Appendix C-1

RUSSIAN RIVER ESTUARY OUTLET CHANNEL ADAPTIVE MANAGEMENT PLAN YEAR 1

Prepared for

Sonoma County Water Agency

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1. INTRODUCTION

Sonoma County Water Agency (the Agency), with the assistance of PWA, has developed a proposed management plan for the Russian River Estuary mouth in response to a recent Biological Opinion (BO) from the National Marine Fisheries Service (NMFS) designed to improve salmonid rearing habitat in the estuary (NMFS, 2008). The proposed plan revises the existing Russian River Estuary outlet channel management plan.

The management plan documented in this report is for the first year (Year 1) of management following issuance of the BO. The BO recognizes several phases of outlet channel management over fifteen years with additional management options specified for each phase. The outlet channel is 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 Year 1 management plan, to be implemented in 2009, is part of the first phase of outlet channel management (Phase 1) specified in the BO.

The approach of the Year 1 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 and recreational resources of the site. It is the Agency's intent to apply for modified permits that allow additional flexibility to manage the outlet channel to meet the goals of the BO for Year 2 onward. It is recognized that the measures developed in the Year 1 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.

The goal of the management plan is to reduce marine influence on the Russian River Estuary (Figure 1) during the management period, May 15th through 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 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 adaptive implementation of this outlet channel is the focus of this management plan. This adaptive management plan, as documented in this report, is initiated with planning that includes: (1) defining project performance criteria, (2) developing a conceptual model of relevant physical processes, and (3) conducting technical analysis to quantify target outlet channel conditions. The resulting operations and management plan derived from these planning steps is also documented in this report. 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 Year 1 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 are no known occurrences of target outlet channel conditions occurring during the proposed management season of May 15 to October 15 for the ten year period of record (1999 to 2008) for which water levels and channel photographs are available. Instead, 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 2008. However, 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 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 the goal of improving salmonid habitat because it immediately exposes 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 2009 instream flows, the predicted local bed shear stress during the management period fluctuates above and below the critical bed shear stress threshold.
8. River discharge at Jenner is a significant source of uncertainty for hydraulic conditions in the outlet channel. Discharge measurements are made at the USGS Guerneville gaging station (11467000), 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 losses between the Guerneville gaging station and the mouth vary from 10% to 53% and averaged 37%. Limited USGS measurements suggest lower losses.

2.2 RECOMMENDATIONS: YEAR 1 MANAGEMENT ACTIONS

1. Initial management actions should be conservative to minimize the risk of breaching, even if this means increased likelihood of closure. Practically, this means an initial preference for smaller, but perhaps more frequent, management actions that are corrections to the existing channel configuration. Based on experience from these initial efforts, larger and less frequent actions may be undertaken.
2. Once the estuary closes, take early actions to widen the channel and increase conveyance 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. It may also result in lower lagoon water levels, particularly early in the season.
3. The Year 1 target outlet channel will be approximately 100 feet wide, 0.5 to 2 ft deep, and occupy a planform alignment within the area occupied by the channel under current management practice. A wider, shallower channel will be more resilient, less likely to 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. 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 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. Marine conditions diminish habitat quality for salmonid rearing by reducing the habitat extent, elevating salinity above optimal levels for salmonids 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 Year 1 management.

Performance criteria for water quality and ecological values in the lagoon are addressed separately and are not included in this document. In addition, management of the outlet channel for steelhead habitat may impact on other species that use the estuary such as seals and birds. The Agency is addressing these impacts through a separate process.

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, the lagoon is subject to undesirable water quality conditions not just during the breached period, but also for some period of time following the breach. "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 management plan should anticipate a permanent reduction in instream minimum flow requirements between the Dry Creek confluence and the mouth starting in 2010. Minimum flows will be reduced from current State Water Resources Control Board Water Right Decision 1610 levels of 125 ft^3/s to 80-85 ft^3/s [1]. The expected reduction in minimum river flows will provide more favorable conditions for channel management to avoid 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 YEAR 1

As discussed above (Section 1), the approach of the Year 1 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 Year 1 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.

The management plan assumes that under existing regulatory permits the Agency may excavate up to 1000 cubic yards of sand per excavation event (as specified in the permits) to create a

[1] 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).

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).

For Year 1, performance objectives for lagoon water levels will be more tolerant of lower water levels (i.e., average water levels may be less than 7 ft NGVD). This approach reduces the risk of uncontrolled breaching within existing permit constraints. The outlet channel may function less frequently as a unidirectional channel. The objective will be to reduce tidal flows to the extent feasible, including creating muted tidal conditions in the lagoon. Lastly, artificial breaching may be required more frequently during Year 1. With this management plan, SCWA seeks to minimize or avoid such breaches during the management period, but recognizes that they may be needed to avoid flooding of adjacent properties.

Because of a multi-year drought, the Agency has petitioned the State Water Resources Control Board to temporarily reduce the minimum instream flow requirements in 2009 to 35 ft^3/s for the reach between the Dry Creek confluence and the mouth. The low river flows expected during the Year 1 management season provide more favorable conditions for channel management within existing permit constraints.

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 are 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 discussion with project stakeholders (the Agency, NMFS, California Department of Fish and Game, and California State Parks). An initial version of this model was presented to a meeting of project stakeholders on March 23, 2009. Based on feedback from that meeting and the technical analyses detailed below, the conceptual model was revised and included in this management plan.

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 and the outlet channel discharge are the two largest flows; evaporation and seepage are minor factors in the water balance. As such, the outlet channel discharge capacity needs to nearly match the river discharge. If the discharge is too low, the estuary water level will rise to flood stage and artificial breaching will be necessary. If the 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. 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 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 significantly 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 considered because it is assumed that management actions would be enacted to return the outlet channel to target conditions prior to a breach. 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). Whether the flow velocity is below the threshold depends on 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 a wider 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 impacts the 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 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 moves in the direction of the littoral transport (Dean and Dalrymple, 2002). However, observations by NMFS suggest that the direction of migration may be reversed for outlet channel such that the mouth moves 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. 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, with an emphasis on 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-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.

Although there were no instances of overflow conditions during the proposed management period, there were five relevant events that occurred just outside of this period during the years 1999-2008. 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 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 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 conveyence:	<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 highwater 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, Year 1**	**Carmel River Lagoon**
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 4. 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 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, 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

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.

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 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 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 Calculation of estuary inflows

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 between Guerneville, a monitoring station approximately 15 miles upstream of the Austin Creek confluence that marks the start of the Russian River estuary. The losses in discharge are believed to be due to diversions, interaction with the adjacent aquifer, and groundwater pumping, although no detailed information 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 2008 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 assumed negligible and not included in the water balance model. This assumption is based on wave conditions and wave event duration during the May through October management period. Wave conditions during the management period are associated with beach berm building, not with extensive overtopping and berm erosion more prevalent during winter storm events. In addition, the duration of wave events are typically shorter than the duration of closure ($\triangle t$), and therefore overtopping, if present, would likely coincide with the start of the closure, not the later portion of the closure from which change in water level ($\triangle h$) observations were used in the water balance model. As an initial check on the assumption of negligible overtopping volume, the potential increase in lagoon water level was calculated for overtopping events only likely to occur during extreme winter storms. FEMA coastal flood guidelines (Jones et al., 2005) suggest 1 ft^3/s per foot of beach berm as the upper range for overtopping rate during extreme events. Applying this rate to the beach berm length and estuary area at the Russian River estuary, the lagoon water level would rise less than 0.5 ft. Since this amount of water level increase is an upper bound not likely to occur during the management period and since even this upper bound is considerably less than the change in water level values ($\triangle h$) applied in the water balance model, the negligible wave overtopping assumption is reasonable. If more detailed wave and berm dimension data are collected at the site, more detailed wave overtopping volumes during the management period can be estimated.

