Arundell Barranca Channel Modifications Ventura Harbor to Harbor Boulevard Alternatives Evaluation

FINAL REPORT

Ventura County Watershed Protection District

April 2015

northwest hydraulic consultants

water resource specialists

Arundell Barranca Channel Modifications
Ventura Harbor to Harbor Boulevard
Alternatives Evaluation

Submitted to:

Ventura County Watershed Protection District
800 South Victoria Avenue
Ventura, CA 93009
Attention: Mr. Kirk Norman

Prepared by:

Northwest Hydraulic Consultants Inc.
80 South Lake Avenue, Suite 800
Pasadena, CA 91101

Contact: Ed Wallace, P.E.
Phone: (626) 440-0080
ewallace@nhcweb.com

PROFESSIONAL ENGINEER - STATE OF NEVADA
EDWARD E. WALLACE
EXP: 6/30/16
CIVIL
No. 7694

Project No: 600001

Submitted on:

30 April 2015

nhc
northwest hydraulic consultants

water resource specialists

Report Prepared by:

Ed Wallace, P.E., Project Manager
Hank Fehlman, P.E. Senior Project Engineer
Nami Tanaka, P.E. Project Engineer
Brian Wardman, P.E. Project Engineer
Andrey Shvidchenko, Senior Engineer
Dawn Lasprugato, GIS Specialist

DISCLAIMER

This document has been prepared by Northwest Hydraulic Consultants in accordance with generally accepted engineering practices and is intended for the exclusive use and benefit of the Ventura County Watershed Protection District and their authorized representatives for specific application to the Arundell Barranca Channel Modifications project. The contents of this document are not to be relied upon or used, in whole or in part, by or for the benefit of others without specific written authorization from Northwest Hydraulic Consultants. No other warranty, expressed or implied, is made.

Northwest Hydraulic Consultants and its officers, directors, employees, and agents assume no responsibility for the reliance upon this document or any of its contents by any parties other than the client for whom the document was prepared.

Table of Contents

List of Figures

List of Tables

List of Appendices

1. EXECUTIVE SUMMARY

Purpose of Study

Arundell Barranca is a major watercourse in the City of Ventura that drains a watershed of approximately 7,400 acres. The channel begins at the Ventura Foothills and flows into a debris and detention basin formed by the Arundell Dam. Downstream of the dam, the channel flows through Ventura and discharges into Ventura Harbor.

As part of the Arundell Barranca-Ventura Harbor to Mills Road Drain Project, the Ventura County Watershed Protection District (District) is planning to increase the capacity of the Arundell Barranca Channel between its outlet at Ventura Harbor and Harbor Boulevard (Phase 1), and between the Union Pacific Railroad (UPRR) Bridge and Mills Road Drain (Phase 2). The project would provide adequate channel capacity to convey the estimated 100-year discharge, thus reducing flood hazards in the residential, commercial and industrial areas near the channel. The District retained Northwest Hydraulic Consultants, Inc. (NHC) to assess the existing conditions in the channel and at its outlet in Ventura Harbor; to develop and evaluate alternatives with respect to project feasibility, potential environmental impacts, costs, and maintenance; to select most promising alternatives; and to conduct detailed analysis of the selected alternatives. This study focuses on the Phase 1 project area.

Existing Conditions

The existing conditions assessment provides a baseline for comparing different alternatives to improve the capacity of the channel. It also provides insight into development of alternatives that could meet multiple objectives in addition to improved flood capacity.

Watershed and Drainage System

The upper watershed of Arundell Barranca primarily consists of steep canyons on the south-facing slopes of the Ventura Foothills. A topographic break occurs at the base of the foothills at an elevation of about 400 feet, and roughly corresponds to the location of Foothill Road. The area downstream of Foothill Road is a broad alluvial fan that is largely developed. Approximately 45 percent of the watershed is urbanized, primarily at elevations between 60 and 400 feet on the alluvial fan and in the area surrounding the Ventura Harbor.

Arundell Barranca begins in the steep canyons of the Ventura Foothills and flows into a debris and detention basin formed by Arundell Dam, constructed by the District in 1995 to replace a smaller structure. The dam has a tributary area of approximately 2.7 square miles (1,754 acres), or about 25 percent of the watershed. The channel system downstream of the dam is a combination of earth, revetted, and concrete channels, and reinforced concrete box culverts. Much of the Arundell Barranca system is designed to operate with high velocities (supercritical regime) during flood flows. The tributary channels are comprised of a combination of channel types and closed conduits.

The Arundell Barranca channel terminates in an arm of the Ventura Harbor (Harbor) referred to as the Stub Channel. The Harbor is operated by the Ventura Port District. The outlet of Arundell Barranca in Ventura Harbor is a tidal channel subject to diurnal variations in water level.

Maintenance

The Arundell Barranca channel system is maintained by the District from the Arundell Dam to the Harbor. Maintenance activities include annual inspections of the channel structures, flushing of weep holes and subdrains, removal of trash and debris, removal of sediment, grading and weed removal on the service roads and cross ditches, and maintenance of fencing. The District reports that significant debris and sediment removal in the concrete portions of the channel is primarily related to large storm events, and that the channel is largely self-cleaning during normal operations.

At the outlet of Arundell Barranca to the harbor, sediment has been removed from the energy dissipator in the past using a crane and clamshell bucket, but removal of sediments in this location is apparently very rare. The District also removes sediment from Arundell Barranca Dam following major events.

Maintenance of the harbor is the responsibility of the Ventura Port District. The Port District boundary in the Stub Channel is located approximately at the northerly side of the Arundell Barranca outlet channel. Northeast of this boundary, the City of Ventura is responsible for maintenance in the Ventura Keys. The Port District conducts regular reconnaissance and informal soundings of the navigation channels in the harbor to determine dredging needs. When shoaling or sediment accumulation becomes significant, the Port District conducts pre-dredging bathymetric surveys and provides pre-dredging reports to the US Army Corps of Engineers, Coastal Commission, State Lands Commission, and Los Angeles Regional Water Quality Control Board (LARWQCB) under permit requirements. Dredging is conducted on an as-needed basis, and large dredging volumes in the vicinity of Arundell Barranca often correspond to large storm events. Dredging of the Stub Channel is coordinated to the extent feasible with City of Ventura dredging in Ventura Keys.

Dredging is done most regularly at the confluence of Arundell Barranca and the Stub Channel. Dredging in Ventura Keys and in other areas of the harbor is less frequent. Dredging in the marinas and berthing areas is very infrequent, and the responsibility of the marina operators. The Ventura Port District reports that the volume of large sediment (large gravel and cobbles) delivered to the harbor constitutes a low fraction of the total, but that this material is particularly problematic in hydraulic dredging operations.

The Corps of Engineers conducts dredging of the channel entrance including dredging of a depressed sand trap area, and advance maintenance (dredging below the navigation depth) in some other areas of the entrance channel. Material dredged from the harbor entrance is coarse-grained and is disposed of on the beaches to the south of the harbor entrance.

The Port District also conducts occasional repairs of the rock slope protection on the harbor side slopes. Repairs to rock slope protection on the Arundell Barranca outlet and Stub Channel were made at some time in the past, but no work has been required in this area for over a decade.

Channel Capacity

A HEC-RAS (US Army Corps of Engineers software) model of the Arundell Barranca was used to assess the capacity of the existing channel. The channel between Harbor Boulevard and the UPRR Bridge was improved by the District in 2006 and has sufficient capacity to convey the 100-year design flow of 7,498 cfs with the water surface approximately one foot below the top of the channel wall. The channel conveys the 50-year design flow with approximately two feet of freeboard, which meets District design standards. The channel between Ventura Harbor and Harbor Boulevard has a capacity of approximately 6,200 cfs with the water surface at the top of the channel wall. With one foot of freeboard, the capacity is estimated at approximately 5,400 cfs. Computed velocities in the channel between Ventura Harbor and Harbor Boulevard vary from 28 feet per second (ft/s) for the 100-year flow to 19 ft/s for the 2-year flow. Computed velocities in the channel between Harbor Boulevard and the UPRR Bridge are slightly higher due to its steeper slope; about 30 ft/s for the 100-year flow and 21 ft/s for the 2-year flow.

The Beachmont Street Bridge has a cross section consistent with the upstream channel section and the capacity of the bridge is similar to the channel. The Harbor Boulevard Bridge and the channel just upstream has variable geometry, a mid-span pier, compound channel geometry, and columns in flow on the upper slope. A rough estimate of existing capacity is approximately 5,000 cfs. The UPRR Bridge is estimated to have a capacity of approximately 5,000 cfs, assuming that supercritical flow is maintained in the channel upstream.

Outlet and Harbor Hydraulics

Hydraulic conditions in the harbor were simulated using the two-dimensional depth-averaged version of the hydrodynamic Adaptive Hydrology/Hydraulics or ADH (US Army Corps of Engineers software) model system.

A high flow condition was simulated, with a 6,000 cfs discharge from the Arundell Barranca channel occurring at MLLW tide elevation (6,000 cfs is the approximate peak of the 1998 event and near the estimated capacity of the existing channel). This combination of inflow rate and tide level is expected to produce the highest velocities within the harbor. Computed velocities range from 20 ft/s in the Arundell Barranca outlet channel, and between 1.5 ft/s and 9 ft/s in the Stub Channel, with the highest velocities occurring near the Arundell Barranca exit and the northwest bank of the Stub Channel. Velocities in the Pierpont Basin downstream of the Stub Channel in the Harbor range from about 0.5 ft/s to 6 ft/s, and velocities in the harbor entrance are as high as 3 ft/s. Velocities in the Ventura Keys area do not seem to be affected by the inflow, but a pronounced eddy occurs upstream of the Arundell Barranca outlet in the Connecting Channel.

A similar simulation was made assuming MHHW tide elevations. In both simulations hydraulic conditions in the Arundell Barranca concrete channel upstream of Beachmont Street are not affected by conditions in the harbor, and hydraulic conditions in the Arundell Barranca outlet channel downstream of Beachmont Street are controlled by flow over the rock sill and sheet pile wall at the downstream end of the energy dissipator. The simulations show that the effects of varying tidal conditions on maximum velocities in the Stub Channel are relatively modest. At lower tides, the maximum velocities are more widely distributed in the Stub Channel and along the northwest bank.

Key observations from the ADH simulations include:

- very high velocities (up to 20 ft/s) and water surface elevations of approximately 12 feet NAVD88 occur in the Arundell Barranca outlet channel for discharges near the existing upstream channel capacity;
- the water surfaces in the concrete channel upstream of Beachmont Bridge are independent of tidal conditions;
- the energy dissipator forces a hydraulic jump, but a second supercritical to subcritical transition occurs for some flow conditions over the rock sill near the confluence with the Stub Channel;
- velocities on the order of 6 ft/s occur in the Stub Channel, with a high velocity flow stream crossing the channel and impinging on the northwest bank;
- an eddy occurs at the confluence with the Stub Channel and upstream water surfaces in Ventura Keys are increased – the magnitude of increase decreases with tidal elevation, and at MHHW the increase is small and water surface elevations are well below those experienced in normal high tides; and
- in general, for the same flood discharge, higher velocities occur at lower tidal conditions and higher water surfaces occur at higher tidal conditions, but differences in maximum velocity in the Stub Channel and maximum water surface elevation in the Arundell Barranca outlet channel are relatively small.

Harbor Sedimentation Simulation

The sediment transport capabilities of ADH were utilized to simulate deposition in the harbor from the peak of the 1998 high flow event. The simulation results show significant deposition located primarily in the Stub Channel near the outlet of the Arundell Barranca. The deposits are composed primarily of sand and gravel. Simulated deposition depths in the Stub Channel are as high as 6 feet. Fine sediment loads remain in suspension throughout the duration of this simulation, though some minor deposition of fines was noted in the Pierpont Basin and the Connector Channel. Fine materials settle over much longer time periods than the coarse materials (sands, gravels and cobbles) in the sediment load, and are subject to re-entrainment and transport with subsequent tidal action.

Water Quality

Water quality in the project area is regulated under the Los Angeles Basin Plan, California Ocean Plan, Water Quality Control Policy for Enclosed Bays and Estuaries, California Toxics Rule, and Clean Water Act Section 303(d) TMDL listings for the Santa Clara River Estuary and Ventura Harbor. The LARWQCB is responsible for carrying out the state and federal clean water acts through water quality control plans, regulations, and enforcement in the area. The US Environmental Protection Agency provides oversight for execution of the federal act and is directly engaged in some programs and policies for water quality control.

Section 303(d) of the Clean Water Act requires states to develop lists of waters with impaired water quality and to develop and implement Total Maximum Daily Loads (TMDLs), an estimate of the amount of specific pollutant(s) that water body can receive while meeting water quality objectives. Section 303(d) listings are in place for bacteria in the Ventura Harbor: Ventura Keys and for DDT/PCBs in tissue on Ventura Harbor Jetties. Section 303(d) listings on the Santa Clara River include ChemA, coliform bacteria, nitrate-nitrogen, toxaphene, and toxicity. A TMDL is in place for bacteria in the Santa Clara

River, including the estuary, and a LARWQCB Order (R4-2010-0816) is in place to implement a TMDL for toxaphene in fish tissue in the Santa Clara River Estuary. The Order includes requirements for water, sediment and fish tissue monitoring for toxaphene, chlordane, and dieldrin in the Santa Clara River estuary and its subwatershed.

Limited data are available on water quality in Arundell Barranca channel and the Harbor. In a 1999 report, Cotton, Shires, and Associates present a summary of data collected on Arundell Barranca in 1998 and 1999 on five dates during dry weather and two during storm events. The summary indicates high concentrations of nitrates, ammonia, and total dissolved solids, consistent with runoff and return seepage from agricultural lands. The data also indicated high total coliform bacteria concentrations. Organic compounds, including US EPA priority pollutants, were also sampled. No organic constituents were found in detectable concentrations during dry weather, but several were measured in the storm samples. Several of the organic compounds were pesticides/herbicides, also consistent with the agricultural land use.

The City of Ventura collected data on bacteria from 2002 to 2009 in Ventura Keys, Ventura Harbor, and Arundell Barranca. In a letter dated 6 July 2010 the City transmitted the City's data and data collected by Ventura County Department of Environmental Health to the LARWQCB and requested that the listing for bacteria impairment be removed. Over 5000 individual samples are included in this data set, and are under review by the LARWQCB.

The District conducted monthly water quality sampling from June to October 2011 (5 sampling times) at locations on Arundell Barranca just downstream of the UPRR Bridge and just upstream of the Harbor Boulevard Bridge. Several water quality objectives were exceeded in one or more samples at both sampling stations. These included bacteria, TDS, chloride, sulfate, total and nitrate nitrogen, dissolved and total copper, total nickel, total selenium, and total zinc.

Comparison of the samples at the two sampling stations suggests that dilution occurred for some constituents between the two stations due to agricultural return flows. Higher concentrations of nitrate and total nitrogen at the Harbor Boulevard site indicate that the agricultural land is a probable source of this constituent.

The 2011 data is reasonably consistent with the data summary presented in the Cotton, Shires and Associates report. With respect to Section 303(d) listings, Arundell Barranca may be a potential source of bacteria to the Ventura Harbor. Concentrations of bacteria and nitrate and total nitrogen exceeded water quality objectives, and would be a concern for discharge to the Santa Clara River.

The District conducted additional monitoring in June to October 2012, collecting grab samples at the same locations sampled in 2011. Water quality objectives were frequently exceeded at both sampling stations for indicator bacteria, total selenium, total copper, nitrate and total nitrogen, total dissolved solids, chloride, and sulfate. In addition, frequent exceedances of pH standards were observed at Harbor Boulevard. Exceedances for dissolved copper, total nickel and total zinc were observed in 2011 but not in 2012. Total copper, nitrate, and total dissolved solids concentrations were lower than in the 2011 monitoring. Comparison of data from the two sampling sites again showed that some constituents are probably diluted by seepage or return flows from agricultural lands, but that nitrate concentrations increase, indicating that agricultural lands are a potential source of this constituent. However, nitrate concentrations measured at the upstream debris and detention basin (up to 24 mg/l), were similar to or higher than the two channel sampling sites, indicating potentially high background levels from the upper watershed. Similarly, concentrations of total copper and total dissolved solids at the detention basin indicate that they are similar to those in the channel in the project area.

The District's water quality report compared measured concentrations to effluent standards for the Ventura Water Reclamation Facility and identified several constituents that exceed effluent limitations (nitrate, bacteria, total copper), but these were not compared to typical influent sewage concentrations, which may greatly exceed the measured values.

Alternatives Considered

Based on comments received on the initial study and during the scoping meeting for the project, the District developed nine alternatives to assess their feasibility and effectiveness in addressing stakeholder comments. The alternatives were developed in sufficient detail to assess their hydraulic design, feasibility, cost, and potential environmental effects. All alternatives were developed to provide protection against a 100-year flood event. Based on the general layout, dimensions, and type of the hydraulic facilities for each alternative, feasibility considerations such as the potential for conflict with major utilities, necessary changes to other infrastructure, and required land acquisition were assessed. Conceptual level construction cost estimates were prepared for each alternative based on unit costs developed from previous District projects, experience on other projects, and cost estimating guides. The alternatives generally are conceived to provide secondary benefits such as improvements in water quality, reduced sediment delivery to the Harbor, and opportunities for riparian corridors or wetlands. Several alternatives also may provide increased opportunity for recreational or transportation facilities such as bike trails. Benefits and disadvantages were identified qualitatively for each alternative and, where feasible, initial quantitative estimates of potential benefits were made.

Alternatives 1 through 4 are stand-alone alternatives, and Alternatives 5 through 9 are a combination of Alternative 1 with additional features to provide secondary benefits to water quality, sedimentation, or Harbor management at the Arundell Barranca outlet. The nine alternatives developed by the District are briefly described below:

Alternative 1 – Enlarged Arundell Barranca Channel from Ventura Harbor to Harbor Boulevard
Concept: Enlarged channel and bridges from Ventura Harbor to Harbor Boulevard using existing channel alignment. The existing bridges at Harbor Boulevard and Beachmont Street would be replaced, and a new energy dissipator would be constructed at the mouth of the channel in the harbor.

Alternative 2 – Complete Diversion to Santa Clara River – The Nature Conservancy (TNC) Property Alignment
Concept: The entire Arundell Barranca flow would be routed to the Santa Clara River approximately 5,000 feet upstream of the Harbor Boulevard Bridge by intercepting the existing channel near the UPRR Bridge and realigning the high speed channel, constructing a coarse sediment trap, passing through agricultural land, crossing Olivas Park Drive, and passing through TNC property east of the golf course to the Santa Clara River. As part of this alternative, a wetland treatment system would be constructed to treat most of the summer flows to reduce potential pollutant delivery to the Santa Clara River during the dry season.

Alternative 3 – Existing Channel with High Flow Diversion to Off-Channel Retention Basin
Concept: The existing channel from Ventura Harbor to Harbor Boulevard would be retained and a high flow diversion would be constructed for flows in excess of the existing channel capacity. The high flows would be routed to a retention basin sized to provide adequate storage for the 100-year design hydrograph. Stored water would eventually be released or infiltrated.
Alternative 4 – Existing Channel with High Flow Diversion
Concept:
Alternative 4a – Diversion Downstream of UPRR**:** The existing channel from Ventura Harbor to Harbor Boulevard would be retained and a high flow diversion would be constructed for flows in excess of the existing channel capacity. The overflow weir would be constructed near the UPRR Bridge to capture excess flows and the channel would be routed to Harbor Boulevard near Olivas Park Drive. An inlet structure would be constructed on the east side of Harbor Boulevard for flow to enter a set of reinforced concrete box culverts (RCBs). Because of a conflict with a sewer trunk line on the west side of Harbor Boulevard, the RCBs are sized as 6 – 4x10 culverts with their tops at grade in Navigator Drive. An energy dissipator would be constructed at the outlet to the Harbor.
Alternative 4b– Diversion from Upstream of Harbor: The lateral weir would be constructed nearer to Harbor Boulevard than in Alternative 4a and the channel would be routed along Harbor Boulevard to a point near Olivas Park Drive and then to the Harbor. An inlet structure would be constructed on the east side of Harbor Boulevard for flow to enter a set of reinforced concrete box culverts (RCBs). Because of a conflict with a sewer trunk line on the west side of Harbor Boulevard, the RCBs are sized as 6 – 4x10 culverts with their tops at grade in Navigator Drive. An energy dissipator would be constructed at the outlet to the Harbor.

Alternative 5 – Alternative 1 with Low Flow Treatment Wetlands
Concept: The existing channel would be enlarged and bridges modified or replaced from Ventura Harbor to Harbor Boulevard as for Alternative 1, plus a low flow treatment wetland or bio-retention area would be constructed along the channel alignment east of Harbor Boulevard. The treatment facility would be sized to treat low flows, and would primarily intercept urban flows during the summer, the leading edge of runoff events, and a small portion of larger runoff events. The treated flows would be returned to the Arundell Barranca channel upstream of Harbor Boulevard.

Alternative 6 – Alternative 1 with Inline Sediment Trap
Concept: The existing channel and bridges downstream of Harbor Boulevard would be enlarged as for Alternative 1, and a coarse sediment trap would be constructed upstream of Harbor Boulevard to minimize delivery of gravel and cobble bed materials to the Harbor. The sediment trap would require an energy dissipator at the upstream end to transition from supercritical to subcritical flow, and a transition for acceleration back to supercritical flow at the downstream end.

Alternative 7 – Alternative 1 with Extension of Arundell Barranca Channel to Pierpont Basin
Concept: The existing channel and bridges downstream of Harbor Boulevard would be enlarged as for Alternative 1, except that the channel would be extended further into the Harbor.

Alternative 8 – Alternative 1 with Modification of Arundell Barranca Outlet Channel and Stub Channel Confluence
Concept: The existing outlet channel in the Harbor would be modified to increase efficiency in trapping coarse sediments and improve maintenance access for removal of material. A deflector would be

installed at the confluence of the Arundell Barranca and Stub Channels to turn the flows more parallel to the Stub Channel.

Alternative 9 – Alternative 1 with Diversion of Low Flows to Ventura Water Reclamation Facility
Concept: A low flow diversion would be constructed in the existing channel upstream of Harbor Boulevard and up to 5 cfs would be diverted by pipe into the Harbor Trunk sewer line for delivery to the Ventura Water Reclamation Facility. The existing channel and bridges downstream of Harbor Boulevard would be enlarged as for Alternative 1.

The costs and benefits of these alternatives are summarized in Table ES-1.

Table ES-1. Costs and benefits of nine alternatives

Alternative	Cost, $M	Incremental Effectiveness[1]	Feasibility Considerations	Environmental Considerations
1. Expanded Channel	$10.5	Base alternative - provides 100-year flood protection	No additional land required, no major utility conflicts	Reduces pollutants generated by flood overflows onto agricultural lands
2. Diversion to Santa Clara River	$50.0 (does not include compensation or replacement for land at treatment wetland site)	Benefits Harbor water quality and dredging by diverting all flow and sediment to Santa Clara River with low flow treatment of water quality Requires high level of channel and treatment wetland maintenance	Requires acquisition of 68 acres of private land (no willing seller) and use of TNC site for treatment wetland conflicts with grant conditions and TNC intentions for property; other routes more expensive or infeasible Permitting difficult or infeasible under water quality and endangered species regulations	Potential effects on endangered steelhead, tidewater goby, least terns Delivers pollutants during flows larger than 50 cfs to sensitive Santa Clara River estuary Loss of productive coastal farmland
3. High Flow Retention Basin	$19.1	Reduces sediment delivery to Harbor in large events, but would require removal of sediment from retention basins and continued dredging Requires occasional sediment removal from retention basins.	Requires acquisition of 41 acres of private land (no willing seller)	Potential for joint (recreation) use of retention basin Reduces pollutant delivery to Harbor for very large events Loss of productive coastal farmland
4. High Flow Diversion to Harbor	$16.4	Reduces sediment delivery and pollutant delivery at existing outlet but delivers at least a portion of this material to an alternate location in the Harbor	Requires acquisition of 17 acres of private land (no willing seller) and easement at Holiday Inn property May conflict with hotel expansion plans Conduit constructions and high pressure oil and sewer utility crossings in Harbor Boulevard difficult	Potential for joint (bike and pedestrian transportation) use of channel alignment New dredging location in Harbor (formalizes 1998 dredge location) Loss of productive coastal farmland
5. Alt 1 with Low Flow Treatment Wetlands	$17.7	Reduces pollutant delivery to Harbor during low flows Significantly increases maintenance for operation of treatment wetlands	Requires acquisition of 10 acres of private land (no willing seller)	Reduced delivery of pollutants to Harbor Loss of productive coastal farmland

Alternative	Cost, $M	Incremental Effectiveness[1]	Feasibility Considerations	Environmental Considerations
6. Alt 1 with In-Line Sediment Trap	$19.7	Reduces delivery of coarse fraction of sediment (cobbles and gravel) to Harbor, facilitating dredging of smaller material Requires sediment removal from in-line sediment trap every 1 to 5 years depending on flows	Requires acquisition of 5 acres of private land (no willing seller)	Slightly reduced delivery of pollutants to Harbor Loss of productive coastal farmland Visual impacts and levee management due to need to elevate embankment above existing ground
7. Alt 1 with Channel Extension in Harbor	$16.6	Potentially reduced deposition at outlet channel confluence – sediments would be distributed to other areas of the Pierpont Basin and Harbor Requires regular cleaning of subtidal channel extension to maintain capacity	Requires acquisition of right-of-way along Harbor parking lot Conflicts with 22" high pressure oil line Harbor crossing May conflict with water and sewer Harbor crossings	Potential effects on benthic habitat at new outlet Increased construction in a marine environment
8. Alt 1 with Modified Outlet Channel	$14.7	Reduced delivery of cobbles and coarse sediment to Stub Channel, facilitating dredging Reduced velocities in outlet channel Increased maintenance for removal of coarse sediment from cobble trap	Conflicts with 22" high pressure oil line Harbor crossing	Increased construction in a marine environment
9. Alt 1 with Low Flow Diversion to VWRF	$33.1	Reduces pollutant delivery to Harbor during low flows Requires increased operations and maintenance at VWRF	Requires small diameter pipeline crossing of Harbor Boulevard	Reduced delivery of pollutants to Harbor

[1] All alternatives provide 100-year flood protection. Incremental effectiveness indicates secondary benefits for maintenance or environmental benefits.

The incremental benefits and disadvantages of the alternatives may qualitatively be compared to Alternative 1 as a baseline.

Alternative 2 addresses many of the comments received in public review by diverting all flow and sediment to the Santa Clara River. Pollutants carried in the flow would also be diverted to the river, although water quality would be improved in a portion of the runoff volume by the treatment wetlands. However, implementation of Alternative 2 is highly uncertain due to the lack of available land for the diversion channel and the treatment wetlands, and regulatory requirements for the Santa Clara River under the TMDL and endangered species regulations. Screening meetings and telephone communications with National Marine Fisheries Service, California Department of Fish and Wildlife, Los Angeles Regional Water Quality Control Board, Coastal Commission, The Nature Conservancy, and City of Ventura, the District identified several factors that each could delay or make infeasible implementation of the alternative. Although it is the highest cost alternative developed, the costs listed in Table 4-2 likely significantly underestimate the probable costs that would be incurred in environmental studies, legal fees, land acquisition costs, and environmental mitigation. This alternative has very high incremental cost, very low or doubtful implementation feasibility, high maintenance requirements, and adverse environmental effects that potentially significantly outweigh the benefits.

Alternative 3 provides some incremental benefits compared to Alternative 1 in reducing sediment and pollutant delivery to the Harbor, but the incremental benefit is low because only very high flows would be diverted. Implementation feasibility is constrained by lack of available land. This alternative has high incremental construction and land cost, low implementation feasibility, and moderate incremental maintenance requirements. Relatively minor positive environmental effects on sediment and pollutants would be countered by loss of productive coastal farmland.

Alternative 4 diverts a portion of the flow (high flows) to another location in the Harbor. Implementation is constrained by lack of available land and potential conflicts with planned commercial land use (i.e., hotel expansion). Overall effects on Harbor sediment and water quality are neutral, but the location of the discharge is distributed, and a small improvement would likely be realized at the existing outlet near the Ventura Keys residential properties. This alternative has moderate incremental construction and land cost, low implementation feasibility, and moderate incremental maintenance requirements. Overall environmental effects in the Harbor are neutral or slightly negative due to addition of a location where dredging would be required (formalizes 1998 dredge location), and the alternative would cause a loss in productive coastal farmland.

Alternative 5 diverts low flows to a treatment wetland and provides an incremental benefit in water quality in the Harbor. Implementation is constrained by lack of available land. During public review, a concept was advanced that combines elements of Alternatives 2 and 5 to create an estuarine section of Arundell Barranca as an environmental benefit. This concept is constrained by the lack of available land at a suitable elevation for an estuarine system. Compared to Alternative 5, the concept also would not provide the benefits of formal treatment of urban runoff prior to discharge to a natural system. Alternative 5 has moderate incremental construction and land cost, moderate implementation feasibility, and moderate incremental maintenance requirements. Wetland treatment and potential open space/recreation benefits would be countered by loss of productive coastal farmland.

Alternative 6 uses an in-line sediment trap to reduce coarse sediment loads to the Harbor, and could potentially have some water quality and sediment delivery benefits in the Harbor. The alternative is difficult to design because of the required transitions between the basin and the upstream and downstream supercritical channel segments. This requires a relatively long structure that would be elevated above the adjacent ground at its downstream end and subject to maintenance requirements typically associated with levees that constrain planting. This alternative has high incremental cost, moderate implementation feasibility, and high maintenance requirements. Relatively minor water quality benefits would be countered by loss of productive coastal farmland.

Alternative 7 would relocate the outlet of Arundell Barranca in the Harbor, reducing water quality and sediment effects at the confluence of the outlet channel and Stub Channel, but potentially transferring these effects to Pierpont Basin, although sediment might be better distributed and easier to dredge in this location. The alternative includes some uncertainty in performance and maintenance requirements due to construction of the outlet channel at subtidal elevations, and implementation is constrained by conflicts with oil, sewer, and water line crossings of the Harbor and the existing Harbor Patrol dock. This alternative has high incremental construction cost, moderate to low implementation feasibility, and moderate to high maintenance requirements. Environmental benefits at the existing outlet would be countered by potential effects on benthic habitat and navigation at the new outlet.

Alternative 8 would reduce coarse sediment loads to the Harbor by trapping cobble and gravel at the energy dissipator for removal from the top of bank by excavator or clamshell. Reduction of the coarse sediment fraction would facilitate dredging of smaller material in the harbor, but would have little effect on total sediment delivery. During public review, this alternative was supported for further evaluation by the Ventura Port District and the City of Ventura. Implementation is constrained by conflict with the existing oil line crossing of the Harbor. This alternative has moderate incremental construction cost, moderate implementation feasibility, and moderate incremental maintenance requirements (for the District). Environmental benefits include reduced dredging difficulty, slightly reduced sediment delivery, and lower outlet channel velocities.

Alternative 9 is similar to Alternative 5, but diverts low flows to the VWRF for treatment. Compared to Alternative 5, construction costs are low and incremental maintenance costs are low for the District. However, the alternative adds incrementally to operations and maintenance at the VWRF, and charges for the treatment service could outweigh the construction costs. VWRF provided a treatment charge for estimating purposes (approximately $1.8M per year at 2 cfs average flow), but actual costs might be negotiated by the District and City at a lower level.

Alternatives Evaluated in Detail

The alternatives carried forward for detailed analysis included two alternatives (Alternative 1 and Alternative 8) for modification of the outlet of the Arundell Barranca channel in Ventura Harbor. Alternative 1 is the channel configuration originally proposed, including improvements to the concrete channel upstream of the Harbor. Alternative 8 is a modification of Alternative 1 that included an enlarged cross section near the outlet to trap very coarse sediment (cobbles). In addition to these alternatives, Alternatives 5 and 9 were carried forward as supplemental improvements to improve water quality. These alternatives are intended to improve existing conditions by diverting and treating low flows. Alternative 5 would treat the low flows in a constructed wetland and Alternative 9 would divert low flows to the Ventura Water Reclamation Facility for treatment.

Results of Detailed Hydraulic Evaluation

The alternatives developed and modeled in the detailed evaluation are focused on potential improvements over the baseline Alternative 1. During the detailed evaluation, Alternative 1 was modified as necessary to achieve adequate flood performance at the Harbor outlet and the estimated costs were adjusted. Compared to existing conditions, Alternative 1 provides 100-year flood protection and prevents overflow of agricultural and urban land during major events that contributes episodic sediment and pollutant loads to the Harbor. Because of the increased channel capacity, Alternative 1 results in slight increases in velocity and delivery of sediment to the outlet channel. These increases occur only during rare events larger than about 6,000 cfs and would occur over a short duration at the peak of the event. The volume difference in the 100-year event for the Alternative 1 channel is estimated at about 51 af, or about 1.7 percent of the runoff volume. Sediment delivery to the outlet during this extreme event is increased by about 6,000 cubic yards, or about 5 percent of the event load. Based on dredging information after the 1998 event, delivery of sediment by flood overflow and overland erosion to other areas of the Harbor under existing conditions is estimated to be as large or larger than this increase, and to carry potentially higher concentrations of sediment and other pollutants than the improved channel discharge.

Velocities in the outlet and Stub Channel are increased by the additional channel capacity, but differences identified in the simulations are relatively subtle. Under MHHW conditions, Alternative 1 reduces velocities at the mouth of the outlet channel and reduces the eddy at the confluence with the Stub Channel, but velocities of the reverse flow along the east bank of the Stub Channel south of the outlet are slightly increased. Under MLLW conditions impingement of flows on the west bank is slightly increased and the strength of the eddy is increased. These changes would occur at the peak of the design 100-year event, which exceeds the present capacity of 6,000 cfs for a duration of about 40 minutes. Given the variability associated with tidal conditions, changes in velocity attributed to Alternative 1 and that would occur in rare events and for short duration do not significantly alter existing conditions in the Stub Channel with respect to navigation or erosion problems, but these problems are also not significantly reduced.

Similarly, changes in sediment deposition in Alternative 1 compared to existing conditions are small. General deposition patterns are similar, and differences in volumes are probably well within the accuracy of the simulations. A sediment delivery volume increase of 6,000 cubic yards occurs in the 100-year event, but is a small volume compared to estimated long term average dredging volume of 28,000 cubic yards, or about 2.8M cubic yards over a 100 year period.

Alternatives 10, 11, 12, and 13 were variations on Alternative 8 that were developed in the course of the detailed evaluation. Alternatives 8, 12, and 13 were the most promising, and were investigated to compare potential improvements to velocity and sediment conditions in the Harbor that could be made at reasonable additional cost compared to the baseline Alternative 1. Alternative 12 used a narrower and deeper channel than Alternative 8 at the outlet to the Harbor with an expansion on the south side approaching the Stub Channel. Alternative 12 reduced the amount of retaining wall required on the north side of the outlet channel compared to Alternative 8. Alternative 13 used a graded slope on the south side of the outlet channel rather than a retaining wall. This reduced retaining wall costs but required modification of the parking area on the south side of the channel.

The alternatives present differences in velocity and sediment distribution, but considering the magnitude of topographic and bathymetric changes involved in the alternatives, these changes are relatively subtle. As in Alternative 1, Alternatives 8, 12, and 13 make improvements in velocity conditions in the MHHW condition compared to existing conditions, and the improvements are significantly increased compared Alternative 1 for some factors such as west bank impingement and eddy strength. In MLLW conditions, improvements are less significant, and generally similar to existing conditions. Similarly, changes in sediment deposition patterns are different for each alternative, but none of the alternatives provides improvements for all of the factors of concern. In overall performance relative to Alternative 1, Alternatives 8 and 12 provide only modest improvement for substantial change in total cost. Alternative 12 is considered highly sensitive to maintenance and difficult to implement due to the deep outlet channel. Alternative 13 provides minor improvements in velocities, but no significant improvement in sediment performance, and requires modification of the existing parking area outside of existing District right-of way.

Low Flow Treatment

Alternatives 5 and 9 would provide water quality benefits during low flows. Both alternatives were sized for a maximum treatment flow of 5 cfs, which would treat approximately 80% of the flow volume in the period May to September and approximately 15 percent of the average annual runoff volume if the diversion to the wetland treatment system or Ventura Water Reclamation Facility (VWRF) were made year-round. Pollutant removals in stormwater wetland treatment systems are highly variable, depending on treatment system design, weather, soils, flow patterns, vegetation types and management, and other factors. For the purposes of this study, potential pollutant removals were calculated based on literature sources that summarize treatment effectiveness data from a large number of sources. The resulting calculations give fairly wide ranges of pollutant removals for Alternative 5, but indicate that substantial reductions in nitrogen, metals, and bacteria loads could potentially be achieved. Alternative 9 assumes that the entire load in the diverted volume would be removed from the Harbor because it would be diverted to the VWRF. Potential load reductions associated with summer diversion and year-round diversion were both calculated because VWRF may only be able to accept flows during dry weather conditions. Both alternatives provide load reductions that are most significant in terms of total load for the summer period. Load reduction estimates for priority pollutants vary from about 15 to 80 percent of the summer load.

Costs

Table ES-2 summarizes the estimated costs for the alternatives evaluated. Capital costs were estimated using 2013 cost levels as the basis. The construction cost estimate for Alternative 1 was originally developed by the District based on preliminary engineering design. After completing simulations in this study, the harbor outlet configuration was modified to provide adequate flood capacity and the energy dissipator was enlarged. These changes were incorporated into the Alternative 1 cost estimate. Costs for Alternatives 8, 12, and 13 were estimated as additions to the Alternative 1 costs based on the modified configuration at the harbor outlet. Costs are considered suitable for comparison of alternatives, but are approximate due to the conceptual level of layout for the alternatives. Geotechnical, structural, and marine construction design development outside the scope of this study is needed to refine the costs associated with the harbor outlet. Alternatives 8,12, and 13 will likely require relocation of the high pressure oil line at the Harbor crossing. Based on the agreement for installation and operation of the oil line with the Port District, relocation is the responsibility of the pipeline owner, and relocation costs are not included in the alternative costs. A twenty percent contingency is included in the estimates.

Costs for Alternative 5 were based on sizing for the wetlands to provide a 24-hour retention time at the maximum design flow and for Alternative 9 to divert flows from the existing concrete channel to an existing trunk sewer near the Arundell Barranca crossing of harbor Boulevard. Preliminary costs for Alternative 5 were increased by 25 percent to account for uncertainty in sizing to achieve pollutant reduction objectives.

Maintenance costs for each alternative were also estimated and average annual costs were converted to a present value using a 30-year maintenance period (modified from 20-year period used in the preliminary alternatives analysis). Land costs were based on estimated facility size (acres) and typical unit costs provided by the District for land acquisition, mitigation for loss of coastal farmland, and severance. In Table ES-2, Alternative 5 is the only alternative that requires acquisition of land. A willing seller has not been identified for this alternative.

Table ES-2. Comparison of alternative costs[1]

Alternative	Construction Cost, $M	Land Cost, $M	Maintenance Cost PV[2], $M	Total Cost, $M	Incremental Cost, $M
1 – Base Alternative	$11.0	$0	$0.9	$11.9	0
8 – Alt 1 with Cobble Trap	$13.8	$0	$1.4	$15.2	$4.2
12 – Alt 1 with Deeper Outlet Channel	$14.5	$0	$1.2	$15.7	$4.7
13 – Alt 1 with Wider Outlet Channel	$12.3	$0	$0.8	$13.1	$2.1
5 – Alt 1 with Wetland Treatment	$16.1	$1.9	$5.3	$23.3	$12.3
9 – Alt 1 with Diversion to VWRF	$11.5	$0	$28.3[3]	$39.8	$22.7
9 – Alt 9 without Treatment Charges	$11.5	$0	$0.8	$12.3	$0.5

[1]Based on 2013 cost levels

[2]Present Value based on 30 year life at 5 percent interest

[3]Based on VWRF standard unit treatment charge and on 2 cfs average flow – actual cost to be negotiated

Conclusions

Alternative 1 provides an increase in flood capacity consistent with District design standards and would remove property in the Phase 1 area from the FEMA 100-year floodplain (Special Flood Hazard Area). Based on relatively small improvements in velocity and sediment conditions relative to Alternative 1, the incremental costs for Alternatives 8, 12, and 13 do not appear to be justified. Alternative 1 hydraulic and sediment performance over a range of tidal conditions is extremely complex, and should be further tested and refined using a physical model to support design efforts.

Alternatives 5 and 9 provide options for addition of water quality benefits to the proposed project and either would be effective at reducing pollutant loads during low flow or summer conditions when temperatures in the Harbor are relatively high and flushing is limited. Although both alternatives have high incremental cost and have potentially significant implementation constraints, further development and implementation of one of the alternatives is recommended to address existing water quality concerns in the Harbor. Implementation of the water quality alternatives is relatively independent of the flood control features.

2. INTRODUCTION

2.1 Background

Arundell Barranca Channel is a major watercourse in the City of Ventura that drains the watershed of approximately 7,400 acres. The channel begins at the Ventura Foothills and flows into a debris and detention basin formed by the Arundell Dam. Downstream of the dam, the channel flows through Ventura and discharges into Ventura Harbor.

As part of the Arundell Barranca-Ventura Harbor to Mills Road Drain Project, the Ventura County Watershed Protection District (District) is planning to increase the capacity of the Arundell Barranca Channel between its outlet at Ventura Harbor and Harbor Boulevard (Phase 1), and between the Union Pacific Railroad (UPRR) Bridge and Mills Road Drain (Phase 2). The project would provide adequate channel capacity to convey the estimated 100-year discharge, thus reducing flood hazards in the residential and commercial and industrial areas near the channel. Improvements to the channel upstream of Harbor Boulevard to the Union Pacific Railroad Bridge were constructed in 2006 to provide 100-year capacity.

During initial environmental review for the proposed project, stakeholders in the project area raised concerns regarding potential impacts of the increased capacity. The District has retained Northwest Hydraulic Consultants (NHC) to develop alternatives to the proposed project and to assess potential impacts. In addition, the District requested that NHC screen several alternatives to the project as proposed, and assess the two or three most promising alternatives in detail. The alternatives evaluation will provide the District and its stakeholders with a comparison of the proposed project's potential impacts and benefits.

2.2 Objectives

The objectives of this project are to develop the baseline information on the existing conditions in the channel and at its outlet in Ventura Harbor in order to use it as a point of comparison against alternatives; to develop and evaluate alternatives with respect to project feasibility, potential environmental impacts, costs, and maintenance; to select the two or three most promising alternatives; and to conduct detailed analysis of the selected alternatives.

Section 3 of this report presents the existing conditions of the channel and Harbor. Section 4 presents the development and analysis of alternatives. Section 5 presents detailed analysis of the selected alternatives, followed by conclusions and recommendations in Section 6.

2.3 Previous Studies

Numerous previous studies and reports have been completed on the Arundell Barranca channel and watershed that provide background information, data, or initial concepts for alternatives for this study. In addition to direct references in this report, Appendix C provides an annotated bibliography of reference information.

3. EXISTING CONDITIONS

3.1 Project Setting and Site Observations

3.1.1 Geomorphic Setting

Arundell Barranca watershed is approximately 7,400 acres in area and consists of largely undeveloped foothill canyons, urbanized residential and commercial lands, and agricultural land. Elevations in the watershed range from about 1480 feet (NAVD 88) to sea level.

The upper watershed primarily consists of steep canyons on the south-facing slopes of the Ventura Foothills. This area is underlain by the highly deformed Tertiary sedimentary rocks of the Transverse Ranges physiographic province, and is formed by the Ventura anticline, an area of active tectonic uplift (Cotton, Shires and Associates, 1999). Soils in this area are primarily well-drained clay loams and silty clay loams that overlie shale and sandstone. The upper watershed is mostly undeveloped.

A topographic break occurs at the base of the foothills at an elevation of about 400 feet, and roughly corresponds to the location of Foothill Road. The area downstream of Foothill Road is a broad alluvial fan that is largely developed. Slopes are moderate in this area and soils are primarily well-drained sandy loams and silty clay loams.

The area downstream of Highway 101 has low to moderate slope and soils are characterized by well-drained to excessively drained sandy loams and silty clay loams. The lower portion of the project area is part of the ancient Santa Clara River delta with Quaternary alluvial deposits up to 500 feet thick.

The Arundell Barranca is a major drainage course through the City of Ventura, and has major tributaries Telephone Road Drain, Reservoir Barranca, Barlow Barranca, and Mills Road Drain. Approximately 45 percent of the watershed is urbanized, primarily at elevations between 60 and 400 feet on the alluvial fan and in the area surrounding the Ventura Harbor.

3.1.2 Existing Channel System

The Arundell Barranca channel begins in the steep canyons of the Ventura Foothills and flows into a debris and detention basin formed by Arundell Dam, constructed by the District in 1995. The dam has a tributary area of approximately 2.7 square miles (1,754 acres), or about 25 percent of the watershed. Lake and Sexton Canyons are tributary to the dam.

The channel system downstream of the dam is a combination of earth, revetted and concrete channels. CH2MHill (2006) categorized the channel into six reaches. Reaches 1 to 5, extending from the mouth to Estates Avenue (approximately 13,000 feet in length) consist entirely of reinforced concrete box, concrete compound channel, and concrete rectangular channel. Upstream of Estates Avenue, approximately 7,900 feet of Reach 6 (13,900 feet long) is categorized as earth-lined or natural, although a portion of this length is stabilized with riprap and grade control structures. Much of the Arundell

Barranca channel system is designed to operate with high velocities (supercritical regime) during flood flows.

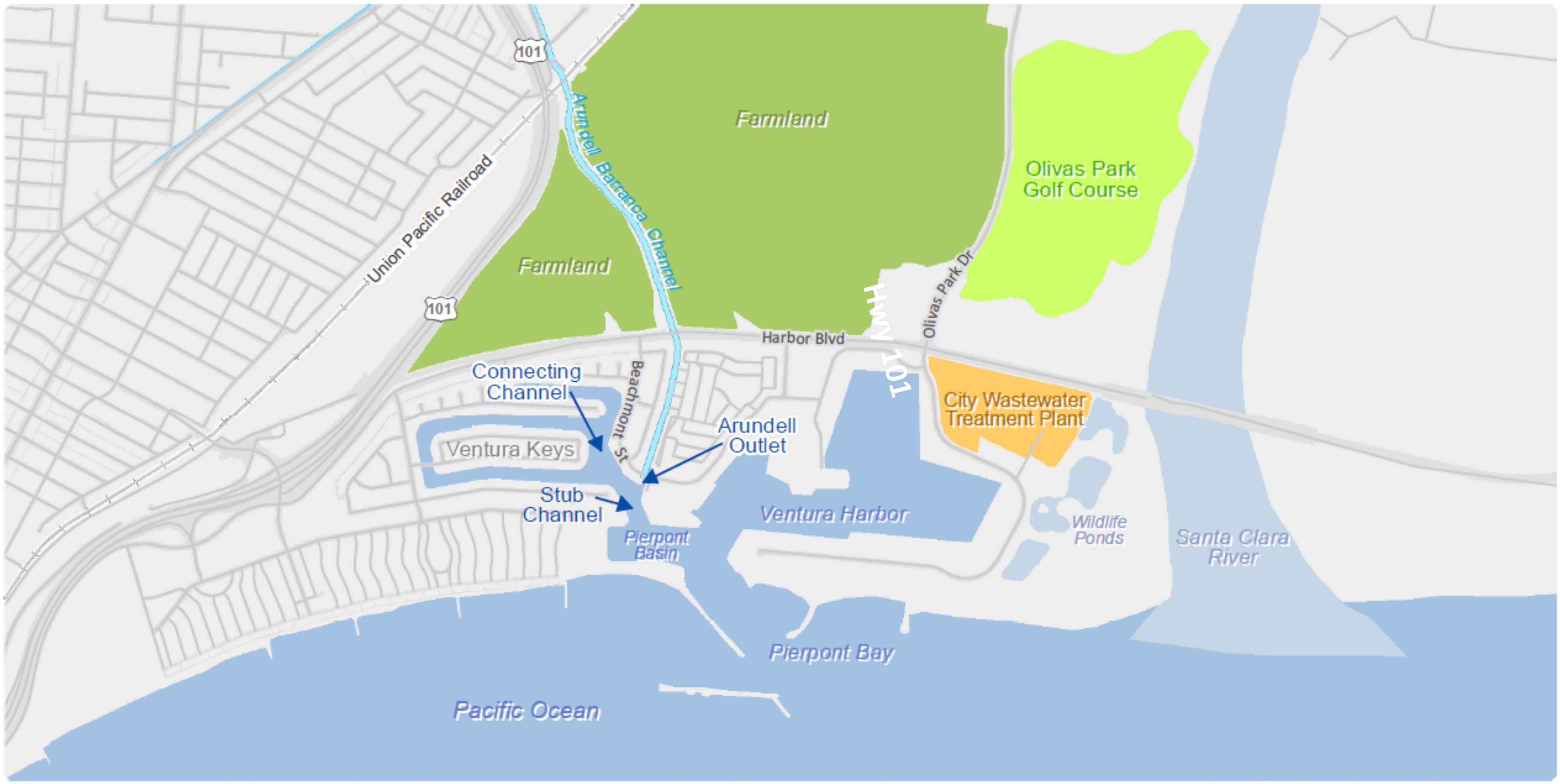
The tributary channels are comprised of a combination of channel types and closed conduits. Reservoir Barranca and Barlow Barranca have significant lengths of natural channels in the undeveloped upper portions of their sub-watersheds in the foothills. The portions of the channel in the urban area are primarily underground conduit, reinforced concrete box, and rectangular concrete channels. Mills Road Drain has a short section of natural channel in its upper watershed, but the remainder of the sub-watershed is heavily urbanized and flows are conveyed n underground conduits. The Telephone Road Drain watershed is entirely urbanized and flows are conveyed in underground conduit.

3.1.3 Ventura Harbor

The Arundell Barranca channel terminates in an arm of the Ventura Harbor, which is operated by the Ventura Port District. The Port District was created in 1952 as a Special District and construction of the 274 acre harbor was completed in 1963. The Port District maintains the harbor, including dredging of the navigation channels.

The Arundell Barranca rectangular concrete channel terminates at an energy dissipator downstream of Beachmont Street. From this point, flows are conveyed in a partially rip-rapped trapezoidal channel that intersects a navigation channel known as the Stub Channel. The Stub Channel and Connecting Channel provide a connection between the Harbor and the Ventura Keys, a single family residential waterfront development. The Connecting Channel and the Ventura Keys channel are outside the Port District boundary and are maintained by the City of Ventura.

Figure 3-1 shows the project area, including the lower Arundell Barranca channel and the harbor.

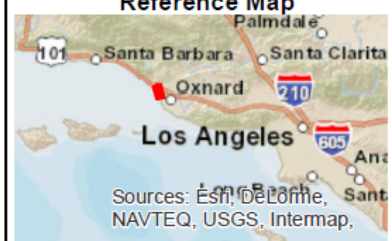


Data Sources: Road areas extracted from Ventura County parcels, downloaded from county website on 1/3/2012. Santa Clara River, Farmland, Wildlife Ponds, City Wastewater Treatment Plant, and Olivas Park Golf Course areas delineated on 1/4/2012 from ArcGIS Online World Imagery. Land Area was revised from the Political Areas shapefile extracted from FEMA's DFIRM database, effective date 1/20/2010, revised land area by nhc 1/4/2012. Arundell Barranca Channel extracted from FEMA's DFIRM database Water Line shapefile, effective date 1/20/2010. Southern Pacific Railroad line from ESRI StreetMap USA, 2008.

Legend

- Arundell Barranca Channel
- Olivas Park Golf Course
- Farms
- City Wastewater Treatment Plant
- River Channel or Ponds
- Ocean, Bay, or Harbor
- Land Area
- Roads
- Union Pacific Railroad

Reference Map



Arundell Barranca Project Area

Figure 3-1. Project Area

Scale - 1:24,000 1 inch = 2,000 feet

0 1,000 2,000 4,000 Feet

CA State Plane, Zone V	horz. datum: NAD 83	horz. units: feet
northwest hydraulic consultants	project no. 600001	February 2015

3.1.4 Site Observations

On 29 November 2011 a site reconnaissance was completed by NHC with District personnel. The reconnaissance included the entire Arundell Barranca channel between the Harbor and the dam and selected locations on the tributaries Mills Road Drain, Barlow Barranca, and Reservoir Barranca. Site observations are briefly summarized below.

Arundell Outlet Channel

Project Area

The project reach is the portion of Arundell Barranca between the Harbor and UPRR, approximately 5,700 feet in length. At the Harbor, the channel passes under Beachmont Street in a reinforced concrete box and terminates at an energy dissipator. The energy dissipator was constructed in 1973 after erosion problems occurred in the channel downstream. The end of the channel is slightly below mean sea level and the energy dissipator is submerged at most tides – the tops of the chute blocks are visible at very low (negative) tides. The channel downstream of the energy dissipator has a combination of grouted rip-rap and loose rip-rap side slopes, and at very low tides this material is also visible in the bed of the channel approximately 80 feet downstream of Beachmont Street. The top of a sheet pile wall, constructed at the time the energy dissipator was installed, is visible running diagonally in the bed. The sheet pile wall and rock bed material form a sill and subsequent drop in the channel profile downstream of the energy dissipator.

Beachmont Street

The channel between Beachmont Street and Harbor Boulevard is a high velocity 25 foot wide rectangular concrete channel that was constructed in 1967. The channel has an access road along its north side and runs through residential areas. At the time of the site visit no sediment or debris was observed in the channel. The channel bed is level, and low flows (non-storm runoff) are distributed at

Looking Upstream at Channel between Beachmont Street and Harbor Boulevard

shallow depth across the channel bottom. The channel bed appears to be in relatively smooth condition, without excessive wear or erosion from coarse sediment transport. Downstream of Harbor Boulevard Bridge there is a drop in channel profile of approximately 3 feet that resulted from connection to the existing channel in 1967.

Harbor Boulevard Bridge

The Harbor Boulevard Bridge is a combination of reinforced concrete box and compound concrete channel (short vertical walls with concrete slopes above). The upstream portion of the bridge has a pier in the channel at about 8 feet from the north wall, and columns in the sloped portion of the channel above the vertical walls. A farm road bridge is located about 90 feet upstream of the Harbor Boulevard Bridge. The channel in this area constricts from an upstream width of 24 feet to a width of 20 feet, then expands again near the upstream face of the Harbor Boulevard Bridge to 24 feet. The combination of contraction and expansion, partial height vertical walls, and piers and columns in the flow make bridge hydraulic conditions very complex at flood flows.

Looking Upstream at Channel between Harbor Boulevard and UPRR

The channel through the reinforced concrete box farm road bridge and for approximately 3,600 feet upstream was constructed in 2006. This section of channel is a high velocity rectangular concrete channel that is 24 feet wide. The channel has a slight "V" shape in the bed, but low flows observed in the site reconnaissance were distributed at shallow depth across much of the channel bottom. The channel has a service road along the north side and a bike trail along the south side.

A second reinforced concrete box farm road bridge is located about 150 feet downstream of the UPRR Bridge. Between the farm road bridge and the UPRR Bridge the channel is 20-ft wide and has approximately 6-ft tall vertical walls with side slopes above. The UPRR Bridge is a girder structure with compound concrete walls and side slopes.

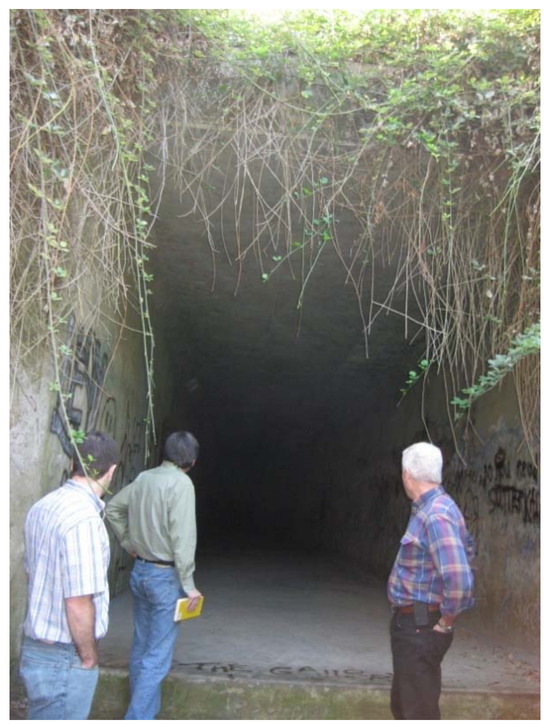
UPRR Crossing

Upstream of Project Area

Observations of channel conditions were made between the UPRR Bridge and Mills Road Drain, near the confluence of Barlow Barranca at Main Street and Highway 101, and upstream of the highway in Camino Real Park. Various sections of the channel were walked between Estates Avenue and Foothill Road, and observations were made at Sexton Canyon Road, Victoria Avenue, and Arundell Barranca Dam. As noted above, multiple channel types are present in these areas. General observations in the sections with unrevetted or partially revetted bed indicate that most of the bed materials are coarse (cobbles and boulders), that there appears to be some tendency for incision of the bed and associated bank erosion, and that there is little coarse sediment in storage in geomorphic features (e.g., bars or depositional features). These observations are consistent with the steep channel slope, confinement by urban land uses, and reduction of sediment supply by the upstream dam. Portions of the channel along Sexton Canyon Road are extremely well-vegetated and have small cross sectional area. Sediment transport capacity in this area is very low under current conditions, but might increase with channel changes associated with flood discharges. District maintenance personnel did not identify any significant change in maintenance needs following dam construction, but a smaller debris facility existed prior to construction of the current dam and the number of significant flood events since dam completion is few.

Typical Partially Revetted Bed

Several transitions between channel types were visited, and these indicate some wear of the concrete surfaces and inlets, but no areas of excessive erosion. The District provided information on repairs conducted at Estates Avenue in 2010, which were apparently necessary to address settlement cracking, but also included installation of a reinforced concrete bed overlay. Photos of the culvert entrance and bed prior to the repair show some wear, and cracking and exposure of the wall rebar near the inlet of the culvert, but no areas of significant bed erosion or rebar exposure. The box culvert and inlet were entirely clear of sediment deposits. The appearance of the channel and box culvert suggests greater sediment transport capacity than supply of sediment, and a sediment load that has a relatively small fraction of particles larger than gravel.



Estates Avenue RCB

Observations were also made at a few points on tributary channels. Conditions at the upstream portion of the Reservoir Barranca at Arroyo Verde Park indicate negligible capacity for coarse sediment transport. Although the watershed appears to be capable of producing very high sediments loads, much of the flow passes through a reservoir upstream of the park, and low flows are conveyed in an 18-inch pipe under a baseball field complex. Higher flows may produce large sediment loads, but they would be largely trapped in surface deposition in the ball field and park areas.

Barlow Barranca was observed near the crossing of Highway 101, at Telegraph Road, and at Foothill Road. The watershed upstream of Foothill Road likely produces large episodic sediment loads, but capacity for transport into the developed drainage system downstream of this point is limited by the size of the well-vegetated channel upstream and conditions at the inlet. Gravel and cobble was observed in the Barlow Barranca conduit at Highway 101, and was the only observation of coarse sediment in the concrete channel system. Because it has a relatively large, steep undeveloped upper watershed, Barlow Barranca is believed to be a primary source of large sediment material delivered to the Arundell Barranca system.

Arundell Barranca Dam

3.2 Hydrology

Flows on Arundell Barranca are dominated by runoff from storm events that occur primarily from November to April. Average annual precipitation is about 18 inches and about 94 percent occurs during this period (VCWPD precipitation normals, http://www.vcwatershed.net/hydrodata/, for Sexton Canyon Gage 230). The south facing slopes of the watershed are subject to periods of intense rainfall and a large fraction of impervious area contributes to flashy storm runoff characteristics. The channel also receives low flows from urban and agricultural land uses throughout the year. The District operates a stream gage (Gage 700) on the Arundell Barranca channel just upstream of Harbor Boulevard, and stream flow records are available for most years from 1963 to present.

3.2.1 Peak Flow

Figure 3-2 shows the annual peak flow records at Gage 700. Prior to the construction of Arundell Dam, the highest peak discharge recorded at Gage 700 is 8,050 cubic feet per second (cfs) in 1980. More recent flood events occurred in 1995, 1998, and 2005 with recorded peak flows between about 3,300 and 6,400 cfs.

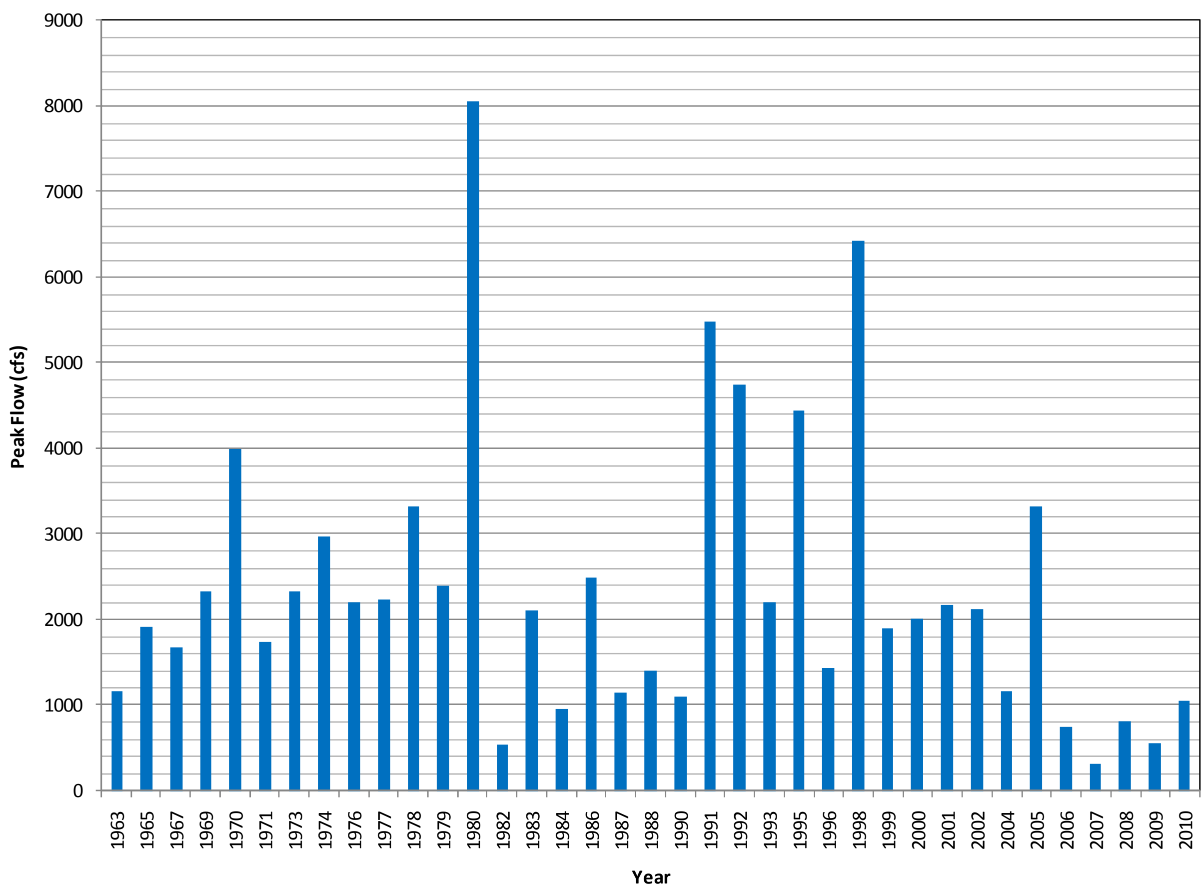


Figure 3-2. Annual peak flows for Arundell Barranca at Gage 700, upstream of Harbor Boulevard

3.2.2 Recent Event Hydrographs

Figures 3-3 through 3-5 show hydrographs of the recent flood events in 1995, 1998, and 2005. Discharges were measured at 5-minute intervals at Gage 700. The hydrographs illustrate the flashy nature of these events, with steep rising and falling limbs and rapid decline to near zero flows.

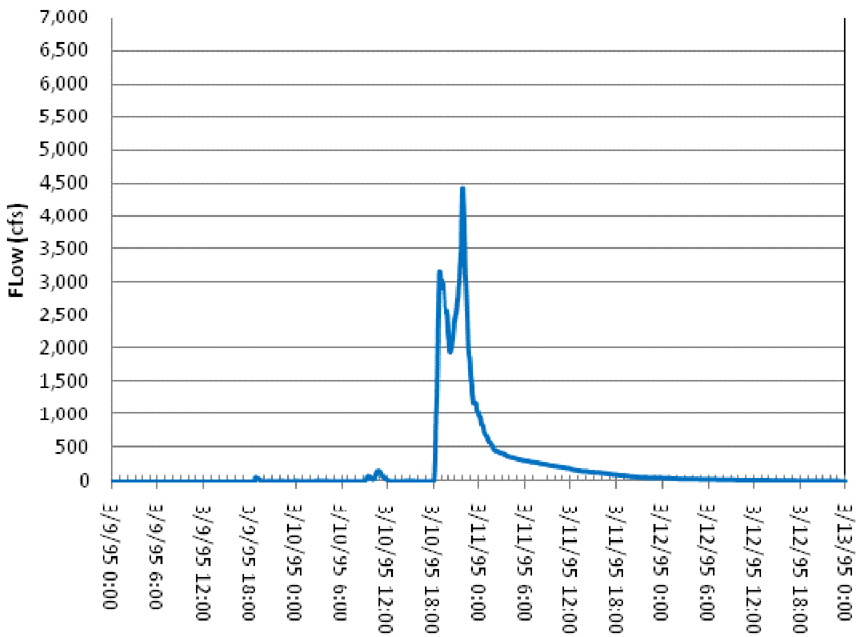


Figure 3-3. 11 March 1995 Storm Event Hydrograph

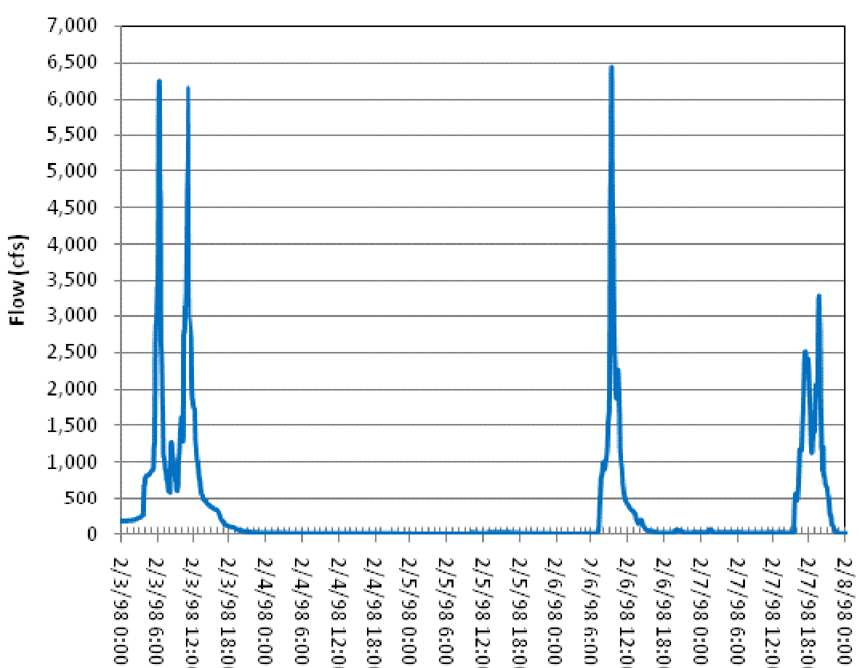


Figure 3-4. 6 February 1998 Storm Event Hydrograph

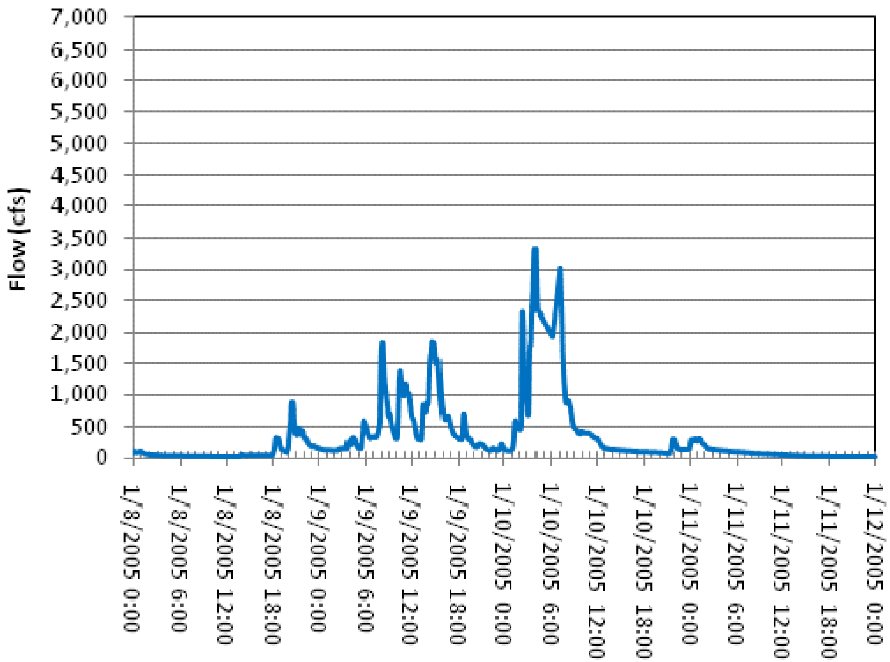


Figure 3-5. 8-10 January 2005 Storm Event Hydrograph

3.2.3 Design Flows

Under current conditions, peak flows tributary to about 25% of the watershed are detained at Arundell Barranca Dam, constructed in 1995. Flow-frequency analysis of gage records will therefore not provide representative design flows for a range of recurrence intervals. The District has produced design flows for the system using rainfall-runoff modeling, and provided 50- and 100- year peak design flows for the channel downstream of UPRR (VCWPD, 2012). The design flows, shown in Table 3-1, were also used in the Arundell Barranca detention basin conceptual design by CH2M HILL (2007). To calculate the peak flows for other recurrence intervals, the District provided peak flow ratios to be applied to the 100-year discharge based on a flow-frequency analysis of Gage 700. The design flows and corresponding ratios are summarized in Table 3-1.

Table 3-1. Peak Design Flows

Return Period (year)	*Ratio ($Q_{return\ period}$ / Q_{100})*	*Peak Flow at UPRR(cfs)*
2	0.262	1,964
5	0.424	3,179
10	0.547	4,101
20	0.674	5,054
50	N/A	6,389
100	N/A	7,498

The District uses a procedure based on the runoff volume estimated using the National Resource Conservation Service rainfall-runoff relationship to develop design flow hydrographs with appropriate volumes for each recurrence interval. Figure 3-6 shows design hydrographs for the 50-year and 100-year recurrence intervals.

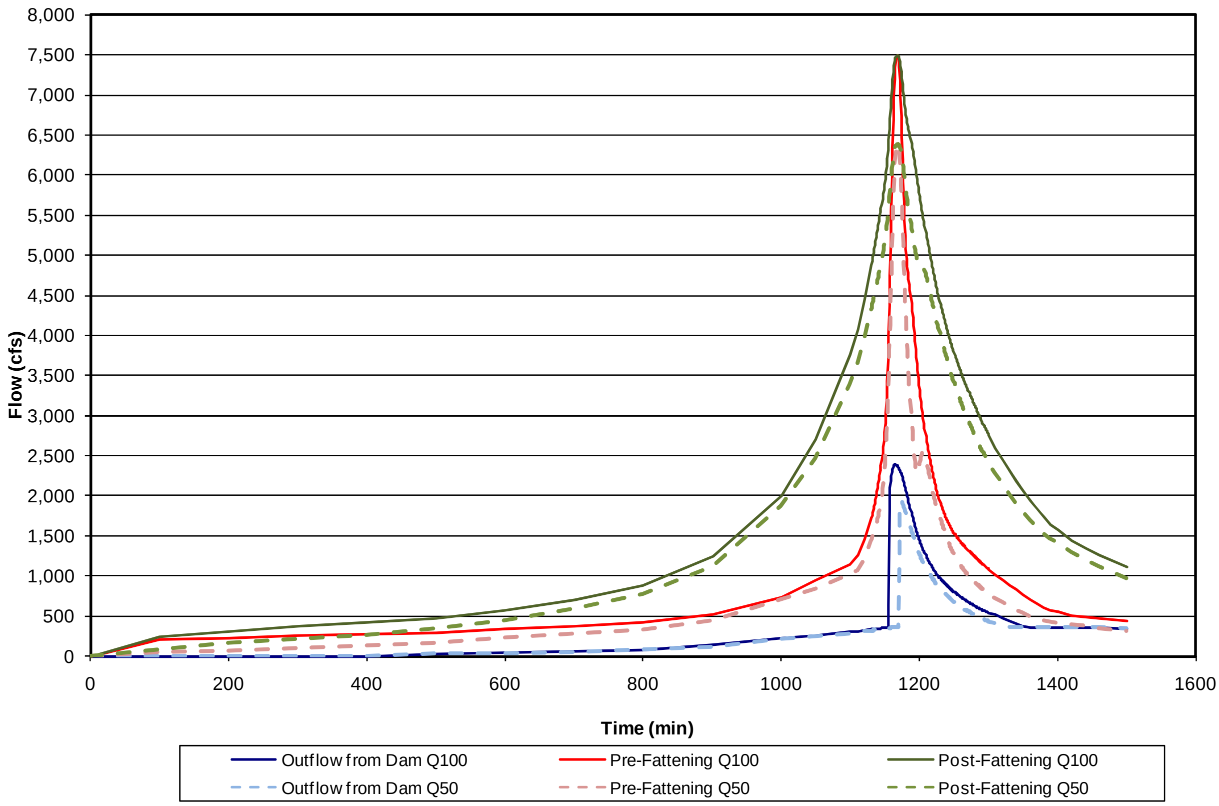


Figure 3-6. 50- and 100-year Design Hydrographs at UPRR

3.2.4 Flow Duration Analysis

For the purpose of characterizing low flows, which are influenced by sources other than storm events such as urban dry weather flows and agricultural return flows, NHC performed a flow-duration analysis on the pre- and post-Arundell Dam periods using the mean daily flows recorded at Gage 700. Figure 3-7 shows the resulting flow-duration curves. The figure indicates that flows are greater than 1 cfs about 50% of the time, but greater than 3 cfs only about 10% of the time. Pre- and post-dam curves indicate a slight increase in low flows following dam construction, which may reflect both longer duration low flows resulting from storage behind the dam and hydrologic differences in the pre- and post-dam periods. The flow duration curve emphasizes low flow characteristics and does not show the attenuating effects of the dam on peak flood flows.

