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Expert Report of Dr. Robert A. Mussetter

Navigability of Mosquito Fork River
Alaska v. United States et al.
Civil Action No: 3:12-CV-00114-SLG

TETRA TECH

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1. Opinions to be Expressed

Based on my understanding of the Federal rules regarding navigability coupled with my evaluation of the available data and information, including my observations during field reconnaissance trips that were conducted in July and August 2013, detailed analysis of data collected by a field crew working under my direction in August 2013, and my knowledge of stream hydrology and hydraulics, fluid mechanics, fluvial geomorphology, boat characteristics and behavior, information obtained from the U.S. expert historian, as well as the information contained in the various expert reports submitted by the State of Alaska, I have formed the following opinions regarding navigability of the disputed reach of the Mosquito Fork River at the time of Alaska's statehood:

1. The disputed reach of the Mosquito Fort River has not changed in a manner that would affect its navigability since January 1959 when Alaska became a state.

2. The approximately 36-mile segment of the disputed reach from about 0.25 miles downstream from the Taylor Highway Bridge to the mouth of Ketchumstuck Creek (RM 3.3 to RM 36.3) and the approximately 5-mile segment between RM 55 and RM 60, just downstream from the area known as Mosquito Flats, contains numerous shallow riffles and rapids that would be significant impediments to navigation under certain flow conditions.

3. Evidence provided by both the United States expert historian and the State's historian for this case indicate that the most likely boats being used for trade and travel on smaller rivers in the Yukon and Tanana River drainages at the time of Alaska's statehood are poling boats and motorized riverboats with total length in the range of 20 to 30 feet. Further, it is the opinion of the United States' expert historian that it would have been necessary for these boats to carry cargo loads of at least one ton to make navigation *a commercial reality* (PPL Montana, LLC v Montana, hereinafter referred to as PPL Montana, p 24).

4. The typical motorized riverboat shown in the drawing provided by the State's historian would have required a minimum draft of 13 to 16 inches when carrying cargo loads of 2,000 to 3,000 lb.

5. The 19-foot, 8-inch long Chicken poling boat shown in the photographs and drawings provided by the State would have also required a minimum draft of 13 inches to 16 inches when carrying cargo loads of 1,000 lb to 2,000 lb.

6. The calibrated hydraulic models for eight study sites that are representative of the numerous shallow riffles and rapids in the segments of the disputed reach discussed in Opinion 2 demonstrate that the minimum discharge required to operate these boats with 1,000- to 2,600-lb loads is as follows:

   a. Poling boat carrying 1,000-lb cargo load in relatively placid water (8-inch minimum draft): 180 cfs to 950 cfs
   b. Poling boat carrying 2,000-lb cargo load in relatively placid water (12-inch minimum draft): 360 cfs to 1,740 cfs
   c. Motorized river boat carrying 2,500- to 2,600-lb cargo load (15-inch minimum draft): 590 to 2,230 cfs.

7. Based on the discharge records for the stream gage located at the Taylor Highway Bridge at the downstream end of the disputed reach, supplemented by flows estimated from equations published by the U.S. Geological Survey and Alaska Department of Transportation, the

segments of the disputed reach discussed in Opinion 2 would have been boatable less than half the time during the open-water season from May 1 through September 1 for even the small poling boat carrying only a 1,000-lb cargo load that is too small to be *a commercial reality*, and these conditions would have occurred during an average of five different, unpredictable periods throughout the open-water season each year for durations of about 12 days.

a. When carrying a 2,000-lb cargo load, the small poling boat would have been able to traverse the reach only about 20 percent of the time during the open-water season, and boatable conditions would have also occurred during an average of five discrete, unpredictable periods each year for durations of 5 days to 6 days each.

b. The motorized riverboat carrying an approximately 2,500-lb cargo load would have only been able to traverse the reach about 10 percent of the time, and boatable conditions would have occurred during an average of 3 discrete, unpredictable periods each year for durations of 5 days to 6 days each.

8. Based on the above information, it is my opinion that the approximately 36-mile segment of the disputed reach of the Mosquito Fork River from about 0.25 miles downstream from the Taylor Highway Bridge to the mouth of Ketchumstuck Creek (RM 3.3 to RM 36.3) and the approximately 5-mile segment between RM 55 and RM 60, just downstream from the area known as Mosquito Flats, was not navigable under the Federal test spelled out in the Daniel Ball case and clarified in subsequent rulings, including the U.S. Supreme Court Ruling in PPL Montana.

2. Basis and Reasons for the Opinions, including Data and Information Relied Upon and Considered in Forming the Opinions

2.1. Introduction

On June 1, 2012, the State of Alaska filed a Complaint to Quiet Title and For Declaratory Judgment for submerged lands underlying the Mosquito Fork of the Fortymile River (Mosquito Fork) based on the assertion that the river is navigable, and therefore, the lands are owned by the State. Under this action, the State specifically claims ownership of *the submerged lands and bed up to and including the ordinary high water lines of the right and left banks of the Mosquito Fork River from its confluence with the Dennison Fork…upstream to just above its confluence with Wolf Creek…, except for those portions of the river that traverse state-owned uplands*[1] (**Figure 1**).

The claims are being made under the "equal footing doctrine" that guarantees newly-admitted states the same rights enjoyed by the original thirteen states and the previously-admitted states, including ownership of lands underlying navigable waters. The standard that is generally applied to determine whether a river was navigable at the date of statehood was set out in The Daniel Ball case (10 Wall. 557, 563), where the court said the following:

> *Those rivers must be regarded as public navigable rivers in law which are navigable in fact. And they are navigable in fact when they are used, or are susceptible of being used, in their ordinary condition, as highways for commerce, over which trade and travel are or may be conducted in the customary modes of trade and travel on water.*

I was retained by the U.S. Department of Justice (DOJ) to provide an opinion as to whether the disputed reach of the Mosquito Fork was susceptible to navigation by customary modes of trade and travel in use at the time of Alaska's statehood (January 3, 1959). In developing this opinion, I performed a series of tasks that included the following:

1. Assessed the hydrologic, hydraulic and geomorphic conditions of the Mosquito Fork in the disputed reach,
2. Assessed whether the characteristics of the disputed reach have been altered from the characteristics that existed on January 3, 1959, when Alaska became a state, through artificial or natural means in a manner that would affect its navigability,
3. Consulted with C. Michael Brown, the expert historian retained by DOJ to assist in this case, regarding the types and characteristics of the boats that were in customary use for trade and travel for commerce in the Upper Yukon River system, the region of Interior Alaska in which Mosquito Fork is located.
4. Using the results from the above tasks, formed an opinion as to whether the disputed reach is currently susceptible to navigation by modes of trade and travel in use at the time of Alaska's statehood, and because it has not changed in a manner that would affect its

[1]The portion of the river subject to the State's claims will be referred to throughout the remainder of this report as the "disputed reach". The excluded portions represent approximately 10.5 miles of the total 80.5-mile reach between the confluence with the Dennison Fork and the mouth of Wolf Creek.

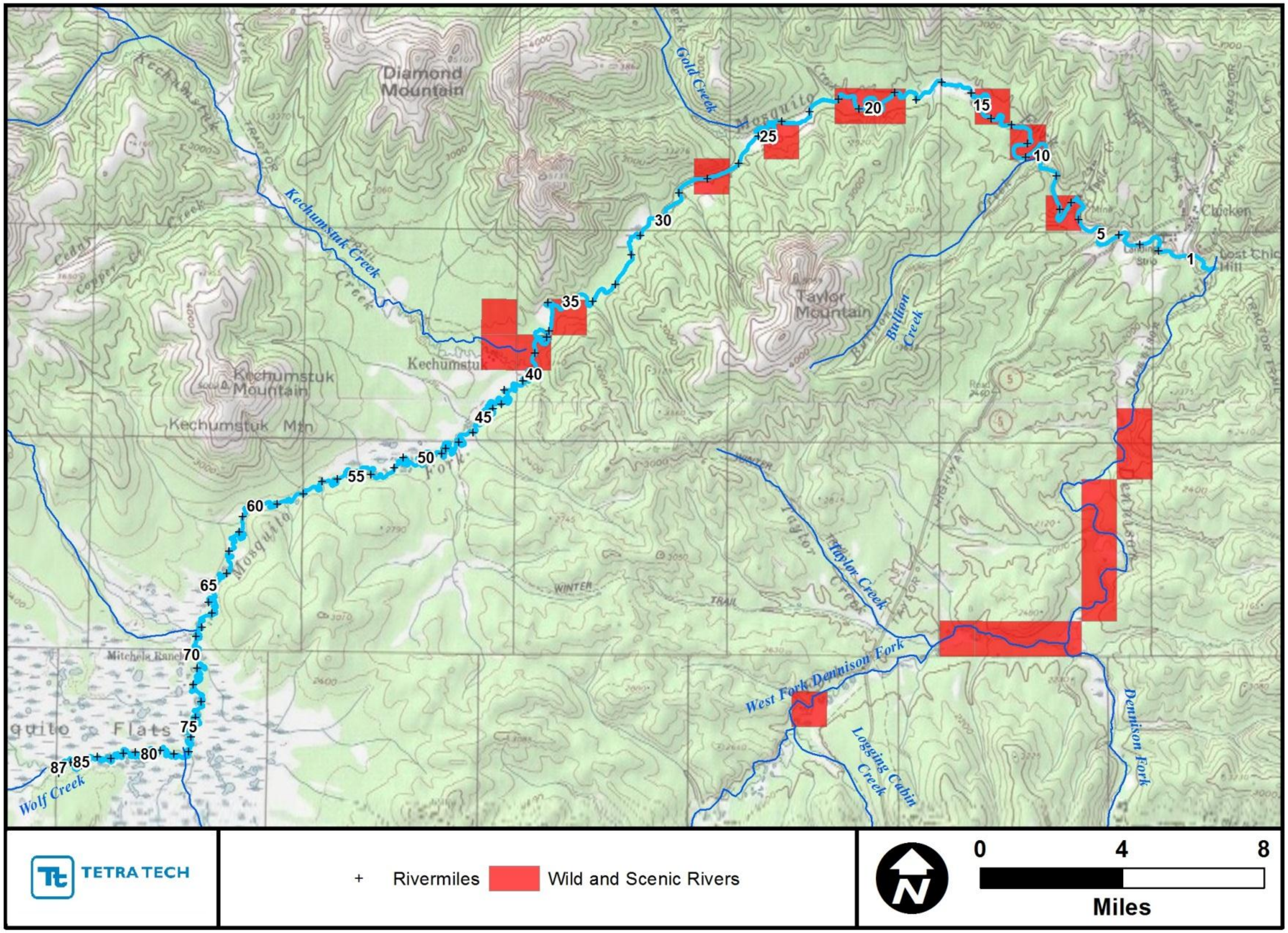


Figure 1. Vicinity map of Mosquito Fork River study reach.

navigability since 1912, whether it was navigable in its ordinary and natural conditions at the date of Alaska's statehood,

5. Reviewed the expert reports and other information provided by the State of Alaska.

The above assessments relied on historical aerial photographs, published information about the character of the river, direct observations of the river throughout the disputed reach, and detailed field data that were collected under my supervision at nine (9) sites within the disputed reach in August 2013 (**Figure 2**). Where appropriate, I have also provide comments or rebuttal of information contained in the State's expert reports.

2.2. Field Data Collection Procedures

An initial reconnaissance of the disputed reach was conducted by helicopter on July 16, 2013. The reconnaissance included an over-flight of the entire reach from Wolf Creek downstream to the confluence with the Dennison Fork and a second upstream pass from Chicken to about the mouth of Ketchumstuk Creek. During the reconnaissance, stops were made at four locations [River Mile (RM) 18.2, RM 24.2, RM 29.3 and RM 37.2] to provide an opportunity to assess conditions on the ground. Information from the field reconnaissance, available topographic mapping, and aerial photography were used to identify locations that could potentially limit navigability. These locations primarily consisted of riffles or rapids where the flows appeared to be very shallow and/or the bed was strewn with large boulders that could create a significant navigation hazard for the types of boats that were in customary use in this area at the time Alaska became a state. With the exception of the most upstream site, the detailed study sites were selected from among the numerous riffles and rapids in the downstream approximately 60 miles of the disputed reach to represent the range of conditions that could potentially limit navigability. The upstream site was selected to represent conditions in the relatively flat-gradient, meandering reach within Mosquito Flats.

In the context of this dispute, it is also significant to note that the initial reconnaissance was intended to include a float trip through the disputed reach; however, it was necessary to cancel the trip because the very low flows in the river would have made it necessary to drag the boats for extended portions of the reach. A second attempt to float the reach was planned for the week of June 2, 2014. Although all of the logistics for this trip were in place, this trip was also cancelled at the last minute due to the low flows that would have prevented effective navigation of the reach, even with a small, modern raft.

Detailed surveys of the selected sites were conducted between August 13 and August 17, 2013 to collect data to evaluate the geomorphic characteristics of the sites and to quantify the hydraulic conditions through the potential navigation hazards over the range of flows that occur in the river (**Table 1**). The surveys generally included ten (10) to 13 cross sections laid out perpendicular to the direction of flow at locations that will capture the hydraulic controls and key hydraulic characteristics of the site. (Site P1 included only 7 cross sections because of its relative simplicity.) The cross sections were spaced at 50- to 200-foot intervals, depending on the planform and longitudinal profile at the site. The field work at each site also included collection of one or more surface samples of the bed material in the riffles/rapids, a surface sample from a typical gravel bar and a discharge measurement. The sites were also thoroughly photographed for later use in interpreting and illustrating the data. Maps showing the cross section layout at each site and plots of the individual cross sections are provided in **Appendices A** and **B**, respectively. The maps in

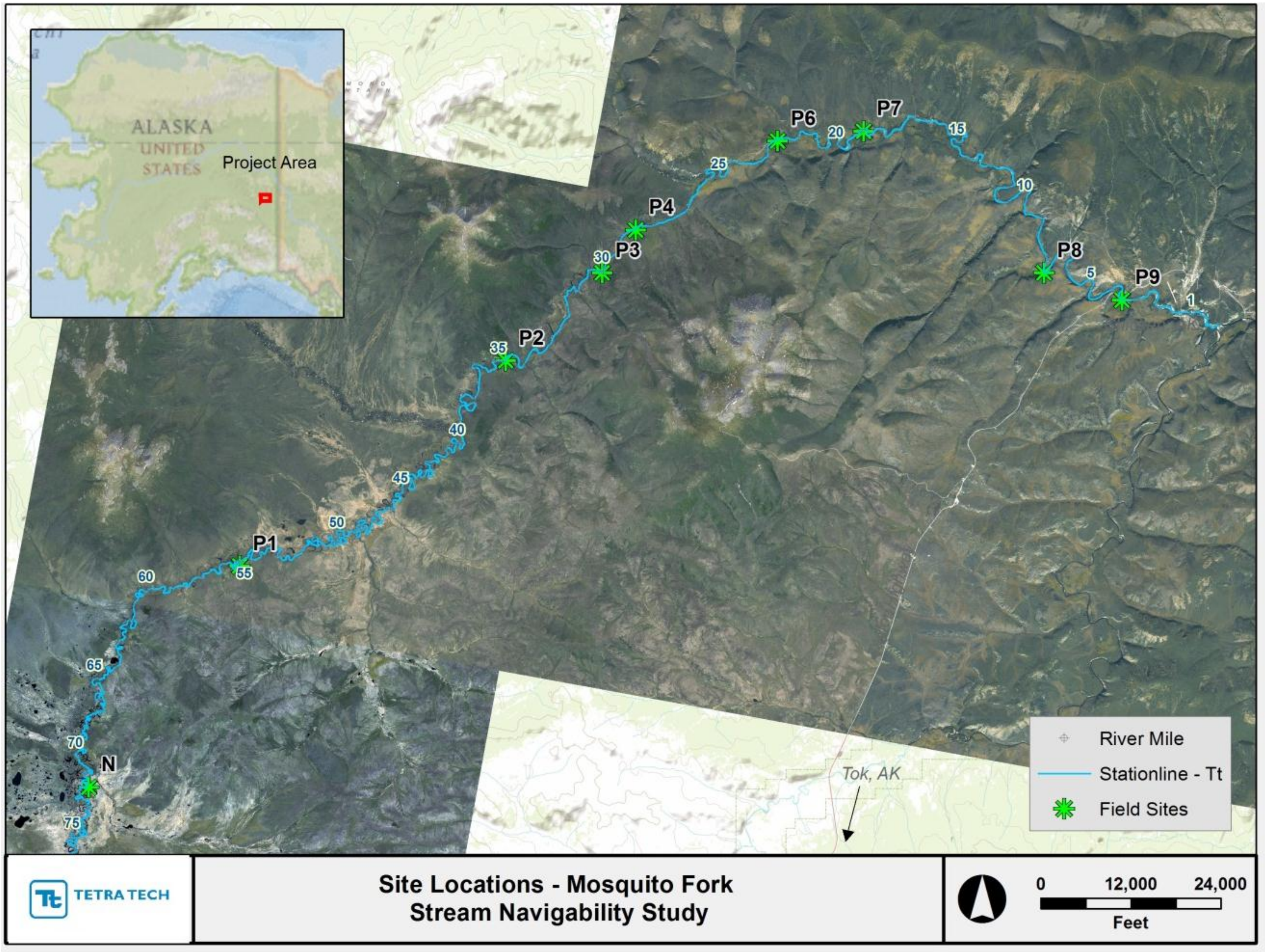


Figure 2. Map showing location of the detailed study sites.

Table 1. Summary of detailed study site surveys.

Site	Approximate Location*	Survey Date	Number of Cross Sections	Number of Sediment Samples	Measured Discharge (cfs)
P9	3.4	August 13, 2013	13	2	58
P8	7.7	August 14, 2013	10	3	62
P7	18.9	August 14, 2013	13	4	68
P6	22.5	August 15, 2013	11	2	61
P4	28.6	August 15, 2013	13	2	47
P3	30.1	August 16, 2013	12	2	63
P2	34.9	August 16, 2013	11	2	79
P1	56.4	August 17, 2013	7	2	9.3
PN	71.5	August 17, 2013	11	1	14

Appendix A also show the locations of the sediment samples, discharge measurements and the GPS base station.

The topographic data were collected using survey-grade RTK-GPS roving units that consisted of a Leica GS14 GNSS receiver and Leica Viva CS15 data collector (**Figure 3**). A separate Leica GS15 GPS antenna was used as a base station set up over a control point to provide real-time correction to the satellite signal being collected by the roving units (**Figure 4**). Since pre-established survey control was not available along the study reach for use with the base station, the absolute horizontal location and elevation of the base station control points were determined by collecting static position data at the base station for the duration of each site survey (typically at least 4 hours). These data were then provided to the National Geodetic Survey (NGS) Online Positioning User Service (OPUS) (http://www.ngs.noaa.gov/OPUS) for adjustment to the state plane coordinate system. Horizontal and vertical coordinates were referenced to the North American Datum of 1983 (AK Zone II, NAD83) and the North American Vertical Datum of 1988 (NAVD88), respectively.

The reported root mean square error (RMS) of the individual points within each site was generally within ±0.05 feet horizontally and ±0.07 feet vertically. Based on the OPUS solution report, the root mean square (RMS) error of the base station control point coordinates was ±1.8 cm (~0.06 feet) and ±0.8 cm (0.03 feet) in the X- and Y-directions (horizontal), respectively, and 1.9 cm (~0.06 feet) in the Z-direction (elevation). From a practical perspective, these accuracy levels mean that the relative horizontal and vertical positions of the vast majority of the surveyed points within each site, and from site to site, are accurate to within less than 0.1 feet. Considering the topographic variability within each site and the relatively long distances between the sites, this level of accuracy exceeds that necessary to represent the local site topography and to quantify the average gradient of the river between sites.

The topographic surveys consisted of detailed profiles of each cross section, including the top-of-bank, location and elevation of the water-surface elevation on either side of the channel at the time of the survey, location and elevation of the thalweg (i.e., minimum bed elevation across the cross section), locations of the bed material samples, and in a few cases, the location of the thalweg at locations away from the cross sections to help refine the bed profile. Average lengths of the cross sections at the eight sites that contained riffles/rapids ranged from about 117 feet at Site P1 to

Figure 3. RTK-GPS roving unit being used by field crew to collect topographic/cross section data at Site P9. Photo by Tetra Tech field crew, August 13, 2013.

Figure 4. Typical GPS base station setup at control point for Site P3.

nearly 290 feet at Site P3 (**Figure 5**). The surveyed profiles generally contain 30 to 50 individual points, spaced at about 5-foot intervals across the channel.

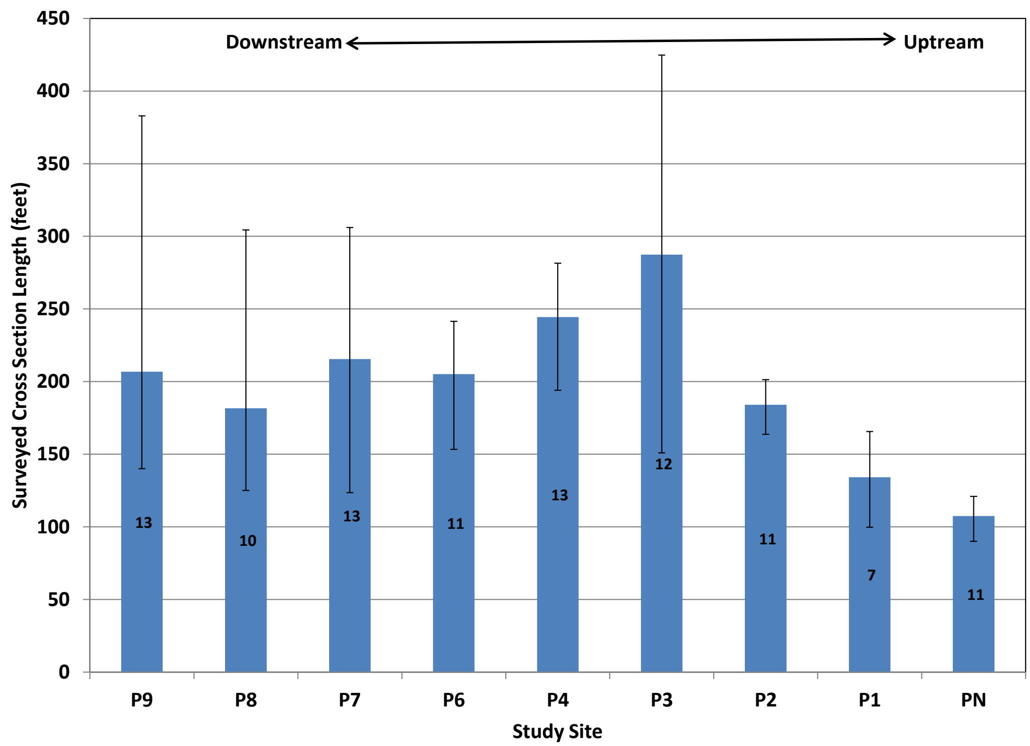


Figure 5. Average length of surveyed cross sections at the study sites. Numbers embedded in the bars are number of cross sections and the whiskers represent minimum and maximum length. Note: These lengths generally exceed the active width of the river to insure that the top-of-bank is well defined by the surveys.

The sediment samples were collected to quantify the range of particle sizes that make up the hydraulic controls at each site and the range of particles sizes that are transported by the river during higher flows. A total of 20 samples were collected in the project reach, with at least one sample from each riffle/rapid and one sample at a representative gravel bar within each site (**Table 2**, Appendix A). The riffle/rapid samples were collected by either stretching a tape measure across the sample area and measuring the median axis of the particles that fell beneath pre-determined increments along the tape (generally, 2 feet) (**Figure 6**), or where the individual particles were sufficiently small, using the standard pebble count technique (Wolman, 1954) that involves pacing across the sample area and measuring the median axis of 100 randomly-selected particles. The bar material samples at Sites P1 through P9 were collected using the pebble count technique. The sizes of individual particles with median diameter less than about 500 mm was measured using a gravelometer (see metal plate held by the crew member in Figure 5), and larger particles were directly measured using a tape measure. A bulk sample of the relatively fine-grained material making up the point bar at Site PN was collected and taken to a local soils laboratory in Fairbanks for gradation analysis. The median (D_{50}) size of the riffle samples ranged from about 50 mm (~1.8 inches) to 180 mm (7.2 inches) and the maximum sizes ranged from 150 mm (~6 inches) to 860 mm (~34 inches) (Table 2). The D_{50} of the bar samples ranged from ranged from 16 mm (0.6 inches) to 87 mm (3.4 inches) and the maximum sizes ranged from 110 mm (4.3 inches) to 400 mm (~16 inches). Plots of the full gradations of these samples are provided in **Appendix C.**

Table 2. Summary of bed material sediment samples collected during the August 13-17, 2013 surveys.

Site	D_{50} (mm)	D_{84} (mm)	Approximate Maximum Size (mm)	Location	Method	Sampled feature
P9	137	236	860	Riffle @XS8	Tagline	Riffle
P9	182	307	860	Riffle @XS5	Tagline	Riffle
P8	98	136	430	Upstream Riffle (Tail) @XS7	Pebble	Riffle
P8	56	100	210	Upstream Riffle (Head) @XS8	Pebble	Riffle
P8	129	272	760	Downstream Riffle @XS3	Tagline	Riffle
P7	65	110	210	Upstream Riffle @XS12	Pebble	Riffle
P7	72	128	210	Middle Riffle @XS6	Pebble	Riffle
P7	46	92	150	Downstream Riffle @ XS4	Pebble	Riffle
P7	61	100	150	Bar between XS4 and XS5	Pebble	Bar
P6	80	126	430	Riffle @ XS8	Pebble	Riffle
P6	87	126	300	Bar @ XS7	Pebble	Bar
P4	82	171	300	Riffle @ XS10	Pebble	Riffle
P4	63	124	300	Bar @XS9	Pebble	Bar
P3	73	134	430	Riffle @ XS5	Pebble	Riffle
P3	76	114	210	Bar @ XS4	Pebble	Bar
P2	53	86	210	Riffle between XS4 and XS5	Pebble	Riffle
P2	41	76	110	Bar @ XS6	Pebble	Bar
P1	59	188	400	Riffle @ XS4	Tagline	Riffle
P1	16	26	400	Bar @ XS3	Pebble	Bar
PN	1.5	3.2	13	Point Bar @ XS2	Pebble	Point Bar

Figure 6. Sediment sample being collected using the tagline technique at the riffle near Site 1, Cross Section 5.

Discharge measurements were collected at each site in conjunction with the surveys using a Teledyne RD Instruments StreamPro Acoustic Doppler Current Profiler (ADCP) mounted on a hand-operated trimaran boat (**Figure 7**). Measurements were made in accordance with USGS protocols (Mueller and Wagner, 2009). In areas where ADCP passes are not possible because of shallow depths (usually at the channel margins), discharge is estimated by measuring the width of the unmeasured area and assuming a horizontal or angled bed profile. The ADCP then extrapolates a flow rate based on the closest measurable conditions. According to Mueller and Wagner (2009), if the total discharge for each pass are within 5 percent of each other, the four values are averaged to produce the final discharge. If the values vary by more than 5 percent, then an additional four passes are made and all eight values are averaged to produce the final result. Measured flows during the surveys were very low, ranging from about 9 cfs at Site P1 (August 17, 2013) and to 79 cfs at Site P2 (August 17, 2013) (Table 1).

Figure 7. Discharge measurement being made at Site P7 using the StreamPro ADCP.

2.3. Physical Characteristics of the Disputed Reach

2.3.1. Overview

The Mosquito Fork drains approximately 1,120 mi^2 of east-central Alaska, flowing into the Dennison Fork of the Fortymile River and ultimately into the Yukon River (**Figure 8**). Elevations in the basin range from about 1,600 feet at the mouth to over 5,000 feet along the northern and western drainage divide. [The highest elevation (5825 feet) occurs on Mt. Veta at the head of the Ketchumstuk Creek drainage.] Precipitation over the basin averages about 15 inches on an annual basis [U.S. Geological Survey (USGS), 1998], with over 70 percent occurring as rainfall during May through September [National Oceanographic and Atmospheric Administration (NOAA) 2014[2]]. Annual snowfall averages 35 to 40 inches at Chicken (NOAA, 2014), and is somewhat greater in the upper parts of the basin. As will be discussed in more detail in a later section of this report, the river is typically frozen from mid- to late-October through late-April to early-May. During the open-water season, discharges above the relatively low baseflows occur episodically in response to individual rainstorms.

The basin is located in the Yukon-Tanana Upland physiographic province that is characterized by rolling topography and typically gentle hillslopes (Brabets et al., 2000). The dominant lithology (i.e., rock type) underlying most the basin is granitic (**Figure 9**, JRTg, Mzg, Mg), although a zone of Quaternary- and Tertiary-age volcanic rocks [i.e., the past ~65 million (M) years] occurs along the south side of the river from about 50 river miles to 70 river miles upstream from the mouth (QTv in

[2]Based on precipitation and snowfall data from Global Hydrologic Climate Network Data Station (GHCND) USC00509313 (Tok, AK US) for 1997 through 2014.

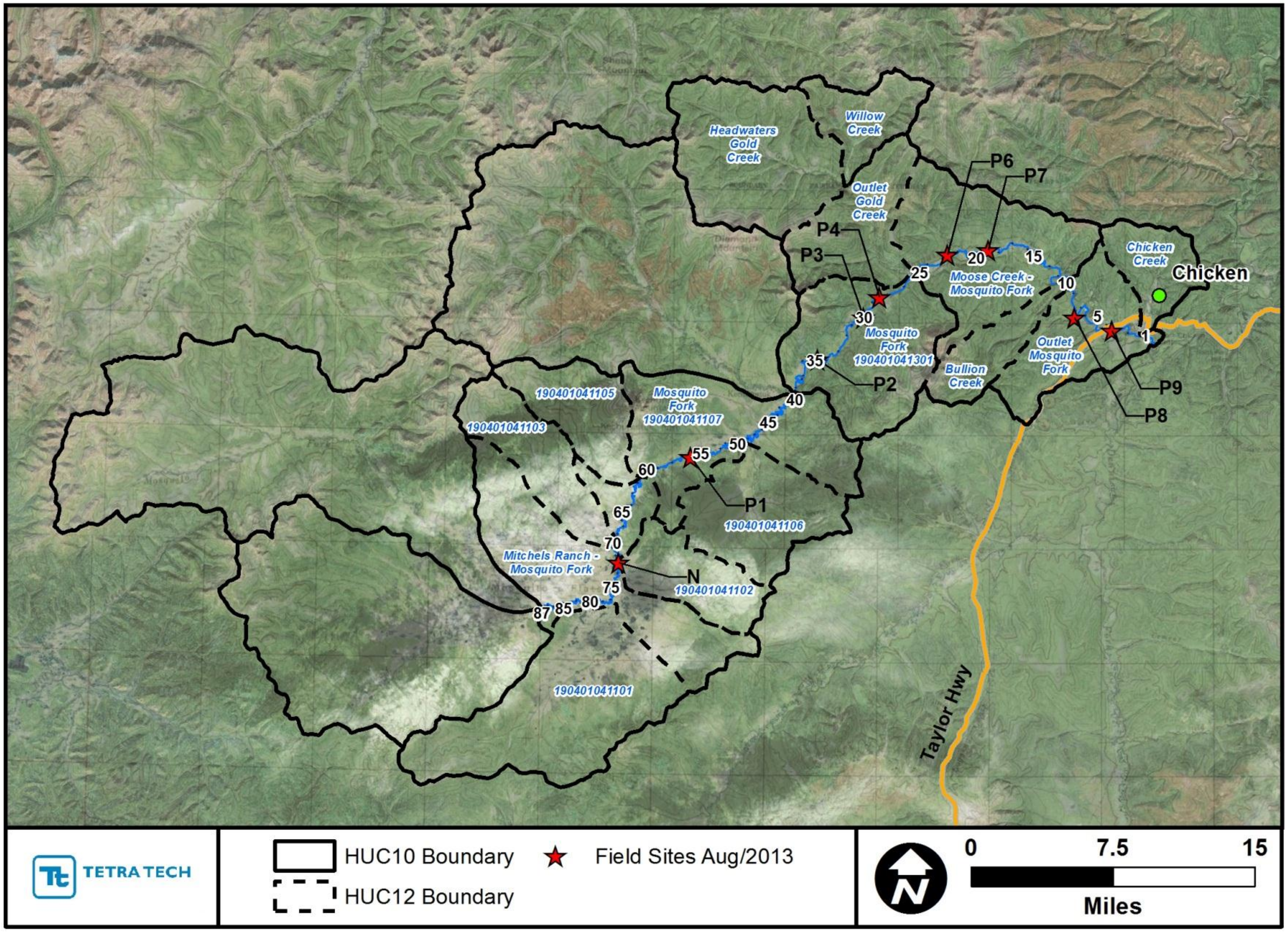


Figure 8. Map of Mosquito Fork watershed.

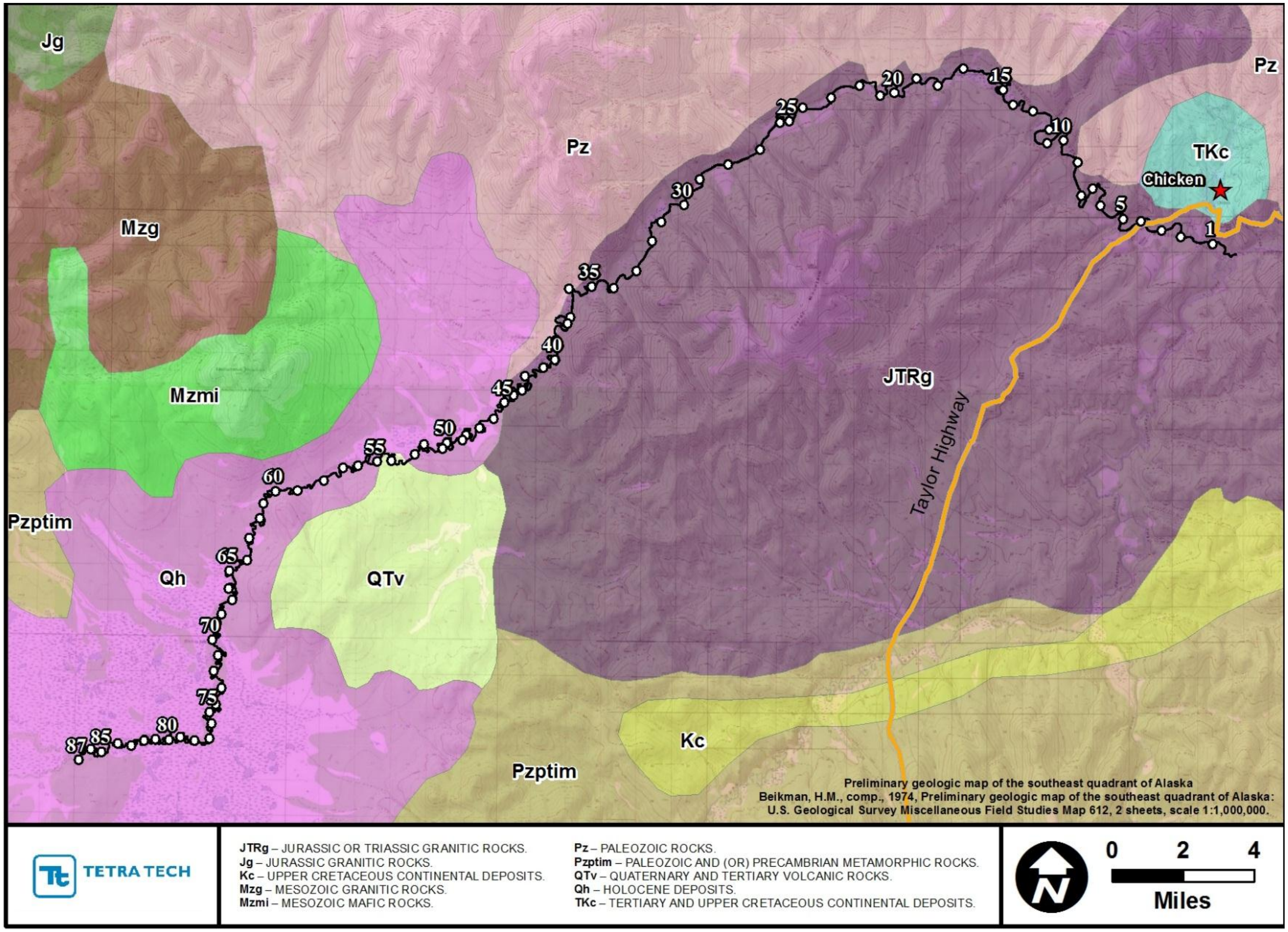


Figure 9. Geology map of the Mosquito Fork drainage basin (modified from Beikman, 1974).

Figure 9). Other rock types that occur in the basin include Paleozoic-age (225M-570M years old) metamorphic schists and gneisses (Pzptim), and the northeastern portion of the basin is underlain by Paleozoic-age metamorphic rocks of varying composition (Pz). The surface material in the wider portions of the valley from about RM 50 upstream through Mosquito Flats consists of modern alluvium. According to Yeend (1995), a pebble count conducted in 1992 of the alluvium in the vicinity of the Taylor Highway Bridge contained 44-percent granite, 38-percent greenstone, 12- percent quartzite, 2-percent quartz, 2-percent schist, and 2-percent basalt, reflecting the general distribution of rock types in the watershed (**Figure 10**). From a practical perspective, the alluvial transport characteristics of these rock types are similar; however, the basalts tend to remain more angular, whereas the other types tend to become rounded and more easily mobilized by the river. A navigation hazard occurs at Site 1 that consists of large, angular basalt boulders that are derived from the adjacent volcanic (Qtv) bedrock (**Figures 11a and b**). These boulders are probably lag deposits into which the river has incised and have likely not been transported a significant distance, if at all, by the river from their original location.

Most rivers have a concave longitudinal profile in which the gradient is flatter in the downstream reaches and steepens in the upstream direction. In contrast, the disputed reach of the Mosquito Fork has a convex profile, with the steepest gradients in the downstream approximately half of the reach and the flattest gradients in the upstream part of the reach (**Figure 12**). Based on the surveyed elevations at Tetra Tech's nine (9) detailed study sites, the approximately 32-mile reach between the Taylor Highway (Site P9) and Site P2 (located about 4.4 miles downstream from Ketchumstuk Creek) is about 14 feet per mile (fpm), while the gradient in the 22-mile reach between Site 2 and the most upstream Site 10 that is located in the area known as Mosquito Flats is less than 4 fpm[3]. Steeper sections occur in both portions of the reach, with gradients of about 19 fpm between Sites P8 and P9 in the downstream portion of the reach and about 22 fpm between Sites P3 and P4.

An attempt was made to assess the variability in the river's gradient between the study sites by cutting a profile along the approximate centerline of the river from the 10 m resolution Digital Elevation Model of this area available from the USGS National Elevation Dataset (NED) (http://datagateway.nrcs.usda.gov) (light black line in Figure 12). The NED for Alaska is mostly derived from the USGS 1:63,360-scale topographic mapping, supplemented in some areas with 5-meter spatial resolution DEMs derived from airborne interferometric synthetic aperture radar (IFSAR) data (Gesch et al., 2014). For the portions of the dataset that are based solely on the topographic maps, the Root Mean Square Error (RMSE)[4] of the elevation data is reported to be +/-4.85 m (15.9 feet), and this decreases to ±1.63 m (5.3 feet) where the IFSAR data are used. Although the metadata are somewhat unclear, it appears that the data along the Mosquito Fork are based only on the topographic maps; thus, it is assumed that the RMSE of the DEM in this area is ±4.85 m. With the exception Site P8, the surveyed elevations at the study sites are within the error bands on the DEM data. The reason for the apparent discrepancy at P8 is not known, and information with which to assess the discrepancy is not available. The stepped nature of the DEM-based profile, however, strongly suggests that the DEM data are not accurate in this area. Considering the care that was taken in establishing the elevation of the control point for P8, the surveyed elevations are believed

[3]The elevations of the study sites (P1-PN) shown in Figure 2.5 are the average elevation surveyed profile through each site. The surveys were conducted using a Leica.

[4]RMSE is a measure of the scatter in the data. The definition assumes that the scatter is normally distributed, and about two-thirds of the data points shown on the mapping should be within the report RMSE of the true elevation (in this case ±4.85 m or 15.9 feet).

Figure 10. Typical view of alluvial cobbles in the Mosquito Fork about 900 feet downstream from the Taylor Highway Bridge, in the general vicinity of the Yeend (1995) pebble count. Photo by R. Mussetter August 13, 2013.

Figure 11a. Basalt boulders in the bed of the Mosquito Fork at Site 1 (RM 56.4). These boulders are derived from the Quaternary- and Tertiary-age volcanic bedrock along the south side of the river in this area.

Figure 11b. Close-up view of the basalt boulders at Site 1.

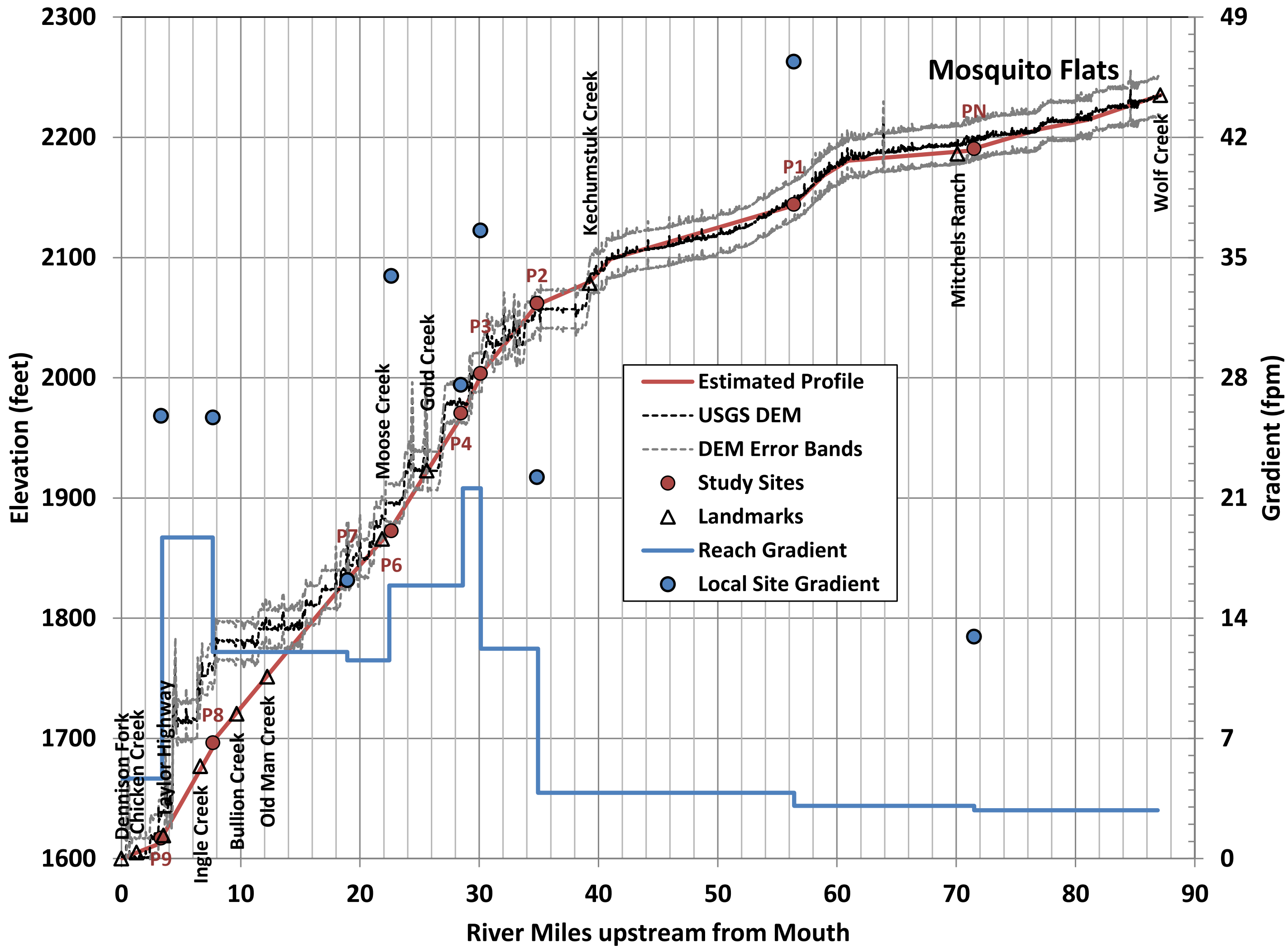


Figure 12. Longitudinal profile of the disputed reach of the Mosquito Fork River.

to be within the accuracy described above in relation to the OPUS correction. As a result, the convexity in the river profile between Sites P7 and P9 indicated by the DEM data may not actually exist.

Both the survey data and the DEM indicate that the gradient of the river is relatively uniform between Sites P2 and P7 at the coarse scale of the plotted profile, while convexities occur in the vicinity of Ketchumstuk Creek and just upstream from Site P1 in the upstream portion of the reach. The convexity is consistent with the geologic and geomorphic setting in these areas. Water and sediment from Ketchumstuk Creek appear to push the river against the right (southeast) valley wall, and the valley width is constricted by the underlying bedrock just downstream (**Figure 13**). A similar constriction in the valley width occurs in the vicinity of the upstream convexity (**Figure 14**). Constrictions of this type often result in steepening of the river profile. The gradient of the steep portion of the convexity near Ketchumstuk Creek is about 11 fpm and the gradient of this portion of the upstream convexity is about 8 fpm.

2.3.2. Detailed Study Sites

The average gradients at the detailed study sites are considerably steeper than the average gradients in the longer reaches in which they are located, and most of these sites include a riffle zone that is significantly steeper than the average through the site (**Figure 15**). This is expected since the study sites were selected in locally steeper reaches where boatability is most likely to be limited. Site P7 is the flattest of the study sites, and Site P1 is the steepest, with average gradients of about 16 and 46 fpm, respectively. The steepest measured riffles occur at Sites P1, P6 and P9, with gradients of 61, 62 and 53 fpm, respectively.

Site P9 is located at the Taylor Highway crossing at approximately RM 3.4, and includes the area locally-known as “the Rock Garden” because of the large boulders and relatively steep gradient (**Figures 16 and 17,** Appendix A). The average channel gradient through the approximately 1,600-foot length of the site is about 26 fpm; however, the Rock Garden portion of the site between approximately XS5 and XS9 drops 5.2 feet over a distance of 520 feet, or a gradient of about 53 fpm. At the relatively low discharge when the surveys were conducted (58 cfs), the steep area was characterized by flow moving through many angular boulders that protruded through the water surface. The sediment samples collected across XS5 had median (D_{50}) and D_{84} (size for which 84 percent of the particles were smaller) of 137 mm (5.4 in) and 236 mm (9.3 inches), respectively, and several particles with median diameter in the range of 3 feet were encountered (Table 2). The sample across XS8 was even coarser, with D_{50} and D_{84} sizes of 182 mm (7.2 inches) and 307 mm (12.1 inches), respectively, and this samples also contained several particles with diameter in the range of 3 feet. The bankfull widths at the surveyed cross sections (ranged from about 140 feet to 254 feet, and averaged about 170 feet (**Figure 18**).

Site P8 is located at a sharp, bedrock-controlled bend in the river at about RM 7.7, approximately 1.5 miles upstream from Ingle Creek (**Figure 19**). The primary navigation hazard at this site is the very wide, shallow riffle just upstream from the bend (**Figure 20**), although hydraulic conditions in the bend would also present a navigation challenge at higher flows for long, narrow boats such as those being commonly used in this area in the late-1950s. The thalweg elevation drops 4 feet over the approximately 810 length of the site between XS 2 and XS10 (average gradient of 26 fpm), and 2.5 feet over about 340 feet in the primary riffle between XS6 and XS9 (gradient of about 39 fpm) (**Figure 21**). (XS1 was not included in the effective overall gradient calculation because it is located in a deep pool created by scour along the toe of the bedrock. Inclusion would indicate a gradient much steeper than the effective gradient through the site.) Sediment samples collected at three locations within this site had median sizes ranging from 56 mm (2.2 inches) to 129 mm (5.1 inches)

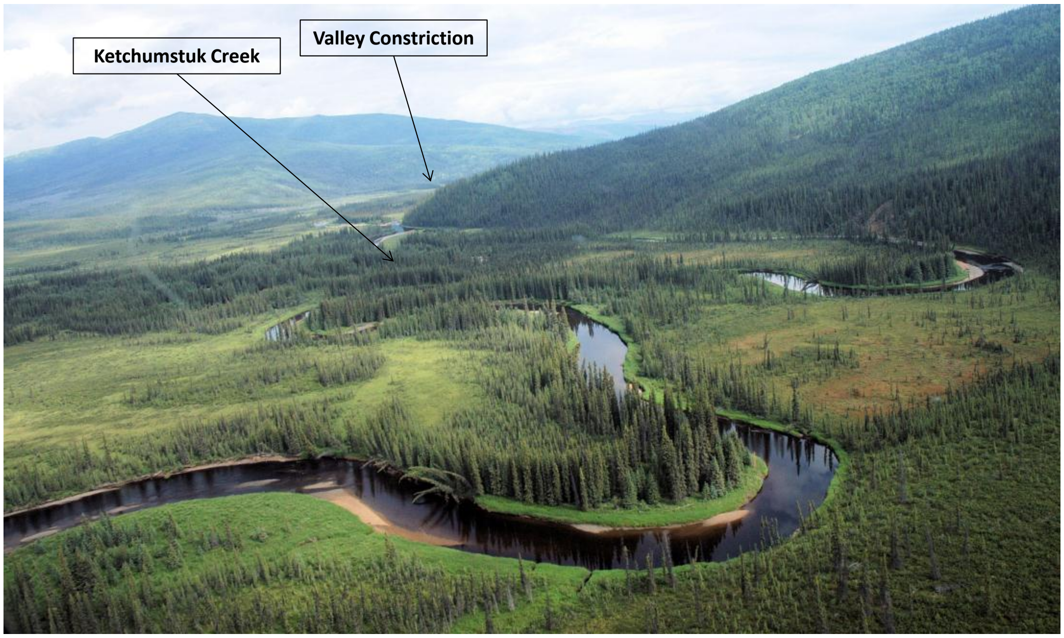


Figure 13. Mosquito Fork River valley looking downstream from approximately RM 41.

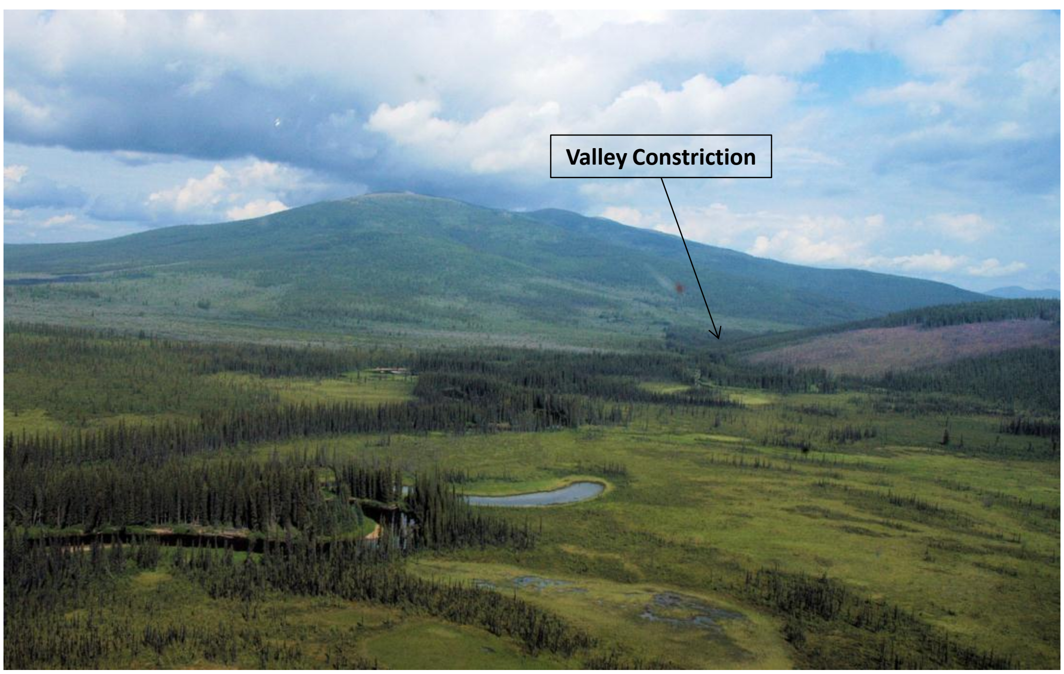


Figure 14. Mosquito Fork River valley looking downstream from approximately RM 63.5.

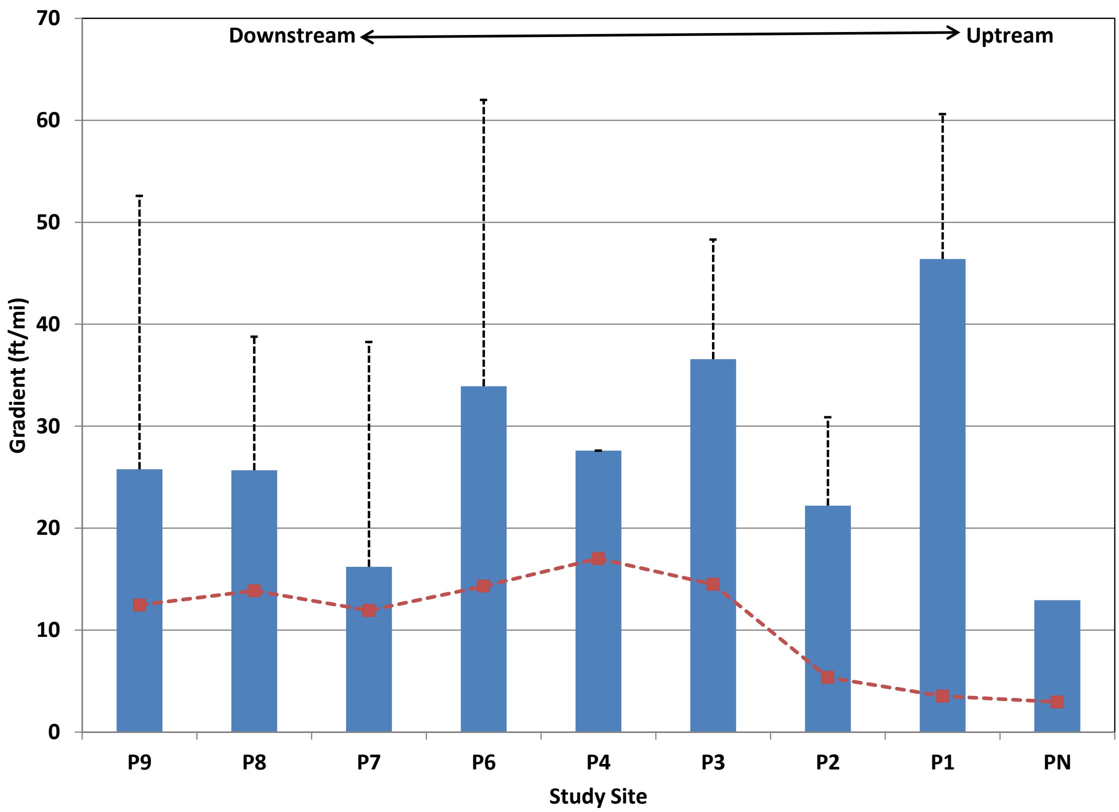


Figure 15. Average channel gradients at the detailed study sites (bars). Whiskers represent the gradient of the key, locally-steep riffle/rapid section within each of the study sites that would limit navigability. Red symbols represent overall reach-averaged gradient within which the site is located.

Figure 16. Site P9 looking upstream from XS5 (See Appendix A) through the area locally-known as “the Rock Garden”. Photo Tetra Tech Field Crew, Aug 13, 2014. Measured discharge = 58 cfs.

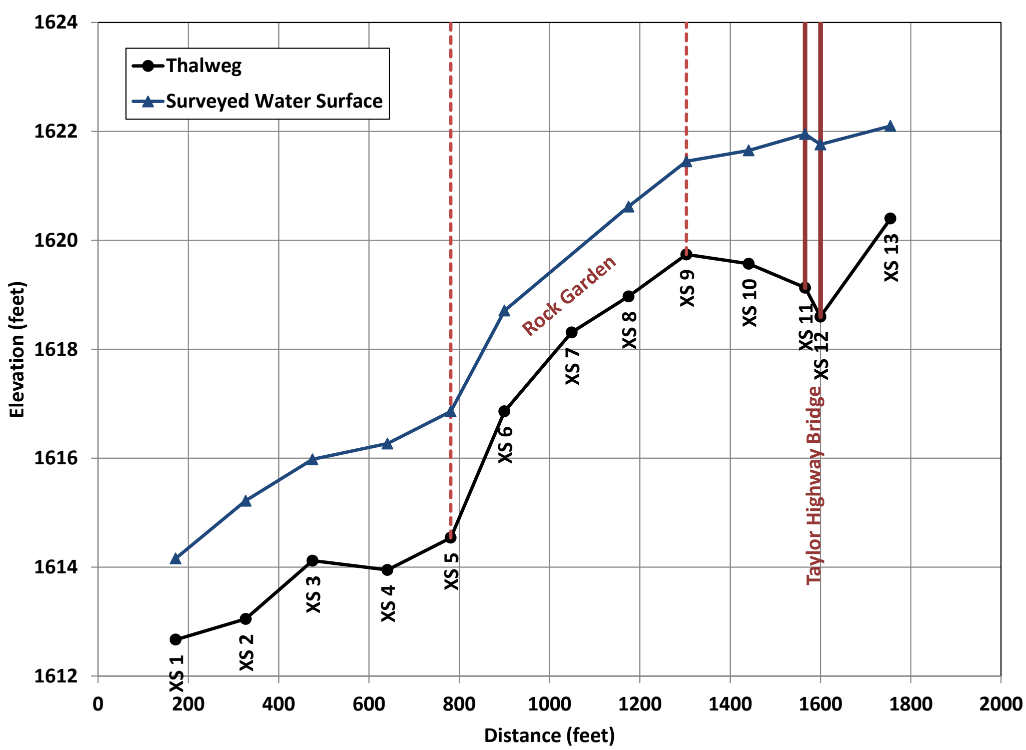


Figure 17. Thalweg (i.e., minimum bed elevation) and surveyed water-surface profile (58 cfs) at Site P9.

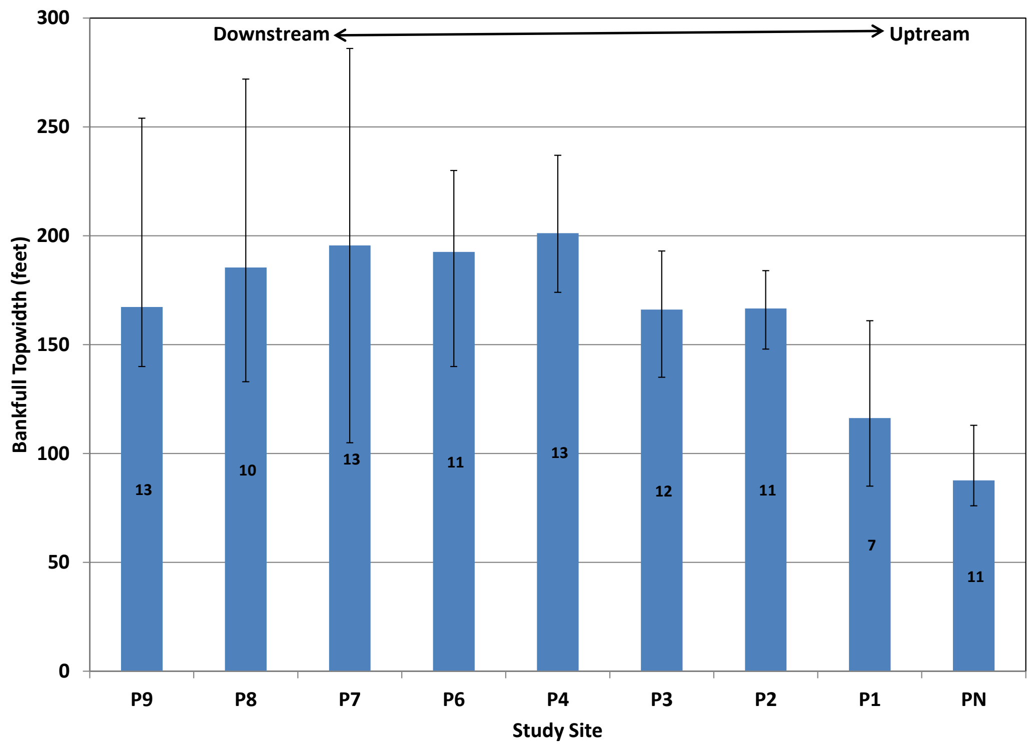


Figure 18. Average bankfull channel width at the surveyed cross sections at each study site. Whiskers represent minimum and maximum width. Embedded numbers represent number of surveyed cross sections at the site.

Figure 19. Oblique aerial view, looking downstream, of the bed-rock controlled bend at Site P8. Photo by R. Mussetter, August 13, 2013. Measured discharge = 62 cfs.

Figure 20. View of wide, shallow riffle in the upstream portion of Site P8. Photo by R. Mussetter, August 14, 2013. Measured discharge = 62 cfs.

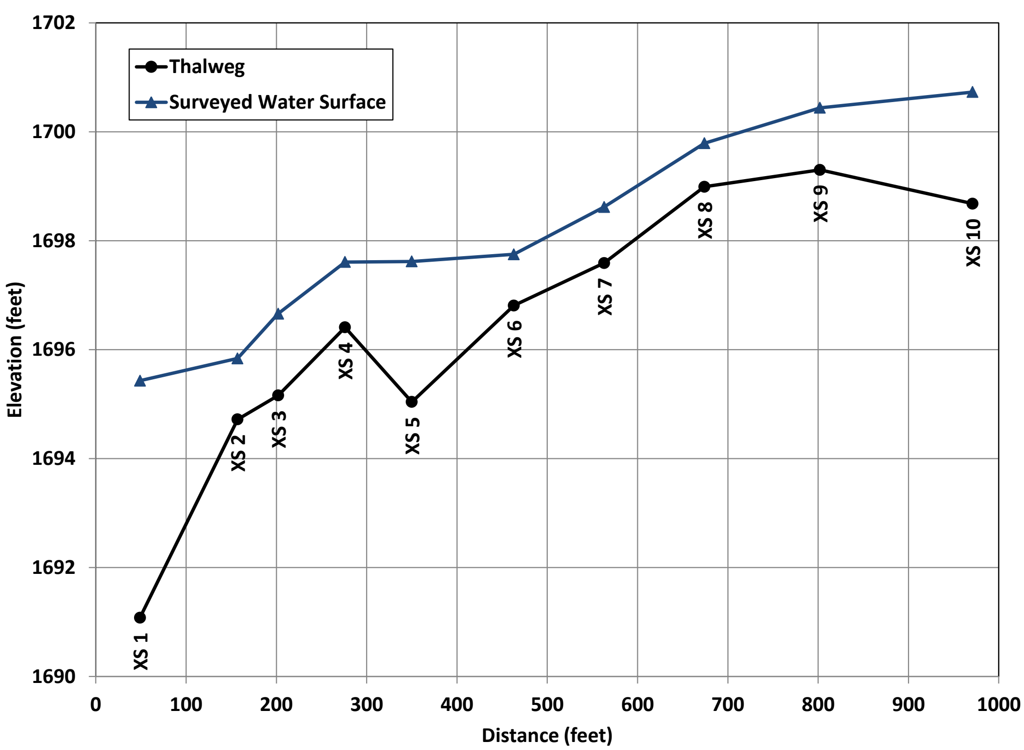


Figure 21. Thalweg (i.e., minimum bed elevation) and surveyed water-surface profile (62 cfs) at Site P8.

and D_{84} sizes ranging from 100 mm (4 inches) to 272 mm (10.7 inches). Maximum sizes ranged from about 9 inches to 2.5 feet (Table 2). The coarsest sample occurred in the steep section just upstream from the apex of the bend near XS3 and XS4. The bankfull widths at the surveyed cross sections ranged from 133 to 272 feet and averaged 185 feet (Figure 18).

Site P7 is located in and upstream from a bend at about RM 18.9, about 3 miles downstream from Moose Creek. Potential navigation hazards at this site include a shallow, cobble riffle near the apex of the bend and XS4 (**Figure 22**), a narrow, cobble riffle adjacent to bedrock outcrop that deflects the flow near XS6 and XS7 (**Figure 23**), and the wide shallow riffle at the upstream end of the site between XS10 and XS12 (**Figure 24**). The overhanging trees along the outside of the bend also create a navigation hazard. The thalweg elevation drops 4.9 feet over the approximately 1,600-foot length of reach between the crest of the upstream riffle (XS12) and XS1 (average gradient of 16.2 fpm) (**Figure 25**). The gradient across the riffle between XS4 and XS7 is about 19 fpm and the gradient of the wide, shallow riffle at the head of the reach is about 38 fpm. The median particle sizes in the riffles near XS4, XS6 and XS11 were 46 mm (1.8 inches), 72 mm (2.8 inches) and 65 mm (2.6 inches), respectively, and the D_{84} sizes were 100 mm (3.9 inches), 92 mm (3.6 inches) and 128 mm (5 inches), respectively. The median axis of the largest particles at all three locations was in the range of 8 to 10 inches. The bankfull widths at the surveyed cross sections ranged from 105 to 286 feet and averaged about 195 feet (Figure 18).

Site P6 is located at bedrock-controlled riffle near RM 22.5, just upstream from the mouth of Moose Creek (**Figure 26**). The shallow riffle, that contains boulders with median diameters of up to 1.5 feet (**Figure 27**), is the primary navigation hazard at this site. The thalweg elevation drops about 3.7 feet over the approximately 580 feet in length of reach from the crest of the riffle (XS8) to XS1 (gradient of about 34 fpm), and the gradient of the main part of the riffle (XS4 to XS7) is 62 fpm (**Figure 28**). The median size of the bed material in the riffle was 80 mm (3.1 inches) and the D_{84} was 126 mm (5 inches). Bankfull widths at the surveyed cross sections at this site ranged from 140 feet to 230 feet, and averaged about 195 feet.

Site P4 is located at a long, wide and shallow riffle at RM 28.6, about 2.5 miles upstream from Gold Creek (**Figure 29**). The riffle, that extends over most of the site with total elevation drop of 6.4 feet over the approximately 1,200-foot length (gradient of 28 fpm), is the primary navigation hazard at this site (**Figure 30**). A vegetated island occurs between about XS3 and XS7 splits the flow at higher discharges, contributing to relatively shallow flow depths. A similar wide, shallow riffle also occurs just downstream from the surveyed reach (**Figure 31**). The median size of the bed material in the riffle was 82 mm (3.2 inches) and the D_{84} was 170 mm (6.7 inches). The individual boulders with median axis of more than 1 to 1.5 feet occur throughout the riffle (**Figure 32**). Bankfull widths at the surveyed cross sections at this site ranged from 175 to 240 feet, and averaged about 200 feet.

Site P3 is located in an expansion zone just upstream from a bend in the river at about RM 30.1 (**Figure 33**). The primary navigation hazard at this site is the wide, shallow riffle that occupies most of the site. This site is a deposition zone for large cobbles and small boulders at high flows when the contraction at the bend causes upstream backwater. A large, vegetated island occurs along the right side of the primary flow path over most of the site that conveys significant flow at higher discharges, contributing to the shallow flows over the riffle. The thalweg elevation drops about 7 feet over the 1,000-foot length of the site between XS11 and XS1 (gradient of 37 fpm) (**Figure 34**). The median size of the bed material in the sampled portion of the riffle was 73 mm (2.9 inches) and the D_{84} is 134 mm (5.3 inches). Individual particles up to 1.5 feet occur in the riffle (**Figure 35**).

Figure 22. Riffle near apex of the bend (XS 4) at Site P7. Also note overhanging trees along the outside of the bend. Photo by R. Mussetter, August 14, 2013. Measured discharge = 68 cfs.

Figure 23. Bedrock-controlled riffle near XS 6 and XS 7 at Site P7. Photo by R. Mussetter, August 14, 2013. Measured discharge = 68 cfs.

Figure 24. Wide, shallow riffle between XS10 and XS12 at Site P7. Photo by R. Mussetter, August 14, 2013. Measured discharge = 68 cfs.

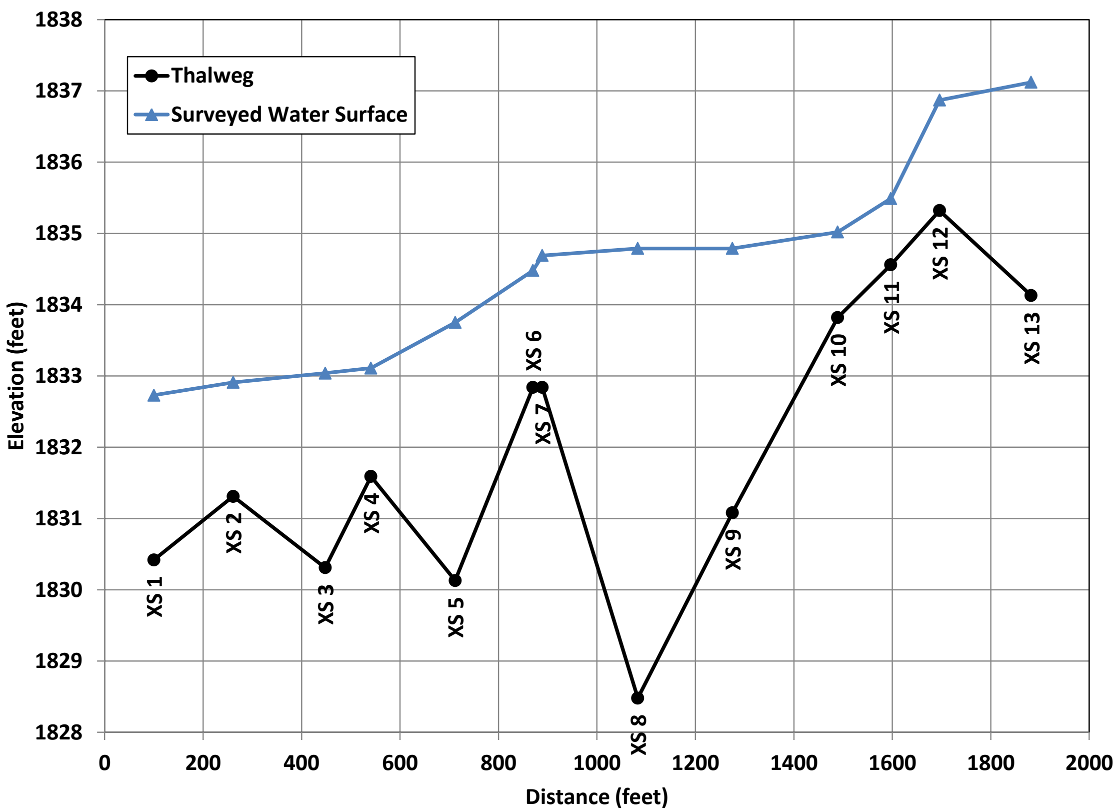


Figure 25. Thalweg (i.e., minimum bed elevation) and surveyed water-surface profile (68 cfs) at Site P7.

Figure 26. Looking downstream across bedrock-controlled riffle at Site P6. Photo by R. Mussetter, July 16, 2013. Discharge at Taylor Highway Bridge ~130 cfs.

Figure 27. Looking downstream across the head of the bedrock-controlled riffle at Site P6. Photo by Tetra Tech field crew, August 15, 2013. Measured discharge = 61 cfs.

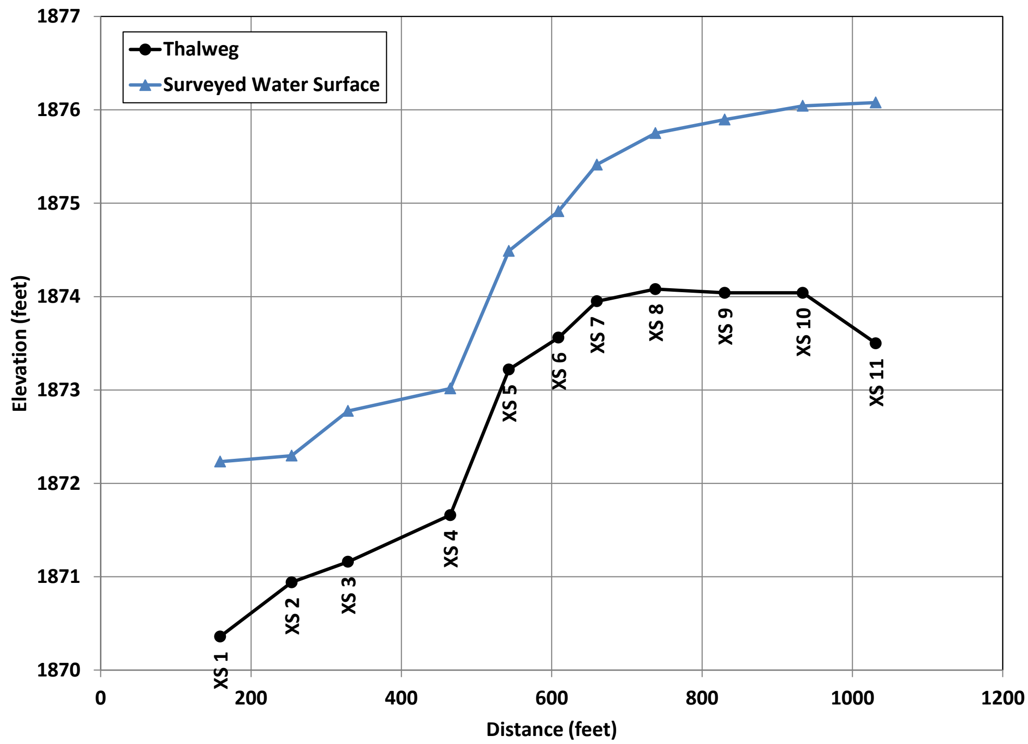


Figure 28. Thalweg (i.e., minimum bed elevation) and surveyed water-surface profile (61 cfs) at Site P6.

Figure 29. Looking upstream at the wide, shallow riffle at Site P4. Photo by R. Mussetter, August 14, 2013. Discharge ~50 cfs.

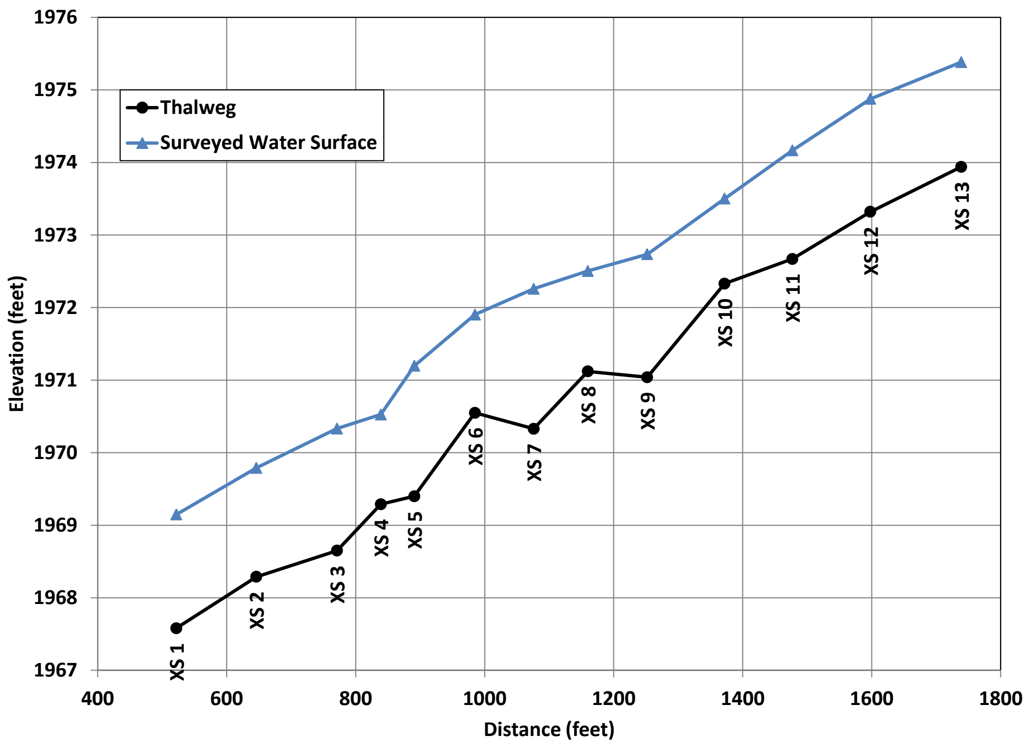


Figure 30. Thalweg (i.e., minimum bed elevation) and surveyed water-surface profile (47 cfs) at Site P4.

Figure 31. Looking downstream at the wide, shallow riffle just downstream from Site P4. Photo by R. Mussetter, July 16, 2013. Discharge at Taylor Highway Bridge ~130 cfs.

Figure 32. Looking upstream through the riffle at Site P4. Photo by R. Mussetter, August 14, 2013. Discharge ~50 cfs.

Figure 33. Looking downstream at the vegetated island and shallow riffle at Site P3. Photo by R. Mussetter, July 16, 2013. Discharge at Taylor Highway Bridge ~130 cfs.

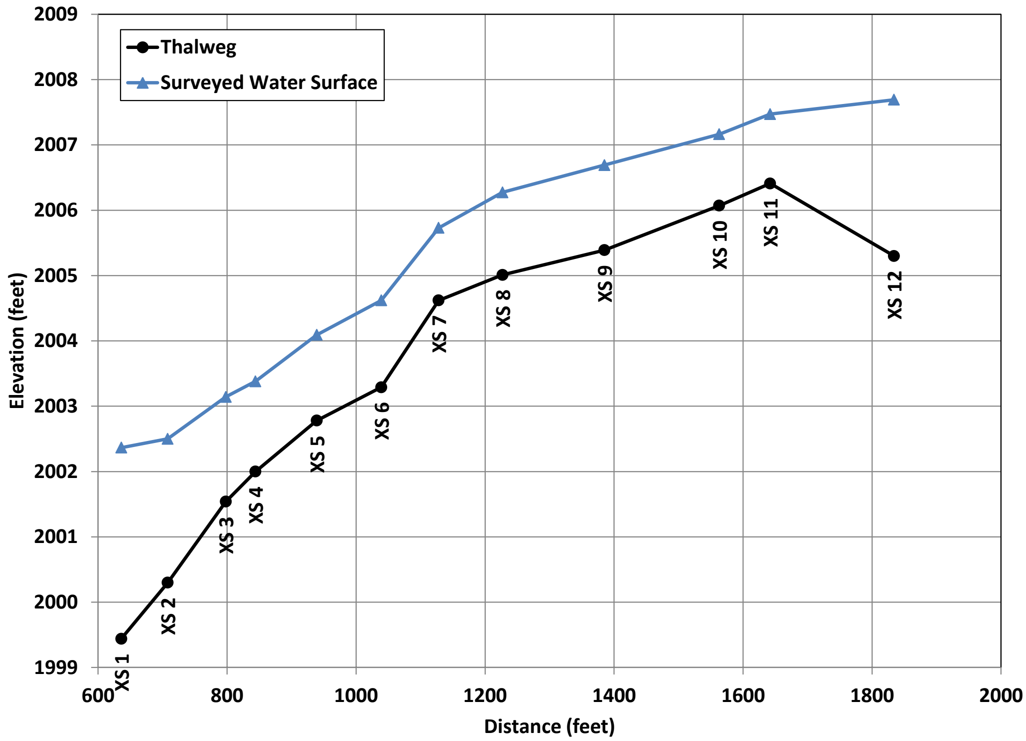


Figure 34. Thalweg (i.e., minimum bed elevation) and surveyed water-surface profile (63 cfs) at Site P3.

Figure 35. Looking downstream across the wide, shallow riffle at Site P3. Photo by R. Mussetter, August 14, 2014. Discharge ~60 cfs.

Bankfull widths at the surveyed cross sections at this site ranged from 135 to 193 feet, and averaged about 165 feet.

Site P2 is located just upstream from a mild bend in the river at about RM 35 (**Figure 36**). The site is characterized by a wide shallow riffle that has formed in an expansion zone upstream from the bend, similar to Site P3. At the time of the surveys, the flow split almost evenly around a large, cobble island in the middle of the site. The riffle and shallow flows around the island are the primary navigation hazard at the site. The thalweg elevation drops 3.2 feet over the approximately 770-foot length of the riffle between surveyed XS1 and XS10 (gradient of 22 fpm), and the gradient of the approximately 470-foot long primary portion of the riffle is somewhat steeper at about 31 fpm (**Figure 37).** The bed material in the sampled portion of the riffle had median size of 53 mm (2.1 inches) and D_{84} of 86 mm (~9 inches), and individual particles with median axis diameters of up to 1 foot are scattered throughout the riffle (**Figure 38**). Bankfull widths at the surveyed cross sections at this site ranged from 115 to 185 feet, and averaged about 150 feet.

Site P1 consists of a basalt boulder rapid at RM 56.4 that has formed in the lag deposits from the adjacent Tertiary- or Quaternary-age volcanics that bound the river on the south site at this location (see geology discussion in previous section) (Figure 11a). The large angular boulders are the primary navigation hazard at this site. The thalweg elevation drops about 2.6 feet over the 230-foot length of the rapid between XS2 and XS5 (gradient of about 61 fpm), the elevation drop (**Figure 39**). The median bed material size in the sampled portion of the site was 59 mm (2.3 inches) and the D_{84} was 188 mm (7.4 inches). Individual particles up to 2.5 to 3 feet in diameter are scattered throughout the rapid (Figure 11b). Bankfull widths at the surveyed cross sections at this site ranged from 85 to 160 feet, and averaged about 115 feet.

The most upstream Site PN is located in the crossing between two bends in the highly sinuous portion of the disputed reach at RM 71.6, in the area known as Mosquito Flats (**Figure 40**). Gradient in this portion of the reach is relatively flat, at about 3 fpm, on average, over the longer reach in which the site is located. The water-surface slope through the site at the time of the surveys was less than 2 fpm (**Figure 41**). The bed material in this portion of the reach is composed primarily of sand and fine to medium gravel. The median size of the material in the bulk sample collected from the point bar at the site was about 1.5 mm (coarse sand), the D_{84} size was about 3.2 mm (fine gravel), and the maximum particle size was in the range of 12 to 15 mm (medium gravel). Based on observations during the field work, this material is believed to be representative of the material throughout most of the Mosquito Flats reach. Bankfull channel widths at the surveyed cross sections at this site ranged from 75 to 115 feet, and averaged about 90 feet. As can be readily seen in Figure 41, flow depths are significantly greater in this part of the reach than the downstream steeper reaches. Thalweg depths at the time of the surveys, when the discharge was only about 14 cfs, ranged from 1.8 to 4.5 feet and averaged about 3 feet. Although overhanging wood on the outsides of the bends presents a safety hazard, a competent boatman should be able to navigate through this relatively low velocity reach without significant challenges.

2.3.3. Changes Affecting Natural Condition of the Disputed Reach

The test of navigability strictly applies to ordinary and natural conditions of the river at the date of statehood. Since the amount of specific information about the river at that time is limited, my assessment of the characteristics of the river was primarily based on modern-day conditions, although I reviewed other available information to help understand whether the river has changed sufficiently in all or part of the reach since 1959 in a way that would affect its navigability. The approximately 2-mile reach near and downstream from the mouth of Chicken Creek has been significantly altered by historic dredging, although most of these activities likely occurred prior to Alaska's statehood (Yeend, 1995). In any case, this part of the reach is not in its natural condition today, nor was it likely in that condition at the date of statehood. It is my understanding that the United States has previously opined that this part of the reach was navigable at the date of statehood; thus, it is excluded from my analysis.

