BEACH OUTLET AND HYDRODYNAMIC EVALUATION REPORT
WILLOW CREEK DAYLIGHT FINAL FEASIBILITY STUDY

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TABLE OF CONTENTS

1 INTRODUCTION 1
2 SITE DESCRIPTION 2
3 BEACH OUTLET CHANNEL EVALUATION 3
3.1 Historical Marsh Outlet Channel 3
3.2 Tidal and Flood Elevation Information 4
3.2.1 Tidal Elevations 4
3.2.2 Published Flood Elevations 4
3.3 Wave Climate 5
3.4 Beach Substrate 7
3.5 Tidal Outlet Reference Site Information 7
3.6 Proposed Beach Outlet Channel Options 9
3.7 Channel Migration Considerations 9
4 HYDRODYNAMIC EVALUATION OF PROPOSED CHANNEL ALIGNMENTS 11
4.1 Model Development 11
4.2 Model Boundary Conditions 12
4.3 Model Results 13
4.3.1 Low-flow Model Runs 13
4.3.1.1 Low-flow Model Sensitivity Analyses 15
4.3.2 High-flow Model Runs 16
4.4 Flooding Considerations 17
5 UNCERTAINTY DISCUSSION 20
6 RECOMMENDATIONS 20
6.1 Beach Outlet Options 20
6.2 Tide Gate Considerations 21
7 REFERENCES 22

List of Tables

Table 1 Tidal Elevations at the Project Site (based on NOAA Gage No. 9447130)........ 4
Table 2 Reference Site Summary Information 8
Table 3 Data Sources Utilized in HEC-RAS Model... 12
Table 4 Low-flow Model Results Summary 14

List of Figures

Figure 1 Site Location Map
Figure 2 T-Sheet for Admiralty Inlet
Figure 3 Wind Speed Distribution for Point No Point Lighthouse (1975 to 1995)
Figure 4 Proposed Conditions (Outlet Option A) Topography and HEC-RAS Model Cross-Section Locations
Figure 5 Proposed Conditions (Outlet Option B) Topography and HEC-RAS Model Cross-Section Locations
Figure 6 Comparison of Water Surface Elevations during 100-Year Flood Event for Existing and Proposed Conditions
Figure 7 Predicted Tide Gate Performance for Various Closure Heights and Storm Durations

List of Appendices

Appendix A Extreme Wind and Wave Summary
Appendix B Proposed Channel Alignments Provided by Shannon and Wilson
Appendix C Time Series Plots of Predicted Velocity and Water Depths at Select Model Cross-Sections
Appendix D Low Flow Model Sensitivity Analyses: In-Flow Rate and Mean Sea Level
Appendix E Summary of Tide Gate Evaluation Results

LIST OF ACRONYMS AND ABBREVIATIONS

1-D	one-dimensional
ACES	Automated Coastal Engineering System
City	City of Edmonds
Confluence	Confluence Environmental
ft/s	foot per second
HEC-RAS	Hydrologic Engineering Center River Analysis System
LiDAR	Light Detection and Ranging
Marsh	Edmonds Marsh
MLLW	mean lower low water
NAVD 88	North American Vertical Datum of 1988
NOAA	National Oceanic and Atmospheric Administration
NOS	National Ocean Service
S&W	Shannon and Wilson, Inc.
SR	State Route
USACE	U.S. Army Corps of Engineers

1 INTRODUCTION

Anchor QEA, LLC, was retained by Shannon and Wilson, Inc. (S&W) to complete an evaluation of coastal processes and tidal hydrodynamics to inform the final feasibility evaluation and conceptual design of proposed daylight channel alignments for Willow Creek/Edmonds Marsh. The primary objective for the Daylight project is to provide (and maximize) juvenile salmon passage into Willow Creek over a range of tidal conditions that occur during the spring and summer rearing period.

This evaluation builds on previous modeling work conducted by Anchor QEA (Anchor QEA 2013) as part of the *Willow Creek Daylight Early Feasibility Study* (S&W 2013). The earlier study characterized existing tidal hydraulics in Willow Creek/Edmonds Marsh and included preliminary modeling of a daylight channel to identify potential for increased fish passage and upstream flooding impacts. The current work, summarized in this report, includes additional one-dimensional (1-D) hydrodynamic modeling of two proposed daylight channel alignments to evaluate potential for fish passage and upstream flooding impacts and a coastal engineering/geomorphic evaluation of Marina Beach Park (and vicinity) as needed to inform selection of the preferred channel alignment and evaluate the long-term sustainability of the design. This current work was completed to support the Willow Creek Daylight Final Feasibility Study being conducted by S&W, Confluence Environmental (Confluence), and Anchor QEA for the City of Edmonds (City).

2 SITE DESCRIPTION

Edmonds Marsh (the Marsh) is an approximately 27-acre estuarine marsh located within the City of Edmonds (Figure 1). It is bordered by State Route 104 to the east, Harbor Square to the north, the BNSF Railroad tracks to the west, and the Chevron/Unocal property (and 216th Street SW) to the south. The Marsh is tidally influenced by Puget Sound; the current connection between the Sound and the Marsh is a complex system of culverts, gates, and storage ponds (SAIC 2013; S&W 2012). The Marsh also receives freshwater runoff from approximately 900 acres, including two creeks and run-off from surrounding properties (Sea-Run Consulting et al. 2007). Elevations within the Marsh range from approximately 4 feet North American Vertical Datum of 1988 (NAVD 88) (6.2 feet mean lower low water [MLLW]) to 13 feet NAVD 88 (15.2 feet MLLW). Detailed information regarding existing and historical site conditions of the Marsh can be found in the *Alignment Alternatives Screening Analysis* (S&W 2012).

3 BEACH OUTLET CHANNEL EVALUATION

The proposed location for the daylight channel for Willow Creek/Edmonds Marsh is through an existing railroad bridge (constructed as part of a previous mitigation effort) and through the City of Edmonds Marina Beach Park, which is a Puget Sound shoreline park to the southwest of the Marsh (see Figure 1). In order to develop a viable design for the daylight channel outlet through Marina Beach Park, an existing coastal processes evaluation was conducted to provide historical context for the project site (Marina Beach Park), evaluate tides and wave climate for the area, and inform design of the beach outlet channel.

3.1 Historical Marsh Outlet Channel

Historical topographic surveys and historical aerial photos are available for the project site and were reviewed to establish the unaltered (pre-development) conditions for the area. Figure 2 shows a historical topographic survey (T-sheet) from 1872 that illustrates the Marsh's original configuration and connection to Puget Sound. The historical mouth of the creek was oriented to the north and was separated from the Sound by a large spit. This suggests that the net littoral drift along the shoreline at the project location is from the south to the north. This is in agreement with the Washington State Department of Ecology's current designation for net littoral drift at Marina Beach Park, which is also south to north (Washington Department of Ecology, 2002).

From the 1890s until 1951, the Edmonds waterfront was characterized by industrial uses, included sawmills and shingle mills; the last of which was closed in 1951. A Unocal bulk fuel terminal began construction on the site in 1923 and the marsh was used for cattle pasture in the 1940s. In the early 1960's, marsh filling was begun and completion of Edmonds Marina (1962) included rerouting of the Willow Creek Drainage south (to its current condition) (Shannon and Wilson, 2013). The creek currently flows to the Sound through a series of outfall pipes (S&W 2012) located along a shore-perpendicular alignment south of Edmonds Marina within the Marina Beach Park. The new daylight channel for the creek will be routed parallel to the BNSF railroad, then through the existing BNSF bridge, south of the Marina across the Marina Beach Park to the daylight point at the Puget Sound. This places the new mouth of the creek south of the location of its historical outlet.

3.2 Tidal and Flood Elevation Information

3.2.1 Tidal Elevations

Tidal elevations for the project site were taken from the National Oceanic and Atmospheric Administration (NOAA) National Ocean Service (NOS) tidal benchmark in Elliott Bay, Seattle, Washington (gage No. 9447130; NOAA, 2003). Conversion between MLLW and NAVD 88 was taken from NOAA's VDATUM software (http://vdatum.noaa.gov/welcome.html). This information is provided in Table 1.

Table 1

Tidal Elevations at the Project Site (based on NOAA Gage No. 9447130)

Tidal Elevation (feet)	Based on MLLW Datum (feet)	Based on NAVD 88 Datum (feet)
Mean higher high water	11.3	9.1
Mean high water	10.4	8.2
Mean tide level	6.6	4.4
Mean low water	2.8	0.6
NAVD 88 (feet)	2.2	0.0
Mean lower low water	0.0	-2.2

Notes:
MLLW = mean lower low water
NAVD 88 = North American Vertical Datum of 1988

Estimates of extreme coastal water levels (in Puget Sound) at the project site were taken from NOAA estimates for NOAA gage No. 9447130. The annual maximum tide (king tide) elevation, represented by the 99% annual exceedance water level, is 12.9 feet MLLW (10.7 feet NAVD88). The 1% exceedance water level (approximate 100-year return period water level) is 14.7 feet MLLW (12.5 feet NAVD88).

3.2.2 Published Flood Elevations

The FEMA flood insurance map for the project area (Map Number 53061C1292E) has an effective date of November 8, 1999 and lists the 100-year floodplain elevation for the coastal areas of the project site as approximately 13.6 feet NAVD88 (15.8 feet MLLW). FEMA flood insurance maps for coastal areas of Snohomish County are in the process of being updated. Preliminary maps of 100-yr flood elevations along the coastal areas of the project site range from

13 to 16 feet NAVD88, with lower elevations predicted for areas north of the project site. Final maps are due for publication in 2015.

The 100-year floodplain elevation in Edmonds Marsh is not provided in the current FEMA floodplain map (Dated November 1999). Therefore, the 100-year floodplain elevation in the marsh is taken from the Dayton Street and SR-104 Storm Drain Alternatives Study completed by SAIC for the City of Edmonds (SAIC, 2013). This study also provided estimates of the 2-year, 10-year, and 25-year return period water surface elevations in the Marsh, as summarized below (see Table 1-3, Node 51 in SAIC, 2013):

- 2-year – 9.1 feet NAVD88 (11.3 feet MLLW)
- 10-year – 10.8 feet NAVD88 (13.0 feet MLLW)
- 25-year – 11.7 feet NAVD88 (13.9 feet MLLW)
- 100-year – 13.1 feet NAVD88 (15.3 feet MLLW)

The preliminary maps of the 100-year flood elevations referenced above provide a 100-year floodplain elevation in the marsh (from coastal processes only) of 12 feet NAVD88 (14.2 feet NAVD88).

For the purposes of comparing proposed conditions to existing conditions in this evaluation, the existing conditions 100-year flood elevation are taken to be 13.6 feet NAVD88 for the beach areas of the site (from November 1998 FEMA flood insurance map) and 13.1 feet NAVD88 for Edmonds Marsh west of SR-104 (SAIC, 2013).

3.3 Wave Climate

Wave data in Puget Sound near the project site are not available. Therefore, the wave conditions at Marina Beach Park were estimated through a wind-wave hindcast using standard methodology outlined in the U.S. Army Corps of Engineers (USACE) Coastal Engineering Manual (USACE 2002). This methodology uses long-term wind data and wind-wave growth formulas to estimate wave parameters from wind information.

For the project site, wind data from the Point No Point Lighthouse Coast Guard weather station (NOAA No. 742065) in Hansville, Washington, were used. The wind data encompassed wind speeds collected every 3 hours (2-minute averages) from the years 1975 to 1990. Figure 3 is a wind rose (frequency of occurrence based on wind speed and wind direction) for the wind data over the period of record. Winds are predominantly from the northwest, south-southwest, and southeast, with large wind speeds recorded for all three of these directions. Based on the wind data, waves will also approach Marina Beach Park predominantly from the northwest, southwest, and southeast. However, Marina Beach Park is somewhat sheltered from direct wave impact from the northwest by the Port of Edmonds breakwater located to the north of the park and from the south-east due to the orientation of the shoreline to the south (Point Edmund). However, waves from the north-west and south-east could have a small impact due to wave refraction (change in wave direction due to influence of bathymetry) that can change the direction of wave approach as it nears the shoreline. But, waves from the south-west to west are anticipated to dominate wave-related coastal processes at Marina Beach Park. This is in agreement with documented net littoral drift rates (from south to north) by the Washington State Department of Ecology (2002).

The wind data were used to predict wind and wave conditions associated with the 2-, 10-, 20-, 50-, and 100-year return period storm events. The extreme wind speeds and wave parameters were evaluated for each 45-degree wind direction bin from true north (e.g., 0 to 45 degrees, 45 to 90 degrees, etc.).

Predicted values of extreme wind speeds were used as input into the Automated Coastal Engineering System (ACES) using the Windspeed Adjustment and Wave Growth module (fetch limited) to predict significant wave heights and peak wave periods generated by the extreme winds (USACE 1992). Results of the wave growth analysis for all directional bins of interest and return periods are provided in Appendix A. The highest predicted waves are from the northwest and west southwest (as shown in both Figure 3 and Table 2) and range from approximately 3 feet for a 2-year wind event to almost 6 feet for a 100-year wind event.

Storm waves are therefore large enough to impact the beach channel alignment that is located within the surf zone during the event. The portion of the channel alignment located in the surf zone during the storm event will depend on the tide at the time of the storm; and the area of impact will include all elevations within the tidal range. Beach areas adjacent to the beach channel alignment lie between -2.2 feet NAVD88 (0 feet MLLW) and 9.1 feet

NAVD88 (11.3 feet MLLW) could be impacted during larger storms (due to waves). Impacts from storm waves on the beach outlet channel include sediment accumulation in the channel, migration of the channel alignment at lower elevations on the beach, and erosion of the channel banks.

3.4 Beach Substrate

A sediment exploration was conducted of the two proposed channel locations and included two borings and five test pits at various locations in Marina Beach Park (S&W 2015). The surface sediments are primarily silty sand with some gravel. The deeper borings revealed more gravel at depths over 40 feet. The surface sediments are expected to be erodible under predicted creek flows and from wind wave conditions (See Section 5). The constructed beach outlet channel will likely develop a somewhat deeper low-flow channel post-construction due to erosion of the surface sediments under creek flows. This is typical of tidal creeks in Puget Sound (see Section 3.5).