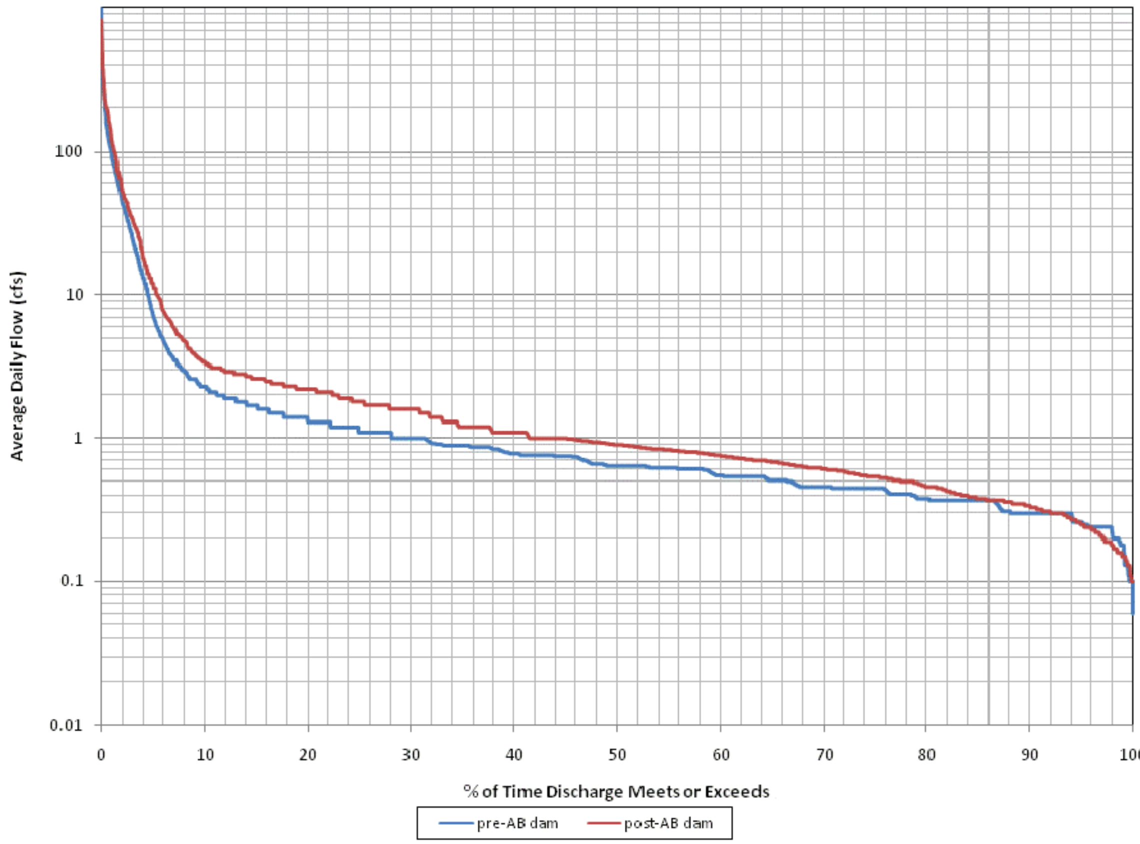


Figure 3-7. Pre- and post-Arundell Dam flow duration curves

For the purpose of considering the magnitude of summer season flows, a similar flow-duration analysis was performed using mean daily flows for the period between 1 May and 30 September. The results of this analysis are presented in Figure 3-8.

Low flows occur in the channel a large fraction of time, but larger events represent a greater fraction of total runoff volume. For the purpose of describing the fraction of runoff volume that occurs above a particular flow rate, the summer flow duration analysis was integrated with flow rate to produce a relationship between discharge and the fraction of total runoff volume. Figure 3-9 shows the results of this analysis. It should be noted that the statistics represented in Figures 3-8 and 3-9 are not solely representative of non-storm event runoff. Storm events in this period are less frequent and generally smaller than for winter months, but their occurrence strongly influences the results because they produce the largest flows. Selecting a different period or screening out rainfall events would produce a different result.

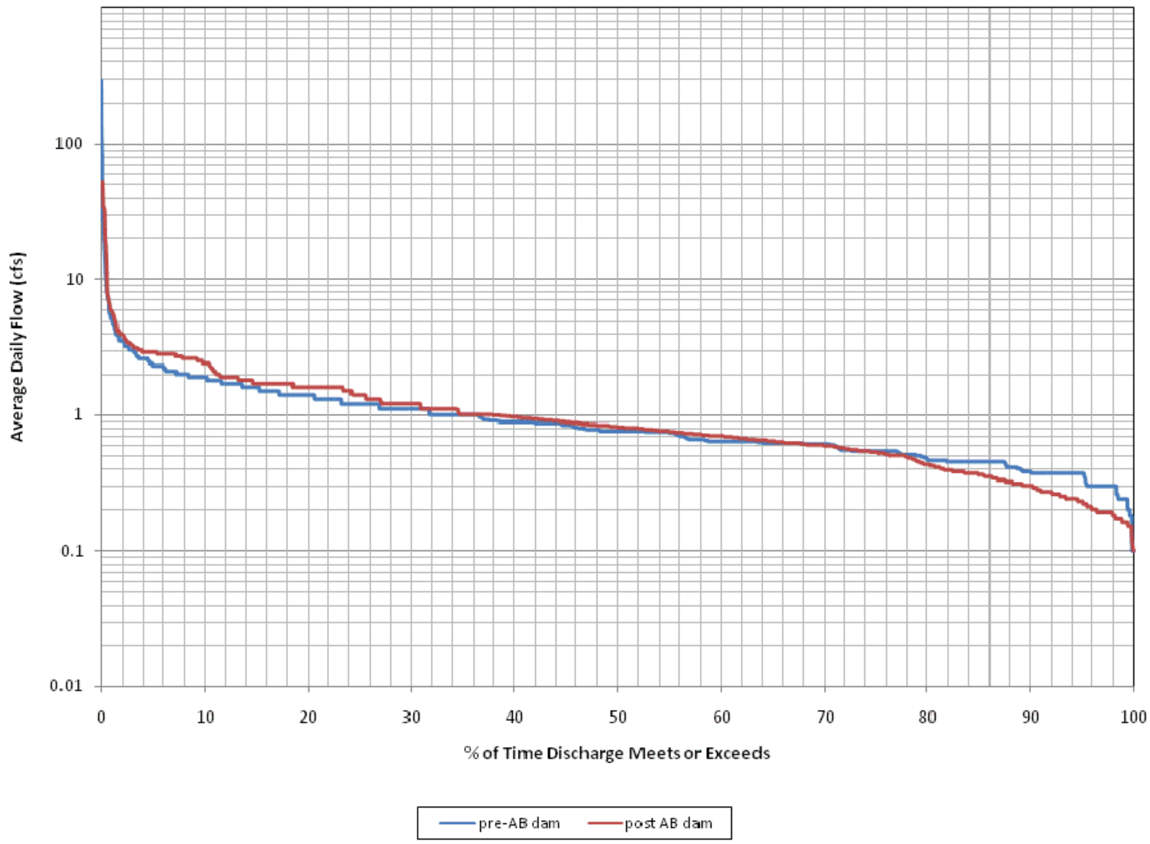


Figure 3-8 Pre- and post-Arundell Dam flow duration curves for the period between 1 May and 30 September

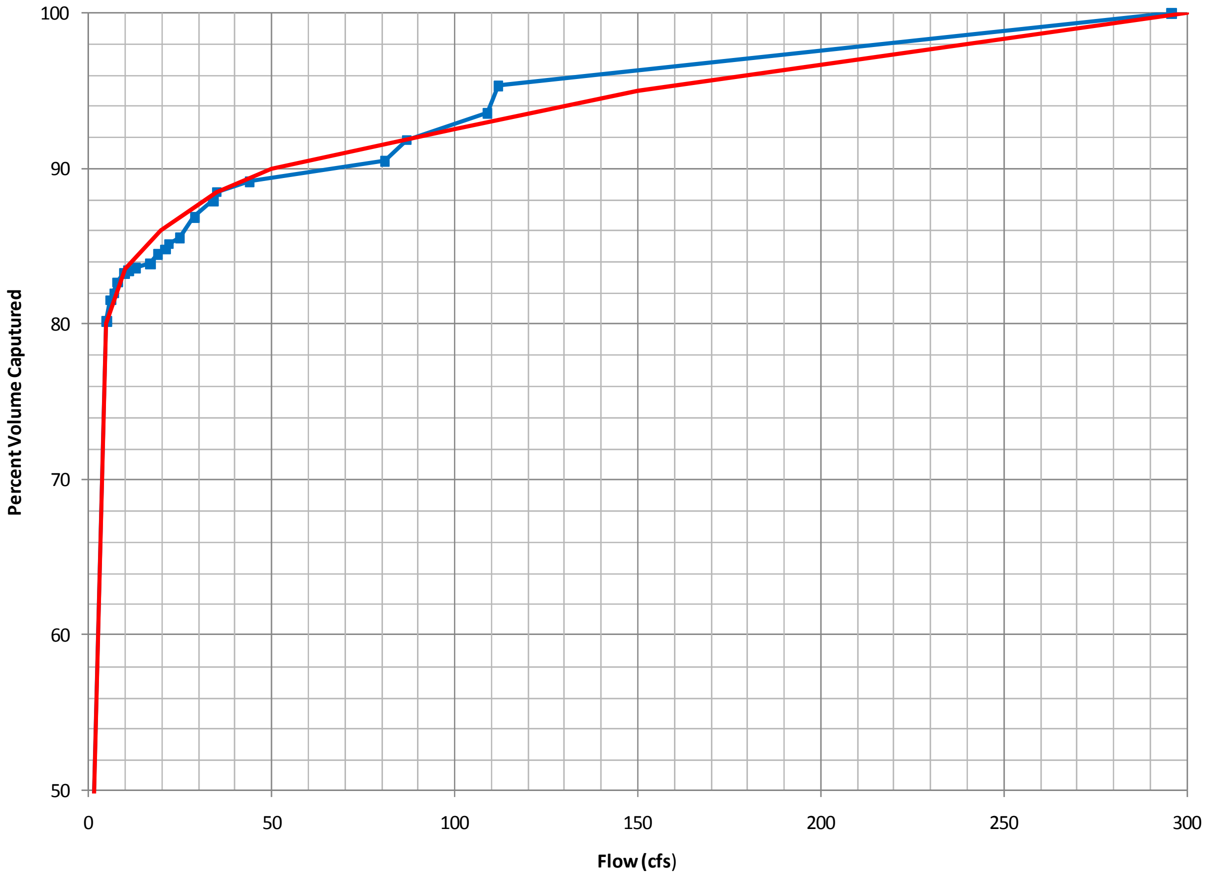


Figure 3-9. Percentage of total runoff volume in the period 1 May to 30 September occurring at or below a specified discharge

3.2.5 Tidal Evaluations

The outlet of Arundell Barranca in Ventura Harbor is a tidal channel subject to diurnal variations in water level. The US Army Corps of Engineers has established bench marks at the Ventura Harbor based on a tidal datum developed by the National Oceanic and Atmospheric Administration (NOAA) at Rincon Island. The Rincon Island datum is based on Mean Lower Low Water (MLLW). For the tidal epoch 1983-2001, the NAVD 88 datum is 0.03 meters (0.1 feet) above the MLLW datum. Mean tide is given as 0.868 meters (2.85 feet) above MLLW and Mean Higher High Water is 1.664 meters (5.46 feet) above MLLW. The maximum recorded tide is given as 2.38 meters (7.81 feet) and the minimum as -0.707 meters (-2.32 feet).

The nearest NOAA tidal gage is at Santa Barbara (Station 9411340). Tides are predicted by NOAA at Ventura Harbor as Station 9411189 and are referenced to the tidal gage at Los Angeles Outer Harbor (Station 9410660) with a 0.0 foot offset for low tide and a -0.1 foot offset for high tide. Time offsets are 16 minutes for low tide and 9 minutes for high tide.

3.3 Hydraulic Modeling

3.3.1 Background

As part of the Lake Canyon Dam alternative analysis, RBF (2008) performed hydraulic analyses to delineate floodplains for five flood protection alternatives using HEC-RAS and WSPG models. RBF's hydraulic model extended from Arundell Dam to Ventura Harbor and was built from a combination of models created by OMRUN Engineering (2005) and CH2M HILL (2006). The updated model included recent channel improvements in the reach between UPRR and Harbor Boulevard, as well as new information obtained from LiDAR base mapping (VCWPD, 2005). The model was also converted to reference the NAVD88 datum to match the LiDAR mapping. NHC adopted this RBF model and, with assistance from the District, further modified the section downstream of the UPRR to develop a HEC-RAS model for the existing conditions.

3.3.2 HEC-RAS Model

The initial model geometry was derived from RBF's HEC-RAS model. With assistance from the District, the reach downstream of the Union Pacific Railroad was modified to better represent the existing conditions. Changes reflect field survey data provided by the District (2008 and 2010). The following changes were made:

- Corrected channel bridge geometry and locations for the UPRR Bridge, two farm road bridges, Harbor Boulevard Bridge, and Beachmont Street Bridge;
- Modified Harbor Boulevard Bridge to include the pier;
- Updated channel invert elevations from Beachmont Street to approximately 100 ft upstream of the UPRR Bridge; and
- Eliminated two waterlines between UPRR Bridge and Mill Street crossing.

Manning's 'n' values of 0.015 and 0.04 were used to represent the channel and overbanks, respectively. The model was run in supercritical flow regime and mixed supercritical/subcritical flow regimes, but supercritical simulations were primarily used to assess existing channel capacity. The upstream boundary conditions were set to normal flow depth using the channel slope. The downstream boundary condition at Ventura Harbor was set to the MHHW elevation of 5.36 feet NAVD88.

3.3.3 HEC-RAS Model Results

The hydraulic model was used to simulate existing conditions with the 2-, 5-, 10-, 20-, 50-, and 100-year peak flows provided by the District. Figures 3-10 and 3-11 show computed water surface profiles from supercritical HEC-RAS model runs.

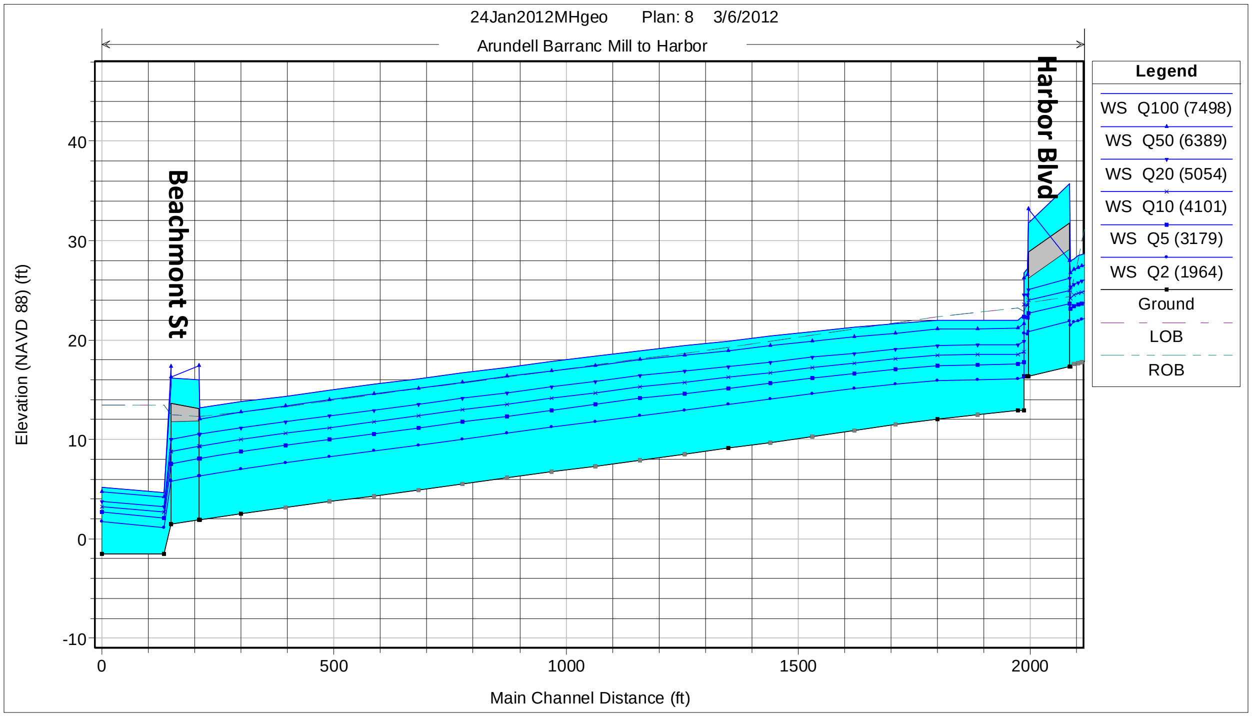


Figure 3-10. Arundell Barranca from Ventura Harbor to Harbor Boulevard - Water Surface Profiles

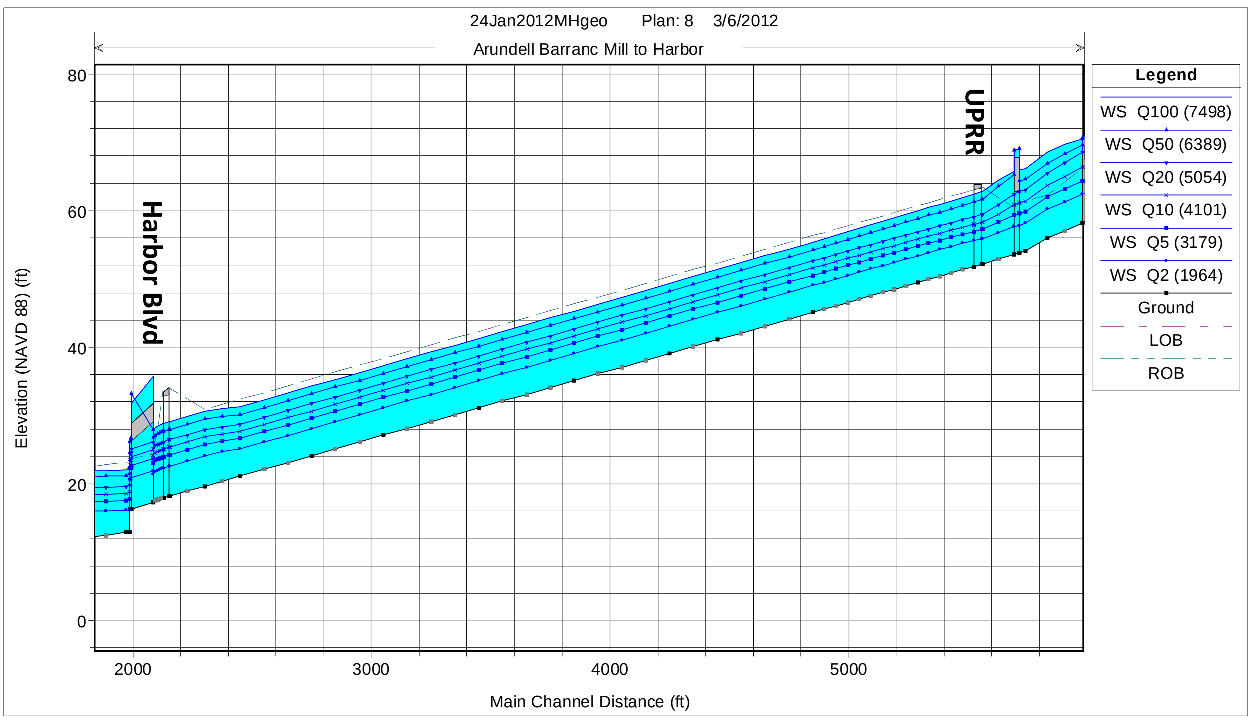


Figure 3-11. Arundell Barranca from Harbor Boulevard to UPRR – Water Surface Profiles

The simulations provide the basis for assessing existing channel capacity. The channel between Harbor Boulevard and the UPRR Bridge has sufficient capacity to convey the 100-year design flow of 7,498 cfs with the water surface approximately one foot below the top of the channel wall. The channel conveys the 50 year design flow with approximately two feet of freeboard, which meets District design standards. The channel between Ventura Harbor and Harbor Boulevard has a capacity of approximately 6,200 cfs with the water surface at the top of the channel wall. With one foot of freeboard, the capacity is estimated at approximately 5,400 cfs. Computed velocities in the channel between Ventura Harbor and Harbor Boulevard vary from 28 feet per second (ft/s) for the 100-year flow to 19 ft/s for the 2-year flow. Velocities in the channel between Harbor Boulevard and the UPRR Bridge were slightly higher due to its steeper slope; about 30 ft/s for the 100-year flow and 21 ft/s for the 2-year flow.

The Beachmont Street Bridge has a cross section consistent with the upstream channel section and the capacity of the bridge is similar to the channel. The Harbor Boulevard Bridge and the channel just upstream has variable geometry, a mid-span pier, compound channel geometry, and columns within the flow area on the upper slope. The capacity of this bridge is difficult to estimate with one-dimensional models because actual performance may depend on the effects of flow disturbances (e.g., cross waves) in the supercritical channel. The existing conditions HEC-RAS model defaults to critical depth upstream of the bridge and estimates the water surface for the 20-year flow capacity approximately at the upstream soffit. A rough estimate of existing capacity is therefore approximately 5,000 cfs. The computed water surface profile is strongly influenced by the contraction in channel width upstream of the bridge. To estimate the impact to capacity associated with the existing contraction in channel width immediately upstream of this bridge, NHC modified the HEC-RAS model to eliminate this constriction and extend vertical walls under the bridge. This model produces an estimate of approximately 6,000 cfs with the water surface near the soffit of the Harbor Boulevard Bridge.

The UPRR Bridge was cited by RBF (2008) as a constraint on existing channel capacity, and CH2MHill (2006) estimated that the channel system between UPRR and Highway 101 has a 5- to 10-year capacity. The updated version of the HEC-RAS model is consistent with these findings. The UPRR Bridge is estimated to have a capacity of approximately 5,000 cfs, assuming that supercritical flow is maintained in the channel upstream. It should be noted that channel and bridge capacities upstream may influence the performance of the UPRR Bridge, but have not been investigated in detail as part of this study.

Table 3-2. Arundell Barranca Channel Capacities

Channel Segment/Structure	***Full Capacity, cfs***	***Capacity with 1 Foot Freeboard, cfs***
Beachmont Bridge	6,200	-
Beachmont to Harbor Boulevard	6,200	5,400
Harbor Boulevard Bridge	5,000	-
Mod. Harbor Boulevard Bridge	6,000	-
Harbor Boulevard to UPRR	>7,500	7,500
UPRR Bridge	5,000	-

3.3.4 Floodplain Mapping

RBF (2008) prepared floodplain mapping for the existing conditions in the project area. Figure 3-12 shows the floodplains developed for the 100-year flood.

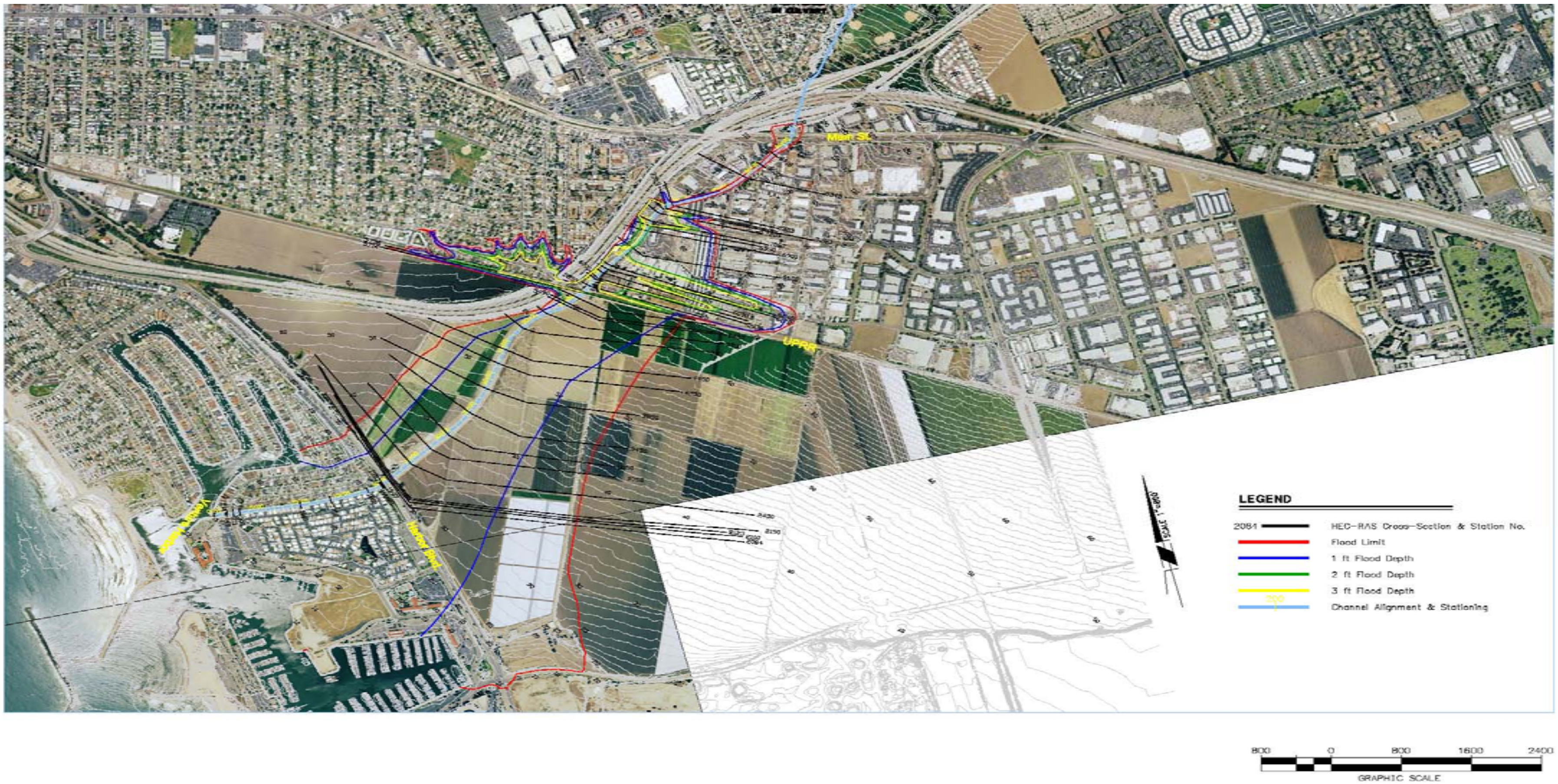


Figure 3-12. Floodplain for lower Arundell Barranca (Source RBF, 2008)

3.3.5 Harbor Hydraulics and ADH Modeling

Hydraulic conditions in the harbor were simulated using the two-dimensional depth-averaged version of the hydrodynamic Adaptive Hydrology/Hydraulics (ADH) model system. ADH was developed by the U.S. Army Corps of Engineers Coastal Hydraulics Laboratory. The model is capable of resolving depth-averaged hydraulic properties in supercritical, subcritical, and trans-critical flow regimes. The model also includes the ability to simulate sediment transport.

Model Development

The Surfacewater Modeling System (SMS) version 11.0 was used as the interface to develop the Ventura Harbor ADH model geometry. A computational mesh composed of linear triangular elements was created to cover all locations of the harbor at Elevation 8 feet (NAVD88) or less. Bathymetric survey data from post-dredging surveys (Fugro, 2007, 2010) was coupled with 2005 LiDAR data (Airborne, 2005) to develop ground elevations in the harbor. This information was brought into SMS and interpolated onto the computational mesh.

Two computational meshes were used to complete the existing conditions modeling. The first mesh represents the entire harbor and includes about a 500 foot section of the harbor entrance channel, Pierpont Basin, Stub Channel, Connecting Channel, Ventura Keys, and the entire harbor and marina to the south of the channel entrance. This mesh does not include the Arundell Barranca concrete channel. Computational nodes (found at the vertex of each triangular element) are spaced about 30 feet apart in the Ventura Keys and Stub Channel area, about 35 feet apart in the Pierpont Basin and harbor entrance, and about 80 feet apart south of the harbor entrance. This first mesh was used to perform initial model calibration and verification.

The second computational mesh was intended to model the local hydraulics in the energy dissipator, Stub Channel, Connecting Channel, Ventura Keys, and Pierpont Basin during high flow events on the Arundell Barranca. This refined mesh did not include areas to the south of the channel entrance (marina and working harbor areas). About 500 feet of the Arundell Barranca channel and energy dissipator were added to this computational mesh. Ground elevations and channel widths for the energy dissipator came from 1974 design drawings and were augmented with updated survey information (Ventura County Watershed Protection District, 2008). The node spacing in this model was reduced to 5 feet through the Arundell Barranca channel and energy dissipator, and 15 feet in the Stub Channel. Other locations of the mesh remained at the same spacing used in the first mesh for the entire harbor.

Model Calibration and Verification

Model calibration and verification was completed in January 2012. An Acoustic Doppler Current Profiler (ADCP) was used to measure velocity throughout the harbor during a spring tide on 9 January 2012. Spring tide refers to a condition in which the sun, earth, and moon are nearly aligned, and occur during new and full moons (i.e., they are not related to the season of Spring). Spring tides are especially strong tides; the tidal elevation swing on 9 January 2012 was between –1.0 feet MLLW and +6.1 feet MLLW. Velocities were measured on both the flood and ebb tide over a period of about 14 hours. Six pressure transducers were also deployed around the entire harbor area to provide continuous monitoring of water levels. During this monitoring, no additional inflow was coming into the harbor from the Arundell Barranca. More information on the field monitoring is provided in Appendix A.

An ADH model simulation using the entire harbor mesh was run using the predicted tide levels from the National Oceanic and Atmospheric Administration (NOAA) tide station in Santa Barbara, CA (Station ID 9411340). The predicted tide level was applied as a boundary condition across the channel entrance. Flow was allowed to enter and exit the model domain only through this boundary. A Manning's 'n' value of 0.020 was specified uniformly throughout the harbor to account for bed roughness. The estimated eddy viscosity routines of ADH were used to estimate eddy viscosity throughout the harbor. The recommended value of 0.5 was used for the eddy viscosity coefficient. The results of this model run were compared to measured values throughout the harbor.

Figure 3-13 shows a comparison of measured vs. computed depth-averaged velocity. The computed depth-averaged velocity was taken from the ADH model at a time step within 15 minutes of when the velocity measurement was taken. The observed measurements and model velocities show that velocities in the harbor are low - generally less than about 0.4 ft/s even during the ebb and flood of a spring tide condition. The measured vs. computed velocity comparison shows some scatter, but all computed values fall within 0.2 ft/s of the measured velocities. Given the low magnitude of the velocities and uncertainties in the measurement and numerical processes, the agreement between the computed and measured velocities is considered adequate.

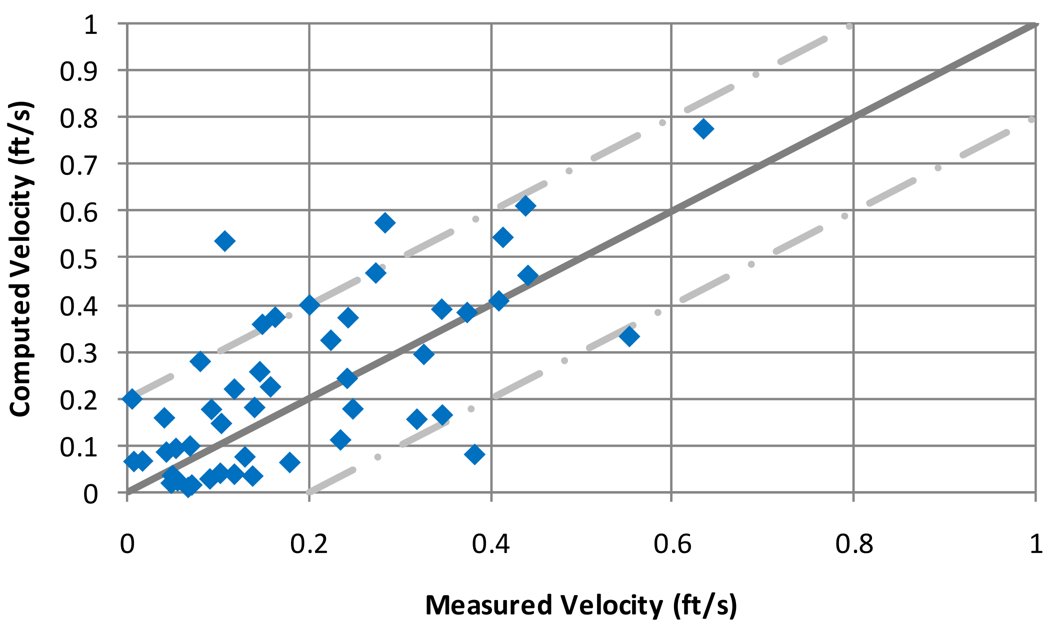


Figure 3-13. Comparison of Depth Averaged Velocity Measured with ADCP and Depth Averaged Velocity Computed in ADH , 9 January 2012 spring tide.

Figure 3-14 shows computed temporal velocity fluctuations at points around the harbor during the 9 January 2012 tidal cycle. Results of the monitoring and harbor model show peak velocities due to tidal fluctuations are less than 0.5 ft/s, and are typically in the 0.0 to 0.3 ft/s range. Figure 3-15 shows a comparison of computed water surface elevations at the northern end of the Ventura Keys and easterly extent of the Ventura Marina. The computed differences in water surface at any time between the harbor mouth and the furthest extent of the harbor is less than 0.1 feet. It should be noted that actual differences at specific times will be affected by wind and waves, and these effects were not explicitly modeled in this simulation. The small difference in computed water surfaces suggests the tidal prism is small enough and the harbor entrance large enough that there is little hydrodynamic effect of the channels in the harbor - water levels are nearly level (absent wind and wave effects) throughout the harbor at any given time during the tidal cycle.

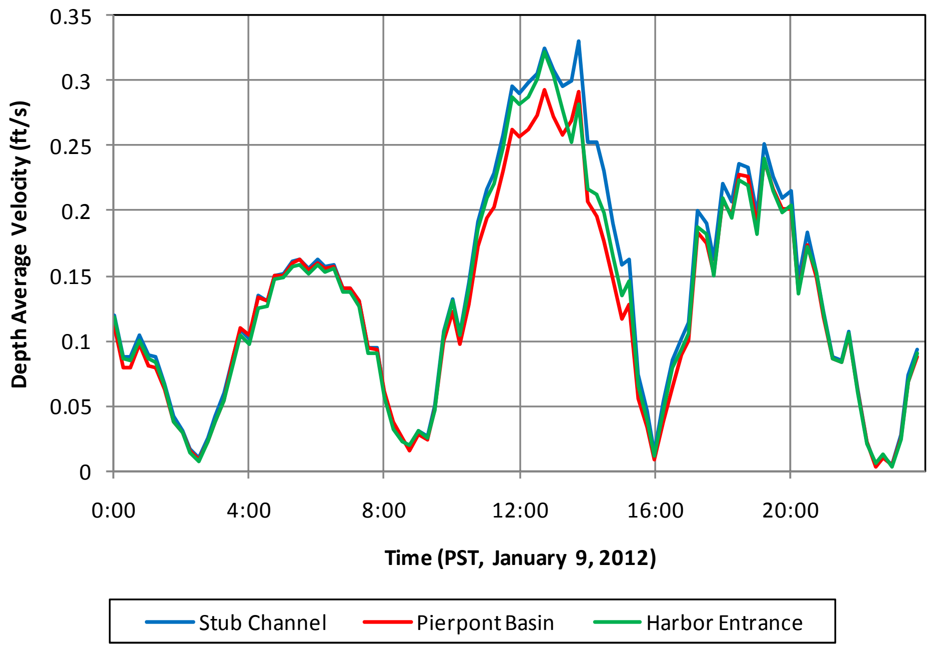


Figure 3-14. Computed velocities through tidal cycle on 9 January 2012 in Stub Channel, Pierpont Basin, and Harbor Entrance

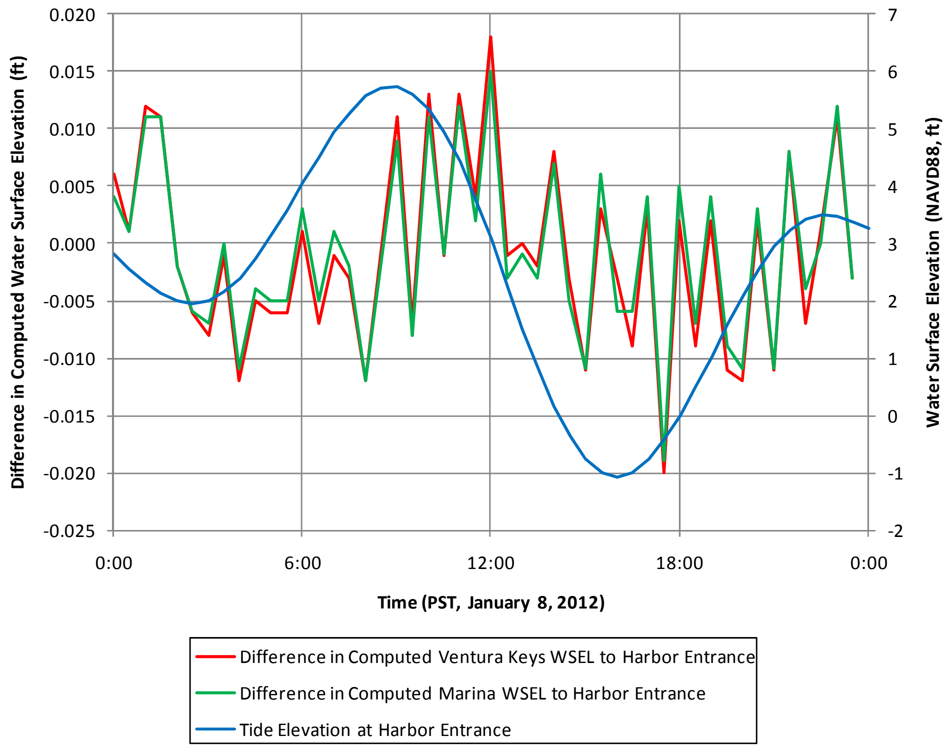


Figure 3-15. Difference between water surface elevations computed at the northerly end of Ventura Keys and easterly extent of the marina relative to the tide elevation at the harbor entrance

Arundell Barranca Inflows

Three models were run to investigate the hydraulic effects of inflows from the Arundell Barranca channel. These models included two steady state (constant flow) runs with an inflow of 6,000 cfs and a tide elevation of either mean lower low water (MLLW) of -0.13 feet or mean higher high water (MHHW) of 5.27 feet. A third unsteady (varied flow) model was run to replicate the 1998 storm event. This model included the peak 3 hours of the flood event with inflows increasing from 1,000 cfs to over 6,000 cfs and decreasing back to 1,000 cfs. The tide was in ebb flow during the peak of this event. All three models were run on the smaller mesh that includes the Arundell Barranca channel.

Material properties in the harbor were kept consistent with the harbor model discussed above. A Manning's 'n' of 0.012 was specified for the concrete Arundell Barranca channel, and a Manning's 'n' of 0.055 was specified for the energy dissipator. In addition to the tailwater boundary condition specified at the harbor entrance, an additional inflow boundary condition was added at the Arundell Barranca channel. This boundary condition specifies a velocity and depth at each node across the top of the channel. These values were determined from normal depth calculations for the given inflows.

Model Results

Figure 3-16A shows the velocities in the harbor during a hydraulic condition with a steady inflow of 6,000 cfs from the Arundell Barranca with the tide at a constant elevation of -0.13 NAVD88 ft. This event is intended to replicate a very large flood event on the Arundell Barranca channel occurring at MLLW tide elevation (6,000 cfs is the approximate peak of the 1998 event and near the estimated capacity of the existing channel). This event is expected to produce the highest velocities within the harbor. Examination of Figure 3-16A shows these velocities to be over 10 ft/s in the Arundell Barranca outlet channel, and to be between 1.5 ft/s to 9 ft/s in the Stub Channel, with the highest velocities occurring near the Arundell Barranca exit and the northwest bank of the Stub Channel. Velocities in the Pierpont Basin range from about 0.5 ft/s to 6 ft/s, and velocities in the harbor entrance are as high as 3 ft/s. Velocities in the Ventura Keys area do not seem to be affected by the inflow, but a pronounced eddy occurs upstream of the Arundell Barranca channel in the Connecting Channel. Figure 3-16B is the same as Figure 3-16A, but enlarged in the vicinity of Arundell Barranca outlet channel. The figure provides a better view of the eddy development in the Connecting Channel, and shows that velocities in the outlet channel can be as high as 20 ft/s.

Figure 3-17 shows a plot of Froude numbers in the vicinity of energy dissipator and Stub Channel with the 6,000 cfs inflow and MLLW tide. Froude numbers greater than 1.0 indicate supercritical flow. Froude numbers of about 1.8 in the Arundell Barranca concrete channel show the flow is supercritical and therefore unaffected by downstream influences. A hydraulic jump exists at the upstream end of the energy dissipator. The location of the jump is comparable to photos of the physical model of the energy dissipator constructed during the design process (City of Los Angeles Bureau of Engineering, 1972). Downstream of the hydraulic jump, flow passes through critical depth across the rock sill and sheet pile wall at the downstream edge of the energy dissipator, and transitions back to subcritical flow at the confluence with the Stub Channel. Flow in the Stub Channel and Ventura Keys is subcritical.

Figure 3-18 shows water levels in the harbor during the 6,000 cfs inflow with the MLLW tide. The inflow from the Arundell Barranca channel increases the water level by about 0.3 feet in the Ventura Keys during this condition.

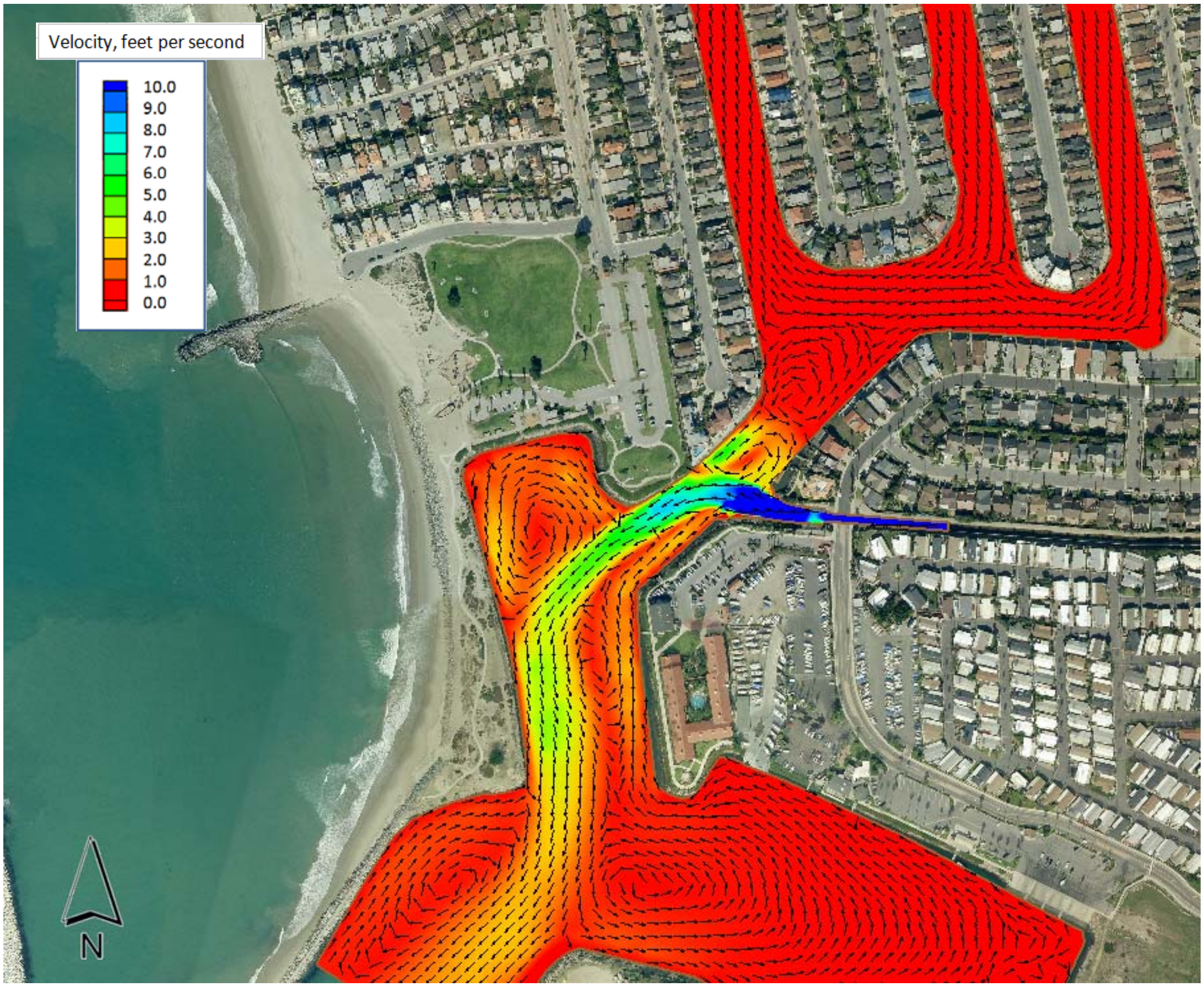


Figure 3-16A. Computed velocity for a constant inflow of 6,000 cfs on the Arundell Barranca channel and tide at Elevation of -0.13 ft NAVD88 (MLLW)

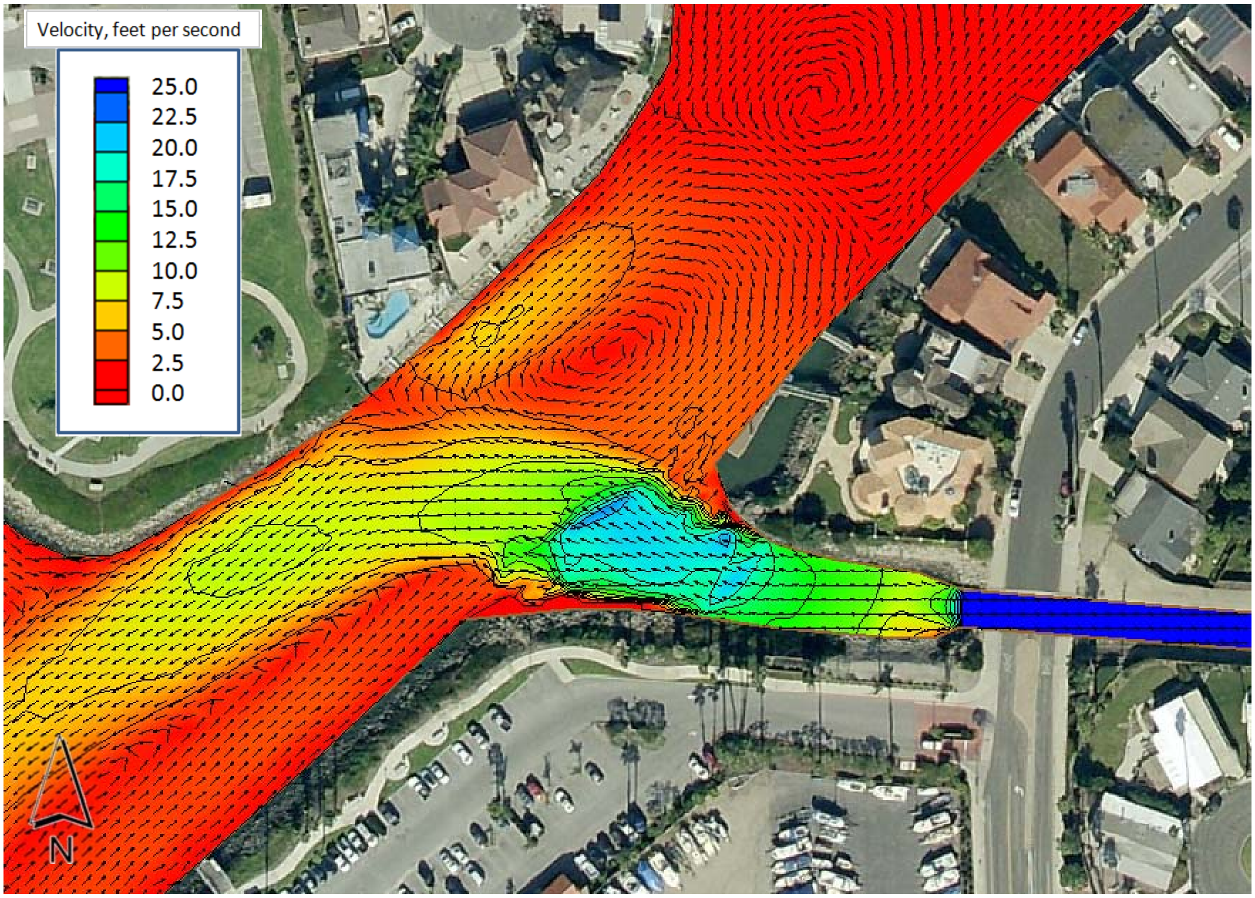


Figure 3-16B. Same as Figure 3-16A (constant inflow of 6,000 cfs with tide at Elevation -0.13 ft NAVD88 (MLLW)), but zoomed in at Arundell Outlet area

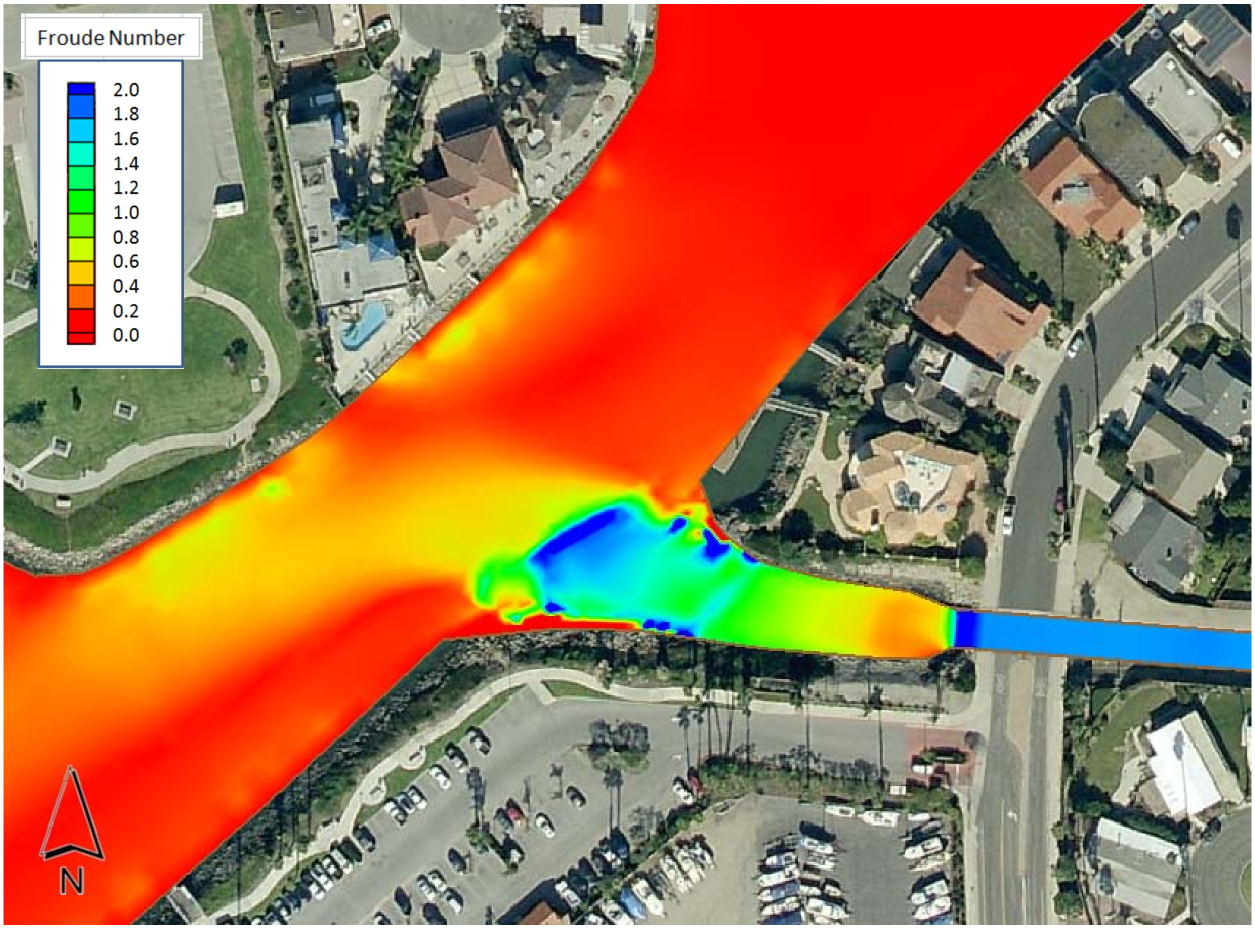


Figure 3-17. Computed Froude number for a constant inflow of 6,000 cfs on the Arundell Barranca and tide at Elevation -0.13 ft NAVD88 (MLLW)

Figure 3-18. Computed water surface elevation for constant inflow of 6,000 cfs on the Arundell Barranca and tide at Elevation -0.13 ft NAVD88 (MLLW)

A second steady state simulation was performed with a constant inflow of 6,000 cfs from the Arundell Barranca channel and the tide at Elevation 5.27 feet (NAVD88). This event is intended to replicate a large flood on the Arundell Barranca channel occurring at MHHW tide elevation. This event is expected to be the most likely to induce high water levels in the Arundell Barranca channel. In addition, comparison of this simulation to the previous simulation with tide at MLLW is intended to investigate the dependence of velocities and circulation patterns in the harbor on depth. Examination of Figure 3-19A shows velocities for this event to be between 1.5 ft/s and 8 ft/s in the Stub Channel, with the higher velocities occurring near the Arundell Barranca channel confluence. Velocities in the Pierpont Basin range from about 0.5 ft/s to 5ft/s, and velocities in the harbor entrance are less than about 2 ft/s. As for the previous simulation, velocities in the Ventura Keys area do not seem to be affected by the inflow. Peak velocities in the Stub Channel are slightly less than those computed with the MLLW tide (Figure 3-16A). Figure 3-19B is the same as Figure 3-19A, but enlarged in the vicinity of Arundell Barranca outlet channel. This figure provides a better view of eddy development in the Connecting Channel, and shows that velocities in the outlet channel can be as high as 18 ft/s even with the higher tide.

Figure 3-20 shows Froude numbers in the vicinity of energy dissipator and Stub Channel with the 6,000 cfs inflow and MHHW tide. Froude numbers of about 1.8 in the Arundell Barranca again show the flow in the channel is supercritical and therefore unaffected by downstream influences. Similar to the MLLW tidal condition, a hydraulic jump exists at the upstream end of the energy dissipator and flow passes through critical depth across the rock sill and sheet pile wall at the downstream end of the energy dissipator. In both simulations hydraulic conditions in the Arundell Barranca concrete channel upstream of Beachmont Street are independent from the tidal conditions in the harbor, and hydraulic conditions in the Arundell Barranca outlet channel downstream of Beachmont Street are controlled by flow over the rock sill and sheet pile wall at the downstream end of the energy dissipator.

Figure 3-21 shows water levels in the harbor during the 6,000 cfs inflow with MHHW tide. The high velocity, low depth outflow from the Arundell Barranca flowing transverse to the Stub Channel slightly decreases the water surface level in the Ventura Keys during this condition.

Figure 3-19A. Computed velocity at a constant inflow of 6,000 cfs on the Arundell Barranca channel and tide at Elevation 5.27 ft NAVD88 (MHHW)

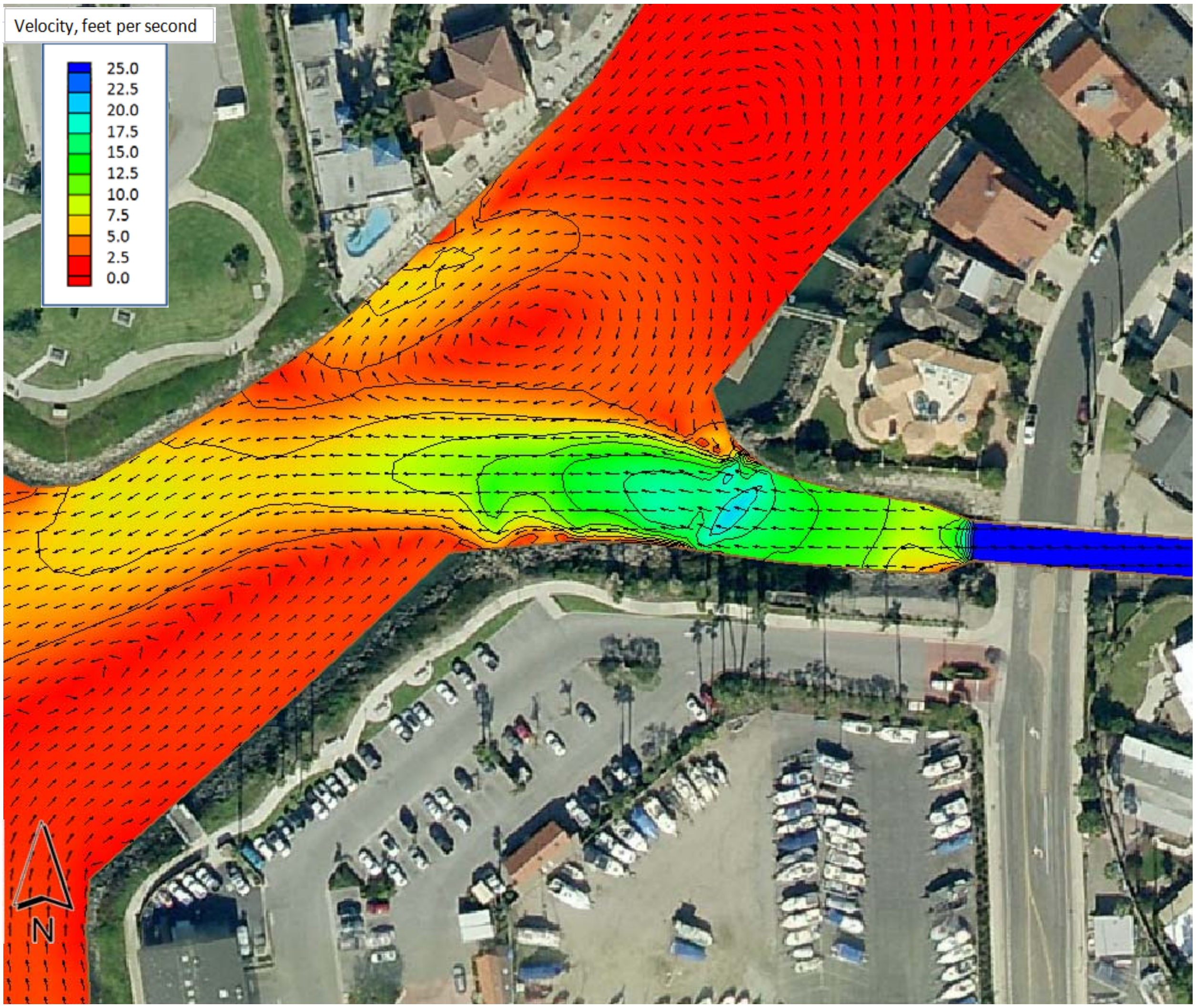


Figure 3-19B. Same as 3-19A (constant inflow of 6,000 cfs with tide at Elevation 5.27 ft NAVD88 (MHHW)), but zoomed in at Arundell Barranca outlet area

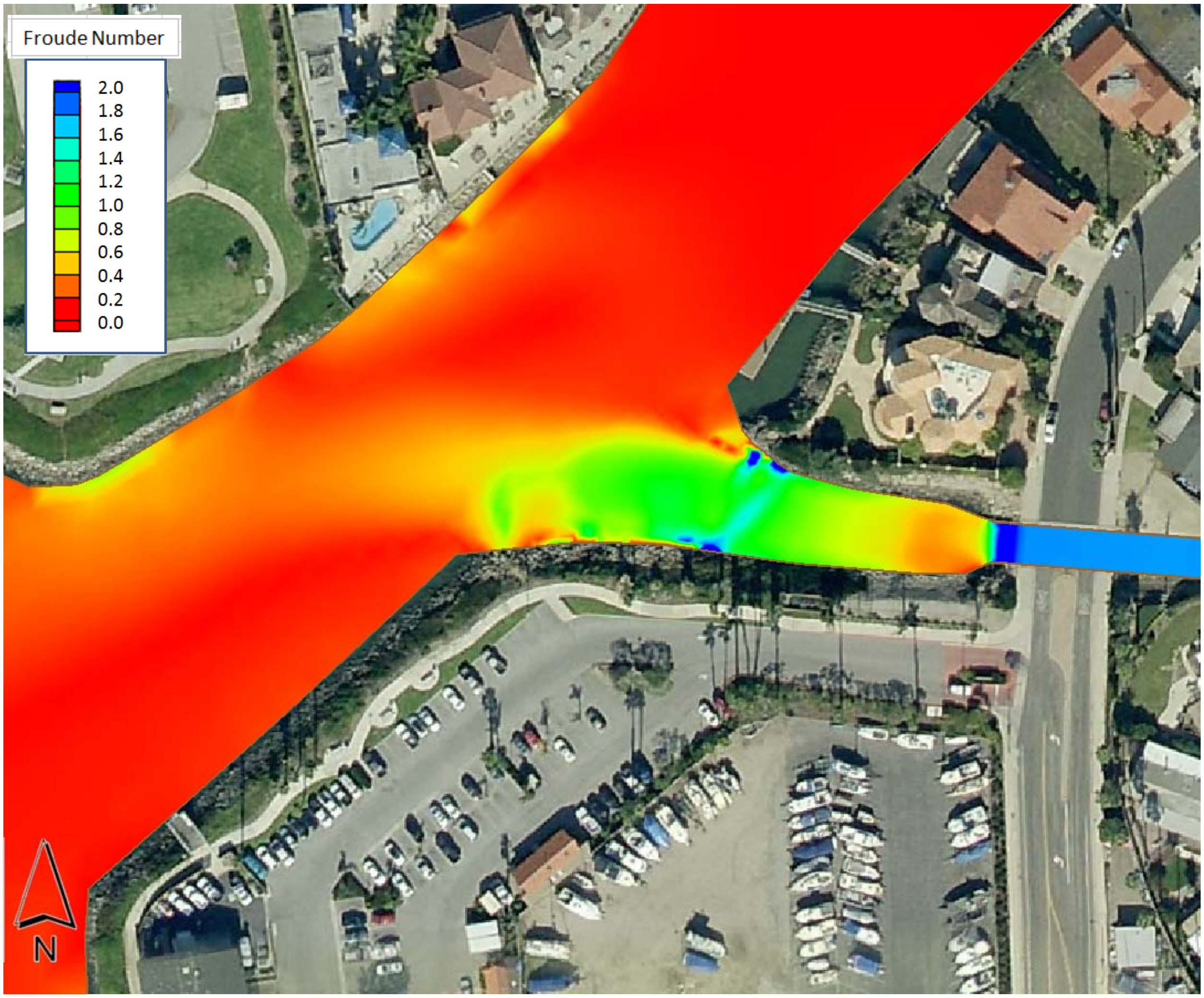


Figure 3-20. Computed Froude number for a constant inflow of 6,000 cfs on the Arundell Barranca and tide at Elevation 5.27 ft NAVD88 (MHHW)

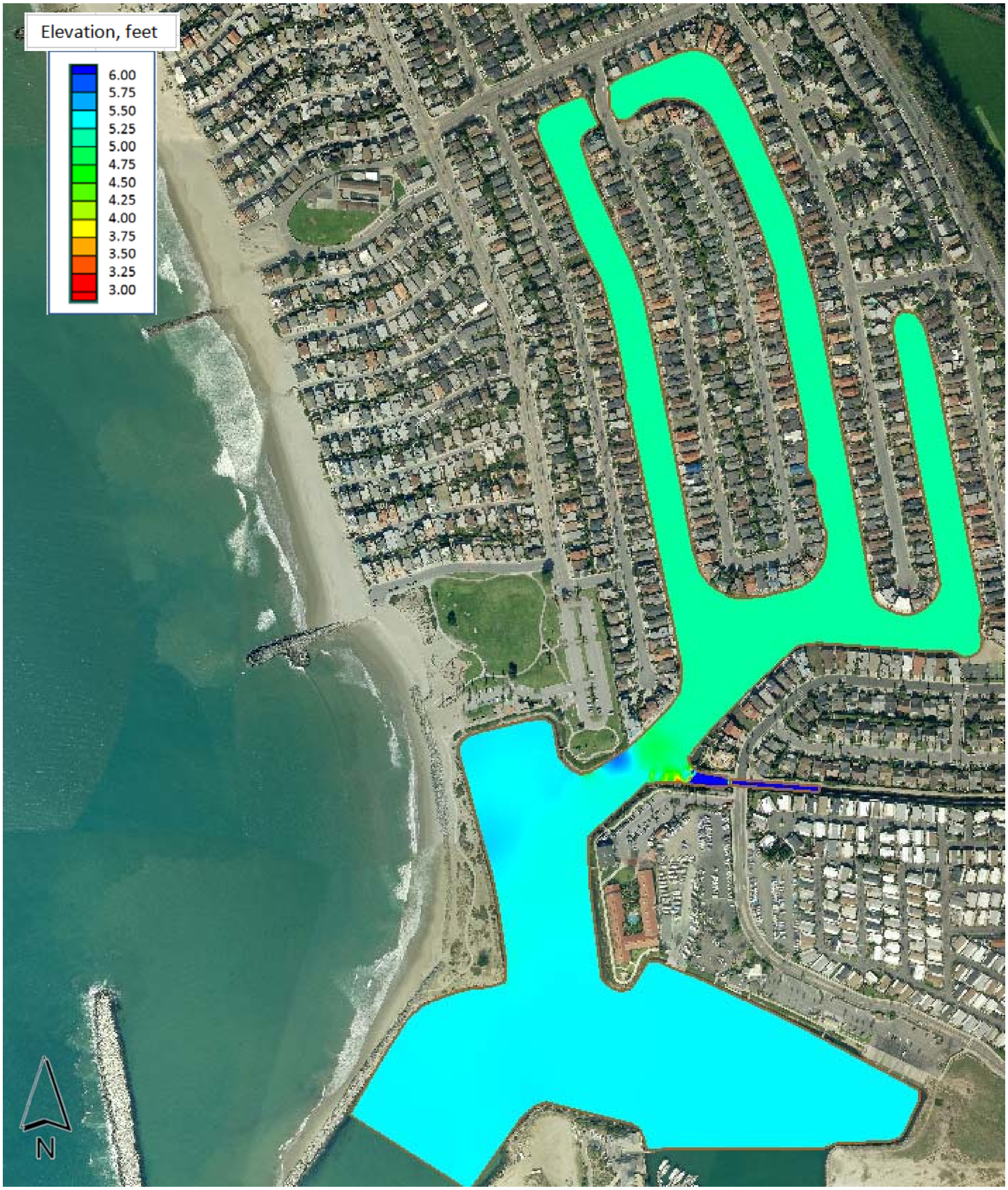


Figure 3-21. Computed water surface elevation for a constant inflow of 6,000 cfs on the Arundell Barranca and tide at Elevation 5.27 ft NAVD88 (MHHW)

In addition to the steady state simulations described above, the peak of the 1998 flood event was selected for an unsteady model simulation of an historical event. The 6 February 1998 event produced the largest peak flow in the last 30 years, but has a relatively short duration of high flows (Figure 3-4). Figures 3-22 through 3-24 show velocities from a simulation of the 1998 high flow event on the Arundell Barranca channel. Tidal elevations decreased from about 4.0 feet to 0.5 feet (NAVD88) during the three hour simulation of the storm peak. Figures 3-22 and 3-24 show velocities in the harbor with inflow at about 2,000 cfs, but on the rising and falling limbs of the hydrograph, respectively. Comparison of the two figures shows velocities in the Stub Channel as high as 6 ft/s for both simulations, but shows a larger extent of the highest velocities in the lower tide condition. Figure 3-23 shows that velocities at the peak of the hydrograph with flows exceeding 6,000 cfs are significantly higher than those at the intermediate flow. Velocities up to 9 ft/s occur across the entire Stub Channel at the confluence and contact the opposite bank. This figure shows velocities consistent with those from the steady state simulations.

The plots shown above represent specific inflow and tidal conditions, and differences in velocity and water level effects are expected for different tidal conditions. However, the simulations show that the effects of varying tidal conditions on maximum velocities in the Stub Channel are relatively modest. At lower tides, the maximum velocities are more widely distributed in the Stub Channel and along the northwesterly bank. The simulations at 6,000 cfs and the peak of the 1998 event (6,430 cfs) exceed the estimate of existing channel capacity, but during flood overflows a portion of the excess flow may be routed over the surface in the vicinity of the channel. The model simulations described above therefore represent a reasonable upper bound on inflows.

Key observations from the ADH simulations for existing conditions with flood discharges on Arundell Barranca include:

- very high velocities (up to 20 ft/s) and water surfaces of approximately 12 feet NAVD88 occur in the Arundell Barranca outlet channel for discharges near the existing channel capacity;
- the water surfaces in the concrete channel upstream of Beachmont Bridge are independent of tidal conditions;
- the energy dissipator forces a hydraulic jump, but a second supercritical to subcritical transition occurs for some flow conditions over the rock sill near the confluence with the Stub Channel;
- velocities on the order of 6 ft/s occur in the Stub Channel, fully crossing the channel and contacting the northwest bank;
- an eddy occurs at the confluence with the Stub Channel and upstream water surfaces in Ventura Keys are increased – the magnitude of increase decreases with tidal elevation, and at MHHW the increase is small and water surface elevations are well below those experienced in normal high tides; and
- in general, for the same flood discharge, higher velocities occur at lower tidal conditions and higher water surfaces occur at higher tidal conditions, but differences in maximum velocity in the Stub Channel and maximum water surface elevation in the Arundell Barranca outlet channel are relatively small.

Figure 3-22. Computed velocities during 1998 storm event (Feb 6, 1998 8:50 a.m. Q=2130 cfs, Tide=3 ft NAVD88)

Figure 3-23. Peak velocities during 1998 storm event (Feb 6, 1998 9:10 a.m. Q=6430 cfs, Tide=2.57 ft NAVD88)

Figure 3-24. Velocities during 1998 storm event (Feb 6, 1998 9:50 a.m. Q=2020 cfs, Tide=1.75 ft NAVD88)

3.4 Sediment

3.4.1 Sediment Load

A review of available information indicates that sediment yield from Arundell Barranca watershed has been previously estimated by several methods, but measurements of transport in the channel are unavailable. Exponent (1999) used the computer program HEC-6 to estimate the rate and volume of sediment flow from the Arundell Barranca watershed. A continuous simulation was performed using a 10-year period (1988-1998) of measured flows from the Arundell Barranca stream gage recorder located near Harbor Boulevard. The results of the analysis indicated that approximately 126,000 tons (or 140,000 cubic yards) per year of total sediment load was transported through the system during the study period. The report states that the estimated sediment yield is higher than what could be expected for watersheds in this region and that it might be adjusted in future based on new sediment monitoring data.

Cotton, Shires and Associates (1999) reviewed dredging records from 1967 to 1998 and concluded that that the average annual sediment accumulation in the northern harbor area (including the Keys Channels, Stub Channel, and Pierpont Basin) during this period was in the range of 35,000 to 40,000 cubic yards. The dredging records also indicated that the annual volume of sediment deposited at the mouth of the Arundell Barranca exceeded 100,000 cubic yards in at least three years in the 30-year period (in 1969, 1974, and 1998). According to Cotton, Shires and Associates (1999), in addition to sediment transported by the Arundell Barranca, sediment accumulated in the Ventura Keys channels from local sources that include erosion of agricultural land and bluff along Harbor Boulevard, sediment entering local storm drain catchbasins from local streets and properties, and windblown silt and sand from nearby beaches. The annual volume of sediment accumulating in the Ventura Keys channels from these other sources was estimated to be approximately 7,000 to 8,000 cubic yards.

The City of Ventura (2005) presented dredging records showing volumes of sediment removed from the waters of the Ventura Keys Connecting Channel and Ventura Harbor’s Stub Channel and Pierpont Basin from 1993 to 2005. These sediment volumes were attributed to the discharges from the Arundell Barranca. The average annual rate of sediment deposition during this period (which included both dry and wet years) was estimated at about 41,000 cubic yards. However, it was indicated that sediment inflows from the Arundell Barranca are episodic in nature and directly related to the major rainfall events in the stream’s watershed. Thus, the early 1998 rainfall event resulted in over 100,000 cubic yards of sediment deposition in the waters adjacent to the stream mouth and the early 2005 rainfall event resulted in over 60,000 cubic yards of deposition. More than 95 percent of the dredged material was relatively fine grained; however, cobbles and quarried stone were also found in the deposits. Though it is possible that a small portion of the finer grained material could be transported into the ocean, the City of Ventura (2005) does not believe that these volumes were significant.