<u>Model Calibration</u>

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 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 discharge measurements provides another estimate of estuary inflow relative to the continuous discharge measurements at Guerneville. These discharge measurements, collected at four stations[2] in the 14 miles below Guerneville, typically fall within 10% of the Guerneville average daily discharge. This suggests that the water balance model may over-predict the reduction in flow losses between Guerneville and the estuary. Since the results of the water balance are used to estimate estuary inflow in the unsteady hydraulic model (see Section 6.2.3 below), the estuary inflow values in the unsteady hydraulic model may under estimate actual estuary inflow. Presently, the existing data are insufficient to explain the discrepancy between the water balance model and the discharge measurements. Higher rates of seepage through the beach berm are one possible explanation. Monitoring to resolve this discrepancy is recommended in Section 7.7. The USGS data was evaluated late in development of the Outlet Channel Adaptive Management Plan. Though consideration of higher estuary inflows is not carried through all aspects of the analysis presented in this report, the implications of potentially higher 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, 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. 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. 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

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

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[2] 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).

(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.

The year 2000 discharge time series was used to generate a synthetic discharge time series to approximate anticipated 2009 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 2009 conditions was derived from the typical discharge time series by scaling the Guerneville discharge to an average summertime flow of 70 ft^3/s. This reduction ratio of approximately 40% is based on the anticipated 2009 emergency instream flow requirements versus historic instream flows. In addition to averaging 70 ft^3/s, short-term variability ranges from about 50-100 ft^3/s during the May to October management period. The resulting discharge time series at Guerneville is shown in Figure 7a for the simulation period.

The anticipated 2009 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. Predicted 2009 inflows to the lagoon vary from approximately 30-50 ft^3/s and average approximately 40 ft^3/s during the summer months. This is consistent with the reduced instream flow requirements obtained by the Agency for 2009 operations (Section 3.2).

<u>*Model Setup*</u>

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. 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.

<u>*Results*</u>

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 predicictions 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 2009 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). This corresponds to flow rates at Guerneville below a threshold discharge of approximately 90 ft^3/s. Bed shear stress fluctuates near the critical shear stress during this period, suggesting some potential for sediment motion and scouring of the channel. Lagoon water levels are relatively constant around 5.5 ft NGVD, resulting in a typical flow depth of approximately 0.5 ft in the channel. Channel velocities are approximately 0.7-0.8 ft/s.

6.3 SENSITIVITY ANALYSIS AND UNCERTAINTY

PWA conducted sensitivity and uncertainty model runs for important variables and parameters to assess the impact on channel performance. Parameters tested were: (1) discharge reduction coefficient, (2) bed roughness (Manning's n), and (3) critical shear stress.

Discharge reduction coefficient

The model calibration procedure for discharge losses from the USGS gaging station at Guerneville to the Russian River lagoon is described in Section 6.2.2. Comparison of observed and predicted rates of water level increase during closure events demonstrated losses of 10-50% relative to the measured upstream discharge. The baseline simulation presented in Section 6.2.3 used a calibrated seasonally-varying coefficient to reduce flow rates into the lagoon, typically resulting in a reduction of 30-50% over the management period. To test channel performance during higher than expected summertime flows (due to lower groundwater losses, diversions, etc), a sensitivity run with a constant reduction factor of 20% was conducted. As discussed above (Section 6.2.2), limited USGS discharge measurements suggest a reduction factor of no more than 10%.

Bed Roughness (Manning's n)

Manning's n is a coefficient that characterizes the surface roughness of the channel bed. For sandy channels, roughness is primarily a function of grain size. Various parameterizations exist for estimating bed roughness (Bray 1979; Bruschin 1985; Julien 2002; Limerinos 1970; Strickler 1923, USGS 1984), yielding Manning's n values of 0.017-0.026 for a grain size of 1 mm (Attachment A-2). A Manning's n of 0.02 was selected for the baseline simulation presented in Section 6.2.3. To test the sensitivity of the results on bed roughness, a sensitivity run with a roughness of 0.025 (25% higher) was conducted.

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. This uncertainty band is shown in Figure 8a to illustrate the uncertainty in the critical shear stress threshold relative to the modeled bed shear stress.

Results

The results of the roughness and discharge reduction coefficient ("Losses") sensitivity model runs are shown in Figure 8a for bed shear stress and Figure 8 b for lagoon water level. Higher than anticipated bed roughness results in a less hydraulically efficient channel that elevates lagoon water levels and channel depths by a small amount (<0.1 ft) during the summer months (July 15-October 15). Average bed shear stress during the same period increased by approximately 25% from 0.60 Pa for the baseline simulation to 0.76 Pa. Higher than anticipated flows (20% diversion scenario) had a similar, but more significant, impact on bed shear stress and water level. Average water levels and channel depth increased by approximately 0.13 ft relative to the baseline simulation. Average bed shear stress increased by approximately 60% to an average value of 0.95 Pa for the summer months, well above the expected critical shear stress threshold. If flow losses between Guerneville and the estuary are even less than 20%, as suggested by the USGS discharge data, bed shear stress would be even higher for a channel constrained to a width of 100 ft.

The results of the sensitivity simulations suggest that while the outlet channel appears to operate in a marginally stable state for the anticipated conditions, variability in sediment grain size, bed roughness, and lagoon inflow could result in channel scour (widening or deepening) or breaching. If necessary, a wider channel could be excavated (or could develop naturally) to reduce bed shear stress below the critical threshold. It should also be noted that the simulations for 2009 anticipated hydrology assume lagoon inflow based on proposed reductions to minimum in stream flow requirements for the summer of 2009. In future years, discharge to the lagoon may be higher than modeled and a wider outlet channel may be required to convey flows below the critical threshold for sand transport.

7. PROPOSED OUTLET CHANNEL ADAPTIVE MANAGEMENT FOR YEAR 1

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

The strategy for outlet channel management is an incremental approach that seeks to minimize the risk of uncontrolled breaching. This strategy includes the following:

- favoring smaller, more frequent modifications over larger, less frequent, modification with less certain outcome
- tolerating more frequent channel closure to avoid the channel breaching to fully tidal conditions
- initially managing estuary water levels at the lower end of the 4-9 ft NGVD range to reduce the scour potential associated with larger water surface differences between the lagoon and ocean

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.

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 dimensions and supporting operations details.

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. It is important to initiate excavation shortly after closure, to prevent lagoon water levels from rising too high above the elevation of the beach berm before the outlet channel can be constructed.

Should the outlet channel close in the weeks immediately preceding the management period, the Agency may initiate excavation to increase the likelihood of entering the management period with the target channel configuration in place. If the channel remains open for some period after May 15th or is breached later in the management season, then begins to show signs of closure (reduced tidal range), the Agency may consider grading to assist channel closure.

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 regrade the channel before the lagoon water level rises too high above the new (higher) beach berm elevation.

7.3 TARGET CHANNEL DIMENSIONS

7.3.1 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. At the start of the management season, lagoon water levels and the channel bed are likely to be lower in elevation, since the system will have recently transitioned from intertidal to closed. 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 channel bed builds higher, it will support higher lagoon water levels while maintaining channel depth within the target range. 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 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 outflow through the outlet channel and that net sand deposition or erosion does not impair the outlet channel's function. However, this goal may not be achievable in Year 1 because additional constraints in place during this first year call for modified performance criteria.

Channel bed elevations are expected to be in the range of 3 to7 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, the minimum beach elevation will be at or just above the tide range when the berm closes, probably between 3 ft and 5 ft NGVD. This minimum beach elevation will be the elevation of the channel bed. The median wave runup elevation during the management period is approximately 6 ft NGVD (Section 5.2.3 and Figure 5). However, intermittent large wave events increase the wave runup to elevations above 9 ft NGVD, during which time closure is more likely. Conceptually, the desired channel bed elevation range is limited on the high end by flooding and on the low end by wave runup (which can close the channel). For example, the upper end of the channel bed elevation, 7 ft NGVD, can be thought of as limited by the flood stage elevation (9 ft NGVD) minus the typical design channel depth (1 ft) and a factor of safety (1 ft). Developing a better feel for these parameters is one objective of the adaptive management plan.

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.

7.3.2 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). 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.

