h$	=	change in water level in the lagoon (ft)
	Q_R	=	river discharge at Guerneville (ft^3/day)
	α	=	discharge reduction factor for groundwater losses
	i_{evap}	=	rate of evaporation from the lagoon (ft/day)
	Q_S	=	rate of seepage loss through the barrier beach (ft^3/day)

All terms in the water balance equation can be measured or approximated to allow calculation of α, the discharge reduction factor, for each closure event. The components and data sources of the water balance model are described below:

- Estuary water level and inlet state ($\triangle h$) – Jenner water level time series, (SCWA, 2000-2007). The inlet was assumed to be closed (no flow) during the calibration, based on periods when the estuary water levels were non-tidal and increasing estuary water levels.
- Guerneville discharge (Q_R) – USGS gaging station 11467000 (Russian River near Guerneville, CA at Hacienda Bridge) (http://waterdata.usgs.gov).
- Evaporation (i_{evap}) – estimated based on climatological evaporation rates for CIMIS evapo-transpiration reference Zone 1 (California coast) (*www.cimis.water.ca.gov*, Attachment A-3).
- Berm seepage (Q_S) – estimated using Darcy's Law based on water level difference between lagoon and ocean (Attachment A-4).
- Lagoon stage-storage curve (A) – determined from 2009 sidescan survey and LiDAR digital elevation model (EDS 2009b).

The volume of water entering the closed lagoon as a result of waves overtopping the beach berm is not included in the water balance model. Two lines of reasoning provide the basis for this exclusion. First, wave conditions during the May through October management period are generally associated with beach berm building, not with extensive overtopping and berm erosion more prevalent during winter storm events. The wave runup analysis in Section 5.2.3 confirms that runup elevations sufficient to overtop the berm are infrequent. Second, the observed water levels used in the water balance model exhibited nearly constant rates of increase, typically over two days or more. Short periods of rapidly changing water levels indicative of overtopping were not used in the water balance analysis.

Model Calibration

The observed rate of water level increase ($\triangle h/\triangle t$) in the lagoon during 18 closure events was calculated from the Jenner gage data. Rates of water level increase ranged from 0.4 ft/day to 3 ft/day and averaged 1 ft/day. The required inflow ($\triangle V/\triangle t$) to yield the observed rates was calculated based on an assumed lagoon surface area (A) at closure of approximately 400 acres. From the observed average discharge at Guerneville (Q_R) over each closure period, a discharge reduction factor, α, was calculated for estuary inflow during each of the closure events. The percent reduction ranged from 10% to 53% and averaged 37% (Attachment A-5). The largest reductions in discharge typically occurred in summer and were less in the spring and fall.

The reduction factors were averaged over each month from May-October to approximate a seasonal trend. The resulting calibration curve (Attachment A-5) was used to reduce the anticipated Guerneville discharge in the unsteady hydraulic modeling discussed in Section 6.2.3 to predict downstream flow rates into the lagoon based on upstream discharge measurements.

Comparison with Discharge Measurements

A limited set of USGS and Agency discharge measurements provides estimates of river flow at other locations besides the continuous discharge measurements at Guerneville. These discharge measurements, collected at four stations[6] in the 14 miles below Guerneville, typically fall within 10% of the Guerneville average daily discharge. For example, Behrens and Largier (2010) found that the longest record, collected by the Agency in 2009 at Vacation Beach, agreed to within 10 ft^3/s of the discharge measurements made at the permanent USGS Guerneville gage. These relatively low losses suggest that the losses calculated to complete the estuary water balance occur downstream of these discharge measurements, in the lower 6 miles of the river. Since the results of the water balance are used to estimate estuary inflow in the unsteady hydraulic model (see Section 6.2.3 below) and have a significant level of uncertainty, the estuary inflow values in the unsteady hydraulic model may not represent actual estuary inflow. Presently, the existing data are insufficient to fully characterize the losses between the discharge measurements and lagoon water levels. Higher

[6] Data available from USGS National Water Information System (http://waterdata.usgs.gov/nwis), Russian River station names (site number): Duncan Mills (11467210), Monte Rio (382757123003801), Vacation Beach (11467006), and Rio Nido (383012122574501).

rates of seepage through the beach berm are one possible explanation. Largier and Behrens (2010) estimate seepage rates to average 60 ft^3/s for all closure data. Their seepage estimates vary from approximately 30 ft^3/s when the estuary is closed and its water level exceeds the ocean water level by 2-3 ft to more than 70 ft^3/s when the water level difference exceeds 5 ft. Substantial uncertainty about the seepage rate, on the order of ±20 ft^3/s, remains; therefore monitoring to resolve this discrepancy is recommended in Section 7.7. The implications of alternative lagoon inflows are discussed in the model sensitivity analysis and outlet channel management sections of this report.

6.2.3 Hydraulic modeling of unsteady mean flow conditions

Using the calibrated water balance model results described in Section 6.2.2, ESA PWA developed a hydraulic model to evaluate the performance of the outlet channel for various hydrologic scenarios. This modeling is a refinement of the steady mean flow calculations described in Section 6.2.1 because it quantifies estuary discharge, explicit channel geometry, and temporal changes in hydraulic parameters. Sources and sinks accounted for in the model include river discharge, groundwater losses, berm seepage, evaporation, and outlet channel discharge (described in more detail in Section 6.2.2 and Figure 7). Flow in the outlet channel is represented by one-dimensional channel hydraulics as a function of estuarine water levels, channel dimensions, channel slope, and bed roughness. Tidally-varying ocean water levels are included in the model, but since these water levels stay below the channel's bed elevation, they do not influence flow in the channel. Initial channel dimensions were based on the results of the preliminary analysis described in Section 6.2.1. Model channel geometry was revised iteratively based on subsequent hydraulic analyses and discussions with the Agency and NMFS. Channel geometry is fixed throughout the simulation, even though the channel may be subject to scour and its mouth lies in the active transport zone created by ocean waves (Section 4). This assumption has been made because currently available data and models cannot adequately characterize the active transport zone. The management implications of this assumption are discussed in Section 7. The model simulates estuary water levels and outlet channel flow for the period spanning proposed outlet channel operations, from May 15 to October 15.

Discharge Boundary Condition

ESA PWA analyzed historic discharge data at Guerneville to select a "typical" water year for the hydraulic model boundary condition. A time series of monthly discharge was obtained from USGS for the time period from 1970 to 2008 and compared to the median monthly discharge for the duration of record to select a typical water year. For each month, the difference between the month's discharge and the median monthly discharge was computed. The sum of the differences (for May-Oct only) was used to rank each year relative to median conditions. Based on this ranking, the 2000 water year was selected as the most typical year (Attachment A-6).

The year 2000 discharge time series was used to generate a synthetic discharge time series to approximate anticipated reduced instream flow conditions. A measured time series is preferable to using the median daily discharge because it retains some of the short-term variability in the observed flow rates. A synthetic discharge time series for anticipated flow conditions was derived from the typical discharge time series by scaling the Guerneville discharge to an average summertime flow of

120 ft^3/s. This reduction to 67% of observed 2000 discharge is based on the anticipated reduced instream flow requirements (Section 3.1) versus historic instream flows. When flows are adjusted to average 120 ft^3/s from July to October, short-term variability ranges from about 85-150 ft^3/s. The resulting discharge time series at Guerneville is shown in Figure 7a for the simulation period.

The anticipated discharge time series at Guerneville was further reduced using the calibration curve developed in Section 6.2.2 to account for downstream losses between the gaging station and the lagoon. The resulting estuary inflow time series is shown in Figure 7a. Anticipated inflows to the lagoon vary from approximately 45-90 ft^3/s and average approximately 55 ft^3/s during the summer months. Once seepage and evaporation losses are subtracted from the lagoon inflow, modeled baseline flows in the outlet channel are 45-85 ft^3/s and average 50 ft^3/s.

Model Setup

The configuration for the unsteady HEC-RAS hydraulic model is very similar to the water balance model described in Section 6.2.2. The unsteady model includes the lagoon, outlet channel, and beach face, and simulations span the duration of the operational period, from May 15-October 15. The outlet channel was parameterized as a prismatic rectangular channel with a width of 100 ft and length of 300 ft. Bed roughness (Manning's n) was set to 0.02. The channel bed was set at 5 ft NGVD and transitions to a 1V:70H slope on the beach face. The actual beach face slope is believed to be closer to 1V:10H; however, a milder slope was required for model stability. Sensitivity runs with a steeper beach face slope indicated negligible influence on velocities in the upstream portion of the outlet channel. Time-varying seepage and evaporation losses from the lagoon were estimated from Darcy's Law and CIMIS climate statistics for coastal areas, as described in Section 6.2.2. The time series of these losses used as model input are shown in Figure 7b. Because these combined losses are less than 10% of the lagoon inflow, the modeled lagoon outflow through the outlet channel is similar to the lagoon inflow (Figure 7a). A downstream water level boundary condition was prescribed for the ocean; however, since the outlet channel bed elevation is above the limit of tidal influence (approximately 4.5 ft NGVD), there was no impact on outlet channel hydraulics.

Results

Model runs were conducted for the operational period from May 15-October 15 for the proposed outlet channel geometry described above. Time series of lagoon water level, channel velocity, and bed shear stress were extracted to evaluate channel performance. Bed shear stress and lagoon water level results for the hydraulic modeling are shown in Figure 8a and Figure 8b, respectively. The bed shear stress values shown in Figure 8a are mean model predictions times 1.5 to account for transverse variations in bed shear stress not captured by the one-dimensional model (Fischenich, 2001).

The results for the proposed channel geometry and the anticipated reduced instream hydrology are shown as the "Baseline" curve. The expected range of critical shear stress (0.4-0.7 Pa) is shown in Figure 8a for reference. After the initial higher flow period during the spring and early summer, both shear stress and lagoon water level are relatively constant throughout the summer and fall (July-October). Bed shear stresses fluctuate during this period, but are always above the critical shear stress, indicating likely sediment motion and scouring of the channel. Lagoon water levels (Figure 8b) are relatively constant around 5.6 ft NGVD, resulting in a typical flow depth of approximately 0.6 ft in the channel. Channel velocities average 1.1 ft/s and range between 1.0-1.3 ft/s.

6.3 SENSITIVITY ANALYSIS AND UNCERTAINTY

ESA PWA conducted sensitivity and uncertainty model runs for important variables and parameters to assess their impact on channel performance. The testing focused on conditions that may encourage a stable channel by reducing predicted bed shear stress below the critical shear stress. Parameters tested were reduced outlet channel flow and critical shear stress.

Reduced Outlet Channel Flow

Anticipated flows in the outlet channel are somewhat uncertain because the losses between upstream observed discharges and the outlet channel are not well characterized, as described in Section 6.2.2. The baseline simulation presented in Section 6.2.3 used a calibrated seasonally-varying coefficient to reduce flow rates into the lagoon. Once seepage and evaporation losses are subtracted from the lagoon inflow, modeled baseline flows in the outlet channel are 45-85 ft^3/s. To test channel performance under conditions with further flow reductions (due to higher losses, groundwater recharge, diversions, or berm seepage), a sensitivity run was conducted with outlet channel flows reduced to 25-45 ft^3/s, approximately 45% less than baseline conditions.

Critical Shear Stress

Uncertainty in the critical shear stress for beach sand at the Russian River mouth is primarily due to the fact that the beach is comprised of a distribution of particles of varying diameter (see Section 6.1), as opposed to a uniform grain size. Grain size analyses indicate a narrow distribution of approximately 0.8-1.25 mm diameter sand, for which the critical shear stress ranges from 0.4-0.7 Pa. The critical shear stress for the typical grain size of 1 mm is 0.5 Pa.

Results

The results of the reduced outlet channel flow sensitivity model run are shown in Figure 8a for bed shear stress and Figure 8b for lagoon water level. The 45% reduction in outlet channel flow resulted in reduced bed shear stress and water level. Average water levels and channel depth decreased by approximately 0.1 ft relative to the baseline simulation. Average bed shear stress decreased by approximately 30% to an average value of 0.58 Pa for the summer months. The range of critical shear stress, 0.4-0.7 Pa, is shown in Figure 8a as a blue band. While the predicted bed shear stress for baseline conditions almost always exceeds this range, the predicted bed shear stress for reduced outlet channel flow falls within the range of critical shear stress.

The results of the sensitivity simulations suggest that while the baseline conditions are likely to cause scour, variability in outlet channel flow and critical shear stress could result in a marginally stable channel. If necessary, a wider channel could be excavated (or could develop naturally) to reduce bed shear stress below the critical threshold. This model was not used to predict sediment transport and therefore the modeled channel geometry was held fixed. Under target conditions, active transport is expected at the channel mouth (Figure 2). In order for the outlet channel to persist, scour caused by the outlet channel flow accelerating down the beach face at low tides needs to be balanced by sediment deposition generated by wave action at high tides. However, if the active transport zone moves upstream into the outlet channel, the channel is likely to breach and return to tidal conditions, as shown in Figure 4.

7. PROPOSED OUTLET CHANNEL ADAPTIVE MANAGEMENT FOR 2013

This section describes the 2013 recommended channel management practices related to the BO requirements. Existing management practices for public safety, operator safety, operational responsibility, and other practices not related to meeting the BO objectives are not discussed here. These existing practices are documented in the Standard Operational Procedures: Russian River Mouth Opening (SCWA, 2002).

The outlet channel management described in this section is based on the performance criteria, conceptual model and technical analysis described in the preceding sections, as well as extensive discussion between the Agency, the resource management agencies, and ESA PWA. In addition, implementation efforts provided practical experience for adapting the plan. An account of the 2010 implementation is provided in Attachment E and an account of physical conditions is provided for 2011 (Attachment F) and 2012 (Attachment G). Some uncertainty remains about the exact outlet channel configuration that may best achieve the target performance criteria. This uncertainty arises from the dynamic natural setting for the outlet channel and from the unquantified tradeoffs between channel specifications which may benefit one performance criterion while impairing another criterion. For example, to reduce the likelihood of closure, it may be beneficial to locate the mouth of the channel further north where the coastline's aspect is more sheltered from waves from the north. However, extending the channel's length to the northern location necessitates narrowing its width to keep excavation within currently-permitted volumes. A narrower channel increases the likelihood of scour-induced breaching. The relative importance of these factors is not known, precluding an exact determination of optimal channel configuration. In addition to these uncertainties, actual conditions at the time of closure, such as beach berm topography, may inform the selected configuration.

The assessment of the outlet channel conducted to date suggests two possible configuration options:

- a wide and short channel that seeks to minimize scour potential; or
- a narrow and long channel aligned to the north that seeks minimize closure potential.

The rationale supporting each of these configurations is described in more detail in Section 7.3 and Attachment D below. The configuration that is selected at the time of closure will be documented to the resource management team in accordance with the communication protocol described in Section 9. Performance of implemented configurations will be monitored and documented to test the conceptual model which guides management and to suggest adaptive changes to future management actions, including some combination of these two configurations.

The strategy for outlet channel management is an adaptive and incremental approach. This strategy favors smaller, more frequent modifications over larger, less frequent, modification with less certain outcome. Once experience is gained from implementing the channel and observing its response, it may be possible to make larger changes during each incremental modification. These larger changes will decrease the duration and frequency of management activity, thereby reducing the disturbance impact over time. Management practices will be incrementally modified over the course of the management period (May 15th to October 15th) in effort to improve performance in meeting the goals of the BO.

The approach may be constrained by an excavation volume limit of 1,000 yd^3 and antecedent beach berm topography prior to implementation. This approach will be implemented to the extent feasible while still staying within the constraints of existing land use permits.

To provide context for the proposed management plan, the first section below describes previous breaching practices for the inlet. Subsequent sections describe the target channel initiation, location, dimensions and supporting operations details. A hypothetical implementation scenario for the outlet channel, based on actual beach berm and ocean conditions observed at the estuary from June 30 to July 6, 2009, is provided in Attachment B.

7.1 PREVIOUS BREACHING PRACTICES

Breaching has historically been performed in accordance with the *Russian River Estuary Study 1992-1993* (PWA, 1993) in effort to minimize flooding of low lying shoreline properties in the Estuary. The beach berm was artificially breached by the Agency when the water surface elevation in the estuary is between 4.5 and 7.0 feet as read at the Jenner gage. Breaching was performed by creating a deep cut in the closed beach berm approximately 100 feet long by 25 feet wide and 6 feet deep by moving up to 1,000 yd^3 of sand. Based on experience and beach topography at the time of the breach, the planform alignment of the breach was selected to maximize the success of the breaches. Breaching activities were typically conducted on outgoing tides to maximize the elevation head difference between the estuary water surface and the ocean. After the last portion of the beach berm was removed, water would begin flowing out the channel at high velocities, scouring and enlarging the channel to widths of 50 to 100 feet. As the channel evolved and meandered, it reached lengths in excess of 400 ft. After breaching, the estuary would be subject to saline water inflow throughout incoming tides.

7.2 INITIATION OF EXCAVATION

Initial channel excavation will be performed when the outlet channel first closes following May 15th, the beginning of the management period. Closure is often preceded by a lengthening and narrowing of the outlet channel, muting of the estuary tide range, and/or an increase in mean tide level within the estuary. The Agency will monitor the estuary for these conditions and initiate planning for a management action when they are observed.

Throughout the management period, the Agency's permits with CSP and the California Coastal Commission dictate that management operations cannot occur on Friday, Saturday, Sunday or a holiday because these days coincide with high public use[7]. The incidental harassment authorization stipulates that management actions cannot occur for more than two consecutive days unless flooding

[7] Exceptions can be made in the event of emergency conditions. See Attachment C for more details.

is threatening. During the marine mammal pupping season (March 15th to June 30th), the initiation of Agency operations is further constrained. Outlet channel management activity must be delayed if a pup less than one week old is on the beach along site access pathways and there must be a week-long break between management actions. More details on timing restrictions are provided in Attachment C.

Should the outlet channel close in the weeks immediately preceding the management period, the Agency, in consultation with NMFS, CDFG, and CSP, may initiate excavation to increase the likelihood of entering the management period with the target channel configuration in place.

The constructed outlet channel may also close during the management season, such as following a large wave event. In such circumstances, it will be necessary to perform maintenance on the outlet channel, to re-connect the channel to the ocean before the lagoon water level rises too high above the new (higher) beach berm elevation.

7.3 CHANNEL LOCATION/PLANFORM ALIGNMENT

Two possible channel configurations within the extent of the existing alignment (Figure 1) may be pursued in 2013 since the location that may best achieve the performance criteria is not certain. Alternative channel alignments may be implemented to test the relationship of mouth location on channel stability.

7.3.1 <u>Wide and short channel alignment</u>

Preference for a wide and short outlet channel assumes that channel failure by scour-induced breaching (Section 4.3) is the controlling failure mode to avoid in selecting the channel's configuration. This assumption is based on the consequences of breaching, which returns the estuary to tidal habitat conditions that will persist until a large wave event occurs to renew the closure. Since these closure events are relatively infrequent during the management period (between 1999 and 2008, there were an average of 2.6 closures per management period), the next opportunity for creating freshwater habitat may be months away. In comparison, if the channel fails by closing, which may be more likely for the wide/short channel because of its mouth's location, another management action can be taken to re-open the outlet channel while preserving the freshwater condition of the lagoon. To reduce the possibility of scour-induced breaching, the hydraulic calculations and modeling in the channel configuration analysis indicates that the excavated channel should be as wide as possible. Under existing permits, the maximum width is 100 ft. The hydraulic modeling indicates that even a width of 100 ft is likely to scour; a narrower channel will further increase bed shear stress and the potential for scour. Once this width is selected, only a relatively short channel that is nearly perpendicular to the beach berm is possible to also stay within the 1,000 yd^3 limit on excavation volume. The actual dimensions of the wide/short configuration will depend on the beach berm topography at the time of management action.

For a given lagoon water surface elevation, the wide/short configuration will have a higher average bed slope than the longer channel because of the channel's shorter length. The wide/short approach attempts to mitigate this by splitting the outlet channel into two reaches with varying steepness, as shown in Figure 2. Across the beach berm, a flat slope is recommended to reduce the contribution of bed slope to flow velocity, thereby minimizing the potential for scour. The entire drop in elevation between the lagoon water level and ocean water level is initially located at the end of the outlet channel, in the active transport zone. In the active transport zone, scour caused by the outlet channel flow accelerating down the beach face at low tides may be balanced by sediment deposition generated by wave action at high tide. As indicated by modeling (Section 6.2.3), it is likely to be difficult to avoid scour even in the portion of the channel with a flat bed because the lagoon water level will set up to create the water surface slope necessary to convey the discharge that maintains constant lagoon water levels. So even if the bed slope is zero, the total energy slope (the combination of bed slope and water surface slope) is likely to generate scouring flow.

Failure by breaching may not be the controlling mechanism if the actual flows conveyed in the outlet channel are less than anticipated or if the channel develops an armored layer of larger particles. As discussed in Section 6.2.2, direct observations of the flow that the outlet channel must convey are not available and have been inferred from upstream discharge observations and lagoon water levels during closure events. The anticipated outlet channel conveyance rates average 50 ft^3/s and range between 45-85 ft^3/s. If actual flow rates are less due to losses elsewhere (e.g. berm seepage), the outlet channel will be less likely to scour. For example, the sensitivity analysis scenario with reduced flow rates between 25-45 ft^3/s exhibited conditions less likely to scour (Section 6.3). Channel armoring is the process by which the smaller sand particles are eroded, leaving behind larger particles that have a higher critical shear stress for erosion. Because of the uniformity of particle sizes observed on the beach berm (EDS, 2009a), armoring is thought to be unlikely within the range of target elevations for the outlet channel. Larger particles have been observed in the channel, but only when its elevation is lower and within the tidal regime.

The wide/short approach will be to construct the channel in the same general location and alignment as the preexisting channel (i.e., the location just prior to closure). When pursuing this approach, excavation will simply widen and connect the channel in place. As the channel migrates during the management season, the location of new excavation may follow this migration.

7.3.2 Narrow and long channel alignment

The narrow/long approach to channel design assumes that wave-induced closure (Section 4.2) is the controlling failure mode to avoid in selecting the channel's configuration. By excavating a longer channel that stretches to the northwest, the channel's mouth can be situated in an area that may be exposed to less wave energy. Because of its aspect, the area to the north is more sheltered from waves originating from the north. When large waves originate from the south, the channel will be oriented perpendicular to the incident wave direction, which may enhance the channel's capacity to transport sand that is washed into the channel's mouth by waves (Attachment D). Observations of lateral mouth migration in both directions (Behrens et al. 2009) suggest that waves from both north and south directions play a role in mouth dynamics. Additionally, the narrow/long alignment provides flexibility to locate the channel mouth at a location with a flatter beach face slope, which may reduce net scour (Attachment D). The narrow/long approach is supported by observations of outlet channels that form at some other California river mouths (Attachment D). However, many of these other river mouths drain smaller watersheds that have lower flow rates into the lagoon, and therefore are less likely to breach. Also, these lagoons may not be constrained by the risk of flooding to adjacent property. Without a flood risk, lagoon water levels can rise higher and possibly drive more seepage through the beach berm rather than through the outlet channel. Finally, a longer channel will reduce the average bed slope, which is hypothesized to reduce scour. However, as discussed for the wide/short channel, it is the total energy slope (the combination of bed slope and water surface slope), which drives flow through the channel. Hydraulic analysis indicates that even if there is no slope to the outlet channel (i.e. it is flat), the water level in the lagoon will increase to create the water surface slope required to maintain the outlet channel's discharge. For the anticipated discharge, the corresponding bed shear stress is predicted to cause scour (Section 6.2.3).

The narrow/long approach will angle the channel to the northwest with an approximate aspect of 30-40 degrees with respect to the beach. This angled alignment tests possible advantages of site features such as areas of reduced wave energy and rocks imbedded in the beach.

7.4 TARGET CHANNEL DIMENSIONS

Prior to excavation the proposed outlet channel will be designed by Agency survey staff using computer-aided design (CAD) software. This design will then be used either to manually stake target channel dimensions or to automatically guide the excavation equipment via a GPS-based equipment controls. This operation protocol will ensure that the channel is excavated to the intended design.

7.4.1 Excavation Volume

The quantity of sand moved will depend on antecedent beach topography. To stay consistent with current permits, the excavated volume will not exceed 1,000 yd^3. Once either the wide/short or narrow/long planform alignment is selected, the limit on excavation volume will largely set channel dimensions. If a wide channel alignment is selected, the channel length will be limited so the total excavated volume remains below the limit. Similarly, if a long channel alignment is selected, the channel width will be limited so the total excavated volume remains below the limit. The actual dimensions at the time of implementation will depend on the beach berm topography at the time of implementation. Monthly surveys of the outlet channel, supplemented by spot checks at the time of management actions, will provide necessary information about beach berm topography.

Any sand excavated from the channel will be placed on the adjacent beach and graded to depths of approximately 1-2 ft higher than the existing grade. The placed sand will be distributed in such a way as to minimize changes to beach topography. If the time available for excavation is limited by uncontrollable factors such as tides, waves, seal use, or days when operations are forbidden, sand placed on the north side of the channel may be left in piles up to 3 ft high and not blended into the existing beach topography. The piles may need to remain un-graded on the north side because equipment access to this side is more difficult and may slow down operations. Once the outlet channel is in place, the north side is also less accessible, reducing the impact of any remaining sand piles on public use.

7.4.2 Bed Elevation

The bed will be excavated 0.5 to 1 foot below the lagoon water level along its entire length, to achieve target channel depths (discussed below) upon initiation of flow. Channel bed elevations are expected to be in the range of 3 to 7 ft NGVD, with corresponding lagoon water levels of 4 to 8 ft, using a typical flow depth of one foot. At the start of the management season, lagoon water levels and the channel bed may be on the lower of this elevation range, since the system will have recently transitioned from intertidal to closed and the beach berm may not yet have built up. As the management season progresses, sand is expected to move onto the beach berm, raising the viable bed elevation for the outlet channel. As the beach berm builds higher, it will support higher lagoon water levels while maintaining channel depth within the target range. The upper end of the bed elevation is governed by the flood stage elevation (9 ft NGVD) minus the anticipated water depth and a factor of safety to buffer against flooding. Frequent maintenance will likely be required early in the management season to maintain an open outlet channel as the beach berm elevation builds. Eventually, the outlet channel may be above the typical wave runup elevation, the elevation at which waves may induce channel closure, and close less frequently.

The bed elevation is a key determinant of lagoon water levels and influences the stability of the outlet channel. Higher bed elevations have the advantage of better meeting the BO's performance criteria of higher lagoon water levels. Higher lagoon water levels would increase seepage through the beach berm, potentially reducing conveyance requirements and the possibility of scour in the outlet channel. A higher outlet channel is also less likely to be closed by waves. On the other hand, lower bed elevations reduce the potential energy which may cause outlet channel scour, provide a greater buffer before flood stage, and may reduce the release of oxygen-depleting organic matter from inundated upstream marshes. Developing a better feel the optimal bed elevation is one objective of the adaptive management plan.

The Phase 1 performance criteria are to develop an outlet channel that supports a stable, perched lagoon with water surface elevations at approximately 7 ft NGVD for several months (Section 3.1). Stable conditions imply that river inflow into the lagoon would be approximately the same as the sum of outflow through the outlet channel and seepage through the beach berm. Stable conditions also imply that net sand deposition or erosion does not impair the outlet channel's function. However, this goal may not be achievable in 2013 because additional constraints in place during this year call for modified performance criteria.

The bed slope should be nearly flat within the outlet channel to minimize the likelihood of scouring the bed. This may be difficult to maintain. In particular, incision within the "flat" channel bottom may occur.

7.4.3 Depth

The target range of water depths, 0.5-2 ft, is constrained on the upper end by the maximum depth at which the channel is likely to be stable (not scour). Larger depths would be associated with a narrower channel. The lower end of the range is constrained by the width; shallower depths would require impractically large channel widths to provide sufficient cross-sectional area to convey flow. Shallower water depths represent a greater factor of safety with regard to preventing bed scour since bed friction retards flow speed more strongly for shallower depths. Prior to implementation the predicted rate of water elevation rise within the estuary will need to be considered to determine the bed elevation to achieve the flow depths desired at the completion of the channel excavation.

7.4.4 Width

The width of the channel is estimated to vary within 25-100 ft for consistency with the existing management permits. For the wide/short configuration, the channel bottom would be excavated to a width of 100 ft, the permitted maximum, to reduce the potential for scour. For the narrow/long configuration, the channel bottom width will be approximately 30 ft to achieve the desired channel length and slope while still staying within the 1,000 yd^3 excavation volume limit.

7.4.5 Length

The channel length is estimated to vary within 100-400 ft, consistent with historic channel lengths observed within the management period (Behrens, 2008). Length will be a function of the channel's planform alignment while also balancing with other channel dimensions in order to keep excavation volumes less than 1,000 yd^3. The wide/short configuration would result in channel lengths between 100-200 ft while the narrow/long configuration would result in channel lengths approaching the maximum of 400 ft.

7.5 EXCAVATION TIMING RELATIVE TO THE TIDAL CYCLE

Under the proposed management plan, channel modifications will be initiated during low tide so that after several hours of work, the channel will be completed near high tide. As per existing practices, a temporary barrier will be left between the ocean and lagoon during excavation. When the last material is excavated, then the temporary barrier will be removed at or near high tide. This will minimize the difference in water levels between the estuary and ocean, reducing the potential for the re-connected channel to scour into a fully tidal inlet.

7.6 EXCAVATION FREQUENCY

Creating and maintaining the outlet channel will probably employ one or two pieces of heavy machinery (e.g. excavator or bulldozer) to move sand on the beach. At the start of the management period (late spring or early summer), when configuring the outlet channel for the first time that year, conditions may require operating machinery for up to two consecutive days (as allowed under the marine mammal incidental harassment permit). The precise number of excavations would depend on uncontrollable variables such as seasonal ocean wave conditions (e.g. wave heights and lengths), river inflows, and the success of previous excavations (e.g. the success of selected channel widths and meander patterns) in forming an outlet channel that effectively maintains lagoon water surface elevations. As technical staff and maintenance crews gain more experience with implementing the outlet channel and observing its response, maintenance during the remainder of the management season is anticipated to be less frequent.

In consideration of the natural beach environment and public access, effort will be made to minimize the amount and frequency of mechanical intervention. Outlet channel management activities cannot last for more than two consecutive days. During the marine mammal pupping season (March 15th to June 30th), the duration and frequency of Agency operations is constrained by restrictions on incidental harassment. Seven days must pass between management events. More details on duration and frequency restrictions are provided in Attachment C.

7.7 UNCERTAINTY AND LIMITATIONS

The proposed operations are based on the analyses documented in this report, input from resource agency staff, and on our professional judgment. Uncertainties about the actual estuary inflow, berm seepage, and outlet channel performance remain. As described in Section 6.2.2, the two methods for estimating estuary inflow, the water balance model and limited discharge measurements, predict disparate estuary inflows. Estuary inflow will fluctuate over the management period and may be greater than the modeled inflow. The seepage through the beach berm is based only on inferred, not observed, estimates of hydraulic conductivity. The outlet channel, particularly its downstream end, will be located in a highly dynamic environment that is influenced by changing river flow, tidal water levels and waves. Since the outlet channel will not include any hard structures, all of these sources of hydrologic forces can readily alter the channel's configuration, which may make it difficult to achieve and maintain the channel's successful function. Modifications of the proposed plan in response to actual conditions will be discussed with the resource agency management team and documented according to the communication protocol described in Section 9. Any modifications will be consistent with existing permit requirements.

Adaptive management once the channel is implemented will further enhance management practice. Actual feasibility with regards to the full range of dynamic conditions has not been determined. Risks associated with outlet channel failure have not been quantified. In addition to the channel's performance criteria, there are also water quality and ecological performance criteria for the perched lagoon. These additional criteria have not been evaluated as part of the outlet channel management plan.

8. MONITORING AND ADAPTIVE MANAGEMENT

Monitoring of the outlet channel should be implemented to facilitate an understanding of the channel's behavior and guide adaptive changes to this initial management plan. Adaptive management changes may be made over the course of the management season, in response to natural processes, outlet channel conditions, and/or outlet channel response. In addition, a more comprehensive review at the end of the management season will employ the monitoring data to recommend management revisions for the following year.

Because relatively few closure events occur per year and each one experiences different river and ocean conditions, a comprehensive monitoring plan is recommended to support adaptive management. The monitoring would quantify changes in the beach and channel elevation, lengths, and widths, as well as flow velocities and observations of the bed structure (to identify bed forms and depth-dependent grain size distribution indicative of armoring) in the channel. If feasible, the required monthly beach topography surveys should be scheduled just in advance of potential closure situations (neap tides, low discharge, and/or large wave events). Staff safety, staff availability, pinniped constraints, and/or rapidly changing physical conditions may preclude optimal scheduling of beach topographic surveys. Because monitoring requires human presence on beach, potentially disturbing the seal population, the monitoring frequency represents a balance between management of the outlet channel and minimizing disruption of wildlife.

A list of recommended monitoring tasks for 2013 is provided below in Table 5.

Table 5 Monitoring tasks associated with outlet channel management

Task	Description	Field Activities	Frequency
Recommended			
Operations log	Record of outlet channel management actions and ambient conditions.	Operations staff to generate written record of operations (excavation method, extent, and location) and ambient conditions (weather, ocean state, estuary water level)	Daily to monthly (Depends on operational activity)
Outlet channel location and state	An automated video or still camera station to capture the outlet channel's location and state.	Field staff to install and service a camera, power supply, and possibly communication system on hillside adjacent to estuary.	Hourly imaging (automated); Weekly servicing
Outlet channel discharge measurements	Collected within the outlet channel to verify the channel's conveyance.	Field staff to complete cross sectional flow velocity surveys using flow meter attached to a wading rod with electronic data logger.	Monthly
Outlet channel bed structure	Observe the bed for bed forms and depth-dependent grain size distribution indicative of armoring.	Field staff to collect sediment sample from the surface of the channel bed.	Monthly

	Sediment sampler used.		
Outlet channel topography	Collect outlet channel elevation and width	Field staff to survey outlet channel features using a total station and prism mounted on a survey rod.	Monthly
Beach topography	Collect beach elevation	Field staff operating rod and staff on beach.	Monthly
Estuary flow dynamics	Integrate cross sectional velocity data in estuary at various locations from mouth to Duncans Mills.	A boat with field staff, collecting cross sectional data from mouth to Duncans Mills.	Weekly

9. COMMUNICATION PROTOCOL

A communication protocol will provide guidance between the Agency and identified points of contact representing key resource management groups in the estuary for the implementation of the Outlet Channel Management Plan during the management period (May 15 – October 15). Primary and alternative points of contact have been identified for each of the key resource management groups. These parties, which together are hereafter referred to as the "Team", include: Sonoma County Water Agency, NOAA National Marine Fisheries Service, California Department of Fish and Game, and California State Parks. A list of contacts for these groups is shown in Table 6.

Table 6 Russian River Estuary Management Team

Contact	Level	Organization	Phone Number	E-mail
Chris Delaney	Primary	Sonoma County Water Agency	707-547-1946 (w) 707-975-5606 (m)	cdelaney@scwa.ca.gov
Jessica Martini Lamb	Secondary	Sonoma County Water Agency	707-547-1903 (w) 707-322-8177 (m)	jessica.martini.lamb@scwa.ca.gov
Gary Tourady	Primary	Agency Operator Sonoma County Water Agency	707-547-1065 (w) 707-975-6285 (m)	garywt@scwa.ca.gov
Jon Niehaus	Secondary	Agency Operator Sonoma County Water Agency	707-521-1845 (w) 707-975-3999 (m)	jon@scwa.ca.gov
Robert Coey	Primary	National Marine Fisheries Service	707-575-6090 (w)	Bob.Coey@noaa.gov
John McKeon	Secondary	National Marine Fisheries Service	707-575-6069 (w)	john.mckeon@noaa.gov
Rick Rogers	Secondary	National Marine Fisheries Service	707-578-8552 (w)	rick.rogers@noaa.gov
Bill Hearn	Secondary	National Marine Fisheries Service	707-575-6062 (w)	william.hearn@noaa.gov
Adam McKannay	Primary	CA Dept. of Fish and Game	707-944-5534 (w)	amckannay@dfg.ca.gov
Richard Fitzgerald	Secondary	CA Dept. of Fish and Game	707-944-5568 (w)	rfitzgerald@dfg.ca.gov
Eric Larson	Secondary	CA Dept. of Fish and Game	707-944-5528 (w)	elarson@dfg.ca.gov
Brendan O'Neil	Primary	California State Parks	707-865-3129 (w)	BONEIL@parks.ca.gov
Damien Jones	Secondary	California State Parks	707-875-3907 (w)	dajone@parks.ca.gov

9.1 IMPLEMENTATION OF OUTLET CHANNEL MANAGEMENT ACTIVITIES

A minimum of 24 hours of notice shall be provided to the Team by the Agency in advance of the excavation and maintenance of the outlet channel. Notice shall be submitted by e-mail (see Attachment B.1 for sample) with a general description of the proposed action to be pursued and will typically include:

- Proposed date and time of implementation;
- Design schematic of proposed channel which shall include:
  - Approximate antecedent beach berm height and width;
  - Proposed location and alignment of outlet channel;
  - Approximate outlet channel dimensions including bed elevation, channel depth, width, length, slope and aspect with respect to beach face
  - Predicted estuary water surface elevation at the time of implementation;
- Current river discharge at USGS Guerneville gage (website: http://waterdata.usgs.gov/nwis/uv?cb_00060=on&cb_00065=on&format=gif_stats&period=21&site_no=11467000)
- Predicted 24 hour precipitation as estimated by the NOAA National Weather Service for Bodega Bay (website: http://forecast.weather.gov/MapClick.php?CityName=Bodega+Bay&state=CA&site=MTR&textField1=38.3333&textField2=-123.047&e=0&FcstType=graphical;
- Predicted deep water swell height, period, and direction at San Francisco as estimated by CDIP (website: http://cdip.ucsd.edu/?nav=recent&sub=forecast&units=metric&tz=UTC&pub=public)
- For maintenance actions a general description of maintenance to be performed;
- Presence of seal pups; and
- Equipment to be used for implementation.

Team members shall provide any comments or suggestions to the approach in writing within 12 hours of the proposed implementation time. If Agency does not receive any comments before this time it is assumed that there are no comments to the proposed action. Comments and recommendations will be recorded for consideration on that management action or future management actions, and the Agency will do its best to respond to comments prior to implementation.

9.2 COMPLETION OF OUTLET CHANNEL MANAGEMENT ACTIVITIES

Within 36 hours of completion of outlet channel excavation or maintenance activities the Agency shall provide the Team a summary of work performed. This summary will be submitted by e-mail and will typically include:

- Date, time and period of implementation;
- Estuary water surface elevation at the time of completion;
- River discharge at USGS Guerneville gage at time of completion
- Deep water swell at CDIP Pt. Reyes buoy at time of completion
- Approximate location of the centerline of the channel mouth in distance along beach berm north of the jetty;
- Approximate orientation of channel along the beach berm;

- Approximate dimensions and orientation of the excavated channel;
- Approximate water depth in the excavated channel;
- For maintenance actions, a general description of maintenance performed;
- Equipment used during implementation;
- Presence of seal pups; and
- Photos documenting work completed.

9.3 OVERRIDING CONDITIONS

Certain conditions such as declines in water quality or imminent flooding to properties and structures in the estuary could drastically change the course of management outlined in this plan and may force the Agency to breach the estuary. The Agency shall stay in close contact with the Team on the development of any conditions which could affect the overall course of management. However, rapidly changing conditions may limit the notification lead time given to the Team in advance of management actions to alleviate flooding or water quality concerns.

9.3.1 Flooding

Based on past management experience in the estuary, the Agency has found that if the estuary is in a closed condition, medium to large storm events can produce very rapid rises in estuary water levels. These storm events are frequently accompanied by large ocean swells which can close the estuary if ouflows through the channel are not high enough to counteract the wave forces produced from the large swells. Management to avoid flooding is complicated by safety concerns; the Agency is unable to operate equipment required for channel management activities if ocean swells are too large. In the past the Agency has typically breached the estuary in anticipation of a large storm in order to prevent flooding.

The high water surface elevations pursued under this plan will diminish the storage capacity of the estuary to handle high inflows. Also, based on past management experience, the Agency believes that the outlet channel as described in this plan will be especially susceptible to closure from large swell events. In an effort to avoid flooding of properties in the estuary during the outlet channel management period, the Agency will consult with the Team regarding the possibility of breaching the estuary in anticipation of a large storm event.

9.3.2 Decline in Water Quality

Declines in water quality could have impacts to salmonids rearing in the estuary, other species which reside in the estuary and the public. Potential water quality concerns include, but are not limited to:

- Dissolved oxygen conditions becoming dangerously low to fish and other species;
- Elevated salinity levels in domestic water wells; and
- Elevated bacterial levels.

The Agency will stay in contact with the Team regarding water quality conditions during the management period. Should conditions get to the point that they are potentially dangerous to salmonids, other species, or the public, the Agency shall consult with the Team on potentially changing the course of management. In cases of high bacterial levels, the Agency will additionally consult with North Coast Regional Water Quality Control Board and the Sonoma County Department of Public Health on potential management actions.

10. REFERENCES

Allen, J. and P. Komar. 2006. Climate controls on US West Coast erosion processes. Journal of Coastal Research 22(3): 511-529.

Aubrey, D.G. and Speer, P.E., 1984. Updrift migration of tidal inlets. Geology 92(5): 531-545.

Battalio B, Danmeier D, Williams P. 2006. Predicting Closure and Breaching Frequencies of Small Tidal Inlets – A Quantified Conceptual Model Proceedings of the 30th Conference on Coastal Engineering, San Diego, CA, USA.

Behrens, D. 2008. Inlet closure and morphological behavior in a northern California estuary: the case of the Russian River. Masters Thesis. University of California, Davis. 160 pp.

Behrens, D., Bombardelli, F., Largier, J. and E. Twohy. 2009. Characterization of time and spatial scales of a migrating rivermouth. Geophysical Research Letters. Vol. 36, L09402, doi:10.1029/2008GL037025.

Behrens, D. and Largier, J. 2010. Preliminary study of Russian River Estuary: Circulation and Water Quality Monitoring - 2009 Data report. Prepared for the Sonoma County Water Agency.

Bray, D.I. 1979. Estimating average velocity in gravel-bed rivers. Journal Hydraul. Div., American Society of Engineers, 105 (HY9), pp. 1103-1122.

Bruschin, J. 1985. Discussion on Brownlie (1983): Flow depth in sand-bed channels. Journal of Hydraulic Engineering, ASCE, Vol. 111: pp. 736-739.

California Department of Parks and Recreation. 2008. Carmel River State Beach Lagoon Water Level Management Project – Initial Study, Mitigated Negative Declaration (Draft).

Dean, R. and Dalrymple, R. 2002. Coastal Processes: With Engineering Applications, 488 p. Cambridge University Press, New York.

DeGraca, H. 1976. Study of the Ocean Beaches Adjoining the Russian River Mouth. Unpublished report, Department of Civil Engineering, University of California, Berkeley.

ECORP Consulting inc. and Kamman Hydrology & Engineering, Inc. 2005. Gualala Estuary and lower river enhancement plan: results of 2002 and 2003 physical and biological surveys.

Prepared for Sotoyame Resource Conservation District and the California Coastal Conservancy. 270 pp.

EDS. 2009a. Unpublished grain size distribution data for the beach at the Russian River mouth. Conducted for Sonoma County Water Agency.

EDS. 2009b. Unpublished sidescan sonar bathymetry data and LiDAR topography data for the Russian River from the mouth to Duncan Mills. Conducted for Sonoma County Water Agency.

Fischenich, C. 2001. Stability thresholds for stream restoration materials. EMRRP Technical Notes Collection (ERDC TN-EMRRP-SR-29). U.S. Army Engineer Research and Development Center, Vicksburg, MS.

Goodwin, P. and Cuffe, K., 1994. Russian River Estuary Study: hydrologic aspects of an estuary management plan. Project 1139. For Sonoma County Department of Planning. October.

Kraus, N., Militello, A. and Todoroff, G. 2002. Barrier Breaching Processes and Barrier Spit Breach, Stone Lagoon, California. Shore and Beach, v. 70, n. 4. 21-28.

Jones, C., Brøker, I., Coulton, K., Gangai, J., Hatheway, D., Lowe, J., Noble, R., and Srinivas, R. 2005. Wave Runup and Overtopping - FEMA Coastal Flood Hazard Analysis and Mapping Guidelines Focused Study Report.

Julien, P.Y. 2002. River Mechanics. Cambridge: Cambridge University Press, UK.

Largier, J. and D. Behrens. 2010. Hydrography of the Russian River Estuary, Summer-Fall 2009, with Special Attention on a Five-Week Closure Event. Bodega Marine Lab, University of California, Davis. Prepared for Sonoma County Water Agency.

Limerinos, J.T. 1970. Determination of the Manning coefficient for measured bed roughness in natural channels. Water Supply Paper 1898-B, U.S. Geological Survey, Washington, D.C.

National Marine Fisheries Service. 2008. 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.

National Resources Conservation Service. 2007. Stream Restoration Design. Part 654 in the National Engineering Handbook.

Pranzini, E. 2001. Updrift river mouth migration on cuspate deltas: two examples from the coast of Tuscany (Italy). Geomorphology Vol. 38, pp. 125–132.

Rice, M.P. 1974. Closure Conditions: Mouth of the Russian River. Shore and Beach, 42(1): 15-20.

Sonoma County Water Agency. 1999. Standard Operational Procedures: Russian River Mouth Opening.

Sonoma County Water Agency. 2000-2007. Unpublished water level data recorded at the Jenner Visitors Center.

Soulsby, R. 1997. Dynamics of Marine Sands. Thomas Telford Ltd. London. 272 pp.

Stockdon, H., Holman, R., Howd, P. and A. Sallenger. 2006. Empirical Parameterization of Setup, Swash, and Runup. Coastal Engineering 53(7): 573-588.

Strickler, A. 1923. Beitrage zur frage der geschwindigkeitsformel und der rauhigkeitszahlen fuer stroeme kanaele und gescholossene leitungen. Mitteilungen des eidgenossischen Amtes fuer Wasserwirtschaft 16. Bern, Switzerland. (in German).

USGS. 1984. Guide for selecting Manning's roughness coefficients for natural channels and flood plains. U.S. Geological Survey Water Supply Paper 2339.

11. LIST OF PREPARERS

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12. FIGURES

Source: Sonoma County Orthophotography (April-May, 2000)

figure 1

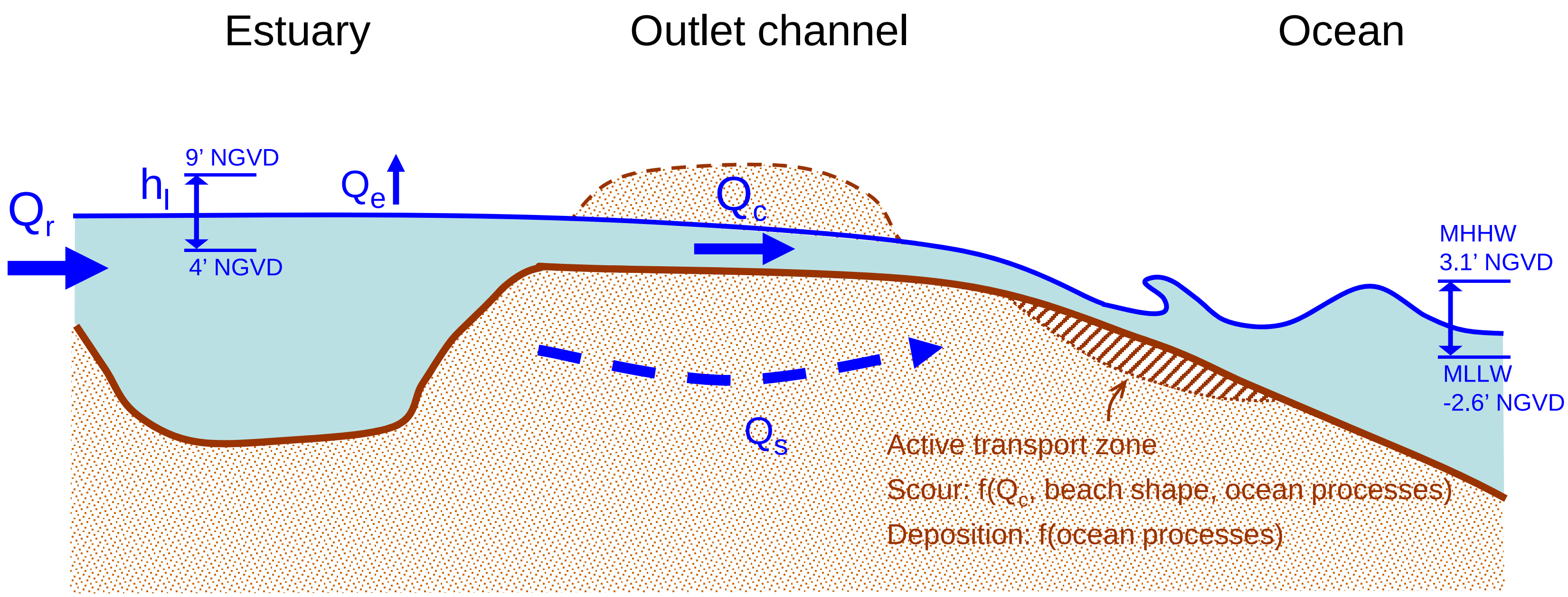
Russian River Estuary Outlet Channel Management Plan

Russian River Estuary Site Location

PWA Ref# - 1958.02

PWA

G:\1958.01_Russian_River\Projects



<u>Parameters</u>

h_l=lagoon water level

Q_r=river discharge

Q_c=outlet channel discharge

Q_s=seepage discharge

Q_e=evaporation from lagoon

<u>Processes</u>

- $Q_r = Q_c + Q_e + Q_s$ (averaged over days)
- No sediment transport within outlet channel
- Active sediment transport outside outlet channel

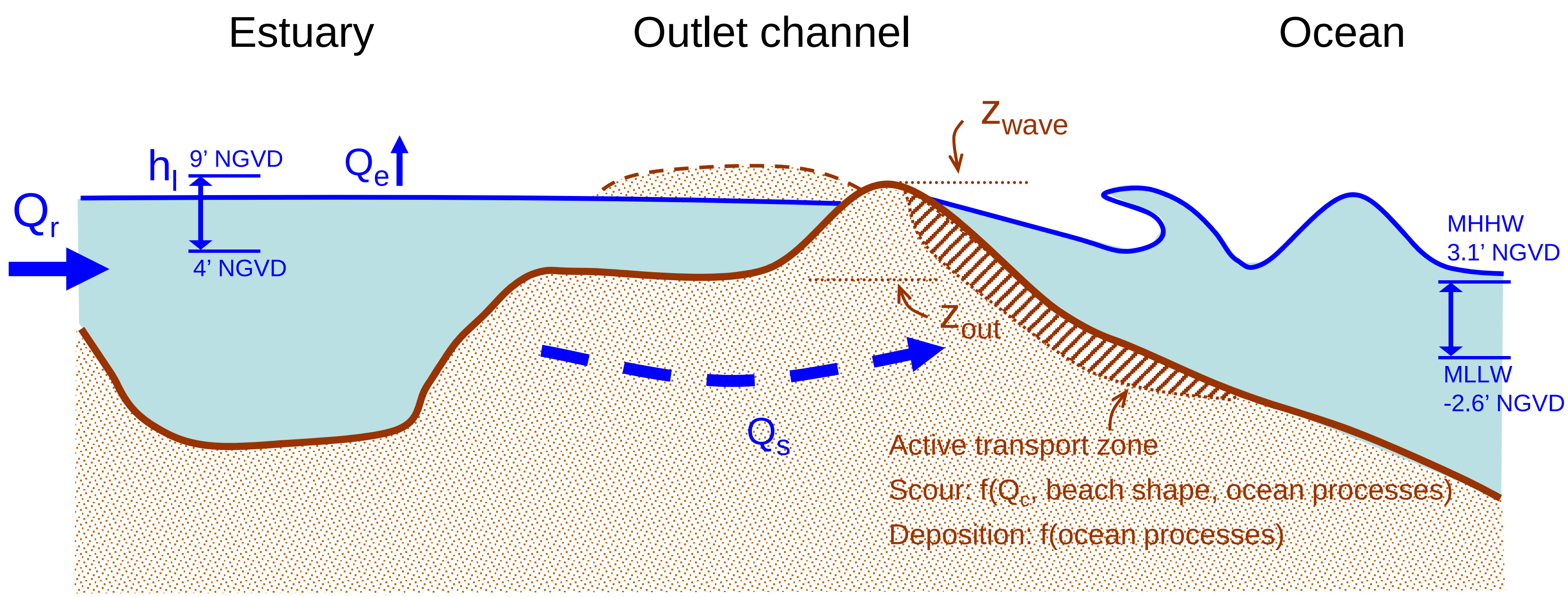
figure **2**

Russian River Estuary Outlet Channel Management Plan

Conceptual model – Target conditions

PWA Ref# 1958.01

PWA



<u>Parameters</u>

z_{out}=target channel bed elevation

z_{wave}=wave runup elevation; f(wave conditions, ocean water level, channel location)

<u>Processes</u>

- $z_{wave} \geq z_{out}$
- wave-induced sediment transport closes outlet channel
- $Q_c \rightarrow 0$
- h_l increasing

figure **3**

Russian River Estuary Outlet Channel Management Plan

Conceptual model – Closure

PWA Ref# 1958.01

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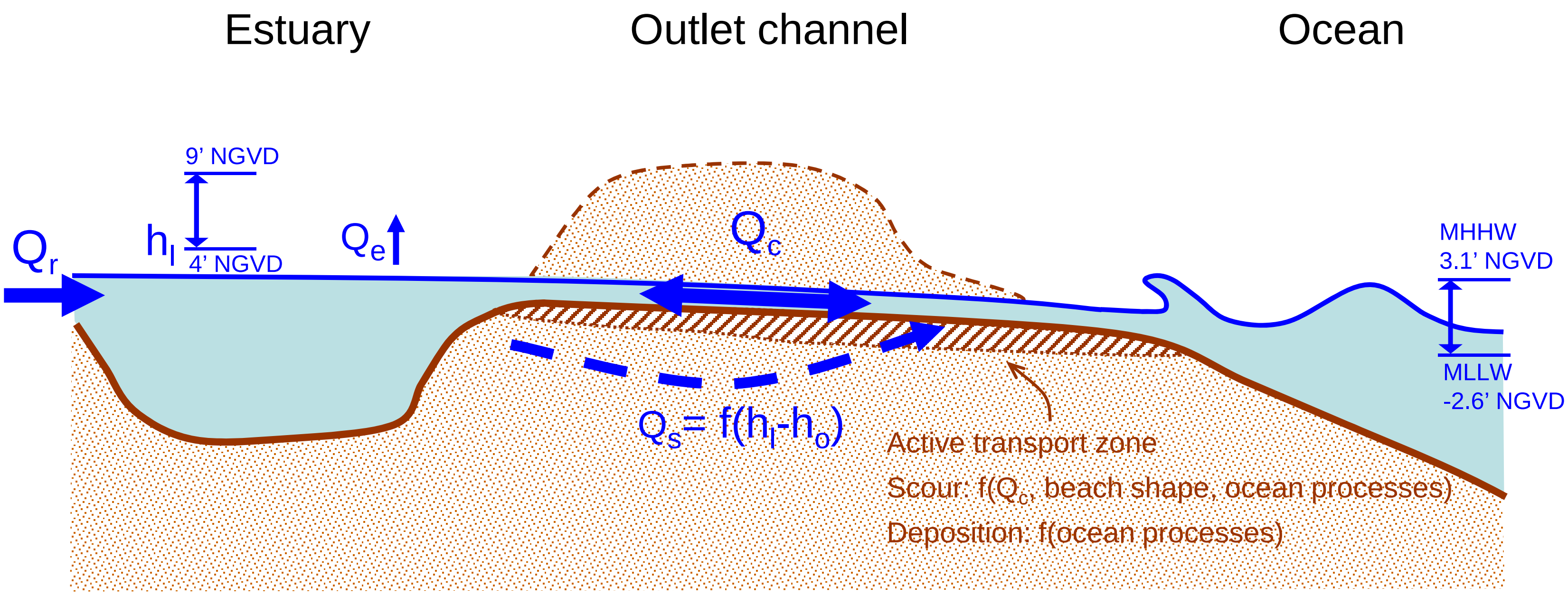

Parameters

u_c= f(channel slope, length, and width; Q_r; ocean water level)
(can be managed to greater or lesser degree)
u_{crit} is f(grain size)

Processes

•$u_c > u_{crit}$; high velocities scour channel
•Q_s increases; high seepage creates groundwater piping and erosion
•sediment transport within outlet channel

figure **4**
Russian River Estuary Outlet Channel Management Plan

Conceptual model – Breaching

PWA Ref# 1958.01

PWA

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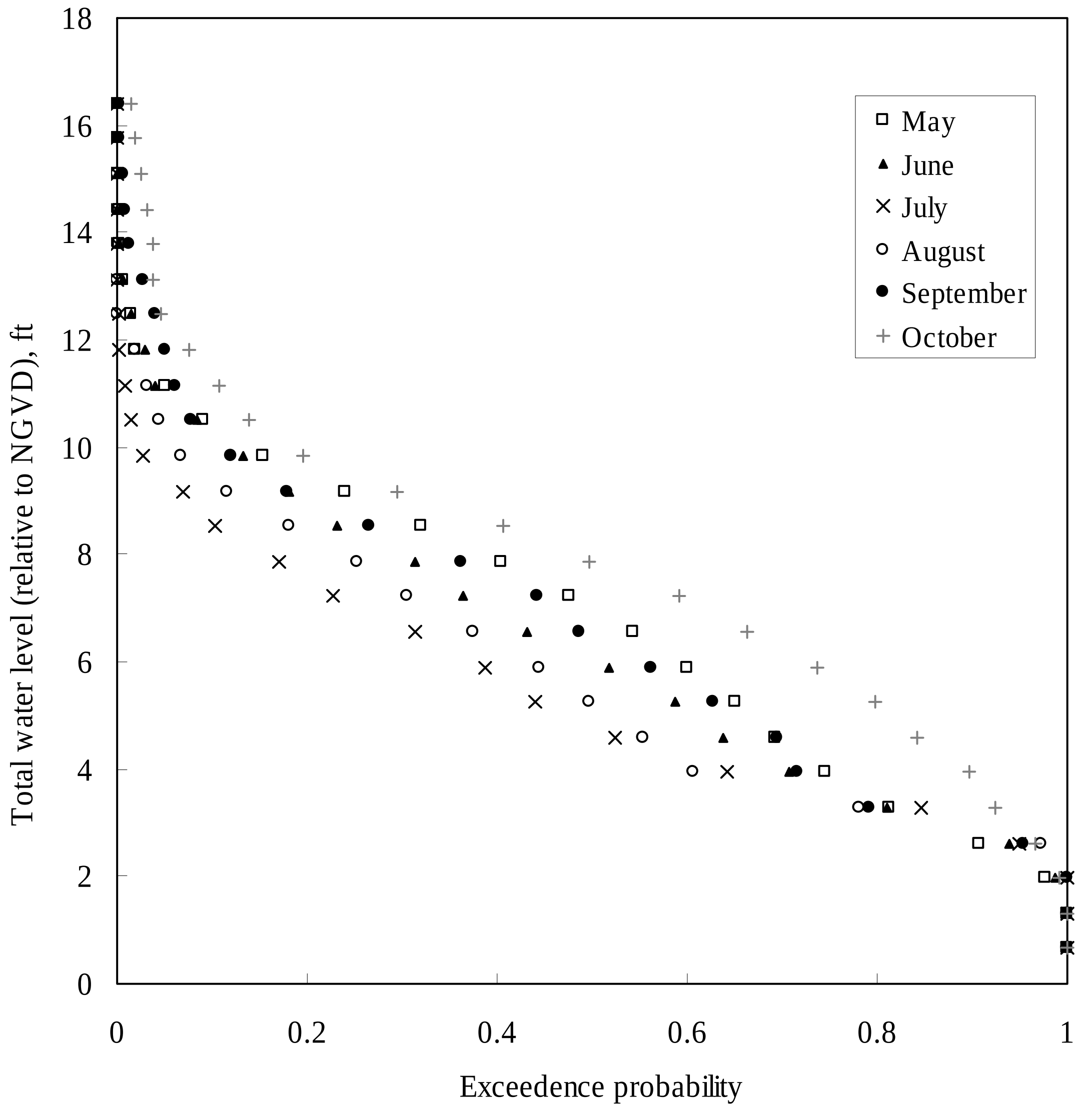


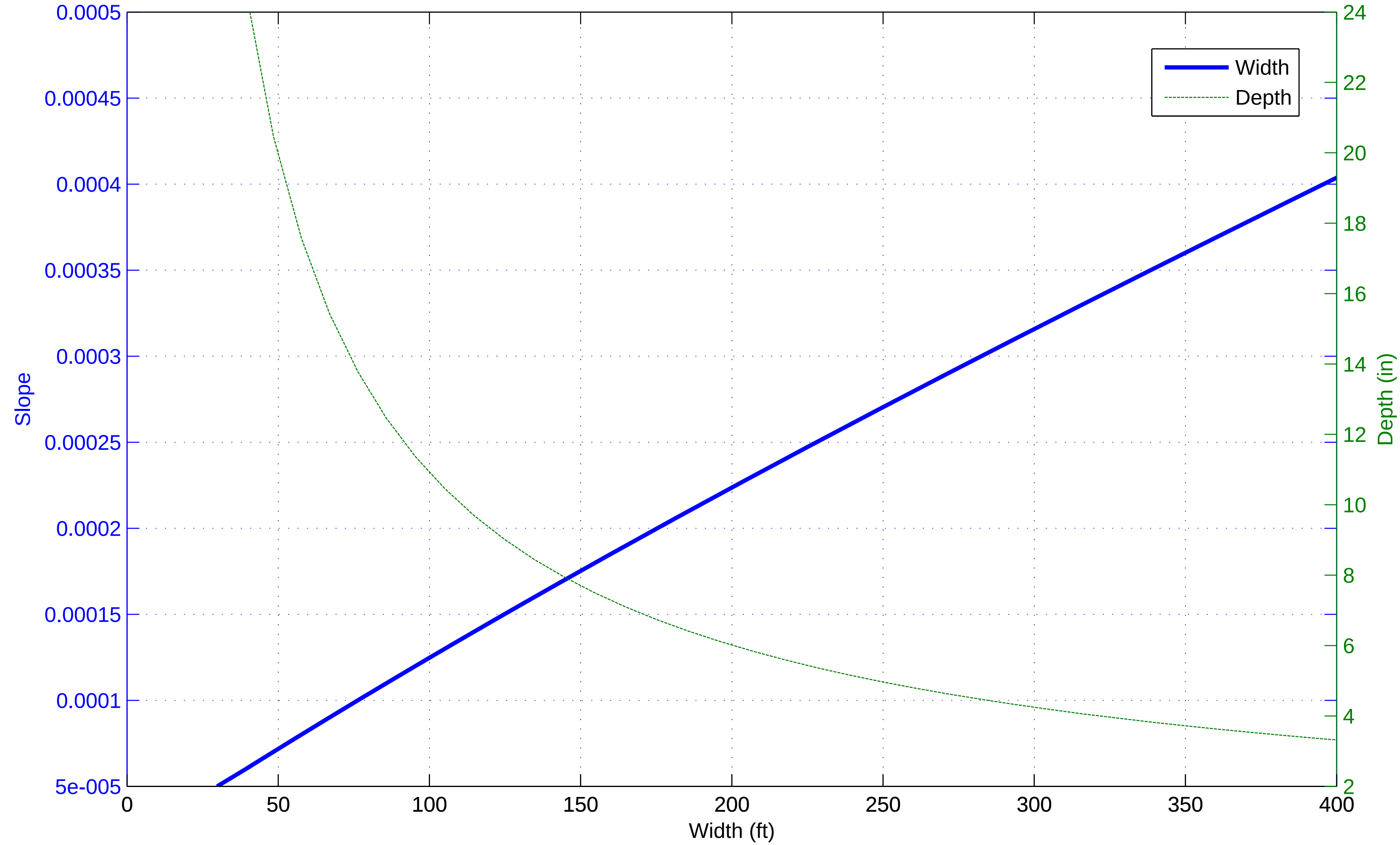
Source: D. Behrens (unpublished). Wave data from CDIP Point Reyes buoy.
Note: Total water level calculated as sum of daily higher high tide and wave runup elevation. Wave runup calculated from Stockdon et al (2006) using estimated de-shoaled deepwater equivalent wave heights.

figure **5**
Russian River Outlet Channel Management Plan

Total Water Level Exceedance, May-Oct

PWA Ref# 1958.01

PWA



Source: Stability curve for local bed shear stress of 0.5 Pa, flowrate of 70 cfs, and Manning's roughness of 0.02.

Figure 6

Russian River Estuary Outlet Channel Management Plan

Slope vs. Width Stability Plot

PWA Ref# 1958.01

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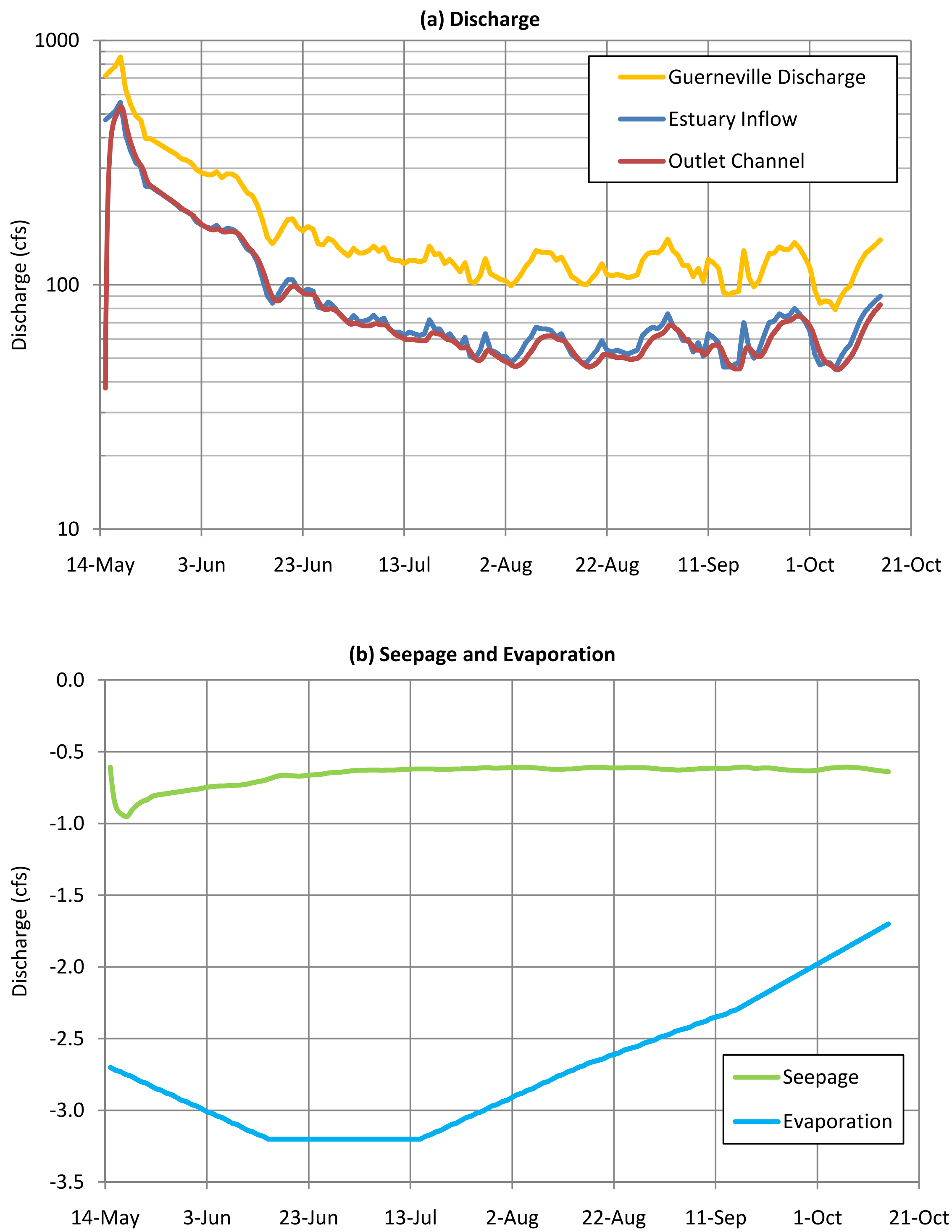


Source: 2010 anticipated discharge at Guerneville and into lagoon calculated by scaling observed 2000 discharge at USGS gage #11467000 (Russian River near Guerneville, CA). Evaporation rates calculated from monthly climatological rates for CIMIS evapotranspiration zone 1 (California coast).