Dredging data reported by Cotton, Shires and Associates (1999) and City of Ventura (2005) are summarized in Table 3-3 and graphically shown in Figure 3-25. Also included in Table 3-3 and Figure 3-25 are estimates of 2006 to 2011 dredge volumes provided by the Ventura Port District. According to the estimate made from published dredging records, the long-term average annual deposition of sediment in Ventura Keys and Ventura Harbor was about 31,000 cubic yards per year. It should be noted, however, that this value is likely to be conservative as the debris basin constructed on the upper Arundell Barranca in 1995 is much larger than the facilities that existed prior to this and therefore reduced sediment inflow to Ventura Keys. According to the data in Table 3-3, average annual dredging during 1997-2011 was about 28,000 cubic yards per year. This value represents current sediment inflow conditions, but the available dredging record for the period after the construction of the debris basin is too short to provide a statistically reliable estimate.

CH2MHILL (2006) utilized the Revised Universal Soil Loss Equation (RUSLE) and the Ventura County Watershed Protection District (VCWPD) Manual method to estimate sediment production within the Arundell Barranca watershed. The RUSLE estimates sheet and rill erosion and excludes sources from channel bed and banks and from slope failures. The equation was calibrated using data collected at the Sexton Canyon/Lake Canyon Dam. The calculated sediment yield of 33,770 cubic yards at this location is in good agreement with the measured value of about 30,000 cubic yards annual average from the Arundell Barranca Dam sediment removal records. Annual sediment yield at the Harbor was calculated using the calibrated RUSLE equation and amounted to 41,000 cubic yards. The calculated value compares favorably an estimate made by the City of Ventura from dredging records in 2005 (same average annual volume), but is considerably higher than the long-term average or the post-1995 average. The results of the RUSLE analysis indicated that under current conditions 85 percent of the sediment load to the harbor area is delivered from the Barlow Barranca and Mills Street Drain (tributaries to the Arundell Barranca) watersheds above Foothill Road.

The VCWPD Manual method employed in CH2MHILL (2006) is based on the watershed area, watershed elongation ratio, fire factor, area of the watershed prone to slipping, and rainfall factor for storms of various return periods. The estimates of sediment production were limited to the areas upstream of Foothill Road because this method is only applicable to predominately undeveloped landscapes. The average annual rate of sediment production within the Arundell Barranca watershed upstream of Foothill Road (excluding Lake/Sexton Canyons) was calculated at approximately 10,000 cubic yards, which is only about 33 percent of the observed dredging volume of about 31,000 cubic yards within the Ventura Harbor.

Table 3-3. Summary of dredging records [a]

Year	Total dredged volume (cubic yards)	Location
1967	---	
1969	192,000	Not specified
1973	89,000	Not specified
1974	130,000	Not specified
1979	90,000	Not specified
1980	92,000	Not specified
1982	40,500	Key Channels
1983	20,000	Stub Channel
1984	98,213	Stub Channel, Pierpont Basin
1986	12,285	Stub Channel, Pierpont Basin
1992	77,109	Key Channels, Stub Channel, Pierpont Basin
1993	7,000	Stub Channel
1996	111,361	Stub Channel, Pierpont Basin
1997	71,147	Connecting Channel
1998	112,005	Connecting Channel, Stub Channel, Pierpont Basin
2001	44,445	Stub Channel, Pierpont Basin
2003	12,850	Connecting Channel, Stub Channel, Pierpont Basin
2004	49,462	Stub Channel, Pierpont Basin
2005	15,340	Connecting Channel, Stub Channel, Pierpont Basin
2006	58,371	Connecting Channel, Stub Channel, Pierpont Basin
2008	2,544	Stub Channel, Pierpont Basin
Remaining (est)	50,000	Connecting Channel, Stub Channel, Pierpont Basin
Long-term average annual	31,264	
Average annual before 1996a	33,085	
Average annual after 1996a	27,744	

Notes: [a] Data from 1967-1992 from Cotton, Shires, and Associates 1999, data from 1993 to2009 from City of Ventura and Ventura Port District from records by RWP Dredging Management; [b] Debris basin on upper Arundell Barranca constructed in 1995; [c] Amount remaining to be dredged in 2005.

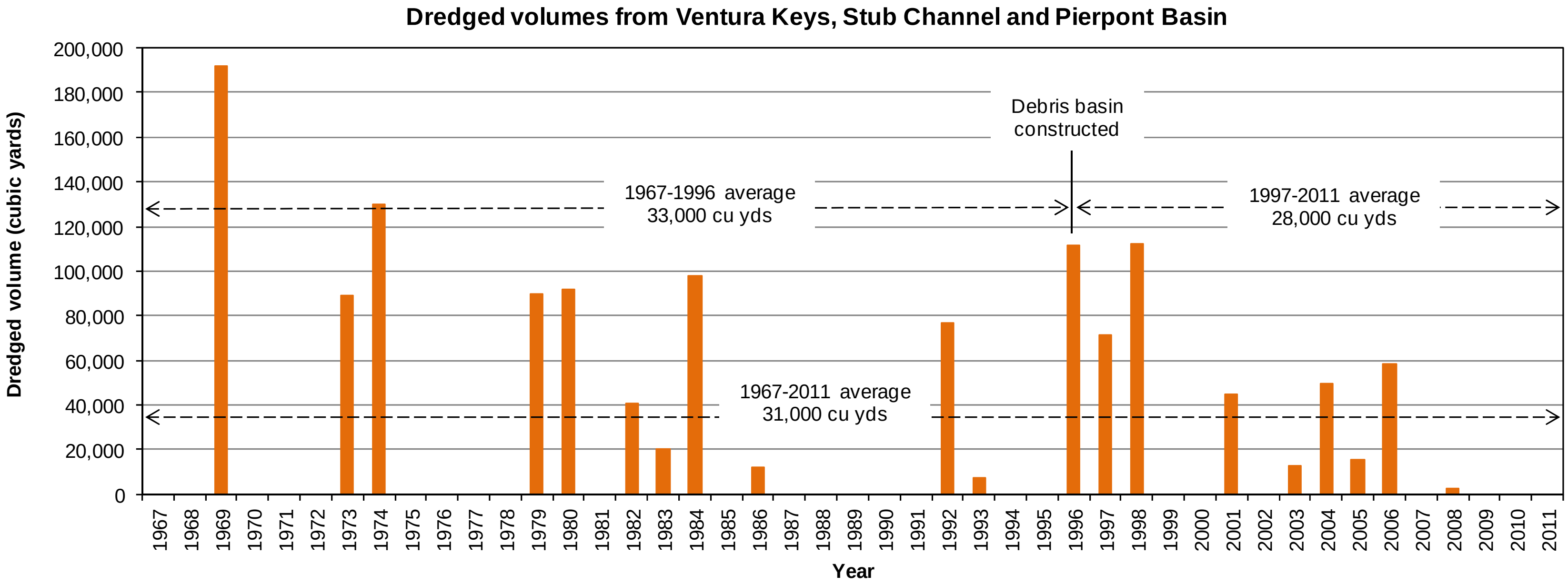


Figure 3-25. Annual volumes of sediment deposits dredged from Ventura Keys, Stub Channel, and Pierpont Basin

3.4.2 Sediment Grain Size

Applied Environmental Technologies (AET) conducted sampling of sediment deposits within the Ventura Harbor and Ventura Keys waterways in February 1994, March 1997, November 1998, May 2002, July 2005, and March 2009. Grain size compositions of sediment deposits measured by AET are summarized in Table 3-4. Following is a brief description of sampling locations and character of sediment deposits as provided in the AET (2009a) and AET (2009b) reports.

In February 1994, sediment cores were collected from 4 locations in the Stub and Pierpont Channels. According to the AET measurements, 63% of the sediment deposits were silt and clay, 36.9% was sand, and only 0.1% was gravel.

In March 1997, the sampling included the collection of sediment cores from 11 locations in Ventura Harbor and 6 locations in the Connecting Channel. During this sediment investigation, the sediments of the harbor consisted generally of silty clay up to 6.5 feet thick, followed by very fine and fine sand, with some gravel layers. Grain size analysis indicated that about 53-69% of the harbor sediments was composed of silt and clay, 30-42% was composed of sand, and up to 6% was composed of gravel. The sediments in the Connecting Channel consisted of silty clay less than 2 feet thick followed by very fine and fine sand. No grain size composition is provided for the Connecting Channel sediments.

In November 1998, sediment cores were collected from 8 locations in Ventura Harbor. The sediment deposits consisted generally of saturated silty clay in the upper 2 feet followed by silty sand or silty clay. Fine to coarse grain sand with occasional gravels were encountered at various locations around the harbor. The harbor sediment samples contained 35-99% silt and clay, 1-56% sand, and up to 10% gravel.

In May 2002, the collection of sediment cores occurred at 4 sample areas in Ventura Harbor. Within each area, 4 sediment samples were collected. According to the AET measurements, 57-81% of the harbor samples were silt and clay, 14-40% was sand, and 3-14% was gravel. In addition, sediment core samples were collected at 4 locations in the Ventura Keys Connecting Channel. The sediment in the Connecting Channel consisted generally of saturated silty clay in the first 2 feet followed by silty sand or silty clay. Fine to coarse sand with occasional gravels were encountered at various locations around the Connecting Channel. The percent of the individual grain sizes of the Ventura Connecting Channel samples were 90% silt and clay, about 8% sand, and about 2% gravel.

In July 2005, sediment core samples were obtained at 4 areas in the harbor. Within each area, 4 sediment samples were collected for compositing into a single sample for analysis. The harbor sediments consisted of 55-82% silt and clay and 18-45% fine to coarse sand. No gravel was present in the samples collected. Sediment cores were also collected at 4 locations within the Connecting Channel. The sediment in the Connecting Channel consisted generally of saturated silty clay, with a small amount of fine to medium sand. According to the AET grain size analysis, about 89% of the sediment sampled in the Connecting Channel was silt and clay.

In March 2009, sediment cores were collected in 6 areas in Ventura Harbor. The number of samples collected in each area ranged from 1 to 4. The samples were combined to obtain composite samples for each area. The sediments in the harbor consisted generally of saturated silty clay and sand, with small amounts of gravel. The percentage of silt and clay in the harbor deposits ranged from about 37% to 61%, the percentage of sand ranged from about 38% to 60%, and the percentage of gravel ranged from 1% to

6%. Maximum grain sizes in the harbor sediment samples ranged from 4.75 mm to 9.5 mm. Sediment cores were also collected in 4 locations within the Connecting Channel of the Ventura Keys. The sediments in the Connecting Channel consisted generally of saturated silty clay, with smaller amount of fine to medium sand. Grain size analysis indicated that about 70% of the sediment deposits in the Keys consisted of silt and clay, about 29% consisted of sand, and 1% consisted of gravel. The maximum grain size in the Ventura Keys deposits was 9.5 mm.

Based on the bed material sampling data presented in AET (2009a) and AET (2009b), the average composition of sediment deposits in Ventura Harbor and Ventura Keys for the 1994-2009 period was computed as: approximately 64% silt and clay, 33% sand, and 3% gravel. The measured maximum grain size in the deposits was on the order of 10 mm.

This long-term grain size distribution is generally consistent with that developed by Exponent (1999). Based on gradation data for deposits in the Ventura Keys, deposits in the Lake Canyon debris basin, and sediment monitoring data, Exponent (1999) developed the following estimate of the total inflowing load gradation for the Arundell Barranca: 21% clay, 48% silt, 21% sand, 9% gravel, and 1% cobbles. Unlike Exponent's grain size distribution, AET data do not include cobbles. According to City of Ventura (2005), cobbles were found in the material dredged from the Ventura Keys and Ventura Harbor. Therefore, some allowance for cobbles needs to be included into the AET grain size distribution for proper modeling of sediment inflow from the Arundell Barranca.

Table 3-4. Measured grain size composition of sediment deposits in Ventura Harbor and Ventura Keys (according to AET 2009a and AET 2009[b])

Year	Location	Silt and clay (%)	Sand (%)	Gravel (%)	Max grain size (mm)
February 1994	Stub and Pierpont Channels	63.0	36.9	0.1	---
March 1997	Stub Channel	53.3	41.1	5.6	---
	Main Channel	69.2	30.4	0.4	---
	Basin Channel	54.6	42.5	2.9	---
November 1998	Pierpont Basin	43.8	56.1	0.1	---
	Main Channel	35.5	54.5	10.0	---
	Main Channel	99.1	0.9	0.0	---
	Main Channel	97.2	2.8	0.0	---
May 2002	Pierpont Channel (Area A)	57.0	40.4	2.6	---
	Main Channel (Area B)	74.7	19.5	5.8	---
	Main Channel (Area C)	61.8	24.3	13.9	---
	Main Channel (Area D)	81.1	14.2	4.7	---
	Connecting Channel	90.0	8.2	1.8	---
July 2005	Pierpont Channel (Area A)	82.4	17.6	0.0	---
	Main Channel (Area B)	55.3	44.7	0.0	---
	Main Channel (Area C)	57.4	42.6	0.0	---
	Main Channel (Area D)	76.5	23.5	0.0	---
	Connecting Channel	88.6	11.4	0.0	---
March 2009	Pierpont Channel (Area A)	57.3	41.7	1.0	9.5
	Main Channel (Area B)	46.2	52.8	1.0	4.75
	Main Channel (Area C)	37.2	56.8	6.0	9.5
	Main Channel (Area D)	60.9	38.1	1.0	4.75
	Main Channel (Area E)	52.8	44.2	3.0	9.5
	Main Channel (Area F)	36.7	60.3	3.0	9.5
	Connecting Channel	70.1	28.9	1.0	9.5
1994-2009 average		**64.1**	**33.4**	**2.5**	**9.5**

3.4.3 Sediment Rating Curve

Exponent (1999) developed a sediment rating table for use in the HEC-6 model. The sediment load versus flow data provided in Exponent (1999) can be approximated by the following relationship:

$$Qt=17.2\ Q^{1.25} \qquad (1)$$

where Qt is the total sediment load (tons per day) and Q is the flow (cfs). The sediment inflow data developed by Exponent (1999) estimate that a total sediment load of approximately 1,400,000 cubic yards was transported through the Arundell Barranca during the 10-year period between 1988 and 1998. However, dredging data presented in Cotton, Shires and Associates (1999) and City of Ventura (2005) indicate that about 380,000 cubic yards were dredged from the Ventura Keys and Ventura Harbor during this period. It appears, therefore, that the Exponent (1999) calculations overestimate sediment inflow from the Arundell Barranca during this period by about 3.7 times. Though some fine sediment delivered from the stream may be conveyed into the ocean, according to City of Ventura (2005) these volumes are unlikely to be significant.

Application of equation (1) to the 5-min flow record for the period January 1998 to March 1998 yields a total sediment inflow from the Arundell Barranca of about 290,000 cubic yards (for bulk sediment density of 90 pcf). The winter of 1998 was characterized by a series of significant flood events with the maximum peak flow of 6,430 cfs (Figure 3-26). According to City of Ventura (2005), approximately 112,000 cubic yards was removed from Ventura Keys and Harbor following this high flow period. Equation (1) overestimates sediment inflow from the Arundell Barranca during this period by 2.6 times.

Application of equation (1) to the 5-min flow record for the period October 2004 to March 2005 1998 yields a total sediment inflow of about 307,000 cubic yards (for bulk sediment density of 90 pcf). The winter of 2005 was characterized by a series of flood events with the maximum peak flow of 3,320 cfs (Figure 3-27). According to the City of Ventura (2005), approximately 15,340 cubic yards was removed from Ventura Keys and harbor following this high flow period, with 72,677 cubic yards remaining (altogether 88,017 cubic yards). Equation (1) overestimates sediment inflow from the Arundell Barranca during this period by 3.5 times.

Thus, it appears that equation (1) tends to overestimate sediment yield from individual flood events as well as long-term sediment inflow from the Arundell Barranca. NHC attempted to modify equation (1) to better fit the dredging data. According to various sources (e.g. Nikitin 1951, Thompson 1985, Romashin 1990, Shvidchenko 1997) the exponent in the equation $Qt=aQ^b$ depends on regional conditions and may vary from 0.9 to 2.9. In the absence of measured instantaneous sediment transport data for the Arundell Barranca, NHC used a range of coefficients within the commonly reported limits and tested the equation against the volumes of sediment dredged from Ventura Keys and Harbor after the 1998 and 2005 winter floods. The sediment volumes dredged after the 1998 and 2005 flood events represent post-dam (i.e. existing) conditions.

Results of the calculations are summarized in Table 3-5. None of the coefficients tested provides ideal fit to all the dredging data, but the best balance between the calculated sediment loads and dredging data is shown by the following relationship:

$$Qt=0.24\ Q^{1.73} \qquad (2)$$

For the 1998 and 2005 winter flood events this equation gives maximum sediment concentrations of about 51,000 mg/L and 31,000 mg/L, respectively. The study area is known for very high suspended sediment concentrations (SSC) in streams, frequently exceeding 30,000-40,000 mg/L (USGS 2012). According to the U.S. Geological Survey (USGS) data, maximum recorded SSC on the nearby Santa Clara River ranges from 51,200 mg/L at the LA-Venture Co. Line gage 111085000 to 70,000 mg/L at the Montalvo gage 11114000. Compared to measured regional data, the maximum sediment concentration values predicted by equation (2) appear reasonable. Equation (2) was adopted by NHC as a first order approximation of sediment inflow from Arundell Barranca and used in ADH modeling of sedimentation in Ventura Keys and Ventura Harbor.

Under existing conditions, very high flows spill out of the channel upstream of Harbor Boulevard and a portion of the flow is conveyed on the floodplain. The channel capacity is estimated at about 6,000 cfs (see Section 3.3.3). The event sediment delivery to the harbor described above for the 1998 event was computed by capping the inflow hydrograph at Arundell Barranca at 6,000 cfs.

When overflows occur, patterns of flooding may vary in each event due to minor changes in floodplain topography and conditions and variability in the performance of bridges due to debris effects. In the 1998 flood event, the channel overtopped upstream of Harbor Boulevard and much of the overflow was conveyed to the south across agricultural land and along Harbor Boulevard, eventually spilling into the arm of the harbor just north of Olivas Park Drive. The Ventura Port District reports that about 5,000 cubic yards of material were dredged from this area of the harbor following the event, and that large quantities of mud were deposited in the parking lot west of Navigator Drive. The portion of the dredged material that was contributed by Arundell Barranca overflows is not known, but overflows through the agricultural lands have the potential to generate very high concentrations of sediment that may exceed the channel concentrations by a factor of 2 or more. Thus, assuming that a significant fraction of the overflow returns to the harbor, total sediment load to the harbor may be increased by the contribution made from the overflows in extreme events.

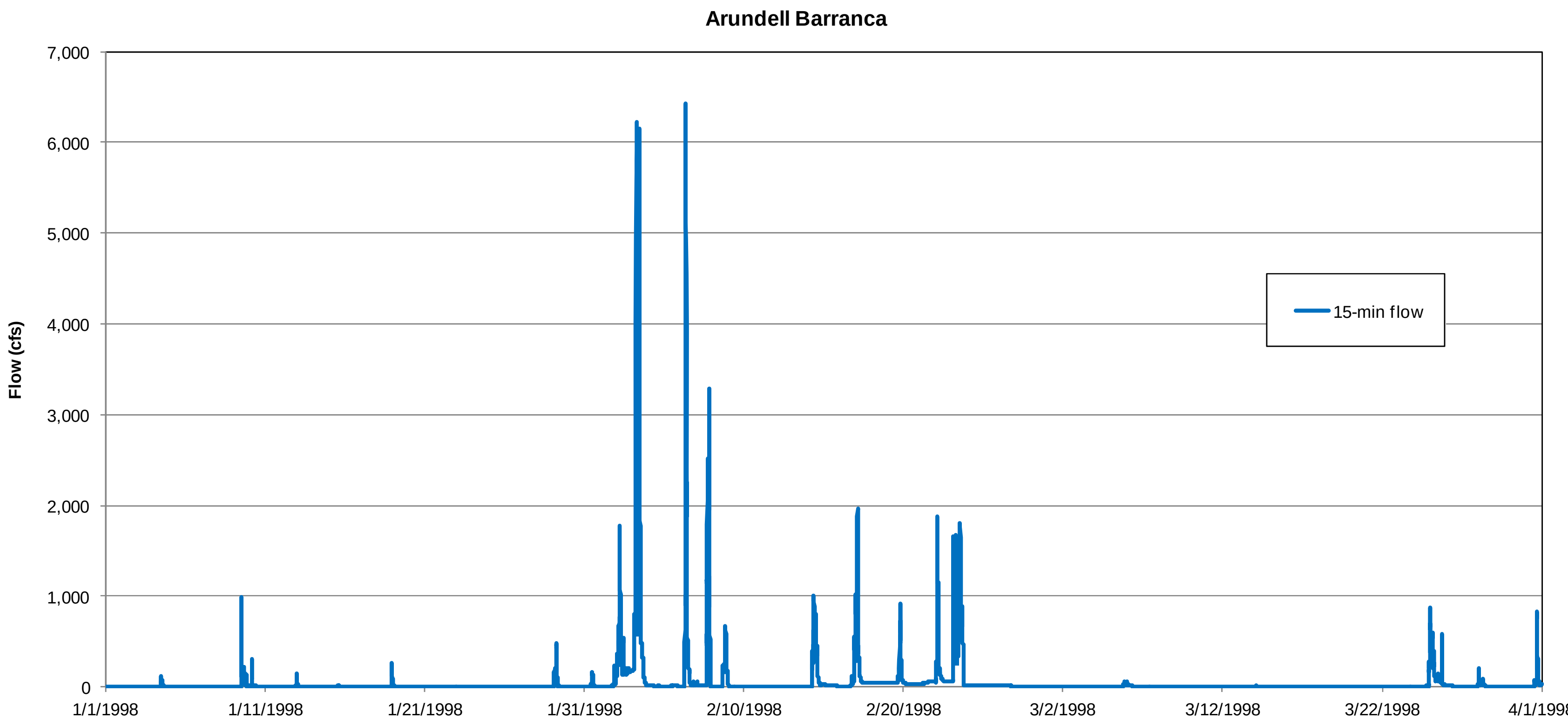


Figure 3-26. Winter 1998 flow hydrograph for Arundell Barranca

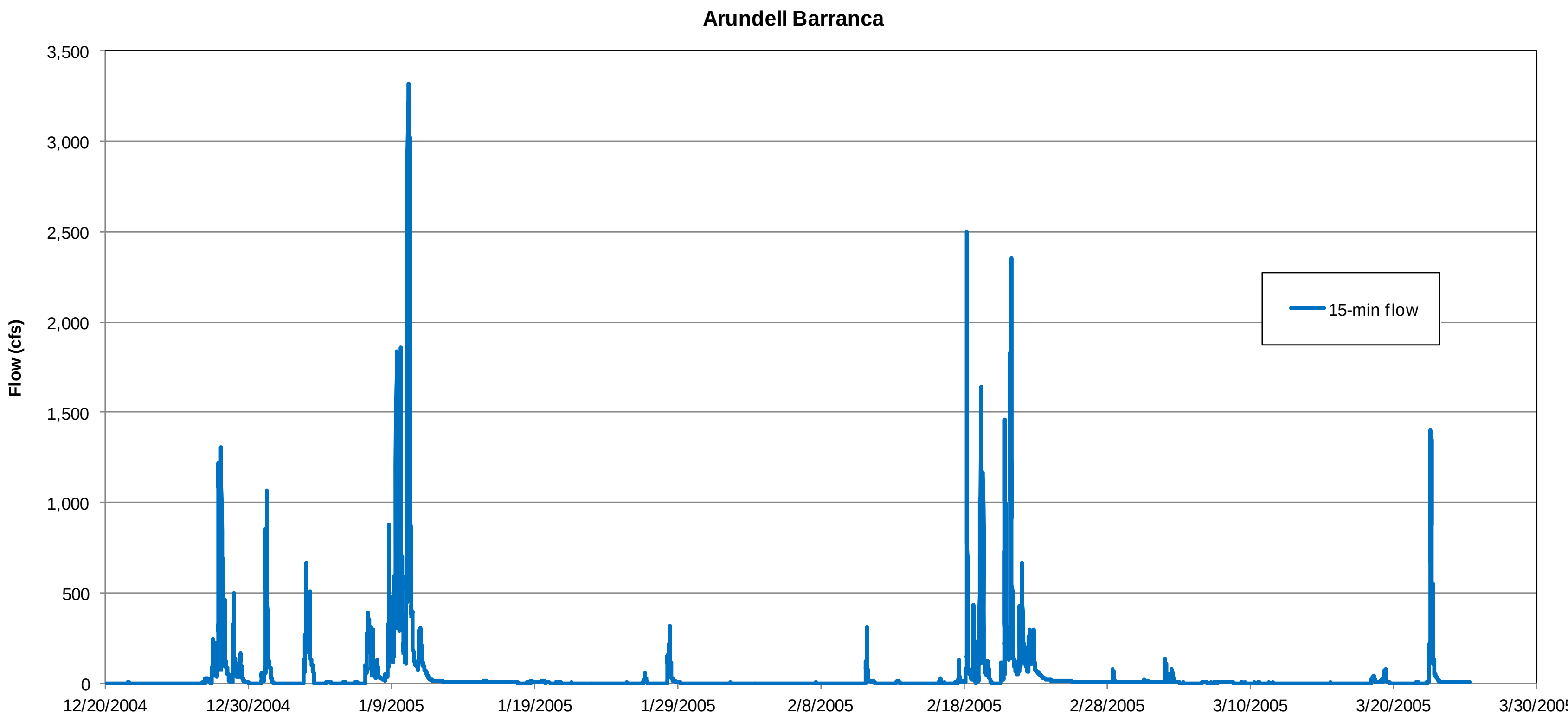


Figure 3-27. Winter 2005 flow hydrograph for Arundell Barranca

Table 3-5. Calculated total sediment yields from Arundell Barranca for different coefficients in equation $Qt=aQ^b$

Period	Dredged volume (cu yds)	Calculated sediment yield (cu yds)					
		a=17.2[c] b=1.25[c]	a=60 b=1.0	a=10 b=1.25	a=1.8 b=1.5	a=0.24[d] b=1.73[d]	a=0.001 b=2.5
Jan 1998 – Mar 1998	112,000[a]	291,000	201,000	169,000	175,000	121,000	208,000
Oct 2004 – Mar 2005	88,000[b]	307,000	233,000	147,000	136,000	83,000	90,000

Notes: [a] Dredged in Apr 1998; [b] Dredged in Mar 2005, includes reported remaining volume; [c] Based on Exponent (1999) sediment load data used in HEC-6 model;
[d] Recommended values.

3.4.4 Sediment Modeling in Harbor

The sediment transport capabilities of ADH were utilized to simulate deposition in the harbor from the peak of the 1998 high flow event. The sediment rating curve described above was used to determine sediment inflows during the peak of the event. The 3-hour hydrograph used in the hydraulic assessment (6 February 1998) was also used for this sediment transport simulation. The distribution of inflowing sediment was assumed to be 64.1% silt and clay, 33.4% sand, 2.45% gravel, and 0.05% cobble by mass. These values were represented by grain sizes of 0.067 mm, 0.5mm, 5mm, and 100mm, respectively. The Wright-Parker relationship for entrainment of suspended non-cohesive sediment multiple grain sizes was used with the Meyer Peter Mueller-Garcia Parker relationship for non-cohesive bedload.

Figure 3-28 shows the computed sediment deposition in feet after the 3 hour event. Deposition is primarily found in the Stub Channel near the entrance of the Arundell Barranca. This deposition is composed primarily of sand and gravel. Deposition depths in the Stub Channel are up to 6 feet. Fine sediment loads remain in suspension throughout the duration of this simulation, though some minor deposition of fines was noted in the Pierpont Basin and the Connector Channel. Fines concentrations at the peak of the event are illustrated in Figure 3-29. For comparison, the concentrations of the sand material at the same time step are shown in Figure 3-30. The coarser material remains in suspension for a much shorter length past the outlet of the Arundell Barranca. Fines concentrations at the end of the simulation are illustrated in Figure 3-31. Note that at the end of the simulation period significant concentrations of fines remain in suspension throughout the lower part of the model domain. These fine materials settle over much longer time periods than the sands, gravels and cobbles in the sediment load, and are subject to re-disturbance and transport with subsequent tidal action.

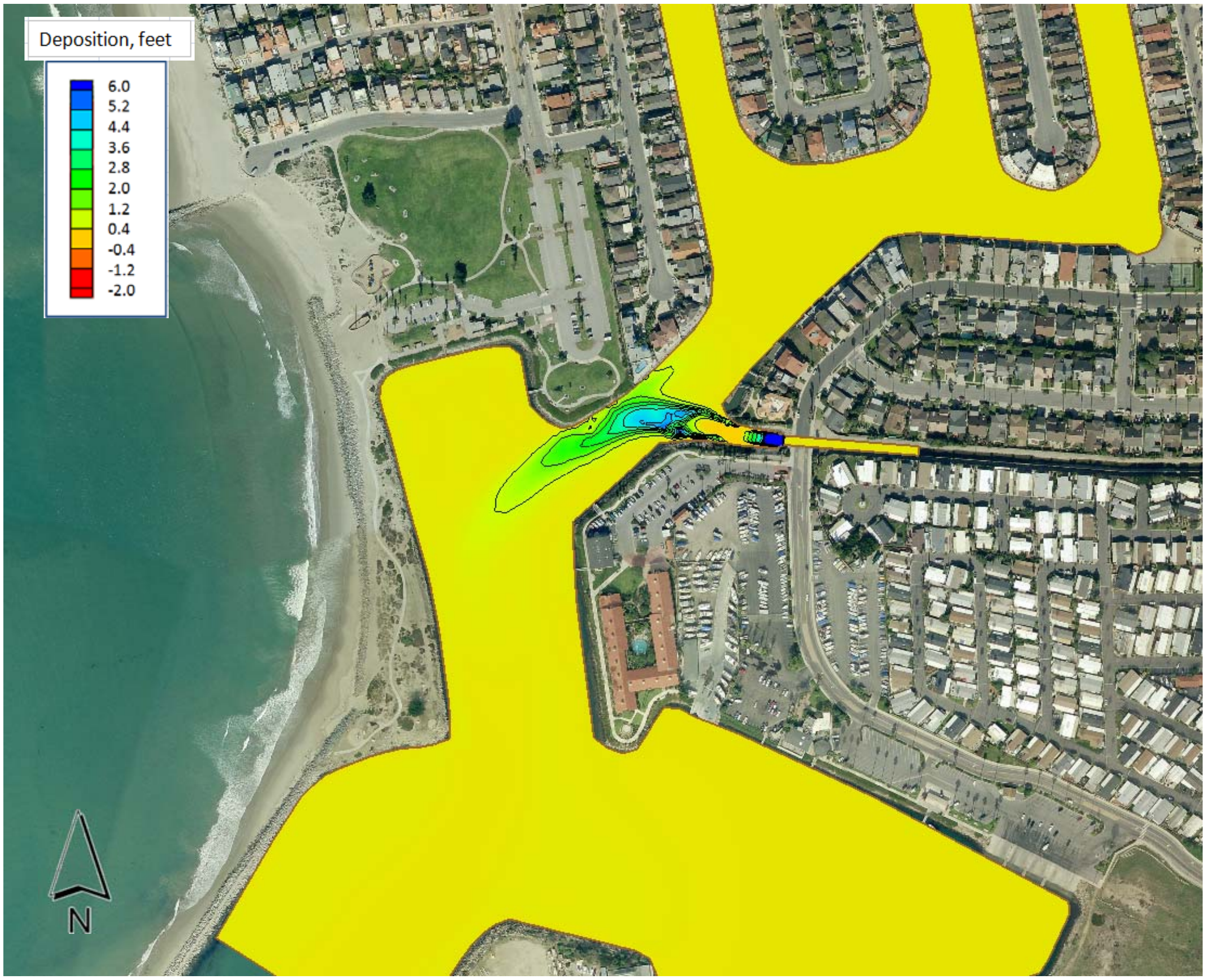


Figure 3-28. Sediment deposition resulting from 1998 high flow event for 6 February 1998 - positive values indicate increase in bed elevation due to deposition of sediment; negative values would indicate decreases in bed elevation due to erosion

Figure 3-29. Fines concentration at peak of 6 February 1998 high flow event

Figure 3-30. Coarse material concentration at peak of 6 February 1998 high flow event

Figure 3-31. Fines concentration at the end of the 6 February 1998 high flow event simulation

3.5 Water Quality

Water quality in the project area is regulated under the Los Angeles Basin Plan, California Ocean Plan, Water Quality Control Policy for Enclosed Bays and Estuaries, California Toxics Rule, and Clean Water Act Section 303(d) TMDL listings for the Santa Clara River Estuary and Ventura Harbor. The Los Angeles Regional Water Quality Control Board (LARWQCB) is responsible for carrying out the state and federal clean water acts through water quality control plans, regulations, and enforcement in the area. The US Environmental Protection Agency provides oversight for execution of the federal act and is directly engaged in some programs and policies for water quality control.

The Basin Plan establishes beneficial uses and water quality objectives for various constituents and water bodies, and describes water quality plans and policies for the region. The California Ocean Plan is a statewide plan that establishes beneficial uses and water quality objectives for waters of the Pacific Ocean outside of enclosed bays, estuaries, and coastal lagoons. The Water Quality Control Policy for Enclosed Bays and Estuaries outlines water quality principles and guidelines for protection of beneficial uses in bays and estuaries. The California Toxics Rule is a federal rule addressing priority pollutants in inland surface waters and enclosed bays and estuaries in California.

Section 303(d) of the Clean Water Act requires states to develop lists of waters with impaired water quality and to develop and implement Total Maximum Daily Loads (TMDLs), an estimate of the amount of specific pollutant(s) that water body can receive while meeting water quality objectives. Section 303(d) listings are in place for bacteria in the Ventura Harbor: Ventura Keys and for DDT/PCBs in tissue on Ventura Harbor Jetties. Section 303(d) listings on the Santa Clara River include ChemA, coliform bacteria, nitrate-nitrogen, toxaphene, and toxicity. A TMDL is in place for bacteria in the Santa Clara River, including the estuary, and a LARWQCB Order (R4-2010-0816) is in place to implement a TMDL for toxaphene in fish tissue in the Santa Clara River Estuary. The Order includes requirements for water, sediment and fish tissue monitoring for toxaphene, chlordane, and dieldrin in the Santa Clara River estuary and its subwatershed.

Limited data are available on water quality in Arundell Barranca channel and the harbor. Cotton, Shires, and Associates (1999) present a summary of data collected on Arundell Barranca in 1998 and 1999 on five dates during dry weather and two during storm events. Average nitrate (12 mg/l as nitrogen) and ammonia (2.6 mg/l as nitrogen) concentrations were high during dry weather and low during storm events. A complete list of dissolved constituents was not provided in the report, but total dissolved solids (TDS) concentrations during dry weather were high (9,650 mg/l). The high nitrate, ammonia, and TDS concentrations are consistent with runoff and return seepage from agricultural lands. The Cotton, Shires and Associates data also indicates high total coliform bacteria concentrations, most notably in the storm samples (MPN 766,351 per 100 ml in storm samples and 17, 533 in dry weather samples). Organic compounds, including US EPA priority pollutants, were also sampled. No organic constituents were found in detectable concentrations during dry weather, but several were measured in the storm samples. Several of the organic compounds were pesticides/herbicides, also consistent with the agricultural land use.

The City of Ventura collected data on bacteria from 2002 to 2009 in Ventura Keys, Ventura Harbor, and Arundell Barranca. Samples were analyzed for fecal coliform, total coliform and *Enterococcus* bacteria.

In a letter dated 6 July 2010 the City (City of Ventura, 2010) transmitted the City's data and data collected by Ventura County Department of Environmental Health to the LARWQCB and requested that the listing for bacteria impairment be removed. Over 5000 individual samples are included in this data set, and are under review by the LARWQCB (pers.comm., Ray Olson, 2012).

The District conducted monthly water quality sampling from June to October 2011 (5 sampling times) at locations on Arundell Barranca just downstream of the Union Pacific Railroad Bridge (UPRR) and just upstream of the Harbor Boulevard Bridge (VCWPD, 2012). Flow rates in the sampling period were estimated at 0.5 to 2.0 cfs. The samples were analyzed for 235 constituents and the October sample was used in chronic sea urchin and algal toxicity tests. The results of this sampling show that the majority of constituents had concentrations below method detection or reporting limits, and a large fraction had concentrations above method detection limits but below water quality objectives. However, several water quality objectives were exceeded in one or more samples at both sampling stations. These include bacteria, TDS, chloride, sulfate, total and nitrate nitrogen, dissolved and total copper, total nickel, total selenium, and total zinc. Table 3-6, reproduced from the report, summarizes the constituents that exceeded water quality objectives.

Comparison of the samples at the two sampling stations suggests that dilution occurred for some constituents between the two stations due to agricultural return flows. Higher concentrations of nitrate and total nitrogen at the Harbor Boulevard site indicate that the agricultural land is a probable source of this constituent.

The 2011 data is reasonably consistent with the data summary presented in the Cotton, Shires and Associates report. With respect to Section 303(d) listings, Arundell Barranca may be a potential source of bacteria to the Ventura Harbor. Concentrations of DDT isomers and PCB congeners were below detection limits. Arundell Barranca is not tributary to the Santa Clara River Estuary, but concentrations of toxaphene and pesticides comprising the Chem A group were below detection limits, and no toxicity was observed in the sea urchin or algal chronic toxicity tests. Concentrations of bacteria and nitrate and total nitrogen exceeded water quality objectives, and would be a concern for discharge to the Santa Clara River.

The District conducted additional monitoring in June to October 2012 (Table 3-7), collecting grab samples at the same locations sampled in 2011. Samples were analyzed for 39 constituents, reduced from the previous year's monitoring based on the numerous constituent concentrations below method detection or reporting limits in the previous year. Two samples were also obtained from the Arundell Detention Basin effluent. Flow rates during monitoring were estimated at 0.33 to 1.2 cfs. Some constituents had concentrations consistently below method detection or reporting limits and below regulatory objective levels: oil and grease, some metals (total and dissolved mercury, total and dissolved silver, total and dissolved zinc), and total chlorine residual. Water quality objectives were frequently exceeded at both sampling stations for indicator bacteria, total selenium, total copper, nitrate and total nitrogen, total dissolved solids, chloride, and sulfate. In addition, frequent exceedances of pH standards were observed at Harbor Boulevard. Exceedances for dissolved copper, total nickel and total zinc were observed in 2011 but not in 2012. The exceedances suggest that coliform bacteria and nitrate-nitrogen concentrations in Arundell Barranca may exceed future TMDL water quality objectives for the Santa Clara River Estuary and Ventura Harbor: Ventura Keys (VCWPD, 2012). Total copper, nitrate, and total dissolved solids concentrations were lower than in the 2011 monitoring. Comparison of data from the two sampling sites again showed that some constituents are probably diluted by seepage or return flows from agricultural lands, but that nitrate nitrogen concentrations increase, indicating that agricultural lands are a potential source of this constituent. However, nitrate concentrations measured at the detention basin (up to 24 mg/l), were similar to or higher than the two channel sampling sites, indicating potentially high background levels from the upper watershed. Similarly, concentrations of total copper and total dissolved solids at the detention basin indicate that they are similar to those in the channel in the project area.

The District's report compared measured concentrations to effluent standards for the Ventura Water Reclamation Facility and identified several constituents that exceed effluent limitations (nitrate, bacteria, total copper), but these were not compared to typical influent sewage concentrations, which may greatly exceed the measured values.

Stillwater Sciences (2011) provides a compilation of water quality data collected since 1999 for the Santa Clara River Estuary and characterizes current water quality conditions. The report notes that water quality in the estuary is highly variable due to the combination of annual meteorological variability, seasonal variability, and interaction with the Ocean. Based upon observed water quality conditions, the report concludes that Basin Plan water quality objectives may be exceeded at times for ammonia, bio-stimulatory substances (including nitrate), bacteria, dissolved oxygen, and pH.

Table 3-6. Constituents exceeding water quality objectives at Arundell Avenue (UPRR) and Harbor Boulevard sampling stations. Total number of exceedances and constituent concentrations are shown for each 2011 exceedance. (Source: VCWPD, 2012)

	Constituent	Total	6/7	7/7	8/4	9/8	10/27	Applicable Standard
Arundell Ave.	*E. coli*	3	272			81,640	464	235 MPN/100 mL (Basin Plan)
	Fecal coliforms	3	1,298			90,000	900	400 MPN/100 mL (Basin Plan)
	Total coliforms	5	900	34,480	224,700	435,200	24,192	10,000 MPN/100 mL (Ocean Plan)
	Enterococci	5	32,550	124	2,005	2,005	288	104 MPN/100 mL (Ocean Plan)
	TDS	5	19,000	8,400	30,000	3,700	3,400	500 mg/L (Basin Plan)
	Chloride	5	9,600	3,100	12,000	310	310	250 mg/L (Basin Plan)
	Sulfate	5	1,900	2,000	2,000	2,500	2,500	400-500 mg/L (Basin Plan)
	Nitrogen	2				11	15	10 mg/L (Basin Plan)
	Cu (d)	3	55	56	55			29.29 μg/L (CTR)
	Cu(t)	5	92	110	55	6.3	9.4	3 μg/L (Ocean Plan)
	Ni (t)	4	18	11	55		18	5 μg/L (Ocean Plan)
	Se (t)	5	8.8	7.8	6.2	5.3	16	5 μg/L (CTR)
	Zn (t)	2	53		40			20 μg/L (Ocean Plan)
	NO_3-N	1					15	10 mg/L (Basin Plan)
Harbor Blvd.	*E. coli*	3	350			4,611	278	235 MPN/100 mL (Basin Plan)
	Fecal coliforms	1				24,000		400 MPN/100 mL (Basin Plan)
	Total coliforms	4	27,550		29,090	241,920	12,997	10,000 MPN/100 mL (Ocean Plan)
	Enterococci	3	1,184			831	124	104 MPN/100 mL (Ocean Plan)
	TDS	5	3,900	7,100	9,000	3,200	2,900	500 mg/L (Basin Plan)
	Chloride	4	530	2,300	9,300	260		250 mg/L (Basin Plan)
	Sulfate	5	1,500	2,200	1,500	1,800	2,100	400-500 mg/L (Basin Plan)
	Nitrogen	5	14	20	17	20	23	10 mg/L (Basin Plan)
	Cu (d)	1		42				29.29 μg/L (CTR)
	Cu(t)	5	31	91	26	6.1	7.3	3 μg/L (Ocean Plan)
	Ni (t)	3	6.5	13			14	5 μg/L (Ocean Plan)
	Se (t)	3		5.6		5.2	11	5 μg/L (CTR)
	NO_3-N	5	13	17	16	20	22	10 mg/L (Basin Plan)
	pH	1		8.68				6.5 – 8.5 pH units (Basin Plan)
	F^-	1				4.9		1.4 mg/L (Basin Plan)

Table 3-7. Constituents exceeding water quality objectives at Arundell Ave. and Harbor Blvd. sampling stations. Total number of exceedances are shown for 2011 and 2012, and constituent concentrations are shown for each 2012 exceedance. (Source: VCWPD 2012)

Location	Constituent	Total 2011	Total 2012	6/5	7/3	8/2	9/4	10/2	Applicable Standard
Arundell Ave.	*E. coli*	3	4	1,450	905		771	1,624	235 MPN/100 mL (Basin Plan)
	Fecal coliforms	3	5	3,500	1,700	500	2,200	5,000	400 MPN/100 mL (Basin Plan)
	Total coliforms	5	5	49,500	1,119,900	488,400	222,200	95,900	10,000 MPN/100 mL (Ocean Plan)
	Enterococci	5	5	728	2,700	818	1,421	3,448	104 MPN/100 mL (Ocean Plan)
	Cu(t)	5	5	3.2	4.1	3.5	3.2	5.5	3 µg/L (Ocean Plan)
	Se (t)	5	5	10	11	6.9	6.9	6.1	5 µg/L (CTR)
	NO_3-N	1	3	12	14		12		10 mg/L (Basin Plan)
	Nitrogen	2	3	12	14		12		10 mg/L (Basin Plan)
	TDS	5	5	3,100	4,900	2,500	4,200	3,800	500 mg/L (Basin Plan)
	Chloride	5	3	260	330		280		250 mg/L (Basin Plan)
	Sulfate	5	5	2,300	3,000	1,800	2,300	1,800	400-500 mg/L (Basin Plan)
	Cu (d)	3	0						29.29 µg/L (CTR)
	Ni (t)	4	0						5 µg/L (Ocean Plan)
	Zn (t)	2	0						20 µg/L (Ocean Plan)
Harbor Blvd.	*E. coli*	3	4	249	327	426		480	235 MPN/100 mL (Basin Plan)
	Fecal coliforms	1	4	500	500	700		3,000	400 MPN/100 mL (Basin Plan)
	Total coliforms	4	5	129,100	1,299,700	686,700	344,800	88,000	10,000 MPN/100 mL (Ocean Plan)
	Enterococci	3	4		1,012	206	262	1,187	104 MPN/100 mL (Ocean Plan)
	Cu(t)	5	4		3.9	3.3	3.3	4.9	3 µg/L (Ocean Plan)
	Se (t)	3	4		8.2	6.5	7.4	6.2	5 µg/L (CTR)
	NO_3-N	5	3		19	17	16		10 mg/L (Basin Plan)
	Nitrogen	5	3		20	17	16		10 mg/L (Basin Plan)
	TDS	5	5	1,900	4,300	2,300	5,200	3,200	500 mg/L (Basin Plan)
	Chloride	4	3		290		310	270	250 mg/L (Basin Plan)
	Sulfate	5	5	1,400	2,600	1,600	2,600	2,400	400-500 mg/L (Basin Plan)
	pH	1	3			8.56	8.54	8.57	6.5 – 8.5 pH units (Basin Plan)
	Cu (d)	1	0						29.29 µg/L (CTR)
	Ni (t)	3	0						5 µg/L (Ocean Plan)
	Fluoride	1	0						1.4 mg/L (Basin Plan)

3.6 Maintenance

The Arundell Barranca channel system is maintained by the District from the Arundell Dam to the Ventura Harbor. Maintenance activities include annual inspections of the channel structures, flushing of weep holes and subdrains, removal of trash and debris, removal of sediment, grading and weed removal on the service roads and cross ditches, and maintenance of fencing. The District reports (pers. comm., John Lagomarsino, 2012) that significant debris and sediment removal in the concrete portions of the channel is primarily related to large storm events, and that the channel is largely self-cleaning during normal operations. When sediment removal is necessary in the channels, it is generally removed by lowering a small loader into the channel and pushing material to a point where it can be removed with an excavator and truck at the top of the bank.

Current maintenance difficulties are largely related to tree fall and breakage during storm events. The most significant vegetation management difficulty is currently in the channel section between Foothill Road and Arundell Barranca Dam. Debris that enters the channel has the potential to catch on structures and cause significant flooding, and the most significant choke point in the existing system is believed to be the UPRR Bridge and the Market Street Bridge upstream. Debris was removed from the UPRR Bridge following the 1998 and 2005 events. A few significant repairs in the channel system have been made over the past decade, including crack/settlement repair of the box culvert at Estates Avenue and revetment of eroded channel slopes near Loma Vista Road.

At the outlet of Arundell Barranca to the harbor, sediment has been removed from the energy dissipator in the past using a crane and clamshell bucket, but removal of sediments in this location is apparently very rare. The District also removes sediment from Arundell Barranca Dam following major events. Removal of sediments is difficult due to clayey materials and high moisture content. The vegetated area downstream of the dam produces backwater on the primary outlet, and causes deposition in this area that is difficult to remove.

Maintenance of the harbor is the responsibility of the Ventura Port District. The Port District boundary in the Stub Channel is approximately on the northerly side of the Arundell Barranca outlet channel. Northeast of this boundary, the City of Ventura is responsible for maintenance in the Ventura Keys. The Port District conducts regular reconnaissance and informal soundings of the navigation channels in the harbor to determine dredging needs (pers. comm. Richard Parsons, 2012b). When shoaling or sediment accumulation becomes significant, the Port District conducts pre-dredging bathymetric surveys and provides pre-dredging reports to the US Army Corps of Engineers, Coastal Commission, State Lands Commission, and LARWQCB under permit requirements. Dredging is conducted on an as-needed basis, and large dredging volumes in the vicinity of Arundell Barranca often correspond to large storm events (see Section 3 for a summary of dredging volumes). Dredging of the Stub Channel is coordinated to the extent feasible with City of Ventura dredging in Ventura Keys. Post-dredging surveys are conducted to complete the reporting for the applicable permits and to quantify the amount of material removed. Dredging is generally done with large hydraulic dredging equipment, and material is disposed of near the mouth of the Santa Clara River. Dredging is conducted between October and March to comply with permit requirements for a minimum flow of 100 cfs in the Santa Clara River and to protect fish and wildlife. Sampling of harbor sediments is done on a regular basis to comply with permit requirements. Dredging is done most regularly at the confluence of Arundell Barranca and the Stub Channel. Dredging in Ventura Keys and in other areas of the harbor is less frequent. Dredging in the marinas and berthing areas is very infrequent, and the responsibility of the marina operators. The Ventura Port District reports that the volume of large sediment (large gravel and cobbles) delivered to the harbor constitutes a low fraction of the total, but that this material is particularly problematic in hydraulic dredging operations.

The Corps of Engineers conducts dredging of the channel entrance including dredging of a depressed sand trap area, and advance maintenance (dredging below the navigation depth) in some other areas of the entrance channel. Material dredged from the harbor entrance is coarse-grained and is disposed of on the beaches to the south of the harbor entrance.

The Port District also conducts occasional repairs of the rock slope protection on the harbor side slopes. Repairs to rock slope protection on the Arundell Barranca outlet and Stub Channel were made at some time in the past, but no work has been required in this area for over a decade.

4. ALTERNATIVES EVALUATION

The primary purpose of the channel improvement project is to provide protection against a 100-year flood and remove properties from FEMA Flood Hazard Zone A. Comments received on the initial study and during the scoping meeting for the project raised concerns about the increased delivery of sediment and pollutants to the Harbor and potential erosion or damage in the Harbor channels due to increased velocities. Comments suggested a diversion to the Santa Clara River, development of a project with benefits in addition to increased flood capacity (e.g., water quality enhancement, groundwater recharge), use of detention facilities on farm land, construction of treatment wetlands, and other conceptual changes. Based on this stakeholder review, the District has developed nine alternatives to assess their feasibility and effectiveness in addressing stakeholder comments. The following sections describe the process of developing and evaluating alternatives.

4.1 Alternatives Development

Nine alternatives were developed in sufficient detail to assess their hydraulic design, feasibility, cost, and potential environmental effects.

4.1.1 Hydraulic Design

All alternatives were developed to provide protection against a 100-year flood event. LiDAR mapping (VCWPD, 2005) of the project site and utility information were used to establish the general plan and profile layout of the proposed facilities for each alternative. Hydraulic calculations were performed to determine appropriate dimensions of the facilities and necessary hydraulic parameters such as bed slope and hydraulic roughness. HEC-RAS models were prepared for several alternatives that include hydraulically complex facilities.

4.1.2 Feasibility

Based on the general layout, dimensions, and type of the hydraulic facilities for each alternative, feasibility considerations such as the potential for conflict with major utilities, necessary changes to other infrastructure, and required land acquisition were assessed. In many cases these considerations resulted in modification of the preliminary alignments or profiles to minimize conflicts and reduce potential impediments to implementation. The major utility lines that are known to be present in the project area are summarized in Table 4-1. Some alternatives included project alignments that resulted in inevitable utility conflicts requiring alteration or relocation of utility pipeline(s).

Table 4-1. Major utilities presenting potential conflicts with alternatives

Utility	Location Within Project Area
36” Harbor Trunk Sewer	Runs along west side of Harbor Boulevard
33” Woolsey Trunk Sewer	Runs south-east across the farmland bounded by existing channel, Harbor Boulevard, Olivas Park Drive, and UPRR
30” Olivas Park Trunk Sewer	Runs along the toe of grade break just south of Olivas Park Drive, turns north, continues along Palma Street
22” Venoco Oil Line	Crosses Stub Channel, runs along south side of existing channel between Beachmont and Harbor, turns south, continues along Harbor Boulevard
12”water	Crosses Stub Channel
12” sewer	Crosses Stub Channel
6” reclaimed water	Crosses Stub Channel
High pressure gas	Runs along Harbor Blvd.

Right-of-way acquisition is needed for most alternatives. Due to the preliminary nature of the alternatives, willingness of landowners to sell right-of-way was not addressed in the preliminary assessment[1]. The costs for right-of-way acquisition have been estimated based on typical values in the area and are included as part of the alternative costs, but the level of land acquisition required may also affect the feasibility of a particular alternative or the time required for implementation.

Effects on other infrastructure such as bridges were also assessed, as these may require coordination with other agencies and have temporary effects on transportation during construction. In addition, significant construction constraints and challenges were identified.

4.1.3 Cost

Conceptual level construction cost estimates were prepared for each alternative based on unit costs developed from previous District projects, experience on other projects, and cost estimating guides (Means, 2010). Quantities were derived from the preliminary plan and profile layouts, and should be considered approximate. Costs are based on June 2012 cost levels and should be escalated for time of construction. A 15% contingency was added to all construction cost estimates. No provision for design or construction administration is included in the estimates.

Maintenance cost estimates were based on estimates of types and frequencies of maintenance activities associated with each alternative and information on District maintenance costs for activities such as sediment removal. Yearly maintenance costs were converted to a present value using an interest rate of 5 percent to facilitate combination with construction and land costs.

Right-of-way acquisition for the alternatives generally involves acquisition of farm land. The cost of acquisition was estimated at $125,000 per acre, and an additional $10,000 per acre (VCWPD, 2012) was assumed for purchase of development rights to mitigate the loss of productive coastal agricultural land.

[1] Since initial development of alternatives the District has contacted the landowner adjacent to the channel in the reach between Harbor Boulevard and the UPRR Bridge. The landowner has informally indicated an unwillingness to sell the property for channel widening or detention/wetland facilities.

Where acquisition of a portion of a parcel may affect the ease of farming on the remainder of the parcel, a severance cost of 50% of the land value (i.e., $62,500) was assumed. Land costs for developed land, where applicable, were estimated by the District based on experience and typical values. The quantity of land to be acquired was increased in some cases to avoid creation of unusable or highly constrained remainder parcels.

4.1.4 Environmental Effects

The alternatives generally are conceived to provide secondary benefits such as improvements in water quality, reduced sediment delivery to the Harbor, and opportunities for riparian corridors or wetlands. Several alternatives also may provide increased opportunity for recreational or transportation facilities such as bike trails. Benefits and disadvantages were identified qualitatively for each alternative and, where feasible, initial quantitative estimates of potential benefits were made. Environmental effects vary widely among alternatives, and benefits such as water quality improvement vary by season, location, type of pollutant, and other characteristics.

4.2 Alternatives Description

Each alternative is briefly described below, including hydraulic design, feasibility considerations, maintenance needs, environmental effects, land and right-of-way acquisition requirements, utilities and infrastructure, maintenance considerations, and cost. Key uncertainties regarding the performance or feasibility of the alternatives are also listed. Figures 4-1 through 4-10 provide preliminary plans of the alternatives.

<u>Alternative 1 – Enlarged Arundell Barranca Channel from Ventura Harbor to Harbor Boulevard</u>
Concept: Enlarged channel and bridges from Ventura Harbor to Harbor Boulevard using existing channel alignment. The existing bridges at Harbor Boulevard and Beachmont Street would be replaced, and a new energy dissipator would be constructed at the mouth of the channel in the harbor.
Hydraulic Design:

- Design Q100= 7,500 cfs
- Supercritical channel, 32 feet wide at S=0.006, D=9.3 feet and V=28 fps at Q100

Feasibility Considerations:

- Right-of-Way **–** Construction in existing right-of-way
- Utilities – Requires crossing high pressure gas line in Harbor Boulevard; oil line constrains width on south side of channel; encroachments in existing easement downstream of Beachmont Street would need to be removed/modified.
- Infrastructure – Requires modifying/replacing bridges at Harbor Boulevard and Beachmont Street
- Construction uses existing channel as form on bottom and one side

Maintenance:

- Similar level and type of maintenance compared to existing system for most flow conditions
- Maintenance road would be switched to southerly side of channel
- Reduced damage and repair following severe flood events

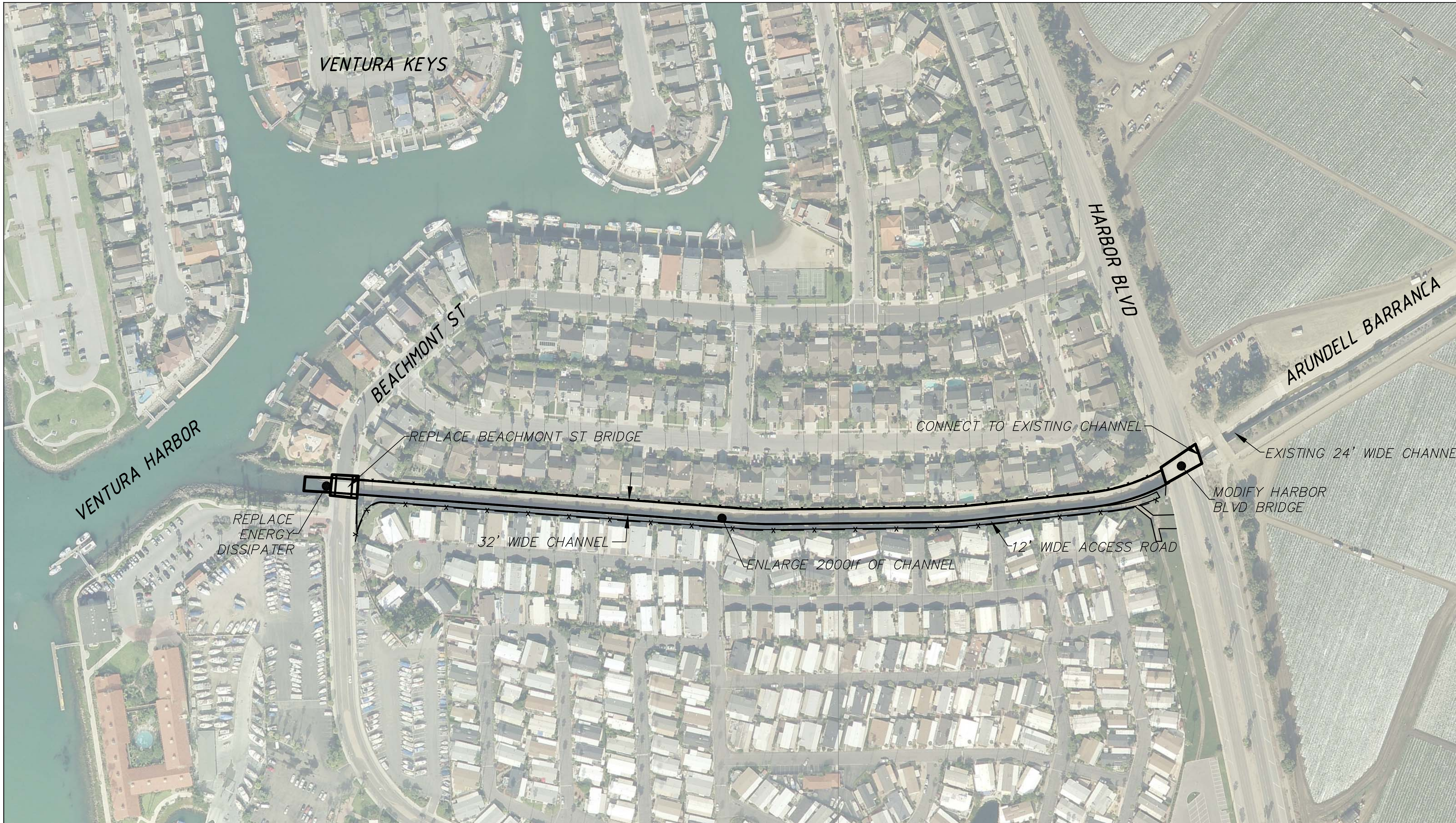
Environmental Benefits:

- No new land required - uses existing channel alignment
- Reduces delivery of pollutants by overland flow to Harbor during extreme (e.g., greater than 10-year) events by preventing flooding of farmland and the market Street industrial area. Currently, these floodwaters drain into Arundell Barranca after the flood peak passes, potentially contributing more agricultural and industrial pollutants than contained in the channel

Environmental Disadvantages:
- Potential for increased velocities in Harbor channels and short-term delivery of sediment to Stub Channel in extreme events
- Potential for increased erosion, damage to property, and disruption of navigation

Approximate Quantities and Costs:
- Channel – 2000 lineal feet of enlarged channel
- Harbor Boulevard Bridge –Construct new channel under westerly portion
- Beachmont Street Bridge – Replace bridge with wider span
- Construction Cost Estimate - $9.9M
- Maintenance Cost Present Value Estimate – $0.5M
- Land Cost - $0
- Total present Value Cost - $10.5M

Uncertainties/Analysis Needed:
- Existing energy dissipator was designed with physical model; new energy dissipator may require advanced numerical or physical modeling to support hydraulic design
- Existing sediment delivery to Harbor during extreme events difficult to quantify and may be highly variable with event



Ventura County Watershed Protection District

nhc northwest hydraulic consultants
80 south lake avenue, suite 800
pasadena, california 91101
phone: (626) 440-0080
fax: (626) 440-1881

Job:
Rev:
Drft: tvs
Date: 11Jul12

Enlarged Channel On Existing Alignment
Ventura Harbor to Harbor Blvd

Figure 4-1. Alternative 1

<u>Alternative 2C – Complete Diversion to Santa Clara River – TNC Property Alignment</u>

Concept: The entire Arundell Barranca flow would be routed to the Santa Clara River approximately 5,000 feet upstream of the Harbor Boulevard Bridge by intercepting the existing channel near the RR Bridge and realigning the high speed channel, constructing a coarse sediment trap, passing through agricultural land, crossing Olivas Park Drive, and passing through TNC property east of the golf course to the Santa Clara River. As part of this alternative, a wetland treatment system would be constructed to treat most of the summer flows to reduce potential pollutant delivery to the Santa Clara River during the dry season.

Hydraulic Design:

- Design Q= 7,500 cfs
- Realignment of approximately 700 feet of supercritical channel at existing slope, depth and width
- Requires stilling basin type energy dissipator at transition between supercritical and subcritical channel
- Requires coarse sediment trap at upstream transition to sub-critical channel
- Subcritical channel, stepped main channel/floodplain design, 4,800 feet long with 80 feet bottom width and approximately 340 to 380 feet top width, 10 to 16 feet deep, at S=0.002; d=4.9 feet, and v=4.3 fps at Q2; d=8.6 feet and v=6.5 fps at Q100
- Slope break (S=0.004) in channel approximately 800 feet upstream of Olivas Park Drive; d=3.5 feet, and v=6.1 fps at Q2; d=7.4 feet and v=8.3 fps at Q100. Higher slope may require channel lining or grade stabilization for approximately 1,500 feet of channel.
- Trapezoidal low flow channel through proposed wetland area, 2,270 ft long and 80 feet bottom width with approximately 100 foot top width, 5 feet deep; at S= 0.004; d=4.5 feet; v=4.7 fps; overflows to floodplain and wetlands (300- 500 feet wide) at flows higher than Q2
- Target design capacity of wetlands is 50 cfs with a one day retention time
- Excavation to daylight tributary channel in Santa Clara River channel
- Large excavation volume near Olivas Park Drive due to flat ground
- Relocation of farm bridge downstream of UPRR

Feasibility Considerations:

- Right-of-Way **–** Requires substantial acquisition of private agricultural land (68 acres)
- Utilities – Alignment crosses two 30-inch gravity sewer trunk lines. For the Woolsey trunk, approximately 1200 feet of sewer trunk must be relocated to remain west of realigned Arundell Barranca.
- Infrastructure – New bridge required on Olivas Park Drive, probable relocation of farm road bridge downstream of UPRR
- Large excavation volume (higher depth) near Olivas Park Drive due to flat ground and drop into river floodplain
- Uncertain whether discharge point to SCR estuary could be permitted by RWQCB due to existing 303(d) listing for nitrogen/nitrate and TMDL for bacteria
- Uncertain whether new discharge point could be authorized under Endangered Species Act

Maintenance:

- Approximately 7,500 feet of new natural bed channel
- Approximately 4,500 feet of concrete channel eliminated
- Subcritical channel may not be as self-cleaning as existing channel
- Coarse sediment trap will need to be periodically cleaned (estimated once per 5 years on average)
- Vegetated channel will require higher annual maintenance than existing concrete channel, including selective vegetation removal
- Wetland/bio-retention treatment system will require maintenance and frequent monitoring
- Dredging requirements in harbor stub channel significantly reduced (estimated 24,000 to 35,000 cy/year as an annual average)

Environmental Benefits:

- Eliminates discharge to harbor, potentially improving water quality, enhancing navigability, and protecting the opposite bank from erosive flows
- Restores more natural sediment supply to ocean
- Reduces dredge disposal requirements due to reduced Harbor dredge volumes
- Provides opportunity for more natural channel and riparian corridor
- Provides opportunity for enhanced wetland and riparian habitat in Santa Clara River floodplain
- Potentially increases freshwater supply to SCR estuary

Environmental Disadvantages:

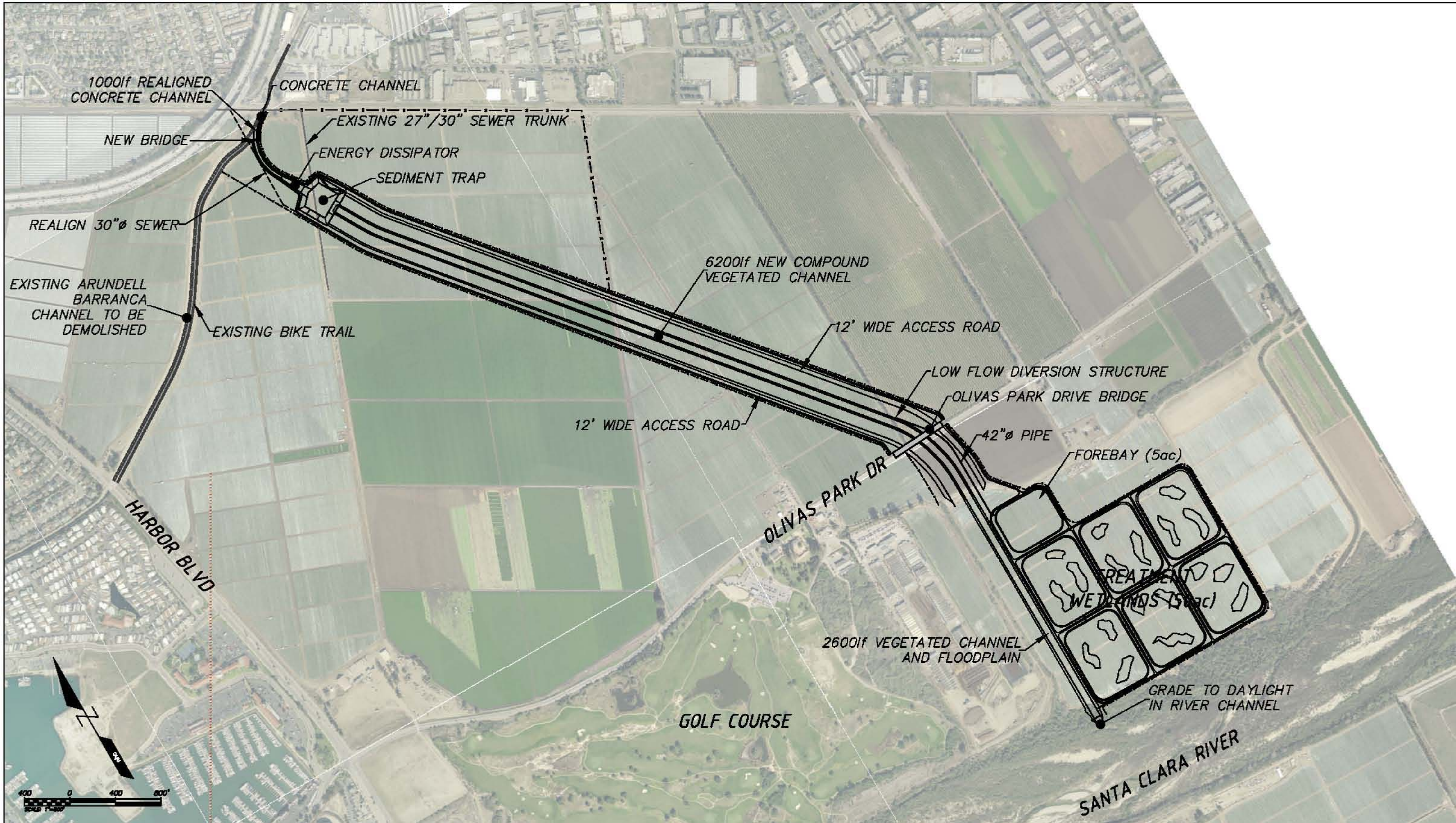
- Potentially delivers urban pollutants and high temperature water to Santa Clara River estuary (mitigated during low flow conditions by wetland/bio-retention treatment)
- Potential attraction flows for endangered steelhead; may require barrier
- May interfere with steelhead ability to reach natal streams using olfactory cues
- New source of freshwater input to estuary may cause it to breach more frequently, with potential adverse effects to endangered tidewater goby, other estuarine fish populations, and endangered California least tern
- During high flow events, tidewater gobies may migrate from SCR to Arundell Barranca and adjacent treatment wetlands, with potential for stranding or mortality
- Probable temporary and permanent impacts on riparian vegetation at connection to SCR channel
- Potential effects on flood inundation for SCR – unlikely to be significant but requires investigation
- Potential short- and long-term adverse effects to threatened, endangered, or other sensitive species (e.g., least Bell's vireo, southwestern willow flycatcher, tidewater goby, etc.)
- Loss of productive coastal farmland
- Conflict with terms of grant that funded TNC's property acquisition as well as TNC's intended use of their property
- Replaces occasional dredging with more extensive annual channel maintenance

Approximate Quantities and Costs:

- Channel construction – 700 feet realigned concrete channel
- Energy dissipator and sediment trap – 550 feet
- Natural bed channel construction – 7,500 feet
- Olivas Park Drive Bridge - 10,000 sf
- Wetland/Bio-retention Treatment - 50 acres, 160,000 cy excavation
- Construction Cost – $32.7M
- Maintenance Cost Present Value Estimate – $3.2M
- Land Cost – $14.2M (does not include acquisition of TNC parcels or purchase of replacement property for restoration)
- Total Present Value Cost - $50.0M

Uncertainties/Analysis Needed:

- Sediment transport in transition and channel
- Coincident flood design criteria for Santa Clara River and Arundell Barranca
- Treatment wetland design for specific pollutants
- Hydraulic and environmental design of confluence with Santa Clara River



Ventura County Watershed Protection District

nhc northwest hydraulic consultants
80 south lake avenue, suite 800
pasadena, california 91101
phone: (626) 440-0080
fax: (626) 440-1881

Job:600001
Rev:
Drft: tvs
Date: 19Jul12

Complete Diversion to Santa Clara River
TNC Property

Figure 4-2. Alternative 2C

Alternative 3 – Existing Channel with High Flow Diversion to Off-Channel Retention Basin

Concept: The existing channel from Ventura Harbor to Harbor Boulevard would be retained and a high flow diversion would be constructed for flows in excess of the existing channel capacity. The high flows would be routed to a retention basin sized to provide adequate storage for the 100-year design hydrograph. Stored water would eventually be released or infiltrated.

Hydraulic Design:

- Existing channel capacity taken as approximately 5,400 cfs; lateral weir diversion of 2,100 cfs required to provide capacity for Q100=7,500 cfs
- Design Q= 2,100 cfs (diversion)
- Improvements to Harbor Boulevard Bridge/channel likely required to achieve 5,400 cfs capacity
- Lateral weir estimated at 1,200 feet long, must be designed to function with supercritical flow in channel
- Grouted rock collector channel, 1,200 feet long, 30 to 50 feet bottom width with 60 to 80 feet top width, 8 feet deep
- Multiple cell retention ponds with storage volume 115 af; to be constructed on 65 acres of agricultural land east of Harbor Boulevard

Feasibility Considerations:

- Right-of-Way **–** Requires acquisition of private agricultural land for retention basin (65 acres).
- Utilities – No identified conflicts
- Infrastructure – Channel/bridge improvements required at Harbor Boulevard

Maintenance:

- New retention basin and lateral weir will require periodic maintenance (VCWPD)
- Reduced dredging at Harbor (Port District and City) due to controlled overflow in large events

Environmental Benefits:

- Reduces delivery of pollutants by overland flow to Harbor during extreme (e.g., greater than 10-year) events
- Reduces overall delivery of sediment and pollutants by trapping a portion in retention basin from extreme events
- Potential for joint use of retention basin area – e.g., recreational fields

Environmental Disadvantages:

- Loss of productive coastal farmland

Approximate Quantities and Costs:

- Lateral weir and collector channel – 1200 feet
- Retention basin – 115 af, 41 acres plus 10 acres severance area, 41 acres mitigation
- Construction Cost – $11.1M
- Maintenance Cost Present Value Estimate – $1.3M
- Land Cost - $6.7M
- Total Present Value Cost - $19.1M

Uncertainties/Analysis Needed:

- Lateral weir design for supercritical flows
- Collector channel design with variable flow rates, energy dissipation for lateral inflows
- Hydraulic improvements for Harbor Boulevard Bridge

Ventura County Watershed Protection District

nhc northwest hydraulic consultants
80 south lake avenue, suite 800
pasadena, california 91101
phone: (626) 440-0080
fax: (626) 440-1881

Job:
Rev:
Drft: tvs
Date: 17Jul12

Existing Channel High-Flow Diversion to Off-Channel Retention Basin

Figure 4-3 Alternative 3

Alternative 4 A– Existing Channel with High Flow Diversion from Downstream of UPRR to Harbor

Concept: The existing channel from Ventura Harbor to Harbor Boulevard would be retained and a high flow diversion would be constructed for flows in excess of the existing channel capacity. The overflow weir would be constructed near the UPRR Bridge to capture excess flows and the channel would be routed to Harbor Boulevard near Olivas Park Drive. An inlet structure would be constructed on the east side of Harbor Boulevard for flow to enter a set of reinforced concrete box culverts (RCBs). Because of a conflict with a sewer trunk line on the west side of Harbor Boulevard, the RCBs are sized as 6 – 4x10 culverts with their tops at grade in Navigator Drive. An energy dissipator would be constructed at the outlet to the Harbor.