3.5 Tidal Outlet Reference Site Information

Reference sites throughout Puget Sound similar to Edmonds Marsh were reviewed to determine the size of the Marsh system and associated outlet channel width and thalweg elevation. This information was used to inform design of the bed elevation (initial) of the Marina Beach Park outlet channel through the existing bridge and out onto the beach.

Seven reference sites within the Puget Sound were analyzed to establish similar conditions for the creek. The seven sites are as follows:

1. Meadowdale Beach County Park (Lunds Gulch Creek) in Edmonds, Washington
2. Race Lagoon in Coupeville, Washington
3. Foulweather Bluff in Hanville, Washington
4. Camp Indianola in Indianola, Washington
5. Point Heyer in Point Heyer, Washington
6. Unnamed west creek on Squaxin Island, Washington
7. Unnamed east creek on Squaxin Island, Washington

The reference sites were chosen to represent similar creeks to the unmodified Willow Creek. Each creek's marsh area, channel width, depth, and outlet elevations were compared using georeferenced aerial photographs and LiDAR elevations. Summary information for the reference sites and proposed geometry for Willow Creek based on review of these sites are provided in Table 2.

Table 2
Reference Site Summary Information

Site Location	Estimated Size of Marsh **(hectares)**	Estimated Elevation of Channel Thelweg at Outlet[1]		**Estimated Wetted Top-Width of Channel[2]**		**Estimated Depth of Channel[3]**	
		(feet, MLLW)	(feet, NAVD88)	**(meters)**	**(feet)**	**(meters)**	**(feet)**
Meadowdale	160.0	9.8	7.6	1.5	5.0	0.6	2.0
Race Lagoon	10.4	6.4	4.2	15.0	49.0	0.6	2.0
Foulweather Bluff	9.6	9.5	7.3	4.5	15.0	0.6	2.0
Indianola, WA	30.8	10.5	8.3	7.6	25.0	0.6	2.0
Point Heyer, WA	2.0	10.5	8.3	3.6	12.0	0.3	1.0
Squaxin Island-west	7.0	6.2	4.0	3.6	12.0	0.3	1.0
Squaxin Island-east	2.3	8.0	5.8	12.1	40.0	1.0	3.3
Willow Creek (Proposed)	8.0	6.0[4]	3.8	4 to 12[5]	13 to 40[5]	n/a	n/a

Notes:
MLLW = mean lower low water datum
NAVD 88 = North American Vertical Datum of 1988
1 = Estimated channel elevation found using 2005 Puget Sound lowlands Light Detection and Ranging (LiDAR). May not represent the actual thelweg elevation.
2 = Estimated channel width found using Google Earth
3 = Estimated channel depth found using Google Earth and various reports on the sites
4 = Willow Creek channel outlet elevation is +6 feet MLLW (+4 ft NAVD88) based on the railway underpass elevation.
5 = Estimated channel width for Willow Creek estimated using reference site comparisons

The estimated size of the marsh at Willow Creek (8 hectares) is closest in size to three reference sites: Race Lagoon, Foulweather Bluff, and Squaxin-Island west. The estimated wetted width of channel at Willow Creek is more in-line with two of those sites; Foulweather Bluff and Squaxin-Island West. For these two reference sites, the estimated elevation of the thelweg at the outlet is approximately 4 feet NAVD88 (6 feet MLLW). Therefore, based on review of these reference sites, the thalweg elevation of the beach outlet at Willow Creek is proposed as 4.0 feet NAVD 88 (approximately 6.0 feet MLLW) at the culvert location and beach outlet, daylighting at that same elevation on the beach, which is roughly the mean tide elevation. This is consistent with the modeled geometry in the initial phase of work conducted for this project (Anchor QEA 2013) and is similar to thalweg elevations of the existing Willow Creek Daylight channel upstream from Marina Beach Park and the BNSF bridge.

3.6 Proposed Beach Outlet Channel Options

S&W, with input from Anchor QEA as documented in this report, developed two options for the beach outlet channel (S&W 2014). Figures developed by S&W for the beach outlet channel options are provided in Appendix B. Option A and Option B channels differ downstream of the bridge, however the channel alignments and geometry upstream of the bridge are identical, and were developed by S&W.

Option A is similar to the original alignment developed as part of the Willow Creek Early Feasibility Study (S&W 2013) and is aligned through the approximate center of the dog off-leash area at Marina Beach Park. This alignment requires the channel to make a 90-degree turn directly downstream of the bridge. The channel is approximately 450 feet long from the bridge to the point where it outlets at +4 feet NAVD88.

Option B is oriented north of Option A and allows for a straighter channel alignment directly downstream of the bridge. The northerly alignment is more similar to the historical channel alignment prior to development in the project area. The channel is approximately 600 feet long from the bridge to the point where it outlets at +4 feet MLLW.

3.7 Channel Migration Considerations

Option A minimizes required excavation to construct the channel by minimizing the channel length between the bridge and the +4 ft NAVD88 contour (see Appendix B). However, the 90-degree bend in the channel downstream of the bridge may need to be armored due to high velocities in the bend during high flow events. In addition, the outlet will be oriented to the south-west and will likely trend towards the north-west in the long-term due to the south to north net littoral drift direction at the project site. This could impact the armored point to the north of the proposed outlet.

Option B requires more excavation downstream of the bridge to construct the channel than Option A, and also bisects the existing parking lot and lawn area. However, the channel has a more natural, straighter alignment downstream of the bridge which should reduce the need for bank armoring downstream of the bridge likely required for Option A. The outlet for Option B is initially oriented to the north-west, which more closely matches the orientation of the historical inlet and the equilibrium location of the inlet channel given net littoral drift is from south to north. Therefore, the channel alignment for Option B may be more stable than Option A in the long-term.

In addition to the longer-term process of littoral drift, large storm waves could cause erosion and sedimentation in and around the portion of the outlet channel subjected to direct wave breaking. Storm waves can mobilize sediment along the beach which could accumulate in the channel mouths reducing conveyance in the channel at lower flows. A large flow event from the creek could mobilize the accumulated sediment and move it out of the channel. However, there will likely be a period of time between a sedimentation causing wave event and channel opening flow event that could result in a constricted flow condition. This is a natural process for tidal creek outlets subject to waves, and is therefore in line with process based restoration efforts. Both Option A and Option B will be impacted by this process; as storm waves can approach either channel obliquely (storm waves can approach the site from the south-west clockwise to the north-west). However, the outlet for Option B is somewhat sheltered from storm waves form the north-west due to the Port of Edmonds breakwater located to the north of the park. The south-west direction is predicted to produce the largest storm waves. The outlet for Option A is oriented with the south-west direction, and may exhibit less sedimentation during storm events from the south-west than Option B, which is aligned almost parallel with that storm wave direction.

4 HYDRODYNAMIC EVALUATION OF PROPOSED CHANNEL ALIGNMENTS

Hydrodynamic modeling (1-D, Hydrologic Engineering Center River Analysis System [HEC-RAS]) was conducted to evaluate low- and high-flow tidal hydrodynamics for the two proposed beach outlet options (A and B). This modeling built upon modeling work conducted by Anchor QEA as part of the early feasibility study, and information regarding development and calibration of the model can be found in the *Final Tidal Marsh Hydraulics Report* (Anchor QEA 2013).

Low-flow model runs for each option were developed to evaluate potential fish passage into the Marsh based on typical spring and summer rearing periods. High-flow model runs for each option were developed to evaluate potential for flooding in the Marsh and upstream in Shellabarger Creek.

4.1 Model Development

The proposed conditions models for Options A and B were developed based on the existing topography and proposed channel geometry developed by S&W (S&W 2014). Data sources used to develop the proposed conditions models are listed in Table 3. Digital terrain models of both options were provided to Anchor QEA by S&W for use in the modeling effort. The thalweg of the beach outlet channel is approximately 4 feet NAVD 88 (6.0 feet MLLW), as discussed in Section 3.6.

Table 3
Data Sources Utilized in HEC-RAS Model

Date Type	Source	Spatial Extent	Temporal Extent
Topography/Stream Geometry	S&W; Digital Terrain Model	Project Area	N/A
Spring Tidal Data	NOAA	Lower Willow Creek	May 1–15, 2008
High-flow Tidal Data	NOAA	Lower Willow Creek	December 17–31, 2007
Spring Flow Conditions	Provided by S&W; taken from SR-104 HSPF Model (SAIC 2013)	Shellabarger Creek and Upper Willow Creek	May 1–15, 2008
High-flow Conditions	Provided by S&W; taken from SR-104 HSPF Model (SAIC 2013)	Shellabarger Creek and Willow Creek	December 1–14, 2007

Note:
HEC-RAS = Hydrologic Engineering Center River Analysis System
NOAA = National Oceanic and Atmospheric Administration
SR = State Route

Surface data from S&W were processed using HEC-GeoRAS, a tool developed for ArcGIS to process geospatial data for use in the HEC-RAS model. HEC-RAS geometry data were developed from HEC-GeoRAS at cross-sections within the project area. The cross-sections and existing surface data are shown in Appendix B for Options A and B, respectively.

Cross-sections were adjusted and the railroad bridge was added using survey data provided by S&W. Manning's roughness values were taken from the original model (Anchor QEA 2013).

4.2 Model Boundary Conditions

The low- and high-flow HEC-RAS models were run as unsteady flow models to simulate tidal cycles during a typical spring period for a typical spring/summer low-flow and predicted 100-year flow. Low flows were provided by the City of Edmonds, Dayton Street flood study model (SAIC, 2013) and represent average flows during May in Shellabarger and Upper Willow creeks (0.5 and 0.3 cubic foot per second, respectively). The high-flow event was provided the City of Edmonds, Dayton Street flood study model, taken from flood modeling work completed by SAIC (SAIC 2013) and represents a flow event in December

2007. To improve the stability of the model, the model was split into three reaches (Upper Willow Creek, Shellabarger Creek, and Lower Willow Creek). Figures 4 and 5 show the Lower Willow Creek model reach. Flow conditions were assumed to be concurrent such that the Lower Willow Creek flow was equal to the sum of the Upper Willow Creek and Shellabarger Creek flows. Simulation time periods were set for 2 weeks. Time-series plots for tidal elevations and 100-year high flow are provided in Appendix C.

4.3 Model Results

Four model simulations were completed: one low-flow and one high-flow simulation for each channel alignment alternative (Option A and Option B). Each simulation was run for a 2-week timeframe with a tidal downstream boundary condition. Results for the low- and high-flow simulations are described in detail below.

4.3.1 Low-flow Model Runs

The purpose of the low-flow model runs was to evaluate in-channel flow velocities in the daylight channel and Marsh to assess potential for fish access. Anchor QEA provided predicted depth and cross-sectional averaged velocities, water surface elevations, and water depths at each model cross-section/station (see Figures 4 and 5) to Confluence Environmental Company (Confluence). Confluence conducted an evaluation that compared the low flow model results with metrics desirable for fish passage. This evaluation is documented in a technical memorandum developed by Confluence for S&W entitled *Analysis of Proposed Fish Habitat with Willow Creek Daylighting and Restoration* (Confluence, 2015). Time series plots of velocity and elevation at various model cross-sections are provided in Appendix C.

A summary of predicted velocities in the daylight channel upstream of the railroad bridge is provided in Table 4 as a percent occurrence of in-channel current speeds greater than or equal to 1 ft/s or 2 ft/s. Cross-section/station numbers reference Option B numbering (Figure 5). Predicted model velocities for portions of the daylight channel upstream of the bridge are identical for both Option A and Option B.

Table 4
Low-flow Model Results Summary; Upstream of the Railroad Bridge
(Options A and B)

Cross-section/Station (Based on Option B)	Percent of Time Velocities ≤ 1 ft/s	Percent of Time Velocities ≤ 2 ft/s
3158.385	97%	98%
3034.243	99%	99%
2824.682	74%	97%
2626.523	71%	86%
2483.468	75%	76%
2292.697	96%	99%
2193.34	83%	98%
2066.47	66%	92%
1973.912	66%	88%
1702.128	65%	87%
1568.822	36%	58%
1382.35	58%	99%
1302.334	62%	99%
1123.483	68%	99%
976.2018	74%	99%
833.6823	80%	100%
737.4906	84%	100%
668.7243	83%	100%
617.8932	81%	100%

Note:
ft/s = foot per second

Plots of predicted in-channel velocities and water depths for select model sections are provided in Appendix C.

A majority of cross-section/station locations have velocities that are less than or equal to 1 ft/s over 60% of the simulation time period. Station locations in the Marsh and at the bridge location meet the 1 ft/s criterion over 70% of the time, with many cross-sections in the 80% and 90% ranges. The highest velocities occur in the straight portion of the channel (Sections 2066 through 1123), and one Station at 1568 meets the 1 ft/s criterion just under 40% of the time. The 2 ft/s criterion is met over 75% of the time for all Stations, with the majority being above 90%, except for Station 1568, which is around 60% of the time. The 0.5 foot depth criterion is met for all stations over 70% of the time, with the majority of locations at over 90%.

The results for sections downstream of the bridge for Options A and B are shown in Table 5. Similar to stations upstream of the railroad bridge, stations downstream of the bridge meet the 1 ft/s criterion over 60% of the time, with many stations over 70% of the time. The 2 ft/s criterion is met over 80% of the time for all stations, with the majority being above 90%.

Table 5
Low-flow Model Results Summary; Downstream of the Railroad Bridge
(Options A and B)

Option A			Option B		
Cross-section/Station	Percent of Time Velocities ≤ 1 ft/s	Percent of Time Velocities ≤ 2 ft/s	Cross-section/Station	Percent of Time Velocities ≤ 1 ft/s	Percent of Time Velocities ≤ 2 ft/s
388	73%	81%	451	66%	98%
233	82%	88%	374	65%	97%
162	89%	93%	285	64%	95%
66	85%	92%	165	61%	91%
			97	60%	89%

Note:
ft/s = foot per second

The higher velocities in the straight portion of the channel are not unexpected, because the channel has a straight alignment for approximately 1,300 feet due to site constraints that limit where the channel can be located. However, during design, rough channel elements (such as large woody debris) can be added to the straight portion of the channel to provide to provide variable velocities, which in turn can help improve fish passage by lowering velocities below those predicted in this model.