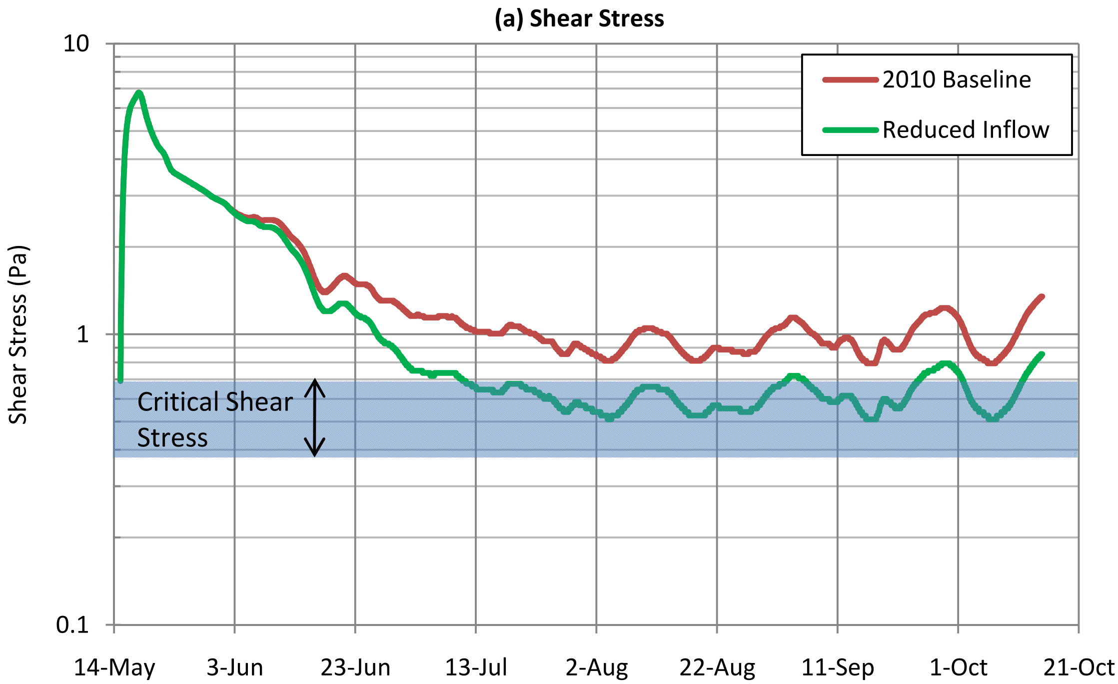
figure 7

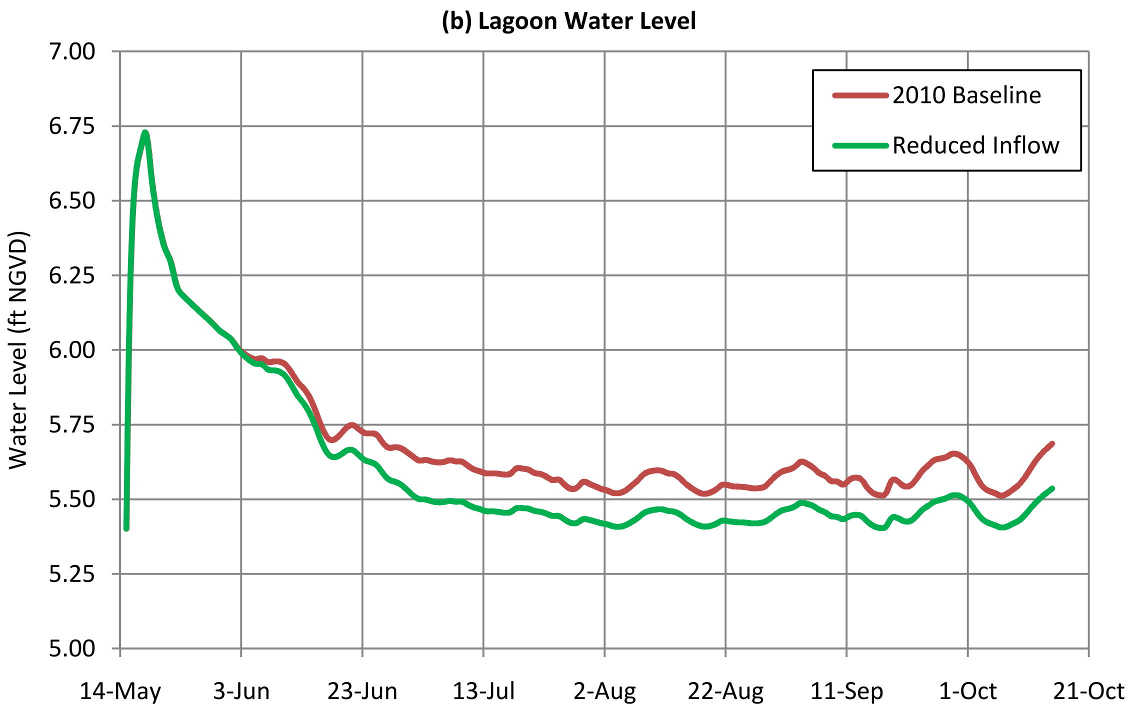
Russian River Estuary Outlet Channel Management Plan

Hydraulic Model Discharge - 2010 Anticipated Hydrology

PWA Ref#: 1958.01

PWA





Notes: Baseline channel geometry: width=100 ft, length=300 ft, bed=5 ft NGVD, n = 0.02.
Source: HEC-RAS hydraulic model results for outlet channel.

figure **8**

Russian River Estuary Outlet Channel Management Plan

Hydraulic Model Results - 2010 Anticipated Hydrology

PWA Ref#: 1958.01

PWA

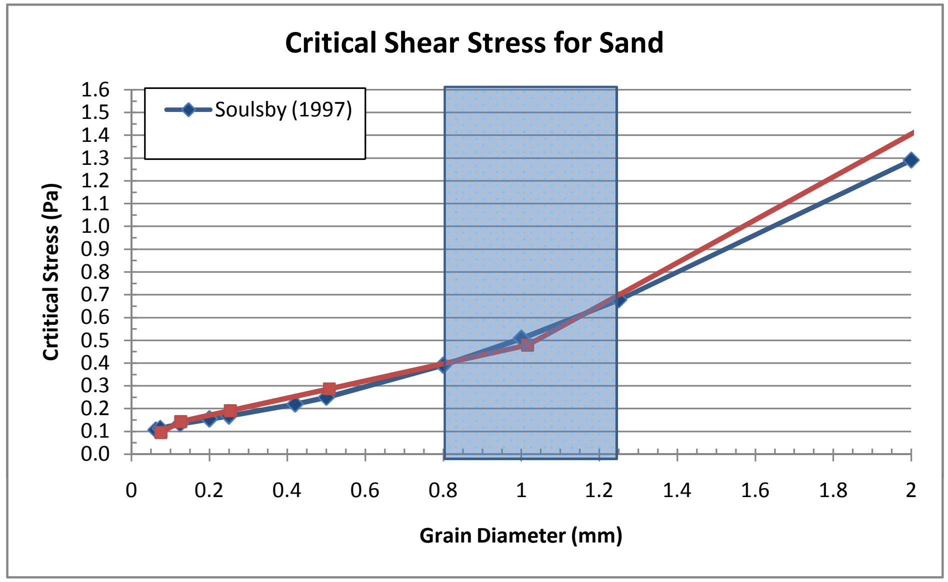
1958.01_2010_Rev_Anticipated_Operations.xls

A-1. Critical shear stress for incipient motion of sand particles

1958.01 Russian River Estuary Outlet Channel
J. Vandever (PWA)
4/1/2009

Variables		
p	1000	kg/m^3
g	9.81	m/s^2
s	2.65	(quartz)
v	1.0E-06	m^2/s

D (mm)	D*	Theta_crit	tau_crit (Pa)	Grain Size
0.0625	1.58	0.105	0.11	Very Fine Sand
0.074	1.87	0.094	0.11	
0.125	3.16	0.066	0.13	Fine Sand
0.20	5.06	0.048	0.15	
0.25	6.32	0.041	0.17	Medium Sand
0.42	10.62	0.032	0.22	
0.5	12.65	0.031	0.25	Coarse Sand
0.8	20.24	0.030	0.39	
1.0	25.30	0.031	0.51	Very Coarse Sand
1.25	31.62	0.033	0.68	
2.0	50.59	0.040	1.29	Granular



Notes: units Pa = N/m^2, assumes density of freshwater, quartz grained sand
Method based on Soulsby (1997) Dynamics of Marine Sand:

Note: does not account for gravitational effects on sloping bed

$$D_* = \left[\frac{g(s-1)}{\nu^2}\right]^{1/3} D$$

$$\theta_c = \frac{0.3}{1+1.2D_*} + 0.055\left[1-\exp\left(-0.020D_*\right)\right]$$

$$\tau_c = \rho(s-1)gd\theta_c$$

A-2. Manning's n worksheet

1958.01 Russian River Estuary Outlet Channel
J. Vandever (PWA)
4/1/2009

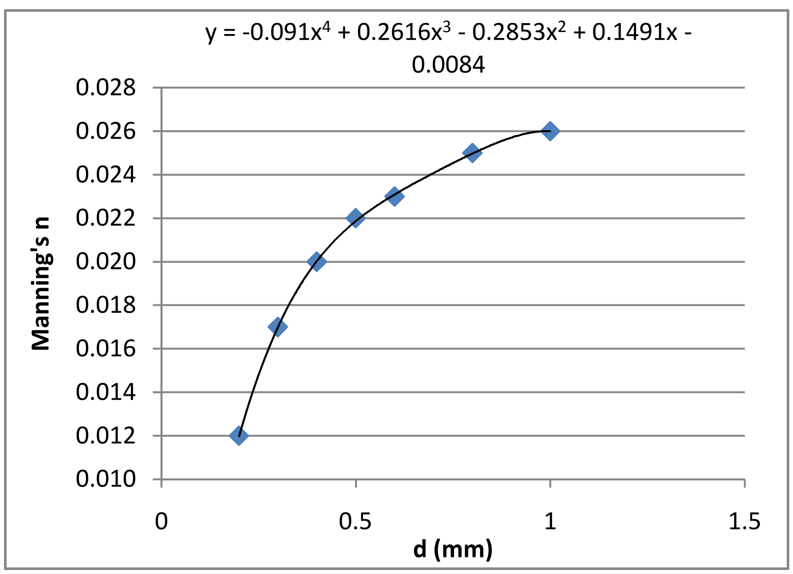
d_{50}	1	mm	0.003281	ft
D	0.84	ft		
Rh	0.83	ft		
S	0.00008	ft/ft		

Equation	n	Notes
Strickler (1923)*	0.018	*valid d range unknown
Limerinos (1970)*	0.021	
Bray (1979)*	0.017	
Bruschin (1985)*	0.018	
Julien (2002)*	0.024	
USGS (WSP2339)	0.026	for 0.2<d<1.0 mm
Average	**0.021**	
Average w/o USGS	**0.020**	

USGS

d (mm)	n
0.2	0.012
0.3	0.017
0.4	0.020
0.5	0.022
0.6	0.023
0.8	0.025
1.0	0.026
2.0	0.035

Polynomial fit to USGS data (d=2.0 mm not included):



A-3. Evaporation Worksheet

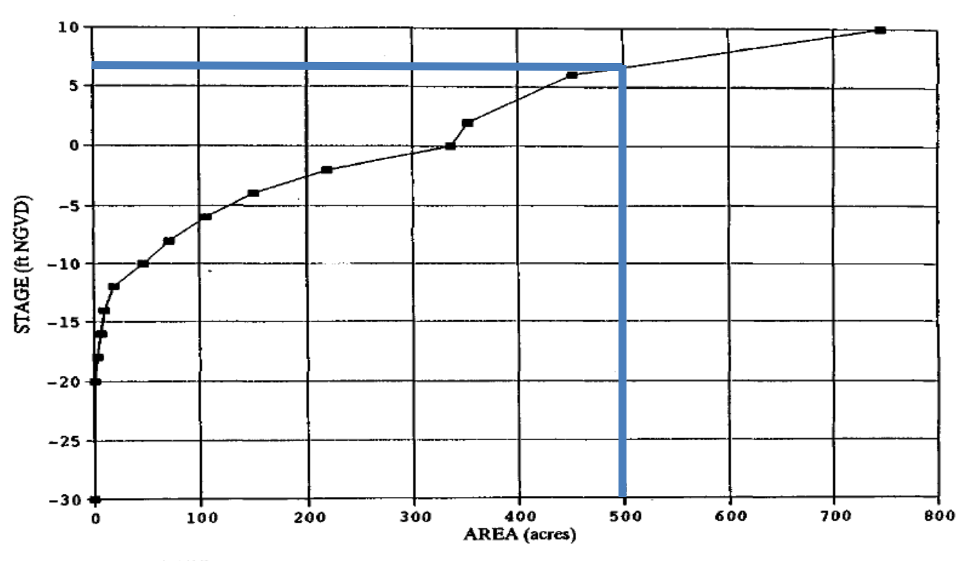
1958.01 Russian River Estuary Outlet Canal
J. Vandever (PWA)
15-Apr-09

CIMIS Reference Evapotranspiration (Eto) Zones
http://www.cimis.water.ca.gov/cimis/images/etomap.jpg

Russian River Estuary is located on California coast in Zone 1
(Coastal plains and heavy fog. Lowest Eto in California, characterized by dense fog)

	in/month	days	in/day	mm/day	cfs
Jan	0.93	31	0.03	0.76	0.6
Feb	1.40	28	0.05	1.27	1.1
Mar	2.48	31	0.08	2.03	1.7
Apr	3.30	30	0.11	2.79	2.3
May	4.03	31	0.13	3.30	2.7
Jun	4.50	30	0.15	3.81	3.2
Jul	4.65	31	0.15	3.81	3.2
Aug	4.03	31	0.13	3.30	2.7
Sep	3.30	30	0.11	2.79	2.3
Oct	2.48	31	0.08	2.03	1.7
Nov	1.20	30	0.04	1.02	0.8
Dec	0.62	31	0.02	0.51	0.4

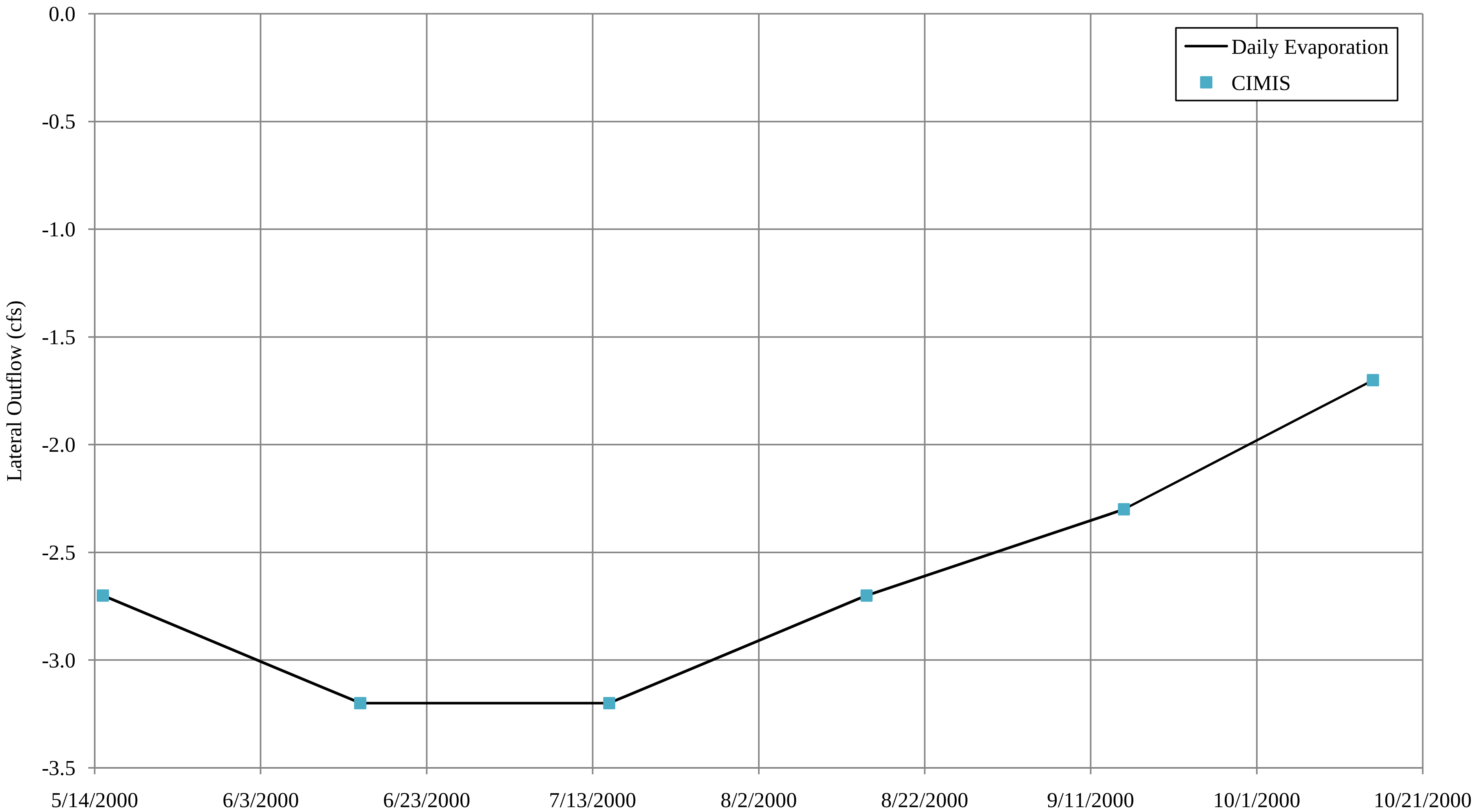
RRE Surface Area	500	acres
	21,780,000	sq ft



Philip Williams & Associates, Ltd.
Consultants in Hydrology

Stage-Area Curve for Russian River Estuary - River Mouth to Monte Rio

Figure 5.6



Notes: Daily evaporation rates for Russian River lagoon interpolated from CIMIS average monthly evapotranspiration statistics for Zone 1 (Coastal plains and heavy fog). Calculations assume lagoon surface area of 500 acres.

Appendix ***A-3***

Russian River Estuary Outlet Channel Management Plan

HEC-RAS model evaporation boundary condition

PWA Ref #: 1958.01

PWA

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A-4. Berm Seepage and Hydraulic Conductivity

1958.01 Russian River Estuary Outlet Canal
J. Vandever (PWA)
16-Apr-09

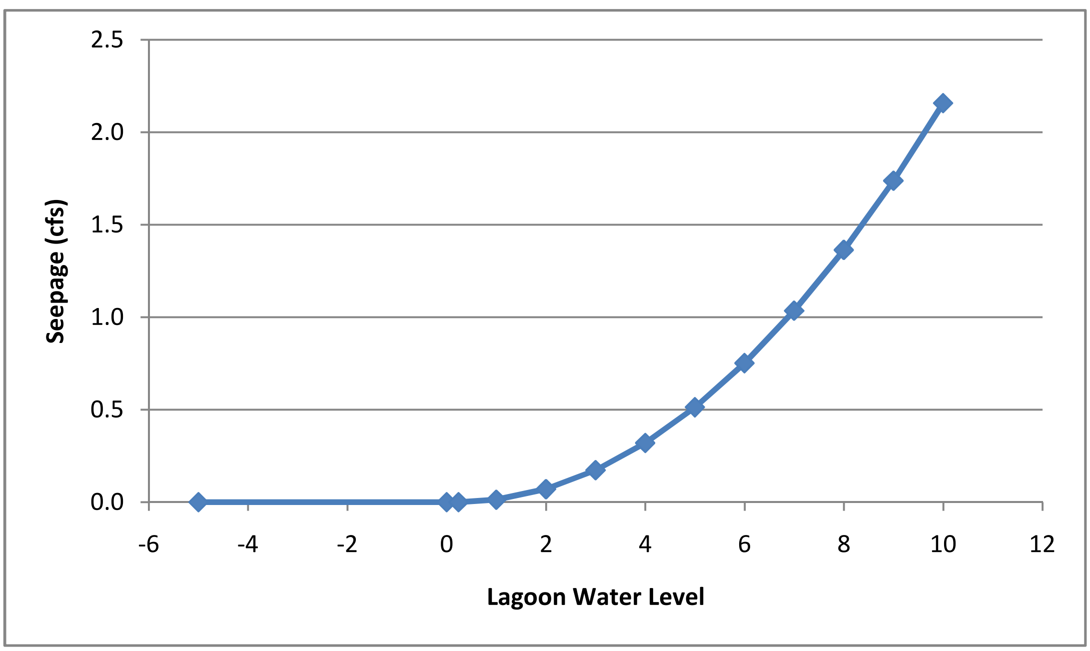
HEC-RAS Diversion Rating Curve

Lagoon WL (ft)	dh (ft)	q (cfs)	
-5	0	0.00	
0	0	0.00	
0.24	0	0.00	(MTL)
1	0.76	0.01	
2	1.76	0.07	
3	2.76	0.17	
4	3.76	0.32	
5	4.76	0.51	
6	5.76	0.75	
7	6.76	1.03	
8	7.76	1.36	
9	8.76	1.74	
10	9.76	2.16	(Flood Stage)
11	10.76	2.62	
12	11.76	3.13	

Darcy's Law

$$\boxed{q = k\frac{\Delta h}{W}A = k\frac{\Delta h}{W}(\Delta h \cdot L)}$$

W	250	ft
L	2500	ft
z_ocean	0.24	ft NGVD (MTL)
k	0.0023	ft/s



A-4. Berm Seepage and Hydraulic Conductivity

1958.01 Russian River Estuary Outlet Canal
J. Vandever (PWA)
7-Apr-09

Bouwer, H. 1978. Groundwater Hydrology. McGraw-Hill, Inc. 480 p.

	Hydraulic Conductivity (m/day)		Hydraulic Conductivity (cm/s)		
	Low	**High**	**Low**	**High**	**Mid**
Fine Sand	1	5	0.001	0.006	0.003
Medium Sand	5	20	0.006	0.023	0.014
Coarse Sand	20	100	0.023	0.116	0.069
Gravel	100	1000	0.116	1.157	0.637
Sand and Gravel	5	100	0.006	0.116	0.061

A-5. Mouth Closure Calibration Worksheet

1958.01 Russian River Estuary Outlet Canal
J. Vandever (PWA)
17-Apr-09

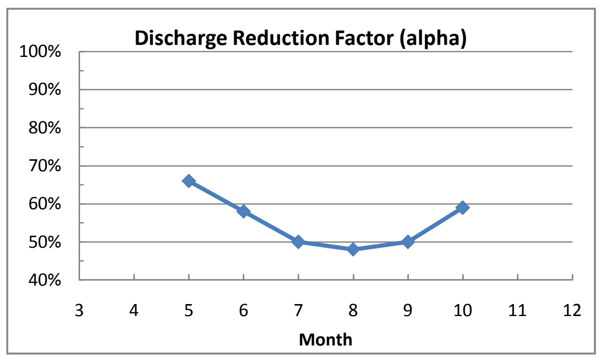
Russian River mouth closure calibrations - HEC-RAS model Years Examined: 2000, 2001, 2003, 2004, 2005, 2007
Accounts for losses between Hacienda Bridge (Guerneville, CA) and the lagoon and the interaction with the aquifer adjacent to the estuary.
No detailed information available for the aquifer groundwater elevations or extraction rates by wells. The loss term is a calibrated variable in the model.

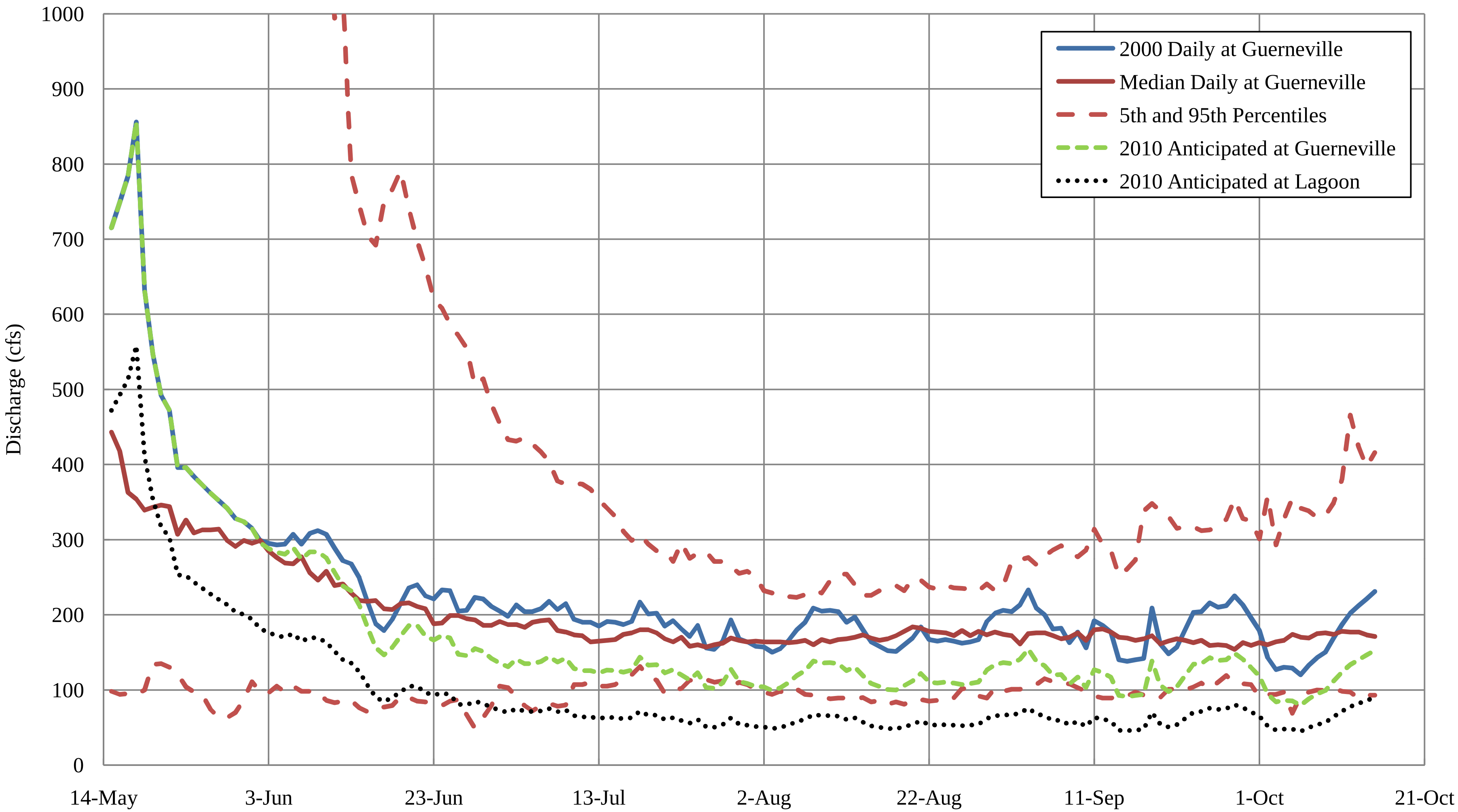
Lagoon Surface Area	400	ac
	17,424,000	sq ft
Evaporation and Seepage Losses	4	cfs

Calibration	Date		Water Level (ft NGVD)		dh	dt	dh/dt (ft/day)	dV/dt (cfs)	USGS Discharge (cfs)	% Reduction	alpha
Closure Event ID	Start	End	Start	End							
06May2000	5/6/2000 18:00	5/9/2000 6:00	3.10	8.40	5.30	2.50	2.12	432	580	26%	74%
24May2000	5/24/2000 8:00	5/25/2000 18:00	3.84	5.76	1.92	1.42	1.36	278	385	28%	72%
16June2000	6/16/2000 13:00	6/21/2000 6:00	4.79	6.90	2.11	4.71	0.45	94	200	53%	47%
25Aug2000	8/25/2000 0:00	9/5/2000 8:00	2.56	7.62	5.06	11.33	0.45	94	195	52%	48%
03Oct2000	10/3/2000 0:00	10/11/2000 12:00	2.85	6.53	3.68	8.50	0.43	91	140	35%	65%
15May2001	5/15/2001 23:00	5/21/2001 21:00	2.14	5.51	3.37	5.92	0.57	119	200	41%	59%
07Apr2007	4/7/2007 13:00	4/11/2007 0:00	1.17	7.68	6.51	3.46	1.88	384	480	20%	80%
13Apr2007	4/13/2007 21:30	4/17/2007 14:30	1.97	7.68	5.71	3.71	1.54	315	465	32%	68%
24Apr2007	4/24/2007 17:00	4/26/2007 14:00	1.51	7.57	6.06	1.88	3.23	656	725	10%	90%
13Oct2007	10/13/2007 2:30	10/22/2007 11:30	2.51	9.15	6.64	9.38	0.71	147	255	42%	58%
9June2003	6/9/2003 17:30	6/12/2003 1:00	2.77	6.47	3.70	2.31	1.60	322	475	32%	68%
9Oct2003	10/9/2003 23:11	10/14/2003 20:40	4.00	6.21	2.21	4.90	0.45	91	170	46%	54%
05Nov2004	11/5/2004 11:00	11/12/2004 4:00	2.40	8.93	6.53	6.71	0.97	196	300	35%	65%
26July2004	7/26/2004 15:41	8/5/2004 0:00	2.27	5.90	3.63	9.35	0.39	78	140	44%	56%
2May2004	5/2/2004 15:40	5/6/2004 19:35	3.44	8.39	4.95	4.16	1.19	240	420	43%	57%
16Apr2004	4/16/2004 9:09	4/18/2004 7:40	4.78	7.98	3.20	1.94	1.65	333	570	42%	58%
3Oct2005	10/3/2005 23:00	10/17/2005 6:30	2.40	8.30	5.90	13.31	0.44	89	170	47%	53%
17Sep2005	9/17/2005 2:00	9/21/2005 13:30	3.37	5.69	2.31	4.48	0.52	104	175	40%	60%

Note: Start and end times represent times used for water level calibration and do not correspond to exact timing of closures and breaches.

Month	Month	% Loss	N	HEC-RAS Multiplier
April	4	26%	4	
May	5	34%	4	66%
June	6	42%	2	58%
July	7	44%	1	50%
Aug	8	52%	1	48%
Sep	9	40%	1	50%
Oct	10	43%	4	59%
Nov	11	35%	1	
			18	





Notes: Median daily discharge calculated from 1970-2008.
Source: USGS gage 11467000 (Russian River near Guerneville, CA). 2010 anticipated discharge at Guerneville calculated from 2000 discharge by scaling factor to obtain typical summertime flowrates of 120 cfs. 2010 anticipated lagoon inflow calculated based on calibrated seasonal losses from Guerneville to lagoon.

Appendix A-6

Russian River Estuary Outlet Channel Management Plan

Daily Russian River Discharge

PWA Ref #: 1958.01

PWA

J:\1958.01RREAMPOutletChannel\Task 5 Hydrologic modeling\Data\Streamflow\USGS_11467000_WaterYear_2000.xls

Attachment B. Hypothetical Implementation Scenario

The following hypothetical implementation scenario is presented to demonstrate how the outlet channel management plan may be implemented. The scenario is based on actual beach berm and ocean conditions observed at the estuary from June 30 to July 6, 2009.

This scenario is purely hypothetical and demonstrates how the adaptive management plan may be implemented based on historical conditions observed in 2009. Actual implementation of the plan may vary in terms of channel geometry, channel location and time required for implementation. The beach environment at the project site is highly dynamic so actual implementation of the plan will be evaluated on a case-by-case basis.

Wednesday, June 30th

Agency personnel have been tracking riverine and ocean conditions on a daily basis during the outlet channel management period. Several days ago, they identified a forecasted ocean swell event with the potential to close the estuary. When it arrives, this medium-sized (2-4 ft.) ocean swell, angled from the southwest, pushes sand into the tidal inlet cutting flow from the estuary to the ocean. Stage in the estuary at the time of closure is approximately 3.5 ft NGVD. Based on river discharge and the time of year, Agency personnel estimate that the estuary water level's rate of rise will be 0.5 ft/day.

Thursday, July 1st

Agency personnel visit the site to assess sandbar conditions. The outlet at the time of closure is just south of Haystack Rock, approximately 550 ft northwest of the jetty, with an alignment roughly perpendicular to the beach face. The preexisting channel slope is steep and would, therefore, be susceptible to scour and wave run-up. Agency decides that this is not the preferable alignment for the outlet channel. In effort to create a channel which has shallower gradient and less susceptible to ocean conditions, it is decided that the channel will be more ideally located to the north of Haystack Rock angled to the northwest. Agency staff collects measurements and limited survey data (e.g. elevation at low point of the berm) in the area to develop a design for the outlet channel.

[Note: If closure had occurred during the pupping season (March 15 – June 30), the site assessment would have included a survey for the presence of seal pups.]

Agency staff returns to their offices to develop a plan and design for the implementation of the outlet channel. Changes between the most recent monthly topographic data and current conditions are assessed using the time-lapse photography and today's survey data. If indicated, today's survey data and judgment may be used to revise the topographic data.

Stage in the estuary is now approximately 4.0 ft. NGVD. Observations from the Jenner gage are used to confirm the previously estimated rate of water surface rise of 0.5 ft/day. Based on current stage and this rate of water surface rise, implementation of the outlet channel is scheduled for Monday and Tuesday, July 5th and 6th so that stage in the estuary will be approximately 6.5 ft. NGVD after the outlet channel is completed.

A design is prepared using the best available topographic data. The outlet channel will be approximately 30 ft wide with 4:1 side slopes, 350 ft long to the mean high tide line, a channel bottom elevation at the inlet of approximately 6 ft NGVD, and a channel design flow depth at time implementation of approximately 0.5 ft. Channel will be aligned to the northwest with an approximate aspect of 35° with respect to the beach face. Estimated material to be excavated is approximated and confirmed to be less than 1,000 yd^3.

Agency staff prepares e-mail to management team to notify them of intention and schedule to construct the outlet channel, provide information regarding current conditions, and provide team with a design schematic according to the Communication Protocol procedure documented in Section 7.8.1 of the management plan. Please see Attachments B.1 and B.2 for an example of e-mail transmittal with attached design schematic. Agency biologists coordinate with Stewards of the Coast and Redwoods to schedule volunteers to assist with pre-, day of, and day after outlet channel creation pinniped monitoring.

Friday, July 2nd

Agency staff receives comments from management team on proposed approach. Time allowing, Agency responds, modifies the proposed approach as needed, and decides on the final approach.

Agency staff reviews rate of water surface rise in the lagoon to confirm that flooding is not expected before proposed management action.

Monday and Tuesday, July 5th and 6th

Agency maintenance crews arrive at the Goat Rock State Beach parking lot early in the morning to prepare for implementation. Agency biologist arrives to begin pinniped monitoring at least one hour prior to crews and coordinates with maintenance crew leader. Agency surveyors stake out designed channel and make corrections to alignment and channel geometry to account for potential changes in beach berm topography since last topographic survey. Outlet channel excavation is carried out according to Section 7.5 of the management plan and according to the plan submitted to the management team. Implementation is also conducted in accordance with the Agency's IHA for harbor seals, northern elephant seals and California sea lions which may be present at the site during excavation activities. Photos are taken to document all implementation activities, and following completion of the outlet channel Agency staff collects measurements of completed channel geometry, flow depth and location.

Wednesday, July 7th

Agency staff sends e-mail to management team to provide documentation of the completion of the outlet channel according to the Communication Protocol procedure documented in Section 7.8.2 of the management plan. Please see Attachment B.3 for an example of e-mail transmittal.

After implementation of the channel, the Agency will monitor performance of the outlet channel according to the monitoring program described in Section 7.7 of the management plan.

Attachment B.1: Sample Proposed Outlet Channel Implementation Email

Date: 7/1/10

Hello Outlet Channel Management Team -

The Russian River Estuary closed on 6/30/10. The Sonoma County Water Agency plans to implement an outlet channel beginning at 7 am on July 5th and potentially extending to the afternoon of July 6th. Details of the proposed outlet channel are the following:

- Channel Width: 30 ft.
- Channel Length: 350 ft.
- Channel Bottom Elevation: 6 ft NGVD
- Design Flow Depth: 0.5 ft
- Location of Channel Inlet Centerline: 970 ft northwest of jetty
- Channel Alignment Aspect: 35 deg. with respect to beach face
- Estimated Estuary WSEL at Time of Completion: 6.5 ft
- Existing Beach Berm Crest Elevation: 10 ft NGVD
- Existing Beach Berm Width: 300 ft
- Excavation Equipment: 1 Excavator, 1 Bulldozer

Attached is a design drawing developed using the most recent topographical survey (6/30/10). Due to the highly dynamic nature of conditions at the site, actual topography at the time of implementation may vary. Implementation of the channel may differ from design in order to account for changed topography.

Current and predicted conditions at the site are the following:

- **River and Estuary:**
  - Russian River near Guerneville Flow (USGS 11467000): 120 cfs
  - Predicted 72 hour precipitation: 0 in.
- **Ocean:**
  - Approximate rate of estuary water surface rise: 0.5 ft/day
  - Current Swell Height and Direction: 5.8 ft @ 10 sec. @ 320 deg.
  - 7/5/10 Predicted Mean Swell Height and Direction: 2.5 ft @ 15 sec. @ 200 deg.

No seal pups were observed on the beach.

For updates on conditions please visit the following URL:

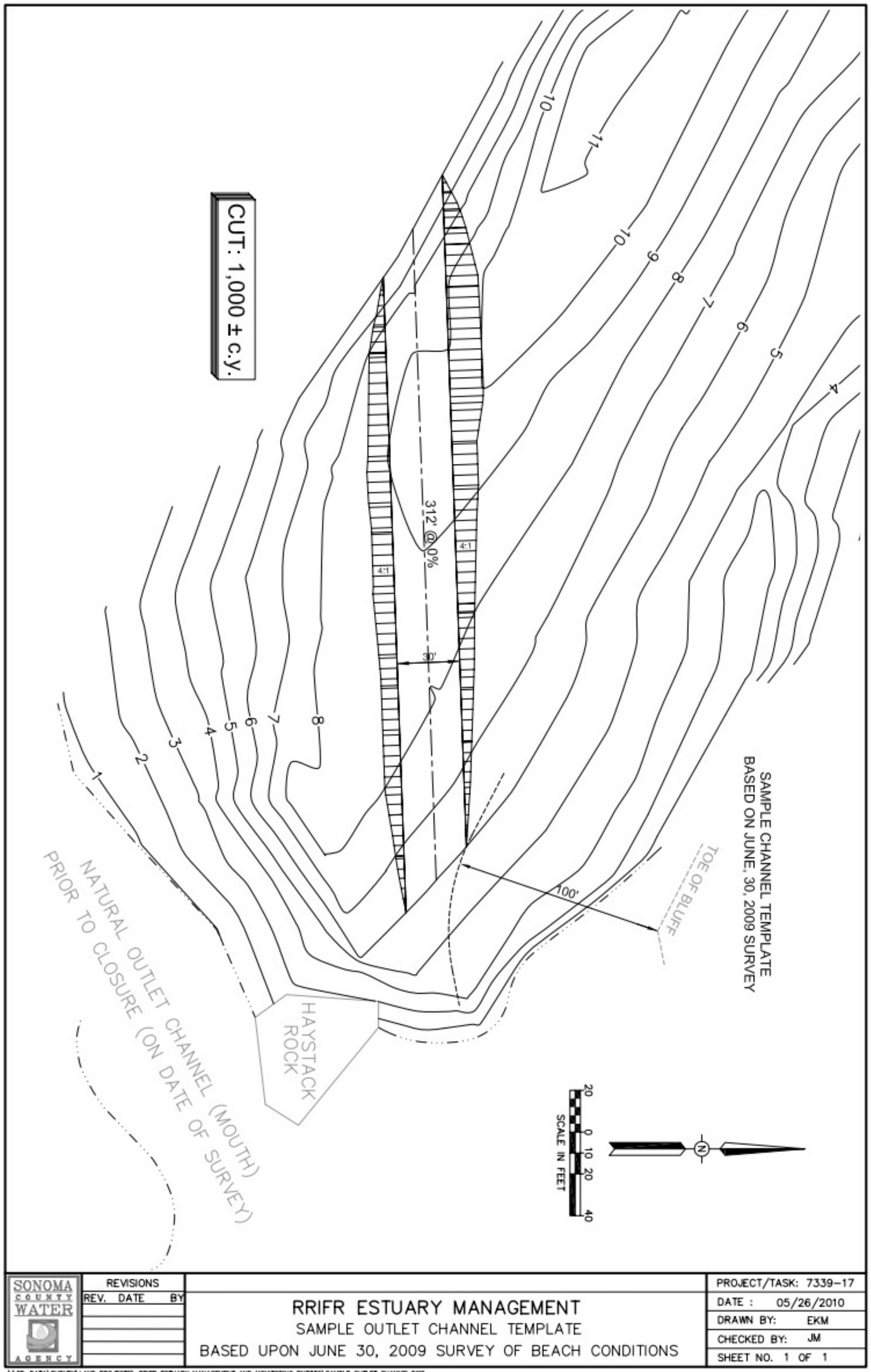
http://www.bml.ucdavis.edu/boon/russianriver

If you have any comments to the proposed implementation plan please provide comments no later than 7/2/10, 5 pm. Should you have any questions or concerns please contact me or Jessica Martini-Lamb at jessicam@scwa.ca.gov, 707-547-1903 (office), 707-322-8177 (mobile).

Sincerely,

Chris Delaney, P.E.
Agency Engineer
Sonoma County Water Agency
707-547-1946 (office)
707-975-5606 (mobile)

Attachment B.2: Sample Proposed Outlet Channel Design Schematic



\\SD-DATA\SURVEY\LAND PROJECTS\ RRIFR ESTUARY MANAGEMENT AND MONITORING PN7339\SAMPLE OUTLET CHANNEL.DWG

Attachment B.3: Sample Proposed Outlet Channel Implementation Email

Date: 7/8/10

Hello Outlet Channel Management Team -

The Russian River Estuary closed on 6/30/10. The Sonoma County Water Agency implemented an outlet channel beginning at 7 am on July 5th and extending to the afternoon of July 6th. Details of the implemented outlet channel are the following:

- Channel Width: 30 ft.
- Channel Length: 350 ft.
- Channel Bottom Elevation: 6 ft NGVD
- Flow Depth: 0.7 ft
- Location of Channel Inlet Centerline: 970 ft northwest of jetty
- Channel Alignment Aspect: 35 deg. with respect to beach face
- Estuary WSEL at Time of Completion: 6.7 ft
- Existing Beach Berm Crest Elevation: 10.2 ft NGVD
- Existing Beach Berm Width: 300 ft
- Excavation Equipment: 1 Excavator, 1 Bulldozer

Attached are photographs of the beach before, during, and after the outlet channel implementation.

Current and predicted conditions at the site are the following:

- **River and Estuary:**
    - Russian River near Guerneville Flow (USGS 11467000): 115 cfs
    - Predicted 72 hour precipitation: 0 in.
- **Ocean:**
    - Current Swell Height and Direction: 2.7 ft @ 14 sec. @ 200 deg.
    - 7/10/10 Predicted Mean Swell Height and Direction: 2.4 ft @ 12 sec. @ 200 deg.

No seal pups were observed on the beach.

For updates on conditions please visit the following URL:

http://www.bml.ucdavis.edu/boon/russianriver

If you have any comments on the implemented channel, please provide comments no later than 7/12/10, 5 pm. Should you have any questions or concerns please contact me or Jessica Martini-Lamb at jessicam@scwa.ca.gov, 707-547-1903 (office), 707-322-8177 (mobile).

Sincerely,

Chris Delaney, P.E.
Agency Engineer
Sonoma County Water Agency
707-547-1946 (office)
707-975-5606 (mobile)

Attachment C. Summary of Land Use Permits

Sonoma County Water Agency
Summary of Special Conditions of Permits for Russian River Estuary Management Activities

Agency / Permit / Expiration	Special Conditions
California Department of Fish and Game Lake and Streambed Alteration Agreement (III-1176-96) - November 6, 1996 Agreement Renewal – November 14, 2001 Agreement Extension – October 17, 2002 Agreement Renewal – November 13, 2003 Agreement Renewal – September 30, 2005 Agreement Extension – December 7, 2009 Agreement Amendment – December 13, 2009 Lake and Streambed Alteration Agreement (1600-2010-0380-R3) - September 8, 2011 Expiration - December 31, 2015	**1. Administrative Measures** Permittee shall meet each administrative requirement described below. 1.1 Documentation at Project Site. Permittee shall make the Agreement, any extensions and amendments to the Agreement, and all related notification materials and California Environmental Quality Act (CEQA) documents, readily available at the project site at all times and shall be presented to DFG personnel, or personnel from another state, federal, or local agency upon request. 1.2 Providing Agreement to Persons at Project Site. Permittee shall provide copies of the Agreement and any extensions and amendments to the Agreement to all persons who will be working on the project at the project site on behalf of Permittee, including but not limited to contractors, subcontractors, inspectors, and monitors. 1.3 Notification of Conflicting Provisions. Permittee shall notify DFG if Permittee determines or learns that a provision in the Agreement might conflict with a provision imposed on the project by another local, state, or federal agency. In that event, DFG shall contact Permittee to resolve any conflict. 1.4 Project Site Entry. Permittee agrees that DFG personnel may enter the project site at any time to verify compliance with the Agreement. 1.5 Work Period Extension. If the Permittee needs more time to complete the authorized activity, the work period may be extended on a day-to-day basis by contacting the DFG representative found within the Contact Information section of this Agreement.

Agency / Permit / Expiration	Special Conditions
California Department of Fish and Game (continued)	1.6 To the extent that any provisions of this Agreement provide for activities that require the Permittee to traverse another owner's property, such provisions are agreed to with the understanding that the Permittee possesses the legal right to so traverse. In the absence of such right, any such provision is void. 1.7 If, in the opinion of the DFG, conditions arise, or change, in such a manner as to be considered deleterious to the stream or wildlife, operations shall cease until corrective measures approved by the DFG are taken. **2. Avoidance and Minimization Measures** To avoid or minimize adverse impacts to fish and wildlife resources identified above, Permittee shall implement each measure listed below. 2.1 In each year that this Agreement is in effect, the Permittee shall provide DFG with an annual lagoon outlet channel adaptive management plan by April 15. 2.2 No excavation of the lagoon outlet channel may occur until DFG has reviewed and approved the annual lagoon outlet channel adaptive management plan. DFG shall provide written comments or approval by May 15 of each year this agreement is in effect. 2.3 The project site has been identified as an area that is potentially inhabited by steelhead trout (Federal Threatened), chinook salmon (Federal Threatened), coho salmon (Federal and State Endangered) and green sturgeon (Federal Threatened). This agreement does not authorize the take, or incidental take of any State or Federal listed threatened or endangered listed species. The Permittee is required, as prescribed in the state or federal endangered species acts, to consult with the appropriate agency prior to commencement of the project. Any unauthorized take of such listed species may result in prosecution. 2.4 To avoid impacts on aquatic and terrestrial species within the immediate work area, prior to implementation of an outlet channel, a qualified biologist will conduct a preconstruction survey to ensure no special-status species are occupying the site. If special-status species are observed within the project site or immediate surroundings, these areas will be avoided until the animal(s) has (have) vacated the area, and/or the animal(s) have been relocated out of the project area by a qualified biologist, upon approval by the regulatory agencies. In addition, the site will be surveyed

Agency / Permit / Expiration	Special Conditions
California Department of Fish and Game (continued)	periodically during construction to ensure that no special-status species are being impacted by construction activities. 2.5 The project biologist will conduct a preconstruction training session for construction crew members. The training will include a discussion of sensitive biological resources within the project area and the potential presence of special-status species, special-status species' habitats, protection measures to ensure species are not impacted by project activities and project boundaries. 2.6 Any material, which could be hazardous to aquatic life and enters a stream or lake (i.e., a piece of equipment tipping-over in a stream and dumping oil, fuel or hydraulic fluid), shall be removed immediately and the DFG shall be notified within 24 hours. 2.7 Any hazardous or toxic materials that could be deleterious to aquatic life that could be washed into State waters or its tributaries shall be contained in water tight container or removed from the project site. 2.8 The Permittee/contractor shall not dump any litter or construction debris within the riparian/stream zone. All such debris and waste shall be picked up daily and disposed of at an appropriate site. 2.9 Refueling of construction equipment and vehicles may not occur within 300 feet of any water body, or anywhere that spilled fuel could drain to a water body. Tarps or a similar material shall be placed underneath the construction equipment and vehicles, when refueling, to capture incidental spillage of fuels. 2.10 Any equipment or vehicles driven and/or operated within or adjacent to the stream/lake shall be checked and maintained daily to prevent leaks of materials that if introduced to water could be deleterious to aquatic life, wildlife, or riparian habitat. 2.11 Any equipment or vehicles driven and/or operated within or adjacent to the stream/lake shall be cleaned of all external oil, grease, and materials that, if introduced to water, could be deleterious to aquatic life, wildlife or riparian habitat.

Agency / Permit / Expiration	Special Conditions
California Department of Fish and Game (continued)	**3. Reporting Measures** Permittee shall meet each reporting requirement described below. 3.1 The Permittee shall notify DFG a minimum of 24 hours in advance of implementing the outlet channel management plan during the lagoon management period (May 15 to October 15). All communications shall be made in the method prescribed within the communication protocol section of the DFG approved annual lagoon outlet channel adaptive management plan. 3.2 The Permittee shall submit an annual report detailing that year's outlet channel management activities. This report may be submitted as a section of the annual lagoon outlet channel adaptive management plan required by May 1 of each year this agreement is in effect.

Sonoma County Water Agency
Summary of Special Conditions of Permits for Russian River Estuary Management Activities

Agency / Permit / Expiration	Special Conditions
California Regional Water Quality Control Board, North Coast Region Section 401 Water Certification (1B04001WNSO) - May 6, 2004 Amendment Extension – October 14, 2009 Amendment Extension – January 20, 2011 Amendment Extension – January 5, 2012 Amendment Extension – December 11, 2012 Expiration - December 31, 2013	Pursuant to 23 CCR3859(a), the applicant shall comply with the following additional conditions: 1. The Regional Water Board shall be notified in writing at least five working days (working days are Monday-Friday) prior to the commencement of grading work, with details regarding the construction schedule, in order to allow staff to be present on-site during construction, and to answer any public inquiries that may arise regarding the project. 2. When operations are completed, any excess material or debris shall be removed from the work area. No rubbish shall be deposited within 150 feet of the high water mark of any stream. 3. A copy of this permit must be provided to the Contractor and all subcontractors conducting the work, and must be in their possession at the work site. 4. If, at any time, a discharge to surface waters occurs, or any water quality problem arises, the project shall cease immediately and the Regional Water Board shall be notified promptly. The Regional Water Board will assess the extent of the problems and determine whether to rescind this Order. 5. The applicant shall submit an annual report, each year this Order is active, summarizing all water quality monitoring results and overall breaching activities to the Regional Water Board by December 31st. 6. This Order is not transferable. In the event of any change in control of ownership of land presently owned or controlled by the Applicant, the Applicant shall notify the successor-in-interest of the existence of this Order by letter and shall forward a copy of the letter to the Regional Water Board at the above address. To discharge dredged or fill material under this Order, the successor-in-interest must send to the Regional Water Board Executive Officer a written request for transfer of the Order. The request must contain the requesting entity's full legal name, the state of incorporation if a corporation, address and telephone number of the person(s) responsible for contact with the Regional Water Board. The request must also describe any changes to the Project proposed by the successor-in-interest or confirm that the successor-in-interest intends to implement the Project as described in this Order. 7. The Applicant shall provide photos documenting the work being conducted and the completed work, to the appropriate Regional Water Board staff person, in order to document compliance.

Sonoma County Water Agency
Summary of Special Conditions of Permits for Russian River Estuary Management Activities

Agency / Permit / Expiration	Special Conditions
California Coastal Commission Coastal Development Permit (2-01-033) – May 15, 2002 Amendment Extension (2-01-033-1A) – June 14, 2010 Monthly Extensions (January - June 2011) Expiration June 30, 2011 Emergency Coastal Development Permit (2-12-002-G) – issued January 9, 2012 and expired April 15, 2012 New Coastal Development Permit Application Submitted – January 23, 2012 Application deemed complete – July 9, 2012 (6 month application review process begins) Emergency Coastal Development Permit (2-13-005-G) – issued February 21, 2013	

Sonoma County Water Agency
Summary of Special Conditions of Permits for Russian River Estuary Management Activities

Agency / Permit / Expiration	Special Conditions
US Army Corps of Engineers, San Francisco District Section 404 & Section 10, Individual Permit (285610N) - July 22, 2005 Permit Modification - October 5, 2009 Time Extension January 5, 2011 Time Extension December 8, 2011 Time Extension December 10, 2012 Expiration - December 31, 2013	<u>Individual Permit Dated July 22, 2005</u> Special Conditions: To ensure compliance with this Department of the Army permit and to further minimize adverse impacts to water quality and aquatic-dependent biological resources, including federally listed threatened salmonid fish species, designated critical habitat, and designated essential fish habitat, the project is subject to the following Special Conditions: 1. To remain exempt from the prohibitions of Section 9 of the Endangered Species Act of 1973, as amended (16 U.S.C. § 1531 *et seq.),* SCWA shall fully implement the non- discretionary terms and conditions for incidental take of Central California Coast threatened coho salmon *(Oncorhynchus kisutch),* Central California Coast threatened steelhead *(Oncorhynchus mykiss),* and California Coastal threatened chinook salmon (*Oncorhynchus tshawytscha)* In the manner stipulated in the Biological and Conference pinion (Pages 33-35) entitled, "Clean Water Act Section 404 Permit for the Russian River Estuary Breaching Activities Conducted 2005-2010" (File No. 151422SWR04SR9206), issued by the National Marine Fisheries Service (NMFS), Southwest Region, on 20 May 2005 (Attachment 3). SCWA shall notify both NMFS and USACE bye-mail or by phone of any known violation of incidental take within twenty-four (24) hours of the occurrence. 2. SCWA shall provide USACE a copy of the approved Estuary Monitoring Plan and all subsequent Annual Monitoring Reports required by the Biological Opinion. 3. All breaching events shall occur only after the estuary water level reaches between 4.5 feet and 7.0 feet NGVD under current flow regimes, as measured by the stage gage at the Jenner Visitor Center. 4. To facilitate adequate inspection of work, SCWA shall notify USACE by e-mail or by phone of the proposed breaching date at least five (5) days prior to the commencement of work. 5. Unless otherwise approved, authorized discharges of dredged material on the sandbar below the high tide line shall consist only of the native sand excavated from the pilot channel. 6. To ensure public safety while minimizing disturbance of harbor seals and other marine mammals during each breaching event, SCWA shall implement a Beach Closure Plan that restricts public access to all areas within 750 feet of the breaching location for a period of 24 hours before and after completion of work. 7. SCWA shall provide USACE a Breaching Activities Report by 31 March for each year of the five-year permit authorization period. Each Breaching Activities Report shall present a tabulation of the breaching events that occurred during the preceding year, including the approximate estuary closure

Sonoma County Water Agency
Summary of Special Conditions of Permits for Russian River Estuary Management Activities

Agency / Permit / Expiration	Special Conditions
US Army Corps of Engineers, San Francisco District (continued)	date, the approximate number: of estuary closure days occurring before the breach event, the breaching event date, and the recorded estuary water level of the breaching event date. 8. The current Coastal Development Permit (CDP 2-01-033) issued by the California Coastal Commission expires on 31 December 2005. The current Section 401 water quality certification (WDID No. IB04001WNSO) issued by the Regional Water Quality Control Board expires on 15 October 2009. SCWA shall obtain requisite time extensions for the Coastal Development Permit and water quality certification prior to the commencement of any work to be performed during the remainder of the five-year Department of the Army permit authorization period. SCWA shall provide USACE a copy of all requisite time extensions to ensure continuing project conformance with State coastal zone and water quality standards. <u>Letter of Modification dated October 5, 2009</u> Under the provisions of33 CFR 325.7(b), Department of Anny Permit No. 285610N is hereby modified to incorporate the following Special Conditions to reflect the recommendations of NMFS and incidental take requirements specified in the Russian River BO (issued September 24,2008): 1. To remain exempt from the prohibitions of Section 9 of the Endangered Species Act of 1973, as amended (16 U.S.C. § 1531 *et seq.)*, the non-discretionary Terms and Conditions for incidental take of Central California Coast endangered coho salmon *(Oncorhynchus kisutch)*, Central California Coast threatened steelhead *(Oncorhynchus mykiss)*, and California Coastal threatened Chinook salmon *(Oncorhynchus tshawytscha)* shall be fully implemented in the manner stipulated in the Biological Opinion entitled, "Water Supply, Flood Control Operations, and Channel Maintenance conducted by the U.S. Anny 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" (File No. 151422SWR2000SRI50) issued by National Marine Fisheries Service on September 24, 2008. 2. All work shall be done in general accordance with SCWA's adaptive management plan for the estuary outlet channel at the mouth of the Russian River, as mandated by NMFS in the Reasonable and Prudent Alternative section of the Russian River BO for alterations to estuary management (pp. 249-50), entitled, "Russian River Estuary Outlet Channel Adaptive Management Plan Year 1" dated July 30, 2009 (Enclosure 1).

Sonoma County Water Agency
Summary of Special Conditions of Permits for Russian River Estuary Management Activities

Agency / Permit / Expiration	Special Conditions
California Environmental Quality Act Environmental Impact Report (EIR) Notice of Preparation – May 10, 2010 Notice of Completion – December 15, 2010 Notice of Determination – August 16, 2011	See EIR for Mitigation Measures.

Sonoma County Water Agency
Summary of Special Conditions of Permits for Russian River Estuary Management Activities

Agency / Permit / Expiration	Special Conditions
California State Lands Commission General Lease, Public Agency Use (PRC 7918.1 R 08103) – June 29, 2004 Lagoon Outlet Channel Authorization – October 13, 2009 (Expiration - December 31, 2010) Monthly Extensions - January 1 to December 31, 2011 General Lease, Public Agency Use (PRC 7918.9) – January 1, 2012 Expiration – May 15, 2015	1. Lessee certified an Environmental Impact Report on August 16, 2011 for the proposed activities authorized in this Lease. Lessee acknowledges that, on September 14, 2011, a lawsuit was filed by the Russian River Watershed Protection Committee against Lessee in the Superior Court of the County of Sonoma alleging that the EIR is inadequate under the California Environmental Quality Act and certification of the EIR be vacated (Case SCV-250347). Notwithstanding any other provisions of this Lease, Lessee further acknowledges that, if certification of the EIR is vacated, the lease shall terminate within 30 days after the EIR is ruled in invalid. 2. Lessee shall comply with all mitigation measures contained in the Mitigation Monitoring Program prepared and adopted by the Sonoma County Water Agency on August 16, 2011. 3. Lessee acknowledges that the land described in Exhibit A of this Lease is subject to the Public Trust and is presently available to members of the public for recreation, waterborne commerce, navigation, fisheries, open space, or other recognized Public Trust uses and that Lessee's proposed construction activities and use of the Lease Premises shall not interfere or limit the Public Trust rights of the public. At least 24 hours prior to and during the breaching activities, Lessee will contact the California Department of Parks and Recreation lifeguards and post signs and barriers to minimize potential hazards to the public. 4. Prior to the start of the initial freshwater lagoon construction on the Lease Premises, Lessee shall submit to Lessor copies of all permits and authorizations from agencies having jurisdiction over the construction of the authorized activities on the Lease Premises. Lessee shall maintain all regulatory permits and authorization required during the term of the lease. 5. All breaching activities shall be carried out in accordance with all applicable safety regulations, permits, and conditions of all other agencies. 6. During the term of the lease, Lessee shall provide Lessor with an annual report on frequency and timing of outlet channel construction and maintenance and breaching occurrences completed each calendar year, including number of days of closure of Goat Rock State Beach. The report should include narrative descriptions and evaluations of outlet channel and breaching events, including any adaptive management changes implemented. 7. Lessee shall submit to Lessor copies of the following: a. Adaptive estuarine water level and barrier beach management plans (as described in 2.1.1 of the Russian River Biological Opinion) after approval by the National Marine Fisheries (NMFS), the California Department of Fish and Game, and the U.S. Army Corps of Engineers.

Agency / Permit / Expiration	Special Conditions
California State Lands Commission (continued)	b. Annual water quality data summary reports (as described in 2.2, Monitoring Estuarine Water Quality: Reporting and Review, of the Biological Opinion). c. Annual report, as specified in the "Russian River Estuary Management Activities Pinniped Monitoring Plan" and distributed to NMFS, the California Department of Parks and Recreation, and the Stewards of the Coasts and Redwoods, on pinnipeds' reaction to the proposed activities authorized in this Lease. 8. All personal property, tools, or equipment taken onto or placed upon the Lease Premises shall remain the property of the Lessee or its contractors. Such personal property shall be promptly removed by the Lessee, at its sole risk and expense upon the completion of the project. Lessor does not accept any responsibility for any damage, including damages to any personal property, including any equipment, tools, or machinery on the Lease Premises 9. No refueling, repairs, or maintenance of vehicles or equipment will take place on the Lease Premises. 10. Lessee shall maintain a logbook on all work vessels during work within the Lease Premises utilized in operations conducted under this Lease to keep track of all debris created by objects of any kind that may fall into the water. The logbook should include the type of debris, date, time and location to facilitate identification and location of debris for recovery and site clearance verification. All debris shall be promptly removed from the Lease Premises. 11. Any equipment to be used on the Lease Premises is limited to that which is directly required to perform the authorized use and does not include any equipment that may cause damage to the Lease Premises.

Agency / Permit / Expiration	Special Conditions
California State Lands Commission (continued)	12. Lessee acknowledges and agrees: a. The site may be subject to hazards from natural geophysical phenomena including, but not limited to waves, storm waves, tsunamis, earthquakes, flooding and erosion. b. To assume the risks to the Lessee and to the property that is the subject of any Coastal Development Permit (CDP) that is issued to Lessee for development on the leased property, of injury and damage from such hazards in connection with the permitted development and use. c. To unconditionally waive any claim or damage or liability against the State of California, its agencies, officers, agents, and employees for injury or damage from such hazards. d. To indemnify, hold harmless and, at the option of Lessor, defend the State of California, its agencies, officers, agents, and employees, against and for any and all liability, claims, demands, damages, injuries, or costs of any kind and from any cause (including costs and fees incurred in defense of such claims), expenses, and amounts paid in settlement arising from any alleged or actual injury, damage or claim due to site hazards or connected in any way with respect to the approval of any CDP that is issued to Lessee involving this property or issuance of this Lease, any new lease, renewal, amendment, or assignment by Lessor.

Sonoma County Water Agency
Summary of Special Conditions of Permits for Russian River Estuary Management Activities

Agency / Permit / Expiration	Special Conditions
California State Lands Commission (continued)	13. Lessor shall have the right to enter upon the property at reasonable times in order to monitor Lessee's compliance with and otherwise enforce the terms of the Lease. 14. Paragraph 9 contained within Section 3 is hereby deleted from this Lease. In the event of any conflict between the provisions of Section 2 and Section 3 of this Lease, the provisions of Section 2 shall prevail.
California Department of Parks and Recreation Temporary Use Permit – December 30, 2003 Permit Extension – September 14, 2009 Permit Extension – December 28, 2009 Expiration – June 30, 2010 Temporary Use Permit – May 15, 2011 Expiration – December 31, 2012 Time Extension - December 31, 2013	**Now therefore**, the State by this Permit hereby grants to the Permittee permission to enter upon State's property, conditioned upon the agreement of the Parties that this Permit does not create or vest in Permittee any interest in the real property herein described or depicted, that the Permit is revocable and non-transferable, and that the Permit is further subject to the following terms and conditions: **1. Project Description:** By this Permit, the State hereby grants to the Permittee permission to enter onto those lands depicted and described on Exhibit "A", Russian River Estuary Management Activities, and Exhibit "B", Russian River Estuary Outlet Channel: Excavation Cut and Fill Locations, attached hereto and herein incorporated by this reference, solely for the purpose of flood control and environmental monitoring. **2. Permit Subject to Laws and Regulatory Agency Permits:** This Permit is expressly conditioned upon Permittee's obtaining any and all regulatory permits or approvals required by the relevant regulatory agencies for the Project and Permittee's use of the Property, and upon Permittee's compliance with all applicable municipal, state and federal laws, rules and regulations, including all State Park regulations. Prior to commencement of any work, Permittee shall obtain all such legally required permits or approvals and submit to the State full and complete copies of all permits and approvals, including documentation related to or referenced in such permits and approvals, along with the corresponding agency contact and telephone numbers, and related California Environmental Quality Act (CEQA) and/or National Environmental Policy Act (NEPA) documentation as applicable. **3. Term of Permit:** This Permit shall only be for the period beginning on 11/15/2011, and ending on 12/31/2012, or as may be reasonably extended by written mutual agreement of the Parties. **4. Consideration:** Permittee agrees to pay State the sum of One thousand five hundred and No/100 Dollars ($1,500.00) as consideration for the rights granted by this Permit. Payment is due upon execution of this Permit. **5. Permit Subject to Existing Claims:** This Permit is subject to existing contracts, permits, licenses, encumbrances and claims which may affect the Property.

Agency / Permit / Expiration	Special Conditions
California Department of Parks and Recreation (continued)	**6. Waiver of Claims and Indemnity:** Permittee waives all claims against State, its officers, agents and/or employees, for loss, injury, death or damage caused by, arising out of, or in any way connected with the condition or use of the Property, the issuance, exercise, use or implementation of this Permit, and/or the rights herein granted. Permittee further agrees to protect, save, hold harmless, indemnify and defend State, its officers, agents and/or employees from any and all loss, damage, claims, demands, costs and liability which may be suffered or incurred by State, its officers, agents and/or employees from any cause whatsoever, arising out of, or in any way connected with this Permit, exercise by Permittee of the rights herein granted, Permittee's use of the Property and/or the Project for which this Permit is granted, except those arising out of the sole active negligence or willful misconduct of State. Permittee will further cause such indemnification and waiver of claims in favor of State to be inserted in each contract that Permittee executes for the provision of services in connection with the Project for which this Permit is granted. **7. Contractors:** Permittee shall incorporate the terms, conditions and requirements contained herein when contracting out all or any portion of the work permitted hereunder. Permittee shall be responsible for ensuring contractor/subcontractor compliance with the terms and conditions contained herein. Failure of Permittee's contractors to abide by State's terms and conditions shall constitute default by Permittee (see Paragraph 20) allowing State to terminate this Permit and seek all legal remedies. **8. Insurance Requirements:** As a condition of this Permit and in connection with Permittee's indemnification and waiver of claims contained herein, Permittee shall maintain, and cause its contractors to maintain, a policy or policies of insurance as follows: Permittee shall maintain motor vehicle liability with limits of not less than $1,000,000 per accident. Such insurance shall cover liability arising out of a motor vehicle, including all owned, hired, and non-owned motor vehicles. Permittee shall maintain statutory Workers' Compensation and employer's liability insurance coverage in the amount of $1,000,000/employee/disease/each accident, for all its employees who will be engaged in the performance of work on the Property, including special extensions where applicable. Said policy shall include a waiver of subrogation in favor of State. If the Permittee is an individual or sole proprietor who is not required by law to have Workers' Compensation insurance, Permittee shall provide State with a written confirmation that Permittee is not required to be, and has elected not to be, covered by Workers' Compensation. Permittee shall procure commercial general liability insurance at least as broad as the most commonly available ISO policy form CG 0001 premises operations, products/completed operations, personal/advertising injury and contractual liability with limits not less than $1,000,000 per occurrence and $2,000,000 general aggregate. Said policy shall apply separately to each insured against whom any claim is made or suit is brought subject to the Permittee limits of liability.

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California Department of Parks and Recreation (continued)	Each policy of insurance required by this provision shall: (a) be in a form, and written by an insurer, reasonably acceptable to State; (b) be maintained at Permittee's sole expense; and (c) require at least thirty (30) days written notice to State prior to any cancellation, non-renewal or material modification of insurance coverage. Insurance companies issuing such policies shall have a rating classification of "A-" or better and financial size category ratings of "VII" or better according to the latest edition of the A.M. Best Key Rating Guide. All Insurance companies issuing such policies shall be licensed to do business in the State of California. Such policies shall contain an endorsement naming the CALIFORNIA DEPARTMENT OF PARKS AND RECREATION as an additional insured at no cost to State. Permittee shall provide to State evidence that the insurance required to be carried by this Permit, including any endorsement affecting the additional insured status, is in full force and effect and that premiums therefore have been paid. Such evidence shall, at State's discretion, be in either the form of an ACORD Form (Certificate of Insurance) or DPR Form 169A (Certificate of Insurance for Concession Contracts/Special Events), or a certified copy of the original policy, including all endorsements. Permittee is responsible for any deductible or self-insured retention contained within the insurance program. Should Permittee fail to keep the specified insurance in effect at all times, Permittee shall be considered to be in default of this Permit, and State may, in addition to any other remedies it has, terminate this Permit.

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California Department of Parks and Recreation (continued)	Permittee shall require and ensure that all contractors and subcontractors have adequate insurance meeting the coverage requirements in this provision. Any insurance required to be carried shall be primary and not excess to any other insurance carried by State. Coverage shall be in force for the complete term of this Permit, including any extension thereof, and for all work being done for which this Permit is required. 9. **Reservation of Rights:** State reserves the right to use the Property in any manner, provided such use does not unreasonably interfere with Permittee's rights herein. 10. **Access Limits and Conditions:** Access to the Property shall be limited to the access designated by State and is illustrated in Figure 2 of Exhibit "A" and as described below. The barrier beach would be accessed from the paved parking lot at Goat Rock State Beach, located at the end of Goat Rock Road off of Highway 1. Equipment would be off-loaded in the parking lot and driven north onto the beach via an existing access point within the parking lot. Additional detail is provided in the attached Russian River Estuary Management Activities. 11. **Notice of Work:** Any required notices to State shall be sent to the State authorities in charge of Sonoma Coast State Park named below. At least 24 hours prior to any entry upon the Property for any of the purposes hereinabove set forth, Permittee shall provide the State contact[s] named below with written notice of Permittee's intent to enter the Property. **STATE:** Contact: Brendan O'Neil Address: 25381 Steelhead Blvd. Duncans Mills, CA 95430 Tel: 707/865-2391 Fax: 707/865-2046 **PERMITTEE:** Contact: Jessica Martini-Lamb Address: 404 Aviation Blvd. Santa Rosa, CA 95403 Tel: 707/547-1903 Fax: 707/524-3782 12. **Limits of Work:** In no event shall this Permit authorize work in excess or contrary to the terms and conditions of any regulatory agency permit or approval. Under no circumstances, whether or not authorized by any regulatory agency, other permit or any person or entity other than State, shall work exceed that which is authorized by this Permit as described in the Exhibit B, Russian River Estuary Management Activities. 13. **Public Safety:** Permittee is responsible for public safety during and after the breaching operation until such time that water velocities and standing waves recede, the sandbar banks stabilize and cease to erode, cave and wash away and heavy equipment has been removed from State Park property. In the interest of public and Park visitor safety STATE reserves the right to require PERMITTEE to provide Peace Officers and/or Lifeguards, at no cost to STATE, to monitor and close the beach to the public for a distance of 750' on each side of the breach as recommended in the Russian River Estuary Study. In the interest of public safety, the preferred days for sandbar breaching are from Monday to Thursday (excluding holidays) when Park visitation is usually at a minimum. In the event of emergency situations, breaching may proceed immediately after notifying the State Park District Superintendent or their designee. 14. **Compliance with Monitoring and Mitigation Measures:** Resource monitoring and mitigation measures identified within the Russian River Estuary Management Project Final Environmental Impact Report, NMFS Biological Opinion, DFG Lake and Streambed Alteration Agreement, Regional Water Quality Control Board Section 401 Water Certification, California Coastal Commission Coastal Development Permit, US Army Corps of Engineers Section 404 and Section 10 Permit, and State Lands Commission General Lease shall be completed in accordance with and to the satisfaction of the District Superintendent or designee. Permittee's activities conducted under this Permit shall comply with all State and Federal environmental laws, including, but not limited to, the Endangered Species Act, CEQA, and Section 5024 of the Public Resources Code. Any of Permittee's archaeological consultants working within the boundaries of the Property shall obtain a permit from the California State Parks Archaeology, History & Museums Division prior to commencing any archaeological or cultural investigations of the Property.

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California Department of Parks and Recreation (continued)	Permittee shall immediately advise State's contact person if any new site conditions are found during the course of permitted work. State will advise Permittee if any new historical resources (including archaeological sites), special status species, threatened/endangered species protocols, or other resource issues are identified within the Project site. Permittee shall abide by District Superintendent or designee's instructions to protect the resource(s) during the permitted work or risk revocation of the Permit. Permittee shall make all excavation activities on the Property available to the State Archaeologist for observation and monitoring. During excavation, the State archaeological monitor may observe and report to the State on all excavation activities. State archaeological monitor shall be empowered to stop any construction activities as necessary to protect significant cultural resources from being disturbed. In the event that previously unknown cultural resources, including, but not limited to, dark soil containing shell, bone, flaked stone, groundstone, or deposits of historic trash are encountered during Project construction by anyone, work will be suspended at that specific location, and the Permittee's work will be redirected to other tasks, until after a State-qualified archaeologist has evaluated the find and implemented appropriate treatment measures and disposition of artifacts, as appropriate, in compliance with all applicable laws and department resource directives. If human remains are discovered during the Project, work will be immediately suspended at that specific location and the District Superintendent or designee shall be notified by Permittee. The specific protocol, guidelines and channels of communication outlined by the California Native American Heritage Commission (NAHC), and/or contained in Health and Safety Code Section 7050.5 and Public Resources Code Sections 5097.9 et seq., will be followed. Those statutes will guide the potential Native American involvement in the event of discovery of human remains. Permittee shall provide a written work schedule to State so that the State archaeological monitor can arrange to be on site on the necessary days. Permittee shall provide reasonable advance notice of and invite the District Superintendent or designee to any preconstruction meetings with the prime contractor or subcontractors. **15. Restoration of Property:** Permittee shall complete the restoration, repair, and revegetation of the Property in consultation with, and to the satisfaction of the State Environmental Scientist should any damage result from permitted activities. Restoration, repair and/or revegetation is required within 30 days after damage or as determined by the State Environmental Scientist. This obligation shall survive the expiration or termination of this Permit. **16. Right to Halt Work:** The State reserves the right to halt work and demand mitigation measures at any time, with or without prior notice to Permittee, in the event the State determines that any provision contained herein has been violated, or in the event that cessation of work is necessary to prevent, avoid, mitigate or remediate any threat to the health and safety of the public or state park personnel, or to the natural or cultural resources of the state park. **17. Use Restrictions:** The use of the Property by Permittee, including its guests, invitees, employees, contractors and agents, shall be restricted to the daytime hours between sunrise and sunset on a day-by-day basis, unless otherwise approved in advance in writing by State. No person shall use or occupy the Property overnight. Activities on the Property shall be conducted only in a manner which will not interfere with the orderly operation of the state park. Permittee shall not engage in any disorderly conduct and shall not maintain, possess, store or allow any contraband on the Property. Contraband includes, but is not limited to: any illegal alcoholic beverages, drugs, firearms, explosives and weapons. Permittee shall not use or allow the Property to be used, either in whole or in part, for any purpose other than as set forth in this Permit, without the prior written consent of the State. **18. State's Right to Enter:** At all times during the term of this Permit and any extension thereof, there shall be and is hereby expressly reserved to State and to any of its agencies, contractors, agents, employees, representatives, invitees or licensees, the right at any and all times, and any and all places, to temporarily enter upon said Property to survey, inspect, or perform any other lawful State purposes. Permittee shall not interfere with State's right to enter. **19. Protection of Property:** Permittee shall protect the Property, including all improvements and all natural and cultural features thereon, including cultural and natural resources, at all times at Permittee's sole cost and expense, and Permittee shall strictly adhere to the following restrictions:

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California Department of Parks and Recreation (continued)	(a) Permittee shall not place or dump garbage, trash or refuse anywhere upon or within the Property, except in self-contained trash receptacles that are maintained to State's satisfaction by Permittee. (b) Permittee shall not commit or create, or suffer to be committed or created, any waste, hazardous condition or nuisance in, on, under, above or adjacent to the Property. (c) Permittee shall not cut, prune or remove any vegetation upon the Property, except as identified in the Project description and herein permitted or subsequently approved in writing by the District Superintendent. (d) Permittee shall not disturb, move or remove any rocks or boulders upon the Property, except as identified in the Project description and herein permitted or subsequently approved in writing by the District Superintendent. (e) Permittee shall not grade or regrade, or alter in any way, the ground surface of the Property, except as herein permitted, or subsequently approved in writing by the District Superintendent. (f) Permittee shall not bait, poison, trap, hunt, pursue, catch, kill or engage in any other activity which results in the taking, maiming or injury of wildlife upon the Property, except as identified in the Project description and herein permitted or subsequently approved in writing by the District Superintendent. (g) Permittee shall not use, create, store, possess or dispose of hazardous substances (as defined in the California Hazardous Substances Act) on the Property except as herein permitted, or subsequently approved in writing by the District Superintendent. (h) Permittee shall exercise due diligence to protect the Property against damage or destruction by fire, vandalism and any other causes. **20. Default:** In the event of a default or breach by Permittee of any of the terms or conditions set forth in this Permit, State may at any time thereafter, without limiting State in the exercise of any right of remedy at law or in equity which State may have by reason of such default or breach: (a) Maintain this Permit in full force and effect and recover the consideration, if any, and other monetary charges as they become due, without terminating Permittee's right to use of the Property, regardless of whether Permittee has abandoned the Property; or (b) Immediately terminate this Permit upon giving written notice to Permittee, whereupon Permittee shall immediately surrender possession of the Property to State and remove all of Permittee's equipment and other personal property from the Property. In such event, State shall be entitled to recover from Permittee all damages incurred or suffered by State by reason of Permittee's default, including, but not limited to, the following: (i) any amount necessary to compensate State for all the detriment proximately caused by Permittee's failure to perform its obligations under this Permit, including, but not limited to, compensation for the cost of restoration, repair and revegetation of the Property, which shall be done at State's sole discretion and compensation for the detriment which in the ordinary course of events would be likely to result from the default; plus (ii) at State's election, such other amounts in addition to or in lieu of the foregoing as may be permitted from time to time by applicable law. **21. State's Right to Cure Permittee's Default:** At any time after Permittee is in default or in material breach of this Permit, State may, but shall not be required to, cure such default or breach at Permittee's cost. If State at any time, by reason of such default or breach, pays any sum or does any act that requires the payment of any sum, the sum paid by State shall be due immediately from Permittee to State at the time the sum is paid. The sum due from Permittee to State shall bear the maximum interest allowed by California law from the date the sum was paid by State until the date on which Permittee reimburses State. **22. Revocation of Permit:** The State shall have the absolute right to revoke this Permit for any reason upon ten (10) days written notice to Permittee. Written notice to Permittee may be accomplished by electronic or facsimile transmission, and the notice period set forth in this paragraph shall begin on the date of the electronic or facsimile transmission, or, if sent by mail, on the date of delivery. If Permittee is in breach of the Permit or owes money to the State pursuant to this Permit, any prepaid monies paid by Permittee to State shall be held and applied by the State as an offset toward

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California Department of Parks and Recreation (continued)	damages and/or amounts owed. Nothing stated herein shall limit the State's exercise of its legal and equitable remedies. 23. **Recovery of Legal Fees:** In any action brought to enforce or interpret any provisions of this Permit or to restrain the breach of any agreement contained herein, or for the recovery of possession of the Property, or to protect any rights given to the State against Permittee, and in any actions or proceedings under Title 11 of the United States Code, if the State shall prevail in such action on trial or appeal, the Permittee shall pay to the State such amount in attorney's fees in said action as the court shall determine to be reasonable, which shall be fixed by the court as part of the costs of said action. 24. **Voluntary Execution and Independence of Counsel:** By their respective signatures below, each Party hereto affirms that they have read and understood this Permit and have received independent counsel and advice from their attorneys with respect to the advisability of executing this Permit. 25. **Reliance on Investigations:** Permittee declares that it has made such investigation of the facts pertaining to this Permit, the Property and all the matters pertaining thereto as it deems necessary, and on that basis accepts the terms and conditions contained in this Permit. Permittee acknowledges that State has made, and makes, no representations or warranties as to the condition of the Property, and Permittee expressly agrees to accept the Property in its as-is condition for use as herein permitted. 26. **Entire Agreement:** The Parties further declare and represent that no inducement, promise or agreement not herein expressed has been made to them and this Permit contains the entire agreement of the Parties, and that the terms of this agreement are contractual and not a mere recital. 27. **Warranty of Authority:** The undersigned represents that they have the authority to, and do, bind the person or entity on whose behalf and for whom they are signing this Permit and the attendant documents provided for herein, and this Permit and said additional documents are, accordingly, binding on said person or entity. 28. **Assignment:** This Permit shall not be assigned, mortgaged, hypothecated, or transferred by Permittee, whether voluntarily or involuntarily or by operation of law, nor shall Permittee let, sublet or grant any license or permit with respect to the use and occupancy of the Property or any portion thereof, without the prior written consent of State. 29. **Choice of Law:** This Permit will be governed and construed by the laws of the State of California.

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US Department of Commerce, National Oceanic and Atmospheric Administration, National Marine Fisheries Service Incidental Harassment Authorization - April 21, 2011 Incidental Harassment Authorization (renewal) - April 21, 2012 Expiration – April 20, 2013	1. This Incidental Harassment Authorization (IHA) is valid from April 2, 2012 through April 20, 2013. 2. This IHA is valid only for activities associated with estuary management activities in the Russian River, Sonoma County, California, including: Lagoon outlet channel management; artificial breaching of barrier beach; geophysical surveys and other work associated with a jetty study; and physical and biological monitoring ofthe beach and estuary as required. 3. General Conditions (a) A copy of this IHA must be in the possession of the SCWA, its designees, and work crew personnel operating under the authority of this IHA. (b) SCWA is hereby authorized to incidentally take, by Level B harassment only, 2.963 harbor seals *(Phoca vitulina)*, 37 California sea lions *(Zalophus caldvmianus)*, and 20 northern elephant seals *(Iirounga angusliroslris)*. (c) The taking by Level A harassment, serious injury or death of any of the species listed in item 3(b) of the Authorization or the taking by harassment, injury or death ofany other species of marine mammal is prohibited and may result in the modification, suspension, or revocation of this IHA. If SCWA observes a pup that may be abandoned, it shall contact the National Marine Fisheries Service (NMFS) Southwest Regional Stranding Coordinator immediately (562-980-3230; Sarah.Wilkin@noaa.gov) and also report the incident to NMFS Office of Protected Resources (301-427-8425; Benjamin.Laws@noaa.gov) within 48 hours. Observers shall not approach or move the pup.

