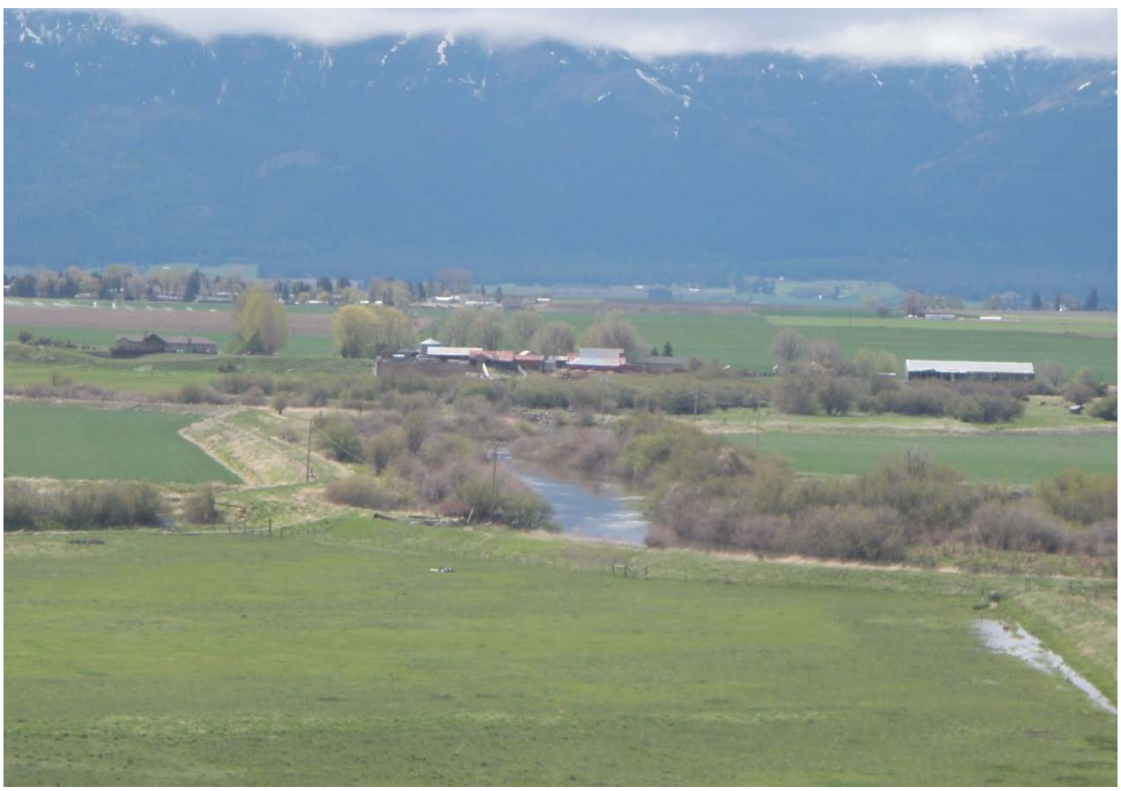
APPENDIX D – HYDRAULICS

RECLAMATION

Managing Water in the West

Catherine Creek Tributary Assessment Hydraulics Appendix

Grande Ronde Project, OR
Pacific Northwest Region
SRH Report 2012-05



U.S. Department of the Interior
Bureau of Reclamation

December 2011

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The U.S. Department of the Interior protects America's natural resources and heritage, honors our cultures and tribal communities, and supplies the energy to power our future.

The mission of the Bureau of Reclamation is to manage, develop, and protect water and related resources in an environmentally and economically sound manner in the interest of the American public.

Photo taken on Grande Ronde River near station 25,265 on April 27, 2010.

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Executive Summary

Reclamation's Sedimentation and River Hydraulics Group at the Technical Service Center developed a one dimensional (1D) hydraulic model to analyze the Catherine Creek Assessment area hydraulic conditions during flood flows. Approximately 60 miles of channel were modeled including a portion of Catherine Creek, Grande Ronde River, and State Ditch.

A steady-flow model was developed to examine the existing hydraulic conditions of Catherine Creek. Steady flow model input consists of a channel geometry, infrastructure dimensions and operating conditions, input discharge, a downstream boundary condition, and roughness values. Terrain models were developed as topographic input to the hydraulic model based on LiDAR data above wetted channel areas and bathymetric surveys within the wetted channel areas. A total of 803 cross-section lines spaced approximately 450 feet apart were applied to cover the 60 river miles modeled across the three streams. Levee elements were assigned manually in HEC-RAS. Twenty-nine bridges and nine diversions were included in the HEC-RAS model.