4.3.1.1 Low-flow Model Sensitivity Analyses

Two model sensitivity analyses were conducted to evaluate the sensitivity of the low-flow model results (velocity and water depth) to incremental changes in upstream in flow volume and changes to mean seal level (sea level rise). The purpose of the sensitivity analysis is to identify potential uncertainty in the low-flow model results based on variability of chosen input boundary conditions.

The low-flow model upstream flow rate was 0.8 cubic feet per second for all runs. For the sensitivity analysis, the in-flow rate was varied by plus or minus 20% (0.64 and 0.96 cubic feet per second). Appendix D provides a comparison of velocities below 2 feet per second and water depths greater than 0.8 feet for all three in-flow rates as predicted by the model. The results of the model varied by less than 2% for based on velocity threshold and less than 3% based on water depth threshold between the three simulations.

Appendix D also provides a similar comparison for low flow model results that used the same in-flow rate (0.8 cubic feet per second) but varied mean sea level. Median predicted increases in sea-level for Seattle (NRC, 2012) for the years 2030 (7 centimeters) and 2050 (17 centimeters) were added to the tidal elevation time series used as the downstream boundary condition for the model. Appendix D provides a comparison of velocities below 2 feet per second and water depths greater than 0.8 feet for the three different mean sea level elevations as predicted by the model. The results of the model varied by less than 2% based on velocity threshold and 3% based on water depth threshold between the three simulations.

4.3.2 High-flow Model Runs

The high-flow model developed as part of the early feasibility study (Anchor QEA 2013) was modified to represent the proposed channel alignments, Options A and B (see Appendix B). Boundary conditions and other model parameters remain unchanged from the previous high flow modeling work (Anchor QEA, 2013), and represent an approximate 100-year hydrograph taken from a storm event in December 2007 (SAIC, 2013). Predicted velocities and water surface elevations from the updated high-flow model are the same upstream of the bridge as the initial high-flow modeling work (for both Options A and B) conducted by Anchor QEA in 2013 as part of the Early Feasibility Study (Anchor QEA, 2013). Figure 6 shows a comparison of water surface elevations in the marsh for existing and proposed (Options A or B) conditions, as well as water surface elevation just upstream of the bridge for proposed conditions (Options A or B). These results are summarized below:

- Water surface elevations in the marsh for exiting conditions reach a maximum of almost 13 feet NAVD88. This elevation compares well with the reported 100-year flood elevation for the Marsh provided in SAIC 2013 (13.2 feet NAVD 88)(see Section 3.2.2).
- Water surface elevations in the marsh for proposed conditions (Options A and B) are lower than existing conditions, reaching maximum elevations of approximately 11 feet NAVD88. This is less than the existing documented and predicted 100-year flood elevation in the marsh by approximately 2 feet.
- Other than at the peak of the flood event (12/4), water surface elevations in the marsh are lower for existing conditions (which include the current outfall system for Willow Creek)) than for proposed conditions (when the channel is daylighted and hasno hydraulic controls).

4.4 Flooding Considerations

The Daylight project high-flow (100-year) model simulation predicts that water surface elevations in the Marsh are not significantly higher than the predicted existing condition 100-year flood elevation in the Marsh provided by SAIC 2013 (13.2 feet NAVD88). However, water surface elevations in the Marsh can reach approximate high tide elevations on a regular basis once the daylight channel is constructed. The mean higher high tide level of 9.1 feet NAVD88 is close to the 2-year flood elevation in the marsh and the king tide elevation of 10.7 feet NAVD88 is close to the 25-year flood elevation in the marsh (SAIC, 2013)(see Section 3.2.2). This will increase the frequency of occurrence of high water in the Marsh and Shellabarger Creek compared to existing conditions, where there are currently hydraulic controls on the creek outlet to attenuate the high tide elevation in the marsh. At present, the City of Edmonds has an existing tide gate, located at the end of the Port of Edmonds pipe in a vault in Marina Beach Park, that is closed manually from October through March each year. For reference, low spots on SR-104 are at elevation 12.0 feet NAVD88 near Harbor Square and as low as 10.6 feet NAVD88 at the SR-104 and Dayton Street intersection.

In order to reduce the risk of flooding at low spots adjacent to the marsh, such as the SR-104 and Dayton Street intersections, due to tidal inundation during large storm events, a self-regulating tide gate could be constructed in Willow Creek. The tide gate could be constructed near the location of the existing Willow Creek channel overflow into the Port of Edmonds storm drain pipes to reduce the propagation of higher tides into the marsh. The tide gate would need to be designed to limit tidal flooding potential to roadways and upland areas within defined operational criteria.

In order to evaluate the potential benefit to flood reduction and impact to fish passage of a tide gate constructed in the channel at the existing overflow (about Model Station 1450 in Figure 5), additional HEC-RAS model runs were conducted with the proposed gate inserted into the model. In addition to modeling, a GIS evaluation was conducted to look at potential storage at different water surface elevations in the marsh above 8.0 feet NAVD88. The tide gate utilized in the model consisted of three 4 foot diameter culverts with invert elevations of 5.5 feet NAVD 88.

A low flow model run was conducted with the tide gate in place to evaluate velocities in the tide gate culvert pipes (with the gate open) over the range of tidal elevations when the gate would remain open. The water surface elevation when the gate would shut was assumed to be 9.5 feet NAVD88 for the low flow run. Water depths and velocities at select stations in the model, including the upstream and downstream end of the culvert were provided to Confluence for inclusion in their fish passage evaluation (Confluence, 2015). Based on preliminary results of the fish passage evaluation, an additional low-flow tide gate simulation was conducted with the middle culvert barrel invert lowered to 4.0 feet NAVD 88. The results of this model run were also provided to Confluence for inclusion in their fish passage evaluation (Confluence, 2015). Plots of water depth and velocity for select sections for the tide gate simulations are provided in Appendix C.

High flow events were simulated in the HEC-RAS model using a series of closure water surface elevations for the self-regulating tide gate and associated time periods when the tide would stay above the closure water surface elevation as the storm duration. The storm inflow was taken to be the approximate average of the 100-year flow hydrograph used in previous modeling work (early December 2007); approximately 72 cubic feet per second. The peak flow during that event was approximately 91 cubic feet per second.

To ground truth and augment model results, a GIS stage-storage evaluate was conducted for the marsh for water surface elevations above 8.0 feet NAVD88. At this elevation, the marsh is basically a bath tub model and the relationship between water surface elevation and storage volume is approximately linear. Appendix E summarizes the augmented results of the HEC-RAS modeling and GIS evaluation, and provides estimates of the storage volume in the marsh above 8.0 feet NAVD88 and predicted water surface elevations for various flow rates and gate closure heights. Figure 7 provides a graphical representation of the summary table in Appendix E.

Figure 7 shows predicted water surface elevations in the marsh based on tide closure elevations of 8.0 feet to 9.5 feet NAVD88. Each closure elevation has an associated time of closure which is equal to the approximate length of time the tide remains higher than the closure elevation over a typical tidal cycle. As the closure depth for the gate is decreased, the time the gate will remain closed increases. While the initial water surface elevation and water volume in the marsh is less when the gate shuts, the marsh must endure a longer period of inflow before the gate can open again and drain the marsh. Therefore, there is a relatively complicated relationship between closure height for the gate and predicted water surface elevation in the marsh.

Water surface elevations remain at least 2 feet below the existing 100-year elevation in the marsh over the range of inflow condition (up to 140 cfs) and storm durations evaluated (up to 5 hours). Closing the gate at 8.0 feet NAVD88, even with the 5 hour closure duration, provides the best performance in terms of flood reduction in the marsh at high flows due to large volume of storage in the marsh above 8.0 feet. Closure heights of 8.5, 9.0 and 9.5 feet NAVD88 all perform about the same due to variable closure durations and all would be viable options for flood control in the marsh. For instance, the predicted water surface elevation in the marsh for the average 100-year inflow (~ 72 cfs) would be 10.15, 10.25, and 10.3 feet, respectively. Each of these predicted water surface elevations is below 10.6 feet NAVD88, which is the elevation of the SR-104 and Dayton Road intersection.

5 UNCERTAINTY DISCUSSION

The results of the tidal hydrodynamic evaluation for this project were based on the best available data at the time and targeted to meet the specific needs of the final feasibility evaluation. Uncertainties in the model are due to limitations of the input data to the model (i.e., topography, flows, and water levels) and assumptions made by the model itself. Specific potential sources of uncertainty with this study include the following:

- Multiple sources of topography information, with different spatial resolutions, coverage areas, and collection times, were used to create the digital elevation models used to develop both the existing and proposed conditions hydrodynamic (HEC-RAS) models.
- Flow data were provided by a run-off model completed by SAIC (SAIC 2013); there are no stream gage data available for the project area.
- The existing conditions model was not calibrated based on synoptic measured flow and water level data in the Marsh due to lack of data.

6 RECOMMENDATIONS

6.1 Beach Outlet Options

Option A and B for the beach alignment have the same hydraulic conditions upstream of the bridge, and very similar hydraulic conditions downstream of the bridge (creek flow velocity and water depth) out onto the beach. Option A is routed through the existing off-lease dog park, whereas Option B is bisecting the existing parking lot and lawn area, which would need to be relocated and/or redesigned.

Option B is aligned in the direction of the historical inlet (to the north-west) and is more aligned with the net littoral drift direction (south to north), which will tend to push the inlet to the north-west. Option A is aligned to the south-west, and would therefore be at higher risk of channel migration as the outlet tries to align itself with the net littoral drift direction. Option B has a straight alignment downstream of the bridge, whereas Option A has a sharp 90 degree turn downstream of the bridge that would likely require bank armoring to remain stable during high creek flows.

References

Therefore, based on hydraulic and coastal processes considerations, Option B is the preferred option for the beach outlet channel. However, as the alignment for Option B greatly impacts the existing Marina Park infrastructure, public usage and park design will need to be taken into consideration when choosing a final preferred alignment.

6.2 Tide Gate Considerations

A self-regulating tide gate set to close at 9.5 feet NAVD88 (11.7 feet MLLW) could be a viable solution to flooding concerns in the marsh, even for low lying areas such as the SR-104 and Dayton road intersection. The proposed elevation for gate closure (9.5 feet NAVD88) is 0.4 feet above mean higher high water at the site. It is expected that once closed, tides can remain higher than 9.5 feet NAVD88 for up to three hours. The gate will provide a fish barrier when tidal elevations are above 9.5 feet NAVD88 and the gate is closed, but this is expected to occur only a few hours at a time on certain days of the month. Elevations in the culvert do not appear to be significantly higher than in the straight channel without the tide gate.

However, the self-regulating tide gate will need to be consistently maintained to ensure that it continues to function as designed. Situations where the gate is stuck open or closed could result in undesirable flooding of lower-lying roadways and upland areas surrounding the marsh.

In addition, water surface elevations in the marsh predicted by the HEC-RAS modeling based on proposed restoration actions at the project site should be used to update the downstream boundary conditions in the flood routing model developed for SR-104 by the City of Edmonds (SAIC, 2013). The flood routing model should be re-run with these updated boundary conditions to verify there are no flooding risks due to proposed hydraulic changes in the marsh upstream of the extent of the HEC-RAS model.

References

7 REFERENCES

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FIGURES

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NOTE: Aerial photo and background data provided by ESRI.

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Figure 1
Site Location Map
Tidal Marsh Hydrodynamics Report
Willow Creek Daylight Early Feasibility Study

NOTE: Aerial photo provided by ESRI.

Figure 2
T-Sheet for Admiralty Inlet
Tidal Marsh Hydrodynamics Report
Willow Creek Daylight Early Feasibility Study