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US Department of Commerce, National Oceanic and Atmospheric Administration, National Marine Fisheries Service (continued)	4. Mitigation Measures In order to ensure the least practicable impact on the species listed in condition 3(b), the holder of this Authorization is required to implement the following mitigation measures: (a) SCWA crews shall cautiously approach the haul-out ahead of heavy equipment to minimize the potential for sudden 1lushes, which may result in a stampede -a particular concern during pupping season. (b) SCWA staff shall avoid walking or driving equipment through the seal haul-out. (c) Crews on foot shall make an et1brt to be seen by seals from a distance, if possible, rather than appearing suddenly at the top of the sandbar, again preventing sudden flushes. (d) During breaching events, all monitoring shall be conducted from the overlook on the bluff along Highway 1 adjacent to the haul-out in order to minimize potential for harassment. (e) A water level management event may not occur for more than two consecutive days unless flooding threats cannot be controlled. (f) Equipment shall be driven slowly on the beach and care will be taken to minimize the number of shut-downs and start-ups when the equipment is on the beach. (g) All work shall be completed as efficiently as possible, with the smallest amount of heavy equipment possible, to minimize disturbance of seals at the haul-out. (h) Boats operating near river haul-outs during monitoring shall be kept within posted speed limits and driven as far from the haul-outs as safely possible to minimize flushing seals. In addition, SCW A shall implement the following mitigation measures during pupping season (March 15-June 30): (i) SCWA shall maintain a one week no-work period between water level management events (unless flooding is an immediate threat) to allow for an adequate disturbance recovery period. During the no-work period, equipment must be removed from the beach.

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	j) If a pup less than one week old is on the beach where heavy machinery will be used or on the path used to access the work location, the management action shal1 be delayed until the pup has left the site or the latest day possible to prevent flooding while still maintaining suitable fish rearing habitat. In the event that a pup remains present on the beach in the presence of flood risk, SCW A shall consult with NMFS and CDFG to determine the appropriate course of action. SCWA shall coordinate with the locally established seal monitoring program (Stewards of the Coast and Redwoods) to determine if pups less than one week old are on the beach prior to a breaching event. (k) Physical and biological monitoring shall not be conducted if a pup less than one week old is present at the monitoring site or on a path to the site. 5. Monitoring The holder of this Authorization is required to conduct baseline monitoring and shall conduct additional monitoring as required during estuary management activities: (a) Baseline monitoring shall be conducted twice-monthly for the term of the IHA. These censuses shall begin at dawn and continue for eight hours, weather permitting; the census days shall be chosen to ensure that monitoring encompasses a low and high tide each in the morning and afternoon. All seals hauled out on the beach shall be counted every thirty minutes from the overlook on the bluff along Highway 1 adjacent to the haul-out using high powered spotting scopes. Observers shall indicate where groups of seals are hauled out on the sandbar and provide a total count for each group. If possible, adults and pups shall be counted separately. (b) In addition, peripheral haul-outs shall be visited tor ten minute counts twice during each baseline monitoring day. (c) During estuary management events, monitoring shall occur on all days that activity is occurring using the same protocols as described for baseline monitoring, with the difference that monitoring shall begin at least one hour prior to the crew and equipment accessing the beach work area and continue through the duration of the event, until at least one hour after the crew and equipment leave the beach. In addition, a one-day pre-event survey of the area shall be made within one to three days of the event and a one-day post-event survey shall be made after the event, weather permitting. (d) Monitoring of peripheral haul-outs shall occur concurrently with event monitoring, when possible. (e) For all monitoring, the following information shall be recorded in thirty minute intervals:

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	i. pinniped counts, by species; ii. behavior; iii. time, source and duration of any disturbance, with takes incidental to SCWA actions recorded only for responses involving movement away from the disturbance or responses of greater intensity (e.g., not for alerts); iv. estimated distances between source of disturbance and pinnipeds; v. weather conditions (e.g., temperature, percent cloud cover, and wind speed); and vi. tide levels and estuary water surface elevation. (f) All monitoring during pupping season shall include records of any neonate pup observations. SCWA shall coordinate with the Stewards' monitoring program to determine if pups less than one week old are on the beach prior to a water level management event. 6. Reporting The holder of this Authorization is required to: (a) Submit a report on all activities and marine mammal monitoring results to the Office of Protected Resources, NMFS, and the Southwest Regional Administrator, NMFS, 90 days prior to the expiration of the IHA if a renewal is sought, or within 90 days of the expiration of the permit otherwise. This report must contain the following information: i. the number of seals taken, by species and age class (if possible); ii. behavior prior to and during water level management events; iii. start and end time of activity; iv. estimated distances between source and seals when disturbance occurs; v. weather conditions (e.g., temperature, wind, etc.); vi. haul-out reoccupation time of any seals based on post activity monitoring;

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	vii. tide levels and estuary water surface elevation; viii. seal census from bi-monthly and nearby haul-out monitoring; and ix. specific conclusions that may be drawn from the data in relation to the four questions of interest in SCWA's Pinniped Monitoring Plan, if possible. (b) Reporting injured or dead marine mammals: In the unanticipated event that the specified activity clearly causes the take of a marine mammal in a manner prohibited by this IHA, such as an injury (Level A harassment), serious injury, or mortality, SCWA shall immediately cease the specified activities and report the incident to the Chief of the Permits and Conservation Division, Office of Protected Resources, NMFS, and the Southwest Regional Stranding Coordinator, NMFS. The report must include the following information: A. Time and date of the incident; B. Description of the incident; C. Environmental conditions (e.g., wind speed and direction, Beaufort sea state, cloud cover, and visibility); D. Description of all marine mammal observations in the 24 hours preceding the incident; E. Species identification or description of the animal(s) involved; F. Fate of the animal(s); and G. Photographs or video footage of the animal(s). Activities shall not resume until NMFS is able to review the circumstances of the prohibited take. NMFS will work with SCWA to determine what measures are necessary to minimize the likelihood of further prohibited take and ensure MMPA compliance. SCWA may not resume their activities until notified by NMFS. ll. In the event that SCWA discovers an injured or dead marine mammal, and the lead observer determines that the cause of the injury or

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	death is unknown and the death is relatively recent (e.g., in less than a moderate state of decomposition), SCWA shall immediately report the incident to the Chief of the Permits and Conservation Division, Office of Protected Resources, NMFS, and the Southwest Regional Stranding Coordinator, NMFS. The report must include the same information identified in 6(b)(i) of this IHA. Activities may continue while NMFS reviews the circumstances of the incident. NMFS will work with SCWA to determine whether additional mitigation measures or modifications to the activities are appropriate. Ill. In the event that SCWA discovers an injured or dead marine mammal and the lead observer determines that the injury or death is not associated with or related to the activities authorized in the IHA (e.g., previously wounded animal, carcass with moderate to advanced decomposition, or scavenger damage), SCWA shall report the incident to the Chief of the Permits and Conservation Division, Office of Protected Resources, NMFS, and the Southwest Regional Stranding Coordinator, NMFS, within 24 hours of the discovery. SCWA shall provide photographs or video footage or other documentation of the stranded animal sighting to NMFS. IV. Pursuant to sections 6(b)(ii-iii), SCWA may use discretion in determining what injuries (i.e., nature and severity) are appropriate for reporting. At minimum, SCWA must report those injuries considered to be serious (i.e., will likely result in death) or that are likely caused by human interaction (e.g., entanglement, gunshot). Also pursuant to sections 6(b)(ii-iii), SCWA may use discretion in detem1ining the appropriate vantage point for obtaining photographs of injured/dead marine mammals. 7. Validity of this Authorization is contingent upon compliance with all applicable statutes and permits, including NMFS' 2008 Biological Opinion for water management in the Russian River watershed. This Authorization may be modified, suspended or withdrawn if the holder fails to abide by the conditions prescribed herein, or if the authorized taking is

Attachment D. Russian River Barrier Beach and Estuary Water Surface Level Adaptive Management in Concert with Physical Processes

(from National Marine Fisheries Service)

Russian River Barrier Beach and Estuary Water Surface Level Adaptive Management in Concert with Physical Processes

John McKeon, National Marine Fisheries Service

To comply with NMFS' BO for adaptive management of the RR estuary, i.e., to manage the beach with the goal of conserving beach sand to allow formation of a stable low-flow season elevated outlet-channel and creating a brackish /freshwater lagoon with marine influence minimized, the Sonoma County Water Agency (SCWA) will need to balance multiple natural physical processes when carrying out flood control activities. The two primary processes to balance are: wave and longshore transport of sand into the channel, dependent on wave direction, height and steepness; and outlet channel river-flow scour determined by slope, depth and roughness. The amount of sand transported by either force is dependent on sand supply. As the channel is likely to be of sand only, the vertical elevation-controls of the outlet channel will be the sum of sand transport out of the channel at low tide by the river outflow, versus transport of sand into the channel on the incoming high tide by wave action and longshore current. As the tide lowers and rises, one of these two physical forces will predominate. Balancing the two transport mechanism rates over a 24 hr tidal cycle will be key to maintaining an over-all stable vertical outlet channel elevation and stable estuary water levels minimally influenced by tidal fluctuation. The wave-face between the low tide line and the top of the wave-face crest (height determined by wave height at high tide) will be the key area of scour and accretion during the cycle.

Calculation of scour in open flume channels is a well studied subject, with critical shear stress of when sediments are mobilized on the channel bottom a function of grain size, water velocity and depth. Velocity is determined by roughness and slope. Channel dimension, slope and roughness can be calculated for predicted flow ranges to minimize sheer stress, bed mobilization, scour, and incision of the channel. However, slope across the wave face will be determined by the beach profile where the river outflow meets the ocean. This is the likely point at which channel headcutting would begin, resulting in significant lowering of the outlet channel elevation and estuary water surface elevation (WSE). Because SCWA cannot influence the slope of the wave face beach profile, strategies to minimize scour potential are limited to: 1) choose a river channel outlet location across the wave face where the beach profile has the least slope between the low tide line and wave-face crest height, and 2) minimize depth with increased channel width across the crest of the wave face. This will both limit scour on the outgoing tide, and increase wave transport of sand into the mouth with a greater length of wave break pushing sand into the channel on high tides. Also, to limit propagation of any headcutting precipitated at low tide, the velocity in the channel above the wave face can be decreased with increased roughness and length, or the depth (and scour potential) decreased by increasing the outlet channel width. The beach size and configuration at the time of closure, and the jetty, will constrain, and in part determine, these three channel characteristics.

However, if flood threats and subsequent breaching actions are to be avoided, minimization of scour in the channel and across the wave face needs to be balanced against the ability of channel outflow to remove the predictable transport of sand into the channel by wave and longshore transport, both of which significantly increase during a beach building event and result in a channel closure event.

Transport of sand by waves on to a beach (and into the outlet channel) occurs when wave height compared to wave length reaches a critical point, which is called critical steepness, expressed as Critical H/L. JW Johnson determined critical steepness in the laboratory as = 0.03; waves with a lower H/L value moved sand offshore, those with a higher value moved sand onshore[2]. Wave length is directly proportional to wave period. Using the acceleration rate of gravity, 32/ft/sec/sec= g; and pi for rough approximation of wave form as sinusoidal, L = g/2pi* T^2 or 5.12T^2 (*e.g.,* 13 ft waves, 9 second period; 9 squared*5.12= 414.72; 13/414.72= 0.0314, steep enough to accrete, or 9 ft waves, 7 second period; 7 squared*5.12= 250.88; 9/250.88= 0.0359).

Because of the coastal aspect of the RR beach and the presence of headlands to the north and south, wave direction is important in determining the height of waves which reach the beach. Wave direction and size also determine the strength of the longshore current, and thus the rate of channel infilling on an incoming tide. The larger the waves, and greater the angle of wave incidence away from perpendicular to the beach, the stronger the longshore current and amount of sand transport.

The incidence of the outlet channel to the wave-face crest will be critical in limiting channel infilling by wave action during a beach building event. When a beach building/closure event is occurring, at high tide waves will be delivering and depositing sand up and over the wave face crest into the mouth of the channel at a rate much greater than the ability of the relatively low flow of the channel to transport sand in opposition to the direction of wave transport. However, a channel behind the wave-face crest and close to perpendicular to the wave direction will be more capable of transporting the sand washed into it by wave action, as flow from the wave will be entrained in the flow of the outlet channel, with the added flow increasing the transport power of the outlet channel. Thus, by orienting the outlet channel near to perpendicular to wave run-up direction, the out-flow channel will be better at limiting or preventing accretion of sand in the channel mouth by successive waves than if the channel is parallel to the wave run-up direction. Strategies for minimizing accretion of sand in the lagoon outlet channel mouth during a beach building event, and limiting likelihood of outlet channel closure events will be: 1) choose a river channel outlet location where the beach profile has the least slope between the low tide line and wave-face crest height, as less slope will mean a greater distance for waves to expend their energy before topping the wave crest, and/or the lower wave-face crest would signify an area of reduced wave size and transport capacity; 2) align the channel from the lagoon outlet, and behind the wave-face crest, to be as near to perpendicular as possible to wave run-up direction in order to minimize sand accretion at the channel mouth during high tide.; 3) insure there is sufficient slope from the lagoon WSE to the point the channel crosses the wave-face crest sufficient to maintain flow across the wave-face crest when waves push the crest above the high tide line (~ 3.3 ft NGVD with a 6 foot high tide). This means planning for the outlet channel invert to be above the lowest point of the wave-face crest height.

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[2] Willard Bascom. 1980. *Waves and Beaches.* Anchor Books Edition. ISBN: 0-385-14844-5

Channel Planform and Slope

In addition to the above described means to balance scour and accretion in the channel mouth and across the wave face, the channel planform will be dictated by beach topography. The entire beach topography above the tide lines is determined by waves and longshore current that will continue to sculpt the beach once the outlet channel has been established. To avoid repetitive heavy equipment excursions on to the beach to reform the outlet channel, the beach topography should dictate both the channel planform and slope of the outlet channel. To determine the most natural channel planform and slope, *i.e.*, the planform location and slope that will most likely be maintained by wave and tidal action subsequent to formation of an outlet channel by SCWA, a detailed topographic survey of the beach will need to be prepared post lagoon-closure, and prior to beach and estuary WSE management actions.

Natural Analogues

When waves reach critical steepness and sand accretion occurs on the beach, the underwater sand bar just outside the wave break is moved onshore with the incoming tide. The beach increases in both width and height, which results in a lengthening of the outlet channel as it has a greater width of beach to cross, and behind the wave-face crest, flows longitudinally along the beach to the lowest point of the crest. The increased length of the channel results in more resiliency to scour and incision during low tide and allows for stabilized lagoon WSE, with tidal influence becoming muted. Lacking subsequent beach building events, the channels may scour back down below the high tide level within weeks, reintroducing tidal influence to the lagoon WSE. However, with continued or subsequent beach building events, the channel continues to elevate and lengthen, and with river inflows declining in spring/summer, the channel loses its ability to incise, and a closed of perched lagoon WSE eventually results.

A short duration event of critically steep waves and beach building occurred along the California Coast the week of May 27th to June 3, 2010. Attached are photos of these river mouth beaches and the channels that resulted from that short duration beach building event. A WSE stage monitor in the Carmel lagoon recorded the effect on lagoon WSE, in which subsequent to the event and the lengthening of the channel, the WSE of the lagoon was maintained above the high tide level and tidal influence became muted. Photos included are of Carmel, San Lorenzo, Scott, Waddell, Pamponio and Navarro river beaches. A plot of the Carmel lagoon WSE for June 2010 can be viewed at http://www.mpwmd.dst.ca.us/wrd/lagoon/webplots/2010/2010webplots.htm

CARMEL, 6/9/2010

San Lorenzo, 6/10/2010

Scott Creek, 6/10/2010

Waddell, 6/10/2010

Pamponio, 6/10/2010

Navarro, 6/6/2010

Navarro, 6/6/2010

Navarro, 6/6/2010 (high tide-/Lagoon WSE ~ 6-7 feet NGVD estimated)

Navarro, 6/6/2010

Attachment E. Implementation of the 2010 Outlet Channel Adaptive Management Plan

At the direction of NMFS, Sonoma County Water Agency (the Agency) has been tasked with creating an outlet channel intended to improve salmonid habitat in the Russian River Estuary while maintaining the current level of flood protection for properties adjacent to the estuary (NMFS, 2008). The adaptive management plan, described in the main body of this report, was developed by the Agency with assistance from ESA PWA and the resource agency management team in 2009 and revised in 2010. Because of permit constraints, the Agency was only able to implement the plan beginning in 2010. This attachment documents the management actions in response to inlet closures that occurred during the 2010 lagoon management period.

During the management period, May 15th to October 15th, Agency staff regularly monitored current and forecast estuary water levels, inlet state, river discharge, tides, and wave conditions to anticipate inlet closure. For the first month and a half, river discharge was somewhat larger than historic daily median conditions due to a wetter-than-average spring, but then receded to nearly replicate historic median flow rates. Average monthly wave energy in 2010 was similar to historic averages for most of the management period and higher for June and October. Two periods of inlet closure occurred (Figure 1), leading the Agency to begin planning for management action to create an outlet channel, in accordance with the plan's communication protocol:

- Starting in late June 2010, physical conditions at the mouth of the Russian River Estuary naturally established an outlet channel that persisted for a week before wave action completely closed the lagoon. In response to this closure, the Agency attempted to create an outlet channel for the first time. This management action briefly re-established outlet channel conditions, but within a half day, wave action re-closed the outlet channel. Before the next scheduled management action could take place, the lagoon breached, returning the estuary to tidal conditions.
- The estuary closed twice more in the management period, during the third week of September and again at the start of October. Although action to create an outlet channel was initially considered after the September closure, an extended period of large waves limited beach access due to safety concerns. As a result, water levels continued to rise, heightening flood risk. Therefore, in consultation with the resource agency management team, the Agency decided to implement full breaching. Two attempts were required for each closure before the lagoon was successfully breached.

The next section of this attachment reviews the process for leading up to and during the July outlet channel implementation. In the following section, the September and October closures are assessed. Although the September and October closures did not result in creation of an outlet channel, the planning process and physical processes are relevant to adaptive management. The last section summarizes lessons learned from the 2010 management period to consider in subsequent years.

JUNE-JULY 2010 OUTLET CHANNEL EVOLUTION

In the second half of June, an outlet channel and perched lagoon were naturally established at the mouth of the Russian River. For about one week, this channel conveyed enough water to the ocean to sustain 4.5 to 5 ft NGVD water levels in the lagoon. Once waves closed the outlet channel and lagoon water levels began to rise, the Agency implemented a management action to create an outlet channel. In the face of strong waves, this outlet quickly closed. Several days later, the lagoon was breached and tidal conditions returned until September. Details of this channel evolution are provided below.

NATURALLY ESTABLISHED OUTLET CHANNEL

Outlet channel conditions (defined as a nearly steady lagoon water levels above ocean water levels and maintained by uni-directional outflow in a channel passing through the beach berm) naturally established over a week-long period in late June. The physical conditions associated with this evolution are described below.

Water level

Water levels in the lagoon, as observed at the Jenner gage, exhibited a muted tide range, indicative of partial closure, starting on June 20th as shown in Figure 2a. The tide range gradually decreased from about 1.5 ft until tidal variations ceased early on the morning of June 27th. Lagoon water levels then increased over the next day to just over 4 ft NGVD. Water levels were then fairly constant at about 4 ft NGVD for three days. On June 30th, the water levels started to decline, probably due to the drop in upstream riverine discharge as compared to higher outlet channel discharge. Water levels declined to a minimum of 3 ft NGVD before the channel closed on July 4th.

Ocean waves and tides

Significant wave height at CDIP's Point Reyes buoy increased above 2 m starting on June 24th as shown in Figure 2b. About the time that tidal influence disappeared from lagoon water levels on June 27th, the significant wave height exceeded 3 m and stayed above 3 m until July 1st. Peak wave period during this time period was approximately 8 seconds and the peak direction was from the northwest. Figure 3 illustrates the wave direction, period, and magnitude from June 16th through July 14th. Astronomic tides were declining from peak spring levels, with the higher high water on June 27th of just over 3 ft NGVD as shown in Figure 2c.

Riverine discharge

Riverine discharge in late June was higher than to median conditions because of late season precipitation and full reservoirs. Figure 2d illustrates how flow dropped rapidly from 325 ft^3/s on June 27th to 225 ft^3/s on June 30th. Flow then continued to drop more slowly at a rate of less than 5 ft^3/s per day for the next two weeks.

Planform alignment

At the time of closure, the channel exited the northwest corner of the lagoon and ran along the foot of the bluff, landward of the berm crest, for approximately 550 ft. The channel then crossed the berm and exiting to the ocean. This alignment was similar to the alignment observed during 1998, an El Nino year (personal communication, C. Delaney). Several days before the closure, the channel was observed further south than its alignment along the bluff once the outlet channel established. Unfortunately, the Agency's automated camera did not collect pictures between June 23-29 due to a power failure, precluding a more detailed analysis of the channel's planform evolution in the days preceding the establishment of the outlet channel.

Beach and channel topography

The beach berm north of the outlet channel and the downstream end of the channel was surveyed by Agency staff on July 1st (Figure 4). The presence of seals on the beach to the south of the channel prevented additional survey data from being collected. On both sides of the channel's mouth, sand had deposited such the intertidal beach protruded approximately 50 feet into the ocean as compared to the beach alignment further south (Figure 4 and Figure 5a). Just north of the outlet channel, the beach face that had been covered by wave runup during the previous high tide extended up to 8 ft NGVD. Then the beach profile stepped up to a bench with elevations above 10 ft NGVD. South of the channel, the berm crest elevation was estimated to approximately 7 ft NGVD, but was not measured directly. The outlet channel was approximately 60 ft wide, with its bed elevation at 0-1 ft NGVD for last one hundred feet before it entered the ocean. The channel flowed around numerous large boulders along much of its length. These boulders may have served as natural grade control inhibiting erosion.

Channel discharge

On June 30th, the Agency collected water depths and point velocities in the outlet channel, which was approximately 60 ft wide. Water in the outlet channel flowed at depths up to 2.7 ft and velocities of at least 5.4 ft^2/s. These velocities are in excess of permissible scour criteria for beach sands, but not sufficient to scour the larger boulders found in the outlet channel (Fischenich, 2001). Integrated water depth and point velocity measurements yielded an estimate the channel's discharge of 297 ft^3/s (SCWA unpublished observations). As shown in Figure 2d, this discharge magnitude was observed upstream at Guerneville approximately two days earlier and was larger than the concurrent Guerneville discharge. This is consistent with the dropping water levels in the lagoon (Figure 2a) and tributary inflows downstream of Guerneville.

WAVE-INDUCED OUTLET CHANNEL CLOSURE

After the week of sustained outlet channel conditions, the wave energy briefly relaxed on July 2nd, and then returned to significant wave heights from the northwest exceeding 3.5 m starting on July 3rd (Figure 2b). This increase in wave height was accompanied by an increase in northwest swell wave period to approximately 10 seconds. This increase in wave energy provided enough landward sand transport to close the outlet channel. Riverine discharge had recently declined, reducing the channel's ability to clear sand and remain open. This closure occurred during a neap tide, when higher high water levels just barely exceeded 2 ft NGVD.

Changes to the wave climate continued for the next several days, with the peak direction shifting to the south and the wave period lengthening to nearly 14 seconds (Figure 3). Significant wave height dropped to less than 1.5 m. This long-period, low-steepness swell is likely to have built the beach berm with onshore sand transport. This likely onshore transport changed the beach topography changed in two ways. The protruding sand deposits at the channel's mouth noticeably diminished in size between July 4th and July 5th, and were essentially gone by July 6th. In addition, the onshore transport probably built the berm crest elevation from the estimated berm crest elevation of 7 ft NGVD on July 1st (C. Delaney) and July 4th (J. Largier) to an elevation of 8.5 ft NGVD as surveyed on July 8th.

Once the outlet channel closed, lagoon water levels began to rise at a rate of approximately 0.5 ft/day. The channel closure and rising water levels initiated the Agency's outlet channel management plan.

MANAGEMENT ACTION

Management action to create an outlet channel was scheduled for July 8th in consultation with the resource management team. The action was scheduled for July 8th because it was a Thursday, the last day that action could be taken before the State Parks permit restrictions on Friday-Sunday operations went into effect. Given the observed rate of lagoon water level rise of 0.5 ft/day, waiting until the following Monday was deemed to be too risky in terms of flood hazard and channel scour. To provide operational flexibility in response to site conditions, two different management options were proposed during planning. Figure 4 shows the alignment of these options, both 30 ft wide, as laid on the topographic surface collected on July 1st. This schematic design was used to discuss management plans with the resource agencies, to estimate volumes of excavated material, and to guide operations staff. Option A, the preferred option, followed the northwest alignment of the natural outlet channel prior closure. In the event that beach surveys indicated a low point in the berm further south or if access to the Option A location was restricted by waves, Option B was proposed just north of Haystack Rock.

Based an assessment of site conditions early on the morning of July 8th, Option A was selected for implementation. Excavation began at approximately 7am on July 8th with a bulldozer and backhoe excavator. The lagoon water level at the time work began was 5.9 ft NGVD.

The excavated portion of the managed channel followed the alignment of the southern half of the naturally established outlet channel, as shown in Figure 5b. This alignment allowed the excavation equipment to avoid rocks embedded in the berm. The backhoe removed sand from the landward portion of the berm, adjacent to a large rock. The bulldozer pushed sand towards the ocean to form the lower portion of the channel. A small berm was preserved between the two pieces of equipment to prevent lagoon outflow before the channel was complete. After approximately two hours of work, wave runup associated with the rising tide started to enter the channel's mouth. Therefore, the middle berm was removed with the excavator at approximately 9:30am, completing the channel.

At the time of completion, the outlet channel was approximately 30 ft wide and had an invert of approximately 4.5 ft NGVD. The estimated volume of excavated sand was 230 yd^3. Water flowed in the channel at a depth of approximately 0.5 ft. Flow was typically uniformly seaward in the upstream portion of the newly excavated channel. However, in the downstream portion, wave runup periodically overwhelmed the outflow, causing the flow to switch direction to landward. The transition between the existing channel and the newly excavated portion created a hydraulic control across which water transitioned from subcritical to supercritical, thereby explaining the channel's lower water level as compared to the lagoon. Bed erosion was observed starting from this transition region and into the new portion.

During the period when the outlet channel was open, water levels in the lagoon continued to increase at a similar rate to the rate before the management action. This constant rate of water level increase indicates that flow in the outlet channel was relatively small compared to riverine inflow to the lagoon.

OUTLET CHANNEL CLOSURE

As ocean tides increased water levels throughout July 8th, the wave runup from the south swell advanced up and over the beach face, as evidenced by the absence of equipment tracks on the beach in July 9th photographs. By the evening of July 8th, this advancing wave runup transported enough sand into the outlet channel that the channel once again closed. Higher high water on the evening of July 8th was above 3 ft NGVD, as tidal conditions were building towards large spring tides.

After reviewing lagoon and beach conditions on July 9th, the Agency scheduled follow-up management for Monday, July 12th, the first day which they were allowed to operate on the beach under their State Parks permit.

BREACHING TO TIDAL CONDITIONS

Lagoon water levels continued to rise at a rate of approximately 0.5 ft/day in the days following closure. On the evening of July 11th, the lagoon breached in the vicinity of Haystack Rock. The lagoon water level at the time of the breach was 7 ft NGVD, which is approximately 1.5 ft below the berm crest elevation surveyed on July 8th. This difference suggests that the breach may have been caused by seepage through the berm. Just before the breach, the water's edge extending towards the breach site, indicating that breach occurred at the low point in the beach berm's crest elevation.

Because the estuary returned to tidal conditions on July 11th, the management action planned for July 12th was cancelled. Tidal conditions persisted in the estuary until September.

SEPTEMBER-OCTOBER 2010 CLOSURES AND MANAGEMENT

In the end of August, coincident with neap tides and increased wave heights, the estuary water levels became muted, diminishing to a tide range of less than one foot (Figure 6a). Shortly afterwards, starting on September 4th, wave energy increased considerably from the northwest (Figure 7b) to sustained wave heights exceeding 3 m and peaking above 4 m (Figure 6b). This combination of muted tides followed by large waves, would seem to have been ideal conditions to prompt closure. However, the inlet stayed open throughout this high wave period. Several factors probably contributed to the inlet's persistent opening. Although large in height, the waves' period was relatively short (below 12 seconds) and from the northwest. Because of the beach faces the southwest, it may be partially sheltered from waves out of the northwest. The tides were transitioning from neap to spring, so the increasing tidal prism would have contributed to scouring the inlet's channel. Wave overtopping also may have contributed to maintaining inlet by adding water to the estuary that then flowed out the inlet, scouring the channel.

After the muted tides in early September, full tide range returned to the lagoon, probably assisted by the arrival of larger spring tides. Around September 18th, during the month's second neap tide, another wave event was observed with significant wave height less than 2 m, nearly half the magnitude of the early September event (Figure 6b). However, the wave period was longer, 16-18 seconds instead of 8-10 seconds, and waves were from the south instead of the northwest. These conditions closed the estuary on September 21st.

After the inlet closed on September 21st, planning to establish an outlet channel began. Based on the most recent beach topography, the projected rate of lagoon water level increase, tides, and wave forecasts, September 28th, was selected for an attempt at creating an outlet channel. Two options for the channel were proposed, one extending to the northwest from the edge of the lagoon, and one just south of Haystack Rock where the inlet had been just before closure. Lagoon water levels were above 6 ft NGVD by the 28th, as anticipated, in part due to wave overwash. Although water levels were rising, runup from large waves made beach access unsafe and operations were postponed to September 29th. Unsafe wave conditions persisted on the 29th, again preventing beach access. Since wave forecasts predicted only a brief lull on the next day before large waves returned and weekend access restrictions loomed, the Agency, in consultation with the resource agency management team, decided on the evening of Wednesday, September 29th, to switch from attempting to create an outlet channel to attempting a full breach.

Wave and tide conditions on the morning of September 30th allowed for beach access and a full breach was implemented. However, waves carried on the rising tide re-closed the inlet that afternoon and lagoon water levels continued to rise. A second attempt at breaching the afternoon of the 30th was cancelled because of unsafe wave conditions on the beach. Because of the impending flood risk (9 ft water levels were projected by Sunday, October 3rd), the Agency sought and received permission from State Parks to access the beach Friday, October 1st. The breach on October 1st was successful, helped by extensive scour coinciding with tides dropping to lower low water during the night. Estuary water levels dropped to 1 ft NGVD on October 2nd.

After a brief lull, wave conditions once again intensified and the inlet closed again on October 4th. Although still within the management period, the proximity to the end of the management season, as well as continuing forecasts for high waves, led the Agency to propose and receive permission from the resource agency management team for a full breach. Breaching was attempted on October 11th, when lagoon water levels had exceeded 7 ft NGVD. This attempt failed as waves pushed sand into the breach before it could enlarge and lower lagoon water levels. A second breach attempt was made on the afternoon of October 12th, successfully creating a sustained breach that lowered estuary water levels to tidal conditions. A third closure occurred on October 21st and naturally breached on October 24th, partly in response to high river discharge. Although this third event was outside the outlet channel management period, it was indicative of the extended period of large waves during September and October 2010.

LESSONS LEARNED AND RECOMMENDATIONS

Based on observations of the estuary, associated physical processes, and the July 8th outlet channel management action, we note the following lessons about implementing the outlet channel management plan.

CONCEPTUAL MODEL

- All four closures discussed above occurred coincident with noticeable wave energy associated with periods greater than 12 seconds. In fact, a long period, but relatively low wave height (less than 2m) event closed the inlet in the third week of September even though a larger wave height, but shorter period wave event two weeks earlier did not close the inlet. In all but one case, the long period waves which caused closure originated from the south or west.
- When wave runup started to progress into the outlet channel and force operations to end, it was decided to favor a deeper outlet channel over a wider outlet channel. Channel depth was sought to facilitate more discharge from the lagoon to counter incoming waves. We recommend continuing to observe channel/ocean dynamics in subsequent outlet channels to inform tradeoff decisions of this nature.

FEASIBILITY

- In hindsight, a better opportunity for establishing an outlet channel in July may have been July 10th or the morning of July 11th, when the long-period south swell had subsided but before the breach occurred. However, based on available information (wave forecasts and no knowledge of the breach) the management action was enacted earlier, on July 8th, because the following days were Friday through Sunday when State Parks restricts beach access. Future outlet channel management opportunities are likely to face similarly constrained time windows: too soon after closure, the wave conditions which caused closure may prevent safe beach access and lagoon water levels will be less than the BO targets; too late after closure and water levels may cause flooding or overtopping the beach berm. In addition to the State Parks weekend access constraints, operations are constrained by IHA rules, particularly before June 15th when pupping season ends.

- If the rocks embedded in the beach are essential for stabilizing against failure by scour, then the elevation of the rocks will largely determine the outlet channel bed elevation and lagoon water level. During the naturally established outlet channel which occurred from June 27th through July 3rd, the channel's bed elevation just before the beach face was 0-1 ft NGVD (July 1st Agency survey) and the lagoon water level was between 4.5 and 5 ft NGVD. Under these conditions, the outlet channel was able to convey approximately 300 ft^3/s.
- If an outlet channel had been in place at the start of the September-October large wave period, it quite likely would have closed since waves frequently overtopped the beach berm and even some full breaches were quickly closed. If the lagoon water level was close to or at the BO target 7 ft NGVD when the closure occurred and beach access was limited by wave conditions for multiple days, e.g. the five day period from September 26th to September 30th, the lagoon would likely have reached flood stage.
- Management actions attempting full breaching, which aim to convert the inlet between two of its stable modes (breached and closed) and which are informed by decades of management experience, still fail quite regularly. For example, in 2010, two of four breach attempts were unsuccessful and historically, one out of every three attempts have been unsuccessful (Behrens et al., in prep). We anticipate that the failure rate of efforts to create an outlet channel, a less common and less stable transitional state, to be at least as frequent, if not more frequent, than the failure rate for full breaches.

COMMUNICATION

- Continue the practice of developing and communicating a backup plan for the outlet channel management action in the event that surf conditions were unsafe at the preferred channel location. Communicating this backup plan ahead of time allowed time for discussion among the resource management team, reducing the potential for last minute disagreement if this option had to be enacted.
- Agency, NMFS, and ESA PWA staff consulted as to the specifics of the outlet channel implementation immediately before and during the excavation. This discussion was necessary because of uncertainty about the actual beach topography, the excavation progress relative to the tides, and the overall development of outlet channel strategy for this initial implementation. It enabled real-time adaptation to on-site constraints. For instance, the excavation's location was shifted slightly south of the prior channel's location to avoid large rocks known to be hidden within the berm. After following this alignment beyond the rocks, the excavation was guided northward so that the mouth of the outlet channel would be as close as possible to the prior location.
- After each management action, we suggest asking State Parks staff if operations had gone in accordance with their expectations with regard to parking lot use, public safety, sand placement, etc.

STAFFING

- The Agency's engineer on site had broad knowledge of the project objectives and operational constraints, enabling him to engage in discussion with the other on-site personnel (particularly the NMFS representative), observe physical conditions, and make real-time decisions about the outlet channel configuration. This presence and decision-making authority was essential since the management action was only defined ahead of time as a strategy, not construction-grade drawings.
- Develop capacity of other Agency staff to manage outlet channel operation so availability of informed decision-makers does not hinder management operations.
- Although equipment operators were new to the site, they adeptly executed outlet channel design as directed by Agency staff. Encourage the contractor to provide staff familiar with the project whenever possible.

EQUIPMENT AND OPERATIONS

- The backhoe excavator was more adept at operations adjacent to rock, the bulldozer was faster for areas with open sand. Particularly if operations occur over two days, consider choice of equipment. For example, on the first day, choose two bulldozers for speed in excavating a larger channel and replace one bulldozer with an excavator on the second day for more precise operations.
- Tides, daylight, and permits all restrict the time available for operations. To maximize time available for implementing management actions, consider the following procedures:
    - When possible, have key resource management team members discuss the operations plan ahead of time, ideally on-site the day before, or by phone if on-site is not practical.
    - Clarify staging procedure between equipment operators and engineering staff to reduce waiting
    - Consider the use of lights to enable equipment to operate under low-light conditions.
- Because rocks limit the outlet channel's alignment; having survey staff on-hand to stake locations of rocks covered by the sand was useful. Agency surveys should continue to monitor rock locations during monthly surveys.
- Equipment operators demonstrated good coordination between the pieces of equipment, with neither piece idle for an extended period. The two pieces smoothly switched the two primary tasks of channel excavation and feathering excavated material onto the beach face.
- Sand cleared from the outlet channel was left as a temporary berm at the mouth of the outlet channel to impede wave runup into the outlet channel. This berm was re-shaped just before finishing to open the outlet channel while still providing some protection from south swell.

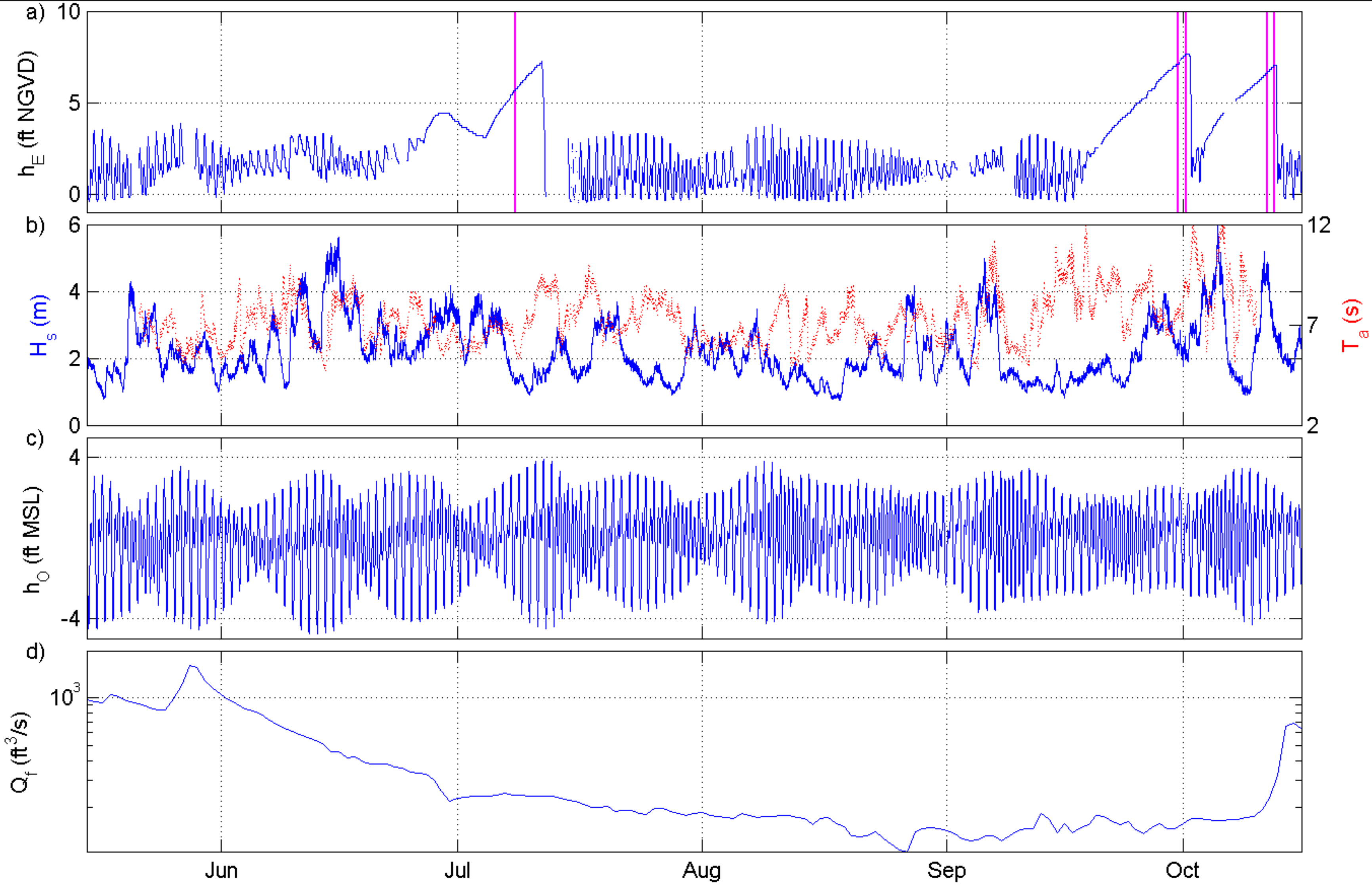
MONITORING

- Because the IHA limits the days available to place people on the beach to collect data, use the full two days allotted for outlet channel creation to collect additional data. For instance, consider having the survey team return at 12-hr intervals to take photographs and survey channel bathymetry and discharge.
- Consider an alternate automated camera placement to capture the northern portion of the beach.

REFERENCES

Behrens, D.K., F. A. Bombardelli, J. L. Largier, and E. Twohy. in preparation. Natural and human influences on tidal inlet closure in small bar-built estuaries

Fischenich, C. 2001. Stability thresholds for stream restoration materials. EMRRP Technical Notes Collection (ERDC TN-EMRRP-SR-29). U.S. Army Engineer Research and Development Center, Vicksburg, MS.



Sources:
a) h_E=estuary water level (SCWA); pink bar = mmgt action
b) H_s=sig. wave height; T_a=avg. wave period (CDIP, Pt. Reyes, #029)
c) h_O=ocean water level (NOAA, Pt. Reyes #9415020)
d) Q_f=river discharge (USGS, Guerneville #11467000)

Figure 1
Russian River Estuary Outlet Channel Management Plan

Estuary and Ocean Conditions, May 15 - October 15 2010

PWA Ref# 1958.01

ESA PWA

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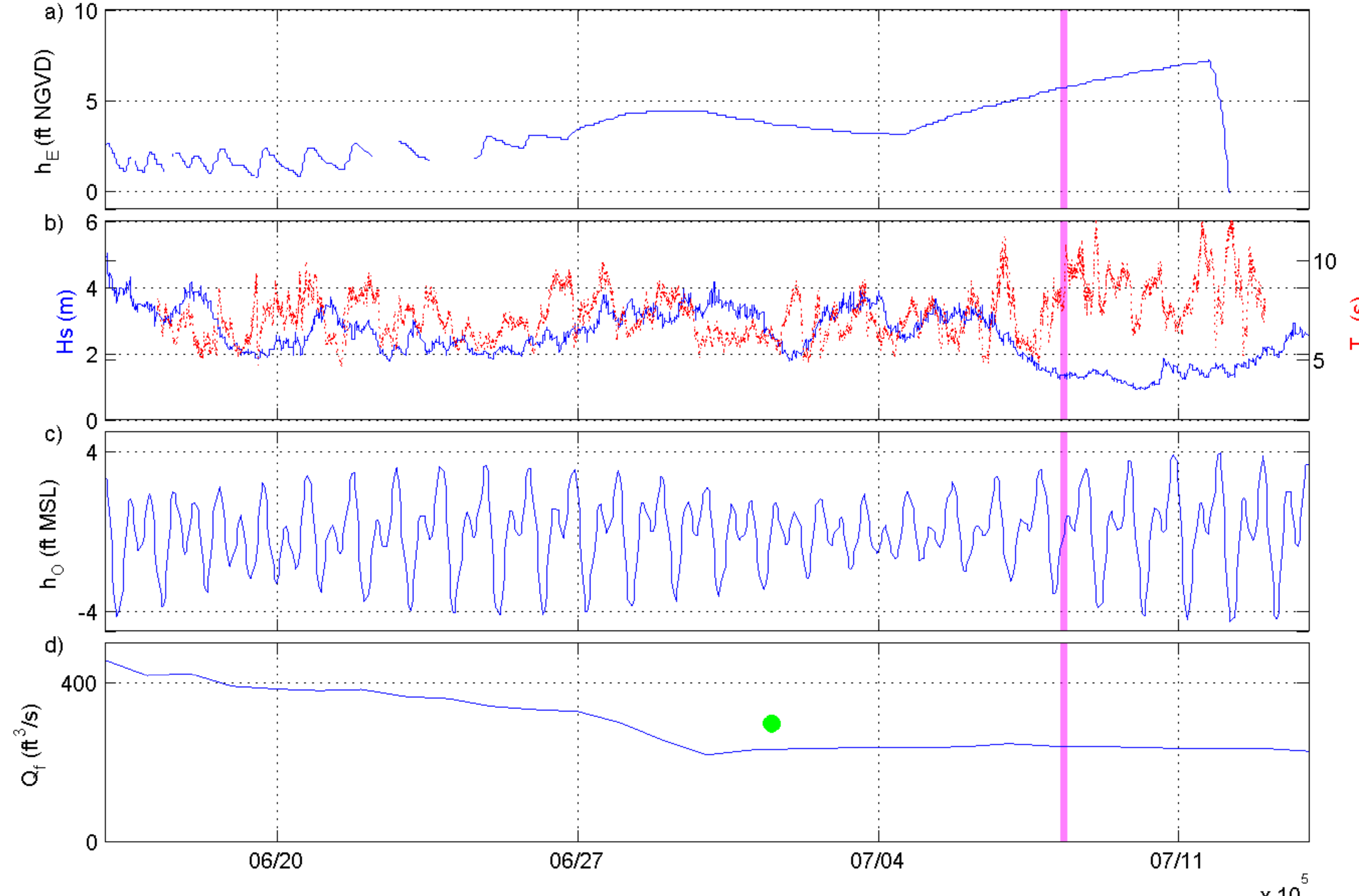

Sources:

a) h_E=estuary water level (SCWA); pink bar = mmgt action

b) H_s=sig. wave height; T_a =avg. wave period (CDIP, Pt. Reyes, #029)

c) h_O=ocean water level (NOAA, Pt. Reyes #9415020)

d) Q_f=river discharge (USGS, Guerneville #11467000)

Figure 2

Russian River Estuary Outlet Channel Management Plan

Estuary and Ocean Conditions, June - July 2010

PWA Ref# 1958.01

ESA PWA

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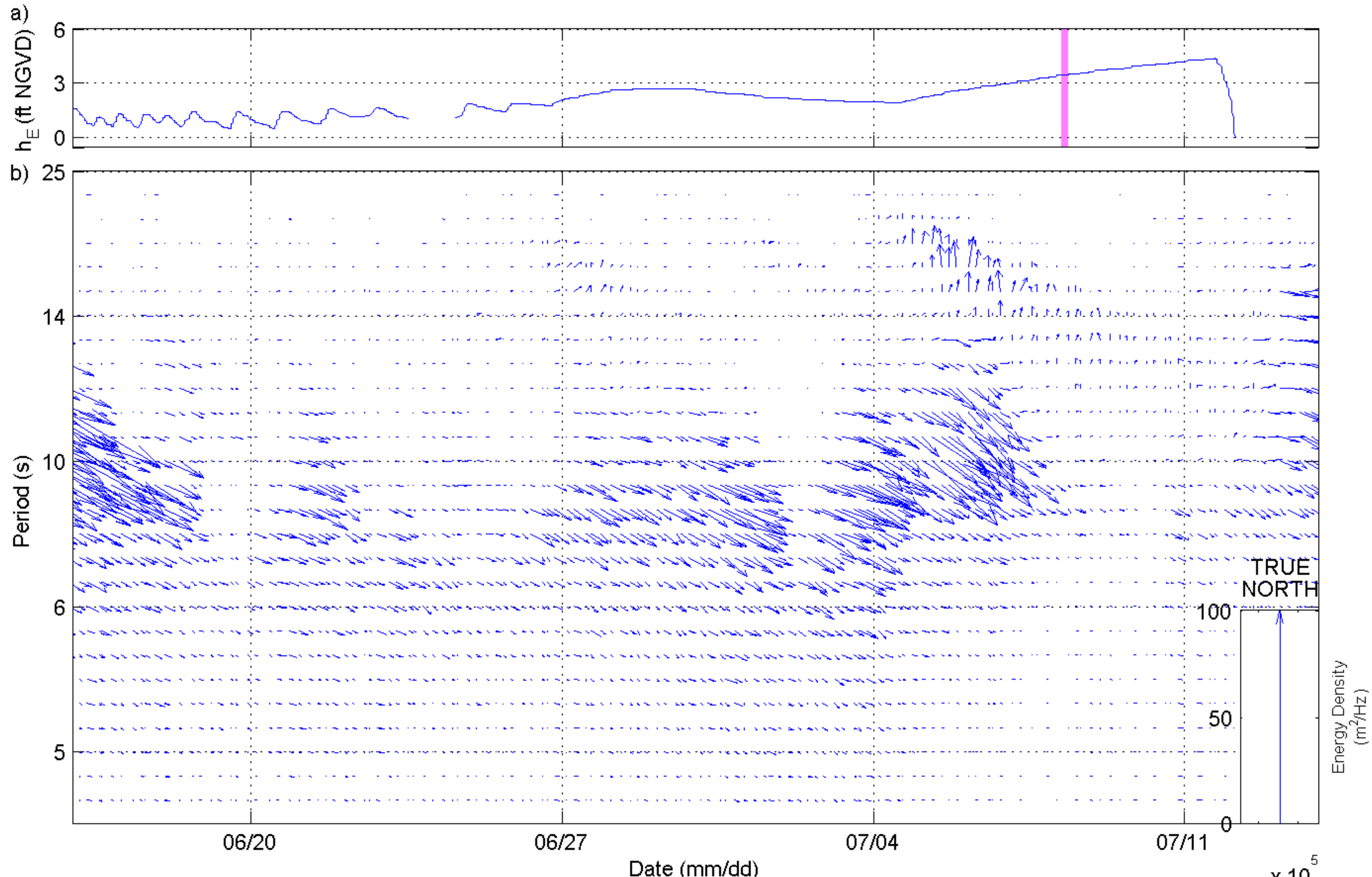

Sources:

a) h_E=estuary water level (SCWA); pink bar = mmgt action

b) Wave magnitude and direction (CDIP, Pt. Reyes, #029)

Figure 3

Russian River Estuary Outlet Channel Management Plan

Estuary Water Level and Wave Energy/Direction Spectrum

JUNE-JULY 2010

PWA Ref# 1958.01

ESA PWA

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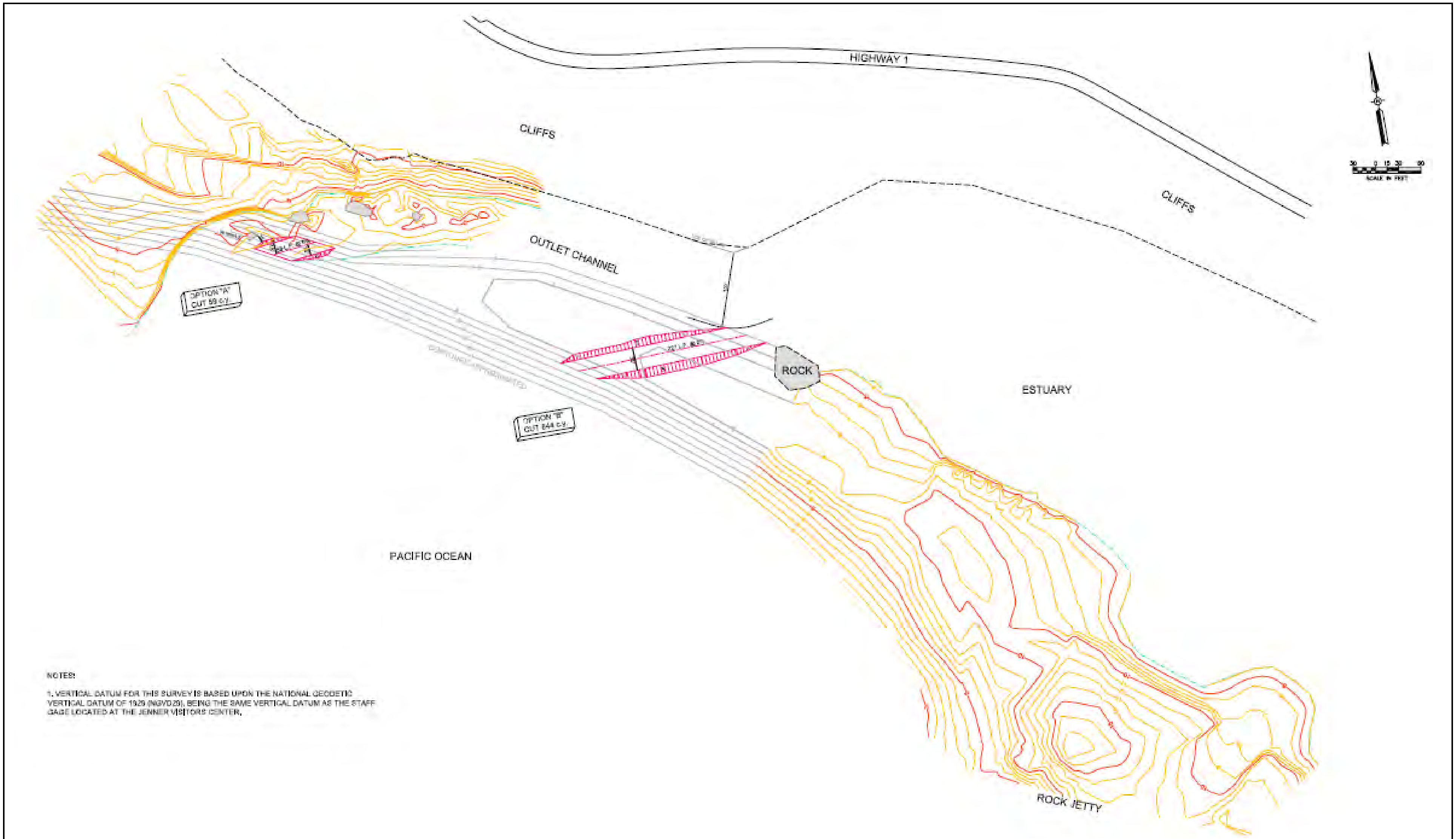


Source: SCWA

figure ***4***

Russian River Outlet Channel Adaptive Management Plan

Beach Topography and Management Options, June 2010

PWA Ref# 1958.01

ESA PWA

a)

b)

Source: C. Delaney, SCWA

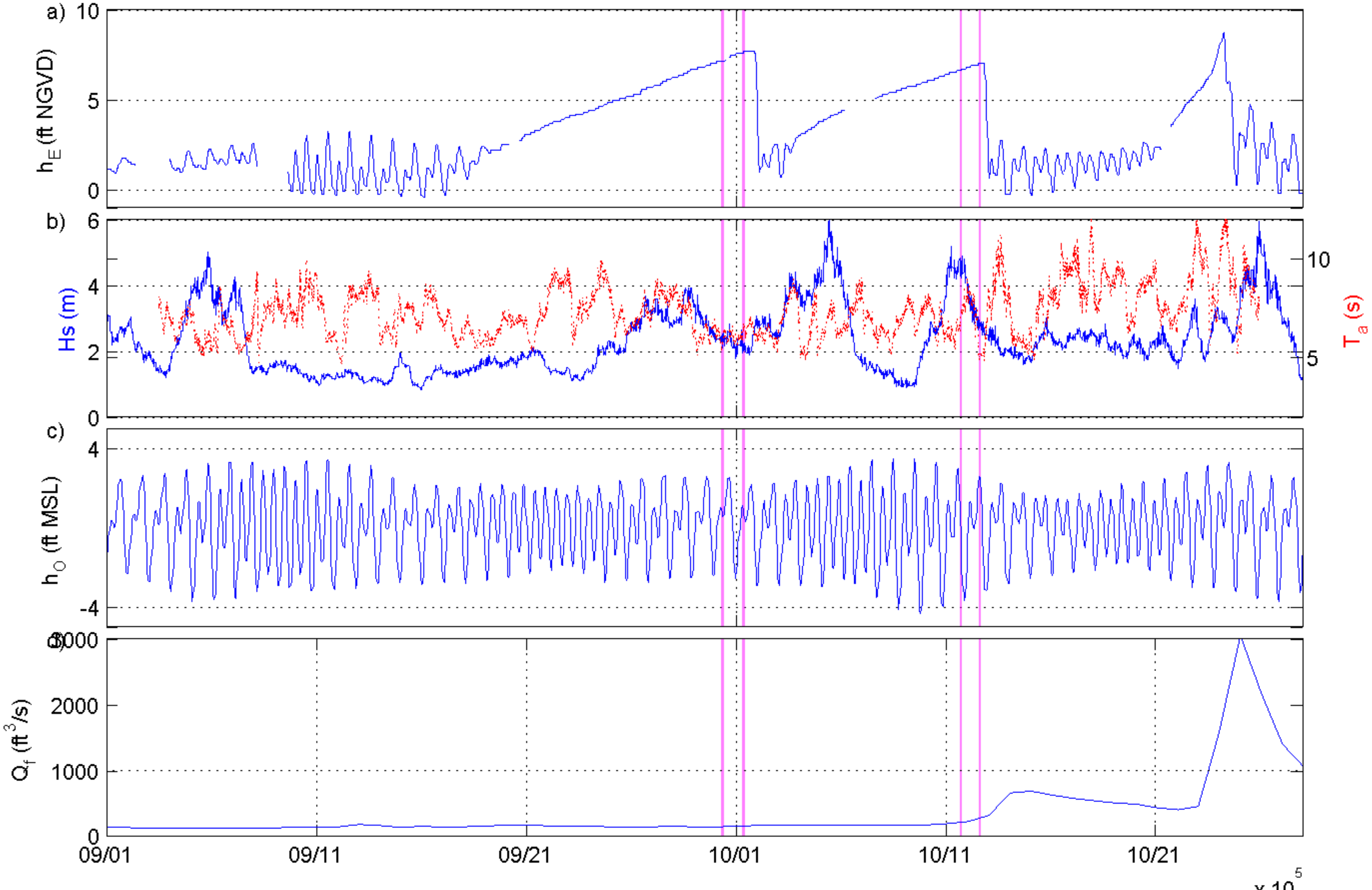
figure 5

Russian River Outlet Channel Adaptive Management Plan

Natural and Managed Outlet Channels

PWA Ref# 1958.01

ESA PWA



Sources:
a) h_E=estuary water level (SCWA); pink bar = mmgt action
b) H_s=sig. wave height; T_a=avg. wave period (CDIP, Pt. Reyes, #029)
c) h_O=ocean water level (NOAA, Pt. Reyes #9415020)
d) Q_f=river discharge (USGS, Guerneville #11467000)

Figure 6
Russian River Estuary Outlet Channel Management Plan

Estuary and Ocean Conditions, September - October 2010

PWA Ref# 1958.01

ESA PWA

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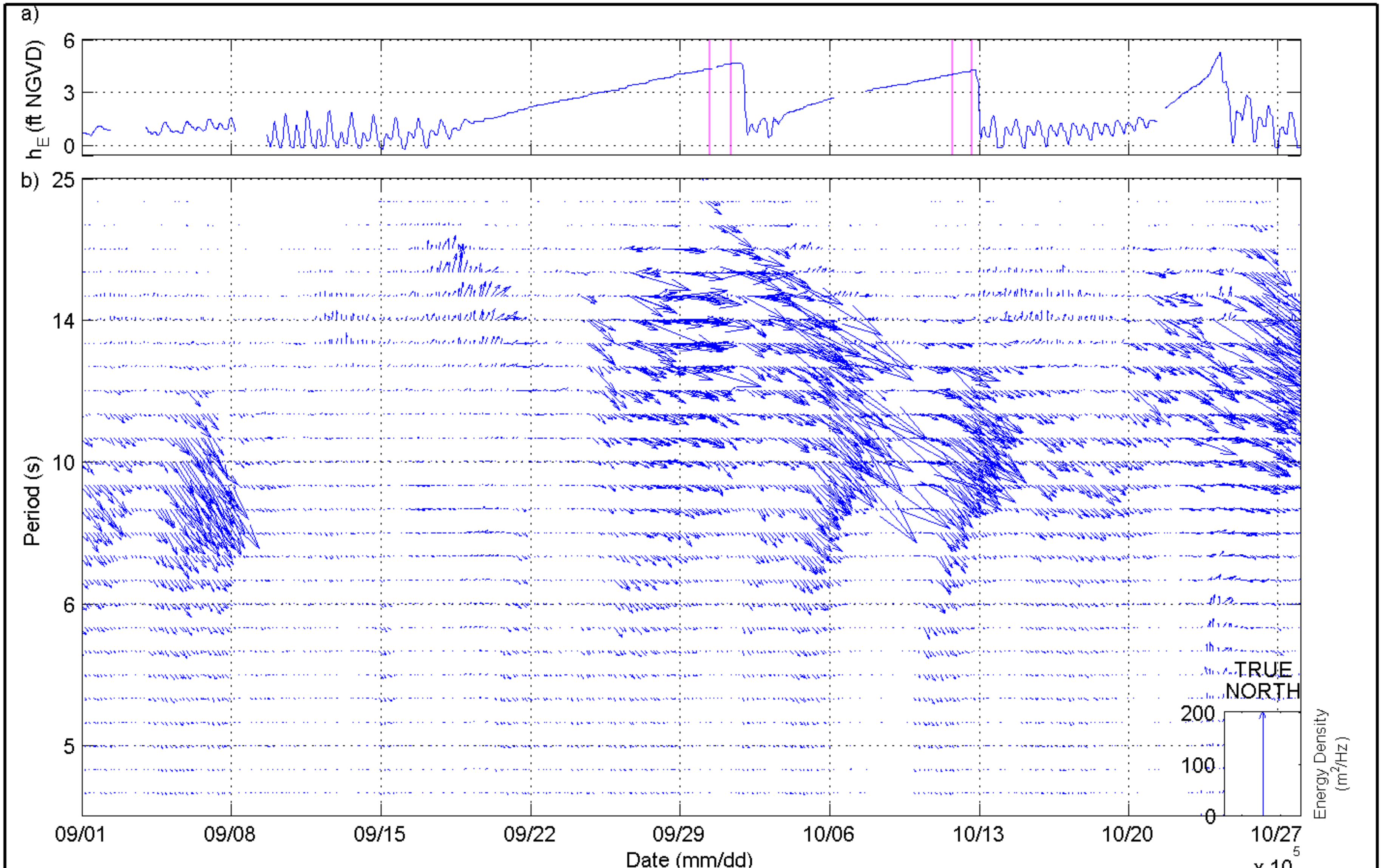

Sources:
a) h_E=estuary water level (SCWA); pink bar = mmgt action
b) Wave magnitude and direction (CDIP, Pt. Reyes, #029)

Figure 7
Russian River Estuary Outlet Channel Management Plan

Estuary Water Level and Wave Energy/Direction Spectrum
September-October 2010

PWA Ref# 1958.01

ESA PWA

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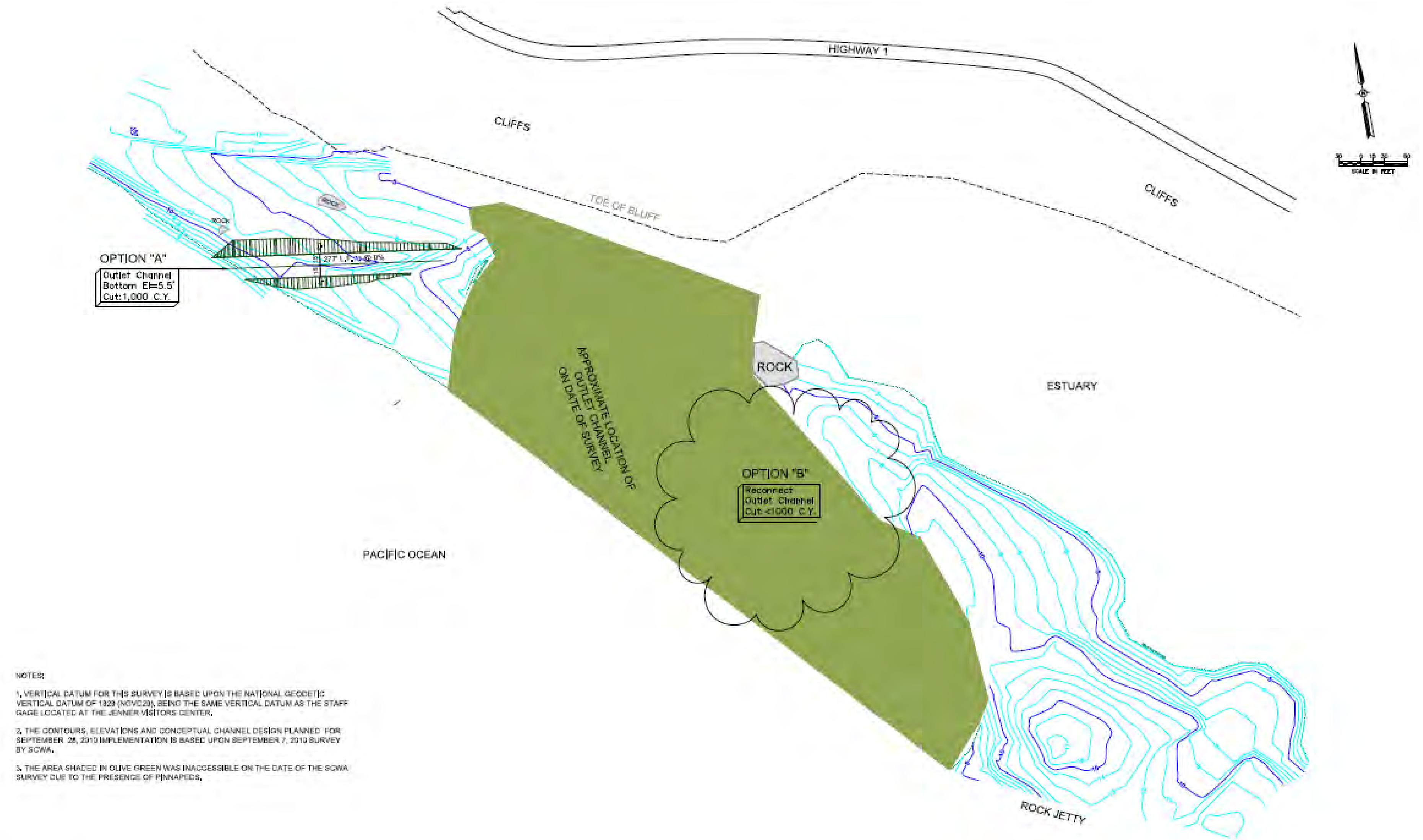


Source: SCWA

figure 8

Russian River Outlet Channel Adaptive Management Plan

Beach Topography and Management Options, September 2010

PWA Ref# 1958.01

ESA PWA

Attachment F. Physical Processes During the 2011 Management Period

As required by the Russian River Biological Opinion, Sonoma County Water Agency (Water Agency) has been tasked with managing a summer lagoon intended to improve salmonid habitat in the Russian River Estuary by creating an outlet channel while maintaining the current level of flood protection for properties adjacent to the estuary (NMFS, 2008). The adaptive management plan, described in the main body of this report, was developed by the Water Agency with assistance from ESA PWA and the resource agency management team in 2009 and revised in 2010 and 2011. Because of permit constraints, the Water Agency was only able to implement the plan beginning in 2010. The revised plan was in effect for 2011, but no opportunities for management action occurred during the management period.

During the 2011 management period, May 15th to October 15th, Water Agency staff regularly monitored current and forecasted estuary water levels, inlet state, river discharge, tides, and wave conditions to anticipate changes to the inlet's state. High river discharge in the first two months of the management period followed by the typical low wave energy conditions during the summer contributed to the inlet staying open for the first four months of the management period. Starting in late September, the inlet went through a succession of perched lagoon conditions and natural breaches, during which the Water Agency closely monitored estuary conditions and considered management options. The perched episodes were short-lived, lasting no more than a week, and included a small outlet channel flowing along and sometimes through gaps in the jetty. The perched episodes ended naturally when lagoon water levels increased, overtopped the beach berm, and scoured a new tidal channel. Since the perched lagoon episodes did not evolve to the point that management action was warranted, the Water Agency did not take any management actions to encourage formation of an outlet channel.

Even though no management actions were implemented to inform the adaptive management process, the physical conditions and inlet response during the management period are reviewed in this attachment to contribute to site understanding and to inform future management actions.

METHODOLOGY

This review of the 2011 outlet channel management period examined water levels, ocean wave conditions, ocean water levels, riverine discharge, beach topography, as well as inlet size and location. The sources for these parameters are listed in Table 1. These data were supplemented with personal observations and discussion with staff from the Water Agency, NMFS, DFG, and the Bodega Marine Laboratory.

Table 1. Data Sources

Parameter	Source
Estuary water level (h_E)	Water Agency Jenner gage*
Wave height (H_s), period (T_a), and direction	CDIP Point Reyes buoy #029
Ocean water level (h_O)	NOAA Point Reyes #9415020
Russian River discharge (Q_f)	USGS Guerneville #11467000
Beach topography, ft NGVD	Water Agency monthly surveys
Inlet size and location	Water Agency and Bodega Marine Laboratory autonomous cameras

*Gage failed near the end of July, and was replaced by early September.

INLET STABILITY PARAMETER AND CLOSURE RISK PROBABILITY

In addition to considering individual parameters, researchers at the Bodega Marine Laboratory have developed a combined parameter to evaluate the stability of the inlet's state, with the aim of predicting closure risk. (Note that the inlet stability parameter does not differentiate between full closure and the perched conditions with a small outlet channel that formed in fall 2010. When discussing this parameter, both states are referred to as a 'closure'.) The inlet stability parameter presented by Behrens et al. (in publication) quantifies the risk of inlet closure based on a sediment balance in the inlet. It considers the daily balance between wave-driven sediment import to the inlet and sediment export driven by tidal fluctuations. The former is estimated from wave measurements and the latter is estimated from tide gage data within the estuary and a stage-storage relation derived from the available bathymetry. Using daily-average values of the stability parameter within the period 1999-2008, Behrens et al. (in publication) showed that high-percentile values of the parameter are closely linked to the risk of the inlet closing within five days. As the percentile of the stability parameter increases, the risk of inlet closure within five days increases exponentially, from risks of roughly five percent when the parameter is at the 50th percentile to a risk of 80 percent when it is measured at the 99th percentile.

FALL PERCHED EPISODES AND NATURAL BREACHES

Time series of estuary water levels, as well as the key forcing factors (waves, tides, and riverine discharge), are shown in Figure 1 for the entire management period. Prior to September, no inlet closures occurred, so lagoon water levels fluctuated in concert with ocean tides (Figure 1a). As shown in Figure 1d, discharge remained high for the first two months of the management period as a result of a wet spring, including precipitation in the start of June. River discharge did not drop below 400 ft^3/s until after June 15th and below 200 ft^3/s until after July 15th. This elevated discharge probably reduced the likelihood of inlet closure during the first two months of the management season even though some sizeable wave events occurred during these months (Figure 1b). In late July and particularly in August, wave energy was at the annual minimum, so tidal exchange was sufficient to maintain an open inlet. As typically occurs on the California coast, wave energy increased starting in September, which eventually caused the estuary to perch six times, starting in late September and into November.

All six inlet perched lagoon episodes in fall 2011 lasted a week or less, ending when the estuary water levels reached 4-5 ft NGVD, overtopped the beach berm, and scoured a new tidal channel. Conditions during the perched lagoon episodes (September 22-29, October 3-8, October 10-14, November 3-8, November 10-12, and November 17-20) are shown in Figure 2. Although the management period ends on October 15th, conditions up through the end of November were reviewed since they were consistent with the inlet behavior that started in late September. Six instances of perched lagoon conditions are slightly higher than the average number of closures, 4.6, in September through November (ESA, 2011). However, a series of repeated perched episodes and natural breaching is not common; since 1996, this pattern has only been observed only one other time, in 2006.

Consistent with the existing conceptual model described in Section 4 of the Management Plan, perched lagoon conditions typically occurred when both wave energy increased and tidal exchange decreased. All perched episodes occurred when the mean wave period was greater than 10 seconds and five perched episodes occurred when significant wave heights were greater than 12 ft. The October 10th episode coincided with wave heights of only 8 ft, but since these waves had long, 16-second periods and originated from the southwest, they still conveyed significant wave energy to the beach. Five of the 2011 episodes occurred during neap tides when the tide range was reduced to less than 5 ft (Figure 2c). When the tide range is less, tidal scour in the inlet is also less, making the inlet more susceptible to infill with sand. Only the November 10-12 episode occurred when the oceanic tide range was greater than 6 ft. All but the first episode occurred with riverine discharge elevated above 250 ft^3/s and the three November episodes occurred when riverine discharge was approximately 400 ft^3/s.

PERCHED LAGOON AND NATURAL BREACH DYNAMICS

As an example of a perched lagoon-breach cycle, Figure 3 shows a sequence of photos of the inlet before, during, and after the October 3-8 episode. As was the case for almost all of the management period, the inlet was located next to the jetty. Shortly before the episode, on September 30 (Figure 3a), the inlet had narrowed in width to approximately 30 feet.

The estuarine water level became muted starting on October 3 with the arrival of some larger, longer-period waves (Figure 2a and b). By October 5, a tidal signal was absent from the estuary and water levels began to rise. The inlet transformed into a small outlet channel running immediately adjacent to and among the rocks at the toe of the jetty (Figure 3b; Figure 4a). The outlet channel was narrow, with a width of approximately ten feet. When the channel reached the portion of the jetty which had been damaged, the channel turned south and flowed through the gap in the jetty (Figure 4b).

The jetty and rocks which had been a part of the jetty may have stabilized the outlet channel, both in sheltering the outlet channel from waves and by providing bank and bed stabilization that minimized channel scour. Sheltering by the jetty probably reduced berm build-up at the inlet's location, leaving a low point in the beach berm that was the site for subsequent overtopping and natural breaching. This small outlet channel, present from the start of the episode, contrasts with other historic closures that were more extensive. For these extensive closures, almost the entire inlet was filled with sand, with only a small indentation on the backside of the berm providing any indication of the inlet's prior location, and no outlet channel was present. All the 2011 episodes were less extensive, which left the beach berm more susceptible to natural breaching.

Natural breaching probably occurred when the estuary water level had risen sufficiently high that it overtopped the beach berm in the vicinity of the outlet channel. This overtopping increased the flow rate through the outlet channel and, in spite of any bank stabilization provided by the jetty and associated rocks, the increased flow rate scoured sand from the channel bed and banks. The enlarged channel was then sufficiently deep to allow tides and salt water to return to the estuary. Shortly after natural breaching, the tidal channel was approximately 50 feet wide (Figure 3c), wider than it had been in the days preceding the episode. This channel enlargement is consistent with the natural breaching mechanism as the higher flow, induced by the elevated estuary water levels during episode, scoured the channel.

CLOSURE RISK PROBABILITY

The 5-day closure risk probability, a derivative of the inlet stability parameter described above, was hindcast for 2011 according to the method described in Behrens et al. (in publication). This hindcast provides an indication of the utility of the stability parameter as a prediction tool for monitoring inlet conditions and planning management action. This parameter integrates wave and ocean forcing conditions, as well as estuary water levels, to provide greater predictive skill than just waves or ocean tides on their own. The stability parameter combines these factors, and the corresponding five-day closure risk time series exceeded 50 percent before each 2011 event (Figure 2a). Some 2011 episodes occurred quickly, transitioning from fully tidal to perched lagoon within a day, so the risk time series did not provide much forewarning in these cases. However the risk was elevated more than two days before the episodes on September 22, November 3, and November 17.

TOPOGRAPHIC CHANGE

The Water Agency has conducted monthly surveys of Goat Rock State Beach that cover a region starting from the jetty and extending approximately 1,500 feet to the north. Typically, the surveys do not include bathymetry within the inlet because flow conditions in the inlet prevent safe access. Also, the survey extent is often limited by the Water Agency's compliance with its marine mammal incidental harassment authorization, which prohibits the survey crew from disturbing the marine mammals hauled out on the beach. Water Agency survey staff collected spot elevations using RTK-GPS and then assembled these elevations into a set of contour lines at 1 ft intervals. The survey elevations are reported in the NGVD29 vertical datum, the working datum for estuary monitoring and management.

To characterize beach berm topographic conditions, ESA PWA assessed data from the Water Agency's 2010 (July to September) and 2011 (May to October) surveys. The locations of five transects selected for analysis are shown in Figure 5. The locations include two transects backed by cliff (Figure 6 and Figure 7), two transects which extend into the estuary (Figure 8 and Figure 9), and a transect just north of the jetty (Figure 10).

This review focuses on the 2011 surveys when the surveys captured a clearer picture of beach evolution. However, the 2010 surveys are included in the transect plots for context. In general the crest elevations in 2010 were lower than 2011. The cause of the lower crest elevations is not known, but may the result of inter-annual variations in wave energy and littoral sediment supply. In addition, the inlet exhibited greater variation in its location in 2010, extending far to the north in July before moving south later by August. As the inlet opened and closed or changed location, it resulted in large changes in beach topography. For example, at Transect 4, the inlet's closure in early July 2010 is readily apparent as substantial increase in the berm's size between the 7/1/2010 and 7/8/2010 transect (Figure 6). The inlet's migration south is evident at Transect 3 (Figure 7) when the crest elevation drops from its 7/8/2010 profile to less than 4 ft NGVD on 8/3/2010. The inlet migration and gaps in the survey data yield little information for evaluating crest elevation evolution at most transects. However, there is sufficient data at Transect 4 to show a trend of increasing crest elevation during summer 2010.