Thirteen model flood flow discharges were simulated in the HEC-RAS model, including the 1.5-, 2-, 10-, 50-, and 100-year flood events. The downstream boundary was set to a normal depth slope of 0.3%. Roughness values were determined using a combination of pebble counts, vegetation and agricultural land use, and professional judgment. The model was compared with measured data from June, 2010 and October, 2010. Comparison between the measured and modeled water surface were variable; Manning's roughness was not adjusted in the model but sensitivity analyses were performed. Several limitations exist with the current 1D model including levees and levee overtopping, missing low flow channel data, and the extent of the LiDAR data.

Four reaches on Catherine Creek were analyzed for the present conditions. Reach 1 (RM 0 - 22.5) can be described as a wide, unconfined valley with an average slope of approximately 0.006%. The channel capacity of the reach is highly variable, with most locations exhibiting bankfull conditions at flows between the 1.5- to 2-year discharges. Average in-channel velocities are very low and are typically around 1.3 ft/s at discharges with recurrence intervals between 1.5 and 100 years. Similarly, shear stresses are very low, indicating the potential to transport only sand sized sediment under flood conditions. Levees are present along most of the reach, limiting floodplain access. In most locations, levees are overtopped at flows equal to or less than the 10-year discharge. There are four disconnected oxbows (RM 10.2, 14, 16.3, and 17.5) in this reach where the levee is overtopped at less than a five year flood. The most notable hydraulic controls in this reach are Elmer Dam at RM 13.1 and the Old Grande Ronde River, which is located in the upstream extent of the reach at RM 22.5. Bridges within the reach, including Booth Lane, Market Lane, and Highway 237, exert local controls at flows exceeding the 100-year discharge but do not appear significant at lower discharges.

Reach 2 is also a wide, unconfined valley with an average slope of approximately 0.04%. A noteworthy break in slope occurs at the confluence of Ladd Creek near RM 31.4, which coincides with changes in hydraulic properties. Channel capacity throughout the reach is variable, with bankfull conditions occurring in most cross sections around 1.5 to 2-year discharges. In-channel velocities below Ladd Creek are generally around 1.7 ft/s. Upstream from Ladd Creek, velocity increases with discharge and averages 3.1 ft/s. Shear stresses in Reach 2 are slightly higher than those in Reach 1, with reach averages ranging from approximately 0.10 to 0.17 lb/ft^2 for discharges between the 1.5- and 100-year recurrence intervals. Levees within Reach 2 are overtopped less frequently than Reach 1 and only 50% of the cross section levees are overtopped at the 100-year discharge. Notable hydraulic controls in this reach include Upper and Lower Davis Dams, Ladd Creek, Wilkinson Lane Bridge, and a Beaver Dam located at RM 24.9. Similar to Reach 1, most bridges in the reach impart some hydraulic control at the 100-year discharge, but their influence appears to be localized.

The downstream end of Reach 3 (RM 37.2 - 40.8) and the upstream end of Reach 2 act as a hydraulic transition zone at the base of the Catherine Creek alluvial fan. The confinement of the valley within Reach 3 increases from downstream to upstream. Average bed slope within this reach is 0.59%. Channel capacity in this reach is high compared to downstream Reach 1 and 2 and also compared with upstream Reach 4. Over 60% of cross sections require a flow of 100-year recurrence interval or greater to exceed the channel banks. Reach-averaged channel velocities range from 4.6 ft/sec for the 1.5 year flood to 6.6 ft/sec for the 100-year flood. Shear stresses in the reach range from about 1 lb/ft^2 for a 1.5-year discharge to 1.75 lb/ft^2 for a 100-year discharge, indicating some potential to transport gravels at higher discharges. Less than 30% of cross sections with levees indicate levee overtopping for flows less than a 500-year discharge. Several of the bridges, such as Main Street Bridge at RM 40 exert hydraulic control on the larger flood flows.

Reach 4 (RM 40.8 – 45.8) is an unconfined valley reach with an average channel slope of 0.83%. The channel capacity for most locations of the reach is between a 5 and 10-year discharge. The reach averaged velocity in Reach 4 is approximately 4.8 ft/sec for the 1.5year discharge and 6.7 ft/sec for the 100-year discharge. Average in-channel shear stresses in the reach range between 1.1 lb/ft^2 for a 1.5-year discharge to about 1.8 lb/ft^2 for a 100-year discharge. Similar to Reach 3, levees present in Reach 4 typically require a discharge of 500-year recurrence interval to overtop. Some localized overtopping of less formidable levees may occur during more frequent floods. The most significant hydraulic control within the reach is the Medical Springs D2 diversion structure.

