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Bear River Restoration Project

Hydrodynamic Analysis of Existing Condition and Alternatives

Pacific International Engineering PLLC

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1.0 Introduction

Pacific International Engineering, PLLC is working under contract with Ducks Unlimited, Inc. to provide surveying and engineering services in connection with the Bear River Restoration Project. The project is located within Sections 8, 17 and 18, Township 10N, Range 10WM in Pacific County, Washington. Generally, the project is located north of the Bear River at Highway 101 at the south end of Willapa Bay (Figure 1-1).

The work is part of an effort to restore, enhance and rehabilitate fish, waterfowl and wildlife habitat within the project area through the restoration of approximately 200 acres of estuarine wetland. Such restoration will be achieved by reestablishing tidal channels that have been partially disconnected from the main estuary since the construction of Highway 101. This technical report describes the collection and analysis of topographic and water level data, documents the development and application of a surface water model to describe the basic hydrology of the estuarine wetland area, outlines the development of conceptual alternatives for restoration of tidal processes to historical tidal channels, and presents the application of the hydrodynamic model for the preliminary analysis of the alternatives relative to the existing condition. The preferred alternative was selected based on the preliminary analysis of alternatives, and the results of the detailed hydraulic analysis of the preferred alternative are presented in this technical report.

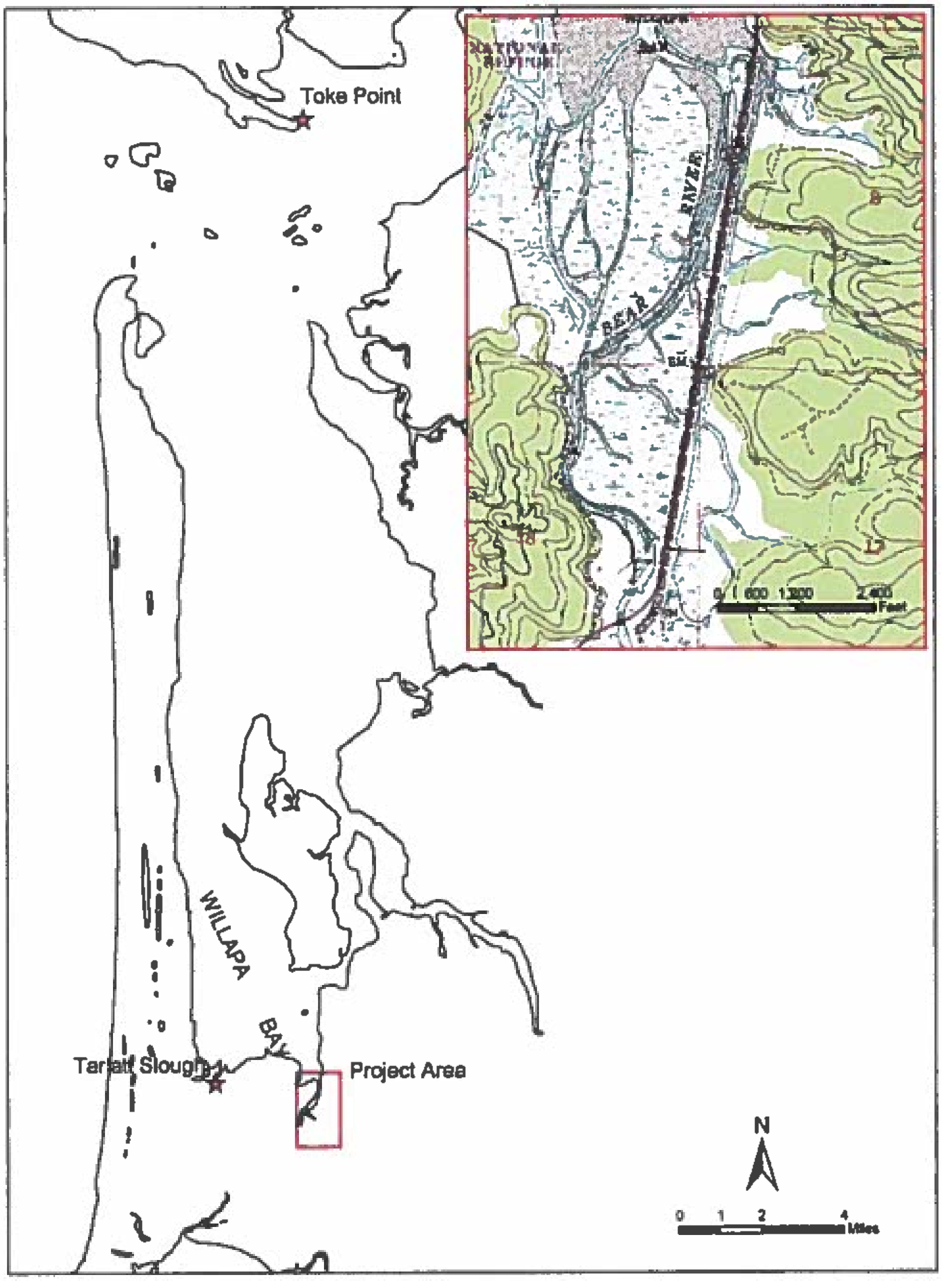


Figure 1-1. Project location map

2.0 Historic Data Review

Existing data relevant to the project were collected from a variety of sources, including:

- topographic survey conducted by the US Army Corps of Engineers in 1999 - included the north and south marshes at the southern end of the project area (Figure 2-1),
- aerial photographs of the project area,
- Geographic Information System layers from Pacific County and US Fish and Wildlife Service that include water courses, boundaries and easements, digital ortho-quad imagery (1999), road and right-of-way delineations, LIDAR data, true color and color IR photos,
- USGS digital elevation data
- tide and water level records obtained from U.S. Fish and Wildlife Service (USFWS) as described in *Tidal Datums for the Bear River Estuary and Marshes*, an unpublished paper by Paul Bakke, hydrologist/ geomorphlogist with USFWS, and Allen Lebovitz, ecologist with Coastal Watersheds Consulting
- Tide and water level records, tidal constituents obtained from the Center for Operational Oceanographic Products and Services, National Oceanic and Atmospheric Administration (COOPS, NOAA) for Toke Point (9440910) and Tarlatt Slough (constituent predictions based on Toke Point).

Data were reviewed to determine the completeness and the sufficiency to provide boundary conditions and validation for numerical modeling. The review determined that existing topographic and tide and water level data for the project site was insufficient to provide boundary conditions and validation data for numerical modeling. Therefore a field measurement program was conducted to acquire additional topographic and bathymetry information, and to acquire additional water level information for the project site.

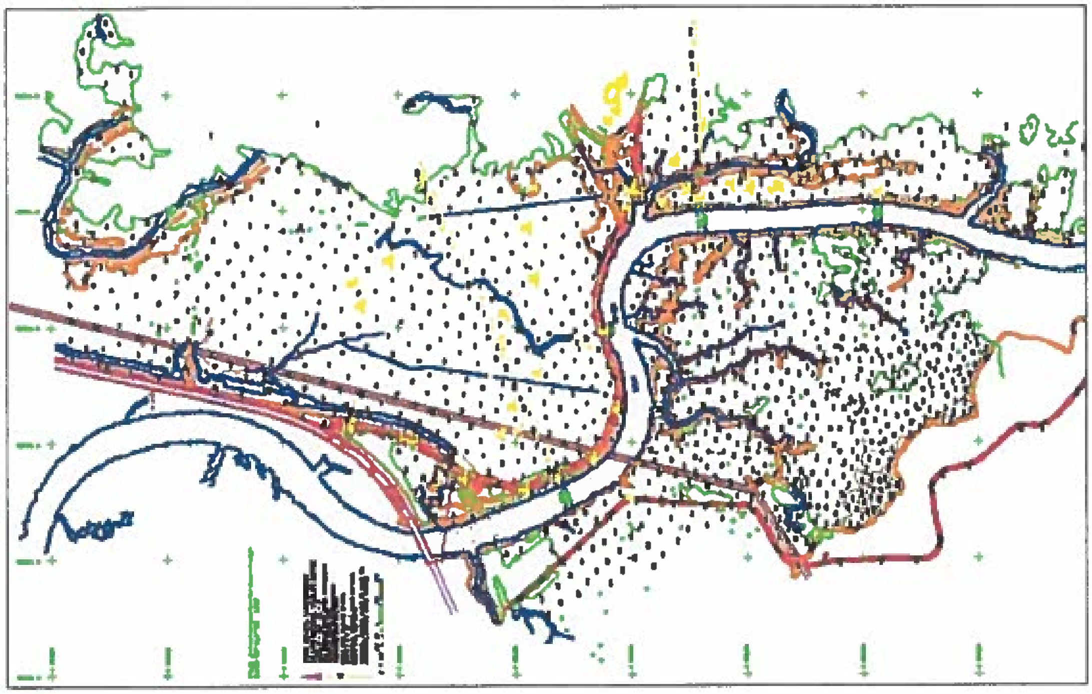


Figure 2-1. **Topographic survey of the north and south marshes by USACE 1999**

3.0 Field Measurements

A field data collection program was conducted to acquire the hydrographic and topographic survey information to properly delineate and prepare a surface water model of the Bear River and the tributary tidal channels to the Bear River in the project area. The field data collection also involved deployment of 3 water level recording gauges for a period of 39 days.

3.1 Topographic Survey

To adequately address modeling needs, a total of 37 cross-sections of the tributary tidal channels, estuarine marshes and the road profile in the project area were surveyed by hydrographic and topographic survey methods (Figure 3-1). The hydrographic survey included the key branches and tributary channels of the Bear River in the vicinity of the proposed project. The survey consisted of measuring river cross sections, from bank to bank and across the estuarine marshes to the point where a significant break in slope occurred. The hydrographic survey was used to develop the data set (computation grid) of river transects for the hydrodynamic modeling described in Sections 4.0 and 5.0 5 below. The tidal and stream channel transects include bathymetric data and topographic data to assess overbank flow conditions and elevations.

The surveying was performed by Real Time Kinematic Global Positioning System (RTK GPS) techniques and referenced to Washington State Department of Transportation (WSDOT) monuments in the region. The RTK GPS system involved deployment of a base station and collection of individual survey point coordinates (Easting, Northing, Elevation) by means of a roving unit. The base station was deployed on a WSDOT monument located on the dike to the north of the Bear River in the south end of the study area (Figure 3-2). Additional WSDOT monuments were used to provide a calibration of the system. For upland portions of the survey a rover unit was carried by backpack with the GPS antenna mounted at a fixed height on a survey range pole (Figure 3-3). For intertidal and sub tidal elevations the rover unit was mounted in a kayak, and elevations were determined through a combination of depth information obtained from an internally recording depth sounder and GPS elevations from the RTK GPS system (Figure 3-4).

Horizontal datum for the survey is the North American Datum of 1983 (NAD83) with horizontal coordinates reported in Washington State Plane South with units of feet. The vertical datum is the North American Vertical Datum of 1988 (NAVD88) in units of feet.

Figure 3-5 shows examples of representative cross-section data obtained in the survey. Transect 21 is a cross section that includes portions of the South tidal channel and the adjacent estuarine marsh elevation. Transect

37 is a cross-section over Highway 101 in the vicinity of the south channel. The individual cross-sections are included in Appendix A.

Final engineering design may require additional survey data to properly tie the project into requirements of WSDOT depending on the selected alternative. More detailed topographic surveying may be required during final design to assess any potential areas of flooding concern identified in the modeling of alternatives.

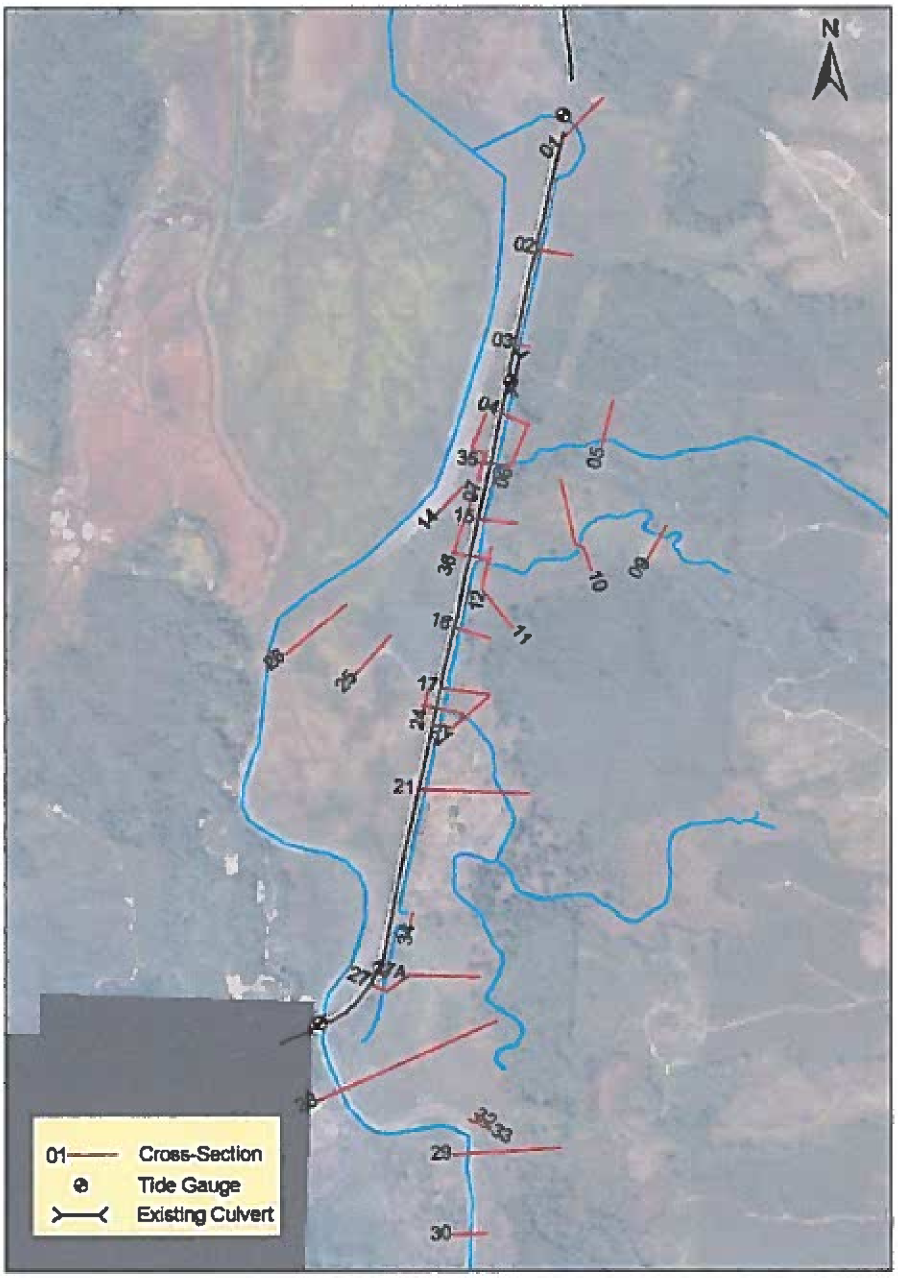
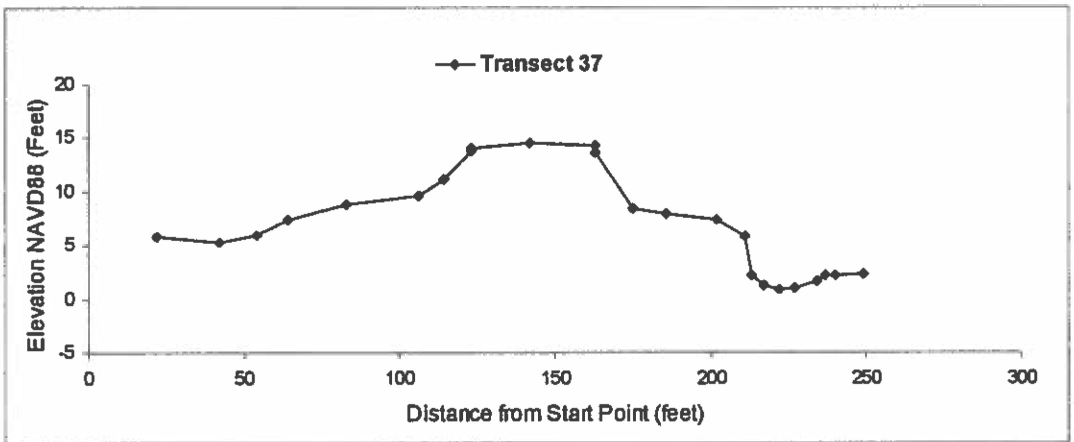


Figure 3-1. Location of survey transects and water level recorders deployed at the project site

Figure 3-2. RTK - GPS base station deployed on the earth dike near Bear River

Figure 3-3. RTK-GPS survey rover station in use at Bear River project site

Figure 3-4. Kayak with GPS Rover unit and internally recording depth sounder



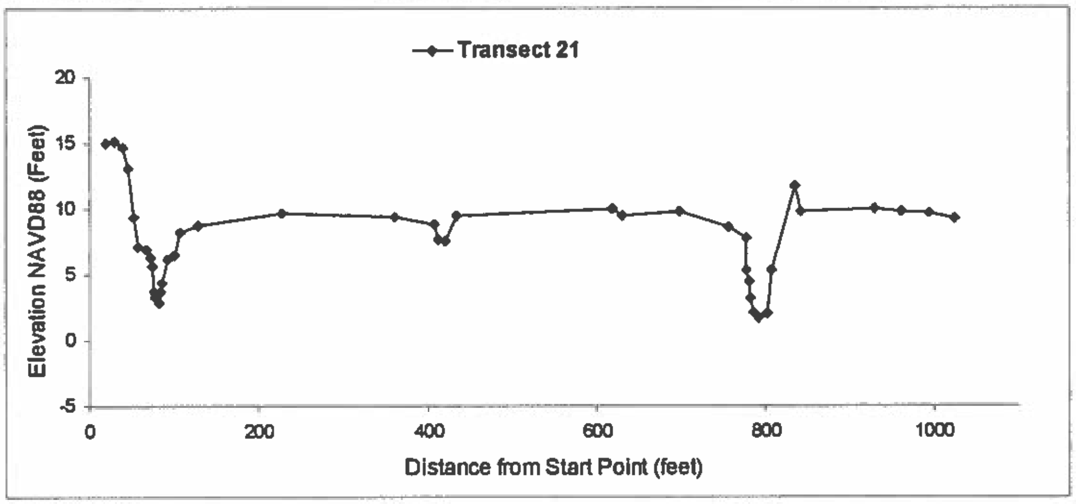


Figure 3-5. Representative topographic cross-sections obtained during the survey

3.2 Water Level Measurements

In addition to survey data, water level information was collected to provide calibration and validation of the surface water model described in Section 4.0. Site specific water surface elevation data were required to provide boundary conditions for the model and verification data in the channel.

Three internally recording pressure gauges were deployed at the following locations:

- Hwy 101 Bridge over Greenhead Slough
- Hwy 101 Bridge over Bear River
- Greenhead Slough upstream of logging road

The locations of the water level gauges is shown in Figure 3-1, and the location coordinates and deployment details are summarized in Table 3-1.

The height of the gauges was surveyed to the NAVD88 datum and adjusted for barometric variation using the Toke Point measurement data. Figure 3-6 shows the water level gauge deployed at the Bear River Bridge.

Tidal fluctuation data was collected for a period of 39 days between May 22 and June 30, 2005. Water levels were recorded at 3-minute intervals.

In addition to the recording gauges, spot elevations of the water surface were obtained at a number of locations and times in the project area during the topographic survey to provide data to verify the tide gauge elevations. The comparison between the measured spot elevations and the time series of tide gauge records is shown in Figure 3-7. There is a very high level of correlation between the gauge measurements and the spot elevations of water surface elevation. The time series shown in Figure 3-7 indicates that the maximum water levels at the Bear River Bridge are 0.2 to 0.3 feet higher than at the Greenhead Slough Bridge. This reflects the amplification of the tidal wave as it propagates into Bear River and illustrates the approximate tidal induced water level difference between the upstream and downstream boundaries of the project area. There is only a slight phase shift between Greenhead Slough and Bear River Bridge.

The time series also shows that the tidal wave is significantly attenuated by the presence of the logging road culvert. Maximum water levels in the borrow ditch are 1 to 2 feet lower than in the natural channels at Greenhead Slough and Bear River. Water level shifts in the borrow ditch also lag the shifts in the main channels at Greenhead Slough and Bear River bridge. The objective of the restoration project is to minimize the attenuation of the tidal hydraulics. An analysis of the site-specific water level data together with the long-term records from Toke Point is provided in section 3.3 below.