The crest elevations of Transects 2, 3, and 4 steadily increased over the 2011 management period. This trend is consistent with seasonal patterns on many California beaches. After some initial increase from May to June, when wave energy was at the annual minimum in July and August, transect changes were minimal. Then berm building accelerated in the fall with the concurrent increase in wave energy (Figure 1), as indicated by the change between the August 15th survey and the September 19th survey. The largest change occurred between the September and October surveys, the period that also experienced the largest wave energy. Over the course of the management period, the crest moved landward at Transect 3 and Transect 4, with the exception of the October survey, when the crest moved seaward at Transect 3. This landward movement is opposite to the typical crest movement at other California beaches (Weigel, 1992) and may be indicative of additional processes affecting these transects, such as supply-limited alongshore transport. At Transects 1 and 2, the crest moved seaward as it built upwards, consistent with typical summer-time response.

Transect 0, which is located just north of and parallel to the jetty, had noticeably different elevations and evolution than the other transects. Compared to the other transects, crest elevations were highest at this transect for both 2010 and 2011. In addition, Transect 0 did not evolve during the management periods, as was observed at the other transects. The only significant change occurred during the winter between the 2010 and 2011 management periods. These two characteristics, the higher crest and lack of management period variability, suggest that the jetty shelters this portion of the beach from small to moderate waves that occur during the management period. Only the larger waves associated with winter storms may be sufficient to re-shape the beach berm near the jetty.

The changes to the beach berm at Transect 1 were intermediate between the monthly changes that occurred to the north (Transects 2-4) and the negligible change in berm elevation adjacent to the jetty (Transect 0). Crest elevations at Transect 1 only increased between the September and October survey, the portion of the management period with the strongest wave energy. This suggests that the jetty may alter wave conditions over some distance from its location: Transect 1 is approximately 200 ft north of the jetty and outside of the area occupied by the inlet during most of the 2011 management period.

LESSONS LEARNED AND RECOMMENDATIONS

Based on observations of the estuary, associated physical processes, and the Water Agency's planning for outlet channel management, we note the following lessons about implementing the outlet channel management plan.

CONCEPTUAL MODEL

- Elevated discharge in the late spring and early summer (greater than 400 ft^3/s until June 15th; greater than 200 ft^3/s until July 15th) reduced the likelihood for inlet closure at that time. However, multiple perched lagoon episodes occurred in the fall when riverine discharged exceeded 250 ft^3/s. This is consistent with Behrens et al. (in publication) that although discharge affects probability of closure, the threshold that prevents closure is likely in excess of 2,000 ft^3/s. A likely contributing factor to the fall perched episodes was the higher wave energy.
- The inlet moved south early in the management period, reaching the jetty in late May or early June, and remained there throughout the 2011 management period and the following winter. This inlet alignment is not common, but has been observed in past years (Behrens et al., 2009).
- During the management period, steady growth of the beach berm was observed north of the jetty, consistent with typical beach berm building that occurs during the summer. However, the rate of berm growth appeared to decrease approximately 200 ft north of the jetty and was negligible immediately adjacent to the jetty.
- Although autumn wave events were large enough to create perched lagoon conditions, the beach berm remained at low elevations, approximately 5 ft NGVD. The inlet then naturally breached when rising estuary water levels overtopped the berm at this low point and scoured a new tidal channel.

OUTLET CHANNEL FEASIBILITY

- The jetty may shelter the inlet, making closure less likely and also limiting berm growth, which then maintains a low point for natural breaching. When the lagoon breaches naturally, management actions cannot be implemented.
- Even if the inlet being near the jetty hinders formation of sustained lagoon and outlet channel conditions, management opportunities for re-locating the outlet channel are limited and constrained. At a minimum, creating an outlet channel further north from the jetty requires a full natural closure, absence of a low point in the beach berm near the jetty, and equipment access to the area north of the jetty.
- A small outlet channel formed during the fall perched lagoon episodes. However, it did not convey enough discharge to prevent lagoon water levels from rising at 0.8 ft/day.

- The outlet channel that formed during the perched lagoon episodes flowed along the jetty and among the disaggregated rock at the damaged end of the jetty. This rock from the jetty may have provided channel stabilization for the outlet channel, increasing the channel's resilience to scour.
- Once outlet channel discharge increased due to rising lagoon water levels, the discharge scoured a new channel, breaching the estuary to the tides. This behavior highlights the susceptibility of a sand bed outlet channel to scour, limiting conveyance capacity.
- The mere occurrence of a perched lagoon is not sufficient to provide an opportunity for outlet channel management; other factors may not permit management action. This point is highlighted by both the 2011 natural breachings and the early fall closures in 2010, when continuing ocean swell precluded outlet channel management action. Over the first two years of effort to implement the outlet channel adaptive management plan, only one closure (July 2010), has been suited for outlet channel management action.

OPERATIONS

- When equipment operators visited the beach to plan a possible management action, they noted that the channel had incised a steep bank in the berm adjacent to the jetty (Figure 11), which would have made equipment access to any areas north of the jetty infeasible.

COMMUNICATIONS

- Although the perched lagoon episodes did not evolve to the point that management action was warranted, the Water Agency began planning management actions as soon as the episodes occurred. Planning included heightened observations of inlet conditions by Water Agency staff, email updates to inform the resource management group, and pre-implementation meetings at the project site to refine plans for management action.

MONITORING

- The Water Agency's upgrades to monitoring the estuary (water levels and photographs available in real-time via the Internet) enhance both management planning and the ability to observe inlet processes.

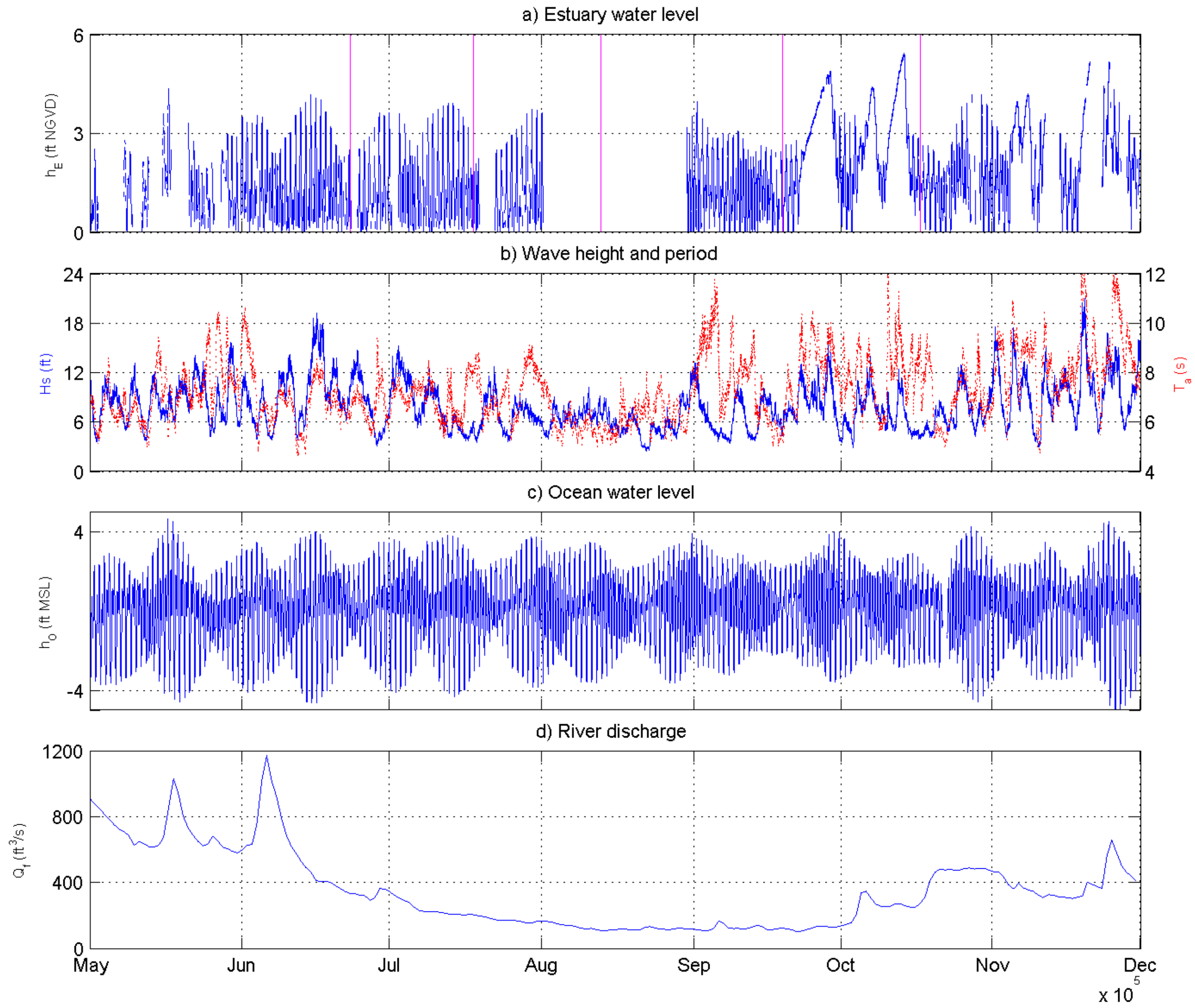
REFERENCES

Behrens, D., Bombardelli, F., Largier, J. and E. Twohy. 2009. Characterization of time and spatial scales of a migrating rivermouth. Geophysical Research Letters. Vol. 36, L09402, doi:10.1029/2008GL037025.

Behrens, D., Bombardelli, F., Largier, J. and Twohy, E. In publication. Episodic closure of the tidal inlet at the mouth of the Russian River – a small bar-built estuary in California. Geomorphology.

ESA. 2011. Russian River Estuary Management Project Environmental Impact Report. Prepared for Sonoma County Water Agency.

Weigel, R. 1992. Oceanographical Engineering.



SOURCE:

a) h_E=estuary water level (SCWA); pink bar = beach survey

b) H_s=sig. wave height; T_a=avg. wave period (CDIP, Pt. Reyes, #029)

c) h_O=ocean water level (NOAA, Pt. Reyes #9415020)

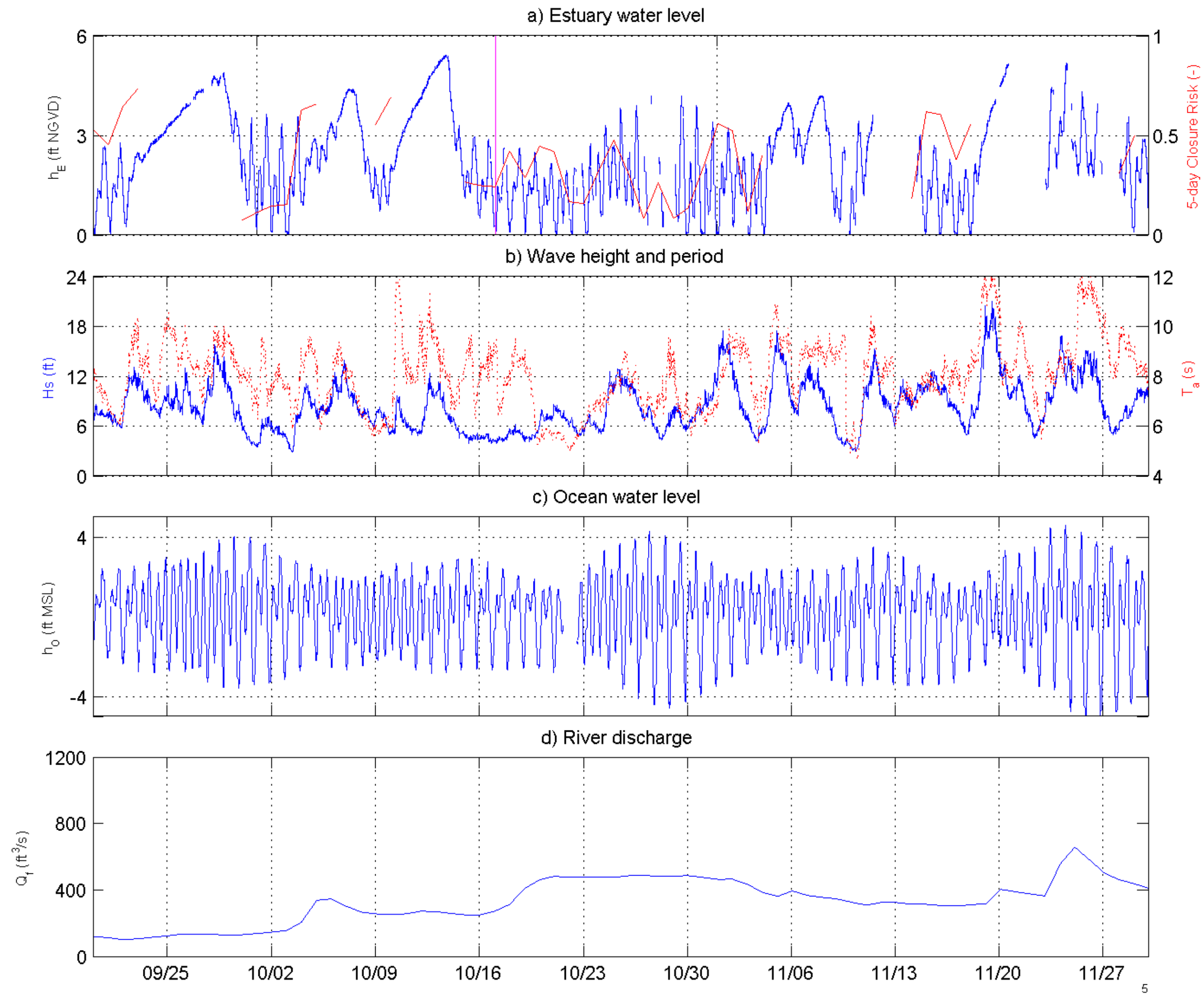
d) Q_f=river discharge (USGS, Guerneville #11467000)

Russian River Estuary Outlet Channel Management Plan . DW01958

Figure 1

Estuary, Ocean, and River Conditions

May - November 2011



SOURCE:

a) h_E=estuary water level (SCWA); pink bar = beach survey

b) H_s=sig. wave height; T_a=avg. wave period (CDIP, Pt. Reyes, #029)

c) h_O=ocean water level (NOAA, Pt. Reyes #9415020)

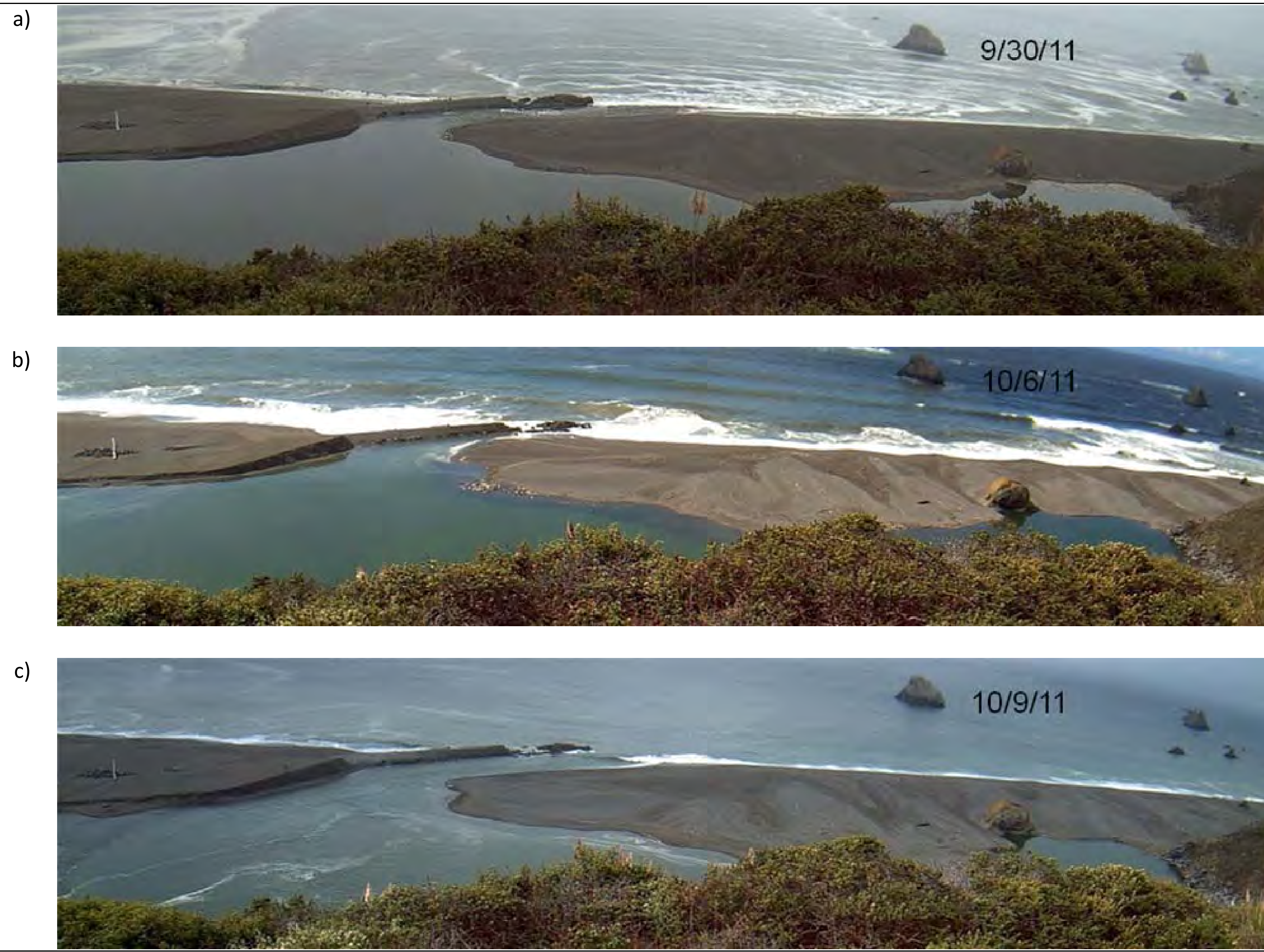
d) Q_f=river discharge (USGS, Guerneville #11467000)

Russian River Estuary Outlet Channel Management Plan . DW01958

Figure 2

Estuary, Ocean, and River Conditions

September - November 2011



SOURCE: Bodega Marine Lab

Figure 3

Inlet State, September 30, October 6, and October 9, 2011 a)

b)

SOURCE: Sonoma County Water Agency

Figure 4

Outlet Channel Along and Through Jetty, September 26, 2011

Source: Sonoma County Water Agency survey data

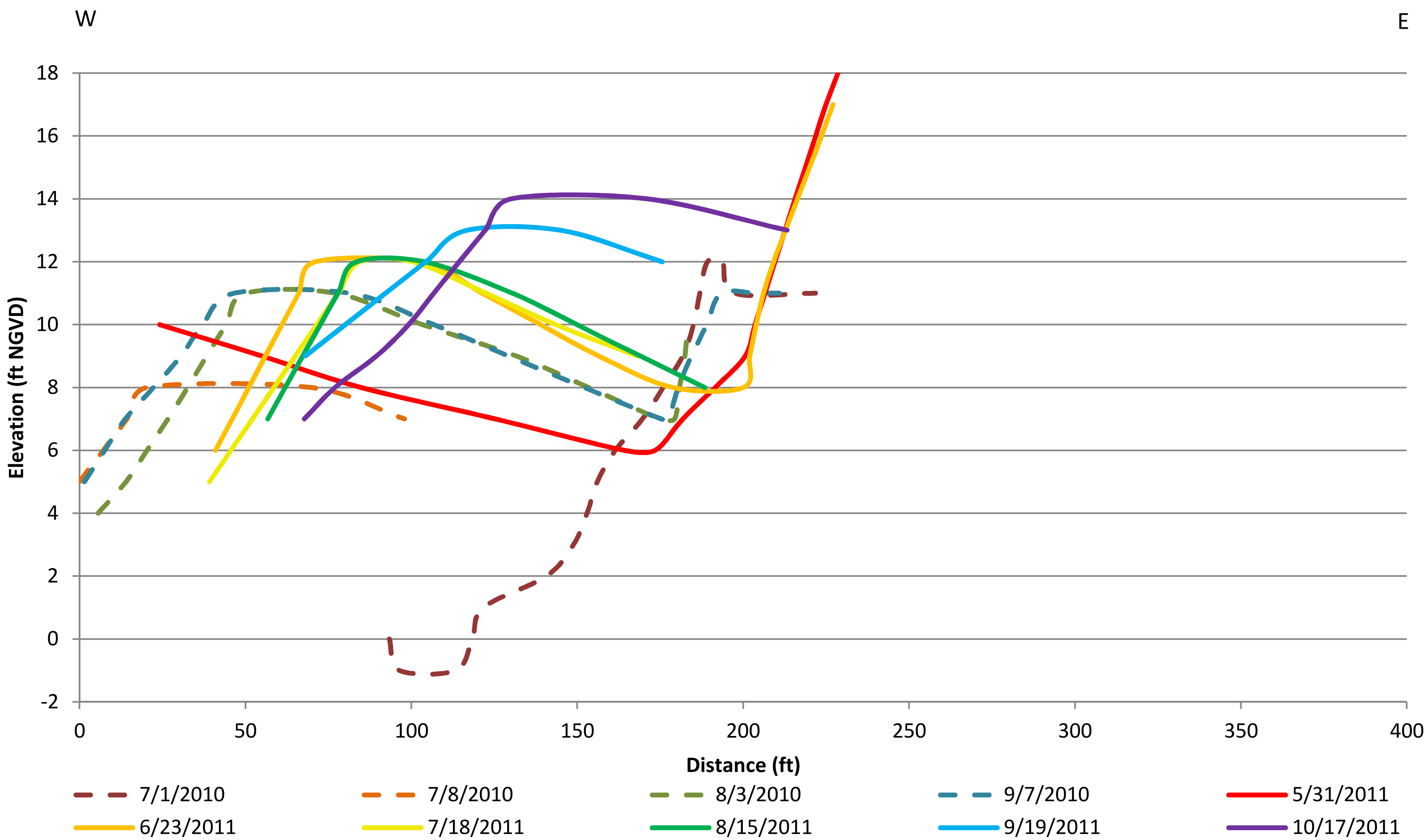
N

figure 5
Russian River Outlet
Channel Management Plan

Beach Topography Transect Locations

ESA PWA Ref# - DW01958

Beach_topo.mxd



Source: Sonoma County Water Agency survey data

figure **6**

Russian River Outlet Channel Management Plan

Beach Transect 4

ESA PWA Ref #: DW01958

ESA PWA

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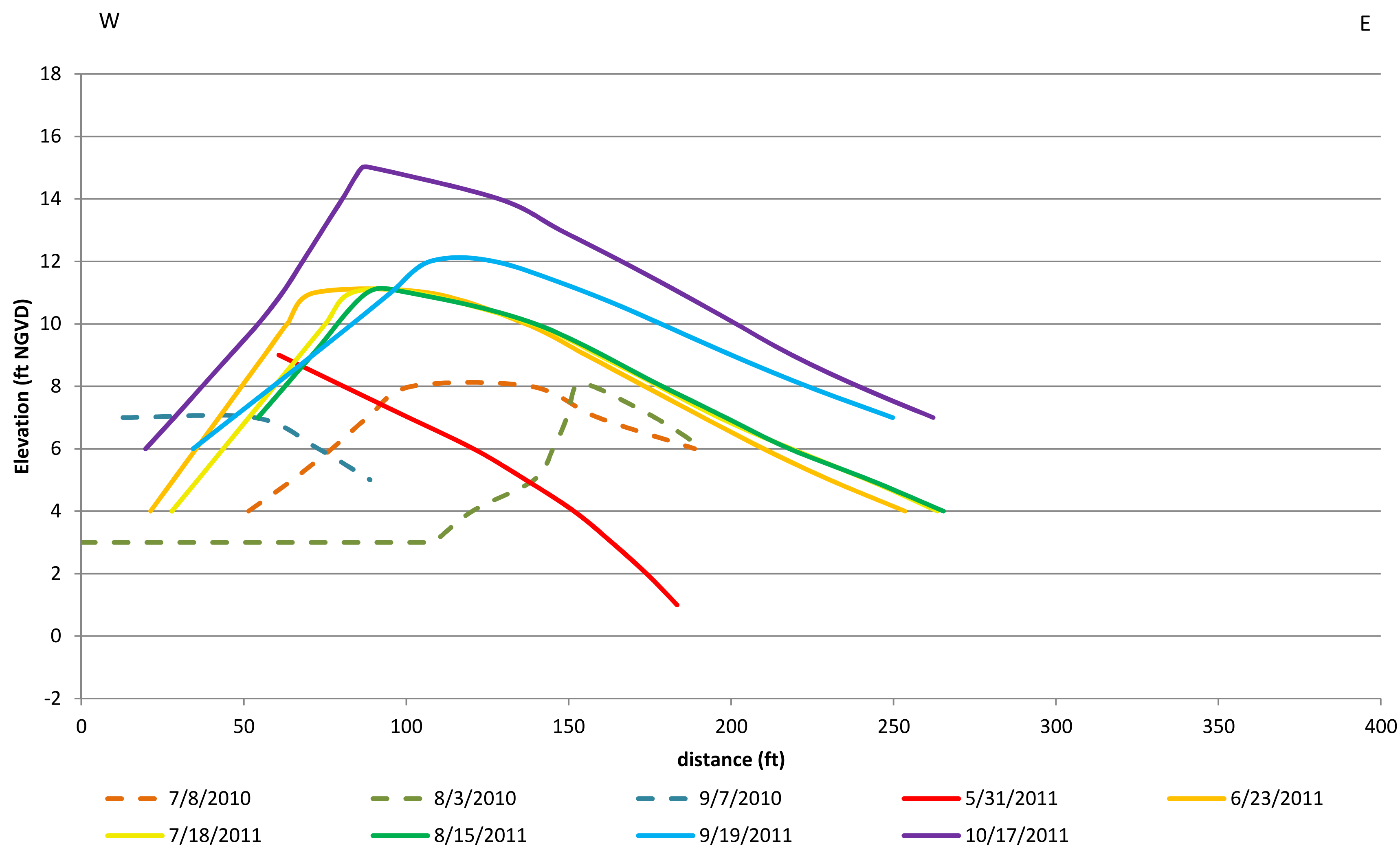

Source: Sonoma County Water Agency survey data

figure ***7***

Russian River Outlet Channel Management Plan

Beach Transect 3

ESA PWA Ref #: DW01958

ESA PWA

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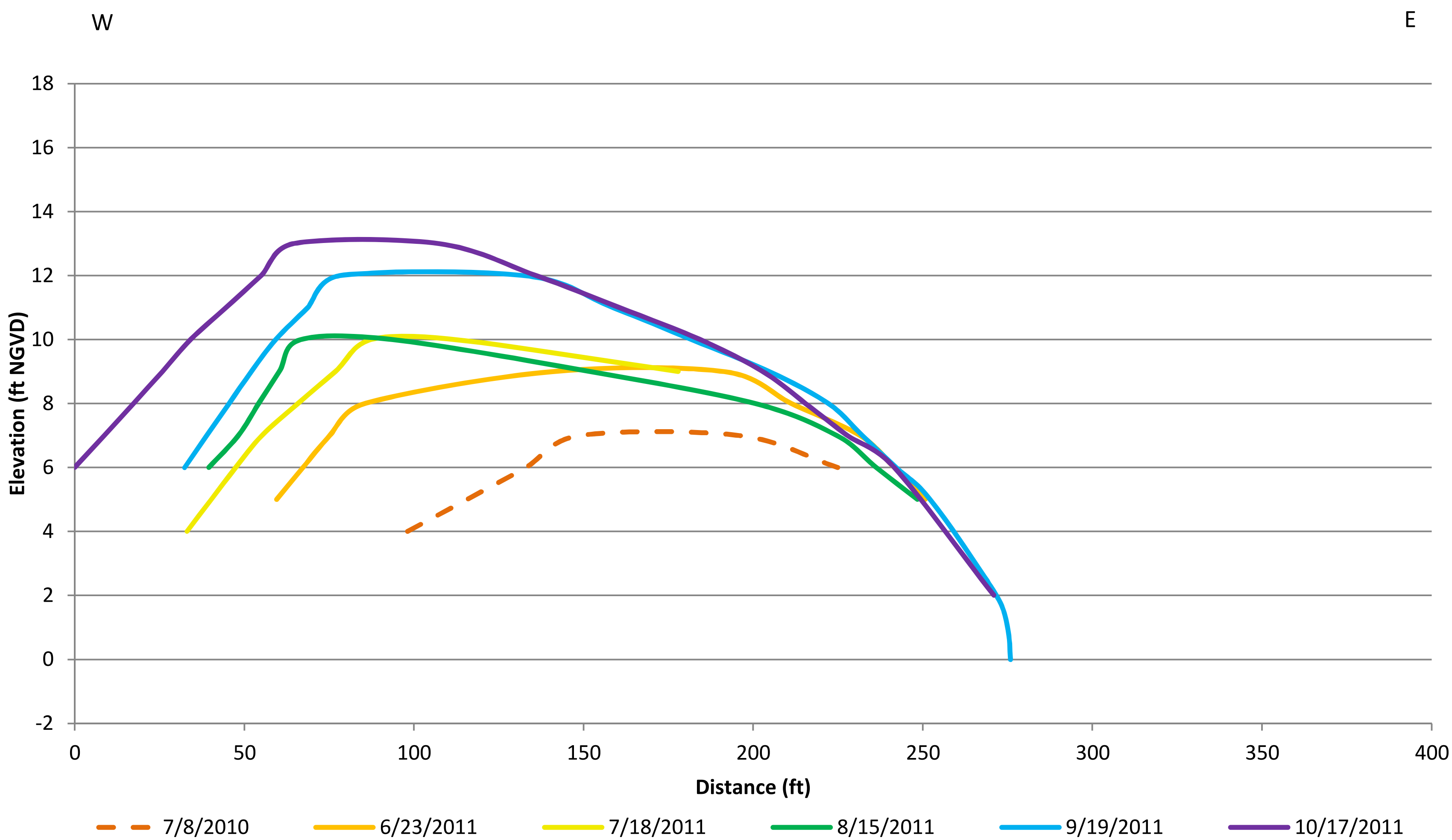

Source: Sonoma County Water Agency survey data

figure **8**

Russian River Outlet Channel Management Plan

Beach Transect 2

ESA PWA Ref #: DW01958

ESA PWA

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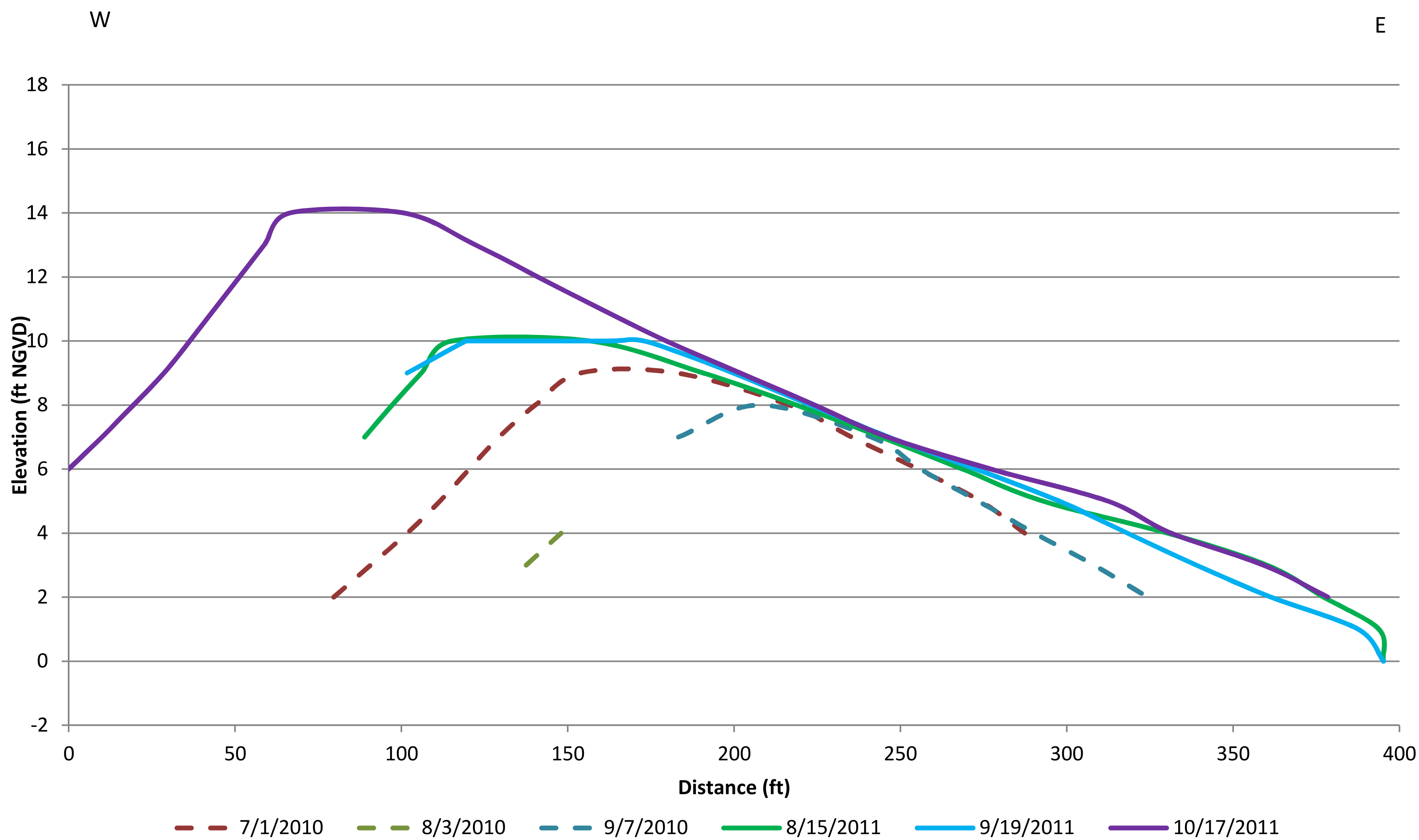


Source: Sonoma County Water Agency survey data

figure **9**

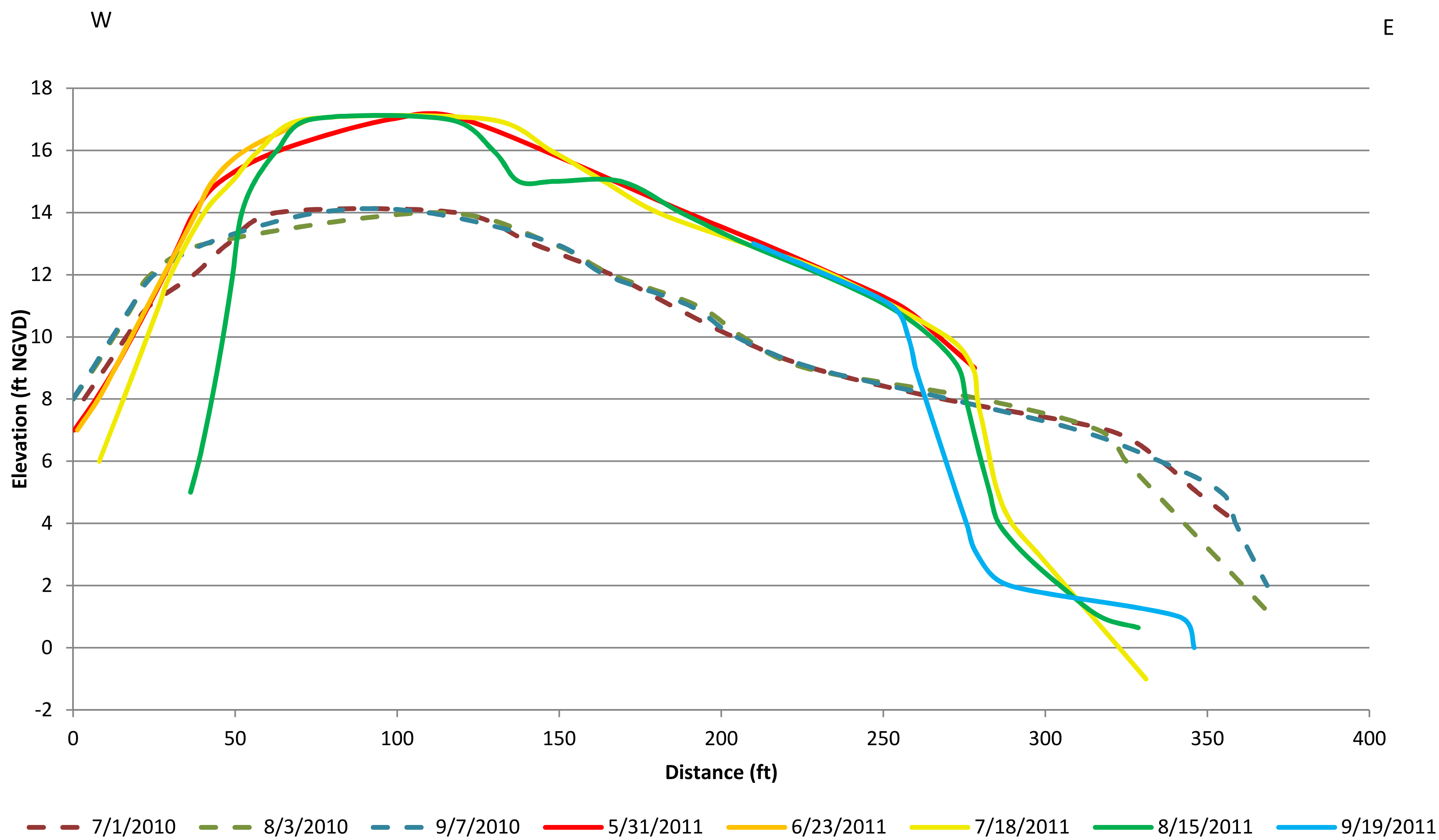
Russian River Outlet Channel Management Plan

Beach Transect 1

ESA PWA Ref #: DW01958

ESA PWA

Bathy_transects.xlsx / Transect1



Source: Sonoma County Water Agency survey data

figure ***10***

Russian River Outlet Channel Management Plan

Beach Transect 0

ESA PWA Ref #: DW01958

SOURCE: Sonoma County Water Agency

Figure 11
Steep Berm Limiting Beach Access, September 26, 2011

Attachment G. Physical Processes During the 2012 Management Period

As required by the Russian River Biological Opinion, Sonoma County Water Agency (Water Agency) has been tasked with managing a summer lagoon intended to improve salmonid habitat in the Russian River Estuary by creating an outlet channel while maintaining the current level of flood protection for properties adjacent to the estuary (NMFS, 2008). The adaptive management plan, described in the main body of this report, was developed by the Water Agency with assistance from ESA PWA and the resource agency management team in 2009 and revised annually in 2010-2013. Because of permit constraints, the Water Agency was only able to implement the plan beginning in 2010. The revised plan was in effect for 2012, but no opportunities for management action occurred during the management period.

During the 2012 management period, May 15th to October 15th, Water Agency staff regularly monitored current and forecasted estuary water levels, inlet state, river discharge, tides, and wave conditions to anticipate changes to the inlet's state. Although the inlet experienced several closures, none resulted in water levels above 5.5 ft NGVD prior to self-breaching. For much of June and July, the inlet was either closed or only allowing heavily muted tides (tide range < 1 ft), but the lagoon water surface never surpassed 5 ft NGVD. During this time, each closure ended when lagoon water levels increased, overtopped the beach berm, and scoured a new tidal channel. Since these episodes did not evolve to the point that management action was warranted, the Water Agency did not take any management actions to encourage formation of an outlet channel. For the remainder of July, all of August, and the first half of September, the estuary was fully tidal. Then the inlet closed twice between September 20th and October 10th. Both closures were short-lived, lasting less than one week, and again the inlet self-breached, precluding any Water Agency management action. The highest lagoon water level of the 2012 management period, 5.25 ft NGVD, occurred at the end of the October closure.

Even though no management actions were implemented to inform the adaptive management process, the physical conditions and inlet response during the management period are reviewed in this attachment to contribute to site understanding and to inform future management actions.

METHODOLOGY

This review of the 2012 outlet channel management period examined water levels, ocean wave conditions, ocean water levels, riverine discharge, beach topography, as well as inlet size and location. The sources for these parameters are listed in Table 1. These data were supplemented with personal observations and discussion with staff from the Water Agency, NMFS, DFG, and the Bodega Marine Laboratory.

Table 1. Data Sources

Parameter	Source
Estuary water level (h_E)	Water Agency Jenner gage*
Wave height (H_s), period (T_a), and direction	CDIP Point Reyes buoy #029
Ocean water level (h_O)	NOAA Point Reyes #9415020
Russian River discharge (Q_f)	USGS Guerneville #11467000
Beach topography, ft NGVD	Water Agency monthly surveys
Inlet size and location	Water Agency and Bodega Marine Laboratory autonomous cameras

*Data transmission failure due to cellular network issues occurred for several 1-5 day periods throughout the management period.

INLET STABILITY PARAMETER AND CLOSURE RISK PROBABILITY

In addition to considering individual parameters, researchers at the Bodega Marine Laboratory have developed a combined parameter to evaluate the stability of the inlet's state, with the aim of predicting closure risk (Behrens et al., 2013). (Note that the inlet stability parameter does not differentiate between full closure and the perched conditions with a small outlet channel. When discussing this parameter, both states are referred to as a 'closure' in that tides are prevented from propagating into the estuary.) The inlet stability parameter presented by Behrens et al. (2013) quantifies the risk of inlet closure based on a sediment balance in the inlet. It considers the daily balance between wave-driven sediment import to the inlet and sediment export driven by tidal fluctuations. The wave-driven import is assessed using nearshore wave estimates derived from a transformation matrix and offshore buoy data (ESA PWA 2012) and the latter is estimated from tide gage data within the estuary and a stage-storage relation derived from the available bathymetry. Using daily-average values of the stability parameter within the period 1999-2008, Behrens et al. (2013) showed that high-percentile values of the parameter are closely linked to the risk of the inlet closing within five days. As the percentile of the stability parameter increases, the risk of inlet closure within five days increases exponentially, from risks of roughly five percent when the parameter is at the 50th percentile to a risk of 80 percent when it is measured at the 99th percentile.

SUMMER AND FALL CLOSURES AND SELF-BREACHES

Time series of estuary water levels, as well as the key forcing factors (waves, tides, and riverine discharge), are shown in Figure 1 for the entire management period. The lagoon water level time series (Figure 1a) summarizes the observed muted conditions in early summer and short-lived closure events that occurred at the end of the management period. As shown in Figure 1d, discharge remained high for the first two months of the management period. River discharge did not drop below 200 ft^3/s until after June 10th, at which time the estuary had already begun its muted tidal phase, leading up to four short-lived closures. This elevated discharge probably reduced the likelihood of inlet closure during the first 30-40 days of the management period (Figure 1d), despite the occurrence of energetic wave conditions in May (Figure 1b). Wave energy reached a minimum in August and early September, but was weaker throughout the 2012 management period than in 2011. The hourly significant wave height was less than 8 ft for the majority of this period.

The conditions leading to inlet closure were consistent with the existing conceptual model described in Section 4 of the Management Plan. All closure events coincided with either moderately high waves ($H_s > 6$ ft) having periods greater than 10 s, or with neap oceanic tide ranges of less than approximately 5 ft. Moderately high waves coincided with the closure events in June, July, September and October. The first closure observed in June and both July closures coincided with neap tide conditions, although long-period swells occurred prior to the former of the two. Closure events that occurred in June and July are examined in more detail in Figure 2, while Figure 3 summarizes conditions that occurred later in September-November.

All closure events occurred with the inlet located adjacent to the jetty. This positioning may have prevented perched conditions from arising by shielding this area of the beach from the wave-driven sediment deposition that caused closure, preventing the beach from accreting to a sufficient height to allow the desired outlet channel elevations from being attained. The low point in the beach berm that was subsequently overtopped and self-breached also persisted immediately adjacent to the jetty.

PERCHED LAGOON AND SELF-BREACH DYNAMICS

During the June and July closures (Figure 2), as well as the late September closure (Figure 3), the lagoon water level only increased at approximately 0.3 ft/day. This slower increase probably occurred because a small outlet channel that flowed over the beach berm and through a gap in the jetty partially balanced inflowing river discharge.

As an example of one of the several inlet closure events that resulted in self-breaching prior to target outlet channel elevations, Figure 4 shows a sequence of photos of the inlet before, during, and after an episode from October 8-15. As was the case for all of the management period, the inlet was located next to the jetty. Prior to closure, the inlet had allowed only muted tides, resulting from a partial breach on October 2nd that did not restore full tidal action. Neap oceanic tides compounded this, and 7-ft high nearshore waves having a dominant period above 20 seconds closed the inlet on October 8th (Figure 3b,c).

After the onset of closure, the estuary water levels began to rise. For the first two days of closure, the water level increased at approximately 0.5 ft/day from 3 to 4 ft NGVD, but this decreased to less than 0.3 ft/day afterwards (lagoon stage above 4 ft NGVD). Waves deposited sediment adjacent to the gap in the jetty structure, blocking outflows from the lagoon that had occurred in prior closures (Figure 4b). This partially-formed barrier berm was overtopped when the lagoon reached approximately 5.25 ft on October 15th (Figure 4c). The outlet channel was narrow, with a width of less than ten feet. This overtopping event coincided with a spring phase of the oceanic tides, which generated a large head difference between the estuary and ocean waters. This head difference presumably contributed to channel flow velocities exceeding the threshold for scouring the beach sand, since the spring lower-low tide on October 16th resulted in the small channel eroding the barrier and creating a new inlet (Figure 4d). After the initial breach, the increased flow rate scoured sand from the channel bed and banks, and the channel increased to more than 20 feet in width (Figure 4d).

The jetty and rocks which had been a part of the jetty appeared to have a significant influence on the geomorphic evolution of the channel. At times, the jetty elements may have stabilized the outlet channel, both in sheltering the outlet channel from waves and by providing bank and bed stabilization that minimized channel scour. Wave sheltering by the jetty probably reduced berm build-up at the inlet’s location, leaving a low point in the beach berm that was the site for subsequent overtopping and self-breaching. Of the six closure events that occurred within the management period, all experienced a similar breaching pattern, self-scouring a tidal inlet before estuary water levels reached 5.5 ft NGVD. This was also true of the two closure events which occurred in November, following the management period (Figure 3). At times, the outlet channel flowed through notch in the jetty (Figure 5), such that the rocks probably provided stabilization that prevented bed scour. The jetty also halted lateral scour to the south. However, once lateral scour is halted, the channel may then maintain its cross-sectional area by scouring downward where it runs parallel to the jetty.

CLOSURE RISK PROBABILITY

The 5-day closure risk probability, a derivative of the inlet stability parameter described above, was hindcast for 2012 according to the method described in Behrens et al. (2013). This hindcast provides an indication of the utility of the stability parameter as a prediction tool for monitoring inlet conditions and planning management action. This parameter integrates wave and ocean forcing conditions, as well as estuary water levels, to provide greater predictive skill than just waves or ocean tides on their own. The stability parameter combines these factors, and the corresponding five-day closure risk time series exceeded 50 percent before each 2012 event (Figure 1e, Figure 2e, and Figure 3e). The closure event initiated on July 1st occurred quickly, transitioning from fully tidal to fully closed within a day, so the risk time series did not provide much forewarning in this case. This was also true of two closure events occurring outside of the management period, in November 2012. However, for all other events observed from June to November, the predicted probability of closure exceeded 50% 2-5 days in advance of each closure. There were no instances during the management period when the predicted probability of closure exceeded 50% and a closure did not occur within 5 days.

TOPOGRAPHIC CHANGE

The Water Agency has conducted monthly surveys of Goat Rock State Beach that cover a region starting from the jetty and extending approximately 1,500 feet to the north. Typically, the surveys do not include bathymetry within the inlet because flow conditions in the inlet prevent safe access. Also, the survey extent can be limited by the Water Agency’s compliance with its marine mammal incidental harassment authorization, which sets guidelines for the survey crew’s approach to marine mammals hauled out on the beach. Water Agency survey staff collected spot elevations using RTK-GPS and then assembled these elevations into a set of contour lines at 1 ft intervals. The survey elevations are reported in the NGVD29 vertical datum, the working datum for estuary monitoring and management.

To characterize beach berm topographic conditions, ESA PWA assessed data from the Water Agency's 2010 (July to September), 2011 (May to October), and 2012 (May to October) surveys. Surveys from November 2011 to May 2012 were also compared, to assess winter-time changes of beach shape. Survey transects from the 2011 analysis were reused (Figure 6), and include two transects backed by cliff (Figure 7 and Figure 8), one transect which extends into the estuary (Figure 9), and two transects just north of the jetty (Figure 10).

This review focuses on the 2012 surveys, although the 2010 and 2011 surveys are included for context. Compared with both 2010 and 2011, the 2012 topographic data indicate that the beach berm was less variable in shape than in previous years. This is especially true of the northern two transects (Figures 7 and 8), and to a lesser extent at Transect 2 (Figure 9). Because of inlet and seal haulout locations, topographic data were not collected in the vicinity of Transect 1 in 2012, so this is not included in the analysis. Adjacent to the jetty groin, Transect 0 showed little monthly change in topography, but extensive inter-annual variability.

During the management period in 2012, the beach berm along transects 2, 3, and 4 showed little variability, changing by less than two feet. The profile along Transect 2 (Figure 9) showed a slight aggradation trend over the course of the management period, but at Transects 3 and 4, the change in shape fluctuated only slightly (Figures 7 and 8). In contrast, between May 2011 and October 2011, the beach berm at these transects built in size by more than 6 feet. The difference in monthly variability at the northern transects between the 2011 and 2012 management periods can likely be tied to the difference in the extent of inlet migration. In 2011, the inlet migrated north of Haystack Rock during the winter, and returned to the jetty in late spring or early summer. This migration resulted in a lower beach profile at all transects. Over the course of the management period, the beach gradually built up to a typical summer profile. Even during the peak winter and spring flows of 2012, the inlet never migrated north of Haystack Rock, leaving a largely-intact beach berm north of Haystack Rock and a lower terrace between Haystack Rock and the jetty groin. Since these northern transects started at a much higher elevation at the start of the management period, the vertical growth of the beach profiles at these locations were several feet less than during the previous year in the same locations.

Transect 0, which is located just north of and parallel to the jetty, had noticeably different elevations and evolution than the other transects during the 2012 management period. Compared to the other transects, crest elevations were highest at this transect for both 2010 and 2011. This was not the case in 2012, when the northernmost two transects were the highest. The crest elevation at Transect 0 did not evolve during the management periods in 2010 and 2011, but was observed to erode between August and October in 2012. Images from the BML stationary camera indicate that this was the result of the inlet shifting from a sinuous alignment (resulting from southward migration) to a straight alignment running nearly parallel to the jetty. The only significant changes occurred during the winter between each of the management periods. The lack of management period variability of this region suggests that the jetty shelters this portion of the beach from small to moderate waves that occur during the management period. Only the larger waves associated with winter storms may be sufficient to re-shape the beach berm near the jetty.

Water Agency surveys taken during the months preceding the 2012 management period (November 2011 to April 2012, Figure 11) show more variability in beach berm height and width than was observed for the 2012 management period (Figure 9). The highest beach crests observed during the 12-month period from November 2011 to October 2012 occurred in November and December 2011, peaking between 14 and 15 ft NAVD88 at Transect 2 (Figure 11). This is consistent with the combination of high-energy, long-period swell waves and generally low fluvial flows during the late fall. By the February 2012 survey, erosion significantly reduced the beach crest elevation. This erosion is likely due to fluvial flows through the inlet at Transect 2. Farther north, at Transect 3, there was less influence from the inlet, and there appeared to be less erosion during winter 2011-12 (Figure 12). The berm crest was highest in late spring (March and May profiles) and in November 2012, peaking between 16 and 17 ft NAVD88. The difference between the evolution of Transects 2 and 3 may be a result of the inlet's lack of migration in 2012, or possibly a difference in the amount of wave exposure between locations.

Water Agency surveys were also used to assess the beach width at Transect 3. We focus on Transect 3, because the influence of the inlet caused the beach to be consistently lower at other transects, sometimes as low as the intertidal zone, where survey data were not consistently collected. The Transect 3 beach width was as the horizontal distance between a particular elevation on the ocean and estuary sides of the beach face, respectively. From November 2011 to June 2012, the beach width at the 12 ft NAVD88 elevation varied from 110 to 145 feet, showing signs of both narrowing and widening during the winter and spring (Figure 13). From June to August 2012, the beach width grew steadily from about 110 ft to 145 ft and appeared to remain at this width though November 2012. At an elevation of 14 ft NAVD88, the width followed the same pattern, but had larger fluctuations, varying from roughly 30 to 110 ft and grew steadily from June 2012 onward. These observations underscore the typical pattern of beach building in summer, but also indicate that waves in winter can build the beach between destructive events.

LESSONS LEARNED AND RECOMMENDATIONS

Based on observations of the estuary, associated physical processes, and the Water Agency's planning for outlet channel management, we note the following lessons about implementing the outlet channel management plan.

CONCEPTUAL MODEL

- Elevated discharge in the late spring (greater than 200 ft^3/s until June 10th) may have reduced the likelihood for inlet closure in May, although the wave climate at this time was also significantly weaker than during the previous year.
- Several short-lived closure events occurred, but waves never built up the minimum crest height (the limiting height for closure) beyond 5.5 ft NGVD, and all events ended with

self-breaches below this elevation. This prevented management actions from being taken during the 2012 season.

- The inlet never migrated north of Haystack Rock during peak winter floods, and returned to the jetty in early spring, much earlier than in most years. This inlet alignment is not common, but has been observed in past years (Behrens et al., 2009).
- During the management period, most of the beach north of Haystack Rock underwent little topographic change. A transect adjacent to Haystack Rock aggraded slightly, consistent with typical beach berm building that occurs during the summer. Adjacent to the jetty, the berm did not aggrade, but rather remained largely unchanged for most of the season and then later eroded between August and October as a result of a shift in the inlet alignment.
- The wave climate remained weak throughout much of the summer and fall, which may have stunted the growth of the beach crest in the vicinity of the jetty (the location of the inlet throughout the 2012 season), preventing lagoon water levels from reaching levels conducive of the planned outlet channel.
- When an outlet channel is present, oceanic tide conditions can encourage scouring and formation of a new tidal inlet. During the spring phase of the tide, the lower-low tide creates a large head difference between the lagoon and ocean, likely increasing the flow velocity in the channel.

OUTLET CHANNEL FEASIBILITY

- The jetty may shelter the inlet, making closure less likely and also limiting berm growth, which then maintains a low point for self-breaching. When the inlet is in a fully or muted tidal condition, options for management become considerably more difficult to implement.
- An outlet channel that was intermittently observed during the 2012 closures conveyed a portion of the inflowing river discharge, slowing the rise in lagoon water levels to approximately 0.3 ft/day. This channel flowed through a gap in the jetty, whose large rocks likely provided some degree of channel stabilization against scour.
- Once outlet channel discharge increased due to rising lagoon water levels or low oceanic tides, the discharge scoured a new channel, breaching the estuary to the tides. This behavior highlights the susceptibility of a sand bed outlet channel to scour, limiting conveyance capacity.
- Even if the inlet being near the jetty hinders formation of sustained lagoon and outlet channel conditions, management opportunities for re-locating the outlet channel are limited and constrained. At a minimum, creating an outlet channel further north from the jetty requires a full natural closure, absence of a low point in the beach berm near the jetty, and equipment access to the area north of the jetty.
- Over the first three years of effort to implement the outlet channel adaptive management plan, only one closure (July 2010), has been suited for outlet channel management action.

COMMUNICATIONS

- Although the perched lagoon episodes did not evolve to the point that management action was warranted, the Water Agency began planning management actions as soon as the episodes occurred. Planning included heightened observations of inlet conditions by

Water Agency staff, email updates to inform the resource management group, and pre-implementation meetings at the project site to refine plans for management action.

MONITORING

- The Agency's month survey methods should be modified to collect specified contours, such as the beach berm ridge line, wetted edge (beach side), and water edge (estuary side).

REFERENCES

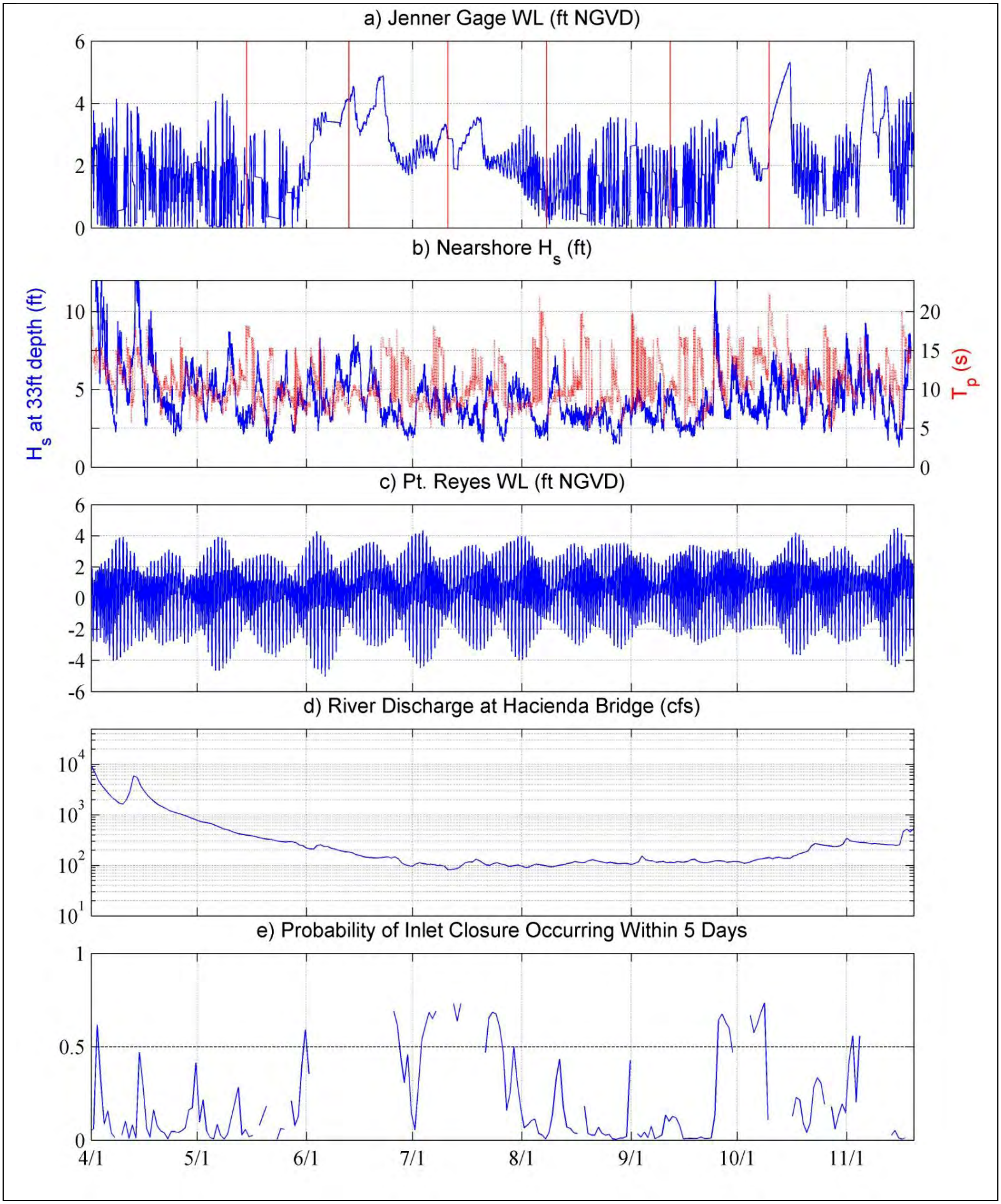
Behrens, D., Bombardelli, F., Largier, J. and E. Twohy. 2009. Characterization of time and spatial scales of a migrating rivermouth. Geophysical Research Letters. Vol. 36, L09402, doi:10.1029/2008GL037025.

Behrens, D., Bombardelli, F., Largier, J. and Twohy, E. 2013. Episodic closure of the tidal inlet at the mouth of the Russian River – a small bar-built estuary in California. Geomorphology.

ESA PWA. 2012. Feasibility of alternatives to the goat rock state beach jetty for managing lagoon water surface elevations: draft existing conditions report. Submitted to Sonoma County Water Agency.

National Marine Fisheries Service (NMFS). 2008. 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.

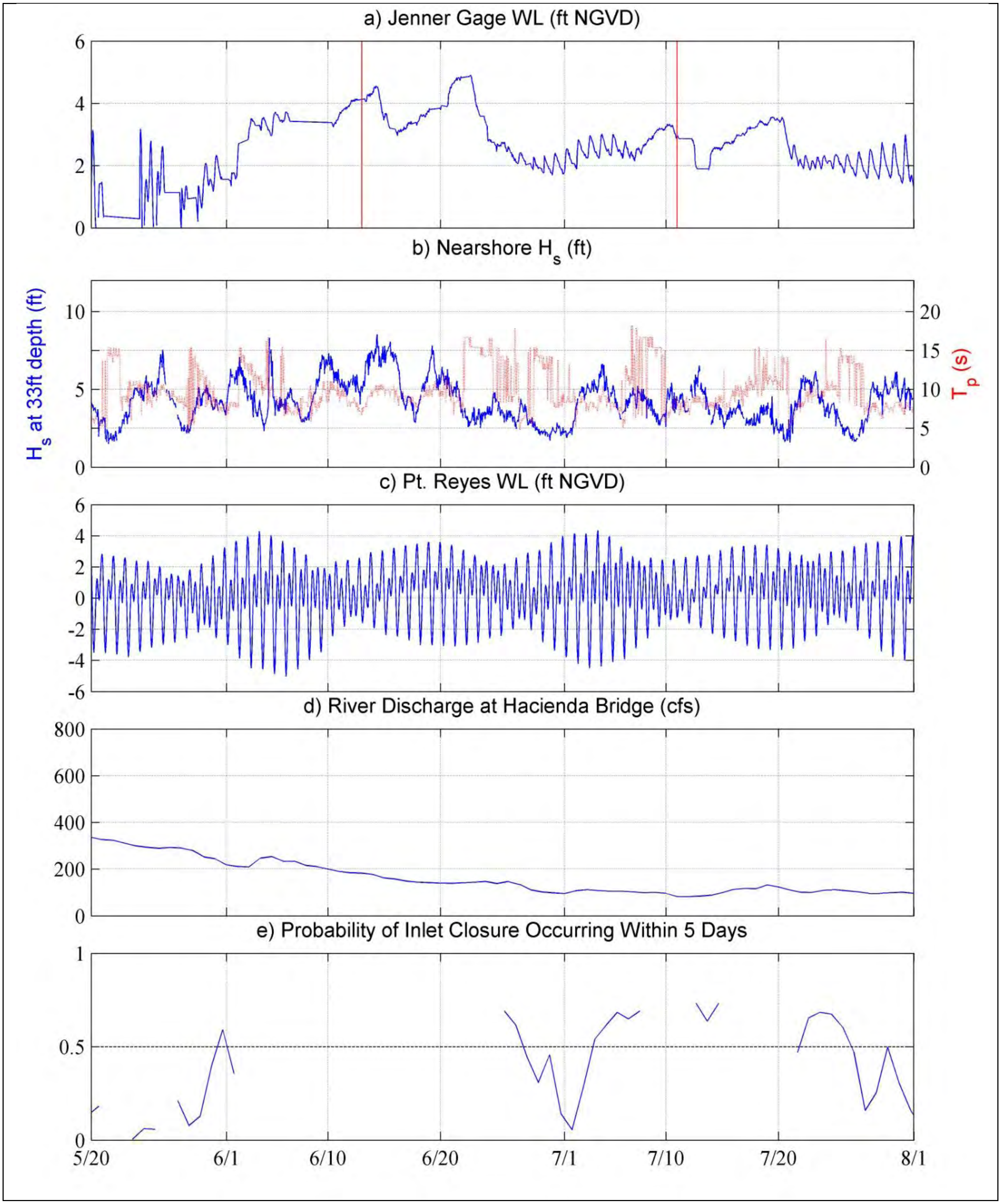
Weigel, R. 1992. Oceanographical Engineering.



SOURCE:

a) Jenner gage water level provided by SCWA; red bar = beach survey
b) H_s = sig. wave height; Tp=peak wave period (CDIP, Pt. Reyes, #029)
c) Ocean water level provided by NOAA (Pt. Reyes #9415020)
d) River discharge provided by USGS (Guerneville #11467000)
e) Five-day closure probability provided after Behrens et al. (2013)

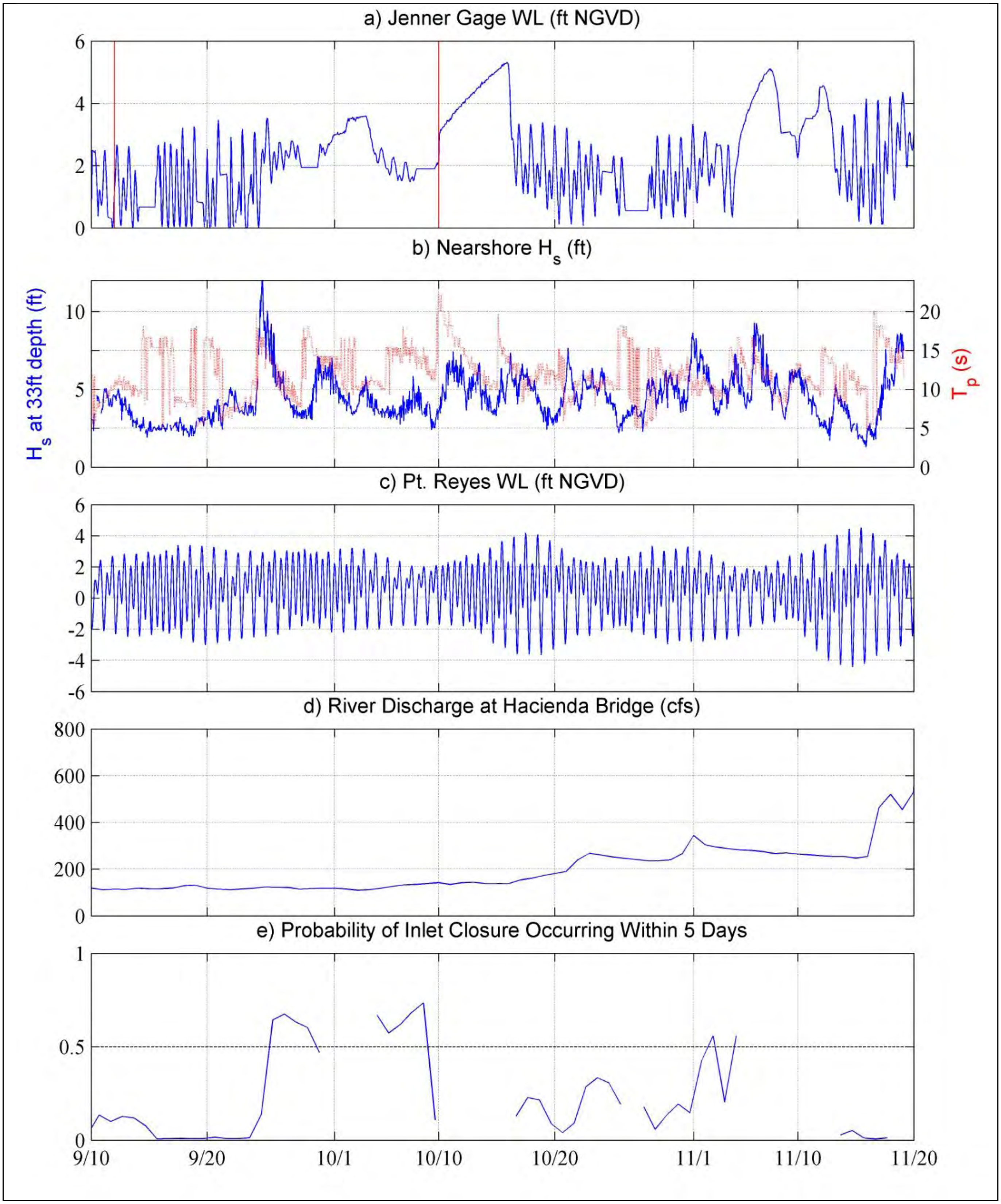
Figure 1
Estuary, Ocean, and River Conditions Compared with Closure Probability: September – November 2012



SOURCE:

a) Jenner gage water level provided by SCWA; red bar = beach survey
b) H_s = sig. wave height; T_p=peak wave period (CDIP, Pt. Reyes, #029)
c) Ocean water level provided by NOAA (Pt. Reyes #9415020)
d) River discharge provided by USGS (Guerneville #11467000)
e) Five-day closure probability provided after Behrens et al. (2013)

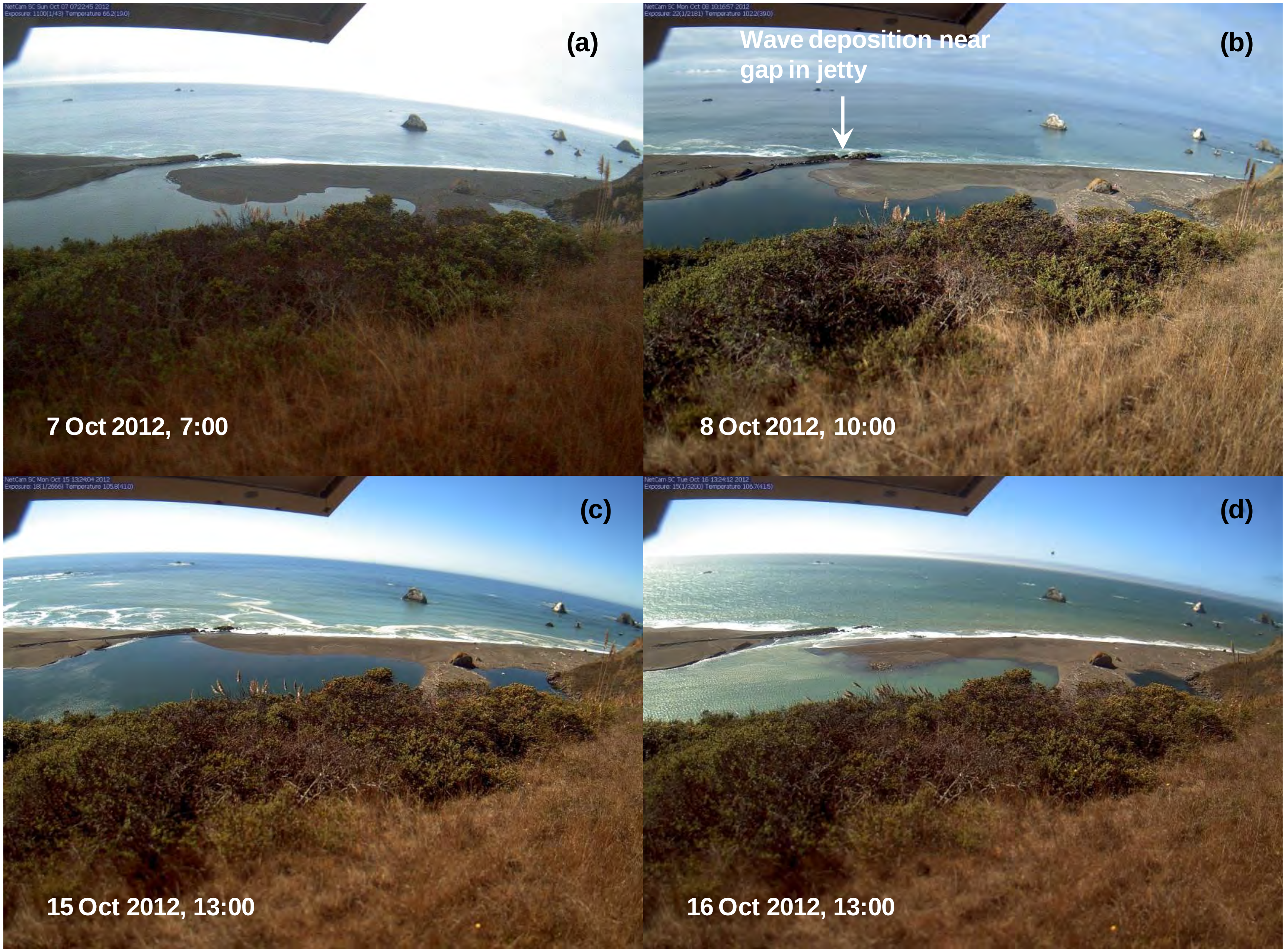
Figure 2
Estuary, Ocean, and River Conditions Compared with Closure Probability: May 20 – August 1, 2012



SOURCE:

a) Jenner gage water level provided by SCWA; red bar = beach survey
b) H_s = sig. wave height; T_p=peak wave period (CDIP, Pt. Reyes, #029)
c) Ocean water level provided by NOAA (Pt. Reyes #9415020)
d) River discharge provided by USGS (Guerneville #11467000)
e) Five-day closure probability provided after Behrens et al. (2013)

Figure 3
Estuary, Ocean, and River Conditions Compared with Closure Probability: September 10 – November 20, 2012



SOURCE: Russian River stationary observation camera (BML)