Hydraulic Design:

- Existing channel capacity taken as approximately 5,400 cfs; lateral weir diversion of 2,100 cfs required to provide capacity for Q100=7,500 cfs
- Design Q= 2,100 cfs (diversion)
- Improvements to Harbor Boulevard Bridge/channel probably required to improve performance for 5,400 cfs capacity
- Lateral weir estimated at 1200 feet long, must be designed to function with supercritical flow in channel
- Grouted rock collector channel parallel to the lateral weir to convey diverted flow to subcritical channel downstream; bottom width 30 to 50 feet and top width 60 to 80 feet, depth 8 feet
- High flow diversion in subcritical channel to Harbor Boulevard near Olivas Park Drive, 4,500 feet long and 60 feet bottom width and 85 feet top width at S=0.006; d=4 feet, v=7.4 fps at Q100 (2,100 cfs diversion), requires bank and bed stabilization
- Inlet structure and reinforced concrete box culvert from upstream of Harbor Boulevard to Harbor; six 4 foot by 10 foot RCBs at S=0.004 and S=0.02, v=11.7 and 19.8 fps at Q100 (2,100 cfs diversion)
- Energy dissipator at Harbor, with pedestrian bridge, approximately 60 feet wide

Feasibility Considerations:

- Land Use – Potentially conflicts with Holiday Inn expansion. Preliminary alignment is in area of proposed building and outlet is near expanded hotel rooms.
- Right-of-Way **–** Requires acquisition of private agricultural land for high flow diversion and channel (18 acres plus 5 acres severance), and acquisition of easement in hotel parking area (0.75 acres).
- Utilities – Requires crossing high pressure gas line, oil line, 24-inch farm drain CMP, and 36-inch sewer line along Harbor Boulevard. Sewer line conflict controls grade of RCBs, and modification of approximately 60 lf of sewer trunk line assumed.
- Infrastructure – Open cut through Harbor Boulevard will be potentially difficult and disruptive; RCB tops will be surfaced for Navigator Drive; existing paths and parking area will need to be modified at harbor outlet; existing drain line may be connected to RCB
- Other – Discharge into harbor is between berthing areas (approximately 100 feet wide between docks) and discharge will cause currents (very infrequently) and potentially some increase in sediment deposition at this location

Maintenance:

- New lateral weir, channel, RCB, and harbor outlet will require periodic maintenance; RCB will be subject to inundation with tidal fluctuations (VCWPD)

Environmental Benefits:

- Reduces delivery of pollutants by overland flow to Harbor during extreme (e.g., greater than 10-year) events

Environmental Disadvantages:

- Requires disruption for construction in Harbor Boulevard, Navigator Drive, and modification of pedestrian walk
- Occasional storm discharge near berthing areas may be problematic
- Potential conflicts with future land use in hotel parking lot
- Loss of productive coastal farmland

Approximate Quantities and Costs:

- Lateral weir and collector channel – 1,200 feet
- High flow diversion channel with bed and bank stabilization – 4,500 feet; 18 acres
- Inlet structure at Harbor Boulevard – 60 feet wide with trash rack
- RCBs to Harbor – 500 feet
- Energy dissipator – 60 feet wide
- Construction Cost – $ 15.0M
- Maintenance Cost Present Value Estimate – $ 0.9M
- Land Cost – $ 3.1M
- Total Present Value Cost - $ 19.0M

Uncertainties/Analysis Needed:

- Lateral weir design for supercritical flows
- Hydraulic improvements for Harbor Boulevard Bridge
- Detailed hydraulic design of energy dissipator and evaluation of effects at Harbor outlet
- Conflict with proposed development

Ventura County Watershed Protection District

nhc northwest hydraulic consultants
80 south lake avenue, suite 800
pasadena, california 91101
phone: (626) 440–0080
fax: (626) 440–1881

Job:	
Rev:	
Drft: tvs	
Date: 17Jul12	

High-Flow Diversion Channel
Agricultural Alignment

Figure 4-4. Alternative 4A

Alternative 4 B– Existing Channel with High Flow Diversion from Upstream of Harbor Boulevard to Harbor

Concept: The existing channel from Ventura Harbor to Harbor Boulevard would be retained and a high flow diversion would be constructed for flows in excess of the existing channel capacity. The lateral weir would be constructed nearer to Harbor Boulevard than in Alternative 4a and the channel would be routed along Harbor Boulevard to a point near Olivas Park Drive and then to the Harbor. An inlet structure would be constructed on the east side of Harbor Boulevard for flow to enter a set of reinforced concrete box culverts (RCBs). Because of a conflict with a sewer trunk line on the west side of Harbor Boulevard, the RCBs are sized as 6 – 4x10 culverts with their tops at grade in Navigator Drive. An energy dissipator would be constructed at the outlet to the Harbor.

Hydraulic Design:

- Existing channel capacity taken as approximately 5,400 cfs; lateral weir diversion of 2,100 cfs required to provide capacity for Q100=7,500 cfs
- Design Q= 2,100 cfs (diversion)
- Improvements to Harbor Boulevard Bridge/channel probably required to improve performance for 5,400 cfs capacity
- Lateral weir estimated at 1200 feet long, must be designed to function with supercritical flow in channel
- Grouted rock collector channel parallel to the lateral weir to convey diverted flow to subcritical channel downstream; bottom width 30 to 50 feet with top width 60 to 80 feet, depth 8 feet
- High flow diversion in subcritical channel along Harbor Boulevard, 4,500 feet long and 60 feet bottom width and 85 feet top width at S=0.003; d=4.9 feet, v=5.8 fps at Q100 (2,100 cfs diversion), vegetated channel acceptable
- Inlet structure and reinforced concrete box culvert from upstream of Harbor Boulevard to Harbor; six 4 foot by 10 foot RCBs at S=0.004 and S=0.02, v=11.7 and 19.8 fps at Q100 (2,100 cfs diversion)
- Energy dissipator at Harbor, with pedestrian bridge, approximately 60 feet wide

Feasibility Considerations:

- Land Use - Potentially conflicts with Holiday Inn expansion. Preliminary alignment is in area of proposed building and outlet is near expanded hotel rooms.
- Right-of-Way **–** Requires acquisition of private agricultural land for high flow diversion and channel (17 acres plus 2 to 11 acres severance), and acquisition of easement in hotel parking area (0.75 acres).
- Utilities – Requires crossing high pressure gas line, oil line, 24-inch farm drain CMP, and 36-inch sewer line along Harbor Boulevard
- Infrastructure – Open cut through Harbor Boulevard will be potentially difficult and disruptive; existing paths and parking area will need to be modified at harbor outlet; existing drain line may be connected to RCB
- Other – Discharge into harbor is between berthing areas (approximately 100 feet wide between docks) and discharge will cause currents (very infrequently) and potentially some increase in sediment deposition at this location

Maintenance:

- New lateral weir, channel, RCB, and harbor outlet will require periodic maintenance; RCB will be subject to inundation with tidal fluctuations (VCWPD)

Environmental Benefits:

- Reduces delivery of pollutants by overland flow to Harbor during extreme (e.g., greater than 10-year) events
- Potential for joint use of channel – greenway

Environmental Disadvantages:

- Requires disruption for construction in Harbor Boulevard, Navigator Drive, and modification of pedestrian walk
- Occasional storm discharge near berthing areas may be problematic
- Potential conflicts with future land use in hotel parking lot
- Potential conflicts with future use along Harbor Boulevard
- Loss of productive coastal farmland

Approximate Quantities and Costs:

- Lateral weir and collector channel – 1,200 feet
- High flow diversion vegetated channel - 2,950 feet; 17 acres
- Inlet structure at Harbor Boulevard – 60 feet wide with trash rack
- RCBs to Harbor – 500 feet
- Energy dissipator – 60 feet wide with pedestrian bridge
- Construction Cost – $ 12.0M
- Maintenance Cost Present Value Estimate – $ 1.4M
- Land Cost - $ 3.0M
- Total Present Value Cost - $ 16.4M

Uncertainties/Analysis Needed:

- Lateral weir design for supercritical flows
- Hydraulic improvements for Harbor Boulevard Bridge
- Detailed hydraulic design of energy dissipator and evaluation of effects at Harbor outlet
- Potential for joint use of channel area
- Conflict with proposed development

Ventura County Watershed Protection District	nhc northwest hydraulic consultants 80 south lake avenue, suite 800 pasadena, california 91101 phone: (626) 440–0080 fax: (626) 440–1881	Job:600001 Rev: Drft: tvs Date: 17Jul12	High-Flow Diversion Channel Harbor Boulevard Alignment

Figure 4-5. Alternative 4B

Alternative 5 – Alternative 1 with Low Flow Treatment Wetlands
Concept: The existing channel would be enlarged and bridges modified or replaced from Ventura Harbor to Harbor Boulevard as for Alternative 1, plus a low flow treatment wetland or bio-retention area would be constructed along the channel alignment east of Harbor Boulevard. The treatment facility would be sized to treat low flows, and would primarily intercept urban flows during the summer, the leading edge of runoff events, and a small portion of larger runoff events. The treated flows would be returned to the Arundell Barranca channel upstream of Harbor Boulevard.
Hydraulic Design:

- Design Q100= 7,500 cfs
- Channel construction downstream of Harbor Boulevard as for Alternative 1 - supercritical channel, 32 feet wide at S=0.006, D=9.3 feet and V=28 fps at Q100
- Gravity diversion of low flows using an intake grate and sump in the bed of the channel leading to an 18-inch pipe
- Target design capacity of wetlands/bio-retention is 5 cfs with a 24-hour retention time
- Wetland/bio-retention in multiple cells to limit excavation and berms; may be a combination of ponds/channel to treat specific pollutants of concern (treatment train)

Feasibility Considerations:

- Right-of-Way **–** Requires acquisition of private agricultural land for treatment system (10 acres)
- Utilities – Same as Alternative 1
- Infrastructure – Same as Alternative 1

Maintenance:

- Low flow diversion and wetland treatment system will require new maintenance activities (VCWPD or City)

Environmental Benefits:

- Potentially improves water quality and reduces total pollutant discharge to Harbor, focusing on summer flows
- Provides potential opportunity for riparian/wetland corridor along a portion of the existing bike trail

Environmental Disadvantages:

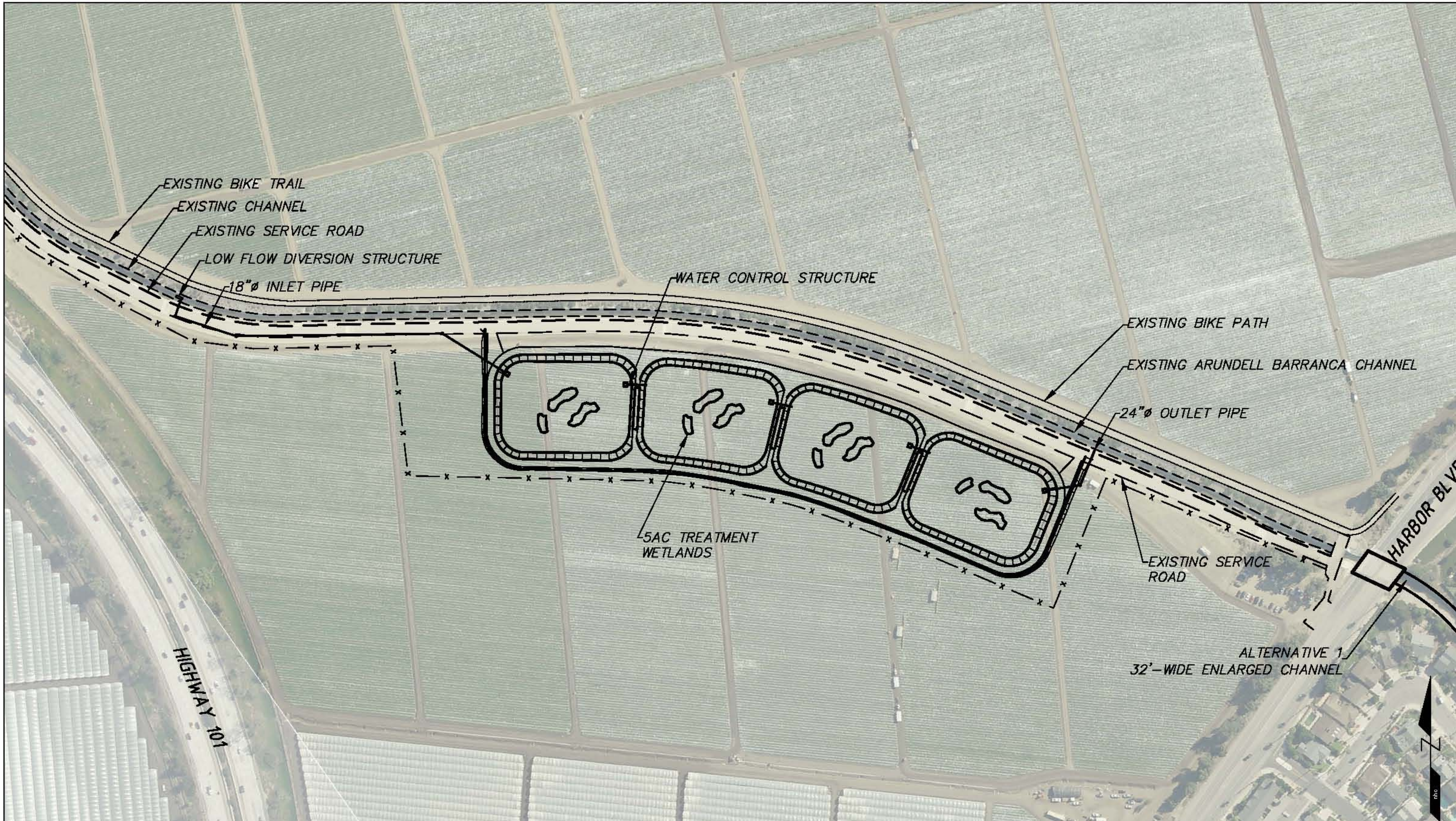
- Loss of productive coastal farmland

Approximate Quantities and Costs:

- Concrete channel and bridges – Same as for Alternative 1
- Treatment wetlands – approximately 10 acres, 40,000 CY
- Construction Cost – $ 12.1M
- Maintenance Cost Present Value Estimate – $ 2.9M
- Land Cost - $ 1.5M
- Total Present Value Cost - $ 16.5M

Uncertainties/Analysis Needed:

- Wetland/Bio-retention treatment design to meet water quality objectives in Ventura Harbor/Ventura Keys



Ventura County Watershed Protection District	nhc northwest hydraulic consultants	80 south lake avenue, suite 800 pasadena, california 91101 phone: (626) 440-0080 fax: (626) 440-1881	Job:600001 Rev: Drft: tvs Date:18Jul12	Alternative 1 With Low-Flow Treatment Wetlands

Figure 4-6. Alternative 5

Alternative 6 – Alternative 1 with Inline Sediment Trap

Concept: The existing channel and bridges downstream of Harbor Boulevard would be enlarged as for Alternative 1, and a coarse sediment trap would be constructed upstream of Harbor Boulevard to minimize delivery of gravel and cobble bed materials to the Harbor. The sediment trap would require an energy dissipator at the upstream end to transition from supercritical to subcritical flow, and a transition for acceleration back to supercritical flow at the downstream end.

Hydraulic Design:

- Design Q100= 7,500 cfs
- Channel construction downstream of Harbor Boulevard as for Alternative 1 - supercritical channel, 32 feet wide at S=0.006, D=9.3 feet and V=28 fps at Q100
- 1100 feet long, 250 feet wide sediment trap; approximately 13 feet deep with levee
- 200 feet of transition section with drop structure from existing supercritical channel to sediment trap
- 500 feet of transition section from sediment trap to Alternative 1 enlarged channel

Feasibility Considerations:

- Right-of-Way **–** Requires acquisition of private agricultural land for the sediment trap (12 acres) along the existing channel alignment.
- Utilities – Same as for Alternative 1
- Infrastructure – Same as for Alternative 1

Maintenance:

- Periodic removal of sediments in sediment trap (VCWPD)
- Reduced dredge quantities in Harbor (Port District and City)and eliminates difficulty with large material
- Increased maintenance for levee/floodwall (see Uncertainties Section below)

Environmental Benefits:

- Reduces the need for dredging in an aquatic environment (Harbor)

Environmental Disadvantages:

- Downstream end of sediment trap will likely be elevated above adjacent ground to provide the required transition back to supercritical flow; may be some aesthetic impact, and would be subject to levee restrictions (no planting of trees on levee or toe)
- Loss of productive coastal farmland

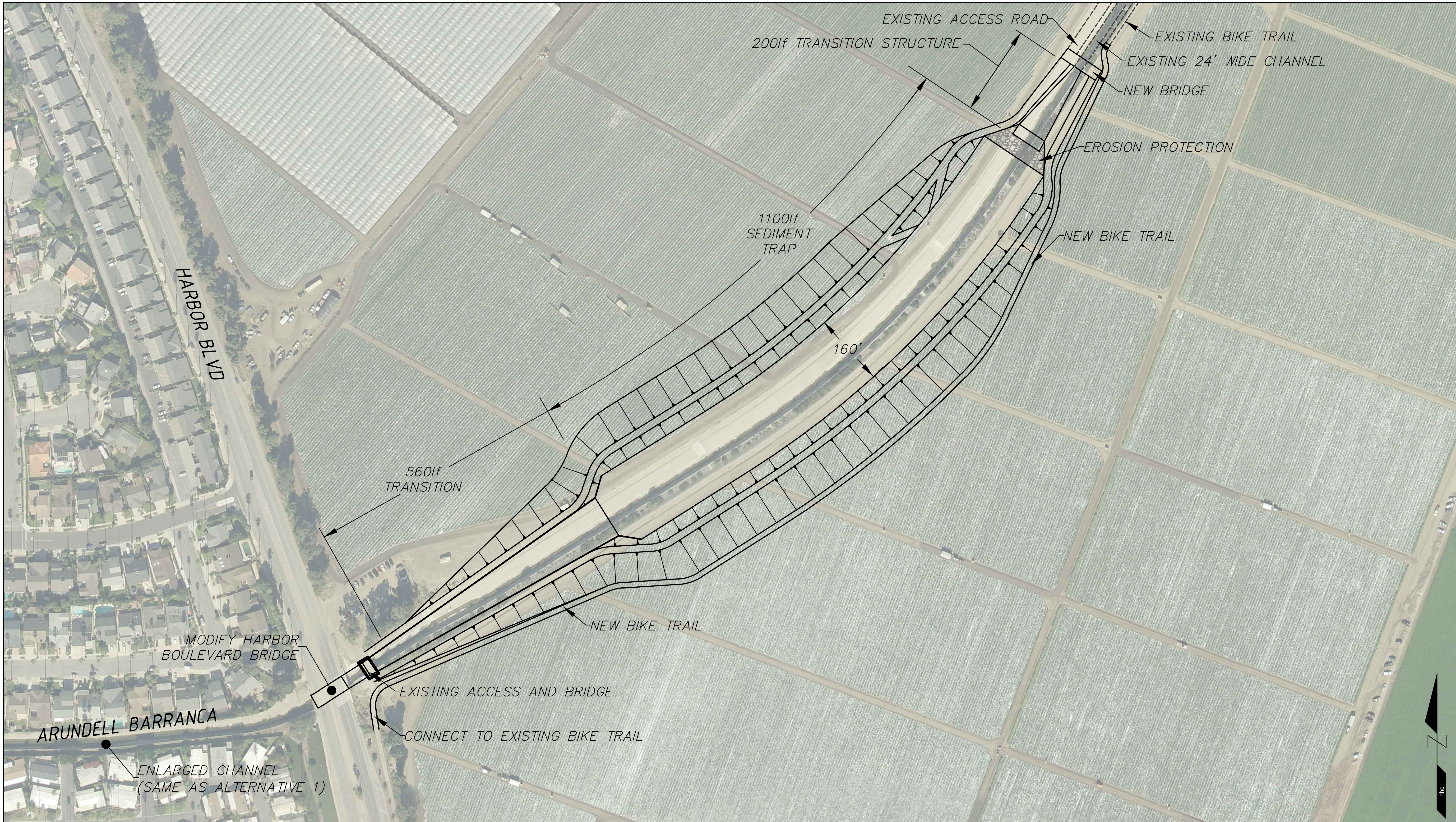
Approximate Quantities and Costs:

- Concrete channel and bridges – same as Alternative 1
- Sediment Trap – 12 acres plus 12 acres mitigation
- Concrete transitions upstream and downstream – 700 lf
- Construction Cost – $ 15.9M
- Maintenance Cost Present Value Estimate – $ 2.2M
- Land Cost - $ 1.6M

- Total Present Value Cost - $ 19.7M

Uncertainties/Analysis Needed:

- Sediment trap transitions will require careful hydraulic design
- Water surface in sub-critical section will be above adjacent grade – levee or floodwall design required



Ventura County Watershed Protection District

nhc northwest hydraulic consultants
80 south lake avenue, suite 800
pasadena, california 91101
phone: (626) 440-0080
fax: (626) 440-1881

Job:	
Rev:	
Drft: tvs	
Date: 17Jul12	

Alternative 1 With In-Line Sediment Trap

Figure 4-7. Alternative 6

Alternative 7 – Alternative 1 with Extension of Arundell Barranca Channel to Pierpont Basin
Concept: The existing channel and bridges downstream of Harbor Boulevard would be enlarged as for Alternative 1, except that the channel would be extended further into the Harbor.
Hydraulic Design:

- Design Q100= 7,500 cfs
- Channel construction Harbor Boulevard to Beachmont as for Alternative 1 - supercritical channel, 32 feet wide at S=0.006, D=9.3 feet and V=28 fps at Q100
- 700 feet long, 32 feet wide, 11 feet deep, supercritical extension channel; maintain S=0.006
- Construction of energy dissipator at the end of extended channel

Feasibility Considerations:

- Right-of-Way - Would require acquisition of right-of-way along Harbor parking lot
- Utilities - Potential conflicts with utility crossings at Harbor; 22" oil line likely conflicts ($1.0M allowance in estimate); may conflict with sewer and water lines
- Infrastructure - Requires modification of existing Harbor Patrol ramp and dock

Maintenance:

- Dredging (Port District) would be similar in volume, but would occur in locations with better dredge access and more receiving volume
- Dredging in Connector Channel (City) may be substantially reduced
- May require periodic cleaning of lower portion of channel if sediments are not self-flushing

Environmental Benefits:

- Reduced dredging and interference with navigation, reduced velocities, and potentially better water quality in the Stub and Connector Channels; potentially better dispersion of sediments in harbor.

Environmental Disadvantages:

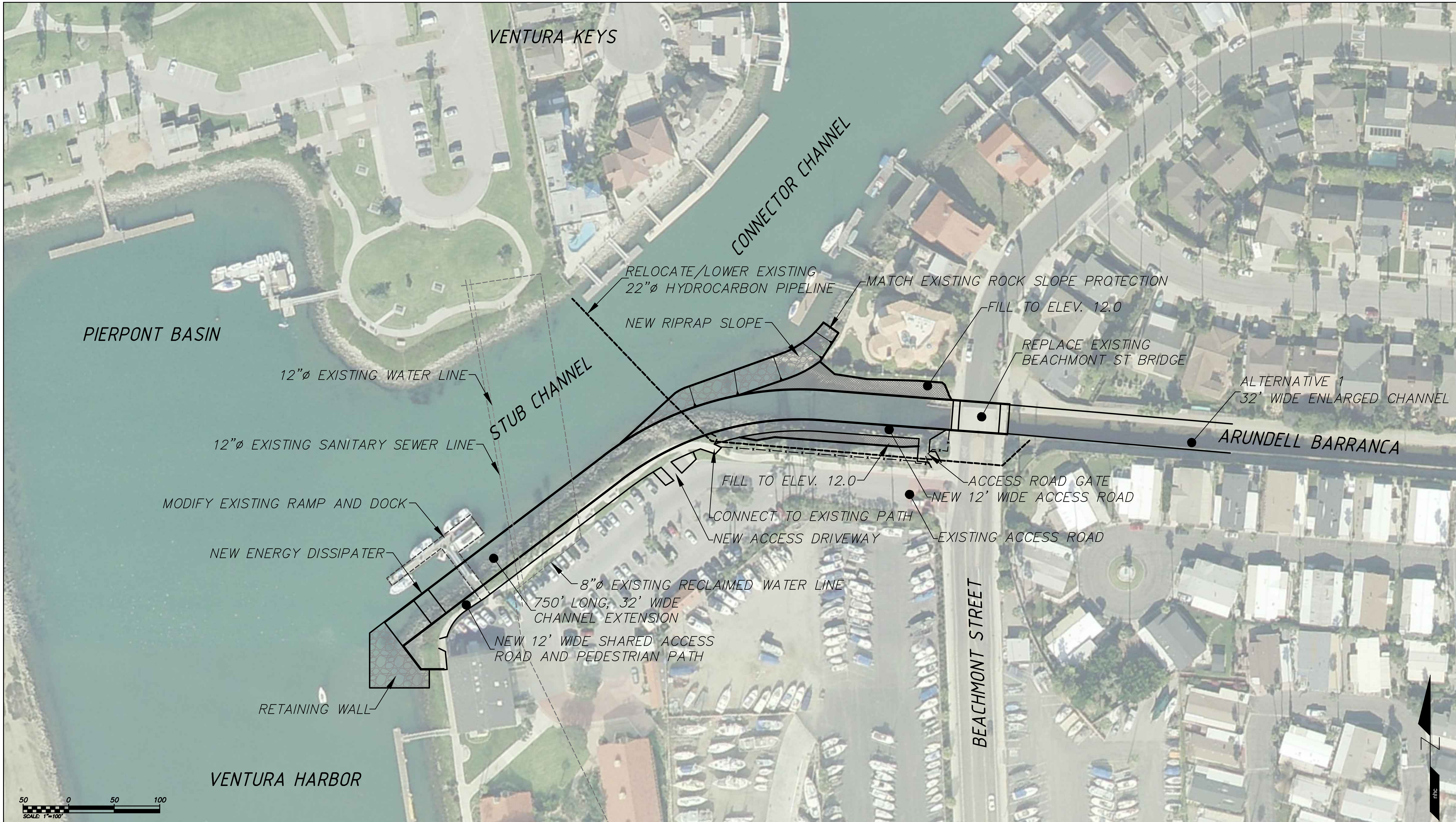
- Requires construction and maintenance within an aquatic environment (Harbor)
- Potential adverse effects on benthic habitat at new outlet

Approximate Quantities and Costs:

- Concrete channel and bridges – same as for Alternative 1 from Beachmont to Harbor Boulevard
- Extended channel – 700 feet
- Construction Cost – $ 16.0M
- Maintenance Cost Present Value Estimate – $ 0.6M
- Land Cost - $0 (with Port District/City cooperation)
- Total Present Value Cost - $ 16.6M

Uncertainties/Analysis Needed:

- Analysis of hydraulics of extended channel resulting from tidal influence
- Analysis of sediment performance of extended channel
- Oil line costs and other utility conflicts



Ventura County Watershed Protection District	nhc northwest hydraulic consultants — 80 south lake avenue, suite 800, pasadena, california 91101, phone: (626) 440-0080, fax: (626) 440-1881	Job: Rev: Drft: tvs Date: 17Jul12	Alternative 1 With Extension of Arundell Baranca Canal

Figure 4-8. Alternative 7B

Alternative 8 – Alternative 1 with Modification of Arundell Barranca Outlet Channel and Stub Channel Confluence

Concept: The existing outlet channel in the Harbor would be modified to increase efficiency in trapping coarse sediments and improve maintenance access for removal of material. A deflector would be installed at the confluence of the Arundell Barranca and Stub Channels to turn the flows more parallel to the Stub Channel.

Hydraulic Design:

- Design Q100= 7,500 cfs
- Channel construction downstream of Harbor Boulevard as for Alternative 1 - supercritical channel, 32 feet wide at S=0.006, D=9.3 feet and V=28 fps at Q100
- Modification of Arundell Barranca outlet channel energy dissipator to increase cross sectional area and capture coarse sediment (cobble trap) – 125 feet long, 80 feet wide, with sill at downstream end
- Construction of flow deflector from right bank of Arundell Barranca outlet to near center of Stub Channel

Feasibility Considerations:

- Right-of-Way - Would require acquisition of right-of-way along Harbor parking lot
- Utilities - Potential conflicts with utility crossings at Harbor; 22" oil line may conflict with wall
- Flow Deflector length constrained to maintain 150 feet wide channel

Maintenance:

- Dredging (Port District) would be only slightly reduced in volume, but reduced in difficulty/cost due to reduction in larger size fractions
- Dredging in Connector Channel (City) may be reduced
- Maintenance of new deflector (Port District)
- Increased removal of coarse sediment in energy dissipator/cobble trap by District from top of bank

Environmental Benefits:

- Reduced dredging and interference with navigation; reduced velocities in outlet channel and confluence; potentially better water quality in the Stub and Connector Channels; potentially better dispersion of sediments in harbor.

Environmental Disadvantages:

- Requires construction and maintenance within an aquatic environment (Harbor)

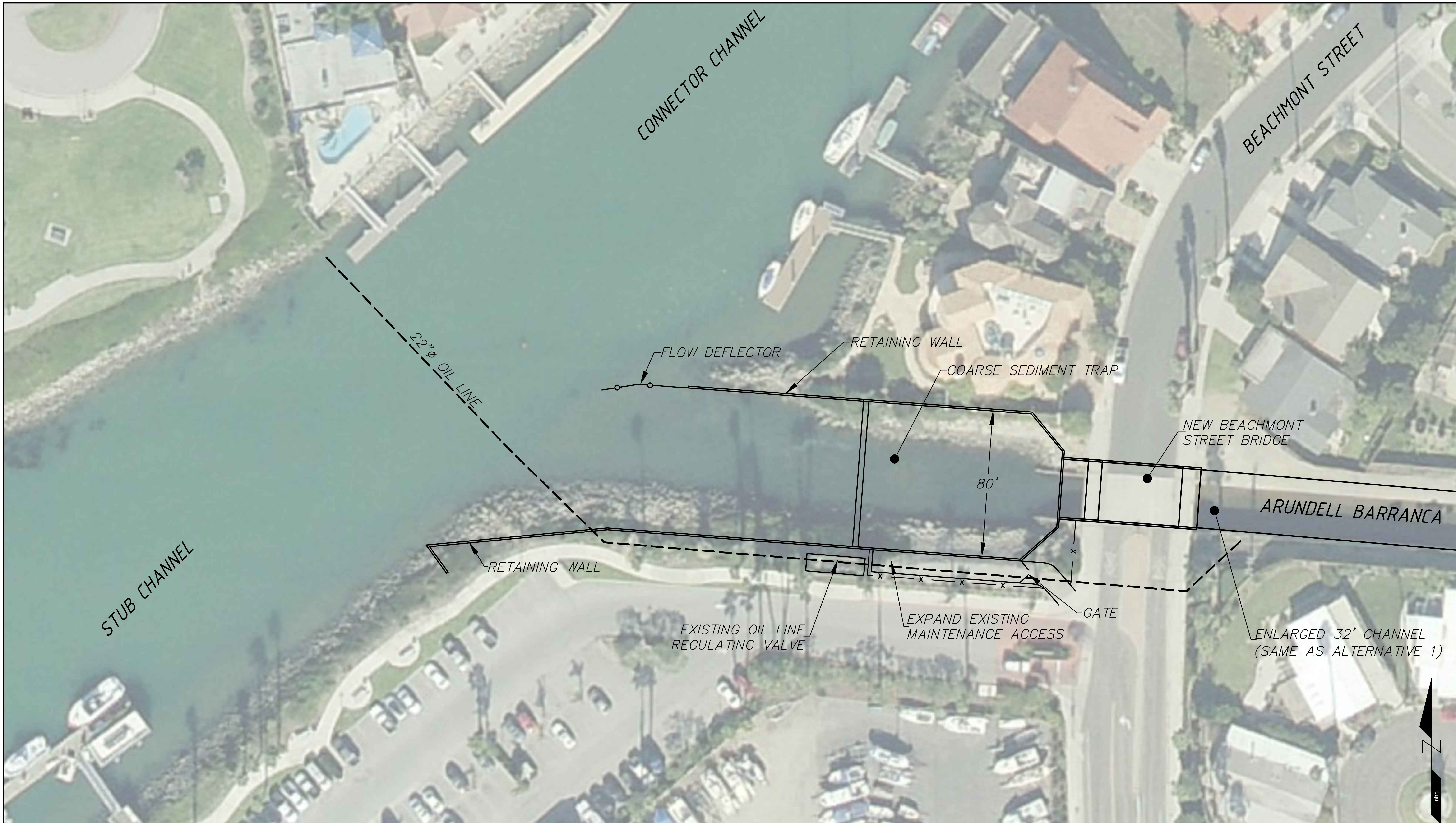
Approximate Quantities and Costs:

- Concrete channel and bridges – same as for Alternative 1 from Beachmont to Harbor Boulevard
- Modified energy dissipator – 125 feet
- Retaining Walls – 650 lineal feet
- Flow Deflector– 50 lineal feet
- Construction Cost – $ 13.4M

- Maintenance Cost Present Value Estimate – $ 1.3M
- Land Cost - $0 (with Port District/City cooperation)
- Total Present Value Cost - $ 14.7M

Uncertainties/Analysis Needed:

- Analysis of hydraulic/sediment performance of deflector during flood events
- Analysis of coarse sediment trapping in modified energy dissipator



Ventura County Watershed Protection District	nhc northwest hydraulic consultants 80 south lake avenue, suite 800 pasadena, california 91101 phone: (626) 440-0080 fax: (626) 440-1881	Job:600001 Rev: Drft: tvs Date: 17Jul12	Alternative 1 With Modification of Outlet Channel

Figure 4-9. Alternative 8

Alternative 9 – Alternative 1 with Diversion of Low Flows to Ventura Water Reclamation Facility

Concept: A low flow diversion would be constructed in the existing channel upstream of Harbor Boulevard and up to 5 cfs would be diverted by pipe into the Harbor Trunk sewer line for delivery to the Ventura Water Reclamation Facility. The existing channel and bridges downstream of Harbor Boulevard would be enlarged as for Alternative 1.

Hydraulic Design:

- Design Q100= 7,500 cfs
- Channel construction Harbor Boulevard to Beachmont Street as for Alternative 1 - supercritical channel, 32 feet wide at S=0.006, D=9.3 feet and V=28 fps at Q100
- Gravity diversion in 18-inch pipe to manhole on Harbor Trunk near existing channel
- Target design flow for diversion is 5 cfs; depends on capacity of plant to accept flows

Feasibility Considerations:

- Right-of-Way – Can probably be accomplished in existing rights of way
- Utilities - Potential conflicts with utility crossings at Harbor Boulevard
- WRF – ability of plant to accept, treat, and discharge flows needs to be checked

Maintenance:

- Minor increase over existing practices to maintain low flow diversion
- Increase in flows and associated operations and maintenance at WRF

Environmental Benefits:

- Potentially improves water quality and reduces total pollutant discharge to Harbor, focusing on summer flows

Environmental Disadvantages:

- Does not address dredging, sediment delivery, velocities, or navigation in Harbor

Approximate Quantities and Costs:

- Construction Cost – $ 10.3M
- Maintenance Cost Present Value Estimate – $ 22.7M

(Based on standard rates for diversion of an average 2 cfs flow to VWRF at $1.78M per year – would be negotiated with City; maintenance cost PV without treatment charges = $0.5M)

- Land Cost - $0 (with Port District/City cooperation)
- Total Present Value Cost - $ 33.0M

Uncertainties/Analysis Needed:

- Ability of plant to accept flows; associated water quality benefits in Harbor; costs of treating diverted flow

Ventura County Watershed Protection District	nhc northwest hydraulic consultants 80 south lake avenue, suite 800 pasadena, california 91101 phone: (626) 440–0080 fax: (626) 440–1881	Job:600001 Rev: Drft: tvs Date:19Jul12	Alternative 1 with Low Flow Diversion to V W R F **Figure 4-10. Alternative 9**

4.3 Alternatives Screening

The alternatives described above were reviewed with stakeholders in a public meeting on 19 July 2012. Comments on the alternatives primarily focused on potential diversions of flow away from the existing harbor outlet and on sediment delivery and water quality near the outlet channel. Several comments suggested improvements to existing water quality were needed, and supported the use of wetlands as a component of the project. Table 4-2 summarizes the alternatives and their estimated construction and maintenance costs, and Table 4-3 provides a qualitative screening based on costs, effectiveness, implementation feasibility, and environmental considerations.

Table 4-2. Alternatives Cost Summary

Alternative	Land Acquisition and Mitigation Cost[1], $M	Construction Cost, $M	Maintenance Cost Present Value[2], $M	Total Cost, $M
1. Expanded Channel	$0.0	$10.0	$0.5	$10.5
2. Diversion to Santa Clara River[3]	$14.2	$32.7	$3.2	$50.0
3. High Flow Retention Basin	$6.7	$11.1	$1.3	$19.1
4. High Flow Diversion to Harbor[4]	$3.0	$12.0	$1.4	$16.4
5. Alt 1 with Low Flow Treatment Wetlands	$1.5	$12.1	$4.0	$17.7
6. Alt 1 with In-Line Sediment Trap	$1.6	$15.9	$2.2	$19.7
7. Alt 1 with Channel Extension in Harbor[5]	$0.0	$16.0	$0.6	$16.6
8. Alt 1 with Modified Outlet Channel	$0.0	$13.4	$1.3	$14.7
9. Alt 1 with Low Flow Diversion to VWRF	$0.0	$10.4	$22.7[6]	$33.1

1Includes acquisition, severance damages, and agricultural land mitigation
2 Includes Construction, Lands, and Maintenance Present Value
3 Does not include acquisition of TNC parcels or their replacement
4 Based on Alternative 4B
5 Based on Alternative 7B
6 Includes $22.2M in City charges for treatment (to be negotiated; $10.9M without treatment charges

Table 4-3. Alternative Screening

Alternative	Cost, $M	Incremental Effectiveness[1]	Feasibility Considerations	Environmental Considerations
1. Expanded Channel	$10.5	Base alternative - provides 100-year flood protection	No additional land required, no major utility conflicts	Reduces pollutants generated by flood overflows onto agricultural lands
2. Diversion to Santa Clara River	$50.0 (does not include compensation or replacement for land at treatment wetland site)	Benefits Harbor water quality and dredging by diverting all flow and sediment to Santa Clara River with low flow treatment of water quality Requires high level of channel and treatment wetland maintenance	Requires acquisition of 68 acres of private land (no willing seller)[2] and use of TNC site for treatment wetland conflicts with grant conditions and TNC intentions for property; other routes more expensive or infeasible Permitting difficult or infeasible under water quality and endangered species regulations	Potential effects on endangered steelhead, tidewater goby, least terns Delivers pollutants during flows larger than 50 cfs to sensitive Santa Clara River estuary Loss of productive coastal farmland
3. High Flow Retention Basin	$19.1	Reduces sediment delivery to Harbor in large events, but would require removal of sediment from retention basins and continued dredging Requires occasional sediment removal from retention basins.	Requires acquisition of 92 acres of private land (no willing seller) [3]	Potential for joint (recreation) use of retention basin Reduces pollutant delivery to Harbor for very large events Loss of productive coastal farmland
4. High Flow Diversion to Harbor	$16.4	Reduces sediment delivery and pollutant delivery at existing outlet but delivers at least a portion of this material to an alternate location in the Harbor	Requires acquisition of 23 acres of private land (no willing seller)[3] and easement at Holiday Inn property May conflict with hotel expansion plans Conduit constructions and high pressure oil and sewer utility crossings in Harbor Boulevard difficult	Potential for joint (bike and pedestrian transportation) use of channel alignment Loss of productive coastal farmland
5. Alt 1 with Low Flow Treatment Wetlands	$17.7	Reduces pollutant delivery to Harbor during low flows Significantly increases maintenance for operation of treatment wetlands	Requires acquisition of 23 acres of private land (no willing seller)[3]	Reduced delivery of pollutants to Harbor Loss of productive coastal farmland

Alternative	Cost, $M	Incremental Effectiveness[1]	Feasibility Considerations	Environmental Considerations
6. Alt 1 with In-Line Sediment Trap	$19.7	Reduces delivery of coarse fraction of sediment (cobbles and gravel) to Harbor, facilitating dredging of smaller material Requires sediment removal from in-line sediment trap every 1 to 5 years depending on flows	Requires acquisition of 24 acres of private land (no willing seller)[3]	Slightly reduced delivery of pollutants to Harbor Loss of productive coastal farmland Visual impacts and levee management due to need to elevate embankment above existing ground
7. Alt 1 with Channel Extension in Harbor	$16.6	Potentially reduced deposition at outlet channel confluence – sediments would be distributed to other areas of the Pierpont Basin and Harbor Requires regular cleaning of subtidal channel extension to maintain capacity	Requires acquisition of right-of-way along Harbor parking lot Conflicts with 22" high pressure oil line Harbor crossing May conflict with water and sewer Harbor crossings	Potential effects on benthic habitat at new outlet Increased construction in a marine environment
8. Alt 1 with Modified Outlet Channel	$14.7	Reduced delivery of cobbles and coarse sediment to Stub Channel, facilitating dredging Reduced velocities in outlet channel Increased maintenance for removal of coarse sediment from cobble trap	Conflicts with 22" high pressure oil line Harbor crossing	Increased construction in a marine environment
9. Alt 1 with Low Flow Diversion to VWRF	$33.1	Reduces pollutant delivery to Harbor during low flows Requires increased operations and maintenance at VWRF	Requires small diameter pipeline crossing of Harbor Boulevard	Reduced delivery of pollutants to Harbor

[1] All alternatives provide 100-year flood protection. Incremental effectiveness indicates secondary benefits for maintenance or environmental benefits.

[2] The District contacted The Nature Conservancy, the owner of the property along the right bank of the Santa Clara River where Alternative 2c wetlands and channel outlet are shown; TNC informally indicated an unwillingness to sell or modify the use of the required land for the purposes of the diversion.

[3] The District has contacted the landowner adjacent to the channel in the reach between Harbor Boulevard and the UPRR Bridge. The landowner has informally indicated an unwillingness to sell the property for channel widening or detention/wetland facilities.

The incremental benefits and disadvantages of the alternatives may qualitatively be compared to Alternative 1 as a baseline. Alternative 2 addresses many of the comments received in public review by diverting all flow and sediment to the Santa Clara River. Pollutants carried in the flow would also be diverted to the river, although water quality would be improved in a portion of the runoff volume by the treatment wetlands. However, implementation of Alternative 2 is highly uncertain due to the lack of available land for the diversion channel and the treatment wetlands, and regulatory requirements under the TMDL and endangered species regulations. In screening meetings and telephone communications with National Marine Fisheries Service, California Department of Fish and Wildlife, Los Angeles Regional Water Quality Control Board, Coastal Commission, The Nature Conservancy, and City of Ventura the District identified several factors that each could delay or make infeasible implementation of the alternative. Although it is the highest cost alternative developed, the costs listed in Table 4-2 likely significantly underestimate the probable costs that would be incurred in environmental studies, legal fees, land acquisition costs, and environmental mitigation. This alternative has very high incremental cost, very low or doubtful implementation feasibility, high maintenance requirements, and adverse environmental effects that potentially significantly outweigh the benefits.

Alternative 3 provides some incremental benefits compared to Alternative 1 in reducing sediment and pollutant delivery to the Harbor, but the incremental benefit is low because only very high flows would be diverted. Implementation feasibility is constrained by lack of availabile land. This alternative has high incremental construction and land cost, low implementation feasibility, and moderate incremental maintenance requirements. Relatively minor positive environmental effects on sediment and pollutants would be countered by loss of productive coastal farmland.

Alternative 4 diverts a portion of the flow (high flows) to another location in the Harbor. Implementation is constrained by lack of available land and potential effects on hotel expansion. Overall effects on Harbor sediment and water quality are neutral, but the location of the discharge is distributed, and a small improvement would likely be realized at the existing outlet near the Ventura Keys residential properties. This alternative has moderate incremental construction and land cost, low implementation feasibility, and moderate incremental maintenance requirements. Overall environmental effects in the Harbor are neutral, but the alternative would cause a loss in productive coastal farmland.

Alternative 5 diverts low flows to a treatment wetland and provides an incremental benefit in water quality in the Harbor. Implementation is constrained by lack of available land. During public review, a concept was advanced that combines elements of Alternatives 2 and 5 to create an estuarine section of Arundell Barranca as an environmental benefit. This concept is constrained by the lack of available land at a suitable elevation for an estuarine system. Compared to Alternative 5, the concept also would not provide the benefits of formal treatment of urban runoff prior to discharge to a natural system. Alternative 5 has moderate incremental construction and land cost, moderate implementation feasibility, and moderate incremental maintenance requirements. Wetland treatment and potential open space/recreation benefits would be countered by loss of productive coastal farmland.

Alternative 6 uses an in-line sediment trap to reduce coarse sediment loads to the Harbor, and could potentially have some water quality and sediment delivery benefits in the Harbor. The alternative is difficult to design because of the required transitions between the basin and the upstream and downstream supercritical channel segments. This requires a relatively long structure that would be elevated above the adjacent ground at its downstream end and subject to maintenance requirements typically associated with levees that constrain planting. This alternative has high incremental cost, moderate implementation feasibility, and high maintenance requirements. Relatively minor water quality benefits would be countered by loss of productive coastal farmland.

Alternative 7 would relocate the outlet of Arundell Barranca in the Harbor, reducing water quality and sediment effects at the confluence of the outlet channel and Stub Channel, but potentially transferring these effects to Pierpont Basin, although sediment might be better distributed and easier to dredge in this location. The alternative includes some uncertainty in performance and maintenance requirements due to construction of the outlet channel at subtidal elevations, and implementation is constrained by conflicts with oil, sewer, and water line crossings of the Harbor and the existing Harbor Patrol dock. This alternative has high incremental construction cost, moderate to low implementation feasibility, moderate maintenance requirements. Environmental benefits at the existing outlet would be countered by potential effects on benthic habitat and navigation at the new outlet.

Alternative 8 would reduce coarse sediment loads to the Harbor by trapping cobble and gravel at the energy dissipator for removal from the top of bank by excavator or clamshell. Reduction of the coarse sediment fraction would facilitate dredging of smaller material in the harbor, but would have little effect on total sediment delivery. During public review, this alternative was supported for further evaluation by the Ventura Port District. Implementation is constrained by conflict with the existing oil line crossing of the Harbor. This alternative has moderate incremental construction cost, moderate implementation feasibility, and moderate incremental maintenance requirements (for the District). Environmental benefits include reduced dredging difficulty, slightly reduced sediment delivery, and lower outlet channel velocities.

Alternative 9 is similar to Alternative 5, but diverts low flows to the VWRF for treatments. Compared to Alternative 5, construction costs are low and incremental maintenance costs are low for the District. However, the alternative adds incrementally to operations and maintenance at the VWRF, and charges for the treatment service could outweigh the construction costs. VWRF provided a treatment charge for estimating purposes (approximately $1.8M per year at 2 cfs average flow),but actual costs might be negotiated by the District and City at a lower level.

Based on the relative benefits and considerations outline d above, the District advanced Alternatives 1,5, 8, and 9 for further evaluation. Alternative 8 was advanced to investigate potential improvements to sediment conditions in the Harbor, and Alternatives 5 and 9 were advanced to consider potential water quality improvements. Alternatives 5, 8 and 9 each include the improvements in Alternative 1.

5. DETAILED ALTERNATIVE ANALYSIS

The alternatives carried forward for detailed analysis included two alternatives (Alternative 1 and Alternative 8) for modification of the outlet of the Arundell Barranca channel in Ventura Harbor. Alternative 1 is the channel configuration originally proposed, including improvements to the concrete channel upstream of the Harbor. Alternative 8 is a modification of Alternative 1 that included an enlarged cross section near the outlet to trap very coarse sediment (cobbles). In addition to these alternatives, Alternatives 5 and 9 were carried forward as supplemental improvements to improve water quality. These alternatives are intended to improve existing conditions by diverting and treating low flows. Alternative 5 would treat the low flows in a constructed wetland and Alternative 9 would divert low flows to the Ventura Water Reclamation Facility for treatment.

Because the Harbor outlet alternatives include the channel improvements upstream of Beachmont Street, and because they are independent of the supplemental water quality improvements in Alternatives 5 and 9, this section is organized into two subsections - Section 5.1 describes the analysis and compares results for the Harbor outlet modifications, and Section 5.2 develops and compares the water quality alternatives.

5.1 Harbor Outlet Modifications

Harbor outlet alternatives were analyzed using a two-dimensional hydraulic model over a range of flow and tide conditions to assess their performance relative to existing conditions. The analyses resulted in modifications to the alternatives over the course of the work to improve performance in terms of flood capacity, avoiding adverse impacts on velocity magnitudes and distributions, and sediment transport. The invert profile of Alternative 1 was modified to achieve flood control objectives, and Alternative 10 was developed as a lower cost option to Alternative 8 and later reconfigured to optimize performance (designated at Alternative 12 in its final configuration). Also, an alternative that reduced the use of retaining walls (Alternative 13) was added to the options considered. The characteristics of the alternatives carried forward and developed in the detailed analysis and their expected performance under simulated flood conditions are presented below.

5.1.1 Lower Arundell Barranca and Outlet Characteristics

Flows in the lower Arundell Barranca upstream of Beachmont Street are supercritical, with velocities exceeding 30 feet per second under with-project, 100-year peak discharge conditions (channel bottom width = 32 feet, discharge = 7500 cfs). The existing rectangular concrete channel ends below the Beachmont Street crossing, and a drop chute connects the channel to an energy dissipator which passes flow to a riprap-lined outlet channel. A sheet pile wall separates the Barranca outlet from the Stub Channel, and a guidance dike projects out along the north boundary of the outlet channel at the confluence. Plan and profile views of the outlet area are presented in Figure 5-1. A perspective view of the energy dissipator structure is shown in Figure 5-2. This more complex set of structures replaced a relatively simple outlet (see Figure 5-3) that had been constructed in Spring of 1969, but was damaged during a flood that occurred in early 1970 (City of Los Angeles, 1972).

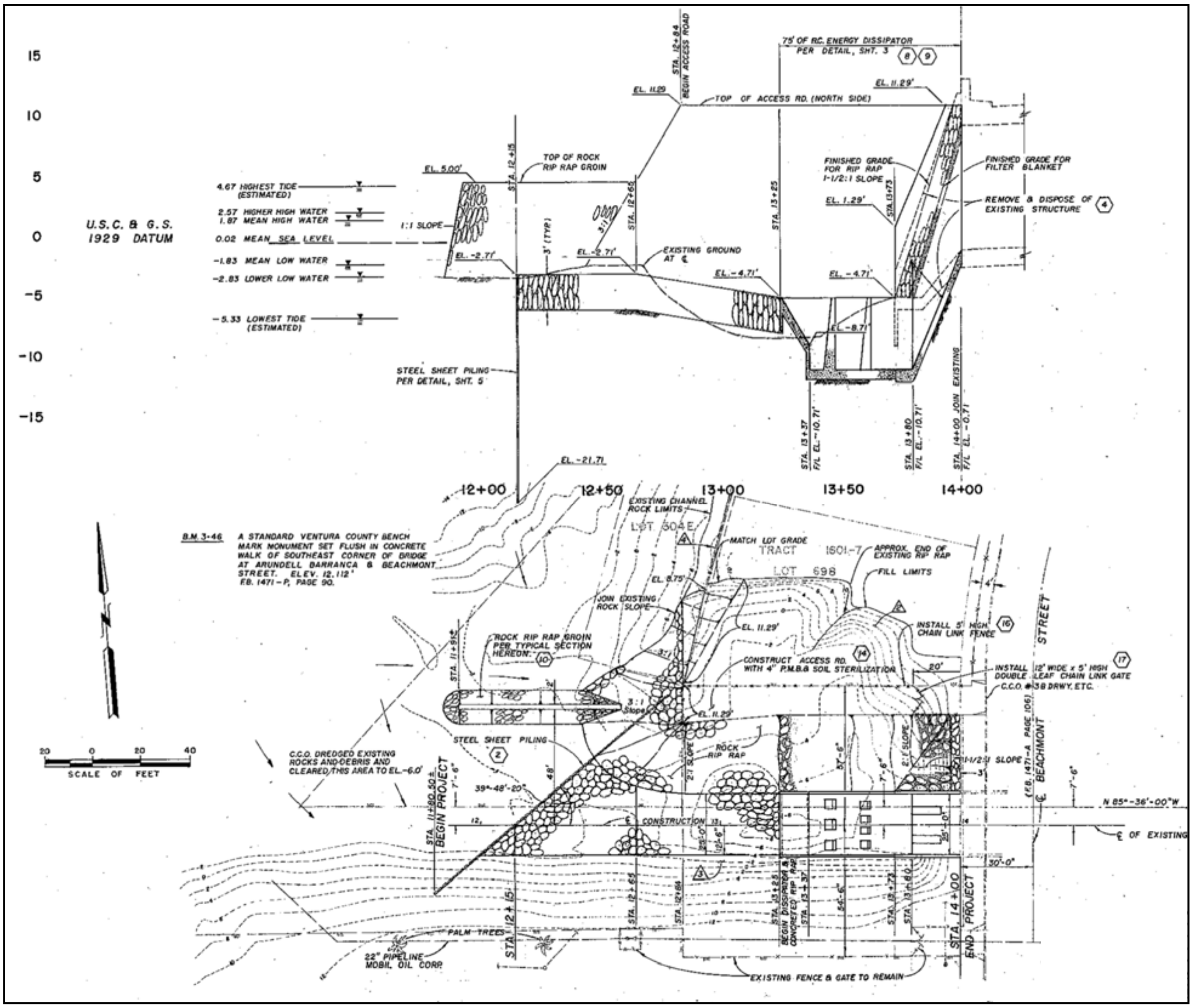


Figure 5-1. Plan and profile view of the existing outlet channel and energy dissipator at the downstream end of Arundell Barranca, constructed in 1974

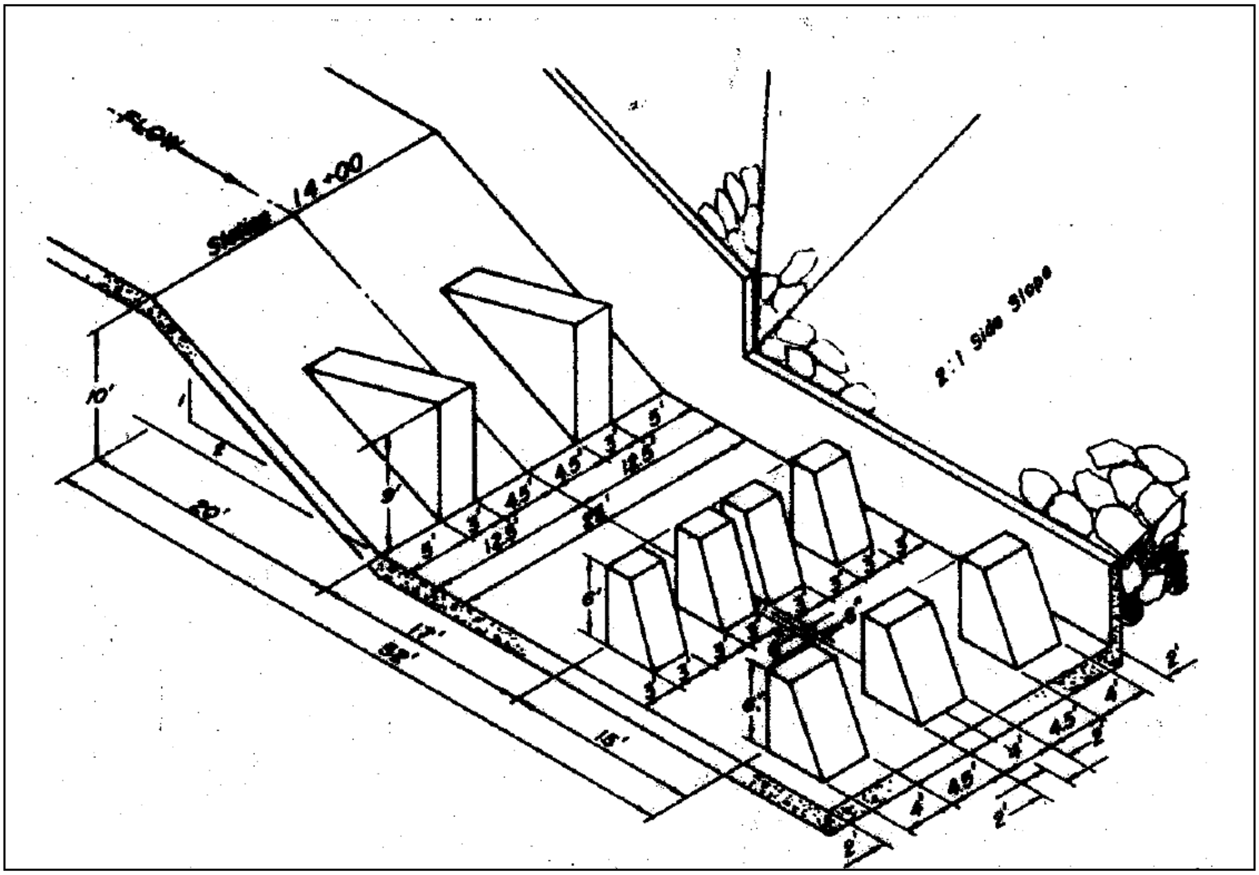


Figure 5-2. Perspective view of the energy dissipator downstream of Beachmont Street on Arundell Barranca

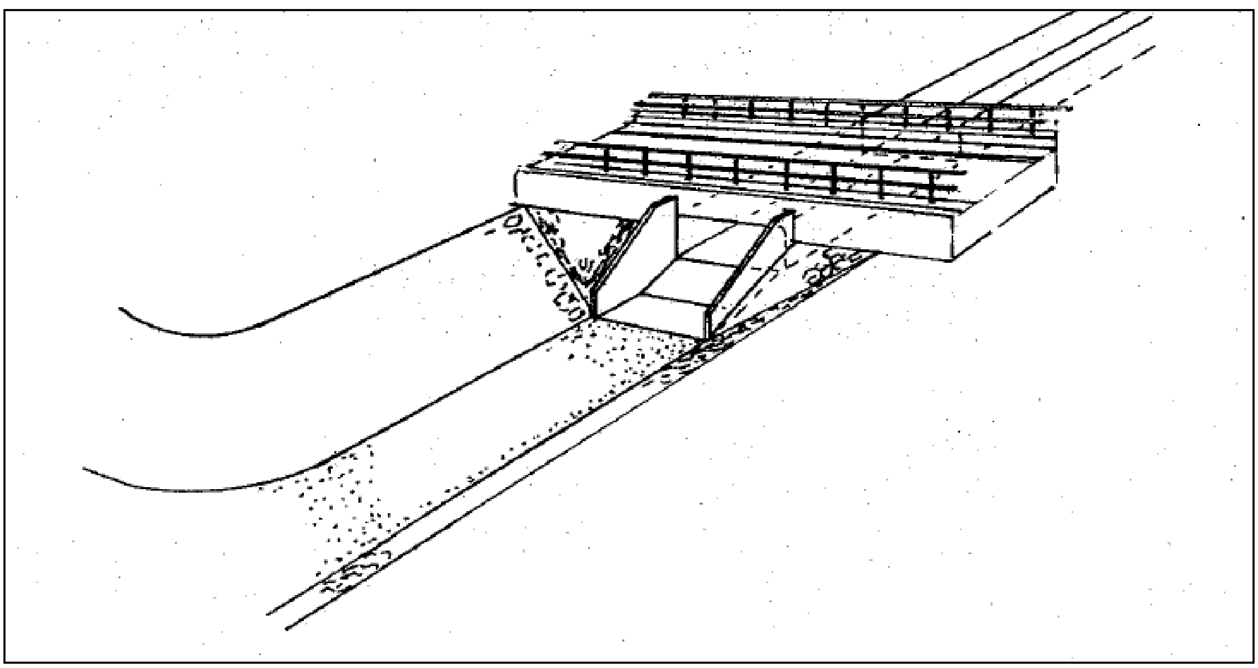


Figure 5-3. The previous outlet configuration (constructed in Spring 1969) at the downstream end of the Arundell

5.1.2 Configuration of the Selected Alternatives

The outlet configuration associated with Alternative 1 is the existing condition outlet, modified to conform to the widening proposed for the channel upstream. In the course of the detailed analyses conducted in this phase of study, it was determined that the profile of the existing outlet would have to be lowered to accommodate the higher design flows expected with improvements upstream. The original outlet was designed to handle design discharges of 4600 cfs. With the design flow rate increased to 7500 cfs, peak design flood water levels could overtop the banks downstream along the outlet channel. Thus, Alternative 1 includes both widening and deepening the outlet geometry, with the general configuration matching that of the as-built condition.

The Alternative 8 outlet configuration was analyzed as originally proposed, with a cobble trap defined by a weir wall across the channel, and an 80-ft wide outlet channel excavated to an elevation of -8 feet NAVD88 and bounded by retaining walls.

An Alternative 10 outlet configuration was developed initially as a modification to Alternative 8, with expected cost reductions associated with a reduced retaining wall height along the south wall. The hydraulic performance of this alternative proved less than optimal, with issues associated with lack of symmetry and ineffective flow zones. Modifications were made to the configuration to enhance its performance, ultimately resulting in a dramatically altered version, which is hereinafter designated Alternative 12. With this alternative, the invert level of the outlet has been lowered to elevations which approximate those in the stub channel, and the south bank has been cut away with a vertical wall of increasing height to aid in directing flows and sediment load southward toward the ocean.

The Alternative 13 outlet configuration combines elements of the Alternative 1 and 12 outlets. The Alternative 13 configuration involves minimal modification to the channel invert profile, and incorporates the slight adjustment to the channel outlet alignment that is part of Alternative 12, but through use of a riprap bank rather than a retaining wall. In part, this alternative was developed to consider ways to reduce impacts on a high pressure oil pipeline that crosses the Stub Channel and runs along the southerly side of the outlet channel.

Plan, profile, and section views of Alternatives 1, 8, 12, and 13 are shown in Figures 5-4 through 5-11.