Table 3-1. Summary of water level recorders deployed in the study

Gauge	Easting, ft	Northing, ft	Deployment Date	Recovery Date
Hwy 101 Bridge over Greenhead Slough	767574.1	389421.2	May 22, 2005	June 30, 2005
Hwy 101 Bridge over Bear River	769871.8	397944.5	May 22, 2005	June 30, 2005
Greenhead Slough upstream of logging road	769348.1	395386.7	May 22, 2005	June 30, 2005

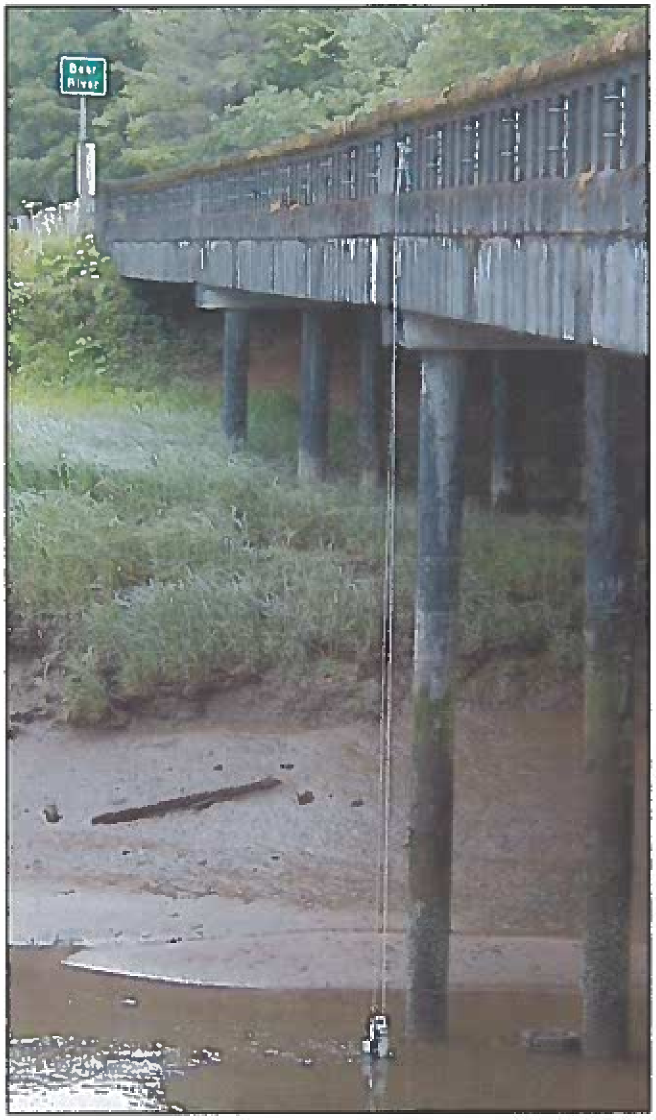


Figure 3-6. Tide gauge deployed at Bear River Bridge

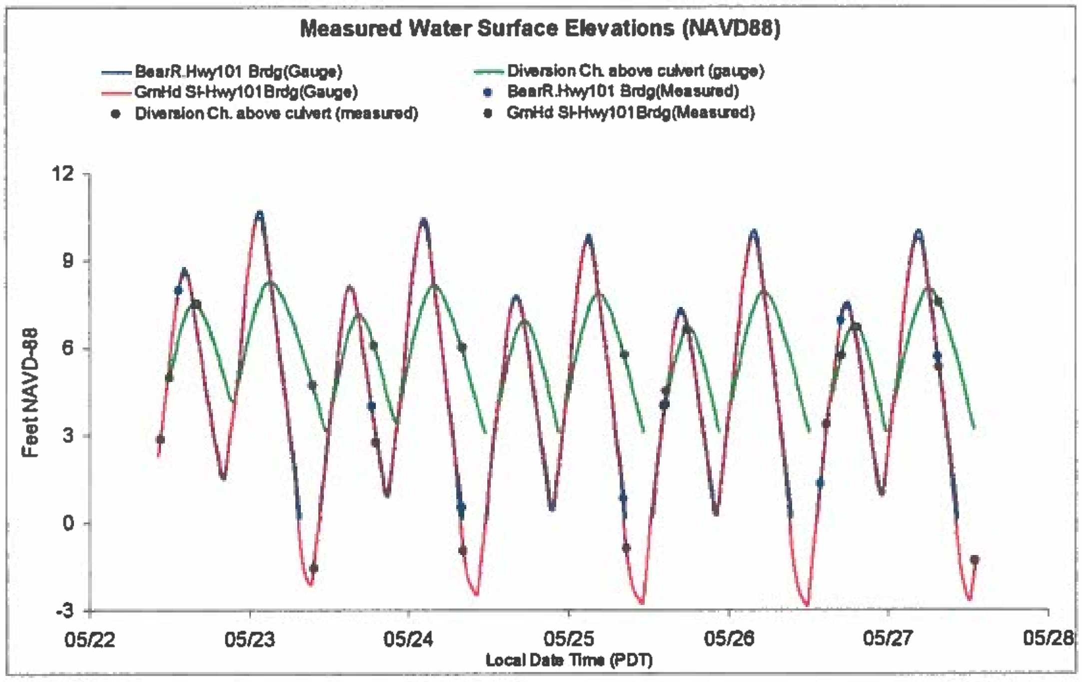


Figure 3-7. Measured spot elevations obtained by RTK GPS and time series of 3-minute water level recorder measurements

3.3 Water Level Analysis

Design water levels and representative tidal fluctuations at the project site are needed for engineering design purposes, and specifically to provide boundary conditions for numerical modeling of surface water processes. A long-term history of water levels is required for developing design water levels for 2-year, 25-year and 100-year return periods. Unfortunately, there is no long-term record of water levels at the project site. To ascertain the design levels, statistical data were obtained from the NOAA Toke Point measuring station (9440910). The results of this analysis were transferred to Bear River based upon a comparison of water levels measured at the project site in the spring of 2005 by PI Engineering and recorded by the NOAA gauge at Toke Point. Further detail on the statistical analysis of data at Toke Point and the comparison between Bear River and Toke Point is given in Appendix B. The results of the water level analysis are presented here for Bear River in the NAVD88 vertical datum. All units are given in feet unless otherwise specified. The design water levels for Bear River are given in Table 3-2.

Table 3-2. Return period water levels for Bear River

Return Period (years)	Water Level (feet)
2	13.24

10	14.03
25	14.42
50	14.68
100	14.95

Based on a Weibull Extreme Analysis of measured daily high water at Toke Point for a 9-year period.

The duration during which various discrete elevations are submerged at Bear River was determined using hourly measured water level data (Toke Point) over a 9-year period to calculate the cumulative distribution curve for water levels in the Bear River area. Figure 3-8 shows the probability of any given elevation remaining dry. Figure 3-9 shows the percent time any given elevation is submerged, which is the inverse of Figure 3-8. Cross-section topographic surveys (Appendix A) indicate that the elevation of the marsh is between 8.5 feet and 10 feet NAVD 88. Figure 3-9 shows that elevations above 9 feet NAVD 88 are submerged approximately 5 percent of the time.

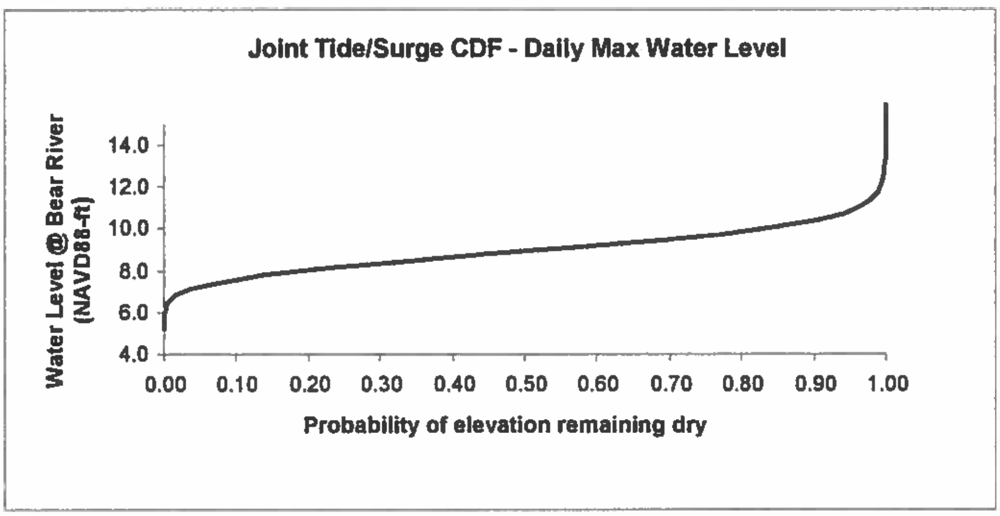


Figure 3-8. Probability of elevations remaining dry at Bear River

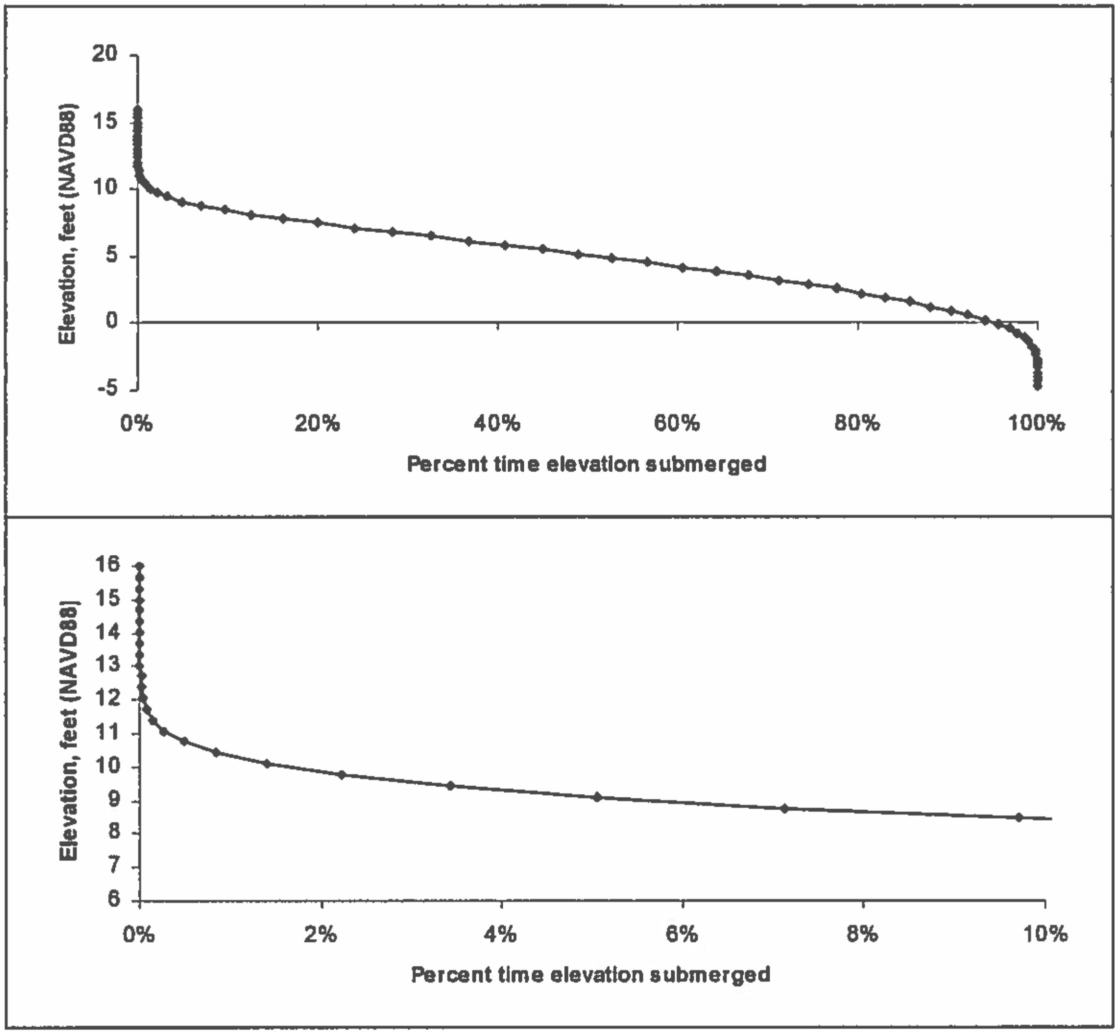


Figure 3-9. Percent time elevations are submerged at Bear River; full range (top); expanded view (bottom).

4.0 Surface Water Modeling

4.1 Model Layout and Geometric Data

The HEC-RAS model (version 3.1.2, USACE, April 2004) used for the Bear River restoration project is a one-dimensional unsteady-flow model. The HEC-RAS model for the existing condition consists of a total of 25 cross-sections that include the borrow ditch, north channel, middle channel, south channel, Bear River and a small creek (no name) flowing into Bear River as shown in Figure 4-1. Geometric data for most of the cross-sections were surveyed by PI Engineering, as described in Section 3-1. Geometric data for Bear River and the upstream section of the south channel and small un-named creek were based on a topographic survey conducted by the US Army Corps of Engineers in 1999 (Figure 2-1). Plots of each of the cross-sections are provided in Appendix A.

4.2 Roughness Coefficient

The flow equations in the HEC-RAS model include the Manning roughness coefficient. The Manning roughness coefficient was estimated to be 0.035 in channel areas and 0.10 to 0.15 in overbank areas. These estimated roughness values were based on several factors, including engineering judgment gained from similar experience and application of the model, observations made during a site visit, and review of available aerial photos.

4.3 Boundary Conditions

The boundary conditions for the HEC-RAS model include inflows at the upstream boundaries, and tailwater conditions at the downstream boundaries. The measured water levels (summer low flow and typical spring tides, see Appendix B) at Hwy 101 bridges over Greenhead Slough and Bear River were used as downstream boundary conditions for the model calibration. Inflow (summer low flow) at the upstream boundary in Bear River was estimated to be 50 cfs, and inflows (summer low flows) in the north channel, middle channel, south channel and the small creek were estimated to be 2 cfs for the period when the water levels were measured. Model calibration runs were performed to validate use of these estimated inflows.

4.4 Model Calibration

The HEC-RAS model was calibrated using the observed stage hydrograph measured by PI Engineering upstream of the logging road (see Appendix B). The calibration procedures primarily involved adjustment of loss coefficients at the logging road culvert and the inflows at upstream boundaries. A comparison of the measured and modeled stage hydrographs upstream of the logging road is shown in Figure 4-1. The comparison indicates satisfactory overall agreement between modeled and observed hydrographs.

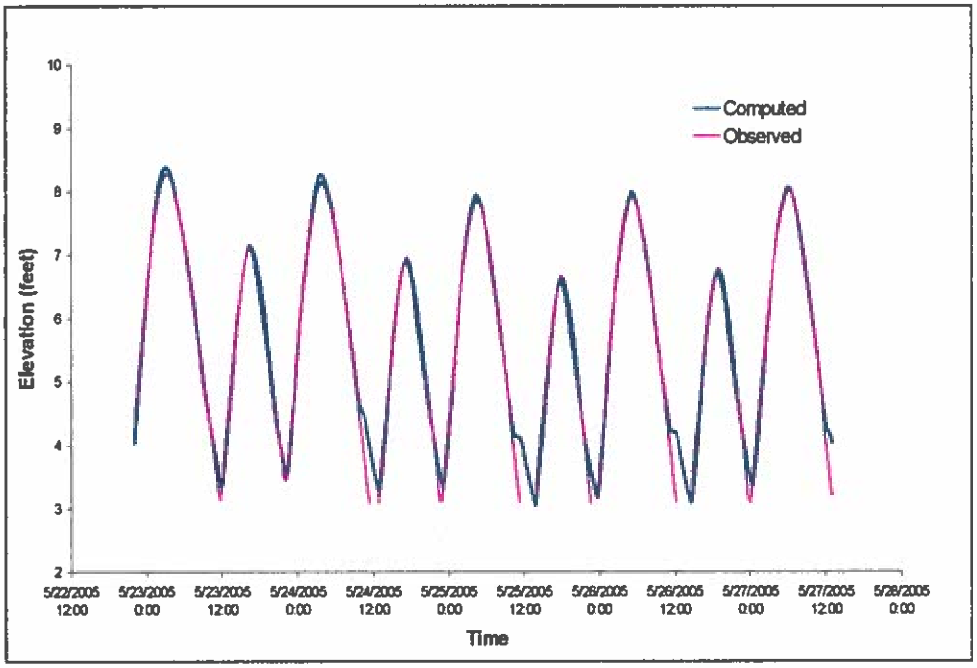


Figure 4-1. Stage hydrographs upstream of the logging road culvert

5.0 Engineering Alternatives

Six engineering alternatives for potentially restoring estuarine wetlands within the project area were originally identified by Ducks Unlimited. These are:

1. Do nothing (existing condition).
2. Removal and replacement of the culvert under the logging road to remove the existing flow constriction.
3. The same as 2 with the addition of a bridge and/or culvert constructed through Highway 101, at the southernmost channel.
4. Removal of the hydraulic connection at the logging road and construction of a bridge and/or culvert through Highway 101, at the southernmost channel.
5. The same as 4 with the addition of a second hydraulic connection through Highway 101, at the middle creek channel.
6. The same as 5 with the addition of a third hydraulic connection through Highway 101, at the northernmost creek channel.

Analysis of water level measurements obtained from the project area (Section 3.2) illustrates that there is a significant attenuation of the tidal fluctuations in the borrow ditch and in the north, middle, and south channels located on the east side of Highway 101 relative to the tidal fluctuations at the Greenhead Slough and Bear River bridges. The goal of the project is simply to restore tidal fluctuations in the north, middle, and south channels such that the attenuation is minimized. The analysis of the field measurements and preliminary modeling with the HEC-RAS model also show that no significant additional increase in water level in the restored channels would be achieved with additional hydraulic connections at the middle and northern channels. It was therefore decided by the project team (July 20, 2005) that Alternatives 5 and 6 be dropped from further analysis. In this Section, conceptual design features for Alternatives 2, 3, and 4 have been developed for the purpose of providing the channel dimensions and elevation details required for surface water modeling of alternatives and to identify any potential constructability considerations such as excavation requirements, bridge pier supports, and scour protection requirements.

5.1 Alternative 2 - Logging Road Box Culvert

Figure 5-1 shows replacement of the existing 4-ft-diameter culvert at the logging road with a set of three larger precast concrete box culverts. These structures would likely require pile foundation supports, based on soil boring data obtained for construction of the nearby Bear River Bridge. Cofferdams would be required in the channel both upstream and downstream of the logging road, to temporarily dewater the site for excavation, pile driving and placement of the culvert sections. Channel flows would be pumped around the project site as required. About 3 months would be required to install the new culverts.

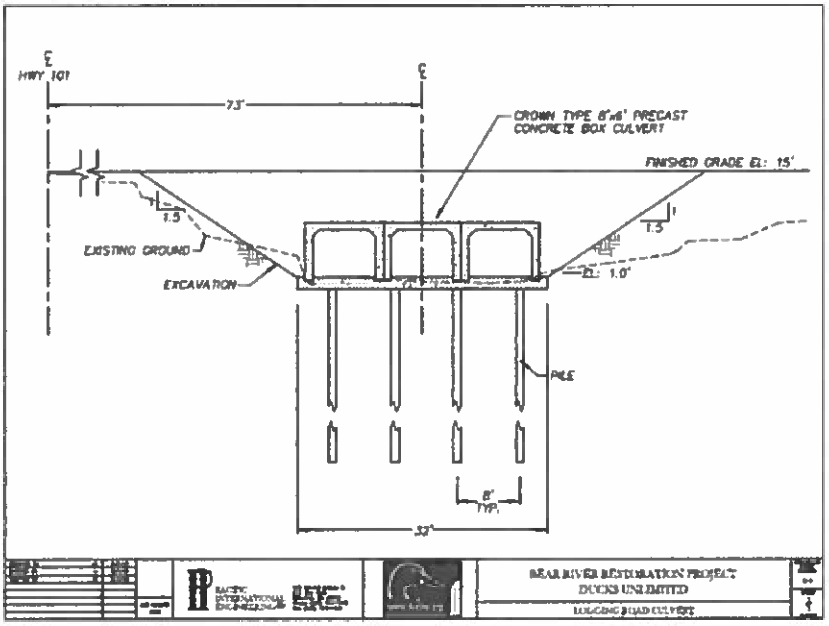


Figure 5-1. Logging road box culvert conceptual design

5.2 Alternative 3 - Logging Road Bridge

Figure 5-2 shows replacement of the existing 4-ft-diameter culvert at the logging road with a single-span bridge. The bridge deck would span a distance of approximately 70 feet providing a channel bed width of approximately 28 feet with 1.5H:1V side slopes meeting the elevation of the existing channel bed at +1 foot NAVD88. The bridge could be constructed without the need for cofferdams and dewatering, provided that the logging road could be closed for about 3 months. Pile supports would be driven followed by construction of the pile caps and bridge abutments. The single-span bridge deck would be set in place following excavation of the new channel through the logging road. About 3 months would be required for bridge construction.

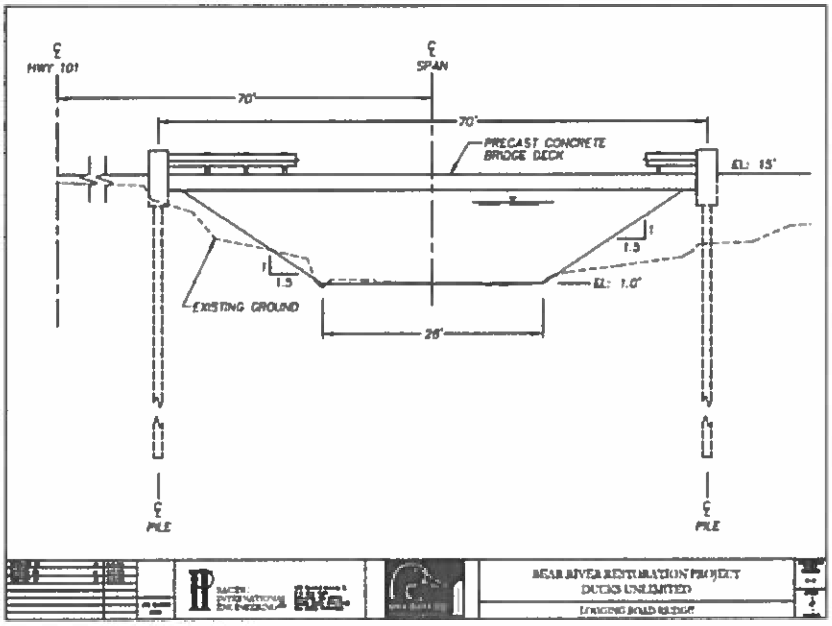


Figure 5-2. Logging road single span bridge conceptual design

5.3 Alternative 4 - South Channel Bridge

5.3.1 Construction

Figure 5-3 shows a new single-span bridge at the south channel. Construction sequence for this bridge would be similar to that for the logging road bridge alternative (Alternative 3), except that a temporary detour for Highway 101 would be required. This bridge would have to be built to Federal Highway specifications, and would be significantly more expensive to construct than the bridge at the logging road. Construction would take 4 to 5 months.

The bridge deck would span a distance of approximately 70 feet providing a channel bed width of approximately 31 feet with 1.5H:1V side slopes meeting the elevation of the existing channel bed at +2 feet NAVD88.

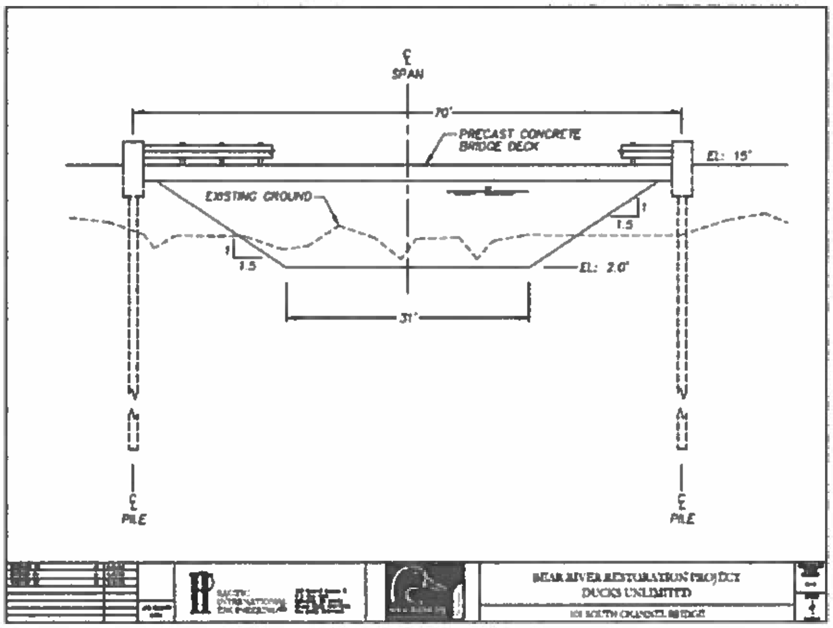


Figure 5-3. Highway 101 at South Channel single span bridge conceptual design

5.3.2 Excavation

Opening the south channel would require approximately 12,000 to 18,000 cubic yards of excavation, extending from the ditch on the east side of the highway to the Bear River. Hauling and disposal of the wet marsh soils would have to be done in accordance with Department of Ecology requirements.

6.0 Evaluation of Alternatives

This section of the report presents an application of the HEC-RAS model for the purpose of conducting a preliminary evaluation of engineering alternatives relative to the existing condition (Alternative 1).

6.1 Evaluation Criteria

The objective of the project is to restore tidal fluctuations to the north, middle, and south channels on the east side of Highway 101 such that the attenuation induced by the existing culvert at the logging road is minimized. The purpose of the evaluation presented herein is to provide a basis for comparison among alternatives leading to recommendations for a preferred alternative from a tidal restoration perspective. Consequently, the evaluation is based on an application of the model for a limited set of relatively typical hydraulic and hydrologic conditions. A subsequent stage of model application will focus on optimization of the preferred alternative and more detailed aspects of the engineering design that will include consideration of a wider range of hydraulic and hydrologic combinations.