Figure 4
Inlet Closure and Self-Breach in October 2012

Source: Sonoma County Water Agency

figure 5
Project Name

Outlet Channel Flow Through Jetty Notch

ESA PWA Ref# DW01958.02

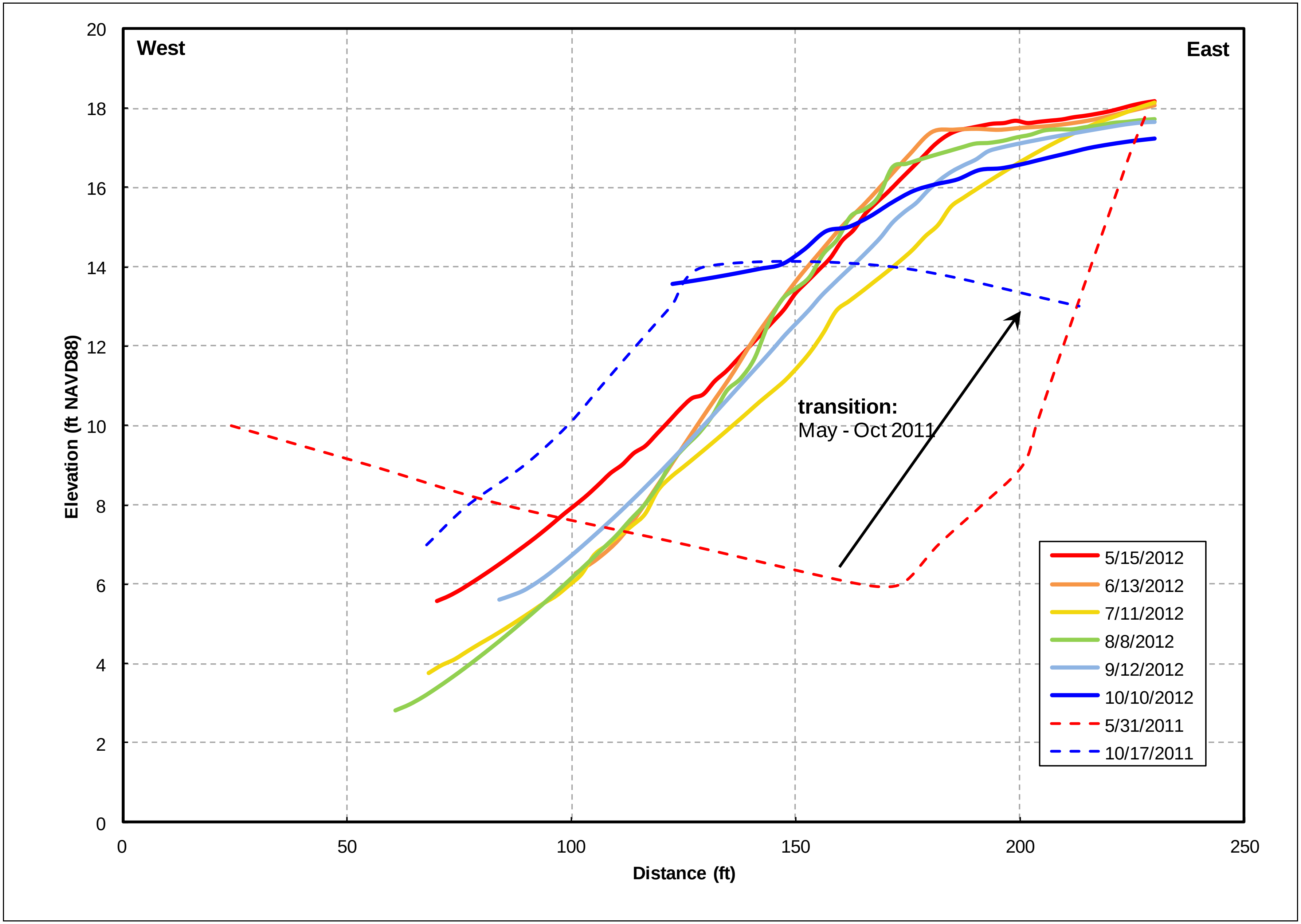
ESA PWA

SOURCE: image from USDA NAIP

Russian River Estuary Outlet Channel Management Plan . DW01958

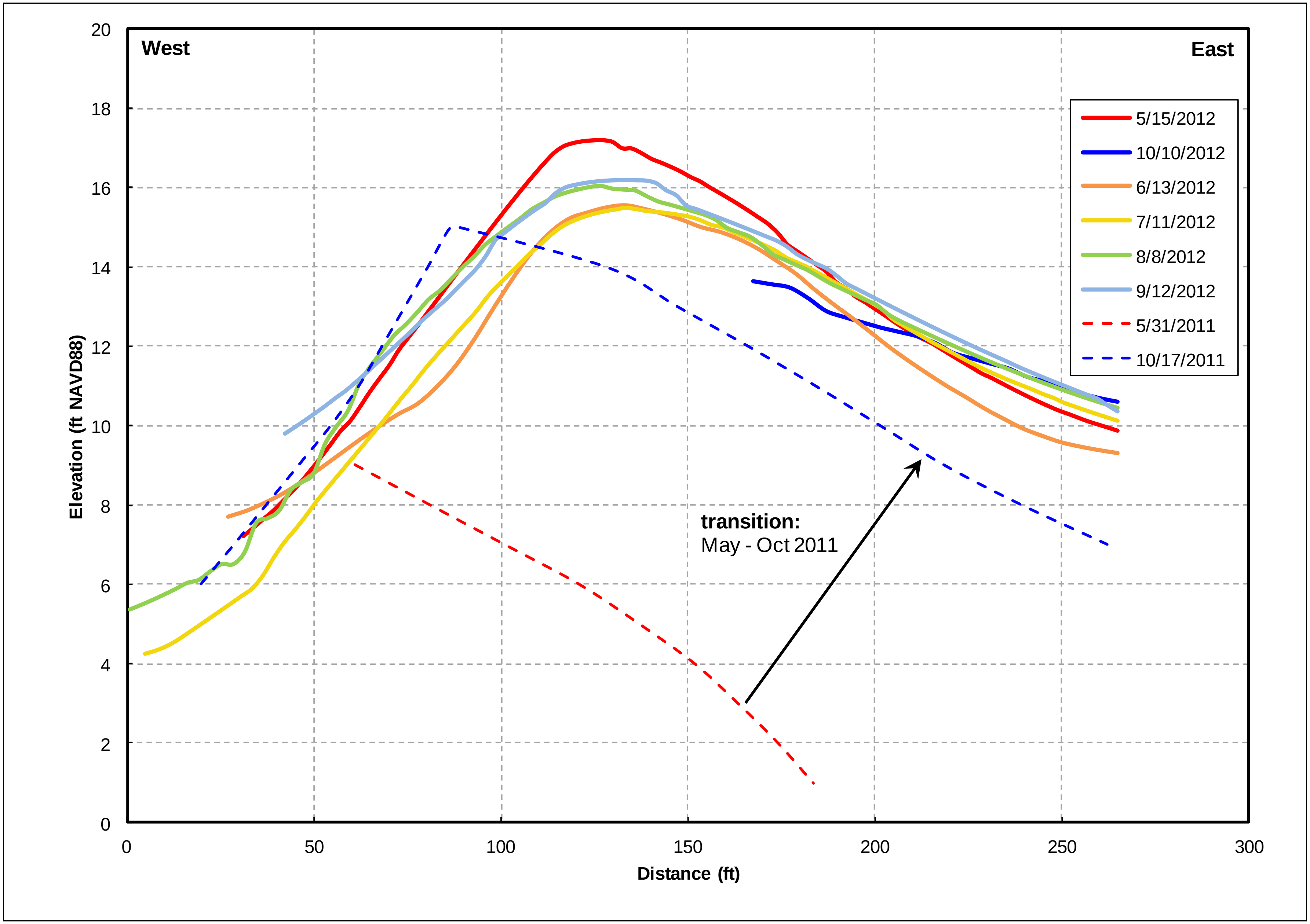
Figure 6

Beach Transect Locations



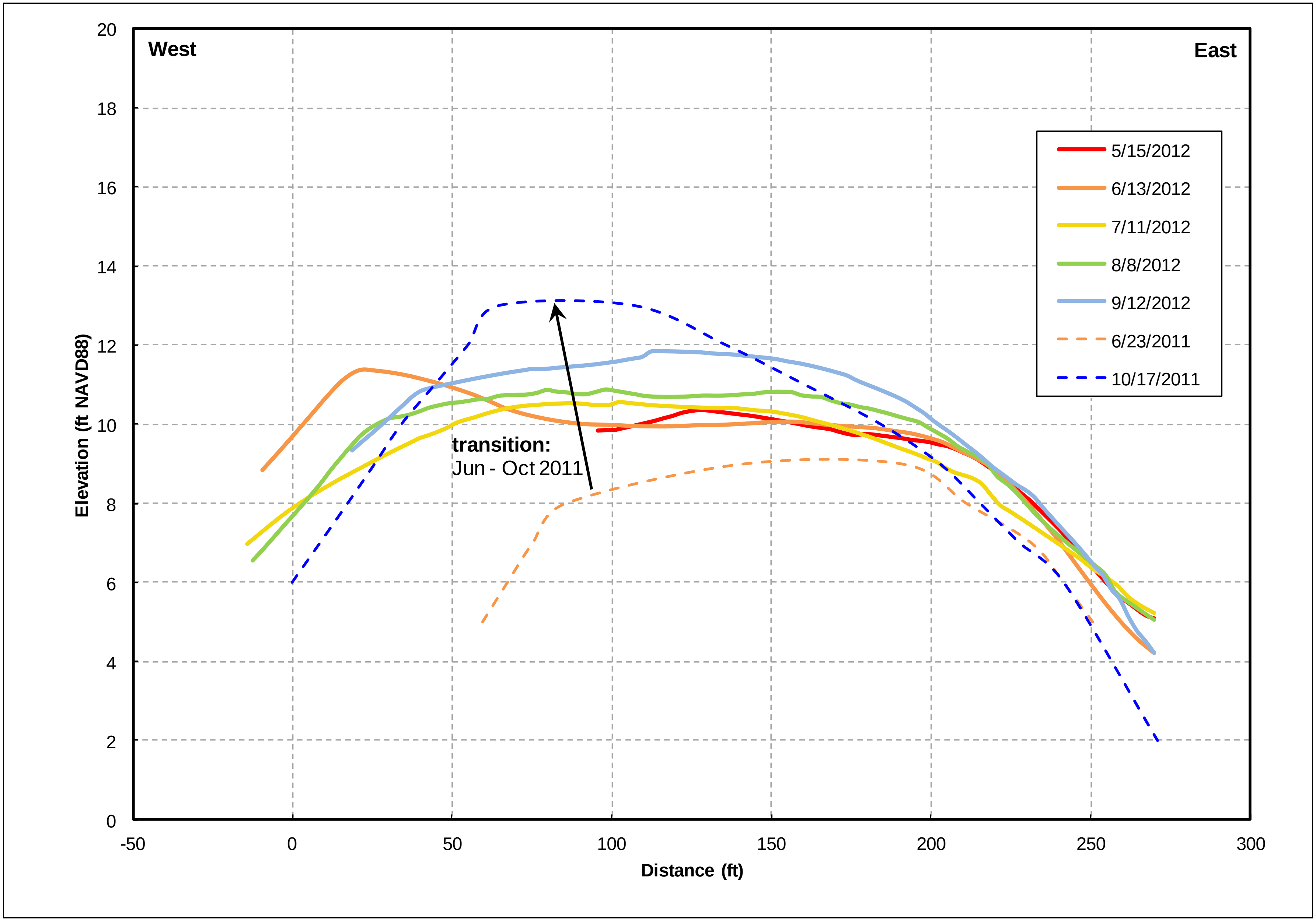
SOURCE: SCWA survey data

Figure 7
Beach Transect 4



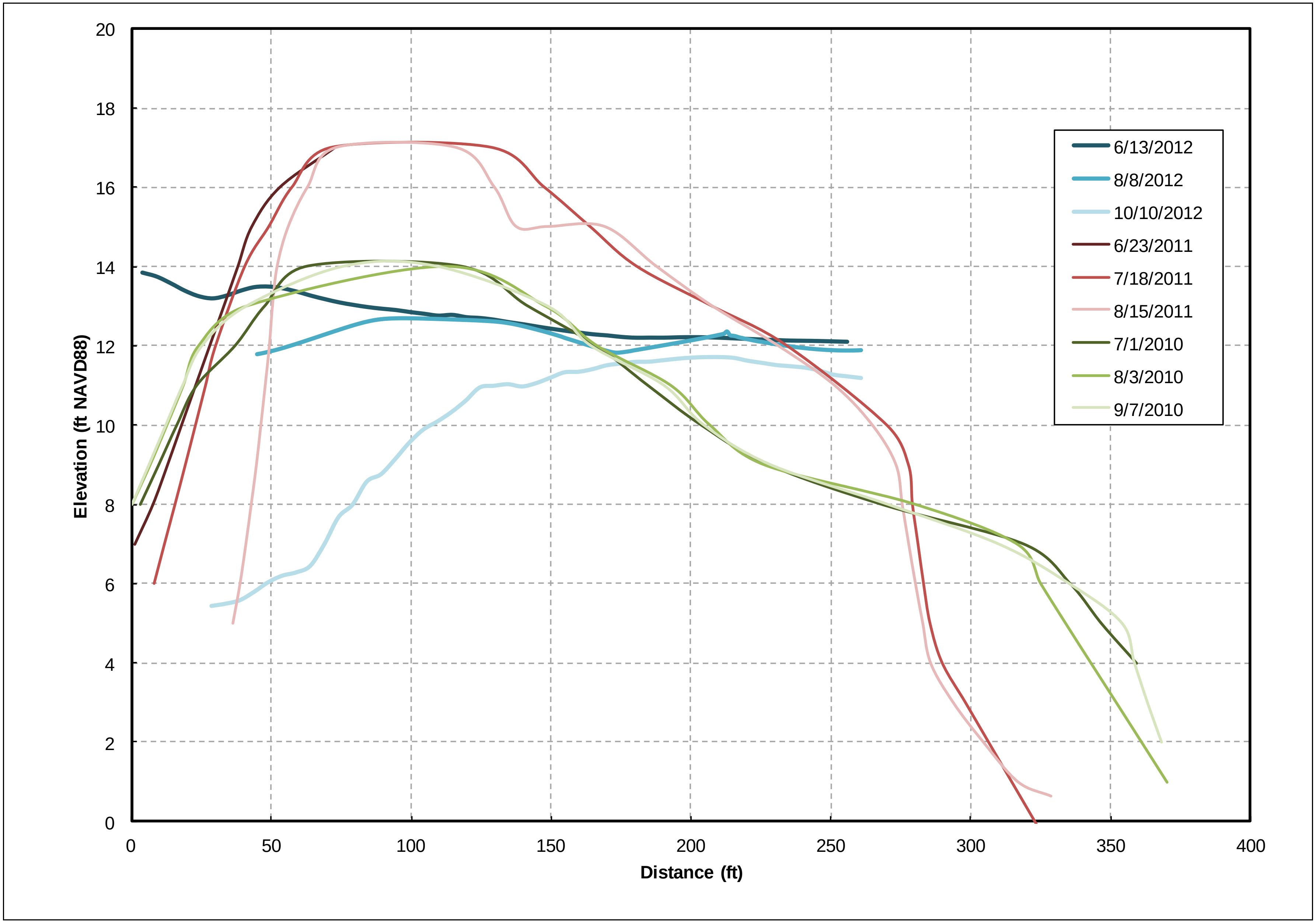
SOURCE: SCWA survey data

Figure 8
Beach Transect 3



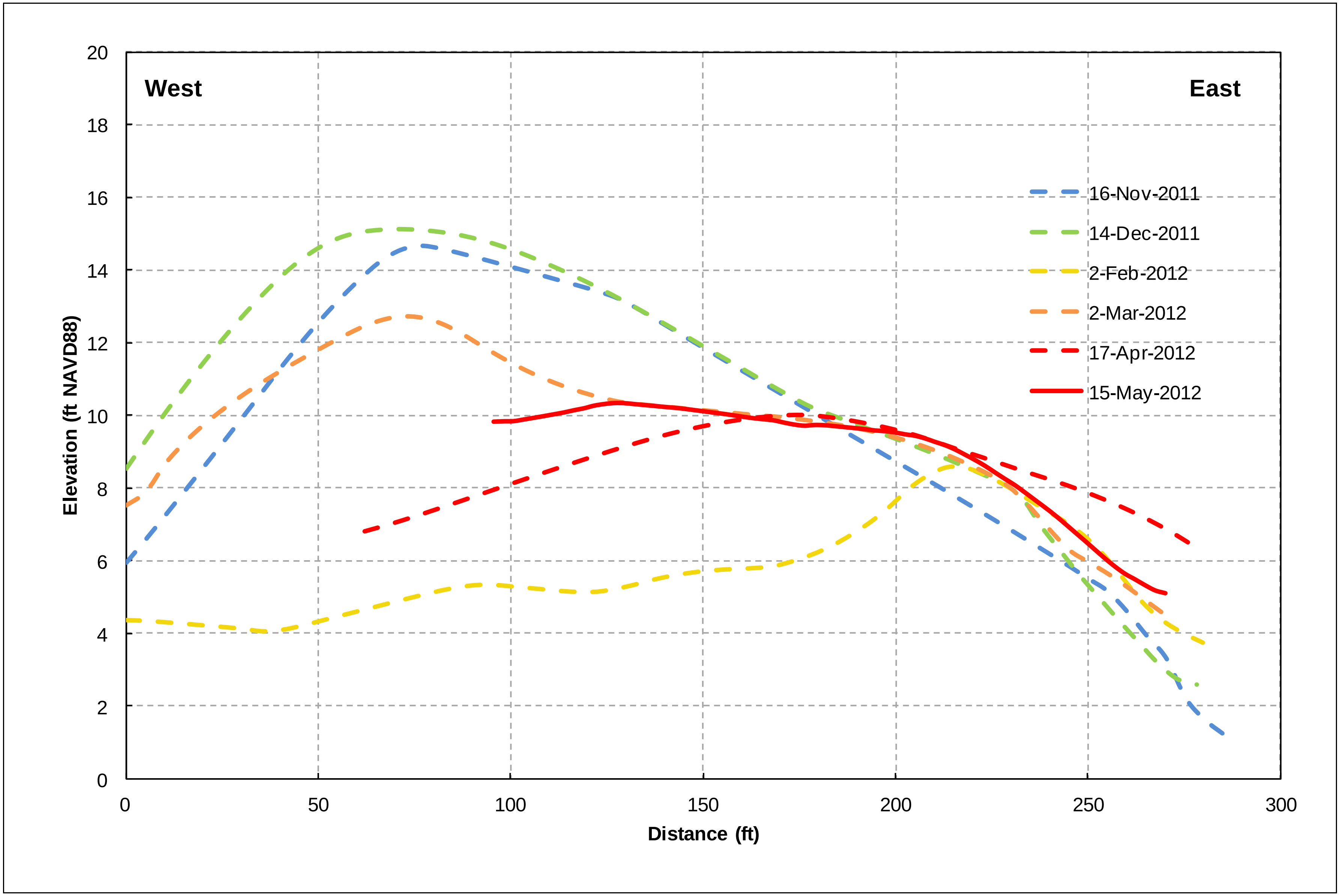
SOURCE: SCWA survey data

Figure 9
Beach Transect 2



SOURCE: SCWA survey data

Figure 10
Beach Transect 0



SOURCE: SCWA survey data

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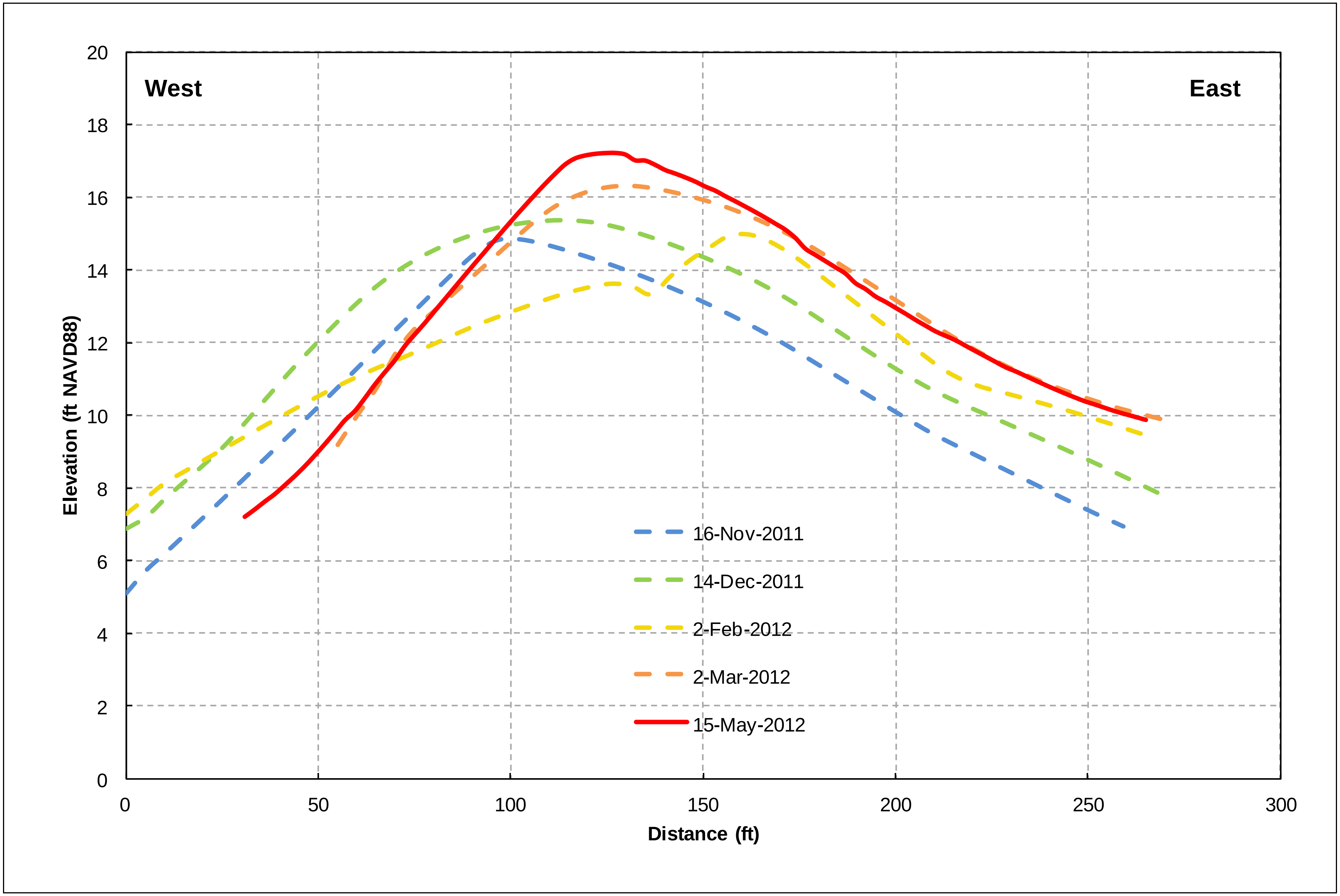
Figure 11

Nov 2011 to May 2012 topographic change at Beach Transect 2

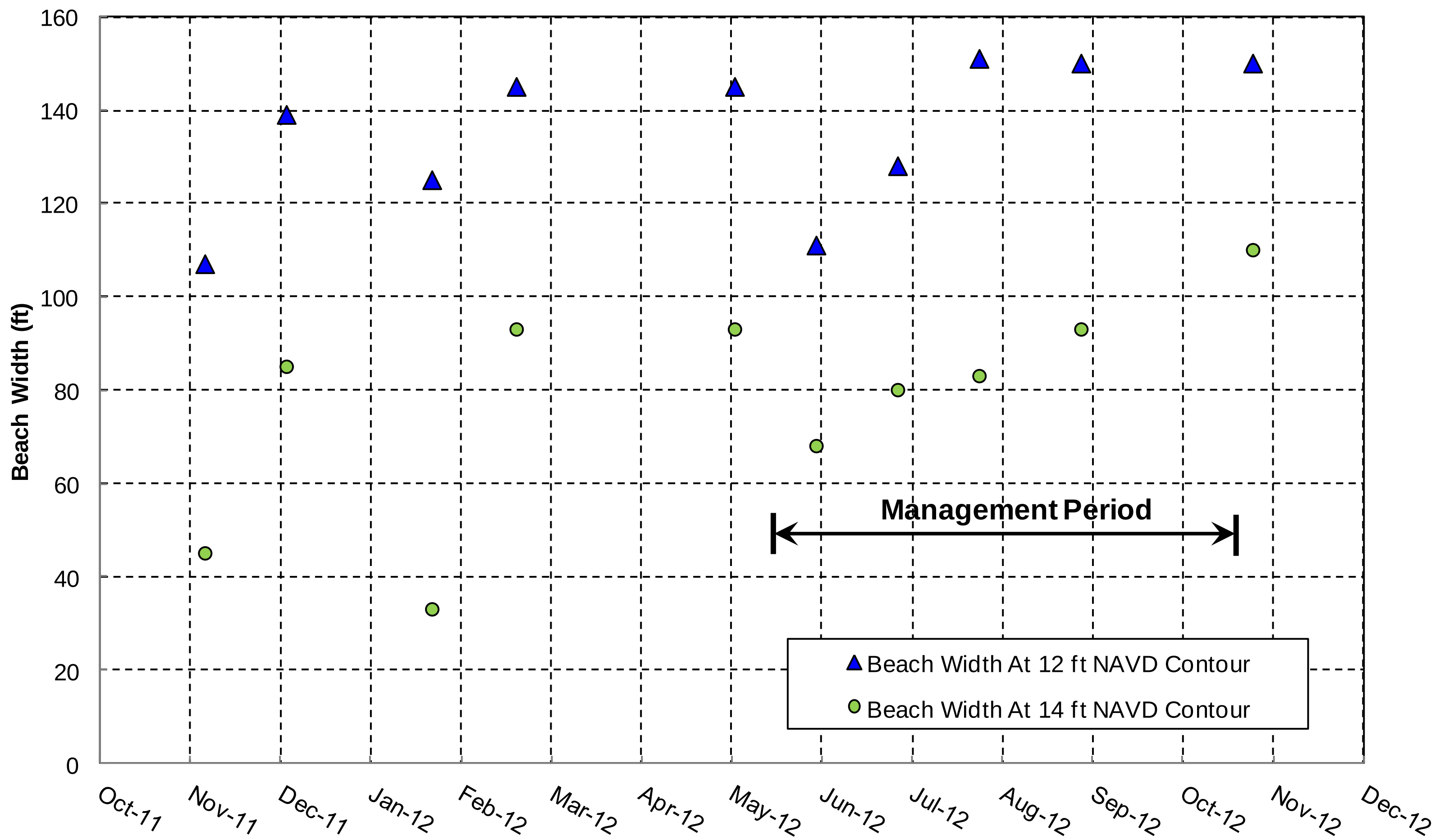


SOURCE: SCWA survey data

Russian River Estuary Outlet Channel Management Plan . DW01958

Figure 12

Nov 2011 to May 2012 topographic change at Beach Transect 3



SOURCE: SCWA survey data

Figure 13
Beach Width at Beach Transect 3.

DRAFT

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

Existing Conditions

Prepared for
Sonoma County Water Agency

December 31, 2012

ESA PWA

DRAFT

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1 INTRODUCTION

1.1 Study Purpose

This existing conditions report, developed by ESA PWA at the request of Sonoma County Water Agency (Water Agency), describes the physical processes that influence water levels in the Russian River Estuary (Estuary) and that may be influenced by the jetty on Goat Rock State Beach (GRSB) (Figure 1-1). The findings of this report will be used as part of a larger feasibility study. This feasibility study will develop and assess alternatives to the jetty that may help achieve target estuarine water surface elevations. As such, this report fulfills 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 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.

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 barrier beach blocking 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 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 ft NGVD[1]. While it is likely 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 BO allows 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 will 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

[1] NGVD=National Geodetic Vertical Datum, a fixed reference elevation adopted as a standard

inlet. Conceptually, the outflow will also 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 flooding of 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, 2012). 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. This document initiates this second phase by describing existing conditions affecting Goat Rock State Beach and the jetty. The third stage further expands the project scope to include flood risk reduction measures for properties adjacent to the Estuary.

Water surface elevations are stated objective of the Biological Opinion’s Estuary Management RPA. 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 will inform 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.

Based on the requirements of the Biological Opinion described in the paragraph above, the goal of the feasibility study is to evaluate alternatives that modify the Goat Rock State Beach jetty with the intent of improving the likelihood of achieving the target lagoon water surface elevations (ESA PWA, 2011). To accomplish this goal, the study objectives include:

- Describe the extent and composition of the jetty

- Understand the jetty's existing effects on the physical processes which affect lagoon water surface elevations, including beach permeability, sand storage, and sand transport
- Evaluate the jetty's influence on flood risk to property adjacent to the Estuary
- Develop and analyze jetty alternatives, such as jetty removal, partial removal, jetty notching and other uses of the jetty which may help achieve target lagoon water surface elevations

The first three objectives are the subject of this existing conditions report. This report is intended to provide an understanding of the physical processes which affect the lagoon water surface elevations and which may be significantly affected by the jetty. The findings of this report will then inform the development of alternatives that may modify the jetty and the feasibility assessment of implementing these alternatives. When the study is complete, it will help inform any future decisions as to whether to proceed with modifications to the jetty. If the technical analyses in this study will provide a basis for future steps; however, additional technical work may be required to inform potential project impacts and design features. The Biological Opinion does not require the Water Agency to implement any recommendations of the jetty study and implementation of any recommended project is currently without a funding agency.

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 and the California Department of Fish and Game (CDFG).

The remainder of this section 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. For completeness, sections on groundwater seepage and inlet morphology are also introduced in this draft report. However, the analyses for these sections have not yet been completed, largely because the on-beach activities required for these tasks have not yet been permitted. Permitting and the assessments to inform these tasks are anticipated for 2013. The results of these two analyses will be included in the revised version of this report. In addition, a site assessment of the jetty will also be conducted in 2013 and its findings added to Section 2.

1.2 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. These connections between the jetty, barrier beach formation, and lagoon water surface elevations need to be quantified to inform the development and evaluation of alternatives in subsequent stages of the feasibility study. The first assessment, which will inform subsequent assessments, is the characterization of the jetty's extent and composition, as much of the jetty is hidden within the beach and poorly documented. Next, the study will look at three key processes by which the jetty may influence barrier beach formation and lagoon water surface elevations. These processes are groundwater permeability, beach morphology, and inlet morphology[1]. 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. We have chosen to focus the study on the morphology since ultimately it is the beach shape that influences lagoon water surface elevations. For example, to achieve the target water levels defined in the Biological Opinion, the barrier beach morphology needs to close the inlet which connects the Estuary to the ocean. We examine the morphology 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 river and 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

[1] As discussed above, groundwater permeability and inlet morphology sections only include introductory material; the analyses for these sections will be completed in 2013 and included in the revised version of this existing conditions report.

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.

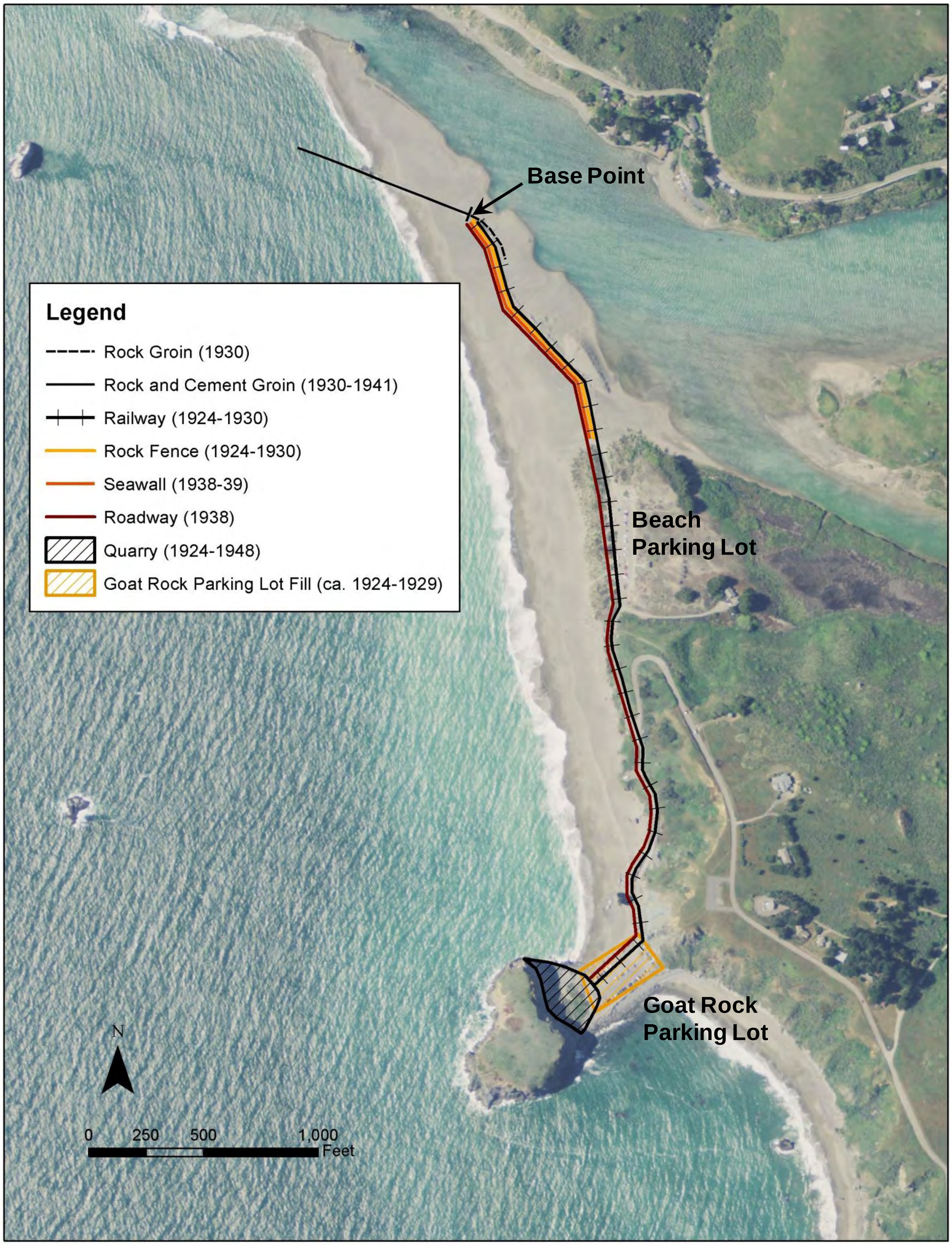
The inlet, the connection between the Estuary and ocean across the barrier beach, shifts between three states – tidal, closed, or outlet. When ***tidal***, the inlet allows the water level oscillations associated with the tides into the Estuary and allows the river inflow to drain out. The fresh river 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 levels 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 artificial or a natural causesto reduce flood risk. 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 Biological Opinion 's 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 Parks. 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.

1.3 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 will inform 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 (GRSB) 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 components north of the Beach Parking Lot because of their proximity to the inlet. These components are hereafter referred to as the following:

- **Jetty, jetty complex, or complex**: All man-made structures on GRSB north of the Beach 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**: The access elements collectively consist of the roadway, seawall, railway, and rock fence, which were built to transport equipment and rocks to the groin for construction and to prevent the inlet from opening south of the groin.

In total, over 100,000 tons of rock were quarried from Goat Rock and 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 existing reports. Once permitting is complete, a topographic survey of and soil samples from the existing jetty structure may be added to this section. 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. The quarry at Goat Rock was first opened 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 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 jetty, in order to prevent the inlet from opening south of the jetty,

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

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 oceanward 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 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 endpiece 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 121 feat 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 endpiece 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 south of the Beach Parking Lot or filled over by the present-day access road. The proposed topographic survey of the jetty structure would give a more complete picture of the extent of the remaining jetty complex. The observations on the beach can be used with the existing historical reports to estimate the size, extent, and material of the remaining structures. These are summarized in Table 2-1.

Existing LiDAR data from September 2010 indicates that the top of the concrete cap of the groin may have subsided several feet since 1948. Datum corrections between mean low low water (MLLW) at the time of the groin construction and the present were obtained using data from the Pt. Reyes tide gage (NOAA, 2012). MLLW at the site shifted by 0.21 ft between the old (1960-1978) and new (1983-2001) epochs at Pt. Reyes. The available LiDAR data suggest that the top of the groin had an approximate elevation of 14-15 ft MLLW in September 2010, compared with the original reported elevation of 17 ft MLLW (Schulz, 1942). If the 1938 planning detail of the groin (Figure 2-1) is accurate, and if subsidence has been uniform across the structure, this would suggest that the base of the jetty structure may have subsided by approximately 2-3 ft from an initial estimate of -12 to -16 ft MLLW (Figure 2-1) to an elevation of roughly -14 to -19 ft MLLW at present. This would suggest that the legs of the concrete cap on the groin may have also subsided to a bottom elevation of about 5-6 ft MLLW on the ocean side and 8-9 ft MLLW on the river side of the groin, with the rock and cement armoring placed in 1948 possibly lying beneath these elevations.

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 2-5). 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. 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 burial 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), facilitating periodic wave overwash during storm events.

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 sand spit 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. Since redwood sheets are known to have been fixed to the top of the posts (which are still present, but may be somewhat degraded), the extent of the sheets in the beach can be estimated with a future survey of the posts.

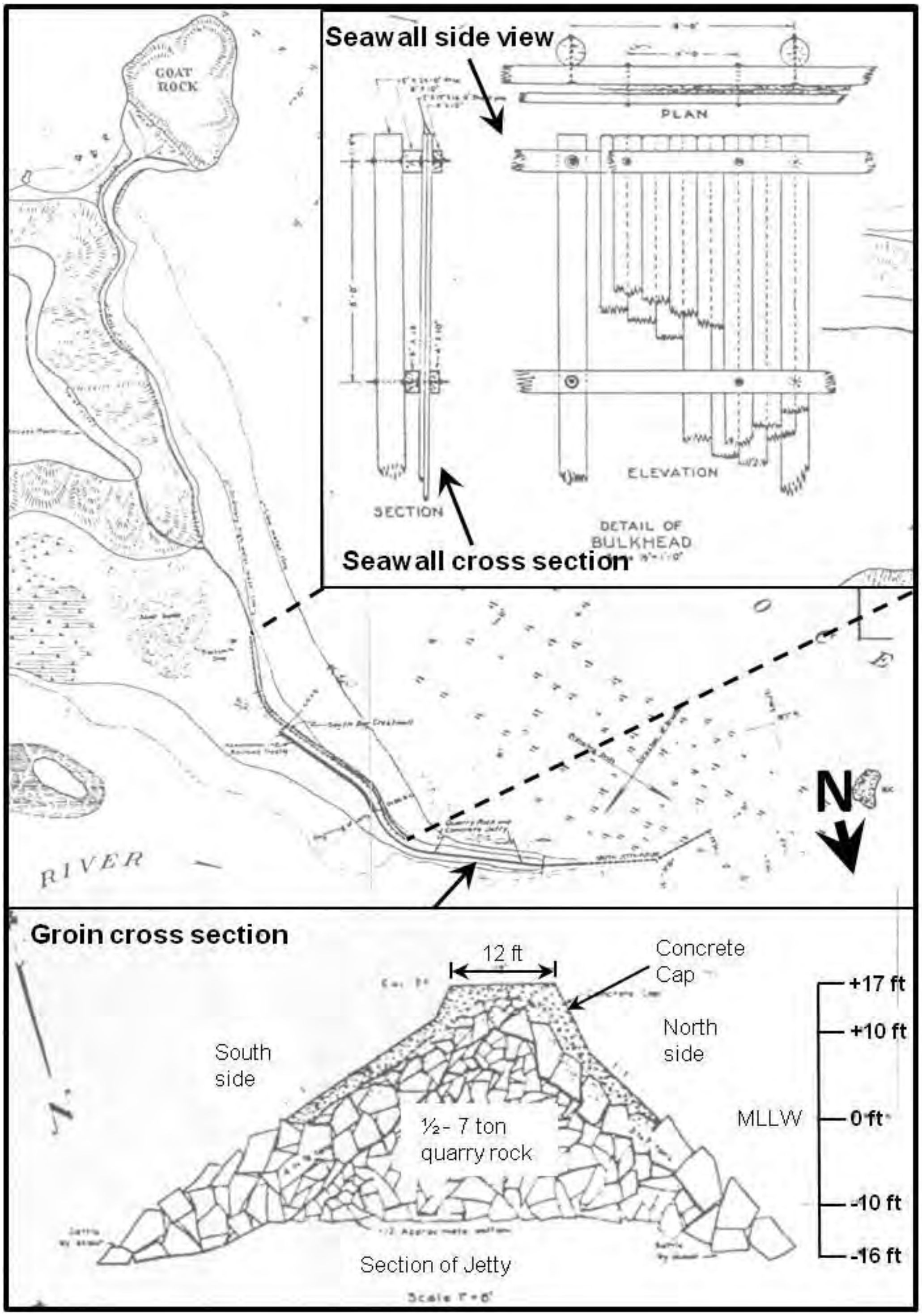
Table 2-1. Extent and material of the components of the jetty complex on GRSB.

Constructed Element (Time of Completion)	Approx. Endpoints (Length)[1]	Elevation Range[2] (MLLW)	Material[3]
Railway(1930)	38°27'00" N, 123°07'42" W; 38°26'52" N, 123°07'35" W; (~1000 ft)	14-24 ft	Timber pilings, steel railings
Rock Fence (1930)	38°27'00" N, 123°07'42" W; 38°26'52" N, 123°07'35" W;	Unknown	Quarried rock (~ 30,000 tons),
Rock and Concrete Groin (1934, 1941)	38°27'00" N, 123°07'43" W; 38°27'02" N, 123°07'49" W; (505 ft)[1,3]	-19 to 14 ft 17 ft at base point	Quarried rock (~70,000 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]	7 – 22.5 ft near groin 8 - 23.5 ft near Beach Parking Lot	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]	22-25 ft near Beach Parking Lot. Unknown near groin[5]	Compacted rock, possibly additional rock base in some areas
Groin Armoring (1948)	Unknown[6] (< 500 ft)	-2-14 ft	Quarried Rock (4,280 tons), cement (651 yd^3)

[1] Based on aerial photographs and map provided by Johnson (1959). These may be revised with expected survey work.
[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).

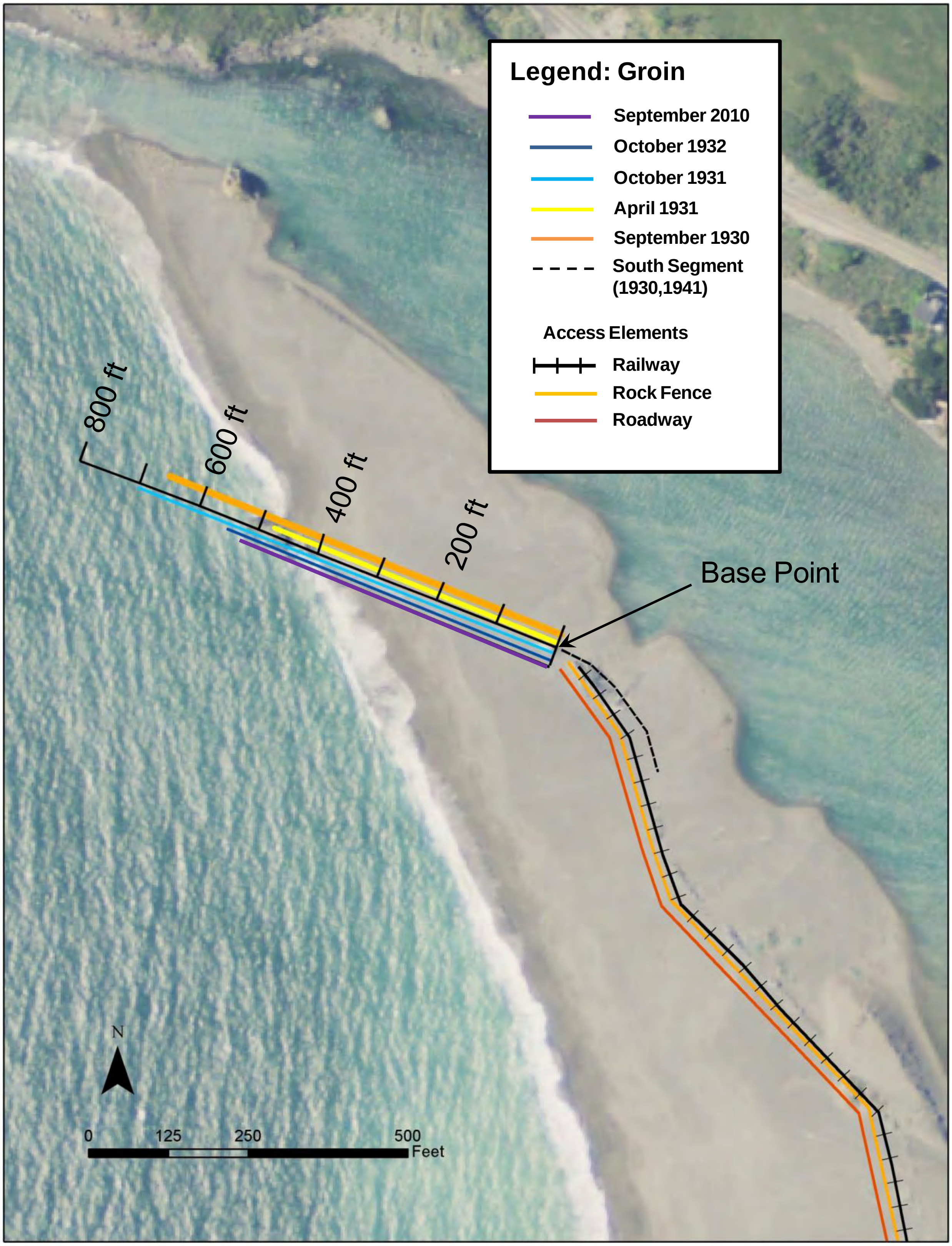
Note: A site assessment of the jetty's current condition, which will include surveying, soil samples, and mapping, is anticipated for 2013. The results of this assessment will be included in this section.

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.



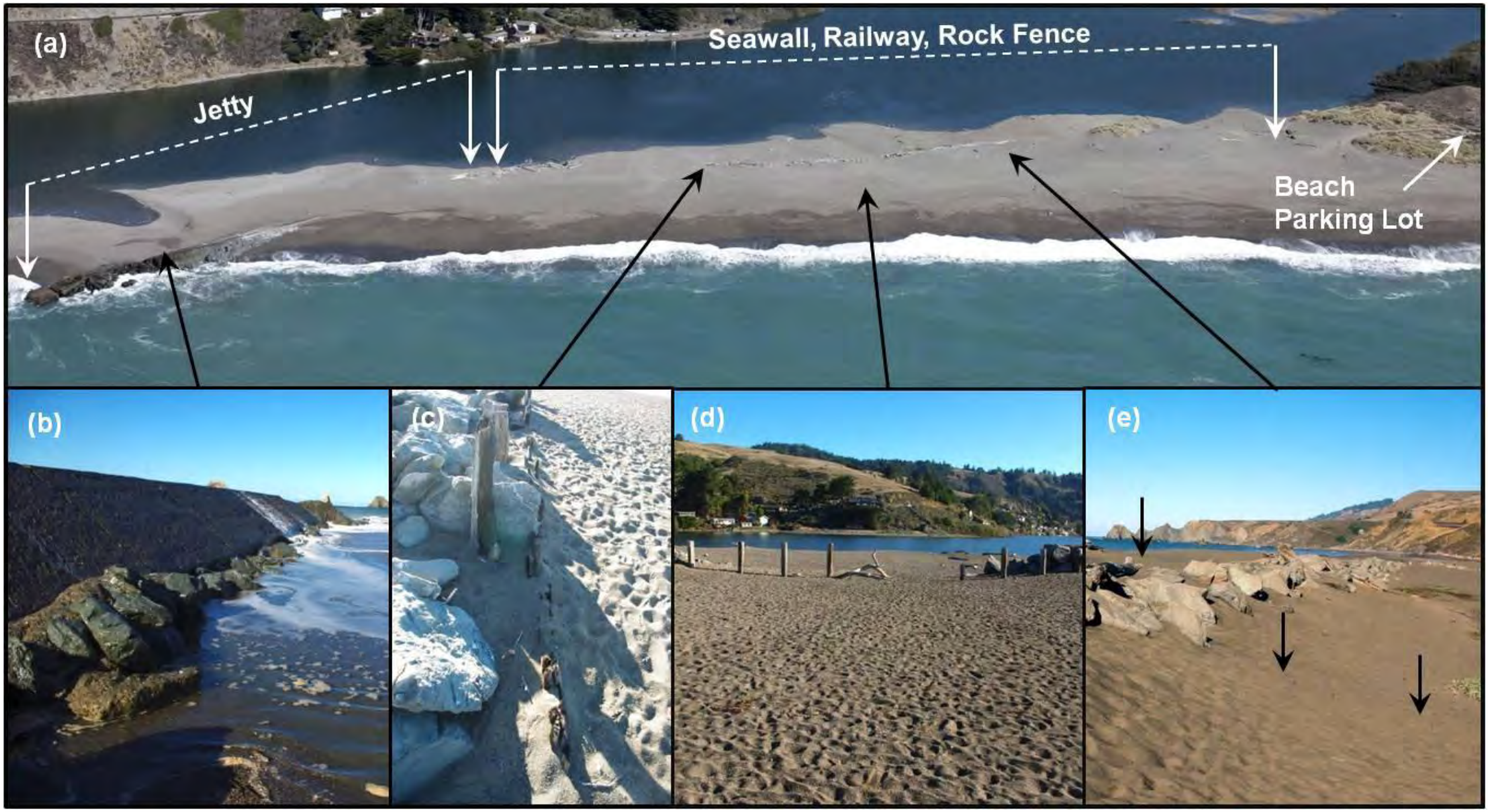
SOURCE:NAIP 2007 aerial image of Sonoma County. Construction diagrams are based on a DWR map provided by Johnson (1959) and historical information from Schulz (1942).

Goat Rock Jetty Feasibility Study . D211669.00

Figure 2-2
Plan view of groin construction: 1930-32

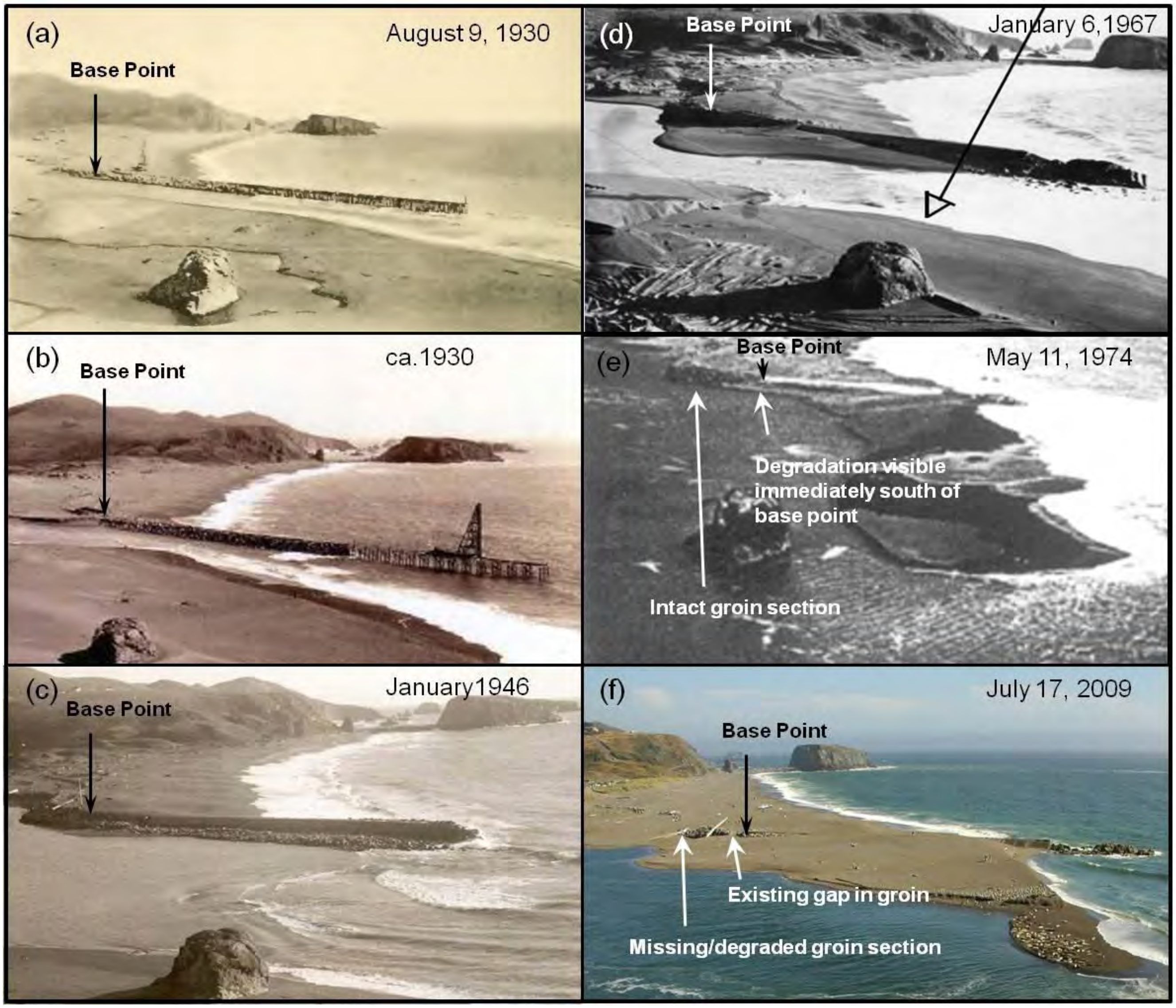
SOURCE: M. Brennan, May 19, 2010.

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: (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 2-5
Site photographs of jetty from 1930 to 2009.

3 GROUNDWATER PERMEABILITY

Note: This section only provides a brief introduction on the relationship between the jetty and groundwater permeability. The technical analyses related to groundwater permeability are anticipated for 2013 and results of those analyses will be summarized in a revised version of this report.

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 level differences between the Estuary and the ocean. Permeability is largely determined by the size, type, and arrangement of the particles within a porous medium. 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).

Because the jetty complex is comprised of various soil types and rock sizes and it was constructed as a permanent structure, the materials comprising the complex probably have different permeability than natural beach sands. However, rates of seepage, and spatial variation of the complex's permeability relative to beach sand are not known. Therefore, the net effect of the jetty, whether it results in an overall increase or decrease in seepage through the jetty-influenced beach as compared to a natural beach, is not known.

Using a water balance approach to analyze closure periods, 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 head difference. Although this analysis demonstrates a direct 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 level difference. Most of the seepage is thought to occur through the upper portion of the barrier beach where the jetty complex resides, but could also be directed laterally into the Estuary's aquifer. Seepage rates within the beach probably decrease with depth as the barrier beach becomes wider. Largier and Behrens (2010) note an increase in seepage rate for lagoon water surface elevations above 3-4 ft NGVD

[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 BO.

and speculate that this may be the result of vertical changes in permeability resulting from the jetty complex. Modeling of the salt field also suggests that most seepage occurs in the upper portion of the Estuary (Largier and Behrens, 2010).

The jetty's permeability is likely to vary between its different components (groin, roadway, seawall, railway, and rock fence) since they are made of different materials, with different methods, and are in varying states of degradation. Besides differences in permeability, the components have different sizes and orientations to the direction of seepage. Relative to a natural barrier beach, some portions of the jetty, such as the large rocks in the groin, may increase the permeability. Other portions of the jetty, such as the compacted roadway and railway fill, may reduce the permeability. In addition, by affecting the setting and duration of beach sand deposition, the jetty may alter the permeability of the beach sands deposited about its flanks. For instance, the portion of beach berm containing the jetty is wider than the portion of beach berm north of the groin that hosts the Estuary inlet. The jetty complex has affected beach berm morphology as discussed in Section 5, and as a result may have also altered the grain size and compaction of sediment which has accumulated around it. The complex's integrated effect on lagoon water surface elevations depends on the magnitude of local permeability and the spatial extent of these variations. Determining these parameters is the focus of the proposed analyses.

Besides reducing lagoon water surface elevations directly, permeability interacts with inlet and beach morphology, jetty-influenced processes discussed below. For instance, permeability affects inlet morphology since water exiting the lagoon as seepage reduces the amount of water that needs to be conveyed by the outlet channel to maintain constant lagoon water levels. 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 (1977 and 2009), 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.

4 OCEAN WAVE CONDITIONS

Wave-driven sediment transport is the constructive process that results in the beach barrier at the Russian River Estuary's inlet. Wave overtopping can also affect breaching of the estuary waters to the ocean. Hence, wave driven processes are key to subsequent assessments of beach and inlet morphology, as well as flood risk. Once wave conditions are characterized, the potential effects of the jetty structure can be evaluated, and provide an assessment of the implications of modifying or removing the existing jetty. This section summarizes the offshore wave climate, discusses the methodology for predicting the nearshore wave climate adjacent to GRSB, and provides estimates of surf zone processes and their sensitivity to beach morphology and the presence of the jetty.

4.1 Offshore Wave Assessment

To study deepwater wave conditions offshore of GRSB, wave data from 1997 through 2011 were collected from the Pt. Reyes buoy (NDBC buoy # 46214) operated by the Coastal Data Information Program (CDIP). The buoy is located in 1700-ft deep water at the edge of the continental shelf, approximately 37 mi. southwest of the inlet and 25 mi. west of Point 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.

We examine the total energy (a function of the wave height squared, measured in cm^2) of the offshore waves, since this is the main determinant of wave heights in the nearshore zone, and is thus related to the total runup and overwash on the barrier beach. We focus on hourly wave energy from May to October (the juvenile steelhead rearing season as identified in the 2008 Biological Opinion by the National Marine Fisheries Service), indicating the amounts of the total wave energy split among waves of different approach direction (θ) and peak period (Tp), the period of the wave components with the most energy. Regarding direction, waves are separated based on their approach direction relative to the orientation of the beach. GRSB has an approximate orientation of 345° (measured from north), so the shore-normal vector is at 255°. The waves are characterized as either northerly (> 255º approach) or southerly (< 255 º approach) based on this vector. Regarding period, waves are taken as either as either swells (Tp > 10 seconds) or seas (Tp < 10 seconds). Applying these classifications, wave energy at this site is separated among four possible combinations of these parameters. To typify the evolution of the wave climate throughout the management period, we found the maximum offshore wave energy observed during each day during the years 1997-2011. The maximum daily wave energies were then aggregated by Julian day (January 1 is Julian Day 1). For example, the maximum wave energy for June 30 is the maximum of the offshore wave energy during this day for each of the years 1997-2011.

Figure 4-1 shows the amount of wave energy associated with each wave type during the management period. For the majority of the period from May to October, northerly seas (Figure 4-1b, upper right panel) are the primary contributor to the offshore wave spectrum. The exception to this is the month of October, when the maximum energy of northerly swell (Figure 4-1a, upper left panel) contributes a larger portion of the wave energy. Northerly seas are greatest in May and June and decrease during the remaining months, whereas northerly swells are moderate in May and June, reach a minimum in July and August, and begin increasing in September. In some years these are also high in May, although this is not consistent. Southerly waves (both sea and swell) were weaker at the Pt. Reyes buoy than their northern counterparts during the management period.

Overall, northerly waves (regardless of wave period) dominate the offshore spectrum between May and October. However, because of wave refraction, offshore and nearshore wave directions might not correlate, especially on coastlines with variable offshore bathymetry. Although swells comprise a smaller portion of the total energy, it is not clear from this analysis alone how important they are to inlet closure events, since closure events are infrequent during this period. While Behrens et al. (2013) have shown that wave energy is a good predictor of closure when shore-parallel sediment transport dominates (suggesting that seas are more important during the management period), past studies (e.g. Ranasinghe and Pattiaratchi, 2003) have also shown that inlet closure can result from relatively weak waves that approach the shoreline from a shore-perpendicular angle. In this circumstance, these waves tend to be long-period swells, which have relatively greater power and are typically associated with beach-building, that close the inlet by pushing offshore sediment bars onto the beach. The impacts of offshore waves on inlet morphology are addressed in Section 6.

Offshore waves are visualized by direction and percent occurrence in Figure 4-2a. This diagram reflects the behavior of the dominant (i.e. highest energy) waves during the months from May to October. Although southerly waves are present during this period (Figure 4-1), this underscores the dominance of waves from the northwest. Wave power (proportional to the product of the wave energy and the wave period) is also shown, as this is correlated to sediment transport in the nearshore zone. For waves of similar height but different periods, those with the longer periods will have greater wave power, and thus, greater capacity for moving sediment. For the waves measured at the Pt. Reyes buoy, the northwesterly waves dominate the wave power spectrum simply because they or more common than long-period southerly waves throughout most of the period from May to October. However, the greatest wave power is associated with more westerly directions.

4.2 Estimating Nearshore Wave Conditions

Approximating surf zone processes requires information about the nearshore wave climate, which in turn requires an understanding of how waves transition from offshore to nearshore conditions. For this study, we approximated nearshore waves using four sources of information:

- Offshore waves measured at the Pt. Reyes Buoy (discussed above);
- A transformation matrix provided by the Coastal Data and Information Program (CDIP) (pers. comm. Bill O'Reilly) based on a numerical refraction/shoaling model;
- Wave transformation information provided by Johnson (1959), based on a wave-tracing method; and
- Wave measurements immediately offshore of GRSB collected by the Bodega Marine Laboratory (BML).

These sources were used to develop a revised wave transformation matrix that maps offshore wave energy at the Pt. Reyes buoy to nearshore wave energy offshore of the inlet at GRSB. The nearshore wave energy was then used to predict nearshore wave heights. The final two sources were used as a check on the predicted nearshore waves. The certainty of the transformed nearshore waves is greatest at the location where the BML wave measurements were collected (38º26'32.6" N; 123º7'45.8" W) near the inlet.

Nearshore wave heights used in the analyses of this section are solely a product of transformed waves derived from the offshore wave information at Pt. Reyes. We assume that wind-wave generation between the Pt. Reyes Buoy and GRSB is relatively unimportant compared with translation of the existing offshore wave field to the nearshore. We also only consider wave refraction and do not account for shoaling[1], since the methods that we use for estimating wave setup, runup and overwash on the beach require inputs of de-shoaled wave heights. Appendix A provides further detail about our use and modification of the transformation matrix.

4.2.1 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

[1] Shoaling is the process of wave steepening due to wave propagation into shallower depths, resulting in generally increased wave height and decreased wave length, and ultimately wave breaking when the depth is similar to the wave height.

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.

Wave transformation can be performed with a numerical model or by hand with bathymetry charts and an understanding of the offshore wave climate (Johnson, 1959). Transformation matrices map wave information from offshore to nearshore, accounting for processes that alter wave energy along the way. These are tables whose entries are ratios of offshore over nearshore wave height or energy, indexed by offshore direction and period (Figure 4-3a,b). These can be constructed using a numerical model or built from direct comparison between offshore and nearshore measurements (when available), or by hand-drawn methods such as wave ray tracing. Transformation matrices were shown to be valuable tools in previous inlet closure studies at Bolinas Lagoon (PWA, 1999) and Crissy Field (PWA, 2007).

The transformation matrix provided by CDIP transforms offshore spectral wave data measured at the Pt. Reyes Buoy to nearshore spectral data adjacent to GRSB, which is then used to estimate the significant wave height (H_s). The transformation matrix uses linear wave theory (USACE, 2002) to trace individual wave rays from GRSB back to the Pt. Reyes buoy by reversing the processes of refraction and shoaling. The model uses a 100 m × 100 m grid of the central California shelf bathymetry (pers. comm. W. O'Reilly). After each simulation, the energy of the individual wave rays is compared between the offshore and nearshore locations to provide information about the transformation.

Johnson (1959) provided a similar estimate of wave refraction at the site using 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 view of the influence of refraction on nearshore wave energy at GRSB (Table 4-1). The greater decrease in energy observed for northwesterly waves may be a result of the northwesterly coastline orientation (which would decrease the energy of northwesterly waves the most), or possibly the shallow bathymetry and offshore rocks located to the northwest of the inlet.

Table 4-1. 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

4.2.2 Comparison Between Nearshore Predictions and Observations

Before using nearshore wave height predictions to estimate surf zone processes, we tested their accuracy against wave measurements near GRSB. Nearshore wave heights were measured by BML from June 4 to August 5, 2009, and 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.

Figure 4-2b underscores the need to estimate nearshore conditions, as opposed to treating offshore wave conditions as representative of nearshore conditions. Nearshore waves measured from May to October (during the lagoon management months) of 2009 and 2012 had significantly different height, power, and direction than those measured at the Pt. Reyes Buoy. This is largely a result of wave refraction between the offshore location and GRSB. The southwesterly approach direction of the nearshore waves is an indication that the bathymetry causes waves to approach the shoreline at a nearly perpendicular angle. This is an expected result, but slight differences between the power of waves north and south of the shore-normal direction (255°) are an indication of net sediment transport along the beach. The majority of nearshore waves at the depth of the wave sensor were smaller than their offshore counterparts. It should be noted that wave direction measurements, especially in shallow water, are approximate.

Figure 4-3c compares nearshore Hs predicted by a modified CDIP transformation matrix and measured by BML during the 2009 data collection period. Without alteration, the transformation matrix under-predicts nearshore Hs when offshore waves have an approach angle greater than 310° (e.g. 6/20-6/25, 7/15-16, and 7/23 in Figure 4-3c). This was true of both the 2009 and 2012 measurement periods. To compensate for this, we constructed a separate transformation matrix for wave approach angles above 310° using the BML data, and superimposed this onto the original matrix. The methods used for this approach are discussed in more detail in Appendix A. The original and modified transformation matrices are compared side-by-side in Figure 4-3a and Figure 4-3b. Since waves were only measured for a combined total of less than five months, corrections to the original transformation matrix may not reflect the full seasonal range of wave conditions at the site.

We found that offshore waves alone are a poor predictor of wave conditions at GRSB. Similarly, the original (unmodified) transformation matrix predicts nearshore waves that have a large root-mean-squared error (RMSE) of 55 cm when compared to observations, due to under-prediction of northerly waves at the site. The modified transformation matrix gives comparatively good results (Figure 4-3c). The RMSE between observations and estimates was 22 cm for the 2009 data, or about 15 percent of the mean value of Hs. For the 2012 measurements, the RMSE between predicted and observed Hs was 33 cm, or about 27 percent of the mean value of Hs.

Qualitatively, the Johnson (1959) refraction/shoaling coefficients compare well with the coefficients of the transformation matrix. The refraction estimates for a location 200 ft south of the groin's base point (Figure 1-1) are closest to the position where the wave measurements used to modify and verify the transformation matrix were made. There are two notable differences between the results of these two approaches:

- Some westerly and northwesterly swell waves are amplified by the transformation matrix compared with a decrease in energy predicted by Johnson (Figure 4-3a, inset box #2).
- Due to the finer resolution of the transformation matrix, a higher degree of variability is present in nearshore wave energy.

The latter point is especially clear for swell waves approaching from angles of 225-292°. The transformation matrix indicates that two wave energy shadows exist within this range: one for approach angles of 240° and another for approach angles of 255-275°. Since the model used in generating this matrix is not available, it is not clear whether these regions are modeling artifacts or real effects caused by offshore bathymetry. Also, the comparison with recently collected nearshore wave data indicate that the coefficients are low in the longer period, northwest direction range. In the future, we may use the BML data to further test the accuracy of the nearshore wave predictions, and possibly produce a transformation matrix with further revisions. The accuracy of the coefficients and associated incident wave climate can be improved with additional wave refraction modeling and/or additional nearshore wave data. However, the updated coefficients both build upon and improve Johnson's prior work.

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 2011 (discussed above) with information about the beach morphology. The beach spit separating the ocean from the Estuary at GRSB undergoes seasonal changes in shape in response to seasonal changes in the nearshore wave climate. As discussed in Section 2 on the jetty structure and Section 5 on beach morphology at the site, the beach has also undergone long-term change, probably in response to the jetty construction and other factors. Since the completion of the seawall on GRSB in 1938, the inlet has never migrated or breached south of the groin. The construction of the quarry and access elements between Goat Rock and the mainland also may have disrupted a natural bypassing mechanism for the net southerly littoral drift at the site.

4.3.1 Beach Characteristics

Surf zone processes such as wave runup and overtopping are strongly dependent on the shape of the beach. We characterized 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 digital elevation models (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, 2012). 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 during the management period (May-October) starting in 2009. Lastly, survey data were collected independently by Behrens (2012) in August and December of 2009, and January and March of 2010.

Figure 4-4 summarizes the beach slope and crest height taken from the available topographic measurements. The lowest point (Figure 4-4a) on GRSB is north of the groin. During periods of closure, this point is sometimes located immediately to the north of the groin. At other times it is located as far as 800 ft north of the groin, where the headland intersects the beach. The beach crest slopes downward from the south

(20-26 ft NAVD) to the north (6-12 ft NAVD). The foreshore slope (Figure 4-4b), measured on the beach face between mean sea level (MSL) and mean higher-high water (MHHW) level, varies with time and location. Most data sets show no trend with location. However, the August 2009 surveys show a steepening of foreshore slope with distance north: from 0.06-0.10 (0.10 slope is a drop of 1ft over a run of 10 ft) at the south end of the profile to 0.06-0.25 at the northern end of the profile.

4.3.2 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 2011 using the modified transformation matrix with the Pt. Reyes Buoy data. Figure 4-5 and Figure 4-6 summarize the predicted nearshore wave conditions during the management period for these years.

Similar to the results for deepwater waves shown in Figure 4-1, Figure 4-5 shows that northerly seas and swells dominate the nearshore wave spectrum. However, there are several key differences between offshore and nearshore wave conditions:

- In the nearshore, swell waves contribute a larger fraction of the total wave energy from May to September than they did at the Pt. Reyes Buoy. The nearshore energy of swell and sea waves are comparable during these months.
- Southerly waves take up a larger fraction of the total energy in the nearshore compared with offshore.

The first point is indicated by comparing the upper panels of Figure 4-1 and Figure 4-5. At the offshore buoy location (Figure 4-1), seas were the dominant source of wave energy between May 1 and September 30. The second point is evident from the differences between the lower panels of Figure 4-1 and Figure 4-5. While northerly waves still account for the majority of the wave energy at GRSB, southerly waves have comparable energy in May, June, and October.

As observed with the offshore wave field, the majority of the nearshore wave energy arrives in the month October, at the end of the management period. While wave energy peaks above 10,000 cm^2 can occur in May and June, October was the only month with consistently high wave energy between 1997 and 2011. Figure 4-6 shows time series of the predicted nearshore wave height and the tide level at the Pt. Reyes tide gage for the ten year period from 1999 to 2009.

4.3.3 Estimating Wave Runup and Overtopping

The predicted nearshore waves were used to provide hourly time series of wave setup, runup and overtopping from 1997 to 2011. To account for the lack of wave period data near the coast, we assumed that the peak wave period (T_p) measured offshore remained the same throughout the transformation (i.e. that offshore and nearshore wave trains have similar peak periods). We accounted for the lack of wave direction data by assuming that breaking waves on the shore face are perpendicular to the shoreline. Sensitivity of runup and overwash estimates to these parameters is discussed in Appendix A.

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 H0 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 Hs 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.

Wave runup is the maximum vertical extent that the leading edge of a breaking wave reaches on the shore (Figure 4-7b), 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, 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 4-7b). 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):

$$\begin{aligned} \frac{Q}{\sqrt{gH_S^3}} &= \frac{A}{\sqrt{B}}\,\xi \exp\left(-B\frac{R_C}{\gamma_r \xi H_S}\right) && \xi \le 2 \\ \frac{Q}{\sqrt{gH_S^3}} &= C \exp\left(-D\frac{R_C}{\gamma_r H_S}\right) && \xi \ge 2 \end{aligned} \tag{4.4}$$

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.

As a baseline, we use topographic data from the September 2010 LiDAR survey (CCC, 2012) to approximate the beach slope and crest height for each 100 ft segment (Figure 4-8a). The beach's total overwash rate was then taken as the sum of the overwash from all segments. Overwash volumes were summed for each month from January 1997 to December 2011. Figure 4-8b shows the average and standard deviation of the overwash volume in each of the months of the management period. The baseline monthly overwash volumes were negligible for all months except for October, which reflects the earlier result in Figure 4-1 and Figure 4-5 showing that wave energy was highest for this month.

4.3.4 Sensitivity of Wave Overwash to Beach Geometry and the Presence of the Jetty

To determine how sensitive the predicted overwash is to seasonal variations in beach morphology, we also calculated the 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 4-2.

To assess the influence of the jetty, we consider 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. A more extensive comparison of these and other sites is provided in Section 5.

The September 2010 LiDAR flight collected data from all three sites within a two-week period, so the beaches likely experienced similar ocean conditions antecedent to the LiDAR data collection. The Russian River 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 4-8a, the beach crest at GRSB is well-approximated by a linear fit 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. James Hall). The slope of the beach 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). If the jetty were not present at GRSB, inlet migration south of the present-day groin could have periodically lowered the beach, influencing the potential for wave overwash.

Figure 4-8b-d shows the expected overwash volume for each of the months in the management period using linear fits of the along-beach crest profiles at the beaches of the Russian, Gualala, and Navarro Rivers. In all cases, the wave conditions are the same January 1997 to December 2011 nearshore (Russian River) conditions described in the preceding section. The GRSB wave conditions were applied to beach crest profiles that represent each of the three sites, to determine the sensitivity of overwash to different beach profiles. The results are also summarized in Table 4-2. By shifting the beach profile from one with limited yearly inlet migration (Russian River) to one with 5-10 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.

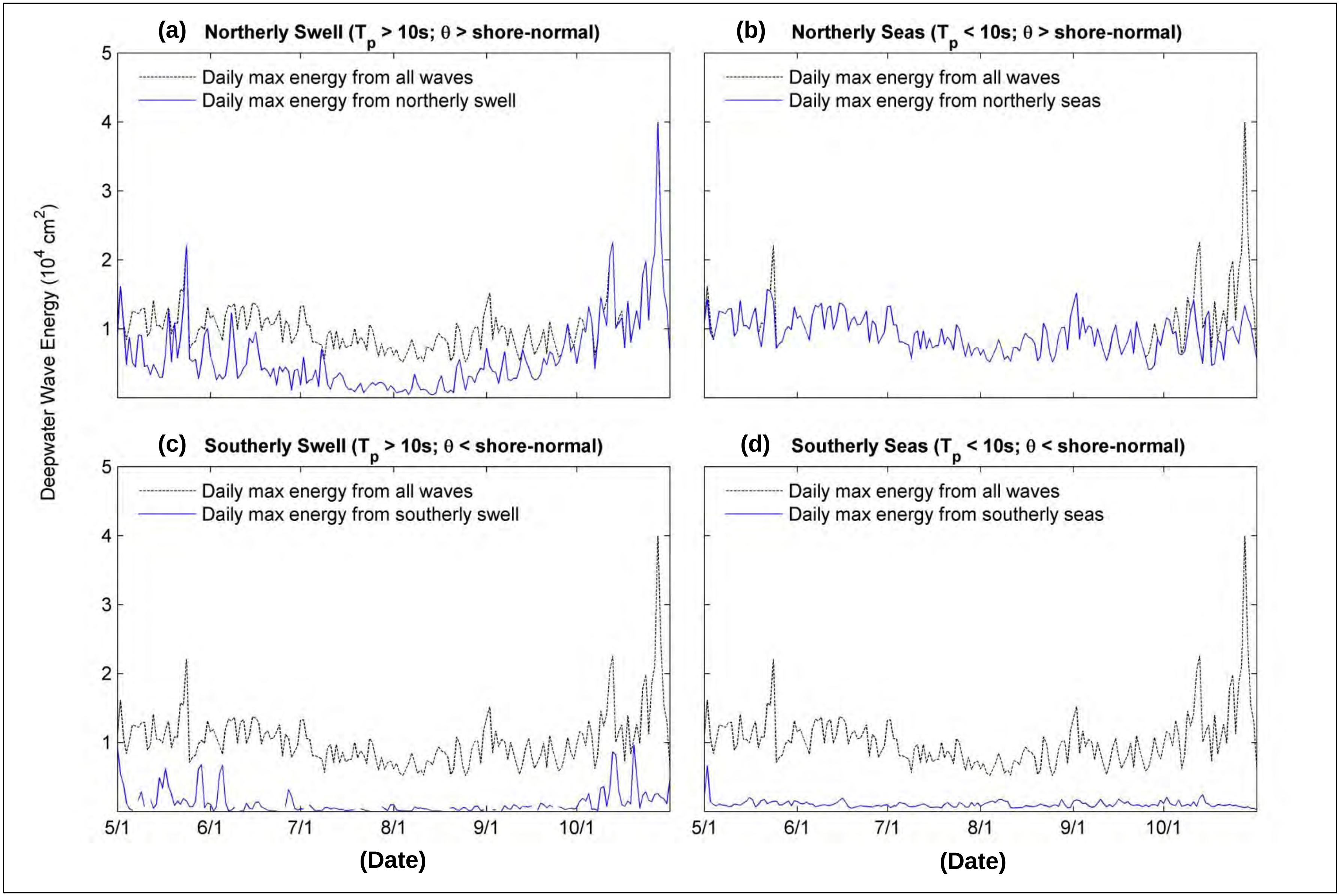
Table 4-2. Expected monthly wave overwash volumes (acre-ft/month).

	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 2011. Reported monthly averages are followed by ± the standard deviation of the sample.