Within the reaches simulated, Grande Ronde River and Reaches 1 and 2 of Catherine Creek have experienced the greatest degree of impact to flood processes. Conversion of floodplain to agricultural land use has resulted in greatly reduced access to high flow habitat, including inundated floodplains and side channels. Constriction of flows between levees has also likely resulted in increased velocities within the channel banks and reduced high flow refugia along the channel margins during more frequent discharges. Overbank areas that do remain accessible between the levees are expected to have reduced complexity compared with unimpaired conditions. Within Reaches 3 and 4 of Catherine Creek, the greatest impacts to river processes results from the presence of low-head diversion structures and bridges. However, the impacts of the structures on floodplain access are less severe since the floodplain extent is much narrower and slope is higher when compared with downstream reaches.

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1. Introduction

1.1. Purpose

An assessment is being conducted by Bureau of Reclamation's (Reclamation) Pacific Northwest Regional Office (PNRO) to define the existing habitat conditions, limiting factors, present use and habitat potential within the Catherine Creek Tributary Assessment Area for Endangered Species Act (ESA) listed salmonids such that project locations can be identified and prioritized for implementation. To help meet the assessment objective, Reclamation's Sedimentation and River Hydraulics Group at the Technical Service Center developed a one dimensional (1D) hydraulic model. The model was used to analyze the Catherine Creek Assessment area hydraulic conditions during flood flows.

1.1.1. Objectives

The objectives of the model were to:

1. Determine what areas are being inundated for discharges with recurrence intervals ranging between 1.05 to 500 years.
2. Evaluate flood storage, water surface elevations, velocities, and shear stresses.
3. Qualitatively compare with historic conditions.
4. Investigate the flow and stage at which inundation of each disconnected oxbow occurs.

1.2. Location

The Grande Ronde River Basin drains the Blue and Wallowa Mountains. The Grande Ronde River enters Grande Ronde Valley from the west and exits towards the north. Catherine Creek is a major tributary to the Grande Ronde River and enters Grande Ronde Valley from the south and combines with Grande Ronde River at the end of a reach known as State Ditch. Upstream of Union, OR, Catherine Creek is a mountainous stream with a narrow valley and slopes approaching 1%, while downstream of Union the river meanders across a wide valley with a nearly flat slope of less than 0.006%.

Approximately 60 miles of channel were modeled (see Figure 1). The model includes a substantial portion of Catherine Creek and a reach of the Grande Ronde River which contains State Ditch. On Catherine Creek, the upstream point of the model is at river mile (RM) 46.6 near Brinker Creek Road Bridge and the downstream point is the confluence with State Ditch at RM 0. The modeled section of State Ditch is from Peach Road Bridge to the confluence with Catherine Creek, a distance of approximately 5.5 miles. Downstream of the confluence of Catherine Creek and State Ditch, 12.6 miles of the Grande Ronde River are modeled. The downstream extent of the model is the canyon known as Rhinehart Gap.