The primary hydraulic criteria for this evaluation include maximum water surface elevations and velocities in the south channel and borrow ditch relative to existing conditions and among alternatives. The HEC-RAS model was applied to calculate maximum water surface elevation profiles along the length of the stream channel in the borrow ditch between the lower boundary conditions at either Greenhead Slough or Bear River and the South Channel. Comparison of maximum water surface elevation provides an indication of the relative levels of inundation that can be expected for each alternative relative to the existing condition and also to ambient conditions in Greenhead Slough and Bear River. The higher the water level that can be achieved in the restored channels and borrow ditch, the more inundation of the estuarine marshes that would be expected to occur in the middle, north, and south channels, and the more successful the channel restoration alternative. The model was also used to calculate horizontal depth-averaged current velocities in the borrow ditch upstream of the logging road for comparison with existing conditions. Higher velocities in this segment of the channel will allow more vigorous and more natural circulation and exchange of saline water within the channels and with the estuarine marshland adjacent to the three channels. Higher velocities will also contribute to increased sediment mobility and allow more natural channel bed and bank dynamics to proceed relative to the existing condition. Maximum velocities are also relevant to fish passage, and care must be taken to avoid alternatives that may result in high local velocities in the vicinity of culverts and structures that may restrict fish passage to the restored channels.

6.2 Hydraulic Analysis for Alternative 2 - Removal and Replacement of the Culvert under the Logging Road to Remove the Existing Flow Constriction

This alternative assumes that the existing logging road culvert would be replaced with a set of new culverts or a bridge. The new set of culverts or bridge would increase the hydraulic conveyance to eliminate or minimize the constriction at the logging road. The HEC-RAS model was run for four different sizes and configurations of culverts and two bridges with the summer low flow and typical spring tide boundary conditions used in model calibration. The culvert configurations include a single 6-ft by 6-ft culvert (6x6), two 6x6 culverts (2x6x6), two 8-ft wide by 6-ft high culverts (2x8x6) and three 8-ft by 6-ft culverts (3x8x6). Two bridges include a single-span, 70-ft bridge and a 110-ft bridge opening. Figure 6-1 shows the maximum water surface elevation profiles along the borrow ditch and south channel for various sets and sizes of culverts and bridges.

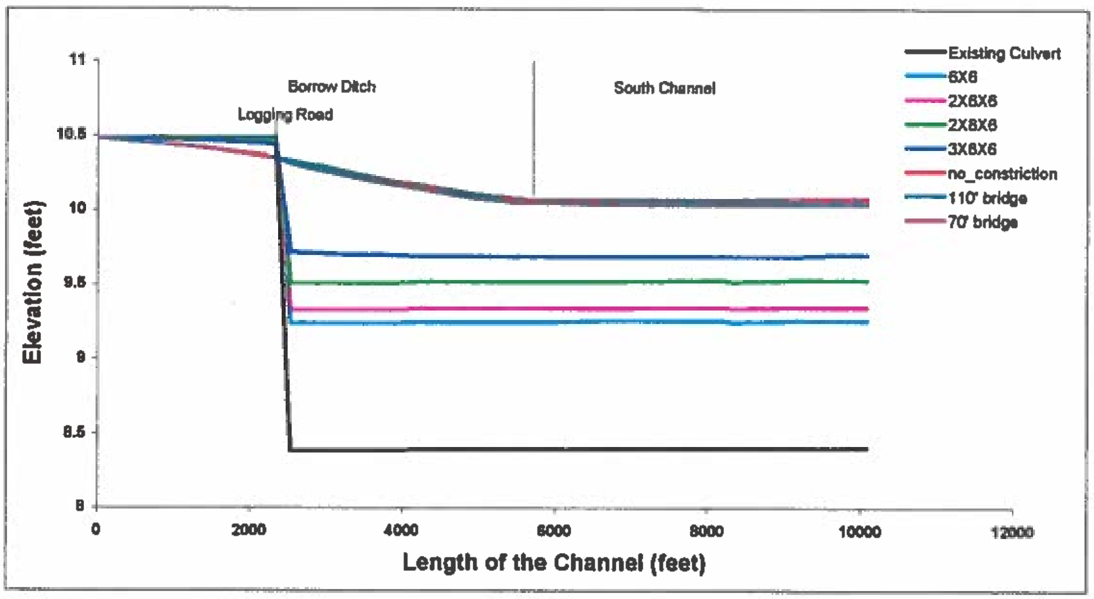


Figure 6-1. Maximum water surface elevation profiles for typical spring tides and summer low flows - Alternative 2

Figure 6-1 shows that this alternative would significantly increase the maximum water surface elevation along the borrow ditch and in the south channel relative to the existing condition. The increased maximum water surface elevations would range from 0.85 to 1.67 feet at the south channel for various sets and sizes of culverts and bridges with increasing water levels achieved with increasing conveyance. The highest water levels are achieved with the bridge alternatives, which provide only slightly less conveyance than an unconstricted condition for these boundary conditions.

The north channel and middle channel would experience similar improvements. There is a loss of approximately 0.4 feet along the one-mile length of borrow ditch for 70-ft and 110-ft span bridges as shown in Figure 6-1. This loss is mainly due to constriction that results in frictional headloss caused by the relatively narrow borrow ditch. Widening the borrow ditch by excavation would help minimize this loss.

The model was also run for the selected sets and sizes of culverts and bridges (2x8x6 & 3x8x6 culverts and 110-ft & 70-ft span bridges) with a winter spring tide as the downstream boundary condition and 2-year flood inflows as the upstream boundary condition. The 2-year flood inflows were estimated based on our recent experience in nearby streams of similar drainage size and hydrologic characteristics. Maximum predicted water surface elevation profiles for these boundary conditions are shown in Figure 6-2. There is a larger increase in the conveyance for the 110-ft span bridge relative to the 70-ft bridge for these flows relative to the summer low flow conditions.

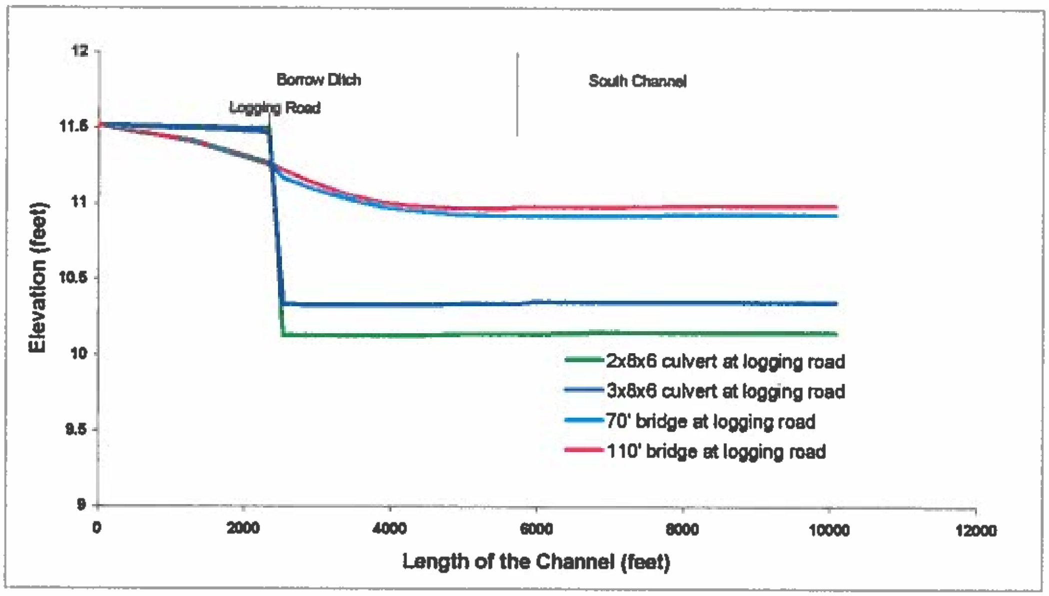


Figure 6-2. Maximum water surface elevation profiles for winter spring tides and 2-year flood inflows - Alternative 2

6.3 Hydraulic Analysis for Alternative 3 - Same as Alternative 2 with the Addition of a Bridge and/or Culvert Constructed through Highway 101, at the Southernmost Channel

This alternative assumes that a set of culverts or a bridge opening would be constructed through Highway 101 at the south channel in combination with the existing culvert or new culverts (3x8x6) at the logging road to achieve the restoration of the estuarine wetland at the project area. Three additional cross-sections surveyed by PI Engineering are included in the HEC-RAS model to represent the configuration of the south channel to the west of Highway 101(see Appendix A) for modeling Alternative 3. The HEC-RAS model was run for various configurations and sizes of culverts or bridges with winter spring tides as the downstream boundary condition and the 2-year flood inflows as the upstream boundary condition. The maximum water surface profiles along the borrow ditch and south channel are plotted in Figure 6-3. Excavation of about 18,000 and 12,000 cubic yards of material at the south channel west of Highway 101 are required for the 110-ft and 70-ft span bridges, respectively, to achieve the results shown in Figure 6-3.

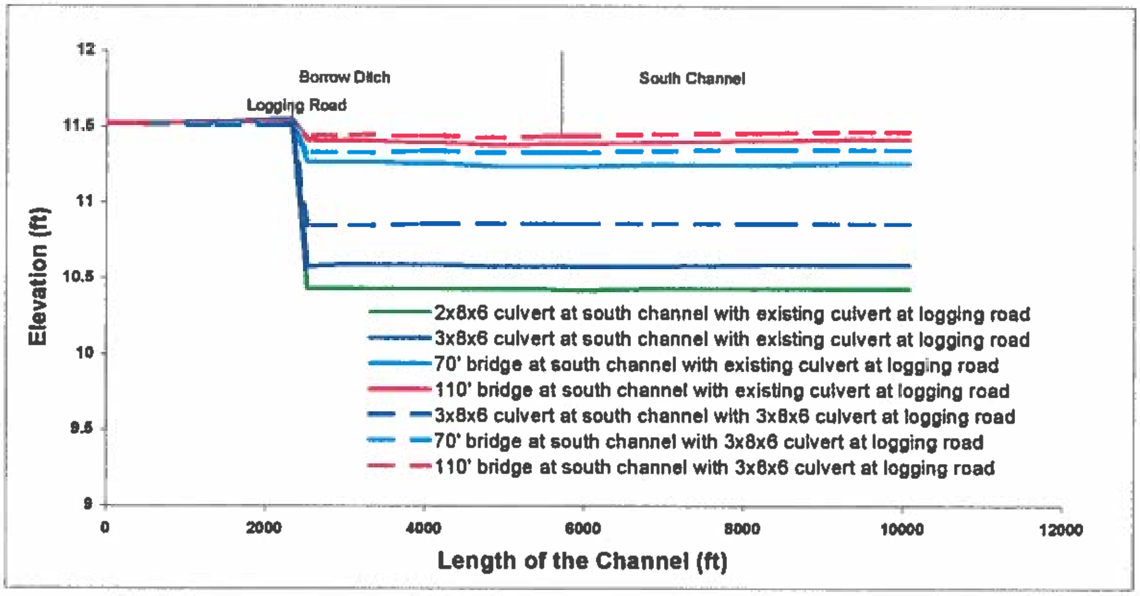


Figure 6-3. Maximum water surface elevation profiles for winter spring tides and 2-year flood inflows - Alternative 3

Figure 6-3 shows that the maximum water surface elevations in the borrow ditch and south channel would increase with the addition of new culverts (3x8x6) at the logging road compared with retaining the existing culvert at the logging road for the same openings at the south channel. The analysis also shows that the bridge alternatives offer more efficient conveyance than the culvert options. It is also apparent that the 70-ft span (Alternative 2) is more efficient than the 3x8x6 culvert (Alternative 3).

6.4 Hydraulic Analysis for Alternative 4 - Removal of the Hydraulic Connection at the Logging Road and Construction of a Bridge and/or Culvert through Highway 101, at the Southernmost Channel

This alternative assumes that a set of culverts or a bridge would be constructed through Highway 101 at the south channel, and the hydraulic connection at the logging road would be removed. The HEC-RAS model was run for 3x8x6 culverts and a 70-ft span bridge with winter spring tides as the downstream boundary condition and the 2-year flood inflows as the upstream boundary condition. The maximum water surface elevation profiles along the borrow ditch and the south channel are plotted in Figure 6-4, along with the maximum water surface elevation profiles for the same set of culverts and bridge with the existing logging road culvert (Alternative 3). Removal of the hydraulic connection at the logging road would decrease the maximum water surface elevation in the borrow ditch and the south channel for the same openings at the south channel through Highway 101. A similar decrease in the maximum water surface elevation would be expected at the middle and north channels. These results suggest that no significant benefit in terms of inundation can be gained by closing the culvert at the logging road. Circulation in the middle and north channels would also be expected to be reduced with this alternative. Therefore this alternative should not be considered further.

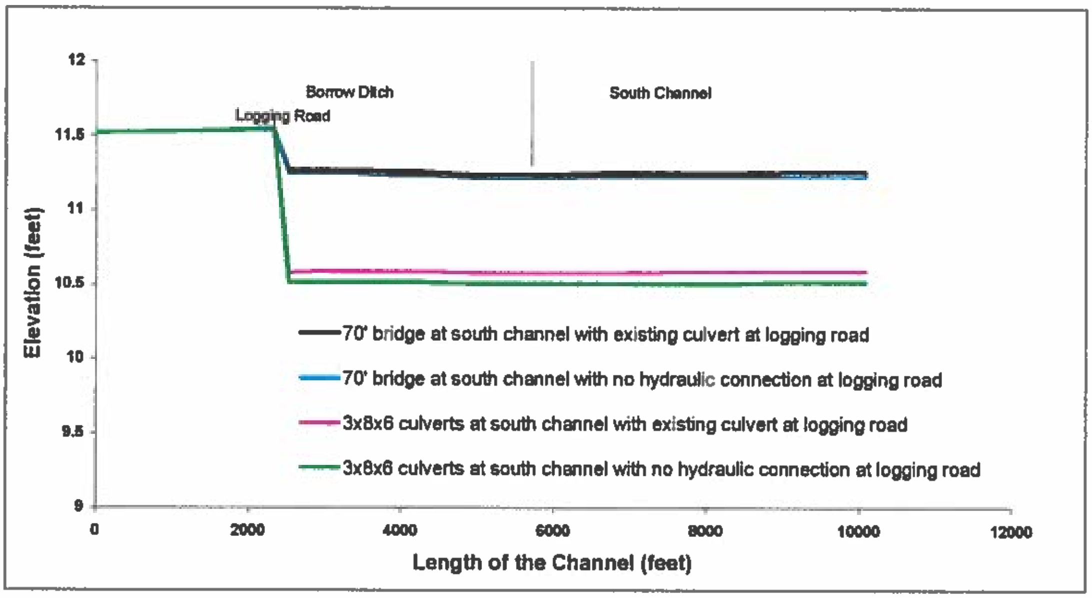


Figure 6-4. Maximum water surface elevation profiles for winter spring tides and 2-year flood inflows - Alternative 4

6.5 Discussion and Comparison of Alternatives 2 and 3

The HEC-RAS model results are summarized in Table 6-1 for Alternatives 2 and 3. Alternative 3 shows an improvement over Alternative 2 for the same sets and sizes of culverts or bridges. For example, the maximum water surface elevation at the south channel would be 0.33 feet higher for a 70-ft span bridge at the south channel in combination with the existing logging road culvert compared with a 70-ft bridge at the logging road. However, it is possible that this water surface elevation difference would be eliminated or minimized if widening the borrow ditch were considered.

The maximum velocities upstream of the logging road and in the south channel are also summarized in Table 6-1. The maximum velocity in the borrow ditch would increase with the increasing conveyance for Alternative 2. The highest maximum velocity would occur with a 110-ft span bridge for Alternative 3 in combination with the existing logging road culvert. The maximum velocity at the borrow ditch would be relatively low, ranging from 0.45 to 0.47 ft/sec. The maximum velocity would increase with the decrease conveyance at the south channel for Alternative 3 in combination with 3x8x6 culverts at the logging road. This effect is most likely a result of localized flow acceleration in the vicinity of the culverts. The highest maximum velocities would occur with the 3x8x6 culverts.

Table 6-1. Summary of HEC-RAS model results

Boundary Conditions	Project Conditions	Structure Size	Maximum W.S.Elevation at South Channel* (feet)	Maximum Velocity at Borrow Ditch** (cfs)	Maximum Velocity at South Channel*** (cfs)
Typical spring tides and summer low flows	Existing condition	Existing culvert	8.4	0.47	0.24
	Alternative 2	6'X6'	9.26	0.92	0.41
		2X6'X6'	9.35	1.76	0.71
		2X8'X6'	9.53	2.14	0.80
		3X8'X6'	9.7	2.64	0.89
		No constrictions	10.08	3.08	1.01
		70' bridge	10.04	2.94	0.95
		110' bridge	10.05	2.97	0.96
Winter spring tides and 2-year flood inflows	Alternative 2	2x8'X6'	10.15	2.45	0.86
		3X8'X6'	10.35	3.43	1.05
		70' bridge	10.93	4.05	1.18
		110' bridge	10.99	4.18	1.21
	Alternative 3 combine with existing logging road culvert	2x8'X6'	10.43	0.47	0.72
		3X8'X6'	10.59	0.47	0.82
		70' bridge	11.26	0.45	1.53
		110' bridge	11.42	0.45	1.85
	Alternative 3 combine with 3x8x6 Logging road culverts	3X8'X6'	10.86	3.20	1.78
		70' bridge	11.35	2.39	2.15
		110' bridge	11.47	1.77	2.29

* Upstream end of south channel
**upstream of logging road
***Transect 21 (see Figure 3-1)

Figure 6-5 shows a series of water level duration curves for the existing condition and for Alternatives 2 and 3 at various locations (only the upper 10 percent of the water level duration shown). The water level duration curve expresses the percentage of time that a given elevation (NAVD88) in feet is submerged (the percentage time wet). The duration curves at Greenhead Slough Bridge and Bear River at the south channel are based on conversion of an 8-yr-long water level time series record from Toke Point to Bear River at these two locations. The duration curves at the south channel for the existing condition, Alternative 2 (70-ft span bridge at logging road) and Alternative 3 (70-ft span bridge at the south channel in combination with the existing logging road culvert) are based on the HEC-RAS model results. Both Alternative 2 and Alternative 3 would significantly increase the maximum water surface elevation at the south channel. The average overbank elevation of the south channel is approximately 9.5 to 10 feet. The water level would be higher than 10 feet at the south channel for 1.1% and 1.3 % of the time for Alternative 2 and Alternative 3, respectively; the water level would be higher than 9.5 feet at the south channel for 2.7% and 2.9 % of the time for Alternative 2 and Alternative 3, respectively.

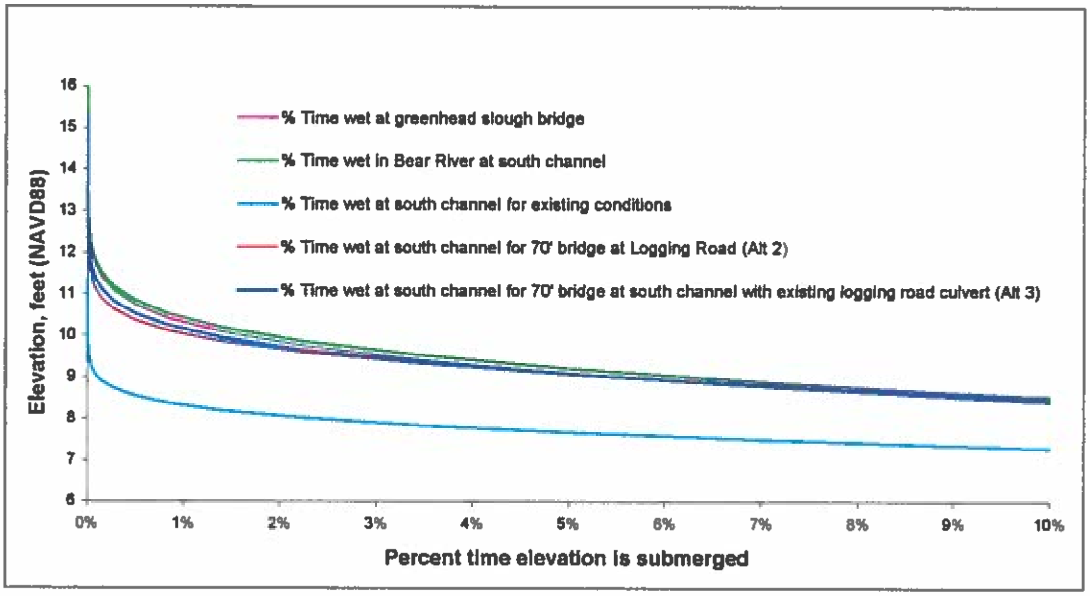


Figure 6-5. Water level duration curves showing the percentage of time wet for various locations and alternatives

The velocity duration curves for Alternative 2 (70-ft span bridge at logging road) upstream of the logging road are plotted in Figure 6-6. The velocity duration curves are based on the HEC-RAS model run results for two completed tide cycles (28 days for each tide cycle): typical spring tides in June and winter spring tides in January. The velocity would be higher than 3 ft/sec upstream of the logging road for 1.66% and 0 % of the time for winter spring tides and typical spring tides, respectively; the velocity would be higher than 3ft/sec upstream of the logging road for 0.83% of the time for the average of the winter spring tides and typical spring tides.

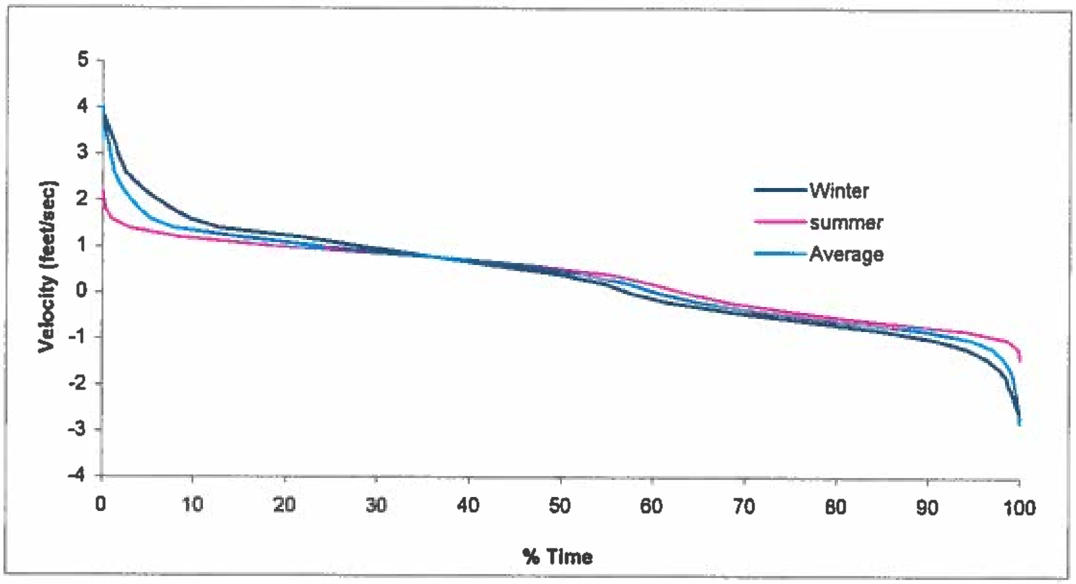


Figure 6-6. **Velocity duration curves Alternative 2, upstream of the logging road**

7.0 Summary of Alternative Analysis and Recommendations for Preferred Alternative

Analysis of water level measurements obtained from the project area (Section 3.3) illustrates that there is a significant attenuation of the tidal fluctuations in the borrow ditch, and in the north, middle, and south channels located on the east side of Highway 101 relative to the tidal fluctuations at the Greenhead Slough and Bear River bridges. This attenuated condition has existed since flows in the three channels were diverted to the borrow ditch as a result of construction of State Highway101 and placement of a culvert at the logging road resulted in constriction of flows in the borrow ditch.