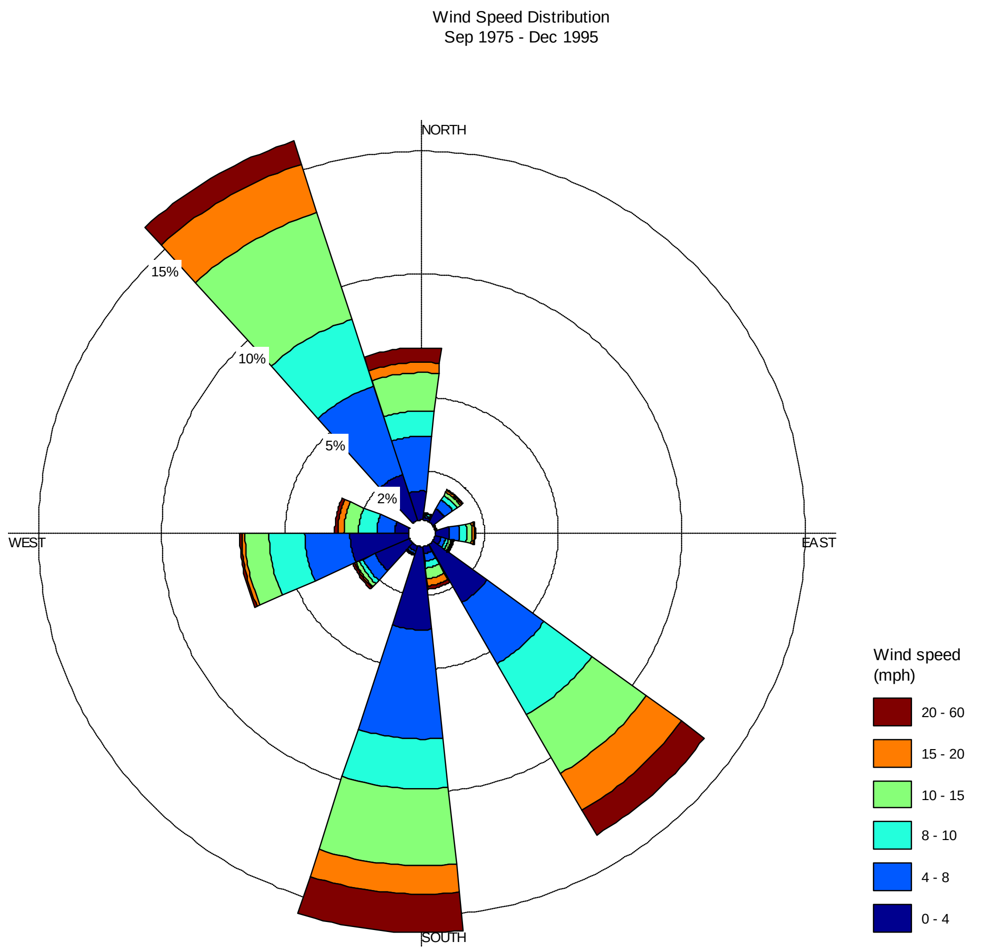

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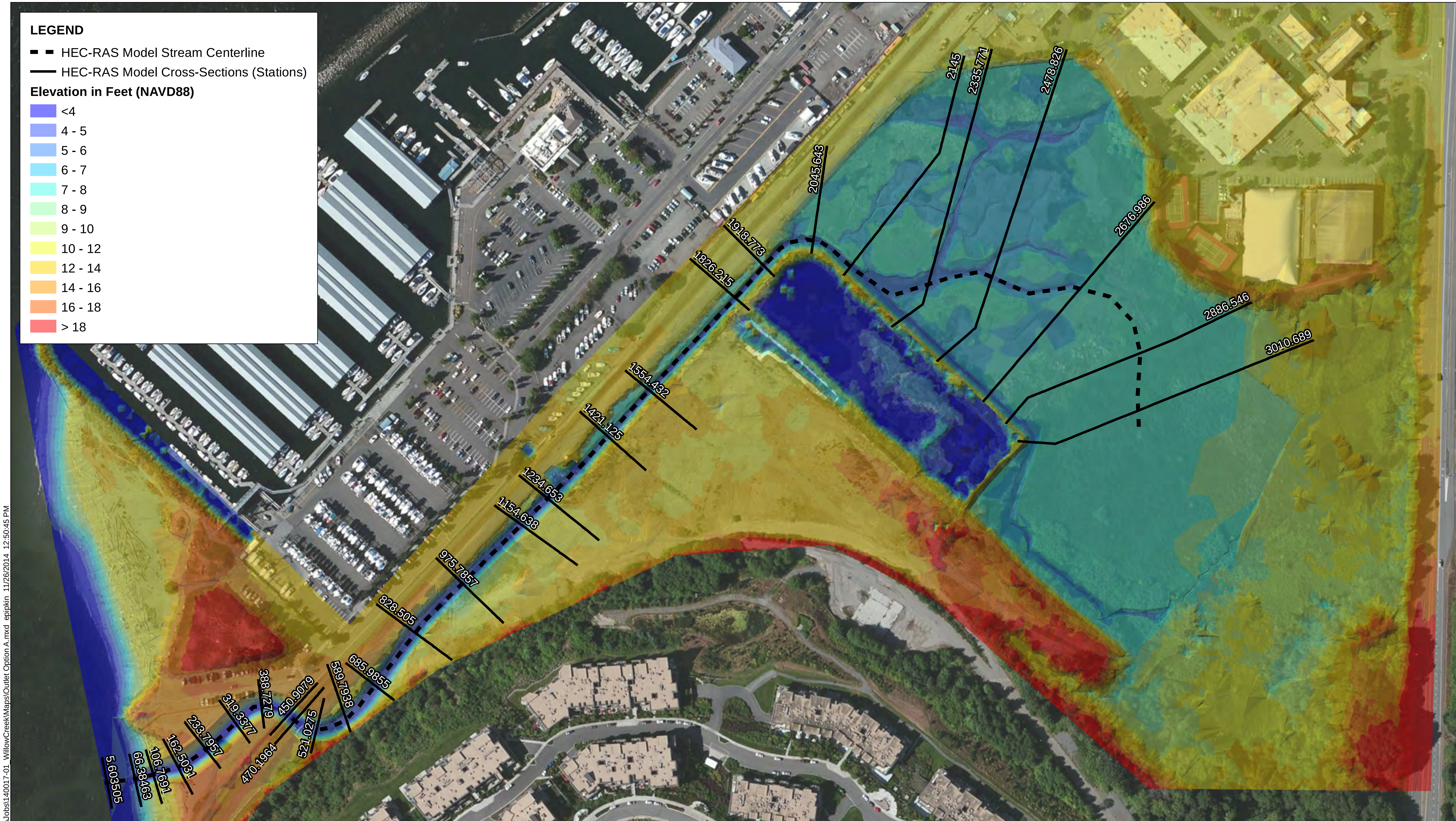
Figure 3

Wind Speed Distribution for Point No Point Lighthouse Wind Speed Distribution (1975-1995)

Hydrodynamic Evaluation Report

Willow Creek Final Feasibility Study



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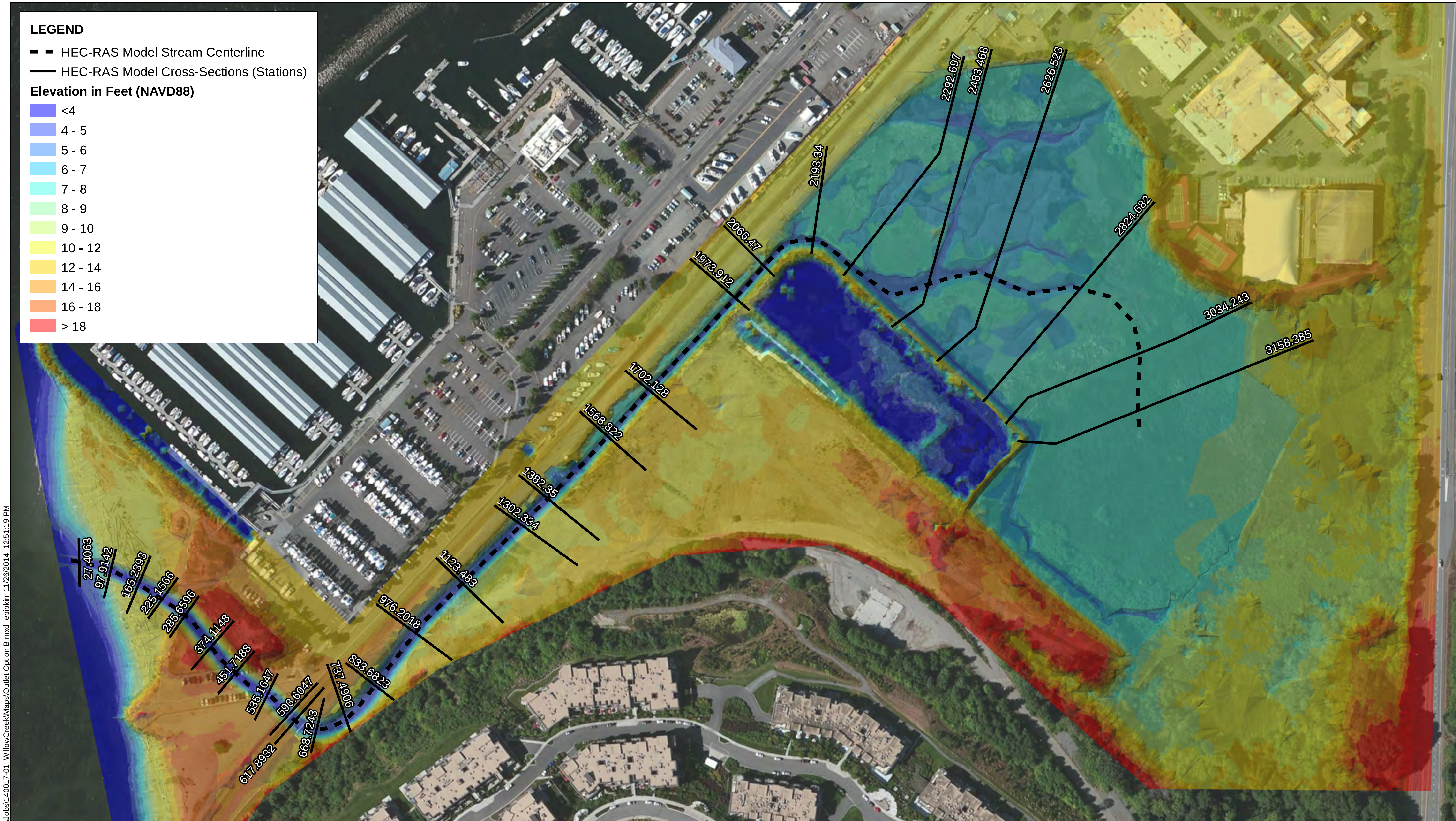
NOTES:
1. Horizontal datum is WA State Plane North Zone, NAD83, feet.
2. Vertical datum is NAVD88, feet.
3. Aerial photo provided by ESRI.

Figure 4
Proposed Conditions (Outlet Option A)
Topography and HEC-RAS Model Cross-Section Locations
Tidal Marsh Hydrodynamics Report
Willow Creek Daylight Early Feasibility Study



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NOTES:
1. Horizontal datum is WA State Plane North Zone, NAD83, feet.
2. Vertical datum is NAVD88, feet.
3. Aerial photo provided by ESRI.

0 100 200
Feet

Figure 5
Proposed Conditions (Outlet Option B)
Topography and HEC-RAS Model Cross-Section Locations
Tidal Marsh Hydrodynamics Report
Willow Creek Daylight Early Feasibility Study

APPENDIX A
EXTREME WIND AND WAVE SUMMARY

Appendix A: Wind-Wave Hindcast Data and Results Summary Table (See Section 3.3)

(Wind Data Source: Point No Point Lighthouse, NOAA #742065, 1975-1995)

				2-year			10-year			20-year		
Start Degrees	**End Degrees**	**Fetch (mi)**	**Depth (ft)**	**Windspeed (mph)**	**Wave Height (ft)**	**Wave Period (s)**	**Windspeed (mph)**	**Wave Height (ft)**	**Wave Period (s)**	**Windspeed (mph)**	**Wave Height (ft)**	**Wave Period (s)**
0	45	n/a	n/a	7	n/a	n/a	12	n/a	n/a	12	n/a	n/a
46	90	n/a	n/a	14	n/a	n/a	24	n/a	n/a	28	n/a	n/a
91	135	n/a	n/a	13	n/a	n/a	25	n/a	n/a	28	n/a	n/a
136	180	n/a	n/a	30	n/a	n/a	37	n/a	n/a	38	n/a	n/a
181	225	12	100	7	0.5	1.5	12	1.1	2.1	14	1.4	2.4
226 [a]	270 [a]	4.3	90	19 [a]	1.1 [a]	2.0 [a]	39 [a]	2.7 [a]	3.1 [a]	49 [a]	3.6 [a]	3.5 [a]
271	315	5.8	90	11	0.6	1.5	29	2.0	2.7	37	2.7	3.1
316	360	12	80	29	3.4	3.6	37	4.7	4.1	39	5.0	4.3
				50-year			**100-year**			**Maximum Observed**		
Start Degrees	**End Degrees**	**Fetch (mi)**	**Depth (ft)**	**Windspeed (mph)**	**Wave Height (ft)**	**Wave Period (s)**	**Windspeed (mph)**	**Wave Height (ft)**	**Wave Period (s)**	**Windspeed (mph)**	**Wave Height (ft)**	**Wave Period (s)**
0	45	n/a	n/a	13	n/a	n/a	13	n/a	n/a	13	n/a	n/a
46	90	n/a	n/a	33	n/a	n/a	36	n/a	n/a	28	n/a	n/a
91	135	n/a	n/a	31	n/a	n/a	32	n/a	n/a	30	n/a	n/a
136	180	n/a	n/a	41	n/a	n/a	42	n/a	n/a	39	n/a	n/a
181	225	12	100	15	1.5	2.5	16	1.6	2.5	15	1.5	2.5
226 [a]	270 [a]	4.3	90	64 [a]	5.1 [a]	4.1 [a]	77 [a]	6.6 [a]	4.6 [a]	60 [a]	4.8 [a]	4.0 [a]
271	315	5.8	90	46	3.6	3.5	53	4.3	3.9	37	2.7	3.2
316	360	12	80	42	5.5	4.4	44	5.8	4.6	40	5.3	4.4

Notes:

n/a Wind direction not applicable for wave generation at the project site

a. Highest observed wind speed of 60 mph may be an outlier. Wave parameters estimated from winds in this directional bin may be over predictions.

APPENDIX B
PROPOSED CHANNEL ALIGNMENTS PROVIDED BY SHANNON AND WILSON

Willow Creek Daylighting
Edmonds Marsh
Edmonds, Washington

FIELD EXPLORATION AND PROPOSED CHANNEL OPTION A

March 2015 | 21-1-12393-404

SHANNON & WILSON, INC.
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

FIG. A

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Willow Creek Daylighting
Edmonds Marsh
Edmonds, Washington

FIELD EXPLORATION AND PROPOSED CHANNEL OPTION B

March 2015 21-1-12393-404

SHANNON & WILSON, INC.
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

FIG. B

Filename: T:\Project\21-1\12393_Edmonds_Marsh\AV_mxd\ExplorAndProposedChannelSurfaces.mxd Date: 3/6/2015 brl