Figure 36. Looking upstream at the wide shallow riffle at Site P2. Photo by R. Mussetter, August 14, 2013. Discharge ~75 cfs.

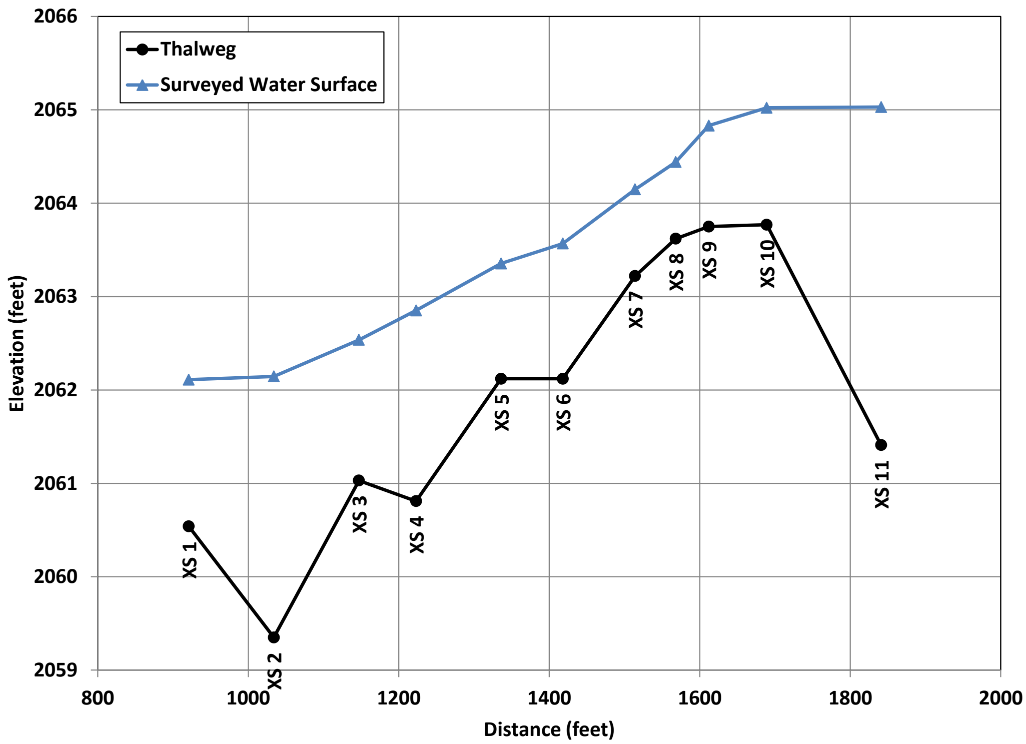


Figure 37. Thalweg (i.e., minimum bed elevation) and surveyed water-surface profile (79 cfs) at Site P2.

Figure 38. Looking upstream across the wide, shallow riffle at Site P2. Photo by R. Mussetter, August 14, 2013. Discharge ~75 cfs.

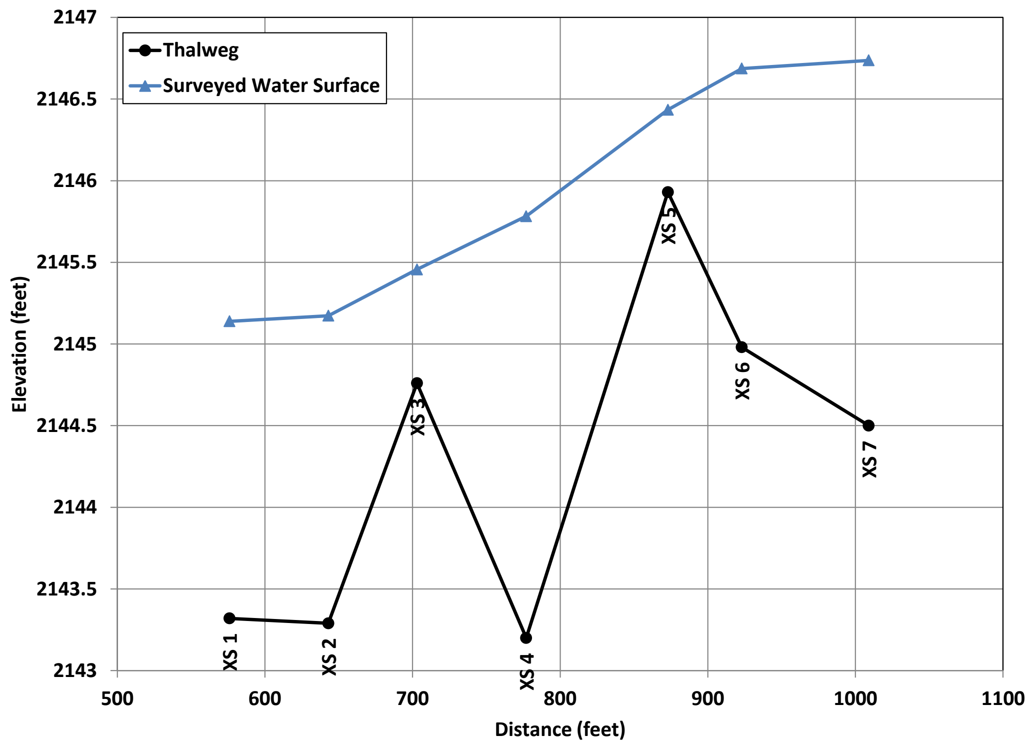


Figure 39. Thalweg (i.e., minimum bed elevation) and surveyed water-surface profile (~10 cfs) at Site P1.

Figure 40. Looking upstream at the Mosquito Fork River in the vicinity of Site PN. (Study site is located along the bare point bar in the lower right of the photo.) Photo by R. Mussetter, August 15, 2013. Discharge ~15 cfs.

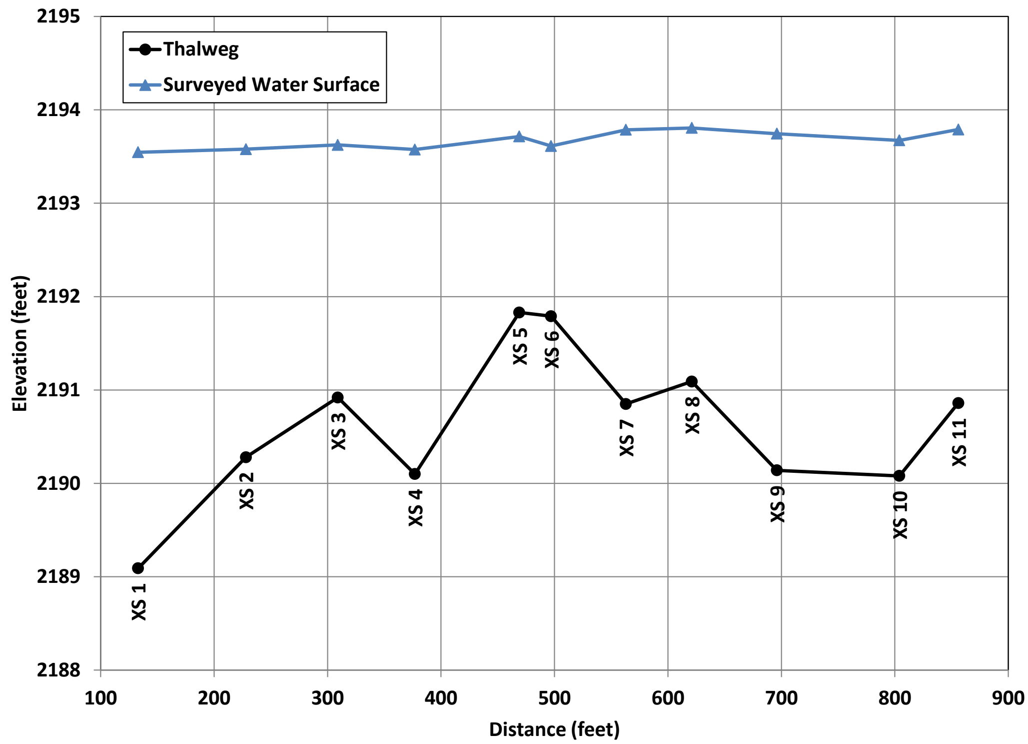


Figure 41. Thalweg (i.e., minimum bed elevation) and surveyed water-surface profile (~14 cfs) at Site PN.

The area from approximately RM 3 to a short distance upstream from the Taylor Highway Bridge has also been subjected to dredging activity, and this activity continues to the present. In addition, construction of the Taylor Highway Bridge has impacted the channel characteristics; thus, this portion of the reach is also not in its natural condition. In spite of these impacts, however, the "Rock Garden" feature described above that limits navigation through this part of the reach appears to be composed primarily of boulders and cobbles that were carried to their present location by the river. While some of this material may have been derived from construction of the bridge, the bulk are of natural origin from upstream sources.

Significant historic in-channel and overbank dredging activity also occurred in the vicinity of Ingle Creek, between RM 6 and RM 7; thus, this area is also not in its natural condition. It is my understanding that most of the dredging activity at this location was also conducted prior to Alaska's statehood. As a result, it is not possible to definitively state whether this area would have been navigable under ordinary and natural conditions.

Beyond these specific areas, I saw little evidence in the field or from the other available information that the remainder of the disputed reach has changed sufficiently as a result of either human activities or natural processes to affect its navigability, either since the date of Alaska's statehood or from its historically, natural condition.

2.4. Hydrologic Conditions in the Disputed Reach

The quantity, duration and timing of flows in the disputed reach are key factors in determining whether the Mosquito Fork was navigable when Alaska became a state. The stream gage at the

Taylor Highway Bridge that is currently operated by the NOAA National Weather Service (NWS) (Gage Designation FMMA2) is the primary source of stage data for the project reach. The NWS has collected a reasonably consistent set of stage measurements at this location during the open-water season since 2006, and they also collected a number of individual stage and discharge measurements during 1999, 2000, 2003 and 2005 (**Table 3**). Stage measurements prior to 2013 were taken manually using the wire-weight gage that is located on the upstream side of the bridge. In 2013, an automated stage recorder was installed on the downstream side of the bridge and hourly readings are available from this instrument for most of the 2013 and 2014 open-water seasons. The BLM also collected data at this location between 1991 and 2004, and the USGS collected a series of measurements during the 1950s and early 1960s.

2.4.1. Analysis of Taylor Highway (FMMA2) Gage Data

The BLM used the available paired flow and stage measurement data to develop rating curves for the Taylor Highway gage, and they used these curves to convert the measured stages for the period from 1998 through 2004 to discharges. Since measured stage data were not available for all of the days in the open-water season, they used an interpolation procedure to fill in the missing days to provide a complete flow record for these years.

Other flow data in the disputed reach include a series of measurements made by the State at various locations along the reach to support their claims in this case in 2009, 2011, 2012 and 2013, and a series of measurements made by the BLM and Tetra Tech to support this study in 2013. While these individual measurements are useful for understanding conditions at the time of the data collection, and to a limited extent, variations in flow along the reach, they do not provide a basis for assessing the long-term flow conditions in the river that are key to assessing whether the river was navigable under ordinary and natural conditions at the date of statehood.

In their hydrology report for this case (Hill et al., 2014), the State presented flow duration curves that are intended to illustrate the percentage of time different levels of discharge were equaled or exceeded on an average annual basis during the May through September period. These flow duration curves are misleading with respect to the question of navigability because they do not reflect the highly episodic nature of flow events during the open-water season that are driven by individual rainstorms.

Although the specific method for developing the curves is not clearly explained, the curves may also be inaccurate because they appear to be based on incomplete records. The caption for Figure 31 in Hill et al. (2014) indicates that the curve is based on "1,617 daily average values for May-September" for the period-of-record data set that appears to include the measurements taken by the USGS in the mid-1950s and early 1960s, the 1999 through 2012 measurements taken by the NWS, and an additional 1,377 daily average values for May-September from the BLM 1996 through 2004 data set. The number of daily average values in the BLM data set makes up less than 45 percent of the days in the indicated period, and the NWS data set contains 672 measured stages in 2006 through 2012, or less than 30 percent of the available days (Table 3). Hourly readings are available for 110 days in 2013 and 101 days in 2014, or about 40 percent of the days.

To overcome the above issues, a complete record of flows for the May through September period from 1996 through 2004 and 2006 through 2014 was developed from the available data. (Sufficient data are not available to develop a similar record for 2005.) The BLM 1996 through 2004 record, discussed above, was used directly in the analysis. For the subsequent period from 2006 through 2014, Tetra Tech converted the available stage records to discharge using the latest

Table 3. Number of available stage and discharge (in parentheses) measurements at the Taylor Highway Bridge.

Year	U.S. Geological Survey	Bureau of Land Management	National Weather Service	Alaska Department of Natural Resources
1954	1 (1)			
1957	1			
1959	3 (3)			
1963	1 (1)			
1964	1 (1)			
1987				21 (4)
1989		1 (1)		
1990		3 (3)		
1991		27 (1)		
1992		26 (2)		
1993		53 (1)		
1994		42 (3)		
1995		70 (3)		
1996		56 (2)		
1997		46 (2)		
1998		41 (3)		
1999		53 (3)		
2000		44 (2)		
2001		32 (2)		
2002		30 (3)		
2003		43 (2)		
2004		15 (2)		
2005			2	
2006			80	
2007			105	
2008			102	
2009			108	14 (14)
2010			114	
2011			81	9 (9)
2012			82	4 (4)
2013*			110	1 (1)
2014*			101	

*NWS - Number of days of available data in 2013 and 2014. Data collected hourly using automated recorder.

(2004) version of the stage-discharge rating curve from the BLM analysis (**Figure 42**), and then filled in the missing days using the Maintenance of Variance Extension (MOVE.1) technique (Helsel and Hirsch, 2002; Hirsch et al., 1993). The Fortymile River near Steele Creek (USGS Gage No. 15348000) was used as the index station for the MOVE.1 analysis. The BLM stage-discharge rating curve is based on 10 paired stage-discharge measurements taken at the Taylor Highway Bridge between June 1992 and September 2004. The reason for selecting these specific measurements as the basis for the curve is not known, but they are very consistent with the other available measurements, and the resulting rating curve is also very consistent with the complete data set. The small variation between the rating curve and the data points that were collected over a 13-year period also indicates that the channel geometry and hydraulic conditions in this area did not change during the measurement period. The rating curve is also very consistent with more recent measurements taken by the State in 2009 and 2012 and by Tetra Tech on August 2013. Based on these data, it appears that the river is relatively stable in this area, and it is reasonable to apply the BLM rating curve to the recent stage data.

The Fortymile River near Steele Creek gage provides an excellent index station because it is located in the same stream system, about 20 miles downstream from the mouth of the Mosquito Fork, and thus, includes the Mosquito Fork flows. As a result, the flows at this location should typically respond to the same basin-wide hydrologic events. Comparison of the hydrographs at the two gages for concurrent periods indicates that the flow patterns are, in fact, very similar (**Figure 43**).In addition, the correlation coefficient (R^2) between the corresponding mean daily flows at the two gages is 0.88, indicating strong correlation between the flows at the two gages (**Figure 44**). In applying the MOVE.1 analysis to fill in the Mosquito Fork data, an initial estimate of the each missing discharge was made using the MOVE.1 equation. In spite of the strong correlation, there is still sufficient scatter in the individual data points to create discontinuities between the estimated values and adjacent (in time) recorded values. To avoid these discontinuities where this occurred, the estimated flows were scaled to the closest (in time) measured discharges at the Mosquito Fork gage. The resulting flow records provide reasonable estimates of the flows in the Mosquito Fork on the days with missing records (**Figure 45**).

To provide a general characterization of the range and duration of flows during the open-water season on an average annual basis, a flow duration curve was developed using the complete flow record (**Figure 46**). This curve indicates that mean daily flows at Taylor Highway are less than 1250 cfs about 75 percent of the time, less than 570 cfs about half the time between May 1 and September 30, less than 280 cfs about 25 percent of the time and less than 120 cfs about 10 percent of the time (Figure 46). The curve shown in Figure 46 indicates slightly higher discharges for a given exceedence frequencies than the curve presented in Hill et al. (2014). For example, the Hill et al. (2014) curve indicates that the median discharge from the incomplete, recorded data is about 475 cfs, and flows are less than 230 cfs about 25 percent and less than 100 cfs about 10 percent of the time, respectively.

As noted above, the flow-duration curve does not reflect the episodic nature of the higher flows, and this must be considered in relation to the question of navigability because the discharge in the river typically rises above the base flows only in response to individual rainstorms and then recedes back to near the baseflows after the storm ends. As a result, the actual time periods during which flows may be sufficiently high for navigation typically occur for relatively short periods throughout the open-water season that are separated by periods of much lower flows. For example, during 2007, the flows were less than 500 cfs for 58 of the 153 days (~38 percent) during the May through September period, but this occurred during 6 different occasions for durations of one to 16 days that were scattered more or less uniformly throughout the period from June 21 through the end of September

(Figure 45). The implications of this issue will be addressed in more detail in a subsequent section of this report.

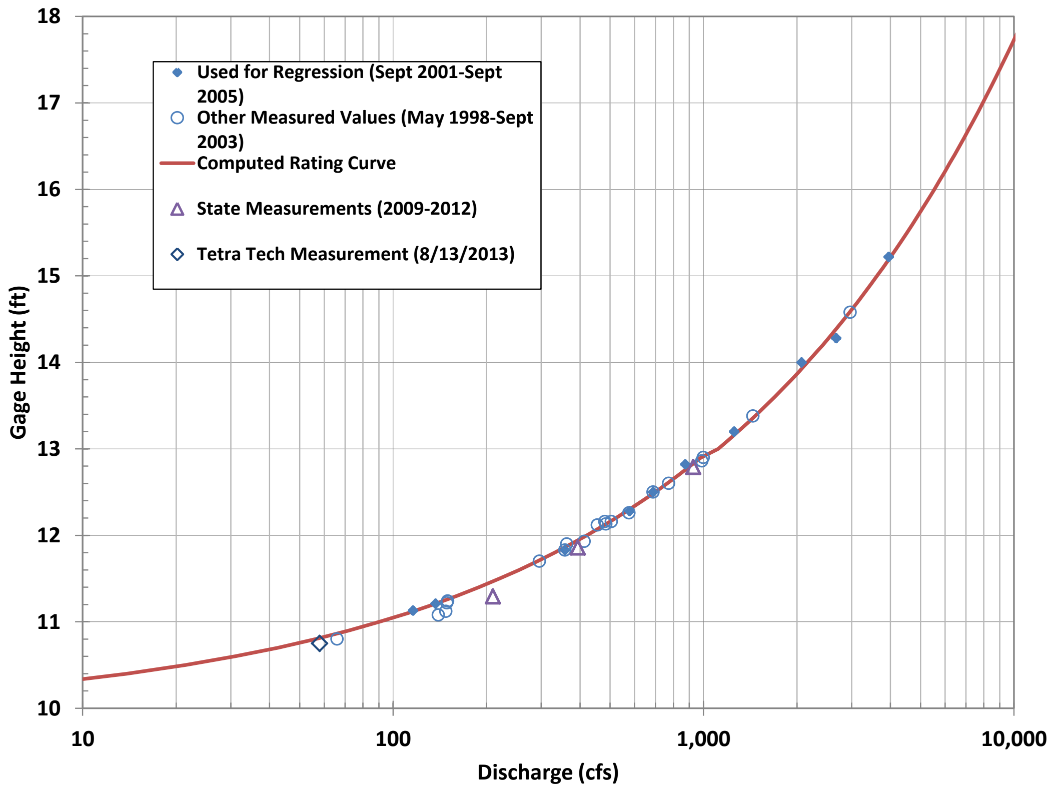


Figure 42. Stage discharge rating curve for the Mosquito Fork River at Taylor Highway Bridge developed by the BLM using stage-discharge measurements collect in 1999 through 2004. Also shown are the data points used to develop the rating curve and other available paired stage-discharge data points.

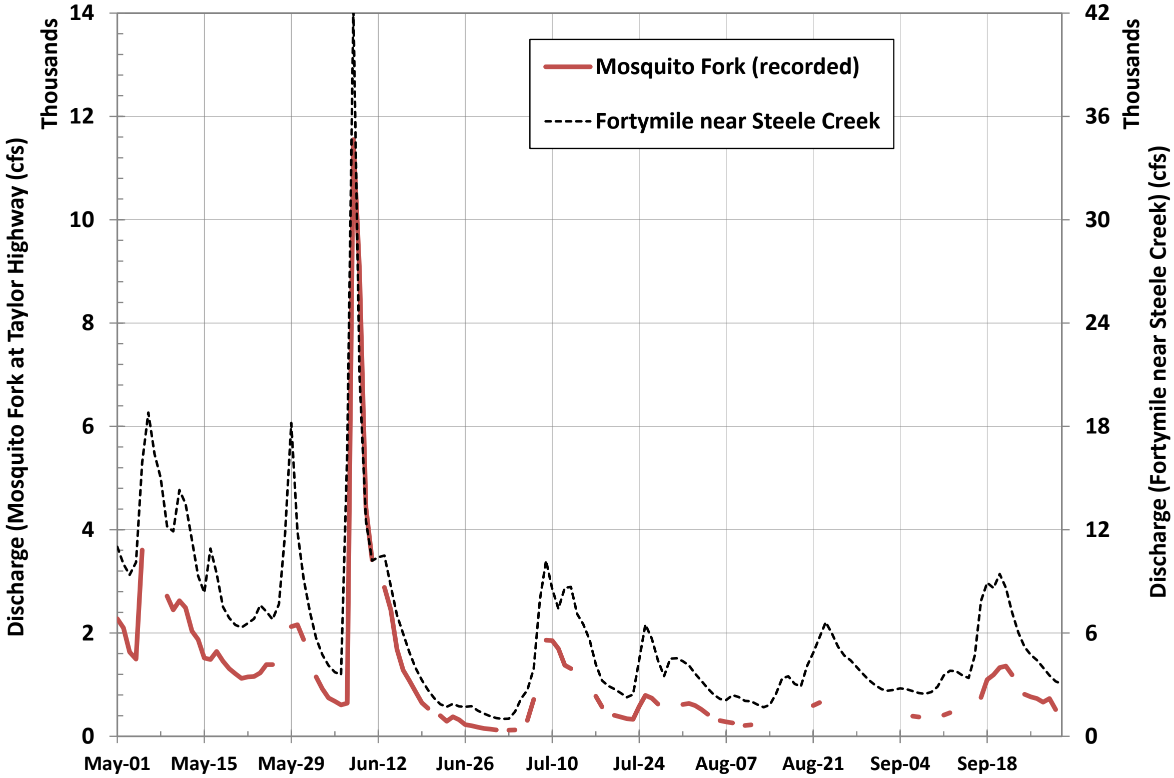


Figure 43. Recorded discharges at Fortymile River near Steele Creek and Mosquito Fork River at Taylor Highway during the 2007 open-water season showing the typical correspondence in flow patterns between the two locations.

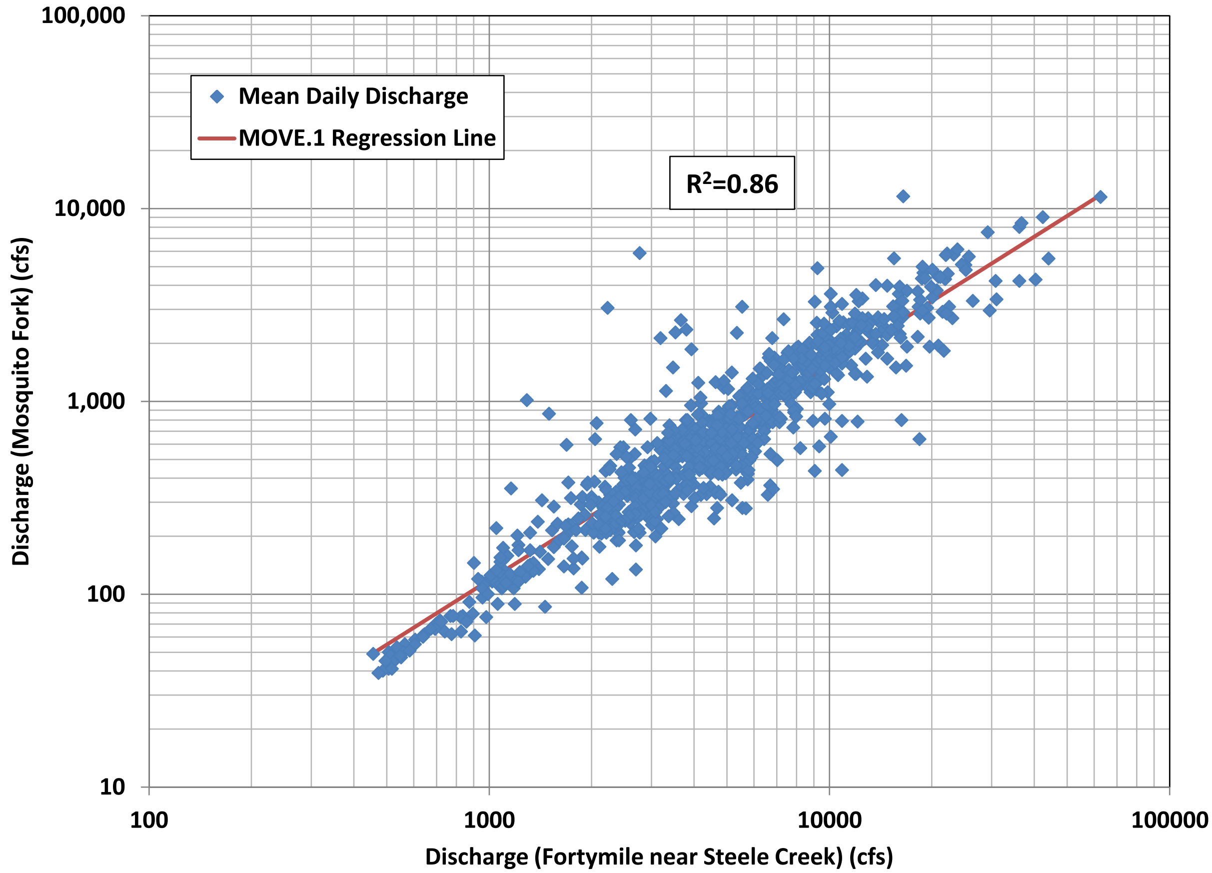


Figure 44. Concurrent measured mean daily flows at Fortymile River near Steele Creek (USGS Gage No. 15348000) and Mosquito Fork River at Taylor Highway (NWS Gage FMMA2) for the period from May 1, 2006 through September 30, 2014. Also shown is the MOVE.1 regression line that was used to fill in the missing discharges in the Mosquito Fork 2006 through 2004 record.

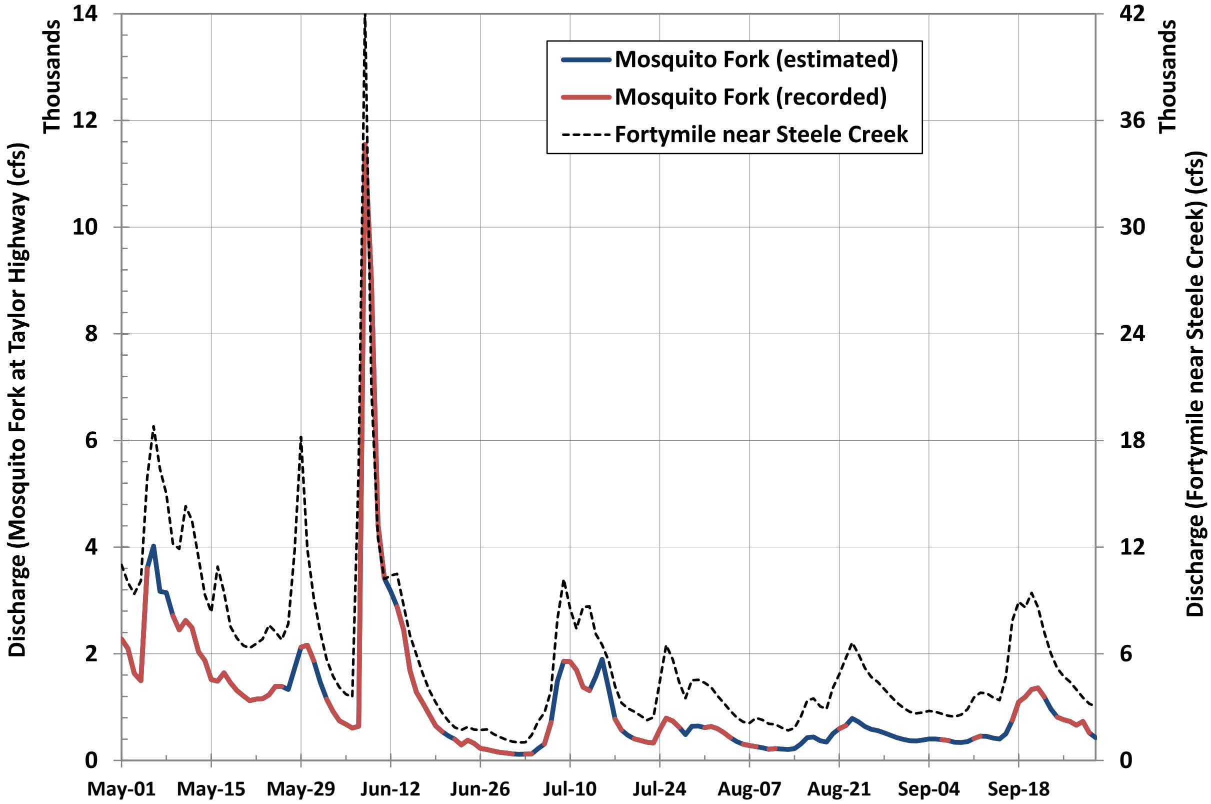


Figure 45. Recorded discharges at Fortymile River near Steele Creek and Mosquito Fork River at Taylor Highway during the 2007 open-water season. Also shown are the discharges for the missing days at the Mosquito Fork gage estimated using the modified MOVE.1 technique.

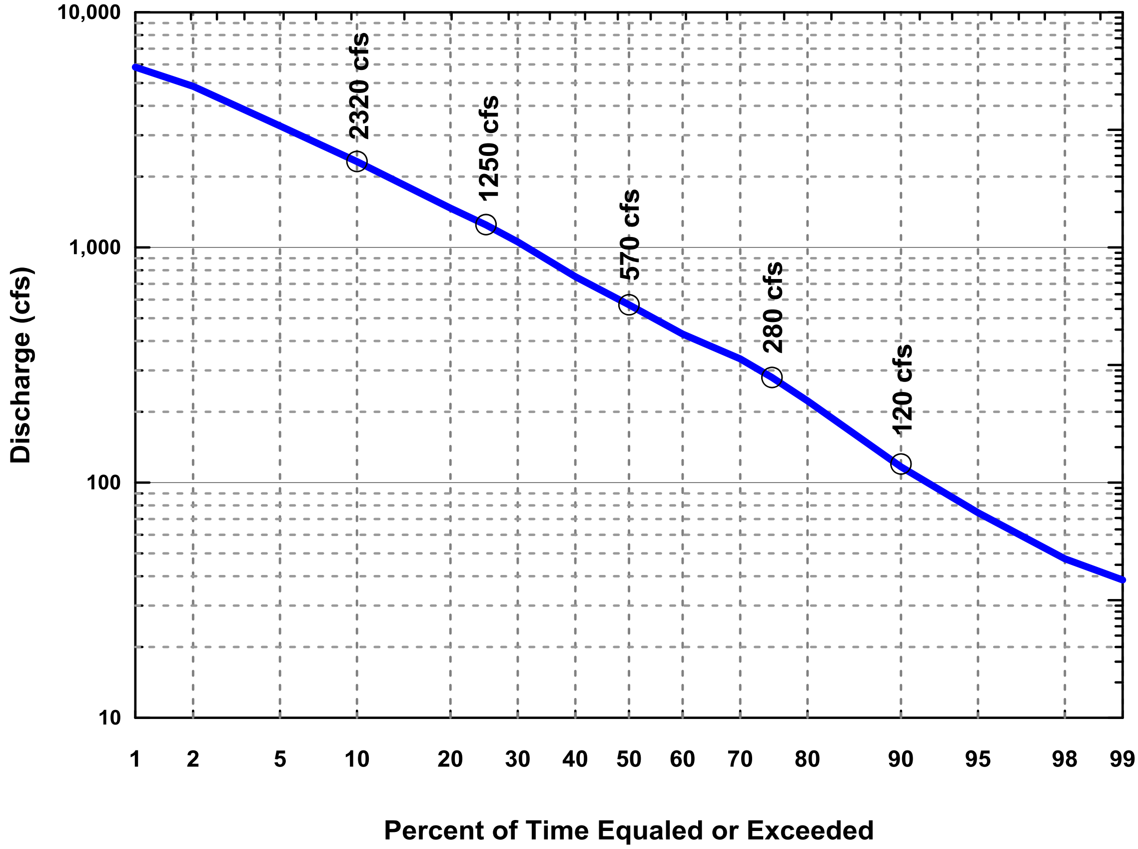


Figure 46. Mean daily flow-duration curve for the period from May 1 and through September 30 based on the filled-in record for the Mosquito Fork River at Taylor Highway Bridge.

2.4.2. Estimated Flows at Locations Upstream from Taylor Highway

The flow record for the Taylor Highway gage discussed in the previous section strictly applies only to Study Site P9. Flows at sites upstream from P9 will vary from those at the gage, primarily because of the reduction in contributing drainage area, although the spatial distribution of the precipitation during any particular event, among other factors, will also have an effect. The USGS (Wiley and Curran, 2003) developed relationships that can be used to predict the magnitude of mean daily flows at ungaged sites for a range of durations using the data from the active stream gages in different regions of Alaska. A set of relationships were developed for high-flow conditions (i.e., durations of exceedence ranging from 1 to 15 percent) and a separate set of low-flow relationships (i.e., exceedance durations ranging from 50 percent to 98 percent) were developed for each of the three months of July, August and September (**Figure 47**). For Region 5, in which the Mosquito Fork is located, the independent variables in high-flow relationships are the drainage area (A), and mean annual precipitation (P), and the low-flow relationships also include the mean basin elevation (E).

The relationships shown in Figure 47 were used to estimate the flow records for each of the upstream sites based on the Taylor Highway flow record. The drainage area and mean basin elevation for each of the sites was developed from a Digital Elevation Model (DEM) obtained from the 7.5"X7.5", 10 m National Elevation Data set (ftp://ftp.ftw.nrcs.usda.gov/), and the mean annual precipitation was estimated from the Precipitation Map for Alaska obtained from Brabets (1998) (http://agdc.usgs.gov/data/usgs/water). The resulting drainage areas ranged from 466 mi^2 at the most upstream Site PN to 1,112 mi^2 at the Taylor Highway Bridge (Site P9 and FMMA2 gage), and the mean basin elevations ranged from 2,900 feet at Site PN to 2,957 feet at the Taylor Highway Bridge (**Table 4**). Mean annual precipitation was relatively constant among the sites, ranging from 15.1 inches at Site PN to 15.3 inches at the Taylor Highway Bridge.

Wiley and Curran (2003) provide recommended procedures for applying the relationships to two categories of ungaged sites: (1) sites on streams for which no gage data are available and (2) ungaged sites on one of the streams used to develop the relationships. An attempt to apply Method 2 was made by computing the relevant flow duration statistics for the complete Mosquito Fork record and then applying the procedure using those statistics for the gage site. This resulted in unreasonable results for many of the sites. As an alternative that produced more reasonable results, ratios of the expected discharge at the ungaged site to the Taylor Highway gage ($R_{ungaged}$) were developed using the following relationship:

$$R_{Ungaged} = \left(\frac{A_U}{A_G}\right)^b \left(\frac{P_U}{P_G}\right)^c \left(\frac{E_U}{E_G}\right)^d$$

Where the subscripts *U* and *G* refer to the ungaged, upstream site and G refers to the corresponding parameter at the Taylor Highway gage, and the exponents *b, c* and *d* are taken from the relevant equation from the tables in Figure 47. (Note that the value of d is 0 for the high-flow equations.) The resulting ratios at each of the sites are essentially constant for all of the relationships, ranging from 0.98 for the most downstream ungaged Site P8 to 0.37 at Site PN (**Figure 48**). Flow-duration curves were developed for each of the sites by scaling the curve for the Taylor Highway gage by the appropriate ratio (**Figure 49**, **Table 5**). The resulting curves indicate that the median (50^{th} percentile) discharge ranges from about 200 cfs at Site PN to about 560 cfs at Site P8. Other values can be read from the table or figure.

Table 2. Estimating equations for annual high-duration flows in Regions 1–7, Alaska and conterminous basins in Canada —*Continued*

[**Estimating equation**: *O-Sn*, daily mean discharge for the water year October–September having an *n*-percent exceedance probability, in cubic feet per second; *A*, drainage area, in square miles; *P*, mean annual precipitation, in inches]

Estimating equation	Coefficient of determination	Standard error of estimate, in percent
Region 5 (34 streamflow gaging stations)		
$O\text{-}S15 = 6.391 \times 10^{-3} A^{1.106} P^{1.779}$	0.96	37
$O\text{-}S10 = 8.746 \times 10^{-3} A^{1.104} P^{1.751}$	0.96	38
$O\text{-}S9 = 9.570 \times 10^{-3} A^{1.103} P^{1.736}$	0.96	39
$O\text{-}S8 = 1.067 \times 10^{-2} A^{1.102} P^{1.718}$	0.96	39
$O\text{-}S7 = 1.215 \times 10^{-2} A^{1.100} P^{1.696}$	0.95	40
$O\text{-}S6 = 1.397 \times 10^{-2} A^{1.098} P^{1.668}$	0.95	41
$O\text{-}S5 = 1.579 \times 10^{-2} A^{1.097} P^{1.648}$	0.95	42
$O\text{-}S4 = 1.977 \times 10^{-2} A^{1.091} P^{1.609}$	0.95	43
$O\text{-}S3 = 2.508 \times 10^{-2} A^{1.087} P^{1.561}$	0.94	45
$O\text{-}S2 = 3.769 \times 10^{-2} A^{1.081} P^{1.468}$	0.94	46
$O\text{-}S1 = 5.859 \times 10^{-2} A^{1.078} P^{1.372}$	0.93	50

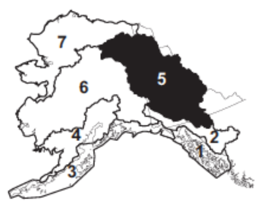


Table 4. Estimating equations for monthly low-duration flows for July, August, and September in Regions 1–7, Alaska and conterminous basins in Canada—*Continued*

[**Estimating equation**: MONTH*n*, *n* percent low-duration flow for the indicated month, in cubic feet per second; *A*, drainage area in square miles; *P*, mean annual precipitation, in inches; *E*, mean basin elevation, in feet above sea level; *G*, area of glaciers, in percentage of total basin area. >, greater than]

Estimating equation	Coefficient of determination	Standard error of estimate, in percent
Region 5 (35 streamflow gaging stations)		
$JULY98 = 4.916 \times 10^{-10} A^{1.250} P^{1.885} E^{1.705}$	0.94	64
$JULY95 = 7.603 \times 10^{-10} A^{1.232} P^{1.687} E^{1.754}$	0.94	62
$JULY90 = 9.022 \times 10^{-10} A^{1.227} P^{1.687} E^{1.753}$	0.94	59
$JULY85 = 1.737 \times 10^{-9} A^{1.220} P^{1.716} E^{1.683}$	0.94	57
$JULY80 = 5.006 \times 10^{-9} A^{1.201} P^{1.695} E^{1.590}$	0.95	54
$JULY70 = 1.964 \times 10^{-8} A^{1.190} P^{1.689} E^{1.452}$	0.95	51
$JULY60 = 7.896 \times 10^{-8} A^{1.181} P^{1.682} E^{1.306}$	0.96	48
$JULY50 = 3.238 \times 10^{-7} A^{1.171} P^{1.642} E^{1.170}$	0.96	45
$AUG98 = 2.021 \times 10^{-10} A^{1.271} P^{1.910} E^{1.755}$	0.93	68
$AUG95 = 1.685 \times 10^{-9} A^{1.257} P^{1.833} E^{1.552}$	0.94	63
$AUG90 = 4.340 \times 10^{-9} A^{1.238} P^{1.644} E^{1.535}$	0.94	60
$AUG85 = 8.224 \times 10^{-9} A^{1.231} P^{1.621} E^{1.481}$	0.94	58
$AUG80 = 3.519 \times 10^{-8} A^{1.219} P^{1.625} E^{1.324}$	0.95	55
$AUG70 = 5.765 \times 10^{-7} A^{1.198} P^{1.598} E^{1.028}$	0.95	52
$AUG60 = 1.740 \times 10^{-3} A^{1.164} P^{1.911}$	0.95	52
$AUG50 = 2.499 \times 10^{-3} A^{1.161} P^{1.819}$	0.95	49
$SEPT98 = 2.076 \times 10^{-9} A^{1.234} P^{1.806} E^{1.516}$	0.94	59
$SEPT95 = 2.731 \times 10^{-4} A^{1.188} P^{2.272}$	0.94	60
$SEPT90 = 5.742 \times 10^{-4} A^{1.182} P^{2.057}$	0.95	53
$SEPT85 = 1.009 \times 10^{-3} A^{1.171} P^{1.917}$	0.95	52
$SEPT80 = 1.188 \times 10^{-3} A^{1.170} P^{1.886}$	0.95	49
$SEPT70 = 1.757 \times 10^{-3} A^{1.166} P^{1.793}$	0.96	46
$SEPT60 = 2.110 \times 10^{-3} A^{1.162} P^{1.775}$	0.96	43
$SEPT50 = 2.506 \times 10^{-3} A^{1.158} P^{1.754}$	0.97	41

Figure 47. High- and low-flow estimating equations for Alaska Region 5 used to estimate the flow records at Study Sites P1 through P8 and PN (from Wiley and Curran, 2003).

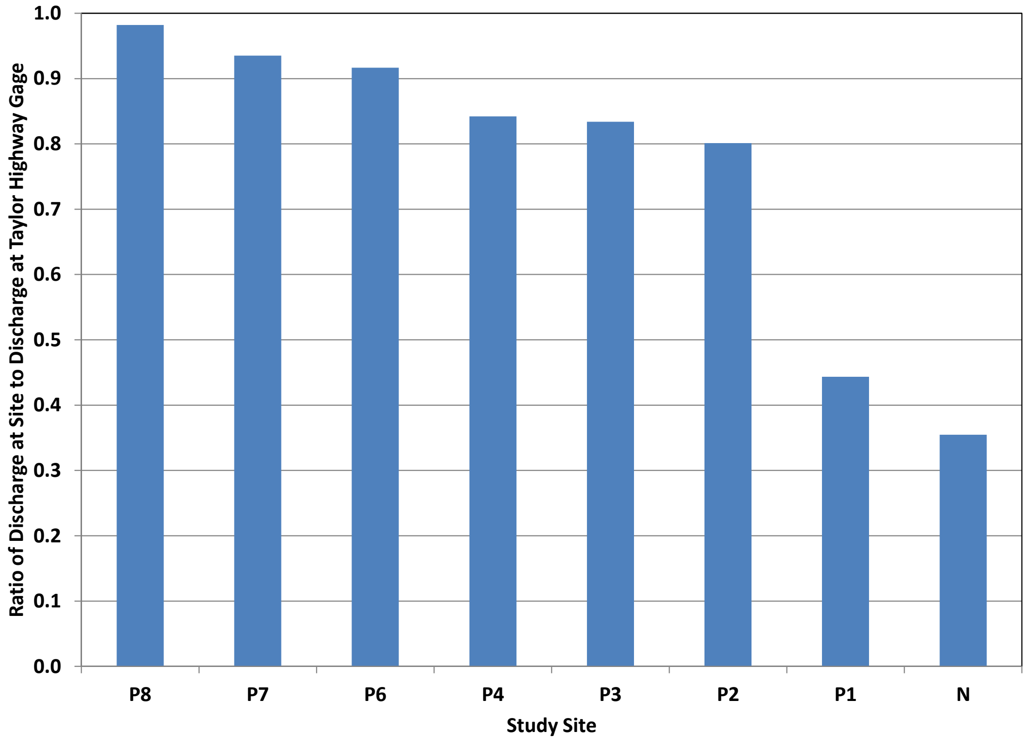


Figure 48. Ratio of expected discharge at each of the study sites to the corresponding discharge at the Taylor Highway Gage (and Site P9) based on the relationships from Wiley and Curran (2003).

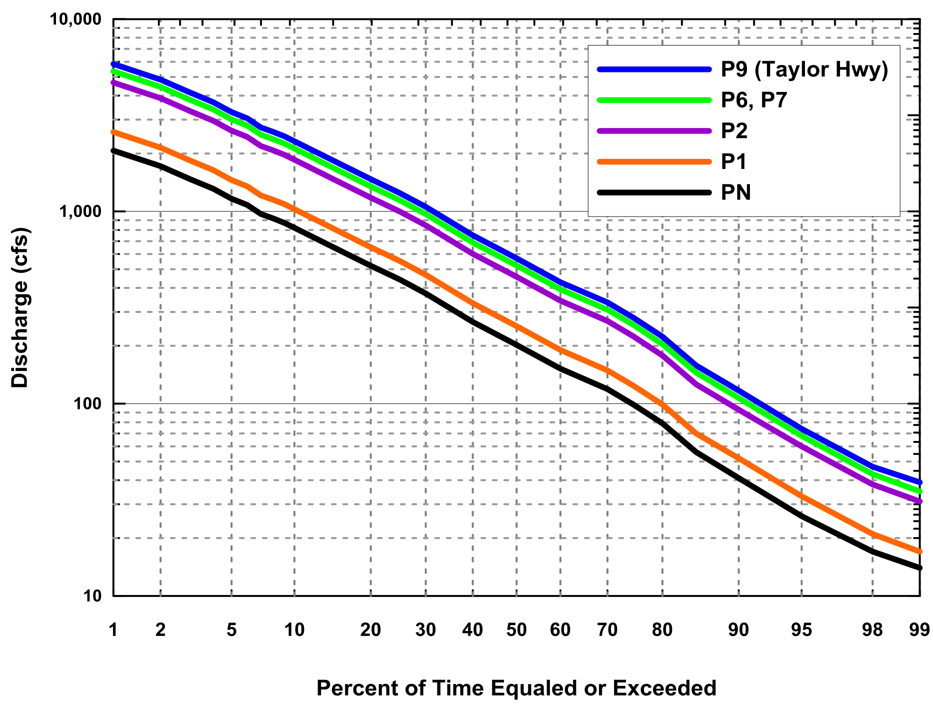


Figure 49. Mean daily flow-duration curves for the study sites developed by scaling the curve for the Taylor Highway gage by the ratios in the previous figure. The curves for Sites P3 and P4 are not shown as they fall between the P2 and P6, P7 curve. Site P8 is not shown because it is essentially the same as P9.

Table 5. Summary of estimated discharges for 25-, 50- (i.e., median), 75- and 90-percent exceedence durations.

Site	Exceedence Duration			
	90%	75%	50%	25%
P9	120	280	570	1,250
P8	115	275	560	1,223
P7	109	262	533	1,165
P6	107	257	523	1,142
P4	98	236	480	1,049
P3	97	233	475	1,038
P2	93	224	457	998
P1	52	124	253	552
PN	41	99	202	442

2.5. Hydraulic Conditions in the Disputed Reach

Hydraulic models were developed for each of the sites using the field data to provide a means of quantifying the flow depths and other relevant hydraulic conditions over the range of flows in the river. The results were then compared with conditions that would have been necessary to navigate the boats that were commonly in use for trade and travel at the time of Alaska's statehood (Section 2.6).

The modeling was performed using the US Army Corps of Engineers Hydrologic Engineering Center's River Analysis System (HEC-RAS) version 4.1 software, a standard among the engineering community for analyzing one-dimensional hydraulic conditions in rivers and open channels. Input for the model consists of a series of cross section profiles that represent the topography of the river, the discharge(s) to be modeled, downstream boundary conditions for each discharge, and a variety of hydraulic parameters that are integral to the equations being solved by the model. Key among these parameters is the channel roughness, typically specified by the semi-empirical Manning roughness coefficient.

Model cross sections for the portion of the river within the bankfull channel were taken directly from the field survey data. The analysis primarily focused on the range of flows that are contained within the limits of the surveyed cross sections; however, most of the cross sections were extended into the overbanks based on the USGS 10M DEM data, field notes and photograph to provide reasonable estimates of the conditions at higher flows that would inundate areas outside the surveyed cross sections. Hydraulic conditions from the model for these higher flows should be used with caution, as the topography is only an approximation of the actual topography at the sites. The approximated topography has no impact on model results for the range of flows that are of primary concern to assessing navigability of the river. To facilitate accurate representation of the distance between the cross sections, the primary flow paths through each site were digitized in ArcGIS using a combination of the field data, photos and field notes. Ineffective flow areas were assigned to the cross-sections, as appropriate, where field observations identified topographic conditions or other features that would prevent downstream flow conveyance.

The step-backwater algorithm used by HEC-RAS requires specification of the water-surface elevation at the downstream cross section for each discharge to be modeled using one of several available methods. For this analysis, the downstream water-surface elevation was computed in the model based on the local slope and estimated roughness values, insuring that the predicted water-surface elevation matched the measured water-surface for the discharge at the time of the survey. The portion of the disputed reach in which the downstream eight study sites are located is generally a riffle-pool reach, with some plane-bed sections, following the classification system of Montgomery and Buffington (1993, 1997). The downstream cross section at most of the sites is located at the head of a pool in the overall riffle-pool structure. Where this occurs, the energy gradient at the low flows when the surveys were conducted is much flatter than at higher flows, a phenomenon that has been long-recognized in riffle-pool channels (Lisle, 1979; Dunne and Leopold, 1978, **Figure 50**). Lisle (1979) found that the typically-flat energy gradient in the pools at low flows increases with discharge until it approaches the average slope of the longer reach represented by the site at flows approaching the bankfull discharge. Based on this finding, the energy gradient used to establish the downstream boundary condition at each site was transitioned from the calibrated value at low flows to the average gradient through the site at the approximate bankfull discharge using semi-logarithmic interpolation.

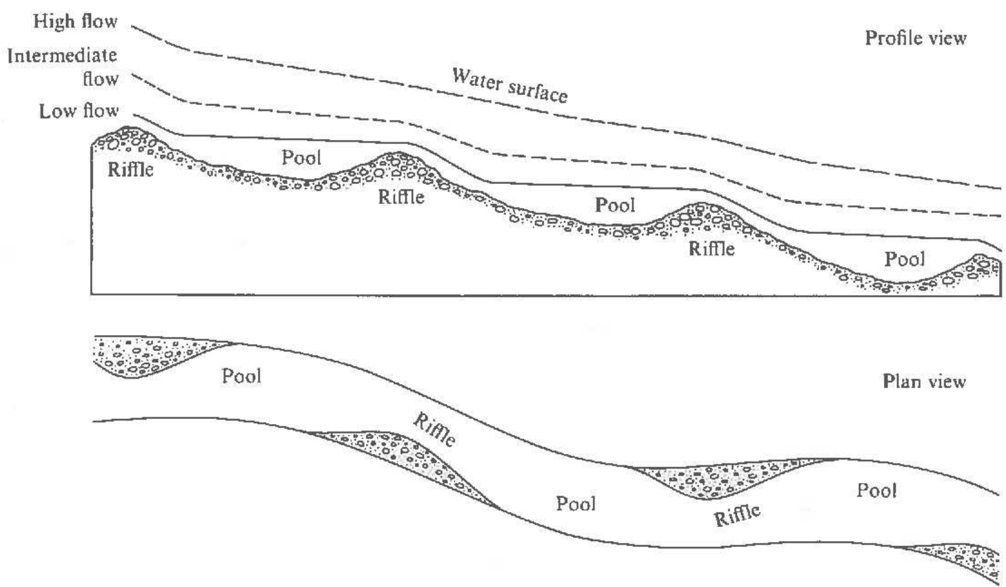


Figure 50. Diagram of longitudinal profile and plan view of a pool-riffle sequence. Water-surface profiles in upper figure represent high, intermediate, and low-flow conditions (Figure 16-20, Dunne and Leopold, 1978).

Specification of the hydraulic roughness values in the models is generally done based on standard references, published relationships, and previous experience with similar streams. When calibration data are available, the initially-specified values are then adjusted within physically reasonable limits so that the predicted and measured water-surface elevations matched within a reasonable tolerance. With the exception of Site 9 that is located at the Taylor Highway Bridge, the only available data with which to directly calibrate the model are the water-surface elevations that were collected during the field surveys when the flows were relatively low (Table 1). The rating curve for the Taylor Highway gage (Figure 42) was used to calibrate the model for Site P9 over the full range of flows.

In coarse-grained rivers such as the Mosquito Fork, the hydraulic roughness (characterized for this study by the Manning *n* roughness coefficient) varies significantly with discharge due to the effects of the large bed material that protrudes into the flow, particularly in the shallow riffles and rapid that would be the most likely impediments to navigation (Ferguson 2007; Bathurst, 2002; Mussetter, 1989; Jarrett 1984). These citations, among others, provide equations that can be used to predict the roughness values based on the characteristics of the modeled reach. Following basic theories of fluid mechanics, the key parameter in most of the roughness equations is the relative submergence of the bed-material particles, quantified by the ratio of the flow depth to a characteristic size of the bed material. The specific size parameter that is used varies by equation, but is typically in the coarse end of the size gradation curve because the larger particles tend to have the greatest impact on hydraulic roughness. The equations from the above-cited references all use the D_{84} (size for which 84 percent of the particles are larger) as the characteristic size.

Tests of the above relationships using the data and model results from Site P9 indicate that the equations from Mussetter (1989) with an adjusted coefficient provide the best calibration when the relative submergence is less than about 4, and the equations from Ferguson (2007) provide the best agreement at higher relative submergence. The agreement between the modeled water-surface and the rating curve for the Taylor Highway gage using this approach is excellent (**Figures 51a and 51b**). Based on the calibration achieved at Site P9, the adjusted equations were then used to estimate the Manning *n* roughness values input to the models, with the D_{84} sizes taken from the coarse riffle gradation curves for each site discussed in Section 2.2. At most of the upstream sites, the riffles are separated by pools in which the relative submergence is significantly greater than 4 over the full range of discharges. In these cases, including all cross sections at Site PN, results from the Ferguson (2007) equation approach a constant value. For these cases, a constant Manning's *n*-value of 0.035 to 0.04 was applied to the affected cross sections for all flows, with the final value selected to provide the best calibration to the relatively low-flow, measured water-surface elevations. The resulting models calibrate very well to the measured water-surface elevations (**Figures 52a through 52i**). Application of the roughness equations to the full range of flows resulted in Manning's *n*-values ranging from 0.035 that is typical of deeper flow in gravel-bed channels to 0.2 for shallow depths that occur at low flows in the riffles.

The resulting models were executed for the range of discharges that have been observed at the Taylor Highway gage. As described in the next section, the results were used, in conjunction with the characteristics of boats that were in customary use in this part of Alaska at the time of statehood to assess whether the Mosquito Fork meets the criteria for navigability.

2.6. Navigability of the Disputed Reach

2.6.1. Boat Characteristics and Required Draft

It is the opinion of C. Michael Brown, the United States' expert historian in this case, that the boats that were typically used or capable of being used for trade and travel on Upper Yukon tributaries at the time of Alaska's statehood were motorized river boats with lengths of at least 28 feet (Brown, 2014, personal communication). These boats were generally constructed from native timber or aluminum, with those made from native timber being the most common in rural areas such as the Fortymile River drainage. It is also Mr. Brown's opinion that it would have been necessary for these boats to carry cargo loads of at least one ton to make their use *a commercial reality* (PPL Montana, p24). In contrast, the State's expert historian has opined that even relatively small poling boats carrying loads in the range of 1,000 lb could have been used for commercial navigation (Buzzell, 2014). A variety of information has been provided by Mr. Brown and the State about the physical

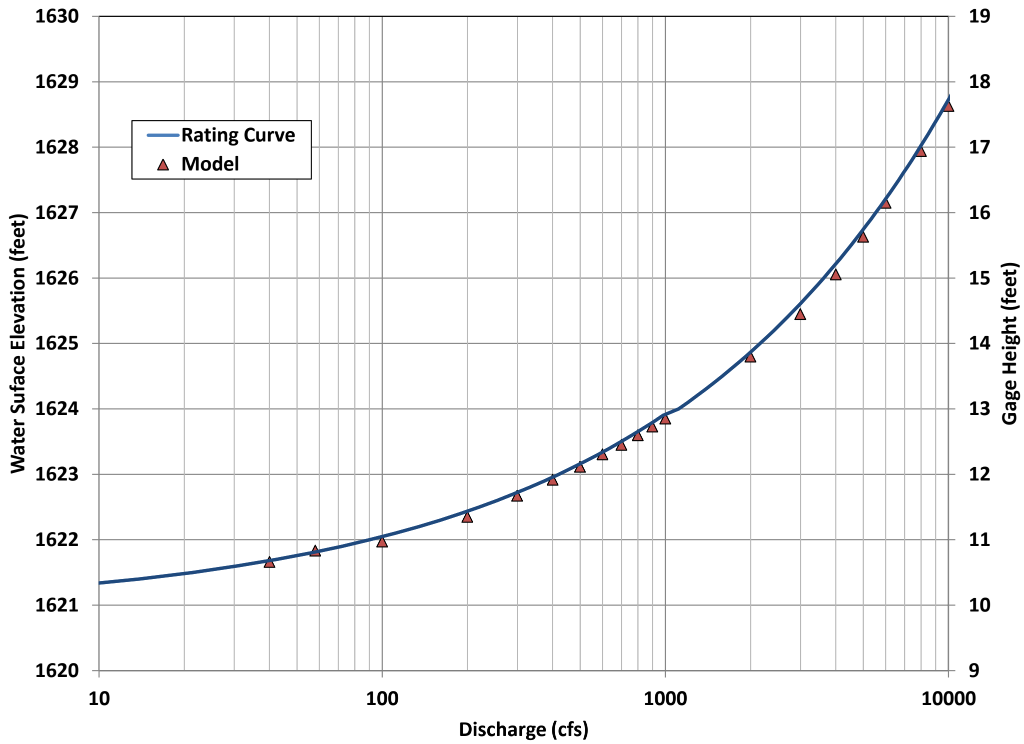


Figure 51a. Discharge versus water-surface elevation rating curve at the Taylor Highway gage and the predicted water-surface elevations from the Site P9 hydraulic model (Gage datum=1,611 feet msl).

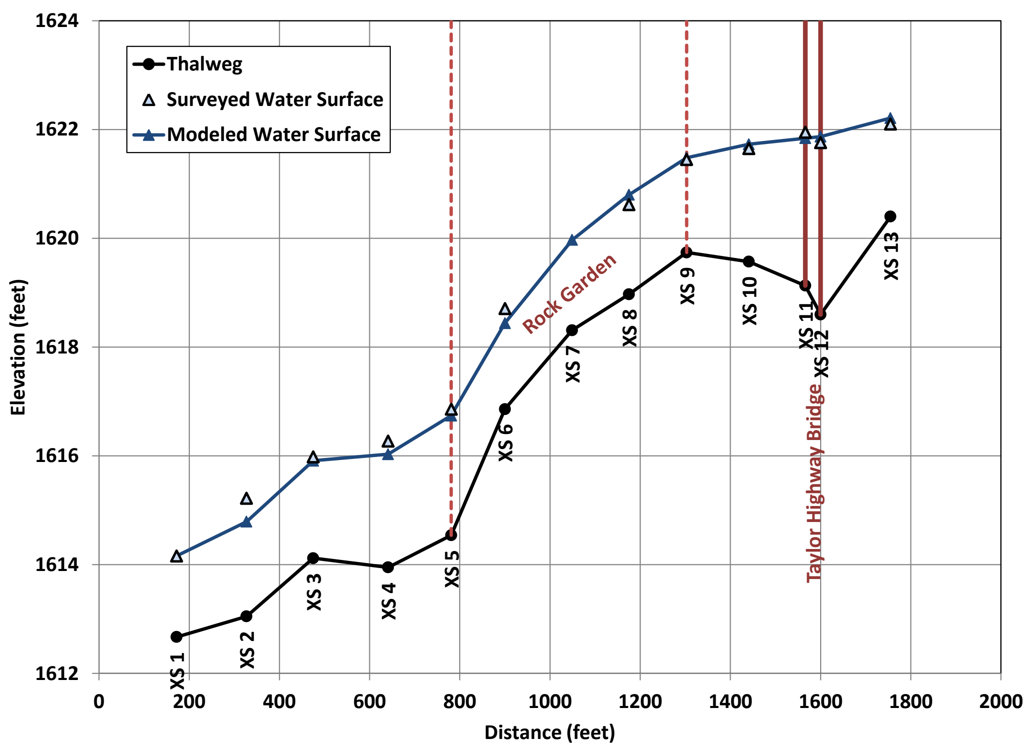
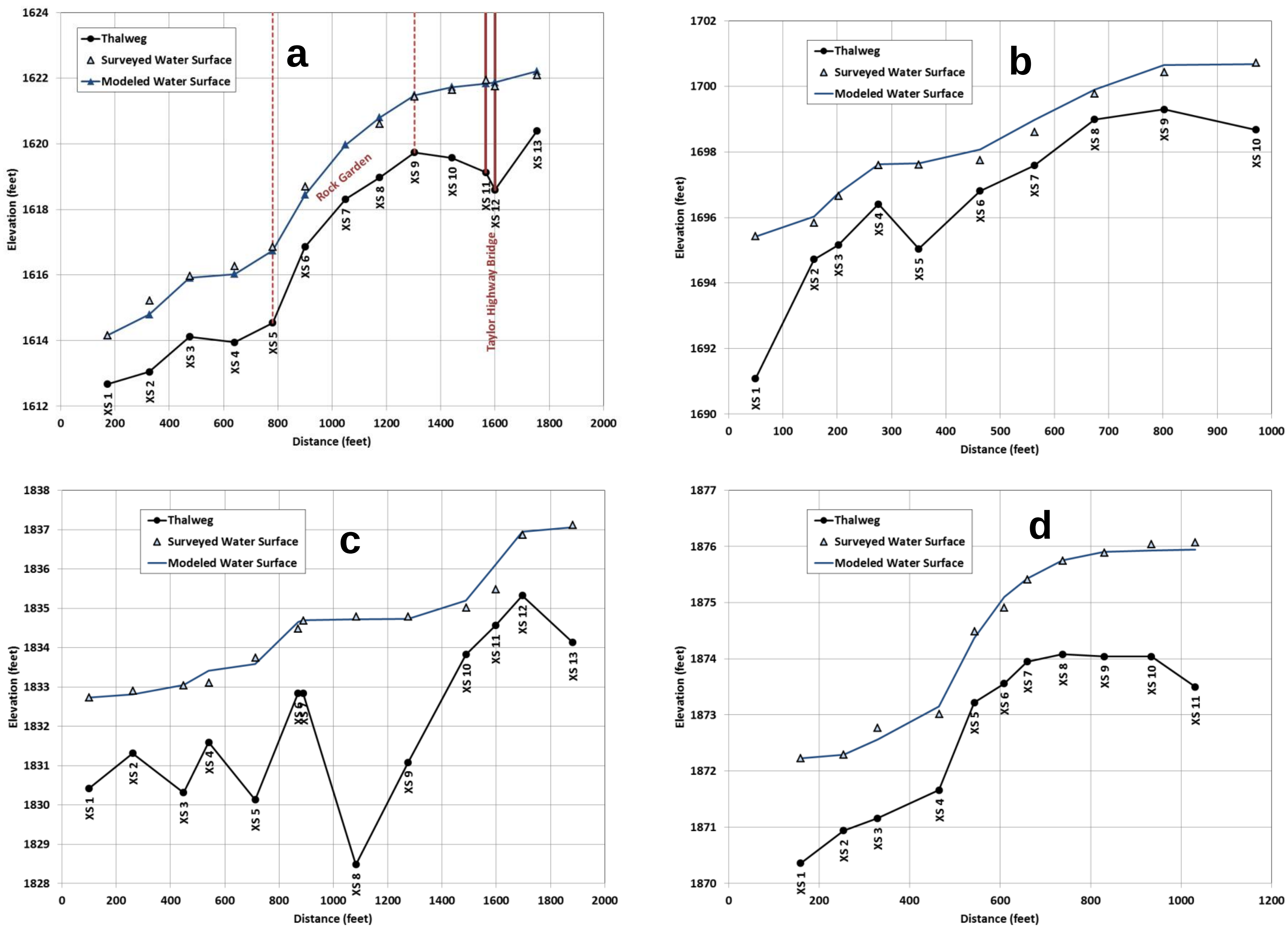
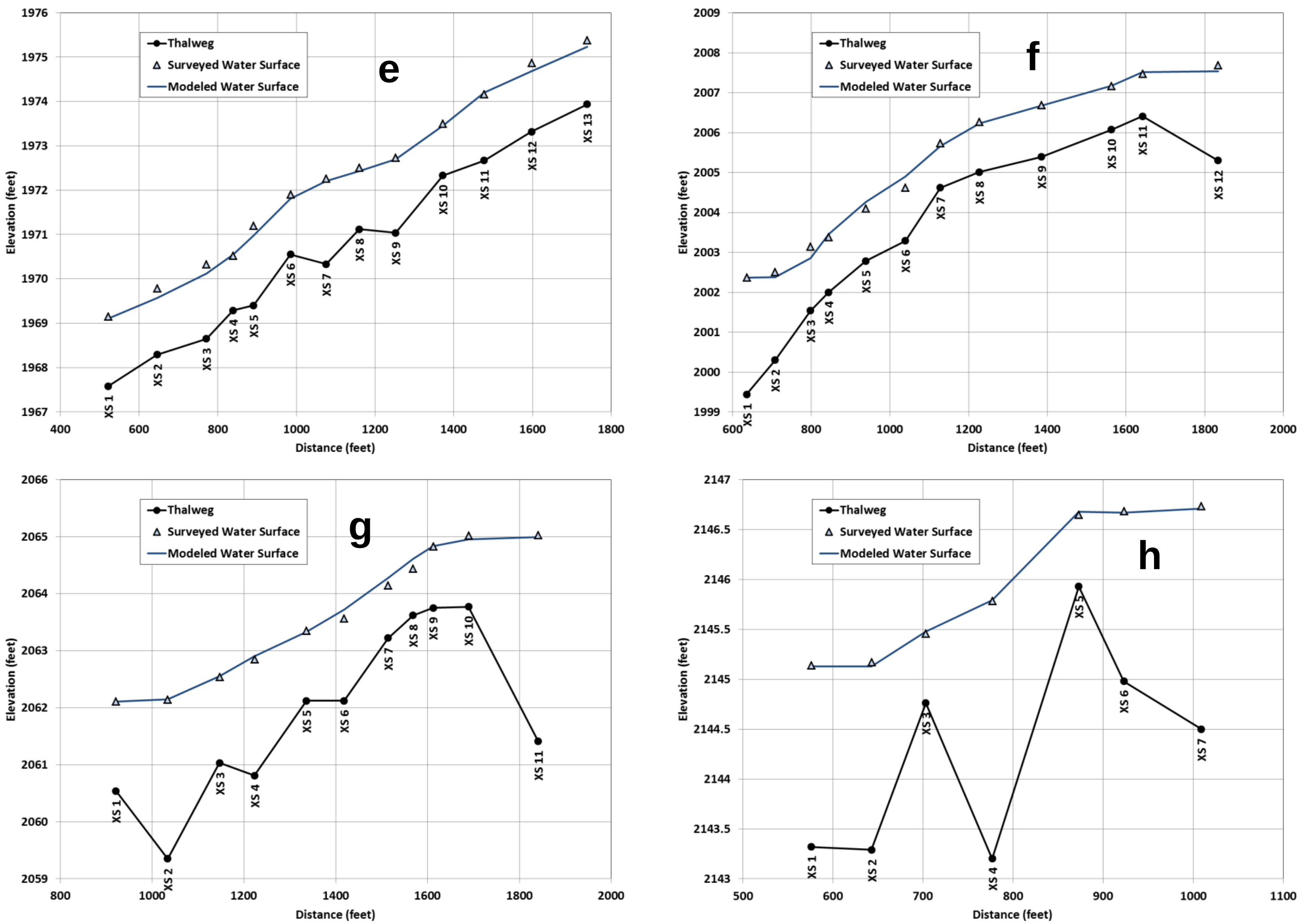


Figure 51b. Thalweg (minimum bed elevation) and modeled water-surface profiles, and measured water-surface elevations for the discharge at the time of the field surveys at Site P9.



Figures 52a-d. Thalweg (minimum bed elevation) and modeled water-surface profiles, and measured water-surface elevations for the discharge at the time of the field surveys: (a) Site P9, (b) Site P8, (c) Site P7, and (d) Site P6.



Figures 52e-h. Thalweg (minimum bed elevation) and modeled water-surface profiles, and measured water-surface elevations for the discharge at the time of the field surveys: (e) Site P4, (f) Site P3, (g) Site P2, and (h) Site P1.

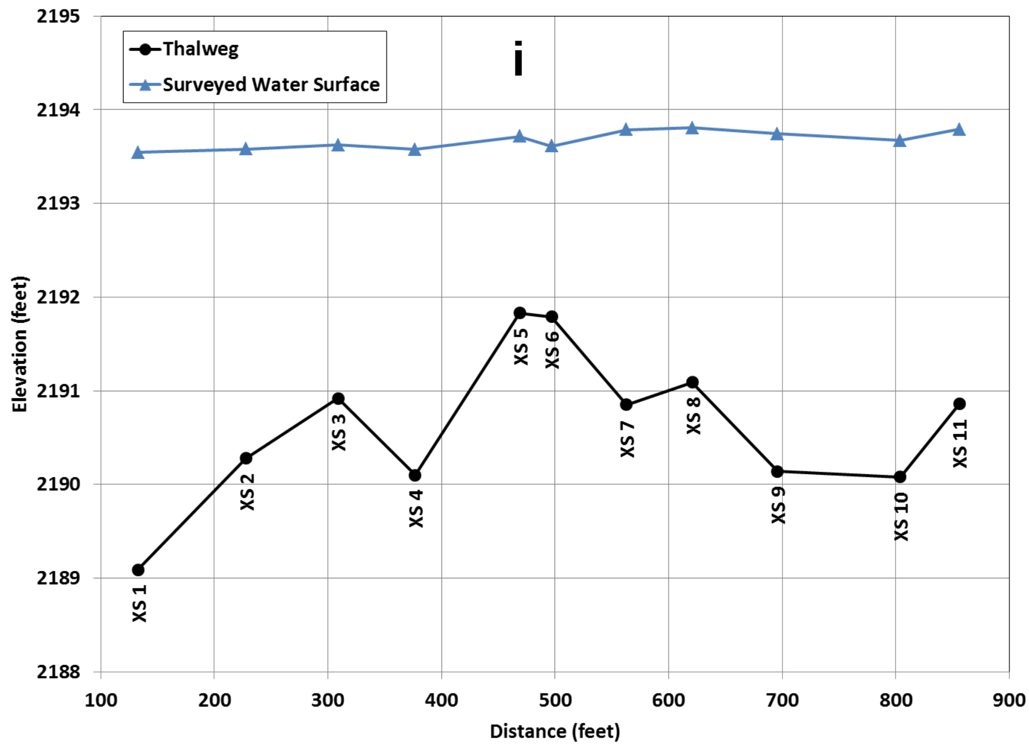


Figure 52i. Thalweg (minimum bed elevation) and modeled water-surface profiles, and measured water-surface elevations for the discharge at the time of the field surveys: (i) Site PN.

dimensions and other characteristics of the range of boats encompassed by Mr. Brown's and Dr. Buzzell's opinions that can be used to assess the hydraulic conditions that would be required to operate these boats. This information includes the drawings of a small (19-foot 8-inch long) poling boat and a 28-foot wooden river boat that were provided as part of the State's document production. While the poling boat is too small to meet the *commercial reality* standard set forth in Montana PPL (C.M. Brown, personal communication, 2014), the draft and operating characteristics of this boat are instructive with respect to conditions necessary to traverse the disputed reach with even small boats carrying relatively small loads. The wooden river boat is consistent with the boats considered by Mr. Brown to be commercially viable.

Although flow velocities, river width, boat maneuverability and the presence of obstacles such as large woody debris (LWD) affect navigability, flow depth in relation to boat draft under loaded conditions is the primary technical factor to be considered in this assessment. Buzzell (2014) provides anecdotal evidence about the typical minimum draft of boats that may have been used in the area, primarily obtained from interviews with individuals who claim to have knowledge of such use. As noted above, most of these boats are too small to be commercially viable. Nonetheless, this evidence, summarized in the following statements, provides information on the minimum drafts that would be necessary to traverse the disputed reach in boats being used for personal (i.e., non-commercial) travel at the date of Alaska's statehood, and set a minimum available draft for which these boat could have been used on the river for any purpose:

Bayless recalled that a typical poling boat could haul a thousand pounds plus the men in the boat. Fully loaded, they displaced about six inches of water.[288] (p89)

> [288]*Summary of interview of William H. Bayless by Rolfe G. Buzzell, Anchorage, July 9, 2014, pp. 9, 13, 20.*

A riverboat with a lift and a 22-horsepower motor could operate in three inches of water unloaded and about nine inches of water with a load of 1,500 pounds.[337] *A flat-bottomed riverboat with a 9-horsepower outboard and small propeller could run in even shallower water. An 800-pound load required six inches of water plus three inches under the boat for a total of nine inches. Operators using a 9-horsepower outboard and lift were also able to drag their boats over gavel bars covered with two to three inches of water.*[338] (p103)

> [337]*Videotaped Deposition of Charles L. Gray, March 7, 2014, pp. 11-12. Jim Reardon, who also used riverboats with lifts in the 1950s, estimated that riverboats drafted 3-4 inches empty and up to six inches carrying a load of 1,500 pounds. Videotaped Deposition of Jim Rearden in Homer, September 18, 2014, Civil Action No. 3:12-cv00114, United States District Court for the District of Alaska, pp. 17-18.*
> [338]*Ibid., pp. 20, 25, 29.*

Blake Gray's flat-bottomed tunnel riverboat drew three inches of water when empty. When carrying two or three people, it would float in three inches of water. When carrying people, hunting gear and a moose, it weighed 1,500 pounds and drew six inches of water. When running at optimal speed on top of the water at 20 miles per hour, the boat drew 1½ to 2 inches of water.[350] (p106)

> [350]*Videotaped Deposition of Charles L. Gray, March 7, 2014, pp. 8-9, 10, 11-12.*

The reasonableness of these statements and the depths required to avoid running aground were assessed by quantifying the boat draft (i.e., depth to which the boat hull, or propeller, in the case of the motorized river boat, project into the water) under a range of loading conditions, based on basic laws of physics and the available information about the size, shape, weight and operating characteristics of the boats.

A drawing provided during the March 7, 2014 deposition of Mr. Charles Gray is believed to be reasonably representative of a typical motorized, wooden river boat (**Figure 53**). This particular boat is 28 feet long, with a 30-inch wide, flat-bottomed hull and top width of 64 inches. The hull curves upward by about 9 inches and narrows to about 14 inches over the front 9 feet of the boat, and a 9-foot long by 14-inch wide tunnel is provided in the stern. The sides of the boat are 16 inches high. These boats were commonly powered by a 35-hp outboard motor that weighs in the range of 140 to 150 lb although there is evidence that 50-hp outboard motors that weigh 190 to 200 lb were also being used (B. Kennedy, personal communication, 2014).

The only known drawing of a poling boat was provided as part of the State's document production (**Figure 54**). Buzzell (2014) described this boat, as follows:

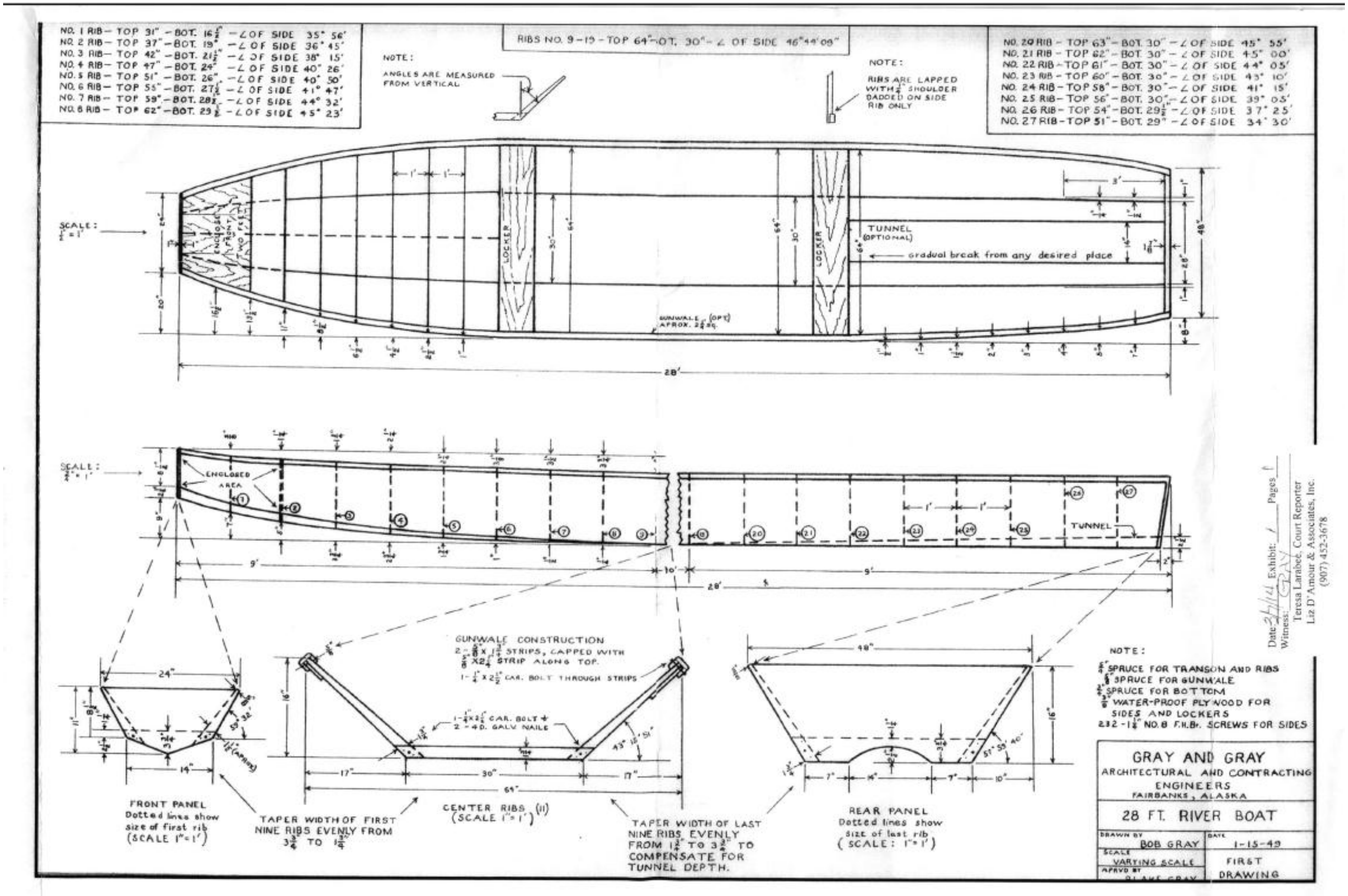
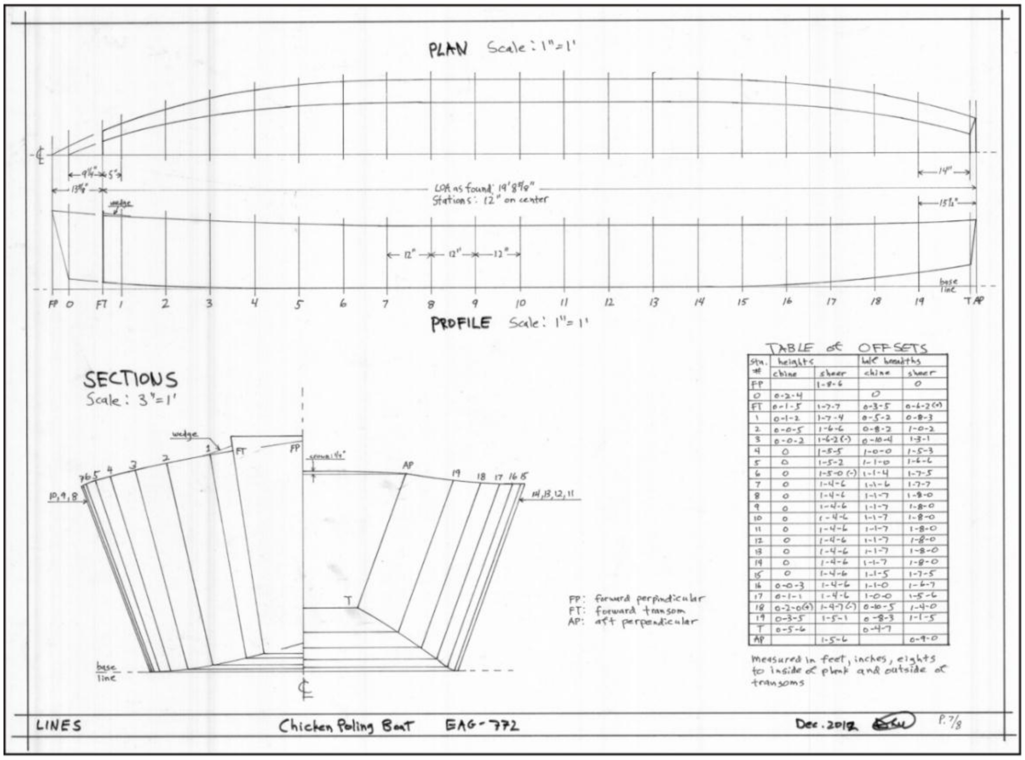


Figure 53. Typical 28-foot river boat (Gray March 2014 Deposition Exhibit 1).



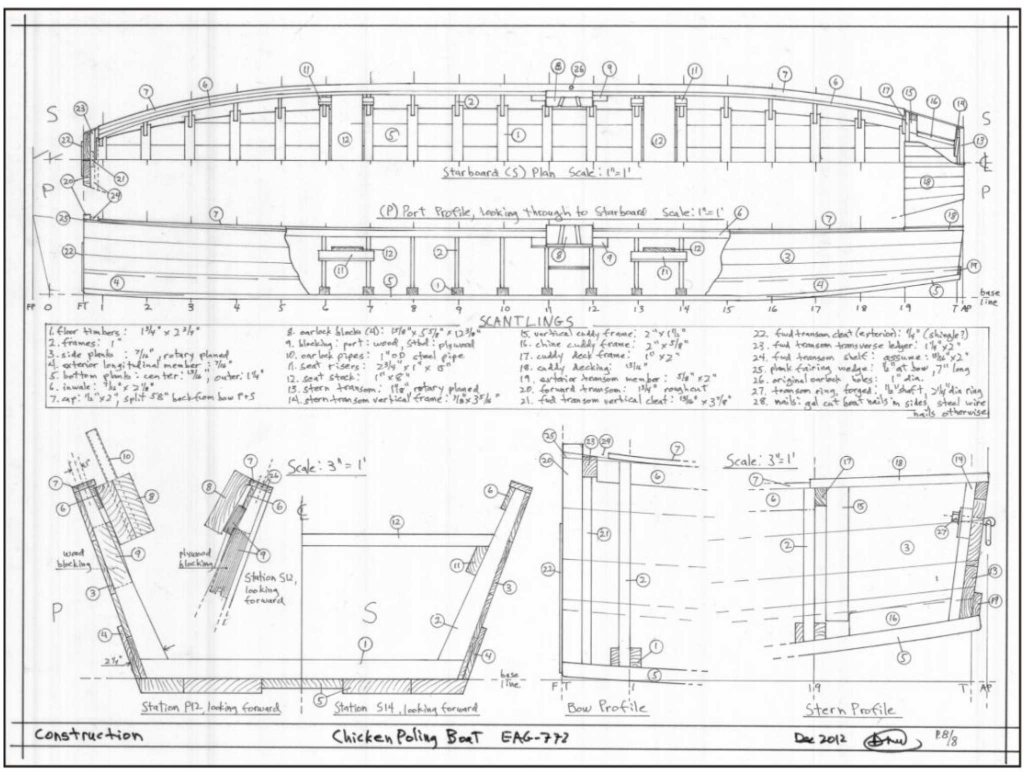


Figure 54. Chicken poling boat (State of Alaska Document Nos. SOA-MF015825 and SOA-MF015826).