Consequently, Ducks Unlimited and partners are seeking to restore tidal fluctuations in the north, middle, and south channels and minimize the attenuation associated with the existing condition. The analysis of the field measurements and preliminary modeling with the HEC-RAS model also show that no significant additional increase in water level in the restored channels would be achieved with additional hydraulic connections at the middle and northern channels relative to efficient alternatives at the logging road and/or the south channel. The analysis of alternatives presented herein was therefore focused on those alternatives that involve openings at the logging road and the south channel (Alternatives 2, 3, and 4), and Alternatives 5 and 6 were eliminated from further consideration. Analysis of Alternative 4, which involves removal of the hydraulic connection at the logging road and construction of a bridge or culvert through Highway 101 at the south channel, reveals that no significant benefit in terms of inundation and circulation can be gained by closing the culvert at the logging road.

The HEC-RAS model results for Alternative 2 and Alternative 3 indicate that Alternative 3 provides an improvement in terms of overall inundation and velocities in the south channel relative to Alternative 2 for the same variations. Alternative 2 provides slightly higher velocities in the borrow ditch and in the vicinity of the north channel relative to Alternative 3, which may improve circulation and sediment dynamics in those areas. Variations on the alternatives tested show that increasing conveyance results in increasing levels of inundation of the restored estuarine marshes. The highest maximum water surface elevations are achieved for the unrestricted condition, with the second highest elevations achieved with a 110-ft span bridge, followed by a 7-ft span bridge, and finally by progressively decreasing numbers of 8-ft by 6-ft culverts.

A significant consideration with respect to Alternative 3 is the need to excavate up to 18,000 cubic yards of sediment between Highway 101 and the Bear River channel in order to accommodate restoration flows. On the other hand, Alternative 2 results in a decrease in the maximum water level of up to 0.55 feet for the winter spring tide and 2-yr flood flow along the length of the borrow ditch and south channel as a result of channel constriction and friction. Excavation of material from the borrow ditch to widen the channel may reduce the decrease in maximum water level. Further analysis is needed to evaluate the excavation of the borrow ditch and to distinguish the performance of Alternatives 2 and 3.

Both Alternatives 2 and 3 would achieve the project goals to restore tidal fluctuations in the north, middle, and south channels and minimize the attenuation associated with the existing condition. The results of the preliminary analysis were presented to Ducks Unlimited and partners at a meeting on August 18th. After review and additional meetings with WSDOT, Ducks Unlimited and partners decided that Alternative 2 (70-ft span bridge at logging road) would be the preferred alternative. Alternative 3 was eliminated from further consideration since the hydraulic connections through the highway were not acceptable to WSDOT at this time. Further discussion with Ducks Unlimited and partners resulted in modifying Alternative 2 as follows:

- A 70-ft span bridge would be constructed at the logging road.
- The centerline of the bridge would be located about 160 feet from the centerline of Highway 101. This distance would allow the bridge to be constructed in the dry, with no impact to existing channel flows during the construction period. This distance would also ensure conformance with WSDOT standards for highway ingress and egress. In addition, construction of the new bridge abutment would be far enough away from Highway 101 so that the highway fill would not be disturbed.
- New channel sections would be excavated upstream and downstream of the new bridge to transition to and from the existing borrow ditch.
- The existing culvert at the logging road would remain.

This modified Alternative 2 is subsequently referred to as the preferred alternative and is shown in Figures 7.1 and 7.2.

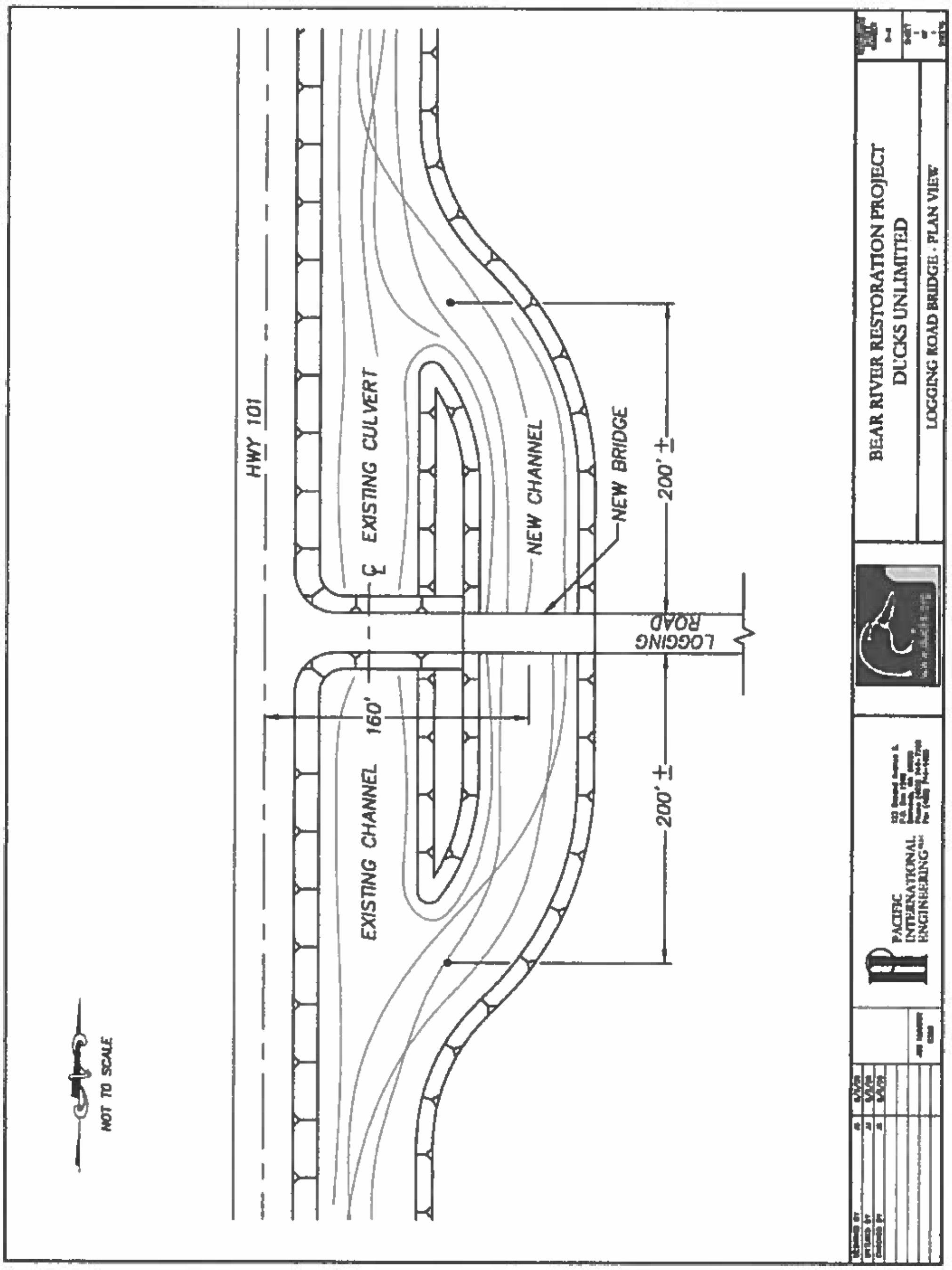


Figure 7.1. Preferred Alternative Logging Road Bridge Plan View

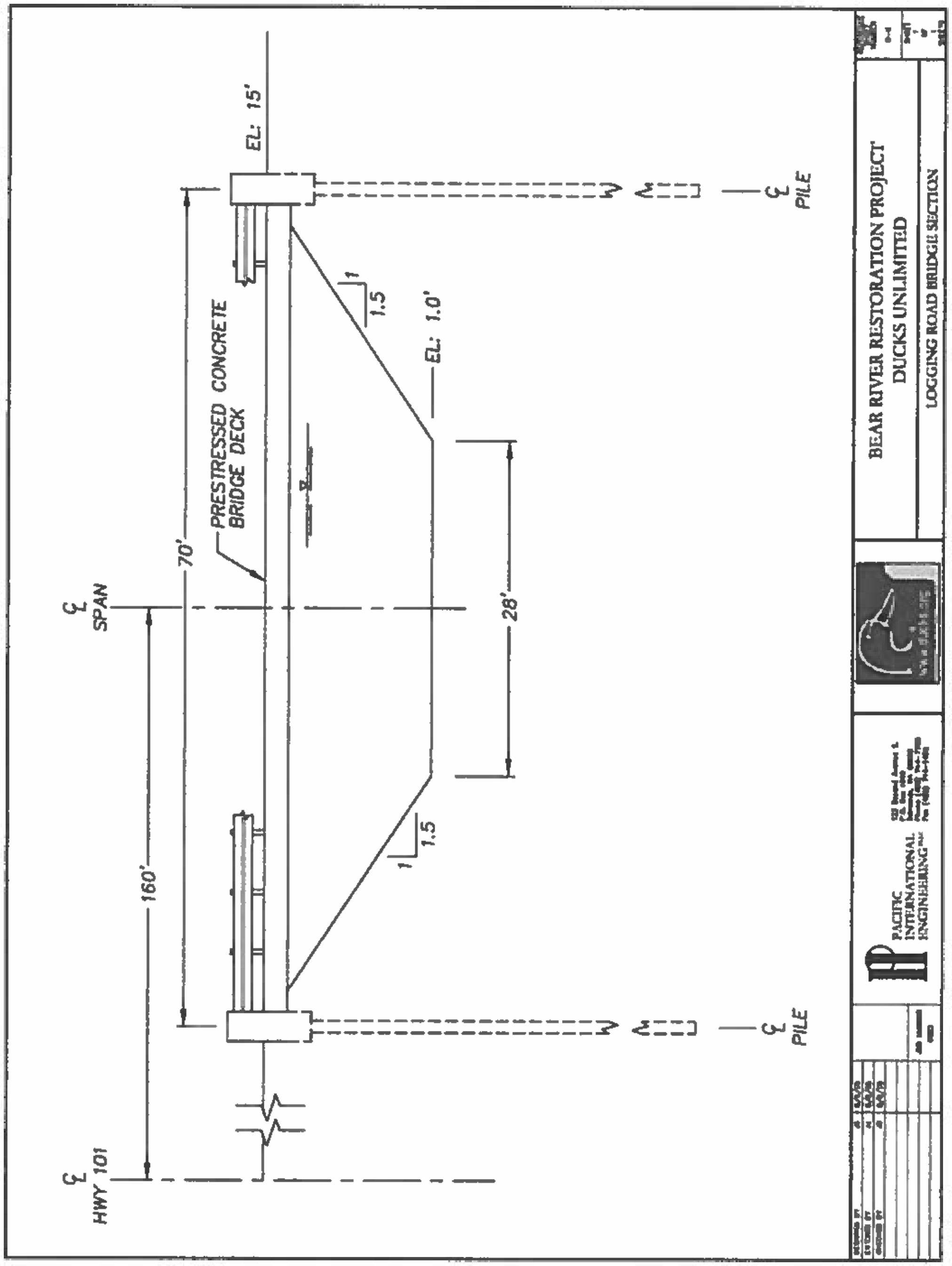


Figure 7.2. Preferred Alternative Logging Road Bridge Section

8.0 Details of Hydraulic Analysis for Preferred Alternative

The hydraulic analysis of the preferred alternative included modifying the HEC-RAS model and running the HEC-RAS for combinations of tidal and surge-induced water levels (25-yr, 50-yr, and 100-yr water levels) and stream flows (25-yr, 50-yr, and 100-yr return period flows) to optimize the preferred alternative and develop more detailed design information. The maximum water surface profiles along the borrow ditch and south channel are plotted in Figure 8-1 to 8-3 for combinations of tides and stream flows.

It appears that the tidal and surge-induced water levels have significant effects on the maximum water surface profiles along the borrow ditch and south channel for both existing and preferred alternatives. The stream flows, on the other hand, have very little effects on the maximum water surface profiles along the borrow ditch and south channel.

The preferred alternative would decrease the maximum water surface elevation along the south channel for 100-year tides in combination with 100-, 50-, and 25-year stream flows relative to the existing condition. The decreased maximum water surface elevations would be 0.01 feet at the upstream end of the south channel for 100-year tides in combination with 100-, 50-, and 25-year stream flows. The maximum water surface along the south channel would increase for 50- and 25-year tides in combination with 100-, 50-, and 25-year stream flows relative to the existing condition. The increased maximum water surface elevations would range from 0.01 to 0.03 feet at the upstream end of the south channel for 50- and 25-year tides in combination with 100-, 50-, and 25-year stream flows.

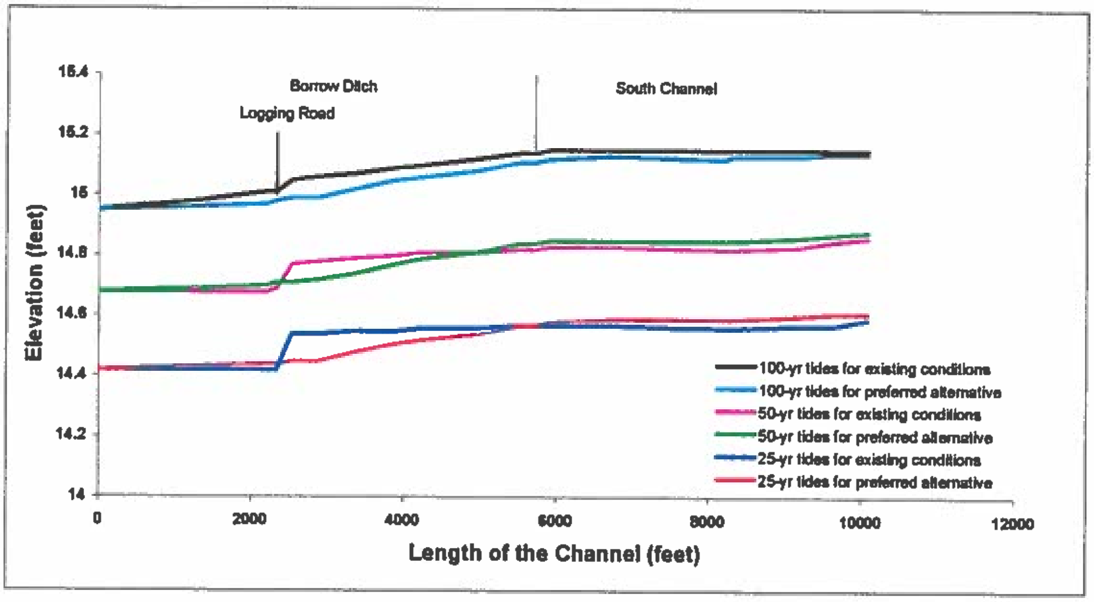


Figure 8-1. Maximum water surface elevation profiles for Preferred Alternative - 100-year stream flow

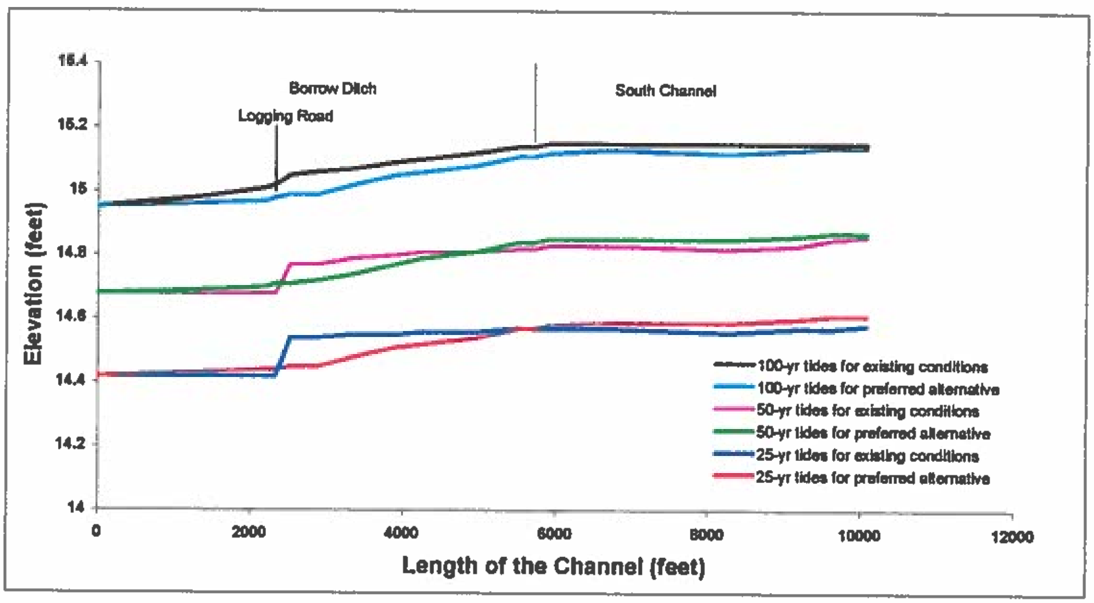


Figure 8-2. Maximum water surface elevation profiles for Preferred Alternative - 50-year stream flow

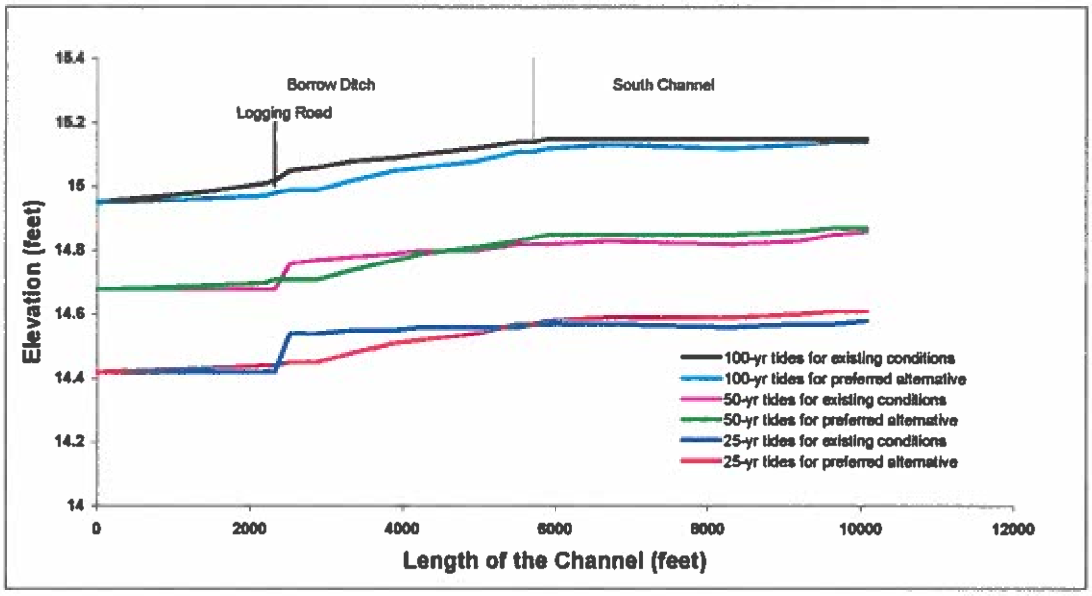


Figure 8-3. Maximum water surface elevation profiles for Preferred Alternative - 25-year stream flow

The HEC-RAS model results are summarized in Table 8-1 for the existing condition and for the preferred alternative. The maximum velocities upstream of the logging road and in the south channel are also summarized in Table 8-1. The preferred alternative would increase the maximum velocity in the borrow ditch and decrease the maximum velocity in the south channel relative to the existing condition. The maximum velocity would range from 5.32 feet/sec to 5.38 feet/sec in the borrow ditch, and from 1.25 feet/sec to 1.31 feet/sec in the south channel for the preferred alternative for combinations of tides and stream flows.

The HEC-RAS models were not rerun for typical spring tides and winter spring tides in combination with summer low flow and 2-year stream flow for the preferred alternative. With the existing culvert remaining in place, the preferred alternative would increase the maximum water surface elevation less than 0.01 feet (estimated based on HEC-RAS model results) in the south channel relative to Alternative 2 (70-ft span bridge at logging road) for typical spring tides and winter spring tides in combination with summer low flow and 2-year stream flow.

The possibility of abandoning the existing culvert and backfilling the borrow ditch at the logging road with material excavated from the new transition was also considered. The difference in maximum water surface elevation in the south channel between "with…" and "without existing culvert at the logging road" would be insignificant (less than 0.01 based on HEC-RAS model results) for typical and winter spring tides, and would be the same for extreme events (25-, 50- and 100-year tides in combination with 25-, 50- and 100-year stream flows).

Table 8-1. Summary of HEC-RAS model results for existing conditions and preferred alternative

Boundary Conditions		Existing Condition			Preferred Alternative			Difference		
Stream Flow Return Period (years)	Tide Elevation Return Period (years)	Maximum WSE* at South Channel** (feet)	Maximum Velocity at Borrow Ditch*** (cfs)	Maximum Velocity at South Channel**** (cfs)	Maximum WSE* at South Channel** (cfs)	Maximum Velocity at Borrow Ditch*** (cfs)	Maximum Velocity at South Channel**** (cfs)	Maximum WSE* at South Channel** (feet)	Maximum Velocity at Borrow Ditch*** (cfs)	Maximum Velocity at South Channel**** (cfs)
100	100	15.5	1.18	2.17	15.14	5.38	1.30	-0.01	4.20	-0.87
	50	14.86	1.00	2.33	14.88	5.35	1.27	0.02	4.35	-1.06
	25	14.59	0.91	2.47	14.61	5.33	1.25	0.02	4.42	-1.22
50	100	15.15	1.19	2.19	15.14	5.37	1.31	-0.01	4.18	-0.88
	50	14.86	1.00	2.34	14.87	5.35	1.28	0.01	4.35	-1.06
	25	14.58	0.90	2.48	14.61	5.33	1.25	0.03	4.43	-1.23
25	100	15.15	1.19	2.20	15.14	5.36	1.31	-0.01	4.17	-0.89
	50	14.86	0.99	2.36	14.87	5.34	1.28	0.01	4.35	-1.08
	25	14.58	0.91	2.50	14.61	5.32	1.25	0.03	4.41	-1.25

* WSE = water surface elevation
**Upstream end of south channel
***Upstream of logging road
****Transect 21 (see Figure 3-1)

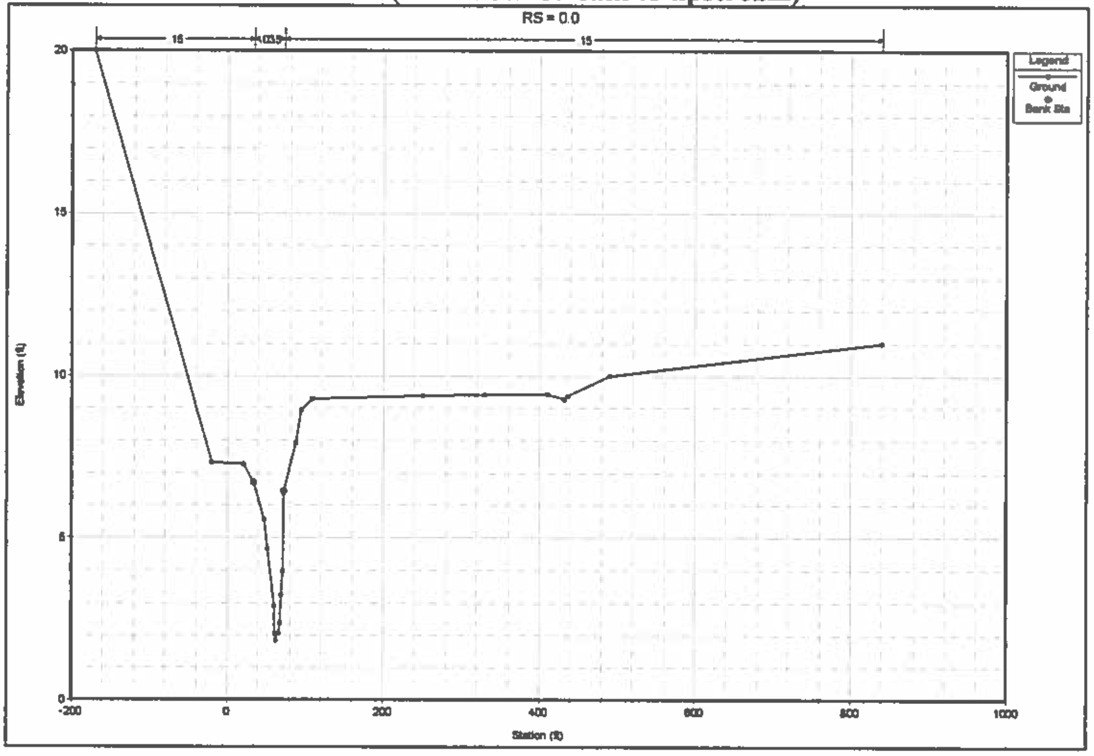
Highway 101 would be overtopped during extreme events. However, no flow exchange would occur between each side of the highway since the water surface elevations on both sides of the highway would be approximately the same when the water level reaches the elevation of the highway for both the existing condition and the preferred alternative. The logging road would be overtopped before the highway overtopped under the existing condition since the elevation of the logging road is approximately 2 feet lower than highway 101. For the preferred alternative, the 70-ft-span bridge at the logging road along with the existing culvert would increase the conveyance of the borrow ditch significantly. Consequently, the water surface elevation on both sides of the highway would be approximately the same for the preferred alternative.