APPENDIX C
TIME SERIES PLOTS OF PREDICTED VELOCITY AND WATER DEPTHS AT SELECT MODEL CROSS-SECTIONS

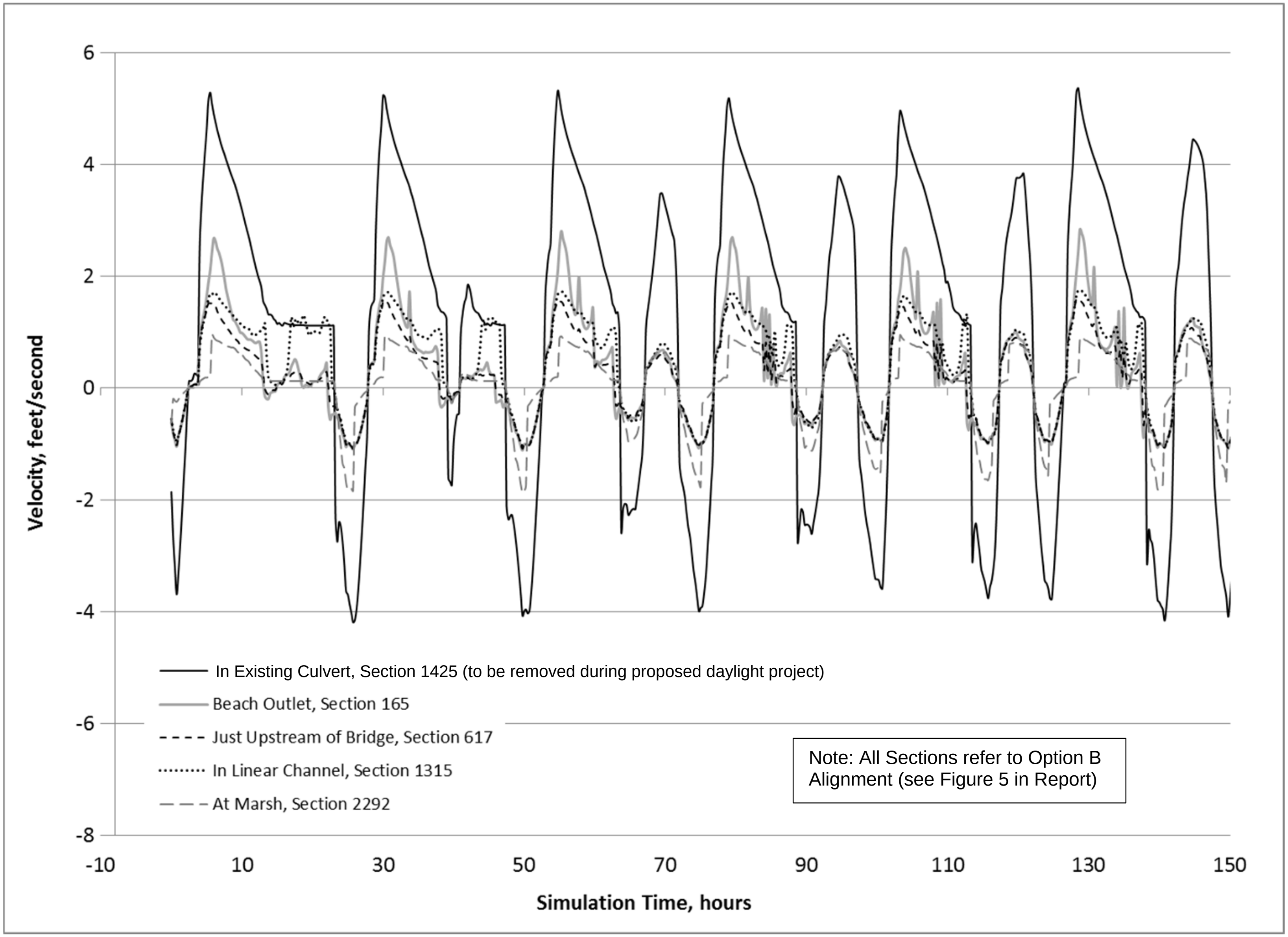

Figure C1
Low-Flow Simulation, Velocities at Select Model Sections
Beach Outlet and Hydrodynamic Evaluation Report
Willow Creek Daylight Final Feasibility Study

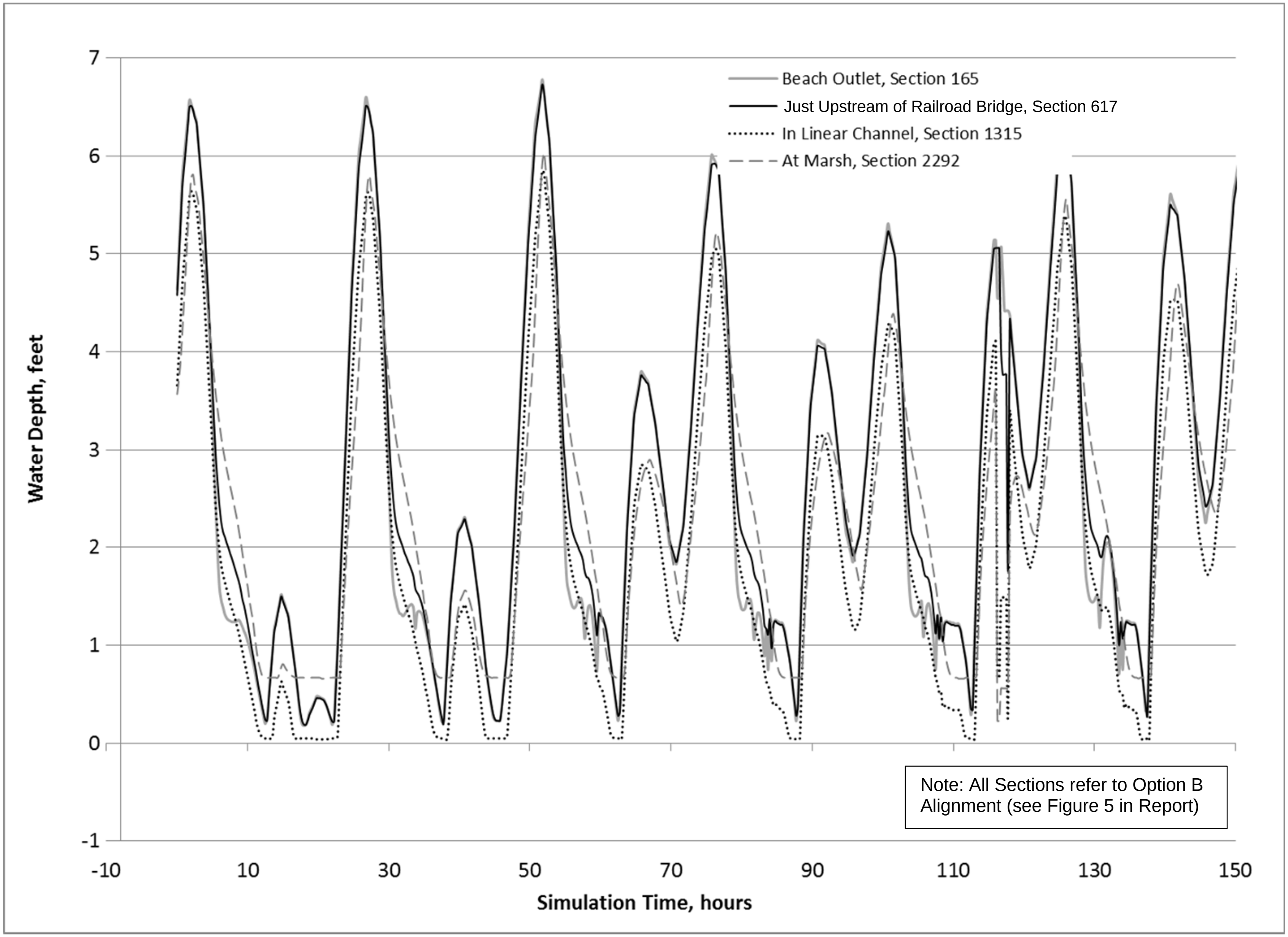

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Figure C2
Low-Flow Simulation, Water Depths at Select Model Sections
Beach Outlet and Hydrodynamic Evaluation Report
Willow Creek Daylight Final Feasibility Study

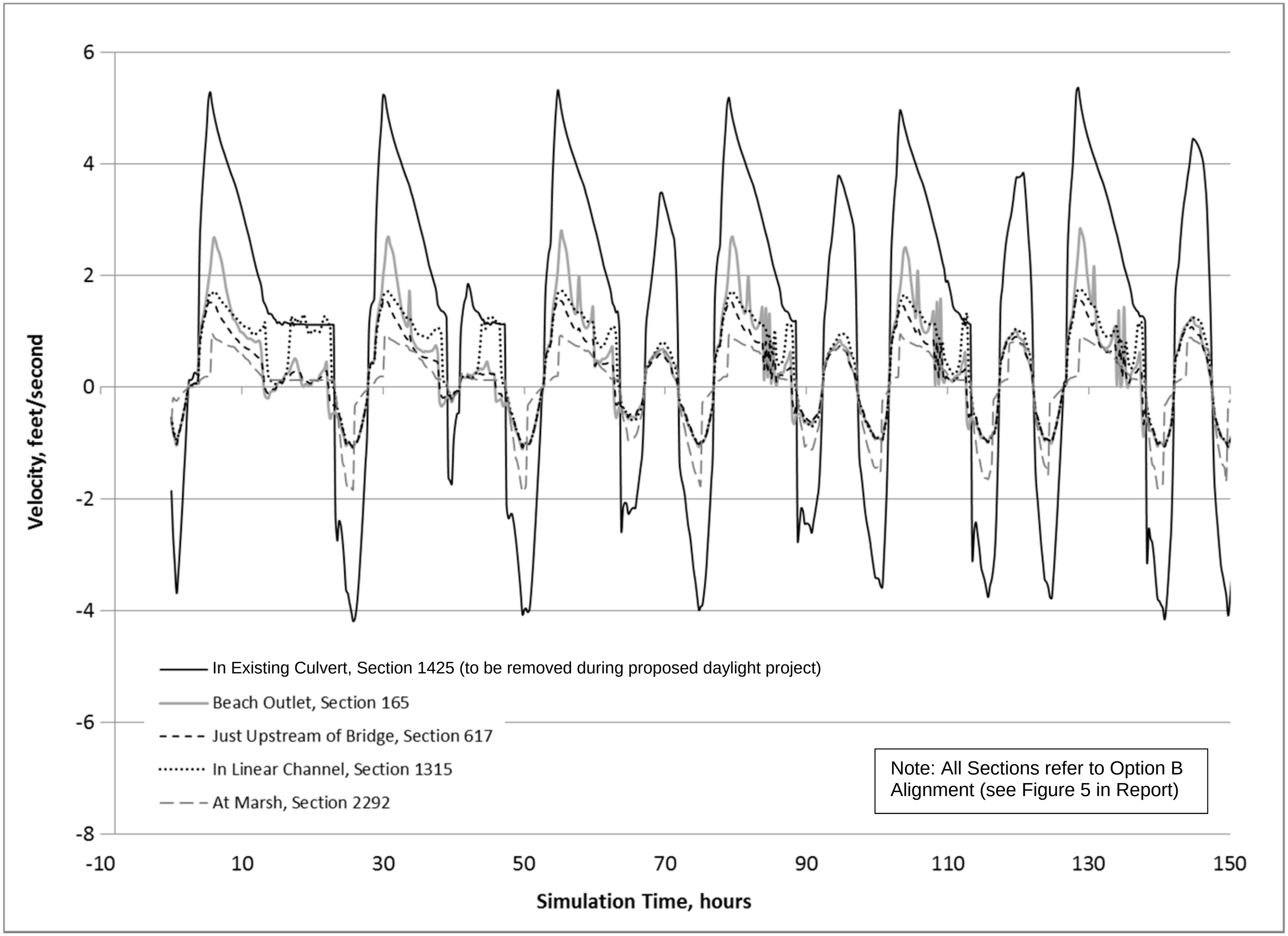

Figure C3

Low-Flow Tide Gate (Invert +5.5 feet NAVD88) Simulation, Velocities at Select Model Sections

Beach Outlet and Hydrodynamic Evaluation Report

Willow Creek Daylight Final Feasibility Study

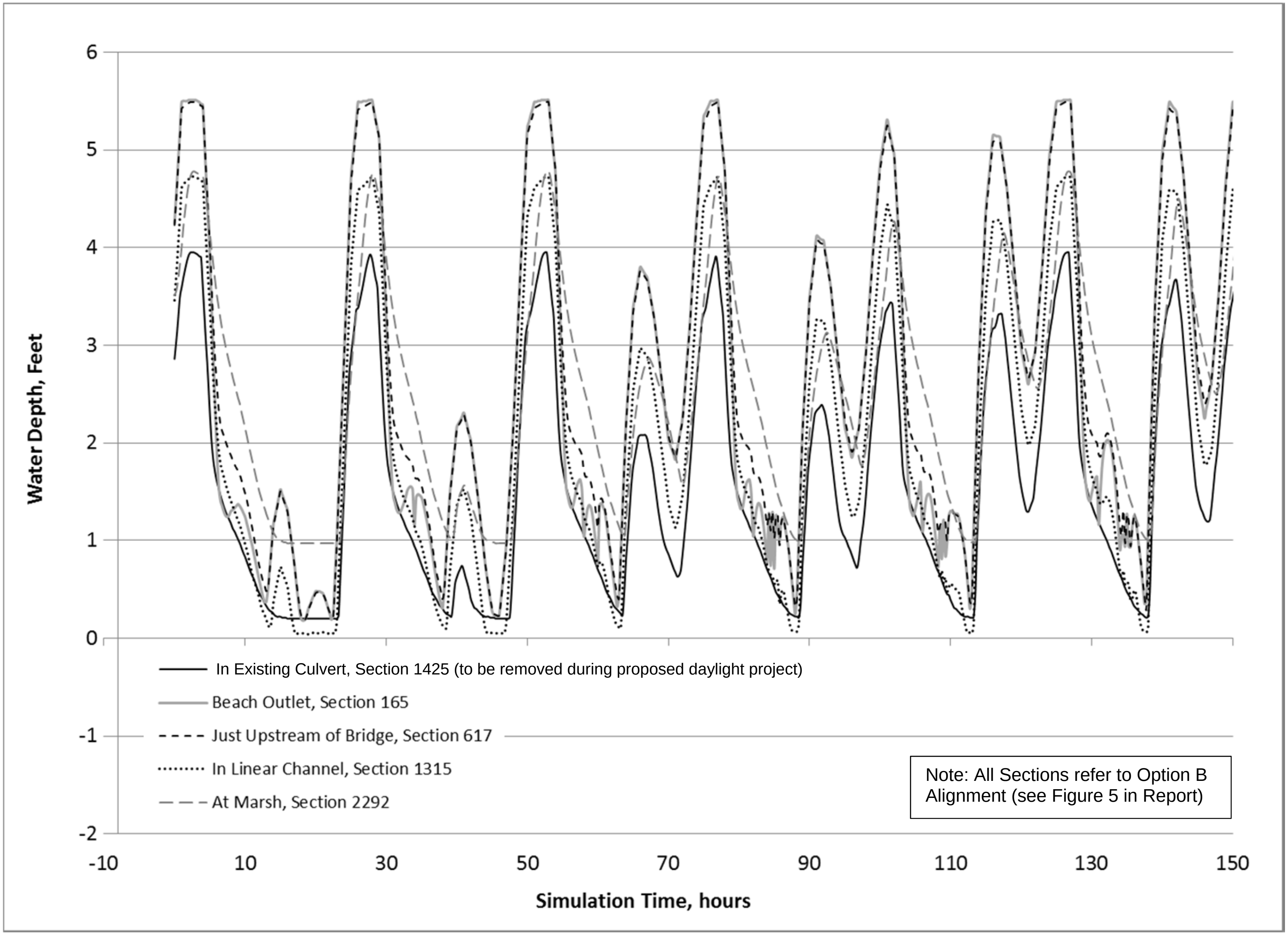

Figure C4
Low-Flow Tide Gate (Invert +5.5 feet NAVD88) Simulation, Water Depths at Select Model Sections
Beach Outlet and Hydrodynamic Evaluation Report
Willow Creek Daylight Final Feasibility Study