The only known extant poling boat in the Fortymile drainage is located on an old river bank on the north side of the Mosquito Fork just upstream from the mouth of Chicken Creek. When examined in 2011, the boat measured 19 feet 6 inches long, 5 feet wide at the top and 2 feet 6 inches wide at the bottom (i.e. tapering sides), and 1 foot 8 inches high on the sides. The stern was 1 foot 6 inches high. The watercraft had characteristics of a small, non-motorized craft used for poling through narrow, shallow streams. The boat had a narrow hull, ideal for negotiating narrow waterways. The fore and aft bottom of the boat was curved. The boat was made of machined lumber. (p90)

To aid in assessing the required draft of these boats, 3-dimensional surface models were created in electronic format using the dimensions shown on the drawings (**Figures 55 and 56**), and the models were used to estimate the boat weights and drafts for a range of loading conditions. The approximate weight of each boat (without crew or cargo) was estimated by determining the volume of wood based on the surface area and approximate thickness of the shell, and adding a factor for fittings and other appurtenances. The unloaded weight of each boat was estimated by adding the boat weight to the estimated weight of the typical boat operator(s). The fully loaded weights were then determined by adding the cargo weight to the unloaded weights.

The river boat in Figures 53 and 55 has a surface area of approximately 167 ft^2. Conservatively assuming an average shell thickness of 1.5 inches, the total volume of wood in the shell is about 21 ft^3. White spruce is one of the more common tree species in this area of Alaska, and would have been a likely source of the wood for locally constructed boats (B. Kennedy, BLM, personal communication, 2014; C. M. Brown, personal communication, 2014). The unit weight of white spruce is about 31 lb/ft^3 (http://commerce.state.ak.us/dnn/ded/DEV/ForestProducts/WhiteSpruce.aspx); thus, the wooden portion of the river boat weighed about 650 lb. Assuming that the fasteners and appurtenances add an additional approximately 10 percent to the weight, the boat would weigh about 720 lbs without the outboard motor. A 1960s vintage 35-hp Mercury outboard motor weighs about 140 lb (http://boatspecs.iboats.com/Mercury 300E 35hp 1960/bpe/57e10864). With a 170-lb operator, motor and 25 gallons of fuel (unit weight of 6.1 lb/gal), the total, unloaded weight of the river boat and operator would be about 1,170 lb.

Similarly, the surface area of the poling boat in Figures 54 and 56 is 101 ft^2. Assuming the boat is constructed from seasoned white spruce with average thickness of 1 inch and adding 10 percent for fasteners and appurtenances, the basic boat would weigh about 290 lb. With the minimum of two crew members required to operate the boat, each weighing 170 lb, the total unloaded weight of the boat and operators would be about 630 lb.

The draft of the boats under various loading conditions was quantified using the basic principle that a solid object floating in a fluid displaces an equivalent weight of the fluid (i.e., Archimedes Principle). The displacement volumes associated with the range of possible boat drafts was computed by determining the volume of the boat below parallel planes set at varying heights above the lowest point on the boat hull, and the equivalent weights were determined by multiplying those volumes by the unit weight of water (62.4 lb/ft^3). The calculations were performed for a range of references planes: (1) assuming the boat is level along the long axis, (2) assuming 1- and 2-

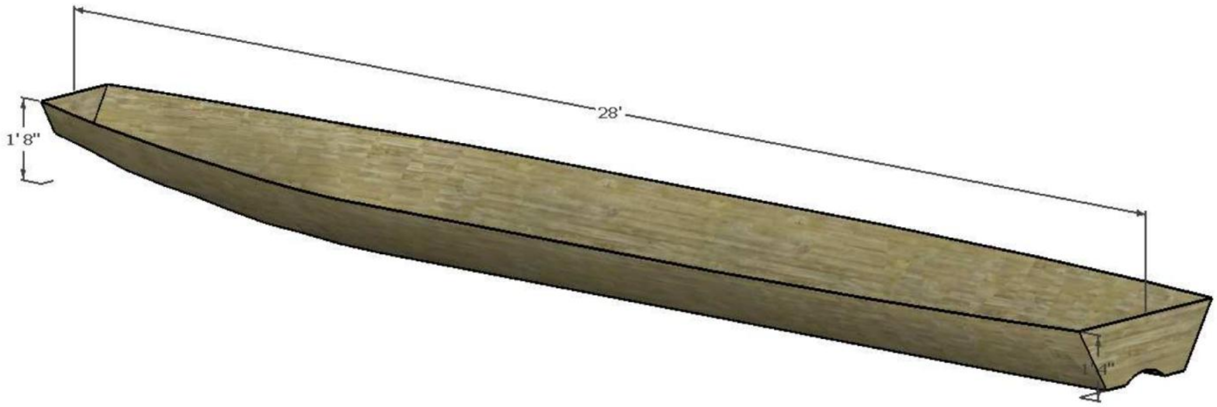


Figure 55. Surface model of typical river boat from drawing in Figure 53.

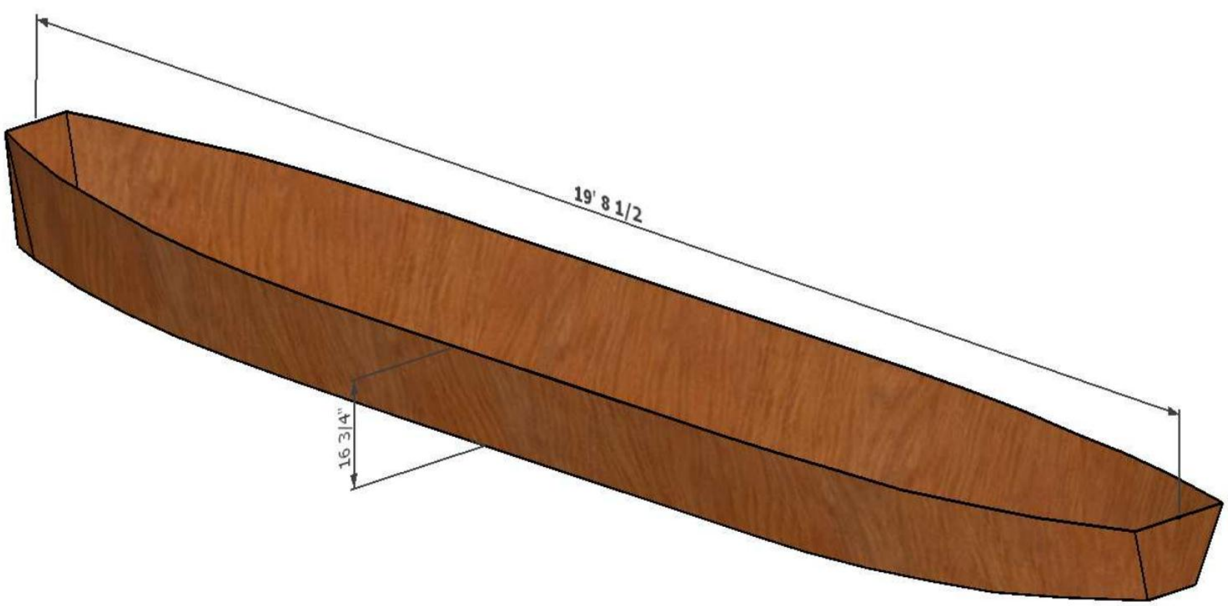


Figure 56. Surface model of Chicken poling boat from drawings in Figure 54.

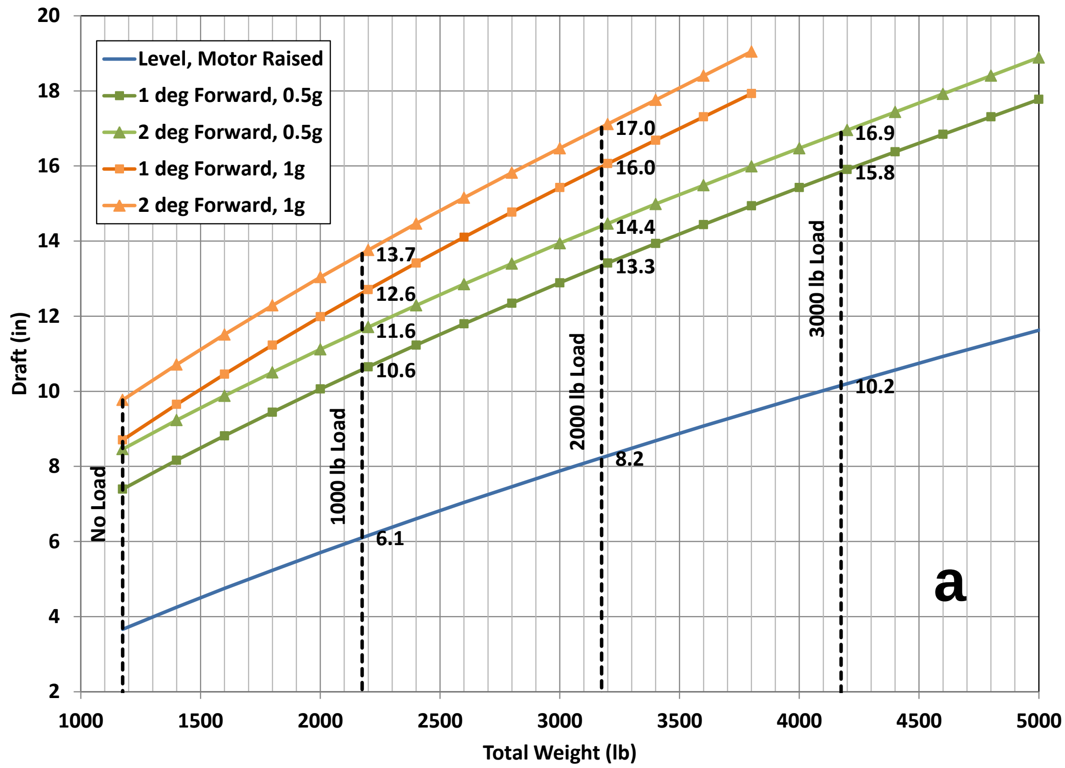
degree forward pitches[5] that would occur when traversing rough water in the downstream direction, and (3) (river boat only) assuming 1 degree and 2 degree reverse pitches (i.e., stern of the boat lower than the bow), that would occur when traversing upstream through a steep riffle or rapid under power.

The results for the river boat indicate that the draft of the hull when sitting level in still water would be 8.2 inches with a 2,000-lb cargo load and 10.2 inches with a 3,000-lb cargo load (**Figure 57a**). Specific information is not available about the depth to which the outboard motor's propeller would project below the boat hull when under power; however, the comment above by Mr. Gray suggests that at least three additional inches of depth are required. Although the tunnel on the bottom side of the boat is effective in limiting the depth to which the propeller must be submerged to provide effective forward thrust without cavitation, the 3-inch submergence below the bottom of the hull is believed to be a minimum for effective operation. As a result, the minimum required depth for 2,000- and 3,000-lb loads when the boat is level from front to back and the motor is lowered for operation would be 11.2 and 13.2 inches, respectively (**Figure 57b**). When under power and traveling in the upstream direction through rough water, the stern of the boat tends to push deeper into the water than the bow, creating a reverse pitch. With a 1-degree reverse pitch when under power and traversing upstream through rough water, the minimum draft, including the propeller, increases to 13.9 inches and 15.8 inches for these two loading conditions, respectively.

Particularly when floating downstream through rough water, the waves create a plunging effect in which the vertical accelerations cause the boat to pitch forward and temporarily increase the effective weight of the boat. Data on the specific magnitude of the vertical accelerations that would occur are not available; however, a reasonable range of accelerations can be estimated from basic physics. The vertical acceleration that would occur if all of the water were to be abruptly removed from beneath the boat would be 1g (i.e., one times the acceleration of gravity). This represents the physical upper limit of the accelerations, and conditions in which this could occur are probably rare, although they could be approached in extremely rough water as the boat passes over large waves. A more reasonable, conservative estimate of the vertical accelerations that could occur in the riffles and rapids along the Mosquito Fork River is in the range of 0.5g. With a 0.5g vertical acceleration, the effective weight of the boat increases by 50 percent, and with a 1g acceleration, the effective weight of the boat doubles. These vertical accelerations and the associated plunging effect cause a commensurate temporary increase in the boat draft (Figure 57a). (Note that the drafts for 1g vertical acceleration are presented here as an upper limit for reference, and are not intended to represent a realistic condition that could be expected under ordinary conditions in a river such as the Mosquito Fork.) For example, with 0.5g acceleration that causes a 1-degree forward pitch of the motorized river boat carrying a 2,000-lb cargo load, the draft would increase to 13.3 inches, and this increases to 15.8 inches when carrying a 3,000-lb cargo load.

Based on a similar analysis, the draft of the small poling boat would be 7.8 inches with a 1,000-lb load and 11.7 inches with a 2,000-lb load when sitting level in still water (**Figure 58**). This increases to 9.8 and 13.7 inches, respectively, with a 1-degree forward pitch with no vertical acceleration. With 0.5g vertical acceleration and 1-degree forward pitch, the draft would

[5] With a 1-degree forward pitch, the reference plane for the 28-foot long river boat would be about 6 inches higher at the stern than at the bow, and this increases to about 12 inches for a 2-degree pitch. The reference plane for the 19' 8-1/2" poling boat would be about 4 inches higher at the stern than at the bow for a 1-degree pitch and about 8 inches higher for a 2- degree pitch. The opposite is, of course, true for reverse pitch. These relatively modest pitches are well within the range of reasonable values.



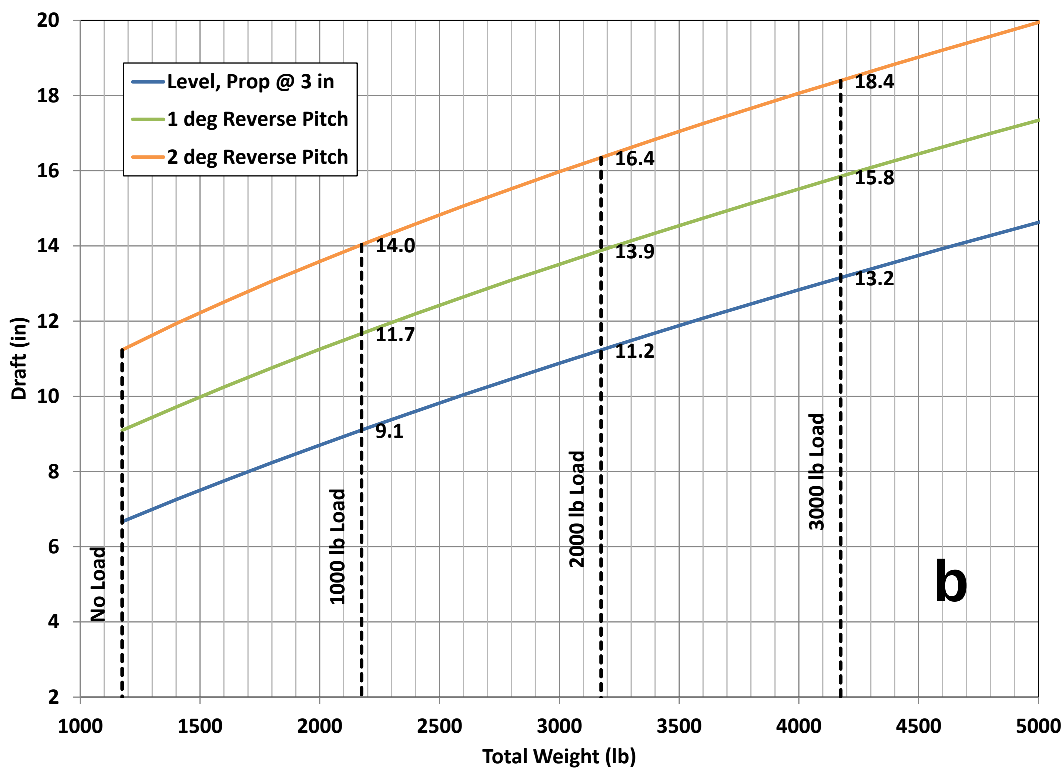


Figure 57. Relationships between draft and total boat weight for the 28-foot motorized boat: (a) sitting level in still water and travelling downstream through rough water with motor raised, and (b) traveling upstream with propeller 3 inches below the boat hull.

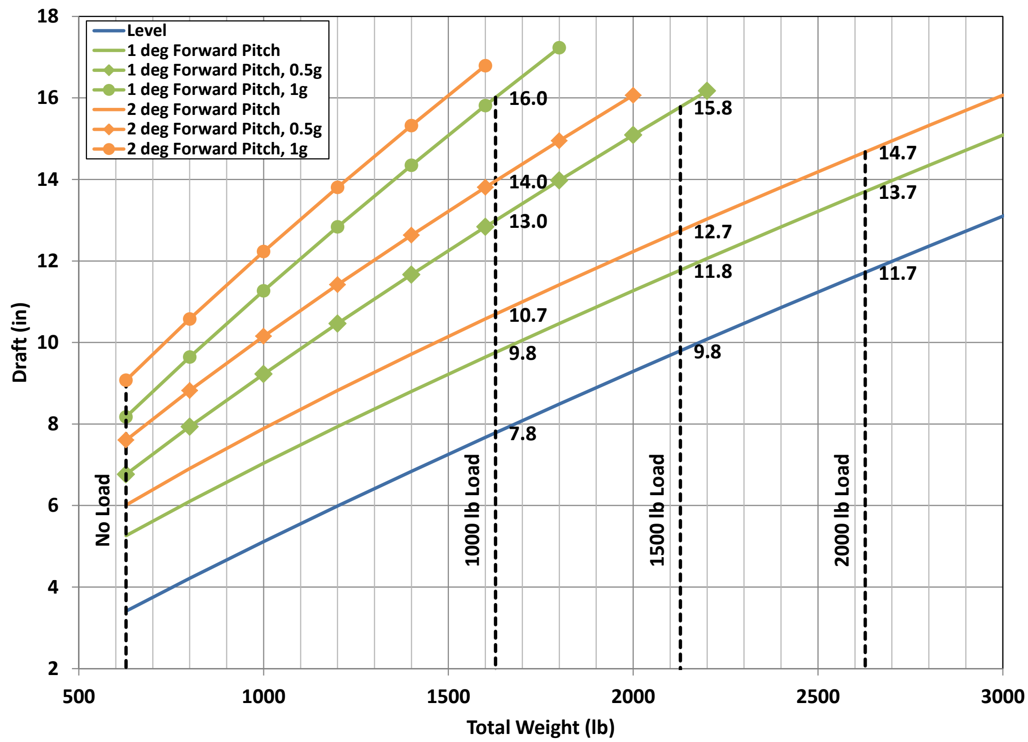


Figure 58. Relationships between required draft and total weight of the small Chicken poling boat.

temporarily and periodically increase to about 13 inches with 1,000-lb cargo load, and to about 15.8 inches with a 1,500-lb load. With a 2,000-lb load under these conditions, draft would exceed the 16-inch height of the boat hull; thus, the boat would take on water.

2.6.2. Flow Depths in the Disputed Reach

The hydraulic models discussed in Section 2.5 provide a means of quantifying the flow depths over the range of flows in the river. The maximum depths at each of the surveyed cross sections can be computed from the difference between the modeled water-surface elevation and the thalweg (i.e., minimum bed) elevation. For example, at Site P9, the thalweg elevation varies from 1612.7 feet at Cross Section (XS)1 to 1620.4 feet at the most upstream XS13, and the water surface at 500 cfs varies from 1614.8 feet to 1623.8 (**Figure 59**). The maximum channel depth from these profiles at 500 cfs varies from about 2.1 feet at XS1 to 4.7 feet at XS12 (upstream side of the Taylor Highway Bridge) (**Figure 60**). Equivalent values of the water-surface elevation and depth for other discharges from 100 cfs to 2,000 cfs are shown in the figures, and similar results for all of the sites were developed and used in the following analysis. Of the cross sections located away from the downstream model boundary (i.e., XS1), XS7 tends to be the shallowest over the range of flows that were analyzed, and thus, would be the limiting location for boating through the site. As can be clearly seen from the cross-section profile, the maximum depth occurs within a relatively narrow zone of the cross section (**Figure 61**). Operation of a boat through a given cross section requires a certain amount of width, particularly where it is necessary to maneuver through areas containing boulders and other obstacles. The typical boats being considered here have hull widths in the range of 2.5 to 5 feet, which would be the absolute minimum width for passage if the boat was perfectly aligned between the obstacles. Because the boulders and other obstacles are not perfectly aligned, it was assumed that a minimum width of 8 feet would be required to

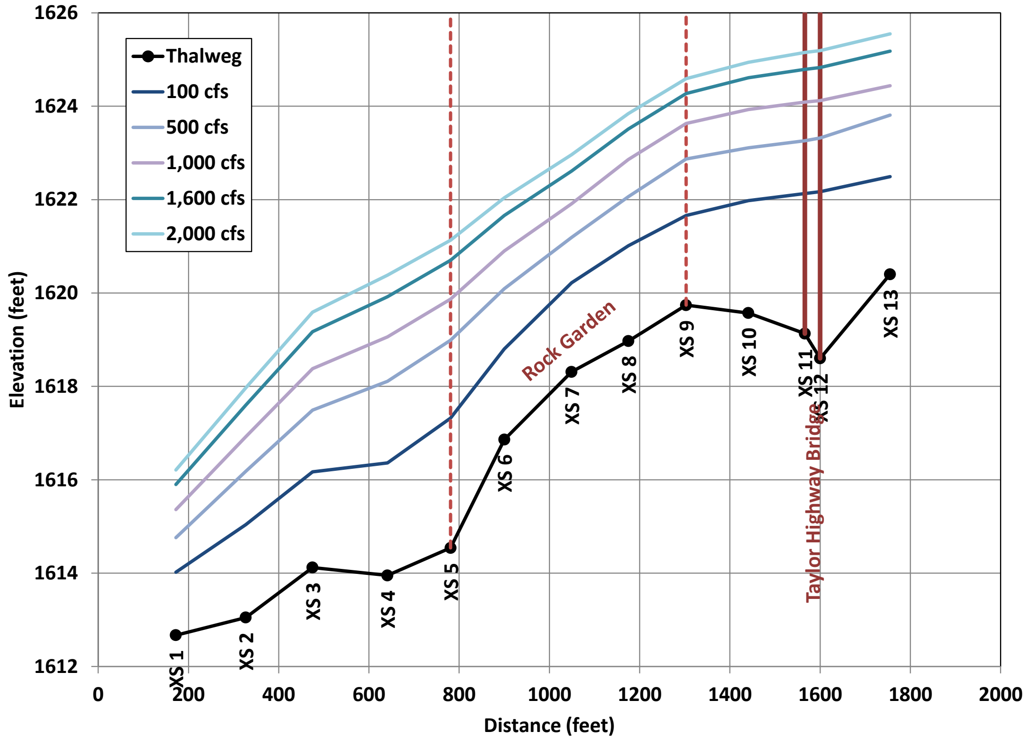


Figure 59. Bed and water-surface profiles for discharges from 100 to 2,000 cfs at Site P9, located near the Taylor Highway Bridge.

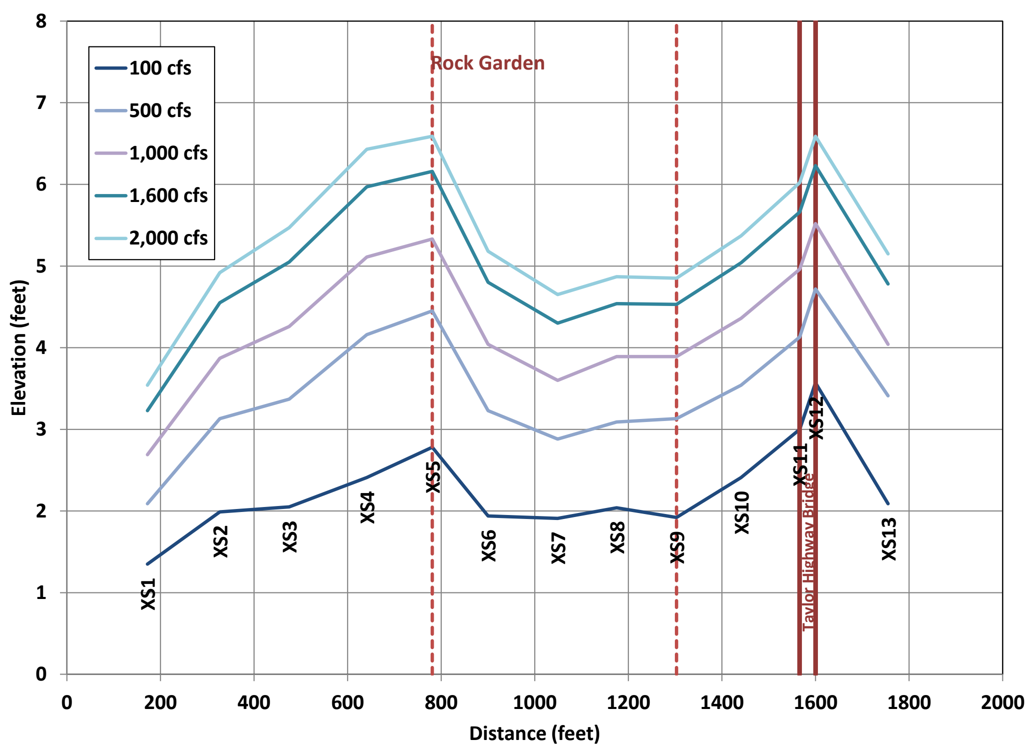


Figure 60. Maximum depth profiles for discharges from 100 to 2,000 cfs at Site P9.

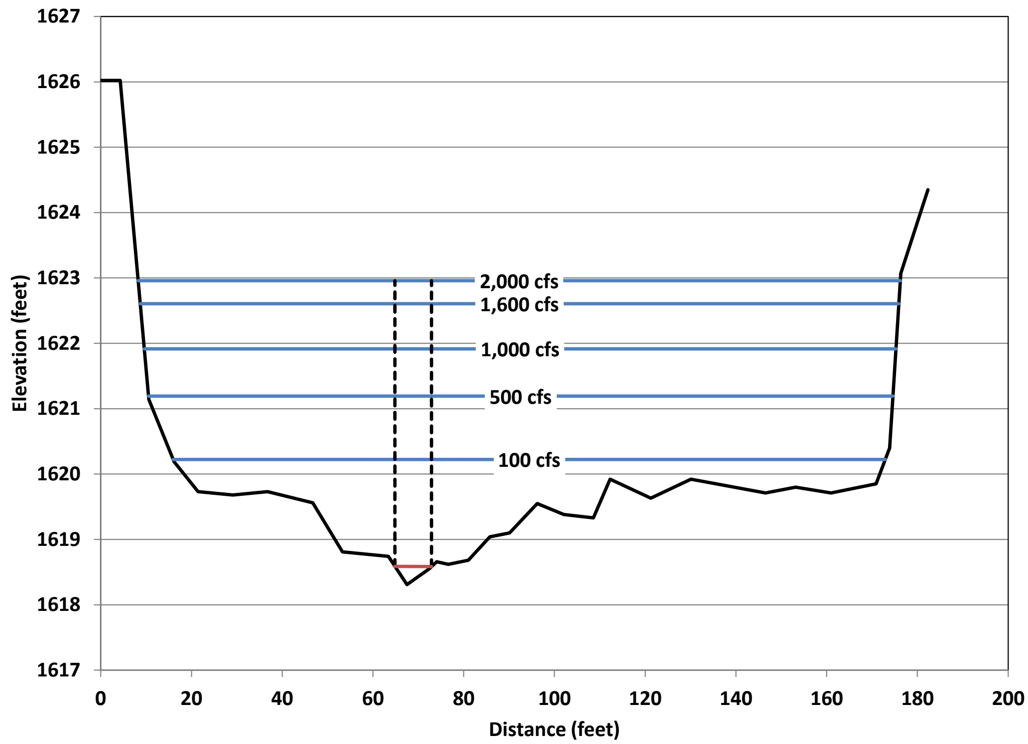


Figure 61. Cross Section XS7 (downstream view) at Site P9, showing water-surface elevations for discharges of 100 cfs through 2,000 cfs and the 8-foot wide zone in the deepest part of the channel.

accommodate the boat and a limited ability to maneuver. At this particular cross section, the effective depth within the deepest 8-foot-wide zone is about 3 inches less than the thalweg depth (i.e., depth below the red line in Figure 61), assuming a smooth bed. A similar evaluation was performed for each of the surveyed cross sections.

The effects of the cobbles and boulders that protrude above the typical bed elevation within the deeper portions of the cross section defined by the survey points must also be considered in evaluating the boatability of the cross sections. The cross-section profiles at the study sites were surveyed to capture the overall bed topography, with surveyed points at an average spacing of 5 to 6 feet. As a result, they do not capture the detailed variability associated with the cobbles and boulders that make up the bed between the points. In addition, the specific topography associated with these materials varies over time due to bed mobilization during high flows. For this reason, even though the average conditions through the site typically remain about the same, the details of the topography at one point in time are not necessarily the same at other times, particularly after high, bed-mobilizing flow events. This variability and the resulting effects on the flow depth available for boating were assessed using the topographic data and the riffle particle-size gradations. A statistical, Monte Carlo simulation was used to develop a range of possible lateral distributions of rock sizes and protrusion heights within the deepest 8 feet of the critical cross sections (i.e., the cross sections that have the shallowest, and thus, limiting depths for boatability) at each study site. For the simulation, the distribution of particles across the target zone of the cross section was determined by randomly sampling the applicable particle-size gradation from the field data and placing the individual particles on the bed starting at the left side of the zone so that each successive particle touches the adjacent particle. A reference plane between successive survey points was established from the data by observing that the survey rod would have been placed on top of the particle that is located at the horizontal location of the survey point. The elevation of the top of each successive particle (and, thus, the height at which it protrudes into the flow) was determined by assuming that the top of the particle is equally likely to be located at any elevation between the reference plane between two survey points (i.e., entirely below the reference plane) and one particle diameter above the reference plane (i.e., the bottom touching, but the entire particle above, the reference plane). This process was repeated 1,000 times for each of the critical cross sections, and the top elevation of the highest rocks within the 8-foot-wide target zone were tabulated for each repetition. Plots of four typical profiles from among the 1,000 trials in the Monte Carlo simulation for XS7 at Site P9 are shown in **Figure 62** to illustrate the process.

The above evaluation applies to only a single cross section at the site. To account for the longitudinal distribution of the particles, the procedure was repeated 20 times, representing 20 cross sections that would span approximately one length of the boat. Based on this assessment, the median height of the highest-projecting rock within the deepest 8-feet of the channel over the approximate length of the boat ranged from 1619.0 to 1621.6 feet, with median and mean values of about 1620.2 (**Figure 63a**). Since it may be possible to avoid one large rock in the zone represented by the 8-foot-wide deep zone at the 20 cross sections, the final analysis was based on the second highest-projecting rock in the zone. This reduced the range of the heights that would restrict navigation from 1618.4 feet to 1621.4 feet, with mean and median values of 1619.7 and 1619.6 feet, respectively (**Figure 63b**). Equivalent results were obtained for the cross section at each study site that would be most limiting to boatability, and these values were used to estimate the discharges required to provide the minimum depth necessary to accommodate the boat drafts presented in the previous section.

2.6.3. Discharges Required for Boatability

The minimum discharge at which it would be possible to traverse through each of each study sites with the above-described boats was determined by identifying the discharge at which the water-surface elevation would be at least the height of the boat draft above the second largest rock in the deepest 8-foot width determined by the procedure described in the previous section. The median height of the second-highest rock above the thalweg elevation from the 1,000 trials at each site ranged from 0.4 feet at Site P6 (XS5) to 1.3 feet at Site P9 (XS7), and the tops of these particles would be submerged at discharges ranging from 3 cfs (Site P6) to 23 cfs (Site P8) (**Table 6**).

Based on the analysis presented in Section 2.6.2, the minimum depths required for the poling boat carrying 1,000- and 2,000-lb loads are about 8 and 12 inches, respectively, when travelling in the upstream direction (i.e., boat pitch would probably not cause an increase in the draft) to 13 inches and about 16 inches when travelling in the downstream direction through the rough water that would occur in the riffles and rapids (i.e., accounting for the likelihood of at least 1-degree forward pitch and 0.5g vertical accelerations) (Figure 58). The minimum depths for the motorized river boat carrying 2,000- and 3,000-lb loads in the upstream direction with the propeller 3 inches below the bottom of the hull and a 1-degree reverse pitch would be about 14 inches and 16 inches, respectively (Figure 57b). The required draft when floating downstream through rough water with the motor raised, but with 1-degree forward pitch and 0.5g vertical accelerations would be about 13 inches and 16 inches with 2,000- and 3,000-lb loads, respectively (Figure 57a). Based on this range of required drafts, the discharge associated with water-surface elevations 8 inches, 12, 15 and 18 inches above the limiting rock heights were estimated using the hydraulic model results. The lowest discharge meeting the depth requirement typically occurred at Site P6 and the highest discharge occurred at Site P8. The resulting discharges ranged from about 180 cfs (Site P6) to about 950 cfs (Site P8) for a draft of 8 inches, from 360 to 1,740 cfs for a draft of 12 inches, and from 590 to 2,230 cfs for a draft of 15 inches (Table 6). The corresponding values for an 18-inch

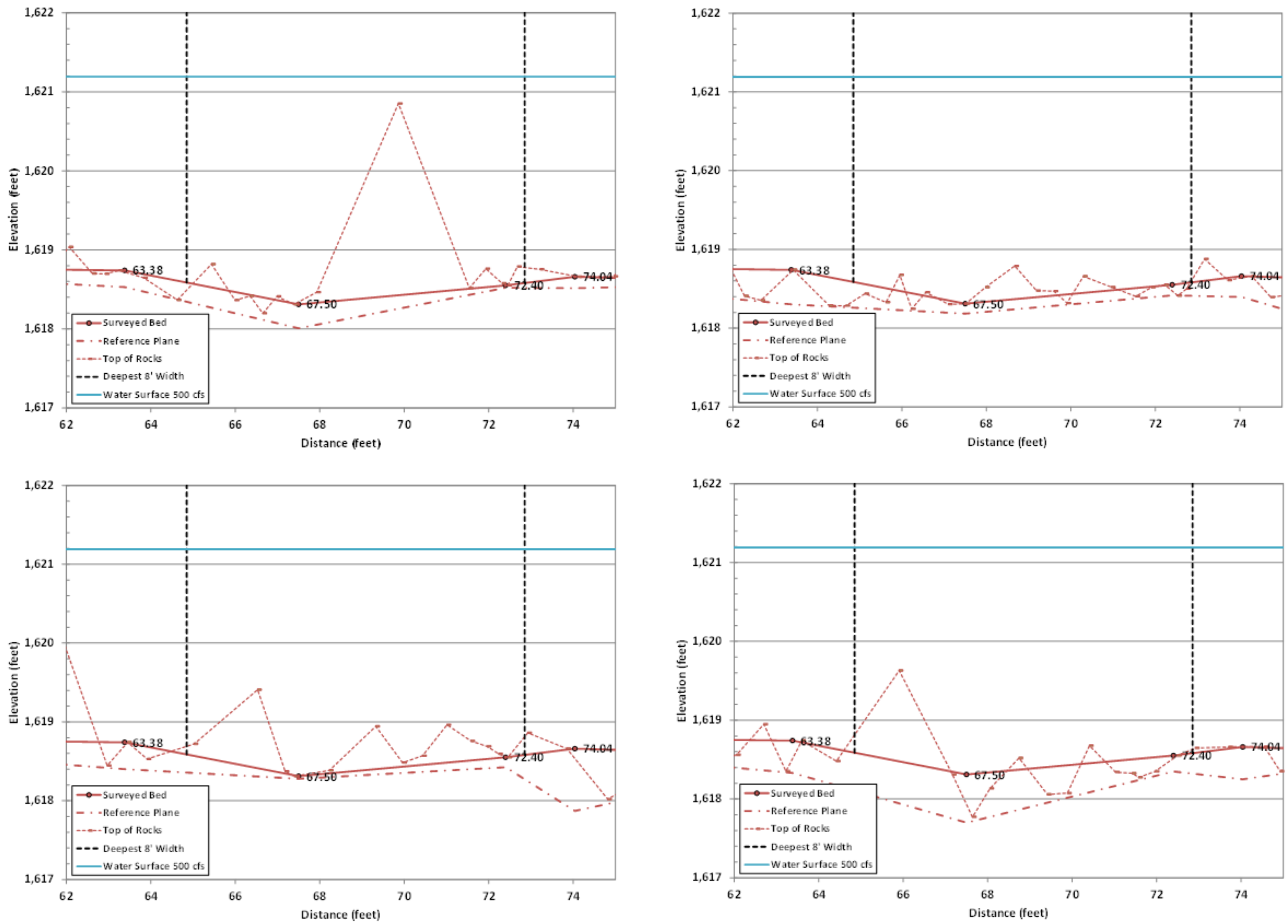
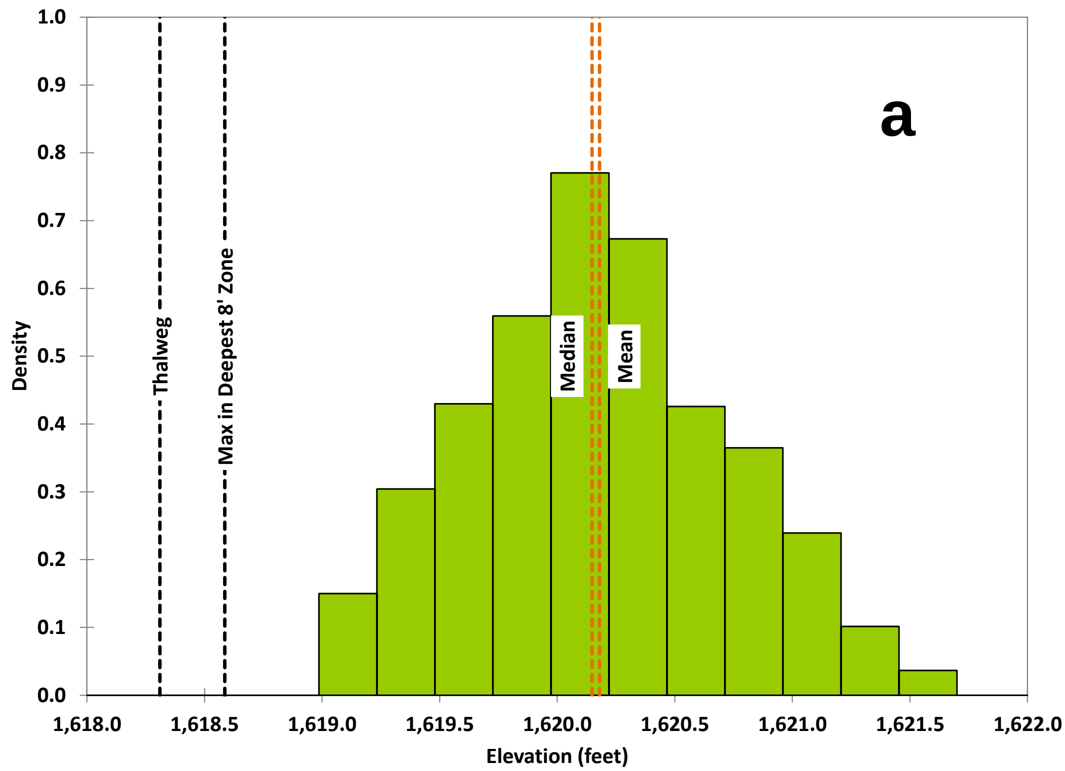


Figure 62. Surveyed bed profile at Site 9, XS7 showing the reference plane and randomly selected top-of-particle elevations for four of the 1,000 samples from the Monte Carlo simulation. The limits of the deepest 8-foot portion of the cross sections and the water-surface elevation at 500 cfs are also shown for reference.



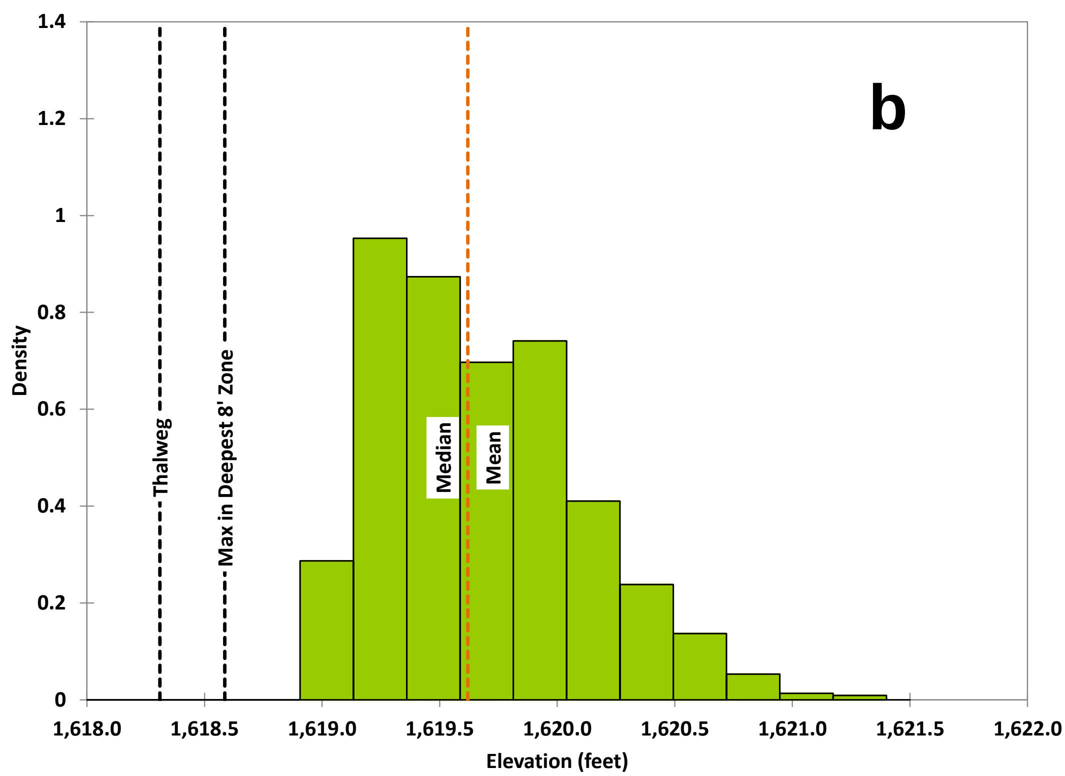


Figure 63. Distribution of (a) highest and (b) second highest top-of-rock elevations within the deepest 8-foot width within approximately one boat length of XS7 at Site P9.

Table 6. Median height of second highest particle within the deepest 8-foot width above thalweg, and discharge required to inundate the particle to the indicated depth.

Site	Limiting Cross Section	Height above Thalweg (feet)	Discharge for Indicated Minimum Draft (cfs)				
			0"	8"	12"	15"	18"
P1	XS5	0.9	17	240	510	1,160	1,550
P2	XS8	0.6	14	660	1,140	1,520	1,920
P3	XS11	0.8	20	450	1,220	1,930	1,520
P4	XS4	0.8	13	210	510	1,020	1,650
P6	XS5	0.4	3	180	360	590	1,300
P7	XS12	1.0	11	280	790	1,300	1,730
P8	XS8	0.6	23	950	1,740	2,230	2,790
P9	XS7	1.3	16	370	770	1,160	1,610

required draft are also shown (1,300 to 2,790 cfs), although this draft approaches the maximum that could be accommodated by the river boat without taking on water, even in placid water.

2.6.4. Time Periods When Navigation Would Have Been Possible

To evaluate the frequency and duration of discharges sufficient to traverse through the study sites based on the required draft, long-term flow records (1998-2004, 2006-2014) were estimated for each site by scaling the completed record for the Taylor Highway gage using the factors shown in Figure 48. The periods when it would have been possible to boat through the site were then determined by comparing the required discharges from the analysis in the previous section with the applicable flow records. For example, these results indicate that it would have been possible to pass through Site P9 with a boat requiring 15 inches of draft during five separate periods in 2007 (May 1 through May 20, May 24 through June 1, June 8 through June 16, July 8 through July 16, and on September 19 and September 23). This represents only 50 of the 153 days (or about 33 percent of the time) during the open-water period from May 1 through September 30 (**Figure 64**). For the full period of record that includes 18 years of data, Site P9 would **not** have been boatable with a boat requiring 12 inches of draft for an average of 93 days per year (~61 percent of the days), and these conditions would have occurred during an average of about 5 discrete periods with durations averaging 4 days per year (**Figure 65**).

An analysis similar to that described above was performed for each of the study sites and each of the three required drafts (**Table 7, Appendix D**). Sites P2 through P9 are representative of conditions at the numerous riffles within the 36-mile segment of the disputed reach from about 0.25 miles downstream from the Taylor Highway Bridge upstream to the mouth of Ketchumstuck Creek, and Site P1 is representative of conditions in the relatively steep, approximately 5-mile reach just downstream from Mosquito Flats. The results indicate that two of the sites (P2 and P8) would have been boatable less than half the time during the open-water season during an average year, even for the small poling boat carrying a 1,000-lb load, and these conditions would have been broken up into about five different periods per year with average lengths ranging from 10 days (Site P8) to 12 days (Site P2). As noted above, the 1,000-lb load would probably have been too small to meet the Federal test for commercial navigation, particularly when considered in conjunction with the episodic and unpredictable nature of the flows at which it would have been possible to traverse the reach with the above described boats. For the 12-inch minimum draft that would be required for the poling boat carrying a 2,000-lb load, the amount of time during the open-water season when Sites P1, P2, and P8 would be boatable decreases relative to the 8-inch draft to 15 percent (Site P8) and 27 percent (Site P1), and these would also occur during an average of five different periods per year for durations of 6 to 10 days each. For the 15-inch minimum draft required for the river boat with a 2,500- to 2,600-lb load, the amount of time Sites P1, P3 and P8 would be boatable decreases even further to 8 percent (Site P1), 10 percent (Site P8) and 10 percent (Site P8). At Site 1, boatable conditions would have occurred during an average of about three different periods per year for durations averaging about 5 days. Similarly, Site P8 would have been boatable during only about three periods per year for average durations of about 5 days.

From the above analysis, it is clear that boatability of the two segments of the disputed reach containing the study sites would have been extremely limited and unpredictable for the wooden boats that were in customary use for commerce in the smaller rivers in the Yukon and Tanana River drainages at the time of Alaska's statehood.

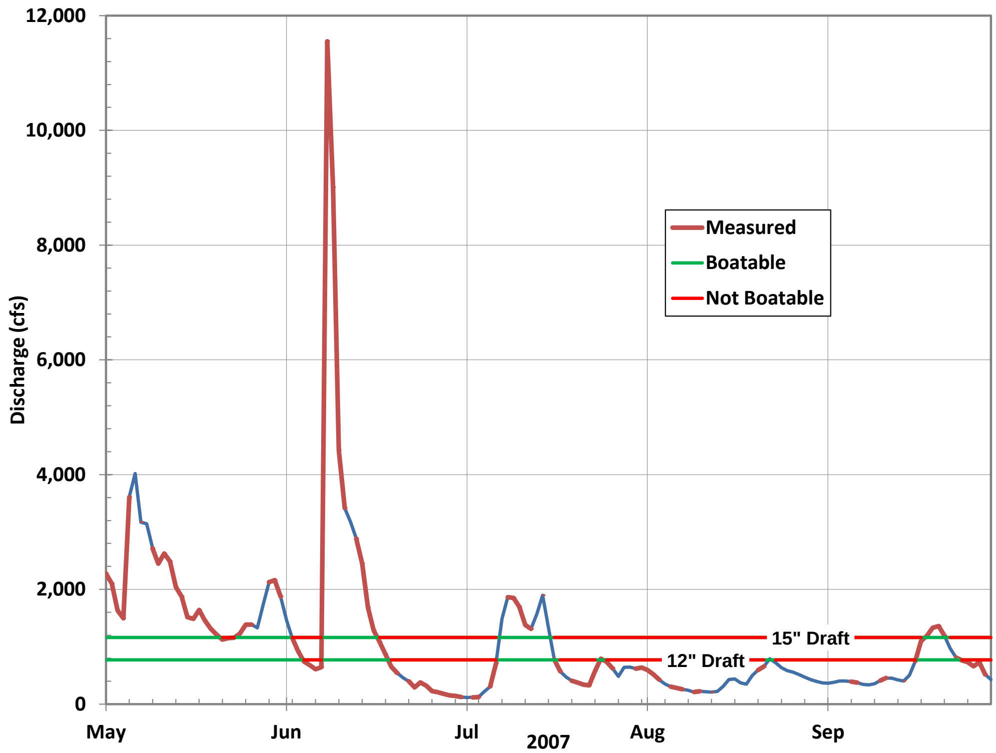


Figure 64. Recorded (and estimated) flows at Site P9 (Taylor Highway Bridge) during the 2007 open-water period, and the period when the site would have been boatable using boats with drafts of 12 and 15 inches.

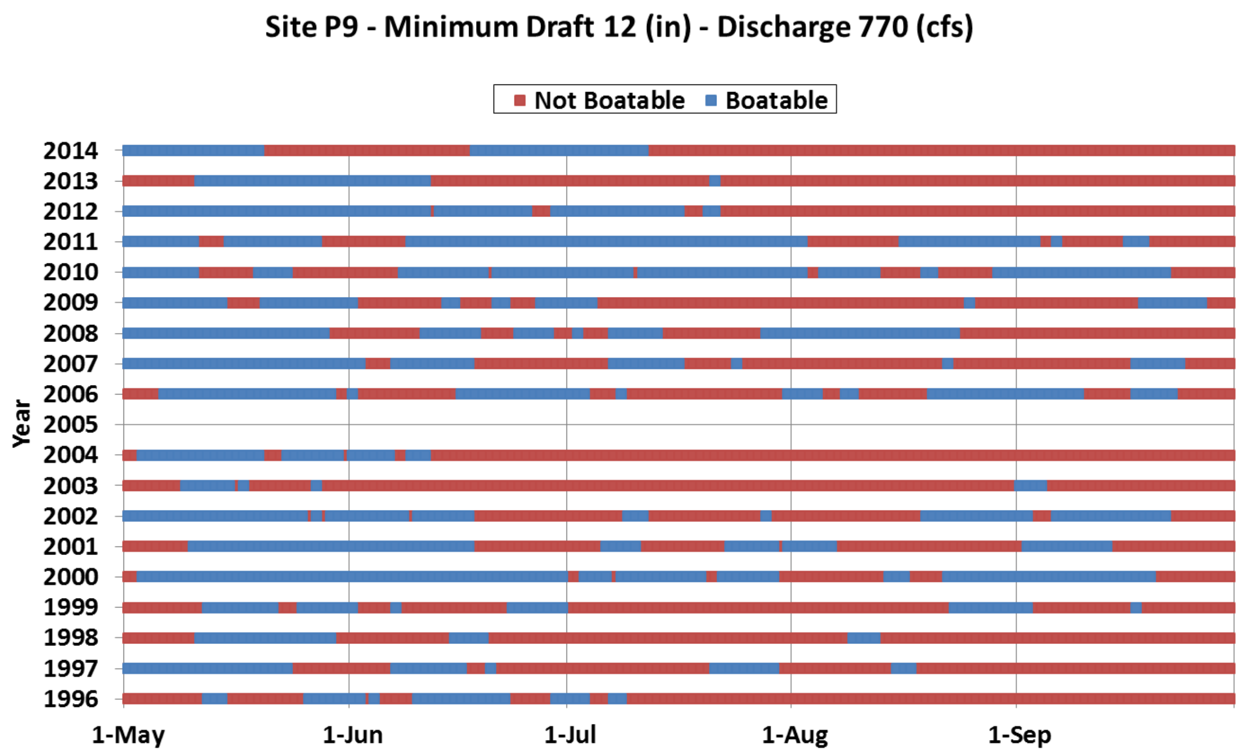


Figure 65. Calculated periods of boatability and non-boatability at Site P9 for a boat requiring 12 inches of draft. Similar plots for each of the study sites for required drafts of 8, 12 and 15 inches are provided in Appendix D.

Table 7. Summary of average annual periods when depths meeting or exceeding the indicated minimum draft would have occurred at each of the eight detailed study sites.

Site	Minimum Draft											
	8 inch				12 inch				15 inch			
	Number of Days	Percent of Days	Number of Discrete Periods	Average Duration	Number of Days	Percent of Days	Number of Discrete Periods	Average Duration	Number of Days	Percent of Days	Number of Discrete Periods	Average Duration
P1	79.2	52%	5.2	21.8	41.7	27%	5.1	8.1	11.9	8%	2.5	5.5
P2	56.6	37%	4.9	12.0	32.1	21%	4.8	6.4	20.9	14%	3.8	5.8
P3	79.2	52%	5.2	21.8	30.7	20%	4.7	6.1	15.3	10%	3.1	5.0
P4	118.5	77%	3.4	61.2	73.2	48%	5.1	16.4	39.3	26%	4.9	7.7
P6	126.7	83%	2.9	73.6	96.7	63%	5.7	26.0	69.4	45%	5.4	14.1
P7	112.4	73%	3.9	43.9	55.2	36%	5.1	11.0	33.1	22%	4.8	6.8
P8	49.0	32%	4.9	9.7	23.4	15%	4.4	5.3	15.8	10%	3.1	5.1
P9	101.0	66%	5.4	30.3	60.4	39%	5.3	11.9	41.2	27%	5	8.0
min	49.0	32%	2.9	9.7	23.4	15%	4.4	5.3	11.9	8%	2.5	5.0
mean	90.3	59%	4.5	34.3	51.7	34%	5.0	11.4	30.8625	20%	4.075	7.3

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Wiley, J.B. and Curran, J.H., 2003. Estimating Annual High-Flow Statistics and Monthly and Seasonal Low-Flow Statistics for Ungaged Sites on Streams in Alaska and Conterminous Basins in Canada, U.S. Geological Survey Water-Resources Investigations Report 03-4114, 69 p.

Wolman, M.G., 1954. A method for sampling coarse river bed material, Transactions of American Geophysical Union, v.35 (6), pp. 951-956.

Yeend, W.E., 1995. Gold Placers of the Historical Fortymile River Region, Alaska. U.S. Geological Survey Bulletin 2125, 84 p.

4. Exhibits which Summarize or Provide Support for Opinions

The listings, tables, and figures set forth above, the documents and publications referenced above summarize or provide support for the opinions expressed.

5. Compensation

Dr. Robert Mussetter's hourly billing rate for his work is $250 per hour. This rate increases to $375 per hour for deposition and trial testimony. Total billings for work on this project through October 2014 have totaled about $223,000.

6. Expert Testimony within Preceding Four Years

Dr. Robert Mussetter has testified as an expert at trial or by deposition in four judicial proceedings within the preceding four years as follows:

Adjudication of All Rights to Use Water in the Gila River System and Source, Superior Court of Arizona and County of Maricopa, Contested Case No. W1-11-3342, Aravaipa Canyon Wilderness Area (Expert report 2013 and deposition 2014).

Navigability of the Gila River Between the Arizona- New Mexico Stateline and the Confluence with the Colorado River, Expert Report and Testimony before Arizona Navigable Streams Adjudication Commission (2014).

7. Qualifications

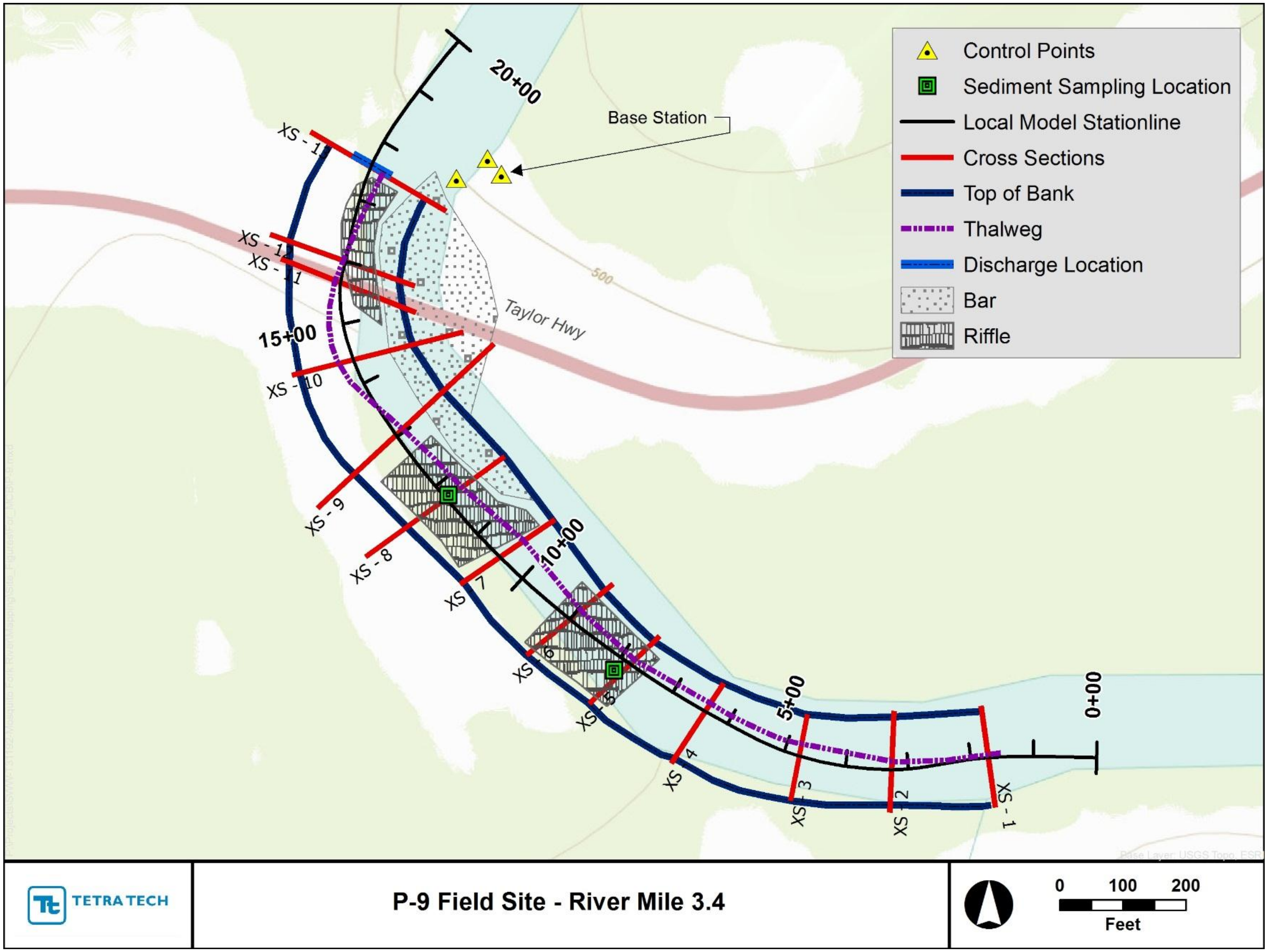
A copy of my curriculum vitae that identifies my expert qualifications, including all publications authored by me within the last 10 years, is attached hereto (Appendix E).

Date December 18, 2014

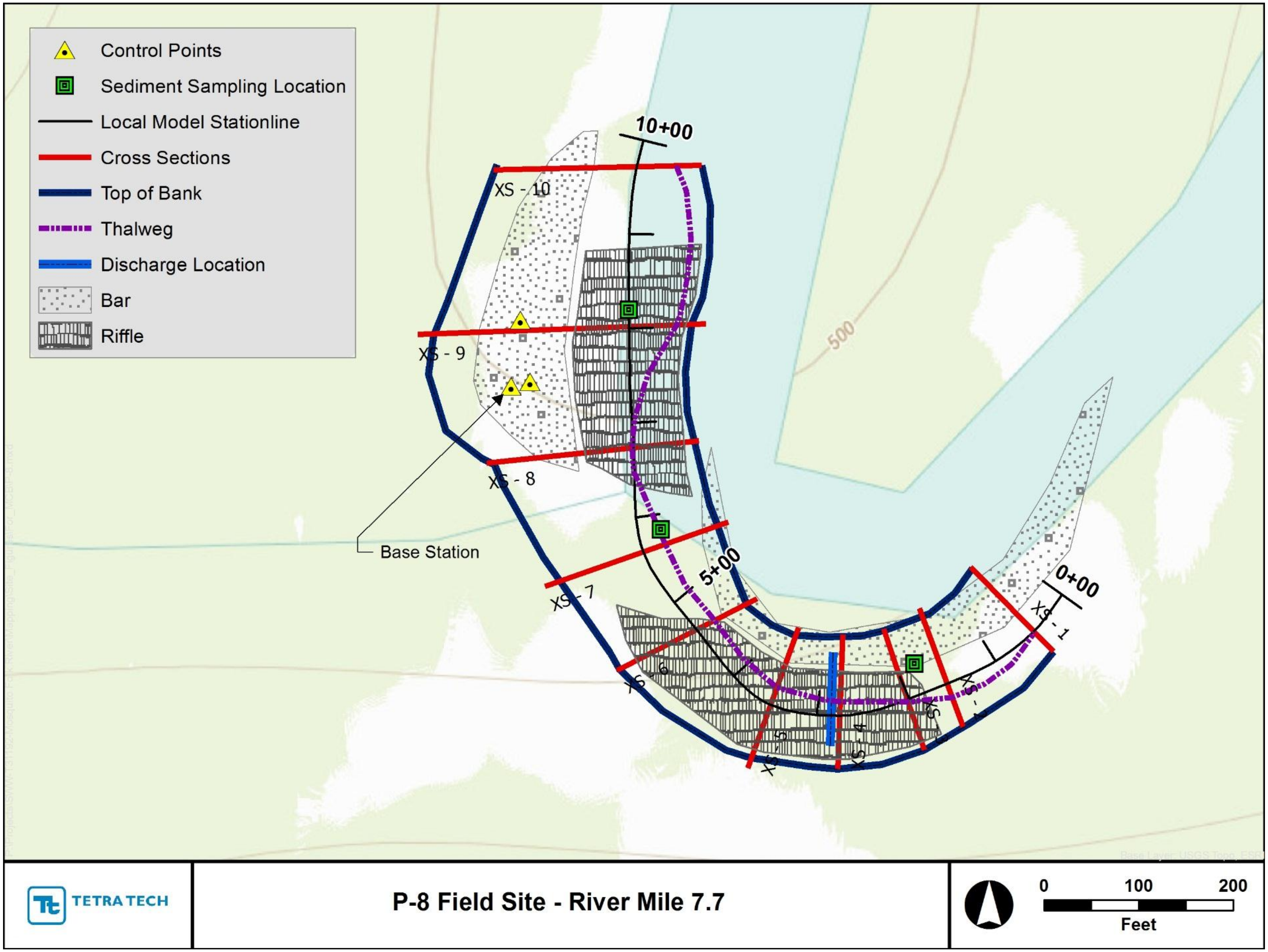
Dr. Robert A. Mussetter ____________________

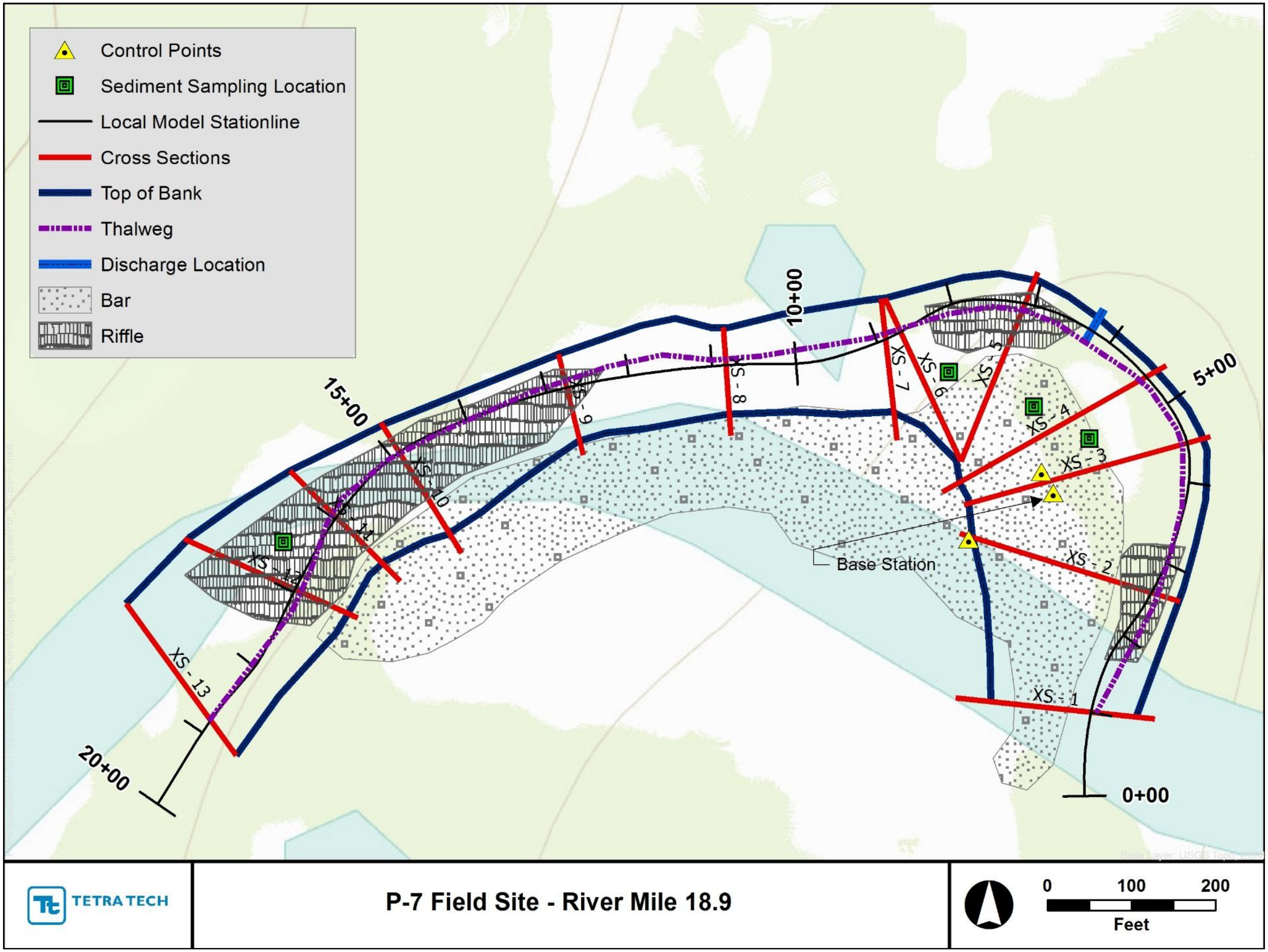
APPENDIX A

Site Figures showing the Locations of the Sediment Samples, Discharge Measurements and the GPS Base Station

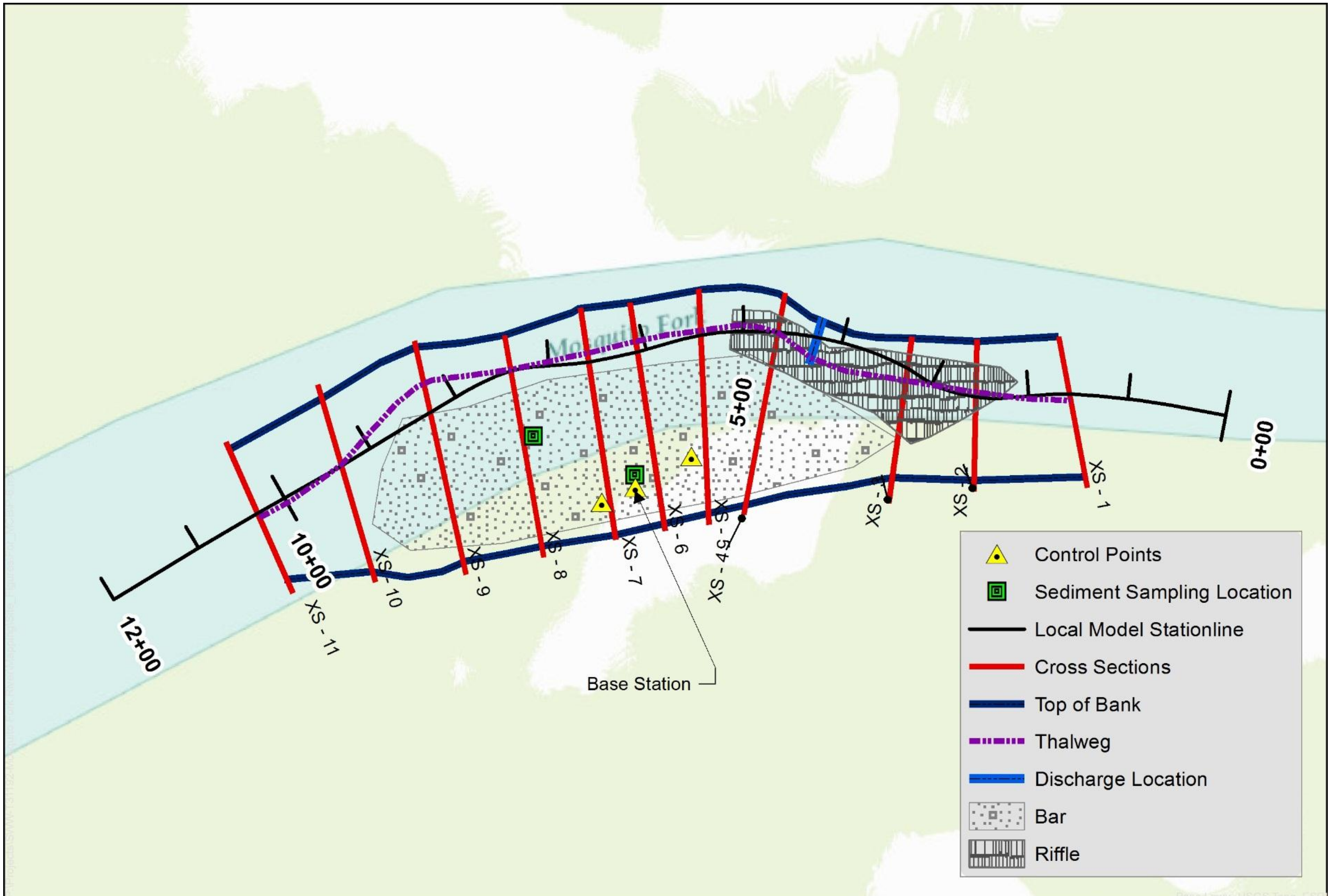


Control Points
Sediment Sampling Location
Local Model Stationline
Cross Sections
Top of Bank
Thalweg
Discharge Location
Bar
Riffle
Base Station
20+00
15+00
10+00
5+00
0+00
Taylor Hwy
500
XS - 13
XS - 12
XS - 11
XS - 10
XS - 9
XS - 8
XS - 7
XS - 6
XS - 5
XS - 4
XS - 3
XS - 2
XS - 1
TETRA TECH
P-9 Field Site - River Mile 3.4
0
100
200
Feet

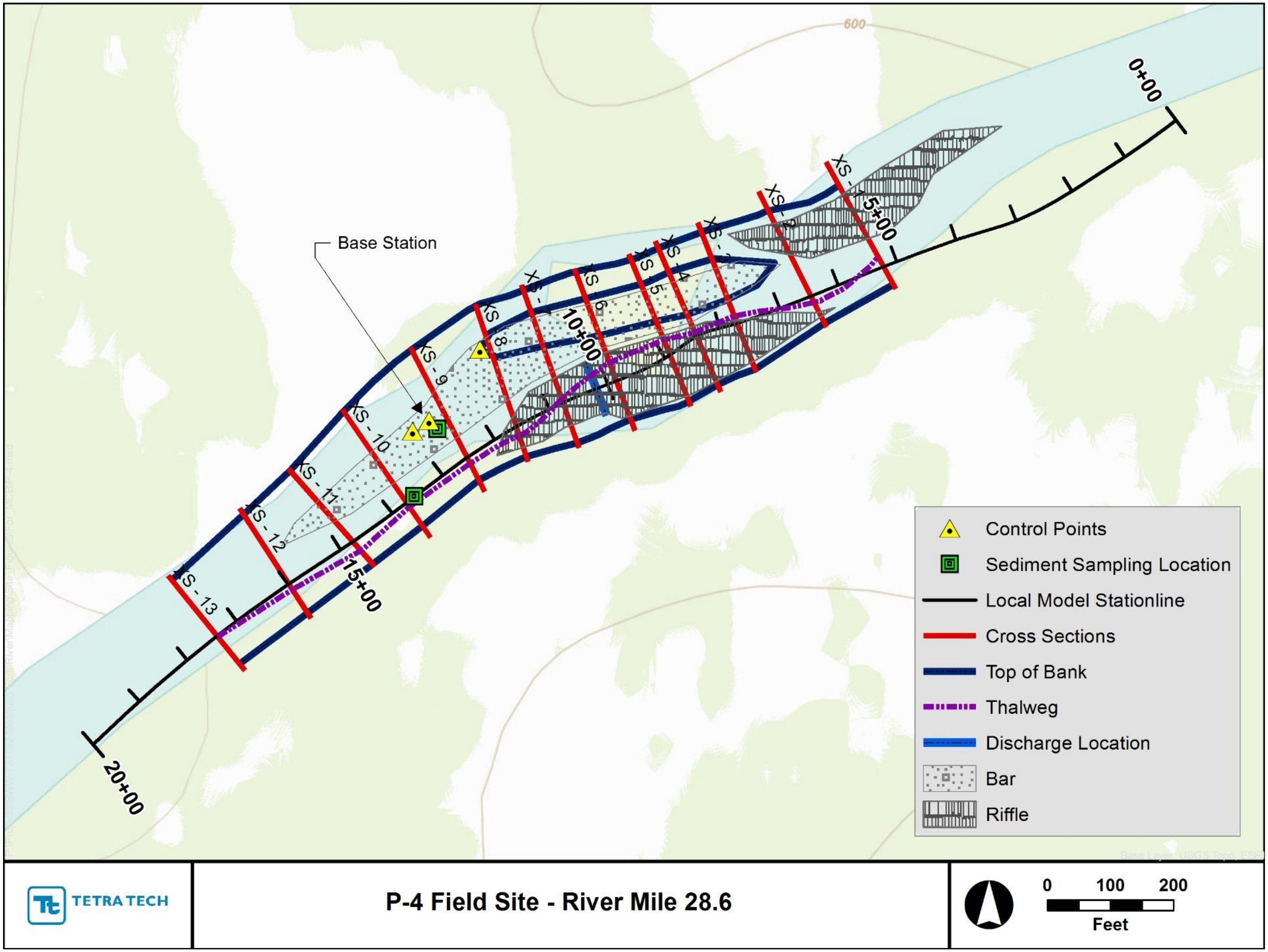

Control Points
Sediment Sampling Location
Local Model Stationline
Cross Sections
Top of Bank
Thalweg
Discharge Location
Bar
Riffle
10+00
5+00
0+00
XS - 10
XS - 9
XS - 8
XS - 7
XS - 6
XS - 5
XS - 4
XS - 3
XS - 2
XS - 1
500
Base Station
TETRA TECH
P-8 Field Site - River Mile 7.7
0
100
200
Feet



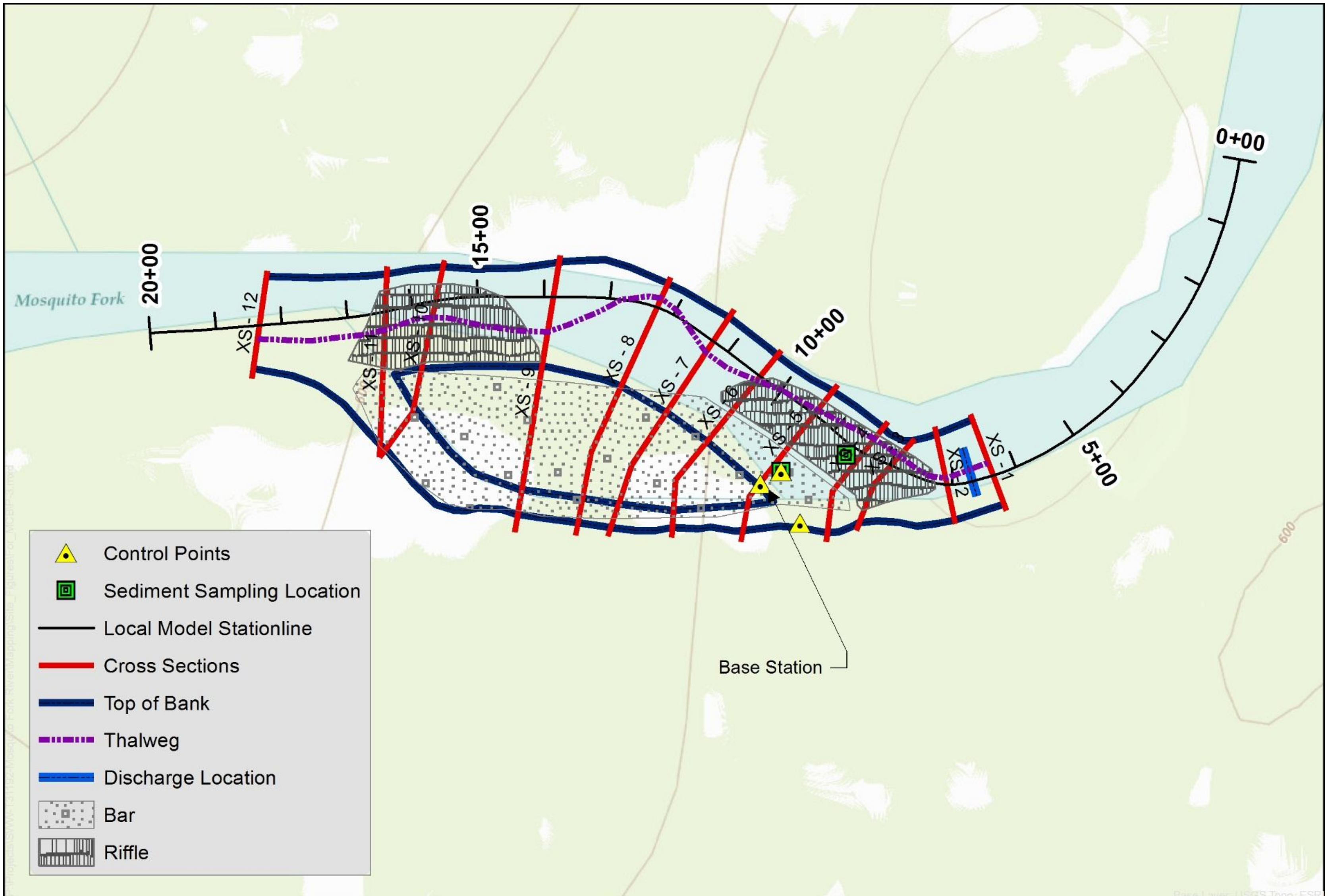
P-7 Field Site - River Mile 18.9



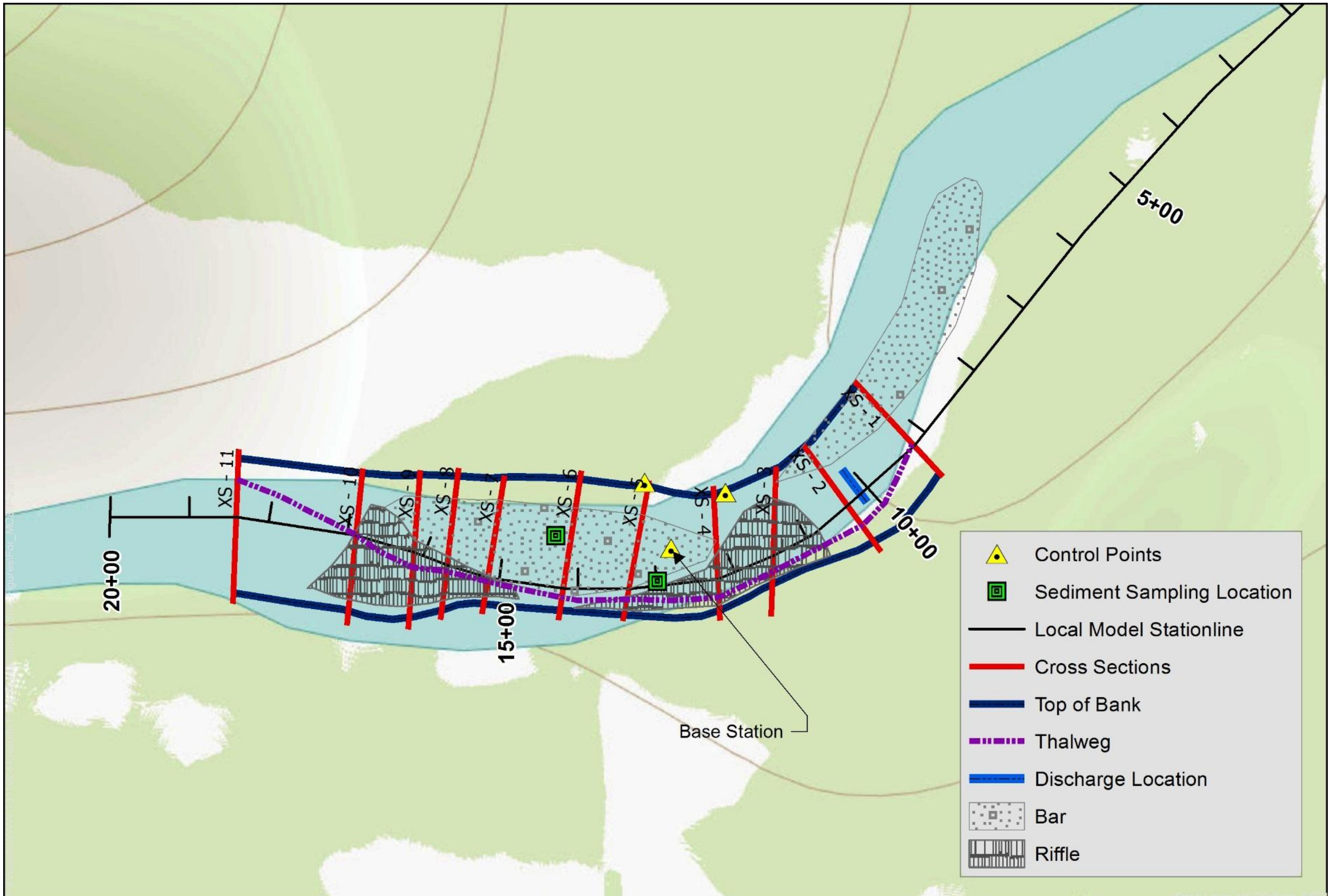
P-6 Field Site - River Mile 22.5



P-4 Field Site - River Mile 28.6

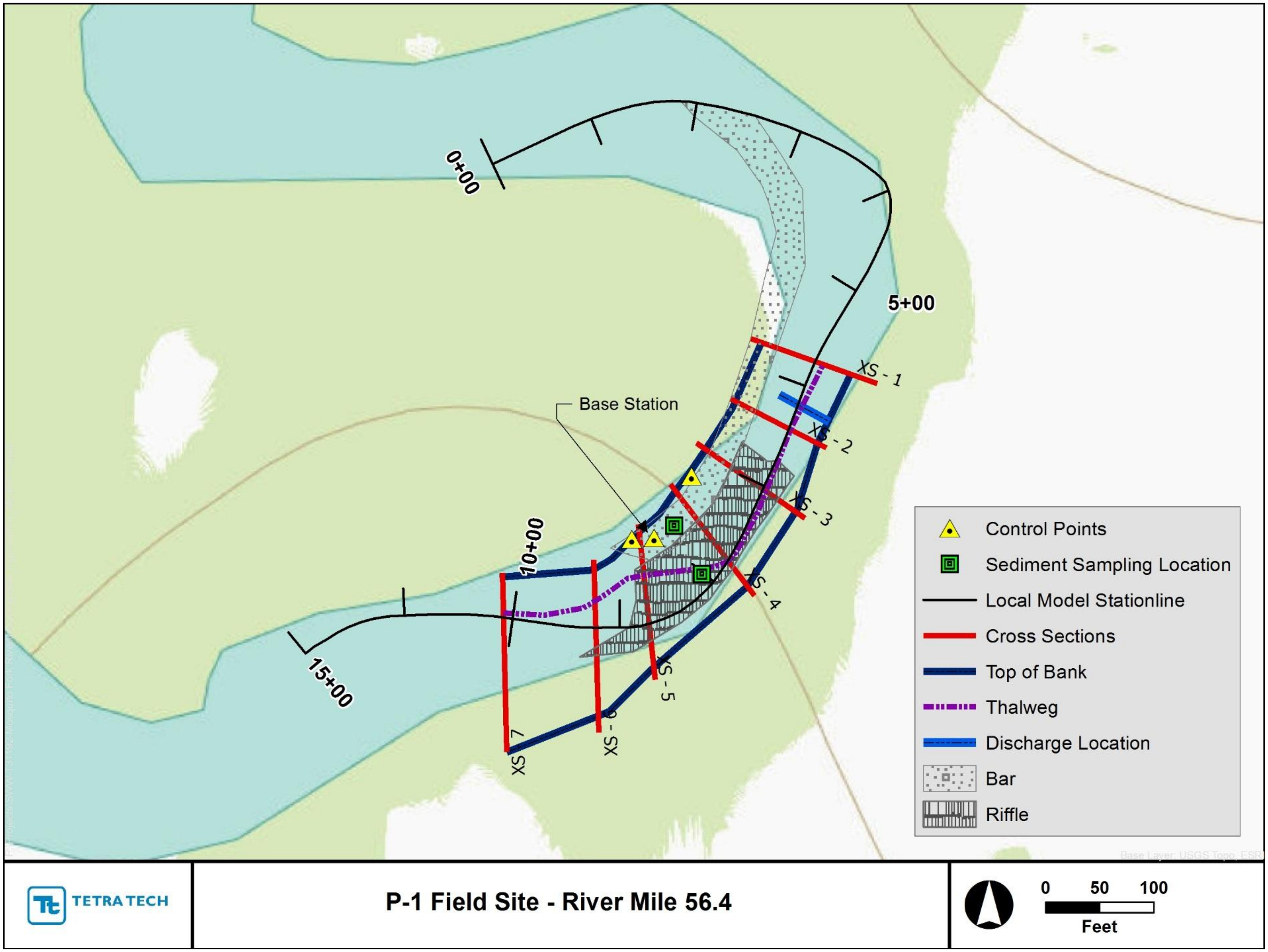


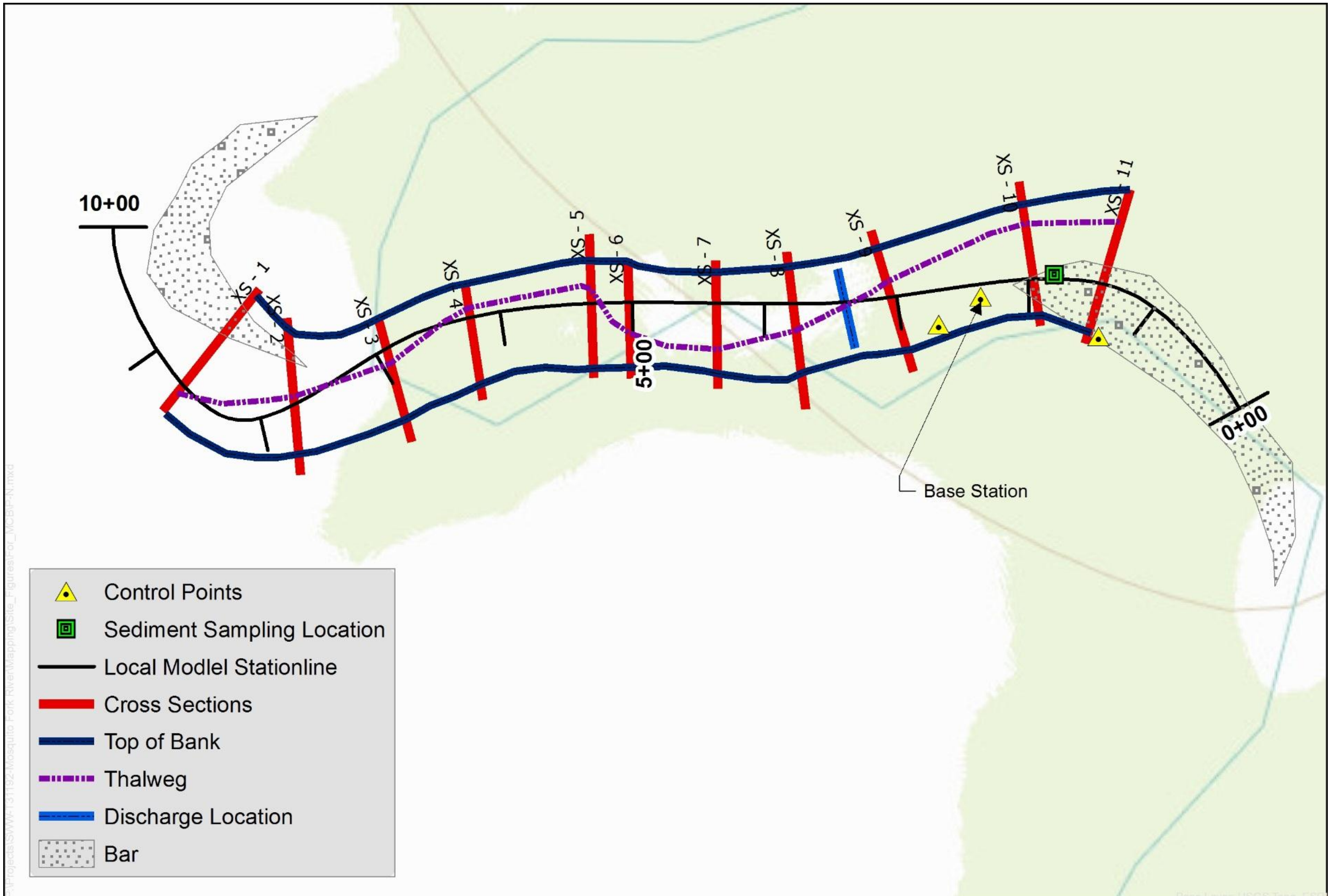
P-3 Field Site - River Mile 30.2



TETRA TECH

P-2 Field Site - River Mile 35


0+00
5+00
10+00
15+00
XS - 1
XS - 2
XS - 3
XS - 4
XS - 5
XS - 6
XS - 7
Base Station
Control Points
Sediment Sampling Location
Local Model Stationline
Cross Sections
Top of Bank
Thalweg
Discharge Location
Bar
Riffle
TETRA TECH
P-1 Field Site - River Mile 56.4
0
50
100
Feet