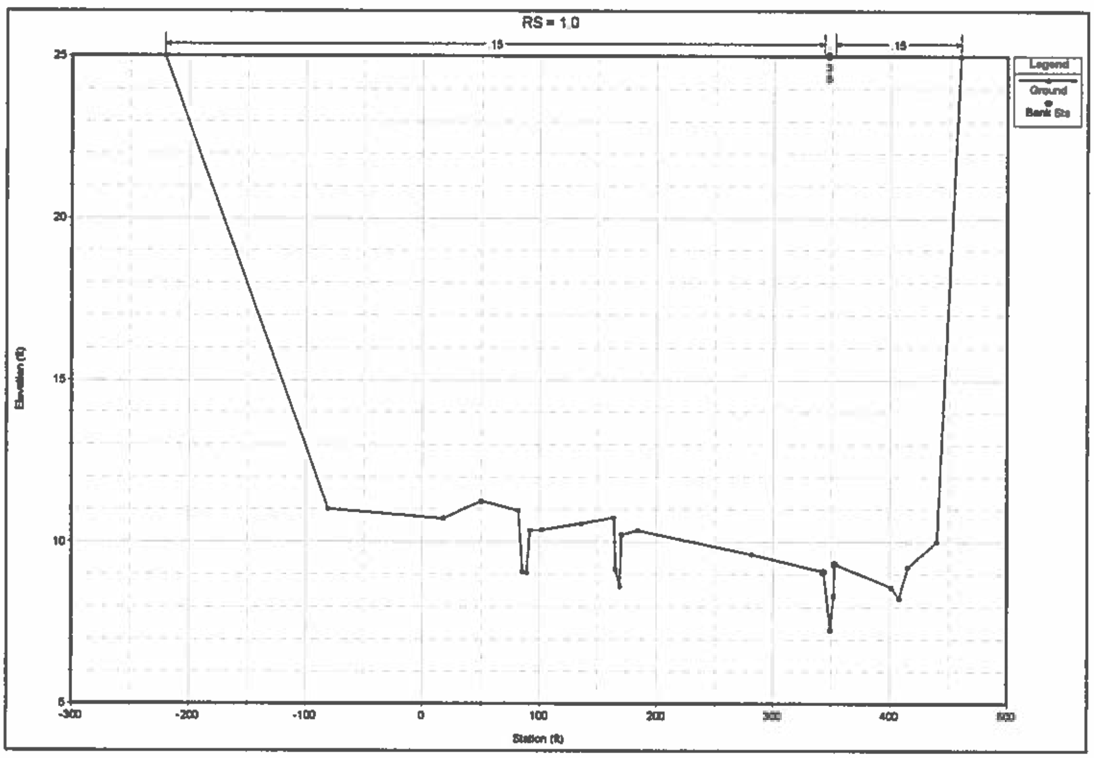
9.0 Conclusions

The preferred alternative that was selected based on the preliminary analysis of alternatives would achieve the project goals to restore tidal fluctuations in the north, middle, and south channels and minimize the attenuation associated with the existing condition. The water level would be higher than 10 feet and 9.5 feet at the south channel for 1.1% and 2.7% of the time, respectively, for the preferred alternative. The velocity would be higher than 3 feet/sec upstream of the logging road for 1.66% and 0 % of the time for winter spring tides and typical spring tides, respectively. The hydraulic analysis of extreme events (25-, 50- and 100-year tides in combination of 25-, 50- and 100-year stream flows) for the preferred alternative indicated that the maximum water surface elevation in the south channel would decrease for 100-year tides and increase for 50- and 25-year tides relative to the existing condition. The increased maximum water surface elevation in the south channel would range from 0.01 feet to 0.03 feet for 50- and 25- year tides relative to the existing condition.

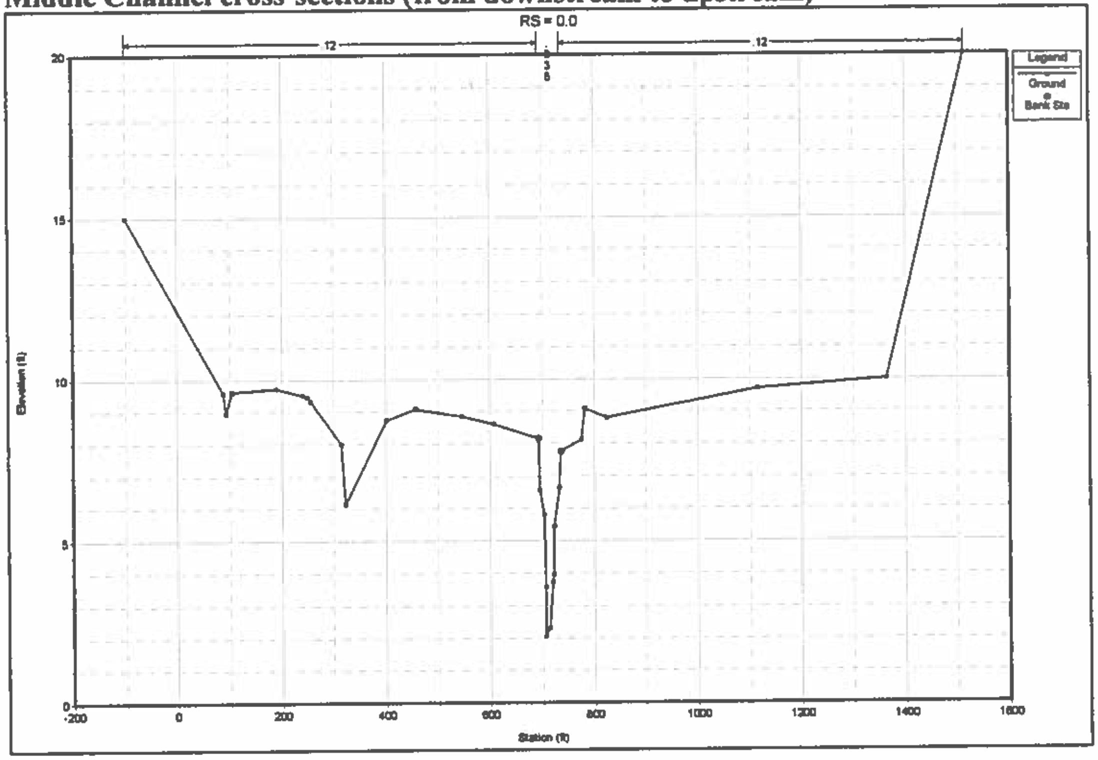
APPENDIX A
TOPOGRAPHIC CROSS-SECTIONS

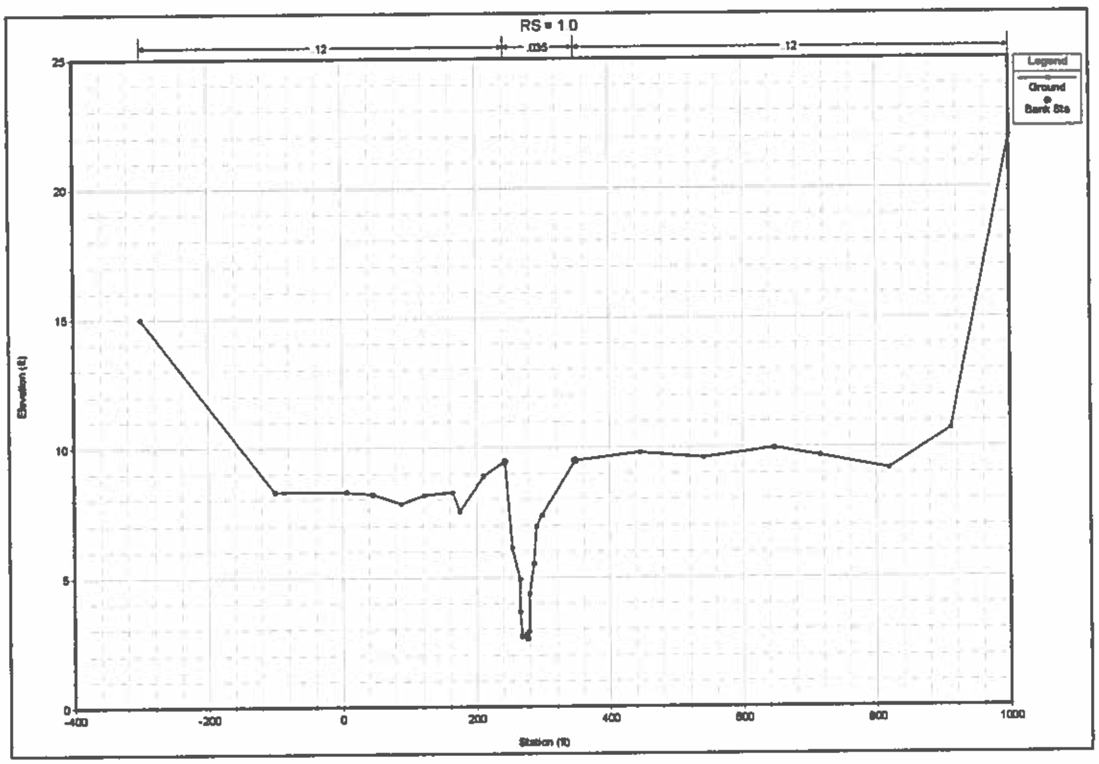
North Channel cross-sections (from downstream to upstream)

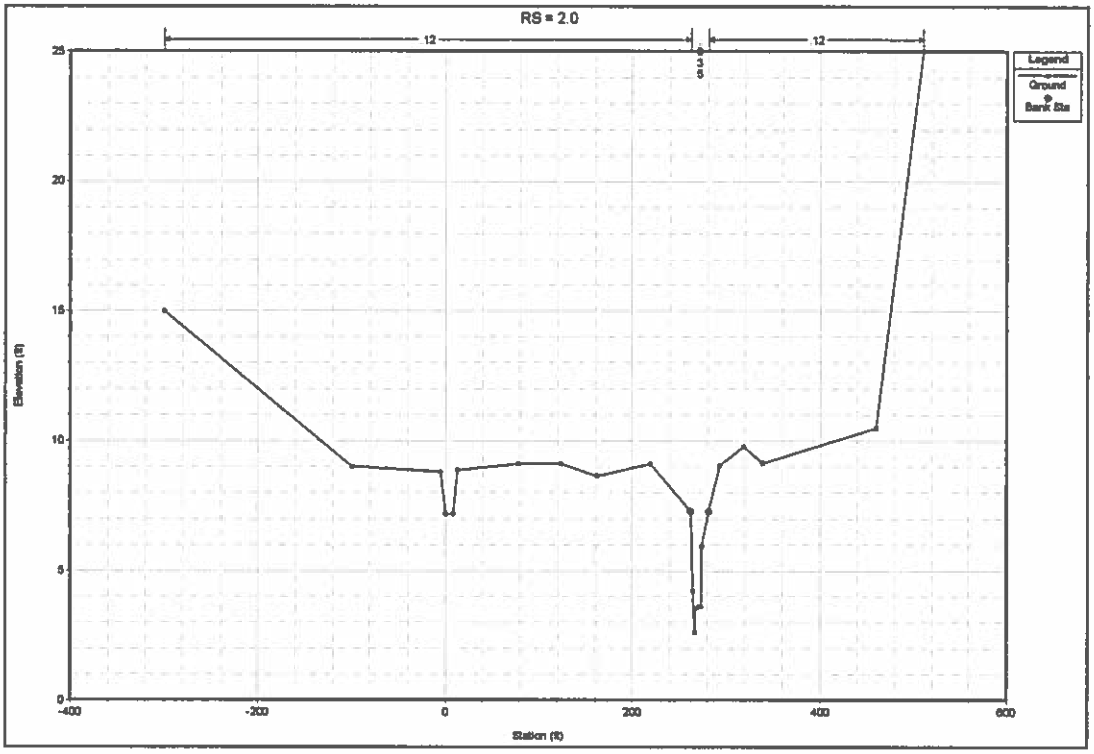




Middle Channel cross-sections (from downstream to upstream)







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Bank Sta
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Station (ft)
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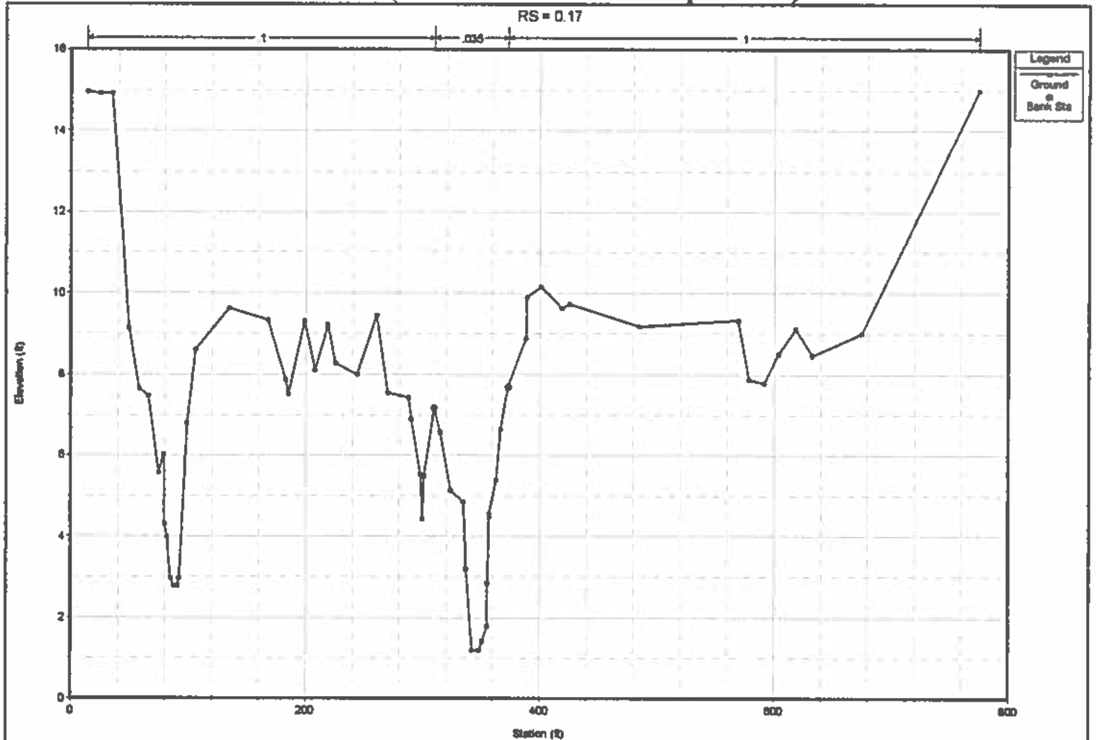
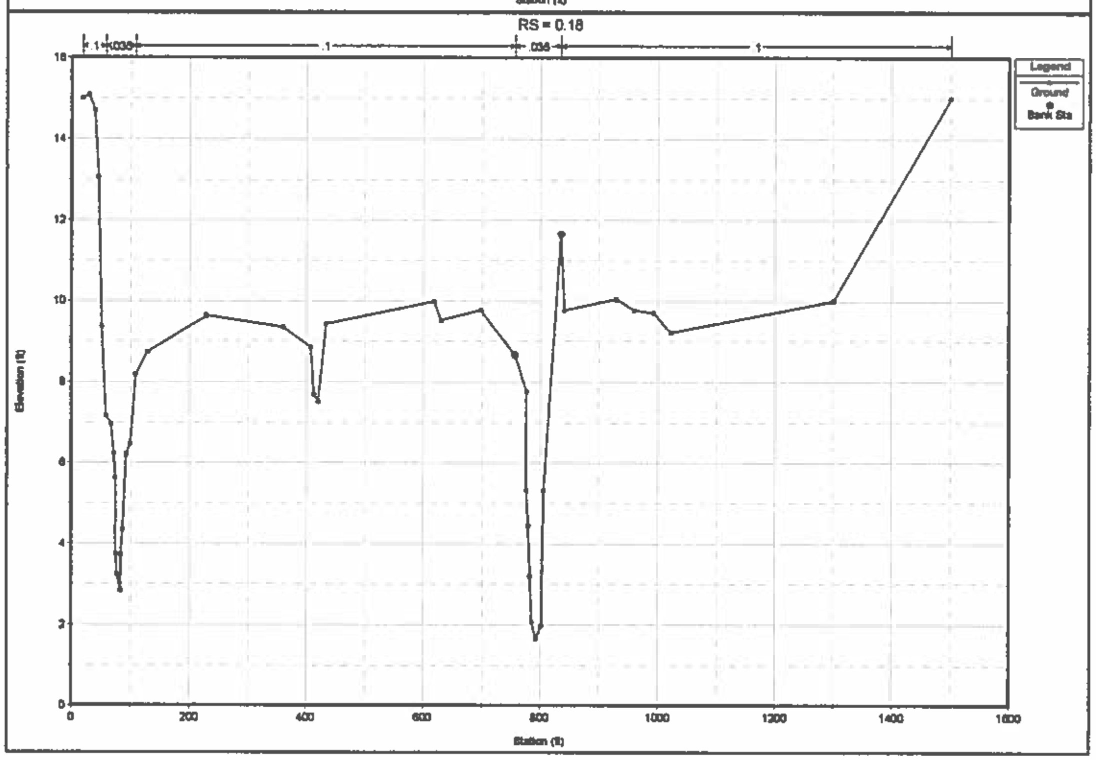
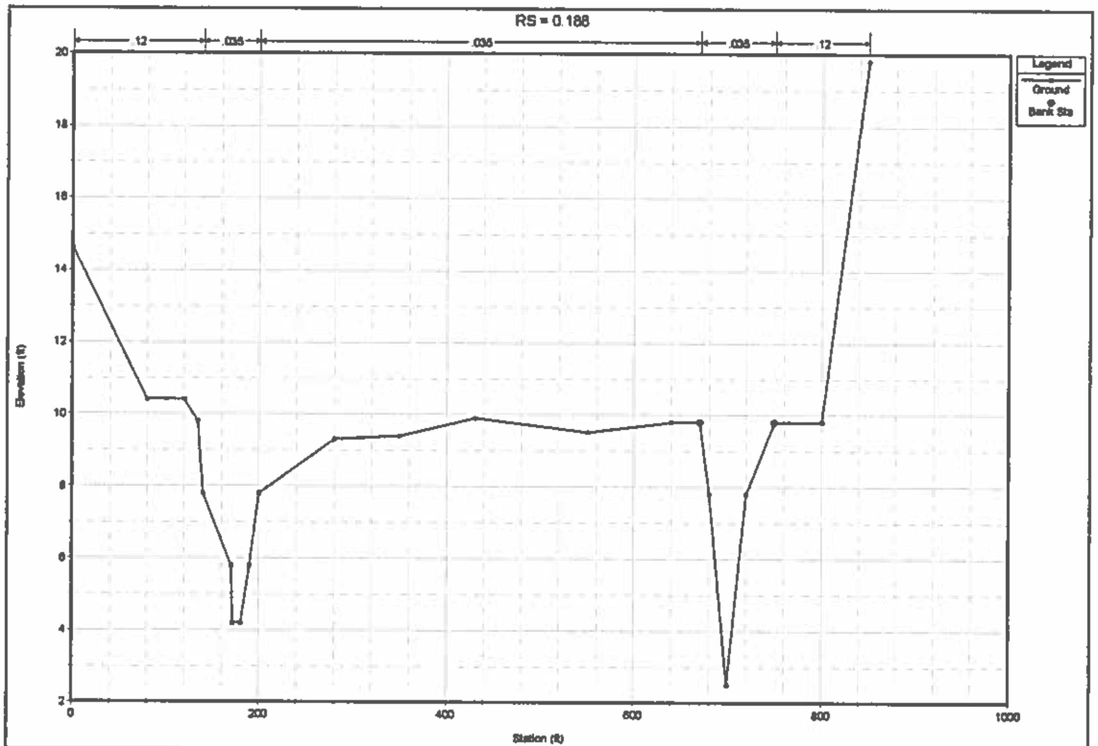
0
200
400
600

South Channel cross-sections (from downstream to upstream)