Ventura County Watershed Protection District

nhc northwest hydraulic consultants
80 south lake avenue, suite 800
pasadena, california 91101
phone: (626) 440–0080
fax: (626) 440–1881

Job:600013
Rev:
Drft: tvs
Date:16Sep13

Alternative 1 at Harbor Outlet

Figure 5-4. Plan View

Alternative 1

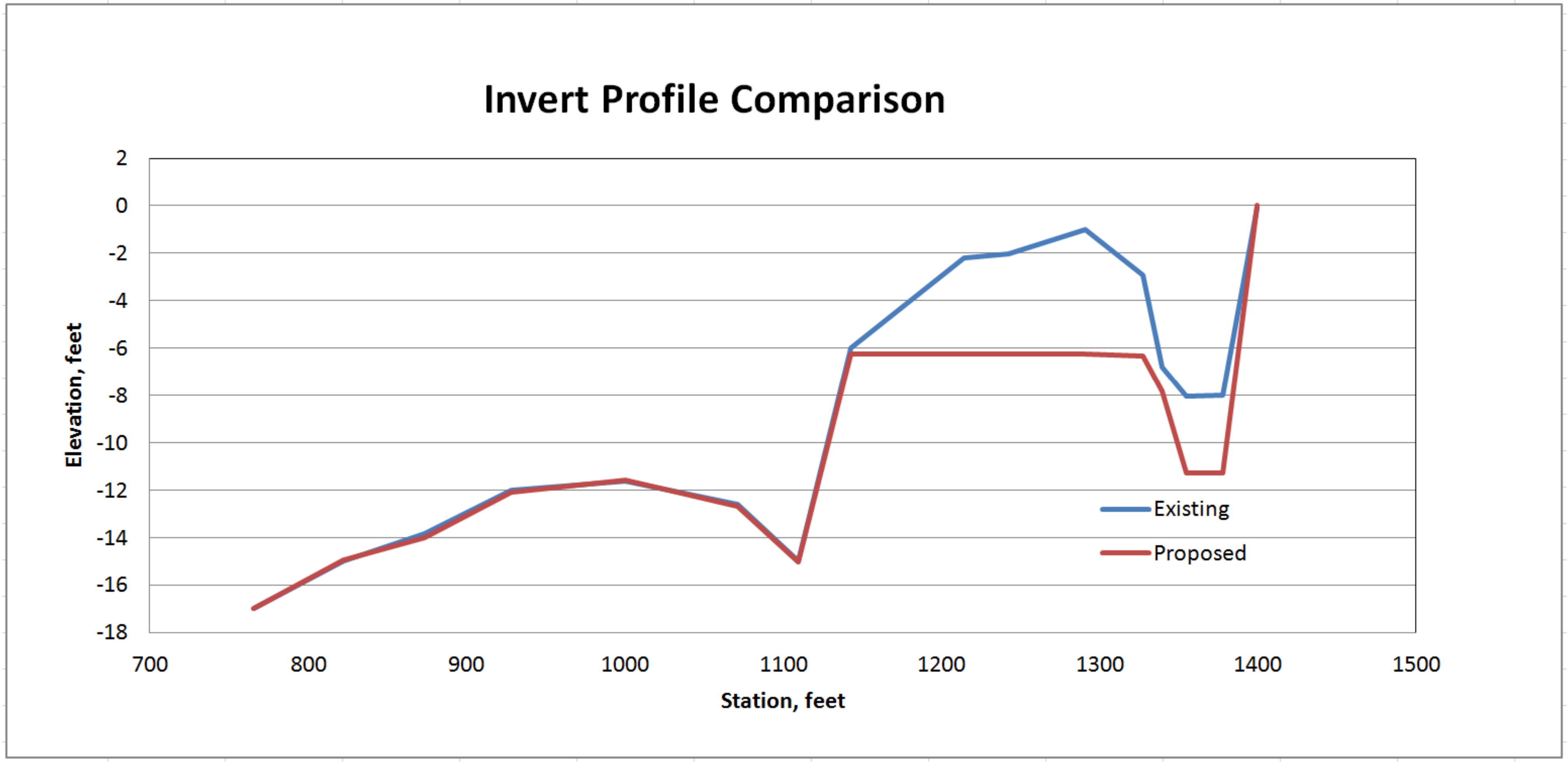
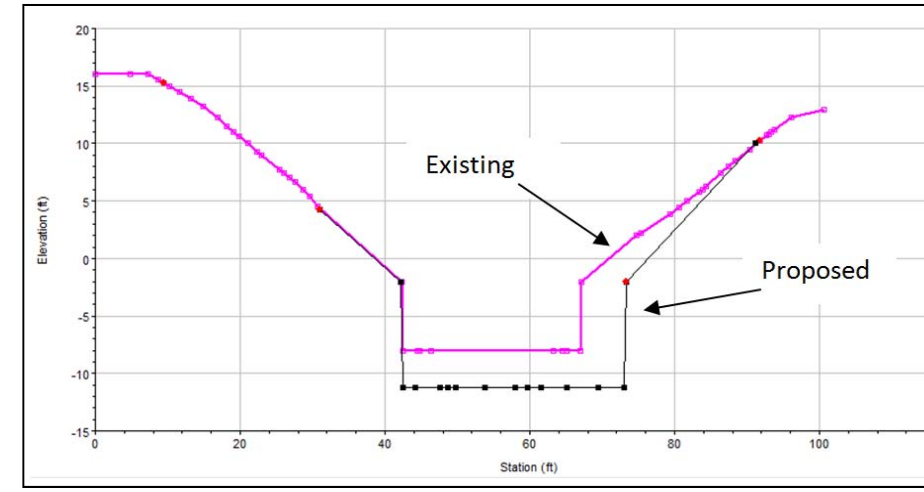
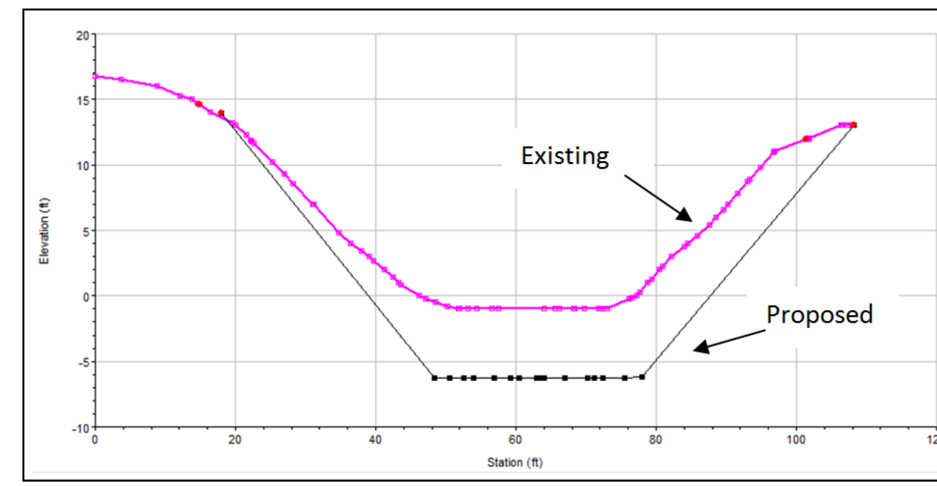


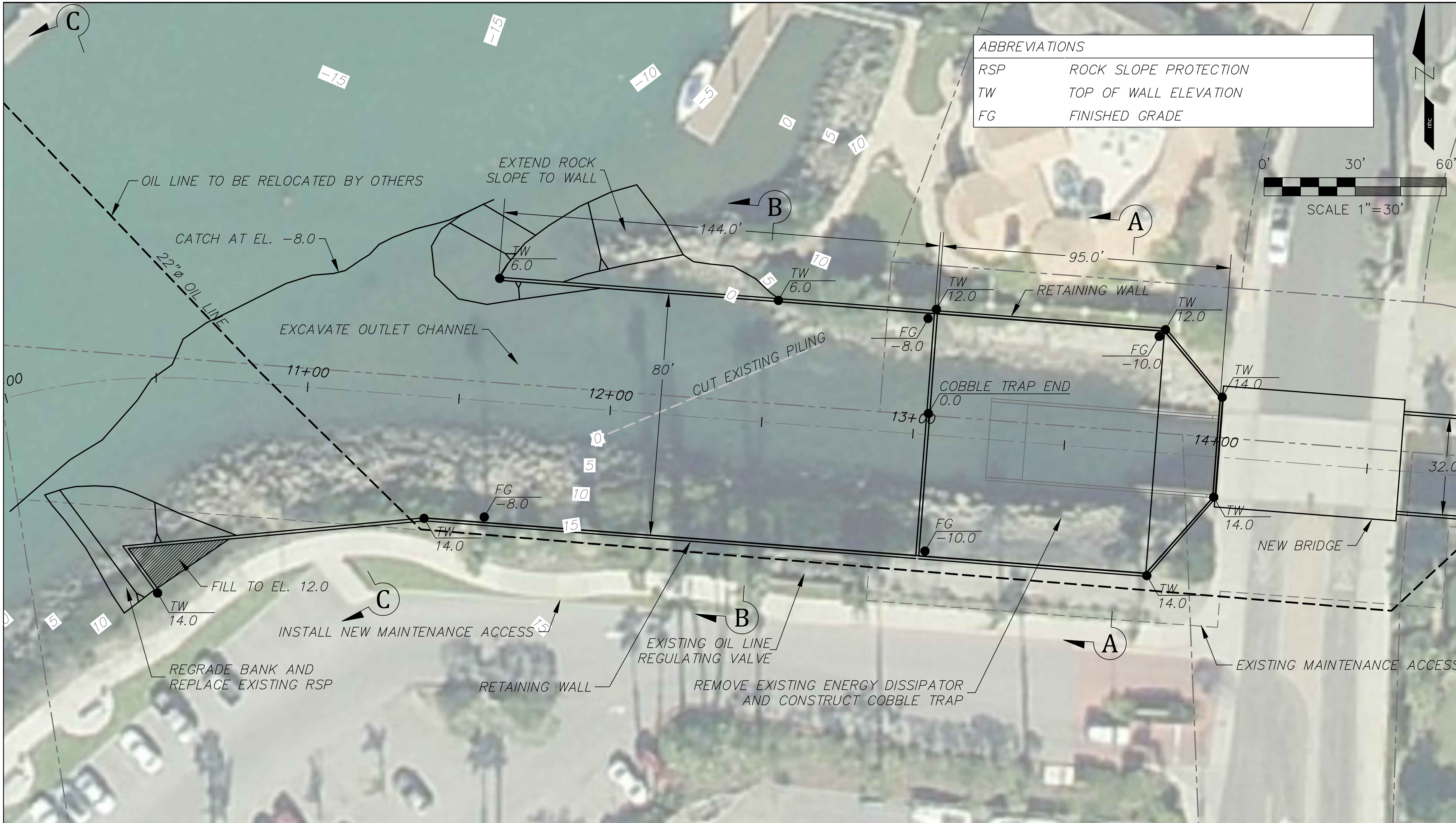
Figure 5-5 Profile and section views of Alternative 1



Section A-A



Section B-B



Ventura County Watershed Protection District

nhc northwest hydraulic consultants

80 south lake avenue, suite 800
pasadena, california 91101
phone: (626) 440-0080
fax: (626) 440-1881

Job:600013
Rev:
Drft: tvs
Date:16Sep13

Alternative 8 at Harbor Outlet

Figure 5-6. Plan View

Alternative 8

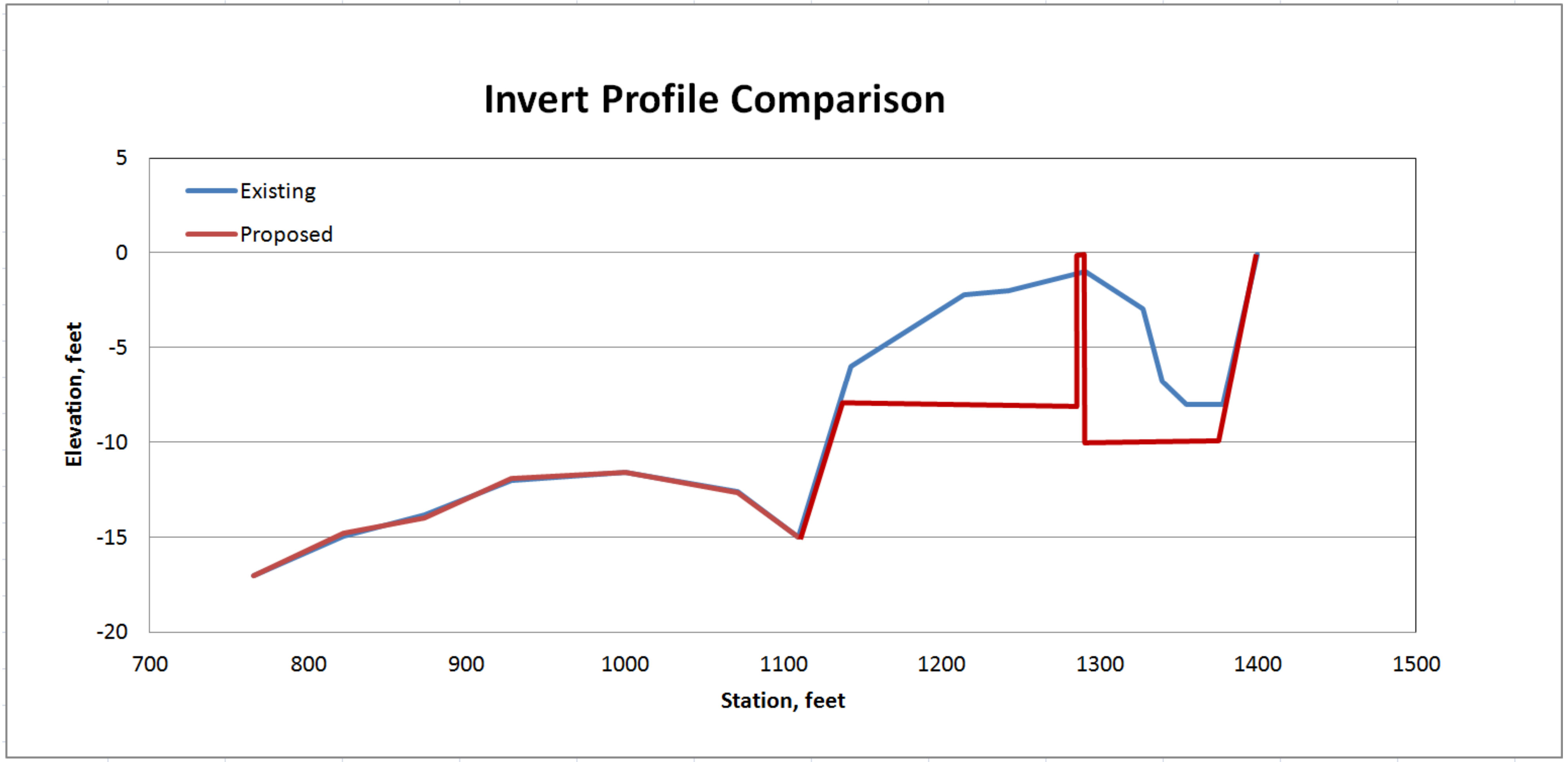
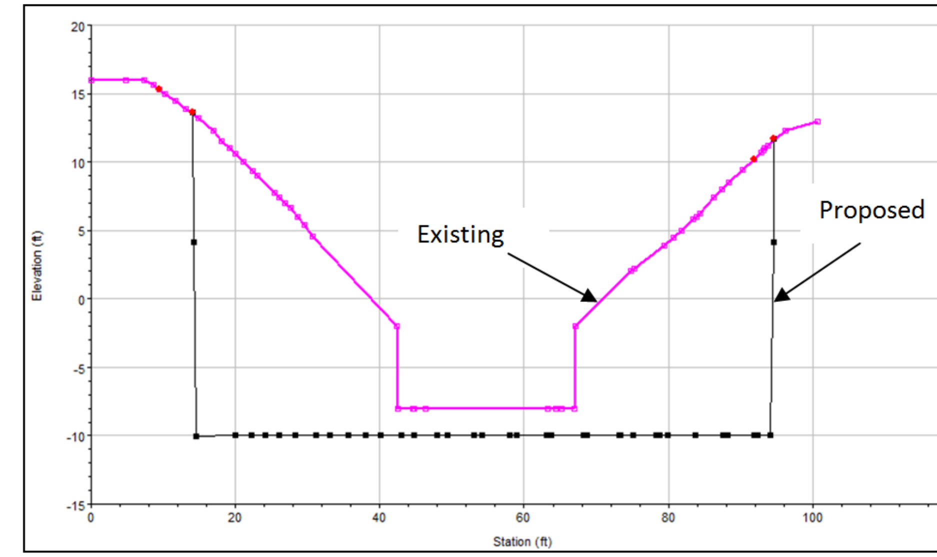
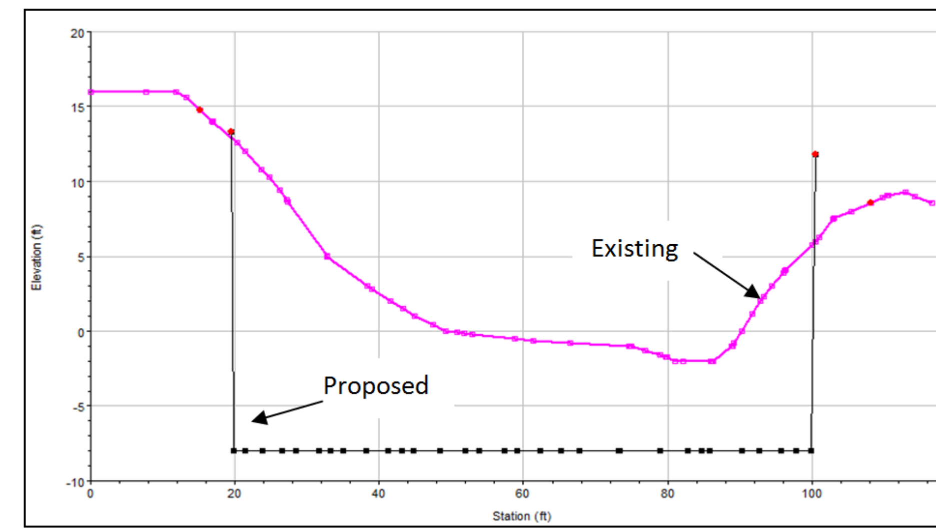


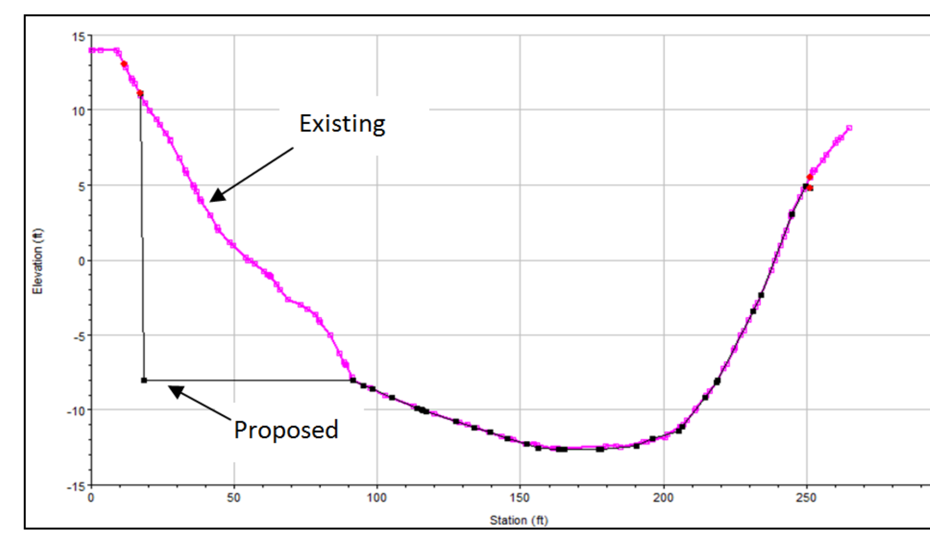
Figure 5-7 Profile and section views of Alternative 8



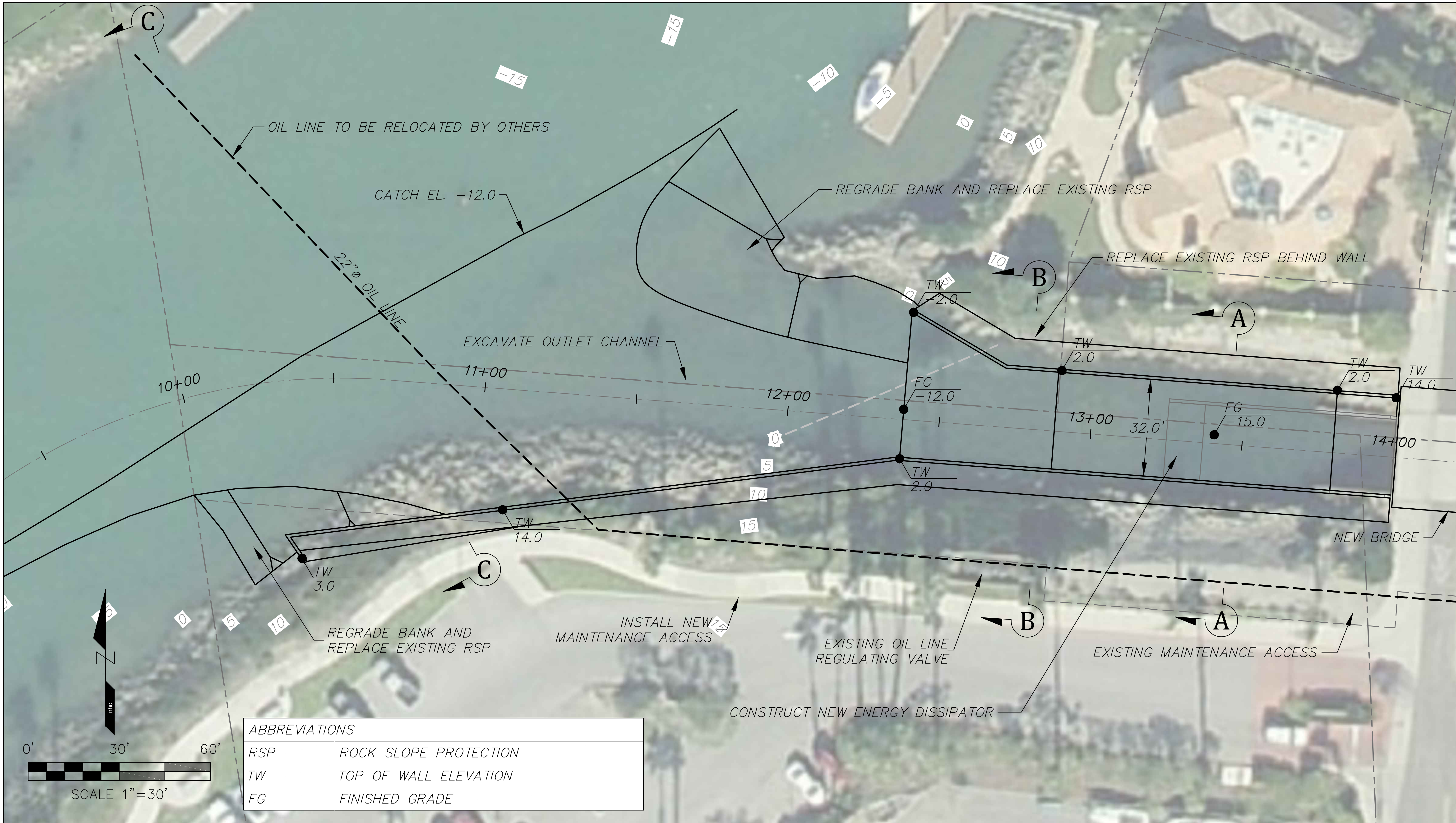
Section A-A



Section B-B



Section C-C



ABBREVIATIONS	
RSP	ROCK SLOPE PROTECTION
TW	TOP OF WALL ELEVATION
FG	FINISHED GRADE

Ventura County Watershed Protection District

nhc northwest hydraulic consultants
80 south lake avenue, suite 800
pasadena, california 91101
phone: (626) 440−0080
fax: (626) 440−1881

Job:600013
Rev:
Drft: tvs
Date:16Sep13

Alternative 12 at Harbor Outlet

Figure 5-8. Plan View

Alternative 12

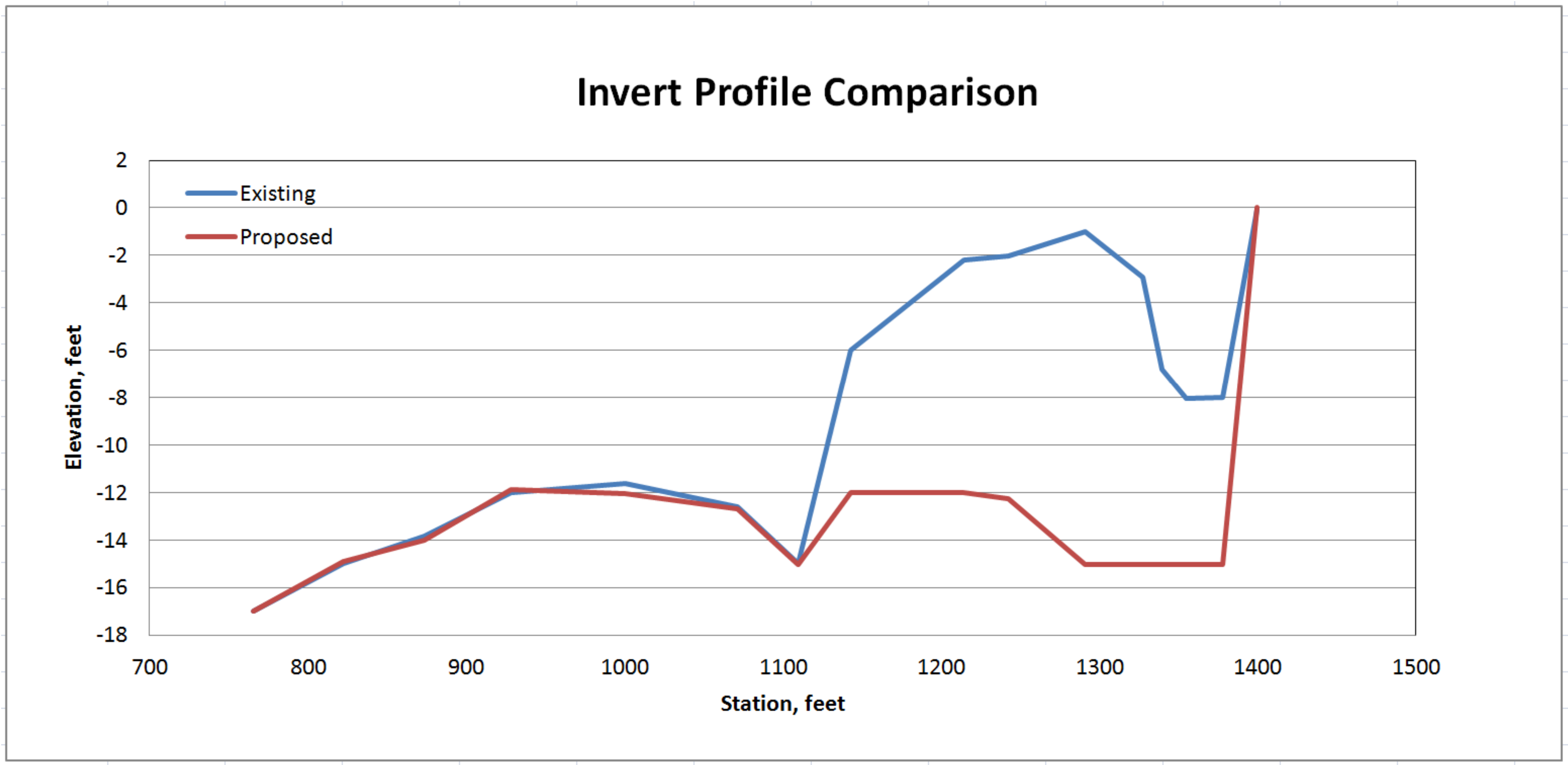
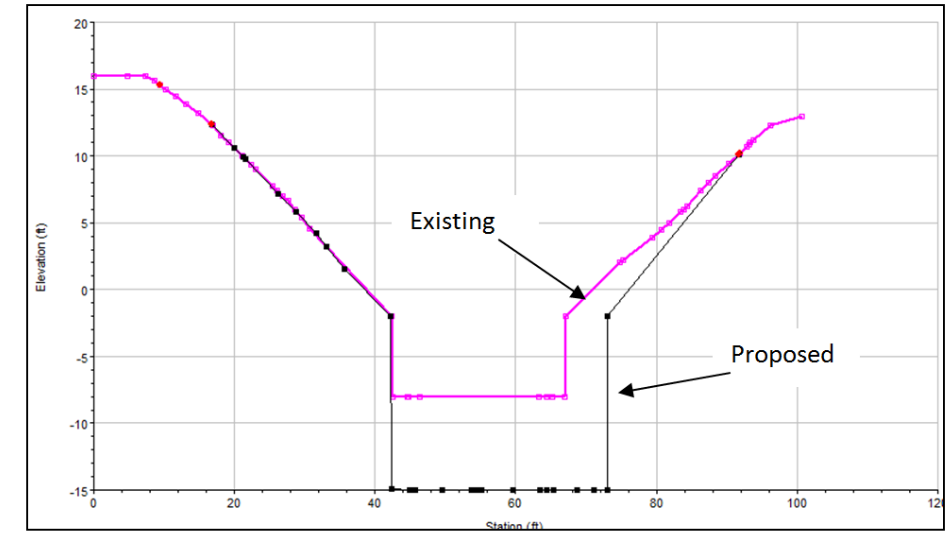
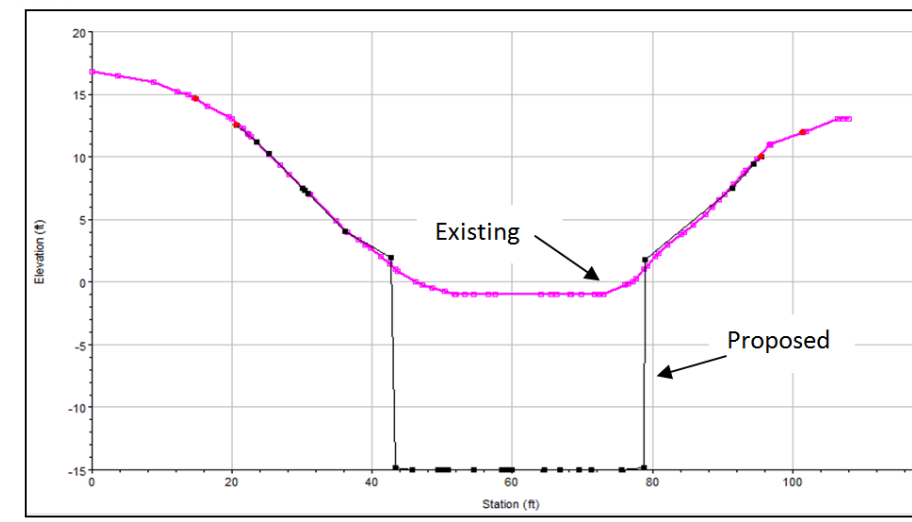


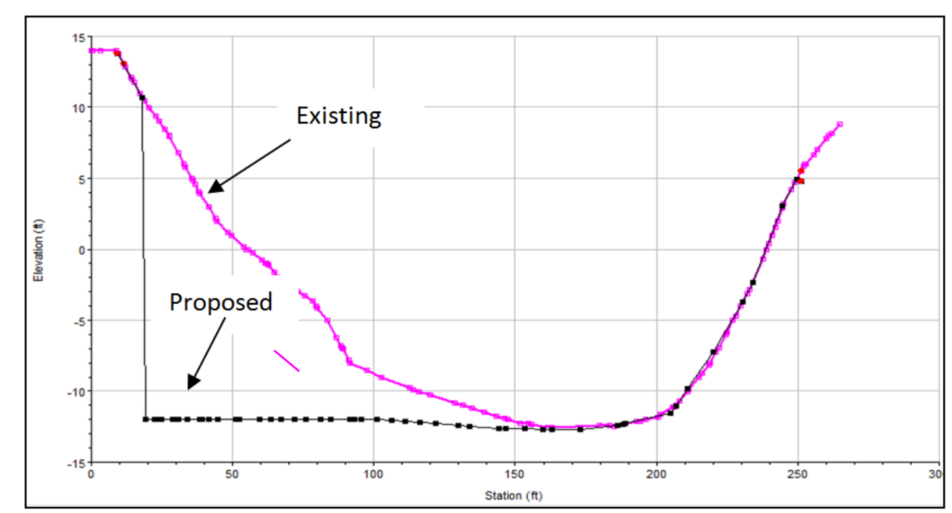
Figure 5-9 Profile and section views of Alternative 12



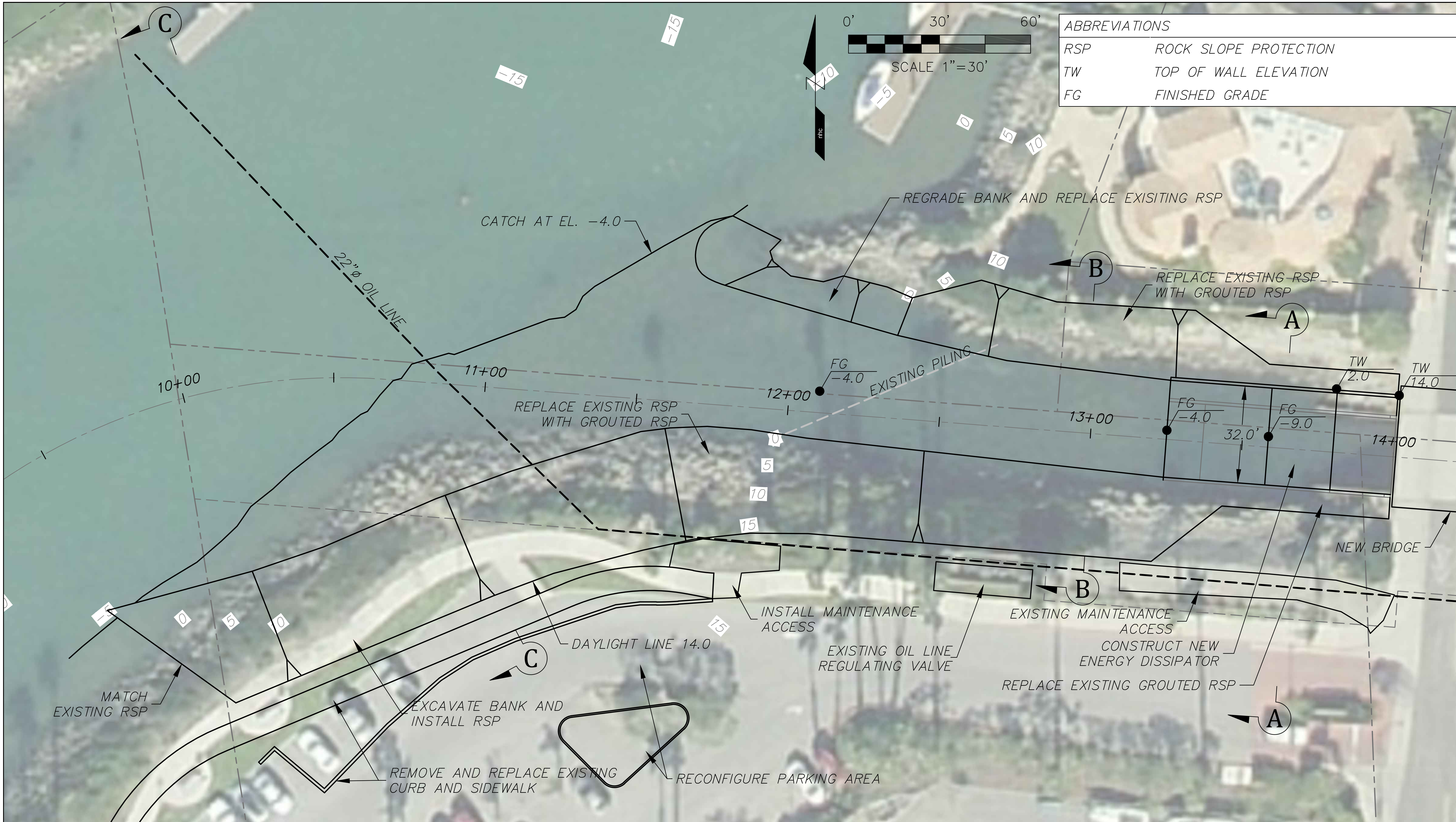
Section A-A



Section B-B



Section C-C



Ventura County Watershed Protection District

nhc northwest hydraulic consultants

80 south lake avenue, suite 800
pasadena, california 91101
phone: (626) 440−0080
fax: (626) 440−1881

Job:600013
Rev:
Drft: tvs
Date:16Sep13

Alternative 13 at Harbor Outlet

Figure 5-10. Plan View

Alternative 13

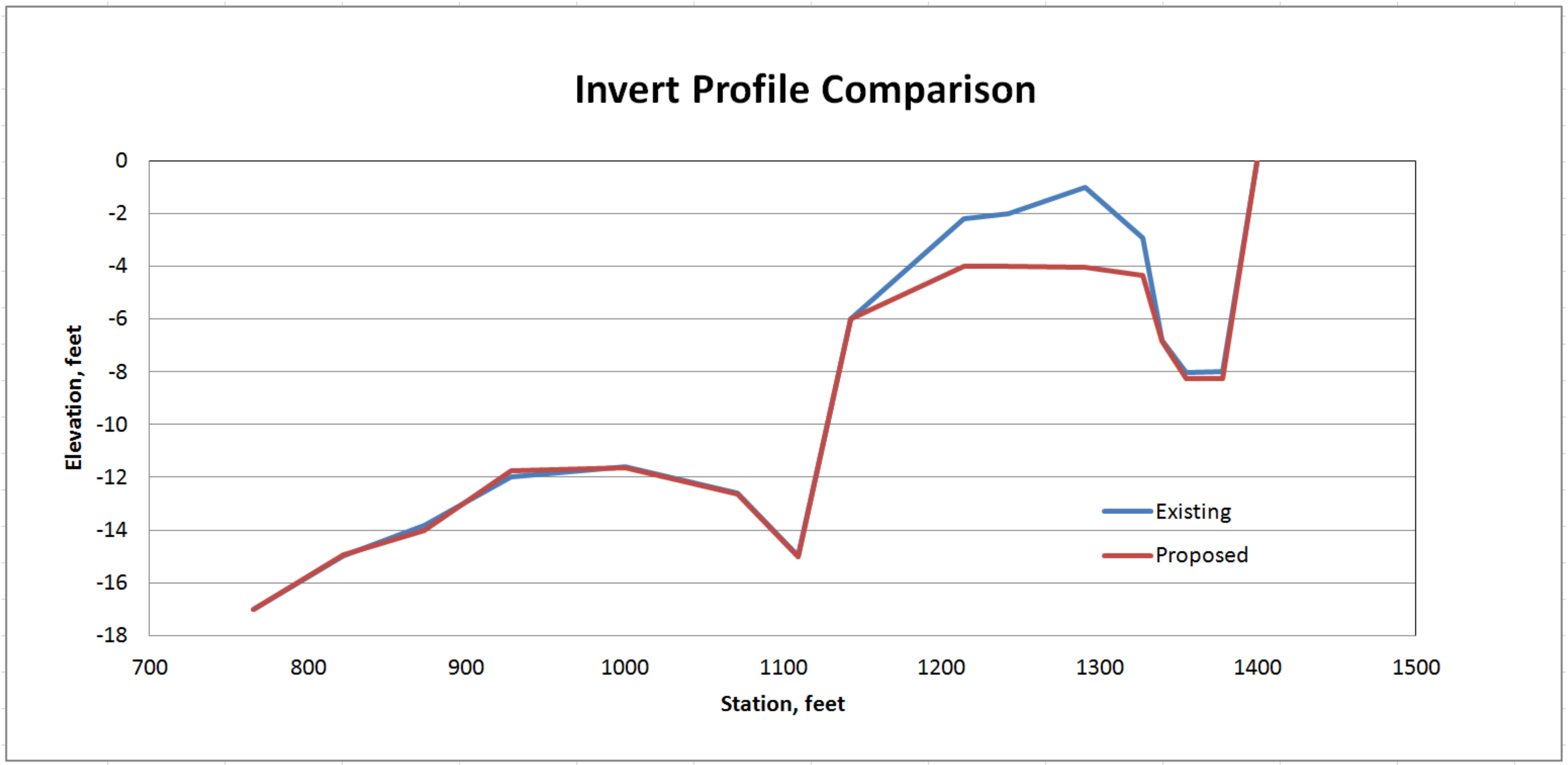
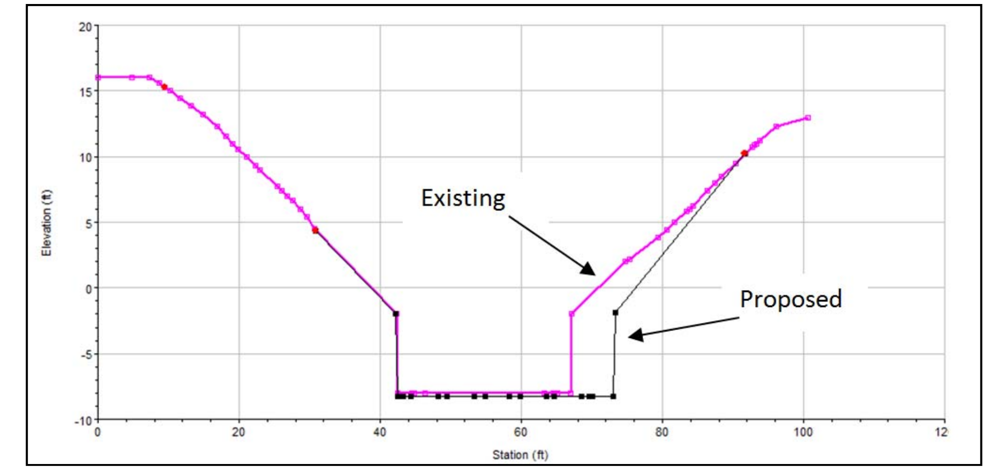
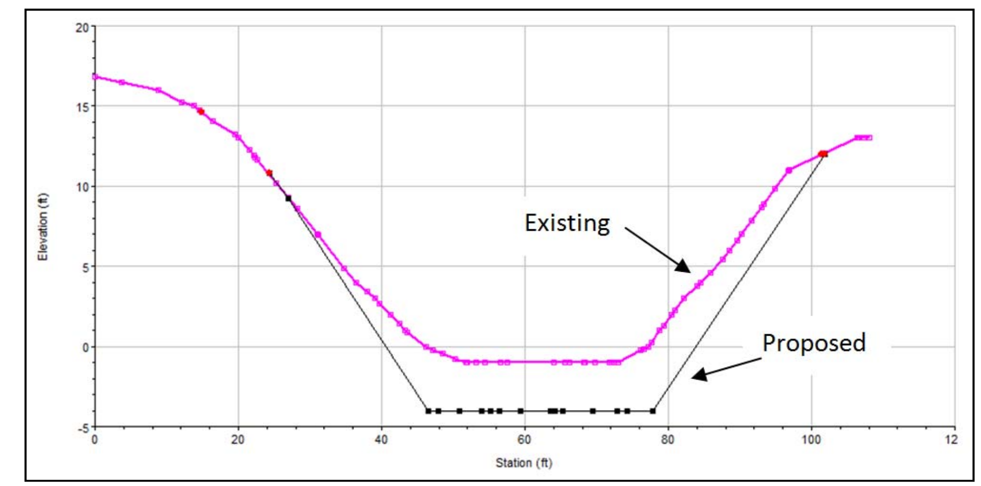


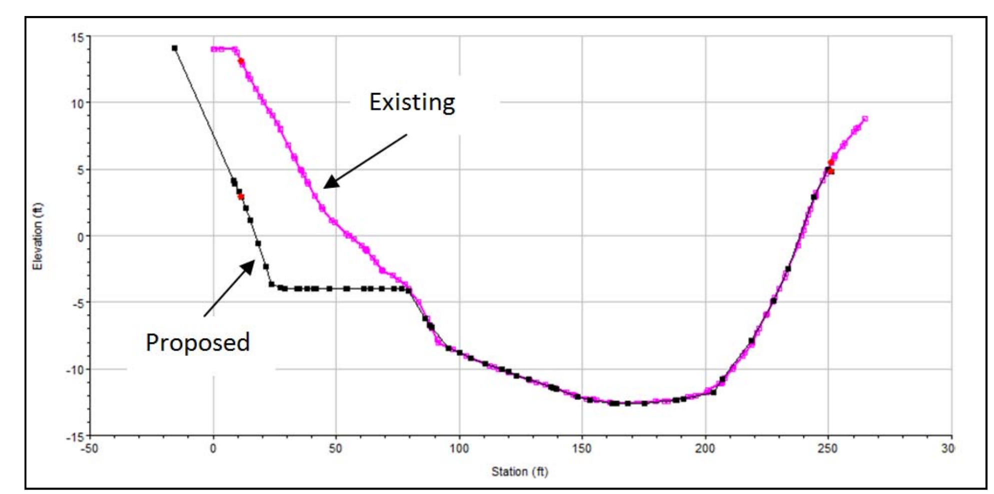
Figure 5-11 Profile and section views of Alternative 13



Section A-A



Section B-B



Section C-C

5.1.3 Analysis Tools and Scenarios

Hydraulic analyses were initially conducted using the one-dimensional computational model HEC-RAS, developed at the Hydrologic Engineering Center of the U.S. Army Corps of Engineers (USACE). This tool aided in development of gross geometries and initial estimates of flow capacities and characteristics. However, development of accurate results was hindered by the one-dimensional computation limitations – the HEC-RAS model is unable to self-identify ineffective flow areas or reflect the multi-directional flow vectors that are generated at the confluence of the Arundell Barranca and the stub channel. The analysis scenarios discussed below were all evaluated using the two-dimensional Adaptive Hydraulics Model (ADH), developed by the Coastal and Hydraulics Laboratory, Engineering Research and Development Center, USACE. The ADH computational mesh and model parameters were developed using the SMS surface water modeling simulation tool, developed by Aquaveo.

The two-dimensional hydraulic performance of the alternatives was first examined using steady-state, rigid boundary assumptions. The effects of high and low tide levels on the hydraulics generated were evaluated, considering steady flow rates up to the 100-year design event. Dynamic simulations of the 24-hour design flood were also completed, with sediment loads applied and erosion/deposition allowed in the outlet, confluence and marina. The results of these analyses are presented below.

5.1.4 Steady-State, Rigid Boundary Simulations – High Tide

The steady state simulations were conducted to examine the performance of the alternatives at selected flow and boundary state conditions. In reality, tide and flow conditions are constantly changing, and the steady state conditions examined may never occur, at least not over the entire model domain at the same instant. They do provide, however, conditions for reference and comparison of hydraulic performance, and typically represent extreme design cases. The high tide case examined in the first of the steady state runs presented below represents the MHHW condition, with a fixed elevation of 5.27 feet (NAVD88) applied at the downstream boundary of the models. The existing condition simulation, which is used for comparison with alternatives simulations, has a steady state flow magnitude of 6000 cfs – an estimate of the capacity of the drainage channel at some point upstream of the Arundell Barranca outlet. For the alternative simulations, the steady state flow magnitude applied was 7500 cfs – the peak flow rate associated with the 100-year design flood.

A detailed view of the outlet performance under existing conditions is presented in Figure 5-12. As previously discussed, design flows alternate through two cycles of supercritical-subcritical transition in the vicinity of the outlet, the first at the drop chute immediately downstream of Beachmont Street, and the second near the location of the sheet pile wall at the downstream end of the outlet.

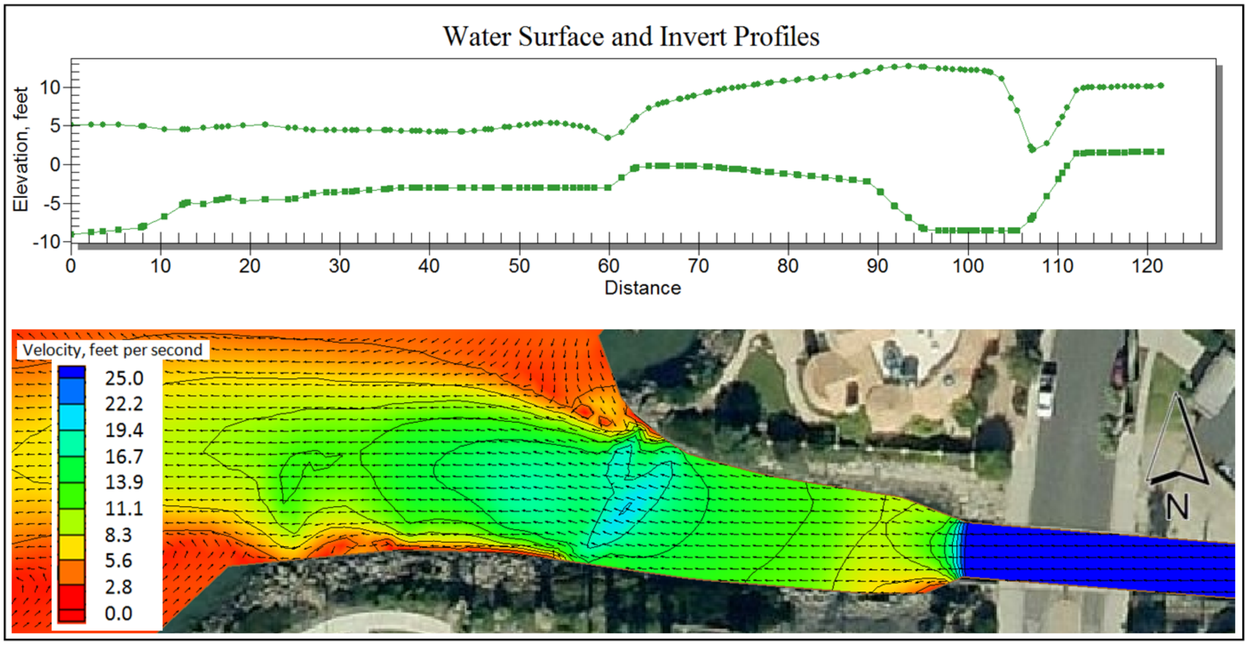


Figure 5-12. Outlet channel water surface profile and velocity variation map, existing conditions, steady state, 6000 cfs, MHHW run

A more extensive view of the velocity variation in the vicinity of the Arundell Barranca outlet under the 6000 cfs MHHW steady state condition is presented in Figure 5-13. Several characteristics of the existing conditions performance may be noted in this figure: the high velocity zone evident in the outlet channel near its downstream end, the impingement of flow vectors against the west bank of the Stub Channel, the reverse gyre north of the outlet that impacts the west bank of the Stub Channel, and the reverse flow paralleling the east bank of the Stub Channel south of the outlet.

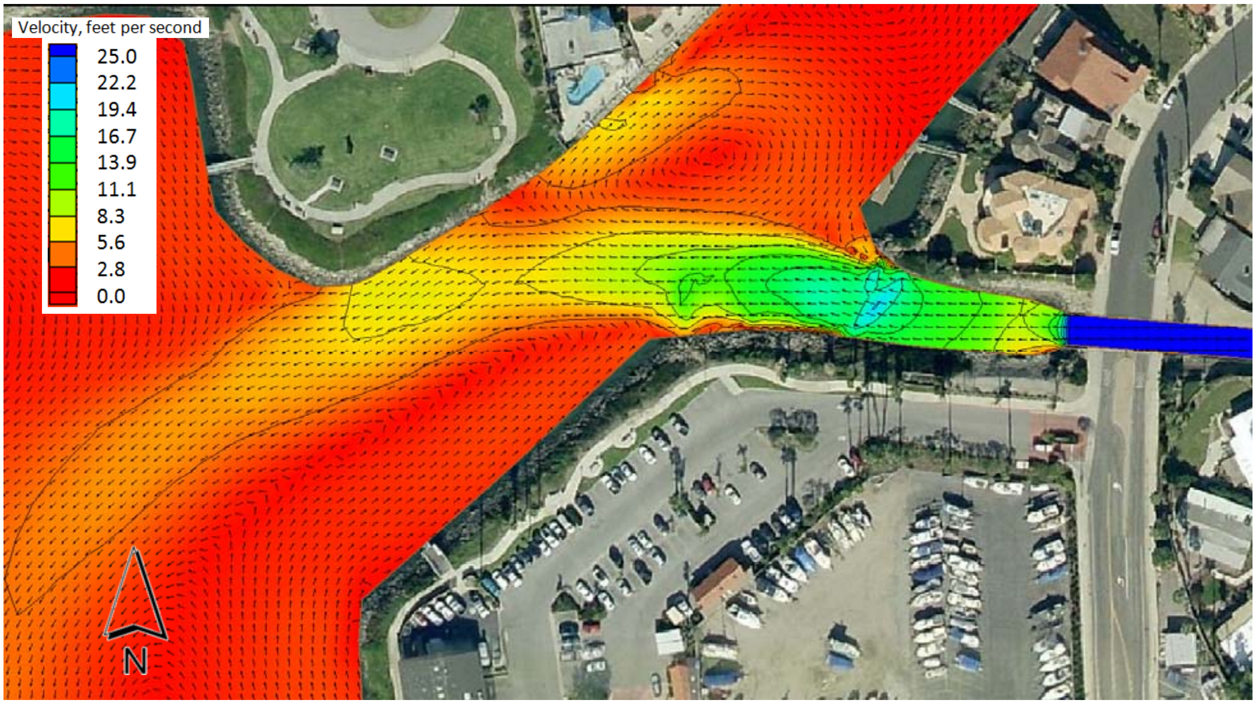


Figure 5-13. Stub channel and outlet channel velocity variation map, existing conditions, steady state, 6000 cfs, MHHW run

A view similar to the above figure is provided for each of the selected alternatives in Figure 5-14. In each case shown, the steady state condition imposed included MHHW at the model boundary and 7500 cfs discharging through the Arundell Barranca. The effects of each alternative on the hydraulic characteristics at the outlet under this flow scenario are summarized in Table 5-1.

Table 5-1. Alternative comparisons, MHHW and steady state high flows

Outlet Configuration	High Velocity Zone in Mouth	Flow Impingement on West Bank of Stub Channel	Reverse Gyre against West Bank of the Stub Channel North of the Outlet	Reverse Flow Paralleling the East Bank of the Stub Channel South of the Outlet
Existing	Local high velocity	Impingement	Well-formed gyre	Diffuse reverse flow
Alternative 1	Slightly reduced velocity	Impingement	Slightly reduced gyre	Slightly increased velocities
Alternative 8	Reduced velocity	No impingement	Reduced gyre	Higher velocities, more focused gyre
Alternative 12	Significantly reduced velocity	No impingement	Significantly reduced gyre	Small gyre only
Alternative 13	Slightly reduced velocity	Reduced impingement	Slightly reduced gyre	Similar to existing

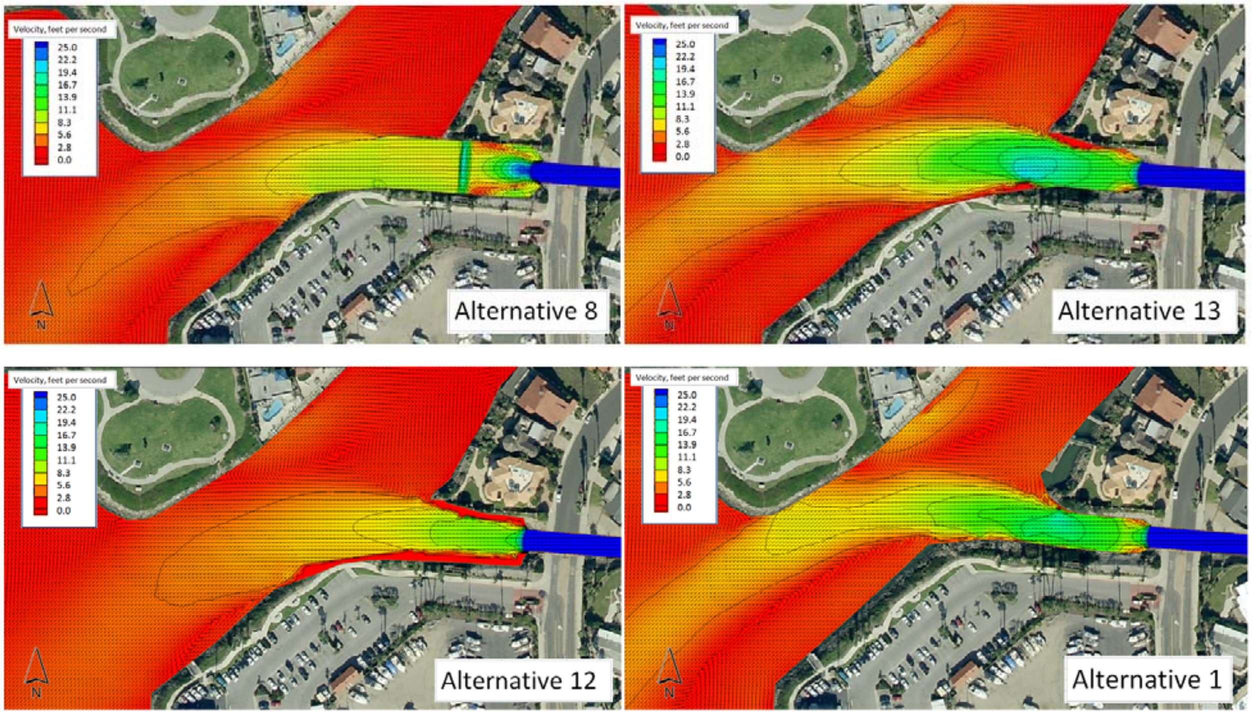


Figure 5-14. Velocity maps for alternatives, 7500 cfs MHHW. Clockwise from top left: Alternative 8, Alternative 13, Alternative 1 and Alternative 12

5.1.5 Steady-State, Rigid Boundary Simulations – Low Tide

The low tide case examined in the set of steady state runs presented below represents the MLLW condition, with a fixed elevation of -0.13 feet (NAVD88) applied at the downstream boundary of the models. As with the MHHW scenario presented in the previous section, the existing condition case used a steady state flow magnitude of 6000 cfs – an estimate of the capacity of the drainage channel at some point upstream of the Arundell Barranca outlet. For the modified Barranca alternatives, the steady state flow magnitude applied was 7500 cfs – the peak flow rate associated with the 100-year design flood.

A detailed view of the outlet performance under existing conditions is presented in Figure 5-15. As with the MHHW condition, design flows alternate through multiple cycles of supercritical-subcritical transition in the vicinity of the outlet, the first at the drop chute immediately downstream of Beachmont Street, and the second located near the location of the sheet pile wall at the downstream end of the outlet. In the MLLW simulation, however, the computed flow depths are much lower and the velocities are higher for an extended distance through the outlet/confluence area, and a third transition zone is formed at the confluence.

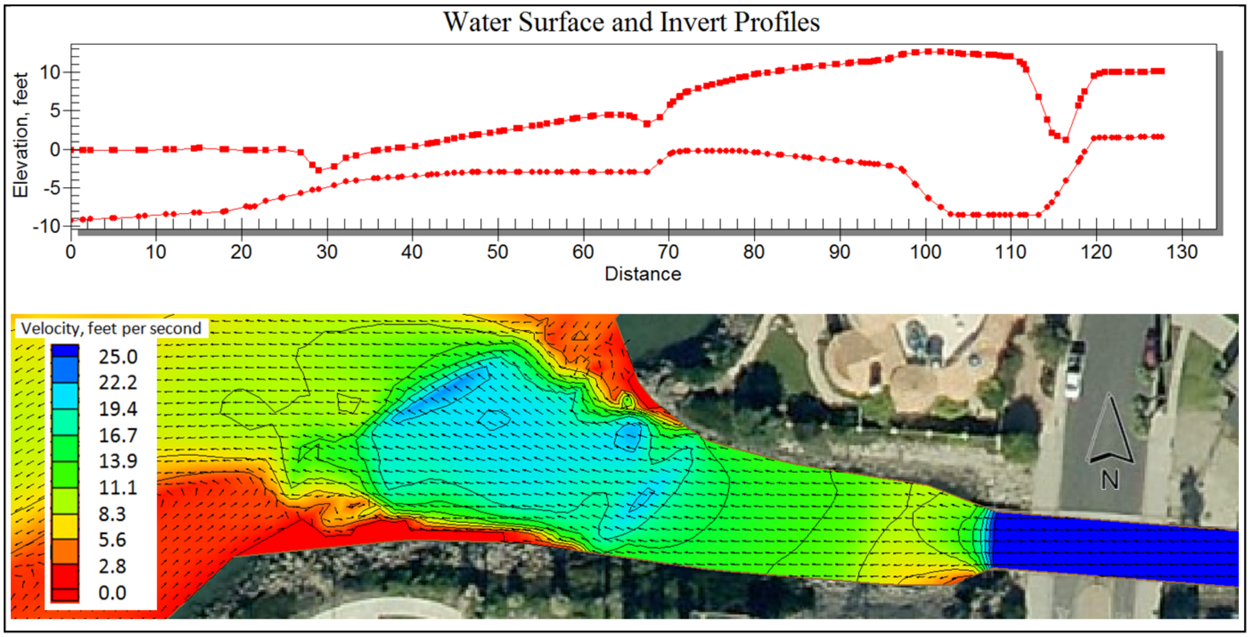


Figure 5-15. Water surface profile and velocity variation map, existing conditions, steady state, 6000 cfs, MLLW run

A more extensive view of the velocity variation in the vicinity of the Arundell Barranca outlet under the 6000 cfs MLLW steady state condition is presented in Figure 5-16. The characteristics noted in the MHHW case are also evident, though more exaggerated in this figure: the high velocity zone evident in the outlet channel near its downstream end, the impingement of flow vectors against the west bank of the Stub Channel, the reverse gyre north of the outlet that impacts the west bank of the Stub Channel, and the reverse flow paralleling the east bank of the Stub Channel south of the outlet.

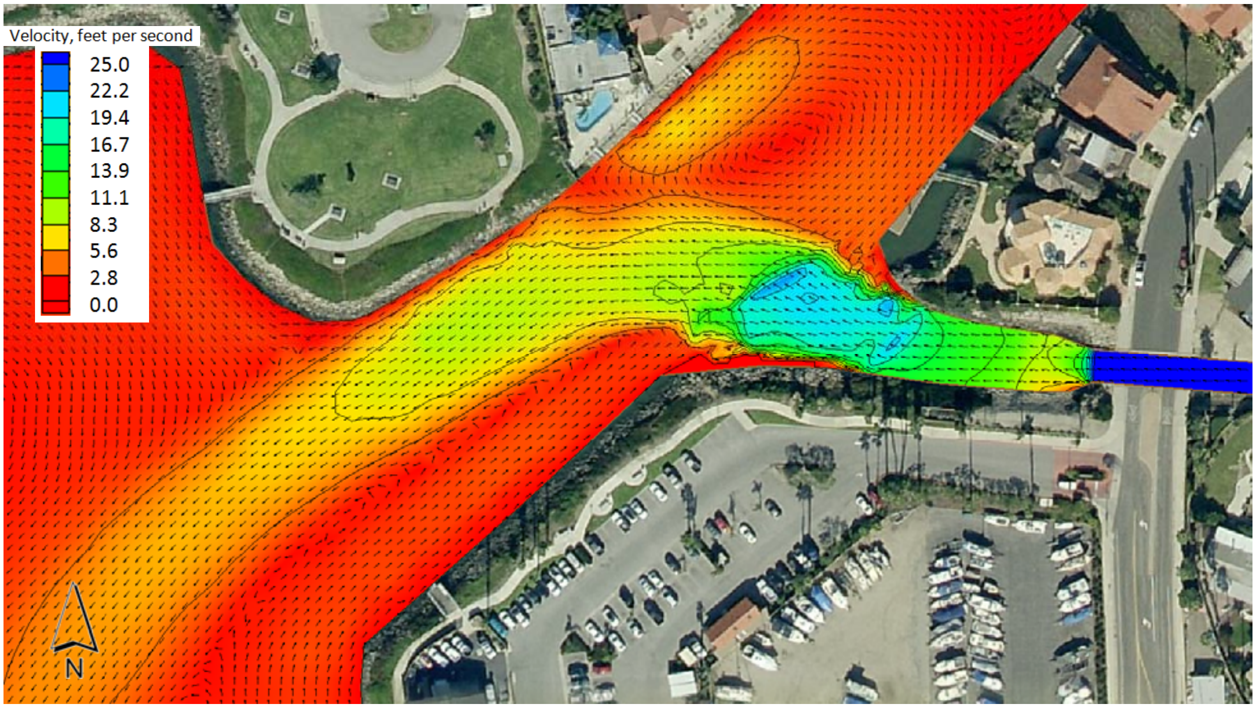


Figure 5-16. Stub channel and outlet channel velocity variation map, existing conditions, steady state, 6000 cfs, MLLW run

A velocity map similar to that presented above is provided for each of the selected alternatives in Figure 5-17. In each case shown, the steady state condition imposed was MLLW at the model boundary and 7500 cfs discharging through the Arundell Barranca. The effects of each alternative on the hydraulic characteristics at the outlet under this flow scenario are summarized in Table 5-2.

Table 5-2. Alternative comparisons, MLLW and steady state high flows

Outlet Configuration	High Velocity Zone in Mouth	Flow Impingement on West Bank of Stub Channel	Reverse Gyre against West Bank of the Stub Channel North of the Outlet	Reverse Flow Paralleling the East Bank of the Stub Channel South of the Outlet
Existing	Local high velocity extends through mouth	Impingement	Well-formed gyre	Semi-focused reverse flow
Alternative 1	Slightly reduced velocity	Stronger impingement	Stronger gyre	Increased velocities
Alternative 8	Reduced velocity	Milder impingement	Reduced gyre	Similar to existing
Alternative 12	Significantly reduced velocity	Milder impingement	Significantly reduced gyre	Smaller gyre
Alternative 13	Slightly reduced velocity	Similar impingement	Similar gyre	Slightly reduced velocities

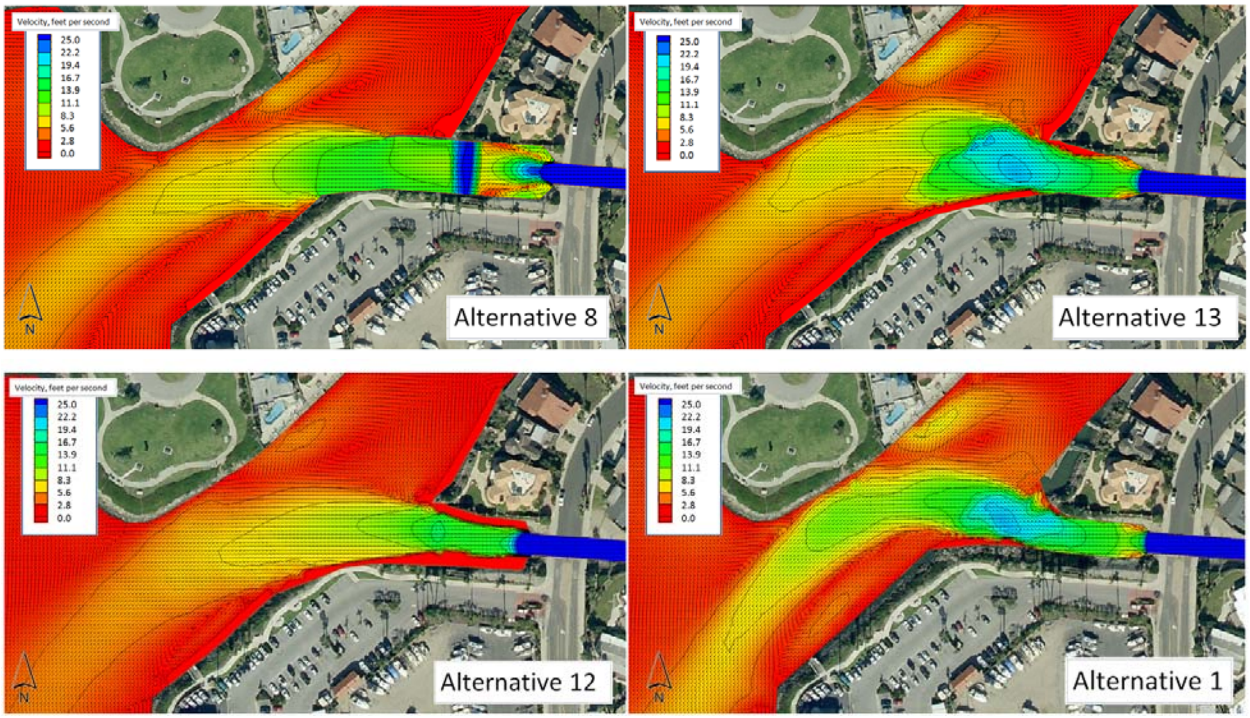


Figure 5-17. Velocity maps for alternatives, 7500 cfs MLLW. Clockwise from top left: Alternative 8, Alternative 13, Alternative 1 and Alternative 12

5.1.6 Movable Boundary Simulations

Dynamic simulations of the 100-year design flood were conducted, with sediment loads representing sediment yield from the Arundell Barranca watershed applied at the upstream end of the model and erosion/deposition allowed in the outlet channel and marina. The with-sediment load design flood analyses were computed assuming two tidal conditions: (1) with flood hydrographs peaking coincident with high tide peaking; and, (2) with flood hydrographs peaking coincident with the occurrence of low tide. The tidal conditions considered are presented in Figure 5-18.

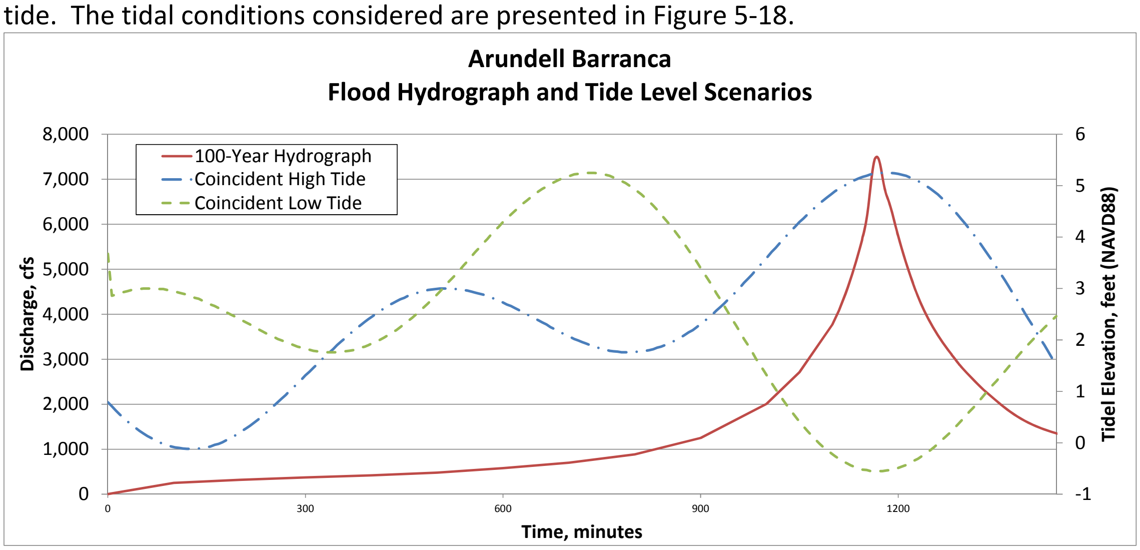


Figure 5-18. 100-year flood hydrograph and two tide level variations used in the dynamic ADH simulations

5.1.7 Movable Boundary Simulations, 24-Hour Design Hydrograph – Peak Coincident with High Tide

The first set of dynamic simulations assumed peak flows from the Arundell Barranca timed to coincide with high tides occurring in the Harbor. Computed deposition patterns for this timing scenario under existing conditions are shown in Figure 5-19. It should be noted that for the existing case peak flood discharges from the Arundell Barranca were capped at 6000 cfs, consistent with the capacity limitations applied in the steady state analyses.

Under the flow and tide condition examined, a significant fraction of the sediment load from the Arundell Barranca accumulates near the confluence, forming a partial dam across the Stub Channel. Deposition depths are greater in the Stub Channel to the north of the confluence, and against the west bank of the Stub Channel. The deposition flare extends southward, along the path of flow toward the Pierpont basin.

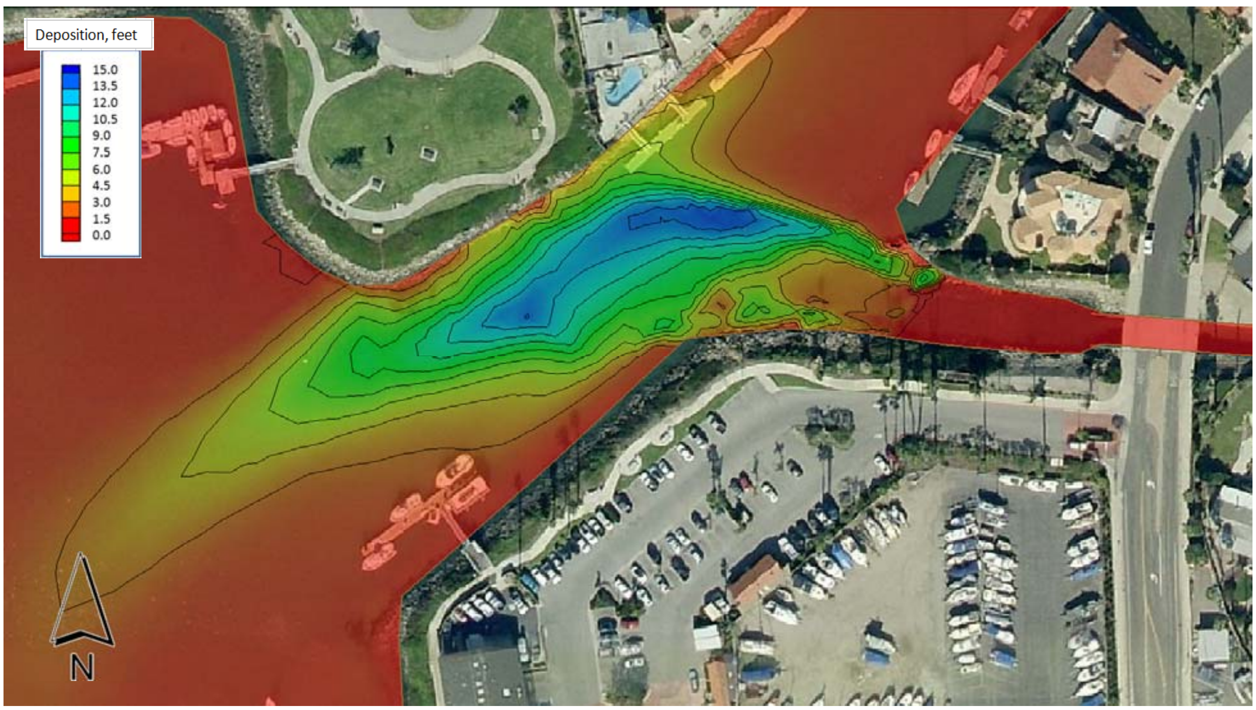


Figure 5-19. Deposition pattern for existing conditions, dynamic simulation of the 100-year hydrograph capped at 6000 cfs, high tide coincides with peak Q

Computed deposition patterns for the alternative conditions under this same timing scenario are presented in Figure 5-20. The flow hydrographs applied in these simulations had a peak discharge of 7500 cfs.

The variation in the outlet configurations has a significant effect on the deposition patterns computed. Alternatives 1 and 13 have deposition patterns that are the most similar to that of the existing condition, though lowering of the outlet invert has moved deposition upstream into the outlet in the case of Alternative 1, and widening the mouth of the outlet has broadened the deposition wedge in the case of Alternative 13. Alternatives 8 and 12 significantly alter the deposition pattern at the mouth.

Both shift the deposition pattern toward the east side of the outlet area. However, in the case of Alternative 8, deposition depths are more uniform throughout the area of accumulation, while with Alternative 12, deposition depths are focused mostly within the outlet itself. The effects of each alternative on the sedimentation characteristics at the outlet under this flow/tide scenario are summarized in the Table 5-3. The expected hydraulic performance sensitivity of each alternative to sediment maintenance, and the ease of maintenance compared to the existing condition are also summarized in this table.

Table 5-3. Alternative comparisons, sedimentation issues

Outlet Configuration	Expected Deposition Location	Potential for Deposition to Block the Stub Channel	Potential for Deposition to Block the Mouth of the Barranca	Potential for Deposition to Impact West Bank Docks	Sensitivity of Hydraulic Performance to Maintenance	Ease of Maintenance (compared to existing)	
						VCWP District	Port District
Existing	Across the Connecting Channel	High	Low	High	Low	Baseline	Baseline
Alternative 1	Similar to Existing, more focus in the mouth of the Barranca	Moderate	Moderate	High	Moderate	Similar to Baseline	Similar to Baseline
Alternative 8	Uniform spread out of mouth into the Stub Channel	Low	Moderate	Low	Moderate to High	Similar in difficulty, but expanded scope and frequency	Less cobble, access improved
Alternative 12	Focused in the mouth of the Barranca	Low	High	Moderate	High	Similar to Baseline	Wider, deeper mouth, easier access
Alternative 13	Very similar to existing, a bit broader wedge at the mouth of the Barranca	High	Low	High	Low	Similar to Baseline	Wider mouth, easier access

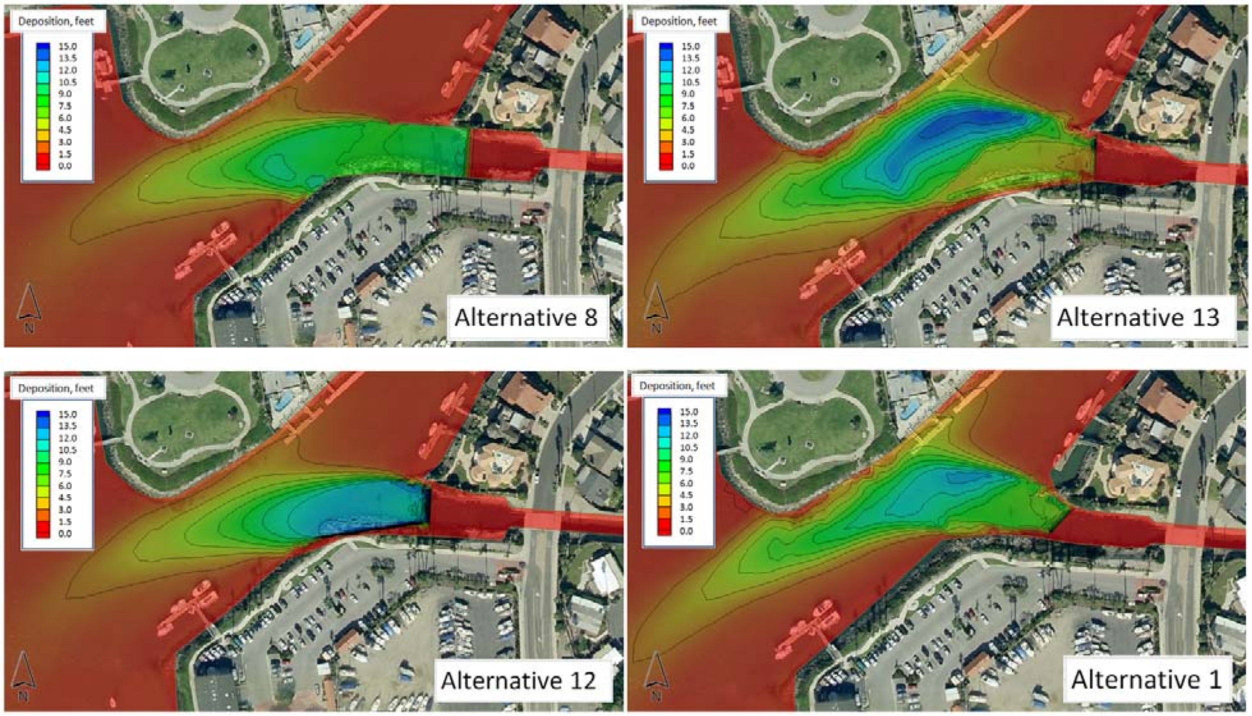


Figure 5-20. Deposition patterns for alternatives, dynamic simulation of the 100-year hydrograph; high tide coincides with peak Q. Clockwise from top left: Alternative 8, Alternative 13, Alternative 1 and Alternative 12

The computed velocity maps for the peak flow time step in the dynamic simulations are presented in Figures 5-21 and 5-22, for existing and alternative conditions, respectively. Comparison of these figures with the steady state maps (Figures 5-19 and 5-20, above), indicates that sediment deposition has some effect on computed velocities and flow patterns at the peak of the simulated event. The strength of the reverse gyre north of the outlet is increased for Alternatives 8 and 12 under the with-sedimentation conditions, but is decreased for existing conditions and Alternative 13, and remains relatively unchanged with Alternative 1. The impingement velocities against the west bank of the Stub Channel are increased for all cases examined under the with-sediment deposition conditions.

Bathymetric changes result in even more significant changes in flow characteristics by the end of the simulation, as indicated in Figures 5-23 and 5-24, where flow vectors for the last time step of the 24-hour simulations are presented for existing and alternative conditions, and in Figures 5-25, 5-26 and 5-27, where flow depths and cross-sectional geometries computed at the last step of the 24-hour simulations are compared. The hydraulic characteristics of subsequent events will be altered as the bathymetry changes. Maintenance of the outlet channel, stub channel and connector channel will be required to avoid impacts to navigation as well as to ensure the containment of flood waters in subsequent events.

Flow velocities expected along the west bank of the stub channel at the point of Arundell Barranca flow impingement will vary by alternative, as well as over time, as illustrated in Figure 5-28. The simulations indicate that slightly higher velocities will be expected at this location under Alternatives 1 and 13, but the duration of this increased peak impact is quite brief, particularly for Alternative 13. Alternatives 8 and 12 are expected to generate significantly lower peak impingement velocities along the west bank of the Stub Channel.