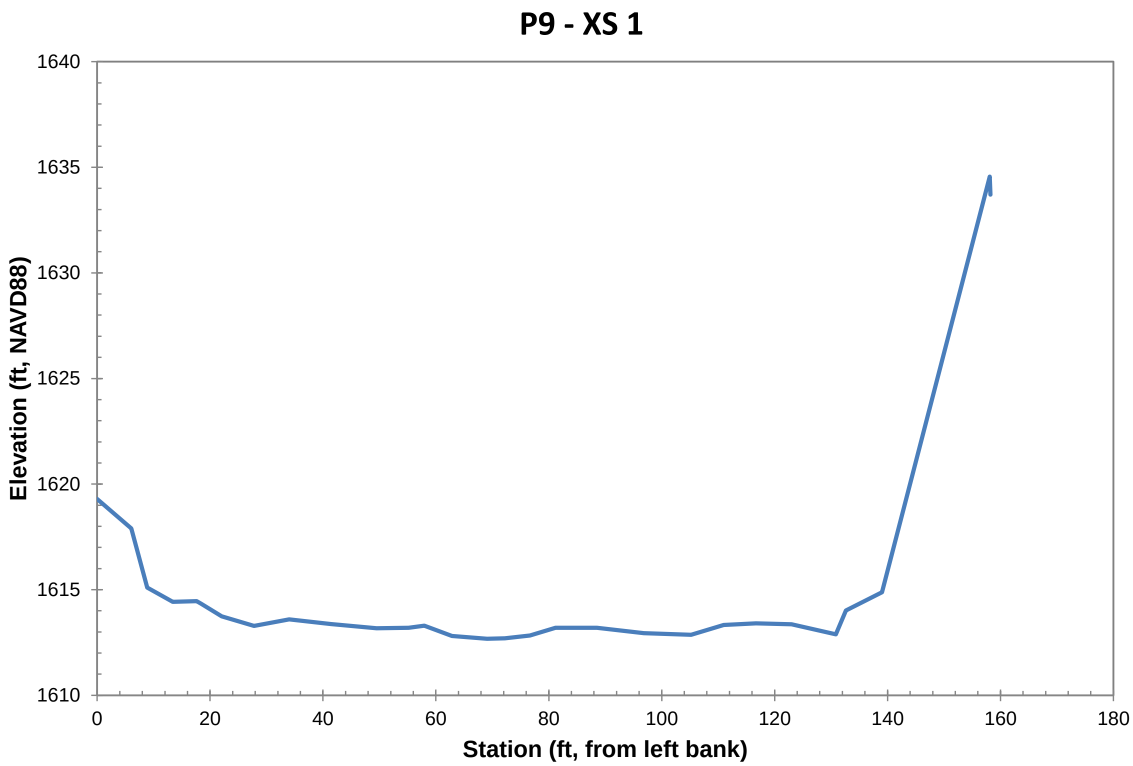


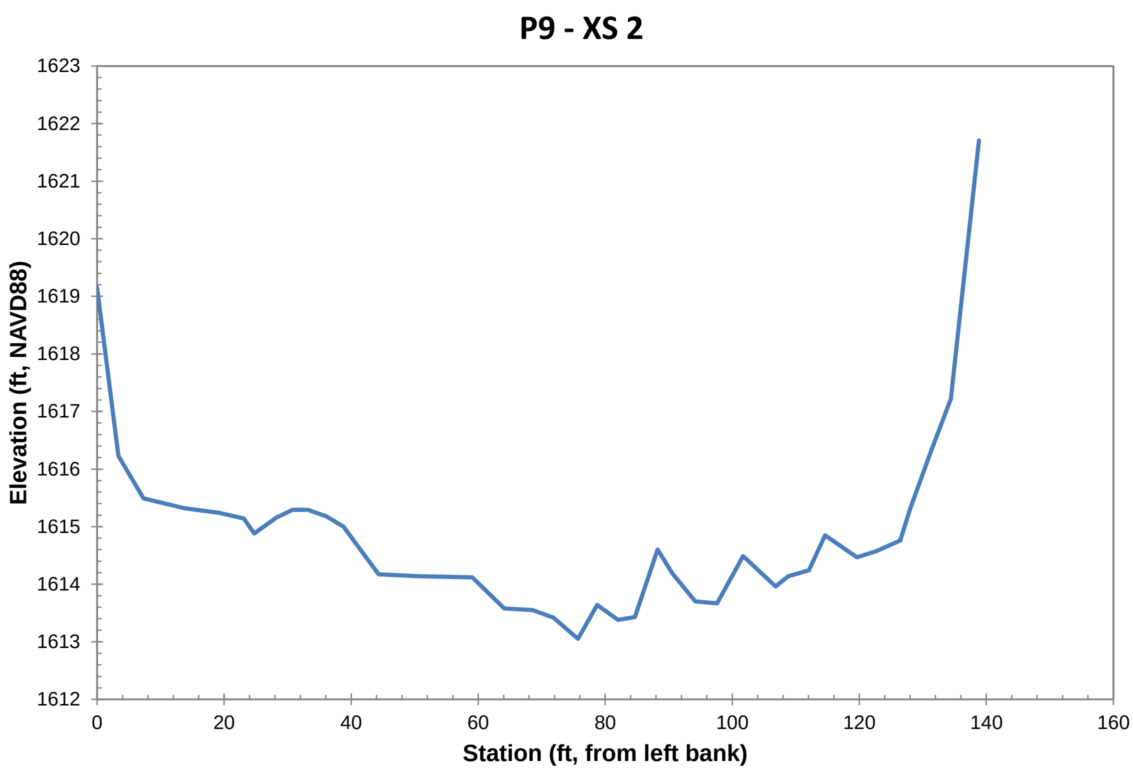
TETRA TECH

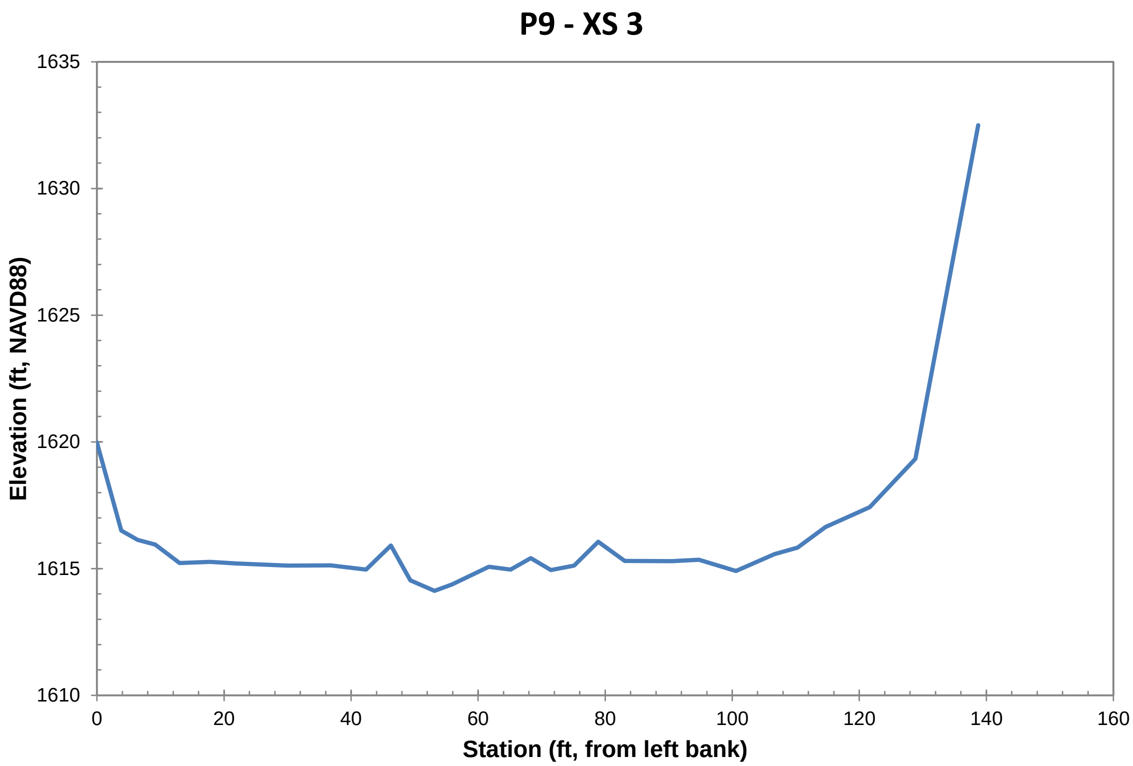
P-N Field Site - River Mile 71.6

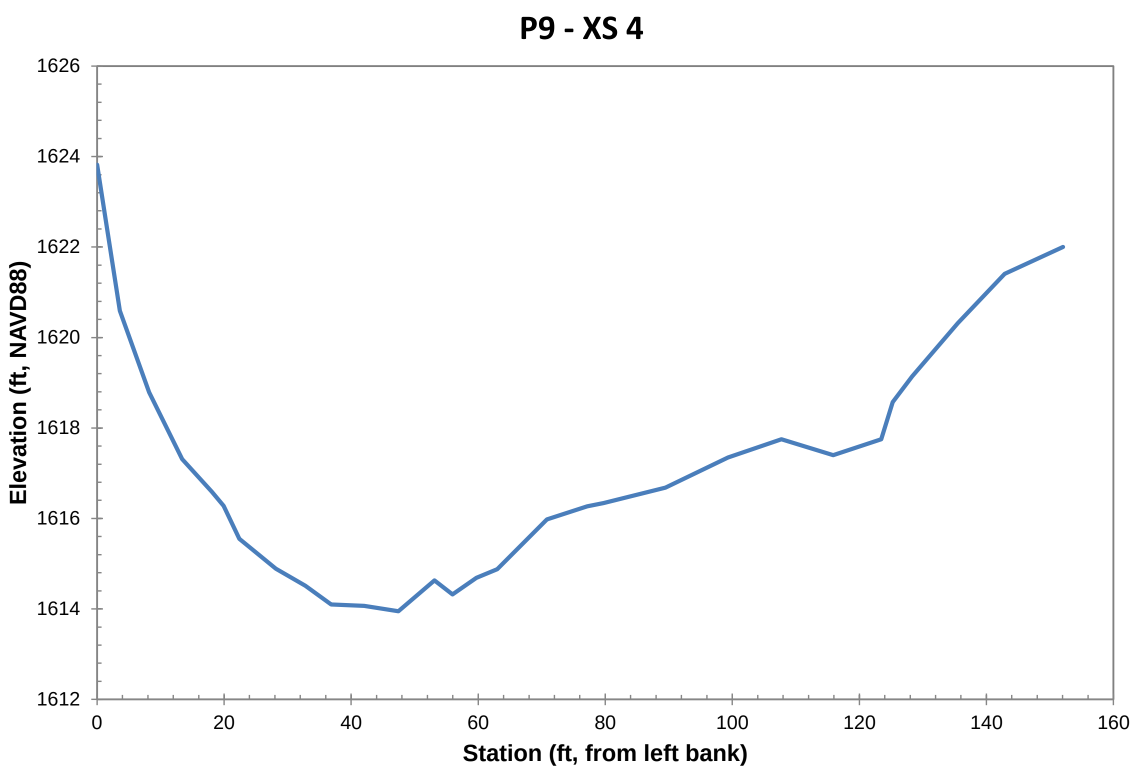
APPENDIX B

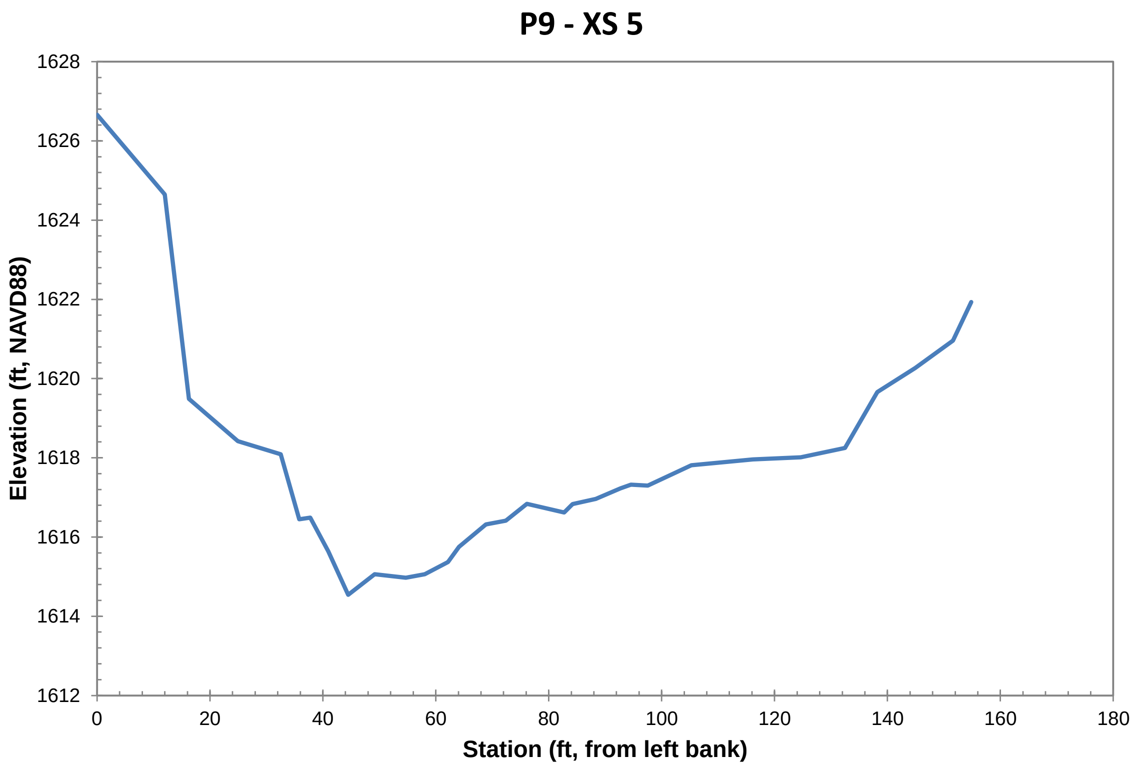
Survey Cross-section Plots of the Individual Cross Sections

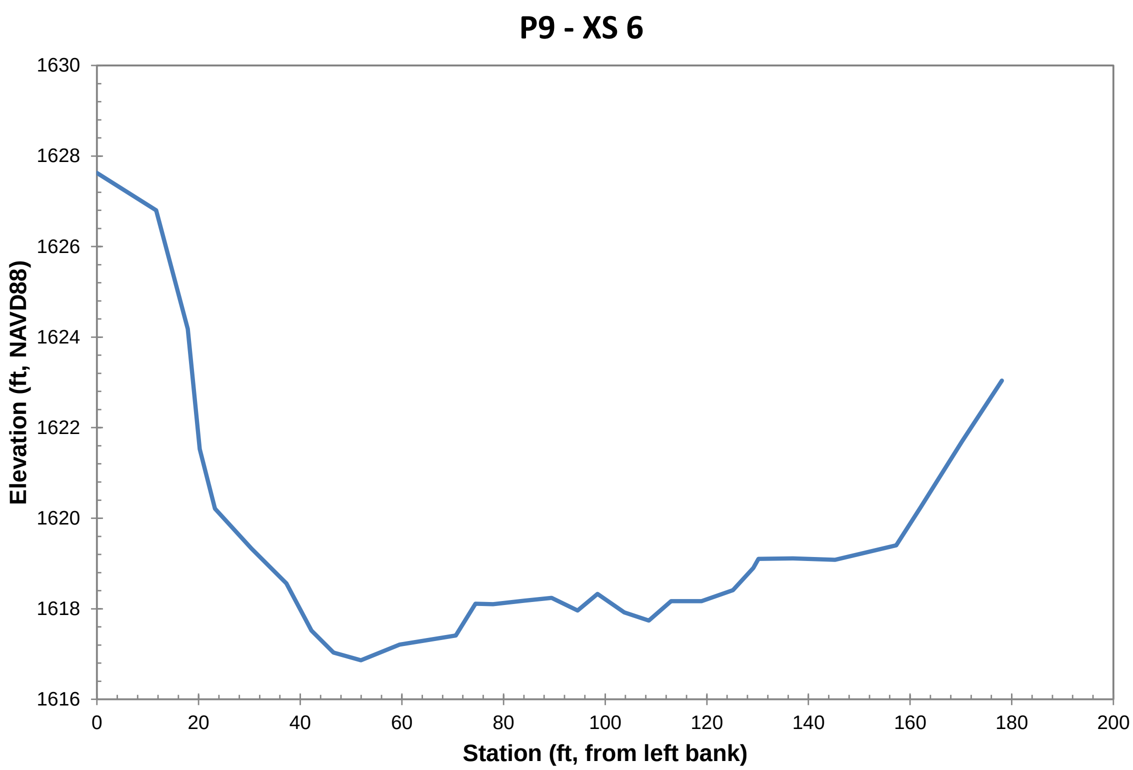

P9 - XS 1
Elevation (ft, NAVD88)
1640
1635
1630
1625
1620
1615
1610
0
20
40
60
80
100
120
140
160
180
Station (ft, from left bank)

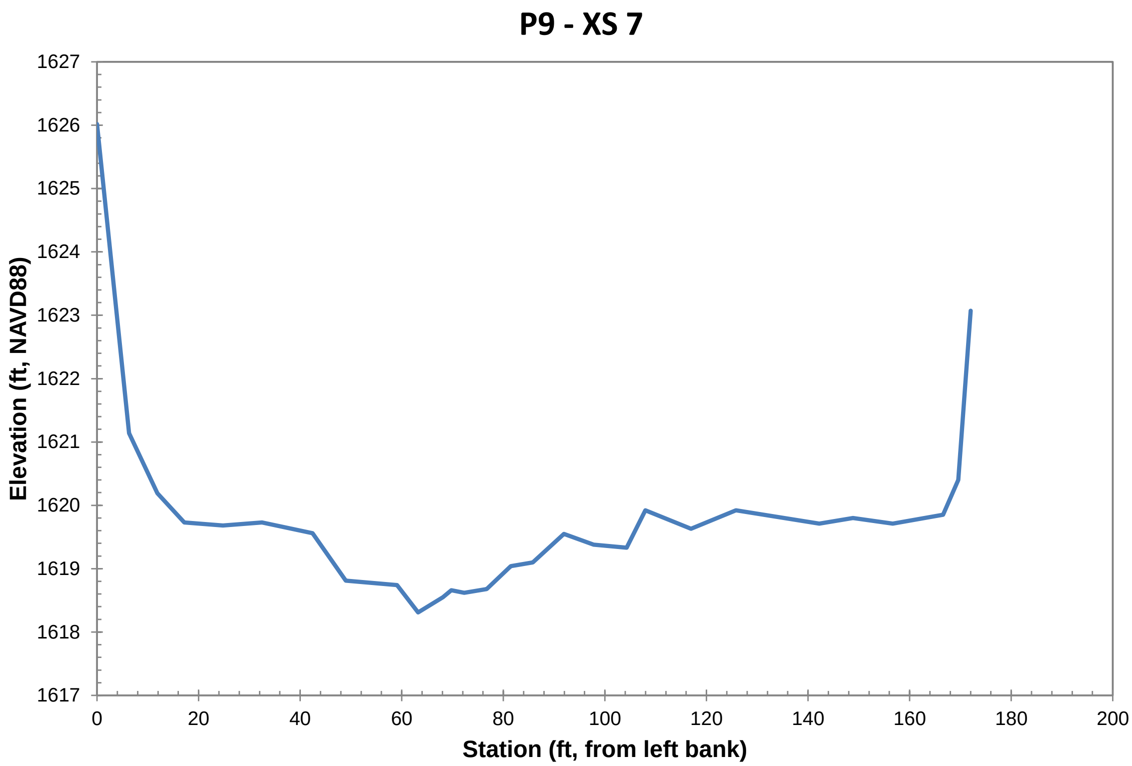

P9 - XS 2
Elevation (ft, NAVD88)
1623
1622
1621
1620
1619
1618
1617
1616
1615
1614
1613
1612
0
20
40
60
80
100
120
140
160
Station (ft, from left bank)

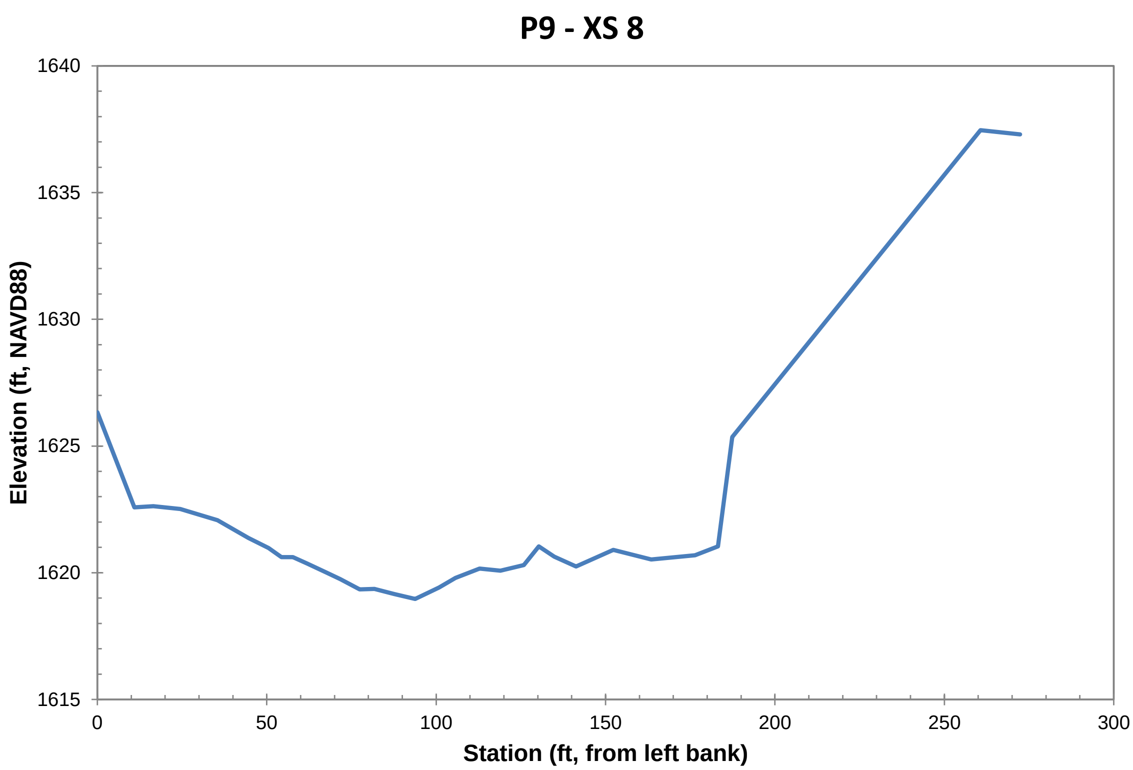

P9 - XS 3
1635
1630
1625
1620
1615
1610
Elevation (ft, NAVD88)
0
20
40
60
80
100
120
140
160
Station (ft, from left bank)

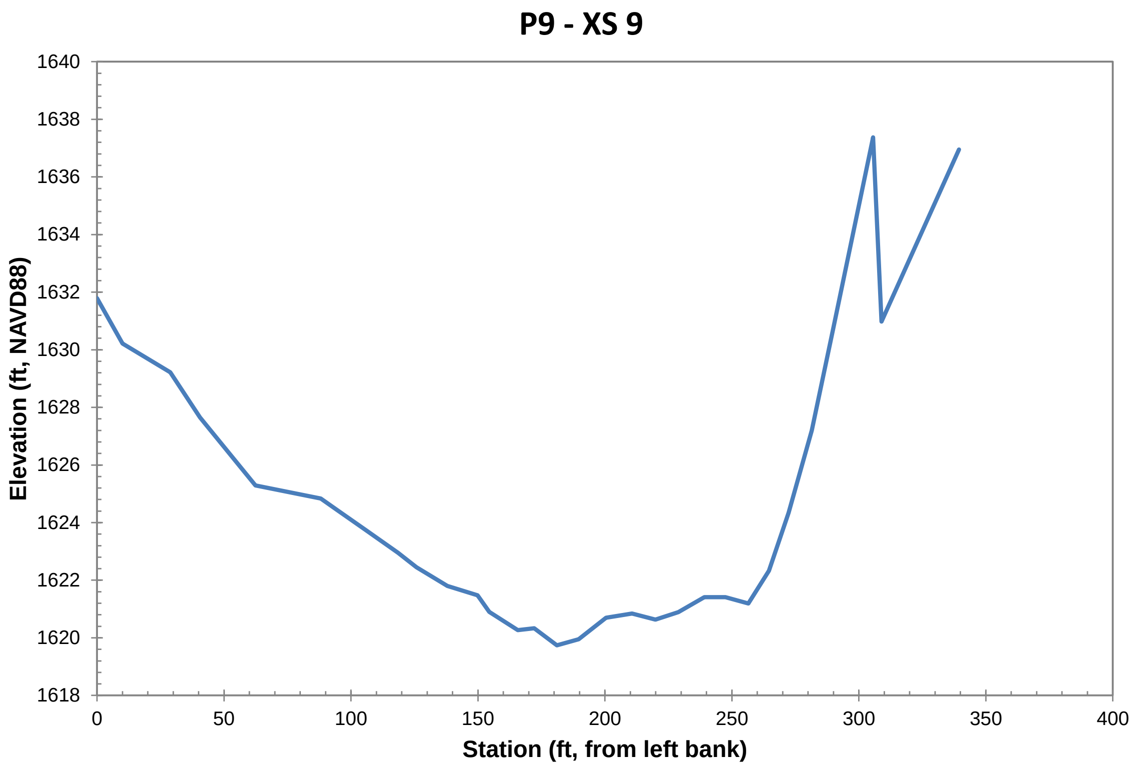

P9 - XS 4
1626
1624
1622
1620
1618
1616
1614
1612
Elevation (ft, NAVD88)
0
20
40
60
80
100
120
140
160
Station (ft, from left bank)

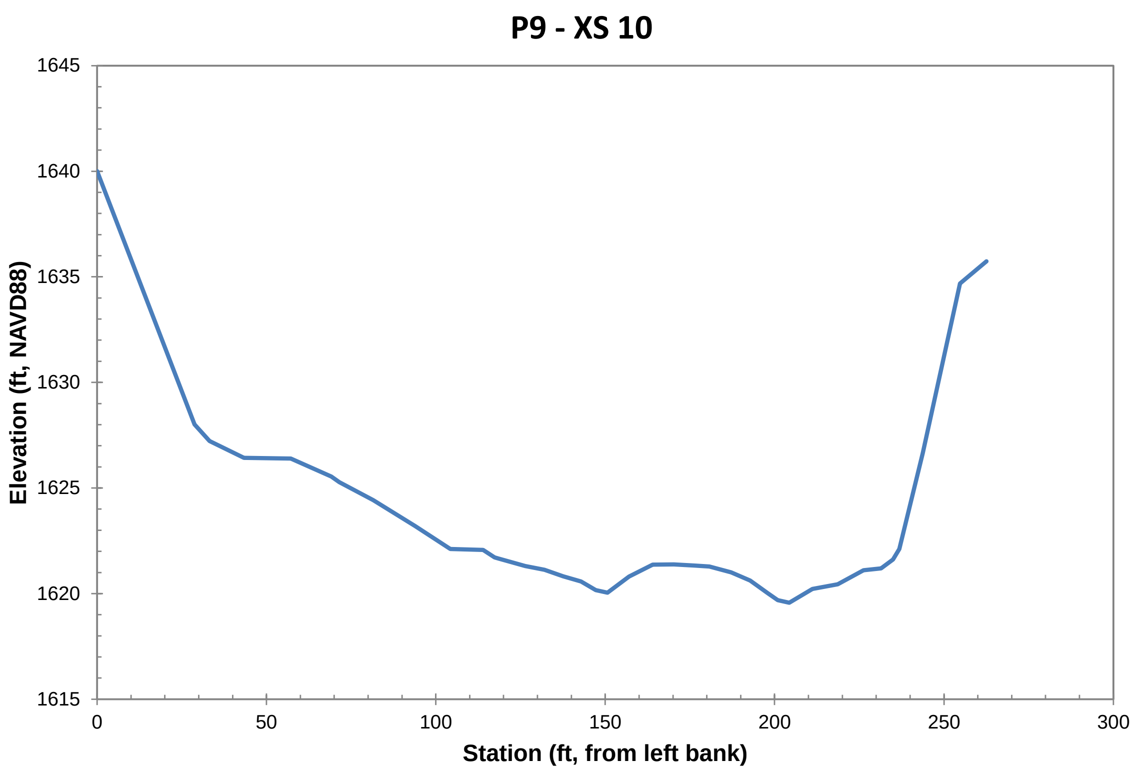

P9 - XS 5
Elevation (ft, NAVD88)
Station (ft, from left bank)
1628
1626
1624
1622
1620
1618
1616
1614
1612
0
20
40
60
80
100
120
140
160
180

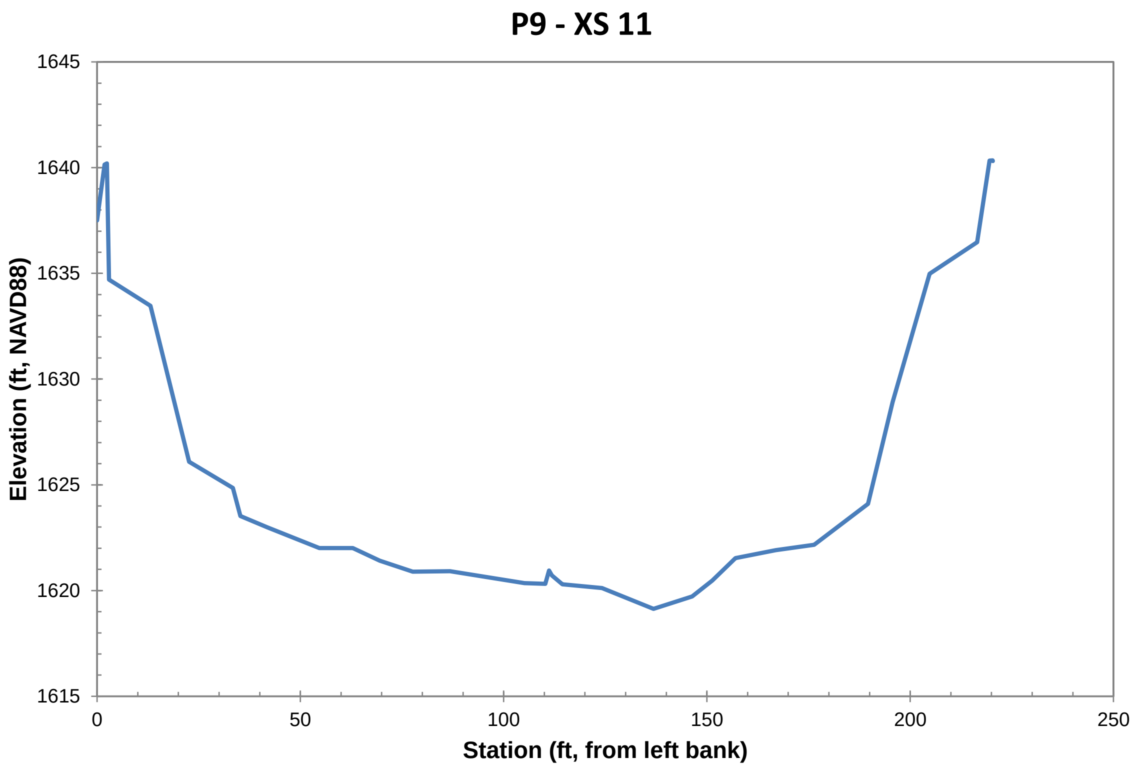

P9 - XS 6
Elevation (ft, NAVD88)
Station (ft, from left bank)
1630
1628
1626
1624
1622
1620
1618
1616
0
20
40
60
80
100
120
140
160
180
200

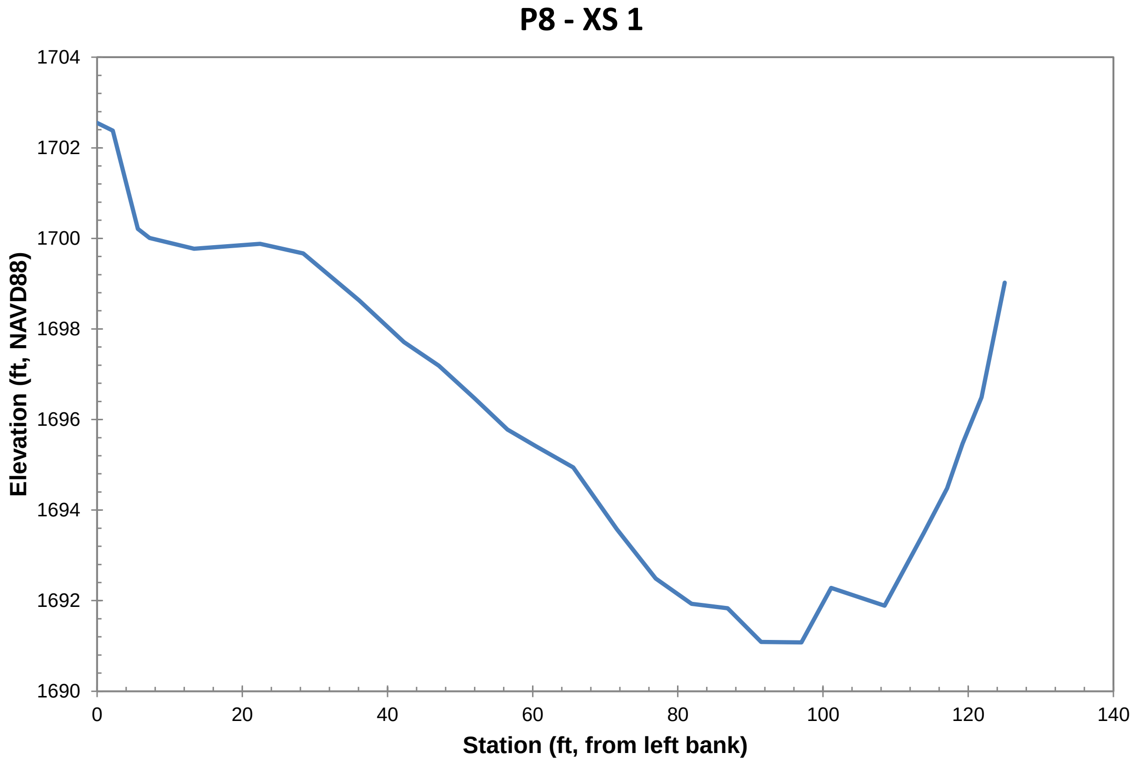

P9 - XS 7
1627
1626
1625
1624
1623
1622
1621
1620
1619
1618
1617
Elevation (ft, NAVD88)
0
20
40
60
80
100
120
140
160
180
200
Station (ft, from left bank)

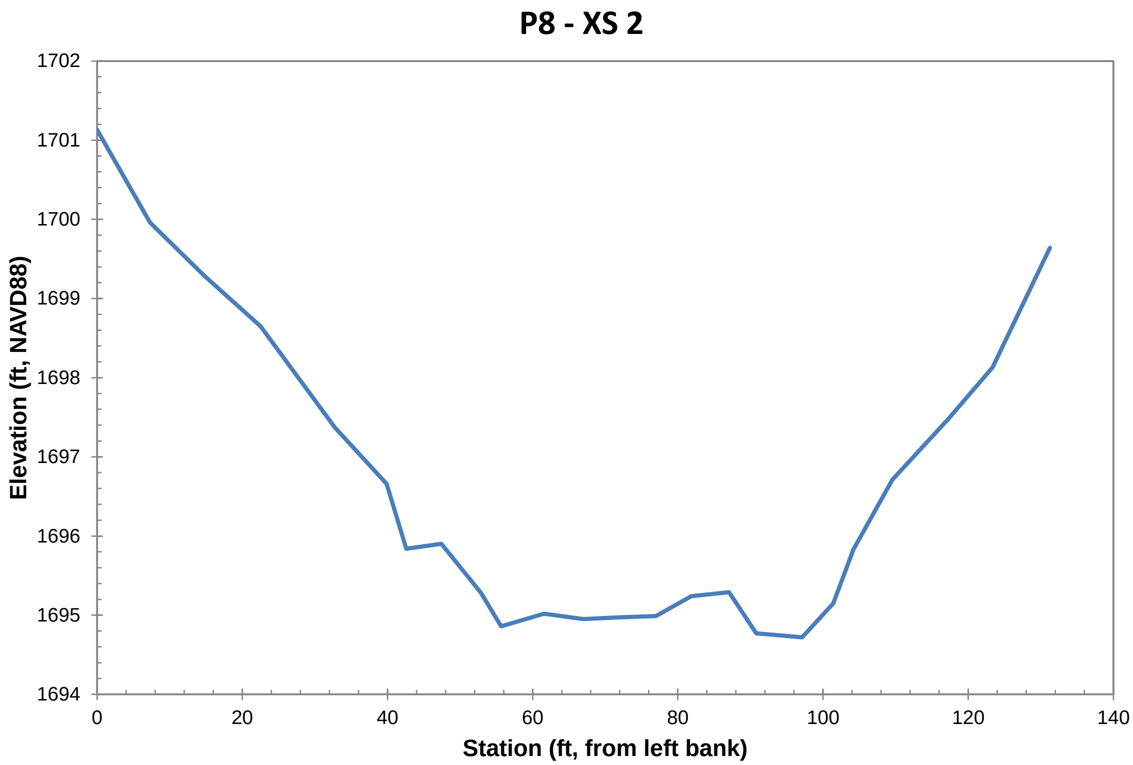

P9 - XS 8
1640
1635
1630
1625
1620
1615
Elevation (ft, NAVD88)
0
50
100
150
200
250
300
Station (ft, from left bank)

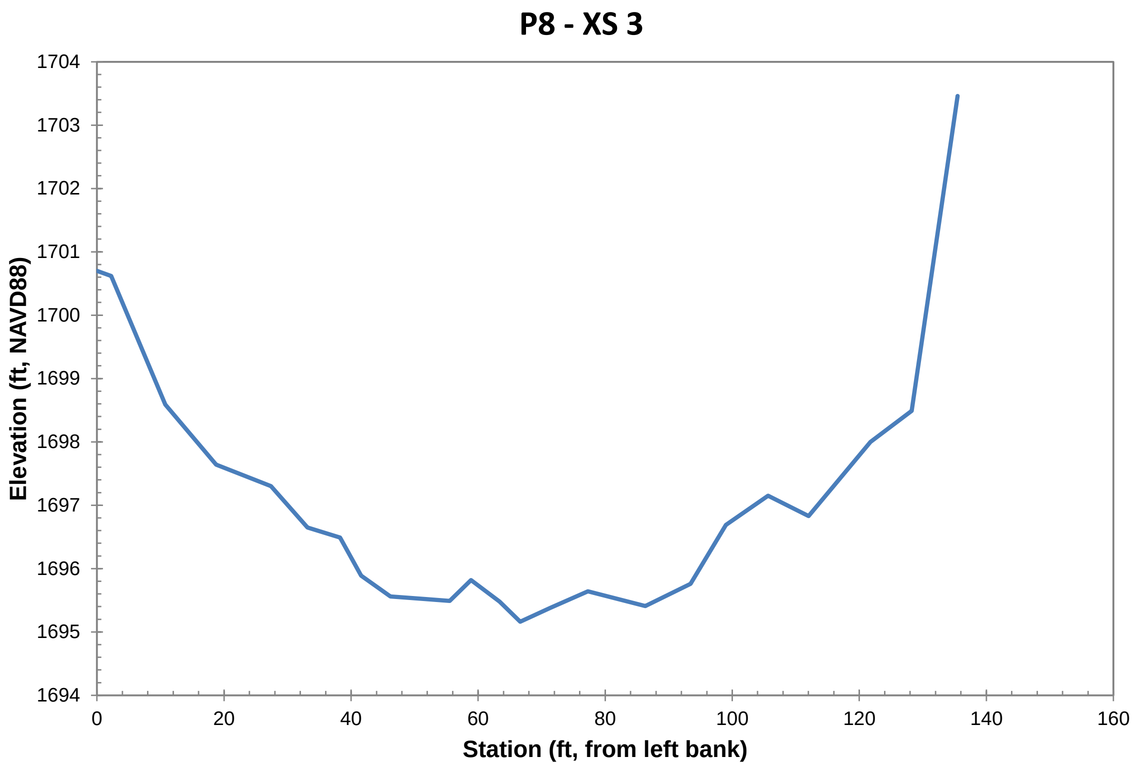

P9 - XS 9
Elevation (ft, NAVD88)
1640
1638
1636
1634
1632
1630
1628
1626
1624
1622
1620
1618
0
50
100
150
200
250
300
350
400
Station (ft, from left bank)

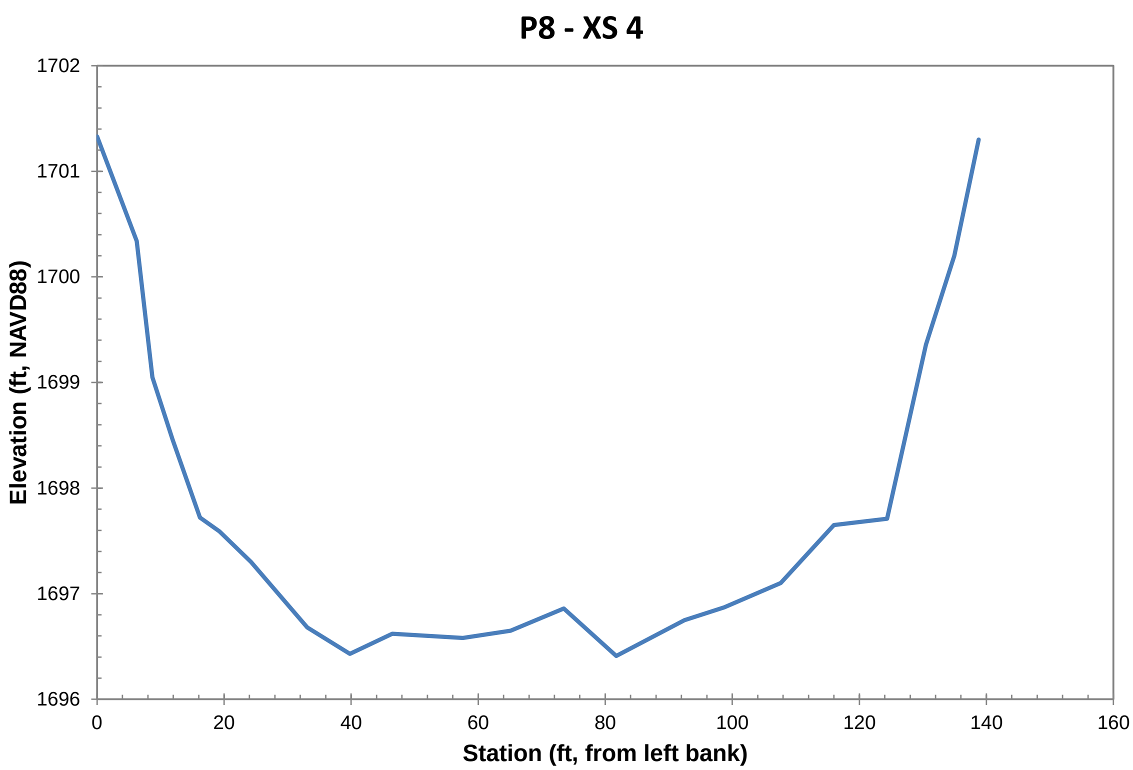

P9 - XS 10
Elevation (ft, NAVD88)
1645
1640
1635
1630
1625
1620
1615
0
50
100
150
200
250
300
Station (ft, from left bank)

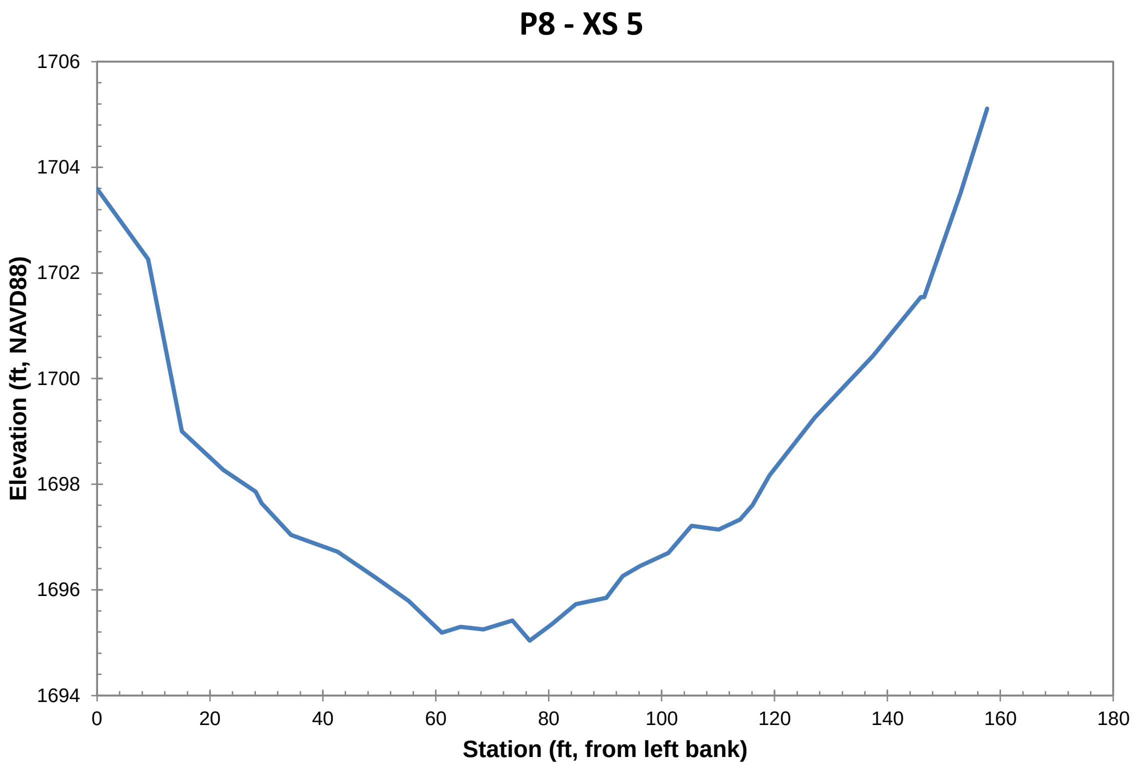

P9 - XS 11
Elevation (ft, NAVD88)
1645
1640
1635
1630
1625
1620
1615
0
50
100
150
200
250
Station (ft, from left bank)


P8 - XS 1
1704
1702
1700
1698
1696
1694
1692
1690
Elevation (ft, NAVD88)
0
20
40
60
80
100
120
140
Station (ft, from left bank)

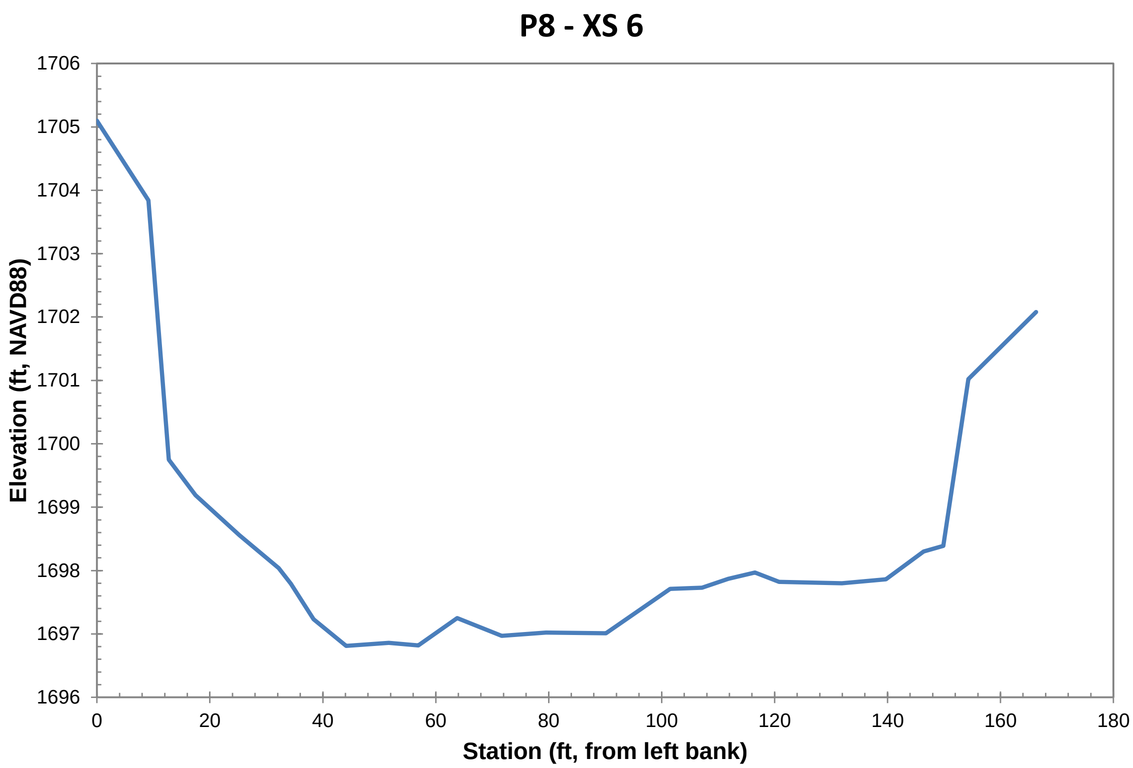

P8 - XS 2
1702
1701
1700
1699
1698
1697
1696
1695
1694
Elevation (ft, NAVD88)
0
20
40
60
80
100
120
140
Station (ft, from left bank)


P8 - XS 3
1704
1703
1702
1701
1700
1699
1698
1697
1696
1695
1694
0
20
40
60
80
100
120
140
160
Elevation (ft, NAVD88)
Station (ft, from left bank)


P8 - XS 4
1702
1701
1700
1699
1698
1697
1696
0
20
40
60
80
100
120
140
160
Elevation (ft, NAVD88)
Station (ft, from left bank)

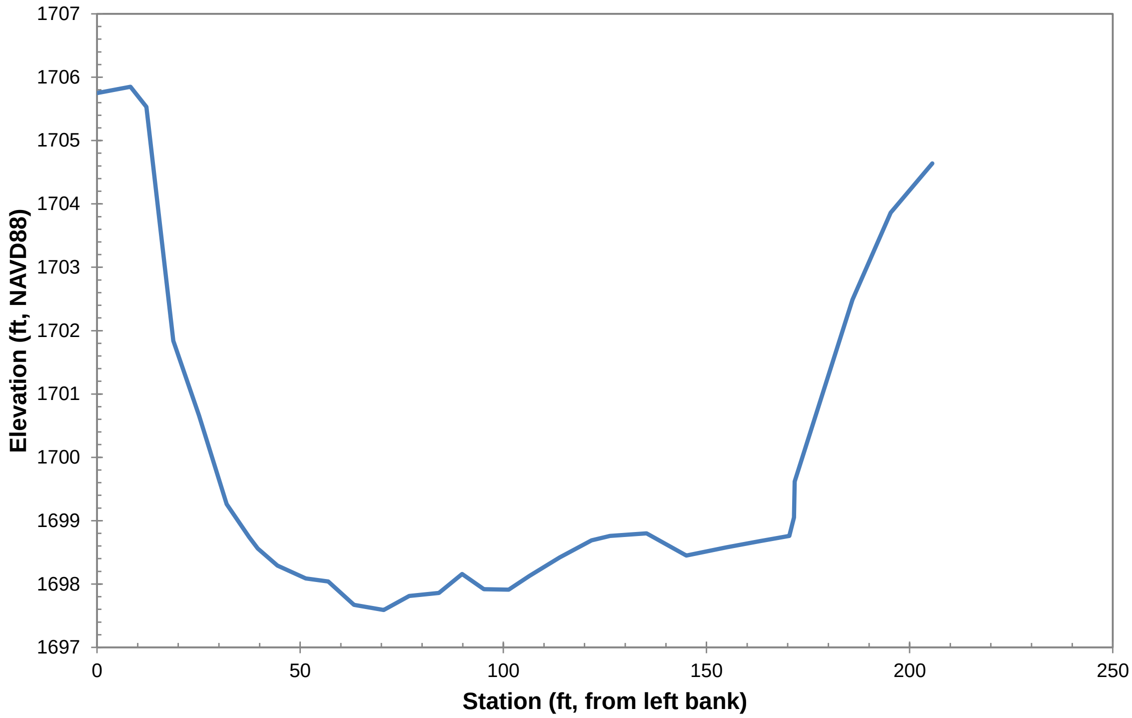


P8 - XS 5
1706
1704
1702
1700
1698
1696
1694
0
20
40
60
80
100
120
140
160
180
Elevation (ft, NAVD88)
Station (ft, from left bank)

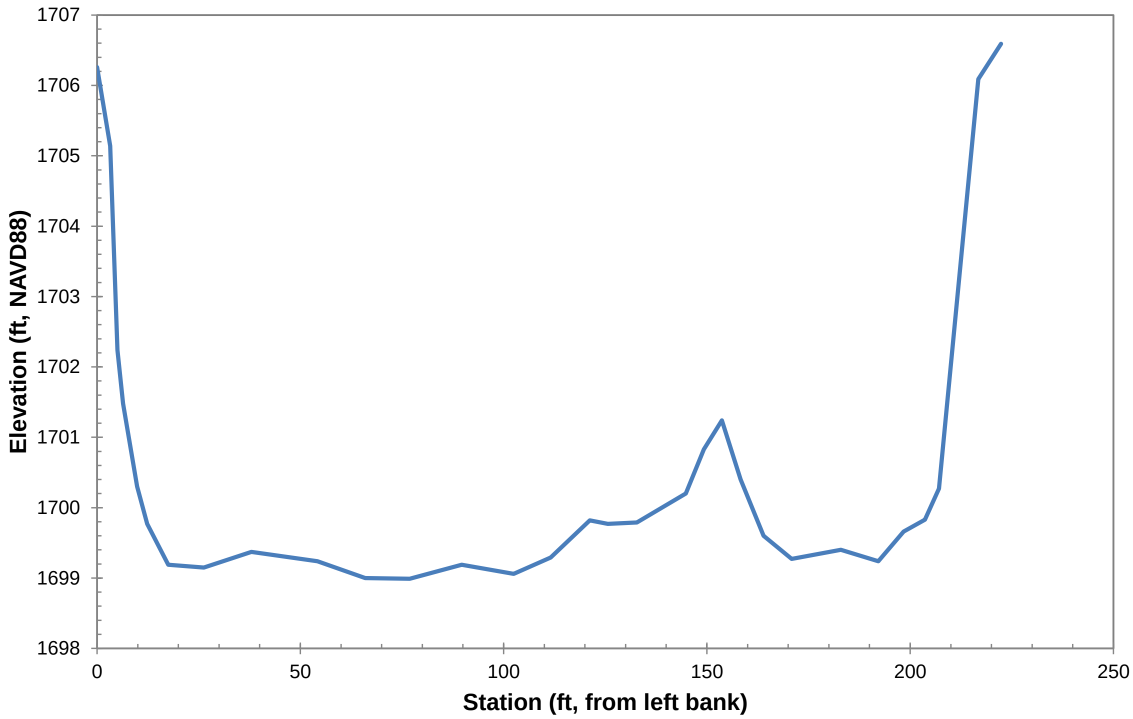


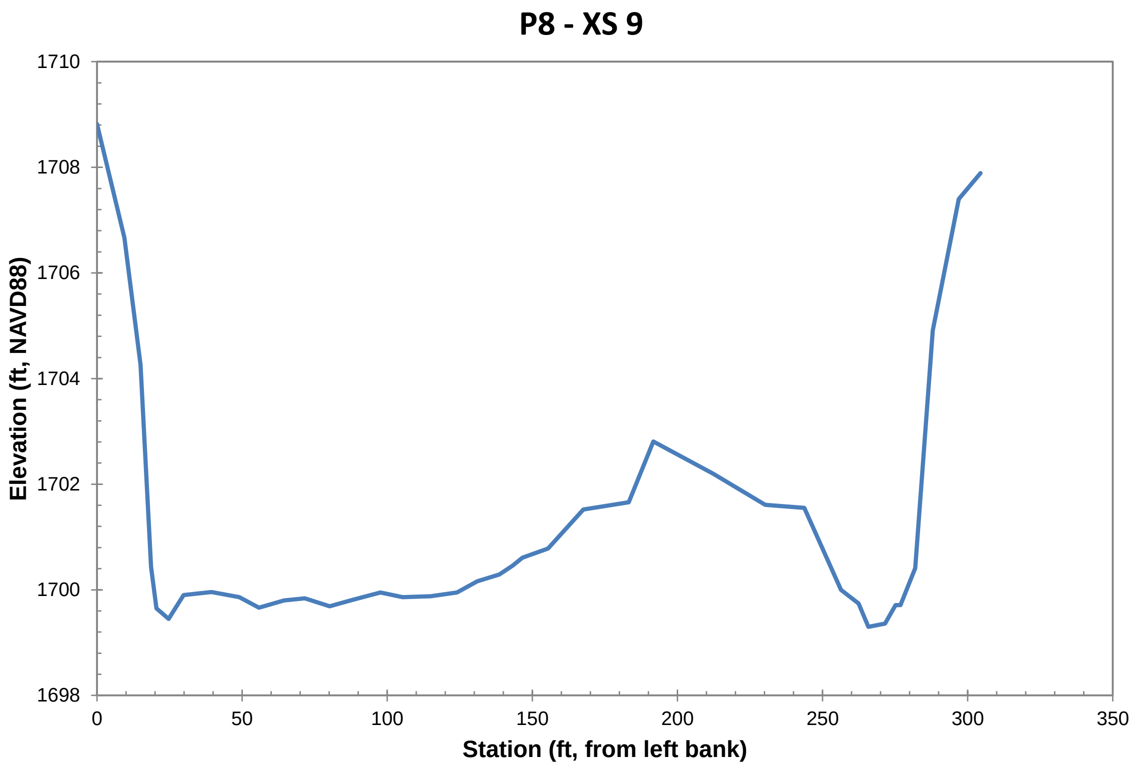
P8 - XS 6
1706
1705
1704
1703
1702
1701
1700
1699
1698
1697
1696
0
20
40
60
80
100
120
140
160
180
Elevation (ft, NAVD88)
Station (ft, from left bank)

P8 - XS 7


1707
1706
1705
1704
1703
1702
1701
1700
1699
1698
1697
Elevation (ft, NAVD88)
0
50
100
150
200
250
Station (ft, from left bank)

P8 - XS 8


1707
1706
1705
1704
1703
1702
1701
1700
1699
1698
Elevation (ft, NAVD88)
0
50
100
150
200
250
Station (ft, from left bank)


P8 - XS 9
1710
1708
1706
1704
1702
1700
1698
0
50
100
150
200
250
300
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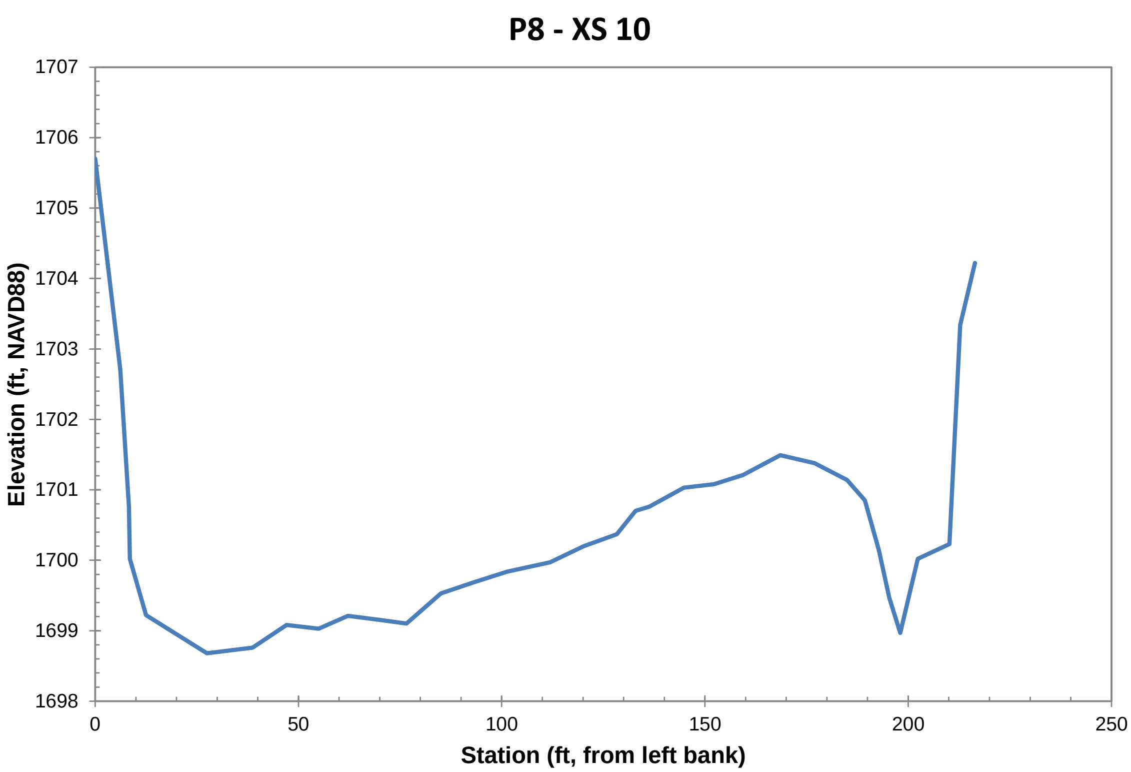
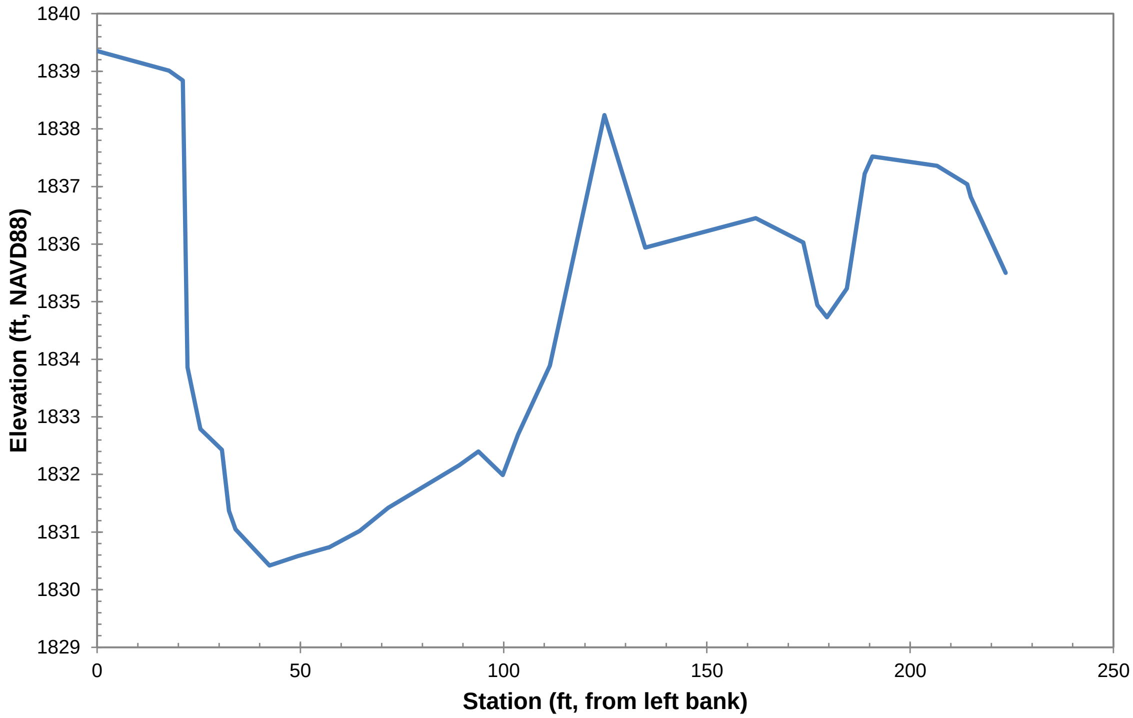
Elevation (ft, NAVD88)
Station (ft, from left bank)

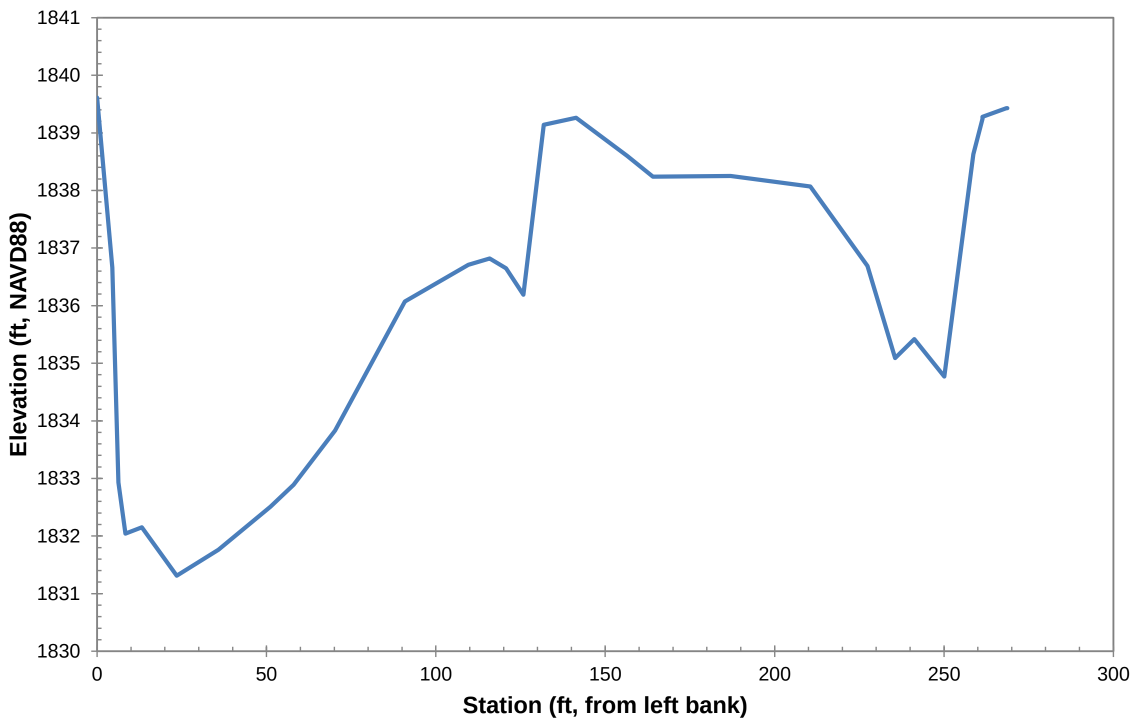

P8 - XS 10
1707
1706
1705
1704
1703
1702
1701
1700
1699
1698
0
50
100
150
200
250
Elevation (ft, NAVD88)
Station (ft, from left bank)

P7 - XS 1

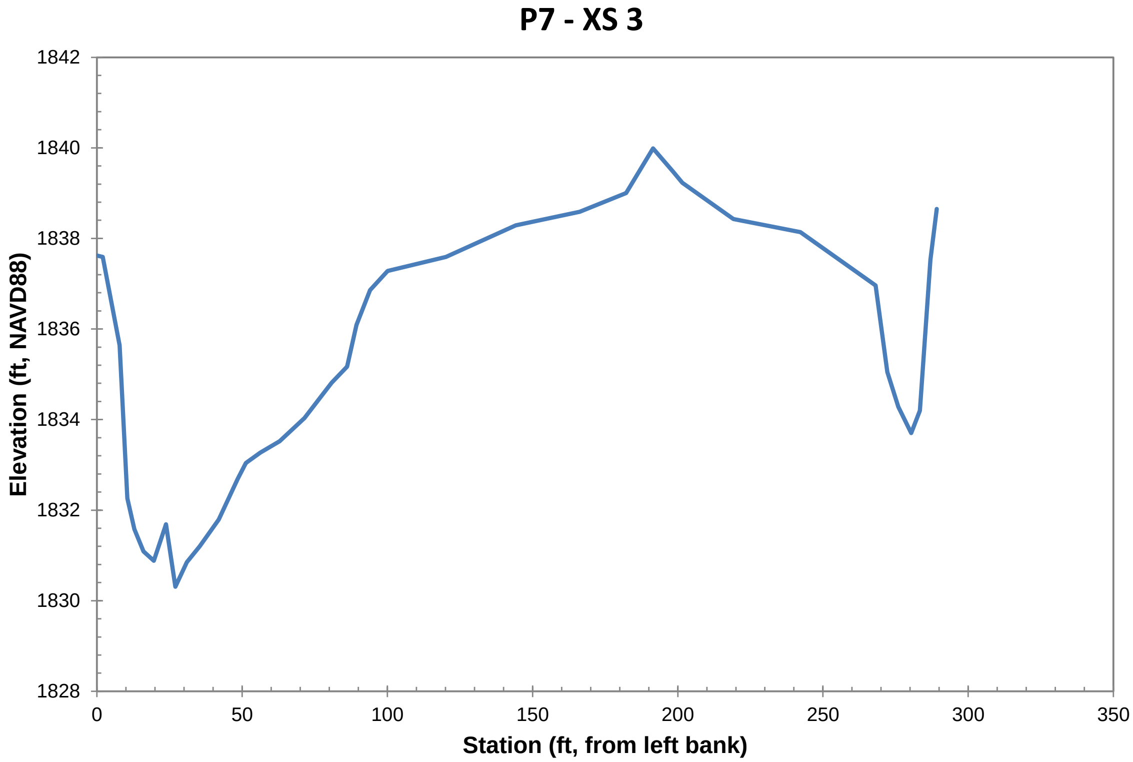


1840
1839
1838
1837
1836
1835
1834
1833
1832
1831
1830
1829
Elevation (ft, NAVD88)
0
50
100
150
200
250
Station (ft, from left bank)

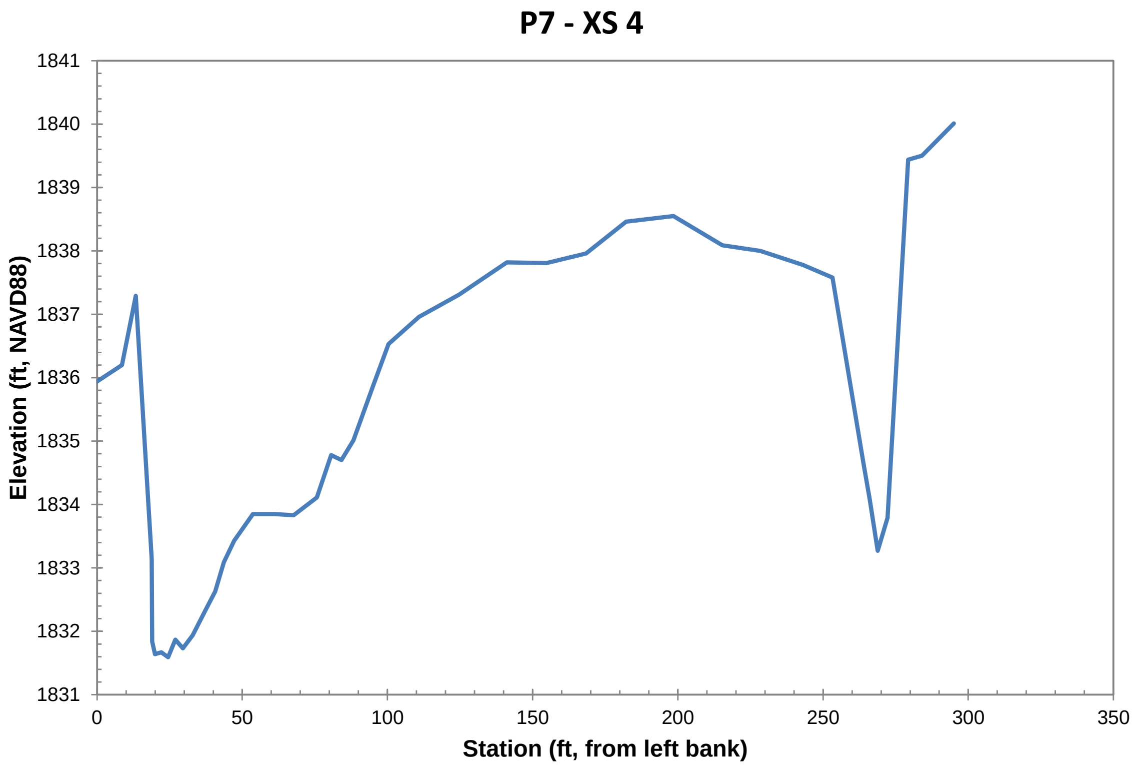
P7 - XS 2



1841
1840
1839
1838
1837
1836
1835
1834
1833
1832
1831
1830
Elevation (ft, NAVD88)
0
50
100
150
200
250
300
Station (ft, from left bank)

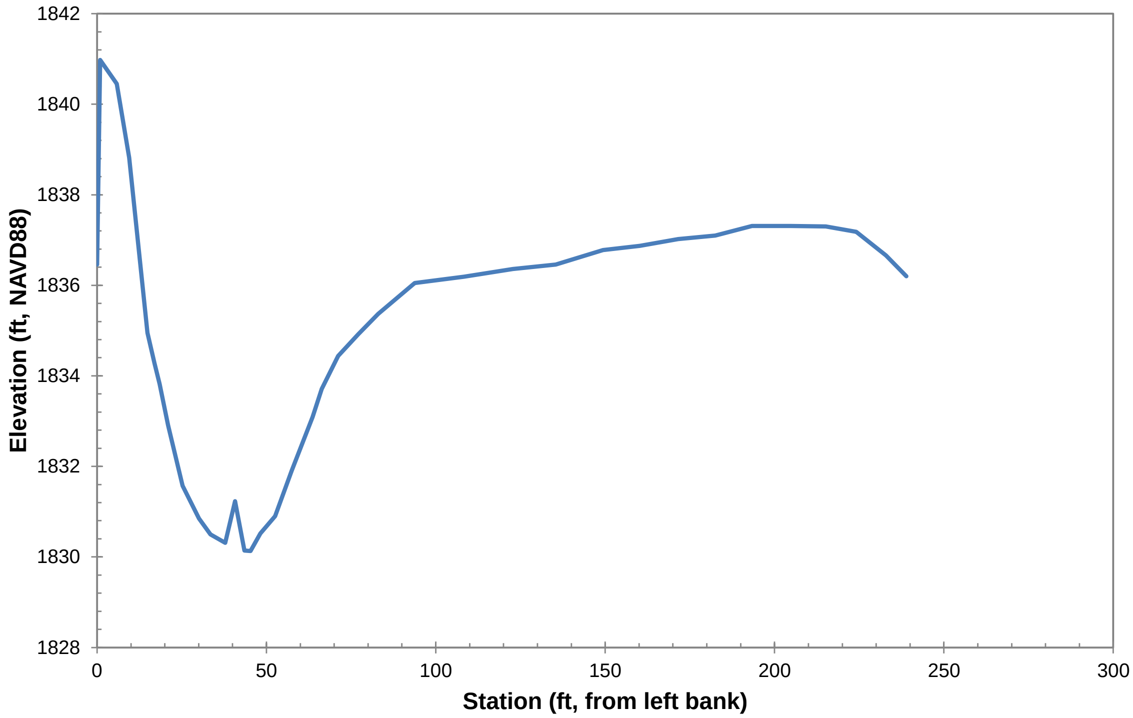


P7 - XS 3
Elevation (ft, NAVD88)
1842
1840
1838
1836
1834
1832
1830
1828
0
50
100
150
200
250
300
350
Station (ft, from left bank)