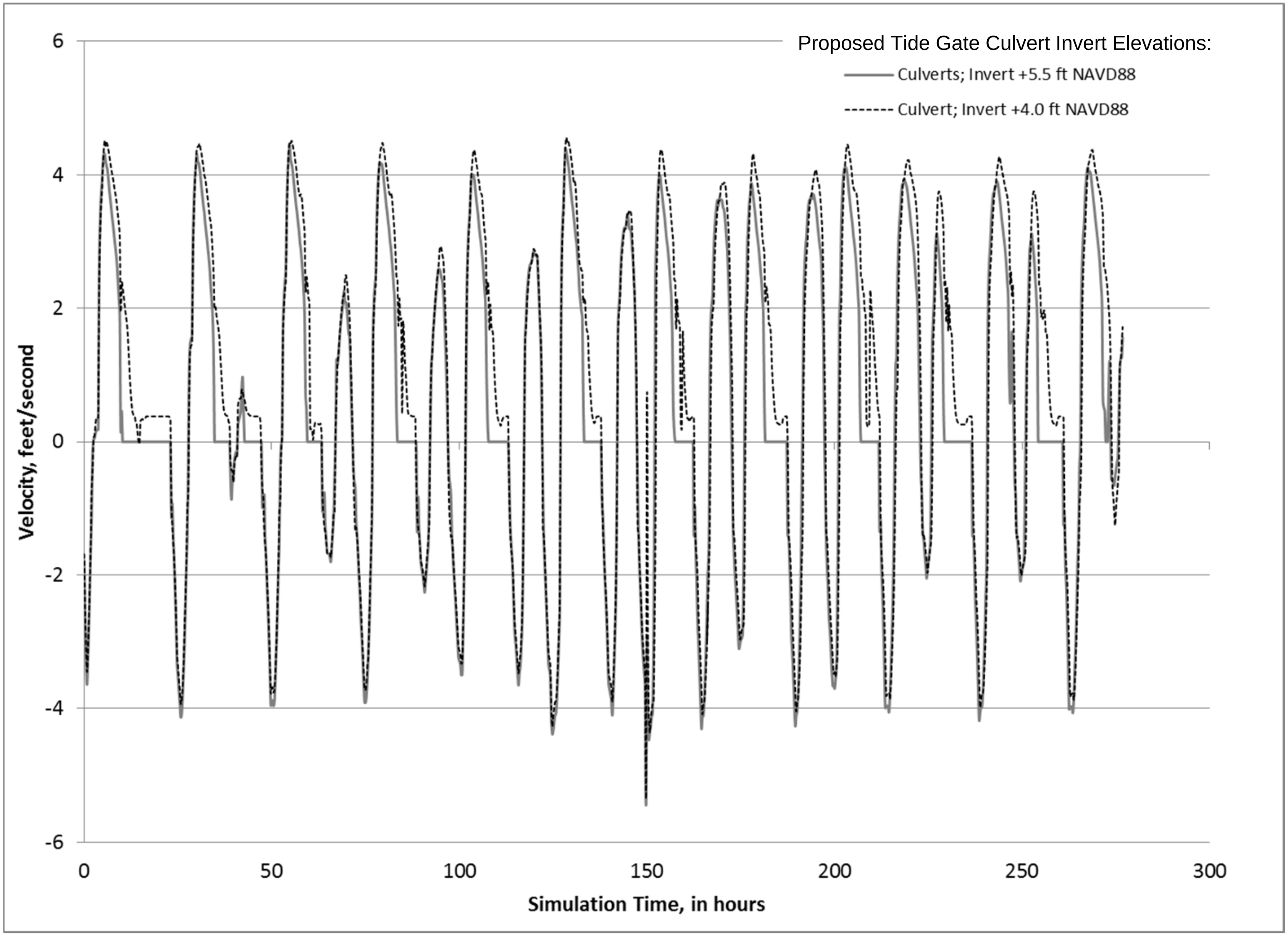

Figure C5
Low-Flow Tide Gate (Variable Inverts +5.5 and 4.0 feet NAVD88) Simulation, Velocities
Beach Outlet and Hydrodynamic Evaluation Report
Willow Creek Daylight Final Feasibility Study

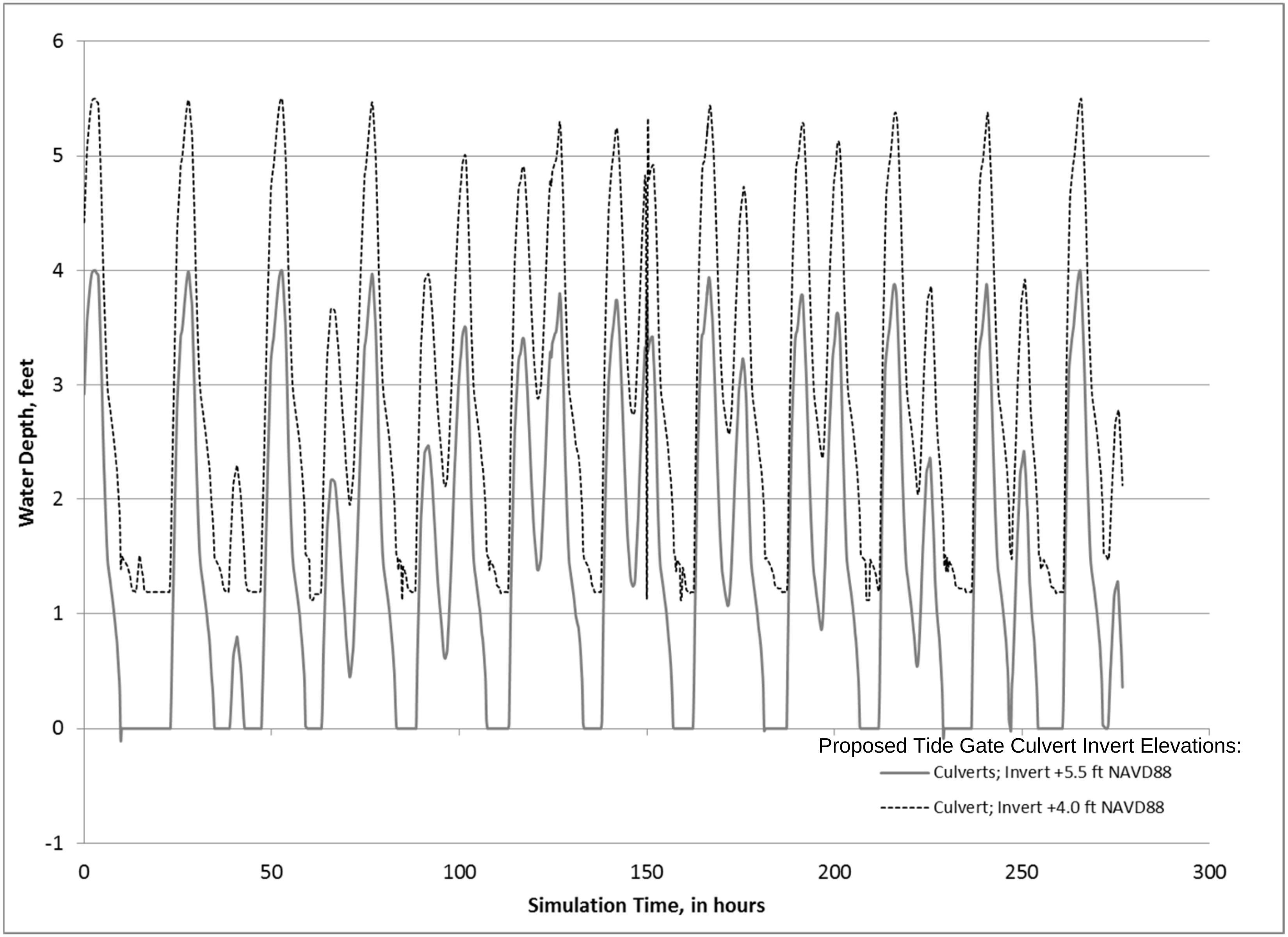

Figure C6

Low-Flow Tide Gate (Variable Inverts +5.5 and 4.0 feet NAVD88) Simulation, Water Depths

Beach Outlet and Hydrodynamic Evaluation Report

Willow Creek Daylight Final Feasibility Study

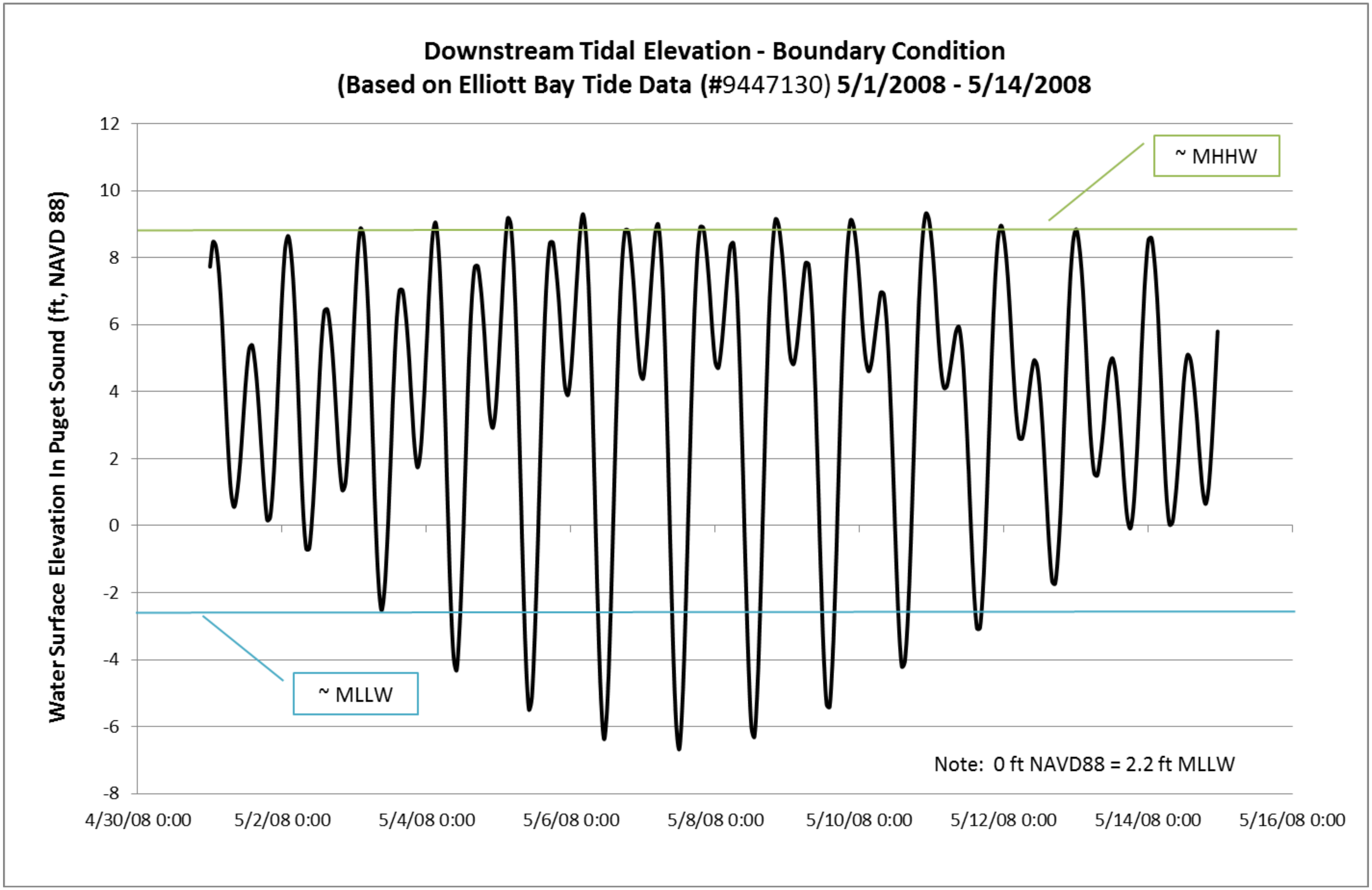

Figure C7

Downstream Tidal Boundary Condition for all Model Simulations
Beach Outlet and Hydrodynamic Evaluation Report
Willow Creek Daylight Final Feasibility Study