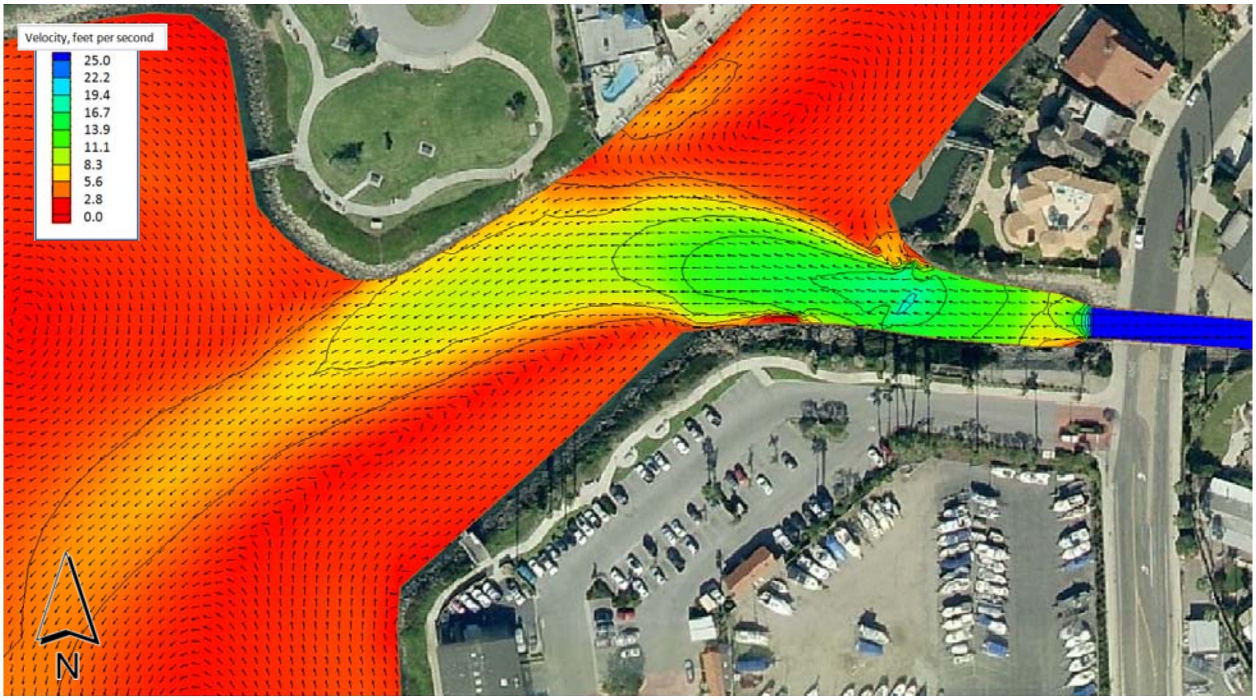


Figure 5-21. Peak discharge velocity map, existing conditions, dynamic simulation of the 100-year hydrograph capped at 6000 cfs, high tide coincides with peak

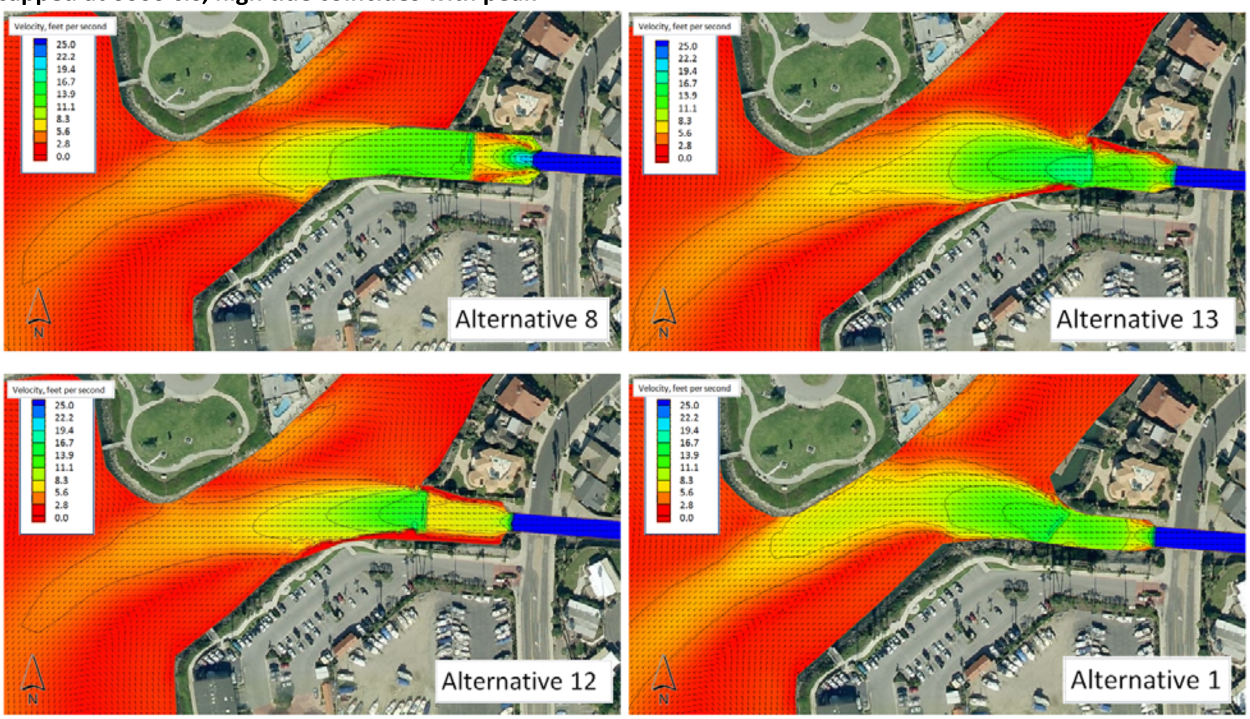


Figure 5-22. Peak discharge velocity pattern, dynamic simulation of the 100-year hydrograph, high tide coincides with peak Q. Clockwise from top left: Alternative 8, Alternative 13, Alternative 1 and Alternative 12

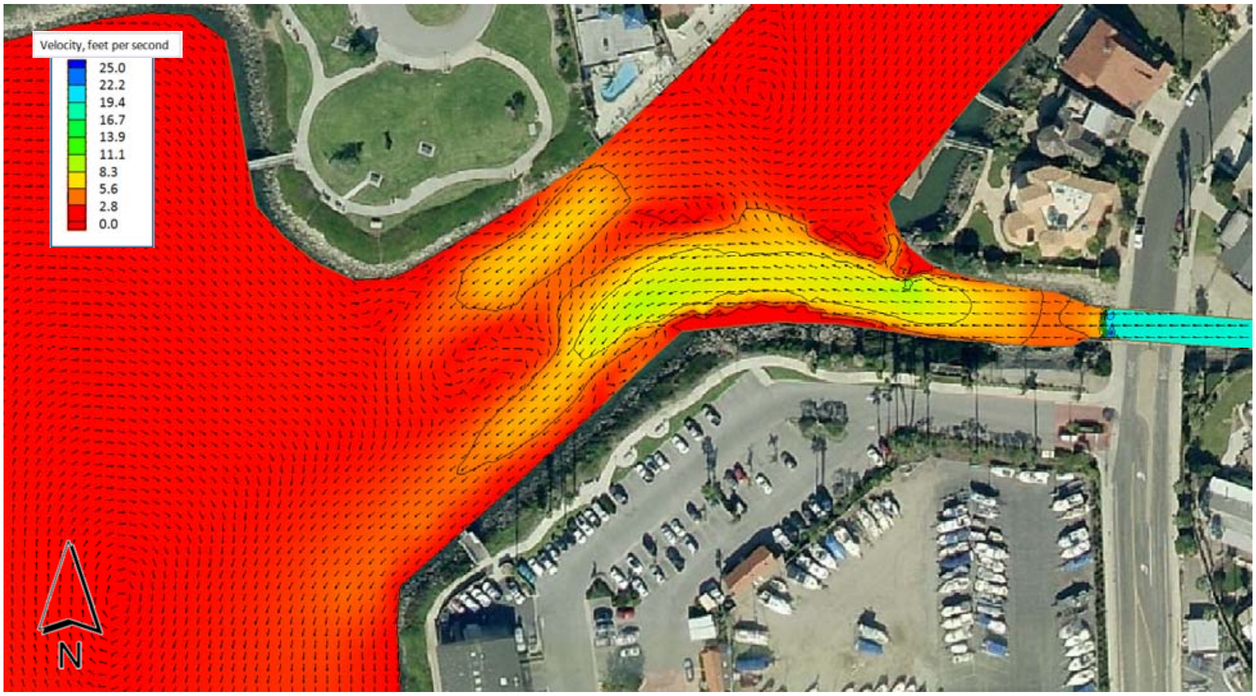


Figure 5-23. Velocity map at the end of the dynamic 100-year (peak flow at high tide) flood simulation, existing conditions – Q at last time step = 1353 cfs

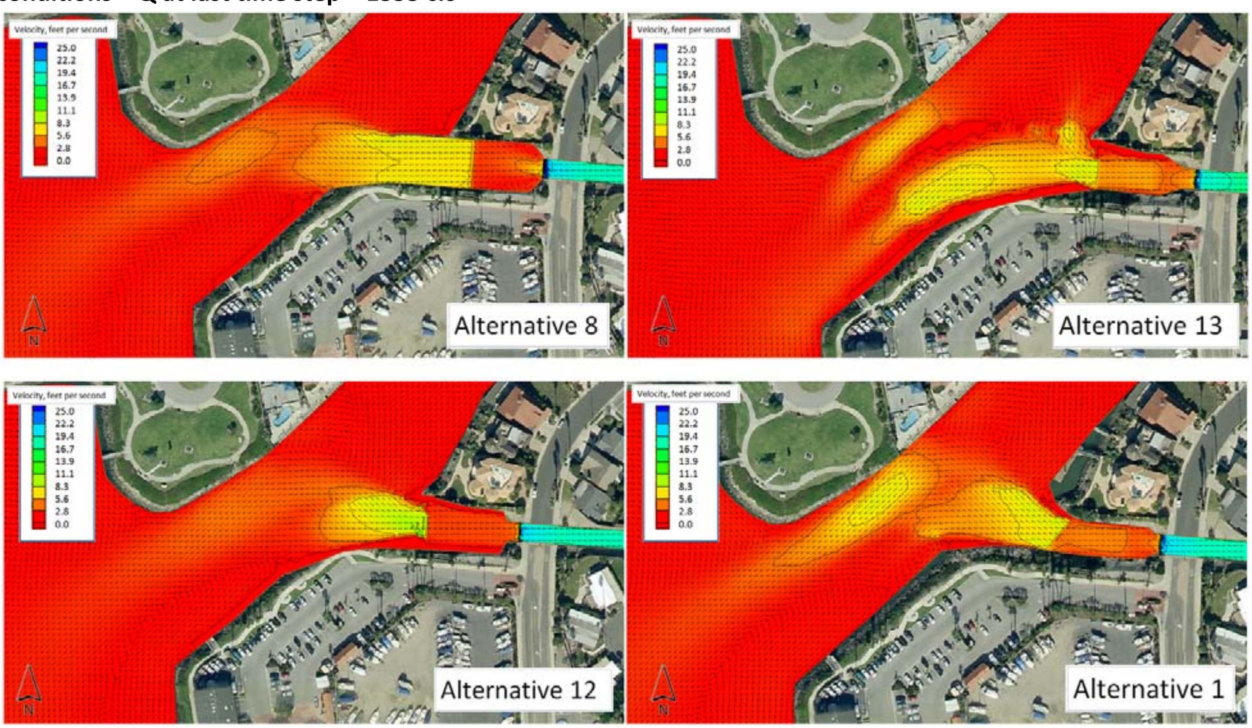


Figure 5-24. Velocity map at the end of the dynamic 100-year (peak flow at high tide) flood simulation, Alternative conditions – Q at last time step = 1353 cfs

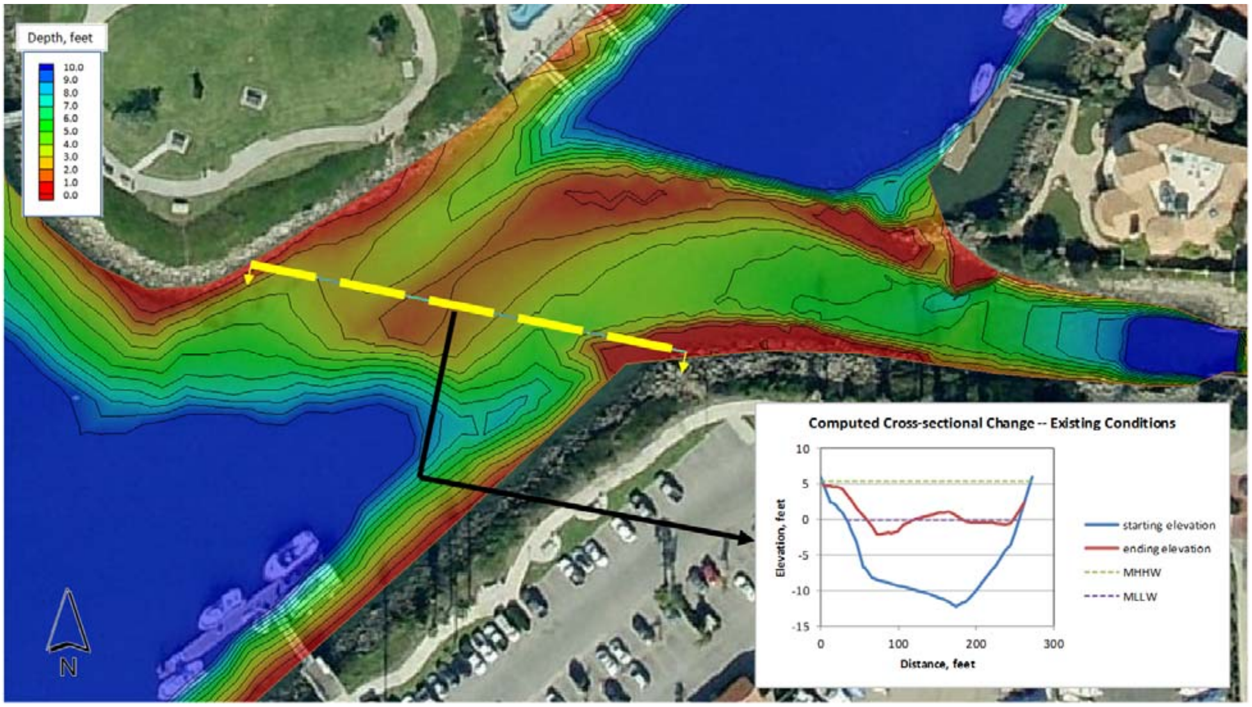


Figure 5-25. Flow depth and cross-sectional change, existing conditions, last time step in the dynamic 100-year (peak flow at high tide) flood simulation – Q at last time step = 1353 cfs

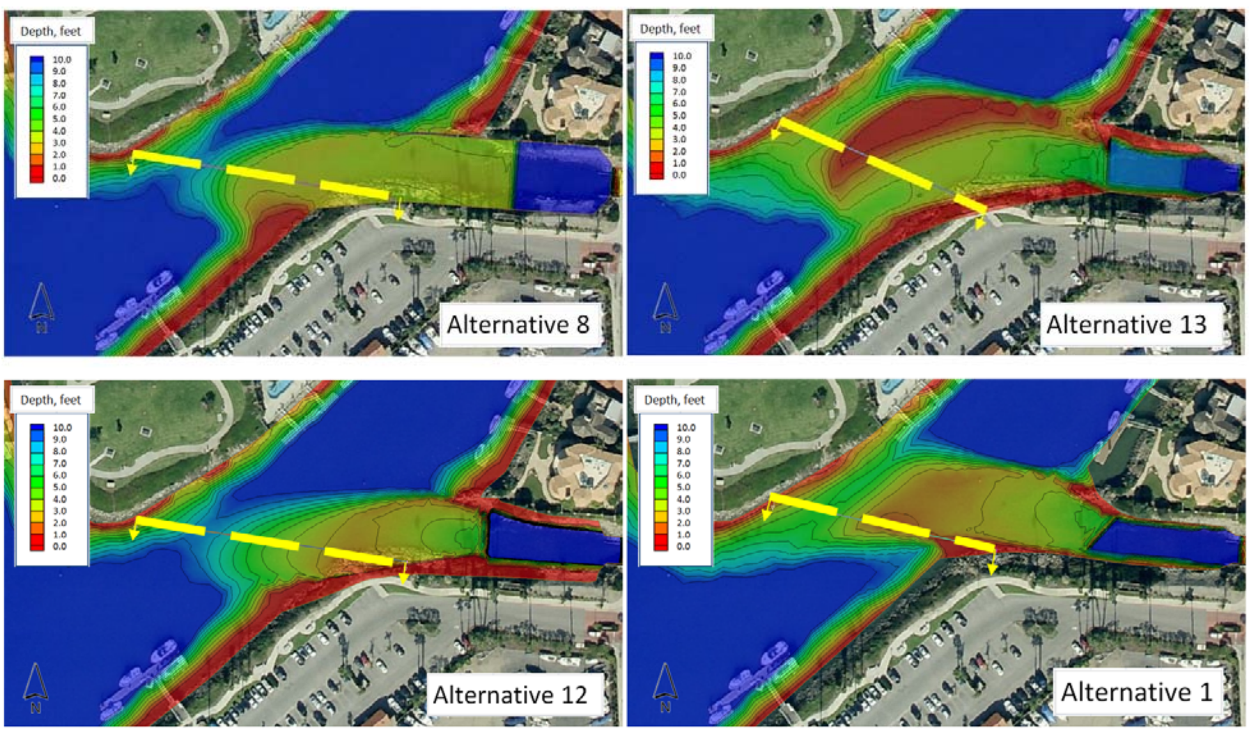


Figure 5-26. Flow depth map, alternative conditions, last time step in the dynamic 100-year (peak flow at high tide) flood simulation – Q at last time step = 1353 cfs (yellow dashed line indicates location of cross-sections shown in Figure 5-23)

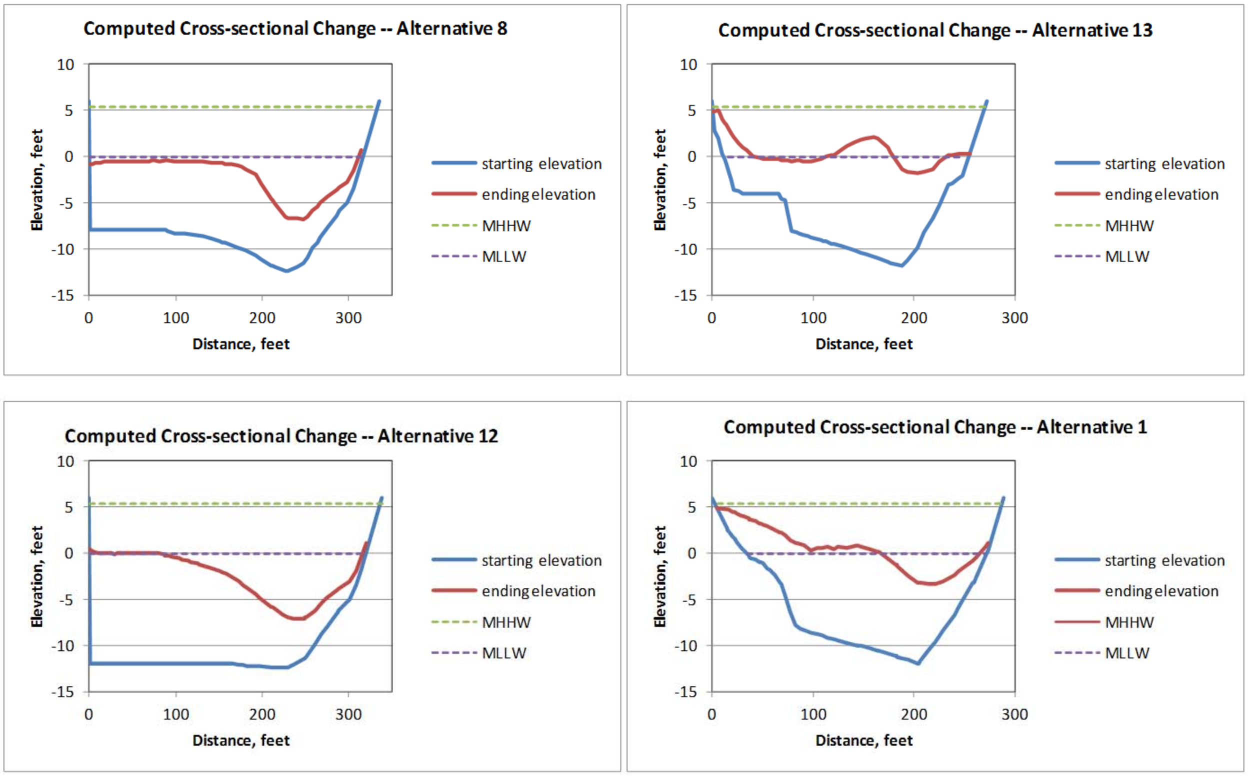


Figure 5-27. Cross-sectional changes computed for the alternatives, dynamic 100-year (peak flow at high tide) flood simulation (section locations are shown in Figure 5-22)

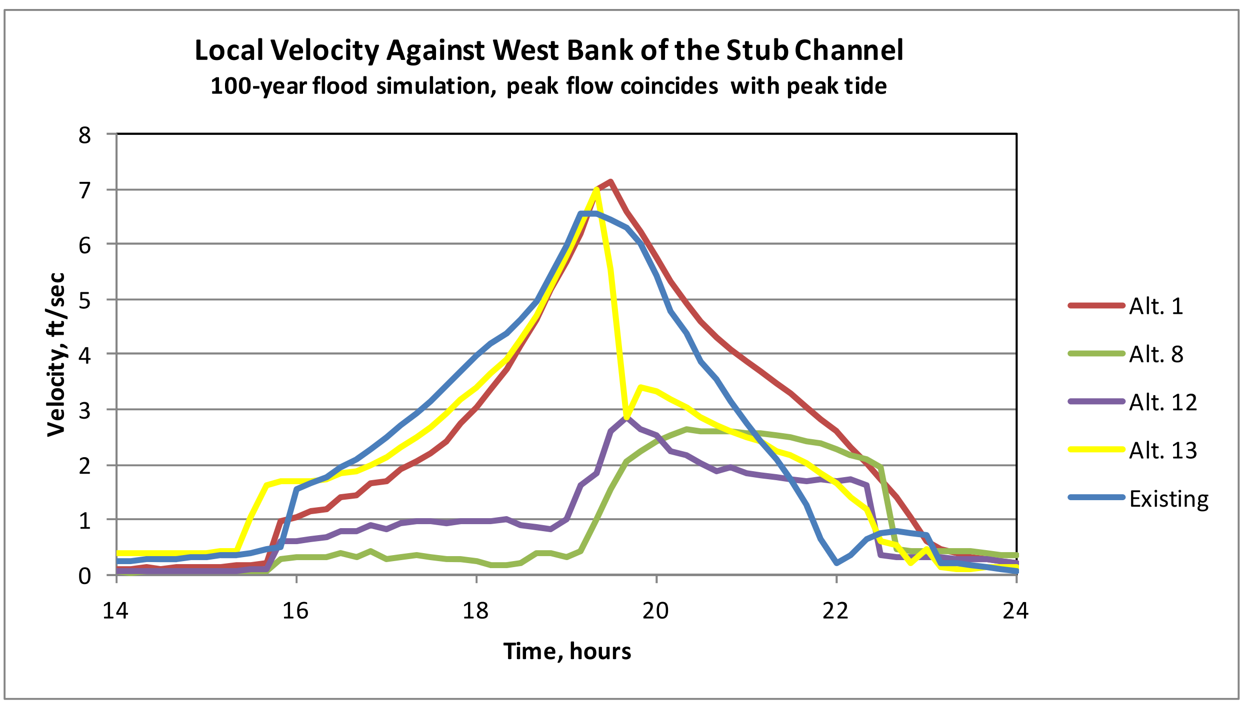


Figure 5-28. Velocity versus time along the west bank of the Stub Channel, existing and alternative conditions, dynamic 100-year (peak flow at high tide) flood simulation

5.1.8 Movable Boundary Simulations, 24-Hour Design Hydrograph – Peak Coincident with Low Tide

The second set of dynamic simulations assumed peak flows from the Arundell Barranca timed to coincide with low tides occurring in the Harbor. Computational difficulties were encountered with nearly all of the model configurations under this scenario, and results were stable only up to the peak of the simulated event. Therefore, results are presented for the peak of the simulation, rather than the end of the event.

Computed deposition patterns for this timing scenario under existing conditions are shown in Figure 5-29. As in the previous simulation, for the existing case flood discharges from the Arundell Barranca were capped at 6000 cfs, consistent with the capacity limitations applied in the steady state analyses. Under this flow and tide condition, the deposition pattern computed through the peak of the simulated event is manifest somewhat uniformly down the length of the Stub Channel, and focused along the west edge. Deposition patterns for Alternatives 1 and 13 are similar, as shown in Figure 5-30. For Alternatives 8 and 12, the deposition patterns are most dissimilar to the existing case, and follow the general trends evident in the high tide dynamic scenario, with less focus along the western edge of the Stub Channel, and more deposition in the outlet channel itself.

Figure 5-29. Deposition pattern for existing conditions, dynamic simulation of the 100-year hydrograph capped at 6000 cfs, low tide coincides with peak Q

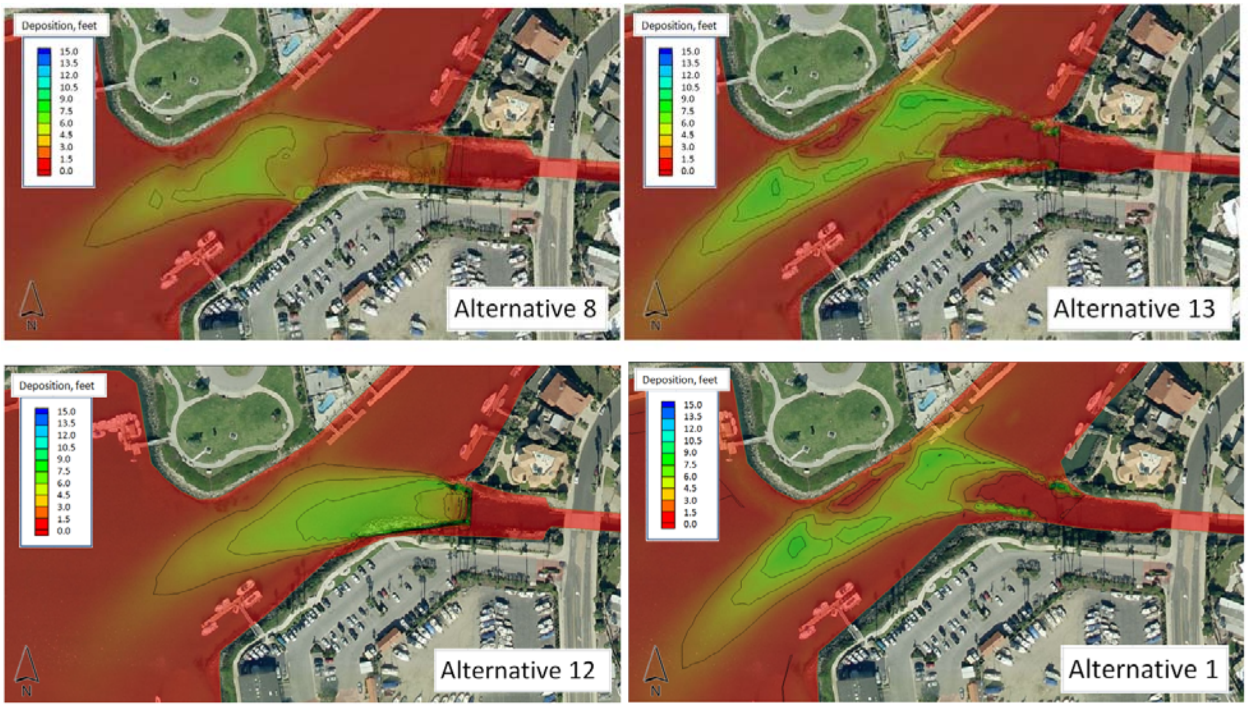


Figure 5-30. Deposition patterns at peak flow time step for alternatives, dynamic simulation of the 100-year hydrograph; low tide coincides with peak Q. Clockwise from top left: Alternative 8, Alternative 13, Alternative 1 and Alternative 12

The computed velocity maps for the peak flow time step in the low tide dynamic simulations are presented in Figures 5-31 and 5-32, for existing and alternative conditions, respectively. As in the case of the high tide dynamic simulations, comparison with the steady state maps (Figures 5-16 and 5-17, above), indicates that sediment deposition does have some effect on computed velocities and flow patterns at the peak of the simulated event. Impingement velocities against the west bank of the Stub Channel are increased, and the reverse gyre north of the outlet is strengthened for all of the cases examined (existing and all alternatives). The reverse flow condition south of the outlet and along the east bank of the Stub Channel is reduced somewhat in the with-sedimentation run with Alternative 1, while slightly strengthened for the existing case, and relatively unchanged with the other alternatives.

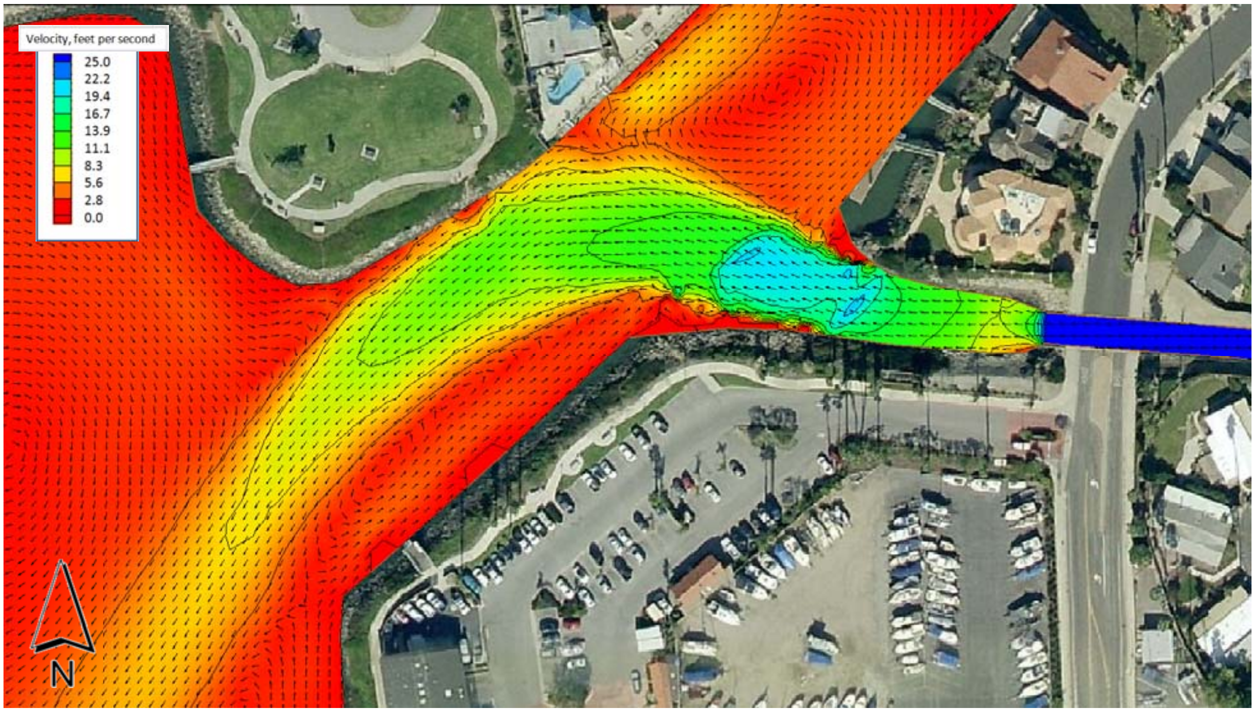


Figure 5-31. Peak discharge velocity map, existing conditions, dynamic simulation of the 100-year hydrograph capped at 6000 cfs; low tide coincides with peak Q

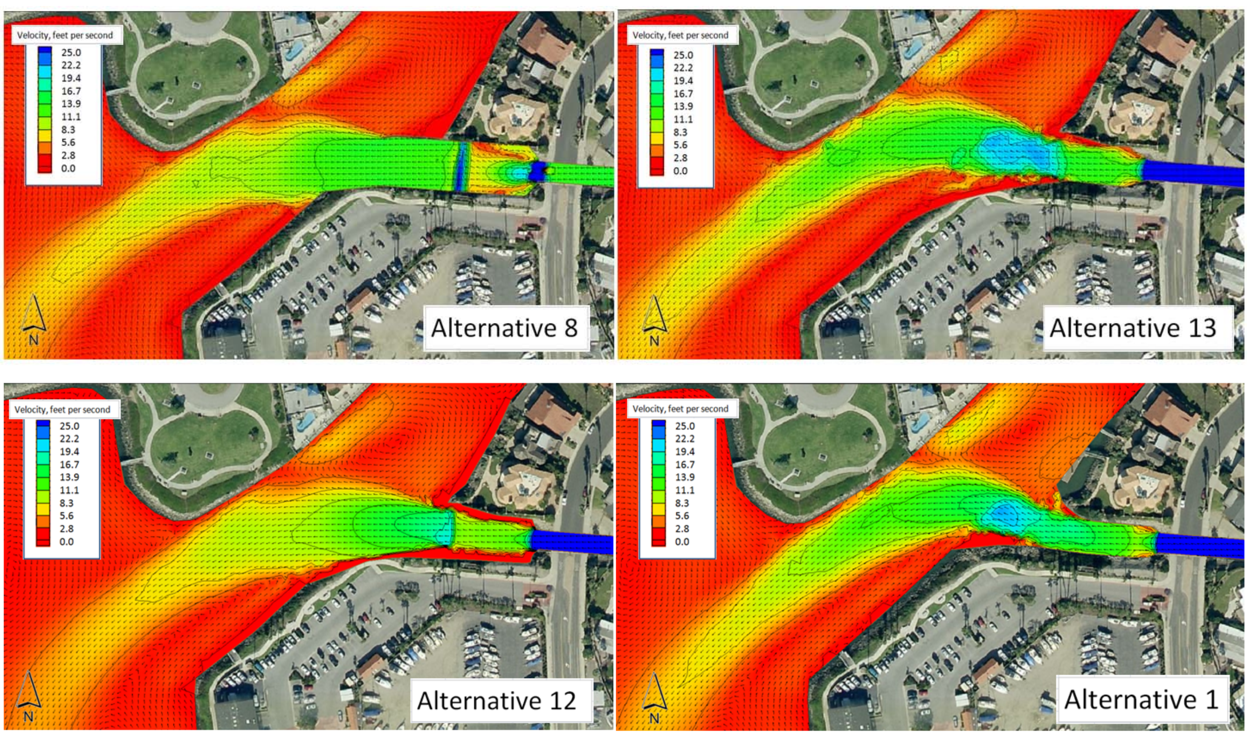


Figure 5-32. Peak discharge velocity pattern, dynamic simulation of the 100-year hydrograph; low tide coincides with peak Q. Clockwise from top left: Alternative 8, Alternative 13, Alternative 1 and Alternative 12

5.1.9 Additional Simulations

Additional simulations were performed to evaluate the effects of the downstream sheet pile configuration, evaluate effects of higher tide levels associated with climate change, and to represent probable sediment deposition conditions under more frequent (5-year) flood events.

Effect of Downstream Sheet Pile Orientation – Alternative 1

As mentioned above, a sheet pile wall was constructed at the downstream end of the outlet as part of the reconstruction project completed in the 1970's. The approximate location and orientation of this sheet pile is shown in Figure 5-33. The top of the sheet pile wall can also be seen in the photograph presented in Figure 5-34. In order to test the sensitivity of the outlet performance to the orientation of this sheet pile wall, a modified version of the Alternative 1 model was developed using the straightened orientation shown in Figure 5-35.

Figure 5-33. View of existing Arundell Barranca outlet with location of sheet pile wall shown

Figure 5-34. Ground view looking downstream through the Arundell Barranca outlet toward the connector channel, with the top of the sheet pile wall shown

Figure 5-35. Potential re-orientation of the sheet pile at the outlet of the Arundell Barranca

The dynamic 100-year flood simulations for the two tidal timing scenarios used previously (flows peaking with tide peaking, flows peaking with tide troughing) were repeated with the modified orientations. Peak flow velocities and flow vectors for these new runs are contrasted with the original results in Figure 5-36. The analyses indicate that the sheet pile orientation does have some effect on the computation results. Under the flow-peaking-at-low-tide simulations, the flows tend to stay more focused with the revised orientation, with stronger impingement on the opposite bank of the stub channel, and a stronger reverse gyre formation along the east bank of the stub channel south of the outlet. The flow-peaking-at-high-tide simulations show slightly decreased velocities in the northwest trending flow vectors on the opposite bank of the connector channel with the revised orientation, and slightly reduced velocities against the west bank of the stub channel, but the overall effect for this scenario appears minor.

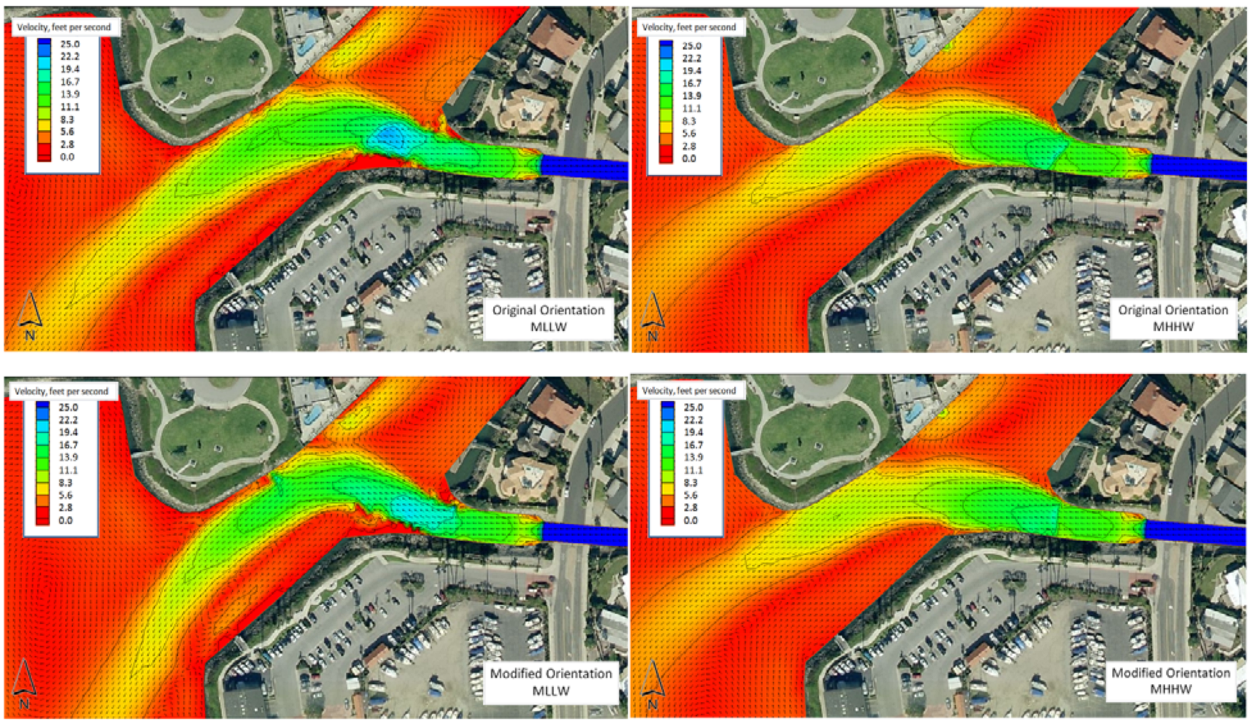


Figure 5-36. Effect of sheet pile orientation on Alternative 1 outlet hydraulics, Q=7500 cfs, high and low tide conditions. Upper left, original orientation, MLLW; Lower left, modified orientation, MLLW; Upper right, original orientation, MHHW; Lower right, modified orientation, MHHW

Effect of Higher Tide Level Associated with Climate Change – Alternative 1

Sea levels are expected to rise in the future as a result of global climate change. Local tide levels may increase by 2 to 5 feet over the next 100 years, according to the planning guidance for the states of California, Oregon, and Washington (CO-CAT, 2010; National Research Council 2012). The sensitivity of the functioning of the Alternative 1 outlet under rising sea level conditions was assessed through additional runs of the ADH model, with downstream boundary conditions modified to reflect this range of potential increase.

The results of these simulations are presented in Figure 5-37. Three scenarios are presented in this figure, with downstream boundary condition levels of 5.27, 7.27, and 10.27 feet (NAVD88), reflecting today's MHHW, and potential future MHHW conditions 2 and 5 feet higher than the current estimate. All of the simulations presented in Figure 5-37 assume a steady state peak discharge of 7500 cfs from the Arundell Barranca.

Sea level rise will have an effect on the hydraulics of the flow as it exits the Arundell Barranca, and the effect will increase as the tide level increases. Higher tide levels will tend to dampen the intensity of the Barranca outflows, and will aid in directing outflows more southward (toward the Pierpont Basin) rather than across the Stub Channel. The effect of a 2-ft rise in the boundary tide level would be limited to areas downstream of the energy dissipator under peak flow conditions, though downstream velocities will be slightly reduced. With a 5-ft rise in the sea level, flood levels are computed to increase by approximately 1 foot in the vicinity of energy dissipator. Flood levels in the Barranca upstream of Beachmont would not be affected under either scenario.

The reduced intensity of the Arundell Barranca outflows expected with sea level rise will decrease the amount of sediment load that will travel out of the outlet area and into the Stub Channel. Maintenance requirements in the outlet area will increase, and maintenance requirements in the Stub Channel will decrease an equivalent amount, as the location of sediment deposition would be expected to shift upstream.

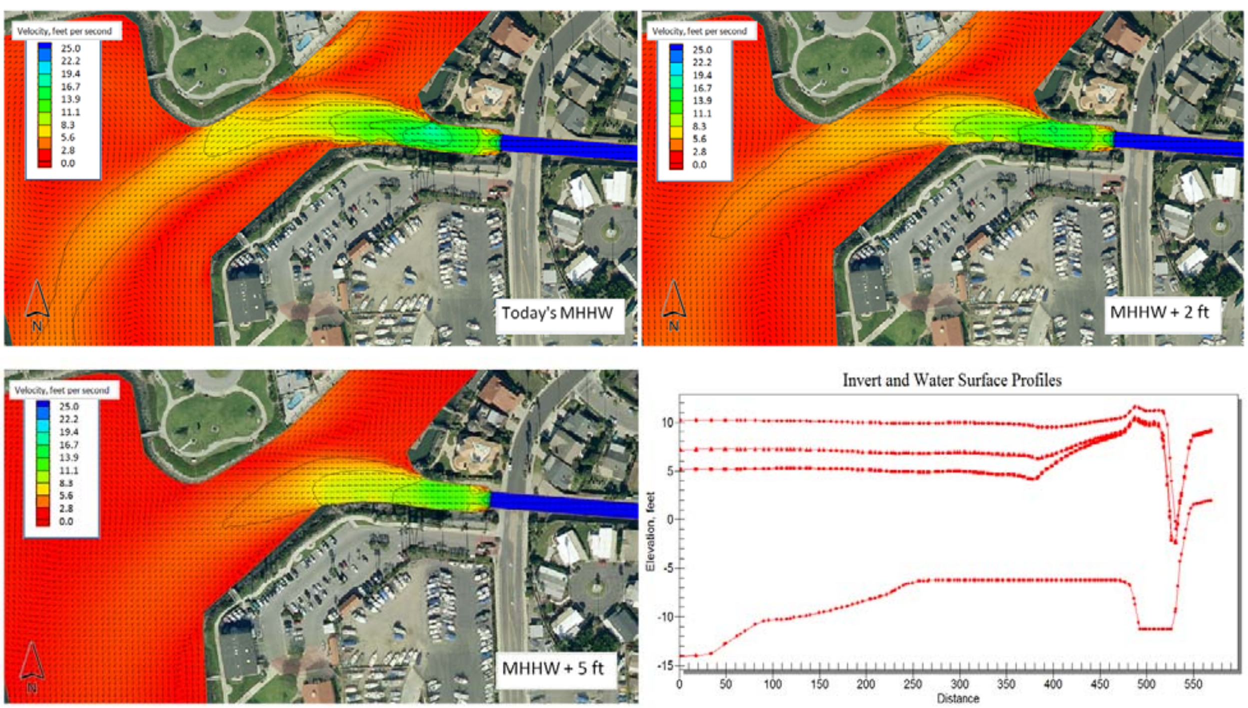


Figure 5-37. Alternative 1 outlet hydraulics, Q = 7500 cfs, for three tidal condition scenarios. Upper left, today's MHHW; upper right, MHHW + 2 ft; lower left, MHHW + 5 ft; lower right, comparison of computed water surface profiles

Dynamic Simulation of the 5-Year Flood Event

An additional set of dynamic simulations were completed to model the expected performance of the existing Arundell Barranca outlet and alternative outlet configurations under the 24-hour, 5-year flood event (peak discharge = 3180 cfs). These simulations are intended to represent events that occur more frequently and therefore may be more relevant to typical dredging and maintenance activities. The simulations were made assuming flood peak timing coincident with high tide (equivalent to the timing scenario applied in the 100-year flood simulation, summarized in Section 5.1.7 of this report).

Computed deposition patterns for this flood and timing scenario under existing conditions are shown in Figure 5-38. Similar to the 100-year flood simulation, a significant fraction of the sediment load from the Arundell Barranca accumulates near the confluence with the Stub Channel. Deposition depths are greatest at along the eastern edge of the stub channel, at the downstream termination of the outlet channel. The deposition flare extends mostly westward across the width of the Stub Channel.

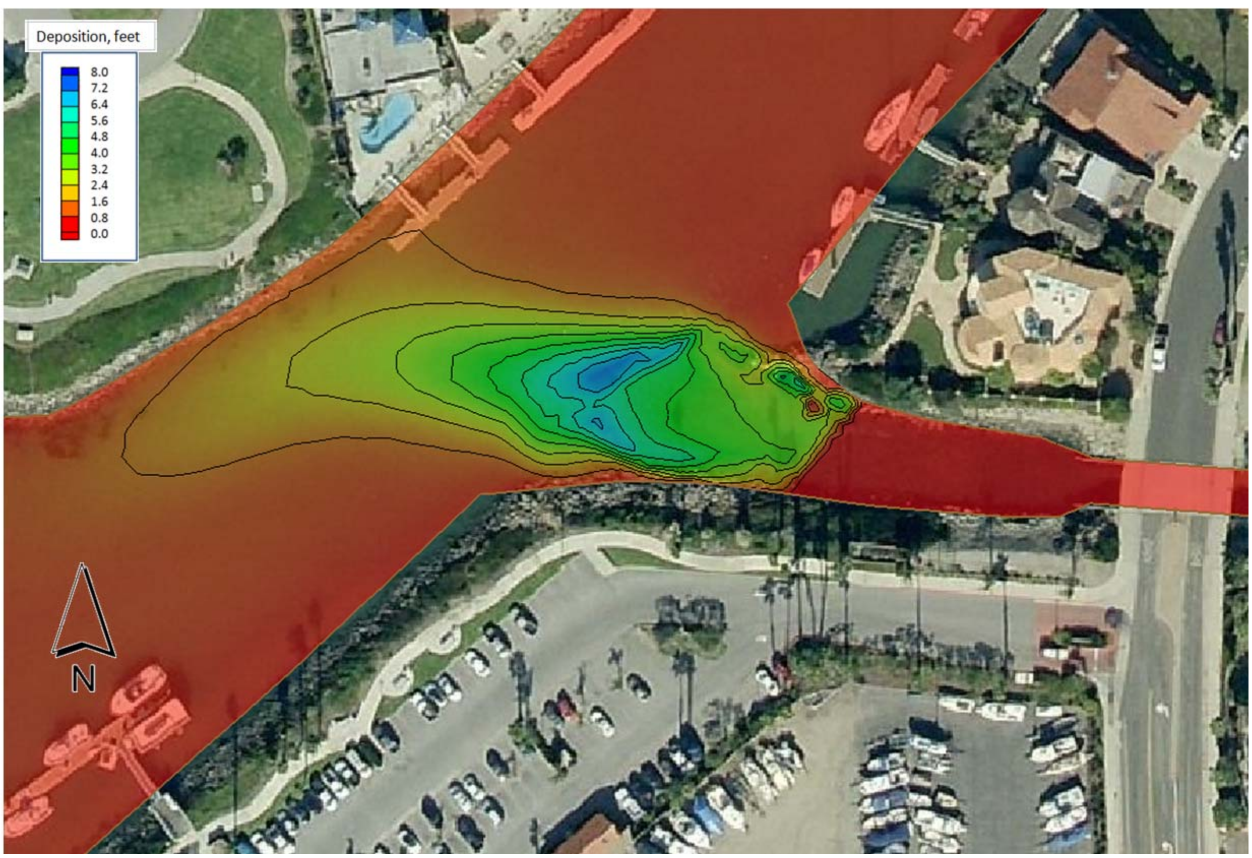


Figure 5-38. Deposition pattern for existing conditions, dynamic simulation of the 5-year hydrograph, high tide coincides with peak Q (end of 24-hour simulation results shown)

Computed deposition patterns for the alternative outlet conditions under this same flood and tide scenario are presented in Figure 5-39.

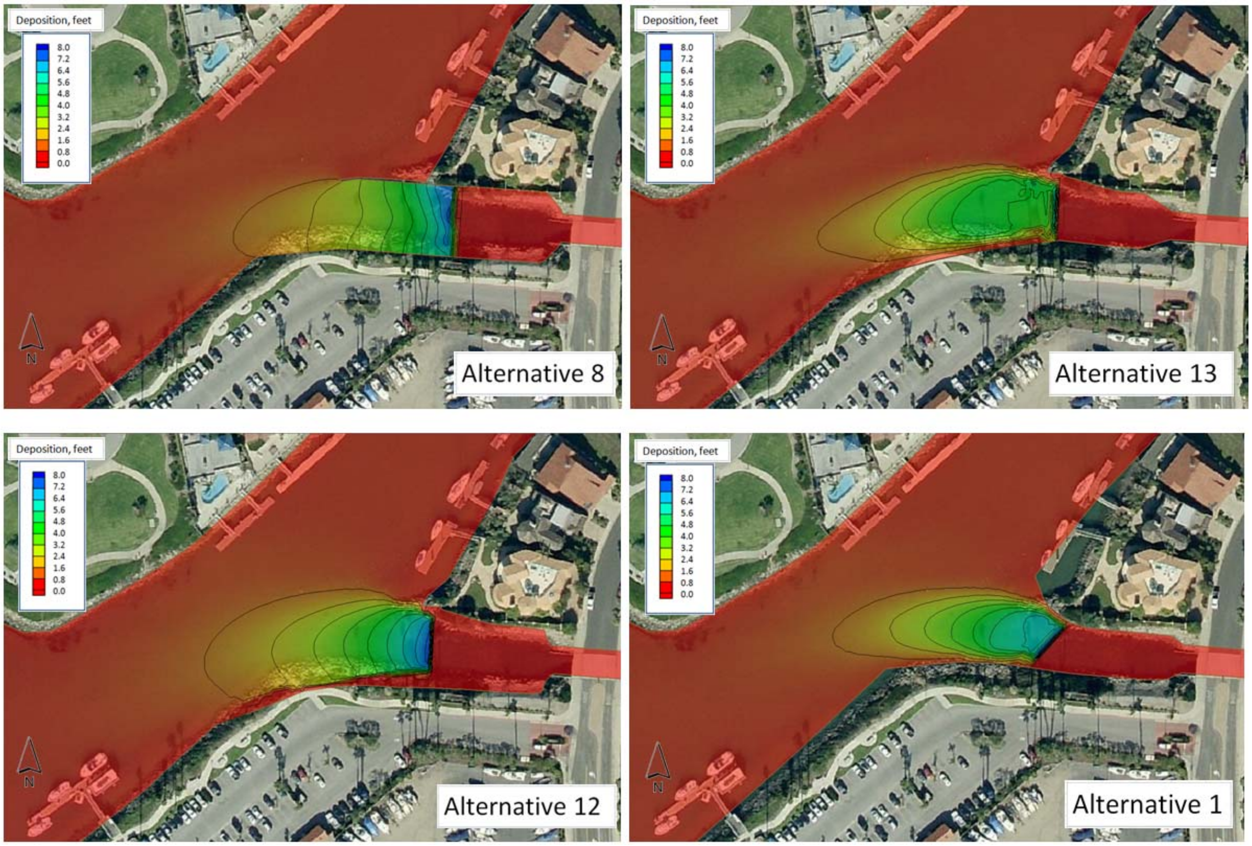


Figure 5-39. Deposition patterns for alternatives, dynamic simulation of the 5-year hydrograph; high tide coincides with peak Q. Clockwise from top left: Alternative 8, Alternative 13, Alternative 1 and Alternative 12 (end of 24-hour simulation results shown)

As with the 100-year event simulation, the outlet configuration is shown to have some effect on the expected sediment deposition pattern at the confluence with the Stub Channel, though less contrast between alternatives is evident under this 5-year flood condition. Each of the alternatives tend to locate the deposition more within the outlet channel itself, rather than in the Stub Channel as is the case under existing conditions. Also, each of the alternatives appear to guide the deposition more in the downstream direction than across the Stub Channel, as was shown to occur in the existing model. Of the four alternative configurations examined, Alternative 8 is expected to most confine the expected deposition within the outlet channel, having the least impact on Stub Channel bathymetry. Alternative 1 has expected deposition patterns that most closely match those of the existing outlet configuration.

5.1.10 Modeling Limitations and Results Discussion

The ADH model is very sensitive to steep slopes in the geometry of the domain. Steep slopes along the sides and invert of the existing and alternative configurations of the Arundell Barranca outlet were adjusted where necessary to meet the model limitations. In areas where vertical containment walls were part of the channel configuration (i.e. the existing channel upstream of Beachmont), the channel sides were represented as no-flow boundaries with an assigned resistance.

The hydraulic and sediment transport performance of the energy dissipator at the outlet of the Arundell Barranca was not evaluated in detail in the simulations conducted for this study. The general geometries and gross resistance conditions associated with the existing and modified energy dissipation structures were approximated, but simulation of the turbulence and three-dimensional complexities was not attempted. It is recommended that the energy dissipator structure to be incorporated into the selected alternative be sized and evaluated in a physical model study, where design features of the dissipator (i.e. chute and baffle block sizes and arrangement, etc.) as well as the outlet channel may be optimized.

The sediment transport and deposition modeling done for this study is an approximation of reality, suitable for comparison of alternative performance, but not for precise determination of bathymetric changes. The transport and deposition of the coarse fraction, which makes up about a third of the historical deposition material, is judged to be reasonably well simulated in the models. The fate of the fine fraction of the sediment load is more approximate, due to re-disturbance and settling processes that involve longer time frames than analyzed in this study. The simulations did not consider the effect of salinity on the deposition rates and patterns of the fine materials passing to the Harbor from Arundell Barranca.

The simulations completed for this study indicate that the tidal conditions occurring at the time of the flood event have a significant effect on the hydraulic characteristics and sediment deposition patterns in the outlet and stub channels. The outlet performance may be very different for identical floods that interact with non-equivalent tidal conditions.

5.2 Low Flow Treatment Alternatives

Alternative 5 and Alternative 9 were advanced from the initial evaluation of alternatives as alternatives that provide potential enhancements of water quality in the Harbor. Both alternatives would be designed to treat typical low flows in the Arundell Barranca Channel. Alternative 5 would treat these flows in series of wetland basins and Alternative 9 would divert the flows the Ventura Water Reclamation Facility.

The Alternative 5 diversion of low flows would occur downstream of the UPRR crossing and requires acquisition of agricultural land for construction of the wetland treatment ponds. An initial layout of the pond system was developed in the initial evaluation of alternatives (Figure 4-6). The location of the treatment ponds is flexible to some degree and could be adjusted based on land availability. The primary constraints on location and configuration are:

- Diversion near UPRR and location of the ponds between UPRR and Harbor Boulevard to provide sufficient topographic fall for diversion and operation of the ponds by gravity;

- Configuration of the ponds to fit the westward sloping land surface without requiring excessive embankment construction – this can be achieved with a series of ponds stepping down in elevation, which may also be advantageous for treatment processes;
- Configuration of the ponds with a shape conducive to uniform flow distribution and prevention of short circuiting – although this might be achieved with in-pond berms, inter-pond piping, or hydraulic controls, a pond length to width ratio of 1.5:1 or greater is desirable.

At the current time, no willing seller has been identified for acquisition of the required land for Alternative 5.

Based on the water quality data collected by the District and the existing and anticipated water quality regulations (see Section 3.5), the primary pollutants of concern for removal in the treatment wetlands are nutrients (especially total and nitrate nitrogen), bacteria, and metals (especially copper and selenium). Although exceedance of water quality standards was also identified in the sampling data for TDS, chloride, sulfate, and pH, concerns over these pollutants are reduced due to discharge to the Harbor where background seawater concentrations provide buffering capacity and the dissolved ion constituents are much higher than those measured in Arundell Barranca.

Wetland treatment has been demonstrated to be effective at removal of all three pollutants of concern. Removal efficiency and effluent quality depend on a number of factors including flow management, vegetation management, residence time, and other factors (Minton, 2002; WERF, 2012). Because the configuration and design of the wetland treatment system depends on land availability, potential removal estimates have been made for this report based on simple assumptions. As presented in Section 4, Alternative 5 would be designed for a target maximum design flow of 5 cfs with a retention time of 24 hours. Based on the flow duration data described in Section 3, these sizing standards would result in diversion of approximately 80% of the total runoff volume in the months May to September and approximately 15% of the total annual runoff volume. This equates to a treatment volume of approximately 239 acre-feet /year during summer months, and approximately 510 af/yr total. A limitation in the pollutant removal estimates is that no data are available form the recent monitoring for flows graeater than 2 cfs. Pollutant concentrations can be expected to vary with flow, but the relationships may be complex and are presently not known. For the purposes of making initial load estimates, characteristic concentrations derived from the water quality data are taken as constant for estimating both the loads and the potential removals. Where removals are based on relationships involving hydraulic residence time or loading, the total load removed was estimated by summing load removals calculated in 1 cfs increments.

Similar to Alternative 5, Alternative 9 was sized for a target maximum diversion of 5 cfs to the VWRF. The viability of Alternative 9 depends upon the ability of the VWRF to accept and treat the diverted flows and on the cost of this treatment. At the time of this report, the VWRF has not confirmed the ability to accept the diversion of up to 5 cfs, but has indicated that only dry weather flows could likely be accepted, requiring seasonal or storm event control of the diversion system. Estimated treatment charges were presented in Section 4 based on information from the City of Ventura, but actual charges are expected to be negotiated between the District and the City if the VWRF is able to accept the flows.

Pollutant removals in most stormwater treatment systems are variable, and wetland treatment is no exception. For the purposes of this report, a range of values reported in the International Stormawater BMP Database (WERF, 2012) and relationships developed from wastewater treatment wetland performance data Minton (2002) have been used to estimate potential removals of pollutants. These methods give fairly wide ranges of potential pollutant removal estimates for Alternative 5. Additional land (surface area for the wetland) would increase hydraulic residence time and reduce hydraulic loading rates in the wetland treatment system, and treatment performance for some parameters is sensitive to these parameters. More detailed design based on weather, flow patterns, storage and buffering capacity of the wetland, soil characteristics, vegetation types, and recreational or aesthetic values might suggest that a larger wetland would provide incrementally cost effective benefits. For this reason, construction cost estimates developed in Section 4 have been modified with an additional contingency of 25% to allow for a potentially desirable increase in size or a change in configuration.

With respect to the Harbor, Alternative 9 results in a complete removal of pollutants in the flows diverted to the VWRF, and thus estimated pollutant removals are a function of the fraction of flow that can be diverted at the 5 cfs target maximum.

Estimated pollutant removal ranges are summarized in Table 5-4. The ranges shown for pollutant removals are derived from applying statistical removal rates or typical effluent concentrations from the International Stormwater BMP Database and applying kinetic or other relationships recommended in Minton and derived from wastewater treatment wetland performance. The relatively large ranges reflect the variability in performance data reported in the literature for constructed wetlands.

Table 5-4. Estimated pollutant removal ranges for Alternatives 5 and 9

Constituent	Characteristic Concentration	Summer Load[1]	Annual Load[2]	Alternative 5 - Wetland		Alternative 9 - VWRF	
				Summer Removal[4]	Annual Removal[5]	Summer Removal[6]	Annual Removal[7]
NO3-N (nitrate)	17 mg/l	6,400 kg	70,900 kg	1,600-3,500 kg	3,700-8,800 kg	5,200 kg	13,100 kg
TN (total nitrogen)	19 mg/l	7,200 kg	79,300 kg	0-1,800 kg	0-4,200 kg	5,800 kg	14,600 kg
Cu (d) dissolved copper	3.3 ug/l	1,200 g	13,800 g	350-400 g	900-1000 g	1,000 g	2,500 g
Cu (t) total copper	3.4 ug/l	1,300 g	14,20 g	370-410 g	950-1050 g	2,600 g	1,000 g
Bacteria (Enterococcus)	500 MPN/100ml	2.09 E+13[3]	1.89 E+12	1.0-1.5 E+12	2.2-3.8 E+12	1.5 E+12	3.9E+12

[1] Summer Load calculated from characteristic concentration from VCWPD sampling 2011 and 2012, and mean daily flow duration calculated from gage near Harbor Boulevard, for period 1964-2005, 1 May to 30 September

[2]Annual Load computed as for summer load, but using mean daily flows over entire calendar year; accuracy limited by lack of water quality data for discharges greater than 2 cfs

[3] Scientific notation - x E+yy indicates x times $(10)^{yy}$

[4] Summer removal calculated using flow volumes and rates in 1 cfs increments; NO3 and TN removal sensitive to loading rate – high end of nitrate range and low end of TN range from ISBMP Database; no adjustment made for temperature.

[5]Annual removal calculated using flow volumes and rates in 1 cfs increments; NO3 and TN removal sensitive to loading rate – high end of nitrate range and low end of TN range from ISBMP Database.

[6]Assumes 100% of load in diverted flow volume is removed from harbor; annual removal includes all flows less than 5 cfs, but VWRF may only be able to accept flows during dry weather periods, which would reduce annual removals.

5.3 Alternative Costs

The construction costs for each of the alternatives were estimated using 2013 cost levels as the basis. The construction cost estimate for Alternative 1 was originally developed by the District based on preliminary engineering design. After completing simulations in this study, the harbor outlet configuration was modified to provide adequate flood capacity and the energy dissipator was enlarged. These changes were incorporated into the Alternative 1 cost estimate. Costs for Alternatives 8, 12, and 13 were estimated as additions to the Alternative 1 costs based on the modified configuration at the harbor outlet. Costs are considered suitable for comparison of alternatives, but are approximate due to the conceptual level of layout for the alternatives. Geotechnical, structural, and marine construction design development outside the scope of this study is needed to refine the costs associated with the harbor outlet. Alternatives 8,12, and 13 will likely require relocation of the high pressure oil line at the Harbor crossing. Based on the agreement for installation and operation of the oil line with the Port District, relocation is the responsibility of the pipeline owner, and relocation costs are not included in the alternative costs. A twenty percent (20%) contingency is included in the estimates. Detailed line item estimates and cost estimating notes are provided in Appendix D.

Maintenance costs for each alternative were also estimated and average annual costs were converted to a present value using a 30-year maintenance period (modified from 20-year period used in the preliminary alternatives analysis). Assumptions for maintenance costs are included in Appendix D.

Table 5-5 presents a comparison of alternative costs. Total cost includes the construction cost and the present value of 30 years of maintenance at an assumed interest rate of 5%. The incremental cost shown represents the increase in total cost for each alternative over the base alternative.

Table 5-5. Comparison of alternative costs

Alternative	Construction Cost, $M	Land Cost, $M	Maintenance Present Value, $M	Total Cost, $M	Incremental Cost, $M
1 – Base Alternative	$11.0	$0	$0.9	$11.9	0
8 – Alt 1 with Cobble Trap	$13.8	$0	$1.4	$15.2	$4.2
12 – Alt 1 with Deeper Outlet Channel	$14.5	$0	$0.8	$15.3	$4.3
13 – Alt 1 with Wider Outlet Channel	$12.3	$0	$0.8	$13.1	$1.2
5 – Alt 1 with Wetland Treatment	$16.1	$1.9[1]	$5.3	$23.3	$11.4
9 – Alt 1 with Diversion to VWRF	$11.5	$0	$28.3	$39.8	$22.7
9 – Alt 9 without treatment charges	$11.5	$0	$0.8	$12.3	$0.5

[1] The District contacted the landowner along the channel between Harbor Boulevard and UPRR Bridge and the landowner informally indicated an unwillingness to sell the property for channel or wetland improvements.

6. CONCLUSIONS AND RECOMMENDATIONS

The alternatives developed and modeled in the detailed evaluation are focused on potential improvements over the baseline Alternative 1. During the detailed evaluation, Alternative 1 was modified from the concept presented in Section 4 to achieve adequate flood performance at the Harbor outlet and the estimated costs were adjusted. Compared to existing conditions, Alternative 1 provides 100-year flood protection and prevents overflow of agricultural and urban land during major events that contributes episodic sediment and pollutant loads to the Harbor. Because of the increased channel capacity, Alternative 1 results in slight increases in velocity and delivery of sediment to the outlet channel. These increases occur only during rare events larger than about 6,000 cfs and would occur over a short duration at the peak of the event. The volume difference in the 100-year event for the Alternative 1 channel is estimated at about 51 af, or about 1.7% of the runoff volume. Sediment delivery to the outlet during this extreme event is increased by about 6,000 cubic yards, or about 5% of the event load. Based on dredging information after the 1998 event, delivery of sediment to other areas of the Harbor under existing conditions is estimated to be as larger or larger than this increase, and to carry potentially higher concentrations of sediment and other pollutants than the channel discharge. Velocities in the outlet and Stub Channel are increased by the additional channel capacity, but differences identified in the simulations are relatively subtle. Under MHHW conditions, Alternative 1 reduces velocities at the mouth of the outlet channel and reduces the gyre at the confluence with the Stub Channel, but velocities of the reverse flow along the east bank of the Stub Channel south of the outlet are slightly increased. Under MLLW conditions impingement of flows on the west bank is slightly increased and the strength of the gyre is increased. These changes would occur at the peak of the design 100-year event, which exceeds the present capacity of 6,000 cfs for a duration of about 40 minutes. Given the variability associated with tidal conditions, changes in velocity attributed to Alternative 1 and that would occur in rare events and for short duration do not significantly alter existing conditions in the Stub Channel with respect to navigation or erosion problems, but these problems are also not significantly reduced.

Similarly, changes in sediment deposition in the Alternative 1 compared to existing conditions are small. General deposition patterns are similar, and differences in volumes are probably well within the accuracy of the simulations. A sediment delivery volume increase of 6,000 cubic yards occurs in the 100-year event, but is a small volume compared to estimated long term average dredging volume of 28,000 cubic yards, or about 2.8M cubic yards over a 100 year period – the expected delivery volume increase represents 0.2 percent of the total deposition expected over 100 years.

Alternatives 8, 12, and 13 were developed to investigate potential improvements to velocity and sediment conditions in the Harbor that could be made at reasonable additional cost compared to the baseline Alternative 1. The alternatives present differences in velocity and sediment distribution, but considering the magnitude of topographic and bathymetric changes involved in the alternatives, these changes are relatively subtle.

As in Alternative 1, Alternatives 8, 12, and 13 make improvements in velocity conditions in the MHHW condition compared to existing conditions, and the improvements are significantly increased compared Alternative 1 for some factors such as west bank impingement and gyre strength. In MLLW conditions, improvements are less significant, and generally similar to existing conditions. Similarly, changes in sediment deposition patterns are different for each alternative, but none of the alternatives provides improvements for all of the factors listed in Table 5-3. In overall performance relative to Alternative 1, Alternatives 8 and 12 provide only modest improvement for substantial change in total cost. Alternative 12 is considered highly sensitive to maintenance and difficult to implement due to the deep outlet channel. Alternative 13 provides minor improvements in velocities, but no significant improvement in sediment performance, and requires modification of the existing parking area outside of existing District right-of way.

Based on relatively small improvements in velocity and sediment conditions relative to Alternative 1, the incremental costs for Alternatives 8, 12, and 13 do not appear to be justified compared to Alternative 1. Alternative 1 hydraulic and sediment performance over a range of tidal conditions is extremely complex, and should be further tested and refined using a physical model to support design efforts.

Alternatives 5 and 9 provide options for addition of water quality benefits to the proposed project and either would be effective at reducing pollutant loads during low flow or summer conditions when temperatures in the Harbor are relatively high and flushing is limited. Although both alternatives have high incremental cost and have potentially significant implementation constraints, further development and implementation of one of the alternatives is recommended to address existing water quality concerns in the Harbor. Alternative 5 is significantly constrained by the lack of available land for wetlands construction. Alternative 9 is constrained by the need to establish a treatment cost at the Ventura Water Reclamation Facility that is acceptable to the District and the City. Implementation of the water quality alternatives is relatively independent of the flood control features.

7. REFERENCES

AET, 2009a. Sampling and Analysis: Ventura Harbor Sediment Investigation, Ventura, California. Final report prepared by Applied Environmental Technologies Inc. for U.S. Army Corps of Engineers, Los Angeles Regulatory Branch; California Regional Water Quality Control Board, Los Angeles Region; and U.S. Environmental Protection Agency, Wetlands Regulatory Office. April 9, 2009.

AET, 2009b. Sampling and Analysis: Ventura Keys Connecting Channel Sediment Investigation, Ventura, California. Final report prepared by Applied Environmental Technologies Inc. for U.S. Army Corps of Engineers, Los Angeles Regulatory Branch; California Regional Water Quality Control Board, Los Angeles Region; and U.S. Environmental Protection Agency, Wetlands Regulatory Office. April 10, 2009.

CH2M HILL, 2006. Arundell Barranca Deficiency Study, Prepared for Ventura County Watershed Protection District. Ventura, CA

CH2M HILL, 2007. Arundell Barranca Detention Basin Conceptual Design, Prepared for Ventura County Watershed Protection District. Ventura, CA

City of Los Angeles Bureau of Engineering Storm Drain Design Division, 1972. Arundell Barranca Hydraulic Energy Dissipator Model Study. Los Angeles, CA.

City of Ventura, 2005. Arundell Barranca Sediment Discharge Volumes. Letter dated August 5, 2005.

City of Ventura, 2010. Letter from Ray Olson, City of Ventura to Jeffrey Shu, State Water Resources Control Board, dated 6 July 2010.

CO-CAT, 201. Sea Level Rise Task Force of the Coastal and Ocean Working group of the California Climate Action Team, State of California Sea-Level Rise Interim Guidance Document, October, 2010.

Cotton, Shires and Associates, 1999. Ventura Keys and Arundell Barranca Watershed Project. Project Alternatives Report, April 2, 1999.

Exponent, 1999. Ventura Keys and Arundell Barranca Watershed Project: Hydraulic and Sediment Transport Design. Draft report prepared for City of San Buenventura by Exponent Failure Analysis Associates, Costa Mesa, CA, March, 1999.

Harbor Data, 2010. Compilation of City of Ventura and Ventura County water quality samples, 2 January 2002 to 2 November 2009, 5175 samples at seven locations, analytes fecal coliform, total coliform, and *Enterococcus*.

Jaoshvili, Sh.V., 1986. Fluvial sediment and formation of beaches of Black Sea coast in Georgia. Sabchota Sakartvelo, Tbilisi, Georgia.

Kadlec, Robert H. and Robert L. Knight, 1996. Treatment Wetlands. CRC Press, Lewis Publishers.

Minton, 2002. Stormwater Treatment. Resource planning Associates, Seattle, WA. 2002.
Mussetter, R.A., Lagasse, P.F., and Harvey, M.D., 1994. Sediment and Erosion Design Guide. Prepared for Albuquerque Metropolitan Arroyo Flood Control Authority (AMAFCA) by Resource Consultants & Engineers, Inc, Fort Collins, Colorado, RCE Ref. No. 90-560, November 1994.

National Research Council, 2012. Sea Level Rise for the Coasts of California, Oregon, and Washington: Past, Present, and Future. Committee on Sea Level rise in California, Oregon, and Washington, Board on Earth Sciences and resources and Ocean Studies Board, National Academies press, Washington, D.C.

Nikitin, Ya.A., 1951. Determination of bed load discharge in rivers of Middle Asia. Gidrotechnika and Melioratsiya, 10, Moscow, Russia, p.54-57.

Pers. Comm., John Lagomarsino, 2012. Telephone conversation with John Lagomarsino, Ventura Watershed Protection District Maintenance Supervisor, 5 March 2012

Pers. Comm. Ray Olson, 2012. Email regarding status of 303(d) review at LARWQCB, 7 Mar 2102.
Stillwater Sciences, 2011. Synthesis Report, Santa Clara River Estuary Subwatershed Study, prepared for City of Ventura. March 2011.

Pers. Comm. Richard Parsons, 2012a. Telephone conversation with Richard Parsons, RWP Dredging Management, 5 March 2012.

Pers. Comm. Richard Parsons, 2012b. Telephone conversation with Richard Parsons, RWP Dredging Management, 7 March 2012.

RBF, 2008. Lake Canyon Dam Alternative Analysis, Prepared for Ventura County Watershed Protection District. Ventura, CA

Romashin, V.V., 1990. Assessment of fluvial sediments supply to beaches of Sochi. In: Issues relating to improvement of coastal protection methods, V.G. Ribka (editor), Moscow, Russia, p. 20-30.

Shvidchenko, A.B., 1997. Field measurements of sediment transport in a piedmont reach of the Laba River. Journal of Hydrology (New Zealand), 36 (2), p.173-181.

Thompson, S.M., 1985. Transport of gravel by flows up to 500 m^3/s, Ohau River, Otago, New Zealand. Journal of Hydraulic Research, 23 (3), IAHR, Delft, The Netherlands, p.285-303.

USGS, 2012. USGS Water Data for the Nation, http://waterdata.usgs.gov/nwis.

VCWPD, 2005. LiDAR topographic mapping. Ventura County Watershed Protection District, Ventura, CA.

VCWPD, 2012. Arundell Environmental Monitoring Final Report, Ventura County Watershed Protection District, 15 February 2012.

Ventura Port District, 2012. Updated estimates of dredging volumes, 1993 to 2011, prepared by Richard Parsons, RWP Dredging Management, February 2012.

WERF, 2012. International BMP Database, Pollutant Category Summary Statistical Addendum for TSS, Bacteria, Nutrients, Metals. Prepared by Geosyntec Consultants and Wright Water Engineers, July, 2012.

Appendix A

Monitoring Information

Water Surface Elevations

Water levels were monitored every minute using Solinst Levelogger Junior at six monitoring locations within Ventura Harbor and Ventura Keys. The locations of the monitoring stations, denoted as LL1, LL2, LL3, LL5, LL6, and LL7 are shown in Figure A1. Barometric pressure was also recorded every minute with Solinst Barologger Edge whose data were used to compensate water level readings recorded by Levelogger Juniors. Leveloggers are still deployed at LL5, LL6 and LL7 to monitor the change in water surface elevation if there happens to be a flood event this winter. No data have been retrieved from LL7.