[1]Behrens (2012)

4.4 Figures

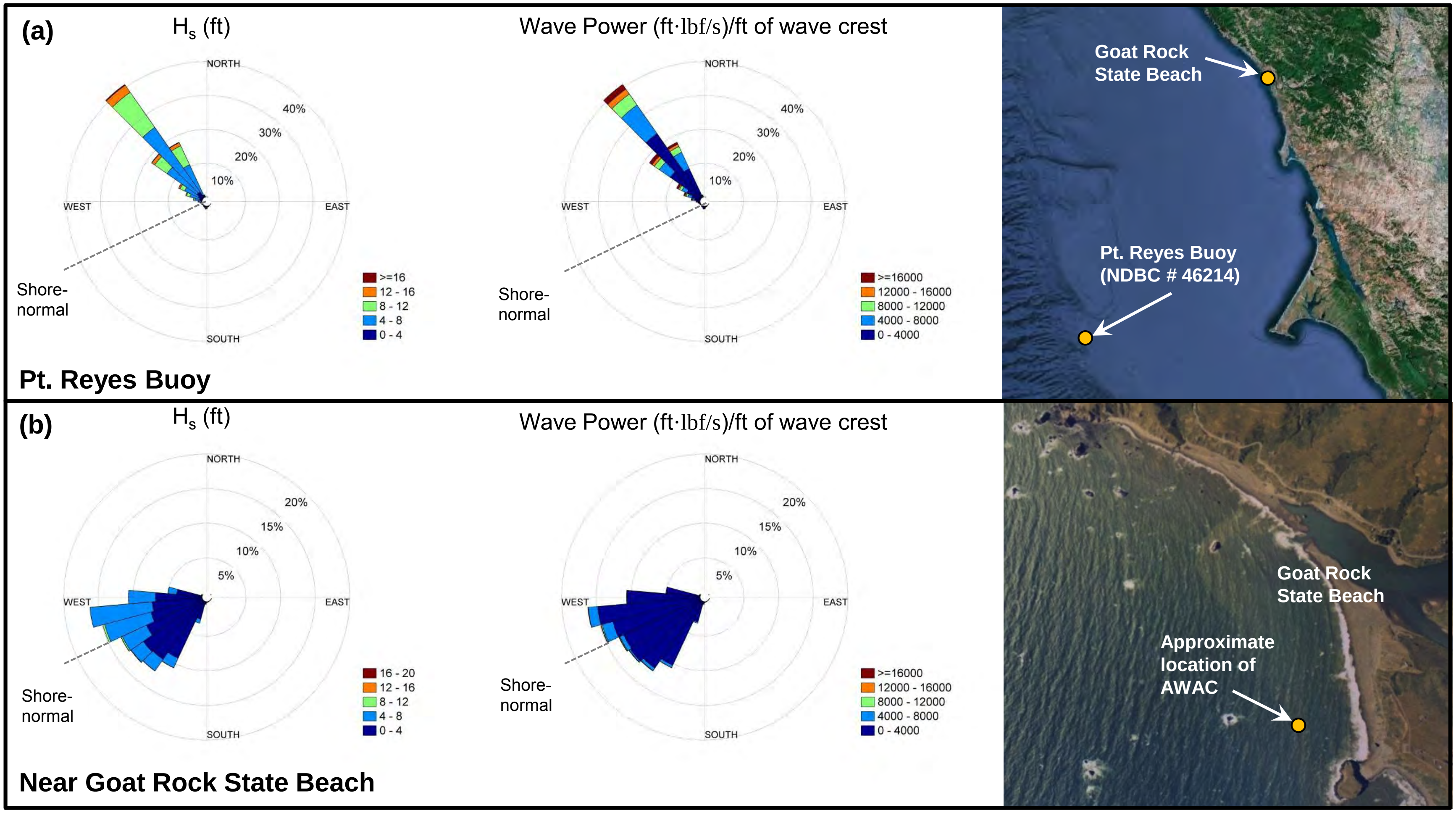


SOURCE: NDBC Buoy # 46214

NOTE: Shore-normal direction is approximately 255° measured from north.

Figure 4-1

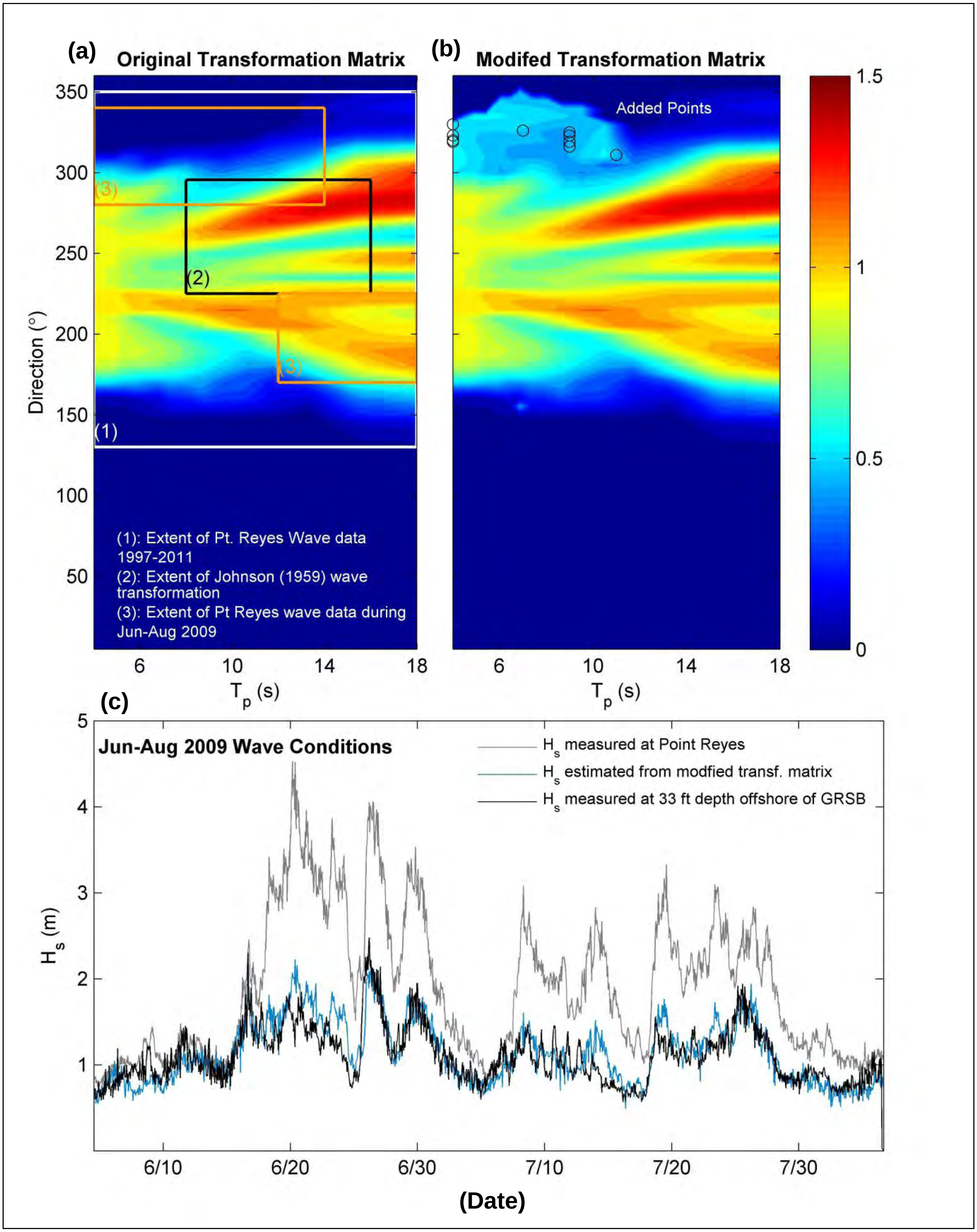
Daily aggregated measurements of offshore (Pt. Reyes) wave energy grouped by northerly and southerly swells and seas



SOURCE: NDBC Buoy # 46214, BML (2012)

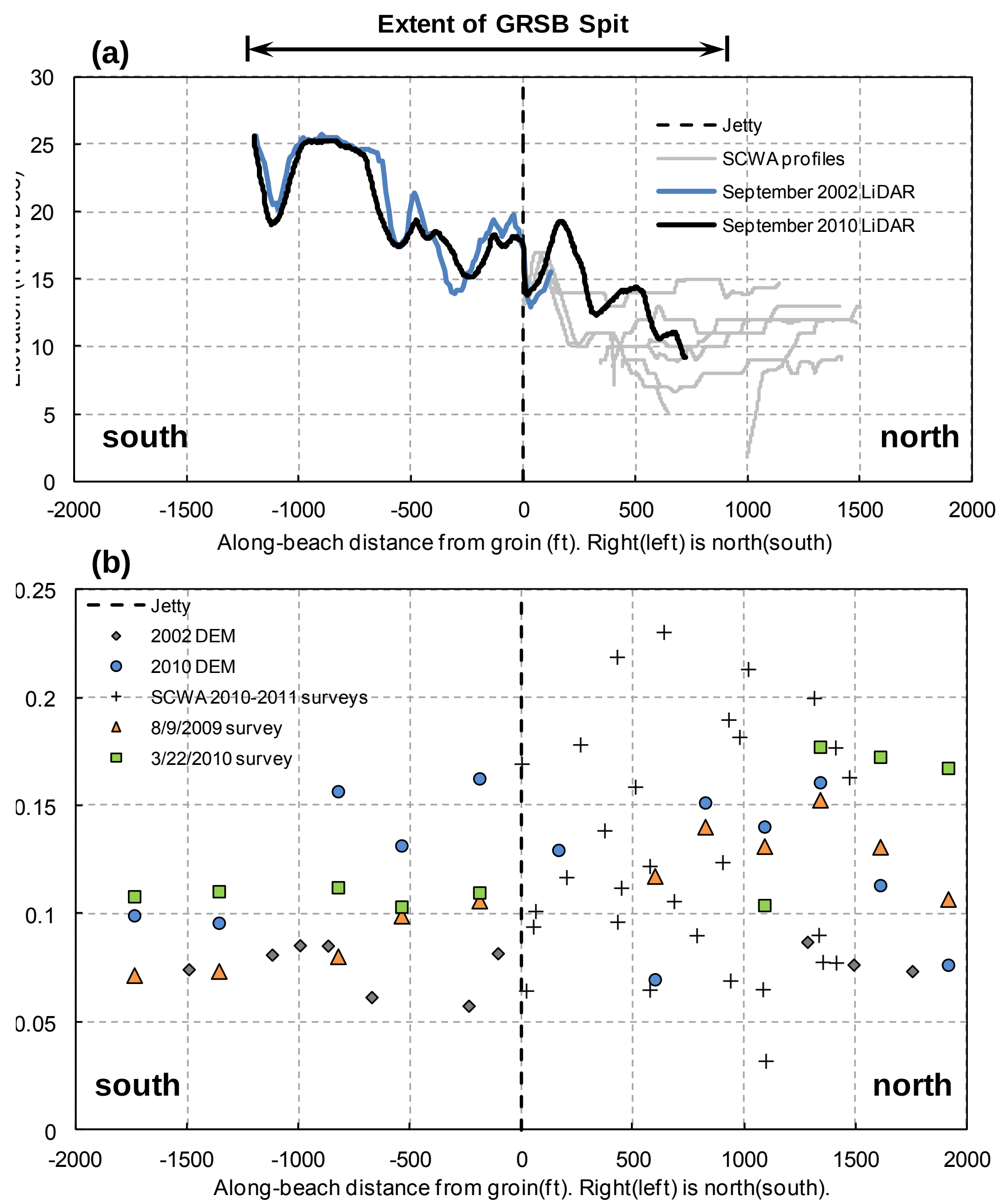
NOTE: Shore-normal direction is approximately 255° measured from north. “AWAC” stands for Acoustic Waves And Currents, referring to a wave and current measuring device produced by Nortek©.

Figure 4-2
Comparison between dominant wave conditions at **(a)** the Pt. Reyes Buoy and **(b)** near Goat Rock State Beach.



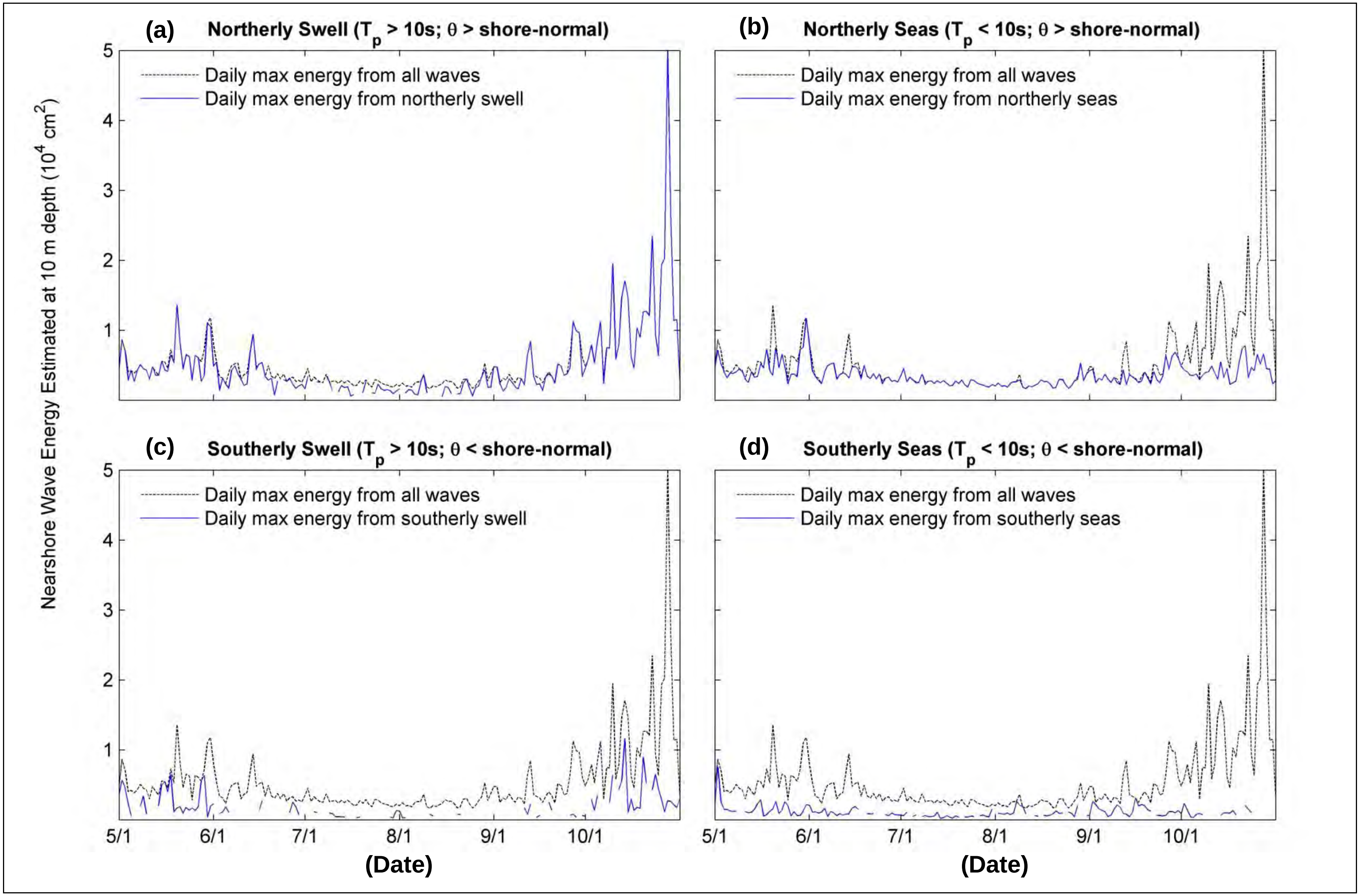
SOURCE: Original transformation matrix (a) obtained from W. O'Reilly of CDIP. Modifications to transformation matrix (b) made using BML wave measurements near GRSB.

Figure 4-3
(a,b) Wave transformation matrices used to obtain nearshore wave estimates and **(c)** Summer 2009 wave conditions



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

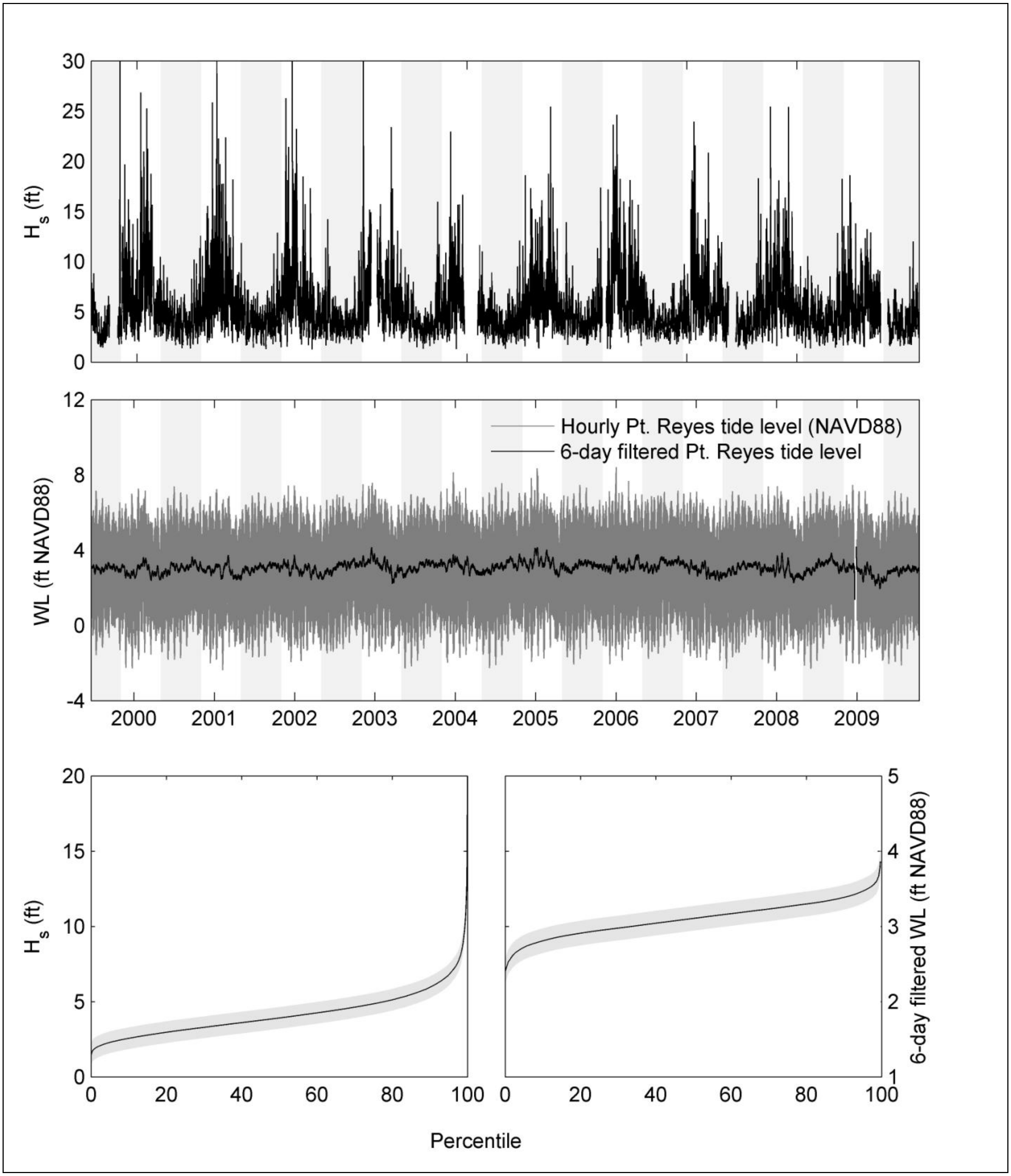


SOURCE: Offshore waves are from NDBC Buoy #46214. These were transformed with the CDIP buoy data to obtain nearshore wave energy.

NOTE: Shore-normal direction is approximately 255° measured from north.

Figure 4-5

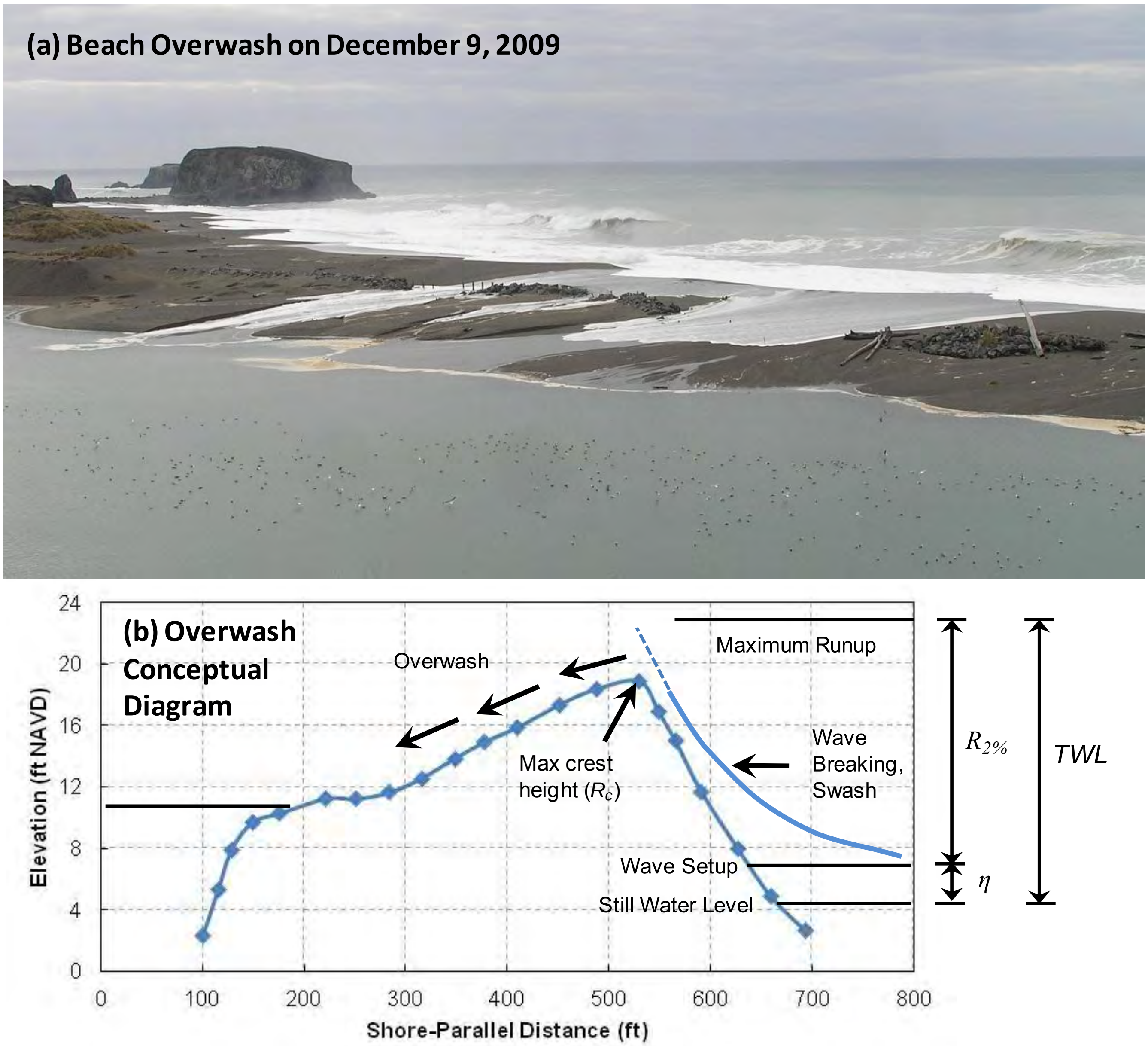
Daily aggregated nearshore estimates of wave energy, grouped by offshore wave direction and period into northerly and southerly swells and seas



SOURCE:NDBC Buoy # 46214, Pt. Reyes Tide Gage. Nearshore estimates obtained using offshore buoy data with a modified version of the transformation matrix provided by CDIP.

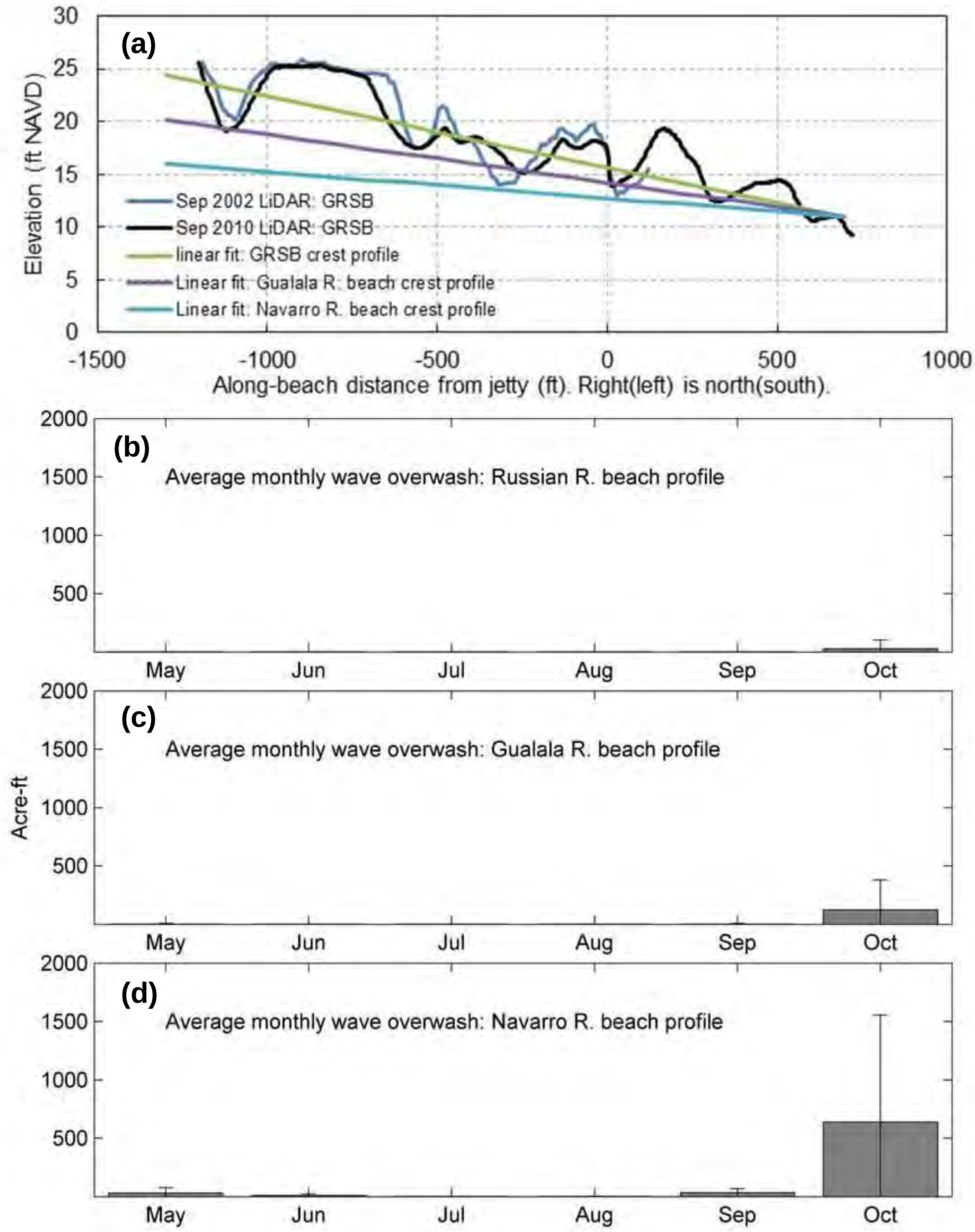
Figure 4-6

(a) Nearshore wave height estimates and **(b)** Pt. Reyes water level from 1999-2009, and **(c,d)** percent exceedance curves for both.



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

Figure 4-7
(a) Example of wave overwash at GRSB and **(b)** conceptual drawing of wave overwash.



NOTE: Average monthly wave overwash volumes calculated using nearshore wave estimates from January 1997 to December 2011.

Figure 4-8
(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 (Johnson, 1959; de Graca, 1976). 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 levels 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 a 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 US 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 levels 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^3yr^{-1} (Slagel and Griggs 2008) and181,000 yd^3yr^{-1} (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 1945-1970 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 are 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 levels 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. Rising sea levels associated with global warming are changing as a result of both thermal expansion of water (e.g. warmer water occupies more volume) and increasing ice melt. 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.

Sea level rise encourages shoreline retreat by raising the elevation of tides and waves relative to the beach. In general, coastal erosion is caused primarily by the combination of high waves and tides. As sea level rises, the amount of time that total water levels (combined mean sea level, tides, and wave runup) exceed a critical value increases. This increases exposure of the dune fields or cliffs behind the beach, leading the back extent of the beach to retreat simultaneously.

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 (1) local atmospheric circulation and (2) 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 San Francisco region vary in magnitude based on the type of climate change scenario used in the prediction: NRC (2012) give an expected range of approximately 16.7 – 65.5 inches of sea level rise by 2100.

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

- The length (*L*) from the crest of the berm to the farthest offshore depth where seasonal erosion or accretion occur.
- The vertical distance (*H*) that spans from the crest of the berm to the location where this offshore point exists
- The long-term change in water level ($\Delta\eta$) associated with SLR

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 SLR values of 1, 3, and 5 ft as representative values within the expected range of NRC (2012). To examine the potential effects of the jetty components, we looked at survey transects with and without jetty components present. Figure 5-7 shows the projected response of the beach to the range of SLR values south of the groin at Transects 7 (north of the groin) and 11 (south of the groin: Figure 5-1, Figure 5-3). If no background beach accretion is present, the beach is predicted to move landward by amounts of 30.1, 60.5, and 150.8 ft for the cases of 1, 3, and 5 ft of SLR, respectively (Figure 5-7).

For the part of GRSB north of the groin and beyond the extent of typical inlet migration (Transects 1-4; Figure 5-1), this 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; Figure 5-1). For transects with no backbarrier dunes or cliffs, the beach would migrate landward as SLR progresses, without any major changes to barrier width. For jetty access elements, such as the seawall, which is embedded in the beach, this change would bury the structure over time (Figure 5-7), which could alter berm seepage depending on the position of the seawall relative to the regions with fastest seepage flows. 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 structure is well above mean sea level at approximately 14.5 ft NAVD88 (as described in Section 2), the exposure of the groin at the seaward extent would allow the groin to interrupt the littoral drift in the vicinity of the inlet. We have not modeled these shape changes in Figure 5-7, but rather for simplicity of presentation and analysis assumed that the profile shape was constant.

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 will move inland versus the historic trend of adequate sediment supply resulting in shore accretion (seaward and upward building) and essentially being "stabilized", in place. If we consider 1930-2012 background accretion rates (Figure 5-3) in the Bruun Rule analysis, the movement caused by sea level rise is slowed, causing the beach to raise in height but adjust more slowly in the horizontal, as has been the case since approximately 1990. As with the previous case (no shoreline accretion), this would result in burial of the jetty elements over time.

5.5 Synthesis

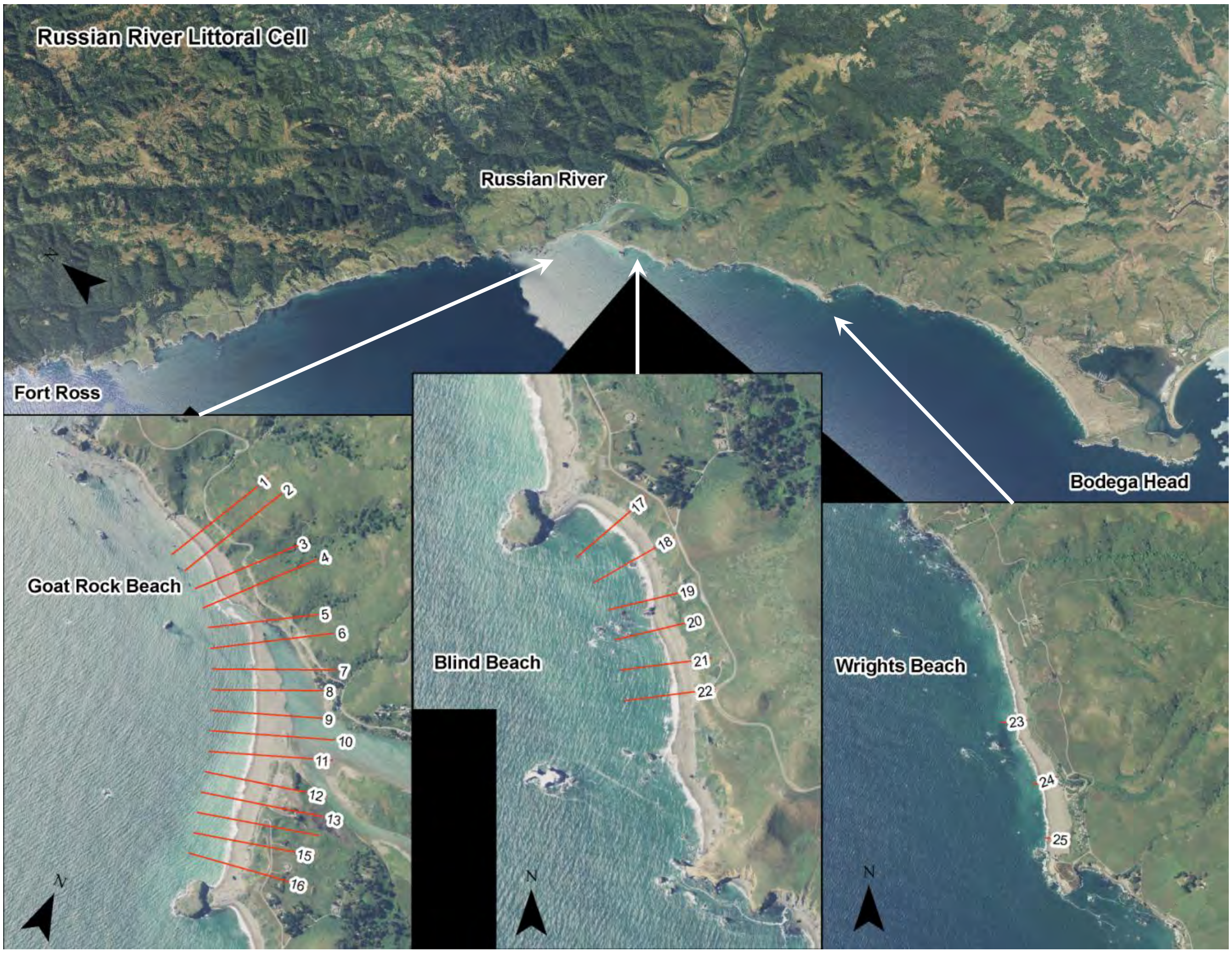
The beach berm at GRSB has changed significantly since the initial construction of the jetty complex between 1924 and 1934. This has implications for the lagoon water level, because the beach acts as the barrier separating the lagoon from the ocean, enabling the lagoon water level to perch above the 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 disruption of the littoral transport behind Goat Rock, but 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 a small proportion 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.

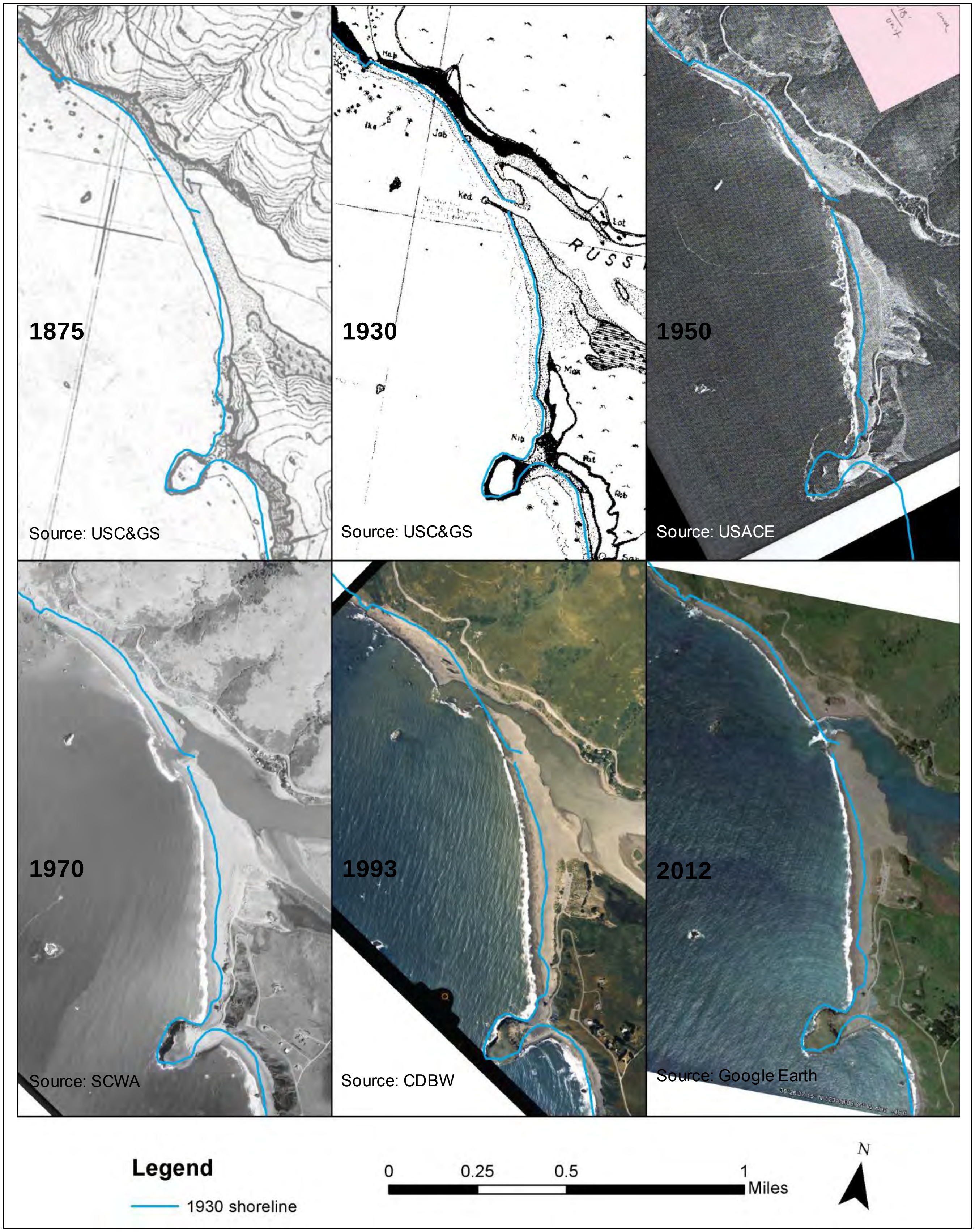
In response to SLR, GRSB is expected to migrate landward. The extent of the landward adjustment will vary depending on future rates of sediment supply to the beach and the continued adjustment of the beach to the disruption of the littoral transport at Goat Rock. The vertical adjustment of the beach will likely bury the existing jetty access elements and most of the groin. To the extent that the beach moves landward, the seaward edge of the groin could become 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 occasional southward migration, and increased water outflow through the sand berm to the ocean.

5.6 Figures



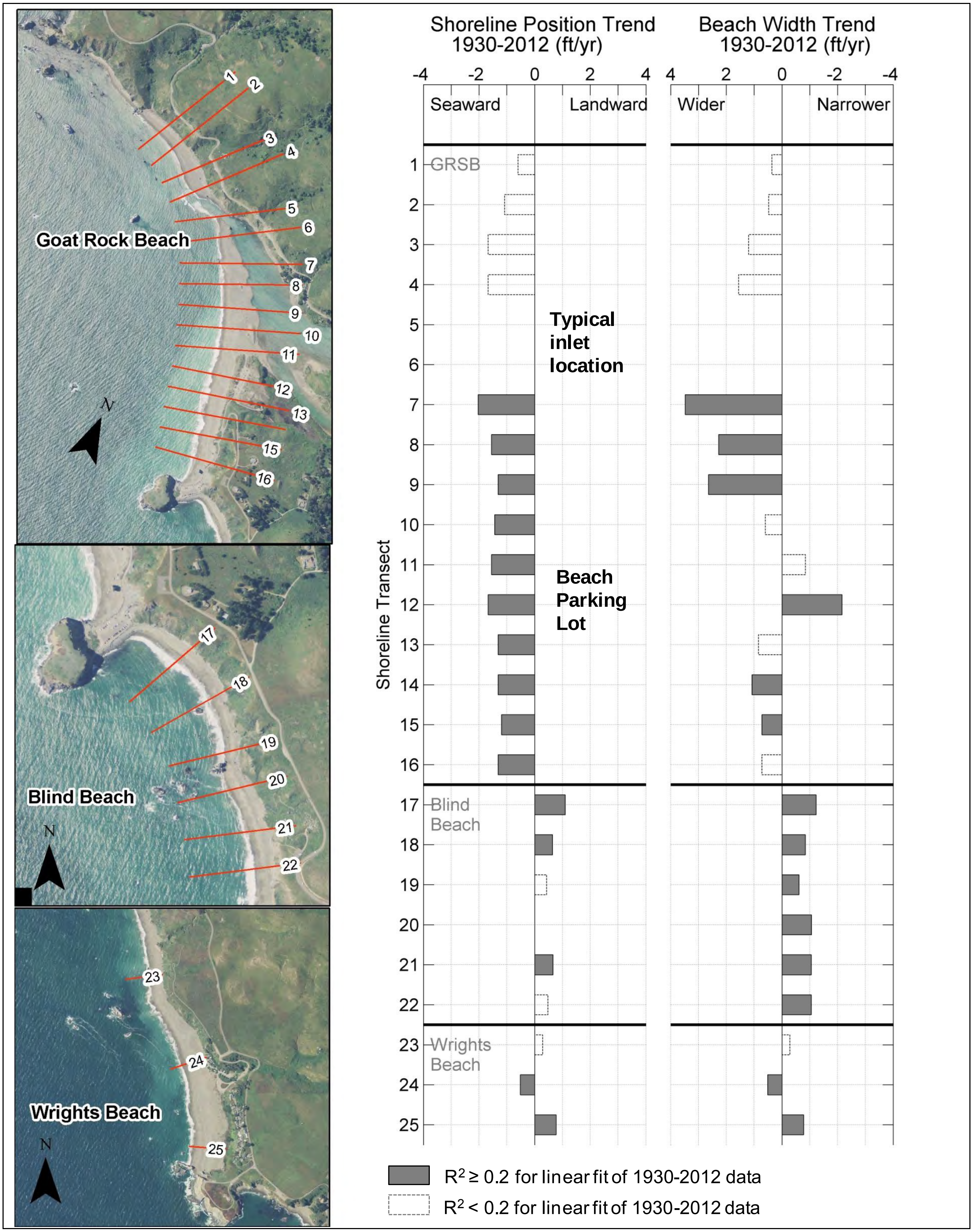
NOTE: Bars shown in insets represent transects used in the analysis of historical images. Background image in all panels is an orthophoto taken in 2009 by the USDA NAIP.

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

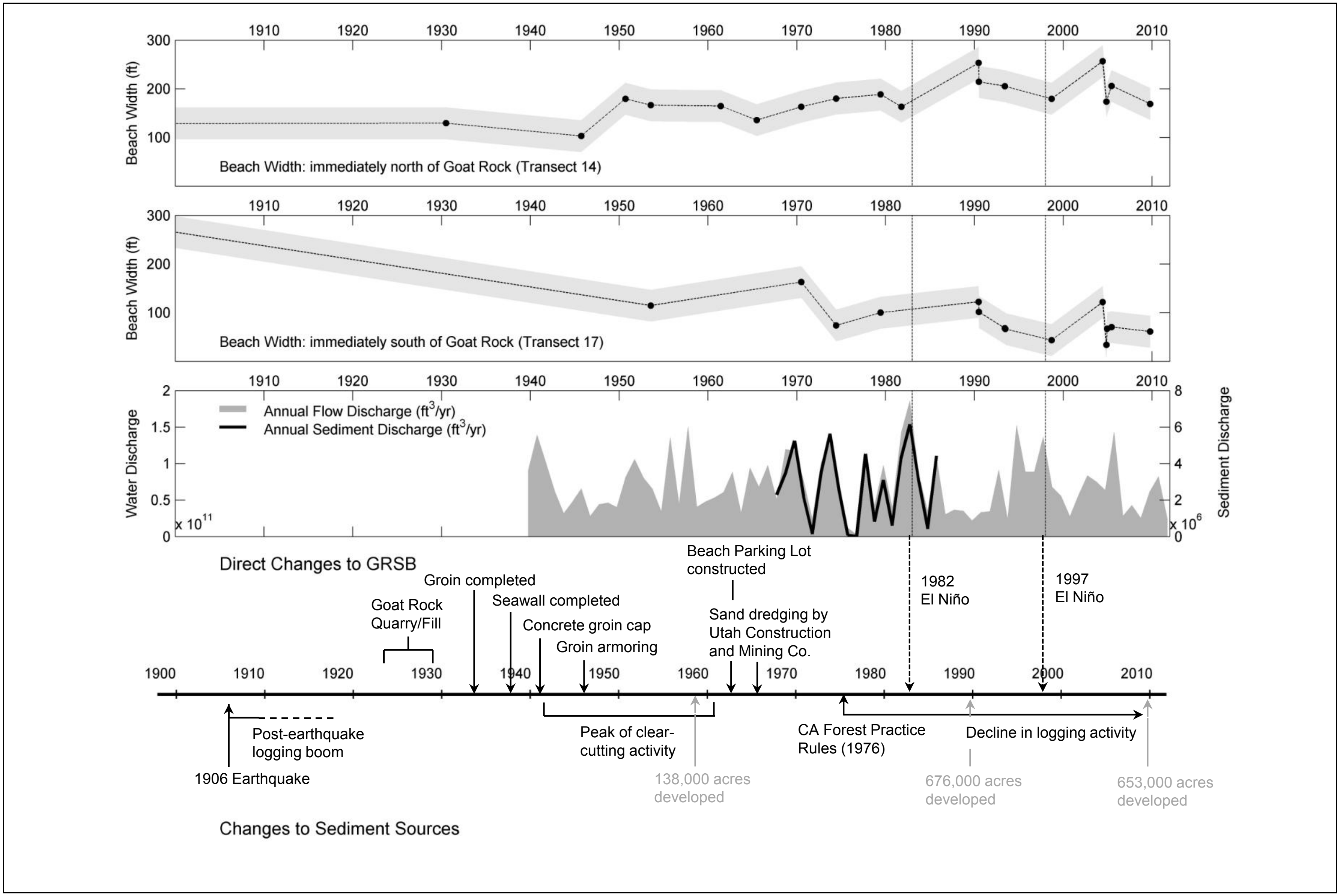
Figure 5-2
Aerial views of GRSB and the northern extent of Blind Beach taken from T-sheets and aerial imagery.



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

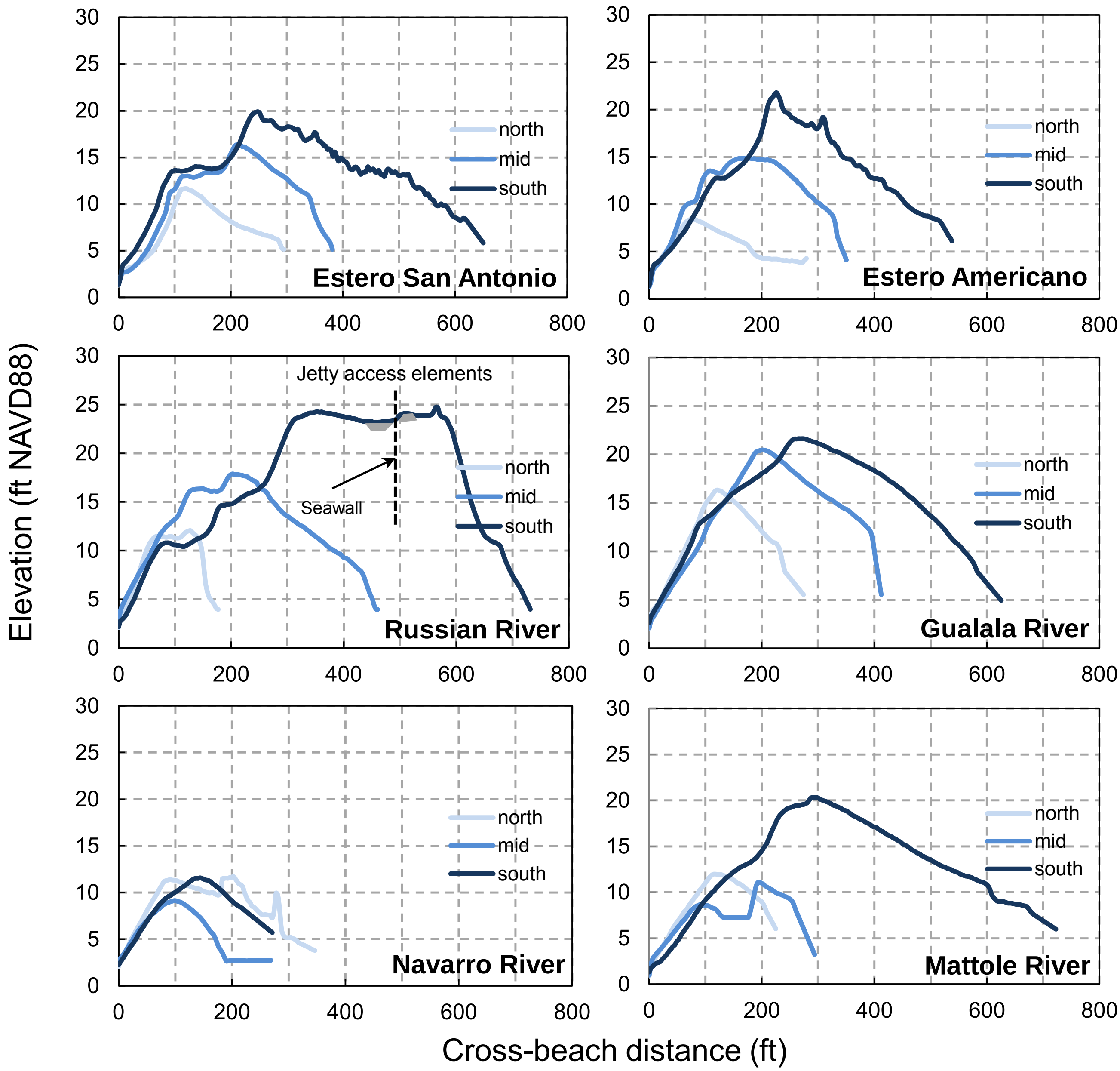
(left) Beach transects and **(right)** corresponding trends in shoreline position and beach width



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

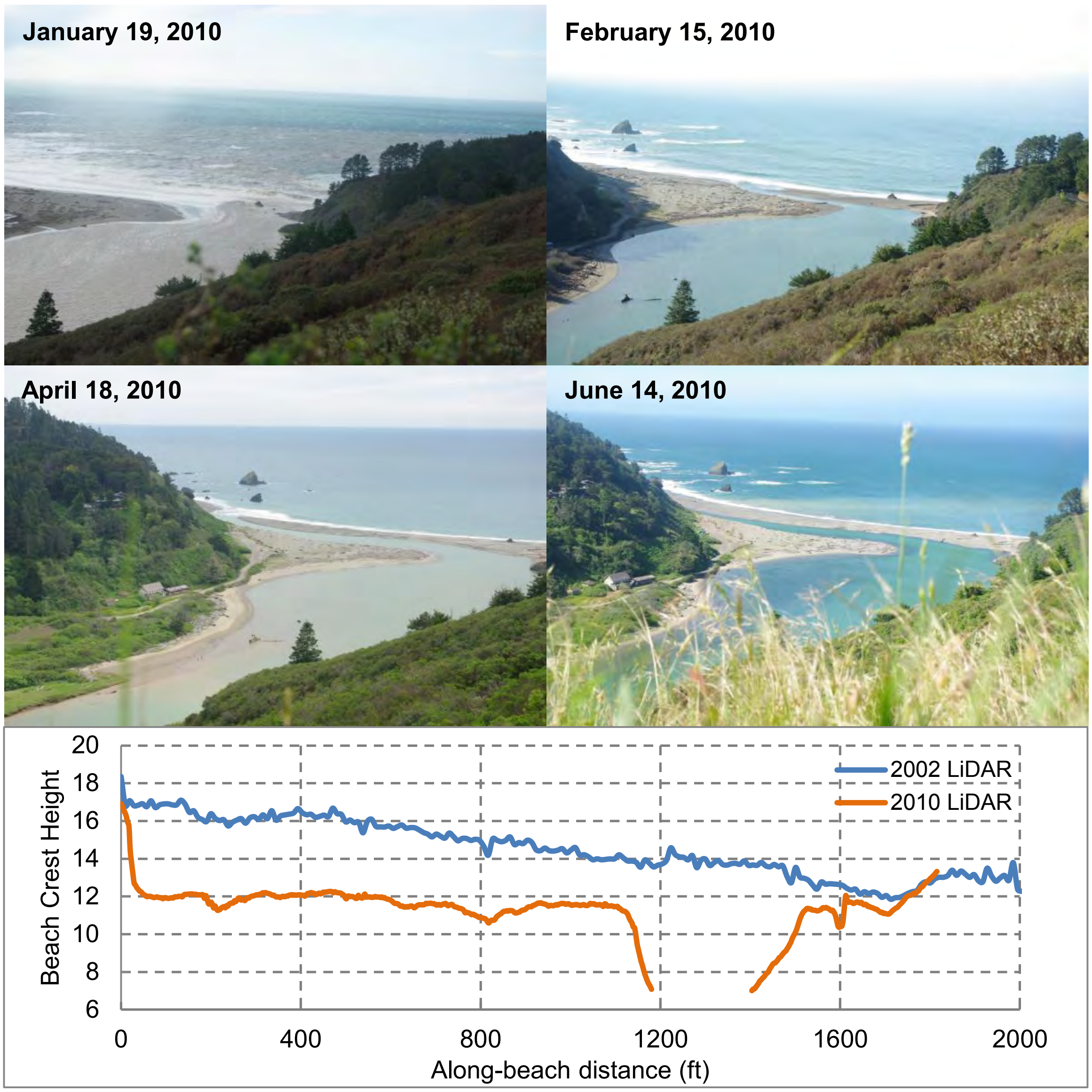
References for Changes to Sediment Sources: Arvola (1976), CDC (2010), DWR (1964), Florsheim and Goodwin (1993), Merenlender et al. (1998), Opperman et al. (2005), Runyan and Griggs (2002), Slagel and Griggs (2008), SEC (1996), Waters (1995), Willis and Griggs (2003)

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



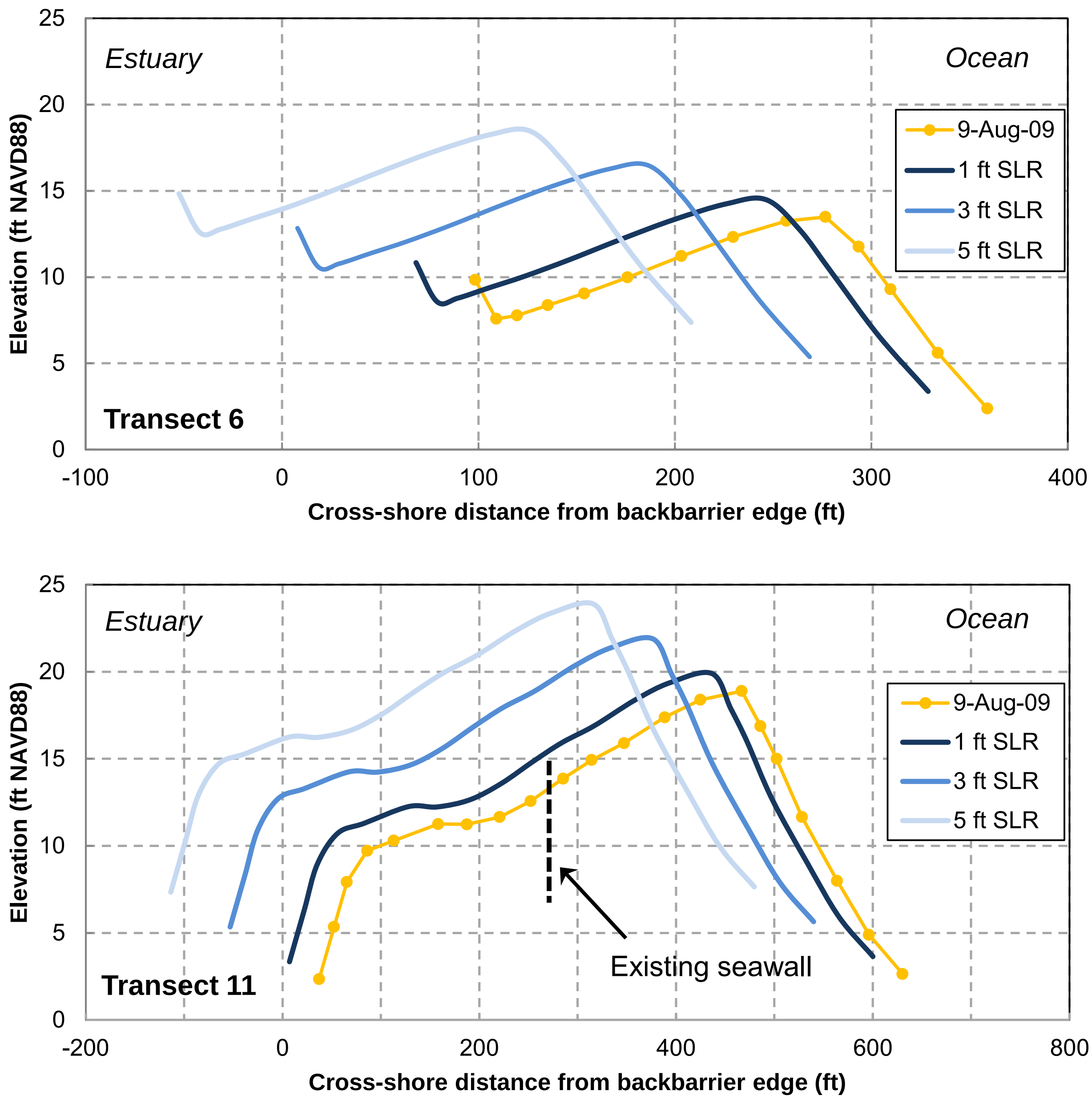
NOTE: Transects were only taken on the spit between the estuary and ocean at each site. Sites were chosen based on similarity to the Russian River Estuary. All sites are located within 200 mi. of GRSB.

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

Note: This section only provides a brief introduction on the relationship between the jetty and inlet morphology. The technical analyses related to inlet morphology are anticipated for 2013 and results of those analyses will be summarized in a revised version of this report.

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. Inlets are very dynamic systems 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. The inlet is characterized by dynamic morphologic change in response to changing conditions. To the extent that the jetty affects these dynamic changes, it could also affect inlet functions such as alongshore sediment transport, which, in turn, can affect groundwater seepage, wave overtopping and breaching characteristics. Hence, interventions such as the jetty can impair some functions, and then require a succession of interventions to mitigate functional degradation. Therefore, removal of the jetty could be beneficial not just to inlet morphology, but also to its related functions, such as sediment transport and ecosystem renewal.

To achieve the Biological Opinion target conditions of a freshwater lagoon perched above tides and below flood stage requires the inlet morphology to be changed from its most common state as a tidal inlet to either a closed or an outlet channel state. For the anticipated river inflows and existing seepage rates, a closed channel would typically lead to lagoon water surface elevations rising beyond flood stage, thereby necessitating management to create an outlet channel. Therefore, this assessment focuses on the jetty's possible effects on the frequency and duration that an outlet channel can be established. The jetty may also affect the frequency and duration of muted tidal conditions. Even though a muted tidal state does not fully achieve the Biological Opinion target conditions, it may provide some benefit to salmonid rearing habitat by increasing the depth of the fresher surface layer. The jetty's potential effects on muted tidal conditions will be assessed, but may be difficult to discern since these conditions are often transitional, and are less distinct from other states.

The jetty may have a direct effect on estuary water surface elevations if it changes the frequency at which the inlet changes between the three morphologic states (tidal/closed/outlet). Whether or not the jetty's effect is favorable to the target estuary water surface elevation depends on which state changes the jetty effects. Transitioning out of tidal inlet conditions is a prerequisite to achieve target water surface elevations and therefore is favorable, particularly if it occurs sooner in the management period. In some instances, this transition may include a period of partial closure and muted tides. The jetty may shorten or prevent these muted tidal periods, perhaps by forcing a southward migrating channel to close. Most likely, tidal conditions will change to closed rather than an outlet channel. Conversely, breaching of closed or outlet state to tidal returns tides and salt to the Estuary and is least favorable from the perspective of Biological Opinion habitat management objectives. A state change from closed to outlet channel is favorable, but probably requires management action and at present does not involve the jetty. Closing of an outlet channel is likely to cause flooding if an outlet channel is not re-established by management action.

Because the changes to inlet state are threshold events within a dynamic system, efforts to create predictive models have taken the form of quantifying geomorphic relationships with empirical data. For example, the state change from tidal to closed has been examined in some detail for the Russian River (Goodwin and Cuffe, 1994; Behrens et al., 2013). In addition to the tidal-closed shift, Battalio et al. (2007) also quantified the change from closed to tidal.

There are several ways in which the jetty may alter estuary water surface elevations by altering state changes between inlet morphologies. As noted above, whether or not these state changes are favorable to achieving target estuary water surface elevations is a function of the specific pair of before/after states. The potentially significant linkages between the jetty, inlet morphology, and estuary water surface elevation include:

- The jetty restricts the southern migration of the inlet. As shown in Behrens et al. (2009), during the management season the inlet typically oscillates between the jetty and 300 ft to the north. When the inlet develops along an alignment exiting the Estuary north of the jetty and angling south across the barrier beach, the jetty may cause the inlet to pinch off and close rather than continue migrating further south. Under these conditions, promoting tidal inlet closure would be favorable; promoting outlet channel closure would be unfavorable (but manageable).
- If the inlet is located parallel to and just north of the jetty and not being forced south against the jetty to the point of closure, the jetty may help the inlet resist closure. This was the original purpose of the jetty (Magoon et al., 2008) and historic wave analysis indicates that the jetty is located near the minimum wave energy for a range of wave conditions (Johnson, 1959). Under these conditions, preventing tidal channel closure would be unfavorable; preventing outlet channel closure would be favorable. Also, the threshold for breaching is probably lower in this condition, given the degraded beach elevation and width that results from recurrent breaching in this one area.

In addition to these direct linkages between the jetty, inlet morphology, and estuary water surface elevation, the jetty may affect other aspects of the system, which then alter water surface elevations. These linkages are noted here and discussed in the other relevant sections on physical process:

- The potential for the outlet channel to scour and convert to a tidal inlet is a function of the flow rate it must convey. The outlet channel's flow rate is dependent on Estuary inflow and seepage flows through the barrier beach, which may be influenced by the jetty if it alters the barrier beach's permeability (Section 3).
- The inlet is a feature incised in the barrier beach, and so depends, to some degree, on the jetty's effects on the overall beach morphology (Section 5). For example, if the jetty alters the width of the beach, this may affect the bed steepness in the outlet channel, which in turn, may influence the channel's sediment transport capacity.

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 levels 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 environmental review under CEQA.

Properties along the Russian River Estuary shoreline are at risk from flooding events within the Estuary. Flood events are associated with heightened water levels 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 levels 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 levels 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 tides or freshwater flows. When the inlet is closed, the Goat Rock State Beach (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 levels 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 s) 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 dams (Florsheim and Goodwin, 1993), which tend to reduce peak flood flows. However, severe fluvial flood events still occur. Some of the most severe floods have occurred during 1982, 1986, 1995, and 2006. These flood events tend to increase the water level in the lower Estuary (Figure 7-2 and Figure 7-3). Since the Estuary is much wider than the channel at the USGS Hacienda Bridge flow gage station

[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.

(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 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 manually breaches the beach berm to prevent flooding when water levels threaten to exceed 9 ft NGVD. 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 level) 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 level 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 (dry-season scenario): A late-fall inlet closure event with beach-building processes causing the beach to impound high water levels. Wind waves generated within the lagoon add to the flood stage.
- Scenario two (wet-season scenario): A winter fluvial flood event elevates estuary water levels.
- Scenario three (wave-transmission scenario): An oceanic wave passes through the inlet and breaks along the shoreline.

For the dry-season flooding scenario, we treat the inlet as closed, with an elevated beach crest creating a lagoon that can support flood water levels. Inside the Estuary, the water level is assumed to reach the height of the beach crest; any higher and the water would overtop the beach, controlling water levels and scouring a new, lower channel. In addition to the lagoon water level, 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 reopening 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 level 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 at flood stage. The stage in the Estuary is taken as the sum of: (1) a typical river stage 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 levels may be higher than actual water levels, 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 height 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 Scenario Components

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

7.2.1 Flood Stage During Dry-Season Conditions

When the inlet is closed by a continuous spit along GRSB, peak water levels in the Estuary 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 Tide Gage from 1999 to 2010.

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 levels (i.e. estuary stage 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 washover 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 levels measured at the Jenner Tide Gage (Figure 7-2) were taken from closure events between 1999 and 2010. Peak water level occurred at the end of closure events (i.e. immediately prior to breaching). We only included water level values taken from closure events that resulted in a natural breach, since stages during manual breach events may not be reflective of the actual beach crest height. We used statistical methods to obtain expected water levels during peak events at certain recurrence intervals. Peak annual water levels 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 (12 years) of the estuary water level data means that predicted values for different recurrence intervals are associated with high uncertainty.

Dry-season flood stage estimates from the two methods described above are summarized in Table 7-1. The two methods result in an overlapping range of beach crest heights (and hence flood stages).

Table 7-1. Potential flood stages during a dry-season flood event.

	Beach crest height[1] (ft NGVD)
From Jenner stage data	
Maximum observed stage[2]	11.1
10-year stage (Gumbel Distribution)[3]	10.5
50-year stage (Gumbel Distribution)[3]	13.5
100-year stage (Gumbel Distribution)[3]	14.8
From LiDAR and Water Agency surveys	
Aggregate of Water Agency surveys	14.5
Back barrier toe heights[4]	15.5-17.5

[1]Impounded water in Estuary assumed to be at this height.
[2]Observed during an inlet-closure period on November 13, 2001, immediately prior to a natural breach event.
[3]Stage during closure conditions. Extrapolated from 1996-2010 stage data in the Estuary. Pre-breach water levels are available prior to 1999, but not continuous water level time series.
[4]Range reflects average back barrier toe heights from 2002 and 2010 LiDAR surveys (CCC, 2012).

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 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 level. 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 levels in the coastal region.

For comparison, the Water Agency's water level gage at the Jenner Visitors Center was used as a reference for fluvial flood elevations. 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 levels 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 shoreface, 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.

A constant value of H/d = 0.78 was used to approximate wave breaking during the travel from offshore coastal waters to the inner Estuary (USACE, 2005), where H is the wave height and d is the local depth. 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 as 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 is stage 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 (Dry-season flood)	• Gumbel Dist. of Jenner peak stages during closures[1]	14.8
	• 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 Comparison with limited lagoon stage data during recent floods	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
	• 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

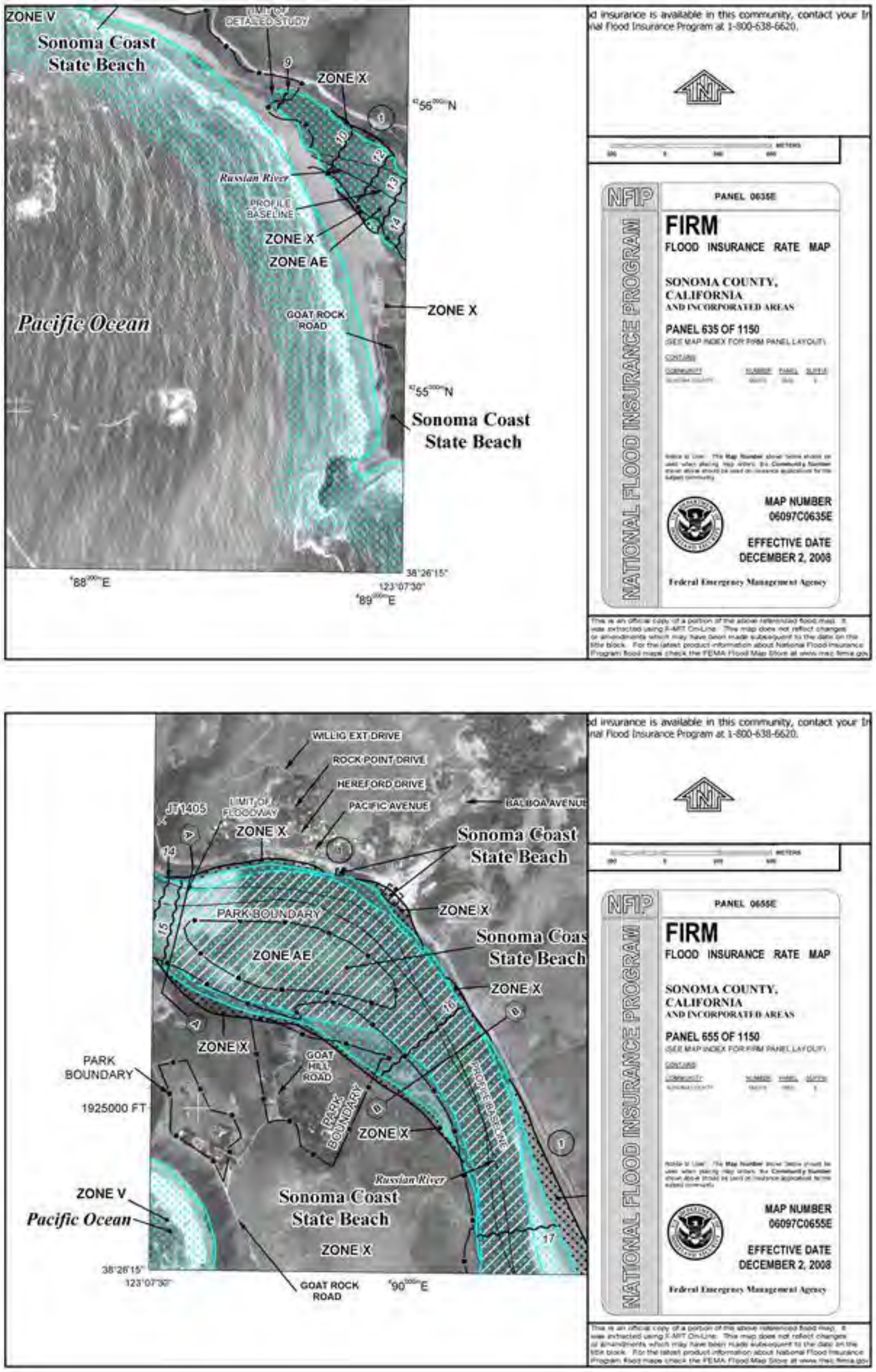


Figure 7-1
FEMA Flood Insurance Rate Map (FIRM) of Jenner, CA.

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.

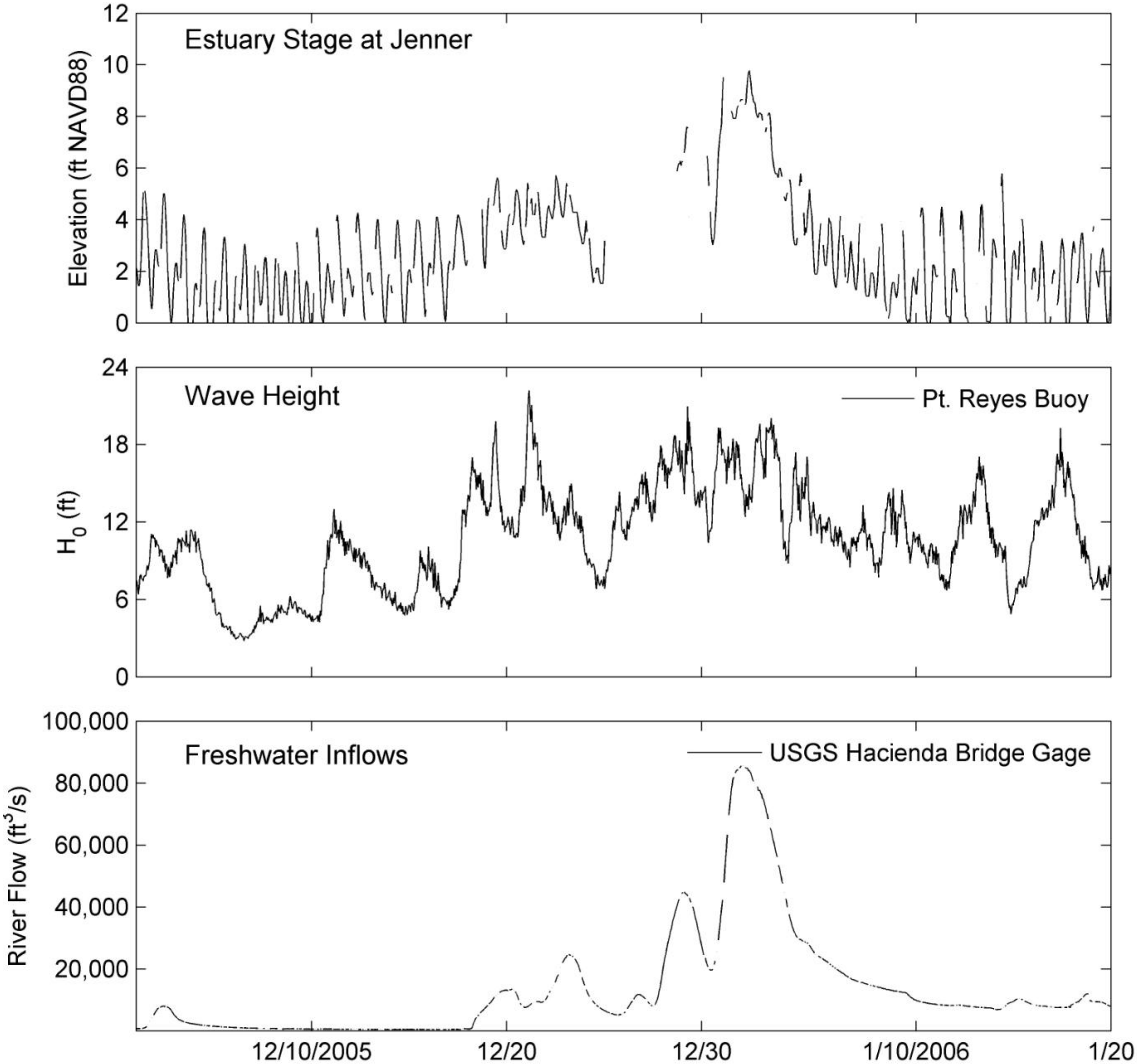


Figure 7-3
Example of a wet-season flooding event, driven by fluvial conditions.

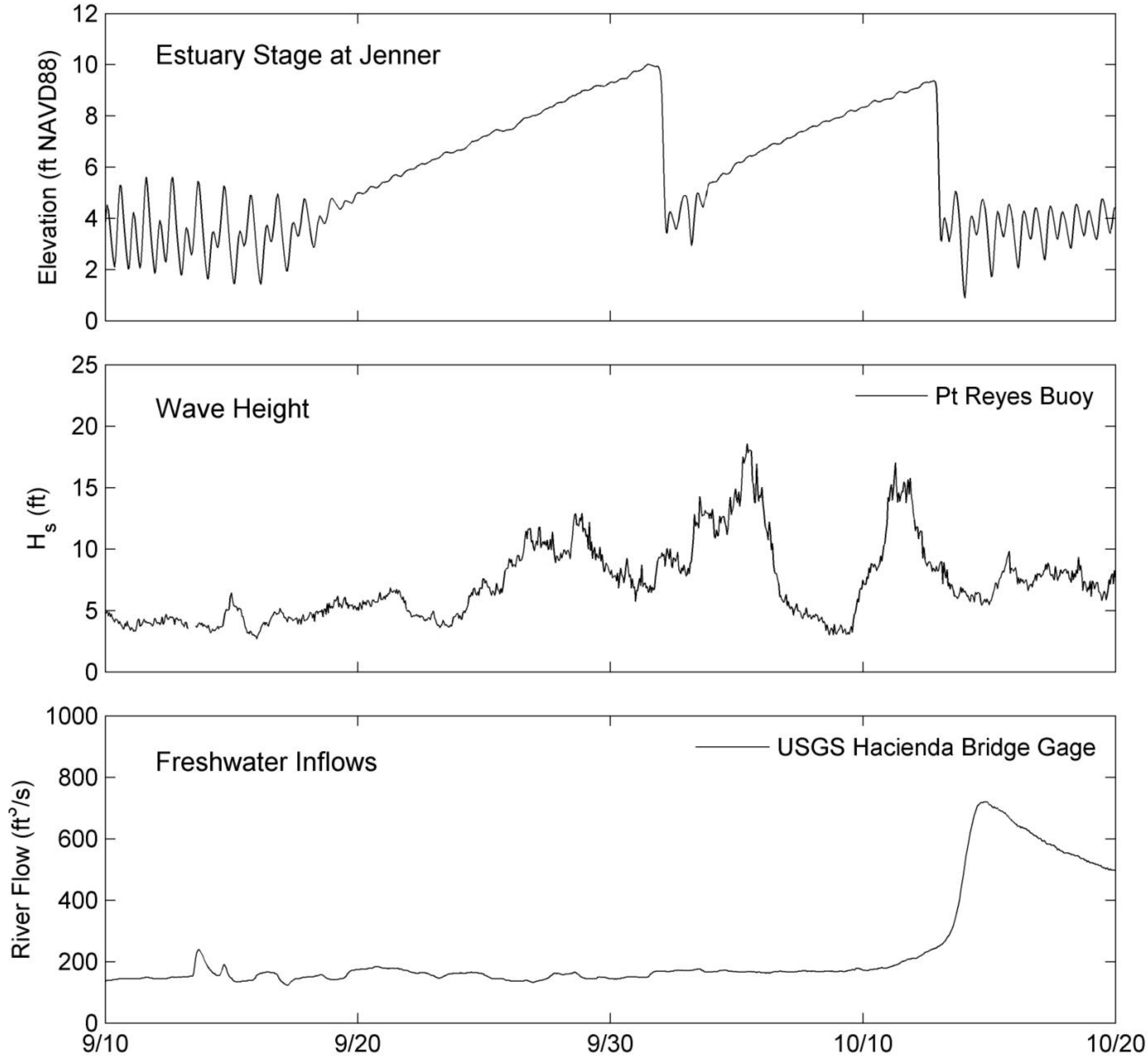


Figure 7-4
Example of dry-season flooding events in 2010, driven by beach conditions.