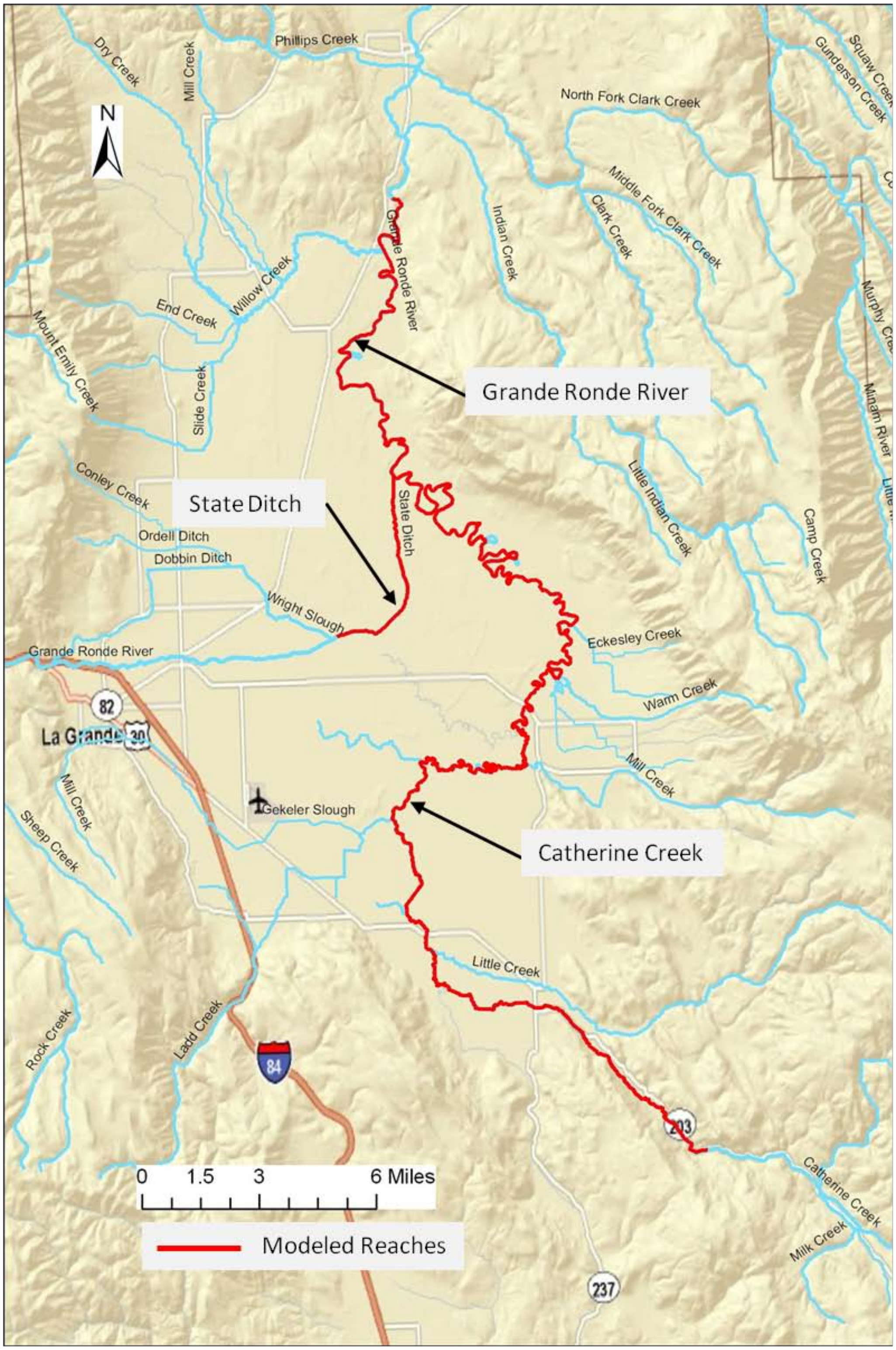

Figure 1. Overview map of Catherine Creek and Grande Ronde River which includes State Ditch.

2. Historic Conditions

European settlement began in the Grande Ronde Basin in the mid-1800s. Prior to that point, several Native American tribes, Nez, Perce, and Umatilla, lived in the basin (Duncan 1998). In 1846, a group of Europeans described the River:

"Grand Round River, which comes in from the West, runs nearly to the middle of the plain in several channels, joins with another branch, bears away to the left and leaves the plain at its Northern extremity, through a now gap…Numerous small creeks and rivulets, run through all parts of the valley, from the surrounding Mountains" (Beckham 1995)

Others that visited the valley during this time period noted the rich soil and good grass in the valley. From 1811-1908, the Grande Ronde River was described as cold and clear, offering habitat for salmon, crayfish, and beavers. In the Grande Ronde Valley, dense vegetation (cottonwood, hawthorn alder, etc.) was present shading the river channel (Beckham 1995). It is assumed that Catherine Creek downstream of RM 37 would have similar characteristics to Grande Ronde River in the Grande Ronde Valley, where river and valley characteristic are similar.

Once Europeans arrived, the Basin was quickly altered. In 1869, a minor pilot channel was constructed along State Ditch (USACE 1957). The channel was initially 6 feet wide and 3 feet deep (Duncan 1998). The purpose of the channel was to reduce annual spring flooding; the ditch replaced 33 miles of river with 4 miles of straightened channel. Other projects were also occurring to drain wetlands. Tule Lake was drained (approximately 2,300 acres of wetland) in 1870 and Catherine Creek was relocated since it originally drained into Tule Lake (Beckham 1995). Irrigation companies were documented as early as 1904. Later projects were also implemented to protect property from flooding.

"Local farmers have in several cases excavated channel cut-offs across narrow reaches of stream meanders, and constructed low earth levees." (USACE 1957)

In addition to stream channelization changes, other land use changes were also occurring, primarily mining, livestock grazing, road building, and timber harvest. Mining has occurred in the headwaters of Grande Ronde River since 1870, and dredge mining was extensive in the early 1900s (McIntosh, 1994). Livestock have been grazing the Grande Ronde River basin since the 1880s. From 1911 to 1990 a decline in domestic livestock occurred mainly due to the sheep industry collapse. However, elk grazing has increased, leading to a similar grazing intensity as in1945 (McIntosh, 1994). Logging activities began in the 1880s as well, and has increased since 1941. Road construction began in the 1920s, and has increased over time (McIntosh, 1994).