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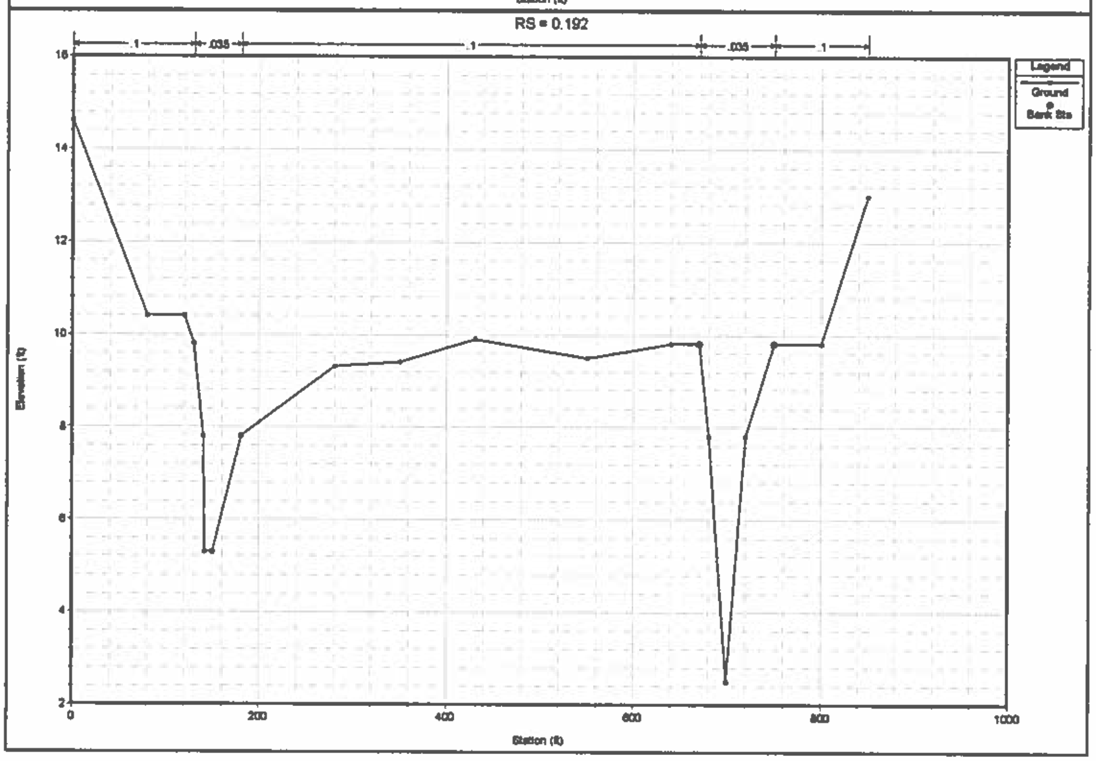
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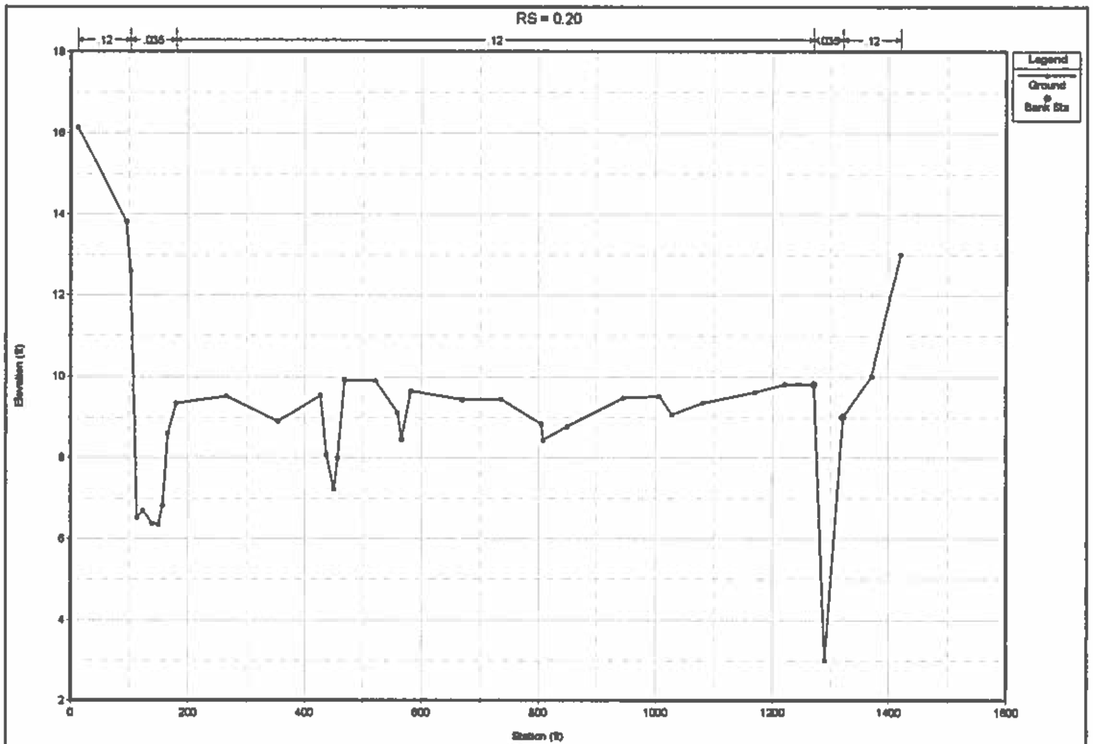
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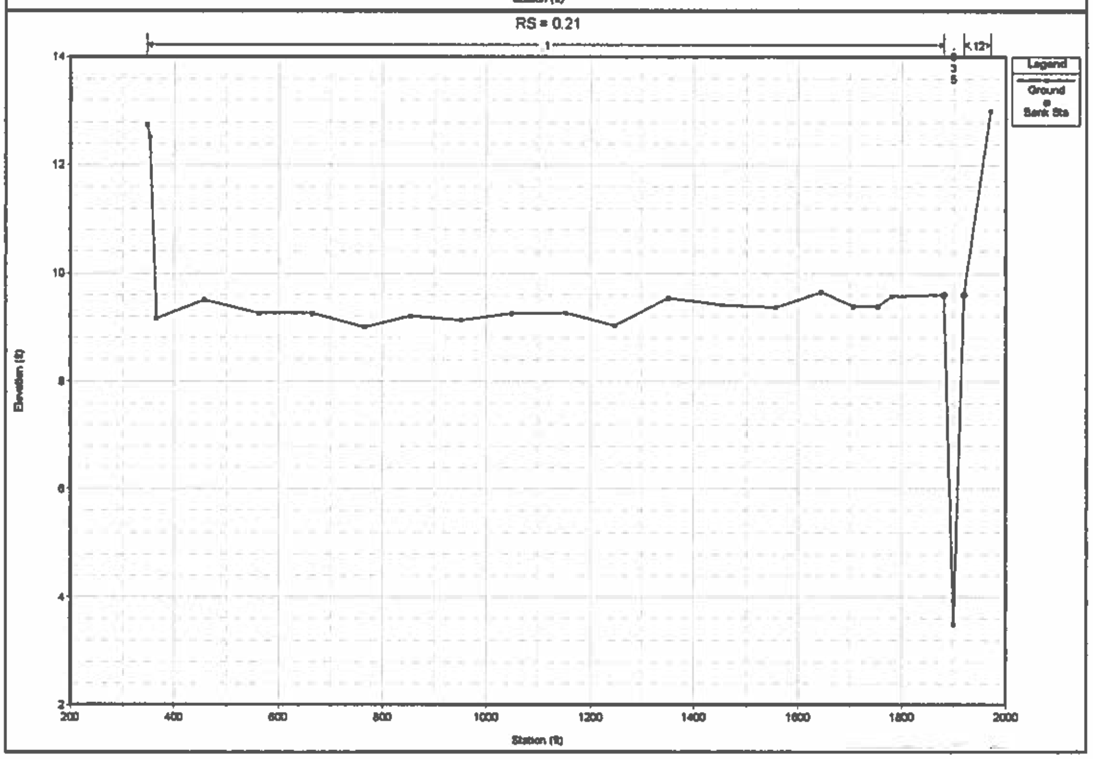
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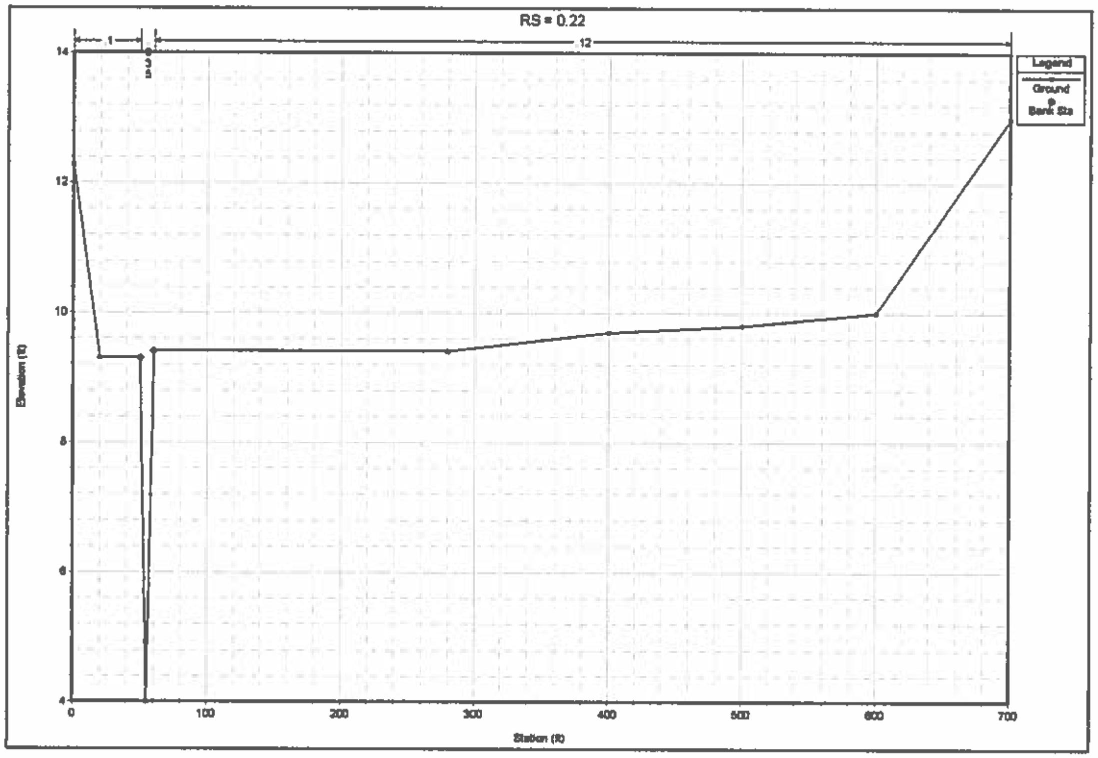
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600
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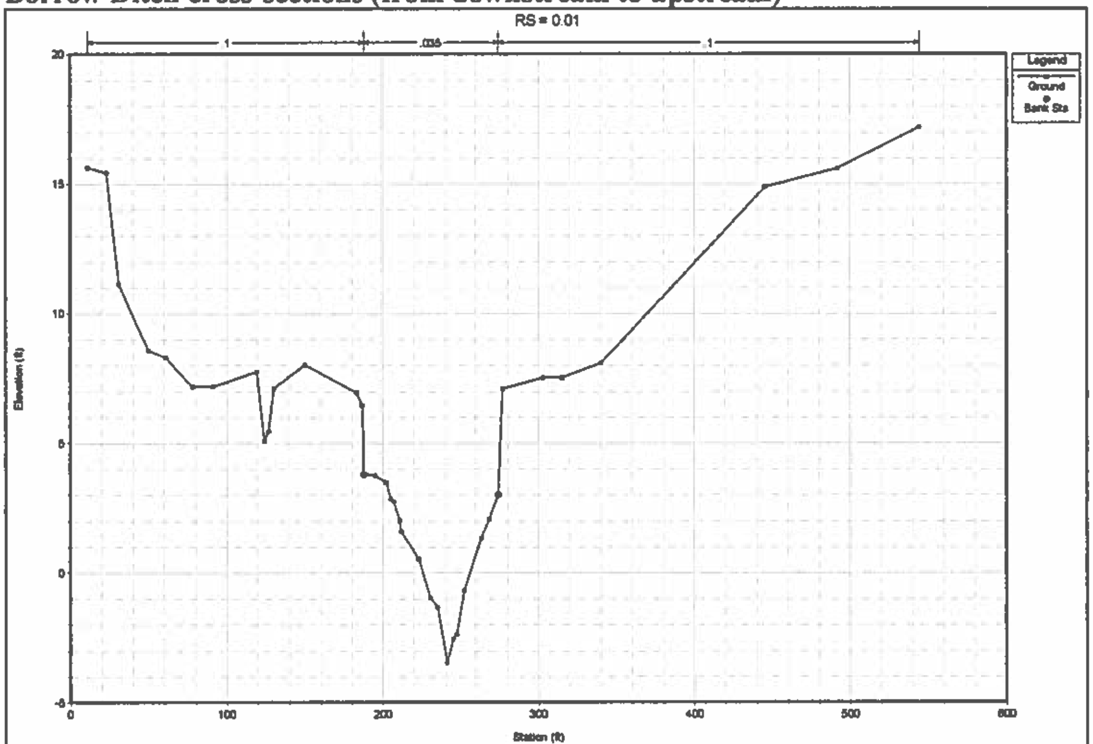
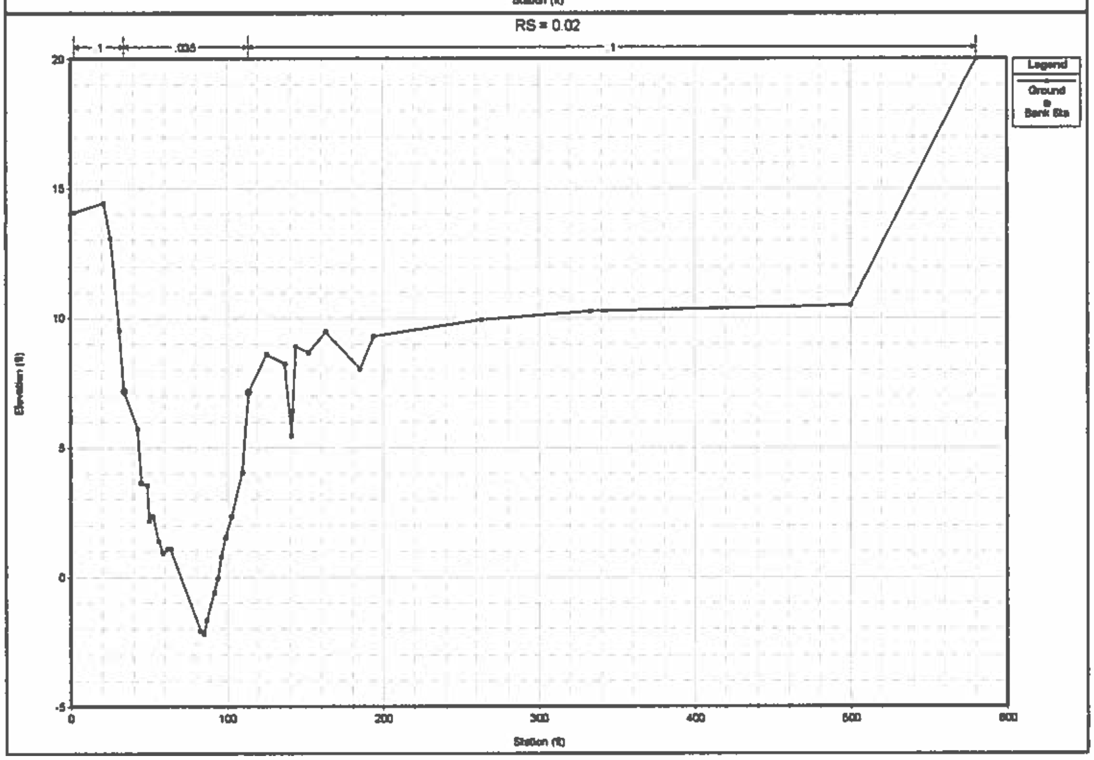
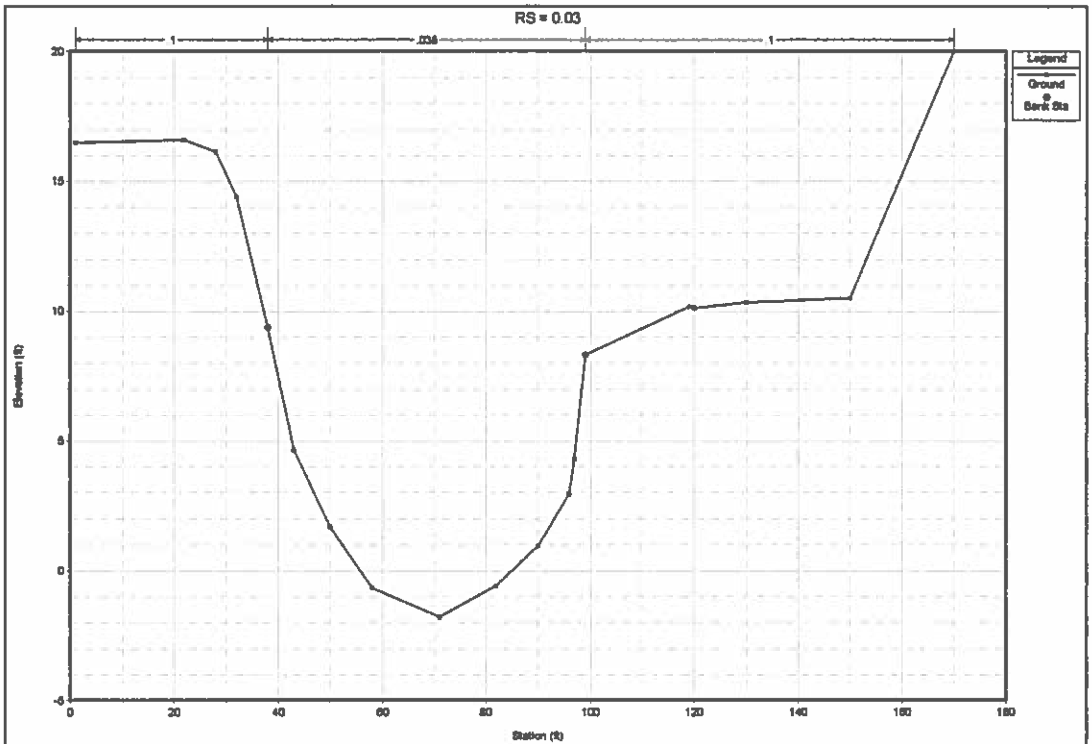
Borrow Ditch cross-sections (from downstream to upstream)



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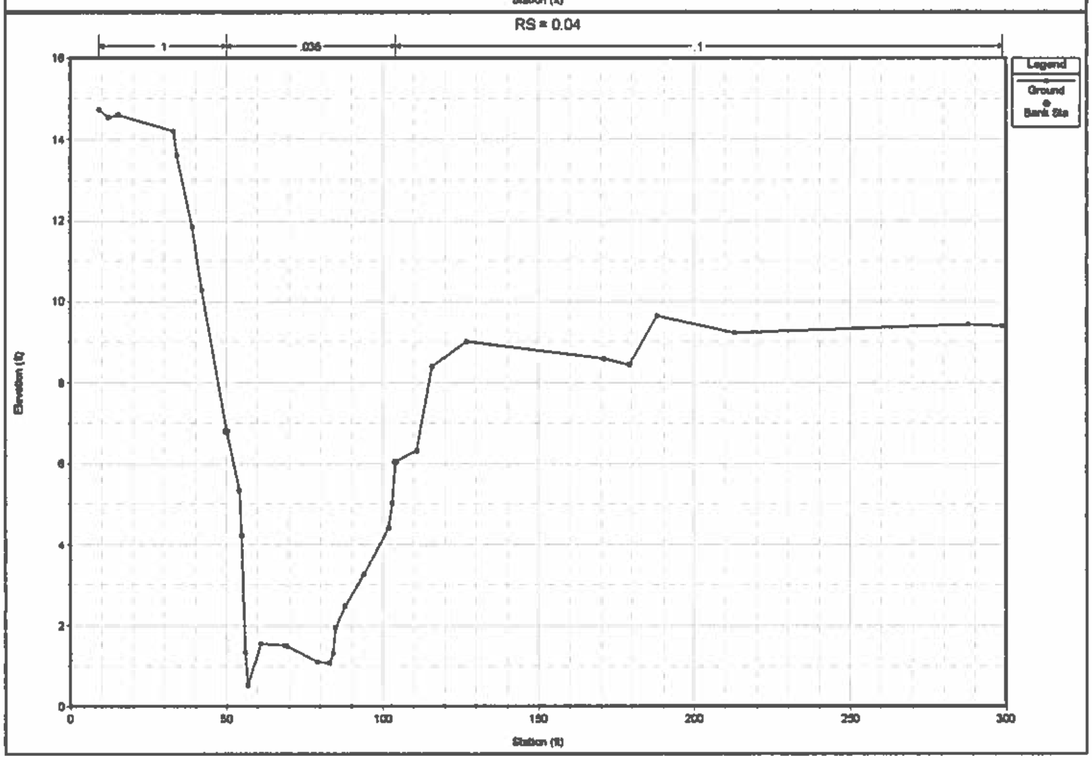
Station (ft)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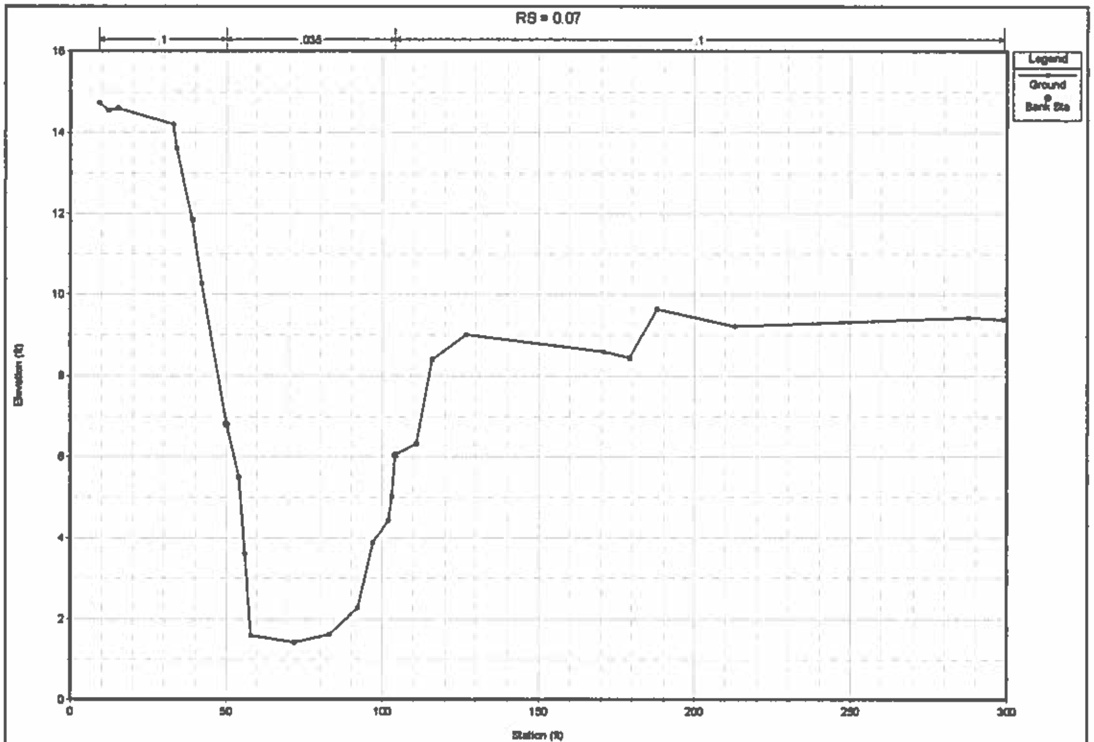

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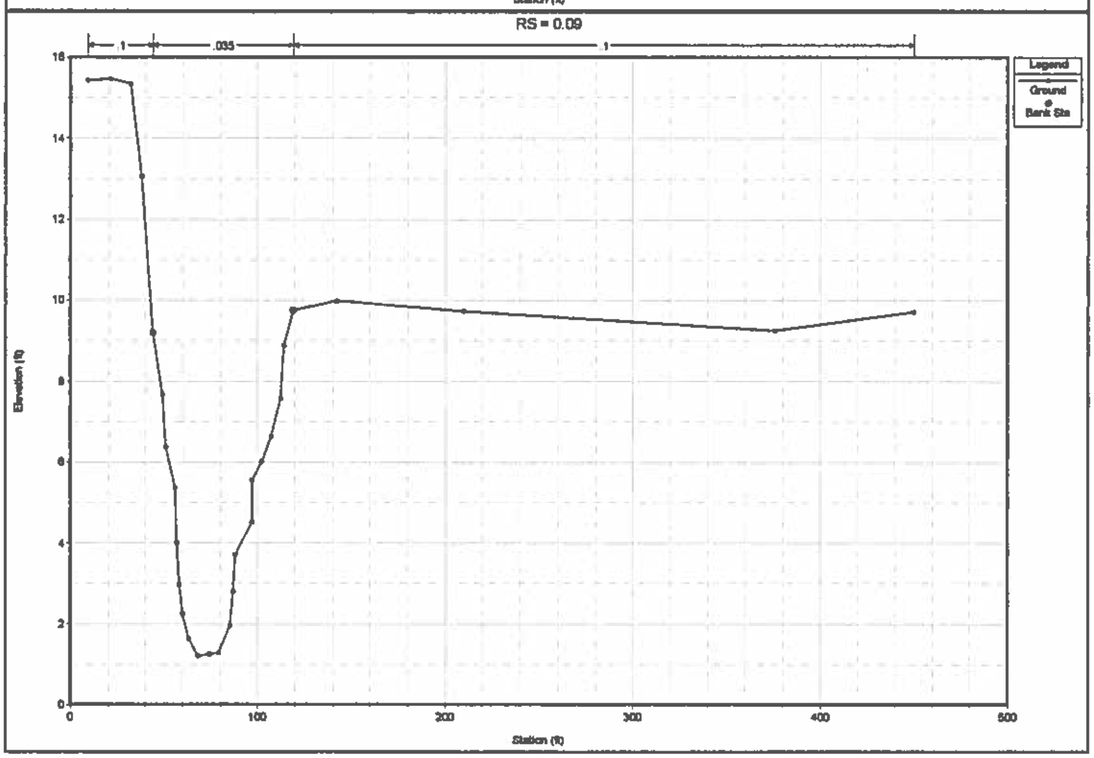