8 REFERENCES

Arvola, T.F. 1976. Regulation of logging in California 1945-1975. California Dept. of Conservation.

Battalio, R.T., D. Danmeier and P. Williams, Predicting Closure and Breaching Frequencies of Small Tidal Inlets –A Quantified Conceptual Model. Proceedings of the 30th International Conference of Coastal Engineering 2006, ASCE 2007, Vol. 4, 3937 - 3949.

Battany, M.C. and M.E. Grismer. 2000. Rainfall runoff and erosion in Napa Valley vineyards: effects of slope, cover and surface roughness. Hydrological Processes 14: 1289-1304.

BCDC. 1987. Future Sea Level Rise: Predications and Implications for San Francisco Bay. Prepared for San Francisco Bay Conservation and Development Commission by Moffatt and Nichol and Wetland Research Associates.

Behrens, D.K., F.A. Bombardelli, J.L. Largier, and E. Twohy. 2009. Characterization of time and spatial scales of a migrating river mouth. Geophysical Research Letters 36: L09402, doi: 10.1029/2008GL037025.

Behrens, D.K. 2012. The Russian River Estuary: Inlet Morphology, Management, and Estuarine Scalar Field Response. Ph.D. Dissertation. Department of Civil and Environmental Engineering, University of California, Davis.

Behrens, D., F. Bombardelli, J.Largier, and E. Twohy. 2013. Episodic closure of the tidal inlet at the mouth of the Russian River – a small bar-built estuary in California. Geomorphology.

Byrne, R.J., R.A. Gammisch, and G.R. Thomas. 1980. Tidal prism-inlet area relations for small tidal inlets. Proceedings of the 17th Coastal Engineering Conference, Sydney, Australia.

Boak, E.H., and I.L. Turner. 2005. Shoreline definition and detection: a review. Journal of Coastal Research 21(4): 688-703.

California Coastal Conservancy. 2012. LiDAR topography obtained as part of the Coastal LiDAR Project. http://csc.noaa.gov/*dataviewer.*

California Department of Conservation (CDC). 2010. 1984-2010 Land Use Summary for Sonoma County. Farmland Mapping and Monitoring Program. http://www.conservation.ca.gov/DLRP/fmmp/Pages/Index.aspx

California Department of Natural Resources, Division of Fish and Game (DFG). 1930. Biennial Report: 1928-1930. 204 pp.

California Department of Water Resources (DWR). 1964. Land and water use in the Russian River hydrographic unit. Bulletin No. 94-11.

Coastal Geologic Services (CGS). 2011. Reference data for small lagoons in Puget Sound. Database provided to ESA PWA, February 1, 2011.

EDS. 2009. Unpublished grain size distribution data for the beach at the Russian River mouth. Conducted for Sonoma County Water Agency.

Environmental Science Associates (ESA). 2010. Russian River Estuary Management Project. Draft Environmental Impact Report. Prepared for Sonoma County Water Agency.

ESA PWA. 2011. Feasibility of Alternatives to the Goat Rock State Beach Jetty for Managing Lagoon Water Surface Elevations – A Study Plan. Prepared for the Sonoma County Water Agency.

ESA PWA. 2012. Russian River Estuary Outlet Channel Adaptive Management Plan. Prepared for the Sonoma County Water Agency.

Federal Emergency Management Agency (FEMA). 2008. Flood Insurance Study: Sonoma County California and Incorporated Areas. Flood Insurance Study Number 06097CV001A..

Florsheim, J. and P. Goodwin. 1993. Geomorphic and hydrologic conditions in the Russian River, California: Historic trends and existing conditions. Discussion document, prepared for California State Coastal Conservancy, Oakland.

Goodwin, P. and C. K. Cuffe. 1994. Russian River Estuary Study. Prepared for Sonoma County Department of Planning and California State Coastal Conservancy.

Griggs, GB, K.B. Patsch, L. Savoy. 2005. Living with the Changing Coast of California. Berkeley, CA: U.C. Press. 525 pp.

Habel, J. S., and G.A. Armstrong. 1978. Assessment and atlas of shoreline erosion along the California coast. California Department of Navigation and Ocean Development.

Hapke, C., D. Reid, B. Richmond, P. Ruggiero, and J. List. 2006. National Assessment of Shoreline Change, Part 3: Historical Shoreline Change and Associated Loss Along Sandy Shorelines of the California Coast. USGS Open-file Report 2006-1219.

Hughes, S.A. 2002. Equilibrium cross sectional area at tidal inlets. Journal of Coastal Research 18(1): 160-174.

Hume, T.M. and C.E. Herdendorf. 1992. Factors controlling tidal inlet characteristics on low-drift coasts. Journal of Coastal Research 8(2): 355-375.

Hunt, L.A. 1959. Design of seawalls and breakwaters. Journal of Waterways and Harbours Division, ASCE 85 (WW3): 123-152.

Johnson, J.W. 1959. Basic oceanographic data for the California coast at the mouth of the Russian River. University of California Water Resources Center Archives.

Julien, P.Y. 1998. *Erosion and Sedimentation*. Cambridge University Press, 300 pp.

Komar, P.D. 1998. Beach processes and sedimentation. Prentice Hall.

Kraus, N.C., K. Patsch, and S. Munger. 2008. Barrier beach breaching from the lagoon side, with reference to Northern California. Shore and Beach 76(2): 33-43.

Largier, J. and D. Behrens. 2010. Hydrography of the Russian River Estuary, Summer-Fall 2009, with Special Attention on a Five-Week Closure Event. Bodega Marine Lab, University of California, Davis. Prepared for Sonoma County Water Agency.

Laudier, N.A., E.B. Thornton, and J. MacMahan. 2011. Measured and modeled wave overtopping on a natural beach. Coastal Engineering 58(9): 815-825.

Magoon, O., B. Edge, D. Treatwell, and P. Atwood. 2008. Lost jetty of California's Russian River. International Conference of Coastal Engineering, Hamburg, Germany.

Marcus and Associates. 2005. Austin Creek Watershed Assessment. Report for Sotoyome Resource Conservation District.

Merenlender, A.M., Heise, K.L., and Brooks, C. 1998. Mapping vineyard expansion provides information on agriculture and the environment. California Agriculture 54: 7-12.

Minard, C.R. Jr. 1971. Quaternary Beaches and Coasts Between the Russian River and Drakes Bay, California. University of California Hydraulic Engineering Laboratory Report No. HEL 2-35.

National Marine Consultants (NMC). 1960. Wave statistics for seven deep water stations along the California Coast. Prepared for the U.S. Army Corps of Engineers, Los Angeles District.

National Marine Fisheries Service (NMFS). 2008. 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.

National Oceanographic and Atmospheric Administration (NOAA). 2012. Tide gage data and datum information. Accessed on November 30, 2012. http://tidesandcurrents.noaa.gov/epoch_datum_check.shtml?stnid=9415020.

National Research Council (NRC). 2012. Sea-Level Rise for the Coasts of California, Oregon, and Washington: Past, Present, and Future. Prepublication. National Academy Press: Washington, D. C.

Noss, R.F. (Ed.). 2000. The Redwood Forest: History, Ecology, and Conservation of the Coast Redwoods. Island Press, Covelo, CA, USA.

Opperman, J.J., K.A. Lohse, C. Brooks, N.M. Kelly, and A.M. Merenlender. 2005. Influence of land use on fine sediment in salmonid spawning gravels within the Russian River Basin, California. Canadian Journal of Fisheries and Aquatic Science 62, 2740-2751. doi: 10.1139/FO5-187.

Patsch, K., and G. Griggs. 2006. Littoral cells, sand budgets, and beaches: understanding California's shoreline. Institute of Marine Sciences, University of California, Santa Cruz.

Patsch, K. and G. Griggs. 2007. Development Of Sand Budgets For California’s Major Littoral Cells. Institute of Marine Sciences, University of California, Santa Cruz. Prepared for California Department of Boating and Waterways, California Coastal Sediment Management Workgroup.

PWA. 1999. Bolinas Lagoon Supplemental Wave Analysis Report. Prepared for the U.S. Army Corps of Engineers, San Francisco District.

PWA. 2007. Crissy Field Monitoring Report. For: Golden Gate National Parks Association. Project 1386-08.

PWA, 2009, California Coastal Erosion Response to Sea Level Rise, Prepared for the Pacific Institute with funding from the Ocean Protection Council, May, 2009. http://www.pwa-ltd.com/projects/pr_cc_cstlErosSLR.html http://www.pacinst.org/reports/sea_level_rise/

Ranasinghe, R., and C. Pattiaratchi. 2003. The seasonal closure of tidal inlets: causes and effects. Coastal Engineering 45(4): 601-627.

Resio, D., S. Bratos, and E. Thompson. 2006. Meteorology and Wave Climate. Chapter II-2 in Coastal Engineering Manual, Part II, Hydrodynamics. Engineer Manual 1110-2-1100. U.S. Army Corps of Engineers.

Revell, D.L., R. Battalio, B. Spear, P. Ruggiero, and J. Vandever. 2011. A methodology for predicting future coastal hazards due to sea-level rise on the California Coast. Climate Change 109 (Supplement 1): S251-S276. doi: 10.1007/s10584-01100315-2.

Rice, M. P.1974. The mouth of the Russian River. M.S. Thesis, University of California, Berkeley.

Runyan, K.B. and G.B. Griggs. 2002. Contributions from coastal cliff erosion to the littoral budget. In: Coyne, M. and K. Sterrett (eds), California Beach Restoration Study. California Dept. of Boating and Waterways and State Coastal Conservancy.

Sallenger, A. H., W. Krabill, J. Brock, R. Swift, S. Manizade, and H. Stockdon. 2002. Sea-cliff erosion as a function of beach changes and extreme wave runup during the 1997–1998 El Niño. Marine Geology, 187(3), 279-297.

Schulz, W.G. 1942. Report on Jenner Jetty at mouth of Russian River. Report for the State of California Department of Public Works, Division of Water Resources. 35 pp.

Slagel, M. J., and G.B. Griggs. 2008. Cumulative losses of sand to the California coast by dam impoundment. Journal of Coastal Research 24(3): 571-584.

Steiner Environmental Consultants (SEC), 1996. A history of the salmonid decline in the Russian River. 86 pp.

Storlazzi, C. D., and G.B. Griggs. 2000. Influence of El Niño–Southern Oscillation (ENSO) events on the evolution of central California's shoreline. Geological Society of America Bulletin, 112(2), 236-249.

Van der Meer, J.W., and W. Janssen. 1995. Wave run-up and overtopping at dikes. In: Kabayashi, Demirbilek (Eds.), Wave Forces on Inclined and Vertical Wall Structures. American Society of Civil Engineers, New York: ASCE, 1-27.

Waters, T.F. 1995. Sediment in streams: sources, biological effects and control. American Fisheries Society Monogram No. 7, Bethesda, Md.

Willis, C.M. and G.B. Griggs. 2003. Reductions in fluvial sediment discharge by coastal dams in California and implications for beach sustainability. Journal of Geology 111: 167-182.

United States Army Corps of Engineers (USACE). 1962. Littoral studies near San Francisco using tracer techniques. Washington, D.C.

United States Army Corps of Engineers (USACE). 1965. Technical Report on Cooperative Beach Erosion Study on Coasts of Northern California, Point Delgada to Point Ano Nuevo. U.S. Army Corps of Engineers, San Francisco District.

United States Army Corps of Engineers (USACE). 1984. Shore Protection Manual, 4th ed., 2 Vol. U.S. Army Engineer Waterways Experiment Station. Government Printing Office, Washington, D.C.

United State Army Corps of Engineers (USACE). 2005. Coastal Engineering Manual. Engineer Manual 1110-2-1100. U.S. Army Corps of Engineers.

United States Geological Survey (USGS). 2012. Digital Shoreline Analysis System (DSAS), v.4.3. http://woodshole.er.usgs.gov/project-pages/dsas/.

United States Geological Survey (USGS). 2012. Russian River flow data measured near Hacienda Bridge. Flow gage 1467000. http://waterdata.usgs.gov/usa/nwis/uv?site_no=11467000.

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APPENDIX A. OCEAN WAVE ANALYSES

Since the Goat Rock State Beach jetty is located in the dynamic, wave-shaped beach setting, the existing conditions report characterizes ocean wave conditions (Section 2). This appendix provides detail about two parts of these analyses, the nearshore wave estimation (including the transformation procedure) and also gives an assessment of the sensitivity of surf zone processes to the uncertainty present in wave periods and incident direction.

A.1. Wave Transformation

Wave transformation is a procedure in which offshore wave conditions are used to estimate nearshore conditions. This is typically done with either a numerical model or hand-drawn wave ray refraction diagrams. These methods account for changes in wave energy between offshore and nearshore sites, which vary in response to refraction and shoaling, 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.

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. For the present study, we did not consider shoaling, so 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:

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

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.

As discussed in Section 4, the original transformation matrix provided by the Coastal Data Information Program (CDIP) (Figure 4-3) was found to under-predict wave heights near GRSB when compared against the available wave measurements provided by the Bodega Marine Laboratory (BML). This discrepancy was especially true of waves arriving from the northwest, which are common at the site (Figure 4-2).

To improve the transformation matrix, we added a limited number of transformation coefficients manually, which we obtained by comparing the Pt. Reyes and GRSB measurements of wave energy. This comparison was only performed for times when the wave spectra at each site exhibited similar characteristics, indicating the passage of a coherent wave train from offshore to nearshore. When this condition was met, the ratio of nearshore energy over offshore energy was taken as the improved transformation coefficient and added to the transformation matrix in the location corresponding to the period and direction of the offshore waves (Figure 4-3b). This is a simple approach used to correct only part of the transformation matrix, and relied on the available wave data from the combined 5-month set of measurements in 2009 and 2012. The matrix could be improved further with additional data spanning a greater range of wave conditions at the site.

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

A.2. Sensitivity of Surf Zone Parameters

Both wave runup on the beach face and overtopping into the lagoon are sensitive to incident wave direction and period. Since the majority of the nearshore waves used in the surf zone assessment (Section 4.3) were obtained by transforming offshore waves, nearshore direction and period come with a level of uncertainty, which is implicit in the overwash predictions given in Table 4-2.

To assess the uncertainty due to variable wave period, we estimated the wave runup ($R_{2\%}$) and overwash rate (Q) for a wave of variable period, but constant height. The majority of nearshore waves recorded by BML in 2009 and 2012 had peak periods (T_p) between 8 and 14 seconds. This is consistent with offshore measurements at the Pt. Reyes Buoy. For a 6 ft wave and a beach foreshore slope of 0.1, increasing T_p results in a linear increase in $R_{2\%}$ from 3.05 to 4.65 (Table A-1). Using the same parameters and a beach crest of 14 ft NAVD, the overwash rate (Q) increases nonlinearly. For $T_p < 11$ s, the change in Q for different periods is small, but increases more rapidly for $T_p > 14$. Despite this sensitivity, the uncertainty resulting from T_p remains small when compared with the error bounds of the overwash volume given in Table 4-2.

Table A - 1 Sensitivity of surf zone parameters to wave period (Tp). Conditions represent wave height of 6 ft, and a beach crest elevation of 14 ft NAVD.

T_p (s)	Wave Runup, $R_{2\%}$ (ft)	Overwash Rate Q (ft^3/hr/ft of beach crest)
8	3.05	0.07
9	3.33	0.25
10	3.61	0.71
11	3.87	1.72
12	4.14	3.68
13	4.40	7.10
14	4.65	12.66

Runup and overwash rate were less sensitive to incident wave direction. Both will decrease with increasing wave angle (relative to the shore-normal direction). Using the same conditions as above and a peak wave period (T_p) of 12 s, the incident direction (ϑ) was increased from zero (i.e. shore-normal waves) to ten degrees. Reductions in both runup and overwash were assessed using the methodology of Van der Meer and Janssen (1995). For the ten-degree change in wave angle, wave runup decreased by roughly two percent, while the overwash rate decreased by roughly four percent.

Table A - 2 Sensitivity wave overwash, Q, to incident wave direction on the beach. Conditions represent wave height of 6 ft, wave period of 12 s and a beach crest elevation of 14 ft NAVD.

ϑ	Wave Runup, $R_{2\%}$ (ft)	Overwash Rate Q (ft^3/hr/ft of beach crest)
0	4.14	3.68
2	4.12	3.65
4	4.10	3.63
6	4.08	3.60
8	4.07	3.58
10	4.05	3.55

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.

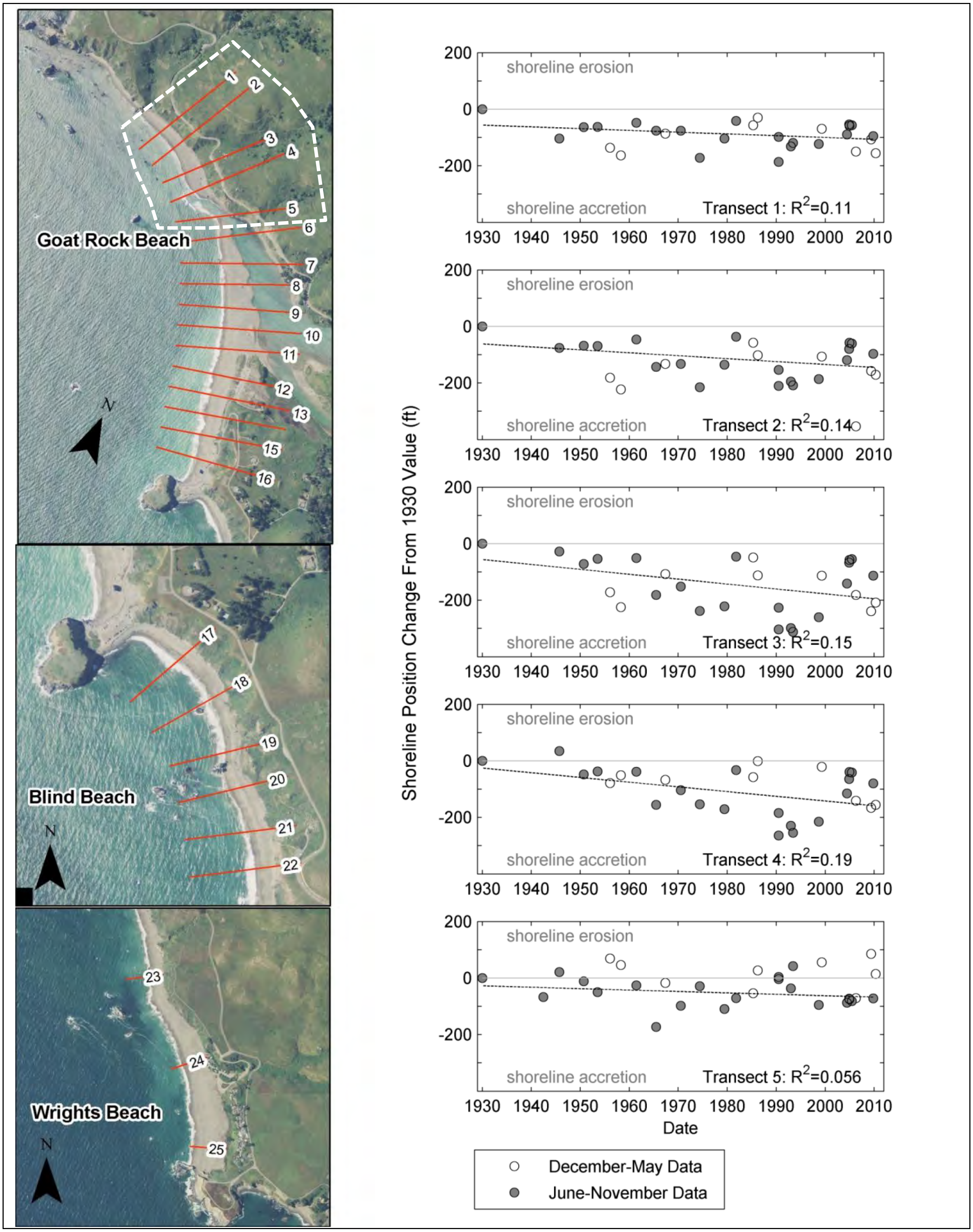
As discussed in the main report, shoreline positions were identified using available aerial images (Table 5-1). Since these were obtained from dozens of separate flights between 1940 and 2012, they were taken at variable heights and resolution. To be able to compare the images directly, each image was 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.

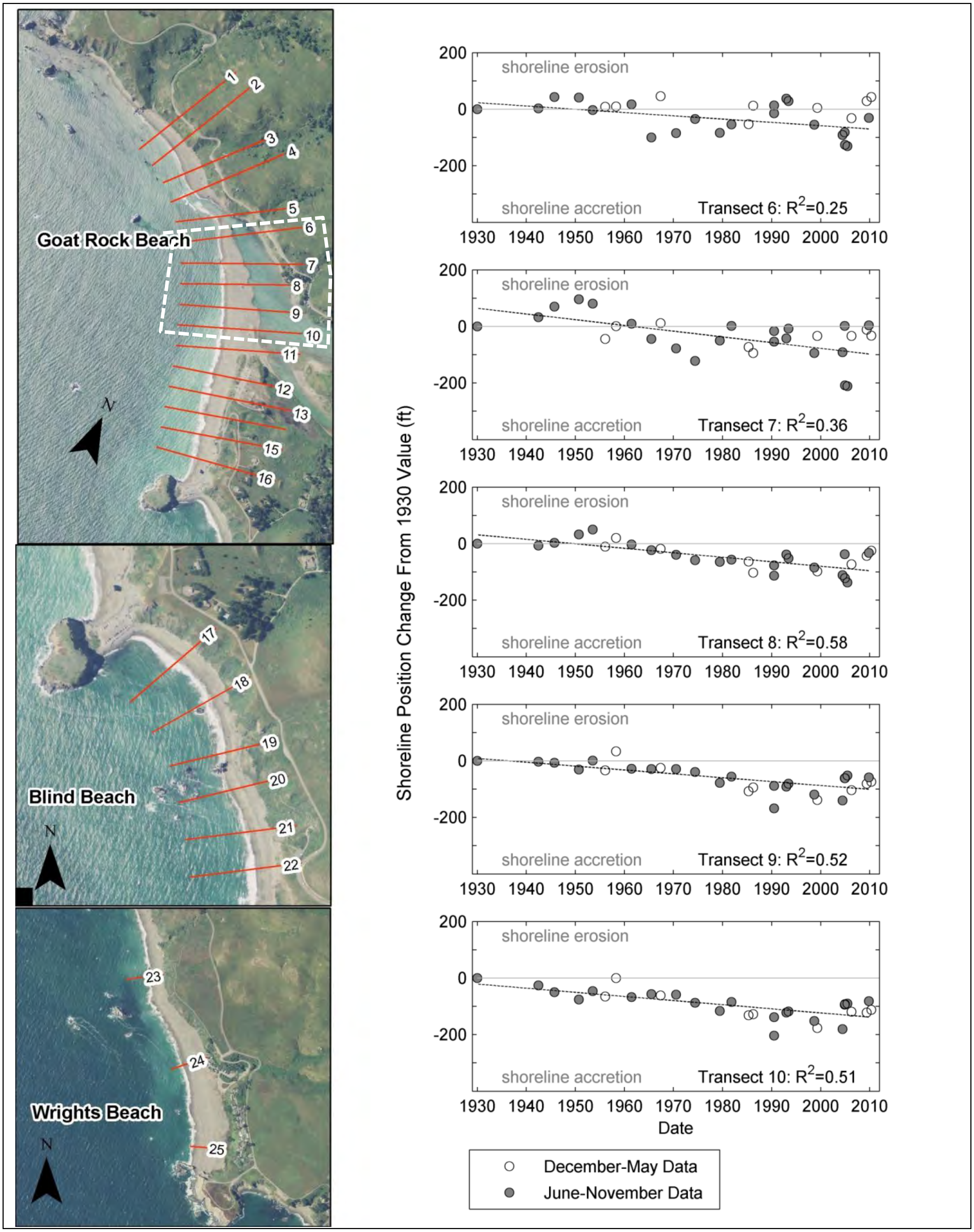
Boak and Turner (2005) and Hapke et al. (2006) discuss these and other errors involved with estimating shoreline features. Digitizing maps and aerial images of limited resolution can produce errors of ~3 ft (Hapke et al., 2006). Errors produced by the georeferencing process range from roughly 6-30 ft 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.

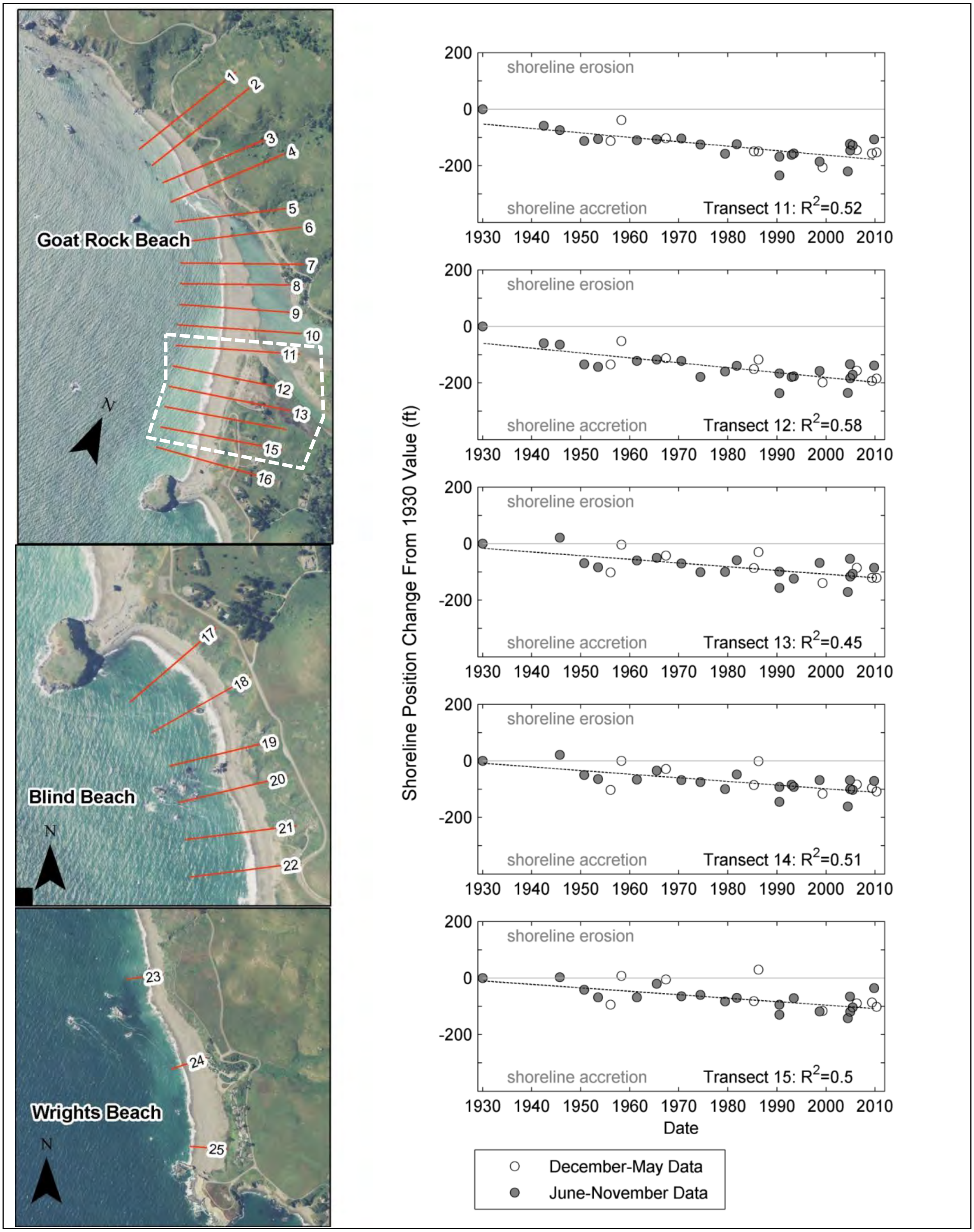
Figure B-1

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



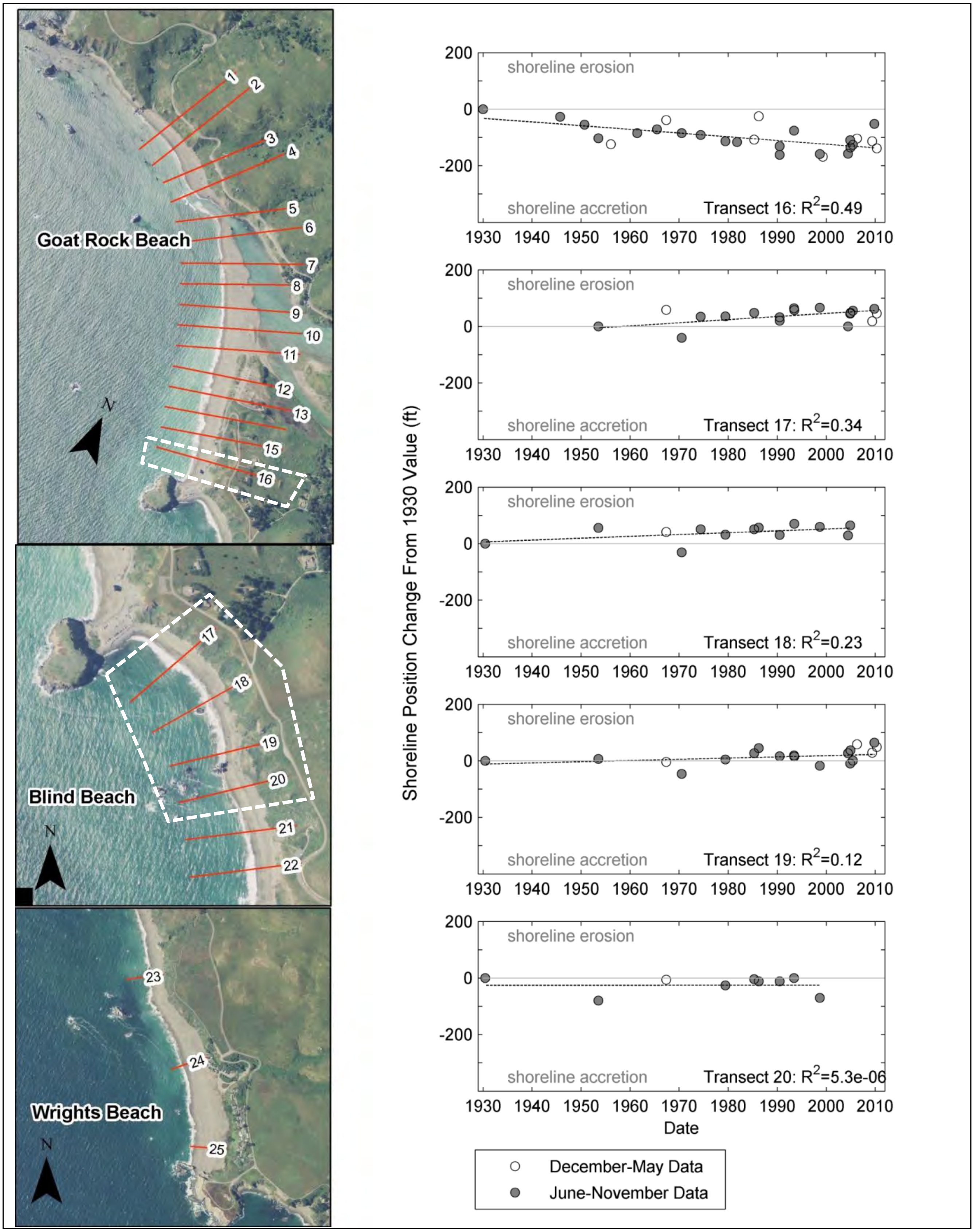
NOTE: All linear trends were obtained using summer-fall data.

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.

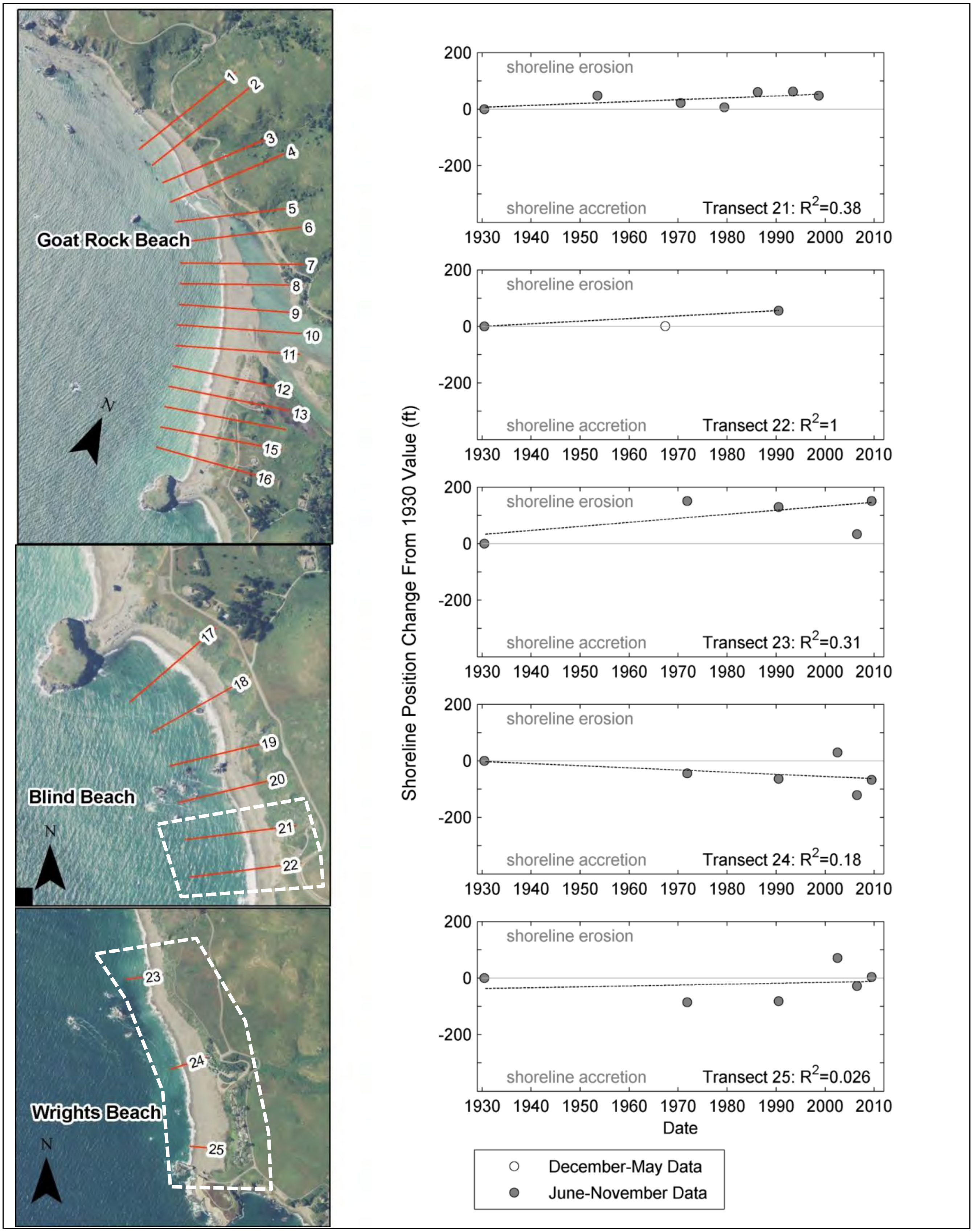
Figure B-3
(left) Index of transects and **(right)** corresponding shoreline position data for transects 11-15.



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

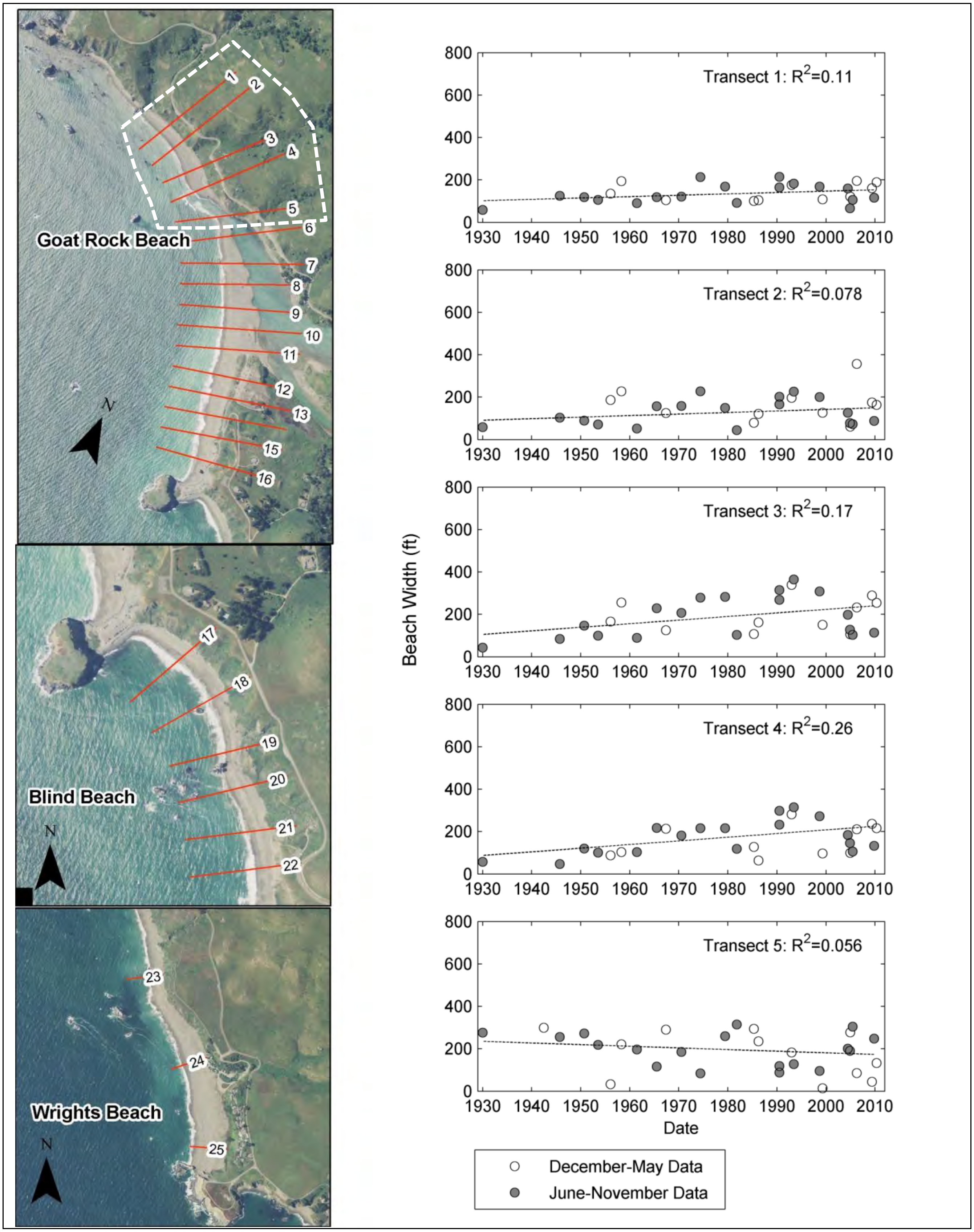
Figure B-4

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



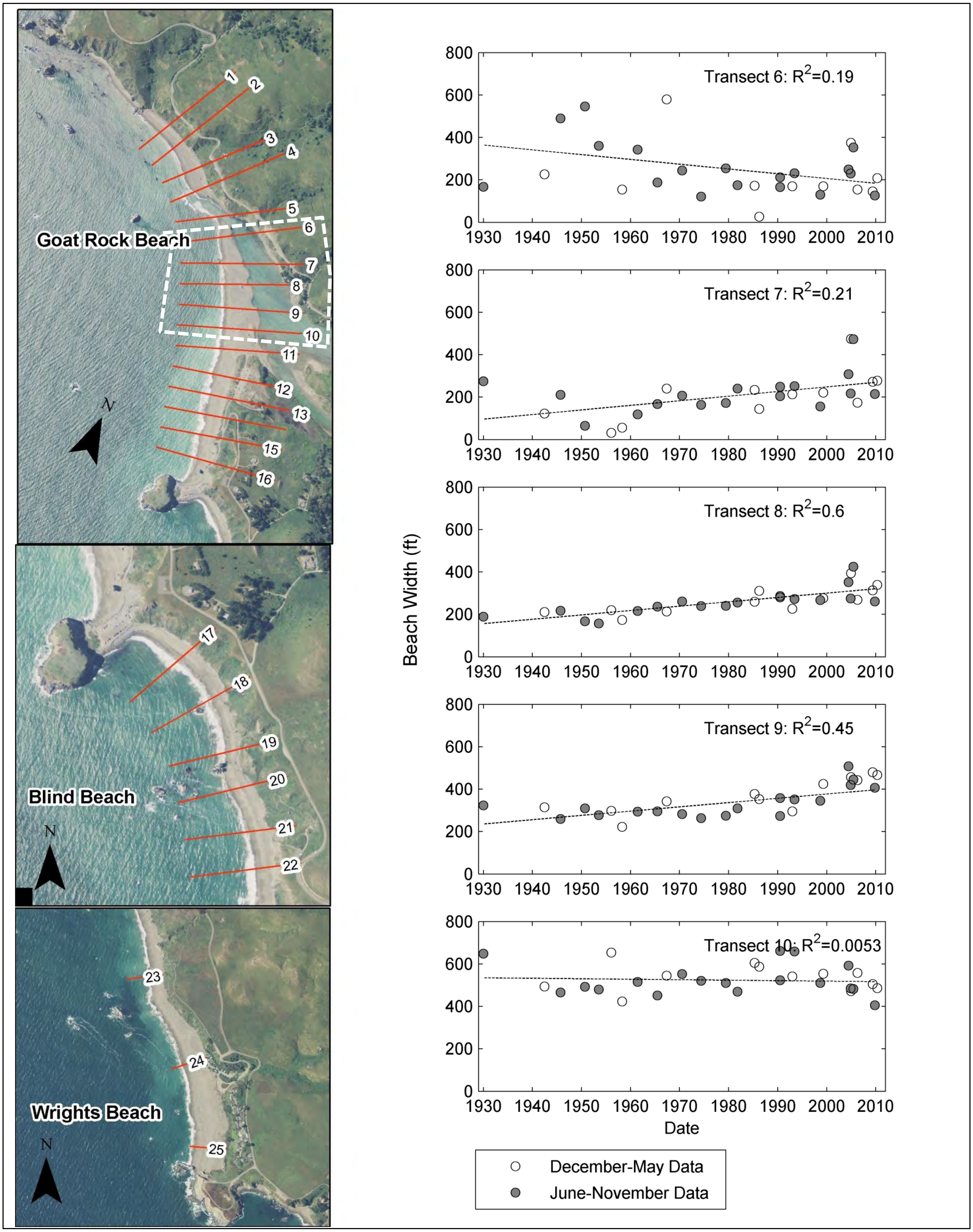
NOTE: All linear trends were obtained using summer-fall data.

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.

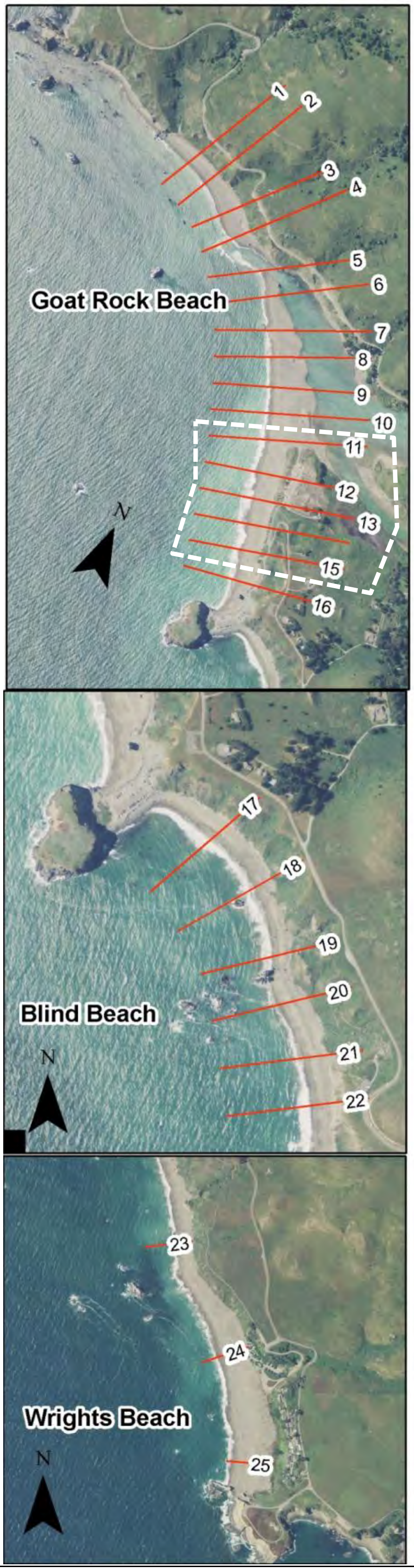
Figure B-6
(left) Index of transects and **(right)** corresponding beach width data for transects 1-5.

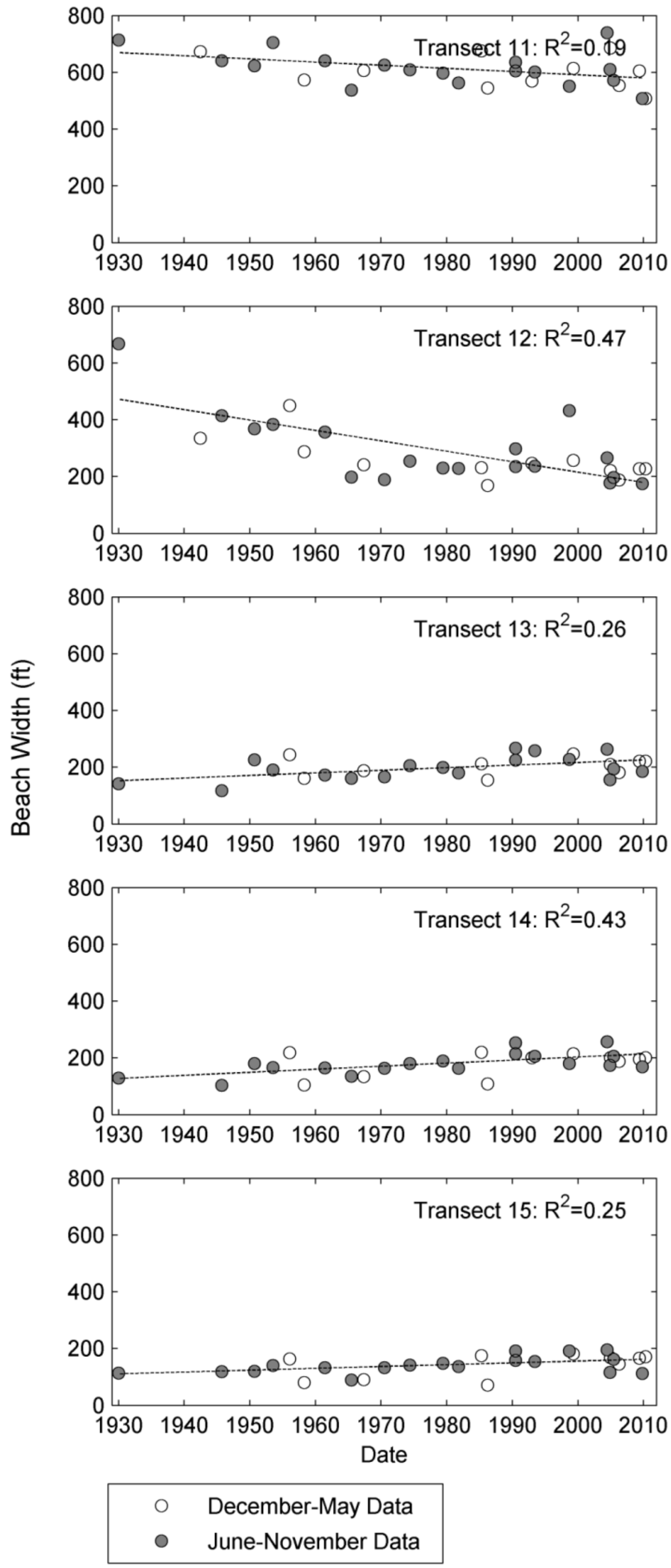


NOTE: All linear trends were obtained using summer-fall data.

Figure B-7

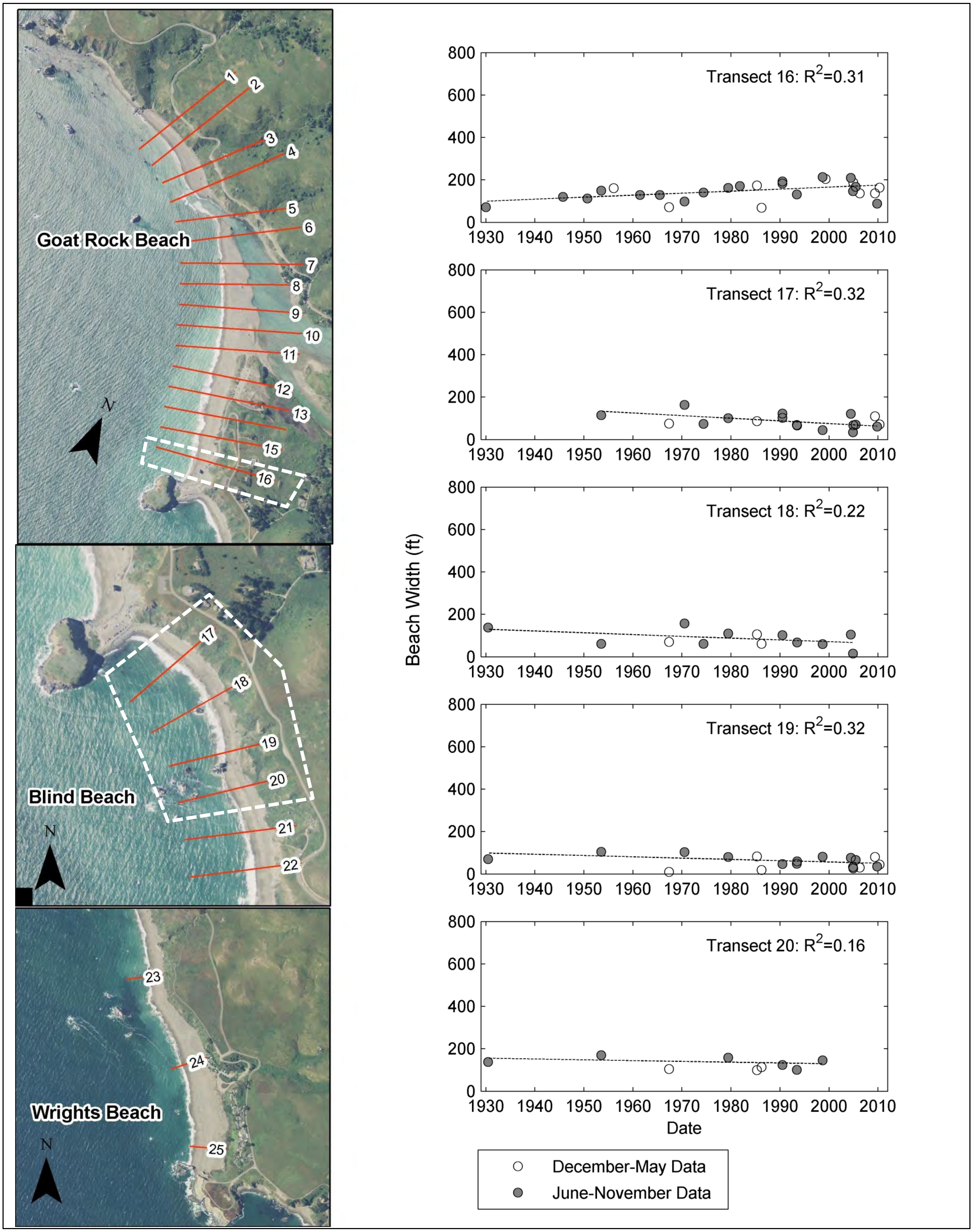
(left) Index of transects and **(right)** corresponding beach width data for transects 6-10.





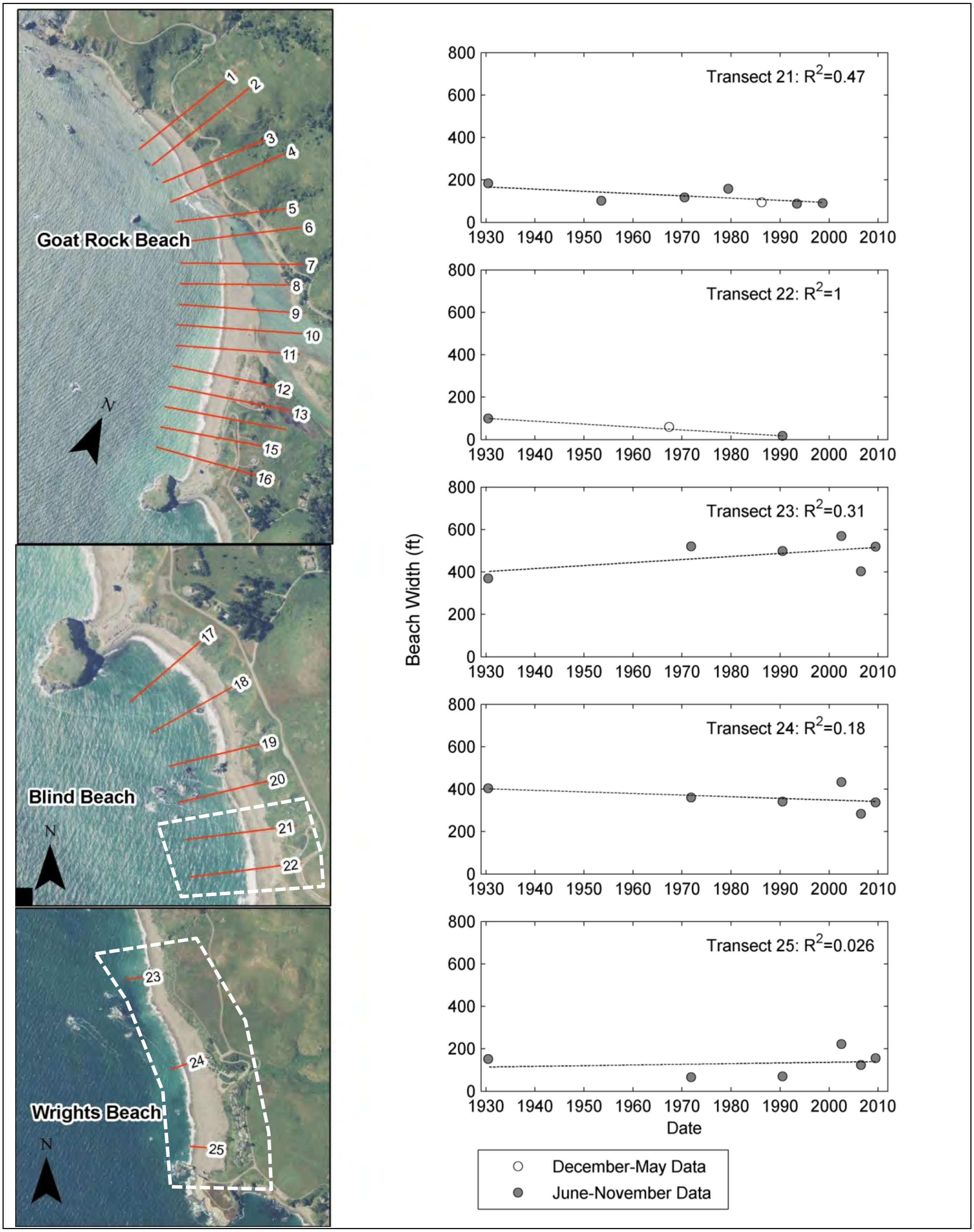
NOTE: All linear trends were obtained using summer-fall data.

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:

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

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_*:

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

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

$$\hat{H} = \frac{gH_{m0}}{u_*^2} \tag{C-3}$$

$$\hat{X} = \frac{gX}{u_*^2} \quad \text{(C-4)}$$

$$\hat{f}_p = \frac{u_* f_p}{g} \quad \text{(C-5)}$$

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:

$$\hat{H} = \lambda_1 \hat{X}^{m_1} \quad \text{(C-6)}$$

$$\hat{f}_p = \lambda_2 \hat{X}^{m_2} \quad \text{(C-7)}$$

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.

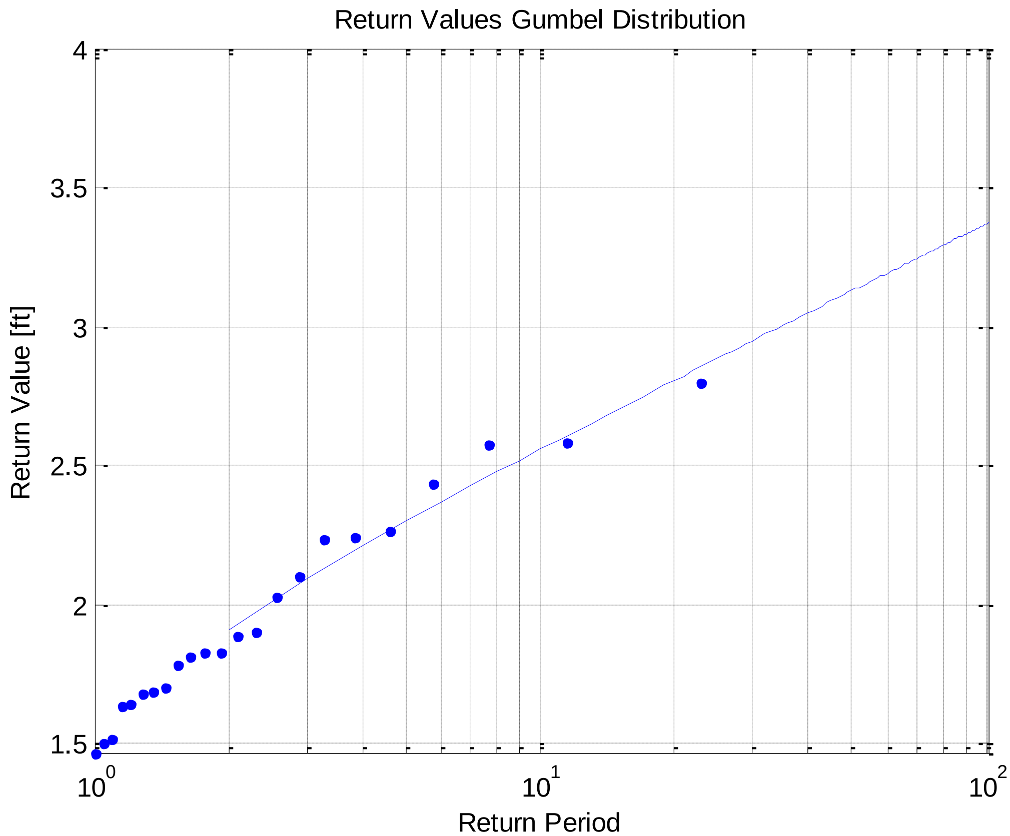
Predicted wind waves were generated using BML wind data from 1988 to 2011, and fit to the Gumbel distribution to estimate wind-wave heights during events with long return periods (Figure C-1). Wave runup was calculated using the same methodology as discussed in Section 4. The estuary side slope influences 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.

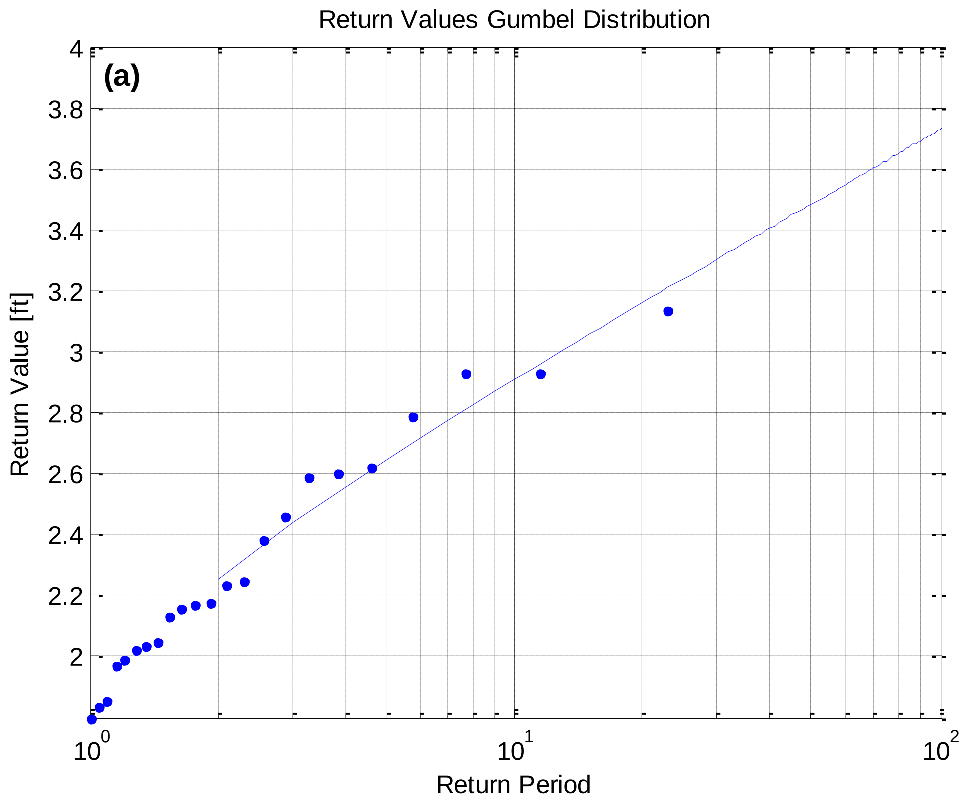
Return Period (Gumbel)	H_{max} (ft)	Runup (ft)			
		Slope 1:2	Slope 1:3	Slope 1:5	Slope 1:10
1	1.91	2.25	1.68	1.23	0.90
5	2.37	2.72	2.03	1.49	1.09
10	2.59	2.94	2.20	1.62	1.19
20	2.82	3.18	2.38	1.75	1.29
50	3.13	3.49	2.62	1.93	1.42
100	3.37	3.73	2.80	2.06	1.52

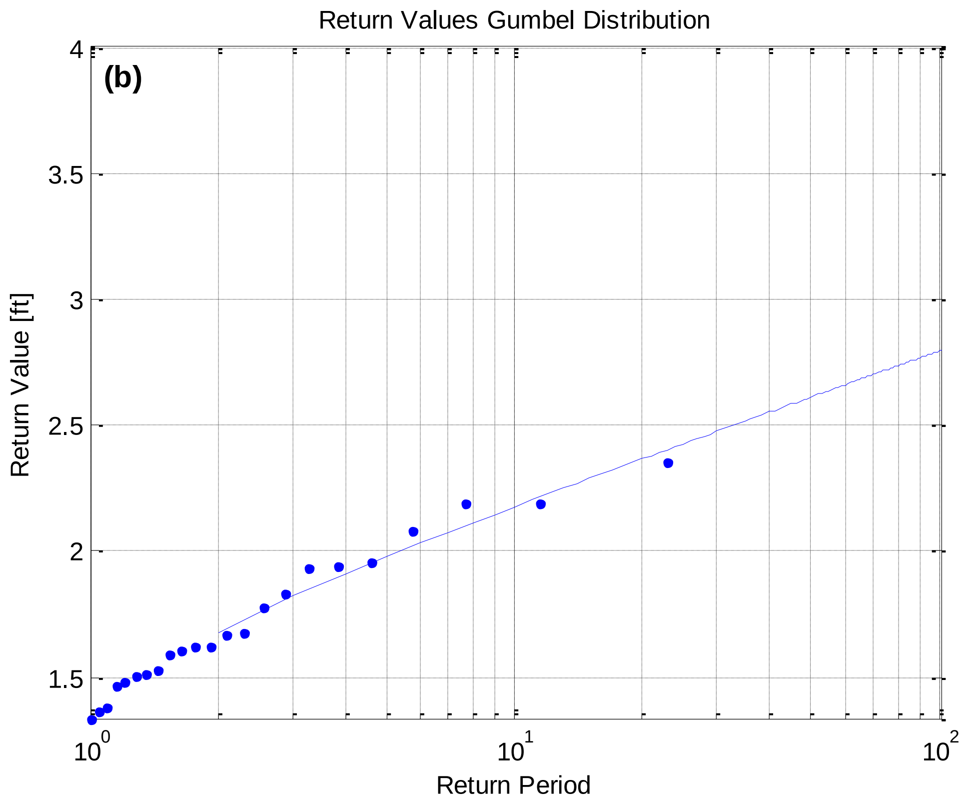


NOTE: Wind-wave heights estimated after Resio et al. (2006) using wind data from the Bodega Marine Laboratory.

Figure C-1

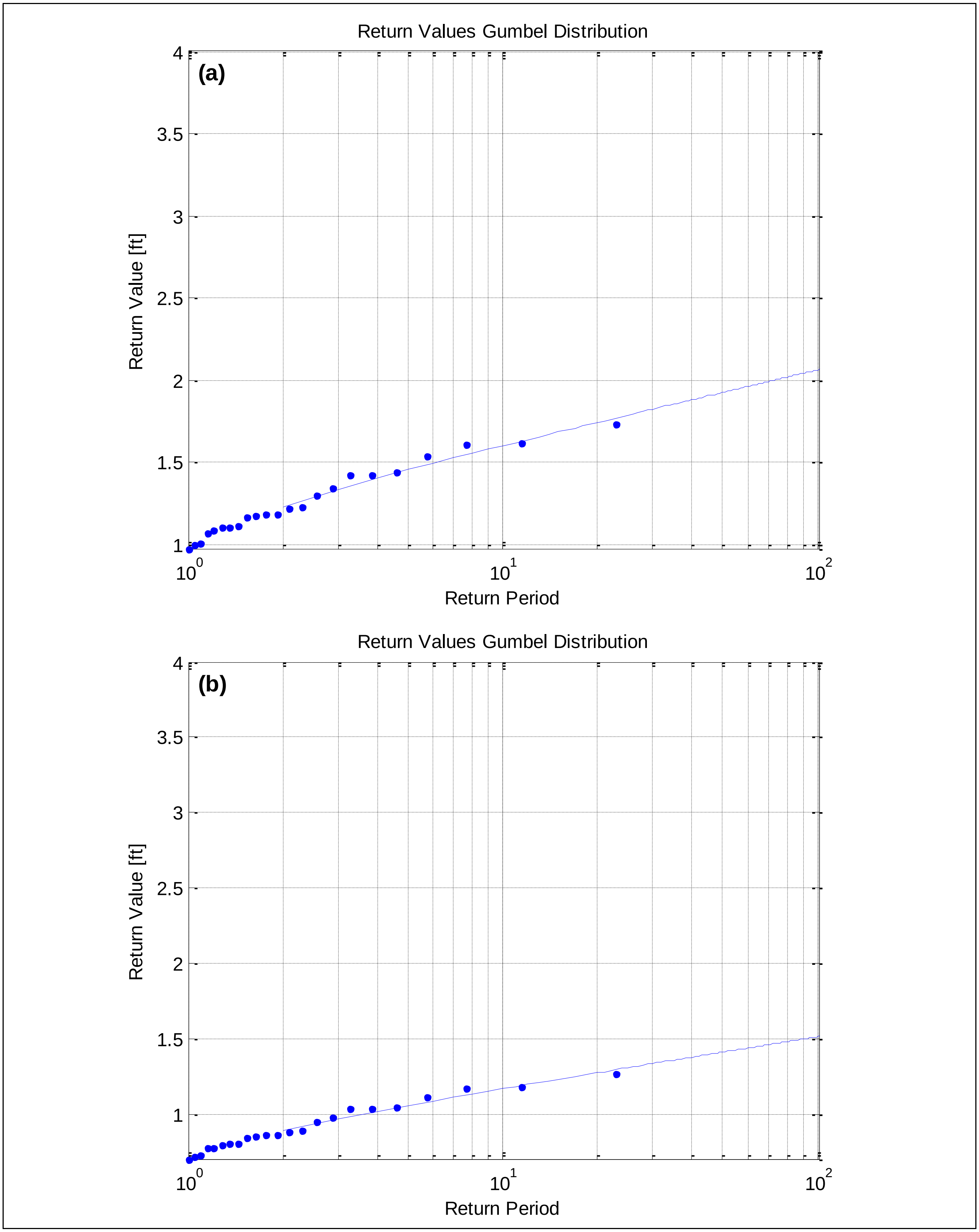
Predicted wind-wave heights and return periods (years).





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.

Expanded Russian River Estuary Preliminary Flood Risk Management Feasibility Study

June 4, 2012

INTRODUCTION

As required by Reasonable and Prudent Alternative (RPA) 2 in the Russian River Biological Opinion (September 24, 2008), the Sonoma County Water Agency (Water Agency) has developed a list of properties, structures, and infrastructure that may be subject to inundation if the barrier beach at the mouth of the Russian River was allowed to remain closed and naturally breach. River front communities in the Russian River Estuary (Estuary) consist of Jenner, Bridgehaven, Freezeout Creek area, and Duncans Mills. Water levels in the Estuary fluctuate when the river mouth is open with tides and can range from below sea level during very low tides and around 6 feet during high tides. The Russian River Estuary Management Project would have a target water elevation of 7 feet and a range from 4 to 9 feet from May 15 to October 15. The number of at risk properties and structures incrementally increases as water levels rise in the Estuary. Below is a preliminary evaluation of river front properties at risk of flooding.

METHODS

The Water Agency created a Geographic Information System (GIS) tool specifically for evaluating inundation risk to properties along the Estuary. An aerial photograph was overlayed with a digital elevation map created during the bathymetric survey of the Estuary in 2008 and 2009. A County of Sonoma parcel permit tool was used by matching the Assessor's Parcel Number (APN) to the parcel to determine the type of structures on the property. These data sources provided the parcel location, and the approximate elevation and location of structures on the property. This information was ground-truthed on December 13, 2011, by visually inspecting properties and structures from a boat on the Russian River. The information provided is considered preliminary and will be revised as the Water Agency moves forward with the flood risk management feasibility requirements of RPA 2. Additional information regarding the exact type and elevation of structures and infrastructure may be necessary for consideration of future water surface elevation management in the Estuary.

RESULTS

This flood risk study evaluated 123 river properties along the Russian River Estuary shoreline from Jenner to Duncans Mills area. The location of parcels and flood elevations are included in the appendix. Table 1 summarizes flood risks including, potential inundation elevation, and type of property, structure, and infrastructure. Structures include houses, garages, and sheds. Infrastructure types included: roads, stairs, tanks, boat docks, etc. Figure 1 shows the number of properties potentially affected by inundation from 4.5 to 14 feet. Estuary water surface elevations between 10 and 12 feet NGVD (the estimated water surface elevation if the barrier beach was allowed to naturally breach) may potentially inundate portions of up to 97 properties, including 16 structures. However, water levels could reach 12-14 feet under extreme conditions, such as sustained heavy surf coupled with large spring tides. Water levels at this height could inundate 23 structures located near the State Highway 1 Bridge over the Estuary downstream to the mouth of the Russian River. Most at-risk structures are located in the Jenner area accounting for 20 properties (Figure 2). There are two at-risk structures in Bridgehaven (Figure 3) and one structure in Goat Rock area. The State Parks Visitor Center in Jenner is estimated to be the first occupied structure to flood at approximately 10.5 feet inundation. Below is a detailed description of at-risk structures grouped by area beginning with Jenner and moving upstream.

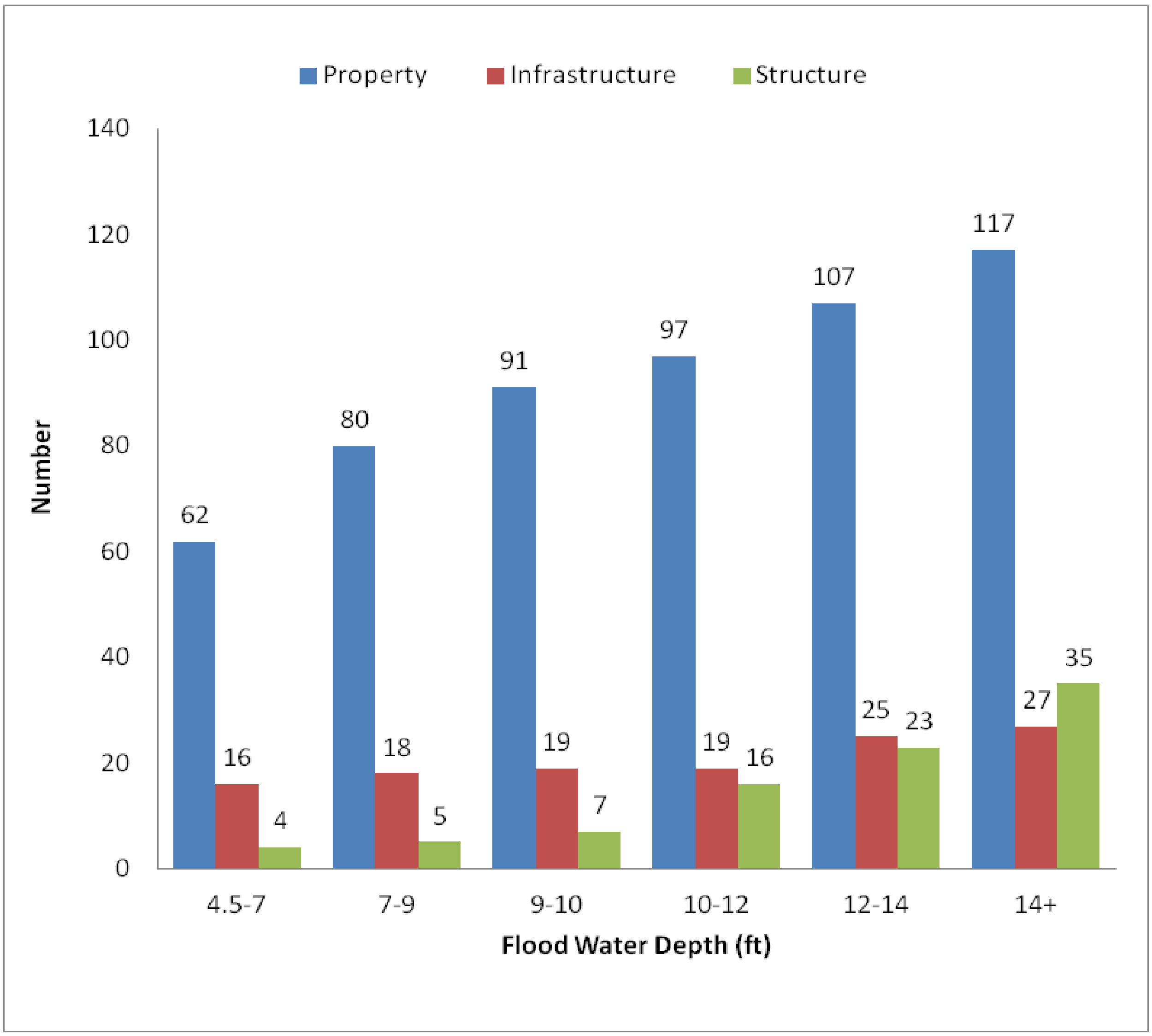


Figure 1: Russian River Estuary flood risk of adjacent properties. A total of 123 properties were evaluated. Definitions: Property parcel boundary; Infrastructure consists of seawalls, docks and stairs, tanks, etc.; Structures are mostly houses and out buildings.