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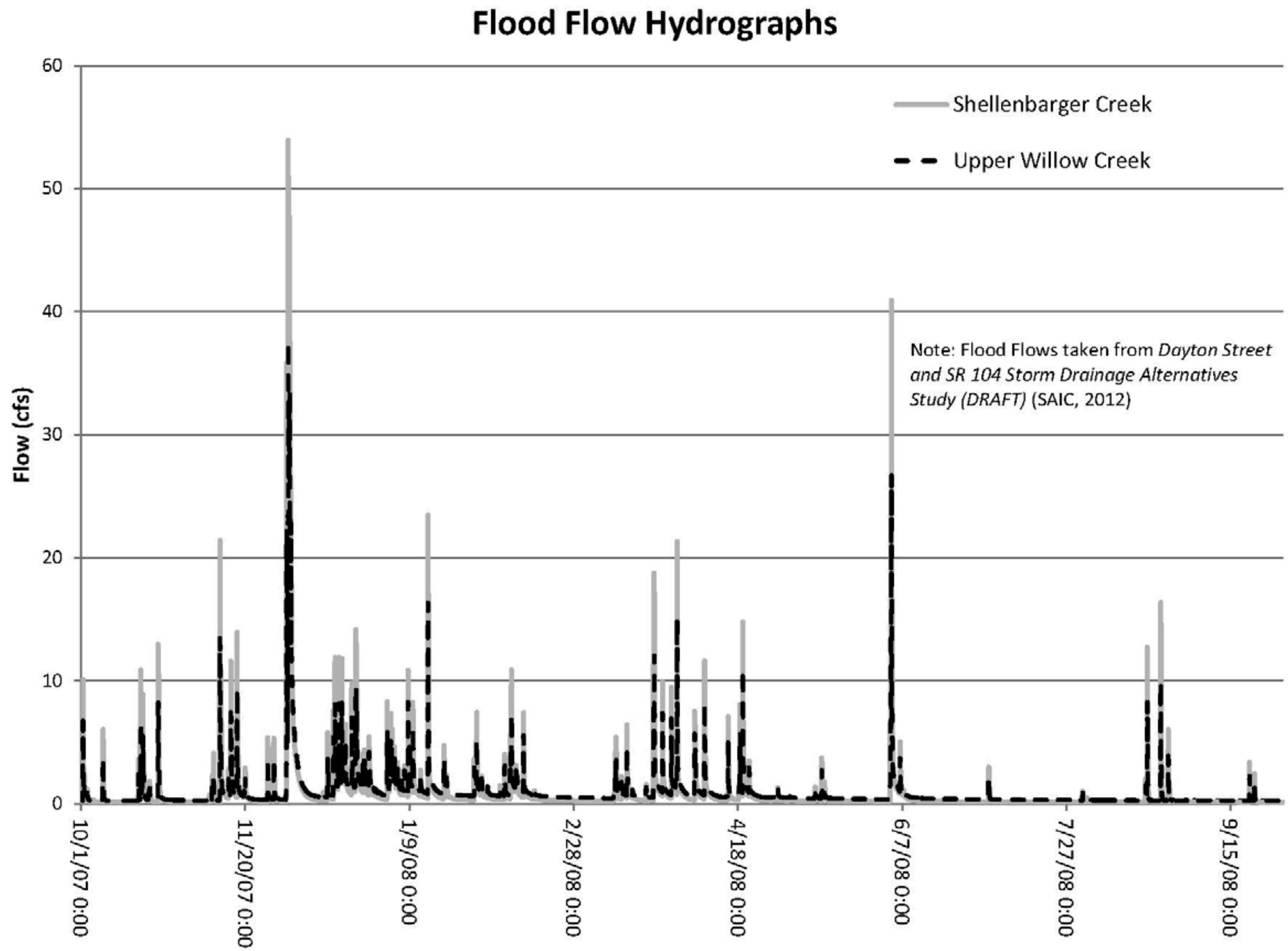

Figure C8
100-Year Flow Hydrographs, Upstream Boundary Condition
Beach Outlet and Hydrodynamic Evaluation Report
Willow Creek Daylight Final Feasibility Study

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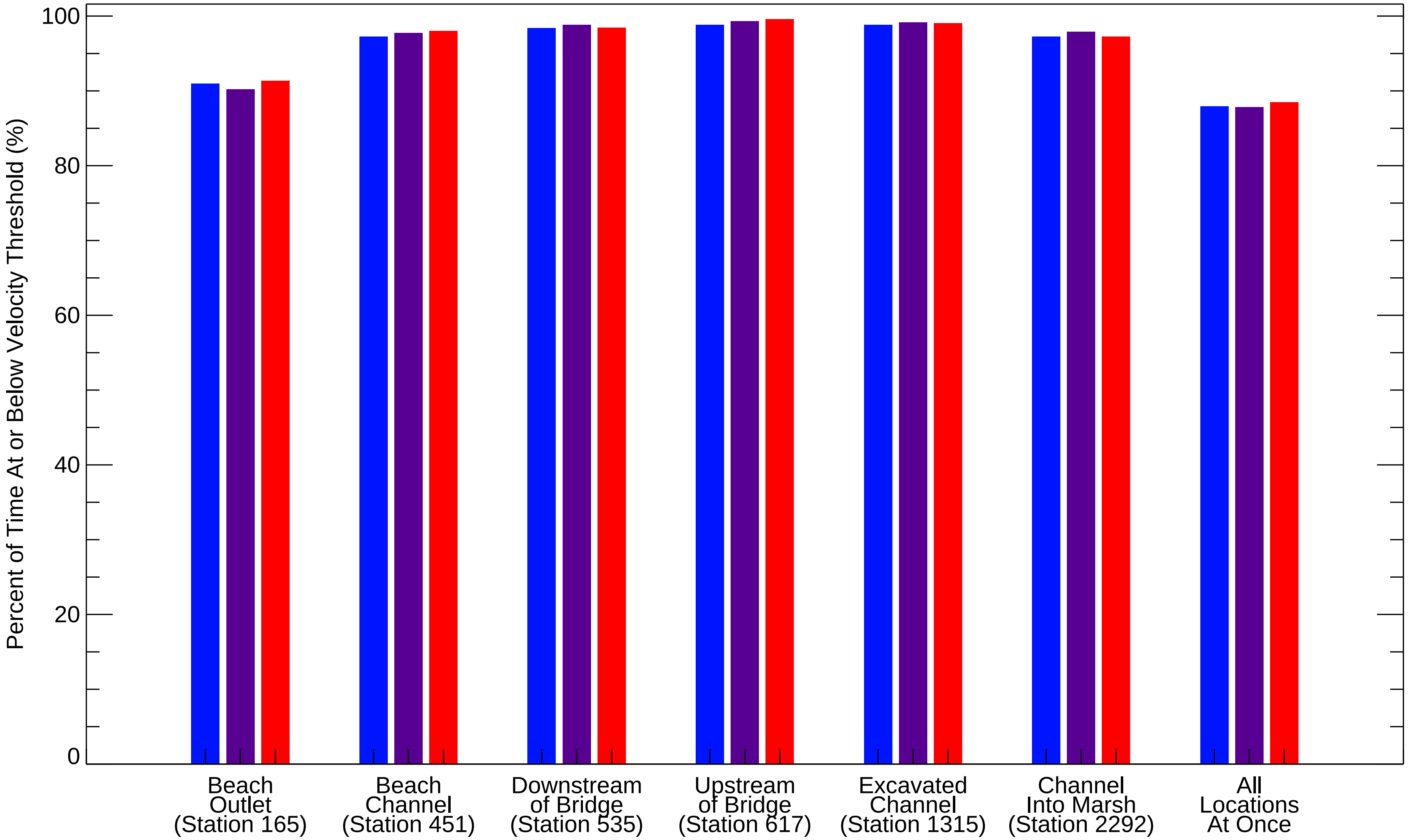
APPENDIX D
LOW FLOW MODEL SENSITIVITY ANALYSES: IN-FLOW RATE AND MEAN SEA LEVEL



Willow Creek - Beach Outlet Option B

Percent of Time At or Below Velocity Threshold

Velocity threshold: 2 ft/s.

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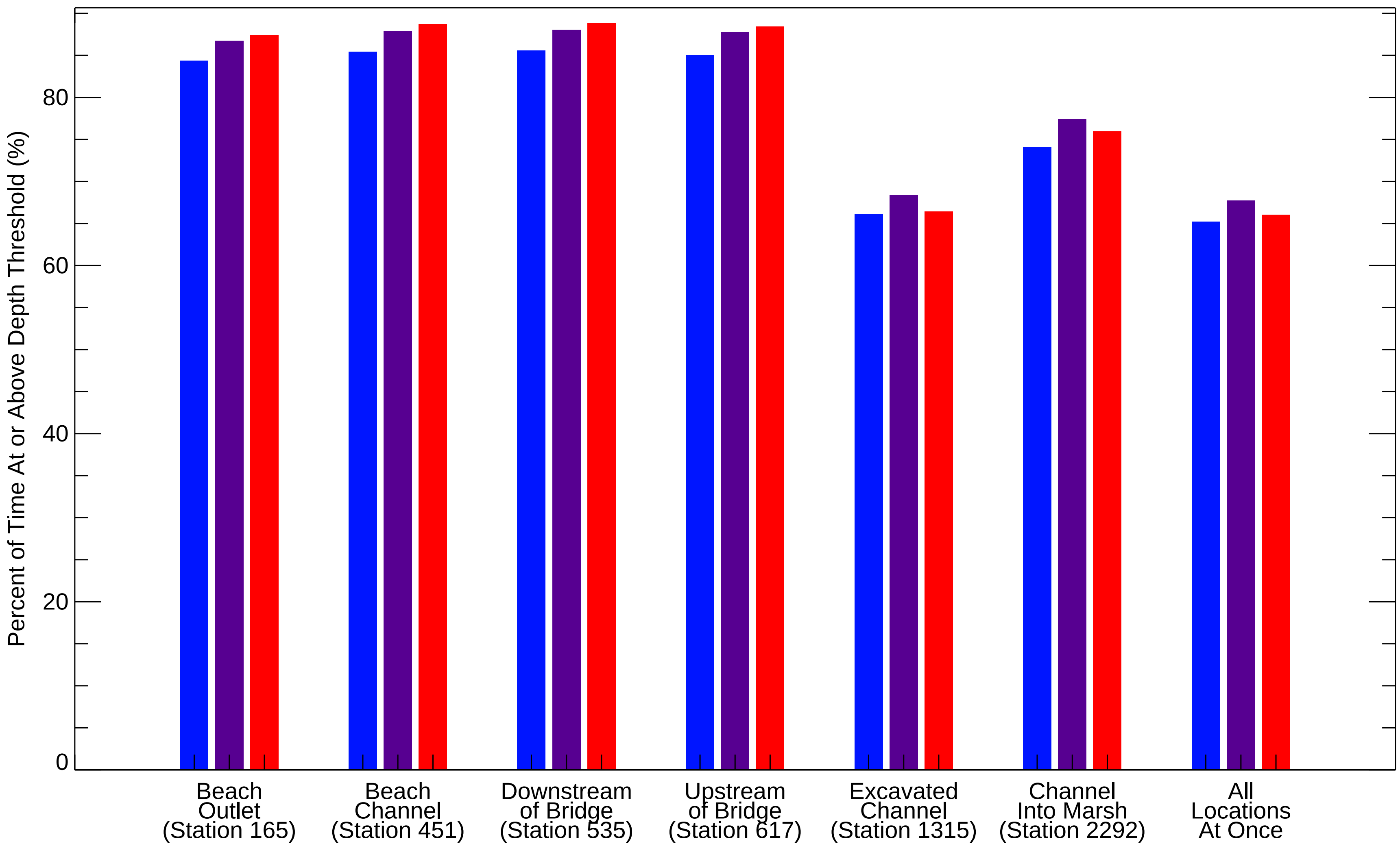
- 0.64 cfs Inflow Rate
- 0.80 cfs Inflow Rate
- 0.96 cfs Inflow Rate



0.64 cfs Inflow Rate
0.80 cfs Inflow Rate
0.96 cfs Inflow Rate

Willow Creek - Beach Outlet Option B

Percent of Time At or Above Depth Threshold

Depth threshold: 0.8 ft.

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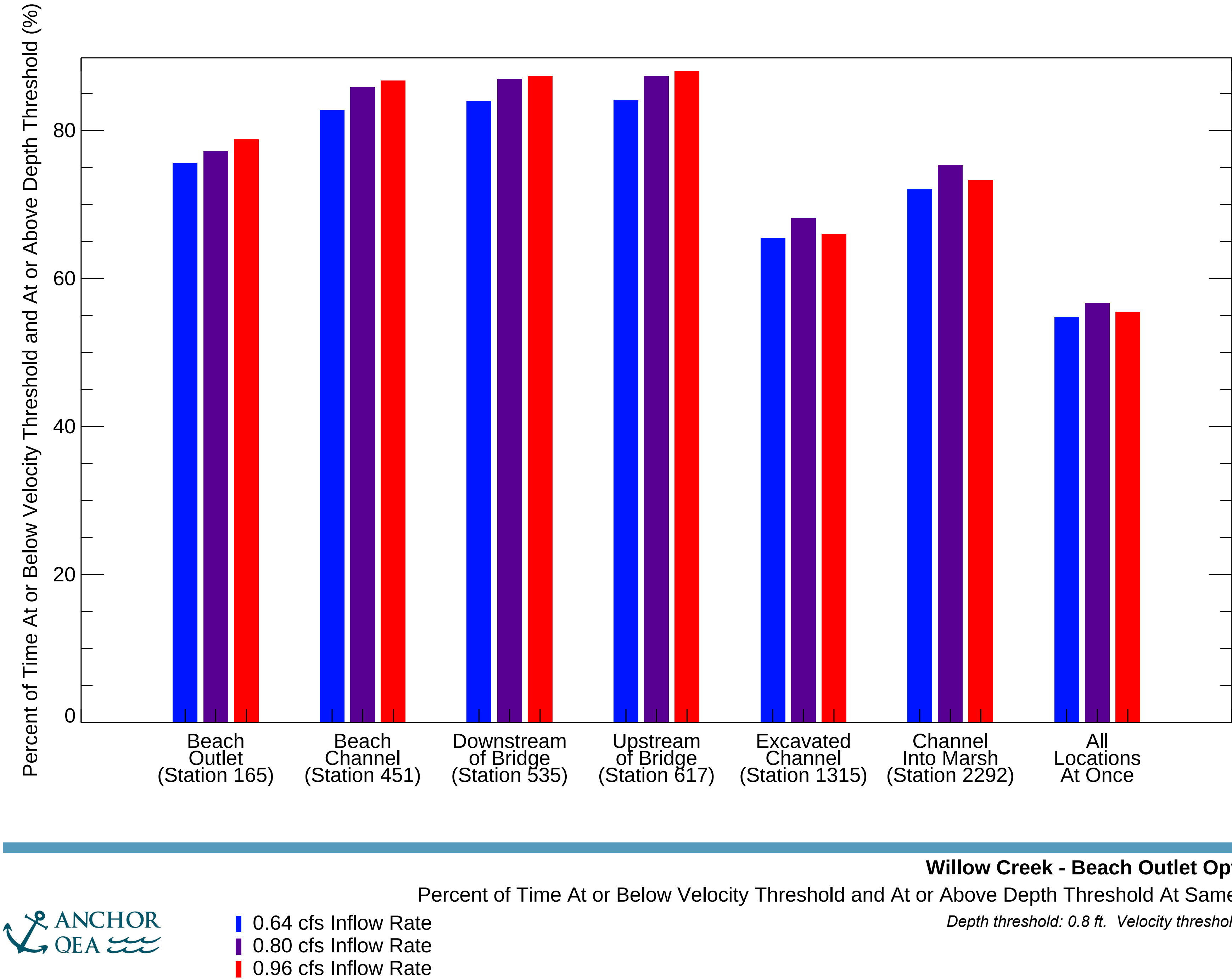

Willow Creek - Beach Outlet Option B

Percent of Time At or Below Velocity Threshold and At or Above Depth Threshold At Same Time

Depth threshold: 0.8 ft. Velocity threshold: 2 ft/s.