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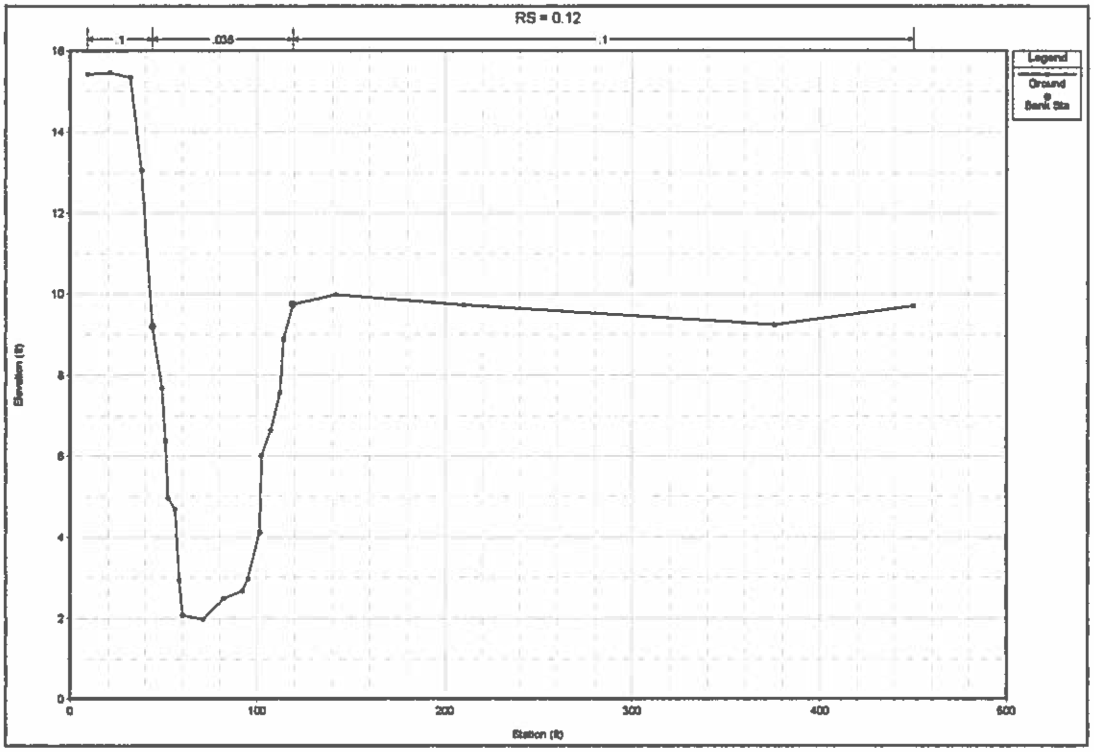
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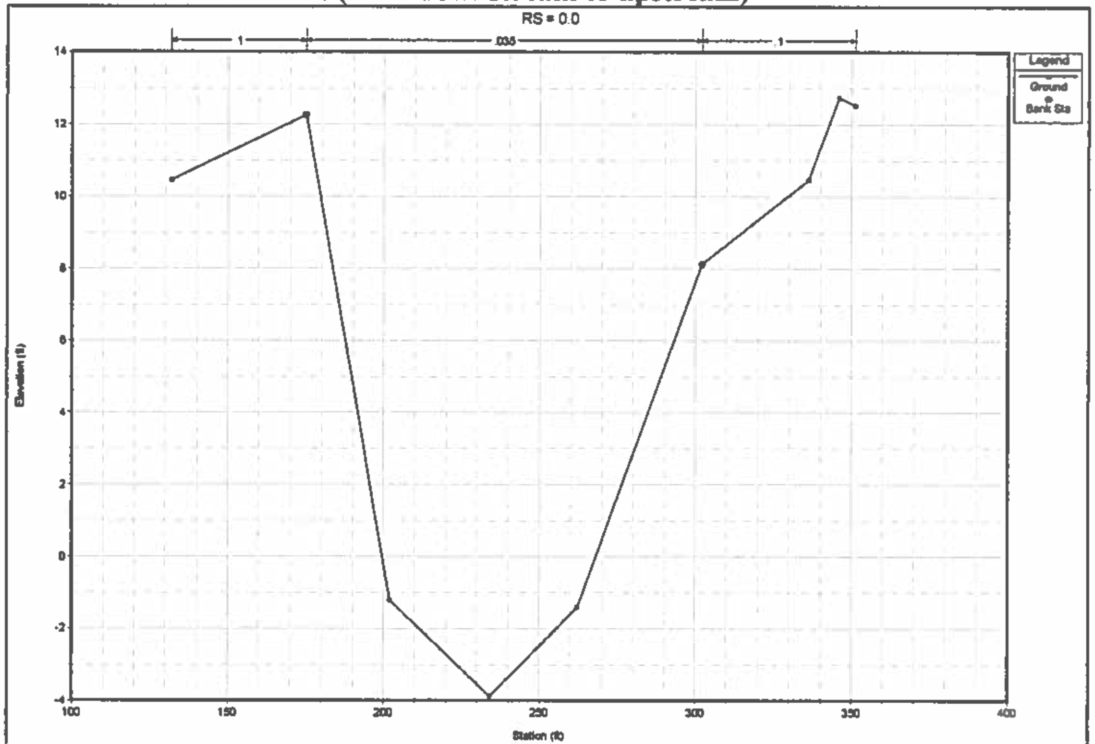
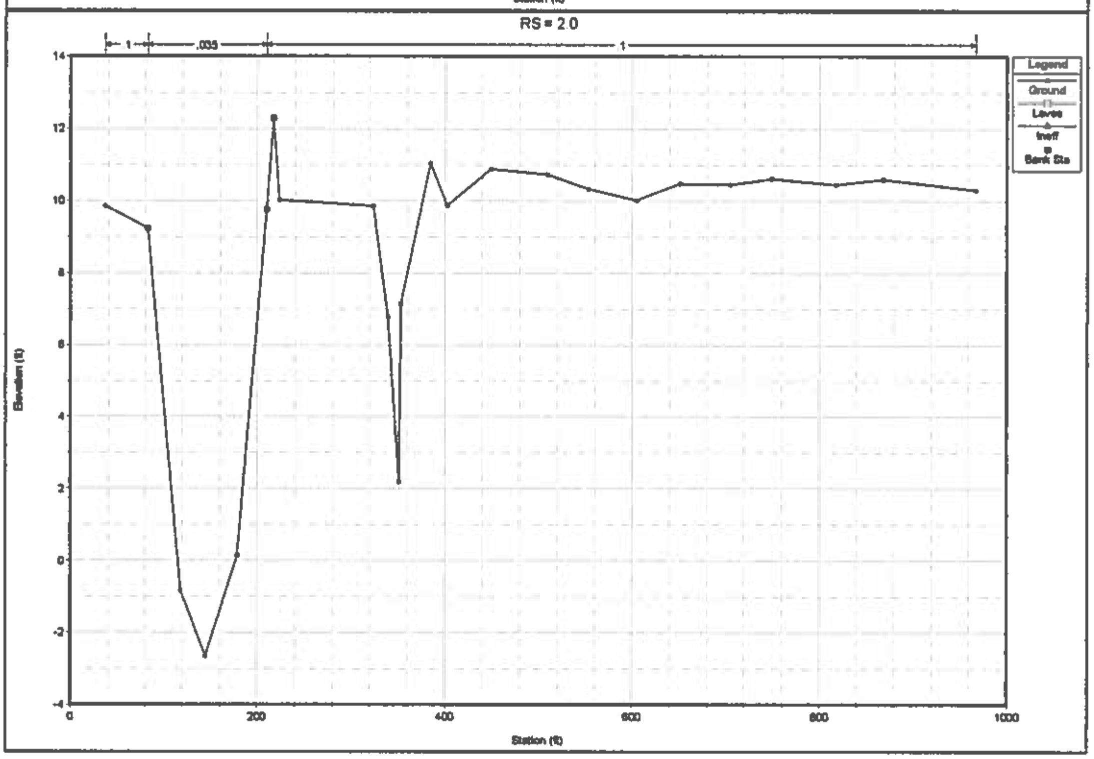
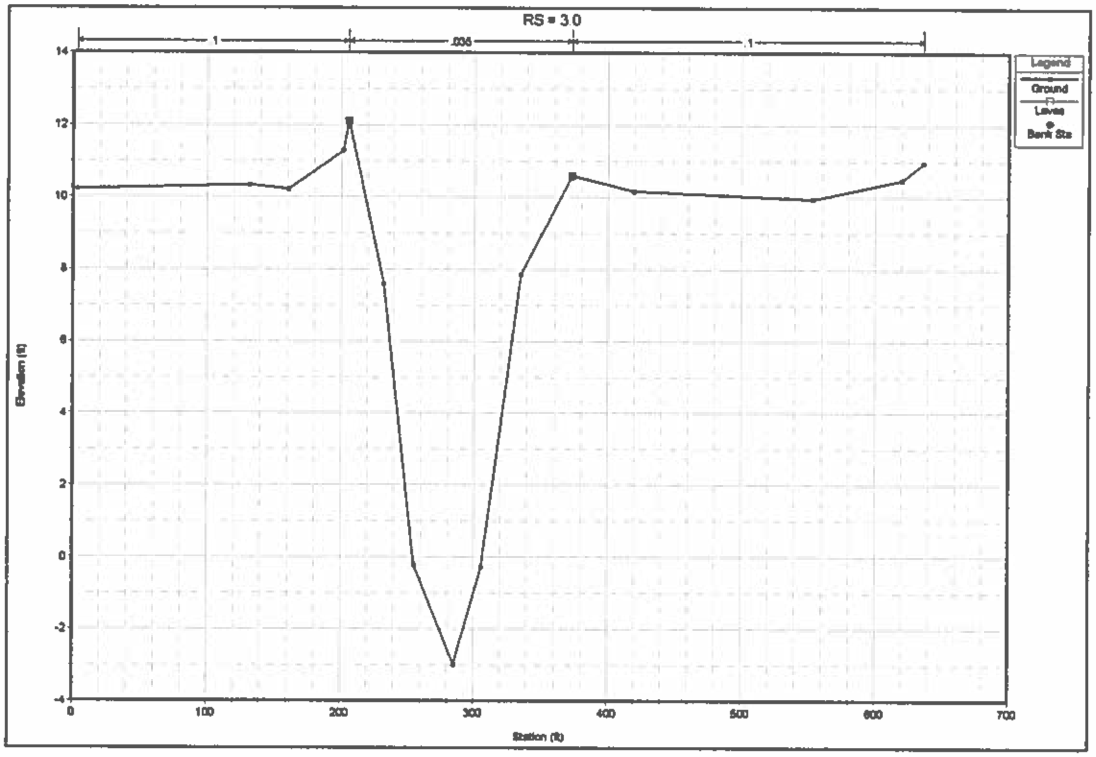
Bear River cross-sections (from downstream to upstream)



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Ground
Levee
Bank Sta
Elevation (ft)
Station (ft)
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-4
0
100
200
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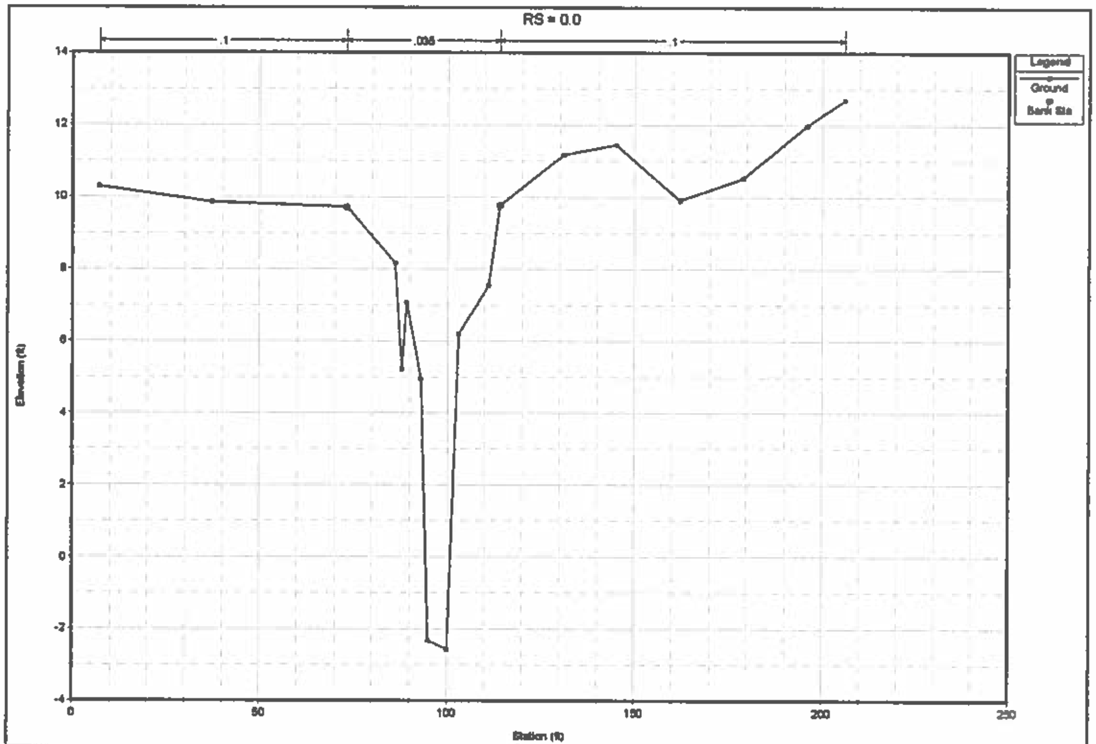
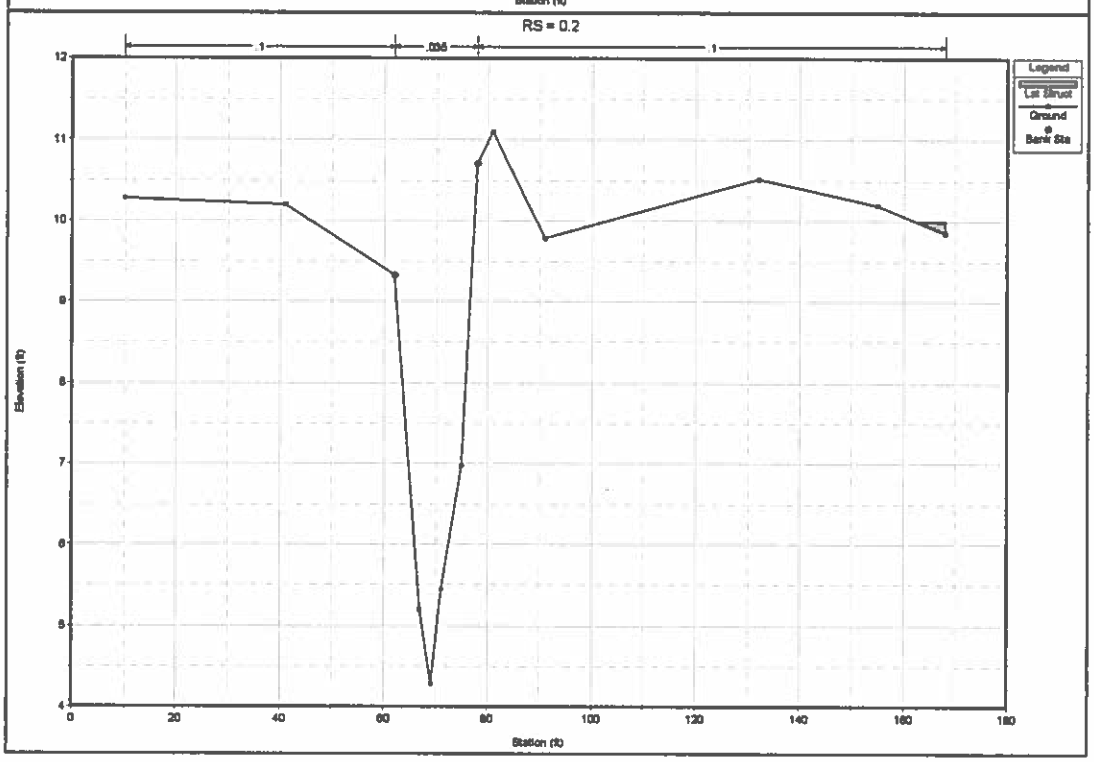
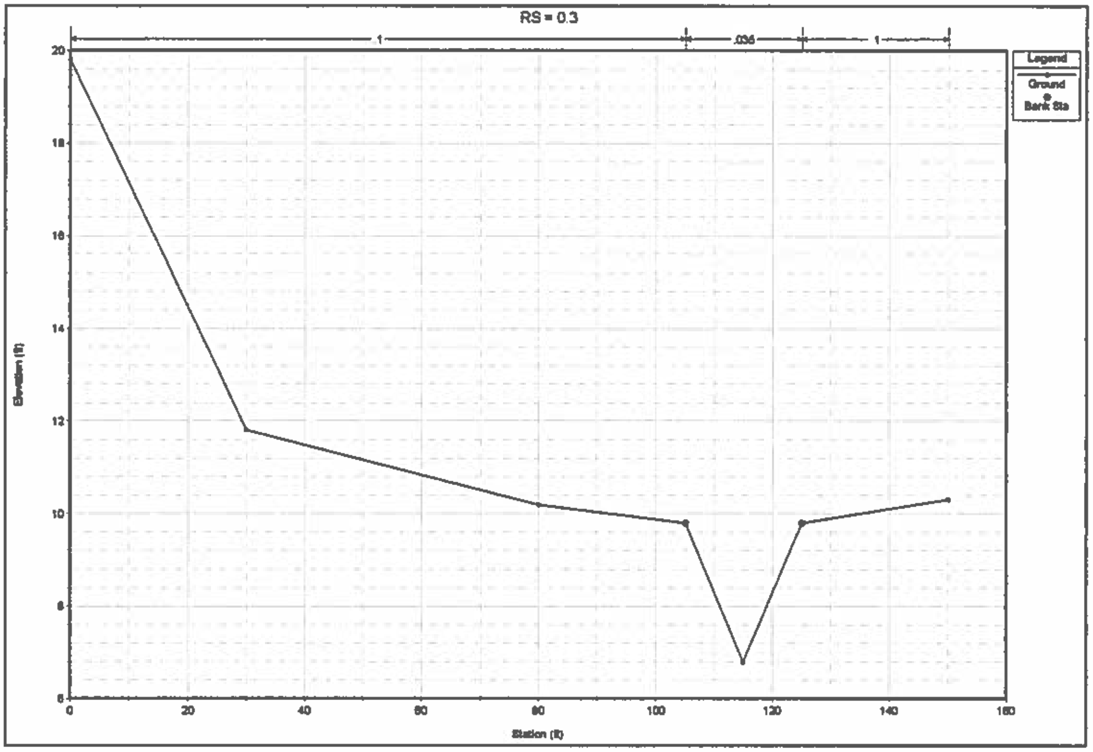
400
500
600
700

No Name Creek cross-sections (from downstream to upstream)