7.3.3 Width

The width of the channel is estimated to vary within 25-100 ft for consistency with the existing management permits. Initial management will start with excavating a channel approximately 100 ft wide to provide maximum feasible capacity to convey discharge without scouring the channel. If experience demonstrates that this wider channel has excess conveyance capacity, subsequent modifications may consist of narrower channel widths. On the other hand, if actual estuary inflows are larger than predicted (see Section 6.2.2), then a wider channel may be required to convey sufficient discharge to prevent rising lagoon water levels while simultaneously avoiding scour in the outlet channel. Because of permitting constraints, a channel wider than 100 ft cannot be implemented in Year 1 (Section 3.2).

7.3.4 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.

7.4 CHANNEL LOCATION/PLANFORM ALIGNMENT

The initial 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). 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. If the channel closes, alternative channel alignments may be implemented to test the relationship of mouth location on channel stability. Various channel locations within the extent of the existing alignment (Figure 1) may be pursued to take advantage of site features such as areas of reduced wave energy and rocks imbedded in the beach.

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 AND VOLUMES

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 daily or near daily from some initial period. 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.

The quantity of sand moved will depend on antecedent beach topography. To stay consistent with current management practices, excavation volumes will not exceed 1,000 yd^3. Any sand excavated from the channel will be placed on the adjacent beach in such a way as to minimize changes to beach topography outside the outlet channel.

7.7 MONITORING

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. 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. 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 Year 1 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.	Every 2 weeks
Outlet channel bed structure	Observe the bed for bed forms and depth-dependent grain size distribution indicative of armoring. Sediment sampler used.	Field staff to collect sediment sample from the surface of the channel bed.	Monthly
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 a reflectorless total station from adjacent hillside.	Monthly
Estuary discharge measurements	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

7.8 UNCERTAINTY AND LIMITATIONS

The proposed operations are based on the analyses documented in this report 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. 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.

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.

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9. LIST OF PREPARERS

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10. FIGURES

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

0 250 500 1,000 Feet

figure 1

Russian River Estuary Outlet Channel Management Plan

Russian River Estuary Site Location

PWA Ref# - 1958.01

PWA

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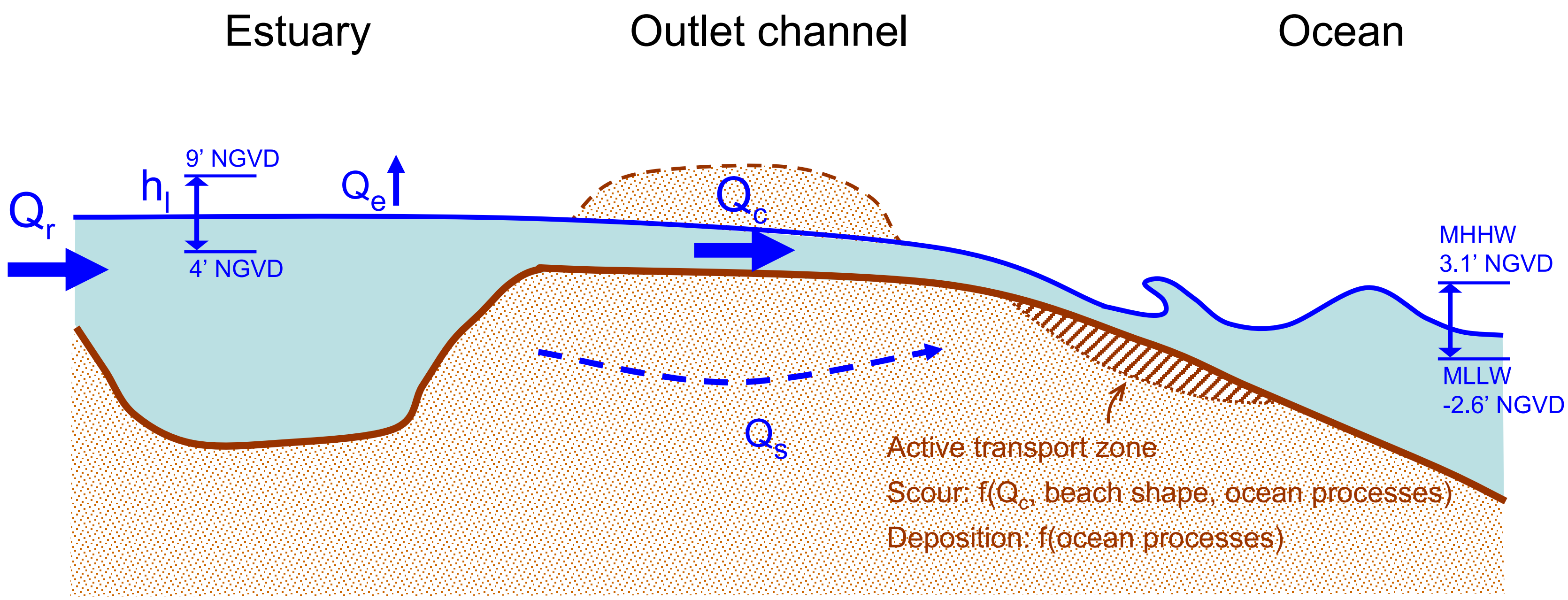