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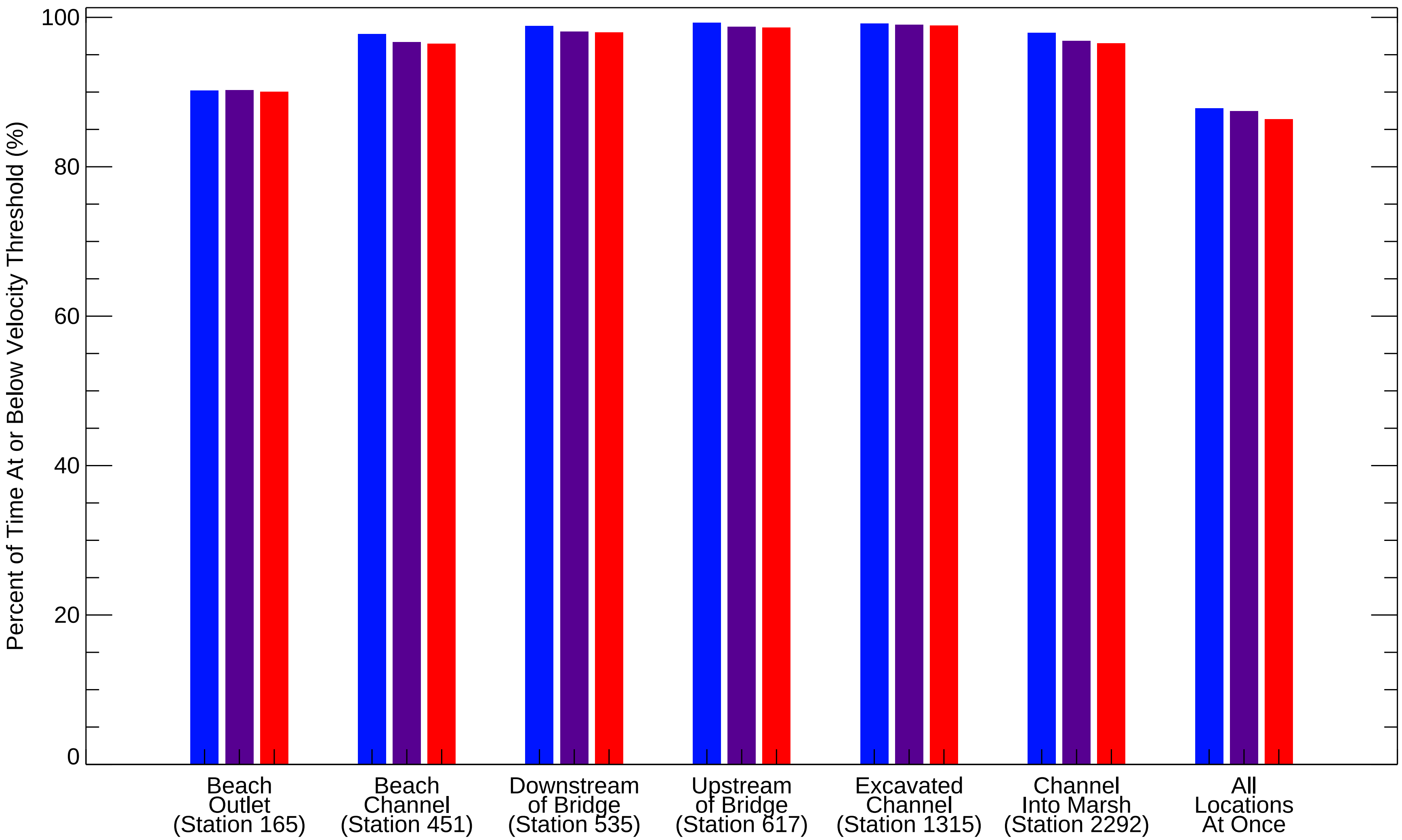

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- Current Sea Level
- 2030 Sea Level
- 2050 Sea Level

Willow Creek - Beach Outlet Option B

Percent of Time At or Below Velocity Threshold

Velocity threshold: 2 ft/s.

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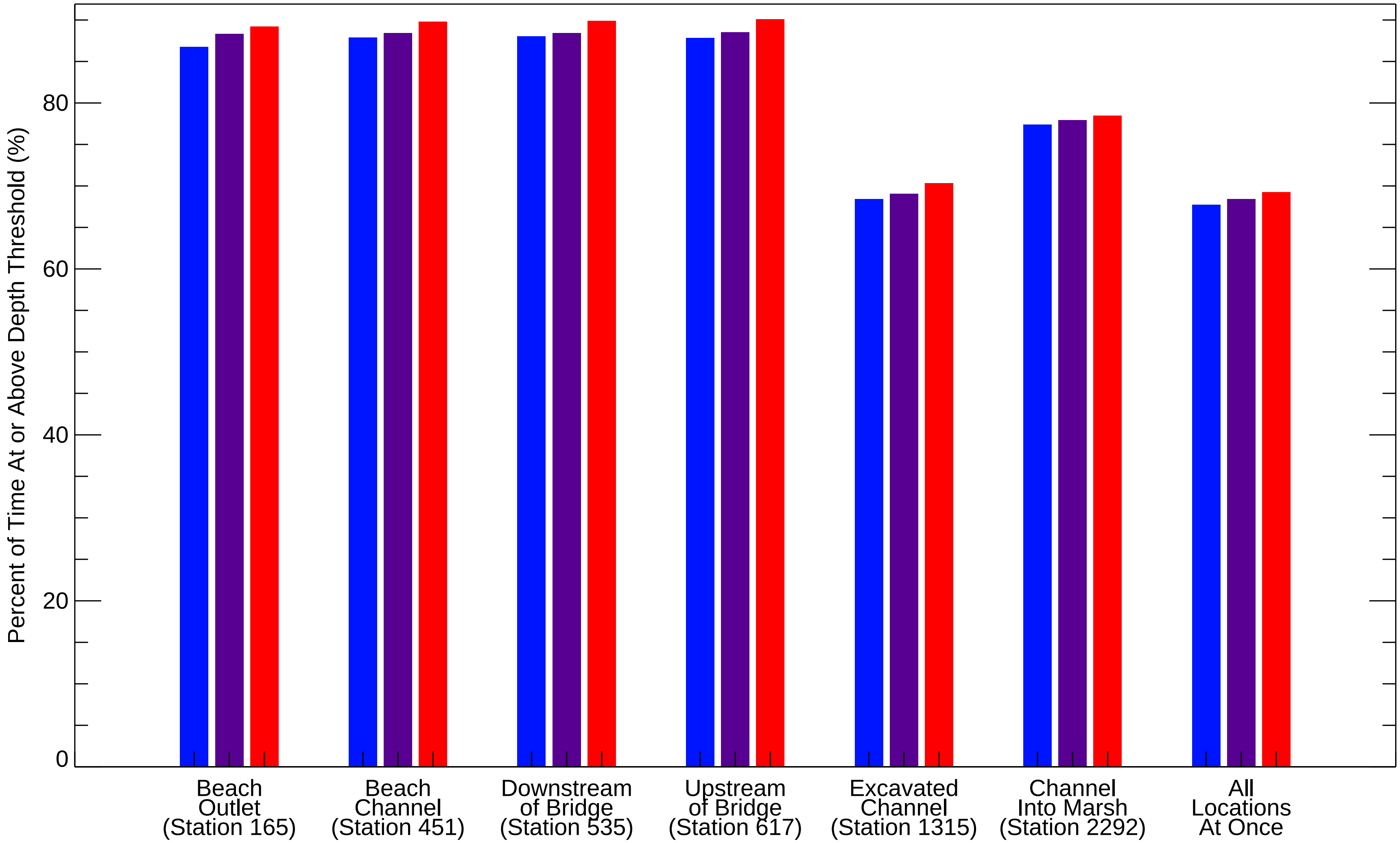

Willow Creek - Beach Outlet Option B

Percent of Time At or Above Depth Threshold

Depth threshold: 0.8 ft.

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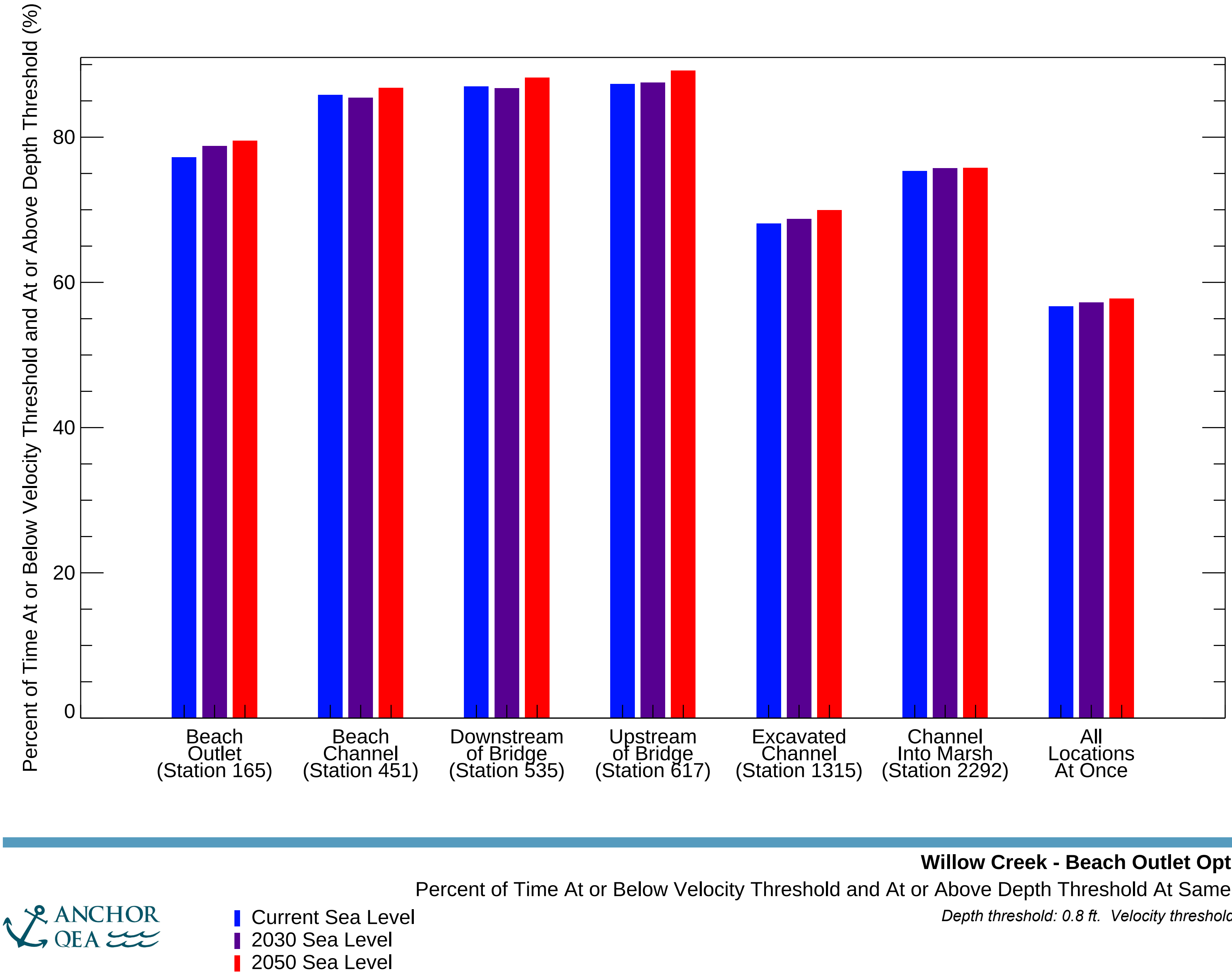

Willow Creek - Beach Outlet Option B

Percent of Time At or Below Velocity Threshold and At or Above Depth Threshold At Same Time

Depth threshold: 0.8 ft. Velocity threshold: 2 ft/s.

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APPENDIX E

SUMMARY OF TIDE GATE EVALUATION RESULTS

APPENDIX E: SUMMARY OF TIDE GATE EVALUATION RESULTS

Water Surface Elevation (ft NAVD88)	Storage Volume in Marsh (cf)	Storage Volume Between WSE Intervals (cf)	Volume (cf) Above Gate Closure Elev. to Fill Marsh to Elev. Intervals				Inflow rate (cfs) to Fill Marsh Above Gate Closure Elev. to Elev. Intervals, Based on Duration of Tidal Level Above Gate Closure Elev.			
			Close at 8.0'	Close at 8.5'	Close at 9.0'	Close at 9.5'	Close at 8.0'	Close at 8.5'	Close at 9.0'	Close at 9.5'
8.00	543,136									
8.25	713,909	170,773	170,773				9			
8.50	892,143	178,234	349,007				19			
8.75	1,075,016	182,873	531,880	182,873			30	10		
9.00	1,261,684	186,668	718,548	369,541			40	21		
9.25	1,451,999	190,315	908,863	559,856	190,315		50	31	13	
9.50	1,646,022	194,023	1,102,886	753,879	384,338		61	42	27	
9.75	1,844,047	198,025	1,300,911	951,904	582,363	198,025	72	53	40	18
10.00	2,046,461	202,414	1,503,326	1,154,319	784,778	400,440	84	64	54	37
10.25	2,284,160	237,699	1,741,024	1,392,017	1,022,476	638,138	97	77	71	59
10.50	2,528,392	244,232	1,985,256	1,636,249	1,266,708	882,370	110	91	88	82
10.75	2,779,180	250,789	2,236,045	1,887,038	1,517,497	1,133,159	124	105	105	105
11.00	3,036,425	257,244	2,493,289	2,144,282	1,774,741	1,390,403	139	119	123	129

Duration (hrs): tidal level above gate closure elev.	5	5	4	3	5	5	4	3
Duration (sec): tidal level above gate closure elev.	18,000	18,000	14,400	10,800	18,000	18,000	14,400	10,800

Estimated 100-year Storm Flows	
Peak Flow, cfs	Average Flow, cfs
91	72