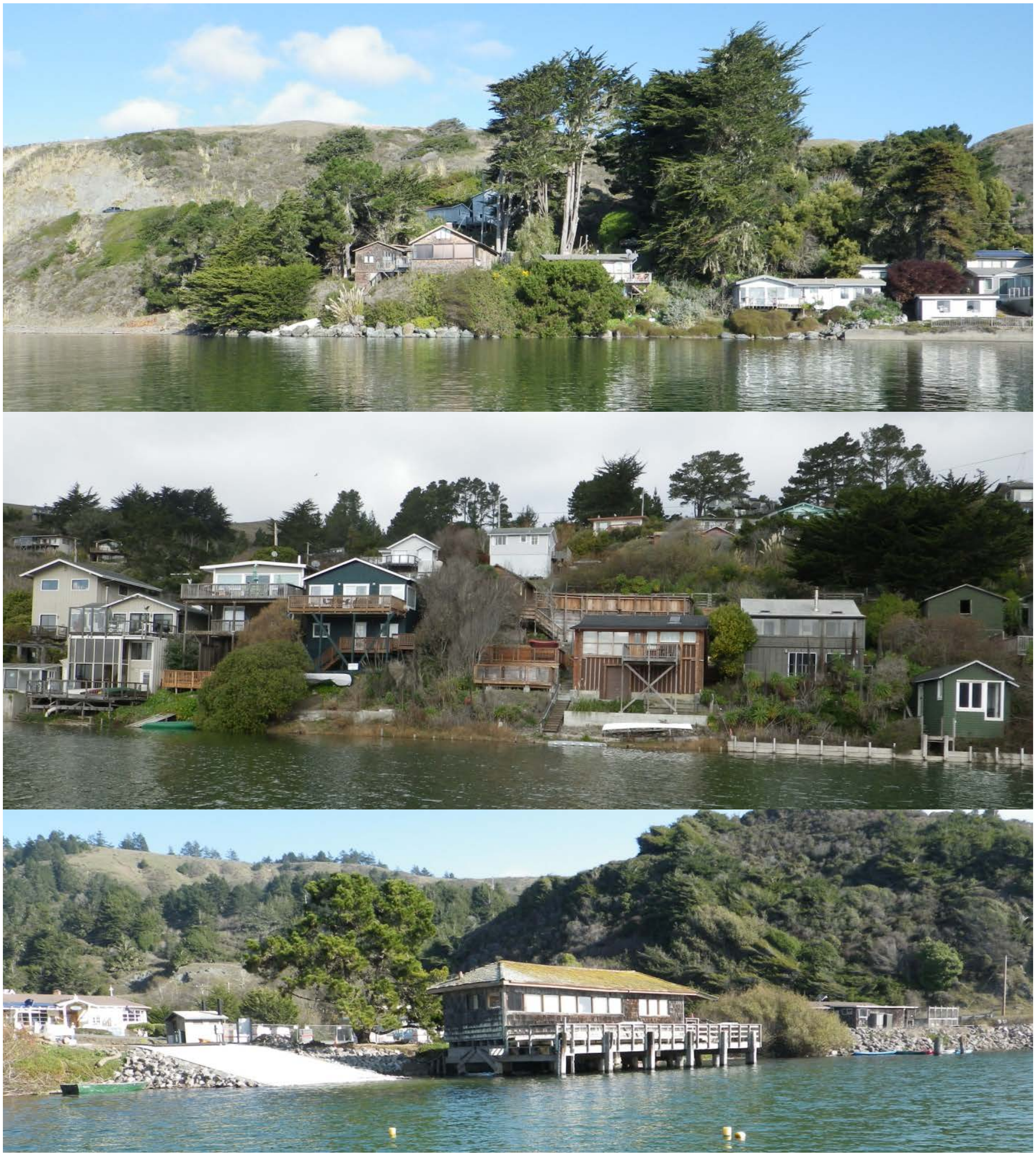


Figure 2: River front properties in the Jenner area. Development consists of residential and commercial properties from the Russian River mouth to Jenner Gulch.

Figure 3: River front properties in the Bridgehaven area near Highway 1 Bridge over the Russian River. Development consists of mostly residential properties and two commercial properties.

Table 1: Russian River Estuary Preliminary Flood Risk Assessment at Varying Water Elevations

APN	Struct-ures	Structure Type	Structure Description	Infrastructure Description	Inundation Elevation (ft)*						Notes
					4.5-7	7-9	9-10	10-12	12-14	14+	
096-090-004	2	Houses	Two single story houses							PS: House	
096-090-005	3	House, out buildings	Single story house and out buildings	Floating Dock		P	P	P	P	PS: House	
096-090-009	1	House	Single story house	Wooden stairs	PI: Stairs	PI: Stairs	PI: Stairs	PI: Stairs	PI: Stairs	PI: Stairs	
096-090-014	None									P	
096-090-015	None								P	P	
096-090-022	None									P	
096-090-023	None								P	P	
096-090-025	1	House				P	P	P	P	P	
096-090-027	None					P	P	P	P	P	
096-090-028	None								P	P	
096-090-030	1	House	Two story house			P	P	P	P	PS: Bottom of house	
096-090-031	None								P	P	
096-090-032	1	House	Single story, appears elevated			P	P	P	P	PS: Foundation of house	
096-100-009	1	House									
096-100-013	None									P	
096-100-014	None										
096-100-035	None										
096-100-079	1	House									
096-120-011	None									P	
096-130-002	1	House								P	
096-130-003	None									P	
096-130-005	None				P	P	P	P	P	P	
096-130-006	None				P	P	P	P	P	P	
096-130-017	None									P	
096-140-003	None									P	
096-140-021	4	Unknown	Camp ground facility						P	P	
096-223-018	None							P	P	P	
097-030-022	None					P	P	P	P	P	
097-030-029	None				P	P	P	P	P	P	
097-030-030	None				P	P	P	P	P	P	

APN	Struct-ures	Structure Type	Structure Description	Infrastructure Description	Inundation Elevation (ft)*						Notes
					4.5-7	7-9	9-10	10-12	12-14	14+	
097-090-023	None					P	P	P	P	P	
097-090-045	5	House, 4 buildings				P	P	P	P	PS	
097-090-046	3	House, 2 buildings				P	P	P	P	PS	
097-130-015	19	Unknown	Camp ground, pasture		P	P	P	P	P	P	
097-130-021	3	House			P	P	P	P	P	P	
097-130-022	None			Road							
097-140-002	None				P	P	P	P	P	P	
097-140-015	9	Unknown	Camp ground facility	Gravel road	P	P	P	P	P	PI: Road	
097-150-004	1	Unknown			P	P	P	P	P	P	No structures near river
097-150-005	8	House, buildings			P	P	P	P	P	SP	No structures near river
097-160-003	None				P	P	P	P	P	P	
099-030-006	None				P	P	P	P	P	P	
099-030-007	None				P	P	P	P	P	P	
099-030-012	None				P	P	P	P	P	P	
099-030-013	1	Building	Unknown building		P	P	P	P	P	P	
099-030-014	None				P	P	P	P	P	P	
099-040-001	3	Buildings	Abandoned ranch buildings		P	P	P	P	P	P	Penny Island
099-040-002	1	House	Goat Rock State Beach	Parking lot, restroom	P	P	P	P	P	P	
099-040-003	None					P	P	P	P		
099-040-010	None						P	P	P	P	
099-040-020	1	House, out building	One story house, appears elevated with stairs				P	P	PI: Stairs, possibly out building	PI: Stairs, possibly outbuilding	
099-040-022	None					P	P	P	P	P	
099-040-031	2	2 Houses	Lower house, two story, first floor appears not inhabited			P	P	P	PS: Bottom of lower house, support columns	PS: Bottom of lower house, support columns	
099-040-032	None							P	P	P	
099-040-033	8	Unknown					P	P	P	P	Buildings not near river

APN	Struct-ures	Structure Type	Structure Description	Infrastructure Description	Inundation Elevation (ft)*						Notes
					4.5-7	7-9	9-10	10-12	12-14	14+	
099-050-002	2	House, out building	House, out building		P	P	P	P	P	P	Buildings not near river
099-050-010	None						P	P	P	P	
099-050-011	None								P	P	
099-050-014	28	Houses, outbuildings	Trailer/ camping park, residential house, hookup spaces	Propane tank near house	P	P	P	P	PIS: Bottom of house, propane tank, driveway	PIS: Bottom of house, propane tank, driveway	
099-050-015	3	House, outbuildings	House, out buildings		P	P	P	P	P	P	Buildings not near river
099-070-010	None				P	P	P	P	P	P	Willow Creek marsh
099-070-011	3	House, out buildings	Single story house, appears elevated	Driveway	P	P	P	P	PI: Driveway	PIS: Driveway, bottom of house	
099-070-012	None						P	P	P	P	
099-080-003	1	Deck and stairs		Wood deck and stairs	PI: Wood stairs	PI: Wood stairs	PI: Wood stairs	PI: Wood stairs	PI: Wood stairs	PI: Wood stairs	No house onsite
099-080-005	None						P	P	P	P	
099-080-006	1	Dock and stairs		Wood stairs, floating dock	PI: Wood stairs	PI: Wood stairs	PI: Wood stairs	PI: Wood stairs	PI: Wood stairs	PI: Wood stairs	
099-080-008	None	Dock and stairs		Dock and stairs				P	P	PI	Dock may be on adjacent lot
099-080-009	1	Deck and stairs		Wood stairs, deck	PI: Wood stairs	PI: Wood stairs	PI: Wood stairs	PI: Wood stairs	PI: Wood stairs	PIS: Stairs, bottom of deck	
099-080-026	1	House	Two story house						P	PS: Bottom of house	
099-080-035	None			Access road			P	P	P	P	
099-080-037	1	House	1.5-story house, bottom appears not inhabited	Wood boat dock and stairs	PI: Boat dock	PI: Boat dock and stairs	PI: Boat dock and stairs	PI: Boat dock and stairs	PI: Boat dock and stairs	PIS: Dock/ stairs, bottom of house	
099-080-038	1	House	Single story house					P	P	PIS: Bottom of house	
099-080-043	None						P	P	P	P	
099-080-044	None							P	P	P	
099-080-045	None				P	P	P	P	P	P	
099-080-048	None						P	P	P	P	
099-080-053	None					P	P	P	P	P	
099-080-054	None						P	P	P	P	
099-080-056	None							P	P	P	

APN	Struct-ures	Structure Type	Structure Description	Infrastructure Description	Inundation Elevation (ft)*						Notes
					4.5-7	7-9	9-10	10-12	12-14	14+	
099-080-060	None									P	
099-080-063	None								P	P	
099-080-064	None									P	
099-080-066	1	House	1.5-story house, bottom appears not inhabited	Stairs		PI: Stairs	PIS: Stairs, bottom of house	PIS: Stairs, bottom of house	PIS: Stairs, bottom of house	PIS: Stairs, bottom of house	
099-090-001	None				P	P	P	P	P	P	Pasture
099-090-004	None				P	P	P	P	P	P	
099-090-005	None				P	P	P	P	P	P	
099-090-018	None				P	P	P	P	P	P	Pasture
099-113-012	None			Gravel parking lot from infill. Bank armored with rock riprap.	P: Bank riprap	P: Bank riprap	P: Bank riprap	P: Bank riprap	P: Bank riprap	P: Bank riprap	
099-120-005	1	Single story business complex	Café and two commercial shops	Parking lot, wooden patio and lawn seating area	P	P	P	P: Parking	SP: Businesses, parking/ patio	SP: Businesses, parking/patio	
099-120-009	3	Building on stilts, restrooms, US post office	State Parks Jenner Visitor Center with viewing deck elevated over Estuary on wooden pylons, concrete vaulted restroom, Postal building on concrete foundation	Concrete boat ramp, riprap banks	PI: Boat ramp, riprap banks	PIS: Boat ramp/ banks, bottom viewing deck	PIS: Boat ramp/ banks, Visitor Center	PIS: Boat ramp/banks, Visitor Center	PIS: Boat ramp/ banks, Visitor Center, Restroom, parking lot, Post Office	PIS: Boat ramp/ banks, Visitor Center, Restroom, parking lot, Post Office	
099-120-011	1	Large Building	Jenner Community Center, metal building with concrete foundation	Gravel parking lot			P	P	PI: Parking lot	PIS: Parking lot, Jenner Community Center foundation	
099-120-013	2	Com-mercial building	Restaurant, building	Roads					P	P	
099-120-017	1	Business	Gas station and store	Underground gas tanks and pumps, paved parking lot	P	P	P	P	PI: Parking lot	PI: Parking lot	Jenner Store
State Highway 1		Road		Highway 1 (two-lane asphalt)					PI: Road	PI: Road	220 ft of Hwy along APN 099-120-005 & -009

APN	Struct-ures	Structure Type	Structure Description	Infrastructure Description	Inundation Elevation (ft)*						Notes
					4.5-7	7-9	9-10	10-12	12-14	14+	
099-140-041	None				P	P	P	P	P	P	
099-140-042	None					P	P	P	P	P	
099-140-043	1	House	Two story house, first floor appears not inhabited	House with concrete foundation/ seawall	PS: Lower foundation/ seawall	PS: Lower foundation/ seawall	PS: Foundation/ seawall, house first floor	PS: Foundation/ seawall, house first floor	PS: Foundation/ seawall, house first floor	PS: Foundation/ seawall, house first floor	
099-140-044	1	House	Single story house elevated on wooded posts		PS: House foundation and posts	PS: House foundation and posts	PS: House foundation and posts	PS: House foundation and posts	PS: House foundation and posts	PS: House foundation and posts	
099-140-046	None					P	P	P	P	P	
099-140-047	None					P	P	P	P	P	
099-140-052	2(3)	House, out buildings	Two story house, unknown lower outbuilding (possible third building)	Wooden seawall/ boat dock, wooden stairs	PI: Seawall/ dock	PIS: Seawall/ dock, stairs, lower outbuilding	PIS: Seawall/ dock, stairs, lower outbuilding	PIS: Seawall/ dock, stairs, lower outbuilding	PIS: Seawall/ dock, stairs, lower outbuilding	PIS: Seawall/ dock, stairs, lower outbuilding	
099-140-053	2	House, out building	Multi-level house, Storage/ boat building	Wooden seawall, wood stairs	PI: Stairs	PIS: Stairs, concrete building foundation	PIS: Stairs, concrete foundation	PIS: Stairs, concrete foundation	PIS: Stairs, bottom of storage/ boat building	PIS: Stairs, bottom of storage/ boat building	
099-140-054	1	House	Two story house, raised foundation	Rudimentary brick seawall, propane tank?	PI: Seawall	PI: Seawall	PI: Seawall, propane tank?	PI: Seawall, propane tank?	PIS: Seawall, propane tank?, bottom of house	PIS: Seawall, propane tank?, bottom of house	
099-140-055	1	House	Three-story house and decks, appears to have raised foundation (stilts)	Rudimentary wood seawall and wood boat ramp, wood stairs	PI: Seawall, ramp, stairs	PI: Seawall, ramp, stairs	PI: Seawall, ramp, stairs	PI: Seawall, ramp, stairs	PI: Seawall, ramp, stairs	PIS: Seawall, ramp, stairs, first floor house and deck	
099-140-058	1	House	Two story house, first floor boat house raised on foundation		P	PS: House foundation	PS: House first story (boat house)	PS: House first story (boat house)	PS: House first story (boat house)	PS: House first story (boat house)	
099-140-059	None	None			P	P	P	P	P	P	
099-140-060	1	House	Single story house, raised on wood stilts		PS: House foundation	PS: House foundation and silts	PS: House foundation and silts	PS: House foundation and silts	PS: House foundation and silts	PS: House foundation and silts	
099-140-063	2	House, out building		Floating boat dock	PI: Dock	PI: dock	PI: Dock	PI: dock	PI: Dock	PI: Dock	

APN	Struct-ures	Structure Type	Structure Description	Infrastructure Description	Inundation Elevation (ft)*						Notes
					4.5-7	7-9	9-10	10-12	12-14	14+	
099-140-064	1	House, boat building	Two story house, boat shed	Stairs, wooden seawall, floating boat dock	PI: Seawall, dock, stairs	PIS: Seawall, dock, stairs, boat shed	PIS: Seawall, dock, stairs, boat shed	PIS: Seawall, dock, stairs, boat shed	PIS: Seawall, dock, stairs, boat shed	PIS: Seawall, dock, stairs, boat shed	
099-140-065	3	House	Three elevated houses or units, first levels appear uninhabited or used for storage	Wooden seawall	PI: Seawall	PI: Seawall	PI: Seawall	PIS: Seawall, lower storage level of one house/ unit	PIS: Seawall, lower storage level of one house/ unit	PIS: Seawall, lower house/unit	
099-140-089	2	Multi-level house/ Bed & Breakfast	Main two story house with raised foundation, and two level house with lower boat house	Wooden seawall, wood stairs	PIS: Seawall, boat house level	PIS: Seawall, boat house level	PIS: Seawall, boat house level, stairs, foundation of main house	PIS: Seawall, boat house level, stairs, foundation of main house	PIS: Seawall, boat house level, stairs, foundation of main house	PIS: Seawall, boat house level, stairs, foundation of main house	Jenner Inn and Cottages, Mystic Landing
099-140-090	1	House	Two story raised foundation	Wood lattice around foundation	P	PS: House foundation	PS: House foundation and lattice	PS: House foundation and lattice	PS: House foundation and lattice	PS: Foundation and lower level of house	
099-140-091	2	House, out building	Three story house, hot tub shed		P	P	PS: House first floor	PS: House first floor	PS: House first floor	PS: House first floor	Second story appears to be main living area.
099-140-092	3	House/ Lodging	Multi-level house, wood deck	Concrete foundation/ seawall, stairs	PI: Foundation/ seawall, stairs	PI: Foundation/ seawall, stairs	PI: Foundation/ seawall, stairs	PI: foundation/ seawall, stairs	PIS: foundation/ seawall, stairs, first floor of house	PIS: Foundation/ seawall, stairs, first floor of house	
099-140-093	None					P	P	P	P	P	
099-150-008	2	House	Two story houses/units connected by deck	Wood stairs and rock riprap at shoreline	P	P	PI: Stairs	PI: Stairs	PIS: Stairs, lower levels of houses	PIS: Stairs, lower levels of houses	
099-150-009	1	House	Two story house		P	P	P	PS: Lower level of house	PS: Lower level of house	PS: Lower level of house	
099-150-012	None				P	P	P	P	P	P	
099-150-013	2	House, garage	Two story house with separate garage		P	P	P	P	P	PS: Lower level of house	
099-150-014	None				P	P	P	P	P	P	
099-150-019	6+	Restaurant, lodging, out building	Restaurant, several storages, and small out building	Gravel parking lot, propane tank	P	PI: Parking lot	PI: Parking lot, propane tank	PI: Parking lot, propane tank	PIS: Parking lot, propane tank, outbuilding	PIS: Parking lot, propane tank, outbuilding	River's End

APN	Struct-ures	Structure Type	Structure Description	Infrastructure Description	Inundation Elevation (ft)*						Notes
					4.5-7	7-9	9-10	10-12	12-14	14+	
099-150-021	2	House, out building	Single story house with raised foundation, outbuilding (garage?)	Rock riprap along shoreline	P	P	PS: House foundation	PS: House	PS: House	PS: House	
099-150-022	2	House, garage	Multi-level house, separate garage	Concrete seawall	PI: Seawall	PI: Seawall	PIS: Seawall, garage and driveway	PIS: Seawall, garage and driveway, lower story of house	PIS: Seawall, garage and driveway, lower story of house	PIS: Seawall, garage and driveway, lower story of house	

*P = property, I = Infrastructure, S = Structure

At-Risk Evaluation of Individual Properties

Below is a description of at-risk structures on river front properties in the Russian River Estuary. Property numbers refer to parcels shown on aerial maps in the appendix. Properties are grouped by area: Jenner, Goat Rock, and Bridgehaven.

Jenner Area

The Jenner area consists of the northern river bank from the mouth of the Russian River to Jenner Gulch, a distance of approximately 0.8 mile. This area contains steep rocky banks, except on the east end in downtown Jenner that is generally flat. Commercial and government buildings (State Parks Visitors Center and US Post Office) occur in downtown Jenner, while residential and rental houses and cottages occur along the steep banks.

Property 1 (APN 099-150-008)
Multi-level residential house. There are two two-story residences connected by a deck. The deck connects the second story of both houses. There is another, smaller deck extended off the lower level of the house further back from the river. A wooden staircase connects the upper deck to the lower deck and extends to ground level near the shoreline. At the shoreline is boulder riprap. At water elevations above 9-10 feet wooden stairs would be flooded. At water levels above 12-14 feet the lower level of the two houses would be inundated.

Preliminary Summary of Structural Flood Risk

Water Elev. (feet)	Structural Flood Risk
4.5-7	No risk
7-9	No risk
9-10	Wooden stairs
10-12	Wooden stairs
12-14	Wooden stairs, lower level of houses
14+	Wooden stairs, lower level of houses

Property 2 (APN 099-150-009)
This property contains a multi-level, residential house. On the first and second stories of the house are wooden decks that extend off the side and back of the house, supported by wooden pillars on concrete pillar foundations. Stairs extend to the ground level. At the edge of the property and shoreline is boulder riprap. Water elevations above 10-12 feet may inundate the lower level of the house and stairs.

Preliminary Summary of Structural Flood Risk

Water Elev. (feet)	Structural Flood Risk
4.5-7	No risk
7-9	No risk

9-10	No risk
10-12	Lower level of house
12-14	Lower level of house
14+	Lower level of house

Property 3 (APN 099-150-021)

This property has a single level house on a raised foundation. A small deck extends off the main living area facing the Estuary. Large boulder riprap borders the property at the water line. There is another smaller outbuilding that may be used as a garage. This building is located behind the main house on higher ground. At water elevations above 9-10 feet the foundation may be inundated and the house may be flooded above 10-12 feet.

Preliminary Summary of Structural Flood Risk

Water Elev. (feet)	Structural Flood Risk
4.5-7	No risk
7-9	No risk
9-10	House foundation
10-12	House
12-14	House
14+	House

Property 4 (APN 099-150-022)

There is a main multi-level house with a separate garage on the property. The main house has a small deck that extends off the second level. The garage is a single level structure and has the lowest elevation of onsite buildings. An asphalt driveway extends from the top of the property to the house and garage. A cement seawall borders the property at the shoreline. The seawall maybe overtopped at a water elevation of 9-10 feet along with the lower driveway and the Estuary side of the garage. At a water elevation of 10-12 feet the garage and the edge of the two-story house may be inundated. At 12-14 feet most of the garage may be flooded and approximately two-thirds of the lower floor of the house may be inundated.

Preliminary Summary of Structural Flood Risk

Water Elev. (feet)	Structural Flood Risk
4.5-7	Concrete seawall
7-9	Concrete seawall
9-10	Seawall, garage and driveway
10-12	Seawall, garage and driveway, lower story of house
12-14	Seawall, garage and driveway, lower story of house
14+	Seawall, garage and driveway, lower story of house

Property 5 (APN 099-150-019)
This property contains a commercial restaurant and lodgings. There are six buildings on the property; a larger, multi-level building, four small cottages and a small outbuilding used for storage. A restaurant is located on the upper most level of the largest building. There is a deck that extents off the restaurant level and overlooks the river. Parking is also available adjacent to the restaurant. There are four small cabins with decks on a steep hillside overlooking the river. The cabins and decks are supported by wooded beams on concrete foundations. The restaurant and cabins are located on the upper hillside and pose no risk of flooding. At the base of the hill, near the shoreline, is a gravel driveway and parking lot containing a propane tank on a concrete pad and a garbage collection area. Near the parking lot at the same lower elevation is a small outbuilding. The propane tank and garbage area would be inundated at 9-10 feet. At an elevation above 12-14 feet the storage outbuilding may be flooded.

Preliminary Summary of Structural Flood Risk

Water Elev. (feet)	Structural Flood Risk
4.5-7	No risk
7-9	Gravel parking lot
9-10	Parking lot, propane tank
10-12	Parking lot, propane tank
12-14	Parking lot, propane tank, outbuilding
14+	Parking lot, propane tank, outbuilding

Property 6 (APN 099-140-065)
This property contains three multi-level houses or units. The building furthest up on the property is well above the shoreline. The two lower buildings on the property are both two stories. The lower levels of these buildings appear to be used for storage. The lowest building has a deck on the top level that faces the water. The deck is supported by wooden pillars on concrete blocks. Along the shoreline is a wooden sea wall that is inundated at 4.5-7 feet. At a water elevation of 10-12 feet the lower storage level of the lowest house may be flooded. At a water elevation of 14+ feet the occupied level of the house may be flooded. The two houses farthest up from the shoreline are not at risk of flooding.

Preliminary Summary of Structural Flood Risk

Water Elev. (feet)	Structural Flood Risk
4.5-7	Wooden seawall
7-9	Wooden seawall
9-10	Wooden seawall
10-12	Seawall, lower storage level of one house/unit
12-14	Seawall, lower storage level of one house/unit
14+	Seawall, lower house/unit

Property 7 (APN 099-140-064)

Multi-level residential home. On this property there is a two story home well above the shore line. There are stairs that extend from the house to the shore. At the base of the stairs is a wooden seawall. Above the seawall is a boat shed. This shed is a roofed open structure supported by wooden pillars and concrete/brick retaining walls on a concrete foundation. Next to the boat shed is a floating boat dock. The wooden sea wall is inundated at 4.5-7 feet. At a water elevation of 7-9 feet the boat shed may be inundated. There appears to be no risk of flooding to the house.

Preliminary Summary of Structural Flood Risk

Water Elev. (feet)	Structural Flood Risk
4.5-7	Wooden seawall, floating boat dock, stairs
7-9	Seawall, floating boat dock, stairs, boat shed
9-10	Seawall, floating boat dock, stairs, boat shed
10-12	Seawall, floating boat dock, stairs, boat shed
12-14	Seawall, floating boat dock, stairs, boat shed
14+	Seawall, floating boat dock, stairs, boat shed

Property 8 (APN 099-140-091)

Multi-level residential house. The house on the property is a three story residence. The second level of the house over hangs the lower level and is supported by wooden pillars anchored to the foundation of the house. The first level may be a separate residence. The second level has a deck that faces the river and appears to be the main living area. The third level is above the flood area. Also, there is a hot tub shed adjacent to the house above the flood zone. At a water elevation of 9-10 feet the first level of the house would be inundated.

Preliminary Summary of Structural Flood Risk

Water Elev. (feet)	Structural Flood Risk
4.5-7	No risk
7-9	No risk
9-10	House first floor
10-12	House first floor
12-14	House first floor
14+	House first floor

Property 9 (APN 099-140-060)

This property has a residential house. The house is single story and is raised on wooden pillars that appear to be supported by concrete pillar blocks. There is a deck that extends off the house and faces the river. At a water elevation of 4.5-7 feet the pillar foundations nearest of the shoreline would be inundated. At a water elevation of 7-9 feet the concrete pillar foundations and wooden pillars would be inundated. The inhabitable house level appears to be above the flood zone.

Preliminary Summary of Structural Flood Risk

Water Elev. (feet)	Structural Flood Risk
4.5-7	House concrete pillar foundation
7-9	House pillar foundation and wooden pillars
9-10	House pillar foundation and wooden pillars
10-12	House pillar foundation and wooden pillars
12-14	House pillar foundation and wooden pillars
14+	House pillar foundation and wooden pillars

Property 10 (APN 099-140-058)

This property contains a two-story residential house on a raised foundation. The bottom level is unoccupied and appears to be a boathouse. There is a small deck off the top level that faces the river. At a water elevation of 7-9 feet the house foundation would be inundated. At a water level of 9-10 feet the first floor boat house would be inundated.

Preliminary Summary of Structural Flood Risk

Water Elev. (feet)	Structural Flood Risk
4.5-7	No risk
7-9	House foundation
9-10	House first story (boat house)
10-12	House first story (boat house)
12-14	House first story (boat house)
14+	House first story (boat house)

Property 11 (APN 099-140-092)

This property contains multi-level buildings used as a residence and/or lodging. The house closest to the shoreline that poses the most risk for flooding appears to be two levels and is on a raised foundation of wooden pillars and concrete pillar blocks. The top level has a deck off the main living area and is on top of the lower level. The lower level has a deck that extends off the side of the house facing the river. It is supported by wooden pillars and concrete foundation. Next to the deck is a glass solarium on a raised concrete foundation that is also a seawall that extends across half of the property shoreline. In between the solarium and deck are wooden stairs to the shoreline. Other buildings and structures are above the flood zone. The concrete seawall and wooded stairs would be inundated at 4.5-7 feet. At a water elevation of 12-14 feet the first floor of the house and solarium may be flooded.

Preliminary Summary of Structural Flood Risk

Water Elev. (feet)	Structural Flood Risk
4.5-7	Concrete foundation/seawall, stairs
7-9	Concrete foundation/seawall, stairs
9-10	Concrete foundation/seawall, stairs

10-12	Concrete foundation/seawall, stairs
12-14	Foundation/seawall, stairs, first floor of house and solarium
14+	Foundation/seawall, stairs, first floor of house and solarium

Property 12 (APN 099-140-054)
This property has a two story house on a raised foundation. The upper and lower levels both have decks facing the river and are supported by the same support pillars. At the base of the house, facing the river is a propane tank. There is also a concrete block seawall near the shoreline. At a water elevation of 4.5-7 feet the seawall would be inundated. The propane tank may be inundated at 9-10 feet. The foundation of the house may be inundated at a water elevation of 12-14 feet.

Preliminary Summary of Structural Flood Risk

Water Elev. (feet)	Structural Flood Risk
4.5-7	Concrete block seawall
7-9	Concrete block seawall
9-10	Seawall, propane tank
10-12	Seawall, propane tank
12-14	Seawall, propane tank, bottom of house
14+	Seawall, propane tank, bottom of house

Property 13 (APN 099-140-053)
There are two houses on the property. The upper house and patio are at the highest elevation on the property, above flood zone. The two-story house closest to the river has a living space on the second floor and a storage/boathouse on the first floor. This house is on a concrete slab foundation with a concrete retaining wall between the house and shoreline. A deck off the second story is supported by wooden pillars anchored to the retaining wall. Wooden stairs extend from the lower level of the building to the shoreline. There is also a wooden seawall at the base of the property that borders the river. At a water elevation of 4.5-7 feet the wooden seawall and the bottom of the stairs would be inundated. At a water level of 7-9 feet the concrete retaining wall/foundation would be inundated. At a water level of 12-14 feet the bottom of the first floor storage/boathouse may be flooded. The second story is above the flood zone.

Preliminary Summary of Structural Flood Risk

Water Elev. (feet)	Structural Flood Risk
4.5-7	Wooden stairs and wooden seawall
7-9	Stairs, concrete foundation
9-10	Stairs, concrete foundation
10-12	Stairs, concrete foundation

12-14	Stairs, bottom of storage/boat building
14+	Stairs, bottom of storage/boat building

<u>Property 14 (APN 099-140-052)</u>

The main house on this property is a two-story residential house. It has a deck off the lower level that faces the river. There is also a garage uphill from the house. The house and garage are located back from the river and poses no risk of flooding. Another building that appears to be a cottage is located near the shoreline. There are stairs from the main house to the cottage and shoreline. At the base of the stair is a small boat dock. A wooded seawall borders the property at the river's edge. At a water elevation of 4.5-7 feet the wooden seawall and boat dock may be inundated. At a water level of 7-9 feet the wooden stairs cottage/outbuildings would be inundated.

Preliminary Summary of Structural Flood Risk

Water Elev. (feet)	Structural Flood Risk
4.5-7	Wooden seawall/boat dock
7-9	Seawall/boat dock, stairs, lower outbuilding
9-10	Seawall/boat dock, stairs, lower outbuilding
10-12	Seawall/boat dock, stairs, lower outbuilding
12-14	Seawall/boat dock, stairs, lower outbuilding
14+	Seawall/boat dock, stairs, lower outbuilding

<u>Property 15 (APN 099-140-089)</u>

Multi-level Bed and Breakfast lodging. This property contains two houses both on raised foundations. The larger house has three levels. The first story has a raised foundation and the second and third levels are living space. Both upper levels have decks that face the water. There are wood lattice panels that enclose the raised foundation of the house. In front of the raised foundation is a grass seating area with patio furniture. A wooden sea wall extends along the shoreline. The second house is two stories with wooden stairs between the two houses that extend to the shoreline. The first floor of this house is a boathouse that is at the shoreline. The second level is living space and has a deck on wood pillars. At a water elevation of 4.5-7 feet the wooden seawall and boathouse would be inundated, although this appears to be by design to provide boat access to the boathouse. At a water elevation of 9-10 feet the boathouse may be flooded above its design and the foundation of the larger house and stairs would be inundated.

Preliminary Summary of Structural Flood Risk

Water Elev. (feet)	Structural Flood Risk
4.5-7	Wooden seawall, boathouse level
7-9	Wooden seawall, boathouse level
9-10	Seawall, boathouse, wooden stairs, foundation of main house
10-12	Seawall, boathouse, wooden stairs, foundation of main house
12-14	Seawall, boathouse, wooden stairs, foundation of main house

14+	Seawall, boathouse, wooden stairs, foundation of main house

Property 16 (APN 099-140-090)

Multi-level residential house or bed and breakfast. This house is two stories on a raised foundation with a deck overlooking the river. The foundation is enclosed by wood lattice panels. At a water elevation of 4.5-7 feet the shoreline would reach the house foundation. At a water level of 7-9 feet the foundation with attached lattice would be flooded. The lower level of the occupied house may be flooded at 14+ feet.

Preliminary Summary of Structural Flood Risk

Water Elev. (feet)	Structural Flood Risk
4.5-7	House foundation
7-9	House foundation and lattice
9-10	House foundation and lattice
10-12	House foundation and lattice
12-14	House foundation and lattice
14+	Foundation and lower level of house

Property 17 (APN 099-140-044)

This property has a single level house elevated on wooden pillars. The area underneath the house appears to be a dilapidated boathouse. The house is in disrepair and may not be occupied. At a water elevation of 4.5-7 feet the house pillars and foundation would be inundated but the living space level is not at risk of flooding.

Preliminary Summary of Structural Flood Risk

Water Elev. (feet)	Structural Flood Risk
4.5-7	House foundation and posts
7-9	House foundation and posts
9-10	House foundation and posts
10-12	House foundation and posts
12-14	House foundation and posts
14+	House foundation and posts

Property 18 (APN 099-0140-043)

This property has a two-story house in which the bottom level appears to be uninhabited. A wooden deck on the top story is connected to the bottom level by a wooden staircase. The foundation of the house is made of rock and cement and also functions as a seawall. There also appears to be a small patio on the seawall. At a water elevation of 4.5-7 feet the seawall and lower foundation of the house would be inundated. At a water level of 9-10 feet the lower level of the house may be inundated.

Preliminary Summary of Structural Flood Risk

Water Elev. (feet)	Structural Flood Risk
4.5-7	Lower foundation/seawall
7-9	Lower foundation/seawall
9-10	Foundation/seawall, first floor of house
10-12	Foundation/seawall, first floor of house
12-14	Foundation/seawall, first floor of house
14+	Foundation/seawall, first floor of house

<u>Property 19 (APN 099-120-009)</u>

This property contains three buildings; the State Parks Jenner Visitor Center, a concrete vaulted restroom and US Post Office building on a concrete foundation. The property is flat and appears to be from imported fill material with riprap along the shoreline. The inland property boundary is Highway 1. The State Parks Visitor Center is a single level building supported by wooden piers over the Russian River. There is a viewing deck on the river side of the Visitor Center that is approximately one foot lower than the building. The postal building was recently constructed on a concrete slab foundation near Highway 1. Recent topographic surveys determined finished floor elevations of the Visitors Center at 10.5 feet and the Post Office at 14.3 feet (John Monahan, Sonoma County Water Agency). Adjacent to the Visitor Center is a concrete boat ramp with boulder riprap flanking each side. The concrete vaulted restroom is located near Highway 1 away from the shoreline. There is also a concrete parking lot between the Visitors Center, bathrooms and Post Office. This property has the highest risk of flooding in the Russian River Estuary. The riprap shoreline and lower boat ramp are inundated at water elevations of 4.5-7 feet. At a water elevation of 9-10 feet the viewing deck of the Visitor Center is inundated and the Visitors Center floods at 10.5 feet. Half of the parking lot would be flooded at 10-12 feet and access to the Post Office would likely be flooded. At a water level of 12-14 feet the entire parking lot and bathrooms would be inundated and the Post Office would be surrounded by flood waters. The Post office would be flooded at 14.3 feet.

Preliminary Summary of Structural Flood Risk

Water Elev. (feet)	Structural Flood Risk
4.5-7	Boat ramp, riprap banks
7-9	Boat ramp/rip rap banks, bottom of Visitor Center deck
9-10	Boat ramp/rip rap banks, Visitor Center deck
10-12	Boat ramp/rip rap banks, Visitor Center
12-14	Boat ramp/rip rap banks, Visitor Center, restroom, parking lot, Post Office entrance
14+	Boat ramp/rip rap banks, Visitor Center, restroom, parking lot, Post Office

Property 20 (APN 099-120-005)

Single story business complex. There is a multi-unit single story building on the property containing a café, real estate office, and tourist store. The property is flat and appears to be from imported fill material with riprap along the shoreline. The inland property boundary is Highway 1. There is a wooden patio and grass seating area adjacent to the café and overlooking the river. There is a small parking lot on the inland side and west side of the building. The riprap shoreline prevents inundation of structures below 9-10 feet water elevation. At a water level of 10-12 feet a portion of the parking lot and grass seating area may be inundated. At a water level of 12-14 feet the commercial building may be flooded and most of the parking lot.

Preliminary Summary of Structural Flood Risk

Water Elev. (feet)	Structural Flood Risk
4.5-7	Riprap banks
7-9	Riprap banks
9-10	Riprap banks
10-12	Riprap banks, parking lot, Café seating area
12-14	Businesses, parking lot, wooden patio/seating area
14+	Businesses, parking lot, wooden patio/seating area

State Highway 1 (downtown Jenner)

The two-lane asphalt State Highway 1 is the only route to access Jenner and to travel along the coastline to the north and south. The highway is adjacent to the Post Office, business complex, and gas station in downtown Jenner. Recent topographic surveys determined elevations of the road in downtown Jenner (John Monahan, Sonoma County Water Agency). The lowest point in the road is at 12.33 feet near the location of the US Post Office. At a water elevation of 12-14 feet approximately 220 feet of roadway would be flooded in downtown Jenner. Access would be restricted to a business complex, gas station, US Post Office, and State Parks Visitors Center.

Preliminary Summary of Structural Flood Risk

Water Elev. (feet)	Structural Flood Risk
4.5-7	No risk
7-9	No risk
9-10	No risk
10-12	May flood road sides
12-14	Road (restricted access to adjacent properties)
14+	Road (restricted access to adjacent properties)

Bridgehaven

The Bridgehaven area is located at the State Highway 1 Bridge over the Russian River approximately two miles upstream of the river mouth. There is a small community of several residences along the steep banks of the river, a restaurant at the south end of the bridge, and a campground in a flat area adjacent to the bridge.

Property 21 (APN 099-050-014)

This property contains a private trailer/camping park with hook-ups. It also contains a residential house. The residential house is closest to the shoreline and poses the most risk for flooding. The house appears to be a one story, manufactured/modular home with a small wooden deck with stairs. There is a driveway at the front of the house, parallel to the river. A propane tank is located near the house and driveway. At a water elevation of 12-14 feet the bottom of the house may be flooded as well as the stairs, propane tank and driveway.

Preliminary Summary of Structural Flood Risk

Water Elev. (feet)	Structural Flood Risk
4.5-7	No risk
7-9	No risk
9-10	No risk
10-12	No risk
12-14	Bottom of house, propane tank, driveway
14+	Bottom of house, propane tank, driveway

Property 22 (APN 099-080-066)

This property has a small elevated house. There is a wooden staircase that extends from the top level to the ground floor. At a water elevation of 7-9 feet the stairs would be flooded and at 9-10 feet the lower level of the house may be inundated.

Preliminary Summary of Structural Flood Risk

Water Elev. (feet)	Structural Flood Risk
4.5-7	No risk
7-9	Stairs
9-10	Stairs, bottom of house
10-12	Stairs, bottom of house
12-14	Stairs, bottom of house
14+	Stairs, bottom of house

Goat Rock Area

The Goat Rock Area is located on the south side of the Russian River Estuary and Penny Island. Most houses in this area are far from the shoreline, but one occurs near the river. The shoreline terrain varies from tidal flatlands to steep hillsides.

Property 23 (APN 099-040-031)

There are two houses on this property. The upper house is above the flood zone. The lower two-story house has wooden support columns at the front of the house and a free standing staircase. The house is near a tidal backwater slough that connects to the Russian River Estuary near Penny Island. The lower level appears to be uninhabited and may be used for storage.

Water elevation at 12-14 feet may flood the lower level of the house and support columns. The living space of the second level is above the flood zone.

Preliminary Summary of Structural Flood Risk

Water Elev. (feet)	Structural Flood Risk
4.5-7	No risk
7-9	No risk
9-10	No risk
10-12	No risk
12-14	Bottom of lower house, support columns
14+	Bottom of lower house, support columns

APPENDIX

Figure 1: Lower Estuary

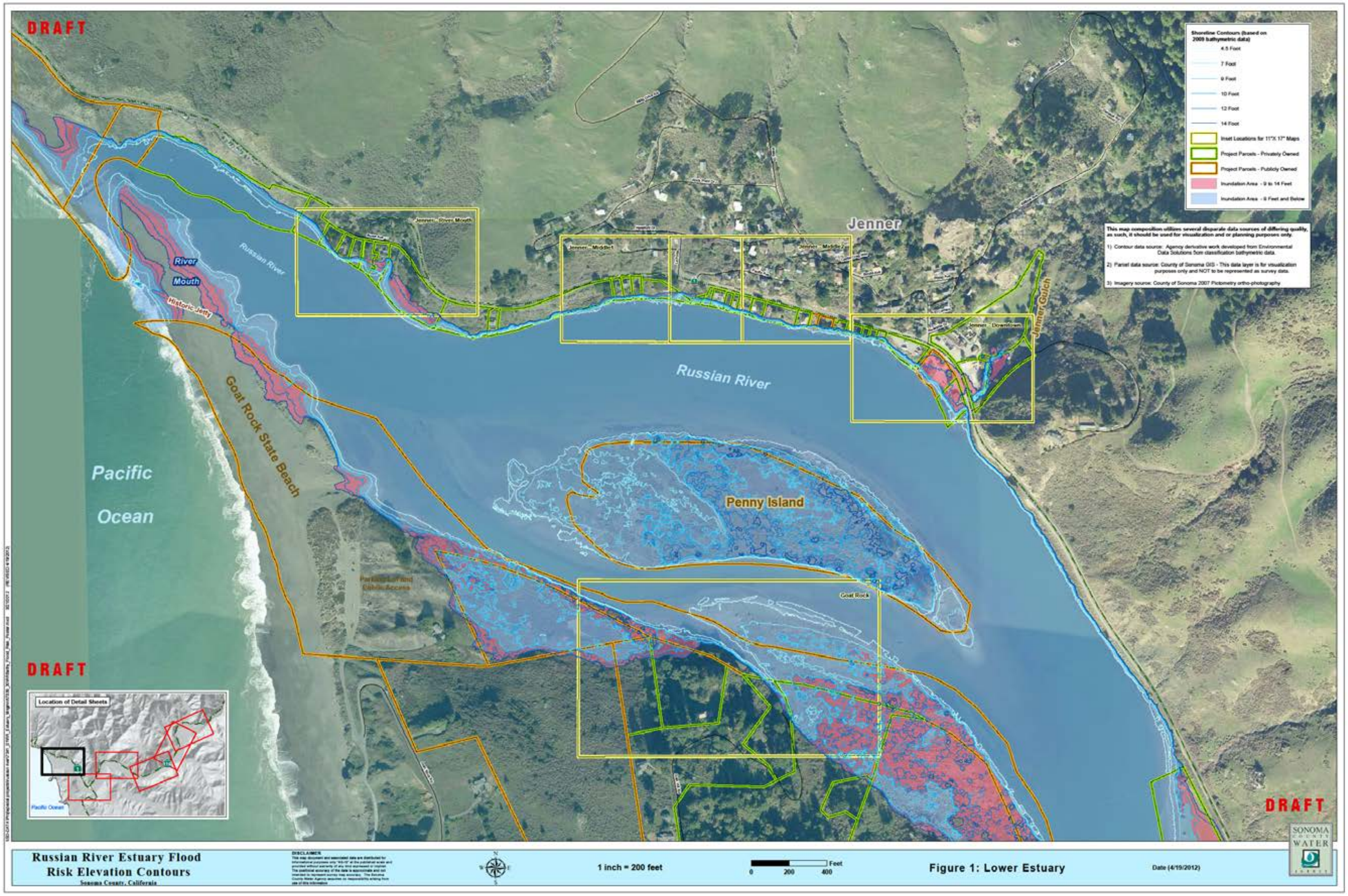
Inset Maps of At-Risk Properties

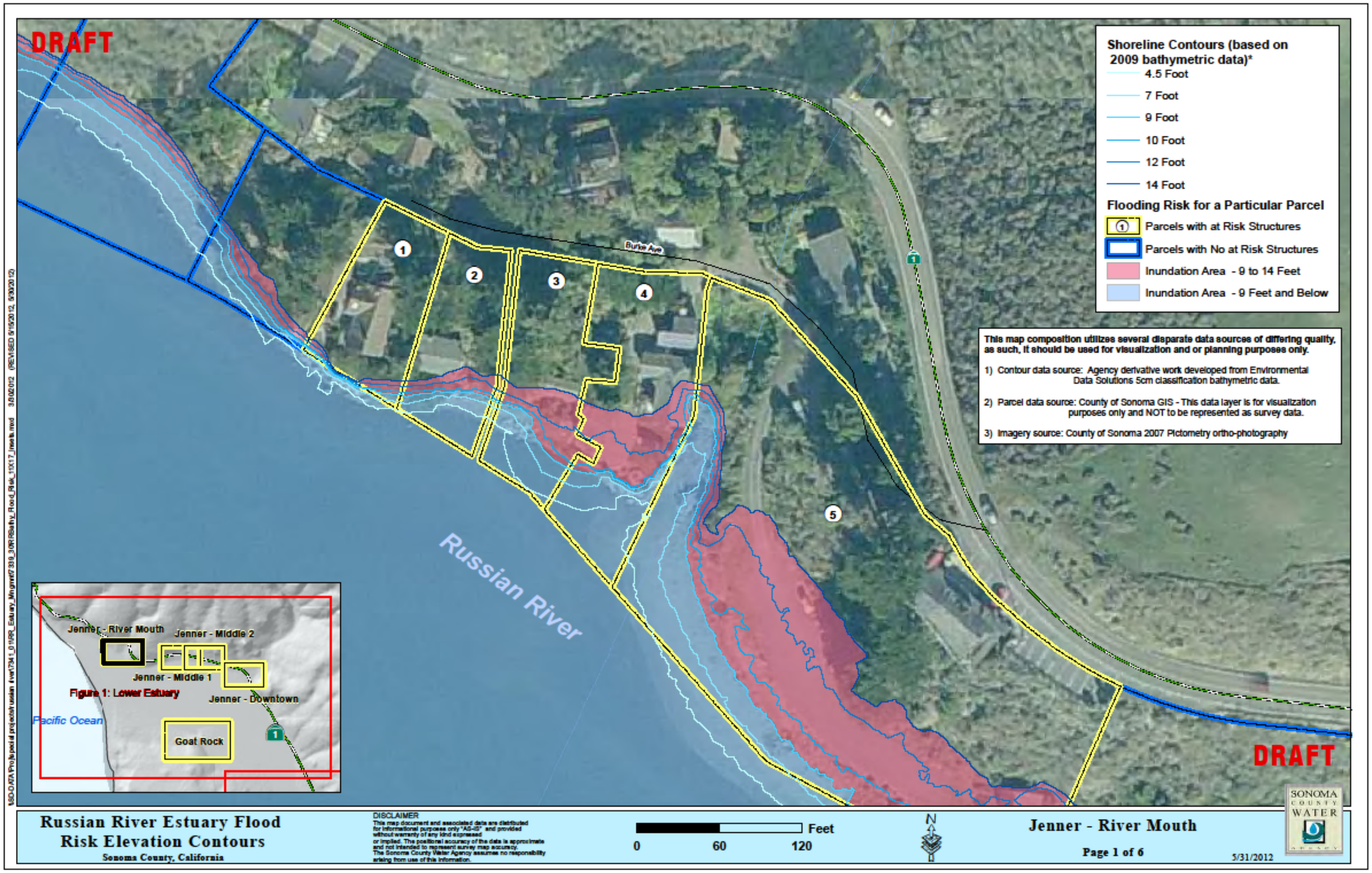
Jenner – River Mouth
Jenner – Middle 1
Jenner – Middle 2
Jenner – Downtown
Goat Rock

Figure 2: Lower Estuary (Bridgehaven)

Inset Map of At-Risk Properties

Bridgehaven


DRAFT
Shoreline Contours (based on 2009 bathymetric data)
4.5 Foot
7 Foot
9 Foot
10 Foot
12 Foot
14 Foot
Inset Locations for 11"X 17" Maps
Project Parcels - Privately Owned
Project Parcels - Publicly Owned
Inundation Area - 9 to 14 Feet
Inundation Area - 6 Feet and Below
This map composition utilizes several disparate data sources of differing quality, as such, it should be used for visualization and or planning purposes only.
1) Contour data source: Agency derivative work developed from Environmental Data Solutions 5cm classification bathymetric data.
2) Parcel data source: County of Sonoma GIS - This data layer is for visualization purposes only and NOT to be represented as survey data.
3) Imagery source: County of Sonoma 2007 Pictometry ortho-photography
Jenner
Jenner - River Mouth
Jenner - Middle1
Jenner - Middle2
Jenner - Downtown
Jenner Gulch
Russian River
River
Mouth
Historic Jetty
Russian River
Goat Rock State Beach
Pacific
Ocean
Penny Island
Goat Rock
DRAFT
Location of Detail Sheets
Pacific Ocean
DRAFT
Russian River Estuary Flood
Risk Elevation Contours
Sonoma County, California
DISCLAIMER
1 inch = 200 feet
0 200 400 Feet
Figure 1: Lower Estuary
Date (4/19/2012)
SONOMA COUNTY WATER AGENCY



DRAFT
Shoreline Contours (based on 2009 bathymetric data)*
4.5 Foot
7 Foot
9 Foot
10 Foot
12 Foot
14 Foot
Flooding Risk for a Particular Parcel
Parcels with at Risk Structures
Parcels with No at Risk Structures
Inundation Area - 9 to 14 Feet
Inundation Area - 9 Feet and Below
This map composition utilizes several disparate data sources of differing quality, as such, it should be used for visualization and or planning purposes only.
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3) Imagery source: County of Sonoma 2007 Pictometry ortho-photography
Burke Ave
1
2
3
4
5
Russian River
Jenner - River Mouth
Jenner - Middle 2
Jenner - Middle 1
Figure 1: Lower Estuary
Jenner - Downtown
Pacific Ocean
Goat Rock
DRAFT
Russian River Estuary Flood Risk Elevation Contours
Sonoma County, California
DISCLAIMER
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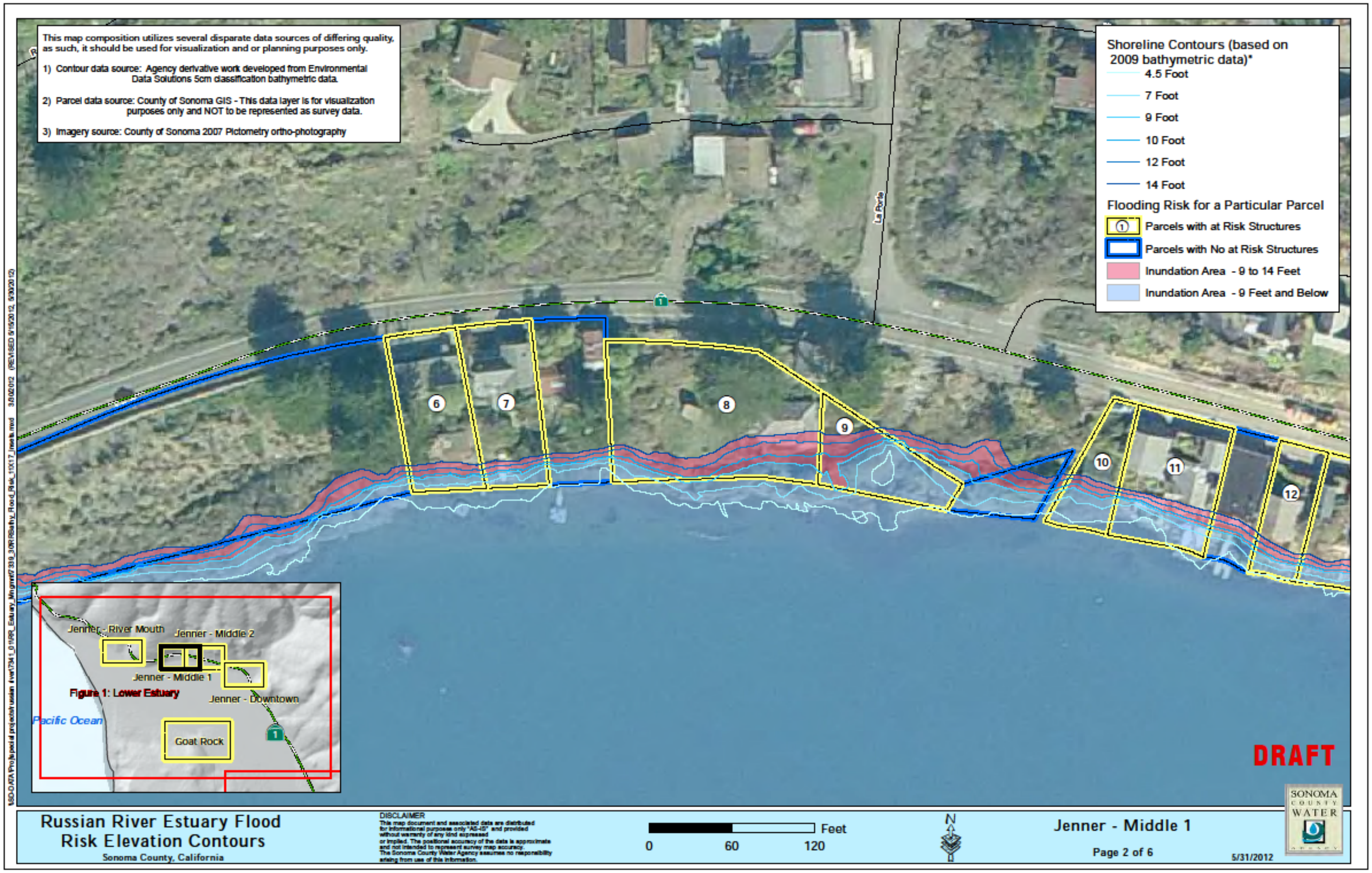
Feet
0
60
120
N
Jenner - River Mouth
Page 1 of 6
5/31/2012
SONOMA COUNTY WATER AGENCY



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2) Parcel data source: County of Sonoma GIS - This data layer is for visualization purposes only and NOT to be represented as survey data.
3) Imagery source: County of Sonoma 2007 Pictometry ortho-photography
Shoreline Contours (based on 2009 bathymetric data)*
4.5 Foot
7 Foot
9 Foot
10 Foot
12 Foot
14 Foot
Flooding Risk for a Particular Parcel
Parcels with at Risk Structures
Parcels with No at Risk Structures
Inundation Area - 9 to 14 Feet
Inundation Area - 9 Feet and Below
6
7
8
9
10
11
12
Jenner - River Mouth
Jenner - Middle 2
Jenner - Middle 1
Figure 1: Lower Estuary
Jenner - Downtown
Pacific Ocean
Goat Rock
DRAFT
Russian River Estuary Flood Risk Elevation Contours
Sonoma County, California
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Feet
0
60
120
N
Jenner - Middle 1
Page 2 of 6
5/31/2012
SONOMA COUNTY WATER AGENCY


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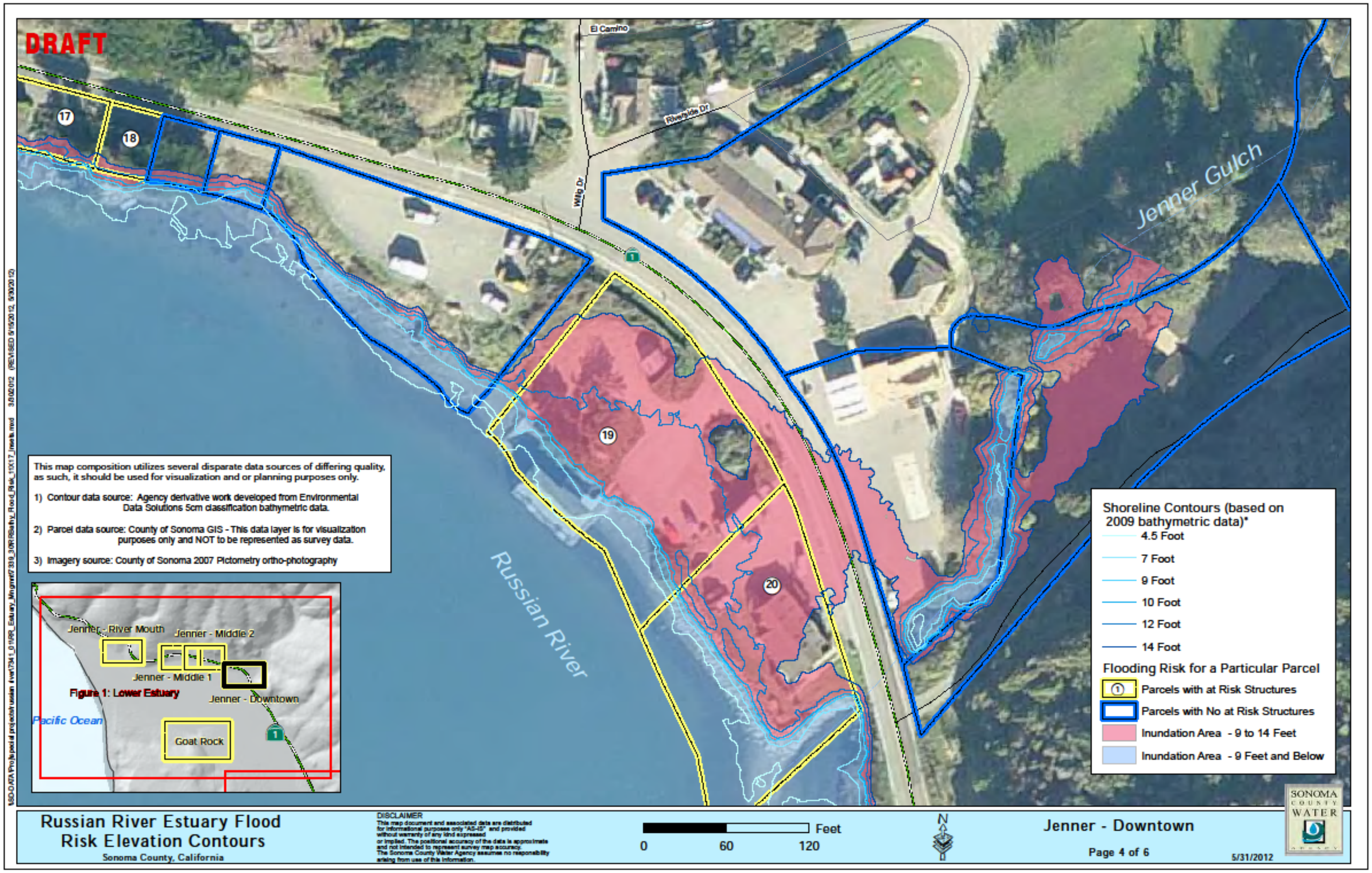
3) Imagery source: County of Sonoma 2007 Pictometry ortho-photography
Shoreline Contours (based on 2009 bathymetric data)*
4.5 Foot
7 Foot
9 Foot
10 Foot
12 Foot
14 Foot
Flooding Risk for a Particular Parcel
Parcels with at Risk Structures
Parcels with No at Risk Structures
Inundation Area - 9 to 14 Feet
Inundation Area - 9 Feet and Below
Pacific Ave
DRAFT
Jenner - River Mouth
Jenner - Middle 2
Jenner - Middle 1
Figure 1: Lower Estuary
Jenner - Downtown
Pacific Ocean
Goat Rock
Russian River Estuary Flood Risk Elevation Contours
Sonoma County, California
Feet
0
60
120
N
Jenner - Middle 2
Page 3 of 6
5/31/2012
SONOMA COUNTY WATER


DRAFT
El Camino
Riverside Dr
Willig Dr
Jenner Gulch
Russian River
17
18
19
20
This map composition utilizes several disparate data sources of differing quality, as such, it should be used for visualization and or planning purposes only.
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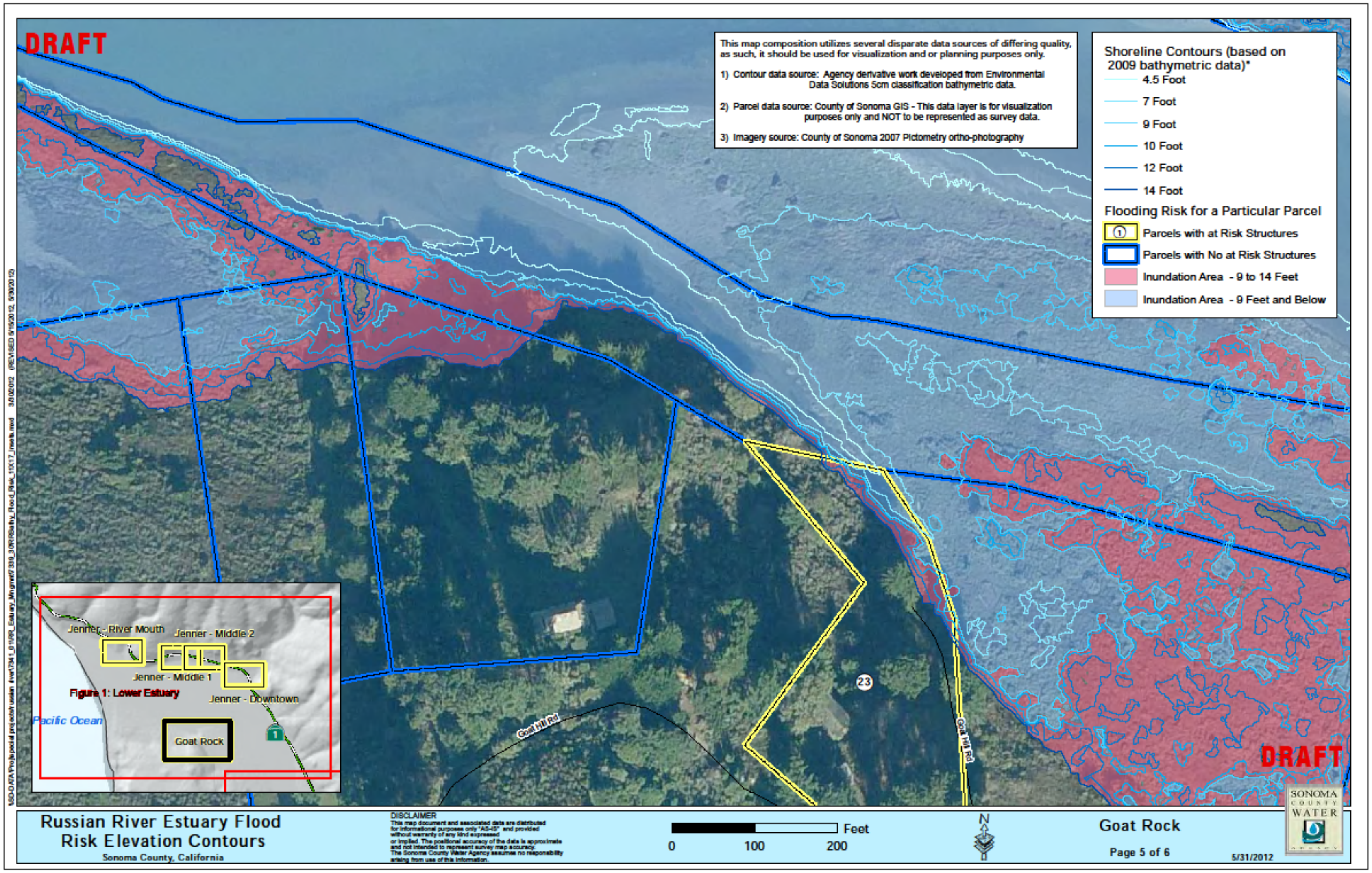
3) Imagery source: County of Sonoma 2007 Pictometry ortho-photography
Jenner - River Mouth
Jenner - Middle 2
Jenner - Middle 1
Figure 1: Lower Estuary
Jenner - Downtown
Pacific Ocean
Goat Rock
Shoreline Contours (based on 2009 bathymetric data)*
4.5 Foot
7 Foot
9 Foot
10 Foot
12 Foot
14 Foot
Flooding Risk for a Particular Parcel
Parcels with at Risk Structures
Parcels with No at Risk Structures
Inundation Area - 9 to 14 Feet
Inundation Area - 9 Feet and Below
Russian River Estuary Flood Risk Elevation Contours
Sonoma County, California
DISCLAIMER
Feet
0
60
120
N
Jenner - Downtown
Page 4 of 6
5/31/2012
SONOMA COUNTY WATER AGENCY



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3) Imagery source: County of Sonoma 2007 Pictometry ortho-photography
Shoreline Contours (based on 2009 bathymetric data)*
4.5 Foot
7 Foot
9 Foot
10 Foot
12 Foot
14 Foot
Flooding Risk for a Particular Parcel
Parcels with at Risk Structures
Parcels with No at Risk Structures
Inundation Area - 9 to 14 Feet
Inundation Area - 9 Feet and Below
Jenner - River Mouth
Jenner - Middle 2
Jenner - Middle 1
Figure 1: Lower Estuary
Jenner - Downtown
Pacific Ocean
Goat Rock
23
Goat Hill Rd
DRAFT
Russian River Estuary Flood Risk Elevation Contours
Sonoma County, California
Feet
0
100
200
N
Goat Rock
Page 5 of 6
5/31/2012
SONOMA COUNTY WATER AGENCY

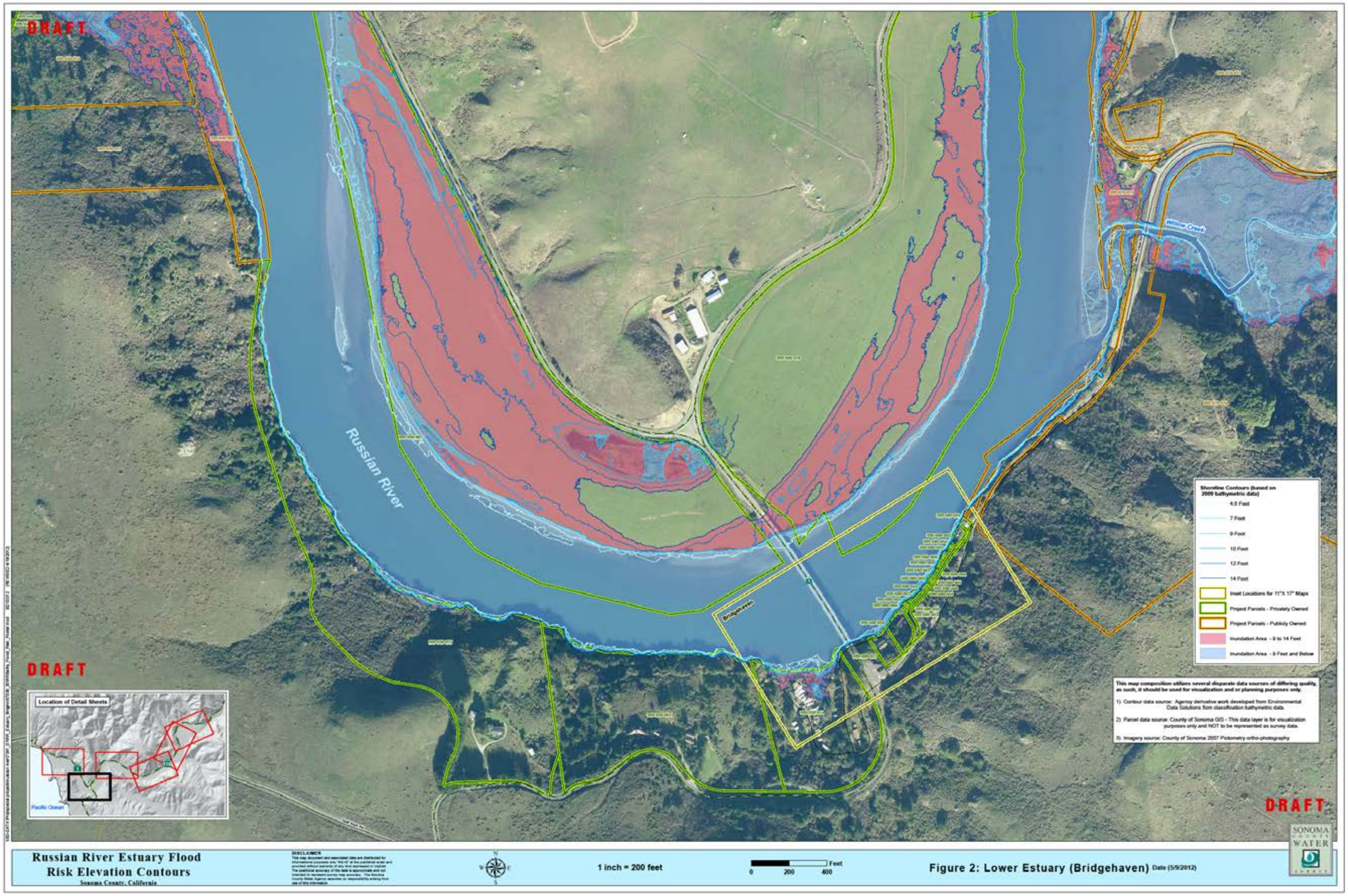

Figure 2: Lower Estuary (Bridgehaven) Date (5/9/2012)

Note: there are no identified at-risk structures upstream of Bridgehaven area.

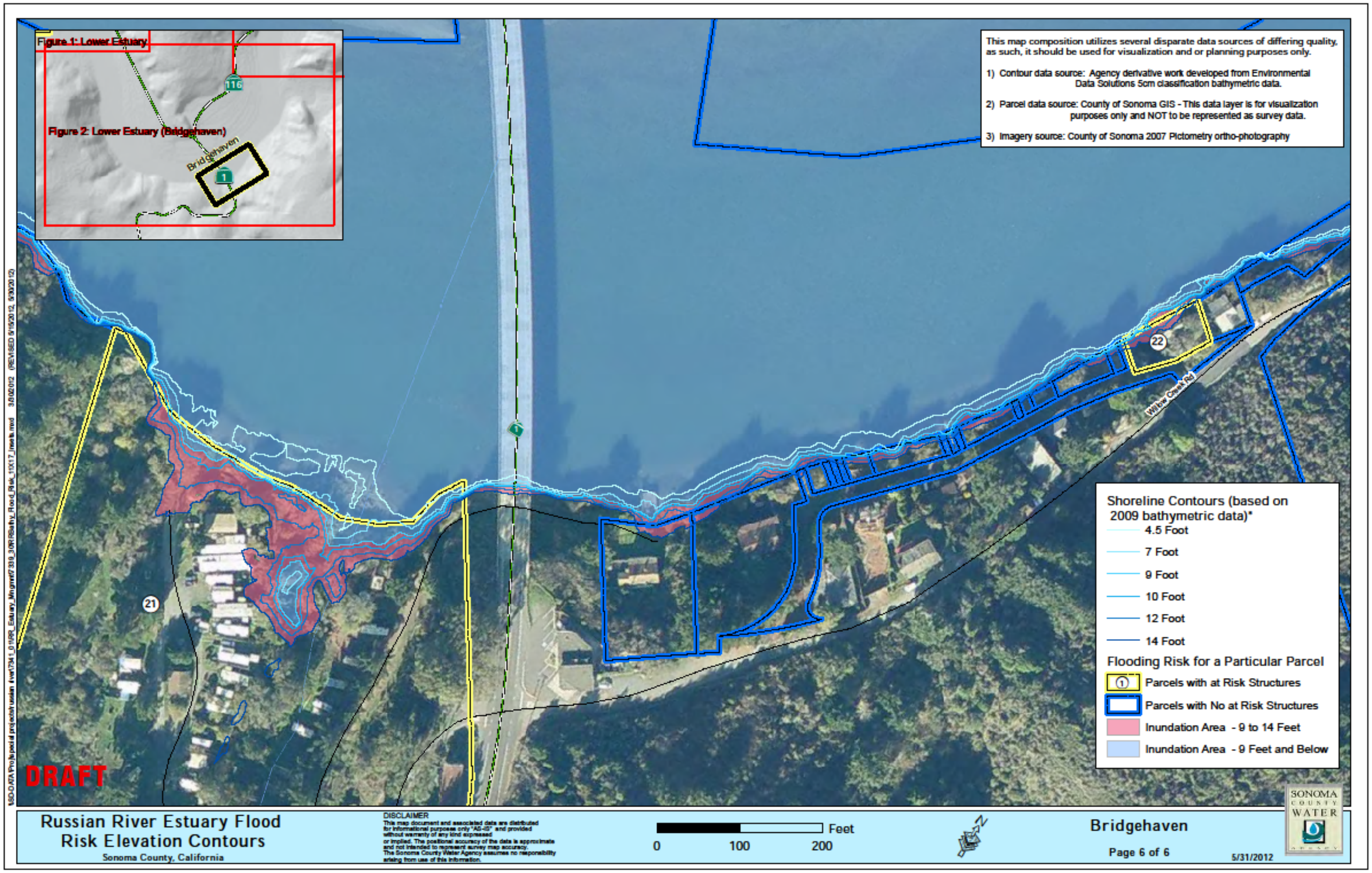

Figure 1: Lower Estuary
116
Figure 2: Lower Estuary (Bridgehaven)
Bridgehaven
1
This map composition utilizes several disparate data sources of differing quality, as such, it should be used for visualization and or planning purposes only.
1) Contour data source: Agency derivative work developed from Environmental Data Solutions 5cm classification bathymetric data.
2) Parcel data source: County of Sonoma GIS - This data layer is for visualization purposes only and NOT to be represented as survey data.
3) Imagery source: County of Sonoma 2007 Pictometry ortho-photography
22
Willow Creek Rd
21
Shoreline Contours (based on 2009 bathymetric data)*
4.5 Foot
7 Foot
9 Foot
10 Foot
12 Foot
14 Foot
Flooding Risk for a Particular Parcel
Parcels with at Risk Structures
Parcels with No at Risk Structures
Inundation Area - 9 to 14 Feet
Inundation Area - 9 Feet and Below
DRAFT
Russian River Estuary Flood Risk Elevation Contours
Sonoma County, California
DISCLAIMER
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Feet
0 100 200
Bridgehaven
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