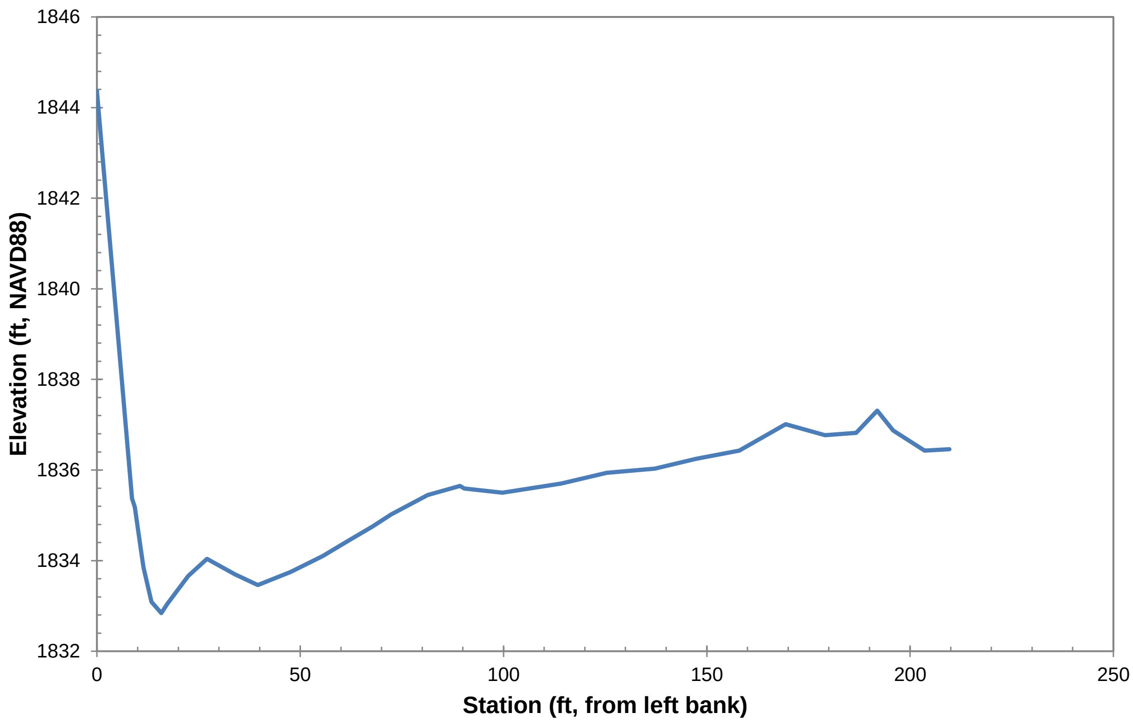


P7 - XS 4
Elevation (ft, NAVD88)
1841
1840
1839
1838
1837
1836
1835
1834
1833
1832
1831
0
50
100
150
200
250
300
350
Station (ft, from left bank)

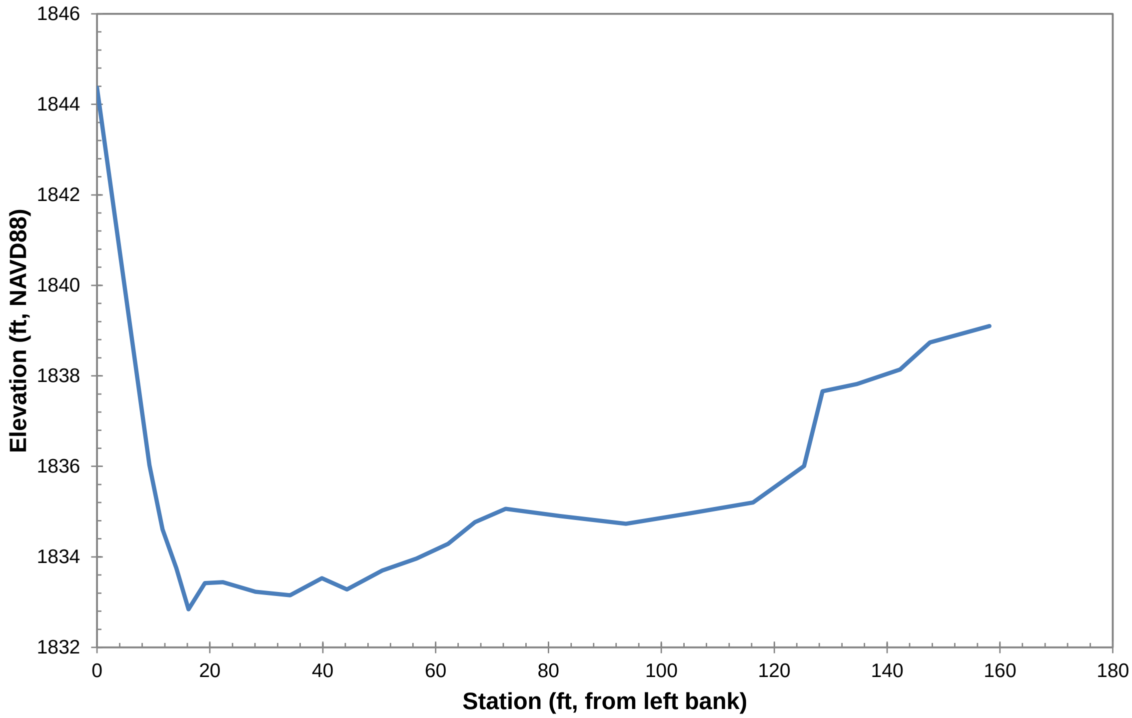
P7 - XS 5


1842
1840
1838
1836
1834
1832
1830
1828
Elevation (ft, NAVD88)
0
50
100
150
200
250
300
Station (ft, from left bank)

P7 - XS 6

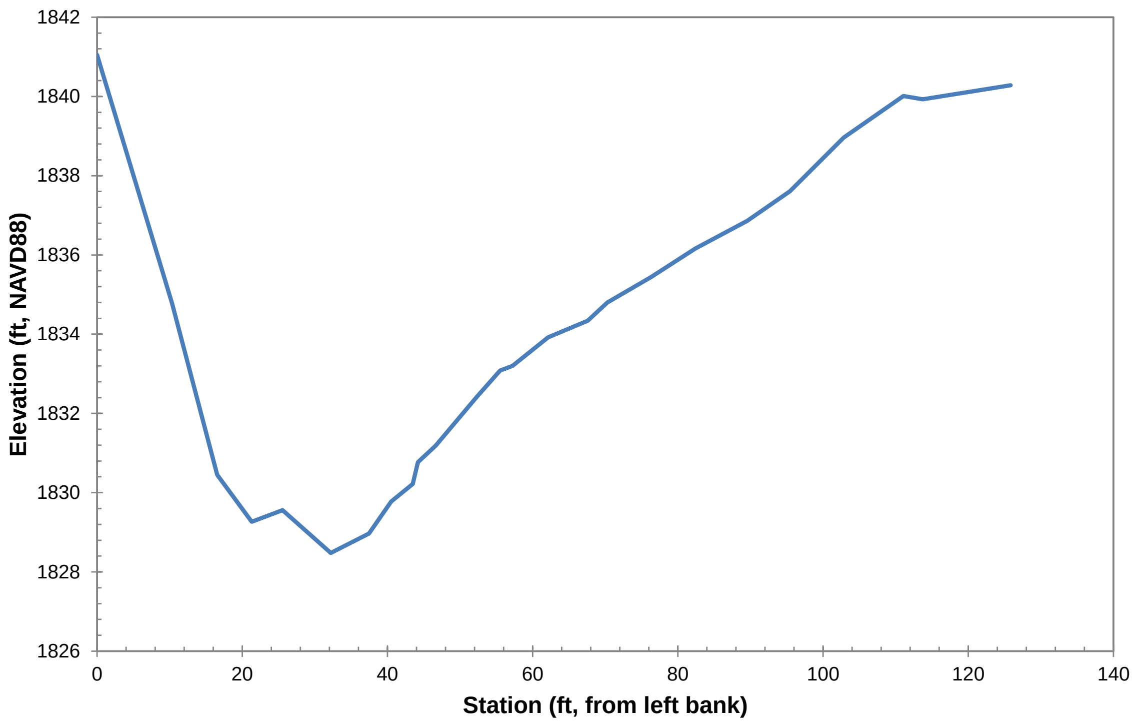

1846
1844
1842
1840
1838
1836
1834
1832
Elevation (ft, NAVD88)
0
50
100
150
200
250
Station (ft, from left bank)

P7 - XS 7

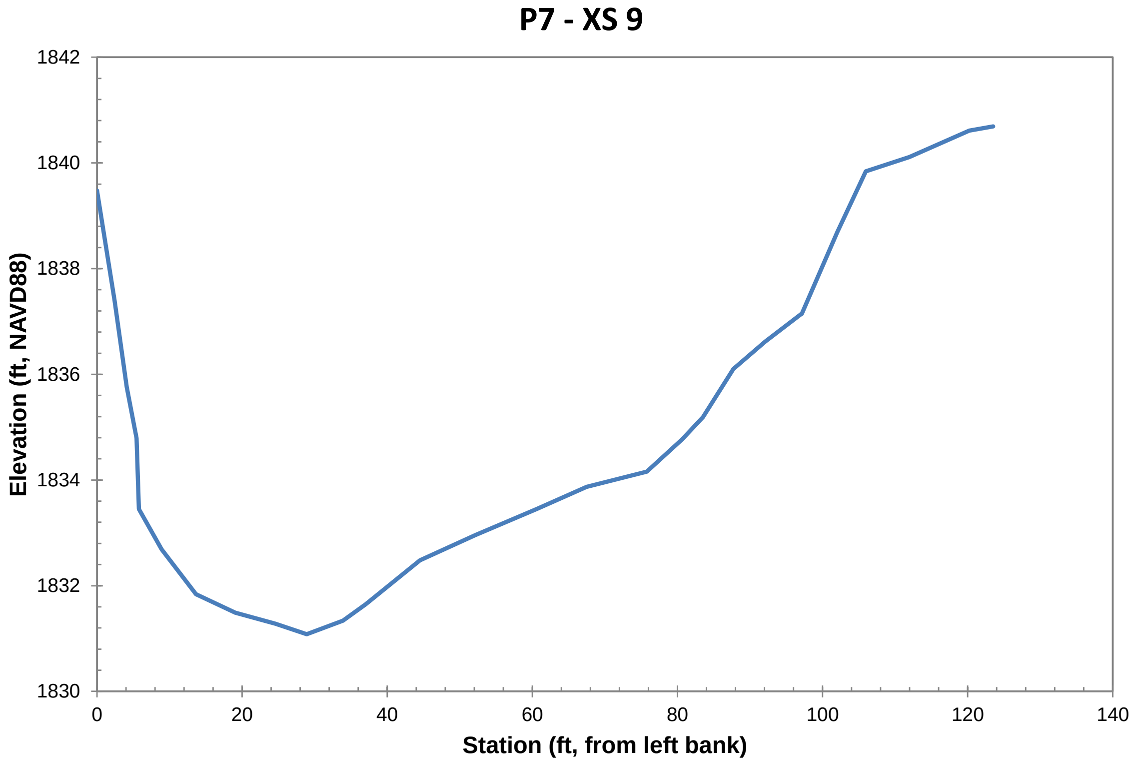


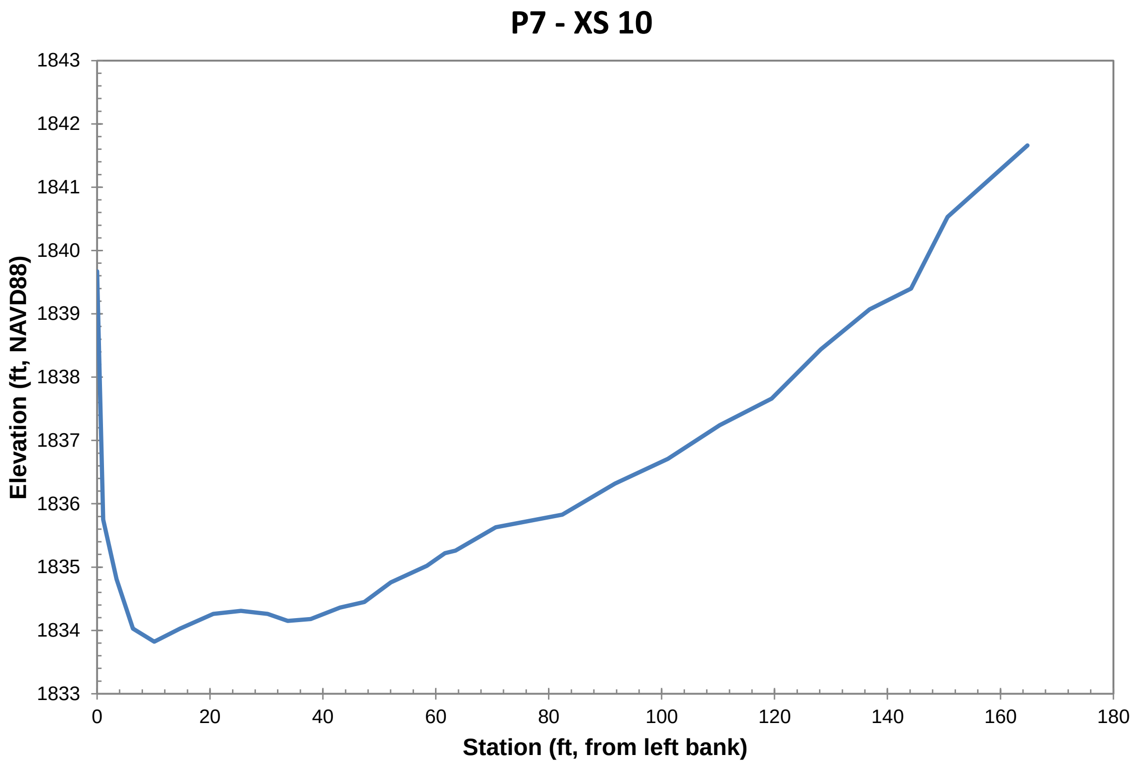
1846
1844
1842
1840
1838
1836
1834
1832
Elevation (ft, NAVD88)
0
20
40
60
80
100
120
140
160
180
Station (ft, from left bank)

P7 - XS 8

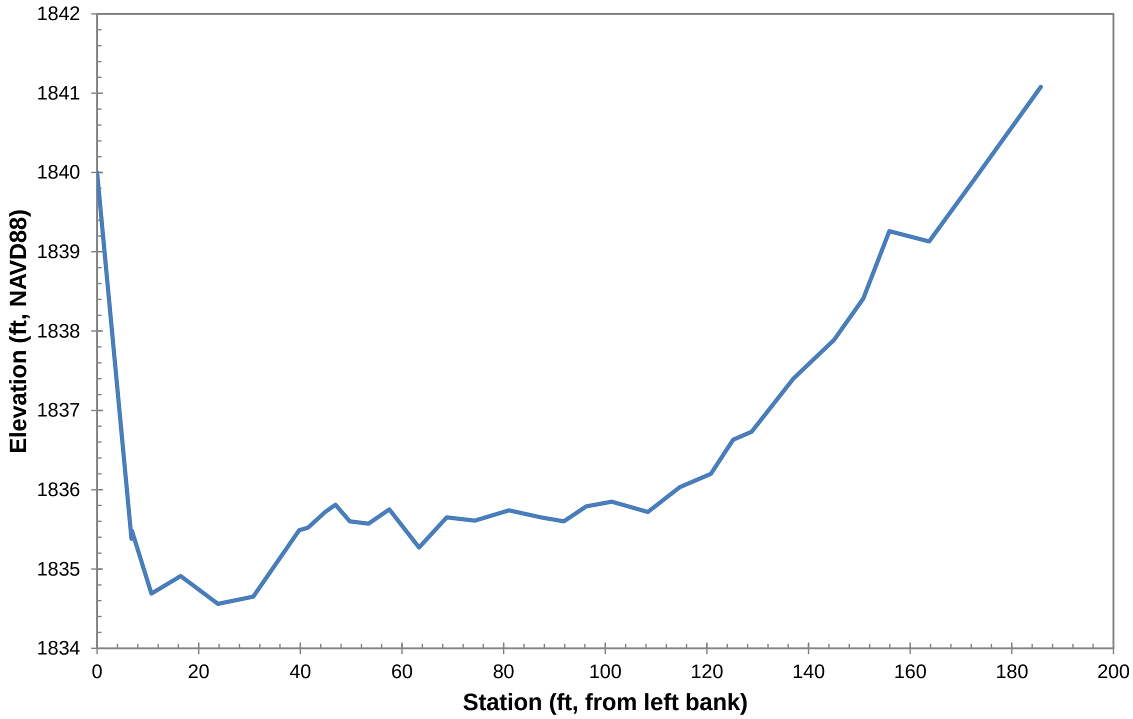


1842
1840
1838
1836
1834
1832
1830
1828
1826
Elevation (ft, NAVD88)
0
20
40
60
80
100
120
140
Station (ft, from left bank)

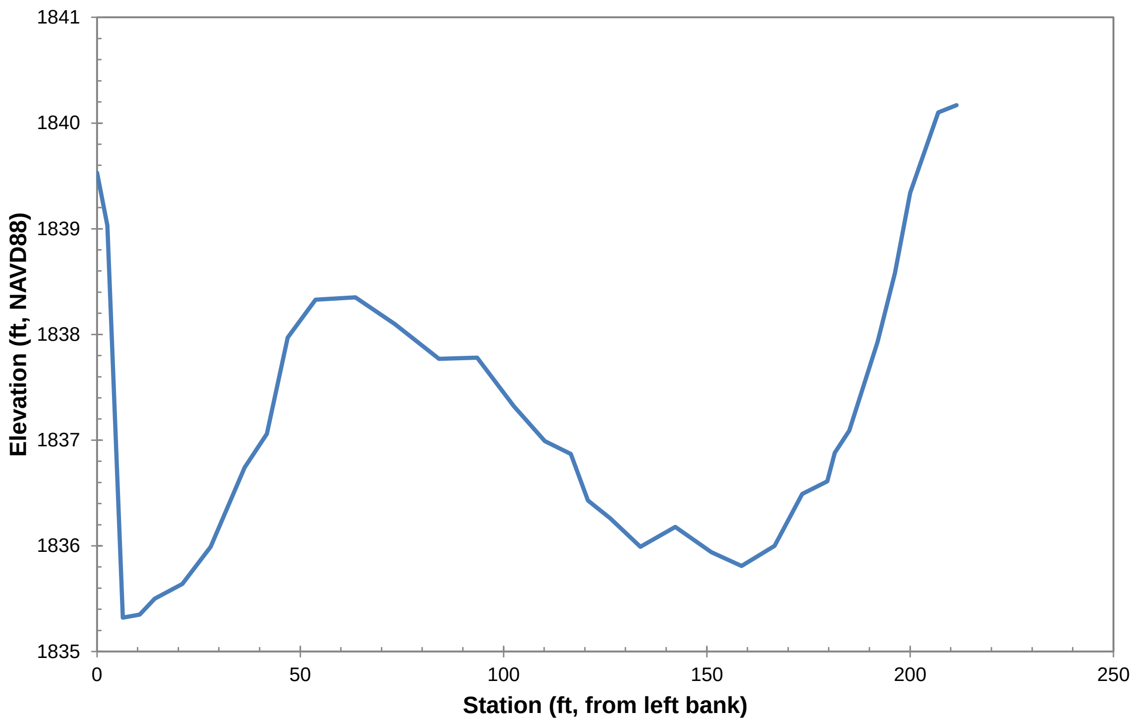

P7 - XS 9
Elevation (ft, NAVD88)
Station (ft, from left bank)
1842
1840
1838
1836
1834
1832
1830
0
20
40
60
80
100
120
140

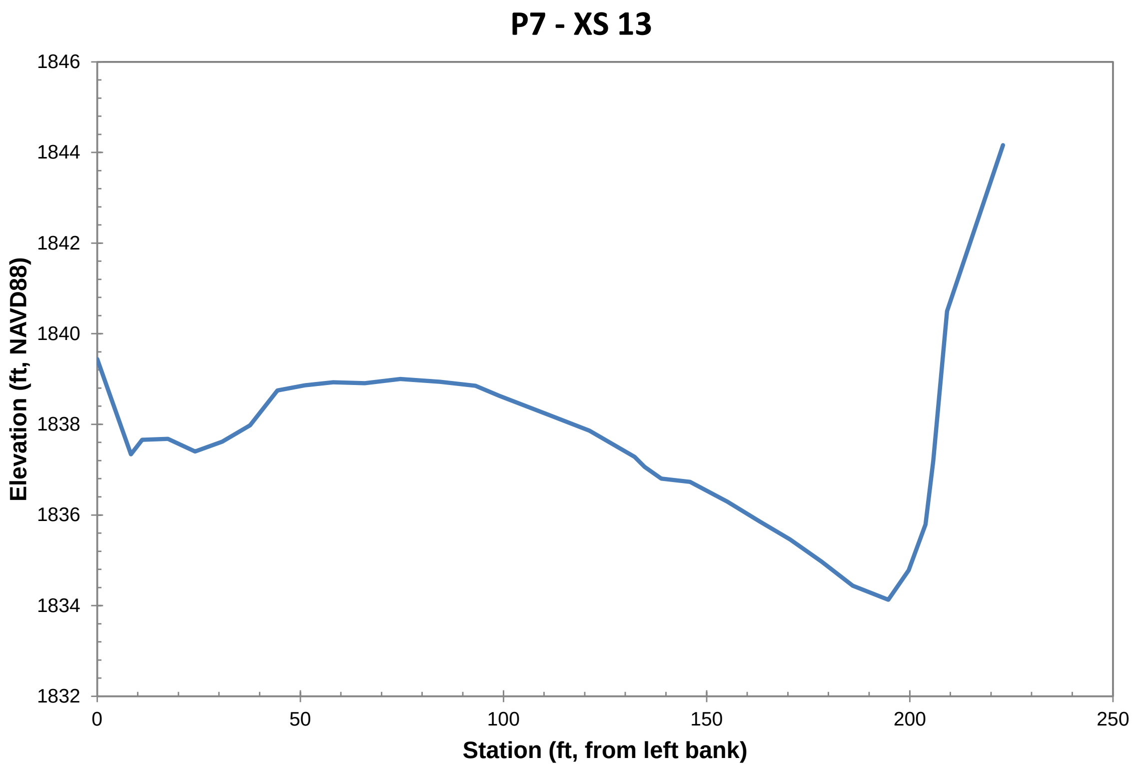

P7 - XS 10
Elevation (ft, NAVD88)
Station (ft, from left bank)
1843
1842
1841
1840
1839
1838
1837
1836
1835
1834
1833
0
20
40
60
80
100
120
140
160
180

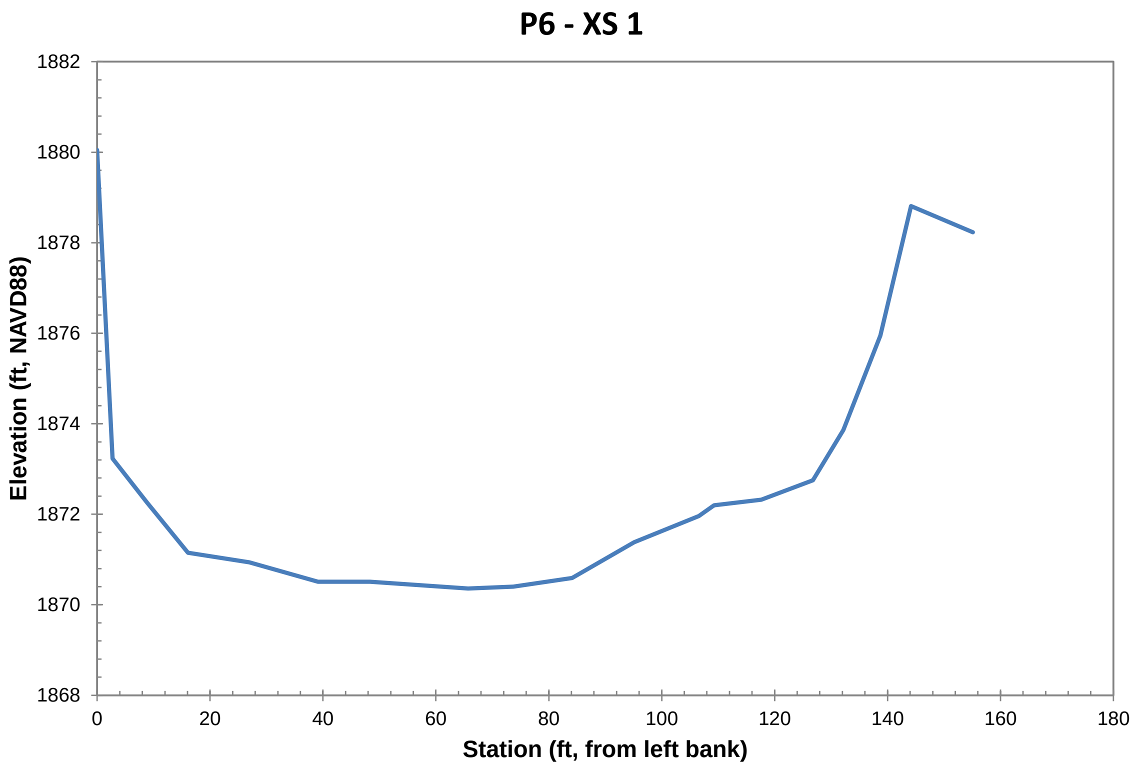
P7 - XS 11

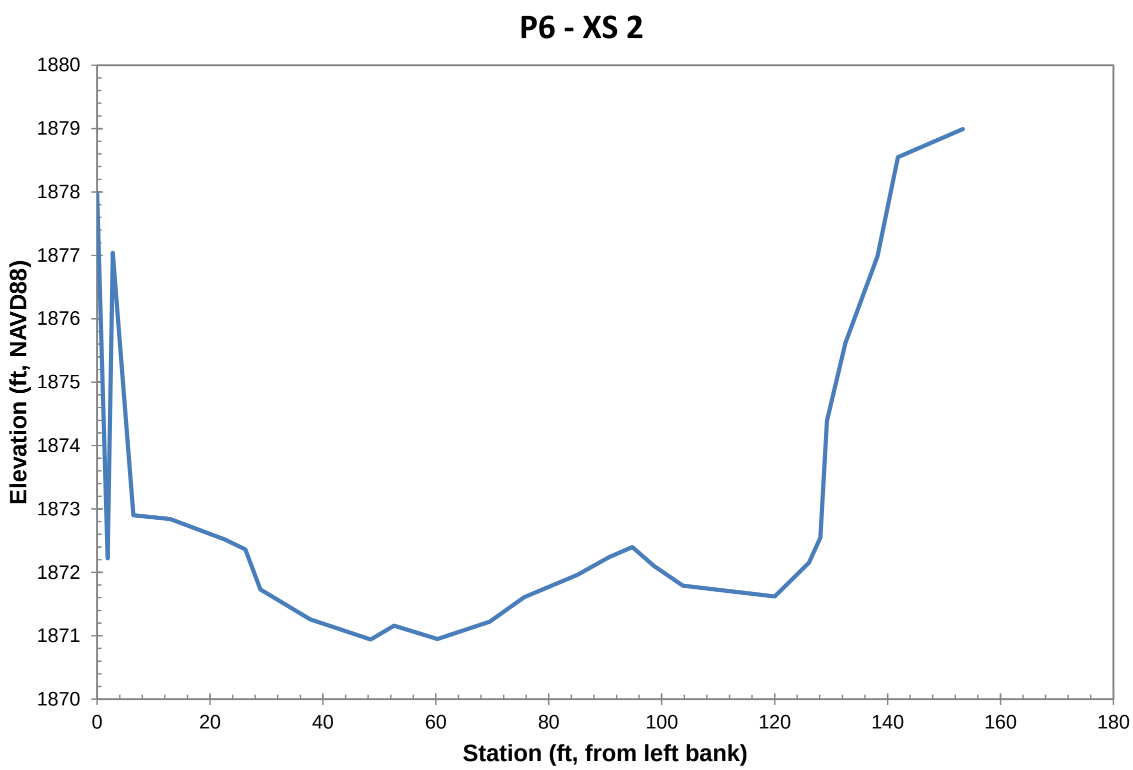


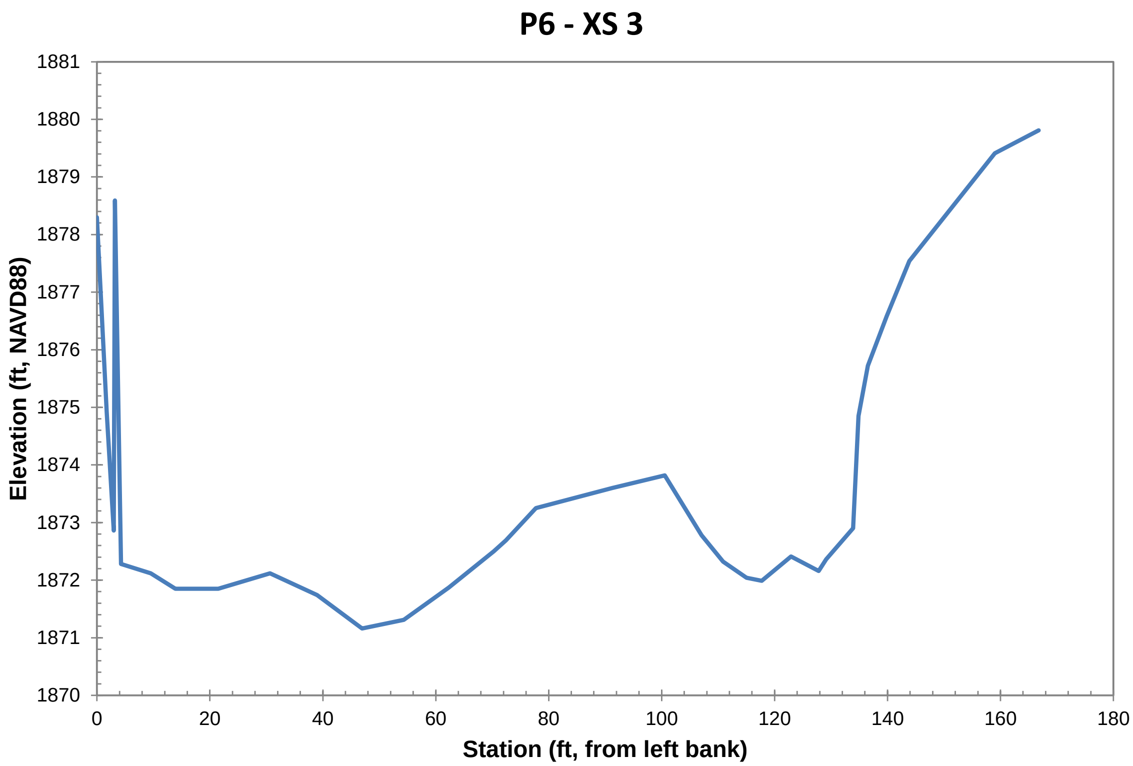
P7 - XS 12

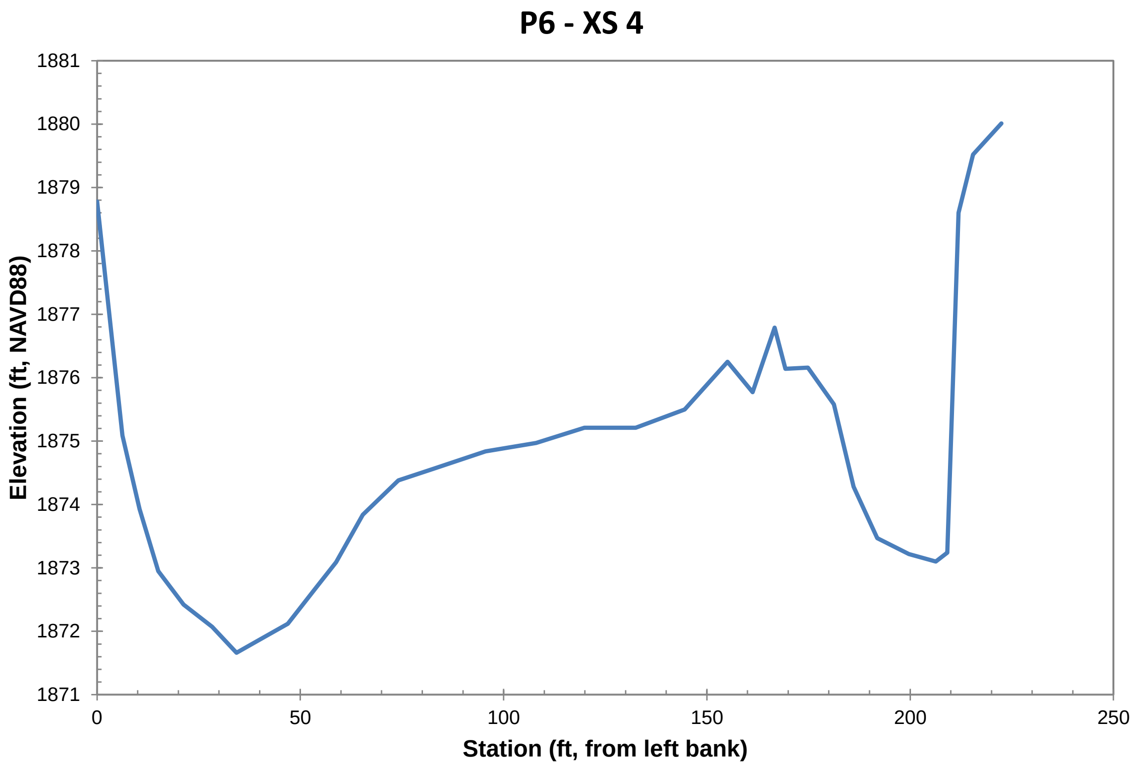



P7 - XS 13
Elevation (ft, NAVD88)
1846
1844
1842
1840
1838
1836
1834
1832
0
50
100
150
200
250
Station (ft, from left bank)

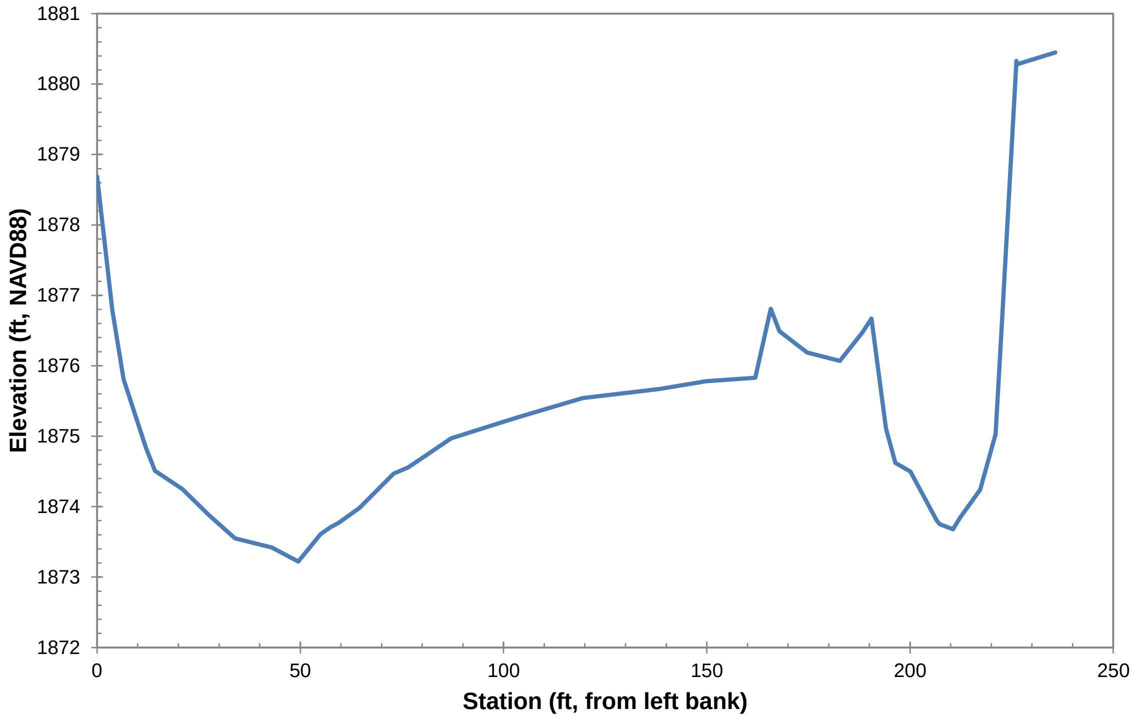

P6 - XS 1
Elevation (ft, NAVD88)
1882
1880
1878
1876
1874
1872
1870
1868
0
20
40
60
80
100
120
140
160
180
Station (ft, from left bank)


P6 - XS 2
Elevation (ft, NAVD88)
1880
1879
1878
1877
1876
1875
1874
1873
1872
1871
1870
0
20
40
60
80
100
120
140
160
180
Station (ft, from left bank)


P6 - XS 3
Elevation (ft, NAVD88)
1881
1880
1879
1878
1877
1876
1875
1874
1873
1872
1871
1870
0
20
40
60
80
100
120
140
160
180
Station (ft, from left bank)

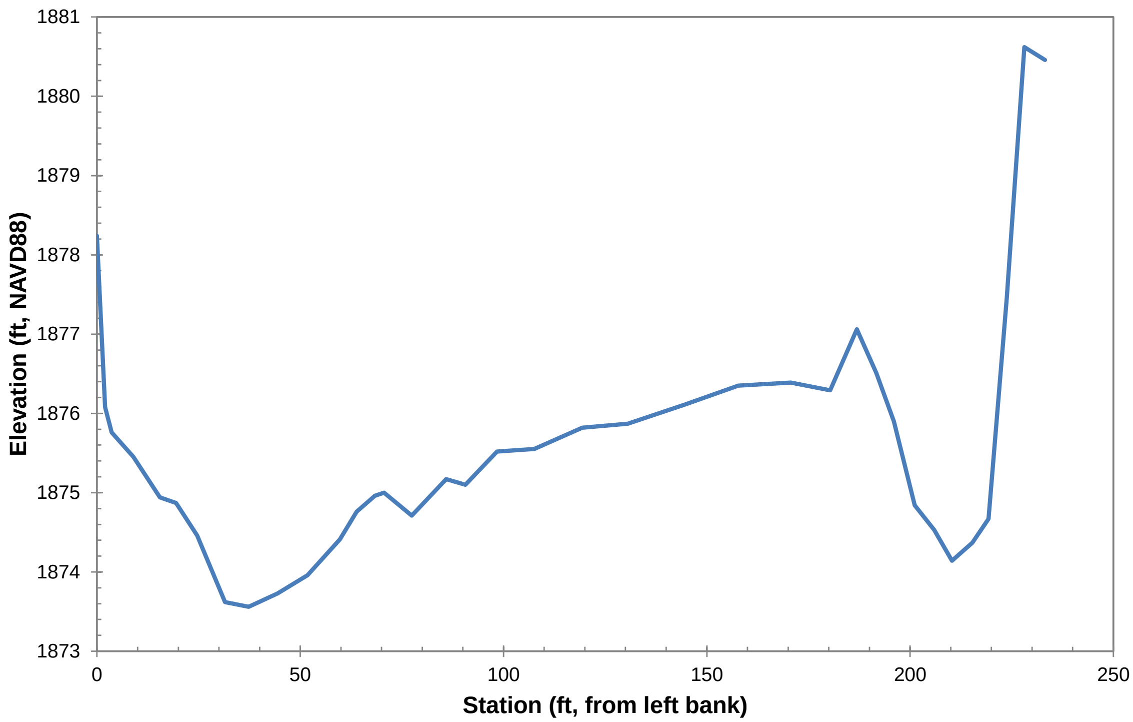

P6 - XS 4
Elevation (ft, NAVD88)
1881
1880
1879
1878
1877
1876
1875
1874
1873
1872
1871
0
50
100
150
200
250
Station (ft, from left bank)

P6 - XS 5

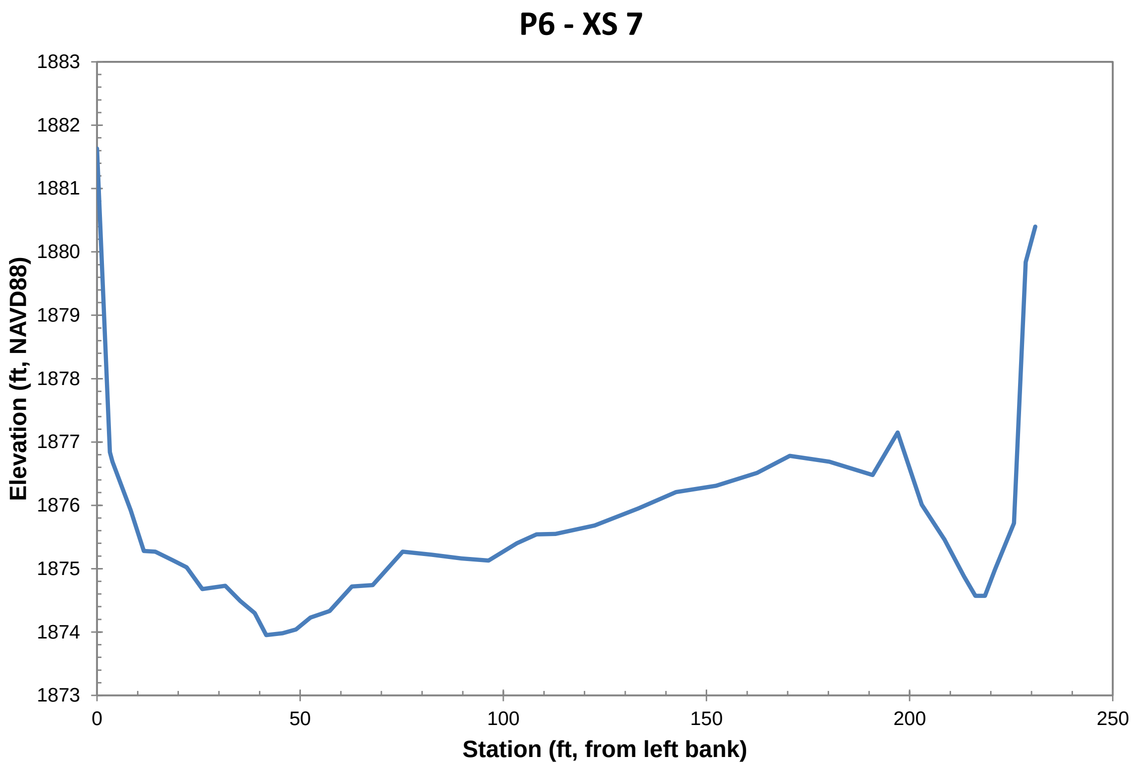


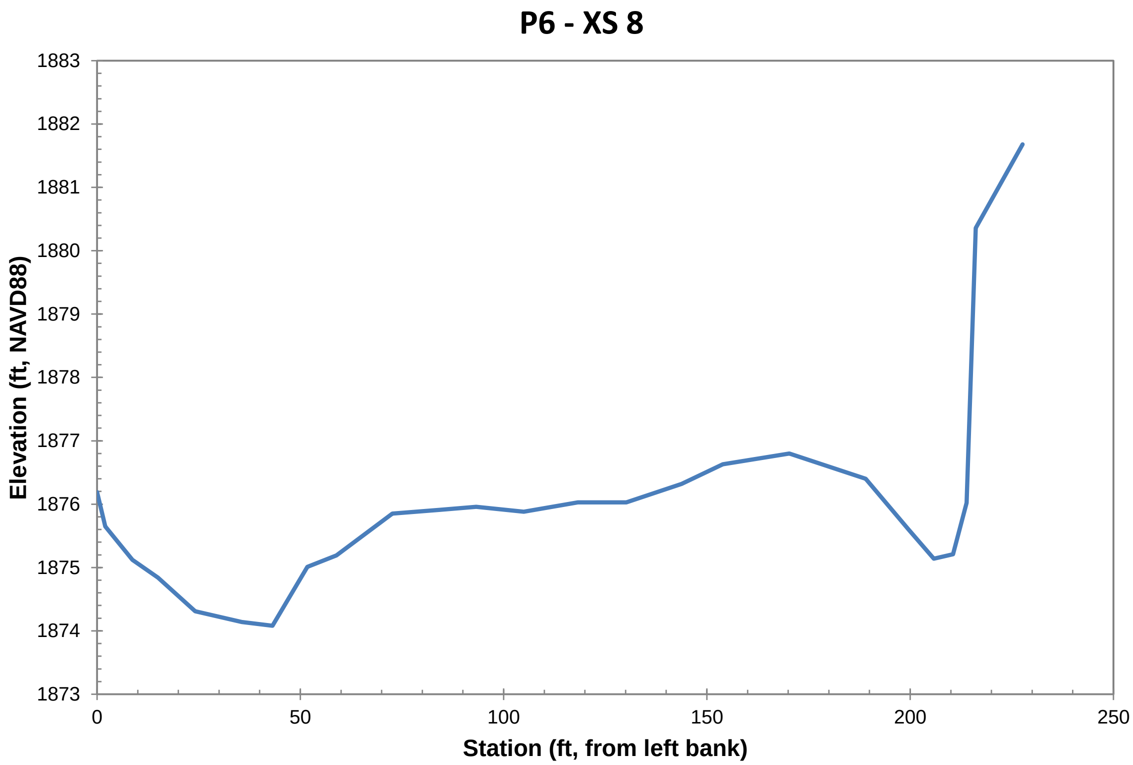
Elevation (ft, NAVD88)
1881
1880
1879
1878
1877
1876
1875
1874
1873
1872
0
50
100
150
200
250
Station (ft, from left bank)

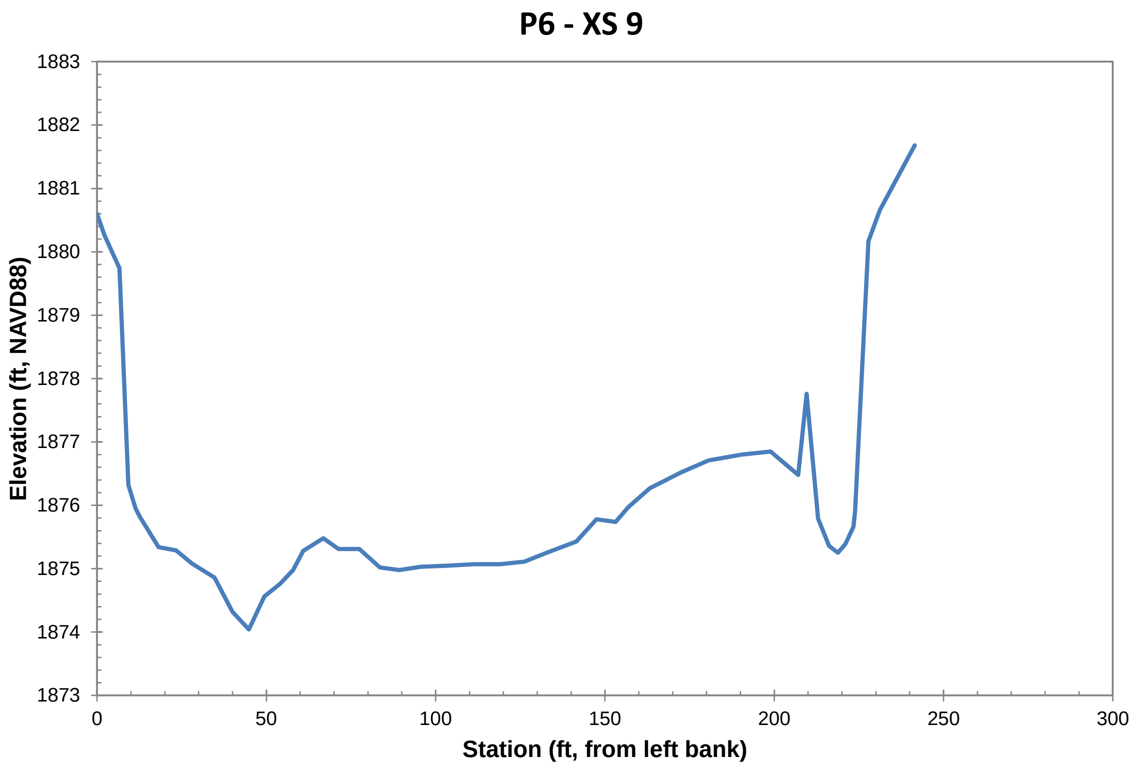
P6 - XS 6

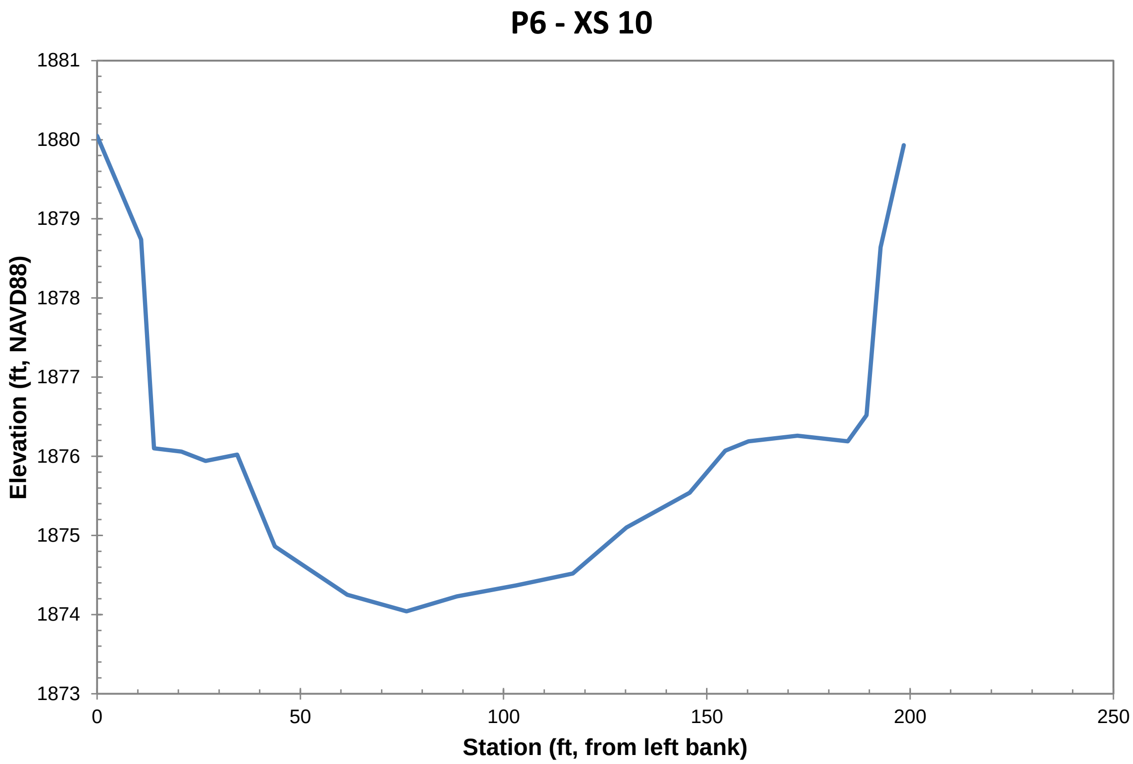


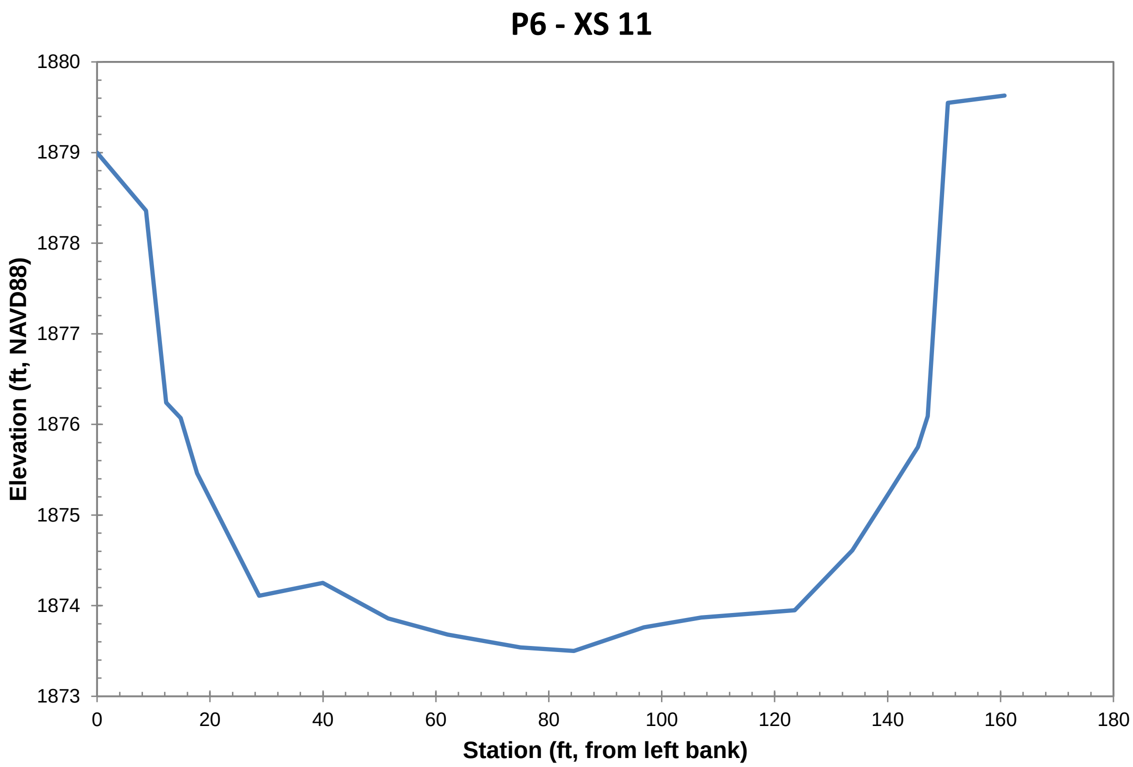
Elevation (ft, NAVD88)
1881
1880
1879
1878
1877
1876
1875
1874
1873
0
50
100
150
200
250
Station (ft, from left bank)

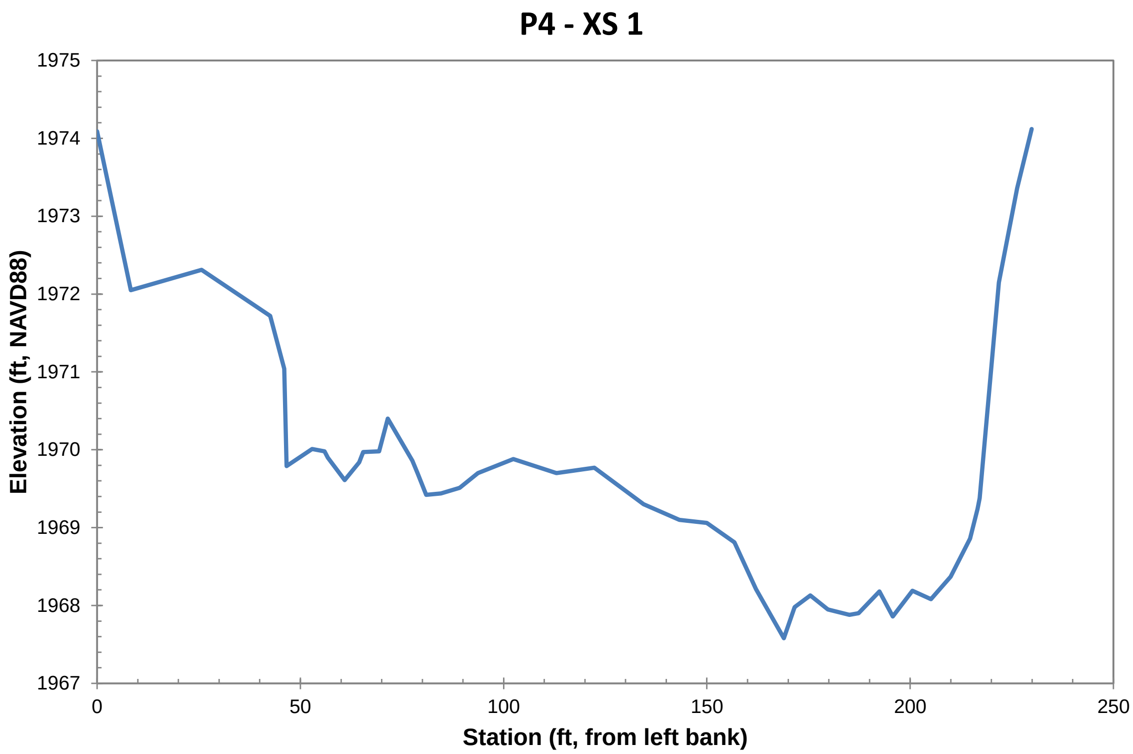

P6 - XS 7
1883
1882
1881
1880
1879
1878
1877
1876
1875
1874
1873
Elevation (ft, NAVD88)
0
50
100
150
200
250
Station (ft, from left bank)

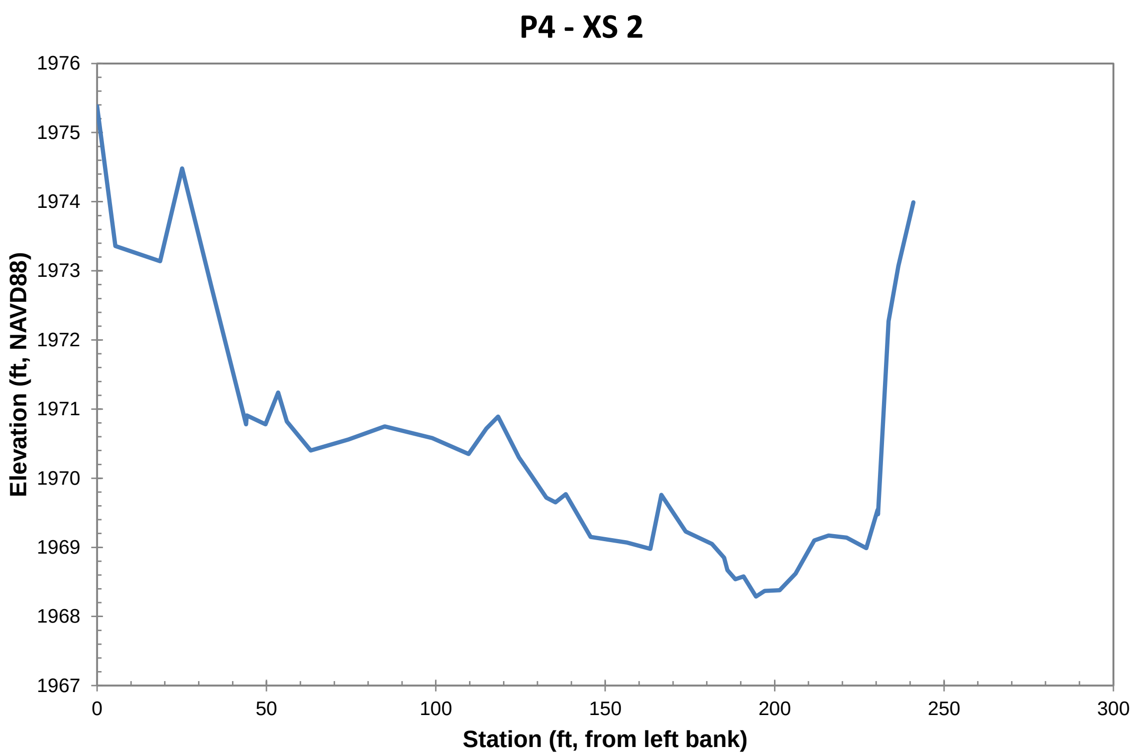

P6 - XS 8
1883
1882
1881
1880
1879
1878
1877
1876
1875
1874
1873
Elevation (ft, NAVD88)
0
50
100
150
200
250
Station (ft, from left bank)


P6 - XS 9
Elevation (ft, NAVD88)
Station (ft, from left bank)
1883
1882
1881
1880
1879
1878
1877
1876
1875
1874
1873
0
50
100
150
200
250
300

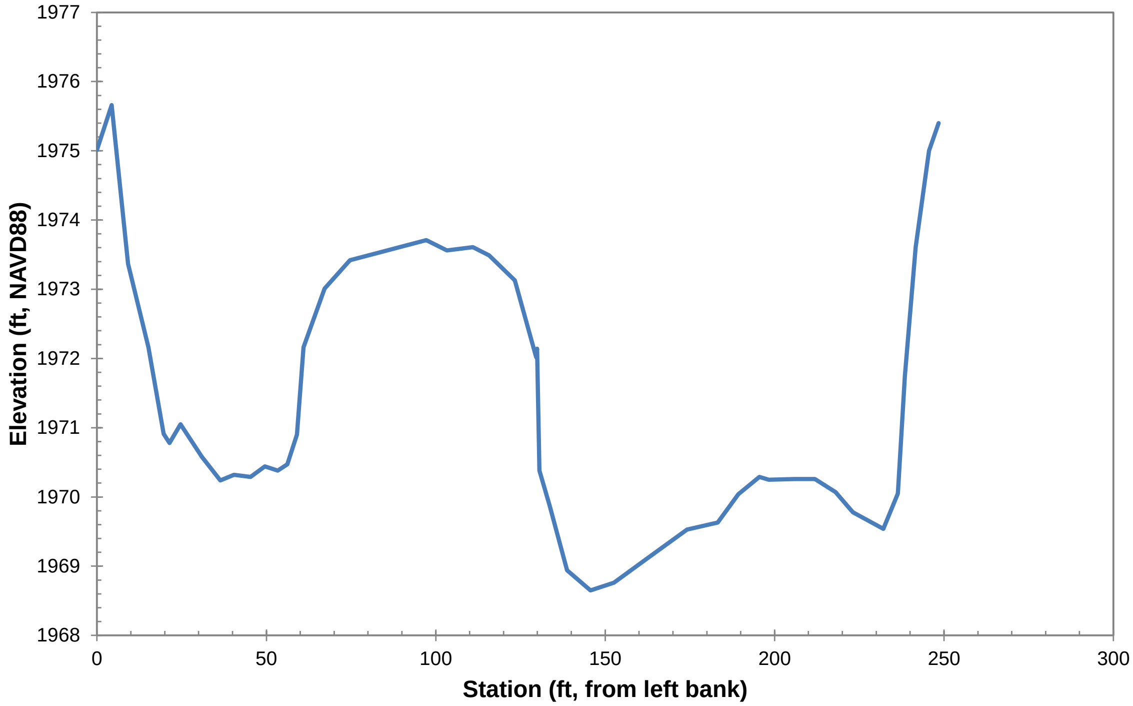

P6 - XS 10
Elevation (ft, NAVD88)
Station (ft, from left bank)
1881
1880
1879
1878
1877
1876
1875
1874
1873
0
50
100
150
200
250


P6 - XS 11
Elevation (ft, NAVD88)
1880
1879
1878
1877
1876
1875
1874
1873
0
20
40
60
80
100
120
140
160
180
Station (ft, from left bank)

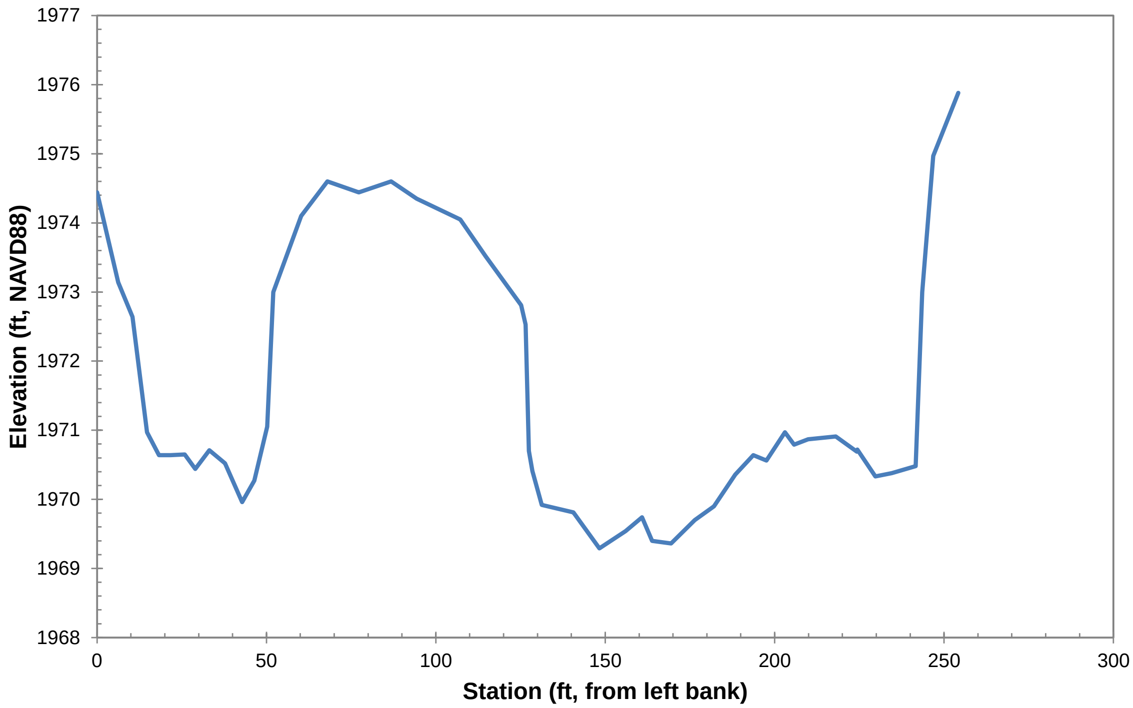

P4 - XS 1
Elevation (ft, NAVD88)
1975
1974
1973
1972
1971
1970
1969
1968
1967
0
50
100
150
200
250
Station (ft, from left bank)


P4 - XS 2
Elevation (ft, NAVD88)
1976
1975
1974
1973
1972
1971
1970
1969
1968
1967
0
50
100
150
200
250
300
Station (ft, from left bank)

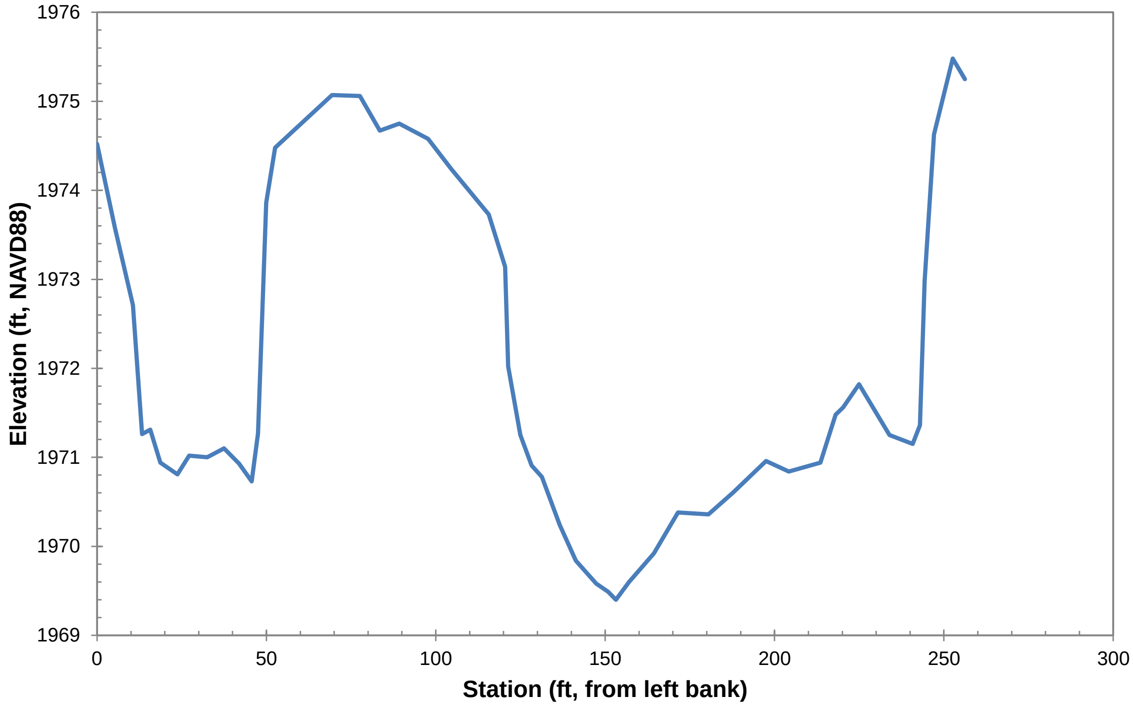
P4 - XS 3


1977
1976
1975
1974
1973
1972
1971
1970
1969
1968
Elevation (ft, NAVD88)
0
50
100
150
200
250
300
Station (ft, from left bank)

P4 - XS 4

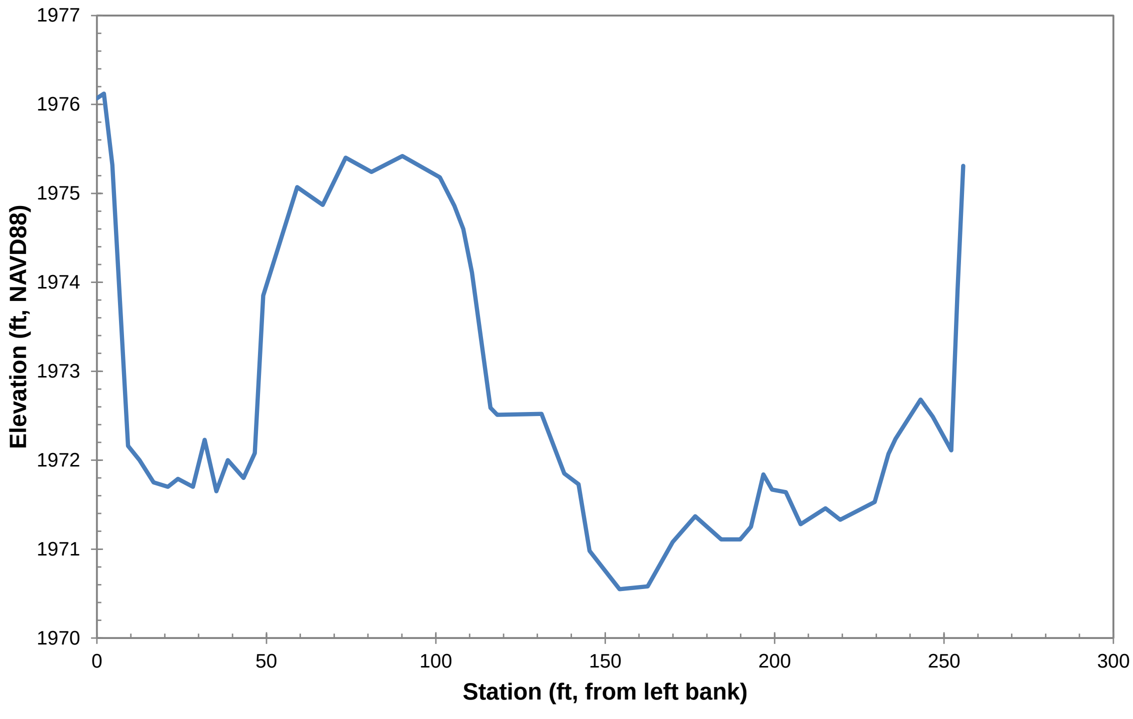

1977
1976
1975
1974
1973
1972
1971
1970
1969
1968
Elevation (ft, NAVD88)
0
50
100
150
200
250
300
Station (ft, from left bank)

P4 - XS 5

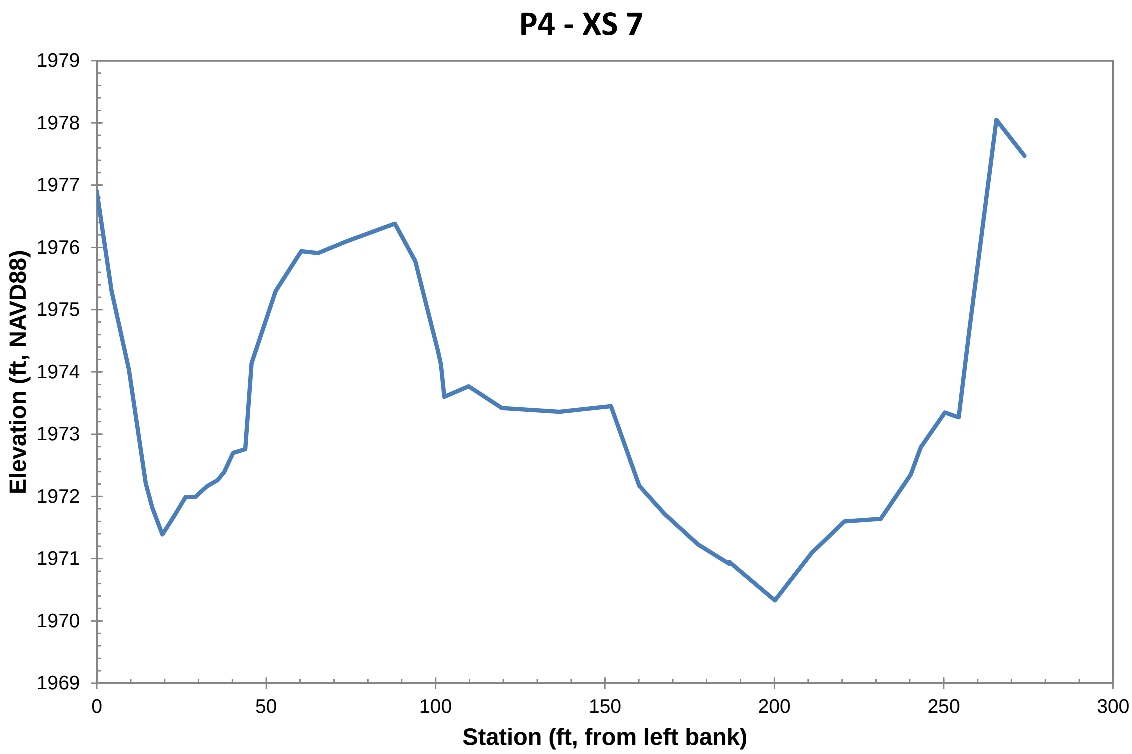


1976
1975
1974
1973
1972
1971
1970
1969
0
50
100
150
200
250
300
Elevation (ft, NAVD88)
Station (ft, from left bank)

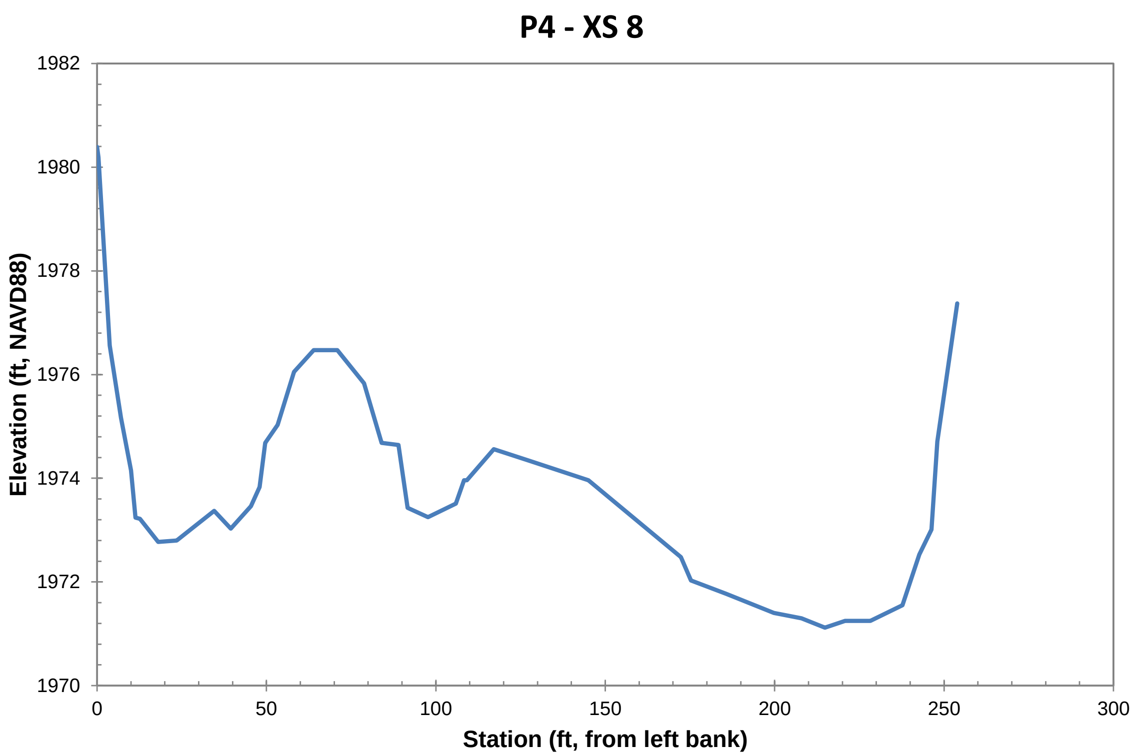
P4 - XS 6



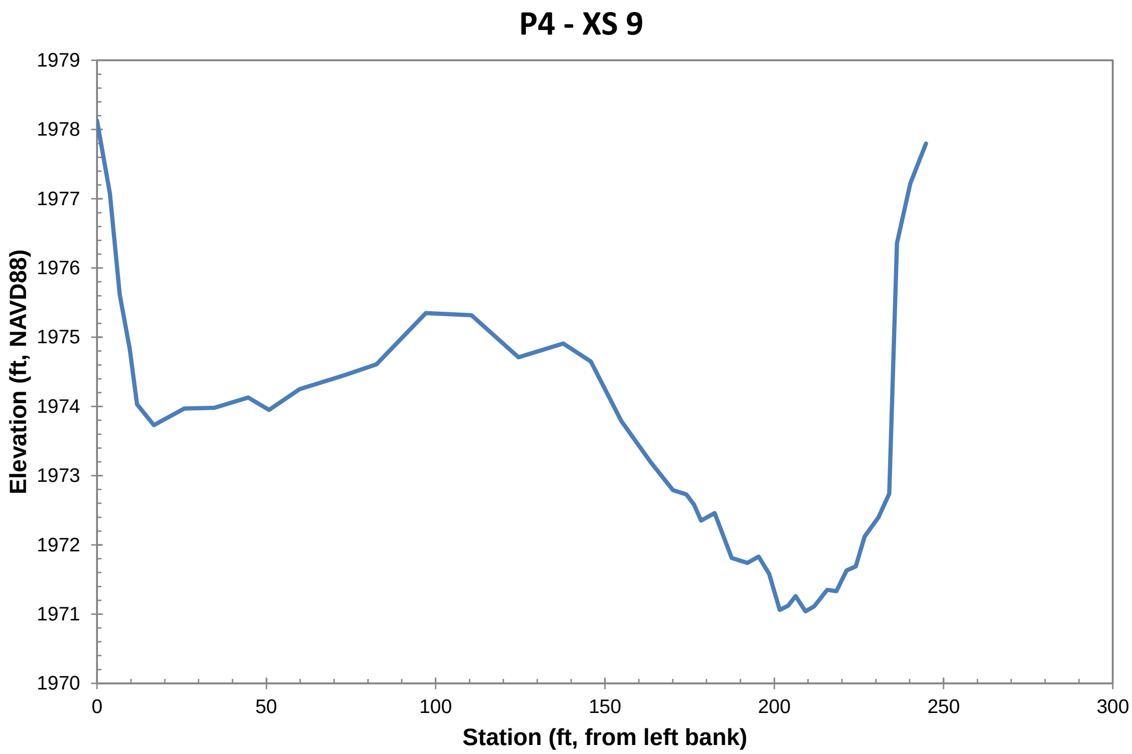
1977
1976
1975
1974
1973
1972
1971
1970
0
50
100
150
200
250
300
Elevation (ft, NAVD88)
Station (ft, from left bank)

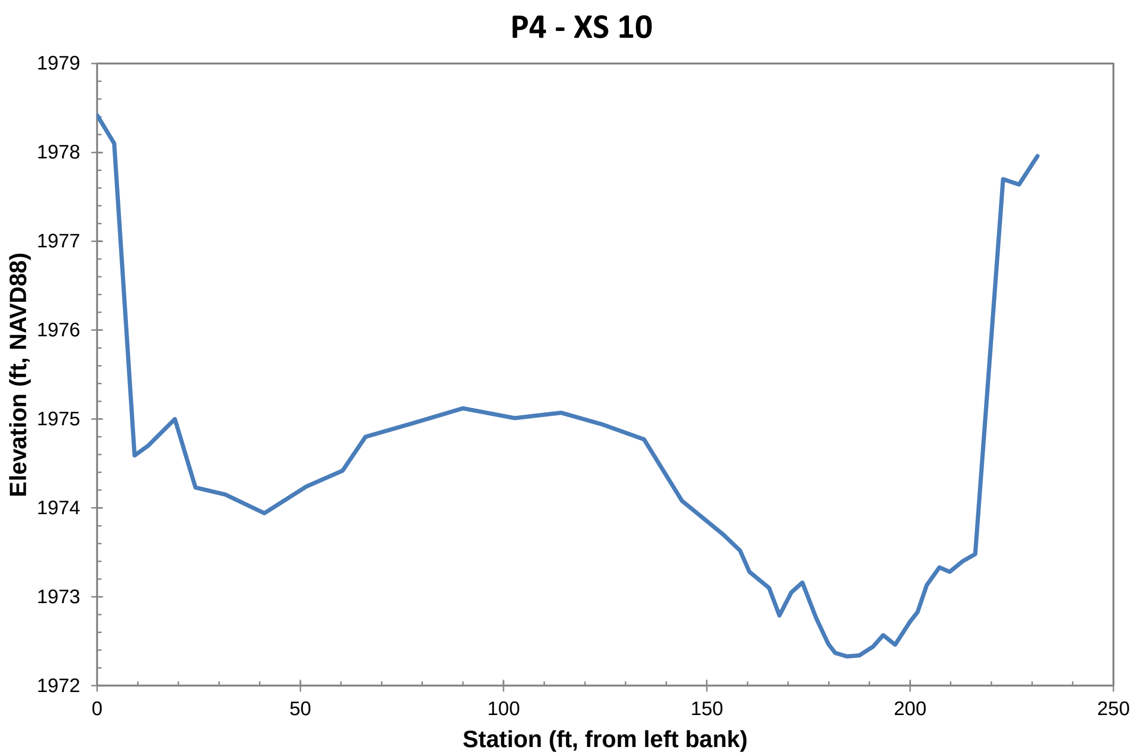


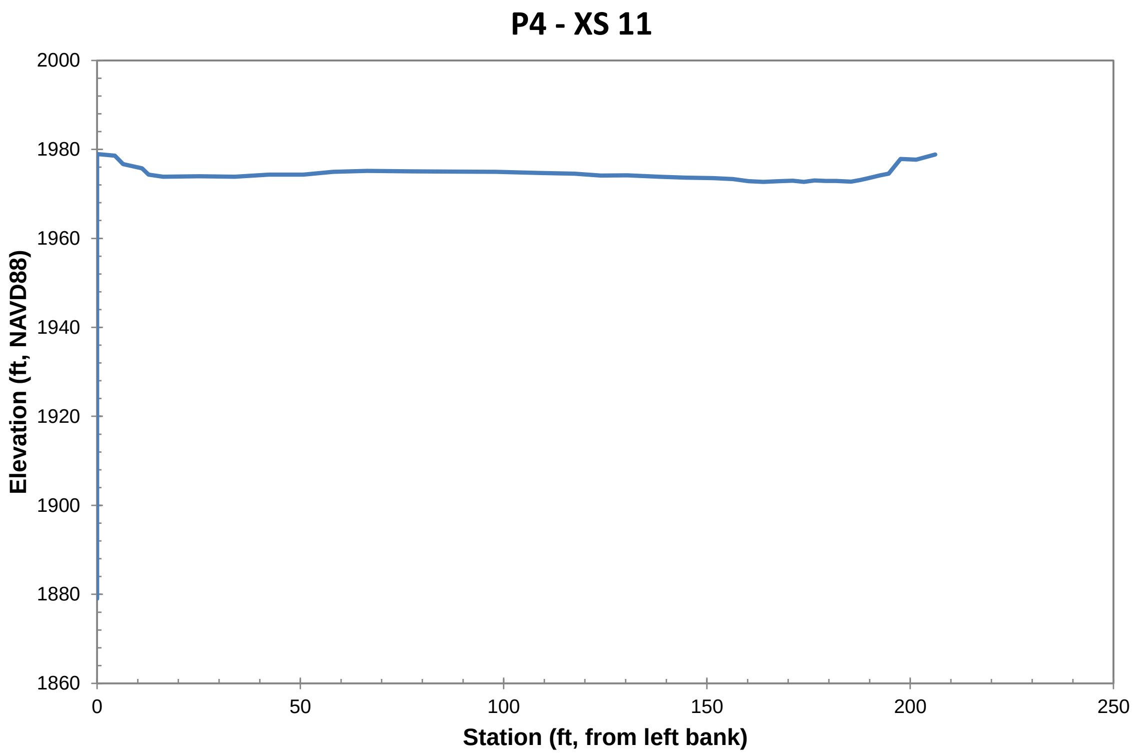
P4 - XS 7
1979
1978
1977
1976
1975
1974
1973
1972
1971
1970
1969
0
50
100
150
200
250
300
Elevation (ft, NAVD88)
Station (ft, from left bank)