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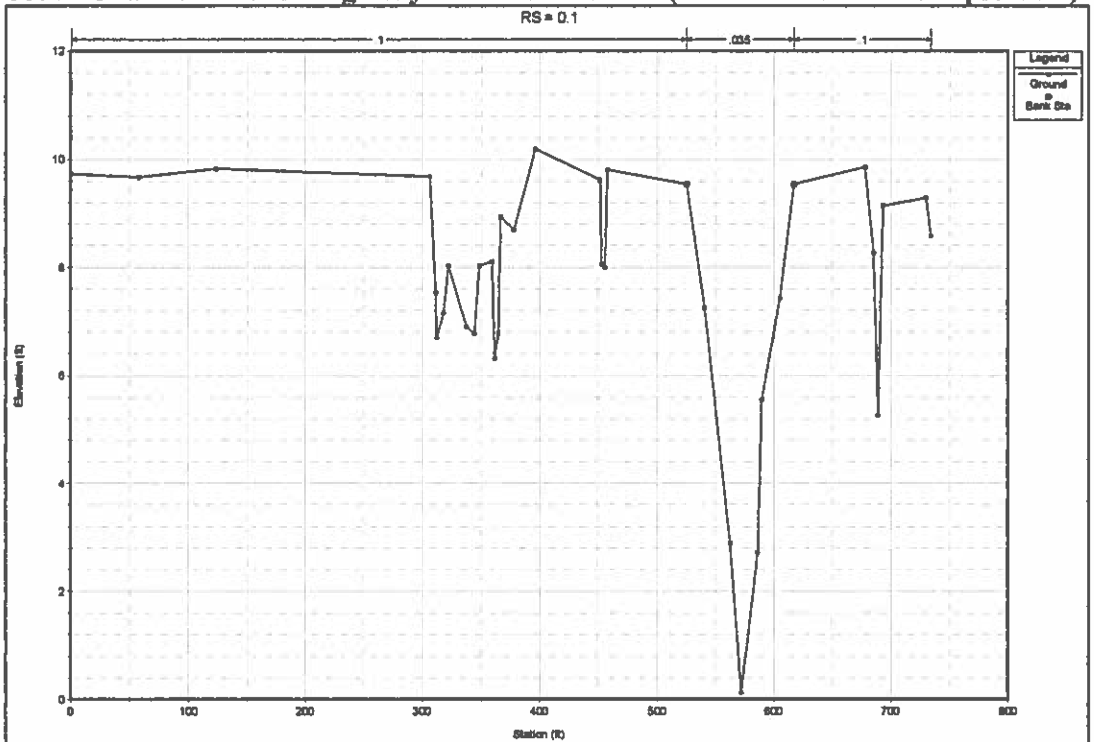
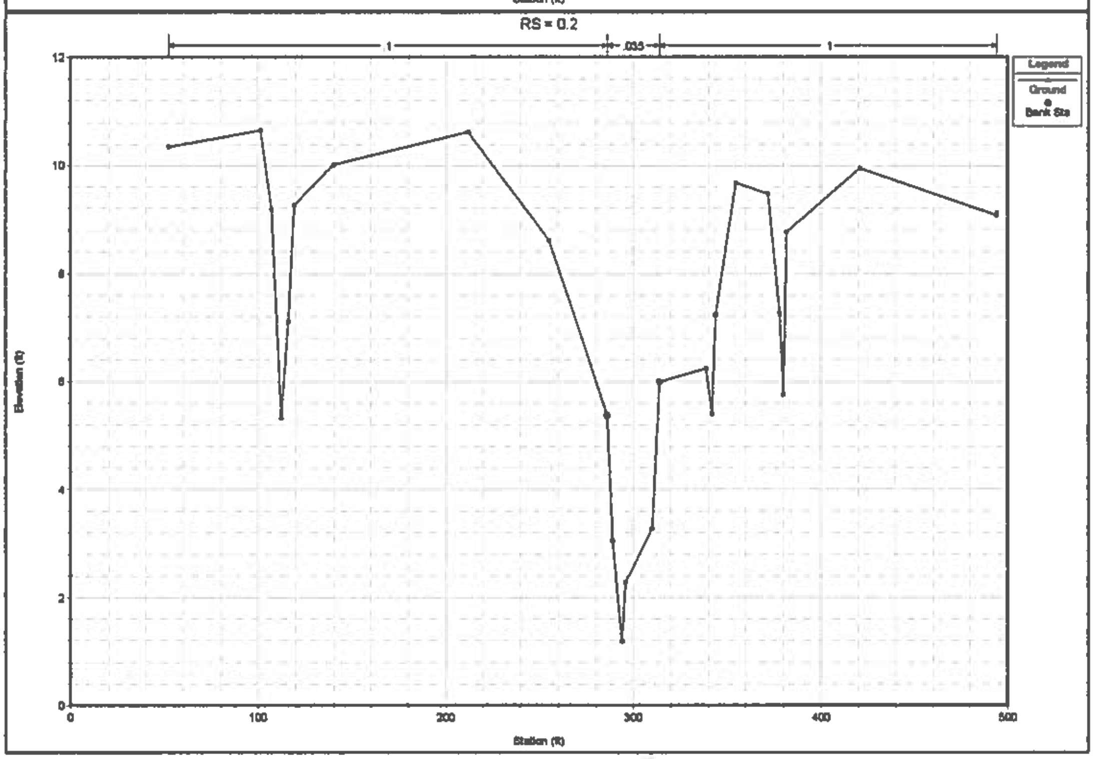
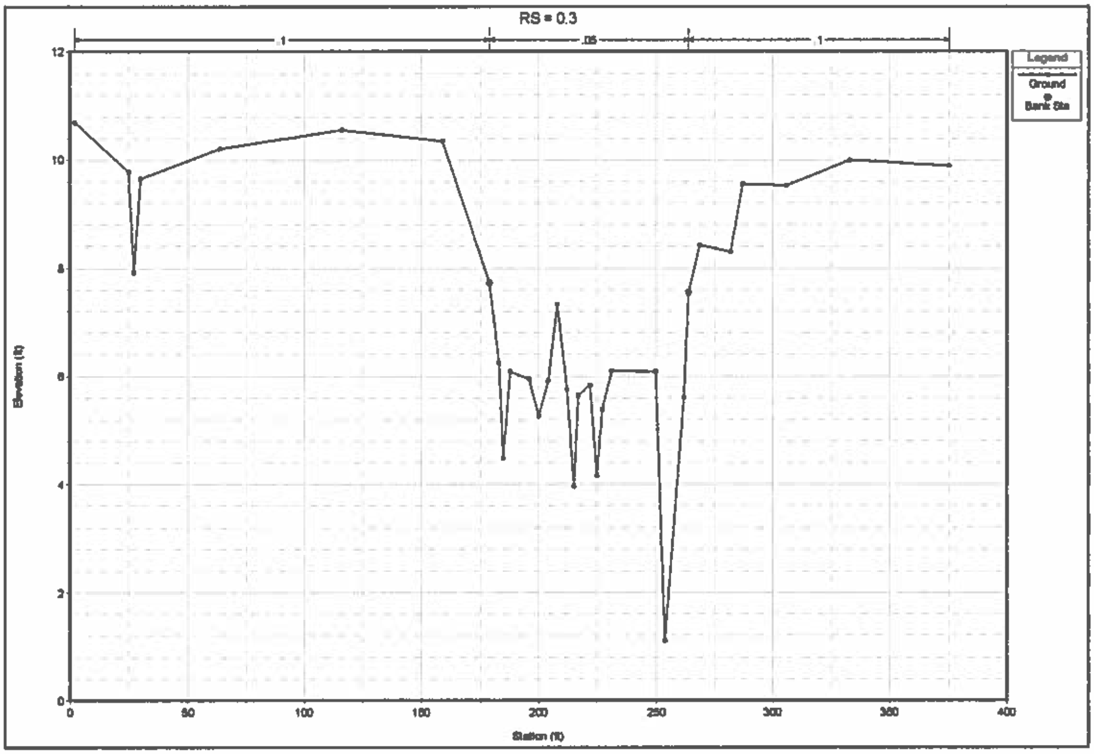
.1
.035
.1
Legend
Ground
Bank Sta
Elevation (ft)
Station (ft)
20
18
16
14
12
10
8
6
0
20
40
60
80
100
120
140
160

South Channel West of Highway 101 cross-sections (from downstream to upstream)






RS = 0.3
.1
.05
.1
Legend
Ground
Bank Sta
Elevation (ft)
Station (ft)
0
50
100
150
200
250
300
350
400
0
2
4
6
8
10
12

APPENDIX B
WATER LEVEL ANALYSIS

This appendix provides a more detailed analysis of the water level measurements from Toke Point tide gauge station (9440910) operated and maintained by the Center for Operational Oceanographic Products and Services, National Oceanic and Atmospheric Administration (COOPS, NOAA). The purpose of the analysis is to develop joint probabilities to estimate the recurrence intervals for extreme water levels induced by tides and storm surges. The appendix also documents the conversion of the long term water level record for Toke Point to the Bear River project site.

Comparison of Toke Point and Bear River water levels

No long term record of water level is available for the project site at Bear River. Tidal constituents developed by NOAA are available at Tarlatt Slough at the south end of Willapa Bay and at Toke Point. The longest relevant series of water level measurements is from the COOPS-NOAA gauge at Toke Point. In this section we compare water level measurements obtained at the project site with predictions at Toke Point and Tarlatt Slough for the purpose of developing a long term water level record for the project site.

Water levels were measured at Greenhead Slough near the mouth of Bear River between May 22 and June 30 2005 by PI engineering. Measured water level data from the Toke Point Tide Gauge was not available for the period of measurement at the time of writing this report, therefore water levels at Toke Point have been predicted using the Harmonic Constants provided by NOAA (Table B-3) for comparison. For this analysis, all data is given in UTC time and uses the NAVD88 vertical datum. To convert from the mean lower low water (MLLW) to the NAVD88 datum at Toke Point, 0.249 meters was subtracted from the MLLW water levels.

A visual examination of the time series of water levels for the two locations indicates they are approximately in-phase with the peaks at Bear River lagging the peaks at Toke Point by around 1 hour. The amplitude at Toke Point is slightly smaller that the amplitude at Bear River (Figure B-1). This is consistent for the 39 days in the measurement interval (Figure B-2). The difference between the daily maximum water level at Bear River and at Toke Point for the measurement interval was found on average to be 1.0656 ft, while the minimum difference was 0.441 ft on June 4 and the maximum difference was 1.64 ft on May 23.

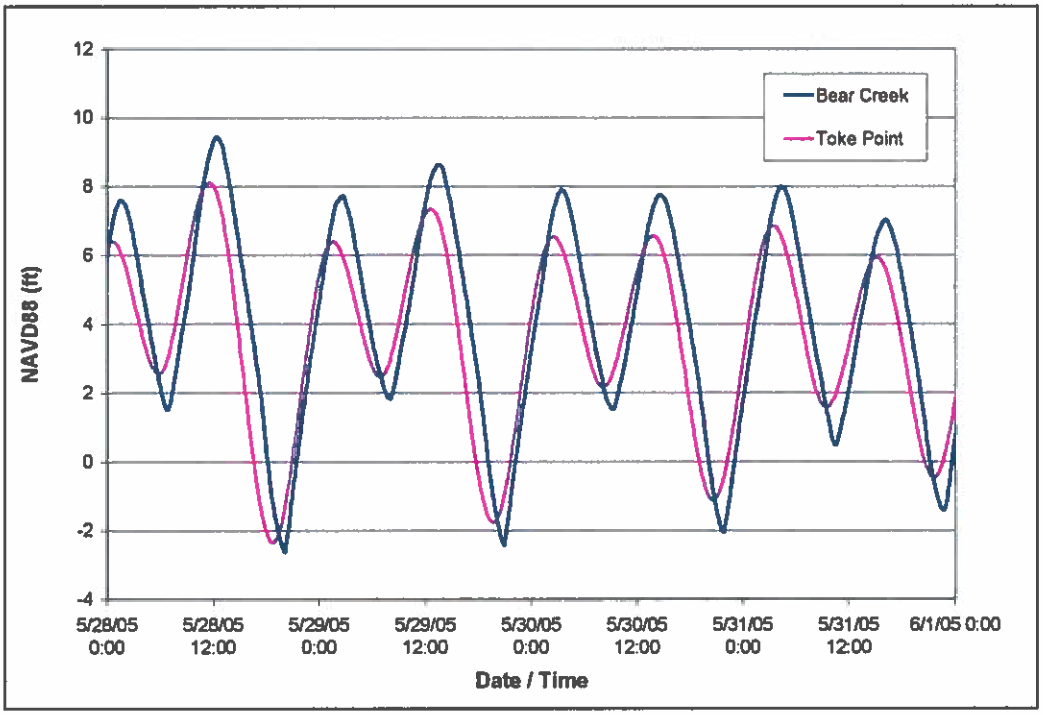


Figure B-1. Water levels at Bear River (measured) and Toke Point (predicted), May 28 to June 1 2005

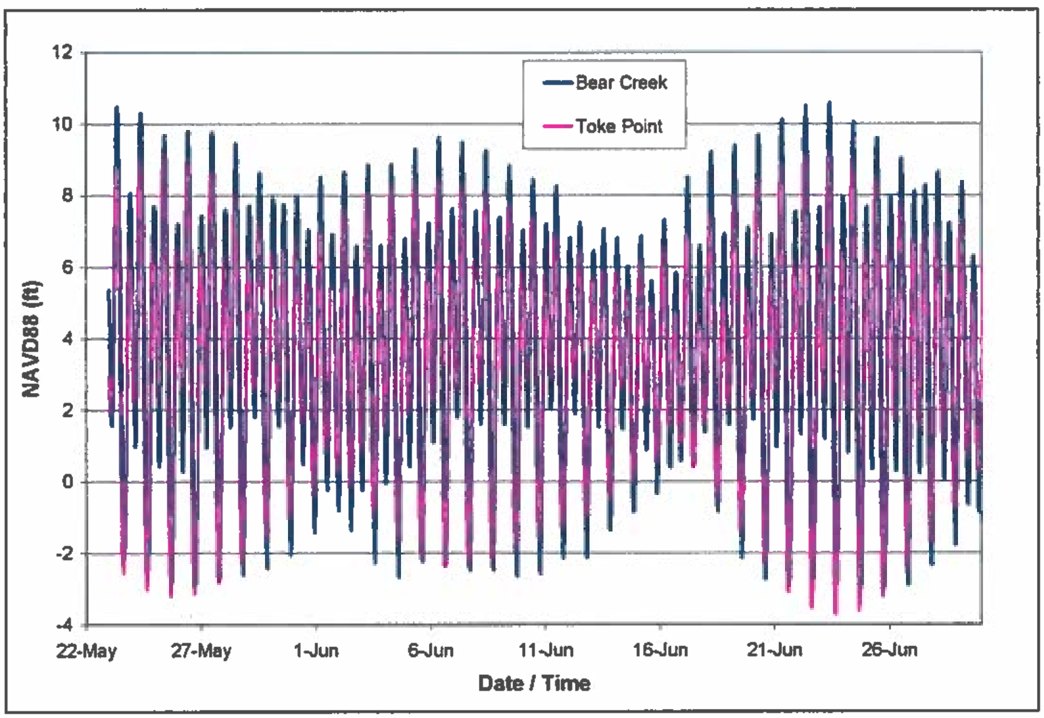


Figure B-2. Water levels at Bear River (measured) and Toke Point (predicted), May 23 to June 30 2005.

Based on the comparison of the measured and predicted time series for the two locations, water levels are transformed from Toke Point (MLLW – meters) to Bear River (NAVD88 – ft) using the following transformation:

(Bear River) = (Toke Point – 0.249) * 3.2808 + 1.0656

0.249 m is the difference between MLLW and NAVD88 datums in m.
3.2808 conversion from meters to feet.
1.0656 ft is added to account for the average difference the in maximum tidal amplitude between Toke Point and Bear River.

In this way, the long term record of water level measurements and predictions at Toke Point can be transformed to the Bear River project site.

Toke Point Measured Anomaly Analysis

Monthly high and low water levels are available from Toke Point gauge 9440910 beginning in January of 1970. Verified 6 minute water levels are available beginning in 1996.

The surge at Toke Point is considered to be the difference between the daily recorded maximum water level and the predicted water level. Figure B-3 shows the difference between the daily maximum surge, and the instantaneous peak surge. The peak surge is not used as it can often be much greater than the daily maximum and present misleading results.

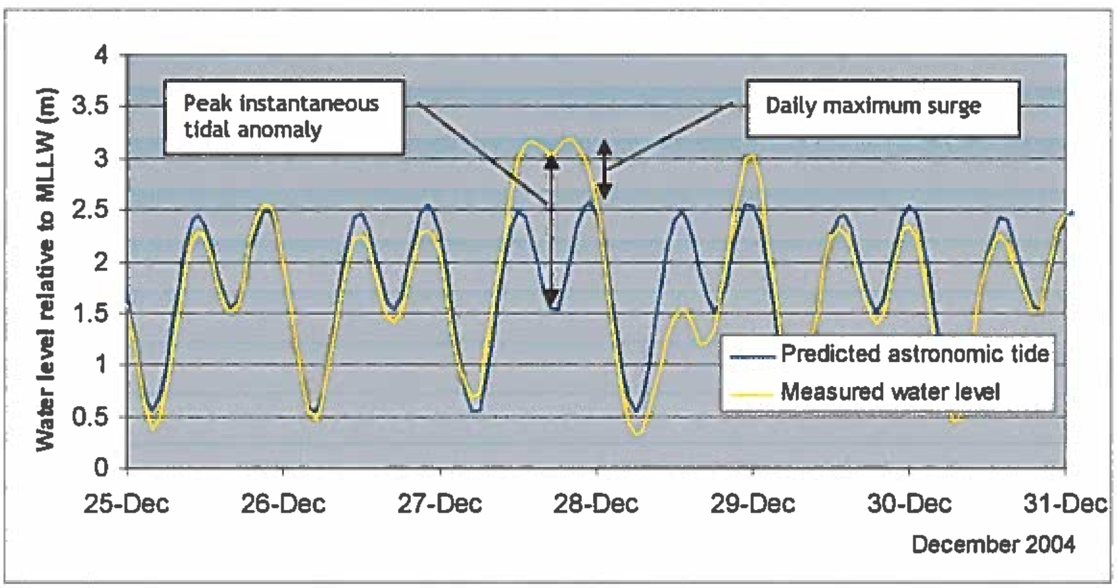


Figure B-3. Example of Daily Maximum Anomaly analysis

Daily measured high water was obtained from the hourly measured high water dataset available for Toke Point on the CO-OPS NOAA website. The predicted hourly water levels were calculated using the NOAA harmonic constants for Toke Point station 9440910. A cumulative distribution function (CDF) of measured surge at Toke Point between 1996 and 2005 is shown in Figure B-4.

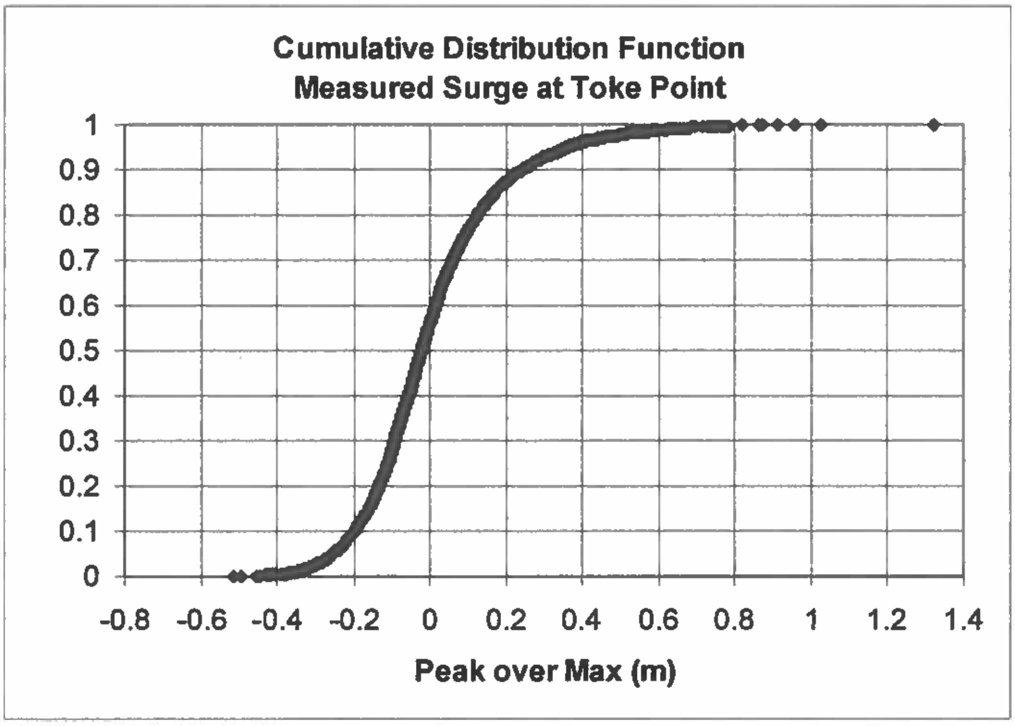


Figure B-4. Cumulative Distribution function for Toke Point surge (1996-2005)

Figure B-4 shows that the surge (difference between measured and predicted water level) is greater than zero 44.1 percent of the time. The data represented here is one value per day for nine years between Jan 1 1996 and Dec 31 2004. Each value shown is the difference between the daily measured high water and the daily predicted high water.

The 90th percentile value for peak of maximum water level difference is 0.243m. The maximum recorded peak over maximum water level was 1.323m. There were 328 events that meet or exceed the 90th percentile peak over maximum water level.

Note, the comparison is between the highest measured and predicted water level each day, but the timing of these events does not necessarily coincide.

An extreme water level analysis was done using the Coastal Engineering Design and Analysis System (CEDAS) for the 328 events above the 90th percentile of peak over maximum water levels yielding the results summarized in Table B-1.

Table B-1 Extreme Surge Analysis

N:	329 NU:	1			
NT:	329 K:	9			
Lambda:	36.56 Mean:	0.41			
Standard deviation:	0.16				
Weibull Distribution					
		FT-I	k=0.75	k=1.00	k=1.40	k=2.0
Correlation:		0.98	0.98	1	0.99	0.96
Sum square of residuals		0.79	2.74	0.13	0.56	1.29
Return Period (Yr)		WL(m)	WL(m)	WL(m)	WL(m)	WL(m)
2		0.85	0.96	0.92	0.86	0.79
5		0.96	1.16	1.06	0.96	0.86
10		1.05	1.31	1.17	1.03	0.91
25		1.16	1.53	1.31	1.12	0.97
50		1.24	1.7	1.42	1.18	1.01
100		1.32	1.87	1.53	1.25	1.05

WARNING: RETURN PERIODS > 27 Yrs may not be meaningful

90% Confidence Interval (Lower Bound - Upper Bound)

Weibull Distribution

Return Period (Yr)	FT-I	k=0.75	k=1.00	k=1.40	k=2.0
5	0.9-1	1-1.3	1-1.2	0.9-1	0.8-0.9
10	1-1.1	1.1-1.5	1.1-1.3	1-1.1	0.9-1
25	1.1-1.2	1.3-1.8	1.2-1.4	1-1.2	0.9-1
50	1.2-1.3	1.4-2	1.3-1.6	1.1-1.3	1-1.1
100	1.2-1.4	1.6-2.2	1.4-1.7	1.2-1.3	1-1.1

Percent chance for water level equalling or exceeding return period

Return Period (Yr)	Period of concern (Yr)				
2	5	10	25	50	
2	75	97	100	100	100
5	36	67	89	100	100
10	19	41	65	93	99
25	8	18	34	64	87
50	4	10	18	40	64
100	2	5	10	22	39

From Table B-1 the 2-yr, 25-yr, and 100-yr surge events are 0.92, 1.31, and 1.53 meters respectively, assuming a shape factor (k) value of 1.0. This estimate may be on the conservative side, as a review of the maximum water level events during the past 35 years shows that the largest surge was just 1.138 meters on March 3, 1999 (see Table B-2). It should be noted here that the values shown in Table A-2 correspond to the highest water levels, which do not necessarily correspond to the peak surges.

Table B-2 Maximum recorded water levels and corresponding surges at Toke Point.

Data recorded from January 1970 to May 2005 (35 years)

Rank	Highest (STND)	Date	Time	High (MLLW)	Predicted (MLLW)	SURGE (m)
1	5.77	19811114	13:30	4.392	3.293	1.099
2	5.605	19731211	13:18	4.227	3.401	0.826
3	5.526	19990303	10:00	4.148	3.01	1.138
4	5.45	19821203	13:54	4.072	3.306	0.766
5	5.41	20011201	20:48	4.032	3.163	0.869
6	5.405	20030102	19:48	4.027	3.322	0.705
7	5.389	19830127	10:42	4.011	3.324	0.687
8	5.367	19780207	0:00	3.989	3.281	0.708
9	5.331	19730118	0:00	3.953	3.325	0.628
10	5.325	19990129	19:00	3.947	2.876	1.071

Table B-3 Table of Harmonic Constants for Toke Point (9440910).

#	Name	Ampl	Epoch	Speed
1	M2	0.981	253.7	28.9841042
2	S2	0.261	284.4	30.0000000
3	N2	0.198	229.7	28.4397295
4	K1	0.435	251.1	15.0410686
5	M4	0.014	352.6	57.9682084
6	O1	0.264	235.4	13.9430356
7	M6	0.009	45.3	86.9523127
8	MK3	0.004	10.9	44.0251729
9	S4	0.000	0.0	60.0000000
10	MN4	0.006	35.6	57.4238337
11	NU2	0.044	228.6	28.5125831
12	S6	0.000	0.0	90.0000000
13	MU2	0.008	253.2	27.9682084
14	2N2	0.022	204.6	27.8953548
15	OO1	0.016	292.6	16.1391017
16	LAM2	0.009	269.3	29.4556253
17	S1	0.006	40.7	15.0000000
18	M1	0.016	270.3	14.4966939
19	J1	0.023	267.6	15.5854433
20	MM	0.000	0.0	0.5443747
21	SSA	0.000	0.0	0.0821373
22	SA	0.158	289.8	0.0410686
23	MSF	0.000	0.0	1.0158958
24	MF	0.024	161.8	1.0980331
25	RHO	0.011	208.7	13.4715145
26	Q1	0.047	230.2	13.3986609
27	T2	0.017	270.5	29.9589333
28	R2	0.002	285.6	30.0410667
29	2Q1	0.006	221.0	12.8542862
30	P1	0.136	249.2	14.9589314

31 2SM2	0.000	0.0	31.0158958
32 M3	0.000	0.0	43.4761563
33 L2	0.033	266.3	29.5284789
34 2MK3	0.005	64.3	42.9271398
35 K2	0.071	279.6	30.0821373
36 M8	0.000	0.0	115.9364166
37 MS4	0.009	30.5	58.9841042

Amplitudes are in Meters
Phases are in degrees, referenced to UTC (GMT)
Latitude: 46° 42.5' N Longitude: 123° 57.9' W
9440910 TOKE POINT, WILLAPA BAY , WA