<u>Parameters</u>

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

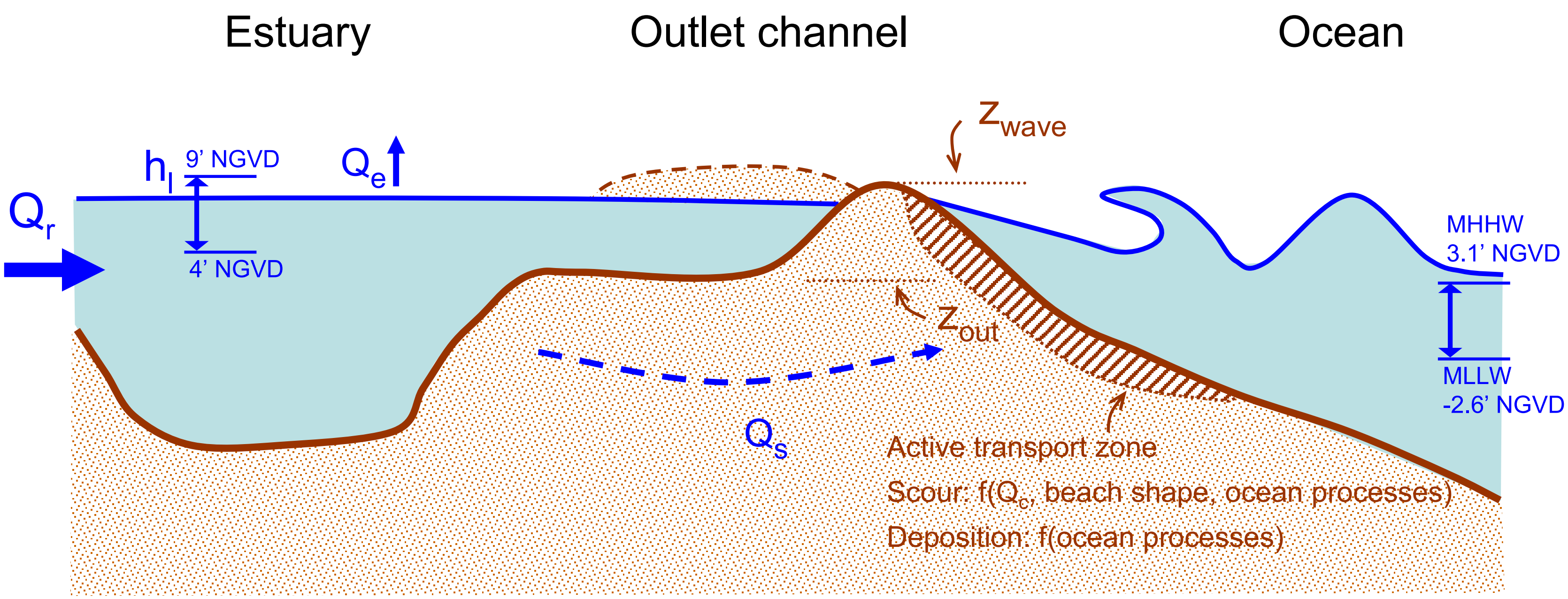
figure **2**

Russian River Estuary Outlet Channel Management Plan

Conceptual model – Target conditions

PWA Ref# 1958.01

PWA



Parameters

z_{out}=target channel bed elevation

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

Processes

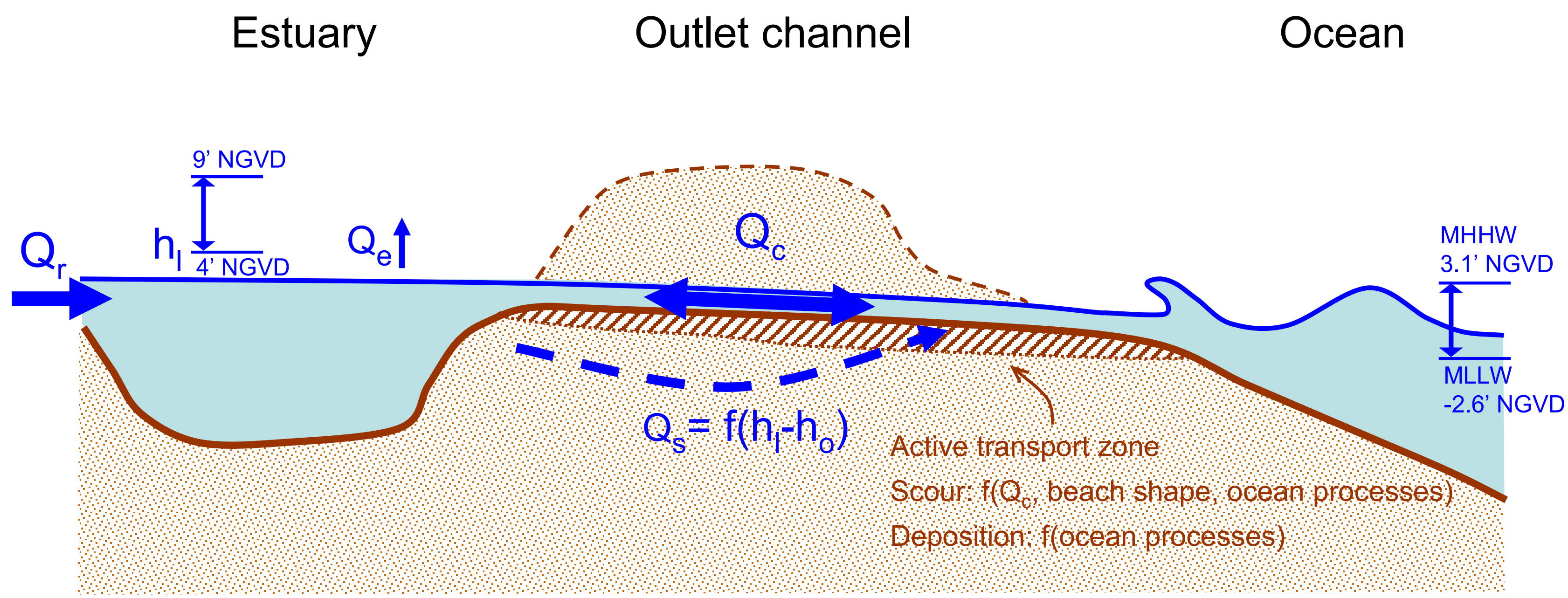
- $z_{wave} \geq z_{out}$
- wave-induced sediment transport closes outlet channel
- $Q_c \rightarrow 0$
- h_l increasing

figure **3**

Russian River Estuary Outlet Channel Management Plan

Conceptual model – Closure

PWA Ref# 1958.01



<u>Parameters</u>

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)

<u>Processes</u>

•u_c > u_{crit}; high velocities scour channel

•Q_s increases; high seepage creates groundwater piping and erosion

•sediment transport within outlet channel

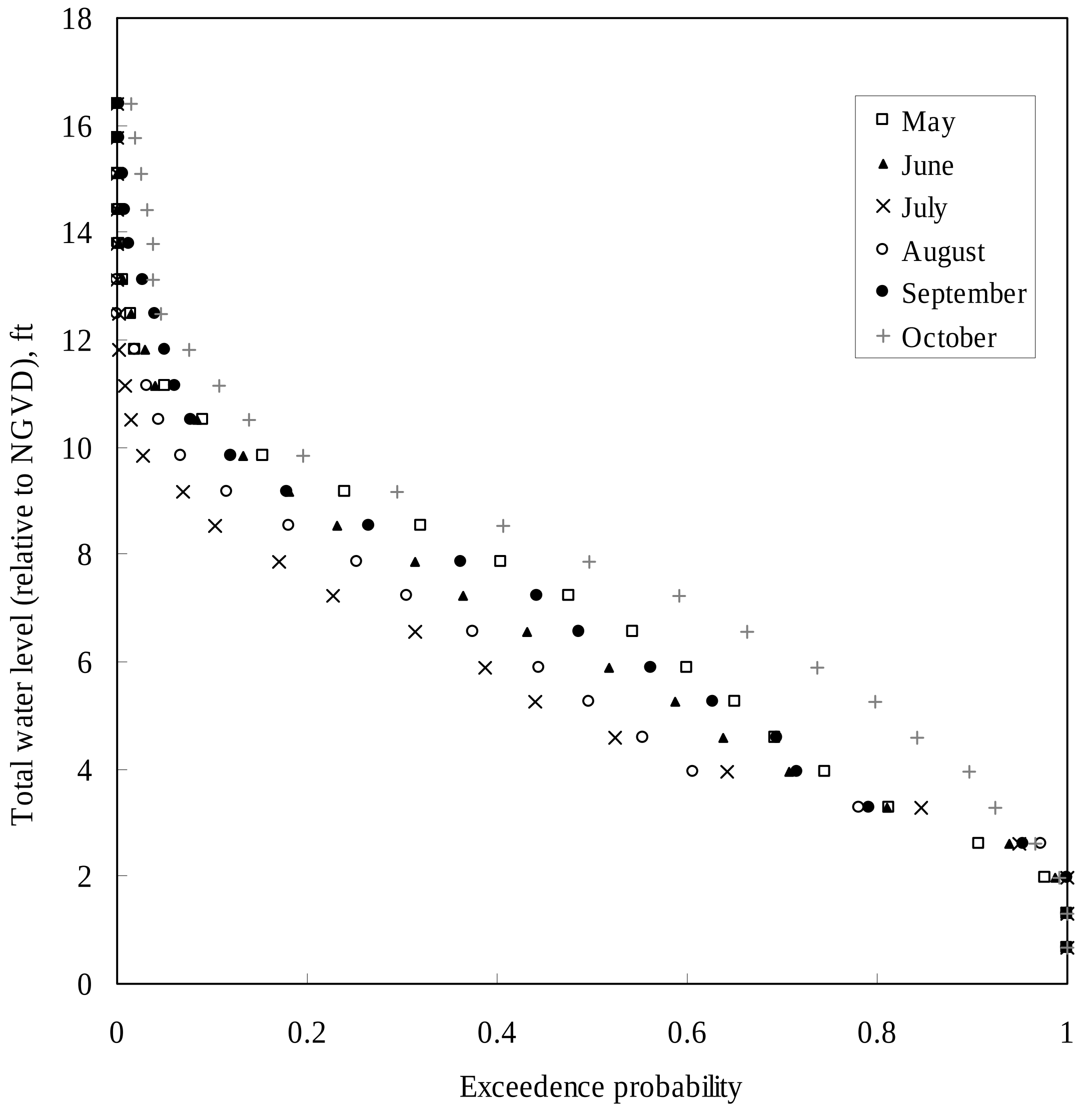
figure ***4***

Russian River Estuary Outlet Channel Management Plan

Conceptual model – Breaching

PWA Ref# 1958.01

PWA



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.