Figure A1. Map Showing Locations of Monitoring Stations and Survey Control Points

Water levels recorded at LL1, LL5, and LL6 were converted to water surface elevations (MLLW) first by computing the Levelogger elevation at each monitoring station. This was done by subtracting water level recorded at a specific time/date from water surface elevation surveyed on the same time/date at a corresponding monitoring station. Recorded water levels were then added to computed Levelogger elevation to convert to water surface elevations. Because water surface elevations were not surveyed near LL2 and LL3, the elevations of leveloggers at these locations were estimated by averaging the differences between water surface elevations at LL5 and water levels recorded at LL2 or LL3. Table 1A is summarizes survey dates and control points (denoted as CP2 and CP 4 in Figure A1) used during the survey.

Monitoring Station	Survey date/time	Control Point (MLLW, ft)
LL1	Jan 23, 2012	CP2, 12.53 ft
LL5	Jan 13, 2012	CP4, 14.42 ft
LL6	Jan 13, 2012	CP2, 12.53 ft

Table A1. Survey dates and control points

Figure A2 presents the five minute running average of measured water surface elevations (MLLW) at LL1, LL2, LL3, LL5, and LL6 from January 9, 2012 through January 11, 2012. The figure also shows measured water surface elevations (MLLW) at NOAA's Los Angeles Outer Harbor station (ID 9410660) with offsets applied for Ventura station (ID 9411189).

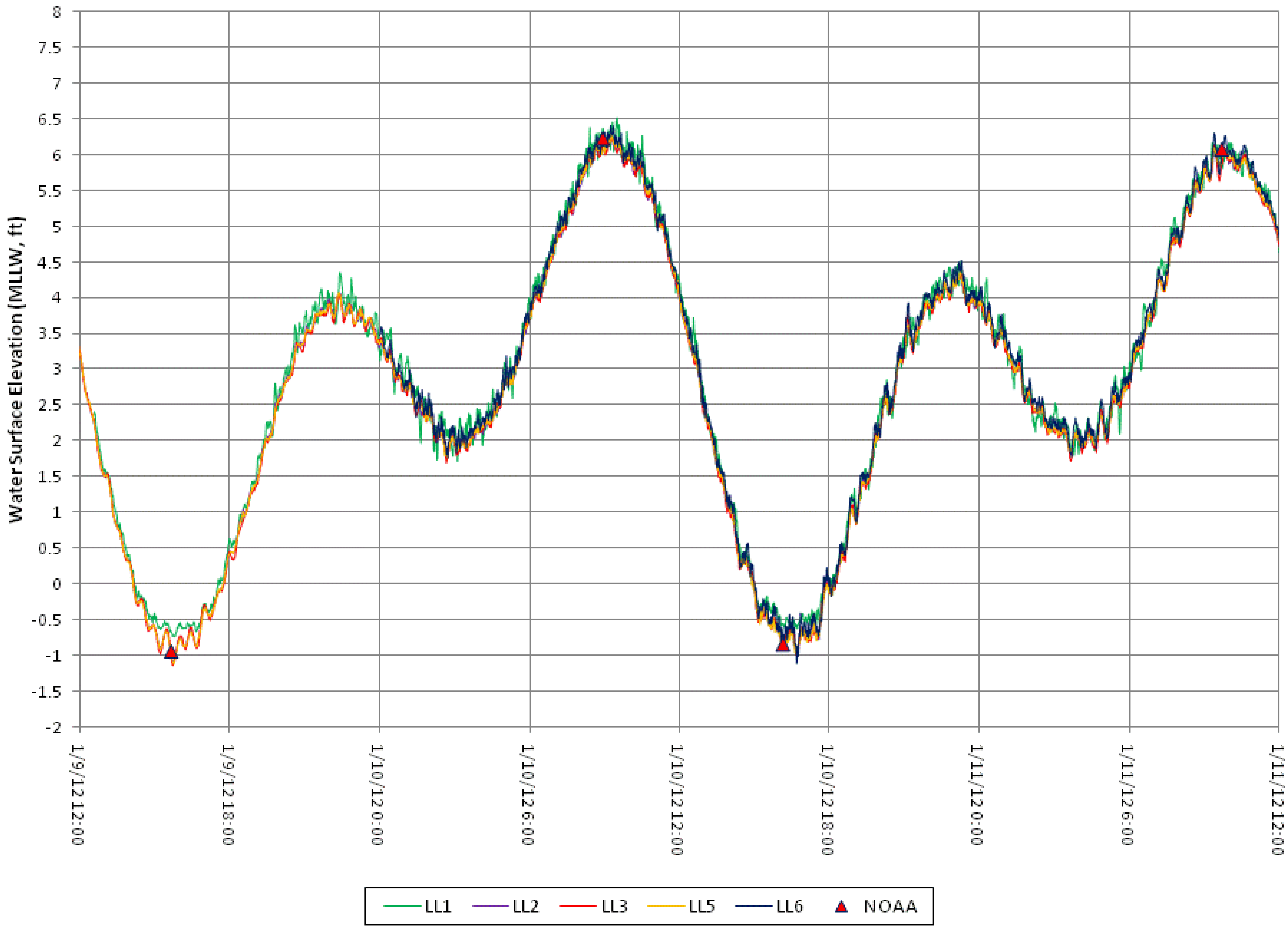


Figure A2. Measured Water Surface Elevations

Velocities

Velocities were measured using Rio Grande Acoustic Doppler Current Profiler (ADCP) throughout the harbor during a spring tide on January 9, 2012. ADCP transects were made across the Stub channel, inlet to Pierpont Basin, and Harbor mouth during the ebb and flood tide.

Figures A3 and A4 show a map of points where the velocities were measured during ebb and flood tides. The table summarizes time, magnitude of velocity, and vector direction for each data point.

ADCP Measurement Summary			
Point #	Time (PST)	Velocity (ft/s)	Flow Direction
19	10:09	0.24	173
20	10:13	0.01	115
21	10:17	0.10	177
22	10:24	0.13	188
23	10:29	0.04	239
24	10:35	0.02	229
25	10:41	0.06	161
26	10:59	0.04	328
35	12:41	0.33	214
36	12:45	0.41	217
37	12:48	0.44	211
38	12:53	0.37	216
39	13:01	0.12	196
40	13:07	0.09	38
41	13:14	0.55	179
42	13:37	0.23	173
43	13:41	0.12	214
44	13:45	0.08	281
45	13:49	0.05	356
46	14:25	0.24	36
47	14:30	0.07	213
48	14:45	0.16	213
49	14:50	0.07	229
50	14:56	0.15	214
51	15:01	0.25	217
52	15:19	0.32	43
53	15:23	0.27	61
54	15:27	0.11	75
55	15:32	0.28	253

Figure A3: ADCP Measurement Locations-Ebb Tide

ADCP Measurement Summary			
Point #	Time (PST)	Velocity (ft/s)	Flow Direction
56	17:03	0.63	243
57	17:07	0.20	226
58	17:12	0.01	271
59	17:16	0.09	55
60	17:21	0.22	42
61	17:25	0.35	27
62	17:29	0.05	338
63	17:33	0.07	193
64	17:37	0.05	344
65	17:41	0.18	345
66	17:46	0.10	352
67	17:53	0.41	37
68	17:57	0.44	35
69	18:01	0.15	42
70	18:05	0.14	219
71	18:09	0.16	225
72	18:12	0.14	30
73	18:15	0.38	41
74	18:19	0.38	34
75	18:20	0.19	26

Figure A4: ADCP Measurement Locations-Flood Tide

Appendix B

Hydraulics

HEC-RAS Results

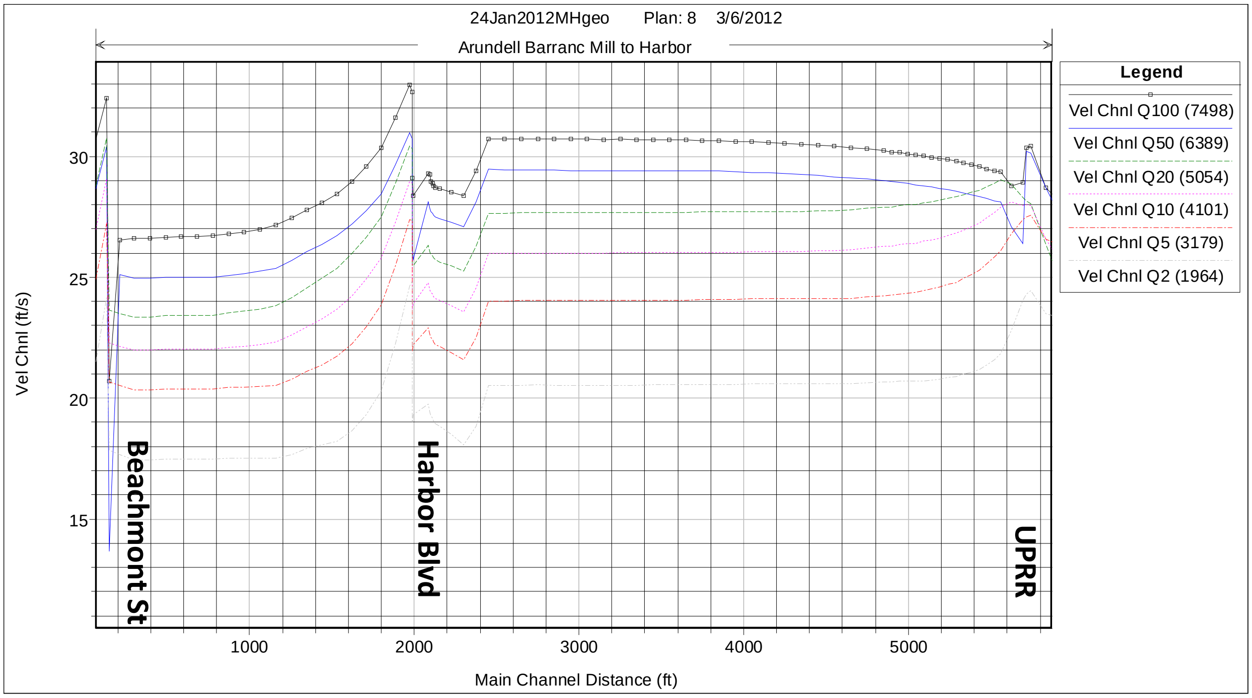


Figure B1. Arundell Barranca from Ventura Harbor to UPRR - Velocity Profiles

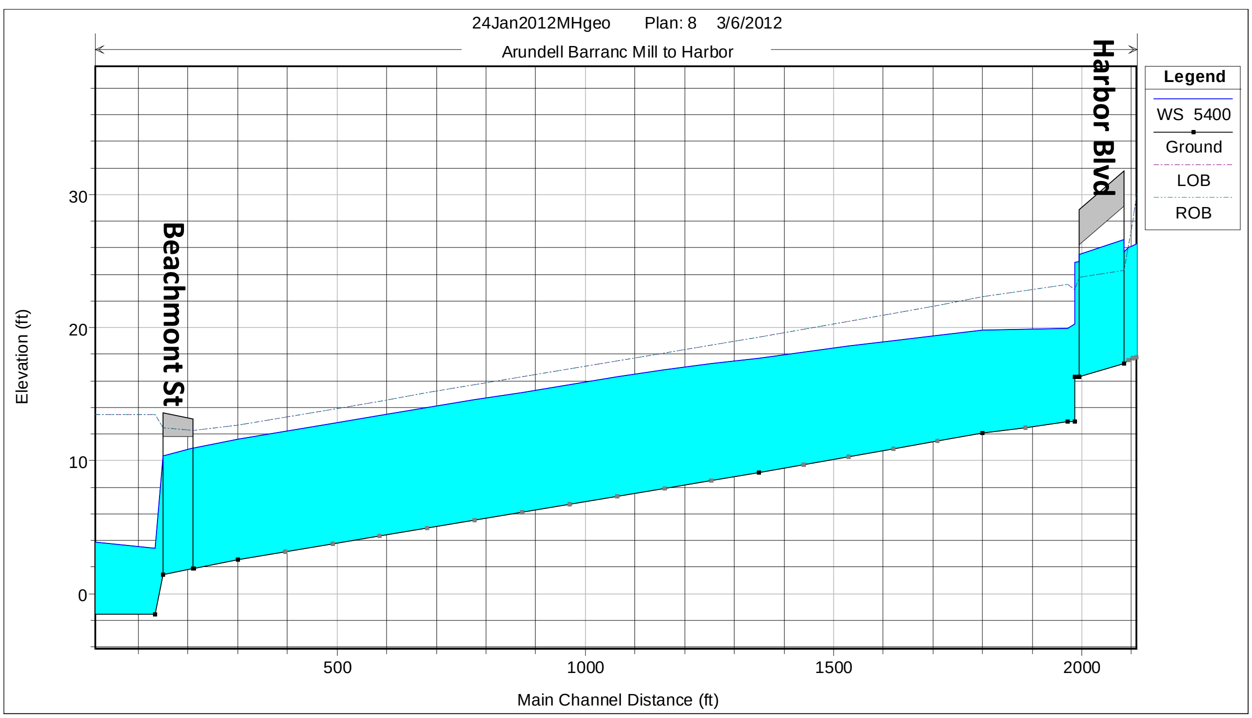


Figure B2. Arundell Barranca from Ventura Harbor to Harbor Boulevard - Water surface profile for 5,400 cfs

HEC-RAS Plan: 8 River: Arundell Barranc Reach: Mill to Harbor

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl	ROB Elev	LOB Elev
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)		(ft)	(ft)
Mill to Harbor	7283	Q100 (7498)	7498.00	72.71	82.48	86.51	97.92	0.009868	31.85	290.51	132.03	1.80	80.71	80.71
Mill to Harbor	7283	Q50 (6389)	6389.00	72.71	81.74	86.00	95.49	0.009604	29.82	226.23	48.78	1.75	80.71	80.71
Mill to Harbor	7283	Q20 (5054)	5053.65	72.71	80.59	85.43	92.06	0.009494	27.18	185.95	23.60	1.71	80.71	80.71
Mill to Harbor	7283	Q10 (4101)	4101.41	72.71	79.49	84.61	89.69	0.009554	25.63	160.03	23.60	1.73	80.71	80.71
Mill to Harbor	7283	Q5 (3179)	3179.15	72.71	78.37	81.02	87.17	0.009648	23.80	133.58	23.60	1.76	80.71	80.71
Mill to Harbor	7283	Q2 (1964)	1964.48	72.71	76.75	78.70	83.35	0.009968	20.61	95.30	23.60	1.81	80.71	80.71
Mill to Harbor	7203	Q100 (7498)	7498.00	71.88	82.80	86.31	96.71	0.009036	31.32	371.67	134.11	1.67	79.88	79.88
Mill to Harbor	7203	Q50 (6389)	6389.00	71.88	82.40	86.03	94.22	0.007847	28.47	321.61	118.66	1.55	79.88	79.88
Mill to Harbor	7203	Q20 (5054)	5053.65	71.88	81.98	85.63	90.62	0.005904	24.04	275.51	102.39	1.33	79.88	79.88
Mill to Harbor	7203	Q10 (4101)	4101.41	71.88	81.63	84.83	88.05	0.004521	20.55	242.05	88.74	1.16	79.88	79.88
Mill to Harbor	7203	Q5 (3179)	3179.15	71.88	83.54	83.54	85.38	0.001162	11.73	527.26	289.87	0.61	79.88	79.88
Mill to Harbor	7203	Q2 (1964)	1964.48	71.88	77.61	78.55	82.18	0.005353	17.15	114.52	20.00	1.26	79.88	79.88
Mill to Harbor	7067	Q100 (7498)	7498.00	70.48	82.36	85.47	95.33	0.007296	29.76	414.85	290.95	1.52	78.48	78.48
Mill to Harbor	7067	Q50 (6389)	6389.00	70.48	82.01	85.15	93.03	0.006246	27.00	324.49	228.30	1.40	78.48	78.48
Mill to Harbor	7067	Q20 (5054)	5053.65	70.48	81.10	84.50	89.84	0.005398	23.76	218.01	30.46	1.28	78.48	78.48
Mill to Harbor	7067	Q10 (4101)	4101.41	70.48	79.87	84.01	87.28	0.005376	21.84	187.76	25.55	1.26	78.48	78.48
Mill to Harbor	7067	Q5 (3179)	3179.15	70.48	78.42	79.70	84.65	0.005629	20.03	158.72	20.00	1.25	78.48	78.48
Mill to Harbor	7067	Q2 (1964)	1964.48	70.48	75.67	77.15	81.24	0.007110	18.94	103.71	20.00	1.47	78.48	78.48
Mill to Harbor	6991		Bridge											
Mill to Harbor	6990	Q100 (7498)	7498.00	69.69	81.84	84.63	92.79	0.006224	27.91	510.02	333.27	1.41	77.69	77.69
Mill to Harbor	6990	Q50 (6389)	6389.00	69.69	81.57	84.23	90.89	0.005254	25.27	424.62	304.13	1.29	77.69	77.69
Mill to Harbor	6990	Q20 (5054)	5053.65	69.69	81.27	83.65	88.07	0.003835	21.22	338.19	271.47	1.10	77.69	77.69
Mill to Harbor	6990	Q10 (4101)	4101.41	69.69	81.54	83.11	85.45	0.002200	16.32	415.60	300.88	0.84	77.69	77.69
Mill to Harbor	6990	Q5 (3179)	3179.15	69.69	77.86	78.92	83.74	0.005129	19.45	163.45	20.69	1.20	77.69	77.69
Mill to Harbor	6990	Q2 (1964)	1964.48	69.69	74.73	76.38	80.63	0.007736	19.50	100.72	20.00	1.53	77.69	77.69
Mill to Harbor	6900	Q100 (7498)	7498.00	68.76	79.62	83.05	92.00	0.008452	30.17	457.11	219.87	1.61	76.77	76.77
Mill to Harbor	6900	Q50 (6389)	6389.00	68.76	79.24	82.44	90.17	0.007514	27.78	379.48	187.76	1.51	76.77	76.77
Mill to Harbor	6900	Q20 (5054)	5053.65	68.76	78.69	81.41	87.44	0.006172	24.29	289.44	141.67	1.36	76.77	76.77
Mill to Harbor	6900	Q10 (4101)	4101.41	68.76	78.37	81.00	84.90	0.004725	20.79	247.86	114.28	1.18	76.77	76.77
Mill to Harbor	6900	Q5 (3179)	3179.15	68.76	76.38	80.21	83.14	0.006305	20.87	152.30	20.00	1.33	76.77	76.77
Mill to Harbor	6900	Q2 (1964)	1964.48	68.76	73.67	75.45	79.88	0.008313	20.00	98.24	20.00	1.59	76.77	76.77
Mill to Harbor	6600	Q100 (7498)	7498.00	65.67	78.33	80.92	89.57	0.005835	27.78	462.08	323.36	1.38	73.67	73.67
Mill to Harbor	6600	Q50 (6389)	6389.00	65.67	76.94	80.71	88.05	0.006571	27.27	285.28	56.73	1.43	73.67	73.67
Mill to Harbor	6600	Q20 (5054)	5053.65	65.67	75.53	80.41	85.39	0.006773	25.34	216.77	40.96	1.42	73.67	73.67
Mill to Harbor	6600	Q10 (4101)	4101.41	65.67	74.35	77.16	82.99	0.006956	23.59	176.33	27.70	1.41	73.67	73.67
Mill to Harbor	6600	Q5 (3179)	3179.15	65.67	72.56	75.30	80.83	0.008328	23.08	137.76	20.00	1.55	73.67	73.67
Mill to Harbor	6600	Q2 (1964)	1964.48	65.67	70.36	72.36	77.16	0.009481	20.92	93.89	20.00	1.70	73.67	73.67
Mill to Harbor	6240	Q100 (7498)	7498.00	61.96	73.63	75.81	86.96	0.007700	30.22	461.07	538.76	1.56	69.96	69.96
Mill to Harbor	6240	Q50 (6389)	6389.00	61.96	73.12	75.62	85.46	0.007177	28.33	269.38	223.78	1.49	69.96	69.96
Mill to Harbor	6240	Q20 (5054)	5053.65	61.96	71.36	75.36	82.59	0.008141	26.89	187.91	33.77	1.55	69.96	69.96
Mill to Harbor	6240	Q10 (4101)	4101.41	61.96	70.04	75.22	80.04	0.008854	25.37	161.66	20.82	1.57	69.96	69.96
Mill to Harbor	6240	Q5 (3179)	3179.15	61.96	68.56	71.16	77.57	0.009397	24.10	131.94	20.00	1.65	69.96	69.96
Mill to Harbor	6240	Q2 (1964)	1964.48	61.96	66.57	68.65	73.63	0.010016	21.32	92.14	20.00	1.75	69.96	69.96
Mill to Harbor	6179		Bridge											
Mill to Harbor	6178	Q100 (7498)	7498.00	61.33	72.95	75.25	84.76	0.007156	29.05	559.11	555.49	1.50	69.33	69.33
Mill to Harbor	6178	Q50 (6389)	6389.00	61.33	72.62	75.15	83.42	0.006428	27.01	396.76	418.15	1.42	69.33	69.33
Mill to Harbor	6178	Q20 (5054)	5053.65	61.33	72.17	74.64	80.48	0.005003	23.19	250.86	233.82	1.24	69.33	69.33
Mill to Harbor	6178	Q10 (4101)	4101.41	61.33	71.88	74.30	77.75	0.003639	19.43	211.07	39.42	1.05	69.33	69.33
Mill to Harbor	6178	Q5 (3179)	3179.15	61.33	67.91	70.55	76.98	0.009485	24.18	131.50	20.00	1.66	69.33	69.33
Mill to Harbor	6178	Q2 (1964)	1964.48	61.33	65.93	68.02	73.00	0.010038	21.34	92.07	20.00	1.75	69.33	69.33
Mill to Harbor	6098.5	Q100 (7498)	7498.00	60.51	70.58	73.36	83.93	0.010316	31.71	459.99	302.44	1.76	68.51	68.51
Mill to Harbor	6098.5	Q50 (6389)	6389.00	60.51	70.18	73.05	82.64	0.009627	29.80	358.29	201.03	1.69	68.51	68.51
Mill to Harbor	6098.5	Q20 (5054)	5053.65	60.51	69.63	72.58	79.79	0.008071	26.26	273.32	134.83	1.53	68.51	68.51
Mill to Harbor	6098.5	Q10 (4101)	4101.41	60.51	69.25	72.14	77.16	0.006485	22.87	225.84	111.55	1.36	68.51	68.51
Mill to Harbor	6098.5	Q5 (3179)	3179.15	60.51	67.06	71.53	76.21	0.009605	24.28	130.91	20.00	1.67	68.51	68.51
Mill to Harbor	6098.5	Q2 (1964)	1964.48	60.51	65.10	67.20	72.20	0.010091	21.38	91.90	20.00	1.76	68.51	68.51
Mill to Harbor	6000	Q100 (7498)	7498.00	58.61	70.58	73.19	82.75	0.006878	29.06	465.20	420.07	1.48	66.61	66.61
Mill to Harbor	6000	Q50 (6389)	6389.00	58.61	69.44	72.93	81.76	0.007612	28.59	255.70	55.07	1.53	66.61	66.61
Mill to Harbor	6000	Q20 (5054)	5053.65	58.61	68.06	72.46	78.93	0.007876	26.55	201.39	37.96	1.52	66.61	66.61
Mill to Harbor	6000	Q10 (4101)	4101.41	58.61	66.96	72.10	76.32	0.007930	24.54	167.85	24.38	1.50	66.61	66.61
Mill to Harbor	6000	Q5 (3179)	3179.15	58.61	64.79	68.26	75.07	0.011314	25.74	123.51	20.00	1.83	66.61	66.61
Mill to Harbor	6000	Q2 (1964)	1964.48	58.61	62.91	65.30	71.01	0.012241	22.84	86.03	20.00	1.94	66.61	66.61
Mill to Harbor	5979		Bridge											
Mill to Harbor	5978	Q100 (7498)	7498.00	58.18	70.56	73.06	81.92	0.006141	28.07	477.19	411.59	1.41	66.18	66.18
Mill to Harbor	5978	Q50 (6389)	6389.00	58.18	69.61	72.82	80.53	0.006306	26.98	285.34	57.72	1.41	66.18	66.18
Mill to Harbor	5978	Q20 (5054)	5053.65	58.18	68.54	72.42	77.27	0.005655	23.93	235.71	45.96	1.31	66.18	66.18
Mill to Harbor	5978	Q10 (4101)	4101.41	58.18	66.37	69.38	76.10	0.008472	25.04	164.01	22.09	1.54	66.18	66.18
Mill to Harbor	5978	Q5 (3179)	3179.15	58.18	64.29	67.83	74.80	0.011658	26.01	122.21	20.00	1.85	66.18	66.18
Mill to Harbor	5978	Q2 (1964)	1964.48	58.18	62.43	64.87	70.72	0.012653	23.10	85.06	20.00	1.97	66.18	66.18
Mill to Harbor	5904.52*	Q100 (7498)	7498.00	57.10	69.73	72.63	81.45	0.005650	28.19	416.54	262.38	1.40	64.25	64.25
Mill to Harbor	5904.52*	Q50 (6389)	6389.00	57.10	68.33	72.28	79.98	0.006404	27.74	267.15	50.15	1.46	64.25	64.25
Mill to Harbor	5904.52*	Q20 (5054)	5053.65	57.10	67.03	71.92	76.74	0.006196	25.15	215.80	32.34	1.41	64.25	64.25
Mill to Harbor	5904.52*	Q10 (4101)	4101.41	57.10	65.03	68.64	75.40	0.008836	25.85	159.95	23.46	1.62	64.25	64.25
Mill to Harbor	5904.52*	Q5 (3179)	3179.15	57.10	63.14	66.58	73.90	0.012062	26.33	120.75	20.00	1.89	64.25	64.25
Mill to Harbor	5904.52*	Q2 (1964)	1964.48	57.10	61.31	63.79	69.76	0.013017	23.32	84.24	20.00	2.00	64.25	64.25

HEC-RAS Plan: 8 River: Arundell Barranc Reach: Mill to Harbor (Continued)

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl	ROB Elev	LOB Elev
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)		(ft)	(ft)
Mill to Harbor	5831.05	Q100 (7498)	7498.00	56.03	68.63	72.23	80.98	0.005491	28.71	353.50	127.25	1.43	62.32	62.32
Mill to Harbor	5831.05	Q50 (6389)	6389.00	56.03	66.90	71.88	79.41	0.006686	28.71	248.89	33.74	1.53	62.32	62.32
Mill to Harbor	5831.05	Q20 (5054)	5053.65	56.03	65.52	71.39	76.17	0.006744	26.33	205.09	29.59	1.51	62.32	62.32
Mill to Harbor	5831.05	Q10 (4101)	4101.41	56.03	63.73	67.27	74.68	0.009064	26.57	157.07	24.24	1.69	62.32	62.32
Mill to Harbor	5831.05	Q5 (3179)	3179.15	56.03	62.01	65.45	72.98	0.012394	26.58	119.60	20.00	1.92	62.32	62.32
Mill to Harbor	5831.05	Q2 (1964)	1964.48	56.03	60.21	62.73	68.78	0.013314	23.50	83.60	20.00	2.03	62.32	62.32
Mill to Harbor	5738.90	Q100 (7498)	7498.00	54.13	66.18	71.18	80.24	0.007084	30.42	274.99	34.30	1.54	61.41	61.41
Mill to Harbor	5738.90	Q50 (6389)	6389.00	54.13	64.63	70.85	78.58	0.008344	30.13	225.47	29.65	1.64	61.41	61.41
Mill to Harbor	5738.90	Q20 (5054)	5053.65	54.13	63.12	66.35	75.31	0.008900	28.05	184.09	25.12	1.65	61.41	61.41
Mill to Harbor	5738.90	Q10 (4101)	4101.41	54.13	61.46	65.35	73.62	0.011621	27.98	146.58	20.15	1.82	61.41	61.41
Mill to Harbor	5738.90	Q5 (3179)	3179.15	54.13	59.90	63.47	71.70	0.013747	27.57	115.31	20.00	2.02	61.41	61.41
Mill to Harbor	5738.90	Q2 (1964)	1964.48	54.13	58.15	60.81	67.42	0.014932	24.43	80.40	20.00	2.15	61.41	61.41
Mill to Harbor	5714.08	Q100 (7498)	7498.00	53.82	65.96	71.09	80.06	0.007057	30.37	260.72	34.17	1.54	61.24	61.24
Mill to Harbor	5714.08	Q50 (6389)	6389.00	53.82	64.30	70.92	78.36	0.008503	30.22	220.01	29.18	1.64	61.24	61.24
Mill to Harbor	5714.08	Q20 (5054)	5053.65	53.82	62.78	66.44	75.07	0.009108	28.16	182.66	24.61	1.66	61.24	61.24
Mill to Harbor	5714.08	Q10 (4101)	4101.41	53.82	61.14	64.90	73.32	0.011698	28.01	146.43	20.00	1.82	61.24	61.24
Mill to Harbor	5714.08	Q5 (3179)	3179.15	53.82	59.60	63.16	71.35	0.013658	27.51	115.58	20.00	2.02	61.24	61.24
Mill to Harbor	5714.08	Q2 (1964)	1964.48	53.82	57.86	60.49	67.03	0.014681	24.29	80.87	20.00	2.13	61.24	61.24
Mill to Harbor	5700		Bridge											
Mill to Harbor	5692.08	Q100 (7498)	7498.00	53.60	65.75	70.02	77.92	0.006119	28.91	388.01	338.00	1.46	60.46	60.46
Mill to Harbor	5692.08	Q50 (6389)	6389.00	53.60	65.24	69.53	75.65	0.005397	26.39	309.02	154.40	1.36	60.46	60.46
Mill to Harbor	5692.08	Q20 (5054)	5053.65	53.60	62.49	67.68	74.87	0.008902	28.31	185.03	27.23	1.67	60.46	60.46
Mill to Harbor	5692.08	Q10 (4101)	4101.41	53.60	60.93	66.80	73.08	0.011238	27.97	147.00	21.68	1.82	60.46	60.46
Mill to Harbor	5692.08	Q5 (3179)	3179.15	53.60	59.41	63.09	71.03	0.013454	27.36	116.19	20.00	2.00	60.46	60.46
Mill to Harbor	5692.08	Q2 (1964)	1964.48	53.60	57.68	60.28	66.67	0.014262	24.05	81.67	20.00	2.10	60.46	60.46
Mill to Harbor	5626.58*	Q100 (7498)	7498.00	52.89	64.52	67.76	77.16	0.023086	28.77	298.55	181.00	1.49	61.94	61.94
Mill to Harbor	5626.58*	Q50 (6389)	6389.00	52.89	63.57	67.29	74.91	0.022895	27.07	239.80	27.83	1.46	61.94	61.94
Mill to Harbor	5626.58*	Q20 (5054)	5053.65	52.89	60.85	66.61	73.77	0.035692	28.84	175.23	22.00	1.80	61.94	61.94
Mill to Harbor	5626.58*	Q10 (4101)	4101.41	52.89	59.52	63.37	71.80	0.039317	28.12	145.85	22.00	1.92	61.94	61.94
Mill to Harbor	5626.58*	Q5 (3179)	3179.15	52.89	58.28	61.54	69.43	0.042623	26.79	118.69	22.00	2.03	61.94	61.94
Mill to Harbor	5626.58*	Q2 (1964)	1964.48	52.89	56.80	59.17	64.88	0.041794	22.81	86.12	22.00	2.03	61.94	61.94
Mill to Harbor	5561.08	Q100 (7498)	7498.00	52.18	62.83	66.61	76.20	0.008738	29.35	255.51	24.00	1.58	63.43	63.43
Mill to Harbor	5561.08	Q50 (6389)	6389.00	52.18	61.65	65.18	73.92	0.008731	28.11	227.27	24.00	1.61	63.43	63.43
Mill to Harbor	5561.08	Q20 (5054)	5053.65	52.18	59.44	63.27	72.51	0.011471	29.01	174.18	24.00	1.90	63.43	63.43
Mill to Harbor	5561.08	Q10 (4101)	4101.41	52.18	58.30	61.85	70.40	0.012279	27.91	146.95	24.00	1.99	63.43	63.43
Mill to Harbor	5561.08	Q5 (3179)	3179.15	52.18	57.26	60.34	67.83	0.012739	26.10	121.80	24.00	2.04	63.43	63.43
Mill to Harbor	5561.08	Q2 (1964)	1964.48	52.18	55.93	58.10	63.33	0.011975	21.83	90.00	24.00	1.99	63.43	63.43
Mill to Harbor	5550		Bridge											
Mill to Harbor	5524.08	Q100 (7498)	7498.00	51.83	62.46	66.30	75.88	0.008782	29.40	255.04	24.00	1.59	63.08	63.08
Mill to Harbor	5524.08	Q50 (6389)	6389.00	51.83	61.29	64.84	73.59	0.008767	28.15	226.93	24.00	1.61	63.08	63.08
Mill to Harbor	5524.08	Q20 (5054)	5053.65	51.83	59.12	62.95	72.07	0.011321	28.88	174.99	24.00	1.88	63.08	63.08
Mill to Harbor	5524.08	Q10 (4101)	4101.41	51.83	58.00	61.50	69.90	0.011992	27.68	148.16	24.00	1.96	63.08	63.08
Mill to Harbor	5524.08	Q5 (3179)	3179.15	51.83	56.96	59.99	67.31	0.012346	25.82	123.11	24.00	2.01	63.08	63.08
Mill to Harbor	5524.08	Q2 (1964)	1964.48	51.83	55.62	57.75	62.85	0.011569	21.58	91.04	24.00	1.95	63.08	63.08
Mill to Harbor	5477.06*	Q100 (7498)	7498.00	51.36	61.96	66.63	75.45	0.008851	29.48	254.30	24.00	1.60	62.61	62.61
Mill to Harbor	5477.06*	Q50 (6389)	6389.00	51.36	60.79	65.66	73.17	0.008843	28.24	226.22	24.00	1.62	62.61	62.61
Mill to Harbor	5477.06*	Q20 (5054)	5053.65	51.36	58.69	64.20	71.50	0.011149	28.72	175.94	24.00	1.87	62.61	62.61
Mill to Harbor	5477.06*	Q10 (4101)	4101.41	51.36	57.59	61.03	69.29	0.011705	27.45	149.41	24.00	1.94	62.61	62.61
Mill to Harbor	5477.06*	Q5 (3179)	3179.15	51.36	56.55	59.52	66.68	0.011954	25.54	124.47	24.00	1.98	62.61	62.61
Mill to Harbor	5477.06*	Q2 (1964)	1964.48	51.36	55.19	57.28	62.27	0.011208	21.35	92.00	24.00	1.92	62.61	62.61
Mill to Harbor	5430.04*	Q100 (7498)	7498.00	50.89	61.46	66.13	75.03	0.008916	29.56	253.62	24.00	1.60	62.14	62.14
Mill to Harbor	5430.04*	Q50 (6389)	6389.00	50.89	60.29	65.35	72.75	0.008914	28.32	225.56	24.00	1.63	62.14	62.14
Mill to Harbor	5430.04*	Q20 (5054)	5053.65	50.89	58.26	64.19	70.94	0.010998	28.59	176.79	24.00	1.86	62.14	62.14
Mill to Harbor	5430.04*	Q10 (4101)	4101.41	50.89	57.16	60.56	68.69	0.011457	27.25	150.53	24.00	1.92	62.14	62.14
Mill to Harbor	5430.04*	Q5 (3179)	3179.15	50.89	56.13	59.05	66.06	0.011626	25.30	125.66	24.00	1.95	62.14	62.14
Mill to Harbor	5430.04*	Q2 (1964)	1964.48	50.89	54.76	56.81	61.72	0.010928	21.17	92.78	24.00	1.90	62.14	62.14
Mill to Harbor	5383.03*	Q100 (7498)	7498.00	50.41	60.95	65.59	74.60	0.008989	29.65	252.85	24.00	1.61	61.66	61.66
Mill to Harbor	5383.03*	Q50 (6389)	6389.00	50.41	59.78	64.89	72.32	0.008993	28.42	224.84	24.00	1.64	61.66	61.66
Mill to Harbor	5383.03*	Q20 (5054)	5053.65	50.41	57.80	63.86	70.42	0.010914	28.51	177.27	24.00	1.85	61.66	61.66
Mill to Harbor	5383.03*	Q10 (4101)	4101.41	50.41	56.72	60.08	68.12	0.011275	27.10	151.37	24.00	1.90	61.66	61.66
Mill to Harbor	5383.03*	Q5 (3179)	3179.15	50.41	55.68	58.57	65.48	0.011386	25.12	126.56	24.00	1.93	61.66	61.66
Mill to Harbor	5383.03*	Q2 (1964)	1964.48	50.41	54.29	56.34	61.20	0.010790	21.09	93.17	24.00	1.89	61.66	61.66
Mill to Harbor	5336.01*	Q100 (7498)	7498.00	49.94	60.45	65.07	74.17	0.009046	29.72	252.26	24.00	1.62	61.19	61.19
Mill to Harbor	5336.01*	Q50 (6389)	6389.00	49.94	59.28	64.43	71.89	0.009055	28.49	224.28	24.00	1.64	61.19	61.19
Mill to Harbor	5336.01*	Q20 (5054)	5053.65	49.94	57.35	63.46	69.89	0.010816	28.42	177.84	24.00	1.84	61.19	61.19
Mill to Harbor	5336.01*	Q10 (4101)	4101.41	49.94	56.28	59.61	67.55	0.011088	26.94	152.25	24.00	1.88	61.19	61.19
Mill to Harbor	5336.01*	Q5 (3179)	3179.15	49.94	55.25	58.10	64.91	0.011150	24.94	127.47	24.00	1.91	61.19	61.19
Mill to Harbor	5336.01*	Q2 (1964)	1964.48	49.94	53.84	55.87	60.68	0.010638	20.99	93.61	24.00	1.87	61.19	61.19
Mill to Harbor	5289	Q100 (7498)	7498.00	49.47	59.96	64.57	73.74	0.009100	29.79	251.71	24.00	1.62	60.72	60.72
Mill to Harbor	5289	Q50 (6389)	6389.00	49.47	58.79	63.95	71.45	0.009112	28.55	223.76	24.00	1.65	60.72	60.72
Mill to Harbor	5289	Q20 (5054)	5053.65	49.47	56.90	63.03	69.37	0.010728	28.33	178.35	24.00	1.83	60.72	60.72
Mill to Harbor	5289	Q10 (4101)	4101.41	49.47	55.84	59.14	67.02	0.010957	26.83	152.89	24.00	1.87	60.72	60.72
Mill to Harbor	5289	Q5 (3179)	3179.15	49.47	54.81	57.63	64.36	0.010953	24.79	128.25	24.00	1.89	60.72	60.72
Mill to Harbor	5289	Q2 (1964)	1964.48	49.47	53.39	55.40	60.17	0.010515	20.90	93.97	24.00	1.86	60.72	60.72
Mill to Harbor	5240.22*	Q100 (7498)	7498.00	48.98	59.44	63.75	73.28	0.009155	29.85	251.15	24.00	1.63	60.23	60.23
Mill to Harbor	5240.22*	Q50 (6389)	6389.00	48.98	58.28	63.35	71.00	0.009172	28.62	223.23	24.00	1.65	60.23	60.23

HEC-RAS Plan: 8 River: Arundell Barranc Reach: Mill to Harbor (Continued)

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl	ROB Elev	LOB Elev
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)		(ft)	(ft)
Mill to Harbor	5240.22*	Q20 (5054)	5053.65	48.98	56.43	60.07	68.83	0.010652	28.26	178.80	24.00	1.82	60.23	60.23
Mill to Harbor	5240.22*	Q10 (4101)	4101.41	48.98	55.37	58.65	66.47	0.010844	26.73	153.44	24.00	1.86	60.23	60.23
Mill to Harbor	5240.22*	Q5 (3179)	3179.15	48.98	54.35	57.14	63.81	0.010821	24.69	128.78	24.00	1.88	60.23	60.23
Mill to Harbor	5240.22*	Q2 (1964)	1964.48	48.98	52.91	54.91	59.65	0.010421	20.84	94.25	24.00	1.85	60.23	60.23
Mill to Harbor	5191.44*	Q100 (7498)	7498.00	48.50	58.95	63.25	72.83	0.009195	29.90	250.75	24.00	1.63	59.75	59.75
Mill to Harbor	5191.44*	Q50 (6389)	6389.00	48.50	57.79	62.91	70.55	0.009214	28.67	222.86	24.00	1.66	59.75	59.75
Mill to Harbor	5191.44*	Q20 (5054)	5053.65	48.50	55.97	59.59	68.30	0.010561	28.18	179.35	24.00	1.82	59.75	59.75
Mill to Harbor	5191.44*	Q10 (4101)	4101.41	48.50	54.92	58.17	65.92	0.010719	26.62	154.06	24.00	1.85	59.75	59.75
Mill to Harbor	5191.44*	Q5 (3179)	3179.15	48.50	53.89	56.66	63.27	0.010678	24.57	129.37	24.00	1.87	59.75	59.75
Mill to Harbor	5191.44*	Q2 (1964)	1964.48	48.50	52.44	54.43	59.14	0.010305	20.76	94.61	24.00	1.84	59.75	59.75
Mill to Harbor	5142.66*	Q100 (7498)	7498.00	48.01	58.44	62.82	72.38	0.009245	29.96	250.25	24.00	1.64	59.26	59.26
Mill to Harbor	5142.66*	Q50 (6389)	6389.00	48.01	57.28	62.53	70.09	0.009267	28.73	222.40	24.00	1.66	59.26	59.26
Mill to Harbor	5142.66*	Q20 (5054)	5053.65	48.01	55.50	59.10	67.78	0.010503	28.12	179.70	24.00	1.81	59.26	59.26
Mill to Harbor	5142.66*	Q10 (4101)	4101.41	48.01	54.45	57.68	65.39	0.010634	26.55	154.49	24.00	1.84	59.26	59.26
Mill to Harbor	5142.66*	Q5 (3179)	3179.15	48.01	53.42	56.17	62.74	0.010586	24.50	129.76	24.00	1.86	59.26	59.26
Mill to Harbor	5142.66*	Q2 (1964)	1964.48	48.01	51.96	53.94	58.64	0.010271	20.74	94.71	24.00	1.84	59.26	59.26
Mill to Harbor	5093.88*	Q100 (7498)	7498.00	47.52	57.93	62.45	71.92	0.009292	30.02	249.78	24.00	1.64	58.77	58.77
Mill to Harbor	5093.88*	Q50 (6389)	6389.00	47.52	56.77	62.31	69.63	0.009316	28.78	221.97	24.00	1.67	58.77	58.77
Mill to Harbor	5093.88*	Q20 (5054)	5053.65	47.52	55.02	58.61	67.26	0.010452	28.07	180.01	24.00	1.81	58.77	58.77
Mill to Harbor	5093.88*	Q10 (4101)	4101.41	47.52	53.97	57.19	64.86	0.010560	26.48	154.87	24.00	1.84	58.77	58.77
Mill to Harbor	5093.88*	Q5 (3179)	3179.15	47.52	52.94	55.68	62.21	0.010506	24.44	130.09	24.00	1.85	58.77	58.77
Mill to Harbor	5093.88*	Q2 (1964)	1964.48	47.52	51.47	53.45	58.14	0.010242	20.72	94.80	24.00	1.84	58.77	58.77
Mill to Harbor	5045.11*	Q100 (7498)	7498.00	47.04	57.43	62.22	71.46	0.009324	30.06	249.47	24.00	1.64	58.29	58.29
Mill to Harbor	5045.11*	Q50 (6389)	6389.00	47.04	56.28	62.09	69.17	0.009349	28.82	221.68	24.00	1.67	58.29	58.29
Mill to Harbor	5045.11*	Q20 (5054)	5053.65	47.04	54.56	58.13	66.74	0.010383	28.01	180.43	24.00	1.80	58.29	58.29
Mill to Harbor	5045.11*	Q10 (4101)	4101.41	47.04	53.51	56.71	64.34	0.010469	26.40	155.33	24.00	1.83	58.29	58.29
Mill to Harbor	5045.11*	Q5 (3179)	3179.15	47.04	52.48	55.20	61.69	0.010408	24.36	130.51	24.00	1.84	58.29	58.29
Mill to Harbor	5045.11*	Q2 (1964)	1964.48	47.04	51.00	52.97	57.64	0.010190	20.69	94.96	24.00	1.83	58.29	58.29
Mill to Harbor	4996.33*	Q100 (7498)	7498.00	46.55	56.93	62.00	71.00	0.009366	30.11	249.05	24.00	1.65	57.80	57.80
Mill to Harbor	4996.33*	Q50 (6389)	6389.00	46.55	55.77	61.76	68.71	0.009393	28.87	221.31	24.00	1.68	57.80	57.80
Mill to Harbor	4996.33*	Q20 (5054)	5053.65	46.55	54.07	57.64	66.24	0.010359	27.99	180.58	24.00	1.80	57.80	57.80
Mill to Harbor	4996.33*	Q10 (4101)	4101.41	46.55	53.03	56.22	63.83	0.010435	26.37	155.51	24.00	1.83	57.80	57.80
Mill to Harbor	4996.33*	Q5 (3179)	3179.15	46.55	51.99	54.71	61.19	0.010374	24.33	130.66	24.00	1.84	57.80	57.80
Mill to Harbor	4996.33*	Q2 (1964)	1964.48	46.55	50.51	52.48	57.15	0.010190	20.69	94.96	24.00	1.83	57.80	57.80
Mill to Harbor	4947.55*	Q100 (7498)	7498.00	46.06	56.42	61.83	70.54	0.009406	30.15	248.67	24.00	1.65	57.31	57.31
Mill to Harbor	4947.55*	Q50 (6389)	6389.00	46.06	55.27	61.55	68.25	0.009434	28.92	220.95	24.00	1.68	57.31	57.31
Mill to Harbor	4947.55*	Q20 (5054)	5053.65	46.06	53.59	57.15	65.73	0.010337	27.96	180.72	24.00	1.80	57.31	57.31
Mill to Harbor	4947.55*	Q10 (4101)	4101.41	46.06	52.55	55.73	63.32	0.010404	26.35	155.67	24.00	1.82	57.31	57.31
Mill to Harbor	4947.55*	Q5 (3179)	3179.15	46.06	51.51	54.22	60.68	0.010343	24.31	130.79	24.00	1.83	57.31	57.31
Mill to Harbor	4947.55*	Q2 (1964)	1964.48	46.06	50.02	51.99	56.66	0.010190	20.69	94.96	24.00	1.83	57.31	57.31
Mill to Harbor	4898.77*	Q100 (7498)	7498.00	45.58	55.93	61.52	70.08	0.009431	30.18	248.42	24.00	1.65	56.83	56.83
Mill to Harbor	4898.77*	Q50 (6389)	6389.00	45.58	54.78	61.19	67.79	0.009460	28.94	220.74	24.00	1.68	56.83	56.83
Mill to Harbor	4898.77*	Q20 (5054)	5053.65	45.58	53.12	56.67	65.22	0.010280	27.91	181.07	24.00	1.79	56.83	56.83
Mill to Harbor	4898.77*	Q10 (4101)	4101.41	45.58	52.08	55.25	62.81	0.010332	26.28	156.05	24.00	1.82	56.83	56.83
Mill to Harbor	4898.77*	Q5 (3179)	3179.15	45.58	51.04	53.74	60.17	0.010269	24.25	131.11	24.00	1.83	56.83	56.83
Mill to Harbor	4898.77*	Q2 (1964)	1964.48	45.58	49.54	51.51	56.17	0.010144	20.66	95.10	24.00	1.83	56.83	56.83
Mill to Harbor	4850	Q100 (7498)	7498.00	45.09	55.43	61.19	69.61	0.009467	30.22	248.08	24.00	1.66	56.34	56.34
Mill to Harbor	4850	Q50 (6389)	6389.00	45.09	54.27	60.90	67.32	0.009497	28.99	220.42	24.00	1.69	56.34	56.34
Mill to Harbor	4850	Q20 (5054)	5053.65	45.09	52.64	56.18	64.72	0.010264	27.89	181.17	24.00	1.79	56.34	56.34
Mill to Harbor	4850	Q10 (4101)	4101.41	45.09	51.60	54.76	62.31	0.010309	26.26	156.17	24.00	1.81	56.34	56.34
Mill to Harbor	4850	Q5 (3179)	3179.15	45.09	50.56	53.25	59.67	0.010248	24.23	131.21	24.00	1.83	56.34	56.34
Mill to Harbor	4850	Q2 (1964)	1964.48	45.09	49.05	51.02	55.68	0.010144	20.66	95.10	24.00	1.83	56.34	56.34
Mill to Harbor	4750.*	Q100 (7498)	7498.00	44.09	54.40	59.72	68.65	0.009528	30.30	247.48	24.00	1.66	55.34	55.34
Mill to Harbor	4750.*	Q50 (6389)	6389.00	44.09	53.25	59.47	66.36	0.009560	29.05	219.90	24.00	1.69	55.34	55.34
Mill to Harbor	4750.*	Q20 (5054)	5053.65	44.09	51.65	55.18	63.69	0.010206	27.84	181.54	24.00	1.78	55.34	55.34
Mill to Harbor	4750.*	Q10 (4101)	4101.41	44.09	50.61	53.76	61.27	0.010233	26.19	156.57	24.00	1.81	55.34	55.34
Mill to Harbor	4750.*	Q5 (3179)	3179.15	44.09	49.57	52.25	58.64	0.010177	24.17	131.52	24.00	1.82	55.34	55.34
Mill to Harbor	4750.*	Q2 (1964)	1964.48	44.09	48.06	50.02	54.67	0.010108	20.63	95.22	24.00	1.83	55.34	55.34
Mill to Harbor	4650.*	Q100 (7498)	7498.00	43.10	53.39	58.31	67.70	0.009571	30.35	247.08	24.00	1.67	54.35	54.35
Mill to Harbor	4650.*	Q50 (6389)	6389.00	43.10	52.25	58.04	65.40	0.009603	29.10	219.54	24.00	1.70	54.35	54.35
Mill to Harbor	4650.*	Q20 (5054)	5053.65	43.10	50.68	54.19	62.66	0.010139	27.77	181.96	24.00	1.78	54.35	54.35
Mill to Harbor	4650.*	Q10 (4101)	4101.41	43.10	49.64	52.77	60.24	0.010152	26.12	157.01	24.00	1.80	54.35	54.35
Mill to Harbor	4650.*	Q5 (3179)	3179.15	43.10	48.59	51.26	57.62	0.010099	24.11	131.87	24.00	1.81	54.35	54.35
Mill to Harbor	4650.*	Q2 (1964)	1964.48	43.10	47.07	49.03	53.66	0.010057	20.60	95.38	24.00	1.82	54.35	54.35
Mill to Harbor	4550.*	Q100 (7498)	7498.00	42.10	52.38	56.91	66.73	0.009621	30.40	246.61	24.00	1.67	53.35	53.35
Mill to Harbor	4550.*	Q50 (6389)	6389.00	42.10	51.23	56.68	64.43	0.009652	29.16	219.13	24.00	1.70	53.35	53.35
Mill to Harbor	4550.*	Q20 (5054)	5053.65	42.10	49.69	53.18	61.65	0.010119	27.75	182.09	24.00	1.78	53.35	53.35
Mill to Harbor	4550.*	Q10 (4101)	4101.41	42.10	48.65	51.77	59.23	0.010128	26.10	157.14	24.00	1.80	53.35	53.35
Mill to Harbor	4550.*	Q5 (3179)	3179.15	42.10	47.59	50.26	56.62	0.010099	24.11	131.87	24.00	1.81	53.35	53.35
Mill to Harbor	4550.*	Q2 (1964)	1964.48	42.10	46.07	48.03	52.66	0.010057	20.60	95.38	24.00	1.82	53.35	53.35
Mill to Harbor	4450	Q100 (7498)	7498.00	41.10	51.36	55.49	65.76	0.009664	30.46	246.20	24.00	1.68	52.35	52.35
Mill to Harbor	4450	Q50 (6389)	6389.00	41.10	50.22	55.44	63.46	0.009696	29.20	218.78	24.00	1.70	52.35	52.35
Mill to Harbor	4450	Q20 (5054)	5053.65	41.10	48.69	52.19	60.64	0.010102	27.74	182.19	24.00	1.77	52.35	52.35
Mill to Harbor	4450	Q10 (4101)	4101.41	41.10	47.65	50.77	58.22	0.010107	26.08	157.25	24.00	1.80	52.35	52.35
Mill to Harbor	4450	Q5 (3179)	3179.15	41.10	46.59	49.26	55.62	0.010099	24.11	131.87	24.00	1.81	52.35	52.35
Mill to Harbor	4450	Q2 (1964)	1964.48	41.10	45.07	47.03	51.66	0.010057	20.60	95.38	24.00	1.82	52.35	52.35
Mill to Harbor	4350.*	Q100 (7498)	7498.00	40.10	50.34	55.08	64.79	0.009703	30.50	245.84	24.00	1.68	51.35	51.35

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl	ROB Elev	LOB Elev
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)		(ft)	(ft)
Mill to Harbor	4350.*	Q50 (6389)	6389.00	40.10	49.20	54.76	62.48	0.009734	29.24	218.47	24.00	1.71	51.35	51.35
Mill to Harbor	4350.*	Q20 (5054)	5053.65	40.10	47.70	51.18	59.63	0.010088	27.72	182.29	24.00	1.77	51.35	51.35
Mill to Harbor	4350.*	Q10 (4101)	4101.41	40.10	46.66	49.77	57.21	0.010090	26.07	157.34	24.00	1.79	51.35	51.35
Mill to Harbor	4350.*	Q5 (3179)	3179.15	40.10	45.59	48.26	54.62	0.010099	24.11	131.87	24.00	1.81	51.35	51.35
Mill to Harbor	4350.*	Q2 (1964)	1964.48	40.10	44.07	46.03	50.66	0.010057	20.60	95.38	24.00	1.82	51.35	51.35
Mill to Harbor	4250	Q100 (7498)	7498.00	39.10	49.33	54.46	63.81	0.009737	30.54	245.52	24.00	1.68	50.35	50.35
Mill to Harbor	4250	Q50 (6389)	6389.00	39.10	48.19	54.21	61.50	0.009767	29.28	218.20	24.00	1.71	50.35	50.35
Mill to Harbor	4250	Q20 (5054)	5053.65	39.10	46.70	50.19	58.63	0.010088	27.72	182.29	24.00	1.77	50.35	50.35
Mill to Harbor	4250	Q10 (4101)	4101.41	39.10	45.66	48.77	56.21	0.010090	26.07	157.34	24.00	1.79	50.35	50.35
Mill to Harbor	4250	Q5 (3179)	3179.15	39.10	44.59	47.26	53.62	0.010099	24.11	131.87	24.00	1.81	50.35	50.35
Mill to Harbor	4250	Q2 (1964)	1964.48	39.10	43.07	45.03	49.66	0.010057	20.60	95.38	24.00	1.82	50.35	50.35
Mill to Harbor	4150.*	Q100 (7498)	7498.00	38.10	48.32	53.60	62.83	0.009767	30.57	245.24	24.00	1.69	49.35	49.35
Mill to Harbor	4150.*	Q50 (6389)	6389.00	38.10	47.18	53.42	60.52	0.009796	29.31	217.96	24.00	1.71	49.35	49.35
Mill to Harbor	4150.*	Q20 (5054)	5053.65	38.10	45.70	49.19	57.63	0.010088	27.72	182.29	24.00	1.77	49.35	49.35
Mill to Harbor	4150.*	Q10 (4101)	4101.41	38.10	44.66	47.77	55.21	0.010090	26.07	157.34	24.00	1.79	49.35	49.35
Mill to Harbor	4150.*	Q5 (3179)	3179.15	38.10	43.59	46.26	52.62	0.010099	24.11	131.87	24.00	1.81	49.35	49.35
Mill to Harbor	4150.*	Q2 (1964)	1964.48	38.10	42.07	44.03	48.66	0.010057	20.60	95.38	24.00	1.82	49.35	49.35
Mill to Harbor	4050.*	Q100 (7498)	7498.00	37.10	47.31	52.71	61.85	0.009794	30.61	244.99	24.00	1.69	48.35	48.35
Mill to Harbor	4050.*	Q50 (6389)	6389.00	37.10	46.17	52.62	59.54	0.009821	29.34	217.76	24.00	1.72	48.35	48.35
Mill to Harbor	4050.*	Q20 (5054)	5053.65	37.10	44.70	48.19	56.63	0.010088	27.72	182.29	24.00	1.77	48.35	48.35
Mill to Harbor	4050.*	Q10 (4101)	4101.41	37.10	43.66	46.77	54.21	0.010090	26.07	157.34	24.00	1.79	48.35	48.35
Mill to Harbor	4050.*	Q5 (3179)	3179.15	37.10	42.59	45.26	51.62	0.010099	24.11	131.87	24.00	1.81	48.35	48.35
Mill to Harbor	4050.*	Q2 (1964)	1964.48	37.10	41.07	43.03	47.66	0.010057	20.60	95.38	24.00	1.82	48.35	48.35
Mill to Harbor	3950.*	Q100 (7498)	7498.00	36.11	46.32	51.78	60.87	0.009798	30.61	244.95	24.00	1.69	47.36	47.36
Mill to Harbor	3950.*	Q50 (6389)	6389.00	36.11	45.18	51.72	58.56	0.009841	29.36	217.60	24.00	1.72	47.36	47.36
Mill to Harbor	3950.*	Q20 (5054)	5053.65	36.11	43.71	47.20	55.62	0.010061	27.70	182.46	24.00	1.77	47.36	47.36
Mill to Harbor	3950.*	Q10 (4101)	4101.41	36.11	42.67	45.78	53.20	0.010060	26.04	157.51	24.00	1.79	47.36	47.36
Mill to Harbor	3950.*	Q5 (3179)	3179.15	36.11	41.61	44.27	50.61	0.010062	24.08	132.03	24.00	1.81	47.36	47.36
Mill to Harbor	3950.*	Q2 (1964)	1964.48	36.11	40.09	42.04	46.66	0.010018	20.57	95.50	24.00	1.82	47.36	47.36
Mill to Harbor	3850	Q100 (7498)	7498.00	35.11	45.31	51.03	59.88	0.009822	30.64	244.74	24.00	1.69	46.36	46.36
Mill to Harbor	3850	Q50 (6389)	6389.00	35.11	44.17	50.66	57.58	0.009861	29.38	217.44	24.00	1.72	46.36	46.36
Mill to Harbor	3850	Q20 (5054)	5053.65	35.11	42.71	46.20	54.62	0.010061	27.70	182.46	24.00	1.77	46.36	46.36
Mill to Harbor	3850	Q10 (4101)	4101.41	35.11	41.67	44.78	52.20	0.010060	26.04	157.51	24.00	1.79	46.36	46.36
Mill to Harbor	3850	Q5 (3179)	3179.15	35.11	40.61	43.27	49.61	0.010062	24.08	132.03	24.00	1.81	46.36	46.36
Mill to Harbor	3850	Q2 (1964)	1964.48	35.11	39.09	41.04	45.66	0.010018	20.57	95.50	24.00	1.82	46.36	46.36
Mill to Harbor	3750.*	Q100 (7498)	7498.00	34.11	44.30	49.89	58.90	0.009842	30.66	244.55	24.00	1.69	45.36	45.36
Mill to Harbor	3750.*	Q50 (6389)	6389.00	34.11	43.17	49.69	56.58	0.009867	29.39	217.39	24.00	1.72	45.36	45.36
Mill to Harbor	3750.*	Q20 (5054)	5053.65	34.11	41.71	45.20	53.62	0.010061	27.70	182.46	24.00	1.77	45.36	45.36
Mill to Harbor	3750.*	Q10 (4101)	4101.41	34.11	40.67	43.78	51.20	0.010060	26.04	157.51	24.00	1.79	45.36	45.36
Mill to Harbor	3750.*	Q5 (3179)	3179.15	34.11	39.61	42.27	48.61	0.010062	24.08	132.03	24.00	1.81	45.36	45.36
Mill to Harbor	3750.*	Q2 (1964)	1964.48	34.11	38.09	40.04	44.66	0.010018	20.57	95.50	24.00	1.82	45.36	45.36
Mill to Harbor	3650.*	Q100 (7498)	7498.00	33.12	43.30	48.94	57.92	0.009856	30.68	244.43	24.00	1.69	44.36	44.36
Mill to Harbor	3650.*	Q50 (6389)	6389.00	33.12	42.18	48.72	55.58	0.009857	29.38	217.47	24.00	1.72	44.36	44.36
Mill to Harbor	3650.*	Q20 (5054)	5053.65	33.12	40.73	44.21	52.62	0.010039	27.68	182.60	24.00	1.77	44.36	44.36
Mill to Harbor	3650.*	Q10 (4101)	4101.41	33.12	39.69	42.79	50.20	0.010035	26.02	157.65	24.00	1.79	44.36	44.36
Mill to Harbor	3650.*	Q5 (3179)	3179.15	33.12	38.63	41.28	47.61	0.010032	24.05	132.17	24.00	1.81	44.36	44.36
Mill to Harbor	3650.*	Q2 (1964)	1964.48	33.12	37.10	39.05	43.66	0.009988	20.55	95.59	24.00	1.81	44.36	44.36
Mill to Harbor	3550.*	Q100 (7498)	7498.00	32.12	42.30	47.91	56.92	0.009862	30.68	244.37	24.00	1.69	43.36	43.36
Mill to Harbor	3550.*	Q50 (6389)	6389.00	32.12	41.18	47.72	54.59	0.009864	29.39	217.42	24.00	1.72	43.36	43.36
Mill to Harbor	3550.*	Q20 (5054)	5053.65	32.12	39.73	43.21	51.62	0.010039	27.68	182.60	24.00	1.77	43.36	43.36
Mill to Harbor	3550.*	Q10 (4101)	4101.41	32.12	38.69	41.79	49.20	0.010035	26.02	157.65	24.00	1.79	43.36	43.36
Mill to Harbor	3550.*	Q5 (3179)	3179.15	32.12	37.63	40.28	46.61	0.010032	24.05	132.17	24.00	1.81	43.36	43.36
Mill to Harbor	3550.*	Q2 (1964)	1964.48	32.12	36.10	38.05	42.66	0.009988	20.55	95.59	24.00	1.81	43.36	43.36
Mill to Harbor	3450	Q100 (7498)	7498.00	31.12	41.30	46.90	55.92	0.009867	30.69	244.32	24.00	1.70	42.36	42.36
Mill to Harbor	3450	Q50 (6389)	6389.00	31.12	40.18	46.63	53.59	0.009870	29.39	217.37	24.00	1.72	42.36	42.36
Mill to Harbor	3450	Q20 (5054)	5053.65	31.12	38.73	42.21	50.62	0.010039	27.68	182.60	24.00	1.77	42.36	42.36
Mill to Harbor	3450	Q10 (4101)	4101.41	31.12	37.69	40.79	48.20	0.010035	26.02	157.65	24.00	1.79	42.36	42.36
Mill to Harbor	3450	Q5 (3179)	3179.15	31.12	36.63	39.28	45.61	0.010032	24.05	132.17	24.00	1.81	42.36	42.36
Mill to Harbor	3450	Q2 (1964)	1964.48	31.12	35.10	37.05	41.66	0.009979	20.54	95.62	24.00	1.81	42.36	42.36
Mill to Harbor	3350.*	Q100 (7498)	7498.00	30.12	40.30	46.41	54.93	0.009873	30.70	244.27	24.00	1.70	41.37	41.37
Mill to Harbor	3350.*	Q50 (6389)	6389.00	30.12	39.17	46.13	52.60	0.009876	29.40	217.32	24.00	1.72	41.37	41.37
Mill to Harbor	3350.*	Q20 (5054)	5053.65	30.12	37.73	41.21	49.62	0.010039	27.68	182.60	24.00	1.77	41.37	41.37
Mill to Harbor	3350.*	Q10 (4101)	4101.41	30.12	36.69	39.79	47.20	0.010035	26.02	157.65	24.00	1.79	41.37	41.37
Mill to Harbor	3350.*	Q5 (3179)	3179.15	30.12	35.63	38.28	44.61	0.010032	24.05	132.17	24.00	1.81	41.37	41.37
Mill to Harbor	3350.*	Q2 (1964)	1964.48	30.12	34.11	36.05	40.65	0.009967	20.54	95.66	24.00	1.81	41.37	41.37
Mill to Harbor	3250.*	Q100 (7498)	7498.00	29.12	39.30	45.93	53.93	0.009878	30.70	244.22	24.00	1.70	40.37	40.37
Mill to Harbor	3250.*	Q50 (6389)	6389.00	29.12	38.17	42.38	51.60	0.009882	29.41	217.27	24.00	1.72	40.37	40.37
Mill to Harbor	3250.*	Q20 (5054)	5053.65	29.12	36.73	40.21	48.62	0.010039	27.68	182.60	24.00	1.77	40.37	40.37
Mill to Harbor	3250.*	Q10 (4101)	4101.41	29.12	35.69	38.79	46.20	0.010035	26.02	157.65	24.00	1.79	40.37	40.37
Mill to Harbor	3250.*	Q5 (3179)	3179.15	29.12	34.63	37.28	43.61	0.010032	24.05	132.17	24.00	1.81	40.37	40.37
Mill to Harbor	3250.*	Q2 (1964)	1964.48	29.12	33.11	35.05	39.65	0.009948	20.52	95.73	24.00	1.81	40.37	40.37
Mill to Harbor	3150.*	Q100 (7498)	7498.00	28.13	38.31	45.33	52.94	0.009872	30.69	244.28	24.00	1.70	39.38	39.38
Mill to Harbor	3150.*	Q50 (6389)	6389.00	28.13	37.19	41.37	50.61	0.009876	29.40	217.32	24.00	1.72	39.38	39.38
Mill to Harbor	3150.*	Q20 (5054)	5053.65	28.13	35.74	39.21	47.62	0.010019	27.66	182.73	24.00	1.77	39.38	39.38
Mill to Harbor	3150.*	Q10 (4101)	4101.41	28.13	34.70	37.80	45.20	0.010013	26.00	157.77	24.00	1.79	39.38	39.38
Mill to Harbor	3150.*	Q5 (3179)	3179.15	28.13	33.64	36.29	42.61	0.010008	24.03	132.28	24.00	1.80	39.38	39.38
Mill to Harbor	3150.*	Q2 (1964)	1964.48	28.13	32.12	34.06	38.66	0.009948	20.52	95.73	24.00	1.81	39.38	39.38

HEC-RAS Plan: 8 River: Arundell Barranc Reach: Mill to Harbor (Continued)

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl	ROB Elev	LOB Elev
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)		(ft)	(ft)
Mill to Harbor	3050	Q100 (7498)	7498.00	27.13	37.31	44.42	51.94	0.009877	30.70	244.23	24.00	1.70	38.38	38.38
Mill to Harbor	3050	Q50 (6389)	6389.00	27.13	36.18	40.30	49.61	0.009882	29.41	217.27	24.00	1.72	38.38	38.38
Mill to Harbor	3050	Q20 (5054)	5053.65	27.13	34.74	38.21	46.62	0.010019	27.66	182.73	24.00	1.77	38.38	38.38
Mill to Harbor	3050	Q10 (4101)	4101.41	27.13	33.70	36.80	44.20	0.010013	26.00	157.77	24.00	1.79	38.38	38.38
Mill to Harbor	3050	Q5 (3179)	3179.15	27.13	32.64	35.29	41.61	0.010008	24.03	132.28	24.00	1.80	38.38	38.38
Mill to Harbor	3050	Q2 (1964)	1964.48	27.13	31.12	33.06	37.67	0.009972	20.54	95.65	24.00	1.81	38.38	38.38
Mill to Harbor	2950.*	Q100 (7498)	7498.00	26.13	36.30	43.15	50.95	0.009883	30.71	244.18	24.00	1.70	37.38	37.38
Mill to Harbor	2950.*	Q50 (6389)	6389.00	26.13	35.18	39.37	48.61	0.009888	29.41	217.23	24.00	1.72	37.38	37.38
Mill to Harbor	2950.*	Q20 (5054)	5053.65	26.13	33.74	37.21	45.62	0.010019	27.66	182.73	24.00	1.77	37.38	37.38
Mill to Harbor	2950.*	Q10 (4101)	4101.41	26.13	32.70	35.80	43.20	0.010013	26.00	157.77	24.00	1.79	37.38	37.38
Mill to Harbor	2950.*	Q5 (3179)	3179.15	26.13	31.64	34.29	40.61	0.010008	24.03	132.28	24.00	1.80	37.38	37.38
Mill to Harbor	2950.*	Q2 (1964)	1964.48	26.13	30.12	32.06	36.66	0.009941	20.52	95.74	24.00	1.81	37.38	37.38
Mill to Harbor	2850.*	Q100 (7498)	7498.00	25.13	35.30	41.96	49.95	0.009888	30.71	244.14	24.00	1.70	36.38	36.38
Mill to Harbor	2850.*	Q50 (6389)	6389.00	25.13	34.18	38.39	47.62	0.009893	29.42	217.18	24.00	1.72	36.38	36.38
Mill to Harbor	2850.*	Q20 (5054)	5053.65	25.13	32.74	36.22	44.62	0.010019	27.66	182.73	24.00	1.77	36.38	36.38
Mill to Harbor	2850.*	Q10 (4101)	4101.41	25.13	31.70	34.80	42.20	0.010013	26.00	157.77	24.00	1.79	36.38	36.38
Mill to Harbor	2850.*	Q5 (3179)	3179.15	25.13	30.64	33.29	39.61	0.010008	24.03	132.28	24.00	1.80	36.38	36.38
Mill to Harbor	2850.*	Q2 (1964)	1964.48	25.13	29.12	31.06	35.66	0.009968	20.54	95.66	24.00	1.81	36.38	36.38
Mill to Harbor	2750	Q100 (7498)	7498.00	24.13	34.30	40.42	48.95	0.009892	30.72	244.09	24.00	1.70	35.38	35.38
Mill to Harbor	2750	Q50 (6389)	6389.00	24.13	33.18	40.14	46.62	0.009898	29.42	217.14	24.00	1.72	35.38	35.38
Mill to Harbor	2750	Q20 (5054)	5053.65	24.13	31.74	35.22	43.62	0.010019	27.66	182.73	24.00	1.77	35.38	35.38
Mill to Harbor	2750	Q10 (4101)	4101.41	24.13	30.70	33.80	41.20	0.010013	26.00	157.77	24.00	1.79	35.38	35.38
Mill to Harbor	2750	Q5 (3179)	3179.15	24.13	29.64	32.29	38.61	0.010008	24.03	132.28	24.00	1.80	35.38	35.38
Mill to Harbor	2750	Q2 (1964)	1964.48	24.13	28.11	30.06	34.67	0.009983	20.55	95.61	24.00	1.81	35.38	35.38
Mill to Harbor	2650.*	Q100 (7498)	7498.00	23.13	33.30	39.75	47.96	0.009897	30.72	244.05	24.00	1.70	34.38	34.38
Mill to Harbor	2650.*	Q50 (6389)	6389.00	23.13	32.18	39.43	45.62	0.009903	29.43	217.10	24.00	1.72	34.38	34.38
Mill to Harbor	2650.*	Q20 (5054)	5053.65	23.13	30.74	34.22	42.62	0.010019	27.66	182.73	24.00	1.77	34.38	34.38
Mill to Harbor	2650.*	Q10 (4101)	4101.41	23.13	29.70	32.80	40.20	0.010013	26.00	157.77	24.00	1.79	34.38	34.38
Mill to Harbor	2650.*	Q5 (3179)	3179.15	23.13	28.64	31.29	37.61	0.010008	24.03	132.28	24.00	1.80	34.38	34.38
Mill to Harbor	2650.*	Q2 (1964)	1964.48	23.13	27.12	29.06	33.66	0.009965	20.53	95.67	24.00	1.81	34.38	34.38
Mill to Harbor	2550.*	Q100 (7498)	7498.00	22.14	32.31	38.97	46.96	0.009896	30.72	244.06	24.00	1.70	33.39	33.39
Mill to Harbor	2550.*	Q50 (6389)	6389.00	22.14	31.19	38.64	44.63	0.009903	29.43	217.10	24.00	1.72	33.39	33.39
Mill to Harbor	2550.*	Q20 (5054)	5053.65	22.14	29.76	33.23	41.62	0.010002	27.64	182.84	24.00	1.76	33.39	33.39
Mill to Harbor	2550.*	Q10 (4101)	4101.41	22.14	28.72	31.81	39.20	0.009995	25.98	157.87	24.00	1.79	33.39	33.39
Mill to Harbor	2550.*	Q5 (3179)	3179.15	22.14	27.66	30.30	36.61	0.009988	24.02	132.37	24.00	1.80	33.39	33.39
Mill to Harbor	2550.*	Q2 (1964)	1964.48	22.14	26.13	28.07	32.67	0.009965	20.53	95.67	24.00	1.81	33.39	33.39
Mill to Harbor	2450	Q100 (7498)	7498.00	21.14	31.31	38.13	45.97	0.009900	30.73	244.02	24.00	1.70	32.39	32.39
Mill to Harbor	2450	Q50 (6389)	6389.00	21.14	30.18	37.80	43.65	0.009927	29.45	216.91	24.00	1.73	32.39	32.39
Mill to Harbor	2450	Q20 (5054)	5053.65	21.14	28.76	32.23	40.62	0.010002	27.64	182.84	24.00	1.76	32.39	32.39
Mill to Harbor	2450	Q10 (4101)	4101.41	21.14	27.72	30.81	38.20	0.009995	25.98	157.87	24.00	1.79	32.39	32.39
Mill to Harbor	2450	Q5 (3179)	3179.15	21.14	26.66	29.30	35.61	0.009987	24.02	132.37	24.00	1.80	32.39	32.39
Mill to Harbor	2450	Q2 (1964)	1964.48	21.14	25.13	27.07	31.67	0.009938	20.52	95.76	24.00	1.81	32.39	32.39
Mill to Harbor	2375.*	Q100 (7498)	7498.00	20.38	31.01	35.68	44.43	0.028465	29.40	255.00	24.00	1.59	31.67	31.67
Mill to Harbor	2375.*	Q50 (6389)	6389.00	20.38	29.85	35.32	42.12	0.028306	28.12	227.22	24.00	1.61	31.67	31.67
Mill to Harbor	2375.*	Q20 (5054)	5053.65	20.38	28.40	31.48	39.11	0.028075	26.26	192.43	24.00	1.63	31.67	31.67
Mill to Harbor	2375.*	Q10 (4101)	4101.41	20.38	27.34	30.05	36.71	0.027599	24.56	166.97	24.00	1.64	31.67	31.67
Mill to Harbor	2375.*	Q5 (3179)	3179.15	20.38	26.27	28.54	34.13	0.026783	22.50	141.28	24.00	1.63	31.67	31.67
Mill to Harbor	2375.*	Q2 (1964)	1964.48	20.38	24.72	26.31	30.24	0.024992	18.85	104.22	24.00	1.59	31.67	31.67
Mill to Harbor	2300	Q100 (7498)	7498.00	19.63	30.65	34.06	43.13	0.007963	28.35	264.45	24.00	1.51	30.94	30.94
Mill to Harbor	2300	Q50 (6389)	6389.00	19.63	29.46	32.64	40.85	0.007888	27.09	235.85	24.00	1.52	30.94	30.94
Mill to Harbor	2300	Q20 (5054)	5053.65	19.63	27.97	30.72	37.87	0.007765	25.25	200.13	24.00	1.54	30.94	30.94
Mill to Harbor	2300	Q10 (4101)	4101.41	19.63	26.88	29.30	35.51	0.007586	23.58	173.93	24.00	1.54	30.94	30.94
Mill to Harbor	2300	Q5 (3179)	3179.15	19.63	25.77	27.79	33.00	0.007316	21.57	147.38	24.00	1.53	30.94	30.94
Mill to Harbor	2300	Q2 (1964)	1964.48	19.63	24.16	25.56	29.23	0.006790	18.06	108.79	24.00	1.49	30.94	30.94
Mill to Harbor	2227.*	Q100 (7498)	7498.00	18.94	29.89	33.40	42.53	0.008097	28.53	262.82	24.00	1.52	32.48	32.48
Mill to Harbor	2227.*	Q50 (6389)	6389.00	18.94	28.70	31.94	40.25	0.008040	27.28	234.21	24.00	1.54	32.48	32.48
Mill to Harbor	2227.*	Q20 (5054)	5053.65	18.94	27.21	30.03	37.28	0.007950	25.47	198.45	24.00	1.56	32.48	32.48
Mill to Harbor	2227.*	Q10 (4101)	4101.41	18.94	26.11	28.61	34.93	0.007815	23.83	172.11	24.00	1.57	32.48	32.48
Mill to Harbor	2227.*	Q5 (3179)	3179.15	18.94	24.99	27.10	32.43	0.007620	21.88	145.31	24.00	1.57	32.48	32.48
Mill to Harbor	2227.*	Q2 (1964)	1964.48	18.94	23.36	24.87	28.69	0.007320	18.52	106.07	24.00	1.55	32.48	32.48
Mill to Harbor	2154	Q100 (7498)	7498.00	18.26	29.16	32.72	41.92	0.008205	28.67	261.54	24.00	1.53	34.02	34.02
Mill to Harbor	2154	Q50 (6389)	6389.00	18.26	27.97	31.26	39.65	0.008161	27.43	232.94	24.00	1.55	34.02	34.02
Mill to Harbor	2154	Q20 (5054)	5053.65	18.26	26.48	29.35	36.67	0.008092	25.63	197.20	24.00	1.58	34.02	34.02
Mill to Harbor	2154	Q10 (4101)	4101.41	18.26	25.37	27.93	34.34	0.008003	24.03	170.68	24.00	1.59	34.02	34.02
Mill to Harbor	2154	Q5 (3179)	3179.15	18.26	24.25	26.42	31.85	0.007876	22.13	143.66	24.00	1.59	34.02	34.02
Mill to Harbor	2154	Q2 (1964)	1964.48	18.26	22.60	24.19	28.12	0.007708	18.84	104.25	24.00	1.59	34.02	34.02
Mill to Harbor	2145		Bridge											
Mill to Harbor	2127	Q100 (7498)	7498.00	18.01	28.89	32.46	41.69	0.008240	28.72	261.12	24.00	1.53	33.61	33.61
Mill to Harbor	2127	Q50 (6389)	6389.00	18.01	27.69	30.98	39.43	0.008212	27.49	232.41	24.00	1.56	33.61	33.61
Mill to Harbor	2127	Q20 (5054)	5053.65	18.01	26.20	29.10	36.46	0.008165	25.71	196.56	24.00	1.58	33.61	33.61
Mill to Harbor	2127	Q10 (4101)	4101.41	18.01	25.10	27.66	34.12	0.008076	24.11	170.13	24.00	1.60	33.61	33.61
Mill to Harbor	2127	Q5 (3179)	3179.15	18.01	23.97	26.15	31.64	0.007967	22.22	143.09	24.00	1.60	33.61	33.61
Mill to Harbor	2127	Q2 (1964)	1964.48	18.01	22.33	23.92	27.91	0.007838	18.95	103.66	24.00	1.61	33.61	33.61
Mill to Harbor	2118.79*	Q100 (7498)	7498.00	17.90	28.75	33.40	41.62	0.008294	28.78	260.49	24.00	1.54	31.75	31.75
Mill to Harbor	2118.79*	Q50 (6389)	6389.00	17.90	27.56	30.88	39.35	0.008270	27.56	231.81	24.00	1.56	31.75	31.75
Mill to Harbor	2118.79*	Q20 (5054)	5053.65	17.90	26.07	28.99	36.39	0.008232	25.79	195.99	24.00	1.59	31.75	31.75
Mill to Harbor	2118.79*	Q10 (4101)	4101.41	17.90	24.97	27.55	34.05	0.008153	24.19	169.56	24.00	1.60	31.75	31.75