The land use changes described above directly and indirectly contributed to the condition of Catherine Creek and Grande Ronde River. Unfortunately, there is little more than anecdotal information to describe the Basin from the late 1800s and early 1900s. In 1941, a Bureau of Fisheries survey collected information on pools and substrate composition of the Grande Ronde River and Catherine Creek. In 1990, portions of the streams were resurveyed. Based on the repeated surveys, the total pools/km has declined by 78% in the Grande Ronde River and 67% on Catherine Creek (McIntosh 1994). The frequency of large pools also dropped 73% and 61%, respectively. Stream flow and discharge records were also examined and showed an increase in base discharge at the gage near La Grande, OR and near Union, OR (McIntosh 1994). Based on the analysis, the average timing of peak flows also shifted to earlier in the year at the La Grande gage from April 10th to March 11^{th}. Changes in base discharge and peak discharge are attributed to timber harvest practices that have reduced evapotranspiration (McIntosh 1994). The cleared areas have greater snowfall accumulations as well as faster snowmelt.

In the present-day Grande Ronde Basin, the summers are typically hot and dry while the winters are cold and wet. Peak flows occur in April or May, while August and September are low flow months (UGRRSLAWQAC 1999). Irrigation withdrawals have reduced low flows in the summer months. Below Union, flow reductions due to water withdrawal are about 25% in June and approximately 50% by mid July (Nowak 2004).

The amount of residence time of water in the valley and the mechanism by which water is transported downstream appears to have changed from the early European settlement days. As stated earlier, multiple channels and creeks historically ran across the valley.

"During presettlement times an estimate 72,000 acres in the middle valley were subject to flooding; up to 60 percent of the valley flood might be inundated for as long as five months. In the 1894 flood, 50,000 acres were covered with floodwaters; in the 1949 flooding, only 5,900 acres were inundated." (Duncan 1998).

Currently, most of the water is transported through a few channels (Catherine Creek and State Ditch), and most of the valley is not inundated throughout the year.

In 1971, the United States Army Corps of Engineers (USACE) evaluated channel capacity on Catherine Creek. Upstream of Union, the flow on Catherine Creek would get out of bank at approximately 1,000 ft^3/sec. Through the town of Union, the capacity of Catherine Creek was 800 to 1,000 ft^3/sec. It was noted that in the past this capacity was as little as 600 ft^3/sec. From the Highway #203 bridge to the Old Grande Ronde confluence, the capacity of Catherine Creek was around 600 ft^3/sec. From the Old Grande Ronde River confluence to the confluence with State Ditch, the capacity ranged from 600 to 1,000 ft^3/sec. Below State Ditch, the capacity of Grande Ronde River was 1,000 to 2,000 ft^3/sec.

During the pre-settlement era, it appears that a large portion of the Grande Ronde Valley was inundated and could be classified as wetlands or wet meadow. After settlement, anthropogenic influences, such as stream channelization, levee development, and agriculture, changed the valley into a few channels that are locked in place where land on both banks has been protected from floods. The channelization has reduced the total length of the river in locations such as State Ditch. A loss of pools from the 1940s to the 1990s has been documented, and riparian area along the channel has declined. Peak spring runoff is occurring earlier than in the 1940s, potentially due to timber harvest, and the summer months are characterized by reduced low flows due to irrigation withdrawals.

3. Methods

A steady-flow model was developed to examine the existing hydraulic conditions of Catherine Creek from RM 0 to RM 46.6. Steady flow model input consists of a channel geometry, infrastructure dimensions and operating conditions, input discharge, a downstream boundary condition, roughness values, expansion and contraction coefficients, and computation parameters. Each model input is described in detail below. The hydraulic model was simulated as a steady-state subcritical flow model.

3.1 Model Geometry

3.1.1 Development of Topographic Data

Topographic data were used to generate cross sections for the model in HEC-RAS. Terrain models were developed for topographic input to the model based on LiDAR data above the wetted channel perimeter and bathymetric surveys within the wetted channel. LiDAR were acquired in four geographic areas within the tributary assessment area. In October 2007, LiDAR data were collected along the Grande Ronde River and State Ditch (combined into a geographic area referred to as “Willow”) and also along Middle Catherine Creek from River Mile (RM) 23.7 to 42.5 (Watershed Sciences, 2007). In 2009, LiDAR were collected along Upper Catherine Creek from RM 42.5 to 52 and Lower Catherine Creek from RM 0 to 23.7 (Watershed Sciences, 2009).

Because bare-earth LiDAR cannot penetrate the water surface and adequately represent bed elevations in the wetted area of the channel, bathymetric surveys were conducted. Surveys were conducted between October 28th and November 2, 2010 along 8 miles of State Ditch upstream of the confluence with Catherine Creek, along approximately 11.7 miles of the Grande Ronde River downstream from the State Ditch confluence, and from RM 0 to 36.5 along Catherine Creek. Two sections of Catherine Creek, from RM 32 to 34.5 and from RM 27 to RM 30, could not be accessed to measure bathymetry.