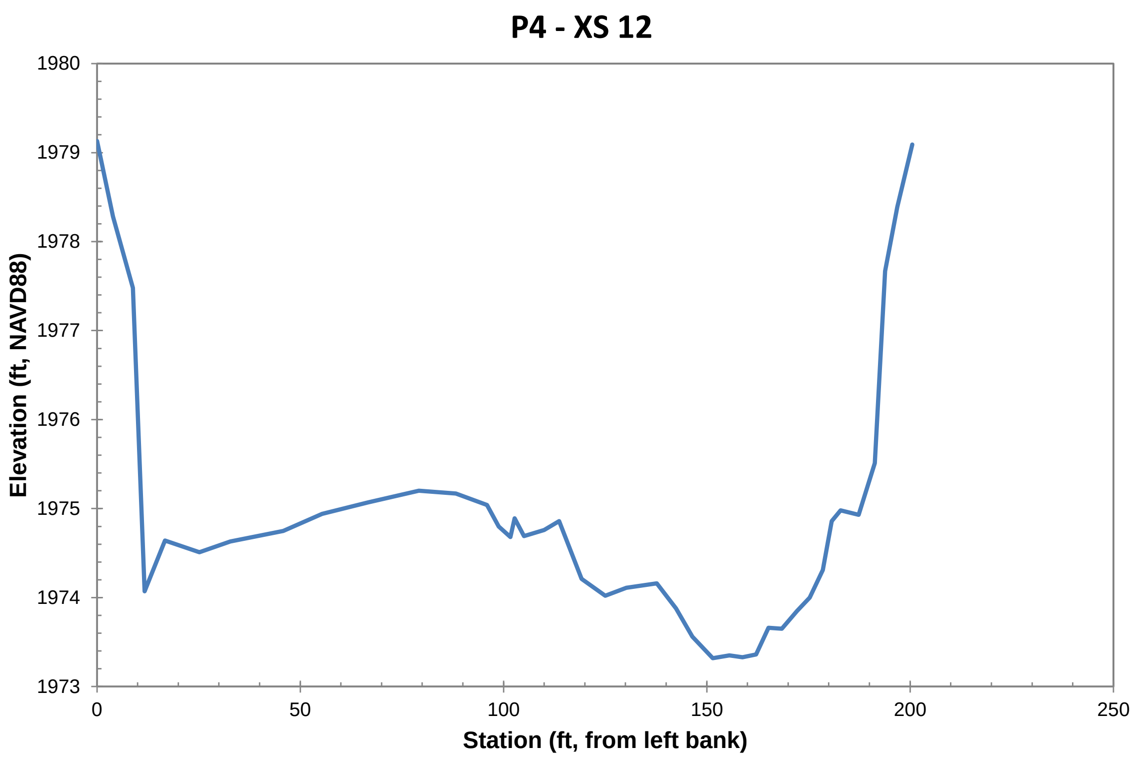


P4 - XS 8
1982
1980
1978
1976
1974
1972
1970
0
50
100
150
200
250
300
Elevation (ft, NAVD88)
Station (ft, from left bank)

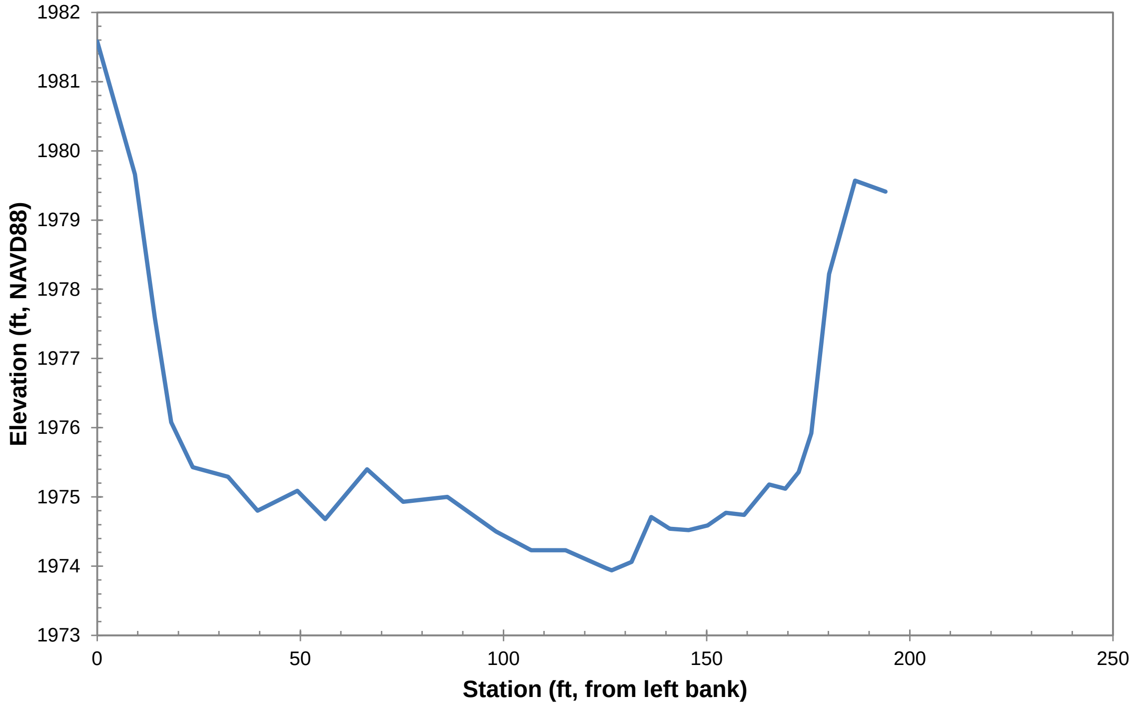

P4 - XS 9
Elevation (ft, NAVD88)
1979
1978
1977
1976
1975
1974
1973
1972
1971
1970
0
50
100
150
200
250
300
Station (ft, from left bank)


P4 - XS 10
Elevation (ft, NAVD88)
1979
1978
1977
1976
1975
1974
1973
1972
0
50
100
150
200
250
Station (ft, from left bank)

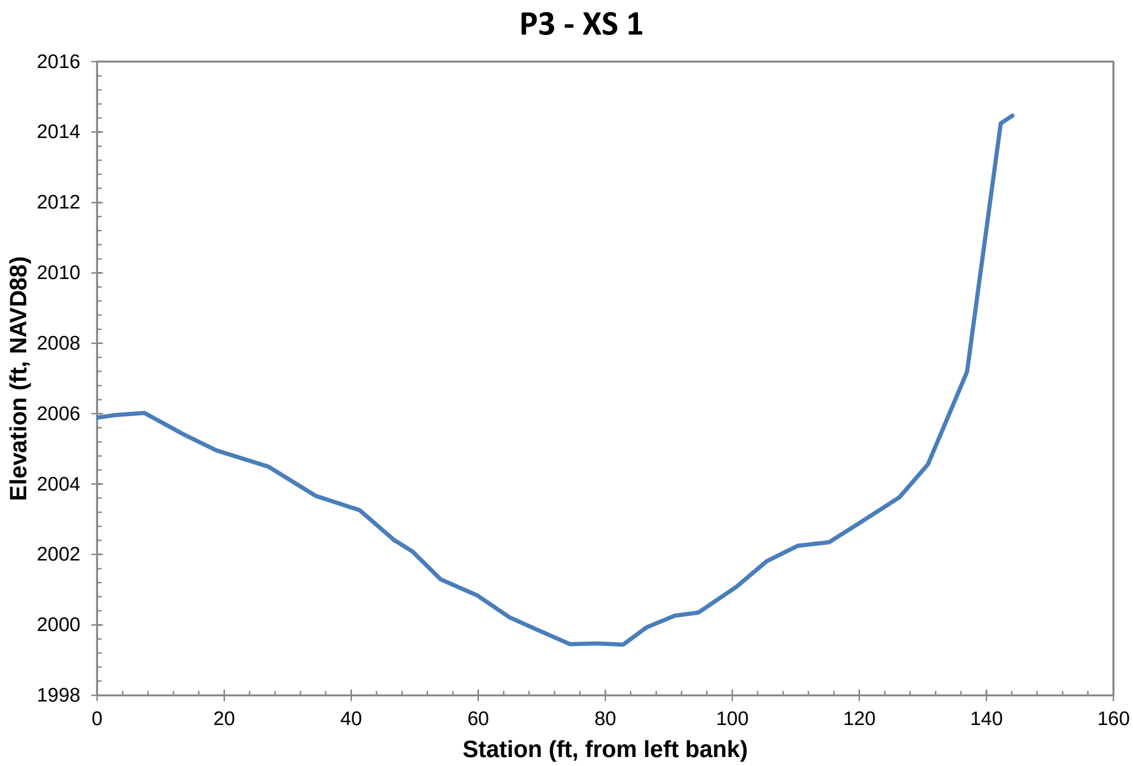

P4 - XS 11
Elevation (ft, NAVD88)
Station (ft, from left bank)
2000
1980
1960
1940
1920
1900
1880
1860
0
50
100
150
200
250

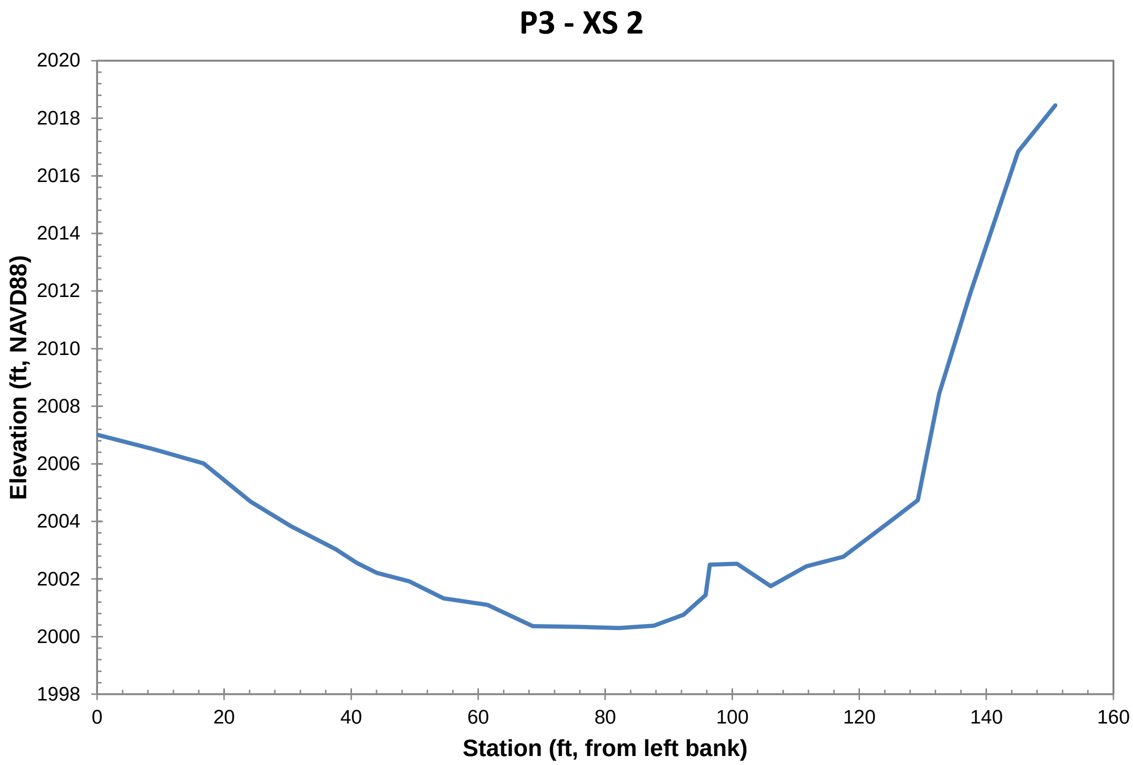

P4 - XS 12
Elevation (ft, NAVD88)
Station (ft, from left bank)
1980
1979
1978
1977
1976
1975
1974
1973
0
50
100
150
200
250

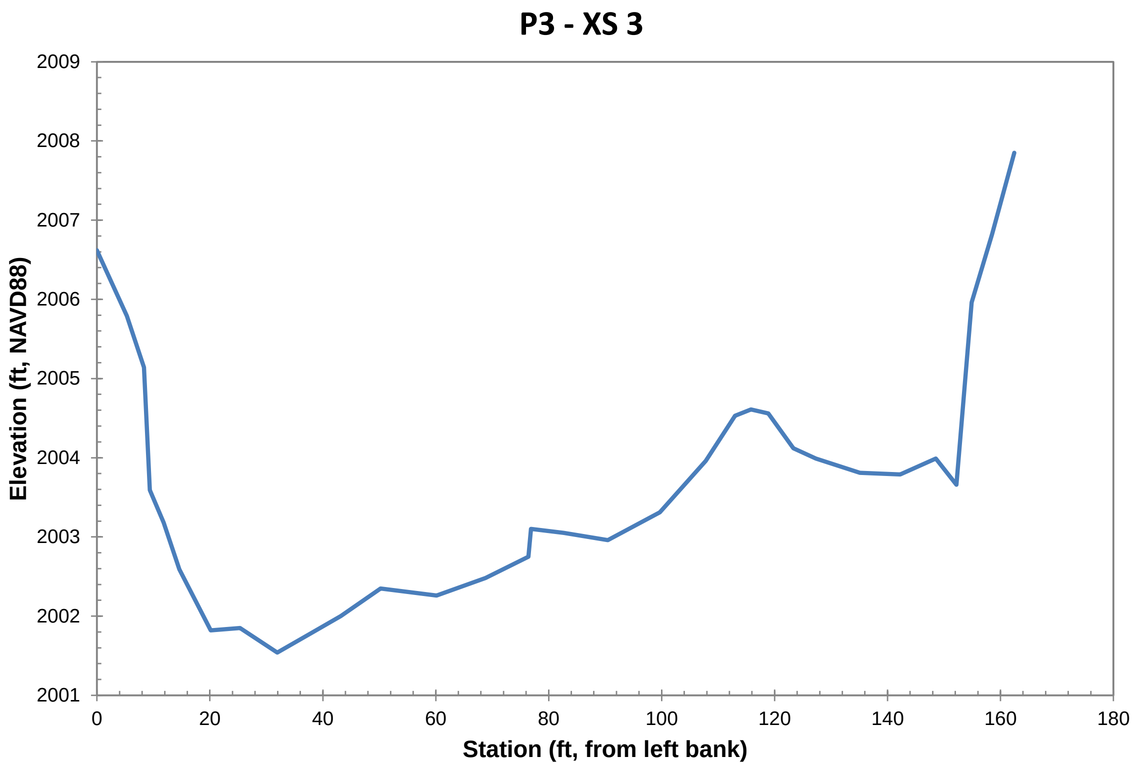
P4 - XS 13



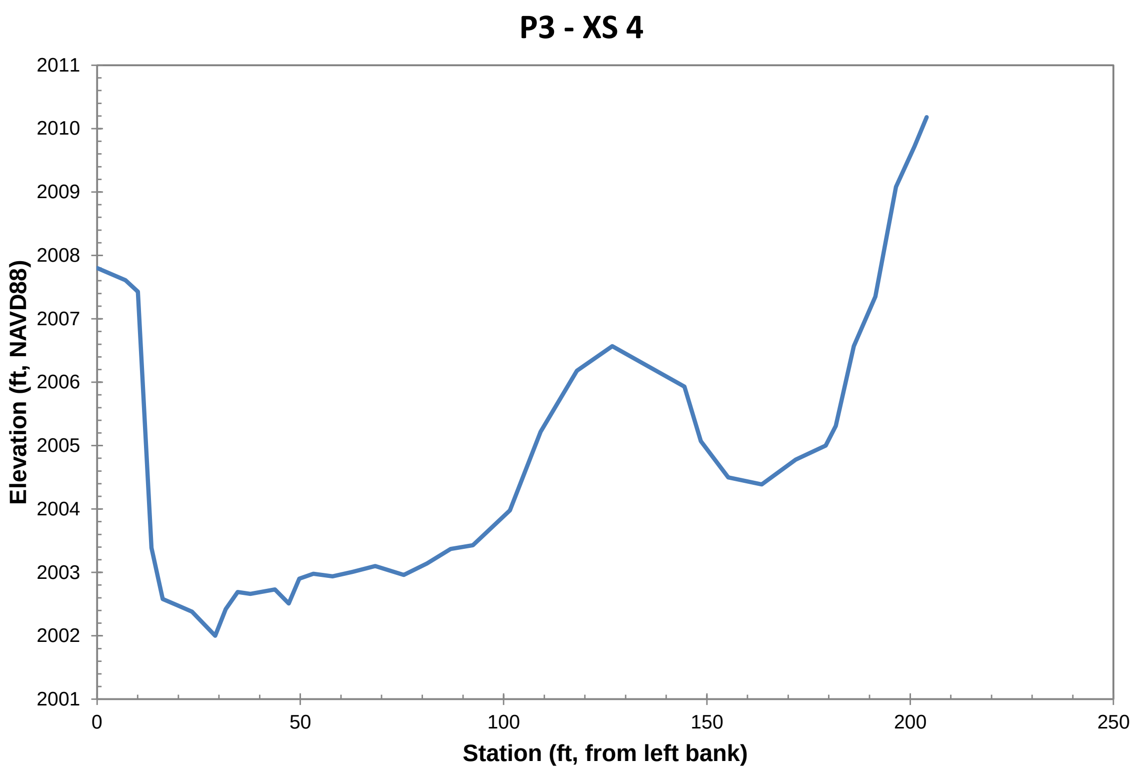
P4 - XS 13
1982
1981
1980
1979
1978
1977
1976
1975
1974
1973
Elevation (ft, NAVD88)
0
50
100
150
200
250
Station (ft, from left bank)


P3 - XS 1
Elevation (ft, NAVD88)
2016
2014
2012
2010
2008
2006
2004
2002
2000
1998
0
20
40
60
80
100
120
140
160
Station (ft, from left bank)


P3 - XS 2
Elevation (ft, NAVD88)
2020
2018
2016
2014
2012
2010
2008
2006
2004
2002
2000
1998
0
20
40
60
80
100
120
140
160
Station (ft, from left bank)

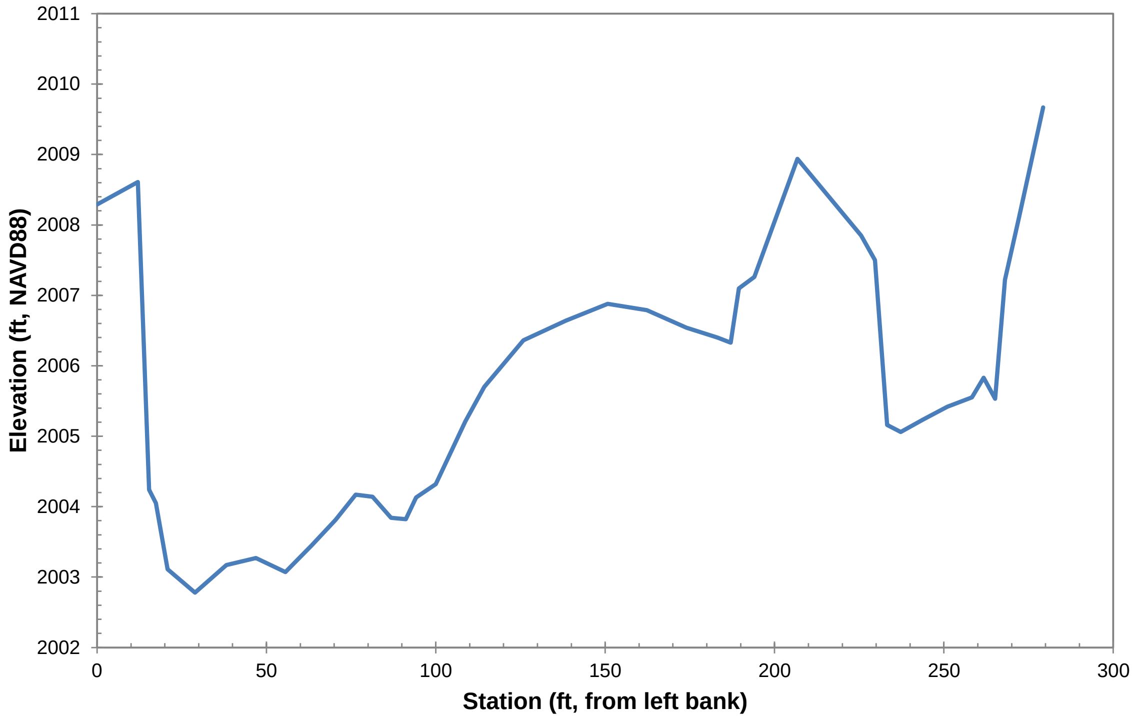


P3 - XS 3
2009
2008
2007
2006
2005
2004
2003
2002
2001
Elevation (ft, NAVD88)
0
20
40
60
80
100
120
140
160
180
Station (ft, from left bank)



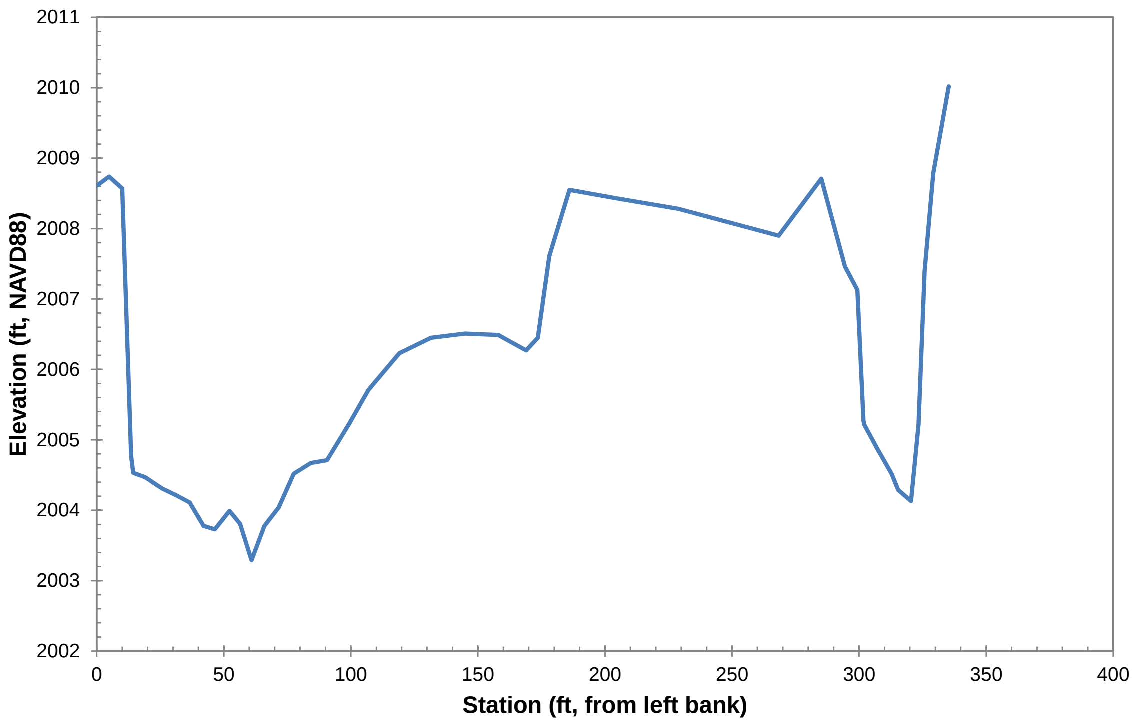
P3 - XS 4
2011
2010
2009
2008
2007
2006
2005
2004
2003
2002
2001
Elevation (ft, NAVD88)
0
50
100
150
200
250
Station (ft, from left bank)

P3 - XS 5



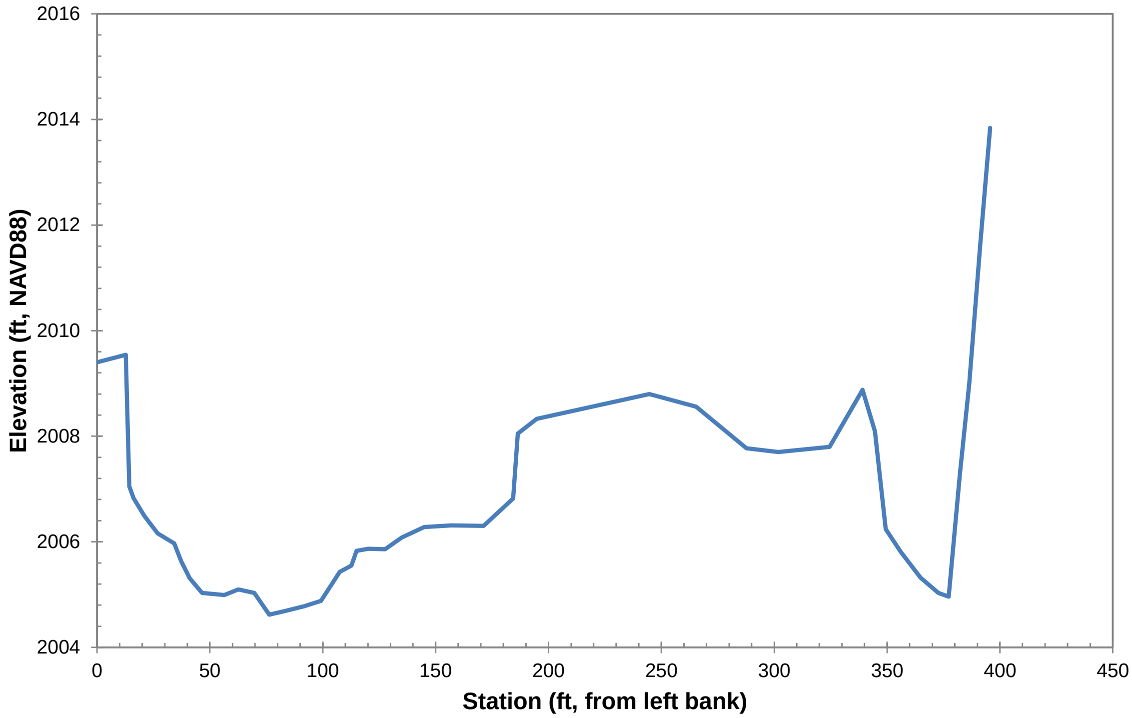
Elevation (ft, NAVD88)
2011
2010
2009
2008
2007
2006
2005
2004
2003
2002
0
50
100
150
200
250
300
Station (ft, from left bank)

P3 - XS 6



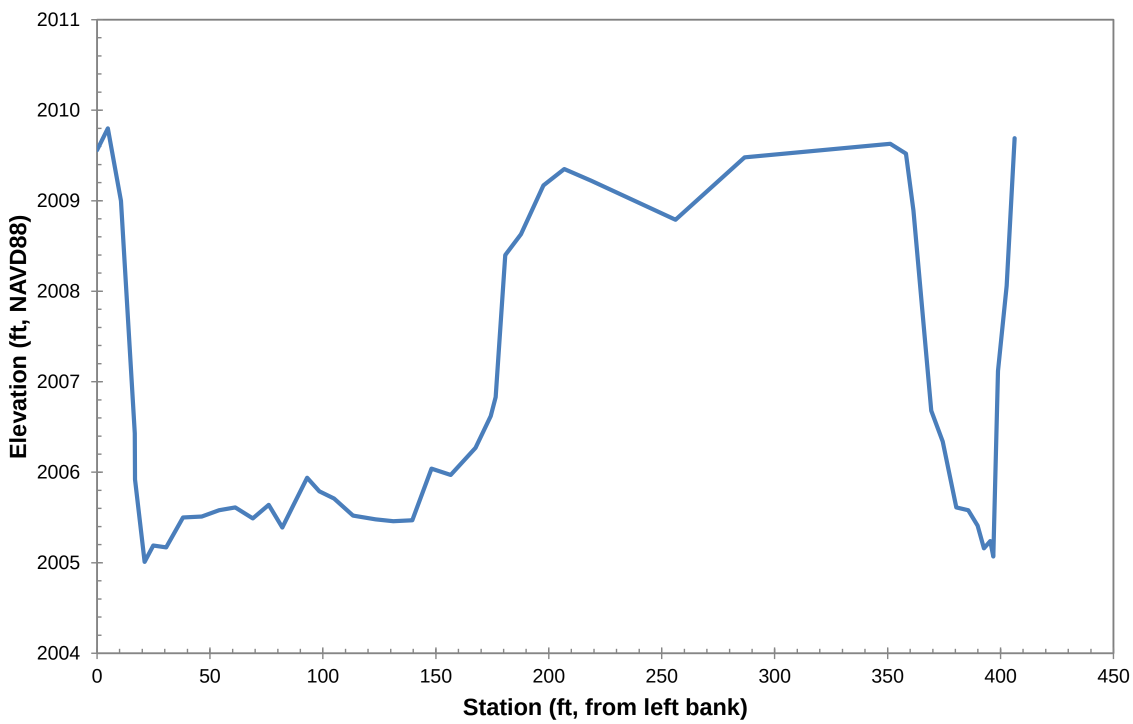
Elevation (ft, NAVD88)
2011
2010
2009
2008
2007
2006
2005
2004
2003
2002
0
50
100
150
200
250
300
350
400
Station (ft, from left bank)

P3 - XS 7

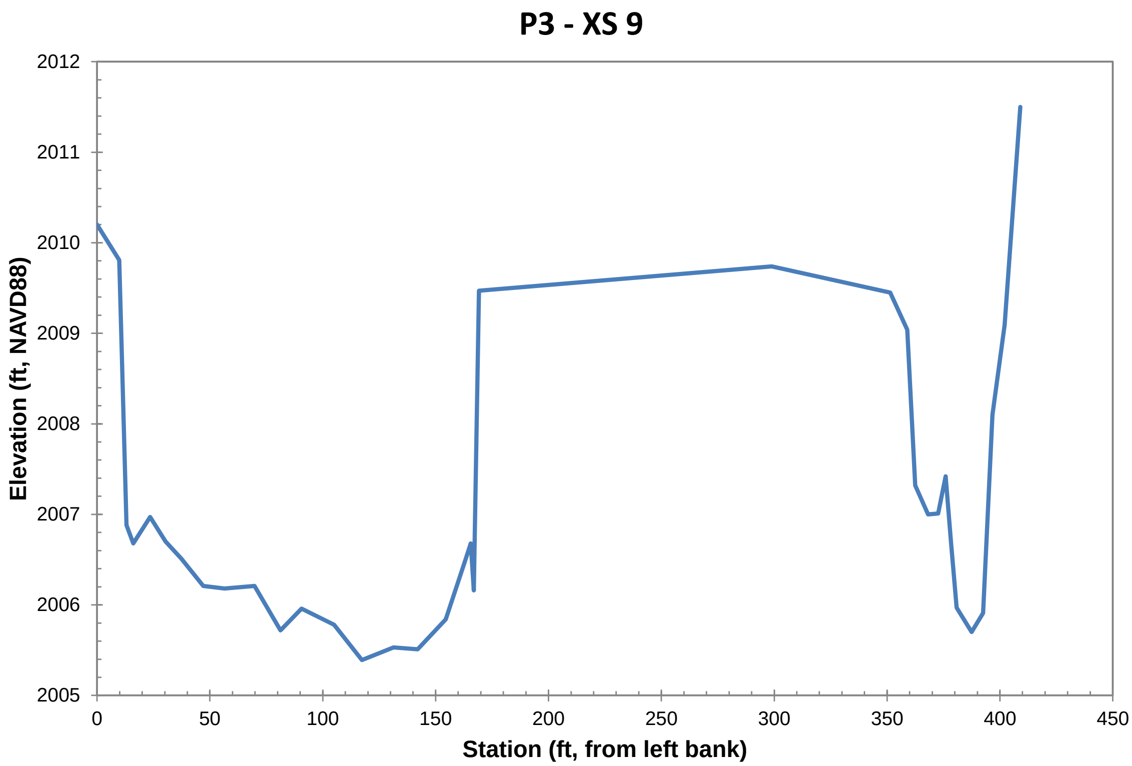


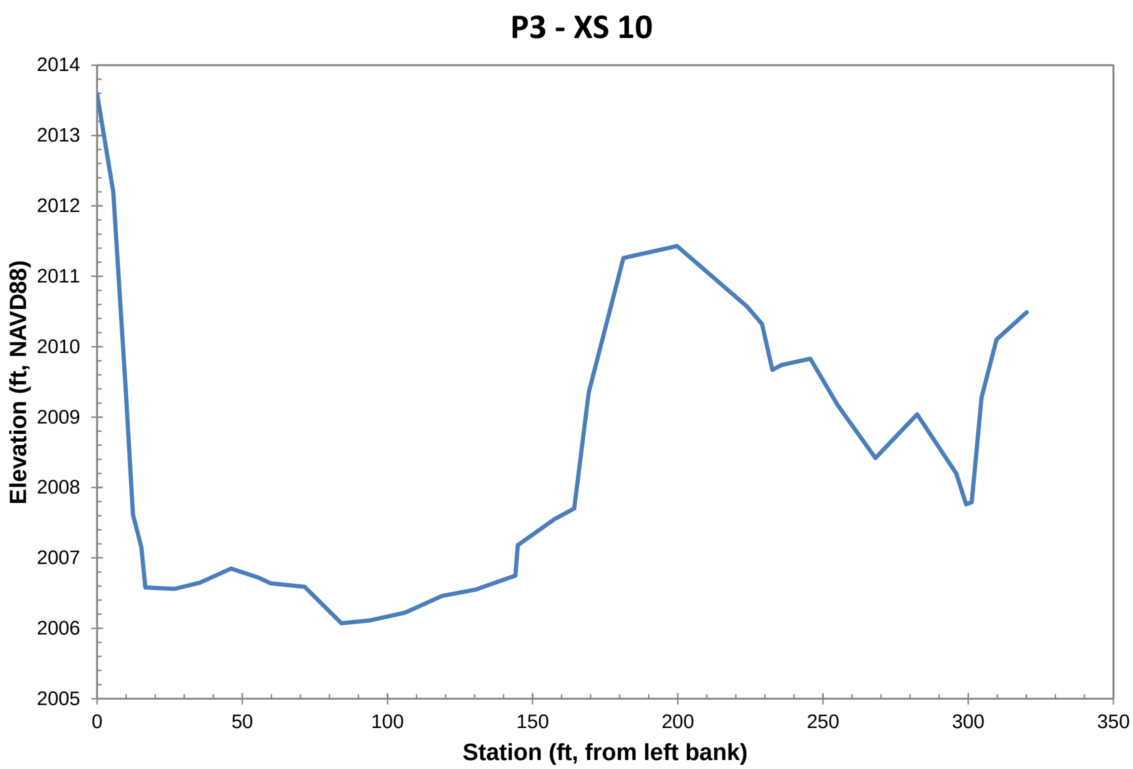
Elevation (ft, NAVD88)
2016
2014
2012
2010
2008
2006
2004
0
50
100
150
200
250
300
350
400
450
Station (ft, from left bank)

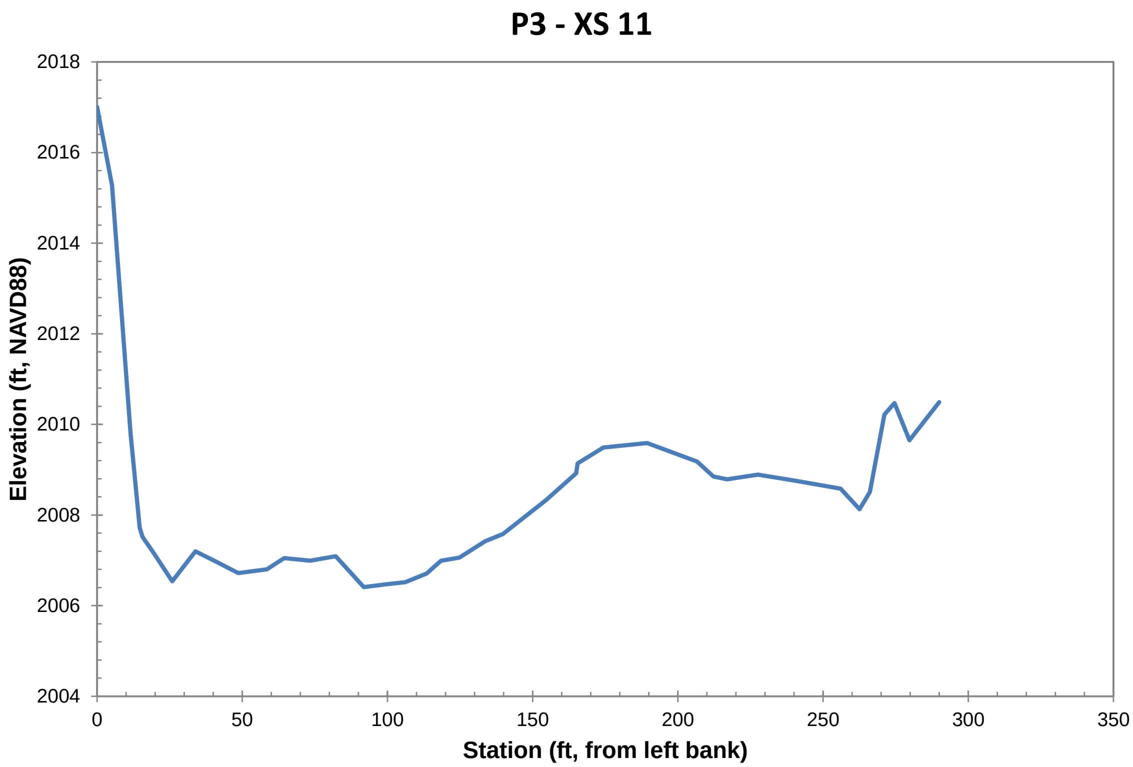
P3 - XS 8

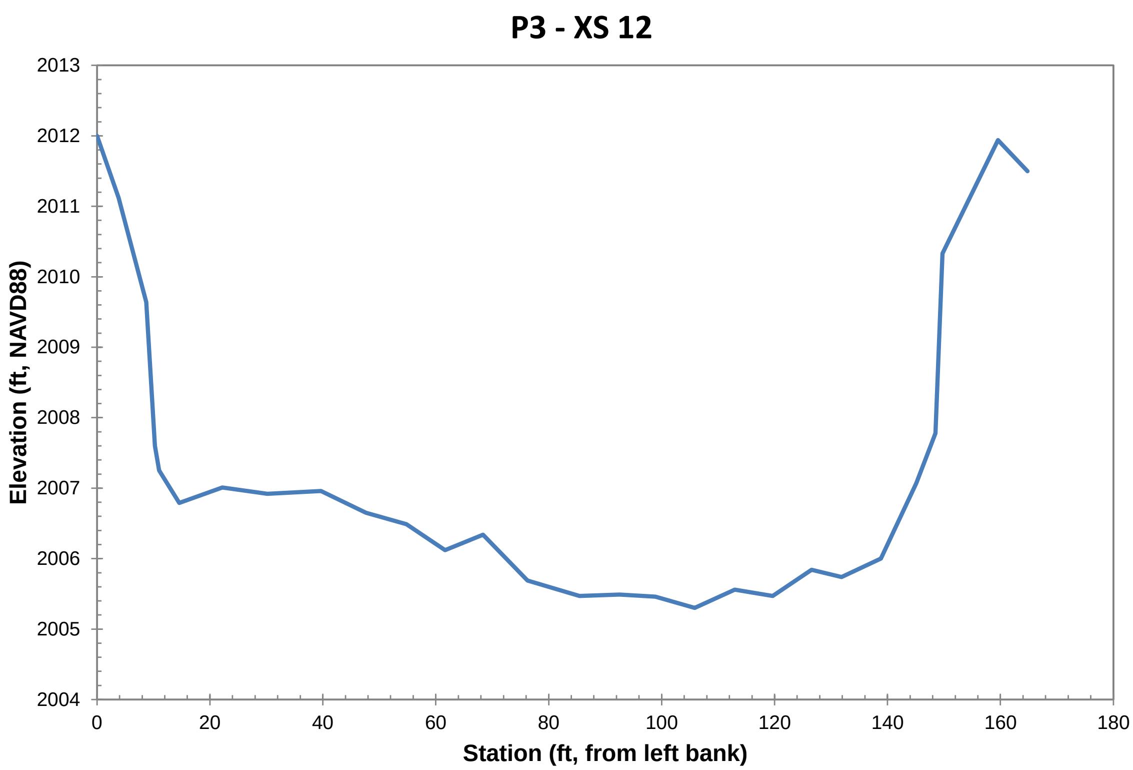


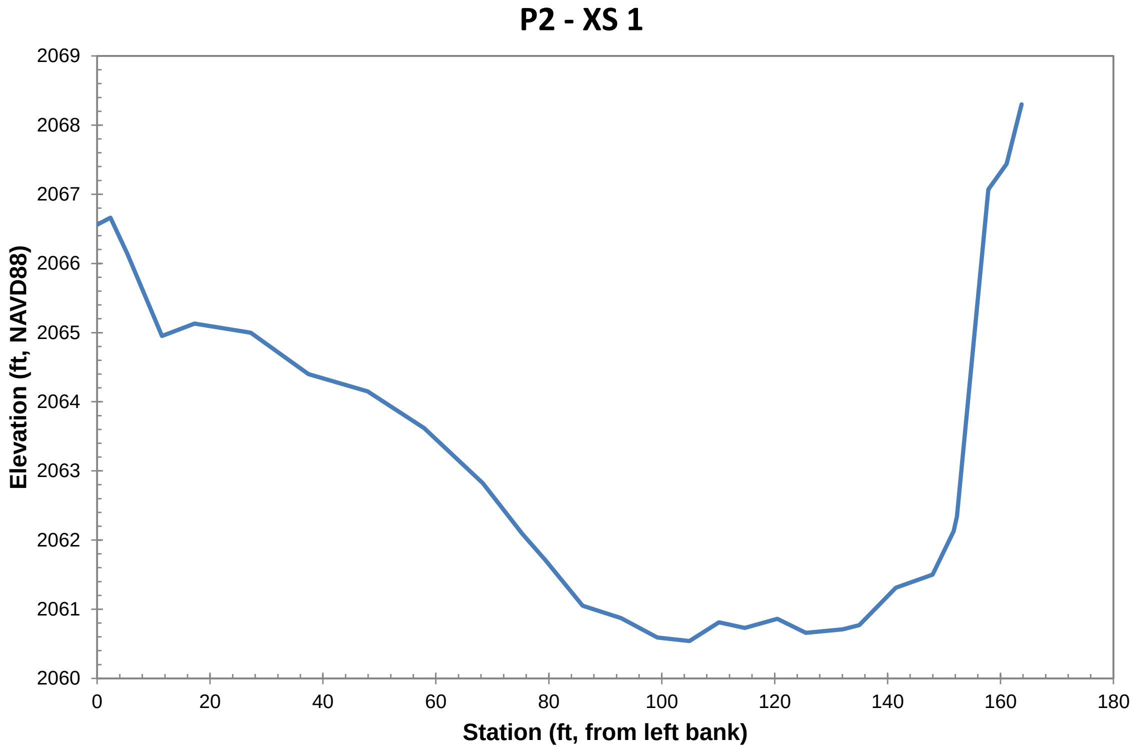
Elevation (ft, NAVD88)
2011
2010
2009
2008
2007
2006
2005
2004
0
50
100
150
200
250
300
350
400
450
Station (ft, from left bank)

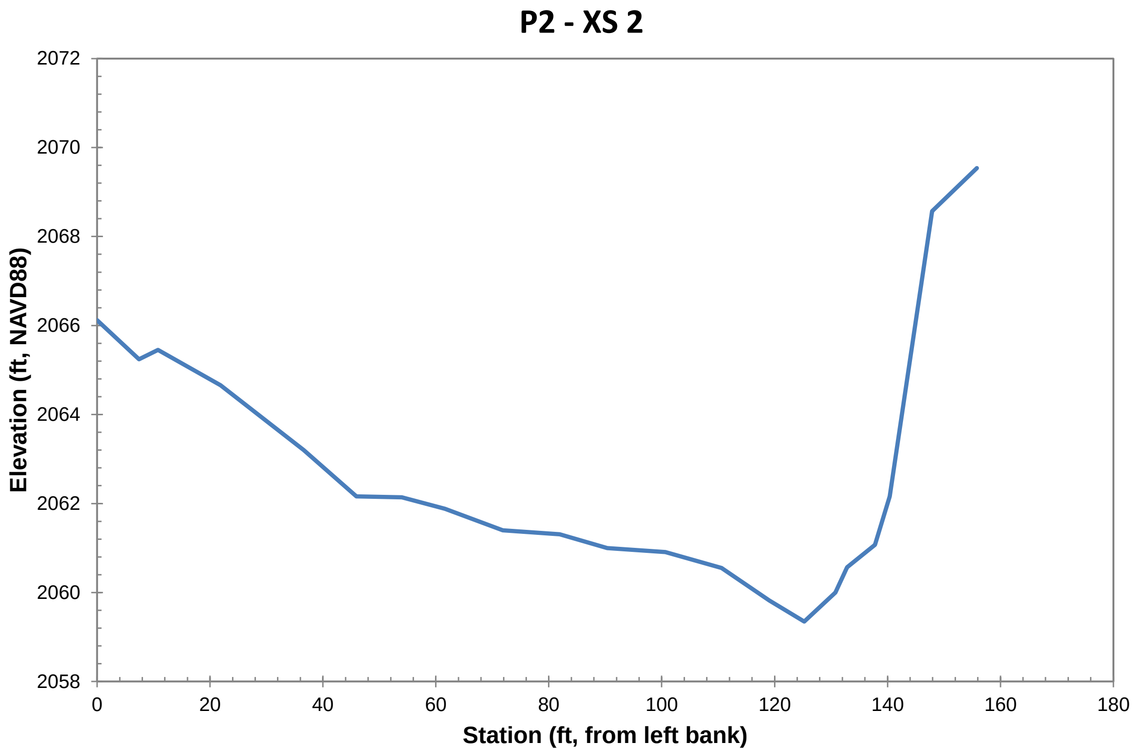

P3 - XS 9
Elevation (ft, NAVD88)
2012
2011
2010
2009
2008
2007
2006
2005
0
50
100
150
200
250
300
350
400
450
Station (ft, from left bank)

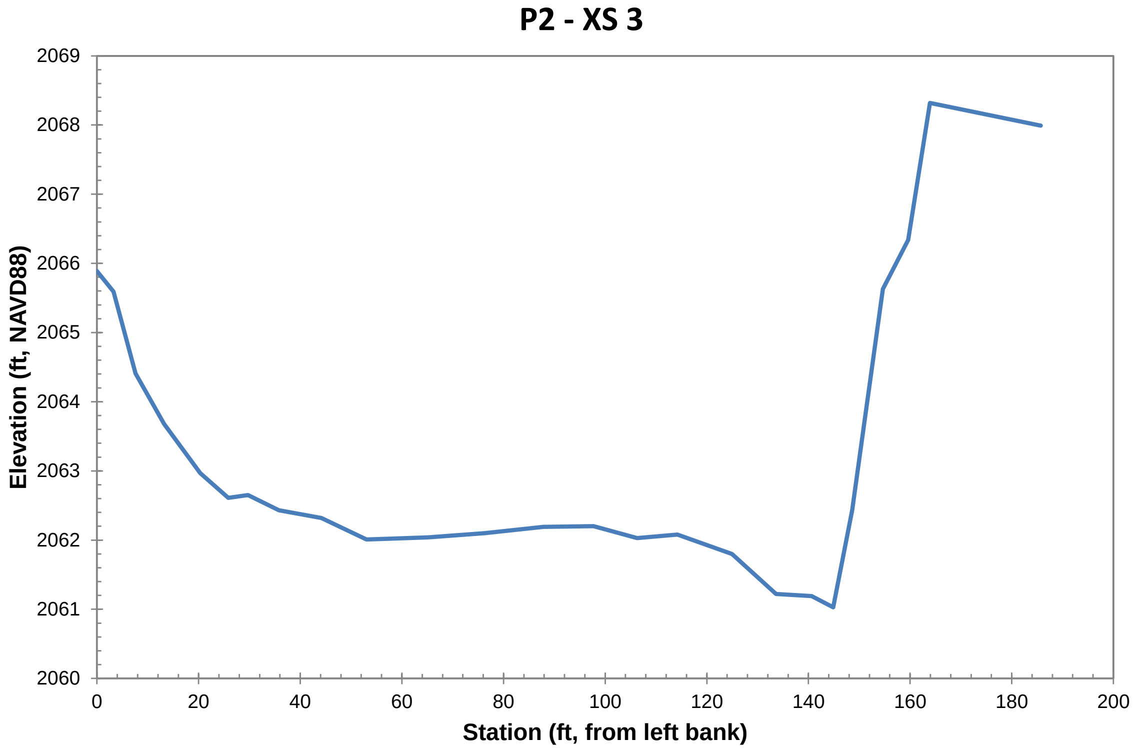

P3 - XS 10
Elevation (ft, NAVD88)
2014
2013
2012
2011
2010
2009
2008
2007
2006
2005
0
50
100
150
200
250
300
350
Station (ft, from left bank)

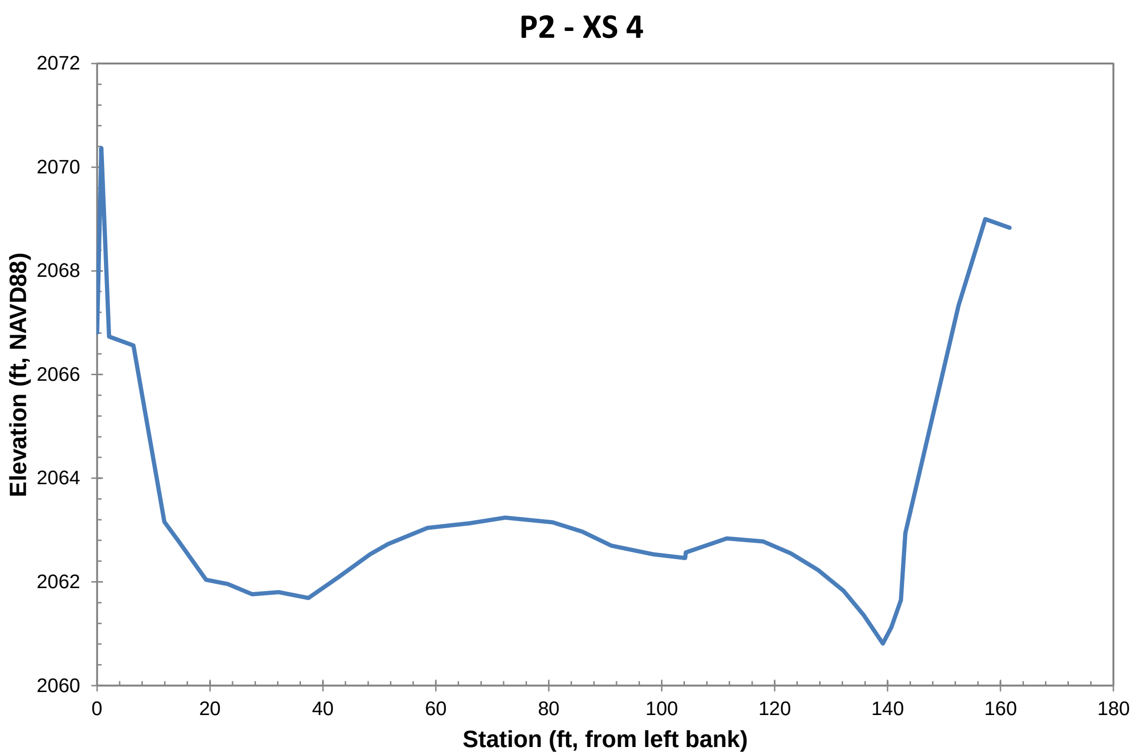

P3 - XS 11
Elevation (ft, NAVD88)
Station (ft, from left bank)
2018
2016
2014
2012
2010
2008
2006
2004
0
50
100
150
200
250
300
350

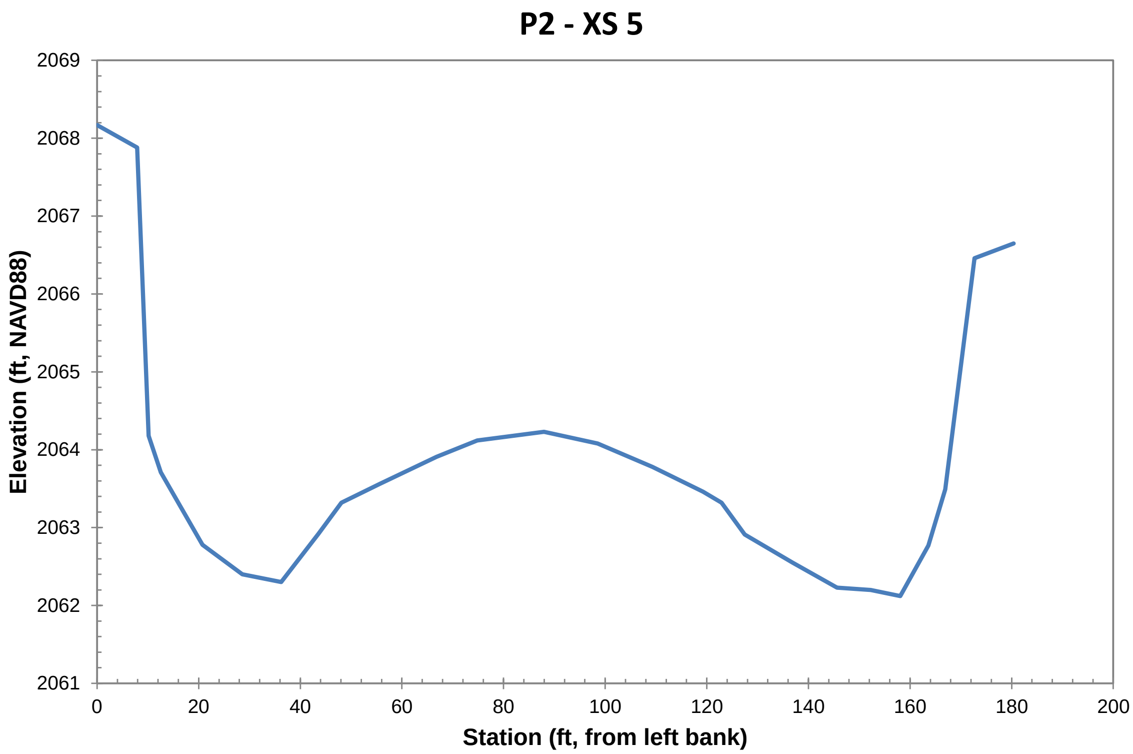

P3 - XS 12
Elevation (ft, NAVD88)
Station (ft, from left bank)
2013
2012
2011
2010
2009
2008
2007
2006
2005
2004
0
20
40
60
80
100
120
140
160
180

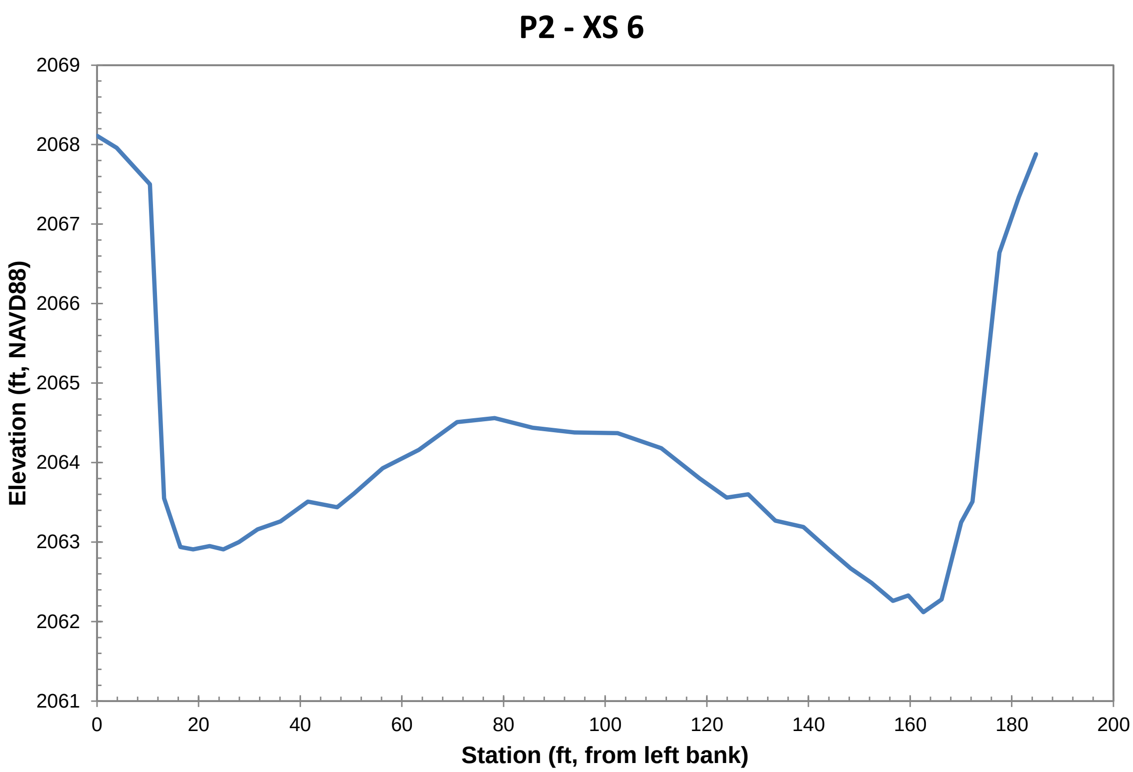

P2 - XS 1
2069
2068
2067
2066
2065
2064
2063
2062
2061
2060
0
20
40
60
80
100
120
140
160
180
Station (ft, from left bank)
Elevation (ft, NAVD88)


P2 - XS 2
2072
2070
2068
2066
2064
2062
2060
2058
0
20
40
60
80
100
120
140
160
180
Station (ft, from left bank)
Elevation (ft, NAVD88)

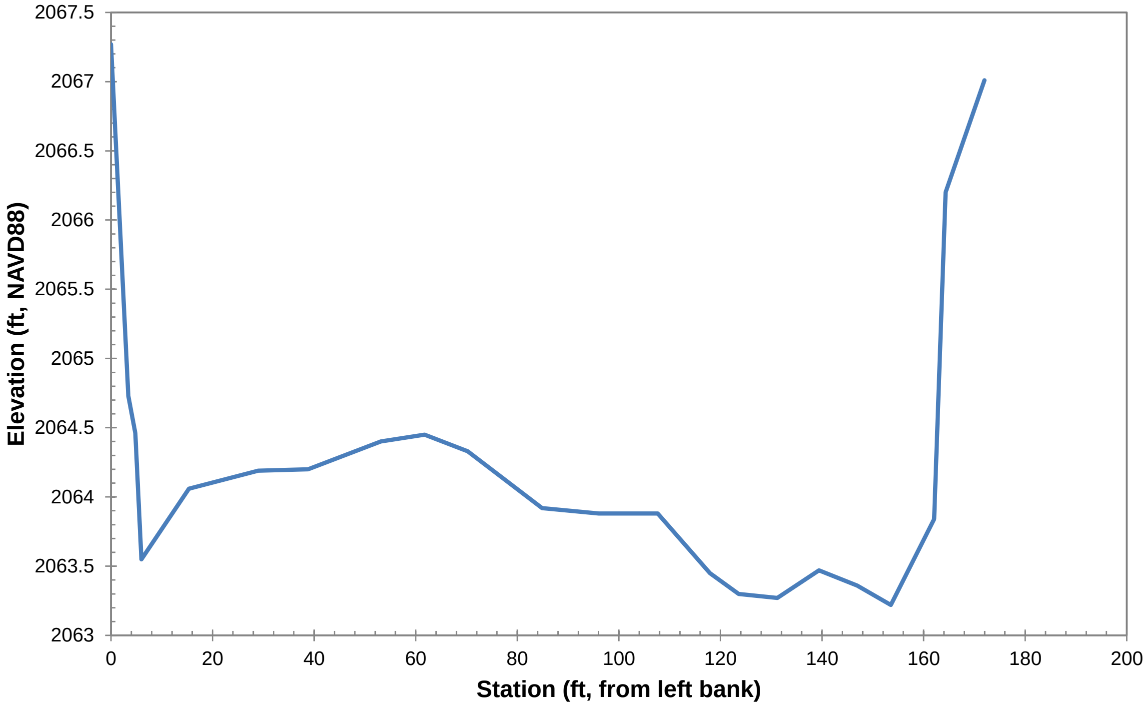

P2 - XS 3
Elevation (ft, NAVD88)
2069
2068
2067
2066
2065
2064
2063
2062
2061
2060
0
20
40
60
80
100
120
140
160
180
200
Station (ft, from left bank)


P2 - XS 4
Elevation (ft, NAVD88)
2072
2070
2068
2066
2064
2062
2060
0
20
40
60
80
100
120
140
160
180
Station (ft, from left bank)


P2 - XS 5
Elevation (ft, NAVD88)
2069
2068
2067
2066
2065
2064
2063
2062
2061
0
20
40
60
80
100
120
140
160
180
200
Station (ft, from left bank)

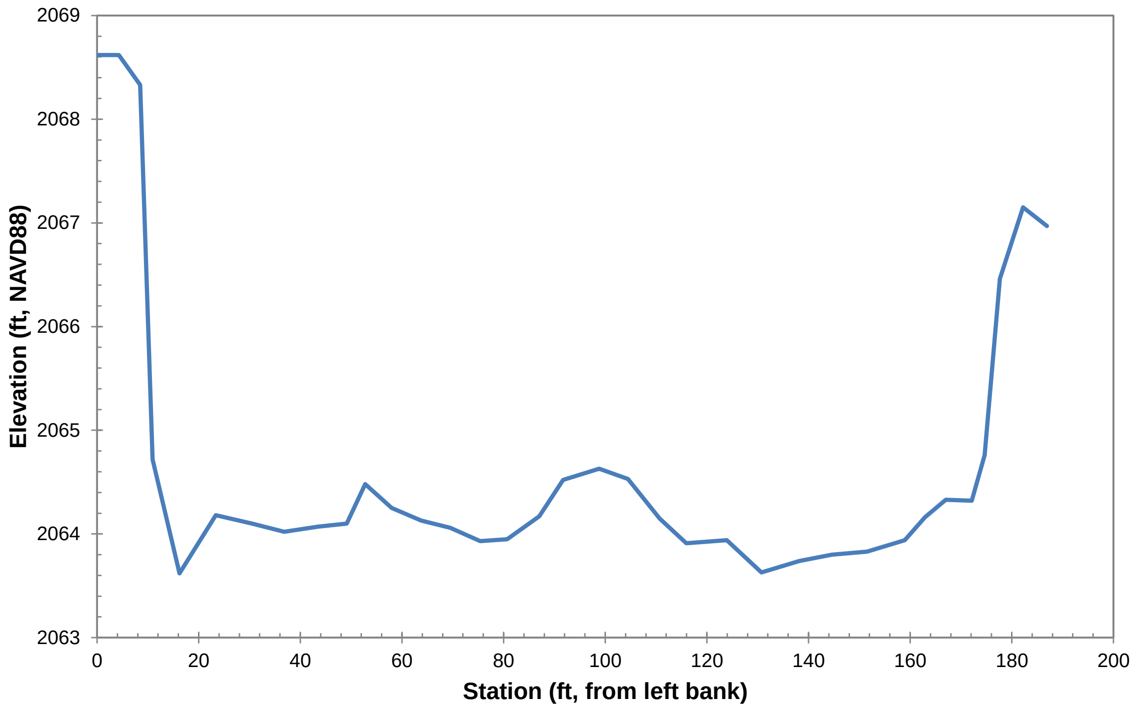

P2 - XS 6
Elevation (ft, NAVD88)
2069
2068
2067
2066
2065
2064
2063
2062
2061
0
20
40
60
80
100
120
140
160
180
200
Station (ft, from left bank)

P2 - XS 7



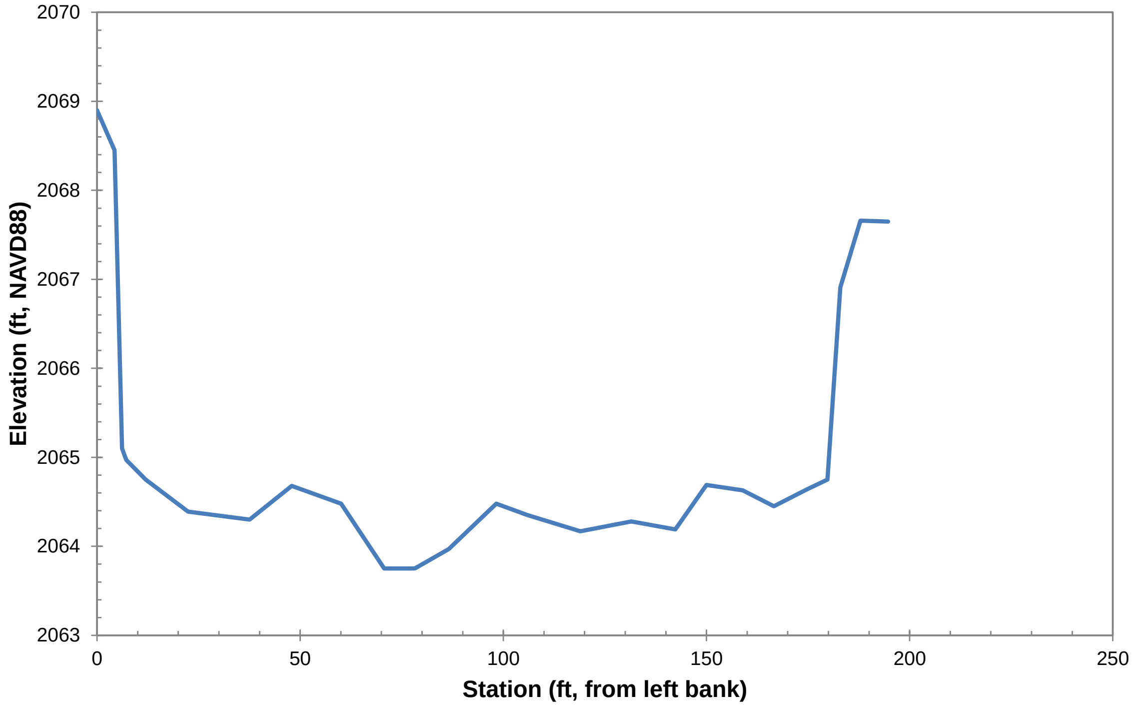
2067.5
2067
2066.5
2066
2065.5
2065
2064.5
2064
2063.5
2063
Elevation (ft, NAVD88)
0
20
40
60
80
100
120
140
160
180
200
Station (ft, from left bank)

P2 - XS 8

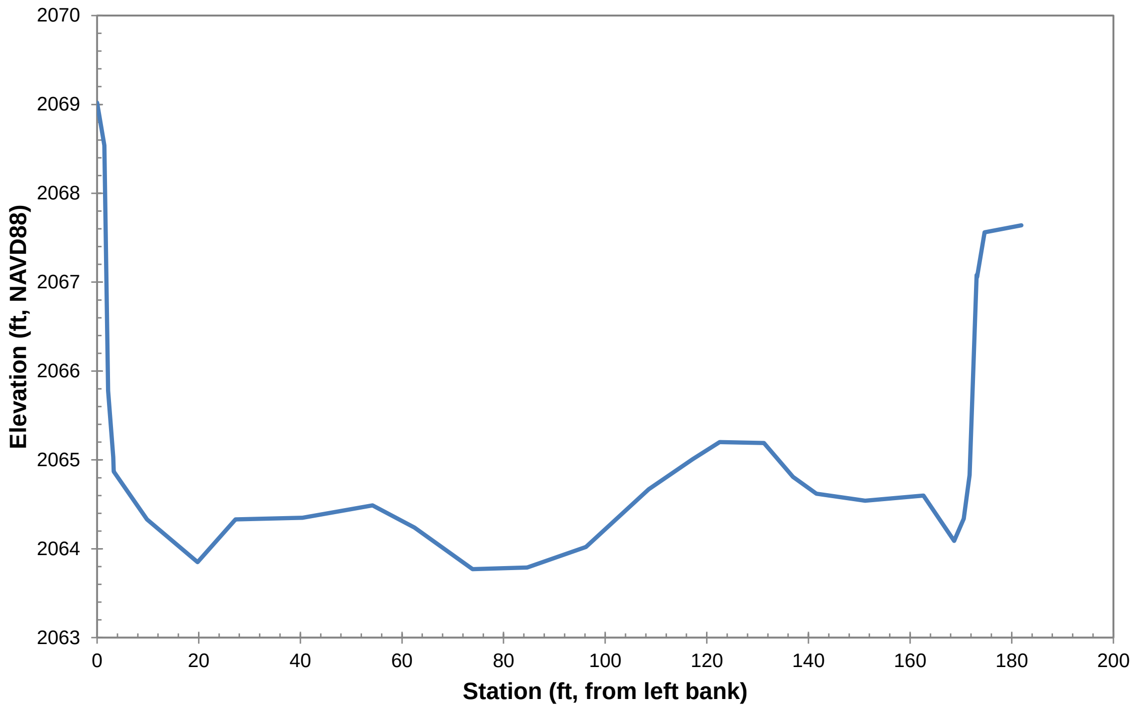


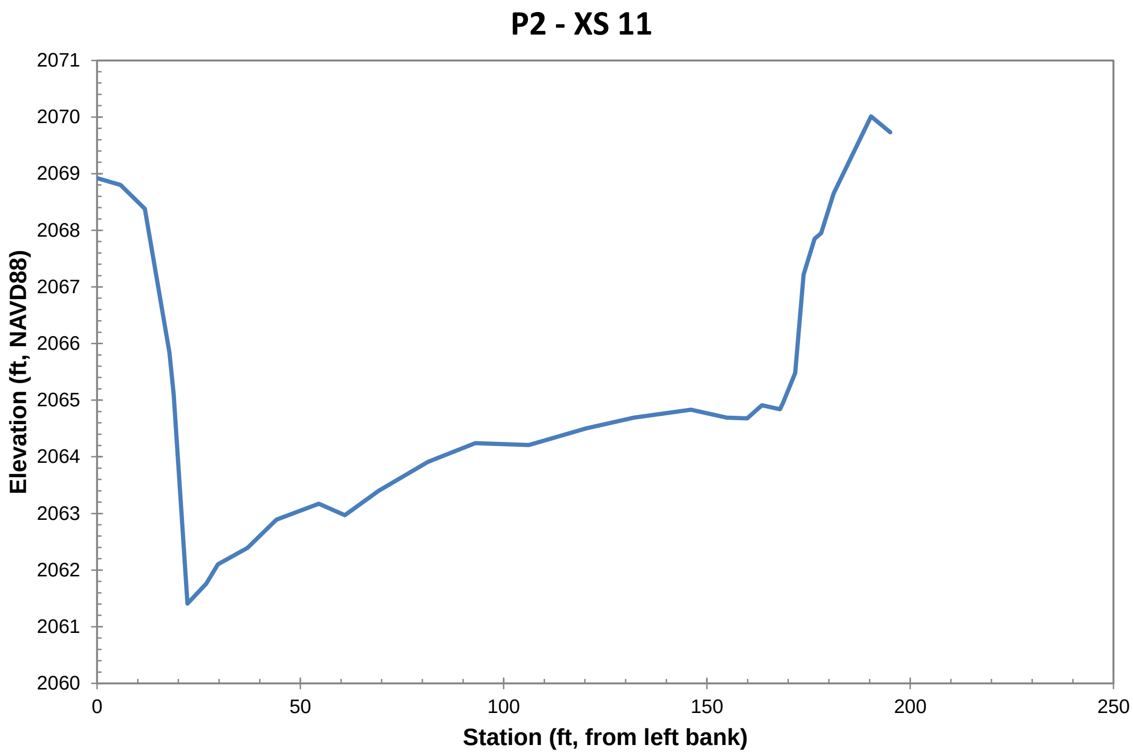
2069
2068
2067
2066
2065
2064
2063
Elevation (ft, NAVD88)
0
20
40
60
80
100
120
140
160
180
200
Station (ft, from left bank)

P2 - XS 9

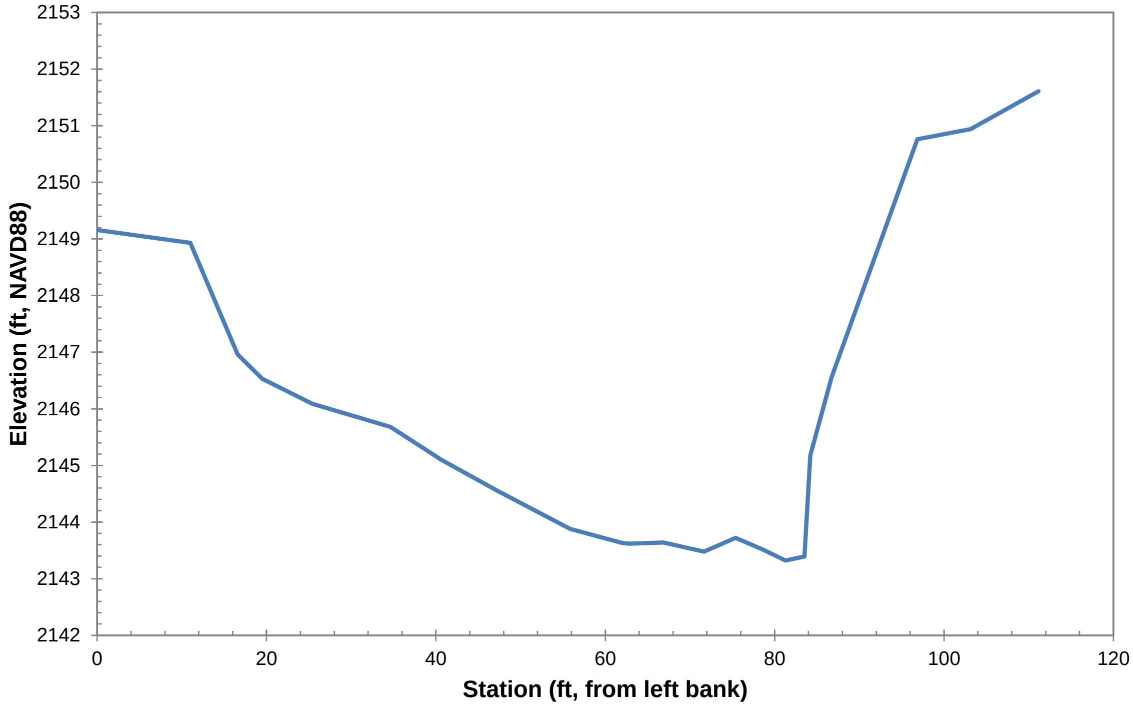


P2 - XS 10

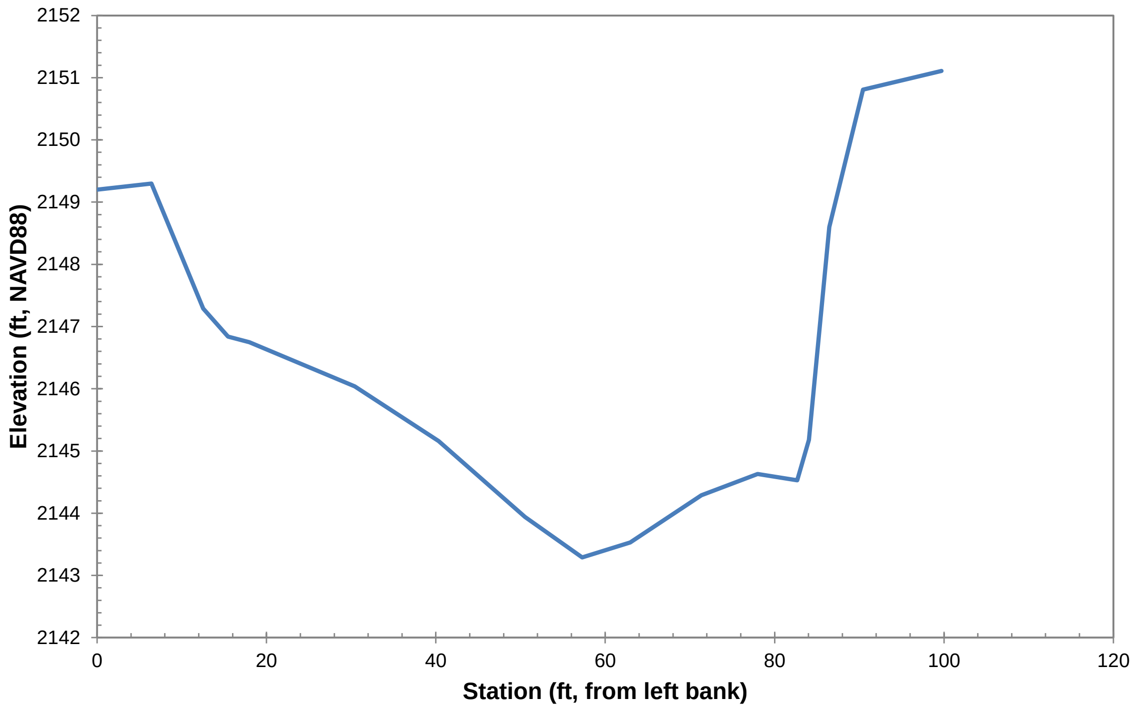


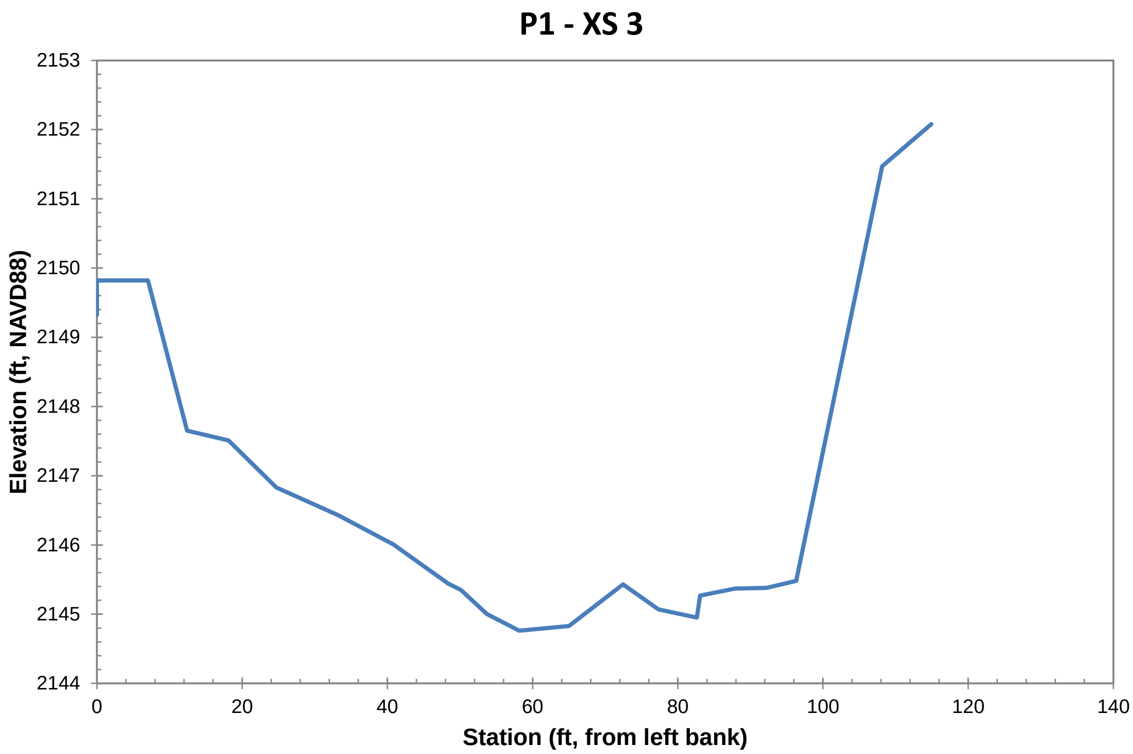

P2 - XS 11
Elevation (ft, NAVD88)
2071
2070
2069
2068
2067
2066
2065
2064
2063
2062
2061
2060
0
50
100
150
200
250
Station (ft, from left bank)

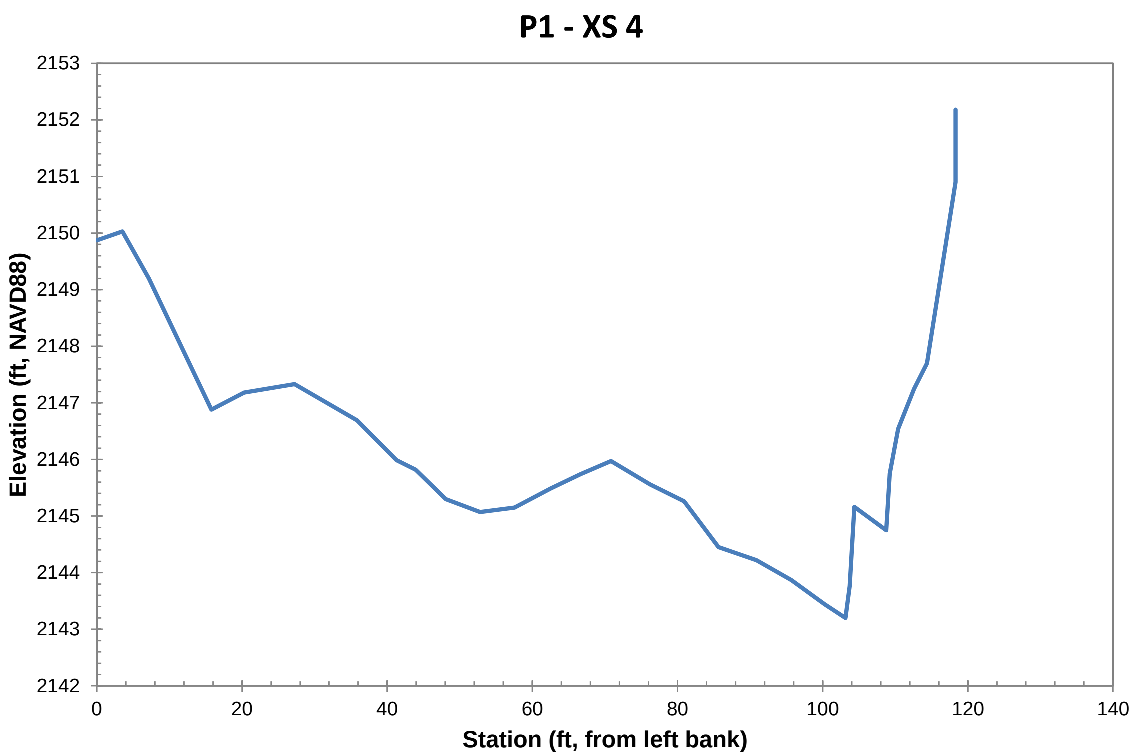
P1 - XS 1


2153
2152
2151
2150
2149
2148
2147
2146
2145
2144
2143
2142
Elevation (ft, NAVD88)
0
20
40
60
80
100
120
Station (ft, from left bank)