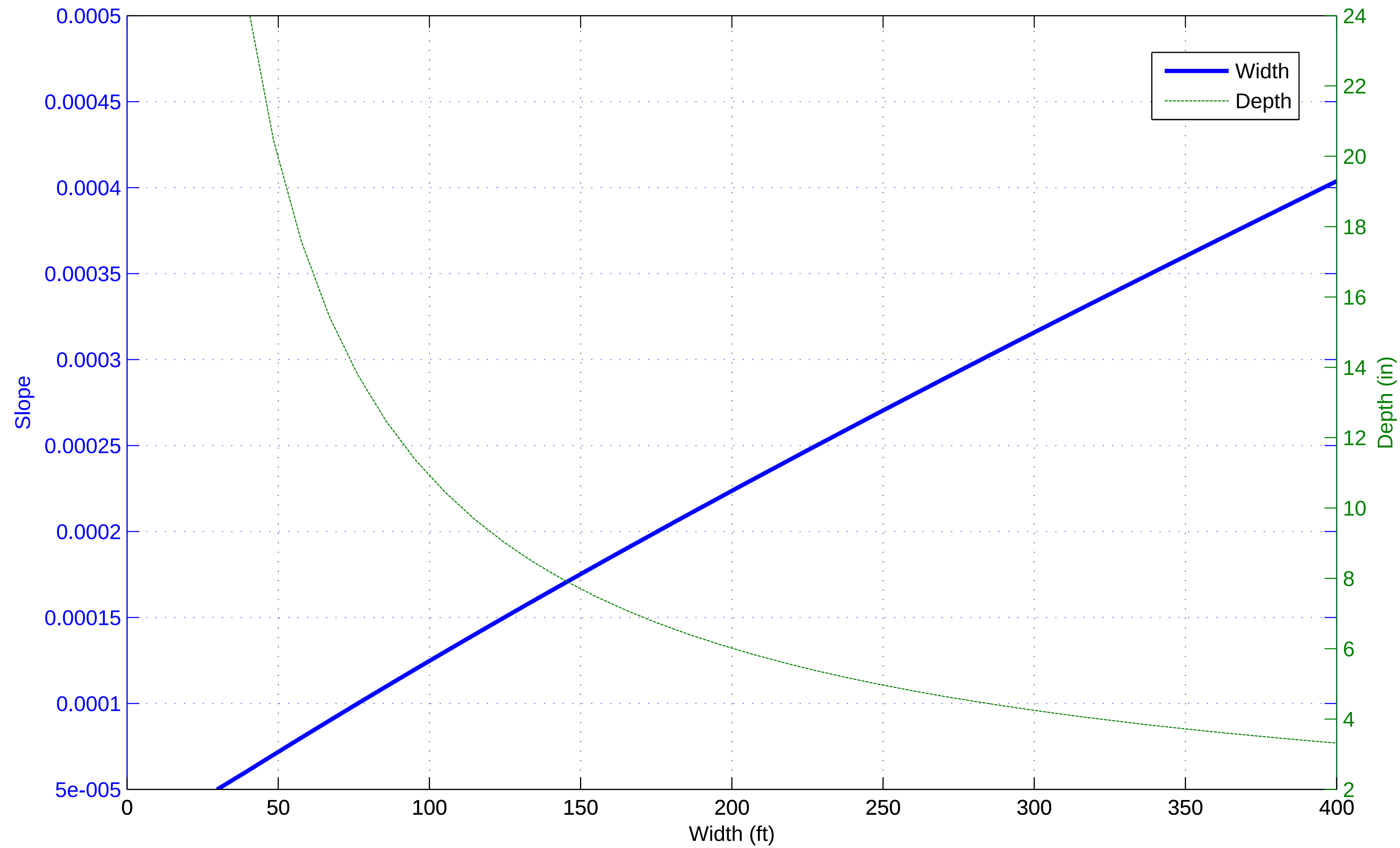
figure **5**

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

PWA

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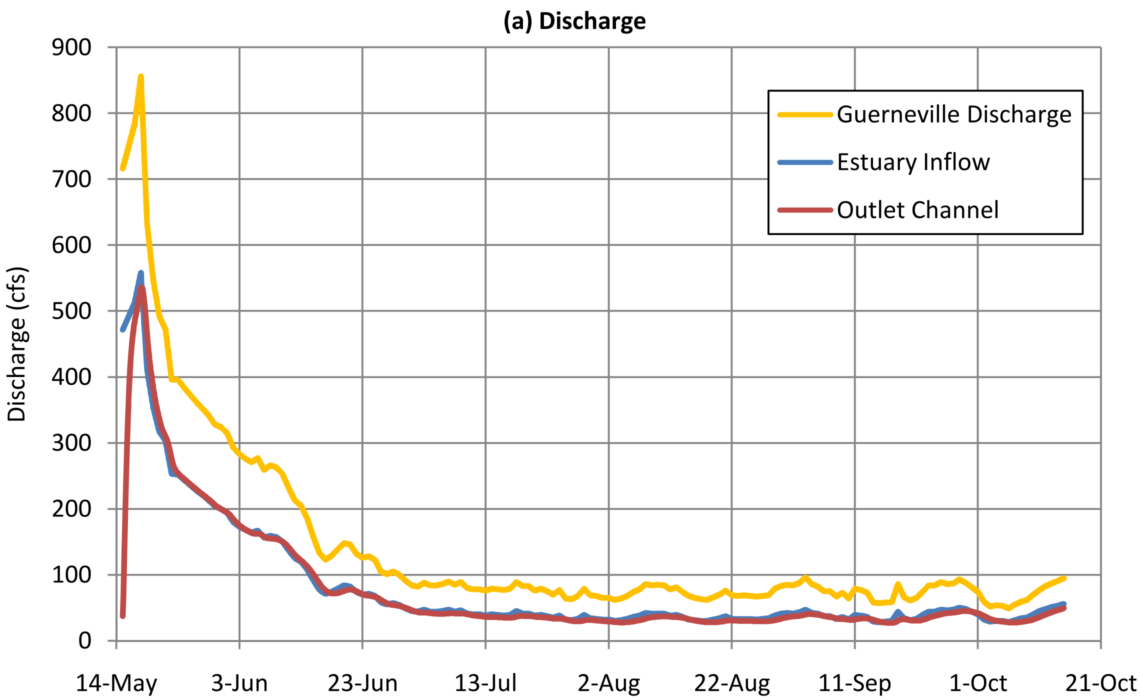
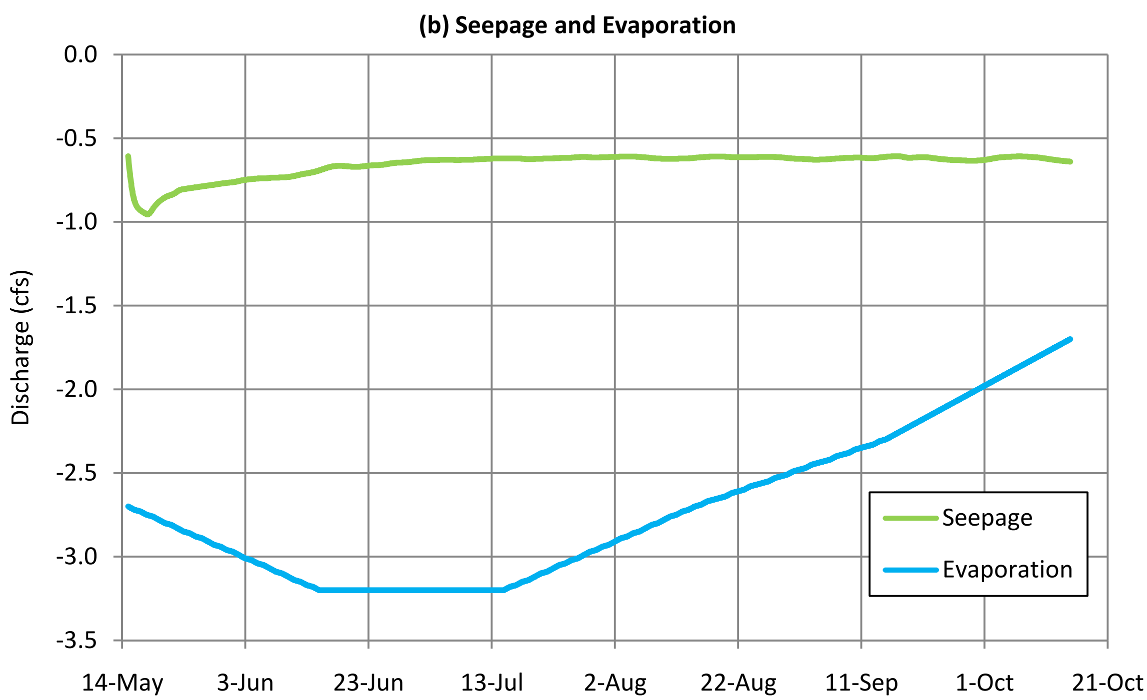




Source: Discharge times series based on observations at USGS gaging station at Hacienda Bridge, Guerneville, CA (#11467000). Evaporation rates calculated from monthly climatological rates for CIMIS evapotranspiration zone 1 (California coast).