The bathymetric survey data were collected using a Sontek River Surveyor M9 Acoustic Doppler Profiler (ADP). Horizontal and vertical position information for the survey was achieved by linking the ADP to a Trimble R8 GPS system operating with a Real Time Kinematic (RTK) survey. Horizontal and vertical accuracies are typically within +/- 0.5 feet.

The GPS and ADP were mounted on an aluminum frame raft with inflatable pontoons and connected to a field computer, which processes information from both instruments. The GPS receiver on the raft was mounted in close proximity to the ADP mounting pole and was set to export the GGA NMEA data string. This data string exports the GPS position data directly to the computer. During the boat surveys, GPS observations were taken to measure the water surface elevation every 20 feet. These measurements were later used to assign a water surface elevation to each ADP measurement.

Data collected in the data controller (on the boat) and in the base station receiver were downloaded to Trimble Business Center (TBC version 2.2). Data logged at the base stations were submitted to OPUS (http://www.ngs.noaa.gov/OPUS/) for post processing. The control point coordinates were adjusted based on these results where necessary. Horizontal positions were reported in NAD 83 State Plane Oregon North International Feet; and vertical positions were reported in NAVD 88 ft. Elevations were derived from GEOID 09. After these adjustments were made, the water surface observations were exported in shapefile format for further use in ArcMap (Version 9.3.1, ESRI, Redlands, CA).

Once the ADP bathymetry data and the GPS water surface elevation data were imported into ArcMap, bed elevations for the ADP measurements were determined. The GPS water surface elevations were used to create a water surface Triangulated Irregular Network (TIN). Using the Functional Surface Tool in 3D analyst, the ADP bathymetry points were assigned a water surface elevation based on each point's position relative to the water surface TIN. Once this process was completed, fields for horizontal position (x,y), water surface elevation, and bed elevation were created and populated in the attribute table of the new 3D feature class.

In addition, the Pacific Northwest Regional Office conducted detailed RTK topographic surveys within the channel from RM 36.8 to 37.9. No additional processing of these data points was required to extrapolate ground elevations. These surveys were combined with the boat surveys for development of the in-channel surface.

3.1.2 Combining LiDAR and survey data

Several processing steps were necessary to combine the LiDAR data with the bathymetry data. First, a terrain surface of just LiDAR was developed for each of the 4 geographic areas (Willow, Lower Catherine, Middle Catherine, Upper Catherine). Since no topographic survey data were collected in Upper Catherine Creek, the final terrain model upstream of RM 42.5 consists only of the processed LiDAR data.

The next task required delineating polygons of the wetted area in ArcMap using a hillshade of the LiDAR and rectified aerial photographs. The LiDAR data were removed from this area within each terrain model if survey data were available to better represent the in-channel surface. This first required converting several LiDAR tiles from multi-point features to single part features, selecting the points intersecting the wetted channel polygons, and deleting the points from the feature class. Within the polygons where in-channel data were collected, the Spline With Barriers tool within ArcMap was used to rasterize the channel surface. Raster cell sizes ranged between 3 and 5 feet depending on the width of the channel and the necessary cell size to represent the width of the channel. These rasterized cells were converted to points. To avoid triangulation issues adjacent to the wetted channel polygons, points located within one cell size (3-5 ft) from the wetted channel polygon were deleted.

Within the two sections of Catherine Creek (from RM 32 to 34.5 and from RM 27 to RM 30) where bathymetry was not collected, channel data were developed by delineating a line along the channel and linearly interpolating elevations along the line based on upstream and downstream surveyed elevations. The Spline with Barriers technique was applied in this area using the interpolated points to develop the rasterized surface as described previously. Although these linearly interpolated data poorly represent the bed through these reaches, they are the best method for representing the bed elevations within the scope of this project.

Comparison of the LiDAR data within the channel with the surveyed data just upstream illustrated the need to lower the bed elevations in these reaches below the elevations captured by the LiDAR (Figure 2). Additional survey data should be collected in these reaches for refined future analyses.