HEC-RAS Plan: 8 River: Arundell Barranc Reach: Mill to Harbor (Continued)

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl	ROB Elev	LOB Elev
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)		(ft)	(ft)
Mill to Harbor	2118.79*	Q5 (3179)	3179.15	17.90	23.84	26.04	31.56	0.008059	22.31	142.52	24.00	1.61	31.75	31.75
Mill to Harbor	2118.79*	Q2 (1964)	1964.48	17.90	22.20	23.81	27.83	0.007966	19.05	103.10	24.00	1.62	31.75	31.75
Mill to Harbor	2110.58*	Q100 (7498)	7498.00	17.78	28.60	33.31	41.54	0.008359	28.87	259.73	24.00	1.55	29.90	29.90
Mill to Harbor	2110.58*	Q50 (6389)	6389.00	17.78	27.41	31.21	39.28	0.008341	27.65	231.08	24.00	1.57	29.90	29.90
Mill to Harbor	2110.58*	Q20 (5054)	5053.65	17.78	25.92	28.89	36.31	0.008313	25.88	195.30	24.00	1.60	29.90	29.90
Mill to Harbor	2110.58*	Q10 (4101)	4101.41	17.78	24.82	27.43	33.97	0.008246	24.29	168.89	24.00	1.61	29.90	29.90
Mill to Harbor	2110.58*	Q5 (3179)	3179.15	17.78	23.69	25.92	31.49	0.008169	22.41	141.86	24.00	1.62	29.90	29.90
Mill to Harbor	2110.58*	Q2 (1964)	1964.48	17.78	22.05	23.69	27.76	0.008114	19.17	102.47	24.00	1.63	29.90	29.90
Mill to Harbor	2102.37*	Q100 (7498)	7498.00	17.67	28.46	32.96	41.47	0.008210	28.94	259.88	27.96	1.55	28.04	28.04
Mill to Harbor	2102.37*	Q50 (6389)	6389.00	17.67	27.27	31.64	39.20	0.008398	27.72	230.51	24.00	1.58	28.04	28.04
Mill to Harbor	2102.37*	Q20 (5054)	5053.65	17.67	25.78	28.95	36.24	0.008378	25.95	194.76	24.00	1.61	28.04	28.04
Mill to Harbor	2102.37*	Q10 (4101)	4101.41	17.67	24.68	27.35	33.90	0.008320	24.36	168.35	24.00	1.62	28.04	28.04
Mill to Harbor	2102.37*	Q5 (3179)	3179.15	17.67	23.56	25.81	31.42	0.008258	22.49	141.33	24.00	1.63	28.04	28.04
Mill to Harbor	2102.37*	Q2 (1964)	1964.48	17.67	21.92	23.58	27.68	0.008236	19.27	101.96	24.00	1.65	28.04	28.04
Mill to Harbor	2094.16*	Q100 (7498)	7498.00	17.55	28.17	32.50	41.38	0.007698	29.25	268.84	38.15	1.58	26.19	26.19
Mill to Harbor	2094.16*	Q50 (6389)	6389.00	17.55	27.10	31.30	39.13	0.008026	27.84	232.30	30.54	1.59	26.19	26.19
Mill to Harbor	2094.16*	Q20 (5054)	5053.65	17.55	25.64	29.17	36.16	0.008457	26.04	194.11	24.00	1.61	26.19	26.19
Mill to Harbor	2094.16*	Q10 (4101)	4101.41	17.55	24.54	27.38	33.82	0.008410	24.45	167.72	24.00	1.63	26.19	26.19
Mill to Harbor	2094.16*	Q5 (3179)	3179.15	17.55	23.41	25.72	31.34	0.008364	22.59	140.70	24.00	1.64	26.19	26.19
Mill to Harbor	2094.16*	Q2 (1964)	1964.48	17.55	21.77	23.46	27.60	0.008378	19.38	101.38	24.00	1.66	26.19	26.19
Mill to Harbor	2085.95	Q100 (7498)	7498.00	17.31	27.97	32.48	41.30	0.006887	29.30	255.93	45.31	1.58	24.33	24.33
Mill to Harbor	2085.95	Q50 (6389)	6389.00	17.31	26.78	30.32	39.04	0.007416	28.10	227.38	38.35	1.61	24.33	24.33
Mill to Harbor	2085.95	Q20 (5054)	5053.65	17.31	25.31	28.42	36.07	0.008161	26.33	191.95	29.72	1.64	24.33	24.33
Mill to Harbor	2085.95	Q10 (4101)	4101.41	17.31	24.21	26.98	33.73	0.008711	24.76	165.66	24.00	1.66	24.33	24.33
Mill to Harbor	2085.95	Q5 (3179)	3179.15	17.31	23.09	25.48	31.25	0.008718	22.92	138.71	24.00	1.68	24.33	24.33
Mill to Harbor	2085.95	Q2 (1964)	1964.48	17.31	21.46	23.22	27.51	0.008859	19.74	99.50	24.00	1.71	24.33	24.33
Mill to Harbor	2039.95		Bridge											
Mill to Harbor	1993.95	Q100 (7498)	7498.00	16.32	27.19	31.81	39.54	0.006485	28.38	279.96	35.00	1.52	23.76	23.76
Mill to Harbor	1993.95	Q50 (6389)	6389.00	16.32	26.56	29.13	36.74	0.005779	25.74	260.98	35.00	1.42	23.76	23.76
Mill to Harbor	1993.95	Q20 (5054)	5053.65	16.32	24.58	27.50	34.63	0.007523	25.45	201.67	35.00	1.56	23.76	23.76
Mill to Harbor	1993.95	Q10 (4101)	4101.41	16.32	23.47	26.11	32.34	0.007882	23.90	171.59	24.00	1.58	23.76	23.76
Mill to Harbor	1993.95	Q5 (3179)	3179.15	16.32	22.29	24.60	29.94	0.007945	22.20	143.23	24.00	1.60	23.76	23.76
Mill to Harbor	1993.95	Q2 (1964)	1964.48	16.32	20.56	22.23	26.35	0.008285	19.31	101.76	24.00	1.65	23.76	23.76
Mill to Harbor	1986.1	Q100 (7498)	7498.00	16.32	26.70	30.87	39.45	0.006795	29.11	301.62	51.06	1.59	22.83	22.83
Mill to Harbor	1986.1	Q50 (6389)	6389.00	16.32	26.20	29.82	36.67	0.005915	26.28	276.86	47.55	1.47	22.83	22.83
Mill to Harbor	1986.1	Q20 (5054)	5053.65	16.32	24.54	27.80	34.56	0.007109	25.48	207.44	35.93	1.57	22.83	22.83
Mill to Harbor	1986.1	Q10 (4101)	4101.41	16.32	23.54	26.43	32.23	0.007295	23.67	174.93	28.93	1.55	22.83	22.83
Mill to Harbor	1986.1	Q5 (3179)	3179.15	16.32	22.37	24.68	29.82	0.007645	21.90	145.15	24.00	1.57	22.83	22.83
Mill to Harbor	1986.1	Q2 (1964)	1964.48	16.32	20.63	22.23	26.23	0.007874	18.98	103.51	24.00	1.61	22.83	22.83
Mill to Harbor	1985.62	Q100 (7498)	7498.00	12.93	22.49	28.90	39.06	0.011702	32.67	229.54	24.00	1.86	22.83	22.83
Mill to Harbor	1985.62	Q50 (6389)	6389.00	12.93	21.60	26.70	36.25	0.011150	30.72	208.00	24.00	1.84	22.83	22.83
Mill to Harbor	1985.62	Q20 (5054)	5053.65	12.93	19.88	24.27	34.14	0.012973	30.30	166.79	24.00	2.03	22.83	22.83
Mill to Harbor	1985.62	Q10 (4101)	4101.41	12.93	18.85	22.59	31.80	0.013548	28.88	142.03	24.00	2.09	22.83	22.83
Mill to Harbor	1985.62	Q5 (3179)	3179.15	12.93	17.77	21.07	29.39	0.014632	27.36	116.20	24.00	2.19	22.83	22.83
Mill to Harbor	1985.62	Q2 (1964)	1964.48	12.93	16.22	18.84	25.82	0.017767	24.86	79.02	24.00	2.41	22.83	22.83
Mill to Harbor	1972.5	Q100 (7498)	7498.00	12.92	22.02	28.12	38.88	0.012068	32.94	227.59	25.00	1.92	23.23	23.23
Mill to Harbor	1972.5	Q50 (6389)	6389.00	12.92	21.17	26.56	36.07	0.011532	30.98	206.23	25.00	1.90	23.23	23.23
Mill to Harbor	1972.5	Q20 (5054)	5053.65	12.92	19.56	23.82	33.95	0.013351	30.44	166.02	25.00	2.08	23.23	23.23
Mill to Harbor	1972.5	Q10 (4101)	4101.41	12.92	18.59	22.33	31.60	0.013923	28.95	141.65	25.00	2.14	23.23	23.23
Mill to Harbor	1972.5	Q5 (3179)	3179.15	12.92	17.56	20.84	29.21	0.015028	27.38	116.10	25.00	2.24	23.23	23.23
Mill to Harbor	1972.5	Q2 (1964)	1964.48	12.92	16.11	18.67	25.53	0.017837	24.64	79.74	25.00	2.43	23.23	23.23
Mill to Harbor	1886.25*	Q100 (7498)	7498.00	12.50	21.99	27.36	37.50	0.010760	31.61	237.23	25.00	1.81	22.76	22.76
Mill to Harbor	1886.25*	Q50 (6389)	6389.00	12.50	21.12	26.11	34.77	0.010202	29.65	215.48	25.00	1.78	22.76	22.76
Mill to Harbor	1886.25*	Q20 (5054)	5053.65	12.50	19.50	23.46	32.45	0.011481	28.88	175.01	25.00	1.92	22.76	22.76
Mill to Harbor	1886.25*	Q10 (4101)	4101.41	12.50	18.52	21.91	30.06	0.011693	27.27	150.41	25.00	1.96	22.76	22.76
Mill to Harbor	1886.25*	Q5 (3179)	3179.15	12.50	17.49	20.42	27.58	0.012152	25.49	124.73	25.00	2.01	22.76	22.76
Mill to Harbor	1886.25*	Q2 (1964)	1964.48	12.50	16.04	18.25	23.71	0.013031	22.23	88.38	25.00	2.08	22.76	22.76
Mill to Harbor	1800	Q100 (7498)	7498.00	12.08	21.96	26.63	36.27	0.009633	30.36	246.97	25.00	1.70	22.30	22.30
Mill to Harbor	1800	Q50 (6389)	6389.00	12.08	21.07	25.40	33.62	0.009073	28.43	224.75	25.00	1.67	22.30	22.30
Mill to Harbor	1800	Q20 (5054)	5053.65	12.08	19.43	23.39	31.17	0.009990	27.50	183.77	25.00	1.79	22.30	22.30
Mill to Harbor	1800	Q10 (4101)	4101.41	12.08	18.44	21.49	28.77	0.009950	25.79	159.04	25.00	1.80	22.30	22.30
Mill to Harbor	1800	Q5 (3179)	3179.15	12.08	17.41	20.00	26.26	0.010019	23.87	133.18	25.00	1.82	22.30	22.30
Mill to Harbor	1800	Q2 (1964)	1964.48	12.08	15.95	17.83	22.35	0.009907	20.30	96.75	25.00	1.82	22.30	22.30
Mill to Harbor	1710.*	Q100 (7498)	7498.00	11.49	21.63	26.06	35.22	0.008980	29.59	253.38	25.00	1.64	21.69	21.69
Mill to Harbor	1710.*	Q50 (6389)	6389.00	11.49	20.70	24.81	32.65	0.008477	27.74	230.32	25.00	1.61	21.69	21.69
Mill to Harbor	1710.*	Q20 (5054)	5053.65	11.49	19.08	22.75	30.10	0.009138	26.65	189.64	25.00	1.71	21.69	21.69
Mill to Harbor	1710.*	Q10 (4101)	4101.41	11.49	18.07	20.90	27.71	0.009011	24.91	164.62	25.00	1.71	21.69	21.69
Mill to Harbor	1710.*	Q5 (3179)	3179.15	11.49	17.03	19.41	25.21	0.008922	22.94	138.56	25.00	1.72	21.69	21.69
Mill to Harbor	1710.*	Q2 (1964)	1964.48	11.49	15.56	17.24	21.35	0.008508	19.30	101.77	25.00	1.69	21.69	21.69
Mill to Harbor	1620.*	Q100 (7498)	7498.00	10.89	21.24	25.47	34.27	0.008395	28.97	259.48	33.49	1.59	21.09	21.09
Mill to Harbor	1620.*	Q50 (6389)	6389.00	10.89	20.29	24.23	31.77	0.008025	27.19	234.94	25.00	1.56	21.09	21.09
Mill to Harbor	1620.*	Q20 (5054)	5053.65	10.89	18.67	22.12	29.15	0.008493	25.97	194.61	25.00	1.64	21.09	21.09
Mill to Harbor	1620.*	Q10 (4101)	4101.41	10.89	17.66	20.30	26.78	0.008328	24.24	169.20	25.00	1.64	21.09	21.09
Mill to Harbor	1620.*	Q5 (3179)	3179.15	10.89	16.60	18.81	24.30	0.008168	22.26	142.82	25.00	1.64	21.09	21.09
Mill to Harbor	1620.*	Q2 (1964)	1964.48	10.89	15.10	16.64	20.51	0.007684	18.66	105.28	25.00	1.60	21.09	21.09
Mill to Harbor	1530.*	Q100 (7498)	7498.00	10.30	20.84	24.91	33.40	0.007905	28.45	266.50	38.61	1.54	20.48	20.48

HEC-RAS Plan: 8   River: Arundell Barranc   Reach: Mill to Harbor (Continued)

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl	ROB Elev	LOB Elev
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)		(ft)	(ft)
Mill to Harbor	1530.*	Q50 (6389)	6389.00	10.30	19.87	23.64	30.95	0.007637	26.71	239.20	25.00	1.52	20.48	20.48
Mill to Harbor	1530.*	Q20 (5054)	5053.65	10.30	18.27	21.52	28.27	0.007961	25.38	199.14	25.00	1.58	20.48	20.48
Mill to Harbor	1530.*	Q10 (4101)	4101.41	10.30	17.23	19.71	25.93	0.007784	23.67	173.25	25.00	1.58	20.48	20.48
Mill to Harbor	1530.*	Q5 (3179)	3179.15	10.30	16.16	18.22	23.48	0.007598	21.71	146.41	25.00	1.58	20.48	20.48
Mill to Harbor	1530.*	Q2 (1964)	1964.48	10.30	14.61	16.05	19.77	0.007146	18.21	107.86	25.00	1.55	20.48	20.48
Mill to Harbor	1440.*	Q100 (7498)	7498.00	9.70	20.37	24.32	32.61	0.007581	28.09	271.48	38.92	1.52	19.88	19.88
Mill to Harbor	1440.*	Q50 (6389)	6389.00	9.70	19.39	23.04	30.19	0.007367	26.36	242.35	25.00	1.49	19.88	19.88
Mill to Harbor	1440.*	Q20 (5054)	5053.65	9.70	17.80	20.90	27.47	0.007585	24.95	202.59	25.00	1.54	19.88	19.88
Mill to Harbor	1440.*	Q10 (4101)	4101.41	9.70	16.75	19.11	25.16	0.007422	23.28	176.16	25.00	1.55	19.88	19.88
Mill to Harbor	1440.*	Q5 (3179)	3179.15	9.70	15.65	17.62	22.74	0.007250	21.37	148.79	25.00	1.54	19.88	19.88
Mill to Harbor	1440.*	Q2 (1964)	1964.48	9.70	14.05	15.45	19.11	0.006957	18.05	108.83	25.00	1.52	19.88	19.88
Mill to Harbor	1350	Q100 (7498)	7498.00	9.11	19.88	23.75	31.86	0.007310	27.79	275.96	39.14	1.49	19.27	19.27
Mill to Harbor	1350	Q50 (6389)	6389.00	9.11	18.92	22.35	29.46	0.007129	26.05	245.25	25.00	1.47	19.27	19.27
Mill to Harbor	1350	Q20 (5054)	5053.65	9.11	17.34	20.31	26.71	0.007272	24.57	205.66	25.00	1.51	19.27	19.27
Mill to Harbor	1350	Q10 (4101)	4101.41	9.11	16.26	18.52	24.44	0.007128	22.96	178.67	25.00	1.51	19.27	19.27
Mill to Harbor	1350	Q5 (3179)	3179.15	9.11	15.14	17.03	22.05	0.006980	21.09	150.75	25.00	1.51	19.27	19.27
Mill to Harbor	1350	Q2 (1964)	1964.48	9.11	13.50	14.86	18.48	0.006796	17.91	109.69	25.00	1.51	19.27	19.27
Mill to Harbor	1254.54*	Q100 (7498)	7498.00	8.51	19.41	23.15	31.08	0.007024	27.45	280.84	39.47	1.47	18.67	18.67
Mill to Harbor	1254.54*	Q50 (6389)	6389.00	8.51	18.46	21.71	28.70	0.006856	25.68	248.77	25.00	1.43	18.67	18.67
Mill to Harbor	1254.54*	Q20 (5054)	5053.65	8.51	16.87	19.70	25.94	0.006939	24.17	209.12	25.00	1.47	18.67	18.67
Mill to Harbor	1254.54*	Q10 (4101)	4101.41	8.51	15.77	17.92	23.70	0.006814	22.59	181.52	25.00	1.48	18.67	18.67
Mill to Harbor	1254.54*	Q5 (3179)	3179.15	8.51	14.63	16.43	21.33	0.006685	20.78	153.01	25.00	1.48	18.67	18.67
Mill to Harbor	1254.54*	Q2 (1964)	1964.48	8.51	12.96	14.26	17.80	0.006515	17.66	111.26	25.00	1.48	18.67	18.67
Mill to Harbor	1159.09*	Q100 (7498)	7498.00	7.92	18.93	22.57	30.35	0.006785	27.16	285.12	39.72	1.44	18.08	18.08
Mill to Harbor	1159.09*	Q50 (6389)	6389.00	7.92	18.00	21.11	27.98	0.006622	25.36	251.94	25.00	1.41	18.08	18.08
Mill to Harbor	1159.09*	Q20 (5054)	5053.65	7.92	16.41	19.11	25.22	0.006667	23.82	212.14	25.00	1.44	18.08	18.08
Mill to Harbor	1159.09*	Q10 (4101)	4101.41	7.92	15.28	17.33	23.00	0.006565	22.30	183.92	25.00	1.45	18.08	18.08
Mill to Harbor	1159.09*	Q5 (3179)	3179.15	7.92	14.11	15.84	20.66	0.006460	20.53	154.83	25.00	1.45	18.08	18.08
Mill to Harbor	1159.09*	Q2 (1964)	1964.48	7.92	12.40	13.67	17.17	0.006367	17.52	112.11	25.00	1.46	18.08	18.08
Mill to Harbor	1063.63*	Q100 (7498)	7498.00	7.32	18.40	21.97	29.66	0.006640	26.98	287.79	39.81	1.43	17.48	17.48
Mill to Harbor	1063.63*	Q50 (6389)	6389.00	7.32	17.45	20.50	27.34	0.006535	25.24	253.17	25.00	1.40	17.48	17.48
Mill to Harbor	1063.63*	Q20 (5054)	5053.65	7.32	15.85	18.51	24.57	0.006562	23.69	213.35	25.00	1.43	17.48	17.48
Mill to Harbor	1063.63*	Q10 (4101)	4101.41	7.32	14.71	16.73	22.36	0.006479	22.20	184.77	25.00	1.44	17.48	17.48
Mill to Harbor	1063.63*	Q5 (3179)	3179.15	7.32	13.52	15.24	20.05	0.006426	20.50	155.11	25.00	1.45	17.48	17.48
Mill to Harbor	1063.63*	Q2 (1964)	1964.48	7.32	11.80	13.07	16.57	0.006367	17.52	112.11	25.00	1.46	17.48	17.48
Mill to Harbor	968.181*	Q100 (7498)	7498.00	6.72	17.84	21.38	29.01	0.006561	26.88	289.26	39.81	1.42	16.88	16.88
Mill to Harbor	968.181*	Q50 (6389)	6389.00	6.72	16.88	19.90	26.70	0.006467	25.14	254.12	25.25	1.39	16.88	16.88
Mill to Harbor	968.181*	Q20 (5054)	5053.65	6.72	15.29	17.91	23.93	0.006483	23.59	214.27	25.00	1.42	16.88	16.88
Mill to Harbor	968.181*	Q10 (4101)	4101.41	6.72	14.14	16.13	21.74	0.006419	22.12	185.38	25.00	1.43	16.88	16.88
Mill to Harbor	968.181*	Q5 (3179)	3179.15	6.72	12.93	14.64	19.44	0.006399	20.47	155.34	25.00	1.45	16.88	16.88
Mill to Harbor	968.181*	Q2 (1964)	1964.48	6.72	11.20	12.47	15.97	0.006367	17.52	112.11	25.00	1.46	16.88	16.88
Mill to Harbor	872.727*	Q100 (7498)	7498.00	6.12	17.27	20.78	28.38	0.006499	26.80	290.41	39.79	1.41	16.28	16.28
Mill to Harbor	872.727*	Q50 (6389)	6389.00	6.12	16.31	19.29	26.07	0.006408	25.07	254.83	26.71	1.38	16.28	16.28
Mill to Harbor	872.727*	Q20 (5054)	5053.65	6.12	14.72	17.30	23.30	0.006426	23.51	214.95	25.00	1.41	16.28	16.28
Mill to Harbor	872.727*	Q10 (4101)	4101.41	6.12	13.54	15.53	21.13	0.006396	22.10	185.61	25.00	1.43	16.28	16.28
Mill to Harbor	872.727*	Q5 (3179)	3179.15	6.12	12.34	14.04	18.83	0.006377	20.44	155.53	25.00	1.44	16.28	16.28
Mill to Harbor	872.727*	Q2 (1964)	1964.48	6.12	10.60	11.87	15.37	0.006367	17.52	112.11	25.00	1.46	16.28	16.28
Mill to Harbor	777.272*	Q100 (7498)	7498.00	5.53	16.71	20.20	27.74	0.006428	26.71	291.78	39.79	1.41	15.69	15.69
Mill to Harbor	777.272*	Q50 (6389)	6389.00	5.53	15.76	18.70	25.45	0.006335	24.99	255.81	28.56	1.38	15.69	15.69
Mill to Harbor	777.272*	Q20 (5054)	5053.65	5.53	14.16	16.71	22.68	0.006355	23.42	215.81	25.00	1.40	15.69	15.69
Mill to Harbor	777.272*	Q10 (4101)	4101.41	5.53	12.98	14.94	20.51	0.006328	22.01	186.31	25.00	1.42	15.69	15.69
Mill to Harbor	777.272*	Q5 (3179)	3179.15	5.53	11.77	13.45	18.21	0.006307	20.36	156.12	25.00	1.44	15.69	15.69
Mill to Harbor	777.272*	Q2 (1964)	1964.48	5.53	10.03	11.28	14.77	0.006319	17.48	112.40	25.00	1.45	15.69	15.69
Mill to Harbor	681.818*	Q100 (7498)	7498.00	4.93	16.12	19.54	27.13	0.006409	26.69	292.09	39.72	1.41	15.09	15.09
Mill to Harbor	681.818*	Q50 (6389)	6389.00	4.93	15.16	18.09	24.85	0.006335	24.99	255.81	28.55	1.38	15.09	15.09
Mill to Harbor	681.818*	Q20 (5054)	5053.65	4.93	13.56	16.11	22.08	0.006355	23.42	215.81	25.00	1.40	15.09	15.09
Mill to Harbor	681.818*	Q10 (4101)	4101.41	4.93	12.38	14.34	19.91	0.006328	22.01	186.31	25.00	1.42	15.09	15.09
Mill to Harbor	681.818*	Q5 (3179)	3179.15	4.93	11.17	12.85	17.61	0.006307	20.36	156.12	25.00	1.44	15.09	15.09
Mill to Harbor	681.818*	Q2 (1964)	1964.48	4.93	9.43	10.68	14.17	0.006319	17.48	112.40	25.00	1.45	15.09	15.09
Mill to Harbor	586.363*	Q100 (7498)	7498.00	4.33	15.53	18.92	26.52	0.006393	26.67	292.36	39.65	1.40	14.49	14.49
Mill to Harbor	586.363*	Q50 (6389)	6389.00	4.33	14.56	17.48	24.25	0.006335	24.99	255.81	28.55	1.38	14.49	14.49
Mill to Harbor	586.363*	Q20 (5054)	5053.65	4.33	12.96	15.51	21.48	0.006355	23.42	215.81	25.00	1.40	14.49	14.49
Mill to Harbor	586.363*	Q10 (4101)	4101.41	4.33	11.78	13.74	19.31	0.006328	22.01	186.31	25.00	1.42	14.49	14.49
Mill to Harbor	586.363*	Q5 (3179)	3179.15	4.33	10.57	12.25	17.01	0.006307	20.36	156.12	25.00	1.44	14.49	14.49
Mill to Harbor	586.363*	Q2 (1964)	1964.48	4.33	8.83	10.08	13.57	0.006319	17.48	112.40	25.00	1.45	14.49	14.49
Mill to Harbor	490.909*	Q100 (7498)	7498.00	3.73	14.94	18.28	25.92	0.006380	26.65	292.59	39.57	1.40	13.89	13.89
Mill to Harbor	490.909*	Q50 (6389)	6389.00	3.73	13.96	16.88	23.65	0.006335	24.99	255.81	28.54	1.38	13.89	13.89
Mill to Harbor	490.909*	Q20 (5054)	5053.65	3.73	12.36	14.90	20.88	0.006355	23.42	215.81	25.00	1.40	13.89	13.89
Mill to Harbor	490.909*	Q10 (4101)	4101.41	3.73	11.18	13.14	18.71	0.006328	22.01	186.31	25.00	1.42	13.89	13.89
Mill to Harbor	490.909*	Q5 (3179)	3179.15	3.73	9.97	11.65	16.41	0.006307	20.36	156.12	25.00	1.44	13.89	13.89
Mill to Harbor	490.909*	Q2 (1964)	1964.48	3.73	8.23	9.48	12.97	0.006319	17.48	112.40	25.00	1.45	13.89	13.89
Mill to Harbor	395.454*	Q100 (7498)	7498.00	3.14	14.37	17.69	25.30	0.006334	26.59	293.46	39.55	1.40	13.30	13.30
Mill to Harbor	395.454*	Q50 (6389)	6389.00	3.14	13.38	16.28	23.05	0.006310	24.96	256.16	29.18	1.37	13.30	13.30
Mill to Harbor	395.454*	Q20 (5054)	5053.65	3.14	11.80	14.31	20.26	0.006303	23.35	216.44	25.00	1.40	13.30	13.30
Mill to Harbor	395.454*	Q10 (4101)	4101.41	3.14	10.60	12.55	18.11	0.006303	21.98	186.58	25.00	1.42	13.30	13.30
Mill to Harbor	395.454*	Q5 (3179)	3179.15	3.14	9.39	11.06	15.81	0.006282	20.34	156.33	25.00	1.43	13.30	13.30
Mill to Harbor	395.454*	Q2 (1964)	1964.48	3.14	7.64	8.89	12.37	0.006283	17.44	112.61	25.00	1.45	13.30	13.30

HEC-RAS Plan: 8 River: Arundell Barranc Reach: Mill to Harbor (Continued)

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl	ROB Elev	LOB Elev
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)		(ft)	(ft)
Mill to Harbor	300	Q100 (7498)	7498.00	2.54	13.77	17.06	24.70	0.006334	26.59	293.42	39.47	1.40	12.70	12.70
Mill to Harbor	300	Q50 (6389)	6389.00	2.54	12.78	15.68	22.45	0.006310	24.96	256.16	29.17	1.37	12.70	12.70
Mill to Harbor	300	Q20 (5054)	5053.65	2.54	11.20	13.71	19.66	0.006303	23.35	216.44	25.00	1.40	12.70	12.70
Mill to Harbor	300	Q10 (4101)	4101.41	2.54	10.00	11.95	17.51	0.006303	21.98	186.58	25.00	1.42	12.70	12.70
Mill to Harbor	300	Q5 (3179)	3179.15	2.54	8.79	10.46	15.21	0.006282	20.34	156.33	25.00	1.43	12.70	12.70
Mill to Harbor	300	Q2 (1964)	1964.48	2.54	7.04	8.29	11.77	0.006283	17.44	112.61	25.00	1.45	12.70	12.70
Mill to Harbor	211	Q100 (7498)	7498.00	1.90	13.20	17.37	24.13	0.006322	26.53	282.71	52.56	1.39	12.26	12.26
Mill to Harbor	211	Q50 (6389)	6389.00	1.90	12.08	16.07	21.87	0.006447	25.11	254.42	25.00	1.39	12.26	12.26
Mill to Harbor	211	Q20 (5054)	5053.65	1.90	10.50	12.70	19.08	0.006425	23.51	214.97	25.00	1.41	12.26	12.26
Mill to Harbor	211	Q10 (4101)	4101.41	1.90	9.31	11.29	16.92	0.006437	22.15	185.19	25.00	1.43	12.26	12.26
Mill to Harbor	211	Q5 (3179)	3179.15	1.90	8.10	9.82	14.63	0.006437	20.51	155.02	25.00	1.45	12.26	12.26
Mill to Harbor	211	Q2 (1964)	1964.48	1.90	6.35	7.65	11.19	0.006511	17.65	111.28	25.00	1.47	12.26	12.26
Mill to Harbor	150		Bridge											
Mill to Harbor	149	Q100 (7498)	7498.00	1.43	15.52	18.03	22.01	0.002989	20.72	465.40	166.36	0.97	12.46	12.46
Mill to Harbor	149	Q50 (6389)	6389.00	1.43	17.30	17.30	19.77	0.001110	13.67	851.84	225.00	0.60	12.46	12.46
Mill to Harbor	149	Q20 (5054)	5053.65	1.43	9.98	12.24	18.66	0.006532	23.65	213.70	25.00	1.43	12.46	12.46
Mill to Harbor	149	Q10 (4101)	4101.41	1.43	8.79	10.84	16.50	0.006553	22.29	184.03	25.00	1.45	12.46	12.46
Mill to Harbor	149	Q5 (3179)	3179.15	1.43	7.59	9.37	14.21	0.006567	20.65	153.95	25.00	1.47	12.46	12.46
Mill to Harbor	149	Q2 (1964)	1964.48	1.43	5.83	7.20	10.78	0.006723	17.84	110.09	25.00	1.50	12.46	12.46
Mill to Harbor	134	Q100 (7498)	7498.00	-1.54	4.65	9.06	20.94	0.014878	32.39	231.51	49.77	2.65	13.46	13.46
Mill to Harbor	134	Q50 (6389)	6389.00	-1.54	4.22	8.19	18.55	0.014146	30.37	210.34	48.04	2.56	13.46	13.46
Mill to Harbor	134	Q20 (5054)	5053.65	-1.54	3.22	7.02	17.91	0.017823	30.75	164.32	44.04	2.81	13.46	13.46
Mill to Harbor	134	Q10 (4101)	4101.41	-1.54	2.68	6.09	15.81	0.018176	29.08	141.06	41.87	2.79	13.46	13.46
Mill to Harbor	134	Q5 (3179)	3179.15	-1.54	2.09	5.07	13.57	0.018785	27.19	116.91	39.50	2.79	13.46	13.46
Mill to Harbor	134	Q2 (1964)	1964.48	-1.54	1.14	3.45	10.20	0.020821	24.16	81.31	35.71	2.82	13.46	13.46
Mill to Harbor	0	Q100 (7498)	7498.00	-1.54	5.17	9.06	18.29	0.010987	29.06	257.99	51.85	2.30	13.46	13.46
Mill to Harbor	0	Q50 (6389)	6389.00	-1.54	4.76	8.19	16.05	0.010114	26.96	237.01	50.21	2.19	13.46	13.46
Mill to Harbor	0	Q20 (5054)	5053.65	-1.54	3.76	7.02	14.91	0.012045	26.79	188.61	46.19	2.34	13.46	13.46
Mill to Harbor	0	Q10 (4101)	4101.41	-1.54	3.23	6.09	12.83	0.011612	24.86	164.95	44.10	2.27	13.46	13.46
Mill to Harbor	0	Q5 (3179)	3179.15	-1.54	2.67	5.07	10.61	0.011032	22.62	140.56	41.83	2.17	13.46	13.46
Mill to Harbor	0	Q2 (1964)	1964.48	-1.54	1.76	3.45	7.27	0.009991	18.83	104.34	38.21	2.01	13.46	13.46

Appendix C

Annotated Bibliography – Arundell Barranca Channel

Prepared by	Prepared for	Date Prepared	Title	File Name	Contents
Cotton, Shires and Associates, Inc.	Ventura County Watershed Protection District	4/2/1999	Project Alternatives Report	Arundell Barranca - combined for Consultants.PDF	Preliminary alternatives analyses including construction cost estimates, operation and land acquisition requirements, rough hydraulic designs, based on Q100=9,000 cfs. AB and SCR hydrology and water quality.
City of San Buenaventura	CH2MHILL	8/5/2005	Arundell Barranca Sediment Discharge Volumes	Arundell Barranca - combined for Consultants.PDF	1993-2005 dredging records in connecting channel and stub channel/Pierpont basin
City of Los Angeles Bureau of Engineering	Ventura County Flood Control District	1972	Arundell Barranca Hydraulic Energy Dissipator Model Study	Arundell Barranca - combined for Consultants.PDF	Results of physical model study of failed energy dissipator after the 1970 storm (1500 cfs). Defined cause of failure and to developed alternatives
CH2MHill	Ventura County Watershed Protection District	9/2006	Arundell Barranca Deficiency Study	CH2M HILL Final Report.pdf	Sedimentation study based on RUSLE equation and VCWPD method. Level of protection in AB based on RAS and WSPG models, flood damage costs, and alternatives description to increase channel capacity. Flow input in RAS model assumes installation of Lake Canyon Dam, and reach between Harbor Blvd and UPRR does not include recent improvement (20 ft wide channel)
City of Ventura Department of Public Works	State Water Resources Control Board Division of Water Quality	7/6/2010	"Info to support reassessment of bacteria listings in Peninsula Beach Ventura Harbor"	City of Ventura Cover.pdf	2002-2009 bacteria data collected in Peninsula Beach (Harbor Cove), Ventura Harbor, Ventura Keys, and AB. Indicated improvement of water quality with respect to bacteria. Coliform and enterococcus measurement results (Excel)
City of San Buenaventura Wastewater Laboratory		3/24/2010	City of Santa Buenaventura Wastewater Laboratory Analytical Quality Assurance Program 2010	Equivalent QAPP Document.pdf	Description of sampling procedures and map of sample locations
Applied Environmental Technologies Inc	USACE; CA Regional Water Quality Control Board; USEPA	4/9/2009	Sampling and Analysis Ventura Harbor Sediment Investigation	Final Rpt_Ven Harbor Connecting Channel_April 2009.pdf	Chemical compounds and physical parameters (sieve analysis) for sediment sampled (Jan 2009) in Ventura Harbor

Applied Environmental Technologies Inc	USACE; CA Regional Water Quality Control Board; USEPA	4/10/2009	Sampling and Analysis Ventura Keys Connecting Channel Sediment Investigation	Final Rpt_Ventura Harbor Sed_April 2009.pdf	Chemical compounds and physical parameters (sieve analysis) for sediment sampled (Jun 2005) in Ventura Keys
Stillwater Sciences	City of San Buenaventura	3/2011	Estuary Sub-watershed Study Assessment of the Physical and Biological Condition of the Santa Clara River Estuary	Final Synthesis Report_03-2011.pdf	Confirm that Ventura Water Reclamation Facility (VWRF) effluent discharge to the Santa Clara River Estuary (SCRE) provides an enhancement of existing beneficial use. Current issue of DO level, nutrient loading, flooding, etc… Geomorphology, hydrology, hydraulics, water quality, vegetation, wildlife habitat, etc in SCRE. 6 VWRF discharge alternatives
Stillwater Sciences	City of San Buenaventura	3/2010	Treatment wetlands feasibility study	Final Treatment Wetlands Feas Study Rpt_03-2010.pdf	Study on estimates of treatment wetland efficiency, evaluation of wetland locations, specs for treatment improvement, cost estimates, and habitat opportunity. Existing water quality.
Exponent	City of San Buenaventura	3/1999	Ventura Keys and Arundell Barranca Watershed Project	Exponent Report_Ventura Keys and Arundell Barranca Watershed Project_1.pdf	AB hydrology (selected hydrographs 1989-1998), RAS model of extension channel alternative, simulation results of RMA2 model to predict circulation pattern in the Keys, 10- year continuous HEC6 simulation for proposed sediment basin (same as Cotton Shires), HEC6 inflow sediment gradation based on deposits in the Keys, Lake Canyon debris basin, and sediment monitoring data. HEC6 estimated 127,000 cy of sediment in the basin over the 10 year period. CORMIX3 for density current
Ventura County Public Works Flood Control District		4/1999	Deficiency Study Barlow Barranca (Arundell Barranca to Foothill Road)	Complete Deficiency Study Report.pdf	VCRAT to model Barlow Barranca watershed for different flood control alternatives
West Consultants	Ventura County Watershed Protection District	6/2011	Sediment/Debris Bulking Factors and Post-Fire Hydrology for Ventura County	Bulking Factor Study - Final Combined Report (6-24-11).pdf	• Results of flow-bulking factor research/analysis • Policy recommendations for pos-tifre hydrology and sediment /debris bulking factor selecting process
CH2MHILL	Ventura County Watershed Protection District	7/2007	Arundell Barranca Detention Basin Conceptual Design	Arundell Barranca Detention Basin Conceptual Design.pdf	RAS model results of alternative involving construction of a 60 ac-ft detention basin with lateral weir. For Q100 (7498 cfs) must divert approximately 1300 cfs to the proposed detention basin. Results of ground water monitoring stations to determine basin depths. PondPack to determine the optimal basin configuration

				1000ftWeir Output.pdf 700ftWeir Output.pdf	
RBF	Ventura County Watershed Protection District	3/2008	Lake Canyon Dam Alternatives Analysis	Lake Canyon Narrative.pdf 5245-Flood Delineation-Existing-Alt1.pdf	AB deficiency based on RAS model. Hydrology based on VCRat. Project alternatives analyses. FEMA FIRM.
Ventura County Watershed Protection District		2011	Multiple documents – Summer and fall water quality data	ABC_Toxicity_10-27-2011.pdf Bacteria_09-08-2011.pdf Bacteria_10-27-2011.pdf Weck_09-08-2011.pdf Weck_10-27-2011.pdf Arundell Field and Monitoring Data.xlsx	Water quality report
Ventura County Watershed Protection District		9/2009	Evaluation of Santa Clara River Design Flows	SCRDesignFlowEval_2009.pdf	• 100 year design peak flow for SCR shows increase, because there have been more incidences of rainfall/storm events in last 30 years. • HSPF model results of SCR and its major tributaries
Ventura County Watershed Protection District		2011		Construction Cost Estimate.xlsx	Preferred alternative construction cost estimate
Ventura County Watershed Protection District		2/2008	Santa Clara Hydraulic Analysis Report	Santa Clara River.pdf SCR_RASPLOT_WITH_LEVEE.dxf	HEC-RAS modeling of SCR to generate maps of flood elevations and zones

Wood Rodgers	Ventura County Watershed Protection District	6/2011	Ventura County Watershed Protection District Santa Clara River Levee System SCR3 Improvements Design Report	Reach 1-3-Improvements Design Entire Report_June-11.pdf	Identify 44 CRF 35.10 criteria deficiency and develop rehabilitation and modification projects for SCR-3 levee system
Ventura County Watershed Protection District		12/2010		2010-048_12272010_TOPO.txt	Field survey of the reach from the UPRR up to the freeway. State plane Zone 5, NAD 27, NGVD29, ft, conversion to NAVD88 is +2.46 ft
Ventura County Watershed Protection District		3/2008		2008-004 Arundell @ Harbor NAD83.dwg	Field survey data DS of Harbor Farmroad. State plane Zone 5, NAD 83, NGVD 29, ft, conversion to NAVD88 is +2.46 ft
Ventura County Watershed Protection District		2011		Arundell5-min2004-05.csv	2005 historical hydrograph 5min data. Peak 3300 cfs
Ventura County Watershed Protection District		2011		Arundell_withUpdatedYield_hydrographPlots1-2012.xls	Pre/post fat Q50 and Q100 hydrographs used in CH2MHILL 2007 report
Ventura County Watershed Protection District		2011		700wy1995, 700wy1998	1995 and 1998 historical hydrograph
Ventura County Flood Control District		1973		Arundell Barranca - combined for Consultants.PDF	Energy dissipator modification as-built
Ventura County Flood Control District		1967		Arundell Barranca - combined for Consultants.PDF	Harbor Blvd to Ventura Harbor as-built
Ventura County Watershed Protection		3/2007		02revised_SH-02-SH-04_PLAN_PROFILE	As Built of channel between UPRR and Harbor Blvd. NGVD29. Conversion to NAVD88 is +2.46 ft

District				.DWG	
Ventura County Watershed Protection District		2011		AB Alt to SCR.dwg	Alternative involving all flow diversion to SCR
Ventura County Watershed Protection District		2011		AB PLAN & PROFILE.dwg AB X-SECTIONS.dwg	Preferred alternative drawings
Ventura County Watershed Protection District		2010		Zone 2 title-mills-arund-wooley_rev5.pdf	Drawings for repairs of Estate St. culvert
Airborne	Ventura County Watershed Protection District	2/2005		ARUNDELL lidar.dwg; 1607.dwg; 1608.dwg; 1668.dwg; 1669.dwg; 1670.dwg; 1729.dwg; 1730.dwg; 1731.dwg; 1789.dwg; 1790.dwg; 1791.dwg	Topographic map derived from Lidar points. State Plan Zone 5, NAD83, NAVD88, ft
West Coast Pipelines	Ellwood Pipeline	6/1970		Oil Line.pdf	22-in oil line plan and profile as-built drawings from Marina Park to Harbor Blvd area
Mobil Oil Corporation West Coast Pipelines		1/1970		OilLinesPlans-001.zip	22-in oil line plan as-built drawings from Marina Park to OPD
Fugro Consultants	Ventura County Watershed Protection District	2010/2007		Final Keys Points.zip; vtaharbor2010all.zip	Ventura Harbor and Ventura Keys survey data

HEC-RAS

Prepared by	Prepared for	Date Prepared	Plan Name	Comments
Ventura County Watershed Protection District		2011	Plan01	RAS model of alternative involving all flow diversion to SCR
RBF	Ventura County Watershed Protection District	3/2008	AlternativeAll_Channel_Junctions-Beachm	RAS from AB Dam to Stub Channel. Reach between Harbor Blvd and UPRR should include 2007 improvement, other reaches do not represent existing conditions. NAVD88
RBF	Ventura County Watershed Protection District	3/2008	Final Current Conditions - Junctions	RAS from AB Dam to Stub Channel, but reach between Harbor Blvd. and UPRR doesn't include 2007 improvement (expansion from 20 ft to 24 ft). NAVD88
Ventura County Watershed Protection District	nhc	2012	Geometry file only was provided: 24Jan2012MHgeo	Modified geometry file of "Final Current Conditions – Junctions" based on referencing 2009 survey data

WSPG

Prepared by	Prepared for	Date Prepared	File Name	Comments
RBF	Ventura County Watershed Protection District	3/2008	AB7.DAT	Model of culvert between Estate to Main.
Ventura County Watershed Protection District		2011	abharbor100.WSW	WSPG preferred alternative

Miscellaneous Data

- Rain Gage (216B and 216C) and Flow (#700) Measurements: http://www.vcwatershed.net/hydrodata/
- Images of damages to AB at Estate Street
- Tide, NOAA Santa Barbara Station 9411340 : http://tidesandcurrents.noaa.gov/noaatidepredictions/viewDailyPredictions.jsp?bmon=01&bday=09&byear=2012&timelength=daily&timeZone=2&dataUnits=1&datum=MLLW&timeUnits=2&interval=highlow&Threshold=greaterthanequal&thresholdvalue=&format=Submit&Stationid=9411340
- Tide, NOAA Ventura Station 9411189: http://tidesandcurrents.noaa.gov/noaatidepredictions/viewDailyPredictions.jsp?bmon=01&bday=13&byear=2012&timelength=daily&timeZone=2&dataUnits=1&datum=MLLW&timeUnits=2&interval=highlow&format=Submit&Stationid=9411189
- Bench Mark, NOAA Rincon Island Station 9411270: http://tidesandcurrents.noaa.gov/data_menu.shtml?stn=9411270 RINCON ISLAND, PACIFIC OCEAN, CA&type=Bench Mark Data Sheets

Appendix D

Cost Estimates

Cost Sumary Table		2013 Cost Levels		Contingency	20%	
15-Sep-13	printed	27-Apr-15		Maintenance PV Factor	15.37245103	(30 years @5%)

Alternative	Construction Cost	Land Cost	Maintenance Present Value	Total Cost	Incremental Cost
1	$11,004,000	$0	$892,000	$11,896,000	0
8	$13,845,000	$0	$1,353,000	$15,198,000	$3,302,000
12	$14,498,000	$0	$1,199,000	$15,697,000	$3,801,000
13	$12,296,000	$0	$853,000	$13,149,000	$1,253,000
5	$16,063,000	$1,890,625	$5,342,000	$23,296,000	$11,400,000
9	$11,485,000	$0	$28,316,000	$39,801,000	$27,905,000
9	$11,485,000		$938,000	$12,423,000	$527,000
			without VWRF charges		

Port District dredging costs vary. 2003 costs selected as representative at $22.41/cy, escalated to 2013 as $25/cy. (R. Parsons pers. comm. 18Jul2012)

Average annual volume =	24,000	cy
Average annual dredging cost @ $25/cy	$600,000	
Present value for n=30 years	$9,223,470.62	

Rounded to M

Alternative	Construction Cost	Land Cost	Maintenance Present Value	Total Cost	Incremental Cost
1	$11.0	$0.0	$0.9	$11.9	$0.0
8	$13.8	$0.0	$1.4	$15.2	$3.3
12	$14.5	$0.0	$1.2	$15.7	$3.8
13	$12.3	$0.0	$0.9	$13.1	$1.3
5	$16.1	$1.9	$5.3	$23.3	$11.4
9	$11.5	$0.0	$28.3	$39.8	$27.9
9	without treatment charges		$0.9	$12.4	$0.5

Arundell Barranca Channel Modifictaions
Alternative 1 - Preferred Alternative - Channel Expansion Harbor Boulevard to
23-Aug-13 printed 27-Apr-15
2013 Cost Levels

ITEM #	DESCRIPTION	QUANTITY	UNIT	UNIT COST	TOTAL COST	COMMENTS
1	Mobilization and Project Management	1	LS	388,440	388,440	
2	Water Pollution Control and Dewatering	1	LS	1,000,000	1,000,000	includes groundwater treatment, cofferdam at Harbor
3	Traffic Control	1	LS	25,000	25,000	
4	Protect Existing Facilities	1	LS	500,000	500,000	
5	Demolition and Clearing	1	LS	20,000	20,000	
6	Shoring and Trench Safety	26625	SF	33	876,295	
7	Excavation	13241	CY	6	80,287	
8	Fill & Backfill	6699	CY	16	107,960	
9	Hauling and Offsite Disposal	6542	CY	8	49,710	
10	Access Roads	18000	SF	2	37,307	
11	Bike Trails and Pedestrian Access	0	SF		0	
12	Fencing and Gates	2000	LF	31	62,166	
13	Concrete Channel	5517	CY	907	5,003,835	high difficulty due to limited space
14	Landscaping and Irrigation	0	LS		0	
15	AC Pavement	0	SF		0	
16	Rock Slope Protection (Grouted)	687	TONS	68	46,373	
17	Harbor Blvd Bridge	1	LS	186,400	186,400	
18	Beachmont Bridge	1	LS	186,000	186,000	
19	Energy Dissipator	1	LS	300,000	300,000	
20	RSP Outlet Channel Downstream of ED	1	LS	300,000	300,000	
	Subtotal				$9,169,774	
	Contingency				$1,833,954.79	
	Total Construction Cost				$11,003,729	
	Lands and ROW				0	
	Total				$11,003,729	

Notes

1 from unit cost formula
2 from VCWPD estimate
3 from VCWPD estimate
4 from VCWPD estimate
5 from VCWPD estimate
6 used lf*12 from VCWPD, unit cost for alts
7 used quantity from VCWPD
8 used quantity from VCWPD
9 used difference between ex and backfill in VCWPD estimate
10 not included in VCWPD estimate - used 10 feet wide, 1800 feet long
11 none included in VCWPD estimate
12 used quantity from VCWPD, added $3 per lf for gates
13 5,000 cy from VCWPD estimate, 1/2 slab 906.9451 cy
14 none included in VCWPD estimate
15 none included in VCWPD estimate
16 used VCWPD quantity, unit cost for alts
17 used VCWPD estimate - low?
18 not included in VCWPD estimate - used $5000 per lf for box culvert plus $50000 roadway improvements
19 from VCWPD, adjusted upward for deeper structure (see quantities to right)
20 increased excavation and concrete quantities to reflect 32' (instead of 29') wide channel

Alternative 1 Maintenance

Maintenance Costs (UPRR to Harbor, 5,600 lineal feet)

Item	Activity	Frequency Times/Year	Cost	Equivalent Ann. Cost	
1	Channel inspection	2	2000	4000	
2	Sediment and trash removal - channel	1	11000	11000	
3	Filter drain cleaning	1	5000	5000	
4	Concrete channel repair	0.1	100000	10000	
5	Coarse sediment removal - Harbor	0.5	40000	20000	1000 yds @40/cy
6	Periodic repairs harbor	0.1	50000	5000	
7	Service road maintenance	1	3000	3000	
				$58,000	

Present Value	$891,602
Total Construction, Land, and Maintenance	$11,895,331
BC	2.84

Arundell Barranca Channel Modifications
Alternative 8 - Alt 1 with Modification of Stub Channel   Cobble Trap
Line Items are in addition to Alternative 1 Costs (Item 20)
23-Aug-13   printed 27-Apr-15
2013 Cost Levels

ITEM #	DESCRIPTION	QUANTITY	UNIT	UNIT COST	TOTAL COST	COMMENTS
1	Mobilization and Project Management	1	LS	68,955	68,955	
2	Water Pollution Control and Dewatering	1	LS	500,000	500,000	dewater and treatment - add to Alt 1 cost
3	Traffic Control	1	LS	0	0	close portion of parking area
4	Protect Existing Facilities	1	LS	50,000	50,000	allowance for parking area and oil line
5	Demolition and Clearing	1	LS	0	0	
6	Shoring and Trench Safety	1	LS	509,004	509,004	continuous shoring at retaining wall
7	Excavation (Coarse Sed Trap, Channel & RWs)	12,111	CY	11.64	140,994	includes surcharge for wet conditions
8	Fill & Backfill (Retaining Wall)	1,289	CY	16.12	20,771	
9	Hauling and Offsite Disposal	10,822	CY	24.67	266,989	
10	Access Roads	2,400	SF	9.96	23,896	
11	Bike Trails and Pedestrian Access	0	SF	9.96	0	
12	Fencing and Gates	550	LF	30.00	16,500	decorative fence along parking area
13	Concrete Channel	0	CY	720	0	
14	Landscaping and Irrigation	6,000	SF	2.79	16,721	
15	AC Pavement	0	SF	5	0	
16	Rock Slope Protection (Grouted)	-687	TONS	68	-46,373	
17	Sed Trap Wall/Sill	1	LS	46,975	46,975	
18	Flow Deflector	1	LS	32,500	32,500	
19	Retaining Wall	650	LF	1,455	945,656	
20	Energy Dissipator/Sed Trap Bed	1	LS	19,938	19,938	
21	Outlet Channel	1	LS	-266,667	-266,667	
22	Vegetation Establishment	1	LS	3,600	3,600	
23	Restore Access Routes	1	LS	18,000	18,000	
	Subtotal				2,367,461	
24	Alternative 1 Construction Cost	1	LS	9,169,774	9,169,774	
	Subtotal				$11,537,235	
	Contingency				$2,307,446.93	
	Total Construction Cost				$13,844,682	
	Lands and ROW				$0	
	Total				$13,844,682	

Notes
1 percentage of other costs
2 allowance, in addition to $1M in Alt 1 costs
4 allowance - parking area and oil line
5 allowance
6 continuous sheet pile
7 confined area
8 native backfill, confined space
9 difference between cut and fill
10 assumes 12 ft wide, serves as access and pedestrian walkway
11 aditional area for sidewalk
12 decorative fence at top of wall
14 landscaping along parking lot and private residence
16 eliminates grouted RSP at Beachmont
20 addition of wider bed; walls accounted for in retaining wall item
21 difference between Alt 1 outlet channel and replacuing RSP in Alt 8
22 allowance
23 allowance - routes through yard and along harbor

Alternative 8 Maintenance
Maintenance Costs (UPRR to Ventura Harbor)

Item	Activity	Frequency Times/Year	Cost	Equivalent Ann. Cost	
1	Channel inspection	2	2000	4000	
2	Sediment and trash removal - channel	1	11000	11000	
3	Filter drain cleaning	1	5000	5000	
4	Concrete channel repair	0.1	100000	10000	
5	Coarse sediment removal	0.5	90000	45000	3000 yds @30/cy
6	Periodic repairs harbor	0.1	100000	10000	
7	Service Road Maintenance	1	3000	3000	
				$88,000	
	Present Value			$1,352,776	
	Total Construction, Land, and Maintenance			$15,197,457	
	BC			2.46	

Arundell Barranca Channel Modifications
Alternative 12 - Alt 1 with Deepening of Outlet Channel
Line Items are in addition to Alternative 1 Costs (Item 20)
23-Aug-13 printed 27-Apr-15
2013 Cost Levels

ITEM #	DESCRIPTION	QUANTITY	UNIT	UNIT COST	TOTAL COST	COMMENTS
1	Mobilization and Project Management	1	LS	84,805	84,805	
2	Water Pollution Control and Dewatering	1	LS	1,000,000	1,000,000	dewater and treatment - add to Alt 1 cost
3	Traffic Control	1	LS	0	0	
4	Protect Existing Facilities	1	LS	50,000	50,000	allowance for parking area and oil line
5	Demolition and Clearing	1	LS	0	0	
6	Shoring and Trench Safety	1	LS	566,957	566,957	continuous shoring at retaining wall
7	Excavation (ED, Channel, and RWs)	12,322	CY	11.64	143,451	includes surcharge for wet material
8	Fill & Backfill (Retaining Wall)	1,413	CY	16.12	22,777	
9	Hauling and Offsite Disposal	10,909	CY	24.67	269,127	includes surcharge for wet material
10	Access Roads	0	SF	9.96	0	
11	Bike Trails and Pedestrian Access	0	SF	9.96	0	
12	Fencing and Gates	250	LF	30.00	7,500	
13	Concrete Channel	0	CY	720	0	
14	Landscaping and Irrigation	6,000	SF	2.79	16,721	
15	AC Pavement	0	SF	5	0	
16	Rock Slope Protection	0	TONS	68	0	
17	Sed Trap Wall/Sill	1	LS	37,410	37,410	
18	Retaining Walls	530	LF	1,550	821,500	
20	Energy Dissipator	1	LS	-108,627	-108,627	deduction because retaining wall cost in item 18
21	Outlet Channel	1	LS	-300,000	-300,000	
	Subtotal				2,911,622	
23	Alternative 1 Construction Cost	1	LS	9,169,774	9,169,774	
	Subtotal				$12,081,396	
	Contingency				$2,416,279.10	
	Total Construction Cost				$14,497,675	
	Lands and ROW				$0	
	Total				$14,497,675	

Notes
1 percentage of other costs
2 allowance, in addition to $1M in Alt 1 costs
4 allowance - parking area and oil line
5 allowance
6 continuous sheet pile
7 confined area
8 native backfill, confined space
9 difference between cut and fill
10 assumes 12 ft wide, serves as access and pedestrian walkway
11 aditional area for sidewalk
12 decorative fence at top of wall
13 assumes wall extends above ground on parking side, decorative finish
14 landscaping along parking side
15 allowance for repaving
16 retained grouted RSP from Alt 1, add for replacing rock above retaining wall
22 deduct outlet channel costs from Alt 1

Alternative 12 Maintenance
Maintenance Costs (UPRR to Ventura Harbor)

Item	Activity	Frequency Times/Year	Cost	Equivalent Ann. Cost	
1	Channel inspection	2	2000	4000	
2	Sediment and trash removal - channel	1	11000	11000	
3	Filter drain cleaning	1	5000	5000	
4	Concrete channel repair	0.1	100000	10000	
5	Coarse sediment removal	1	40000	40000	1000 yds @40/cy
6	Periodic repairs harbor	0.1	50000	5000	
7	Service road maintenance	1	3000	3000	
				$78,000	

Present Value	$1,199,051.18
Total Construction, Land, and Maintenance	$15,696,726
BC	2.42

Arundell Barranca Channel Modifications
Alternative 13 - Alt 1 with Modification of Stub Channel
Line Items are in addition to Alternative 1 Costs (Item 20)
23-Aug-13 printed 27-Apr-15
2013 Cost Levels

ITEM #	DESCRIPTION	QUANTITY	UNIT	UNIT COST	TOTAL COST	COMMENTS
1	Mobilization and Project Management	1	LS	28,749	28,749	
2	Water Pollution Control and Dewatering	1	LS	124,000	124,000	dewater and treatment - add to Alt 1 cost
3	Traffic Control	1	LS	0	0	close portion of parking area
4	Protect Existing Facilities	1	LS	100,000	100,000	allowance for parking area and oil line
5	Demolition and Clearing	1	LS	15,000	15,000	
6	Shoring and Trench Safety	1	LS	0	0	continuous shoring at retaining wall
7	Excavation (ED, Channel, and RWs)	10,607	CY	9.45	100,271	includes excavation ds of sill
8	Fill & Backfill (Retaining Wall)	0	CY	16.12	0	
9	Hauling and Offsite Disposal	11,514	CY	17.21	198,132	combined dry and wet material
10	Access Roads	0	SF	9.96	0	
11	Bike Trails and Pedestrian Access	0	SF	9.96	0	
12	Fencing and Gates	300	LF	30.00	9,000	decorative fence along parking area
13	Concrete Channel	0	CY	720	0	
14	Landscaping and Irrigation	6,000	SF	2.79	16,721	
15	AC Pavement	3,000	SF	10.00	30,000	
16	Rock Slope Protection	2,189	TONS	53	114,915	assumes 50% portion is salvaged from existing rock
17	Grouted Rock Slope Protection	2,952	TONS	64	189,070	
18	Channel Sill	1	LS	31,666	31,666	
19	Sidewalk, picnic area, and parking lot island	1	LS	25,931	25,931	
20	Vegetation Establishment	1	LS	3,600	3,600	
21	Restore Access Routes	1	LS	90,000	90,000	
22	Outlet Channel Alt 1	1	LS	-300,000	-300,000	
	Subtotal				1,077,054	
23	Alternative 1 Construction Cost	1	LS	9,169,774	9,169,774	
	Subtotal				$10,246,828	
	Contingency				$2,049,366	
	Total Construction Cost				$12,296,194	
	Lands and ROW				$0	
	Total				$12,296,194	

Notes
1 percentage of other costs
2 allowance, in addition to $1M in Alt 1 costs
4 allowance - parking area and oil line
5 allowance
6 continuous sheet pile
7 confined area
8 native backfill, confined space
9 difference between cut and fill
10 assumes 12 ft wide, serves as access and pedestrian walkway
11 aditional area for sidewalk
12 decorative fence at top of wall
13 assumes wall extends above ground on parking side, decorative finish
14 landscaping along parking side
15 allowance for repaving
16 24-36" rock; assume half of material is salvaged from existing
19 allowance
20 allowance - routes along harbor
21 deduct Alt 1 outlet channel - covered in other items

Alternative 13 Maintenance
Maintenance Costs (UPRR to Ventura Harbor)

Item	Activity	Frequency Times/Year	Cost	Equivalent Ann. Cost	
1	Channel inspection	2	2000	4000	
2	Sediment and trash removal - channel	1	11000	11000	
3	Filter drain cleaning	1	5000	5000	
4	Concrete channel repair	0.1	100000	10000	
5	Coarse sediment removal	0.5	40000	20000	1000 yds @$40/cy
6	Service Road Maintenance	1	3000	3000	
7	Periodic repairs harbor	0.1	25000	2500	
				$55,500	
	Present Value			$853,171	
	Total Construction, Land, and Maintenance			$13,149,365	
	BC			2.68	

Arundell Barranca Channel Modifications
Alternative 5 - Alt 1 with Low Flow Diversion to Treatment Wetlands
Line Items are in addition to Alternative 1 Costs (Item 21)
23-Aug-13 printed 27-Apr-15
2013 Cost Levels

ITEM #	DESCRIPTION	QUANTITY	UNIT	UNIT COST	TOTAL COST	COMMENTS
1	Mobilization and Project Management	1	LS	152,659	152,659	
2	Water Pollution Control and Dewatering	1	LS	50,000	50,000	assumes excavation above water table
3	Traffic Control	1	LS	0	0	
4	Protect Existing Facilities	1	LS	50,000	50,000	allowance for irrigation facilities
5	Demolition and Clearing	1	LS	4,000	4,000	
6	Shoring and Trench Safety	1	LS	15,000	15,000	misc shoring for diversion structure, piping
7a	Excavation (Piping)	1,200	CY	6.06	7,276	
7b	Excavation (Treatment Wetland)	40,000	CY	3.40	135,820	
8a	Fill & Backfill (Misc)	500	CY	16.12	8,058	
8b	Fill & Backfill (Pipe)	1,100	CY	4.17	4,586	
9	Hauling and Offsite Disposal	38,400	CY	7.60	291,771	
10	Access Roads	23,400	SF	2.07	48,500	
11	Bike Trails and Pedestrian Access	0	SF		0	
12	Fencing and Gates	1,800	LF	13.24	23,839	
13	Concrete Channel	0	CY		0	
14a	Landscaping and Irrigation - Wetland	262,000	SF	2.79	730,145	treatment wetland
14b	Landscaping and Irrigation - Perimeter	79,000	SF	1.20	94,800	hydromulched and seeded basin
15	AC Pavement	0	SF		0	
16	Rock Slope Protection	0	CY	155	0	
17	Diversion Structure	1	LS	40,000	40,000	
18	Diversion Pipe	1	LS	27,500	27,500	
19	Water Control Structures	1	LS	64,000	64,000	
20	Vegetation Establishment - Wetland	1	LS	160,000	160,000	
Subtotal					1,907,953	
21	Alternative 1 Construction Cost	1	LS	9,169,774	9,169,774	
	Subtotal				$11,077,727	
	Contingency				$2,215,545.39	
	Sizing contingency				$2,769,431.74	
	Total Construction Cost				$16,062,704	
	Lands and ROW				$1,890,625	
	Total				$17,953,329	

Notes

1	percentage of other costs
2	allowance
4	allowance - no known conflicts
7a	trench unit cost
7b	very large volume unit cost
8b	native backfill
9	difference between cut and fill
12	assumes 1 gate
14a	assumes landscaping and temporary irrigaton for wetland area
14b	assumes hydromulch, but irrigated
17	assumes channel bottom can be cut
19	weir, headgate, and 18" piping between cells plus outlet riser and piping

Lands	Acres	$/ac	
Ponds	10	125000	1250000
Mitigation	10	10000	100000
Severance	2.6	62500	162500
			1512500
	sizing contingency		378125
			1890625

Alternative 5 Maintenance
Maintenance Costs (UPRR to Ventura Harbor)

Item	Activity	Frequency Times/Yr	Cost	Equivalent Ann. Cost	
1	Concrete channel inspection	2	2,000	4,000	
2	Sediment and trash removal - channel	1	11,000	11,000	
3	Filter drain cleaning	1	5,000	5,000	
4	Concrete channel repair	0.1	100,000	10,000	
5	Coarse sediment removal	0.5	30,000	15,000	1000 yds @$30/cy
6	Service Road maintenance	1	7,500	7,500	
7	Periodic repairs harbor	0.1	50,000	5,000	
8	Wetland - water control, trash, sediment, veg	12	15,000	180,000	
9	Wetland monitoring - water quality	12	5,000	60,000	
10	Wetland periodic major maintenance/repairs	0.2	250,000	50,000	
				$347,500	

Present Value	$5,341,927
Total Construction, Land, and Maintenance	$23,295,256
BC	1.87

Arundell Barranca Channel Modifications
Alternative 9 - Alt 1 with Low Flow Diversion to VWRF
Line Items are in addition to Alternative 1 Costs (Item 20)
23-Aug-13 printed 27-Apr-15
2013 Cost Levels

ITEM #	DESCRIPTION	QUANTITY	UNIT	UNIT COST	TOTAL COST	COMMENTS
1	Mobilization and Project Management	1	LS	108,049	108,049	
2	Water Pollution Control and Dewatering	1	LS	25,000	25,000	piping from AB to MH
3	Traffic Control	1	LS	6,000	6,000	Harbor Blvd
4	Protect Existing Facilities	1	LS	15,000	15,000	utilities Harbor Blvd
5	Demolition and Clearing	1	LS	5,000	5,000	
6	Shoring and Trench Safety	1	LS	48,000	48,000	shoring for 18" pipeline
7	Excavation	725	CY	9.70	7,034	
8	Fill & Backfill	650	CY	36.12	23,475	backfill for pipe
9	Hauling and Offsite Disposal	500	CY	7.60	3,799	
10	Access Roads	0	SF	0.00	0	covered in Alt 1, existing roads
11	Bike Trails and Pedestrian Access	0	SF	0.00	0	
12	Fencing and Gates	0	LF	0.00	0	decorative fence along parking area
13	Concrete Channel	0	CY	0	0	
14	Landscaping and Irrigation	0	SF	0.00	0	
15	AC Pavement	0	SF	5	0	
16	Rock Slope Protection	0	CY	0	0	
17	Diversion Structure	1	LS	50,000	50,000	includes valve or gate
18	Diversion Pipe	1	LS	85,000	85,000	
19	Tie-in to Gravity Sewer	1	LS	25,000	25,000	
	Subtotal				401,357	
20	Alternative 1 Construction Cost	1	LS	9,169,774	9,169,774	
	Subtotal				$9,571,131	
	Contingency				$1,914,226.22	
	Total Construction Cost				$11,485,357	
	Lands and ROW				$0	
	Total				$11,485,357	

Notes
1 percentage of other costs
2 allowance, in addition to $1M in Alt 1 costs
4 allowance - Harbor Blvd
5 allowance
6 continuous sheet pile
7 confined area
8 import backfill, confined space
9 difference between cut and fill, assumes about 30% of native used in backfill
17 same as Alt 5
18 same as Alt 5

Alternative 9 Maintenance
Maintenance Costs (UPRR to Ventura Harbor)

Item	Activity	Frequency Times/Year	Cost	Equivalent Ann. Cost		
1	Channel inspection	2	2000	4000		
2	Sediment and trash removal - channel	1	11000	11000		
3	Filter drain cleaning	1	5000	5000		
4	Concrete channel repair	0.1	100000	10000		
5	Coarse sediment removal - Harbor	0.5	30000	15000		1000 yds @$30/cy
6	Periodic repairs harbor	0.1	50000	5000		
7	Service road maintenance	1	3000	3000		
8	Operations and monitoring - pipeline diversion	6	1000	6000		
9	Pipeline and diversion structure maintenance	0.1	20000	2000		
				$61,000		
Estimated Charges for diverson to VWRF at 2 cfs average flow				$1,781,000	per year	would be negotiated
	Present Value			$28,316,055		
				$14,158,027	for 1 cfs average flow	
					1 cfs=	246 MG
	Total Construction, Land, and Maintenance			$39,801,412		
				$25,643,385	for 1 cfs	
	BC			1.25		
				$937,719.51	maintenance present value without City charges	
	Total without City charges			$12,423,077		