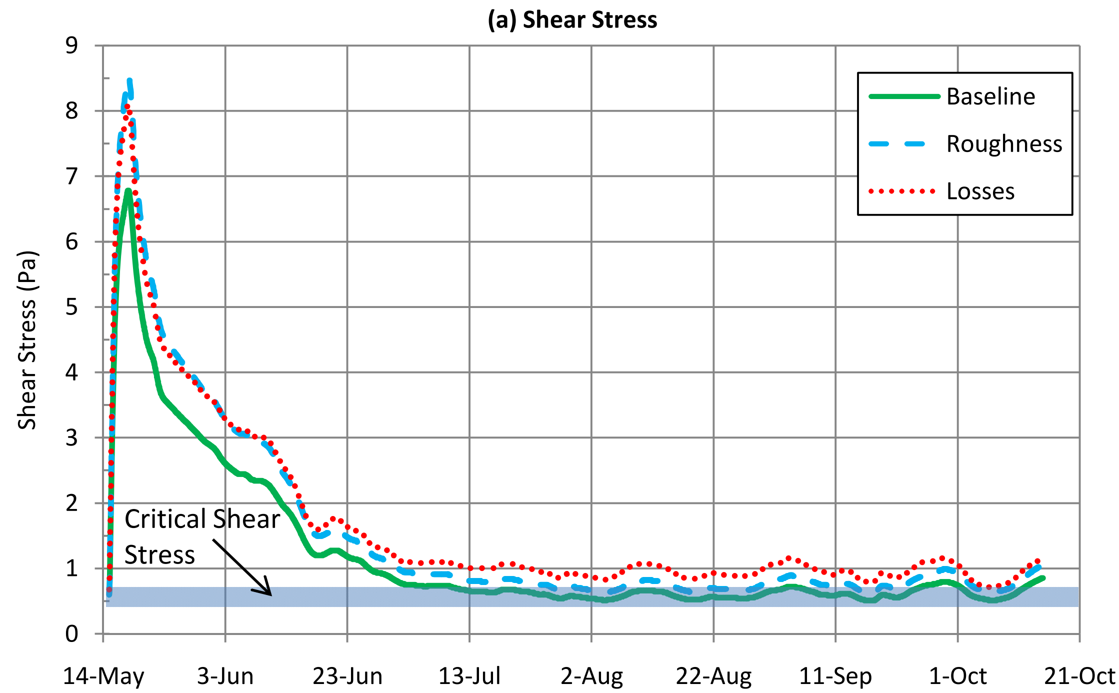
figure **7**

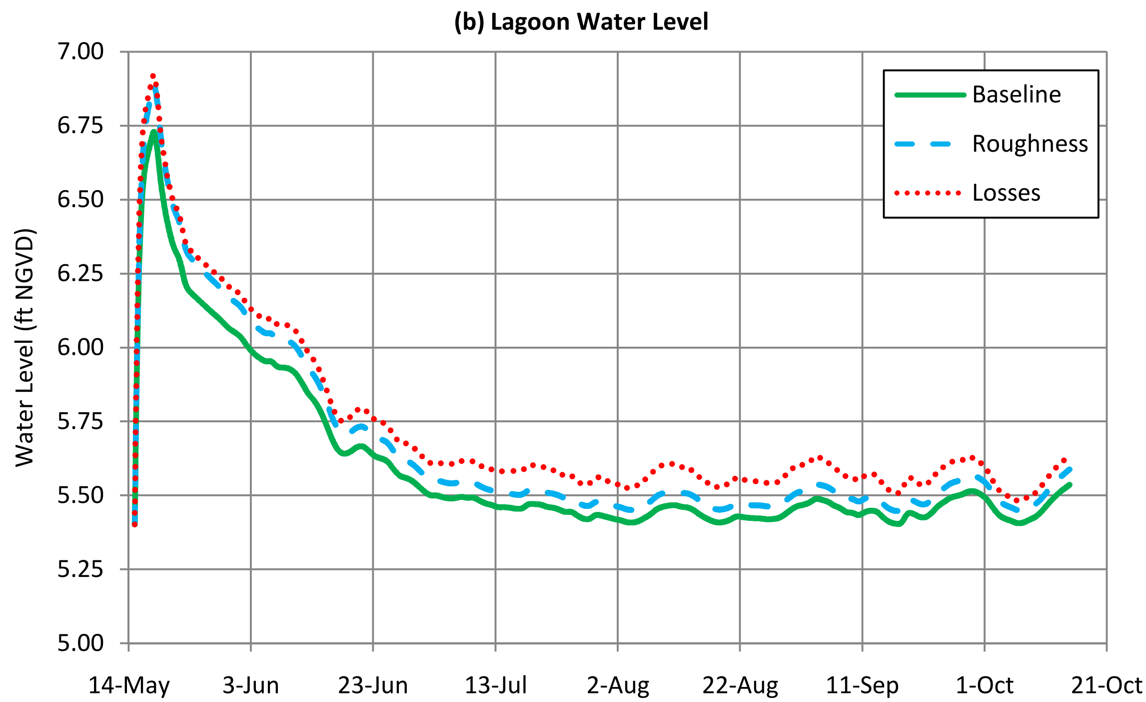
Russian River Estuary Outlet Channel Management Plan

Hydraulic Model Discharge - 2009 Anticipated Hydrology

PWA Ref#: 1958.01

PWA





Notes: Baseline channel geometry: width=100 ft, length=300 ft, bed=5 ft NGVD, n = 0.02. Sensitivity results shown for roughness scneario (n=0.025) and losses scenario (20% uniform reduction in discharge from Guerneville to lagoon).
Source: HEC-RAS hydraulic model results for outlet channel.

figure **8**

Russian River Estuary Outlet Channel Management Plan

Hydraulic Model Results - 2009 Anticipated Hydrology

PWA Ref#: 1958.01

PWA

ATTACHMENT A: SUPPORTING WORKSHEETS FOR CHANNEL CONFIGURATION ANALYSIS

Worksheets

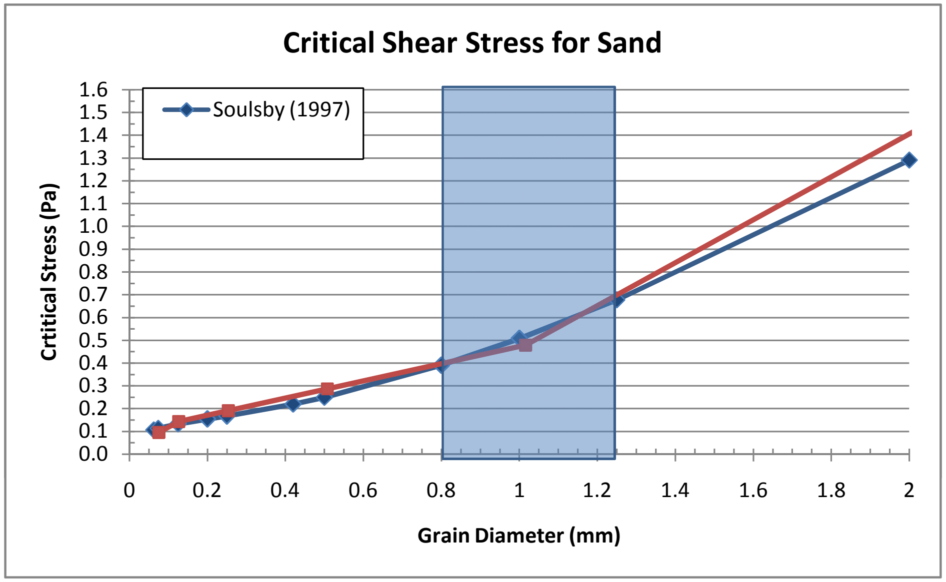
A-1. Critical shear stress for incipient motion of sane particles
A-2. Manning's n
A-3. Evaporation
A-4. Berm seepage
A-5. Mouth closure
A-6. Russian River discharge