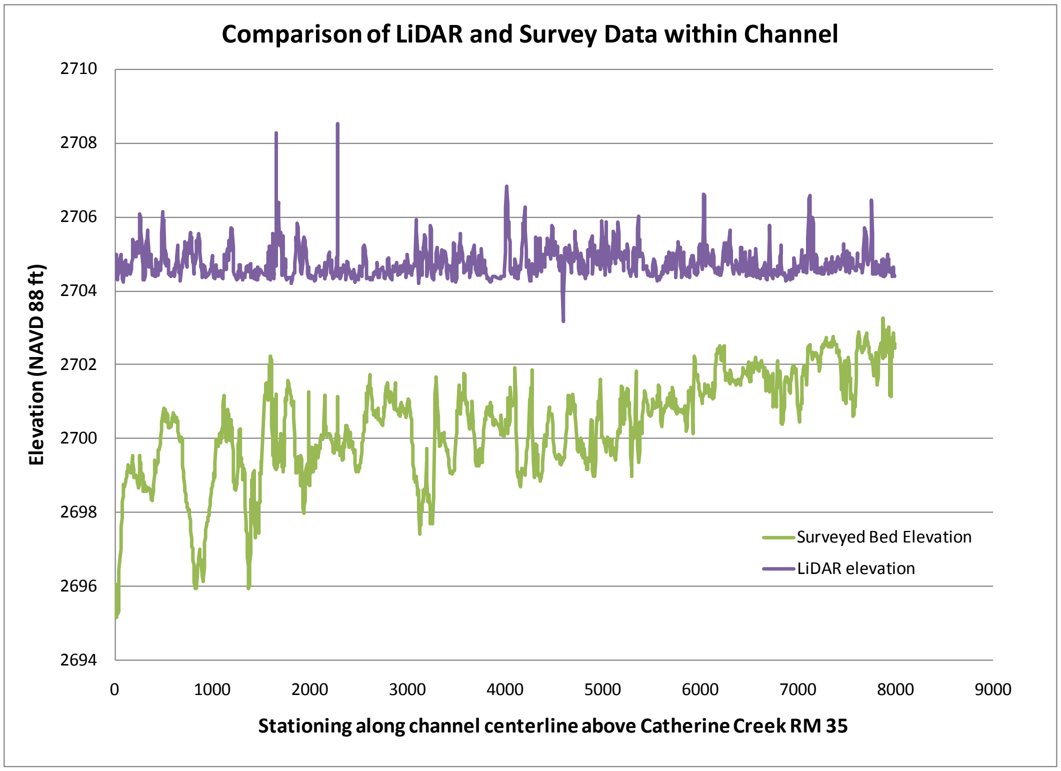

Figure 2. Comparison of LiDAR data within the channel and surveyed bed elevations.

The downstream boundary of the hydraulic model is approximately 6,000 ft downstream of the coverage of the bathymetric surveys. Within this section of the model, the bed elevations are only represented by LiDAR data. Additional surveys will be needed in this reach for refined future analyses. Manipulation of the bed elevations at this downstream end are discussed in Section 3.1.6. Sensitivity analyses were conducted to evaluate the longitudinal extent of the downstream boundary condition and are also discussed in Section 3.1.6.

Final terrain surfaces for the State Ditch, Grande Ronde River below State Ditch, Lower Catherine Creek, and Middle Catherine Creek were developed using the points within the channel developed from the spline with barriers models, the polygons delineating wetted channel areas (soft edges), and the LiDAR data outside of the wetted channel areas.

3.1.3 Cross Section Development

HEC-GeoRAS is a custom interface between HEC-RAS and Geographic Information System (GIS) that provides tools to process geospatial data for use with HEC-RAS. The HEC-GeoRAS program (version 4.2.93 for ArcGIS 9.3) was utilized to delineate cross sections, banklines, flowpaths, and a centerline along the modeled reaches. A total of 803 cross-section lines spaced approximately 450 feet apart were applied to cover the 60 river miles modeled across the three streams. Figure 3 shows a portion of the cross sections delineated upstream of Union, OR near RM 44. Figure 4 shows a portion of the cross sections delineated on Catherine Creek near RM 10. A module within HEC-GeoRas was then utilized to convert all of the delineated line work and topographic information into a HEC-RAS format. The stream channel is extremely sinuous and the dominant flow paths may be different at bankfull and flood flows. There was an attempt to represent the main channel flow paths and overbank flow paths as accurately as possible. However, because of the complex stream channel alignment, this is difficult and it may be necessary to alter the over bank representation to better represent flood flows.

Figure 3. Example portion of upper Catherine Creek with delineated cross sections.