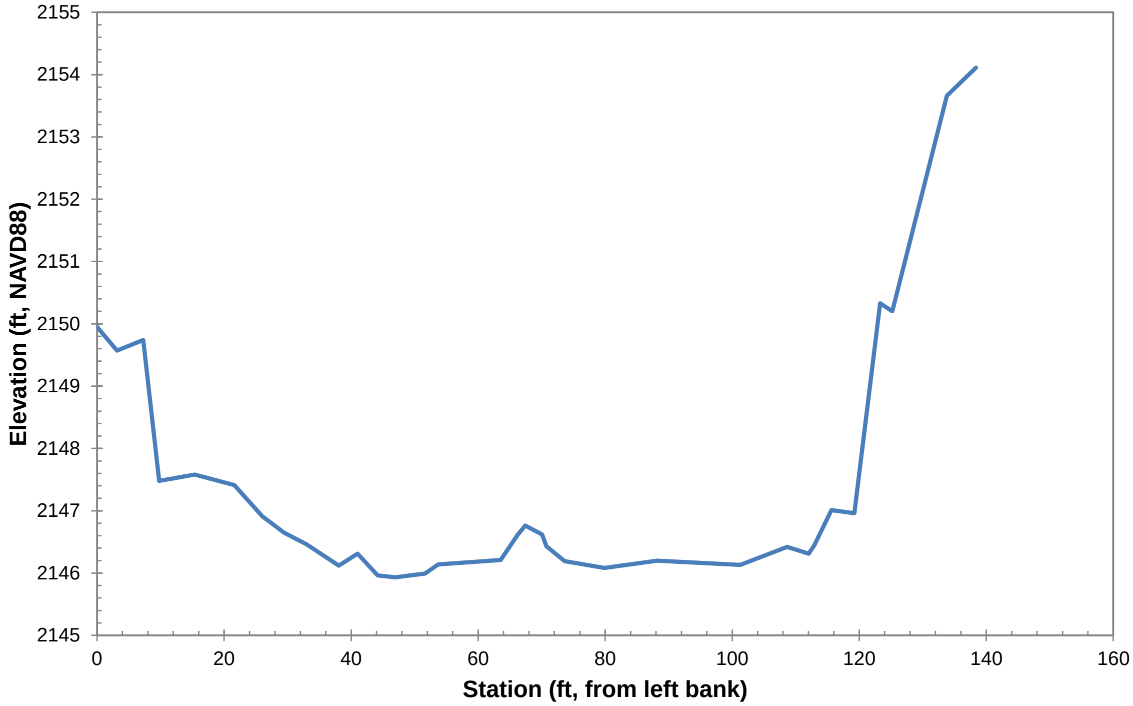
P1 - XS 2


2152
2151
2150
2149
2148
2147
2146
2145
2144
2143
2142
Elevation (ft, NAVD88)
0
20
40
60
80
100
120
Station (ft, from left bank)


P1 - XS 3
Elevation (ft, NAVD88)
Station (ft, from left bank)
2153
2152
2151
2150
2149
2148
2147
2146
2145
2144
0
20
40
60
80
100
120
140

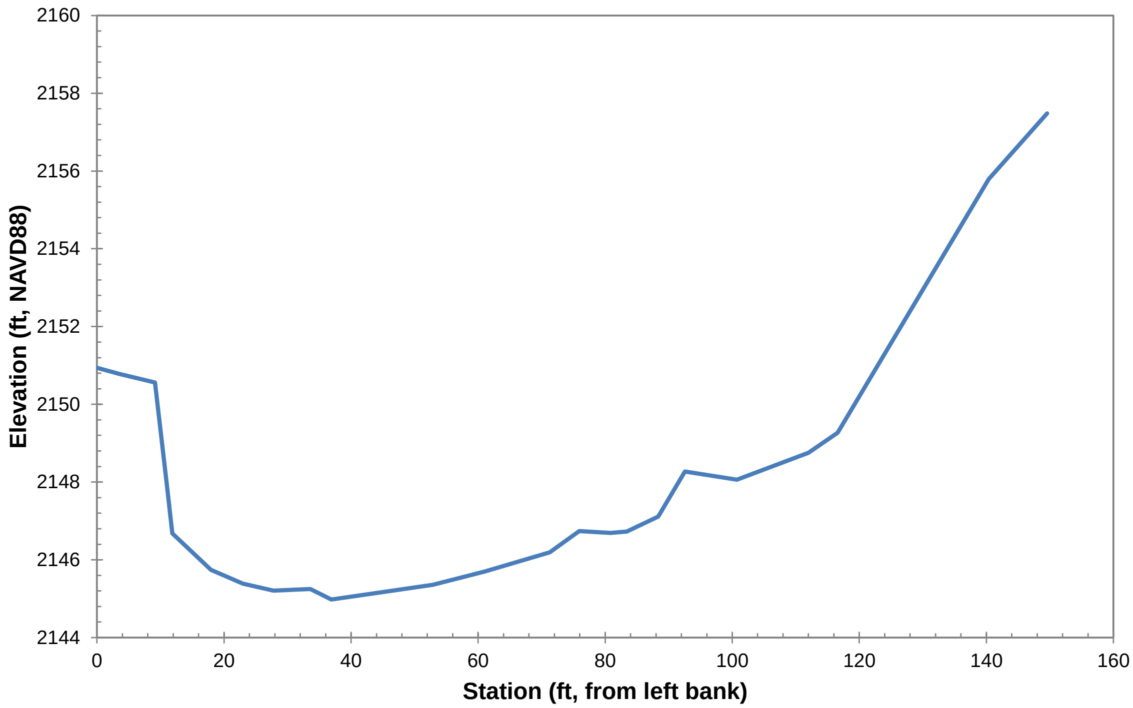

P1 - XS 4
Elevation (ft, NAVD88)
Station (ft, from left bank)
2153
2152
2151
2150
2149
2148
2147
2146
2145
2144
2143
2142
0
20
40
60
80
100
120
140

P1 - XS 5

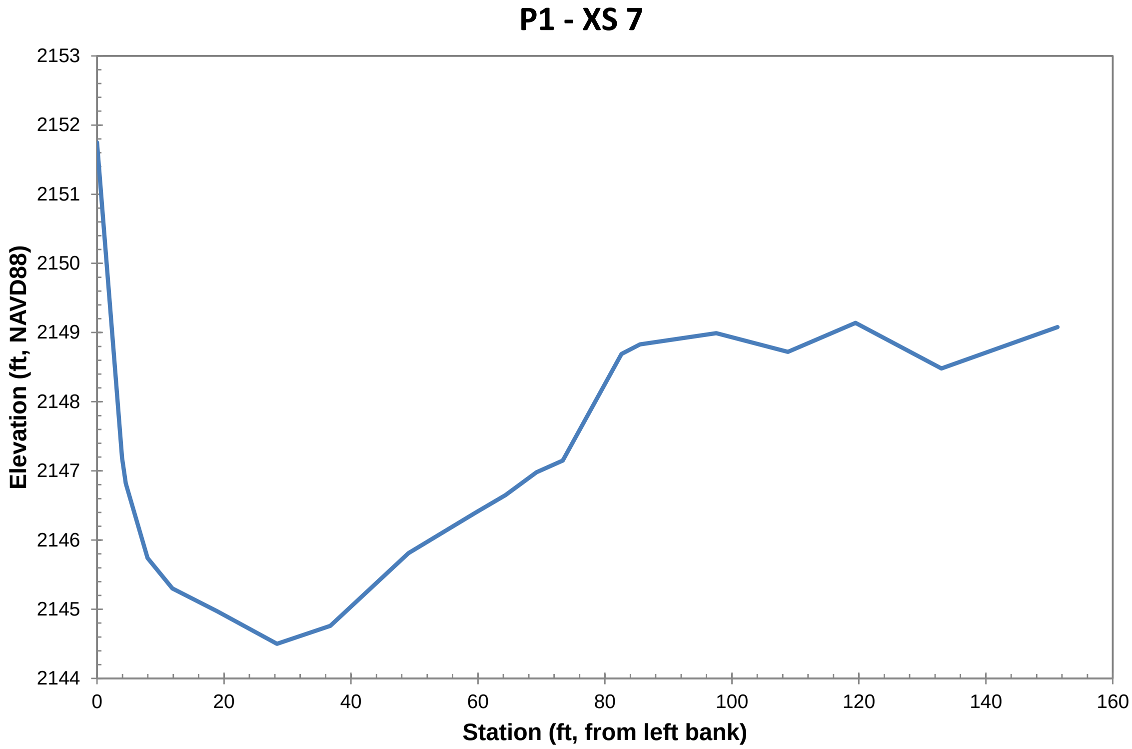


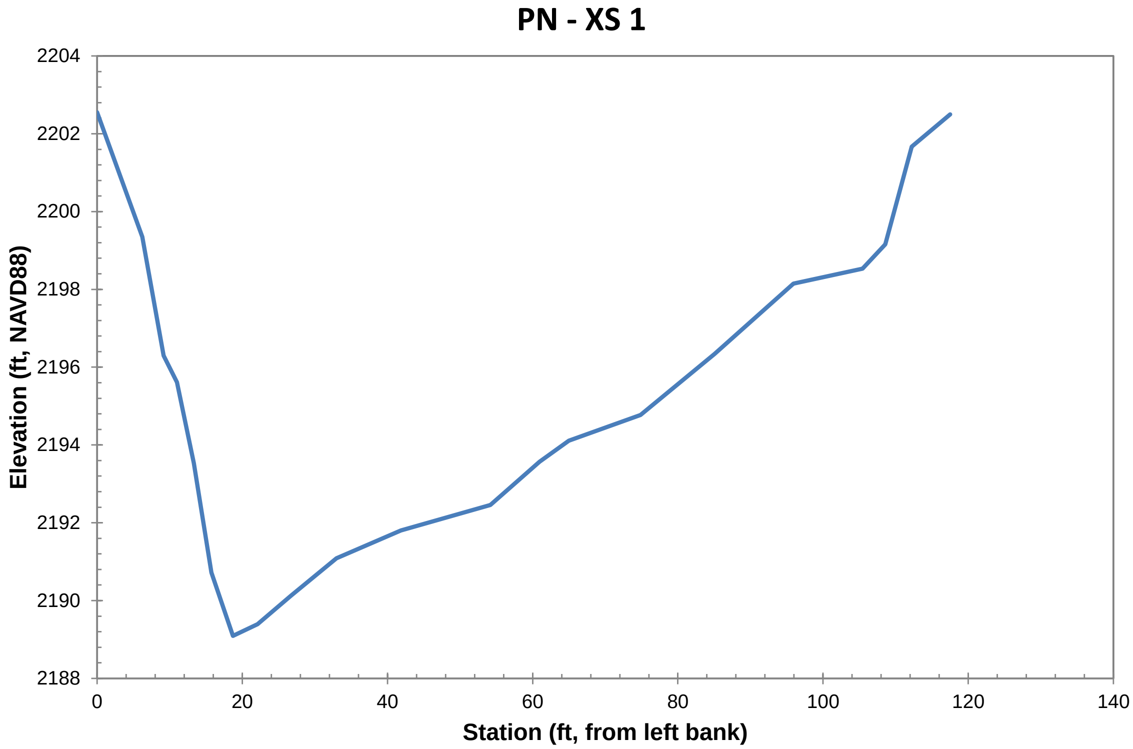
2155
2154
2153
2152
2151
2150
2149
2148
2147
2146
2145
Elevation (ft, NAVD88)
0
20
40
60
80
100
120
140
160
Station (ft, from left bank)

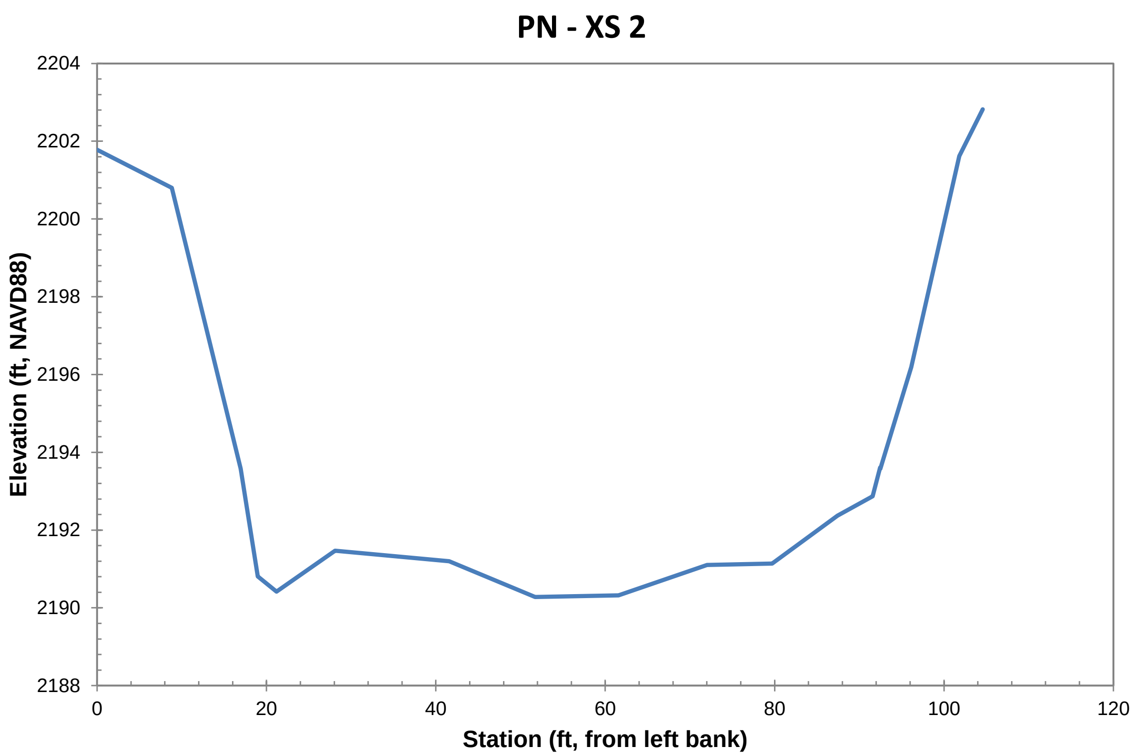
P1 - XS 6

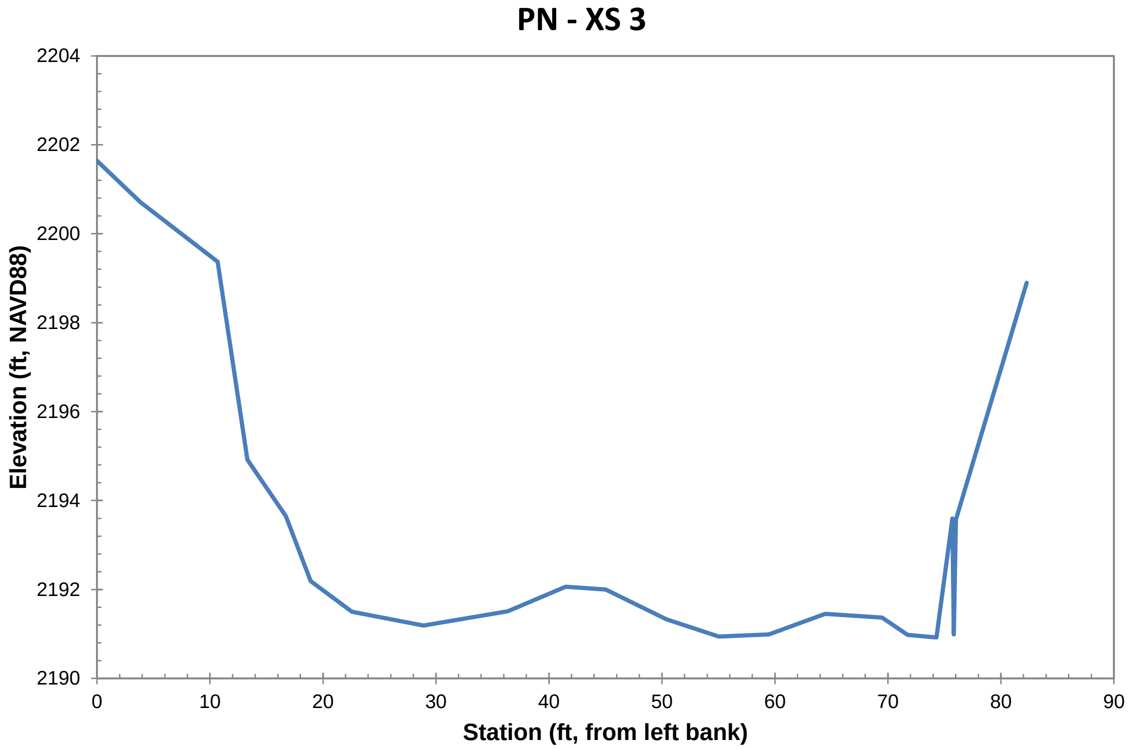


2160
2158
2156
2154
2152
2150
2148
2146
2144
Elevation (ft, NAVD88)
0
20
40
60
80
100
120
140
160
Station (ft, from left bank)

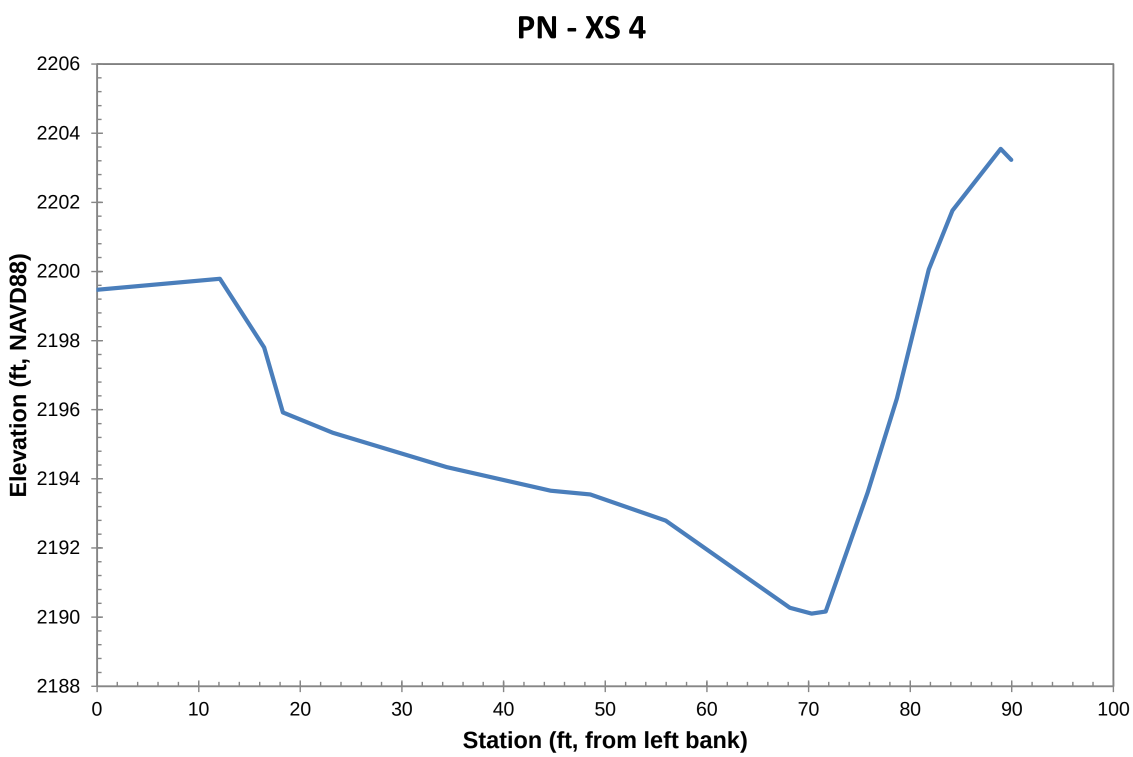

P1 - XS 7
Elevation (ft, NAVD88)
2153
2152
2151
2150
2149
2148
2147
2146
2145
2144
0
20
40
60
80
100
120
140
160
Station (ft, from left bank)


PN - XS 1
2204
2202
2200
2198
2196
2194
2192
2190
2188
Elevation (ft, NAVD88)
0
20
40
60
80
100
120
140
Station (ft, from left bank)

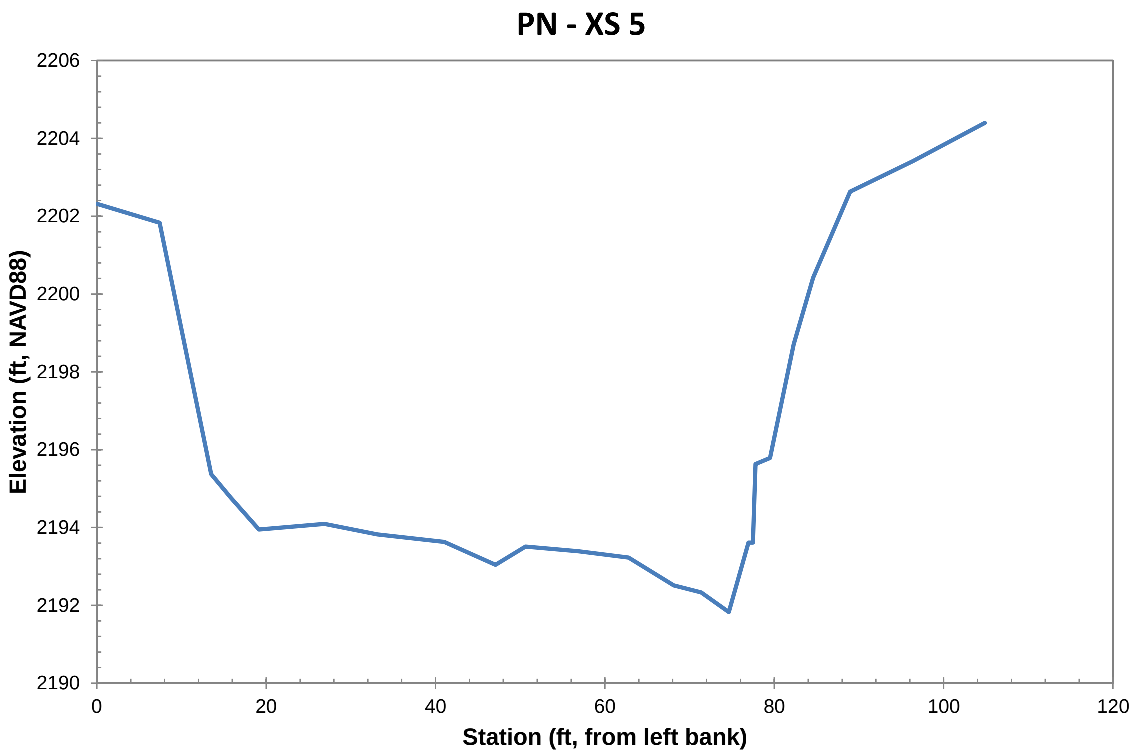

PN - XS 2
2204
2202
2200
2198
2196
2194
2192
2190
2188
Elevation (ft, NAVD88)
0
20
40
60
80
100
120
Station (ft, from left bank)

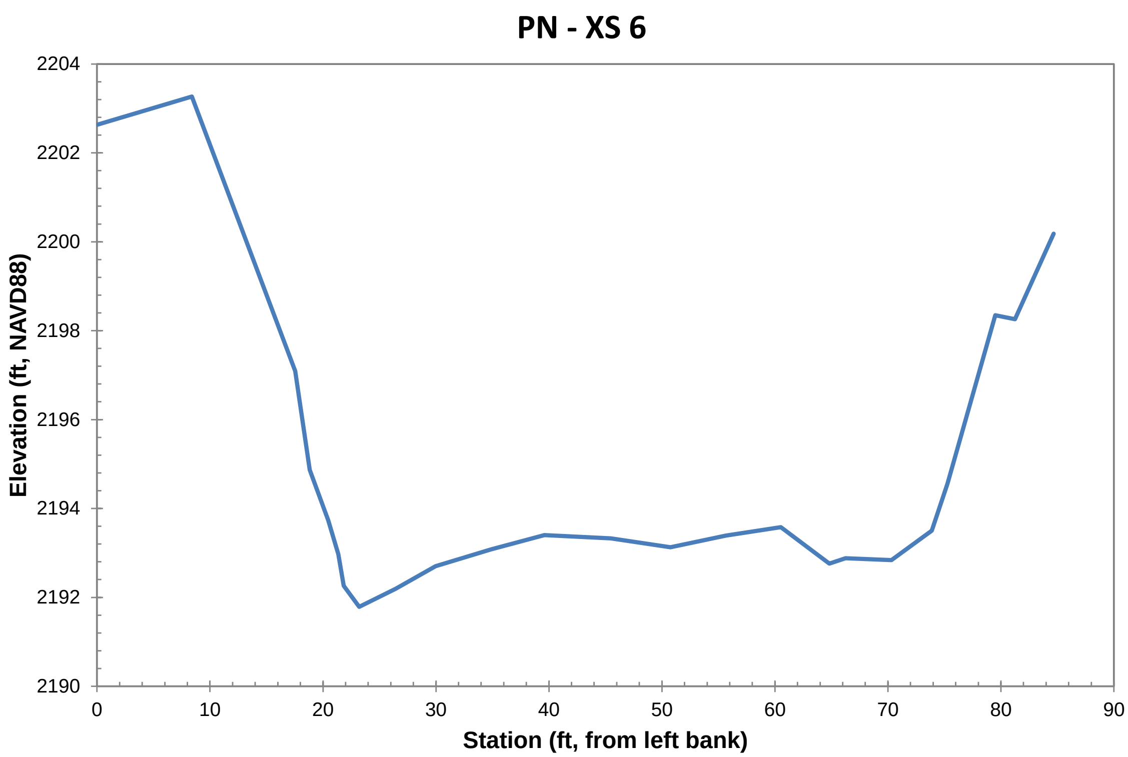


PN - XS 3
2204
2202
2200
2198
2196
2194
2192
2190
Elevation (ft, NAVD88)
0
10
20
30
40
50
60
70
80
90
Station (ft, from left bank)

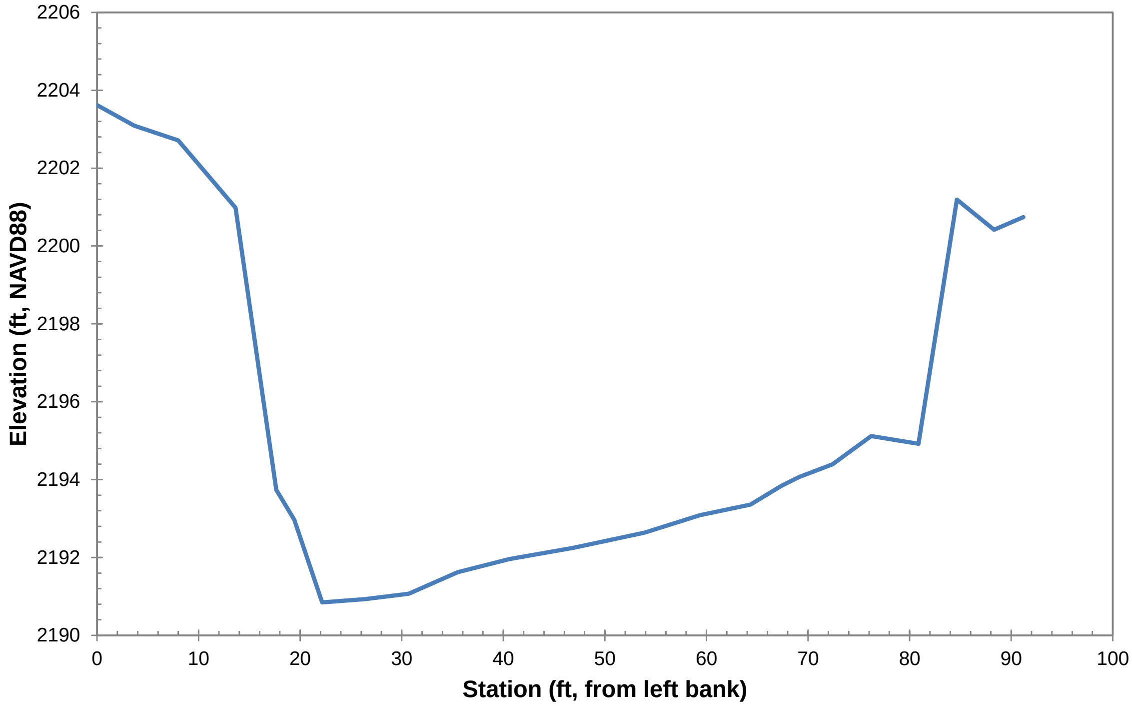


PN - XS 4
2206
2204
2202
2200
2198
2196
2194
2192
2190
2188
Elevation (ft, NAVD88)
0
10
20
30
40
50
60
70
80
90
100
Station (ft, from left bank)


PN - XS 5
Elevation (ft, NAVD88)
2190
2192
2194
2196
2198
2200
2202
2204
2206
0
20
40
60
80
100
120
Station (ft, from left bank)

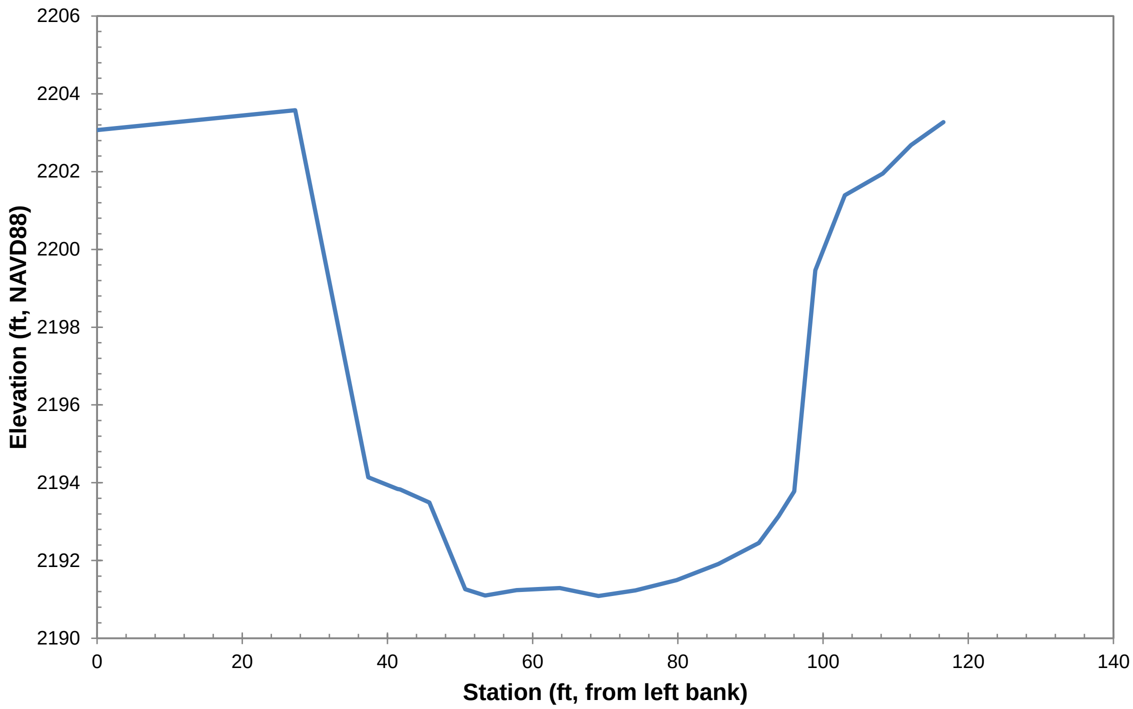

PN - XS 6
Elevation (ft, NAVD88)
2190
2192
2194
2196
2198
2200
2202
2204
0
10
20
30
40
50
60
70
80
90
Station (ft, from left bank)

PN - XS 7



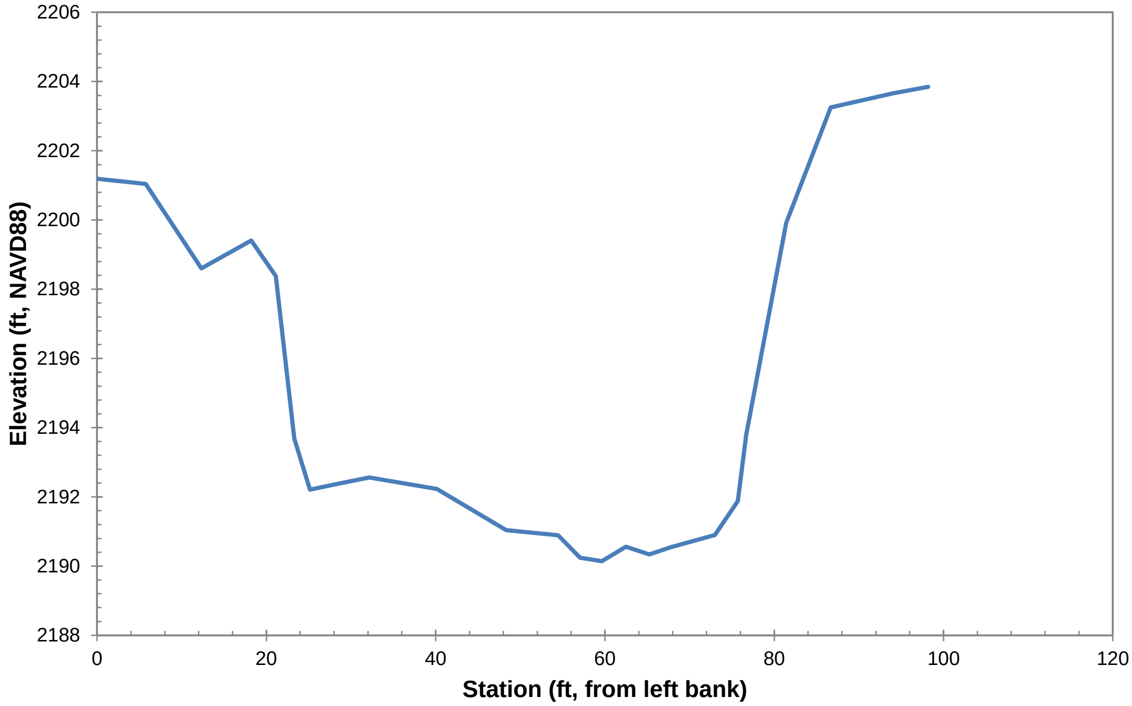
2206
2204
2202
2200
2198
2196
2194
2192
2190
Elevation (ft, NAVD88)
0
10
20
30
40
50
60
70
80
90
100
Station (ft, from left bank)

PN - XS 8

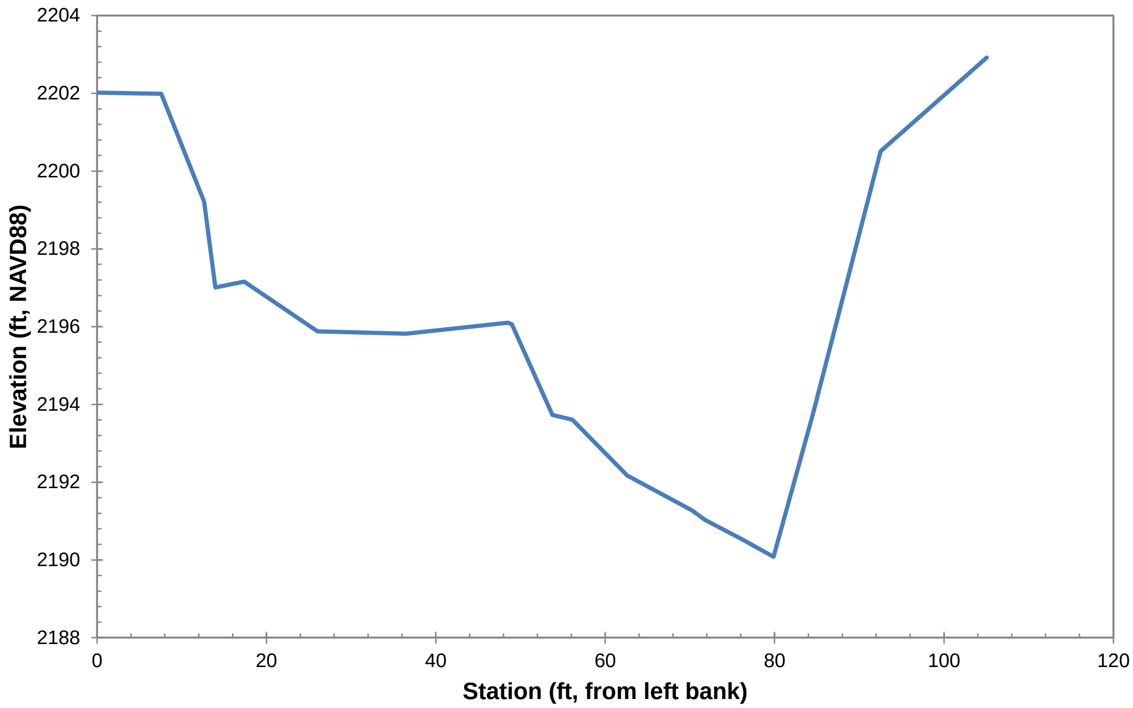


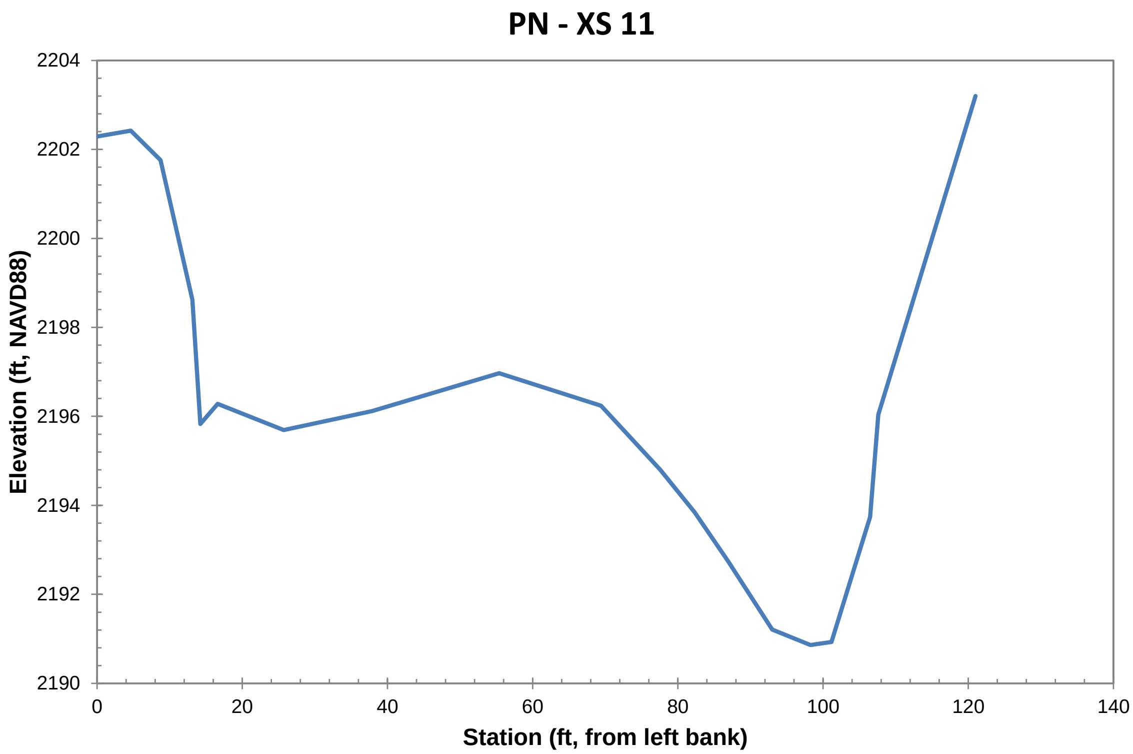
2206
2204
2202
2200
2198
2196
2194
2192
2190
Elevation (ft, NAVD88)
0
20
40
60
80
100
120
140
Station (ft, from left bank)

PN - XS 9



PN - XS 10

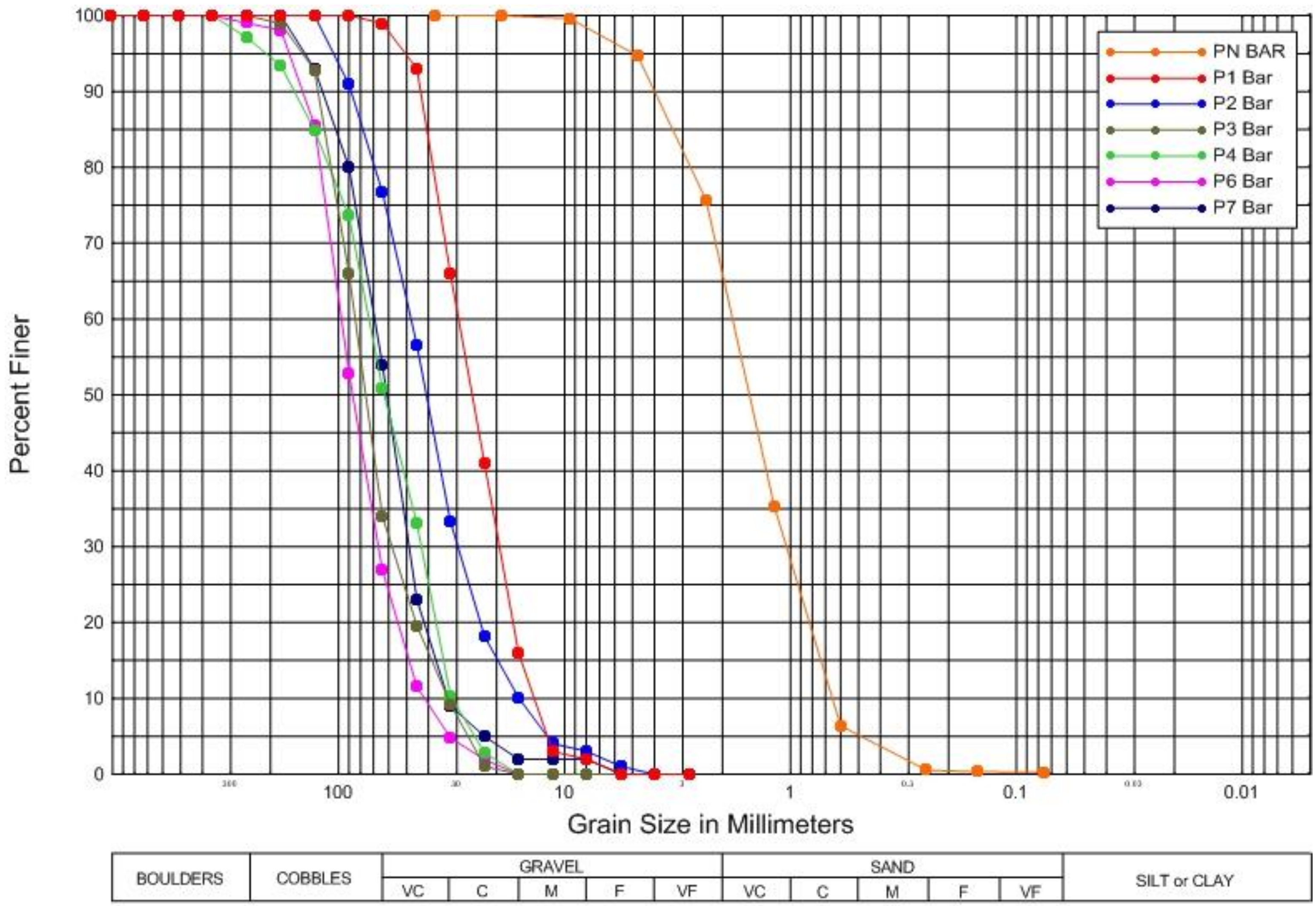



PN - XS 11
Elevation (ft, NAVD88)
2204
2202
2200
2198
2196
2194
2192
2190
0
20
40
60
80
100
120
140
Station (ft, from left bank)

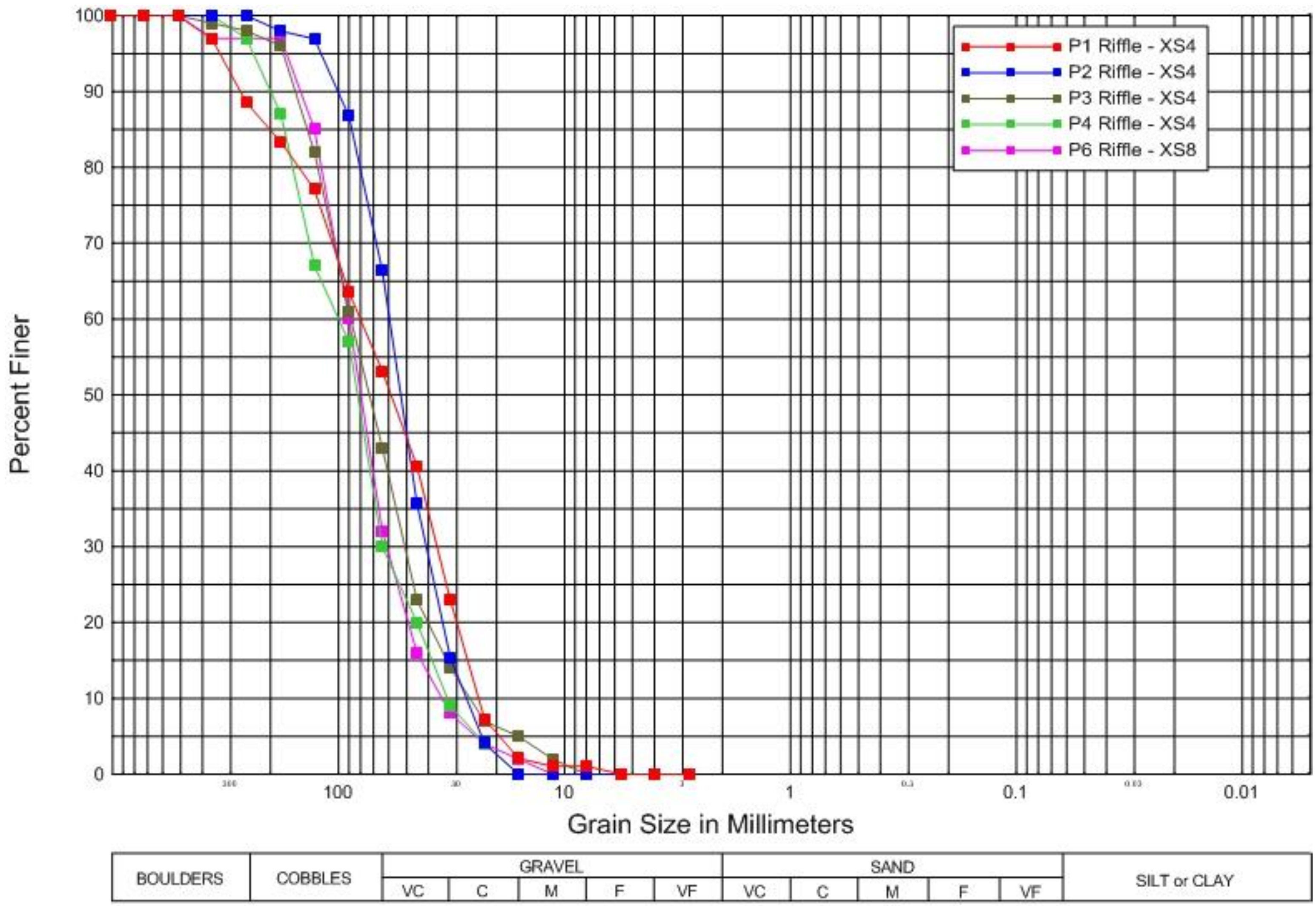
APPENDIX C

Particle Size Distribution Plots of the Full Gradations

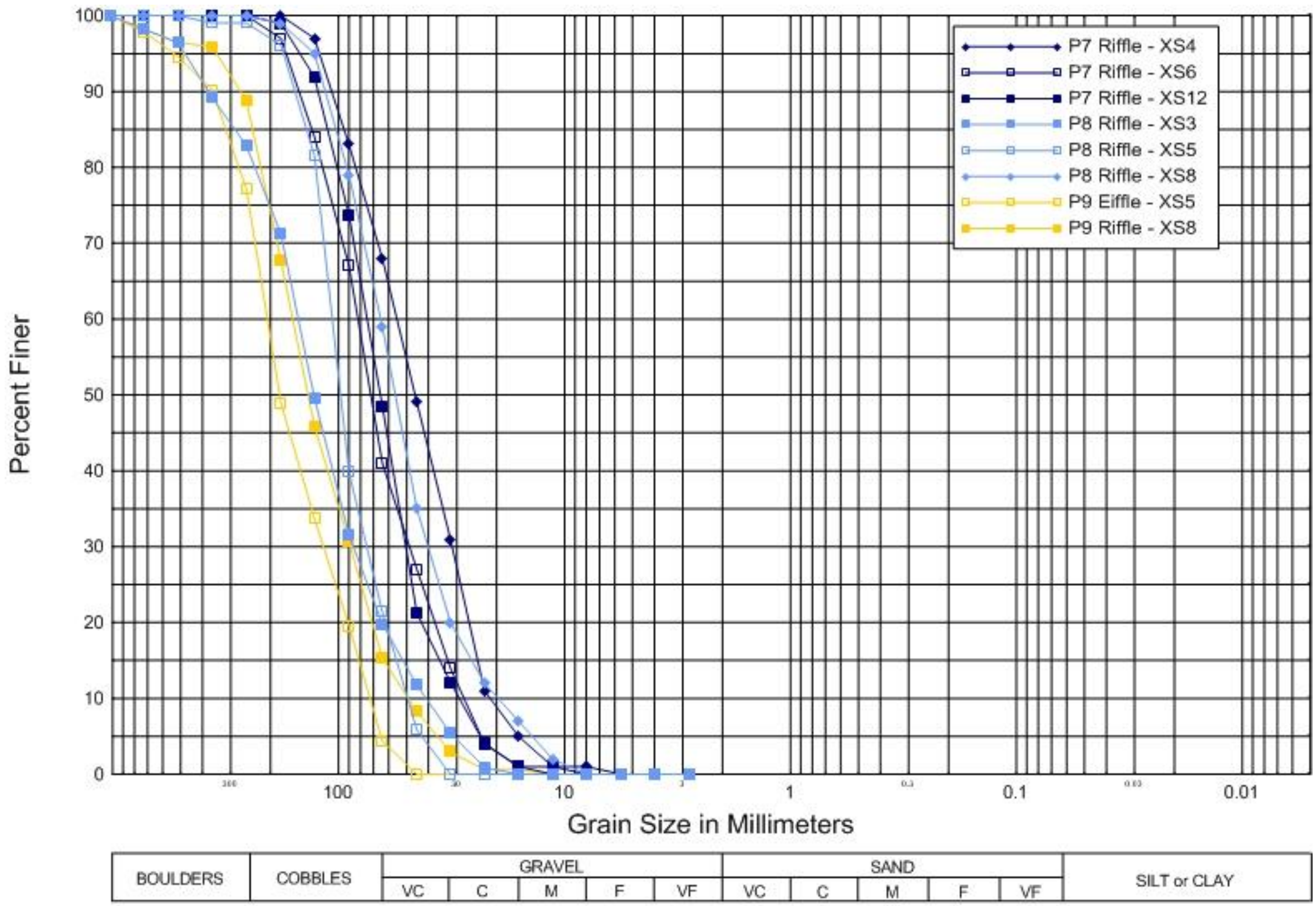
Bob Mussetter, Tetra Tech
December 18, 2014



Percent Finer
Grain Size in Millimeters
PN BAR
P1 Bar
P2 Bar
P3 Bar
P4 Bar
P6 Bar
P7 Bar
100
90
80
70
60
50
40
30
20
10
0
100
10
1
0.1
0.01
BOULDERS
COBBLES
GRAVEL
VC
C
M
F
VF
SAND
VC
C
M
F
VF
SILT or CLAY



Percent Finer
100
90
80
70
60
50
40
30
20
10
0
100
10
1
0.1
0.01
Grain Size in Millimeters
P1 Riffle - XS4
P2 Riffle - XS4
P3 Riffle - XS4
P4 Riffle - XS4
P6 Riffle - XS8
BOULDERS
COBBLES
GRAVEL
VC
C
M
F
VF
SAND
VC
C
M
F
VF
SILT or CLAY

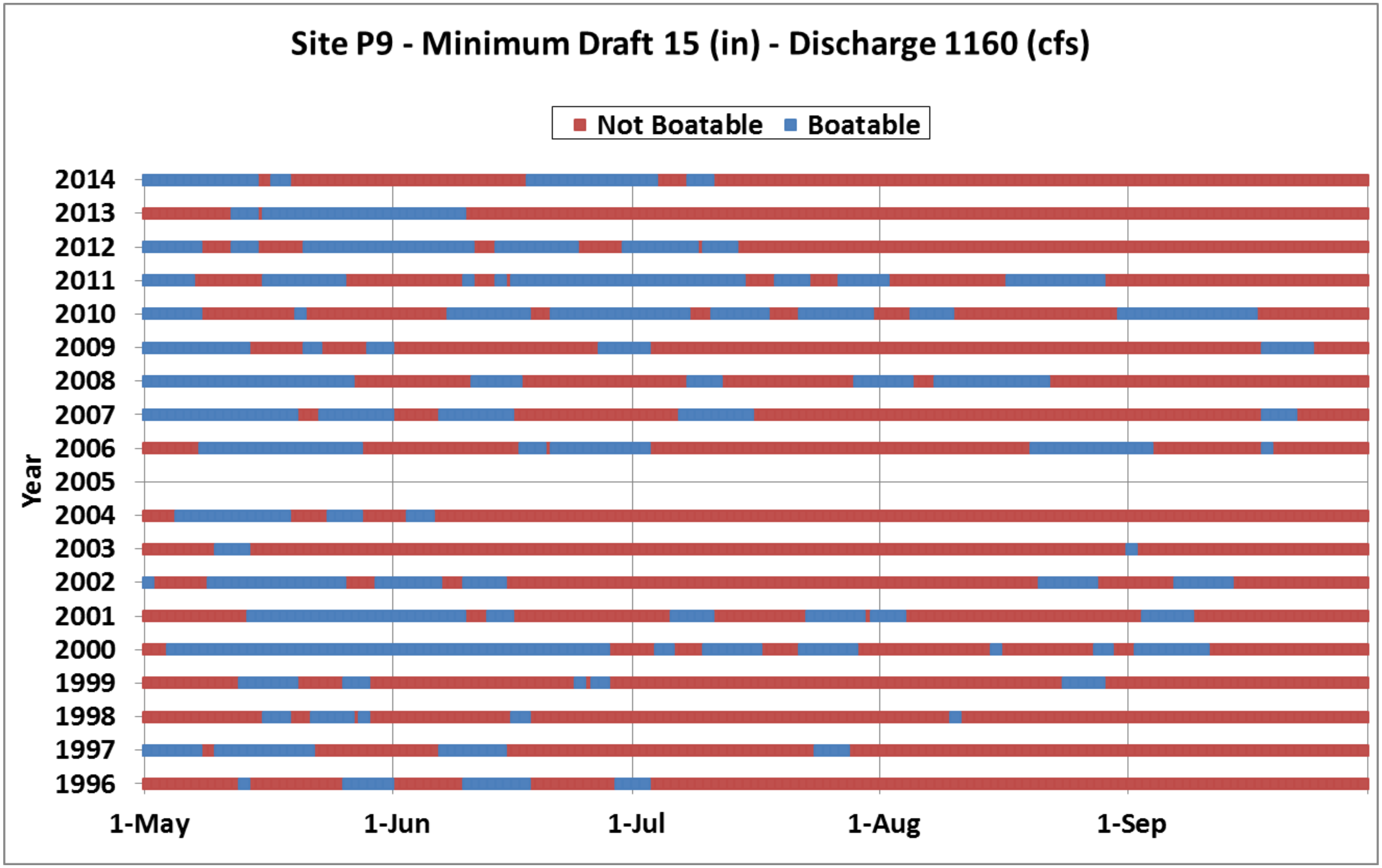


Percent Finer
100
90
80
70
60
50
40
30
20
10
0
100
10
1
0.1
0.01
Grain Size in Millimeters
P7 Riffle - XS4
P7 Riffle - XS6
P7 Riffle - XS12
P8 Riffle - XS3
P8 Riffle - XS5
P8 Riffle - XS8
P9 Eiffle - XS5
P9 Riffle - XS8
BOULDERS
COBBLES
GRAVEL
VC
C
M
F
VF
SAND
VC
C
M
F
VF
SILT or CLAY

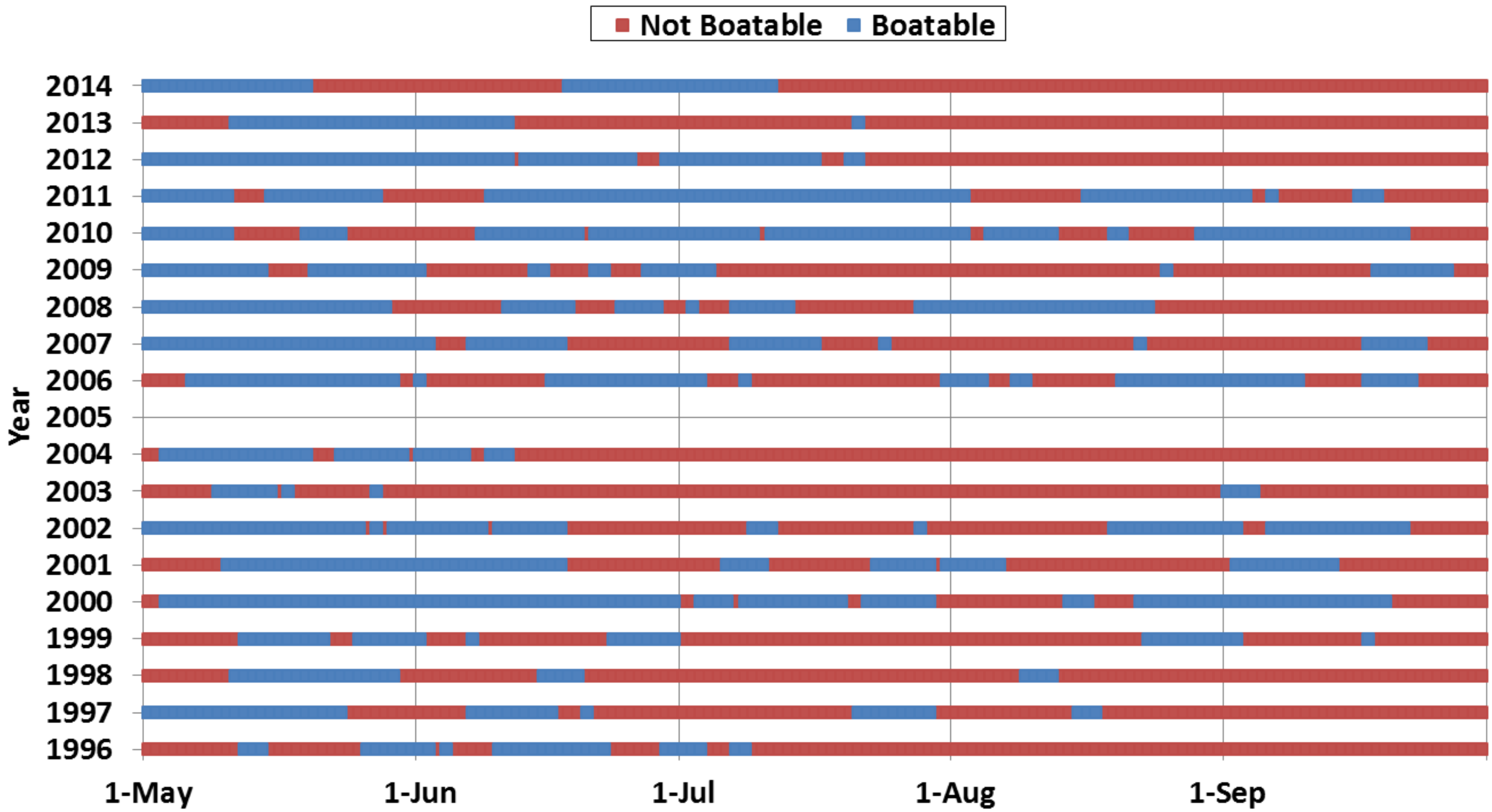
APPENDIX D

Calculated Periods of Boatability and Non-Boatability

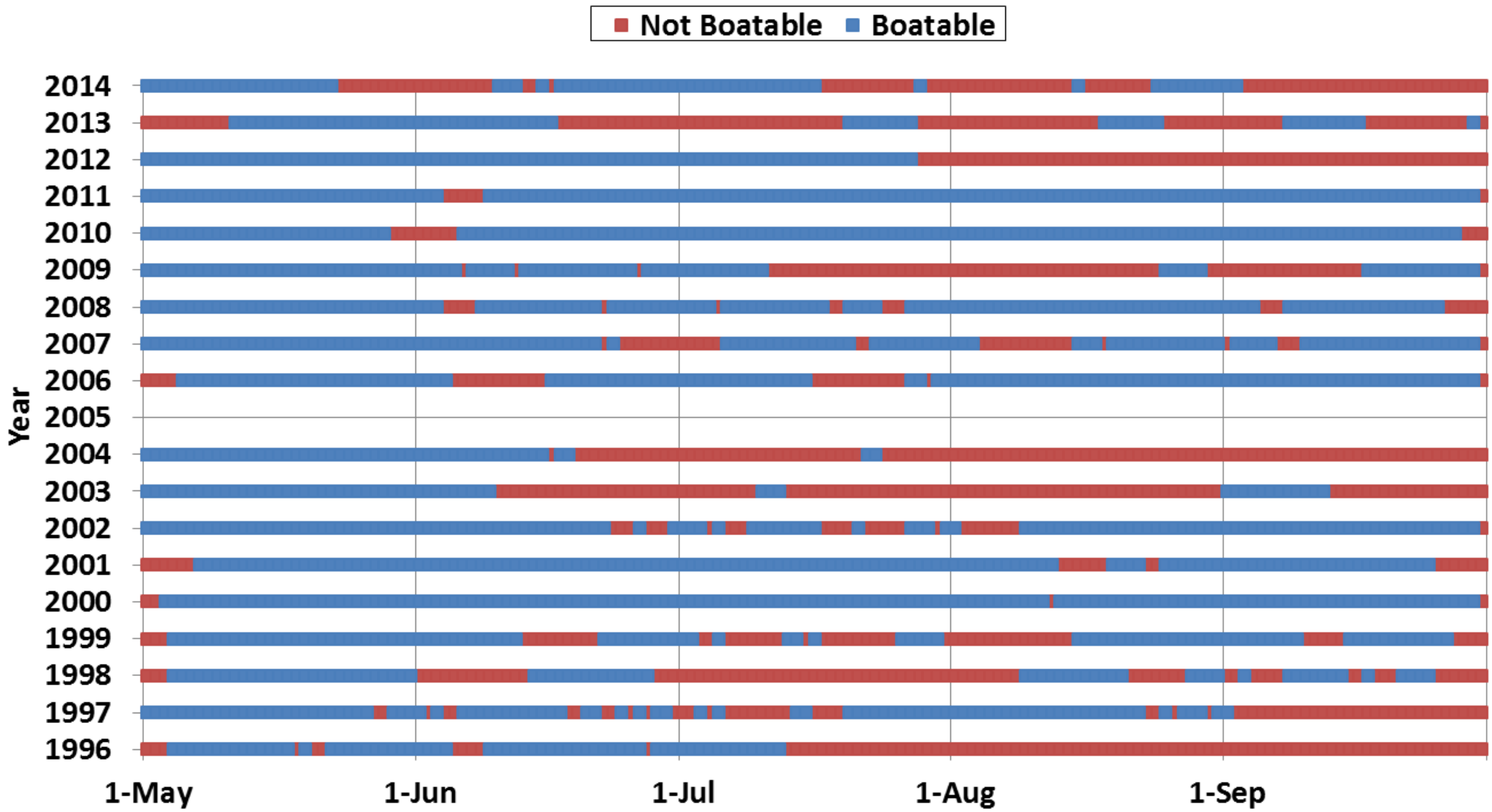
Bob Mussetter, Tetra Tech
December 18, 2014

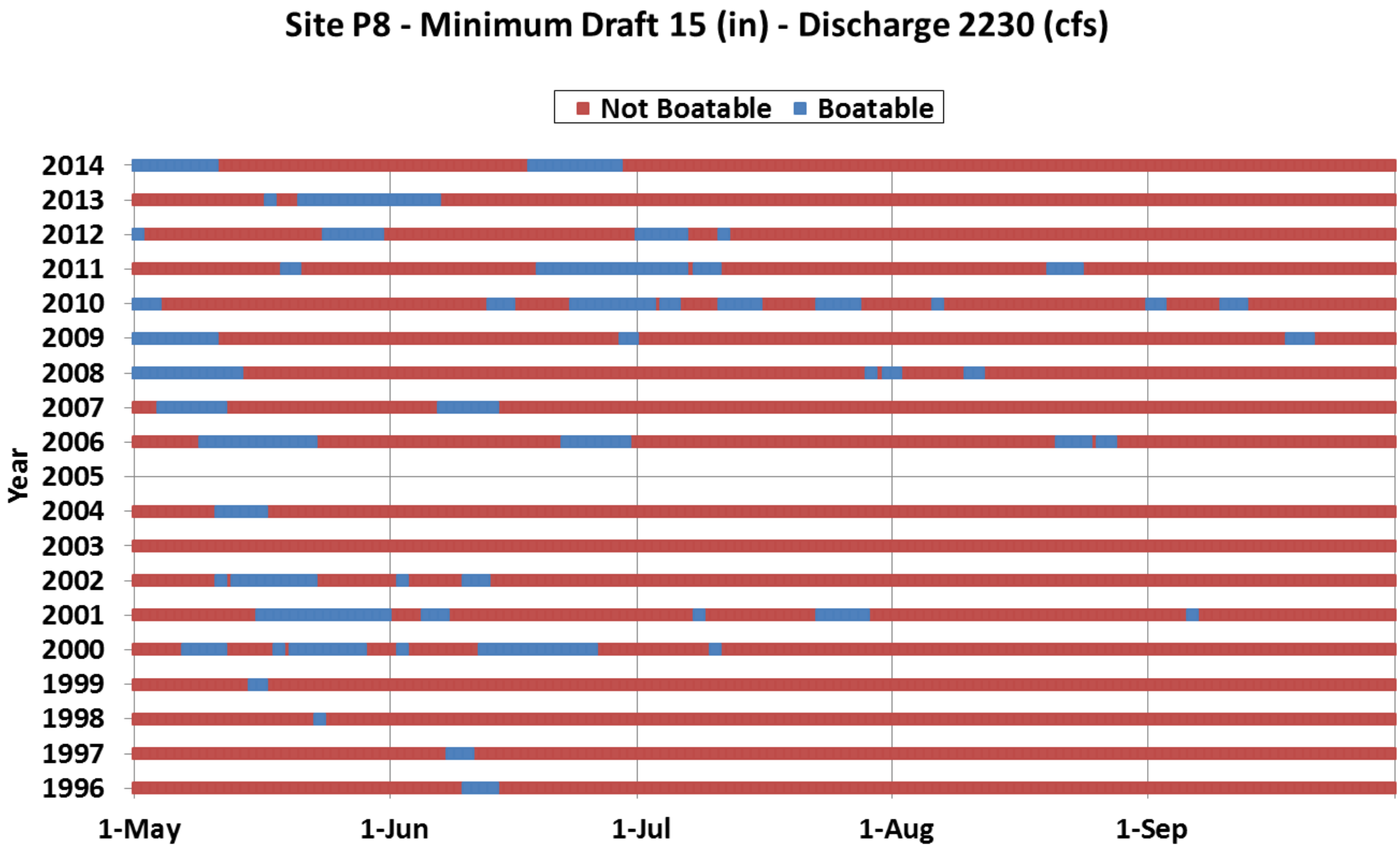

Site P9 - Minimum Draft 15 (in) - Discharge 1160 (cfs)
Not Boatable
Boatable
Year
2014
2013
2012
2011
2010
2009
2008
2007
2006
2005
2004
2003
2002
2001
2000
1999
1998
1997
1996
1-May
1-Jun
1-Jul
1-Aug
1-Sep

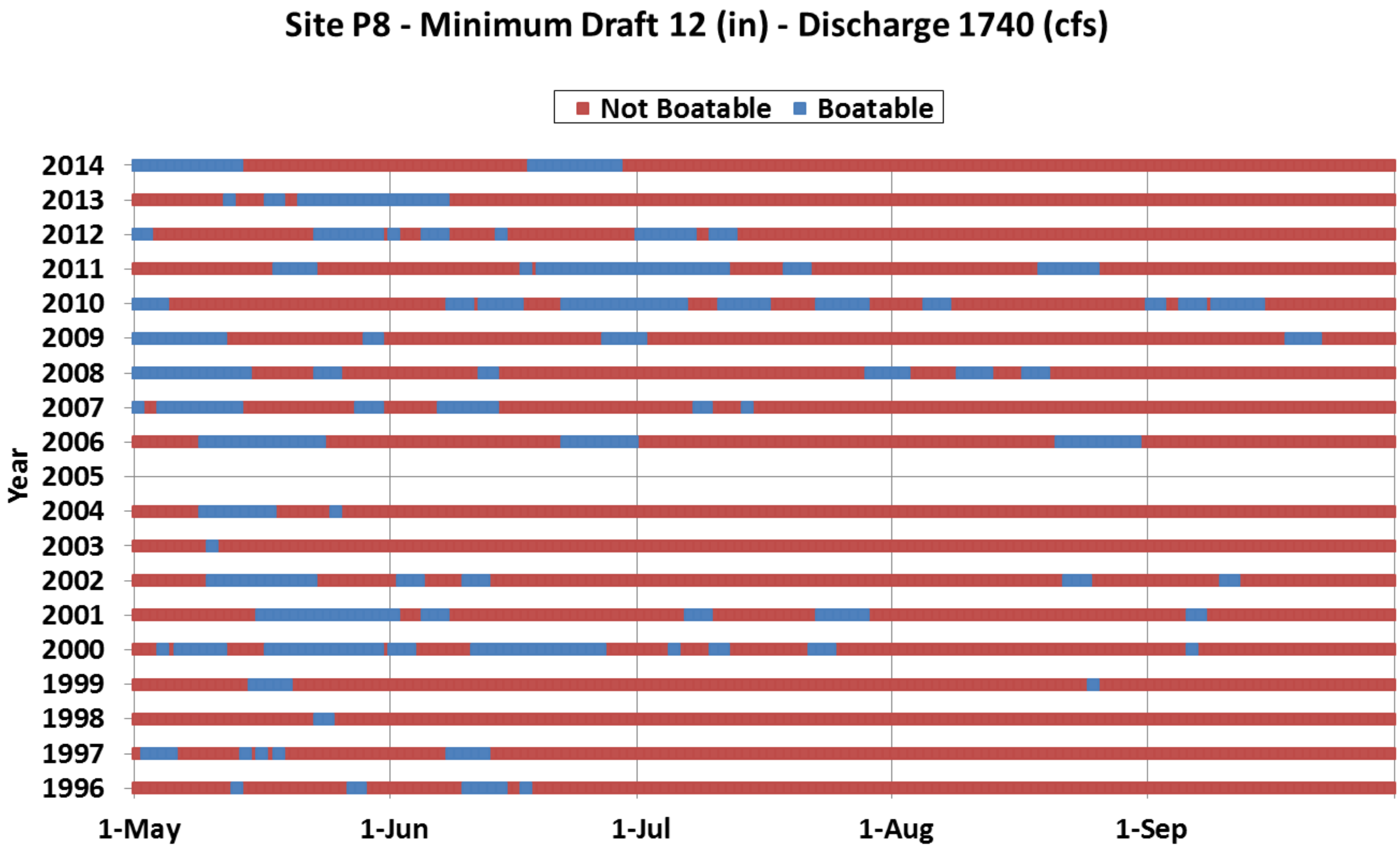
Site P9 - Minimum Draft 12 (in) - Discharge 770 (cfs)



Site P9 - Minimum Draft 8 (in) - Discharge 370 (cfs)

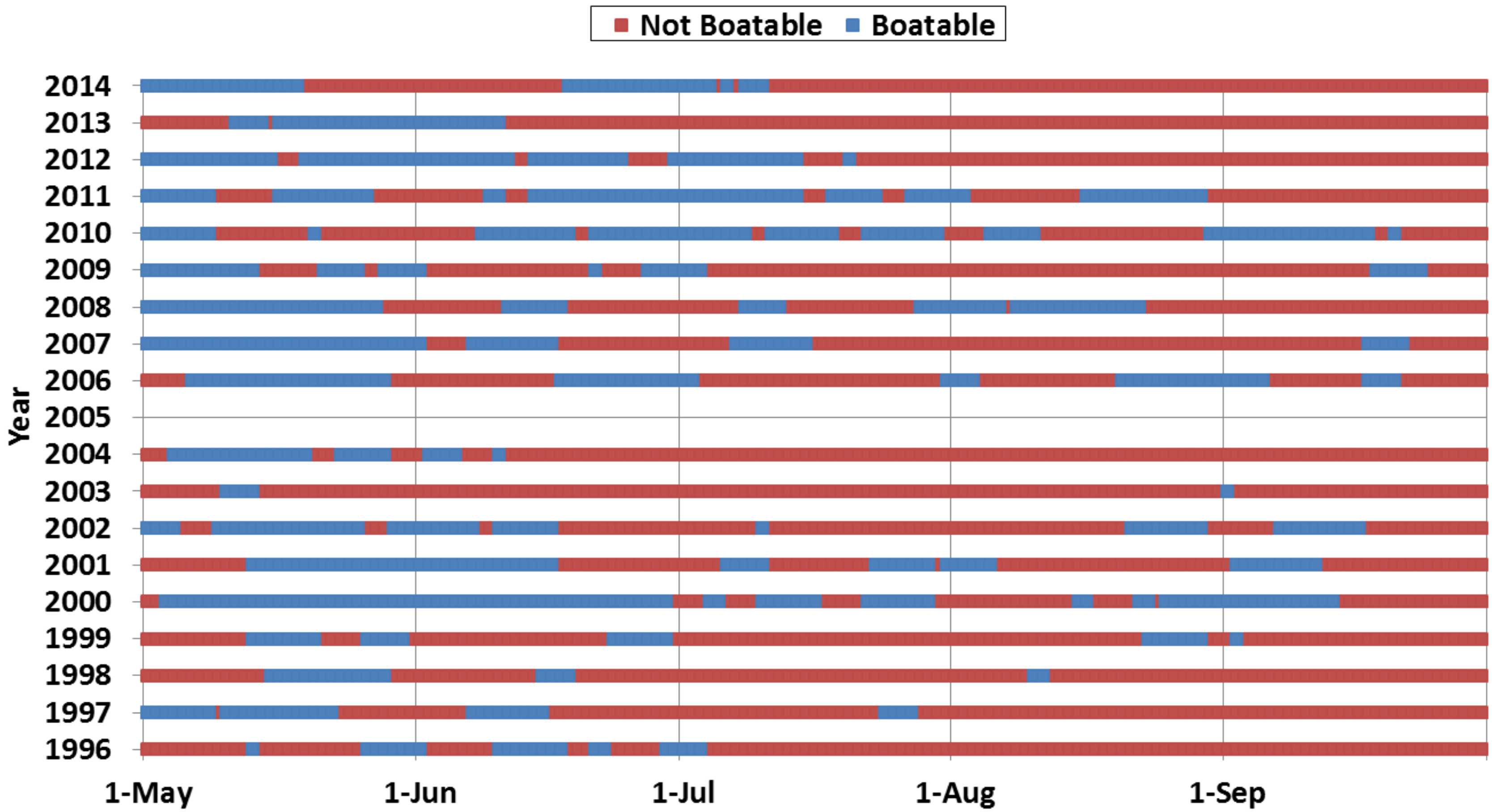


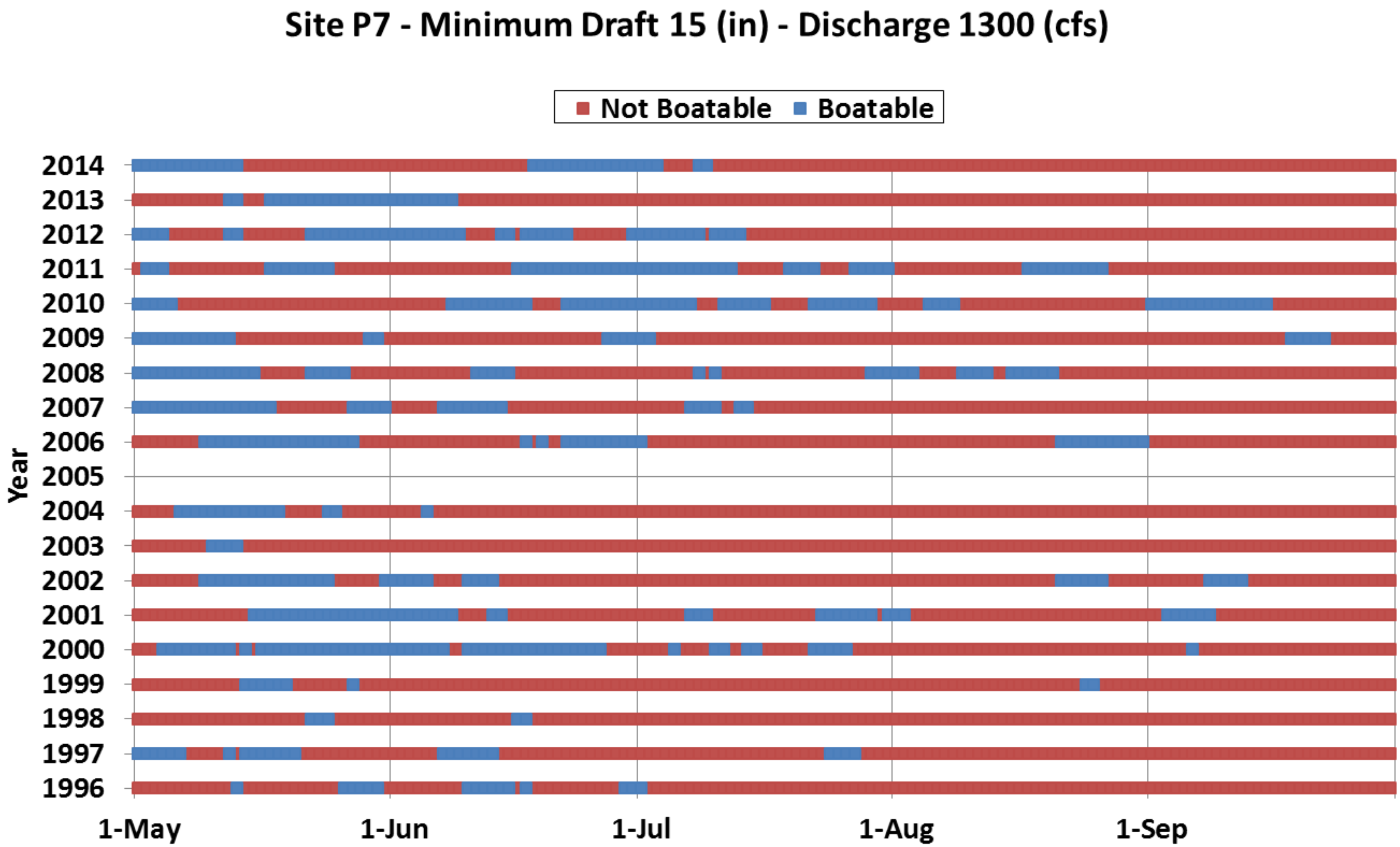

Site P8 - Minimum Draft 15 (in) - Discharge 2230 (cfs)
Not Boatable
Boatable
Year
2014
2013
2012
2011
2010
2009
2008
2007
2006
2005
2004
2003
2002
2001
2000
1999
1998
1997
1996
1-May
1-Jun
1-Jul
1-Aug
1-Sep

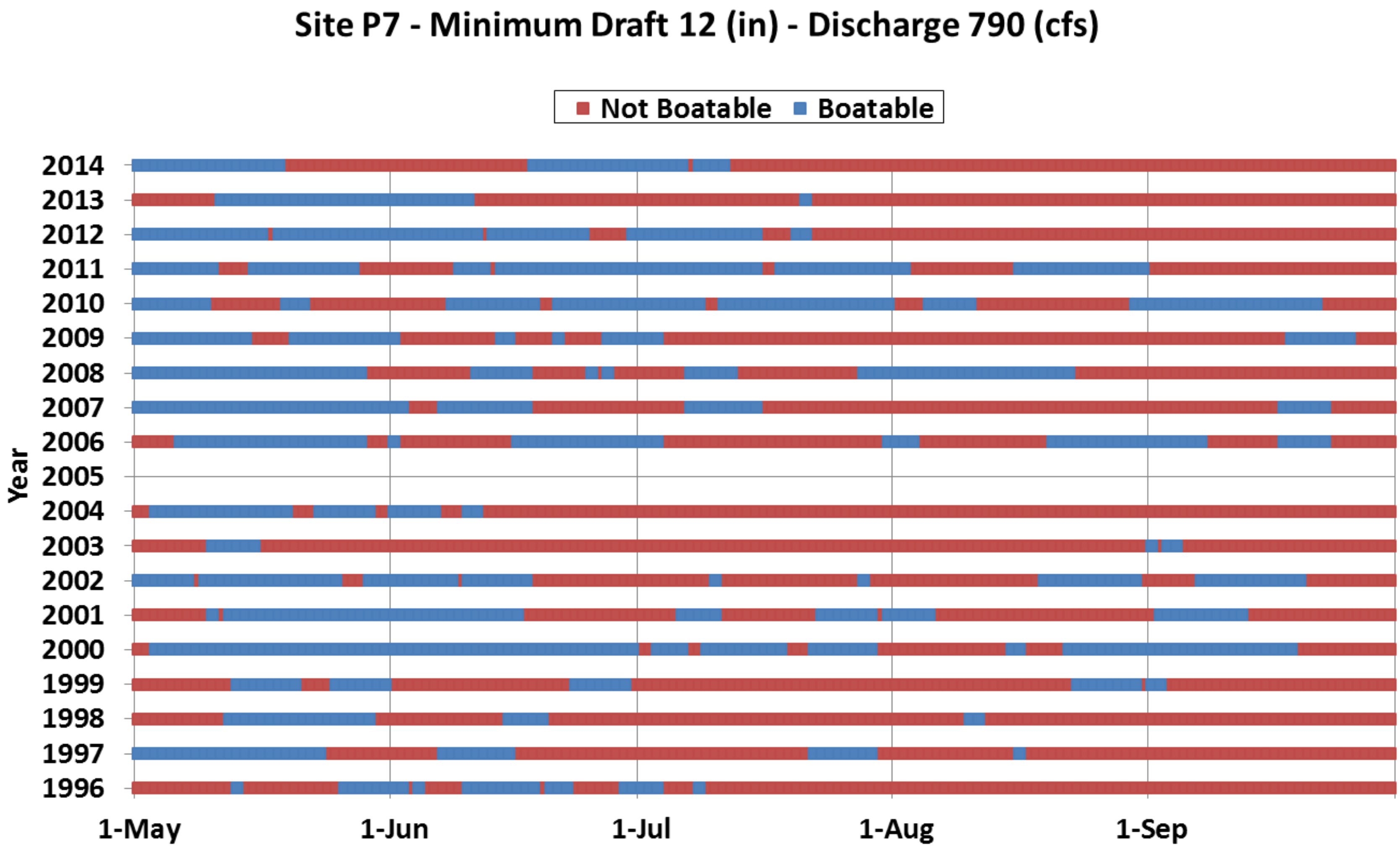


Site P8 - Minimum Draft 12 (in) - Discharge 1740 (cfs)
Not Boatable
Boatable
Year
2014
2013
2012
2011
2010
2009
2008
2007
2006
2005
2004
2003
2002
2001
2000
1999
1998
1997
1996
1-May
1-Jun
1-Jul
1-Aug
1-Sep