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

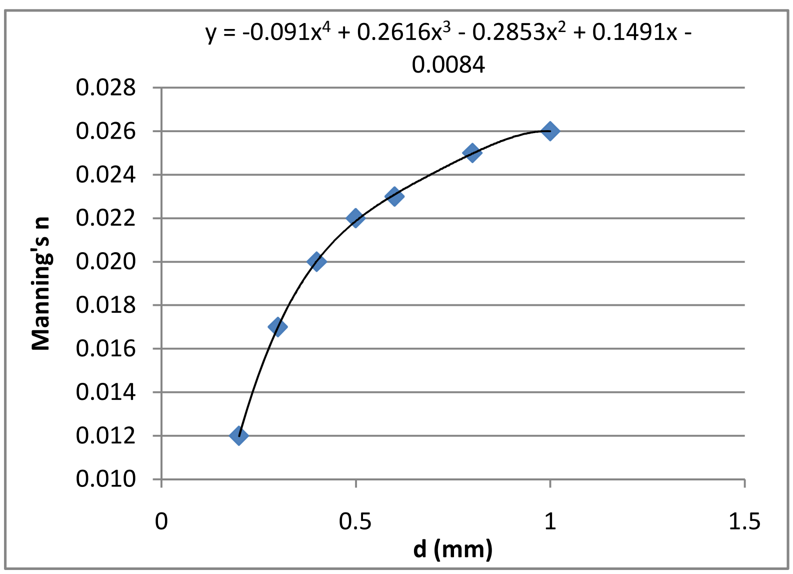
d_{50}	1	mm	0.003281	ft
D	0.84	ft		
Rh	0.83	ft		
S	0.00008	ft/ft		

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

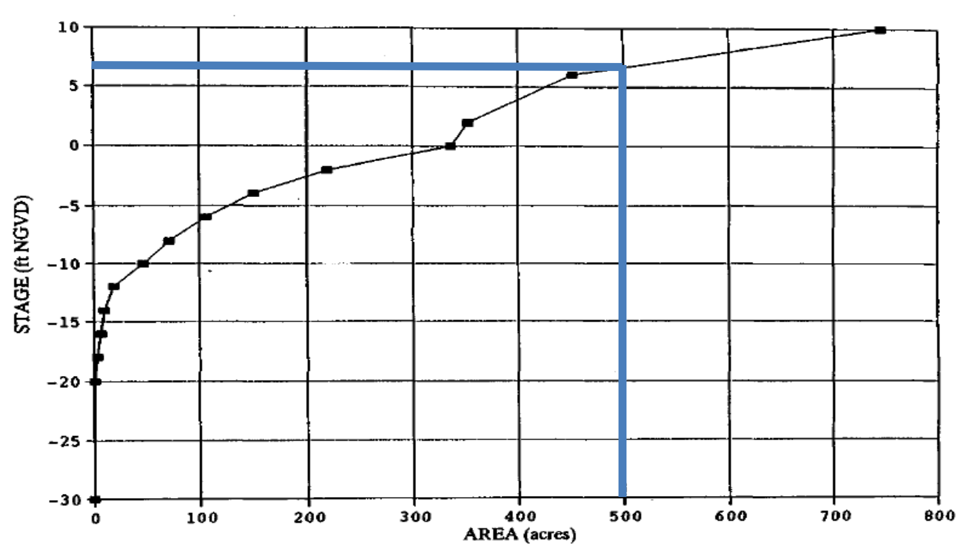
1958.01 Russian River Estuary Outlet Canal
J. Vandever (PWA)
15-Apr-09

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

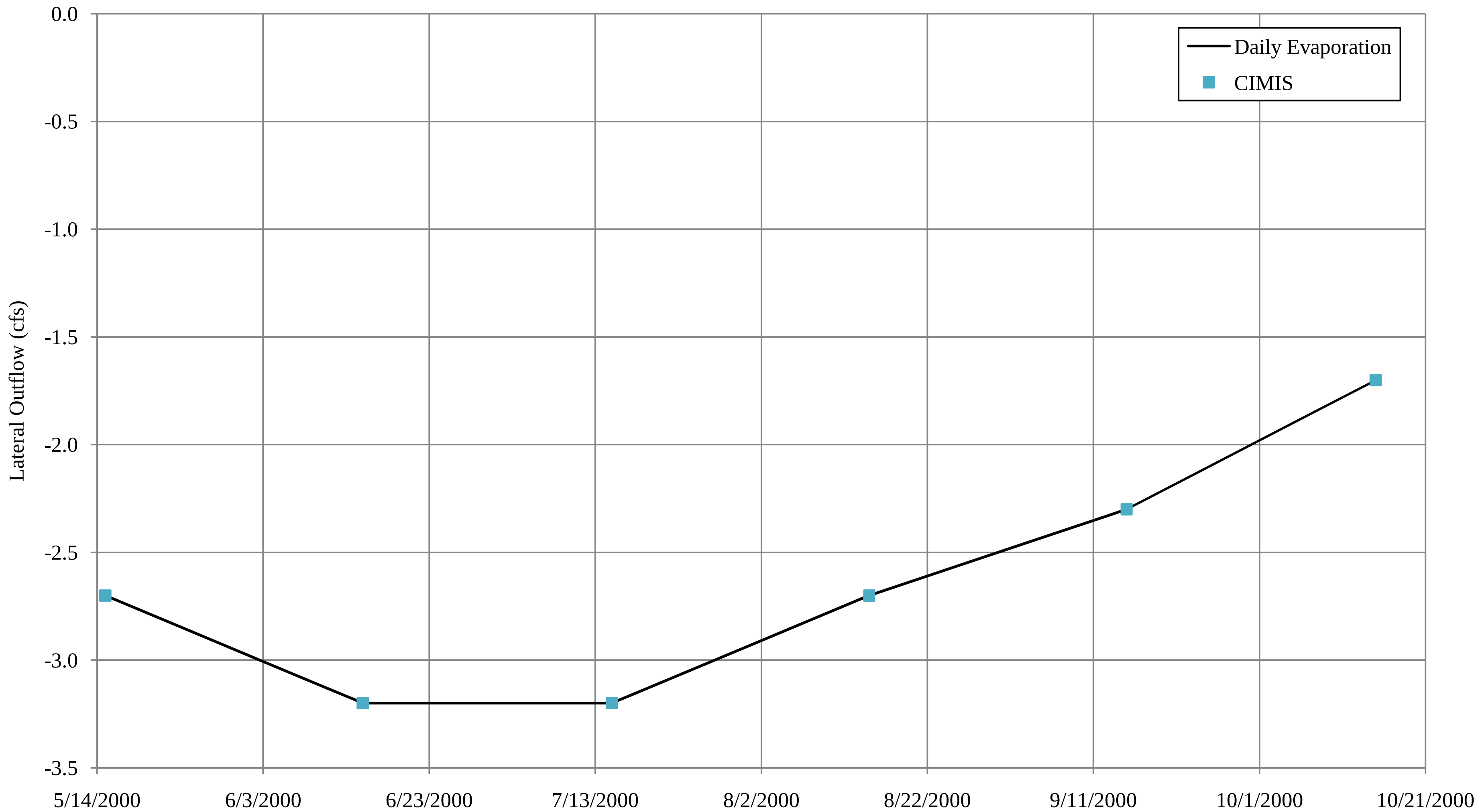


CKC/RUSSNRVR/RRSTOR.WQ1/AREA/2-25-93

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

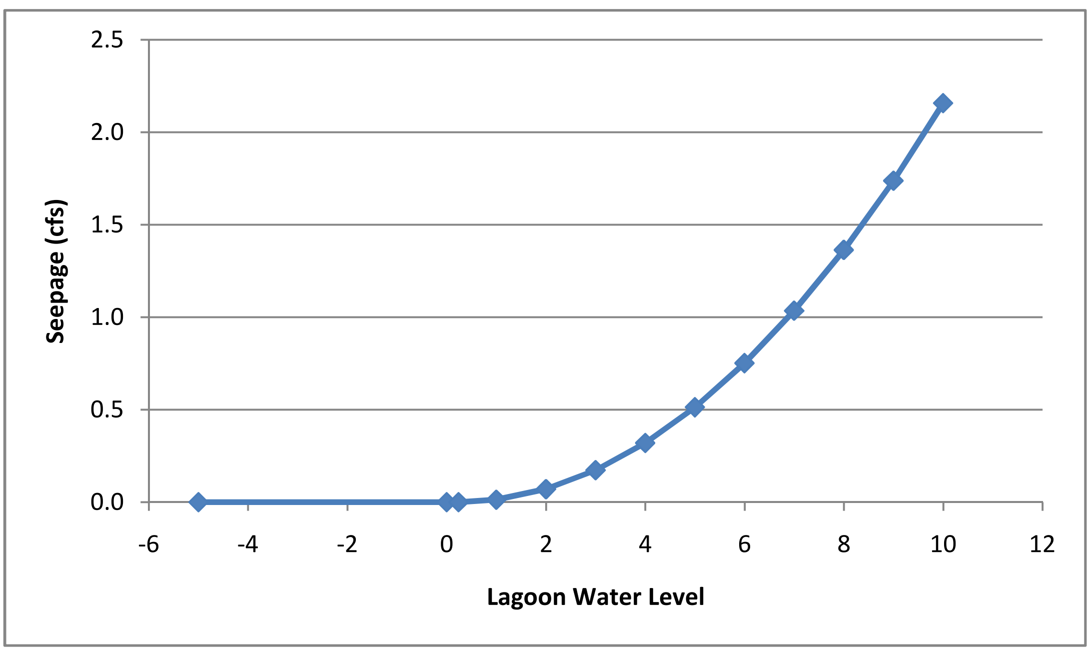
HEC-RAS Diversion Rating Curve

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

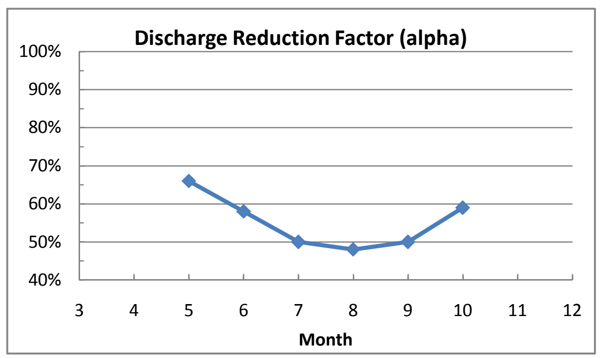
Accounts for losses between Hacienda Bridge (Guerneville, CA) and the lagoon and the interaction with the aquifer adjacent to the estuary.
No detailed information available for the aquifer groundwater elevations or extraction rates by wells. The loss term is a calibrated variable in the model.

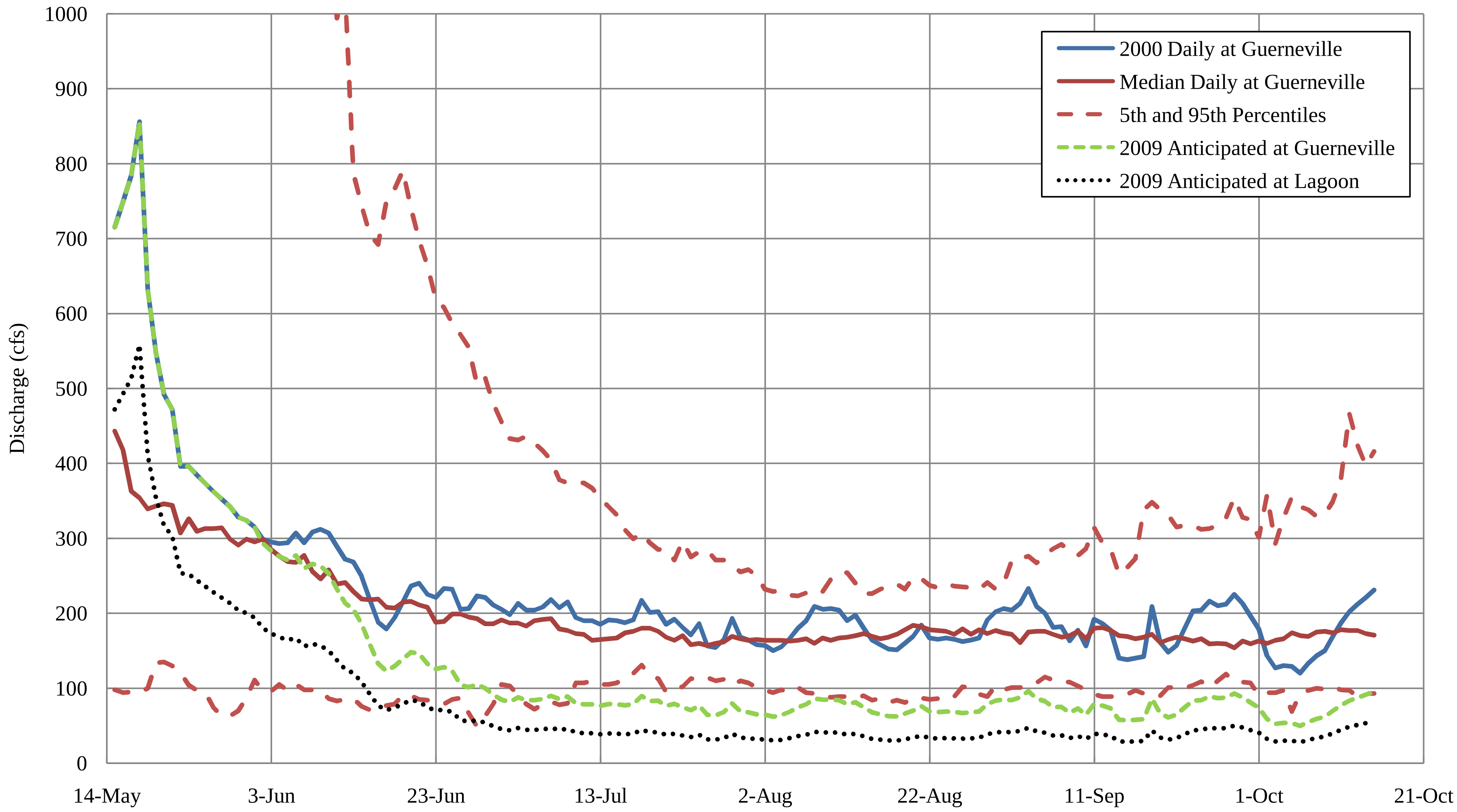
Lagoon Surface Area	400	ac
	17,424,000	sq ft
Evaporation and Seepage Losses	4	cfs

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). 2009 anticipated discharge at Guerneville calculated from 2000 discharge by scaling factor to obtain typical summertime flowrates of 70 cfs. 2009 anticipated lagoon inflow calculated based on calibrated seasonal losses from Guerneville to lagoon.

Attchmnt A-6

Russian River Estuary Outlet Channel Management Plan

Daily Russian River Discharge

PWA Ref #: 1958.01

PWA

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