Site P8 - Minimum Draft 8 (in) - Discharge 950 (cfs)

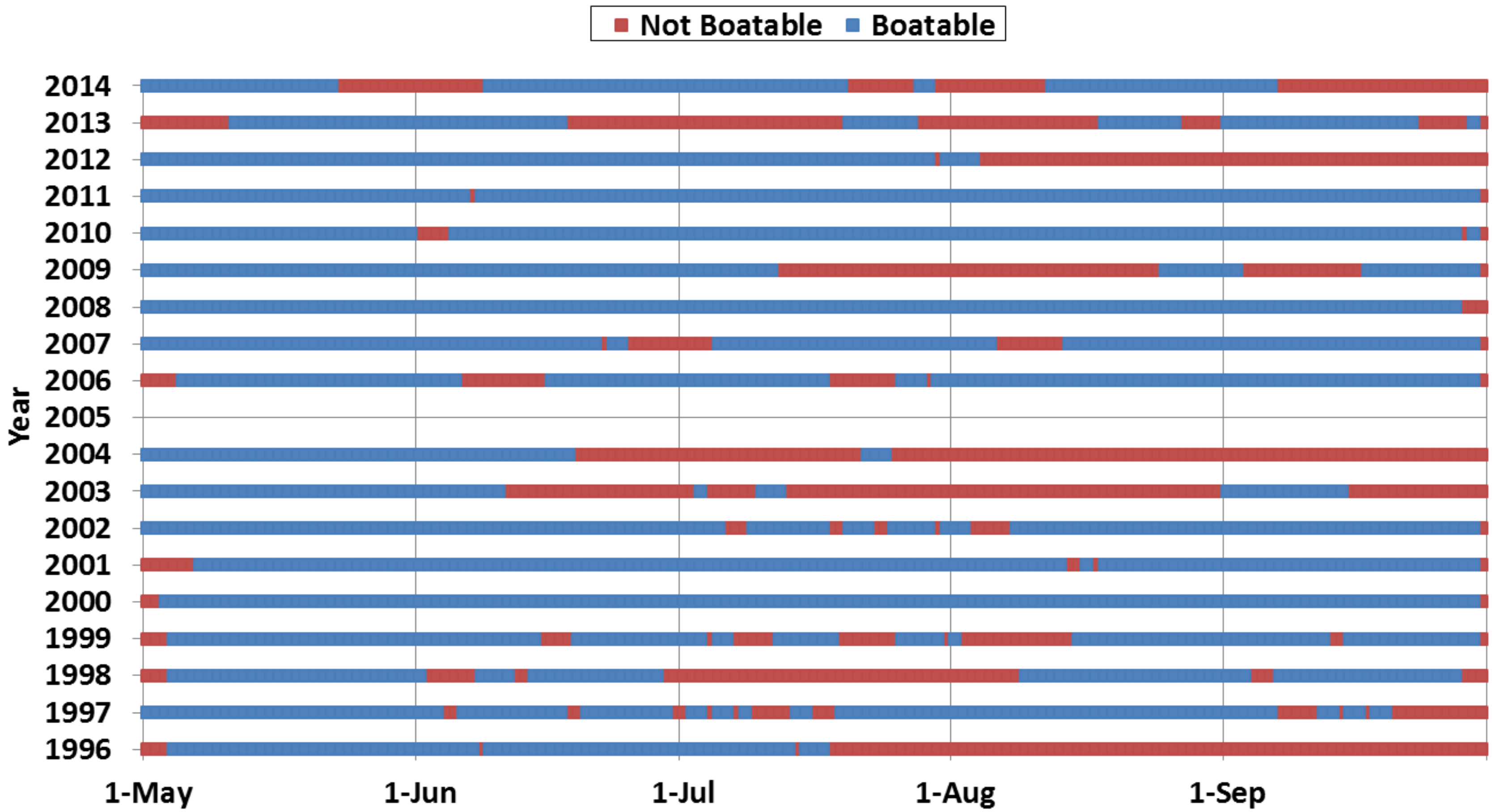


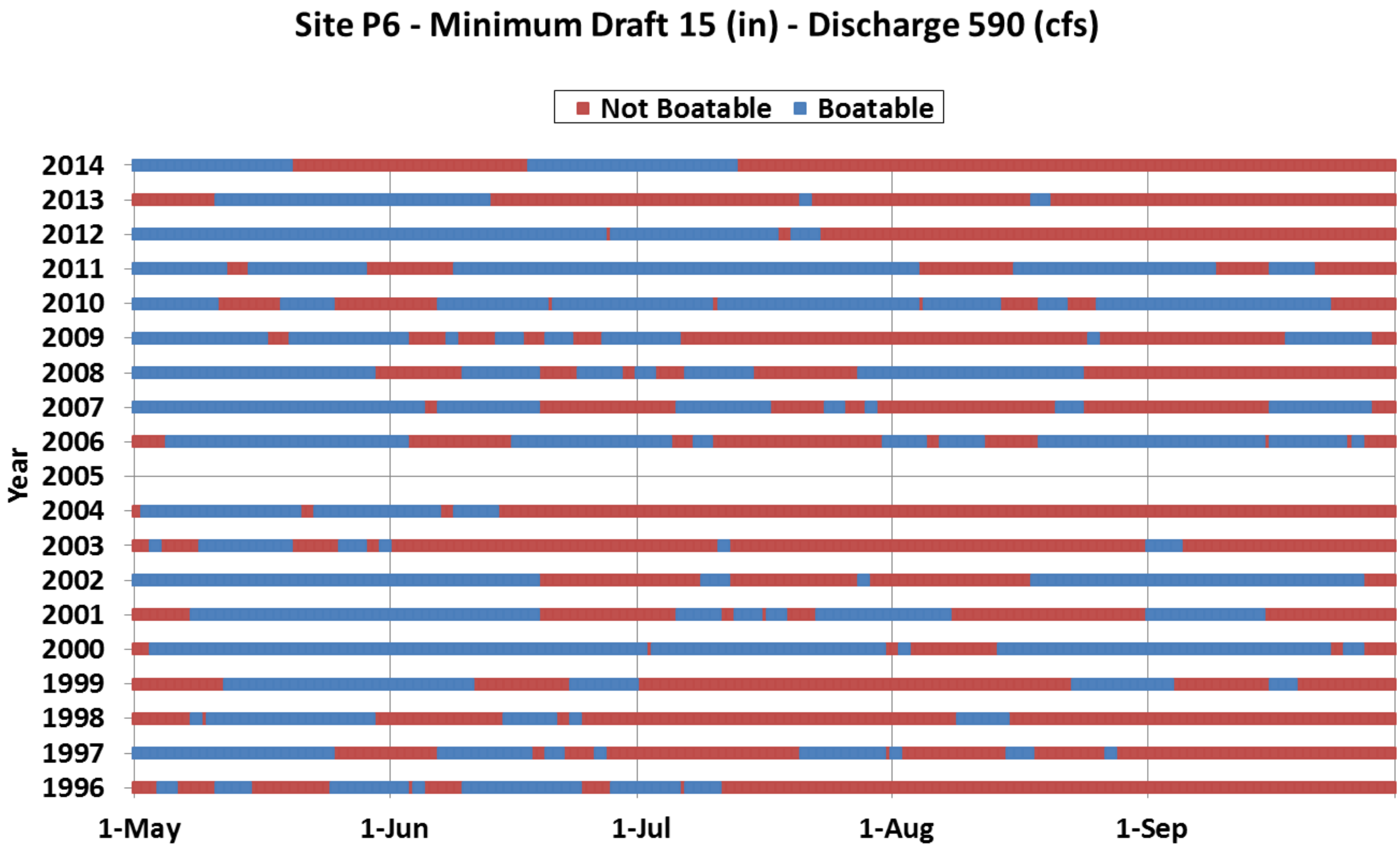

Site P7 - Minimum Draft 15 (in) - Discharge 1300 (cfs)
Not Boatable
Boatable
Year
2014
2013
2012
2011
2010
2009
2008
2007
2006
2005
2004
2003
2002
2001
2000
1999
1998
1997
1996
1-May
1-Jun
1-Jul
1-Aug
1-Sep

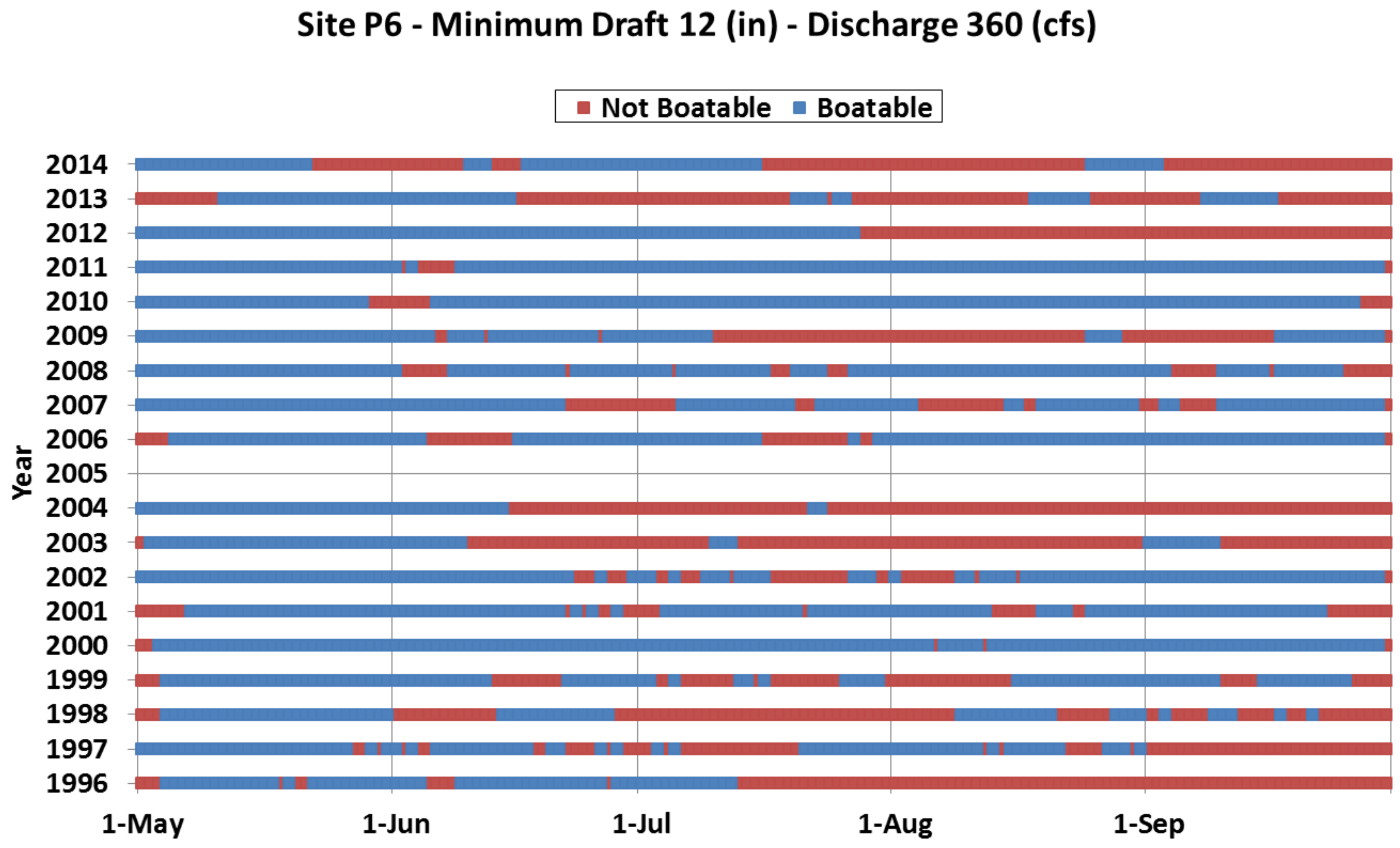


Site P7 - Minimum Draft 12 (in) - Discharge 790 (cfs)
Not Boatable
Boatable
Year
2014
2013
2012
2011
2010
2009
2008
2007
2006
2005
2004
2003
2002
2001
2000
1999
1998
1997
1996
1-May
1-Jun
1-Jul
1-Aug
1-Sep

Site P7 - Minimum Draft 8 (in) - Discharge 280 (cfs)

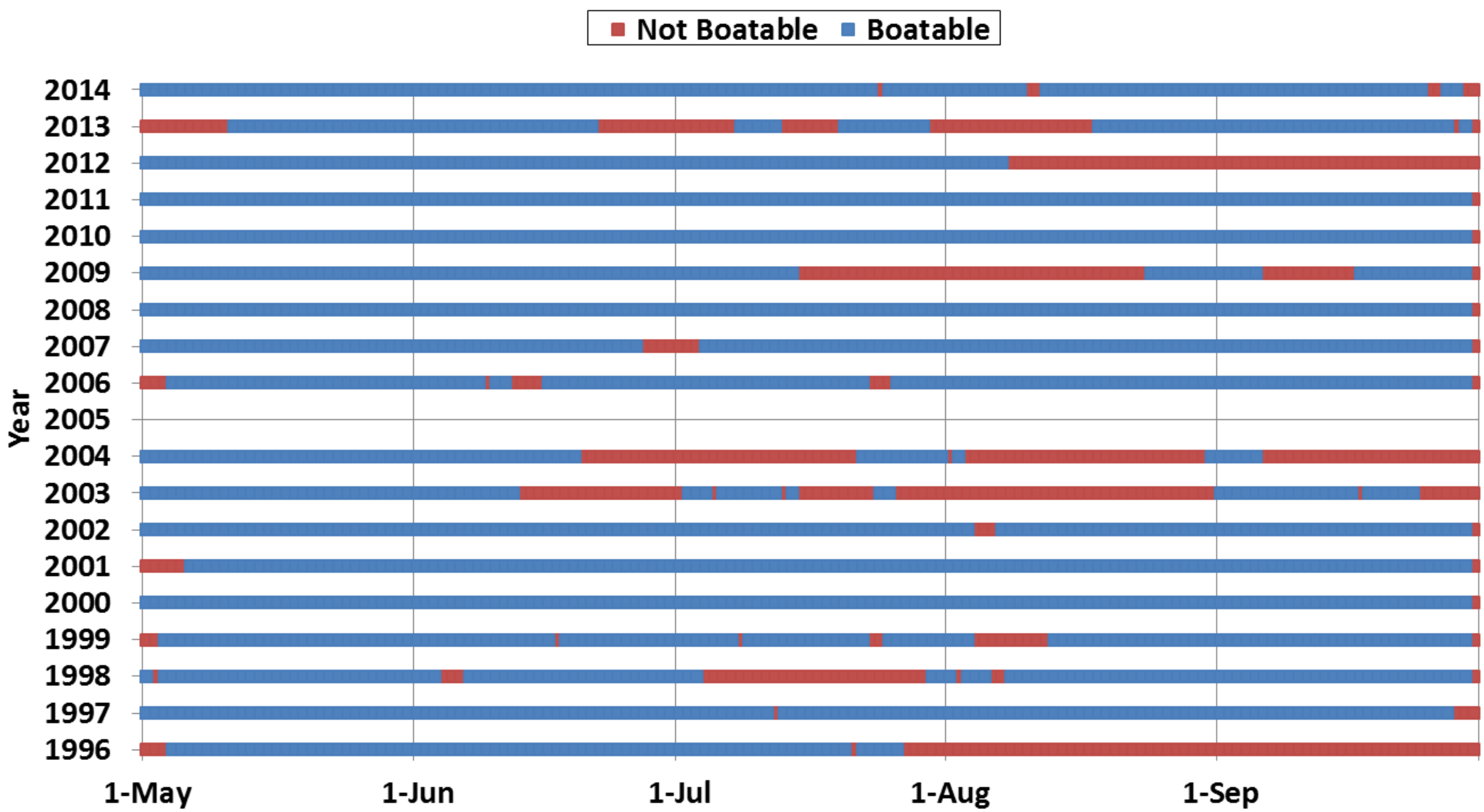


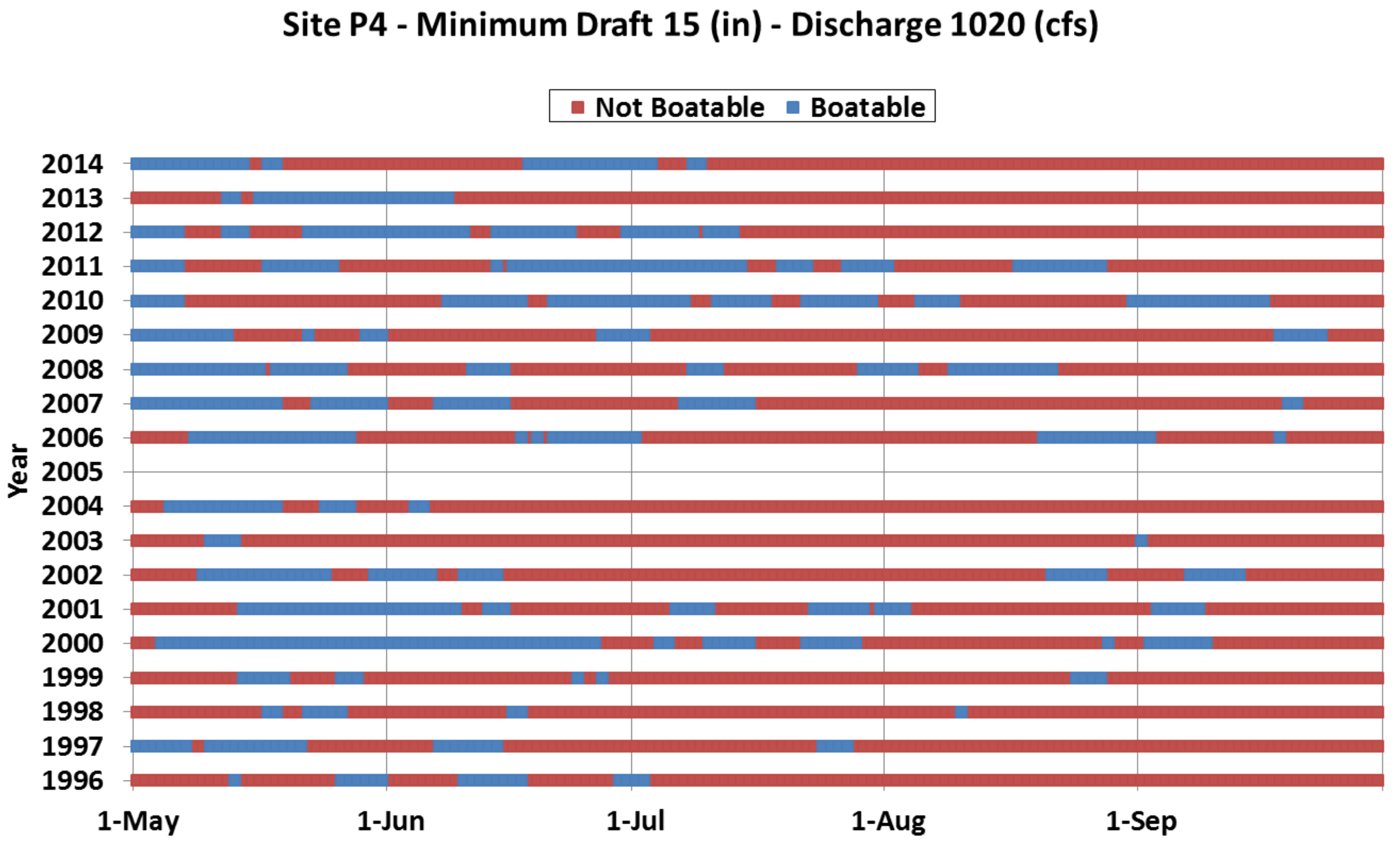

Site P6 - Minimum Draft 15 (in) - Discharge 590 (cfs)
Not Boatable
Boatable
Year
2014
2013
2012
2011
2010
2009
2008
2007
2006
2005
2004
2003
2002
2001
2000
1999
1998
1997
1996
1-May
1-Jun
1-Jul
1-Aug
1-Sep

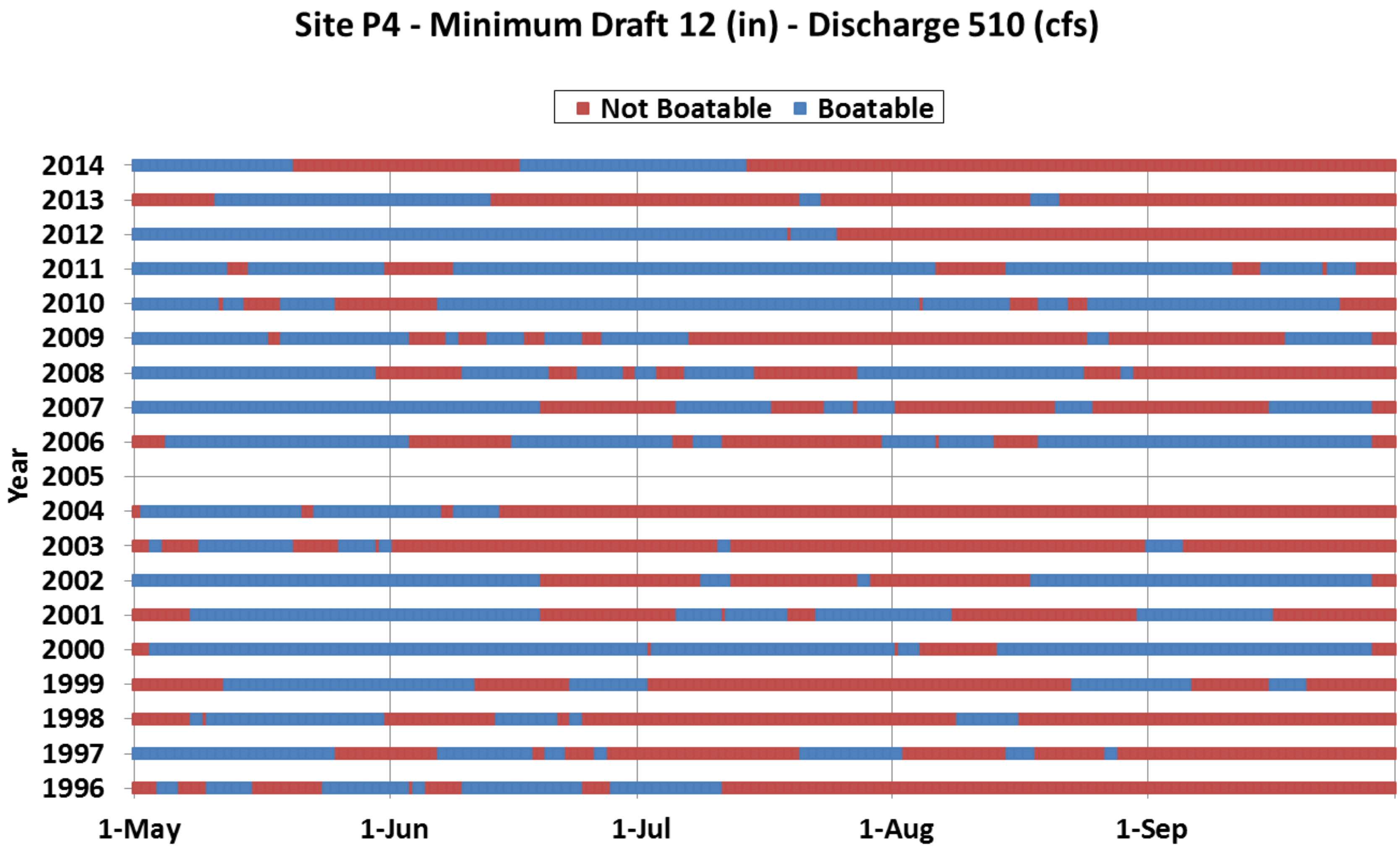


Site P6 - Minimum Draft 12 (in) - Discharge 360 (cfs)
Not Boatable
Boatable
Year
2014
2013
2012
2011
2010
2009
2008
2007
2006
2005
2004
2003
2002
2001
2000
1999
1998
1997
1996
1-May
1-Jun
1-Jul
1-Aug
1-Sep

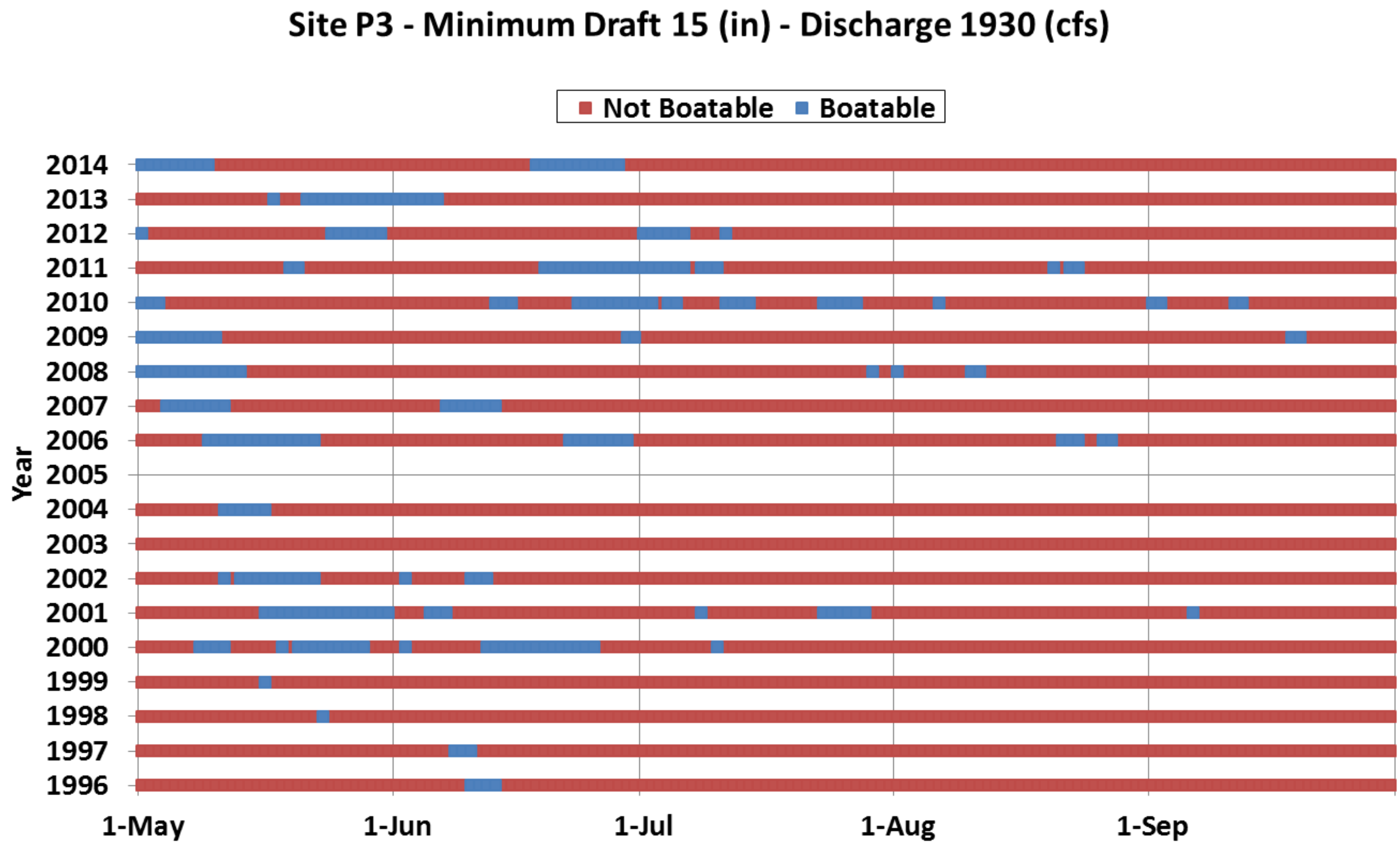
Site P6 - Minimum Draft 8 (in) - Discharge 180 (cfs)



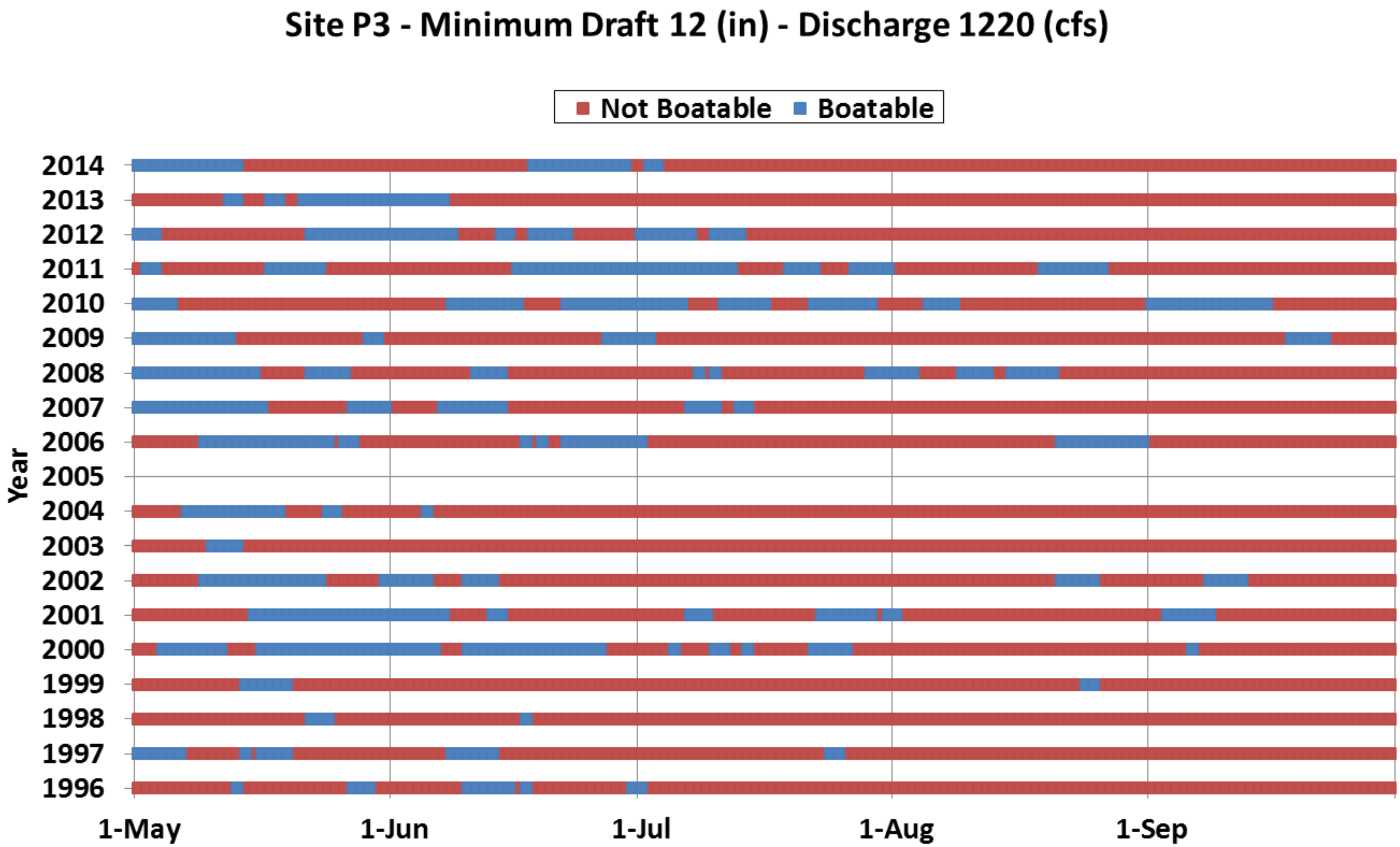

Site P4 - Minimum Draft 15 (in) - Discharge 1020 (cfs)
Not Boatable
Boatable
Year
2014
2013
2012
2011
2010
2009
2008
2007
2006
2005
2004
2003
2002
2001
2000
1999
1998
1997
1996
1-May
1-Jun
1-Jul
1-Aug
1-Sep


Site P4 - Minimum Draft 12 (in) - Discharge 510 (cfs)
Not Boatable
Boatable
Year
2014
2013
2012
2011
2010
2009
2008
2007
2006
2005
2004
2003
2002
2001
2000
1999
1998
1997
1996
1-May
1-Jun
1-Jul
1-Aug
1-Sep

Site P4 - Minimum Draft 8 (in) - Discharge 210 (cfs)
Not Boatable
Boatable
Year
2014
2013
2012
2011
2010
2009
2008
2007
2006
2005
2004
2003
2002
2001
2000
1999
1998
1997
1996
1-May
1-Jun
1-Jul
1-Aug
1-Sep



Bob Mussetter, Tetra Tech
December 18, 2014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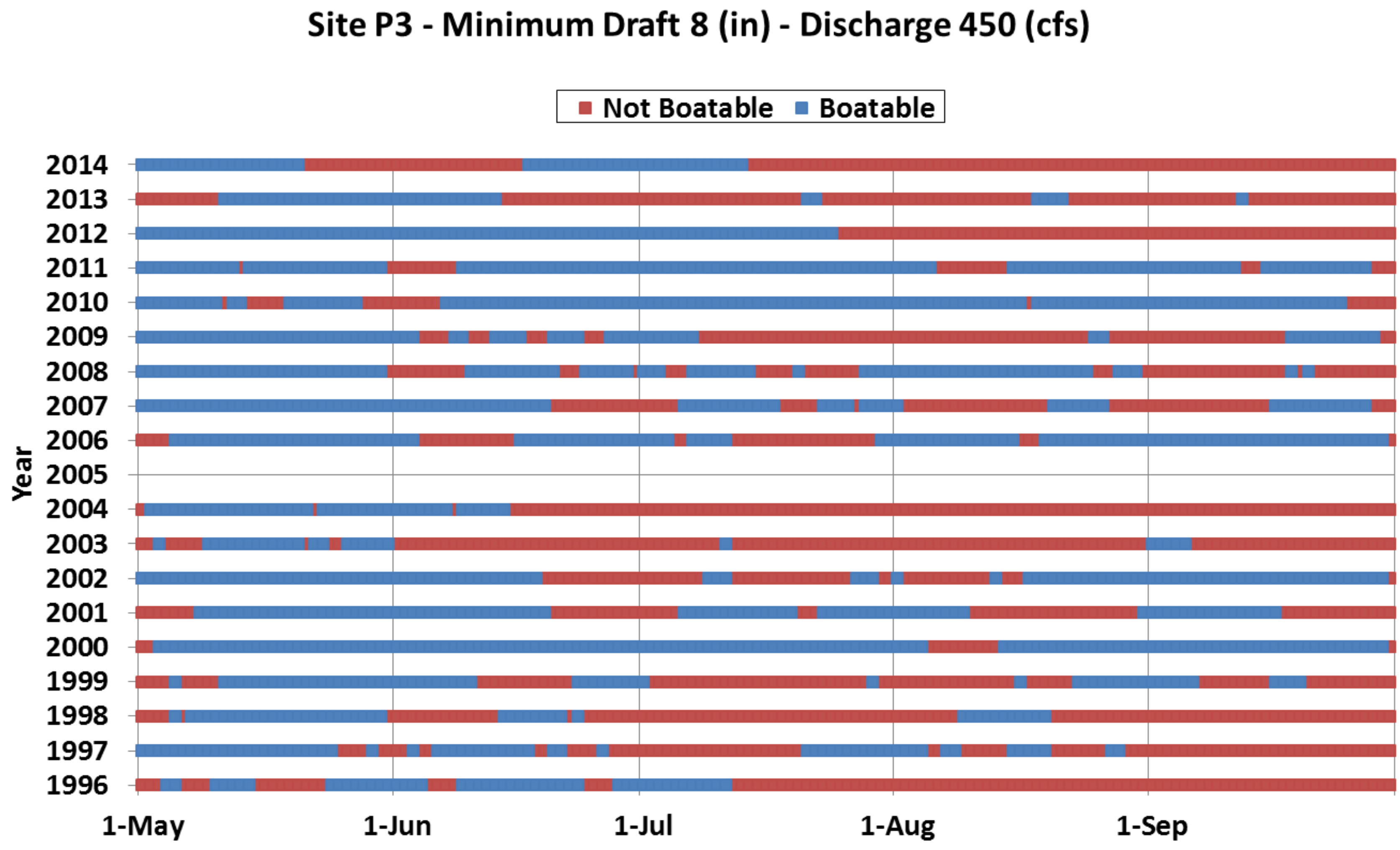

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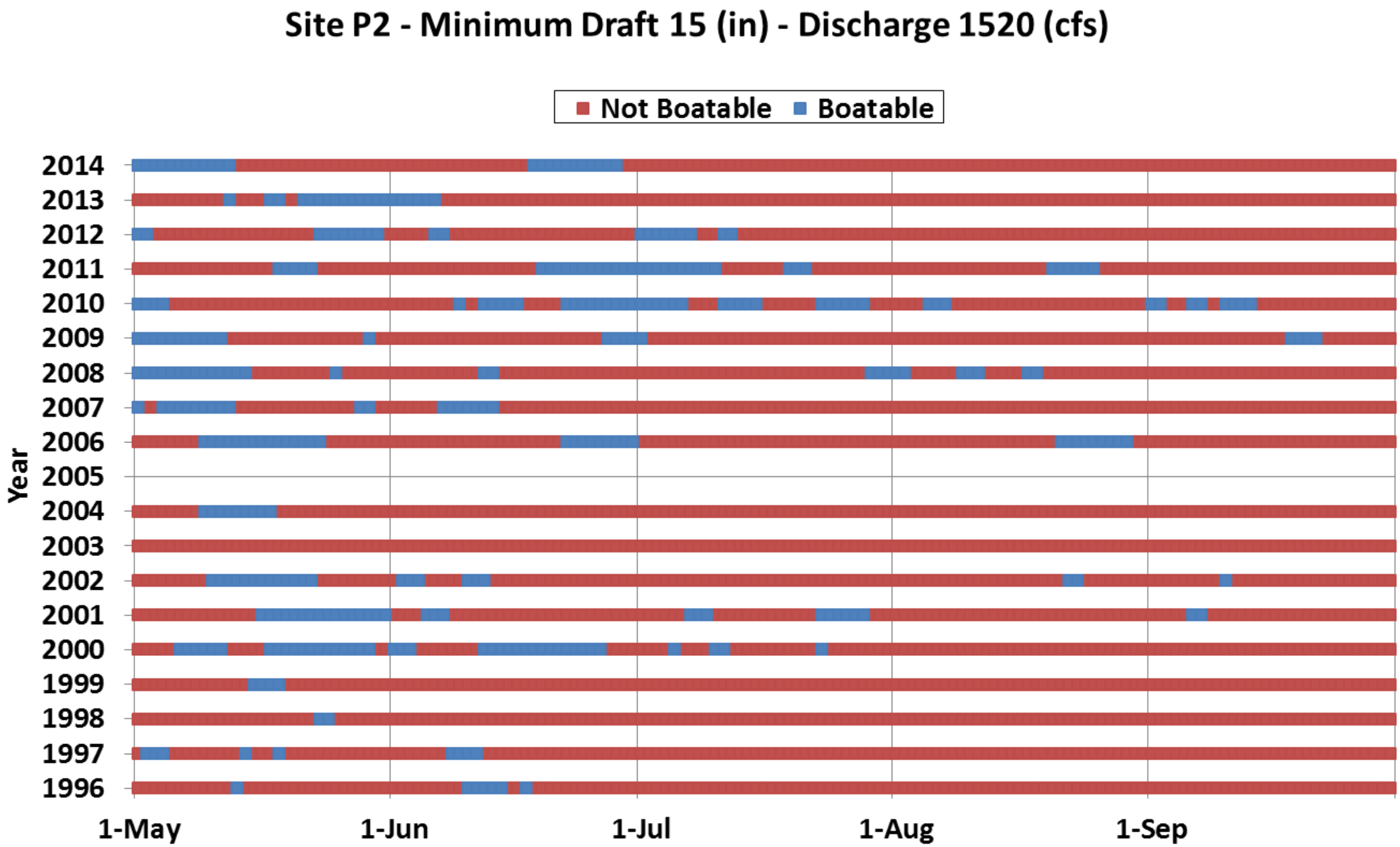

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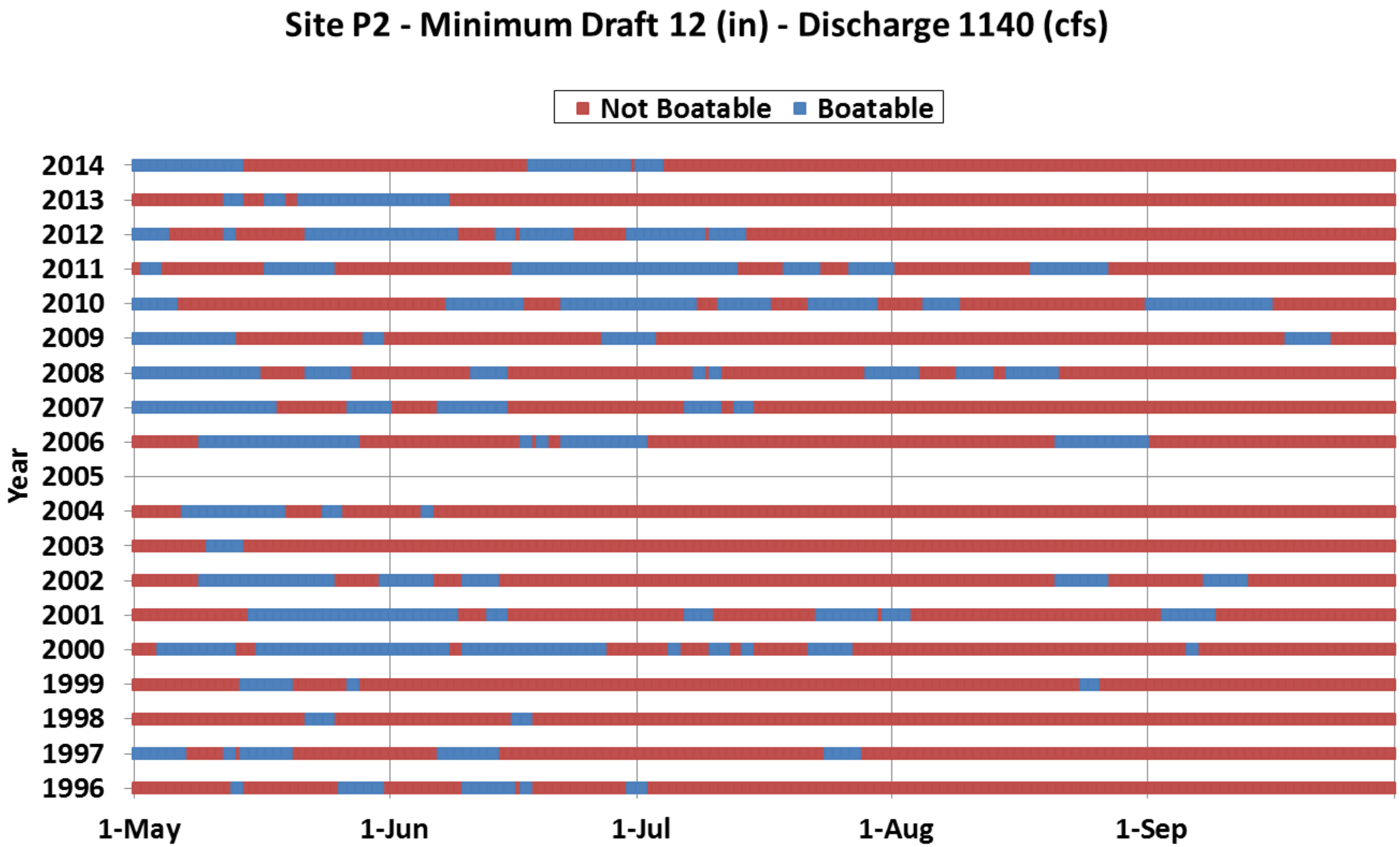
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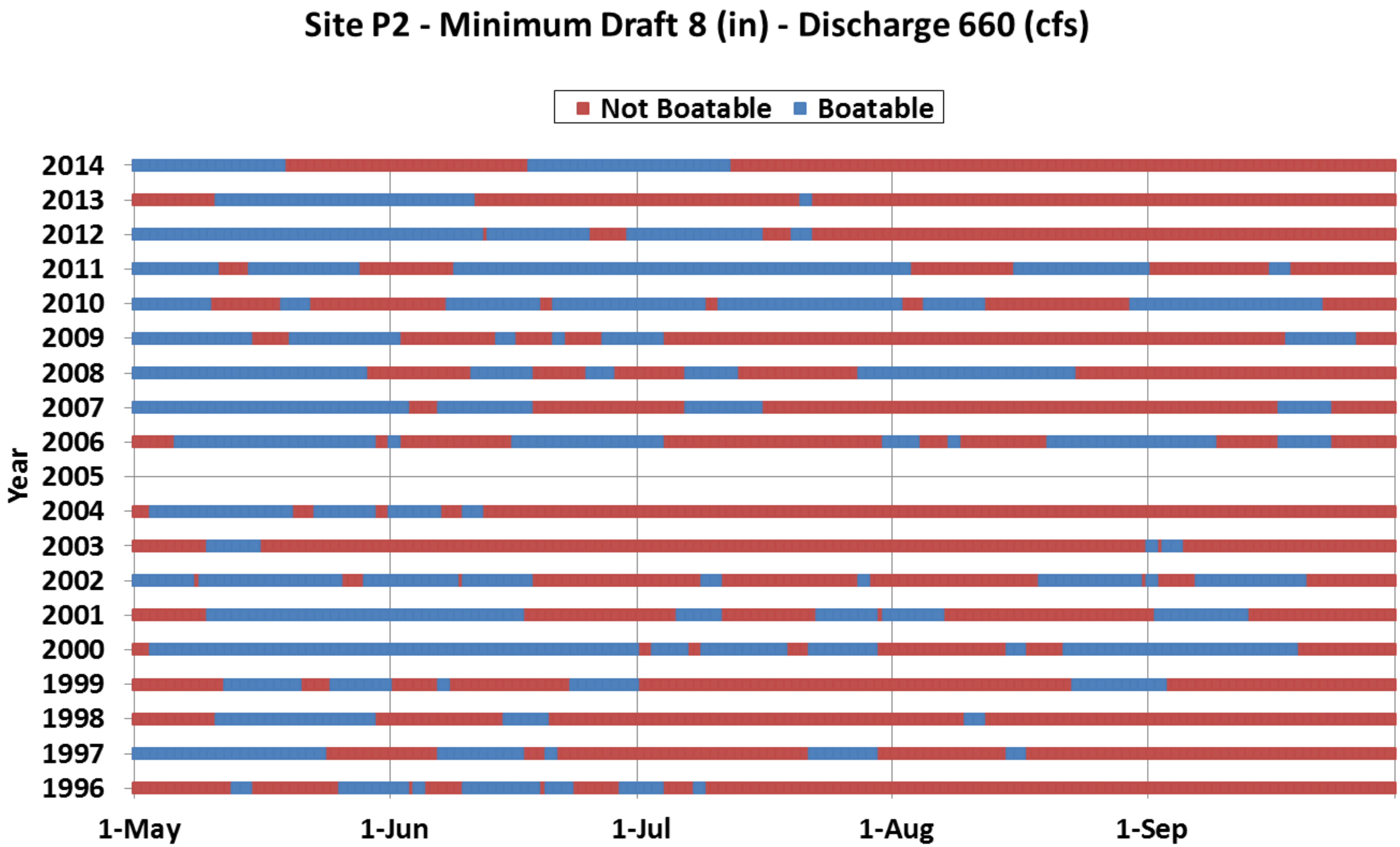
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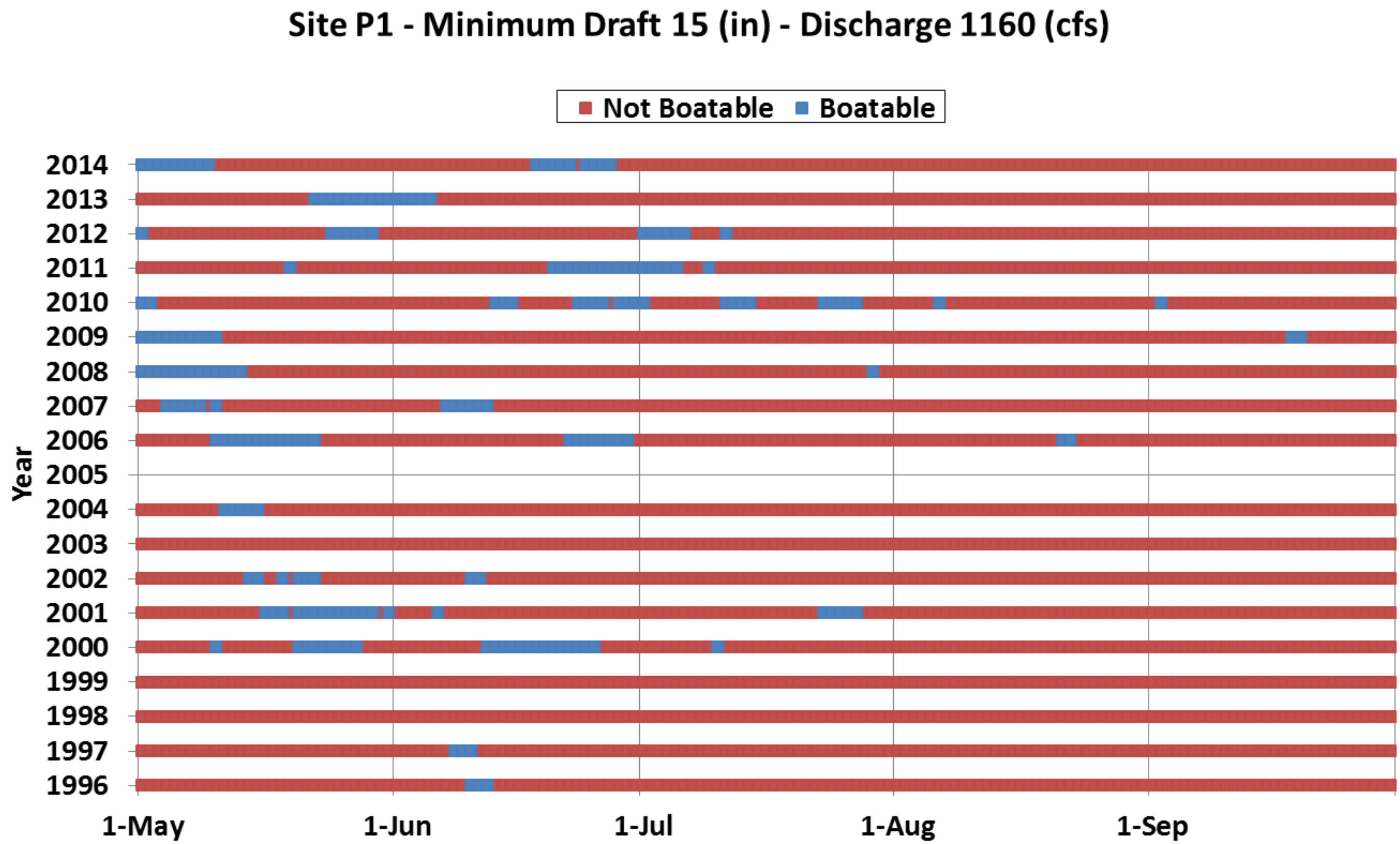
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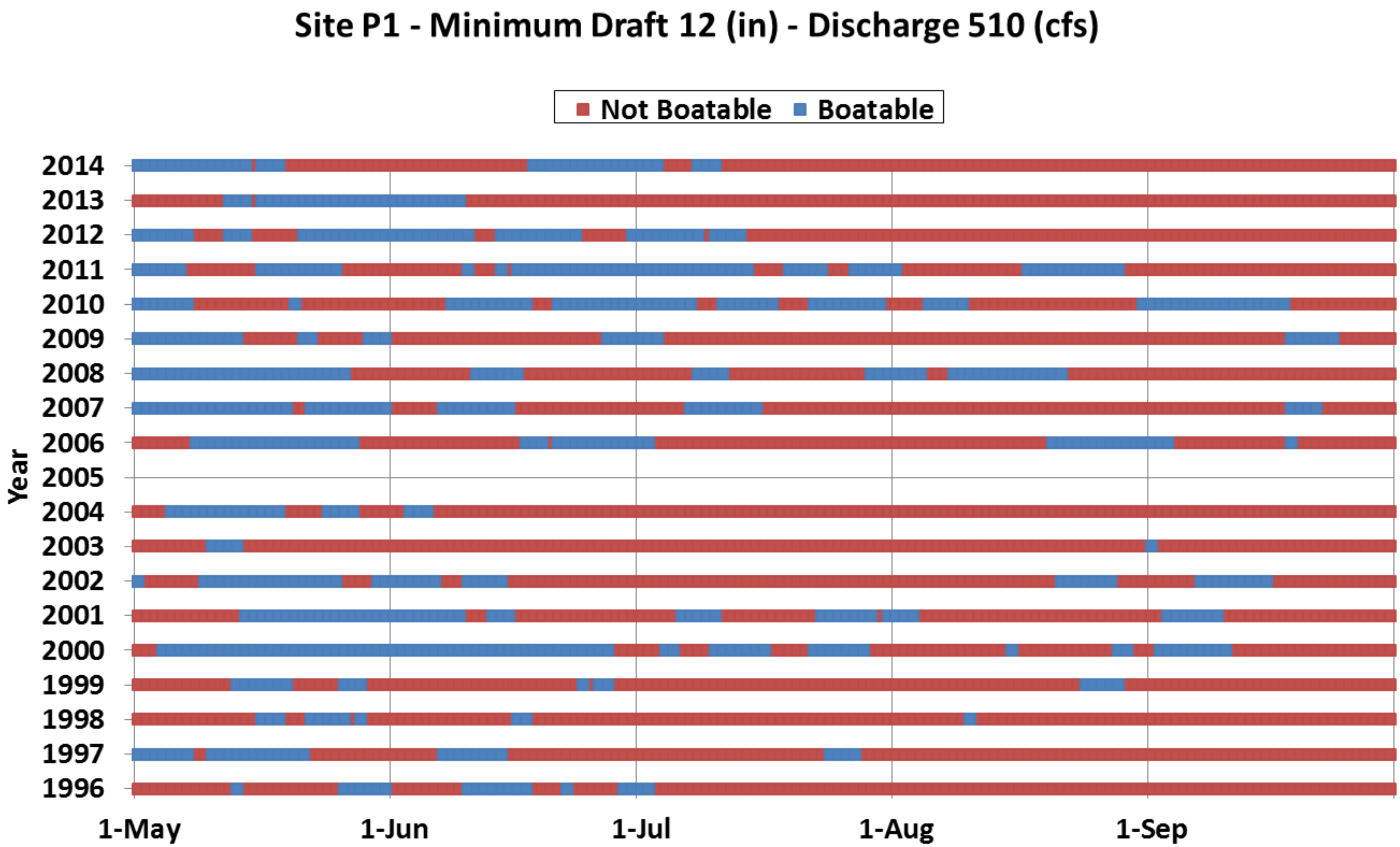

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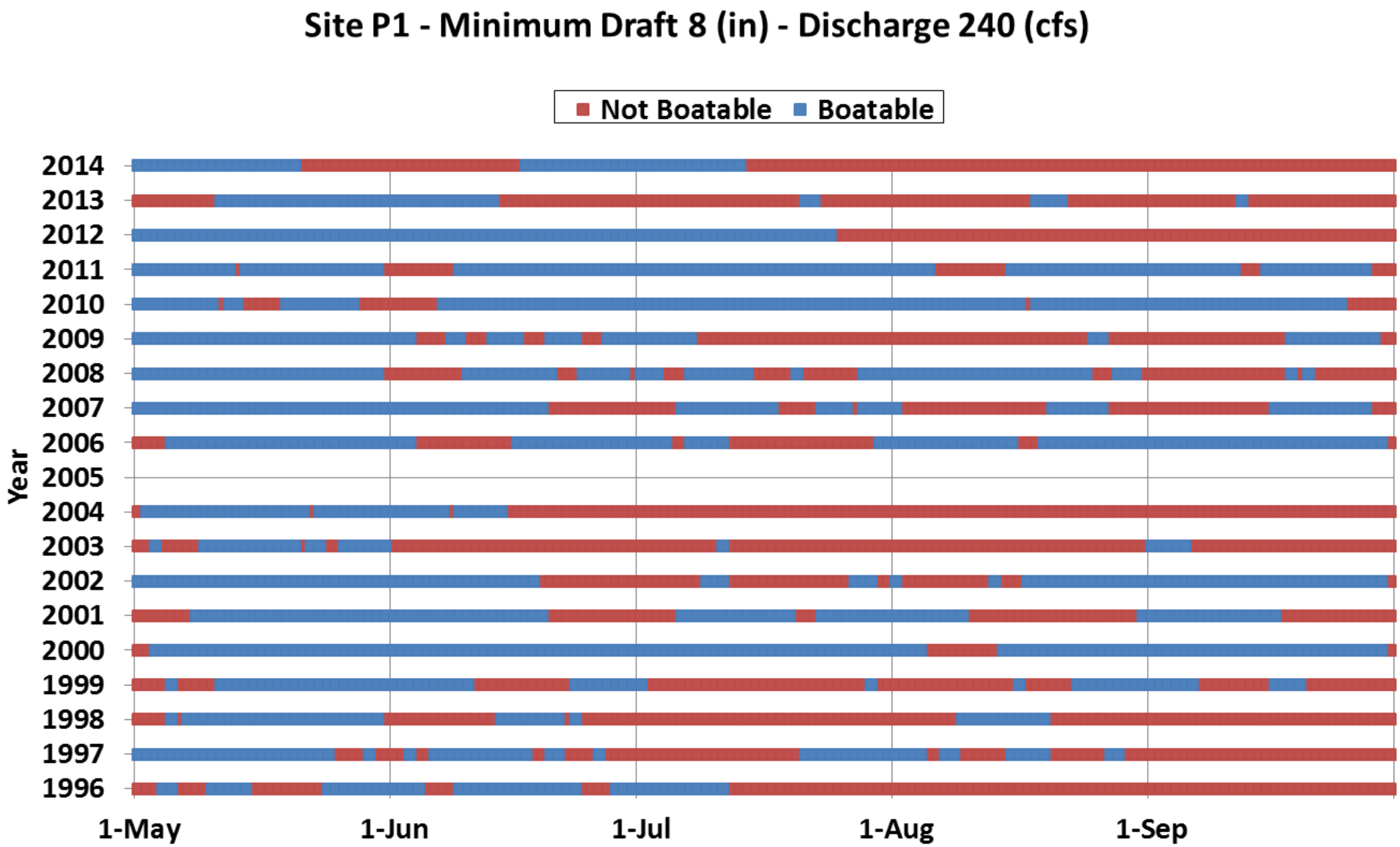
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APPENDIX E

Curriculum Vitae of Robert A. Mussetter

TETRA TECH

ROBERT A. MUSSETTER

POSITION: Program Manager and Discipline Lead
Tetra Tech, Inc.

EDUCATION:

1989 Ph.D. Civil (Hydraulic) Engineering
Colorado State University, Fort Collins, CO

1982 M.S. Civil (Hydraulic) Engineering
Colorado State University, Fort Collins, CO

1976 B.S. Civil Engineering
Montana State University, Bozeman, MT

PROFESSIONAL CAREER:

May 2009 – present	Program Manager and Discipline Lead, Tetra Tech, Inc.
Jan 1994 – May 2009	President and Principal Engineer, Mussetter Engineering, Inc.
Sept 1992 - Jan 1994	Vice President, Resource Consultants & Engineers, Inc.
Jan 1991 - Sept 1992	Principal, Resource Consultants, Inc.
Sept 1989 - Jan 1991	President, Mussetter Engineering, Inc.
Sept 1987 - Sept 1989	Associate and Manager of Fort Collins office, Simons, Li & Associates, Inc.
Sept 1986 - Sept 1987	Senior Engineer and Manager of Fort Collins office, Simons, Li & Associates, Inc.
Mar 1984 - Sept 1986	Senior Engineer and Project Manager, Simons, Li & Associates, Inc.
Dec 1981 - Mar 1984	Hydraulic Engineer, Simons, Li & Associates, Inc.
Aug 1980 - Dec 1981	Research Assistant, Colorado State University, Fort Collins, CO.
Jun 1976 - Aug 1980	Officer, U.S. Army Corps of Engineers, Fort Eustis, VA (Platoon leader, facility engineer, highest grade achieved: Captain).

PROFESSIONAL SOCIETIES AND HONORS:

Diplomate, Water Resource Engineer, American Academy of Water Resources Engineers
American Society of Civil Engineers
American Geophysical Union
American Water Resources Association

TETRA TECH

REGISTRATION: Registered Professional Engineer:

Arizona (1985) #17918
California (1999) #59128
Colorado (1983) #20758
Idaho (1997) #8809
Louisiana (2006) #32687
Montana (1984) #4803-PE
New Mexico (1995) #12603
South Dakota (1995) #6001
Texas (2001) #89604
Wisconsin (2005) #37449

COMMITTEES AND OTHER AFFILIATIONS:

Faculty Affiliate	Colorado State University, Civil Engineering, Department
Member	American Society of Civil Engineers, Urban Erosion Technical Committee
Member	Federal Emergency Management Agency, Riverine Erosion Hazard Area Project Working Group

REPRESENTATIVE PROJECT EXPERIENCE (LITIGATION):

Alaska v. United States et al., Civil Action No: 3:12-CV-00114-SLG, Navigability of Mosquito Fork River (ongoing).

Adjudication of All Rights to Use Water in the Gila River System and Source, Superior Court of Arizona and County of Maricopa, Contested Case No. W1-11-3342, Aravaipa Canyon Wilderness Area (Expert report 2013 and deposition 2014).

Navigability of the Upper Salt River between the Confluence of the Black and White Rivers and the Confluence with the Gila River, Expert Report (2014) and Testimony (anticipated for December 2014) before Arizona Navigable Streams Adjudication Commission (anticipated for early 2015).

Navigability of the Verde River between the Sullivan Lake and the Confluence with the Salt River, Expert Report (2014) and Testimony (anticipated for December 2014) before Arizona Navigable Streams Adjudication Commission (anticipated for December 2014).

Navigability of the Gila River Between the Arizona- New Mexico Stateline and the Confluence with the Colorado River, Expert Report and Testimony before Arizona Navigable Streams Adjudication Commission (2014).

Allman et al. v. Grand River Dam Authority, Case No. CJ-2001-381. Flooding Impacts along the Neosho River Associated with the Operation of Grand Lake and Other Factors, City of Miami, Oklahoma (Expert report and deposition, 2008).

Application for Approval of Absolute Groundwater Rights of the United States of America, Case No. 04CW35. District Court, Water Division 3, Colorado (litigation support, deposition, 2007)

United States v. ASARCO Inc. et al., Evaluation of the Coeur d'Alene and South Fork Coeur d'Alene Rivers for Mine Tailings Impacts. Case No. CV 96-0122-N-EJL (litigation support and deposition, 2004)

U.S. District Court of Kansas, Jacqueline Seyler v. Burlington Northern Santa Fe Corporation and AMTRAK – Case No. 99-2342-KHV, Kansas (deposition, 2002)

Superior Court of Arizona, County of Mariposa, the Burlington Northern and Santa Fe Railway Company v. The State of Arizona et al.—Case No. CV98-14172, Arizona (deposition, 2002)

Souza Property Flooding vs City of Fort Collins, Colorado (litigation support, expert testimony at trial, 2001)

Age of Islands in the Snake River Sector of the Deer Flat National Wildlife Refuge in State of Idaho v. United States of America, United States District Court for the District of Idaho, Case No. CIV97-0426-S-BLW, (expert report, litigation support, 1997-2001)

Evaluation of Stream Channel Processes and Water Rights Claims for Channel Maintenance Flows by the U.S. Forest Service and U.S. Fish and Wildlife Service and Various Indian Tribes Throughout the Snake River Basin, Idaho (litigation support and affidavits, 1994-2001)

Evaluation of instream flows for channel maintenance purposes for National Forest streams in Colorado (extensive litigation support, affidavits, deposition and expert testimony at trial.) (1986-2000)

Evaluation of the Roughness Characteristics of the Neosho River, Associated with Backwater from Grand Lake of the Cherokees, Miami, Oklahoma (litigation support, expert testimony, 1998)

REPRESENTATIVE PROJECT EXPERIENCE (CONSULTING):

Principal Engineer for analysis and design to support the San Joaquin River Restoration Project, California.

Principal Engineer for Hydraulic and Sediment-transport Analysis of the Carmel River Bypass Option, California. Submitted to California American Water, Monterey, California.

Principal Engineer and author of Feasibility Study for Alternatives to Mitigate Flooding Effects on Boxelder Creek. Submitted to Land Acquisition and Management, Centennial, Colorado.

Principal Engineer and author of Preliminary Assessment of Boat Wakes on Beach and Shoreline Erosion in the Hells Canyon Reach of the Snake River. Submitted to Davis Wright Tremaine, LLP, Washington, DC.

Principal Engineer and author of FLO-2D Development, Albuquerque Reach, Rio Grande. Submitted to U.S. Army Corps of Engineers, Albuquerque District.

Principal Engineer and author of Bathymetric Survey of Eagle Nest Lake, New Mexico. Submitted to Bohannan-Huston, Inc., Albuquerque, New Mexico.

Principal Engineer and author of Skunk Creek Channel Stabilization Recommendations and Preliminary Design. Submitted to the City of Sioux Falls, South Dakota, Sioux Falls, South Dakota.

Principal Engineer and author of San Clemente Reservoir and Carmel River Sediment-Transport Modeling to Evaluate Potential Impacts of Dam Retrofit Options. Submitted to American Water Works Service Company, Voohees, New Jersey.

Principal Engineer and author of Carmel River Sediment-Transport Study, Monterey County, California. Submitted to California Department of Water Resources, Fresno, California.

Principal Engineer and author of Indian Bar Sediment Disposal Site Study, Ralston Afterbay, California. Submitted to Jones & Stokes, Sacramento, California, and Placer County Water Agency, Foresthill, California.

Principal Engineer and author of Hydraulic and Channel Stability Analysis, and Design of Erosion Control Plan for Auburn Ravine between Highway 65 and Highway 193, Lincoln, California. Submitted to Kleinfelder, Inc., Sacramento, California.

Principal Engineer and author of Hydrologic, Hydraulic, Erosion and Sediment Transport Analysis, Rummel Creek Watershed Studies, Texas. Submitted to SWA Group, Houston, Texas.

Author of an "Erosion and Sediment Design Guide" for the Albuquerque Metropolitan Arroyo Flood Control Authority for use by public agencies and consultants in designing flood control and erosion protection measures in urbanized areas of the Southwestern U.S.

Analysis of channel stability and determination of an erosion risk line (prudent line) along Calabacillas Arroyo, Albuquerque, New Mexico.

Geomorphic and sediment yield analysis for arroyos feeding to the North Diversion channel, Albuquerque, New Mexico to support a sedimentation study being performed by Waterways Experiment Station.

Instructor for the National Highway Institute's "Stream Stability and Scour at Highway Structures" training course (presented the course nine times, to-date).

Design of the Standard Project Flood channelization for a 2-mile reach of the Agua Fria River, Arizona.

Preparation of Master Drainageway Plan for Cache la Poudre River corridor in Fort Collins, Colorado.

Evaluation of flooding impacts on the Neosho River associated with backwater from Grand Lake of the Cherokees, Miami, Oklahoma (litigation support).

Development of a reclamation plan to restore approximately two miles of Whitewood Creek near Deadwood, South Dakota. The project reach has been heavily impacted by the discharge of mine tailings and subsequent placer mining of the deposits.

Evaluation of the impacts to stream channel stability in the Uncompaghre River between Montrose and Delta, Colorado (□35 miles) of a proposed hydropower operation.

Evaluation of the channel stability and flooding impacts along the Genesee River near Rochester, New York associated with subsidence of the river valley caused by underground salt mining.

Evaluation of stream channel processes and water rights claims for channel maintenance flows by the U.S. Forest Service and various Indian Tribes throughout the Snake River Basin in Idaho (litigation support).

Sediment engineering and channel stability analysis of the Feather and Yuba Rivers, California between the Sutter Bypass and Daguerre Point Dam (□35 miles) to evaluate potential impacts to lateral and vertical stability associated with increased in-levee capacity.

Sediment engineering and channel stability analysis of the Lower American River, California between Sacramento River confluence and Folsom Dam (□23 miles) to evaluate potential impacts to levees, bank protection and riparian habitat associated with various operation scenarios for Folsom Dam.

Sediment engineering and channel stability analysis of the Middle Fork American River, California to evaluate to distribution of sediment deposits upstream of the proposed Auburn Dry Dam and the potential for coarse sediment entrainment through the Dry Dam sluices.

Assessment of cobble bar dynamics and sediment movement in the North Fork Feather River, California to evaluate the potential impacts to fish and riparian habitat associated with sediment-pass-through operations from hydropower facilities.

Evaluation of critical spawning habitat for endangered fishes in the Upper Colorado River basin, including studies on the Yampa River in Dinosaur National Monument, Green River in Desolation and Gray Canyons and the Mineral Bottoms area, and the Colorado River near Grand Junction, Colorado.

Evaluation of instream flows for channel maintenance purposes for National Forest streams in Colorado (extensive litigation support, including expert testimony at trial.)

Evaluation of channel maintenance flows on North Boulder Creek, Colorado to support an Environmental Report on the potential impacts of rehabilitation of the Lakewood Pipeline.

Channel stability analysis and design of environmentally sensitive channel protection measures for Hoop Creek, a steep mountain stream draining the south side of Berthoud Pass, Colorado.

Channel stability, floodplain analysis and hydraulic analysis to support design of a greenbelt/community park along the Laramie River, Laramie, Wyoming.

Geomorphic, hydraulic, and sedimentation analysis to evaluate post-mining stability and develop environmentally sensitive channel protection measures for Whitewood Creek, Lead, South Dakota.

Design of a river stabilization plan and assessment of environmental impacts for an approximately 6-mile reach of the Elkhorn River, Nebraska.

Geomorphic, hydraulic, and sedimentation analysis of Cache Creek, Yolo County, California.

Analysis of bed degradation and channel stability on the lower Kansas River, Kansas.

Analysis of failure of a natural gas line crossing the Truckee River near Carson City, Nevada (litigation support).

Evaluation of the relationship between stream channel processes and the growth of riparian vegetation in Bishop Creek, California.

Sedimentation study to evaluate the impacts on navigation resulting from construction of locks and dams and channel meander cutoffs on the Red River between Shreveport, Louisiana, and Index,

Arkansas. Geomorphic, sediment engineering and channel stability analysis of the American River, California.

Qualitative and quantitative analysis and development of erosion- and flood-control plan for Upper Hickahala and Senatobia Creek Watershed, Mississippi.

Analysis of reservoir operations at Lahontan Dam, Nevada to determine causes of erosion damage at a construction site (litigation support).

Hydraulic and sediment routing study to determine the impacts of dredging and channelization of tributaries on the mainstem Yazoo River, Mississippi.

Analysis of baseline conditions and development of pollution cleanup plans for Erjen Creek in southwestern Taiwan.

PUBLICATIONS AND LECTURES:

Mussetter, R.A., 2013. Fire Effects on Runoff and Sediment Delivery from Rio Grande Watersheds, presentation to Rio Grande Basin Meeting, Albuquerque, September 17.

Harvey, M.D., Mussetter, R.A., Haggerty, G.M., Lundahl, A., and Pegram, P., 2012. Instream and Riparian Habitat Restoration in the Albuquerque Reach of the Middle Rio Grande, New Mexico. Presented at the 2012 Work Environmental & Water Resources Congress, Albuquerque, New Mexico, May 20-24.

Thomas, D.B., Mussetter, R.A., and Boberg, S.A., 2012. FLO-2D Modeling to Support the Rio Grande Bosque Restoration Feasibility Study, Albuquerque Reach of the Middle Rio Grande, New Mexico. Presented at the 2012 Work Environmental & Water Resources Congress, Albuquerque, New Mexico, May 20-24.

Trabant, S.C., Mussetter, R.A., and Shafike, N., 2012. Sediment Routing Model of the Middle Rio Grande. Presented at the 2012 Work Environmental & Water Resources Congress, Albuquerque, New Mexico, May 20-24.

Harvey, M.D., Mussetter, R.A., Fullerton, W.T., Rolland, C., and Donnelly, C., 2012. Sediment Plug Formation in the San Acacia Reach of the Middle Rio Grande. Presented at the 2012 Work Environmental & Water Resources Congress, Albuquerque, New Mexico, May 20-24.

Harvey, M.D., Mussetter, R.A., Marcus, M., and Kuhn, W., 2011. Multipurpose Channel Relocation, Middle Rio Grande, New Mexico. Abstract presented at the 14th International River Symposium, Brisbane, Australia, September 26-29.

Harvey, M.D., Mussetter, R.A., Trabant, S.C., and Fowler, D., 2010. Insensitivity of the North Branch Root River, Milwaukee County, Wisconsin: Implications for Restoration. Poster presented at the PRRSUM First Annual Upper Midwest Stream Restoration Symposium, La Crosse, Wisconsin, February 21-24.

Mussetter, R.A. and Harvey, M.D., 2010. Physical and ecological challenges facing restoration of 240km of the Upper San Joaquin River from Friant Dam to the Merced River, California. 13th International River Symposium, Perth, Western Australia, 11-14 October.

Harvey, M.D. Mussetter, R.A., Fullerton, W.T., Bailey, P., and Kuhn, W., 2010. Ecosystem restoration in four water-short rivers of the Western USA. 13th International River Symposium, Perth, Western Australia, 11-14 October.

Mussetter, R.A. and Harvey, M.D., 2009. Relationship between Physical Characteristics, Flow Regime and Riparian Vegetation in Coarse-grained Streams. Proceedings of the 7th International Symposium on Ecohydraulics (pending), Concepcion, Chile, Jan 12-16.

Harvey, M.D. and Mussetter, R.A., 2009. Modeling of Fine and Coarse Sediment Dynamics in the Upper Colorado River, USA: Implications for Biological Productivity. Proceedings of the 7th International Symposium on Ecohydraulics (pending), Concepcion, Chile, Jan 12-16.

Mussetter, R.A., Trabant, S.C., and Morris, C.E., 2008. Sediment Routing Model of the Middle Rio Grande. Presented at the 50 Years of Soil and Water Research in a Changing Agricultural Environment Conference, September 3-5, Oxford, Mississippi.

Mussetter, R.A., Harvey, M.D., and Parkinson, S., 2007. Boat Wake Erosion of Sand Bars in Hells Canyon of the Snake River, Idaho and Oregon. World Environmental and Water Resources Congress 2007, ASCE, Tampa, Florida, May.

Mussetter, R.A., 2007. An Introduction to Rivers of the Arid Southwest. In Price, L.G., Johnson, P.S., and Bland, D., (eds), Water Resources of the Middle Rio Grande, San Acacia to Elephant Butte, New Mexico Bureau of Geology and Mineral Resources, pp. 14-18.

Mussetter, R.A., Harvey, M.D., and Harner, R.F., 2005. Physical characteristics, flow regime and riparian vegetation in coarse-grained streams, Idaho Batholith, USA. Poster presented at the Sixth Gravel-bed Rivers Conference, Austria, September 5-9.

Mussetter, R.A. and Trabant, S.C., 2005. Analysis of Potential Dam Removal/Retrofit Impacts to Habitat, Flooding and Channel Stability in the Carmel Valley, California. Proceedings of the Watershed 2005 Management Conference, ASCE, Williamsburg, Virginia, July 19-22.

Mussetter, R.A. and Harvey, M.D., 2005. Design Discharges for Arroyos in an Urban Setting. Proceedings of the EWRI 2005 World Water and Environmental Resources Congress, Anchorage, Alaska, May 15-19.

Harvey, M.D. and Mussetter, R.A., 2005. Difficulties of Identifying Design Discharges in Steep, Coarse-Grained Channels in the Arid Southwestern US. Proceedings of the EWRI 2005 World Water and Environmental Resources Congress, Anchorage, Alaska, May 15-19.

Armstrong, S., Miller, W., Mussetter, R.A., Harvey, M.D., and Thomas, D.B., 2004. Aquatic Habitat and Hydraulic Modeling Study, Rio Grande at Bosque del Apache National Wildlife Refuge. Poster session for the 2004 Festival of Cranes, San Antonio, New Mexico, November.

Mussetter, R.A. and Harvey, M.D., 2004. Geomorphic, Hydrologic, Hydraulic and Sediment Transport Analyses: Tools for Evaluating In-channel and Channel-margin Habitat Dynamics. Proceedings of the 3rd Missouri River and North American Piping plover and Least Tern Workshop, Sioux City Iowa, April 12-14.

Harvey, M.D. and Mussetter, R.A., 2004. Fine Sediment Dynamics in Coarse-Grained Streams; Implications for Biological Productivity in Urbanized Western Streams. Proceedings of the EWRI Environmental Resources Congress 2004, Salt Lake City, Utah, June.

Mussetter, R.A. and Harvey, M.D., 2004. Maintaining Natural Conditions in Urban Arroyos: Is It Possible? Proceedings of the EWRI Environmental Resources Congress 2004, Salt Lake City, Utah, June.

Mussetter, R.A., Trabant, S.C., and Wolff, C.G., 2004. Analysis of Potential Dam Removal Impacts to Habitat, Flooding, and Channel Stability in the Carmel Valley, California. Proceedings of the EWRI Environmental Resources Congress 2004, Salt Lake City, Utah, June.

Mussetter, R.A., Wolff, C.G., Peters, M.R., Thomas, D.B., and Grochowski, D., 2004. Two-Dimensional Hydrodynamic Modeling of the Rio Grande to Support Fishery Habitat Investigations. Proceedings of the EWRI Environmental Resources Congress 2004, Salt Lake City, Utah, June.

Peters, M.R., Mussetter, R.A., Thomas, D.B., and Wolff, C.G., 2004. Two-Dimensional Hydrodynamic Modeling of the Rio Grande to Support Fishery Habitat Investigations. Abstract for the proceedings of the American Geophysical Union, Hydrology Days 2004, Colorado State University, Fort Collins, Colorado, March.

Lunger, J.R., Harvey, M.D., and Mussetter, R.A., 2004. Investigation of Habitat Formation and Fish Use during a Range of Flows in a Sand-bed Stream, the Pecos River, New Mexico. Abstract for the proceedings of the American Geophysical Union, Hydrology Days 2004, Colorado State University, Fort Collins, Colorado, March.

Thomas, D.B., Harvey, M.D., and Mussetter, R.A., 2004. Sediment Yield Estimates from Ungaged Tributaries to the Middle Rio Grande, New Mexico. Abstract for the proceedings of the American Geophysical Union, Hydrology Days 2004, Colorado State University, Fort Collins, Colorado, March.

Wolff, C.G., Mussetter, R.A., and Harvey, M.D., 2004. Evaluation of the Effects of Dam Re-operation on Establishment of Riparian Vegetation, Verde River, Arizona. Abstract for the proceedings of the American Geophysical Union, Hydrology Days 2004, Colorado State University, Fort Collins, Colorado, March.

Mussetter, R.A., Harvey, M.D., Anthony, D.J., 2003. Identification of the Ordinary High-water Mark of the Snake River, Western Idaho, USA. Abstract: Proceedings of Hydrology Days 2003, American Geophysical Union, Fort Collins, Colorado.

Harvey, M.D., Mussetter, R.A., Anthony, D.J., 2003. Island Aging and Dynamics in the Snake River, Western Idaho, USA. Abstract: Proceedings of Hydrology Days 2003, American Geophysical Union, Fort Collins, Colorado.

Harvey, M.D., Mussetter, R.A., Morris, C.E., 2003. Fine Sediment in the Upper Colorado River during Spring Runoff and Summer Baseflows: Implications for Flow Recommendations and Biological Productivity. Abstract: Proceedings of Hydrology Days 2003, American Geophysical Union, Fort Collins, Colorado.

Wolff, C.G., Mussetter, R.A., Bucher, B., 2003. Channel Remediation and Restoration Design for Silver Bow Creek, Butte, Montana. Abstract: Proceedings of Hydrology Days 2003, American Geophysical Union, Fort Collins, Colorado.

Harvey, M.D., Mussetter, R.A., Morris, C.E., 2003. Fine Sediment Dynamics in the Upper Colorado River during Spring and Summer Baseflows. Presented to the Upper Colorado River Basin Researcher's Annual Meeting, Grand Junction, Colorado, January 16.

Miller, W.J., Rees, D.E., Ptacek, J.A., Harvey, M.D., Mussetter, R.A., and Morris, C.E., 2002. Ecological and Physical Processes during Spring Peak Flow and Summer Base Flows in the Colorado River above the Gunnison River, Volume I, Draft Report. Prepared for the Colorado River Water Conservation District, Glenwood Springs, Colorado.

Mussetter, R.A., Gessler, D., and Wolff, C.G., 2002. Modeling of Potential Dam Removal Impacts to Habitat, Flooding, and Channel Stability on the Carmel River, California. Hydrology Days, Colorado State University, Fort Collins, Colorado, April 1-2.

Mussetter, R.A., Harvey, M.D., and Trabant, S.C., 2002. Historical and Present Day Sediment Loads in the Middle Rio Grande, New Mexico. Proceedings of Hydrology Days, 2002 American Geophysical Union, Colorado State University, Fort Collins, Colorado, April 1-2.

Mussetter, R.A., Harvey, M.D., Zevenbergen, L.W., and Tenney, R.D., 2001. A Comparison of One- and Two-Dimensional Hydrodynamic Models for Evaluating Colorado Squawfish Spawning Habitat, Yampa River, Colorado. In Applying Geomorphology to Environmental Management, Anthony, D.J., Harvey, M.D., Laronne, J.B., and Mosley, M.P. (eds), Water Resource Publications, Englewood, Colorado, p. 361-379.

Mussetter, R.A. and Harvey, M.D., 2001. The Effects of Flow Augmentation on Channel Geometry of the Uncompahgre River. In Applying Geomorphology to Environmental Management, Anthony, D.J., Harvey, M.D., Laronne, J.B., and Mosley, M.P. (eds), Water Resource Publications, Englewood, Colorado, p. 177-198.

Mussetter, R.A. and Harvey, M.D., 2001. Composite *n*-Value Estimation: Implications for Estimating the Hydraulic Impacts of Riparian Habitat Restoration. Presentation at Seventh Federal Interagency Sedimentation Conference, Reno, NV, March 25-29.

Harvey, M.D. and Mussetter, R.A., 2001. Restoration of the San Joaquin River: Constraints and Opportunities. Presentation at Seventh Federal Interagency Sedimentation Conference, Reno, NV, March 25-29.

Wolff, C.G., Harvey, M.D., and Mussetter, R.A., 2000. San Miguel River Restoration: Geomorphology and Hydraulic Engineering as a Basis of Design. 2000 Joint Conference on Water Resources Engineering and Water Resources Planning and Management, Minneapolis, Minnesota, July 30-August 2.

Mussetter, R.A., 2000. Bed Material Transport Equation for High Suspended Sediment Concentrations, 2000. 2000 Joint Conference on Water Resources Engineering and Water Resources Planning and Management, Minneapolis, Minnesota, July 30-August 2.

Thomas, D.B., Abt, S.R., Mussetter, R.A., and Harvey, M.D., 2000. A Design Procedure for Sizing Step-Pool Structures. 2000 Joint Conference on Water Resources Engineering and Water Resources Planning and Management, Minneapolis, Minnesota, July 30-August 2.

Mussetter, R.A., Harvey, M.D., Wolff, C.G., and McDowall, D.G., 2000. Whitewood Creek Reclamation Plan: A Sound Basis for Design. 2000 Joint Conference on Water Resources Engineering and Water Resources Planning and Management, Minneapolis, Minnesota, July 30-August 2.

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