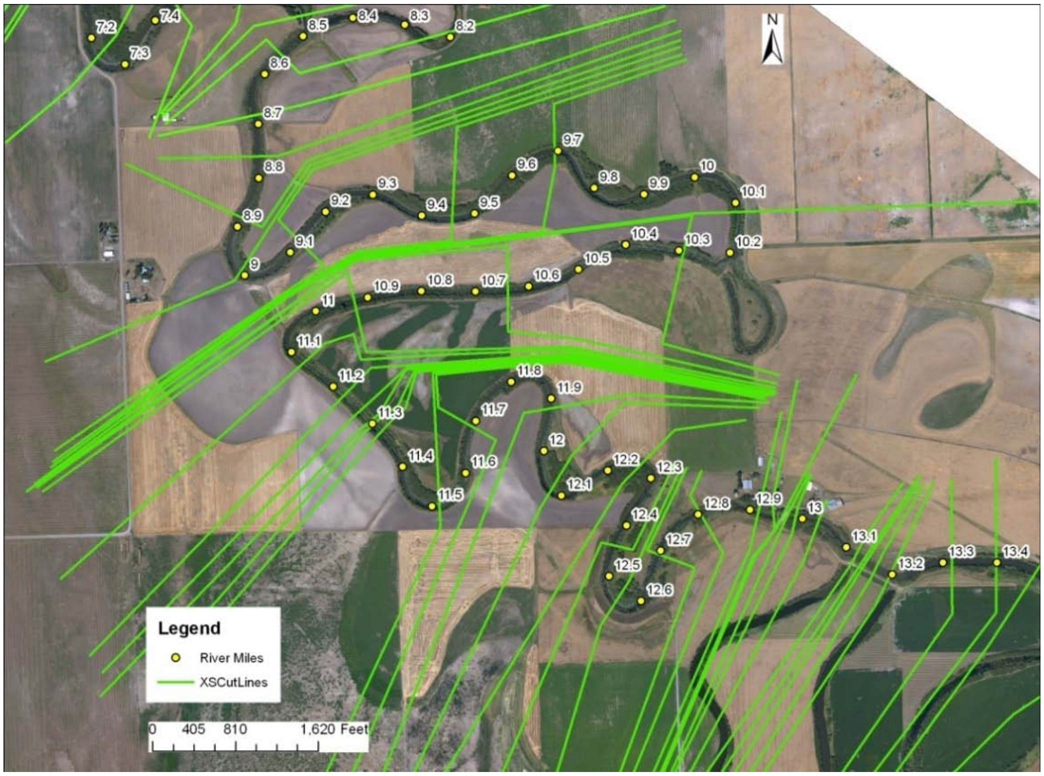

Figure 4. Example portion of lower Catherine Creek with delineated cross sections.

Banklines were manually adjusted where necessary in HEC-RAS to ensure that the top of bank was captured. Levee elements were also assigned manually in HEC-RAS. Levee elements do not allow flow to be conveyed outside of the levee station until the levee elevation is exceeded (Brunner 2008). They were assigned at bank locations or manmade levees as appropriate. Although multiple levees were often present along a single cross section, HEC-RAS does not allow the assignment of more than one levee on each side of the channel. Therefore, the closest, visibly unbreached levee to the main channel was assigned. More detailed explanation of the limitations of the levee assignments and potential impacts to model results are provided in Section 5.1.

3.1.4 Infrastructure

Anderson Perry and Associates, Inc. (AP) surveyed 52 structures in the Grande River Basin in 2010 including four river cross sections at each structure. Twenty-nine bridges and nine diversions were included in the HEC-RAS model (Table 1). Each of these structures is discussed in more detail below. The bridge structure dimensions from the AP survey were manually input to the HEC-RAS model. LiDAR data were utilized to incorporate the bridge deck and road surface information when necessary. For the diversion structures, only the grade control features were incorporated into the model geometry as weirs. Fish ladders, gates, and flow diversions were not included in the model.

Table 1. Bridge and diversion structures included in the HEC-RAS model.

Name	River	Model Station (ft)	River Mile (mi)
Brinker Creek Road Bridge	Catherine Creek	246853.4	46.5
Hwy 203 #B1 (Private Bridge)		241903.4	45.6
Medical Springs #D2		225243.4	42.5
Hwy 203 #B2 (Private Bridge)		223943.1	42.3
Medical Springs #D3		223510.4	42.2
Swackhammer Diversion		215110.1	40.7
Hwy 203 #B3		215421.6	40.6
Bellwood Bridge		212546.1	40.1
Main St. Bridge		212028.1	40.0
Godley Diversion		211803.1	39.9
Townley Dobbin Diversion		211140.1	39.8
5TH St Bridge		210396.1	39.7
Hempe-Hutchinson Diversion		209911.1	39.6
10TH St. Bridge		209060.1	39.5
Pond Slough (Private Bridge)		199263.8	37.6
Miller Bridge		192511.1	36.5
HWY 203 #B4 Bridge		186024.7	35.3
Upper Davis Diversion		184402.1	35.0
Lower Davis Bridge		181367.1	34.4
Lower Davis Diversion		181252.4	34.4
Woodruff Bridge		178262.1	33.8
Wilkinson Bridge #1		168313.1	32.0
Godley Lane Bridge #1		139058.1	26.6
Gekeler Bridge #1		123575	23.7
HWY 237 Bridge #1		110275	21.3
Booth Lane Bridge #1		78300	15.3
Elmer Dam		66934	13.1
Elmers Bridge #1		65975	13.0
Elmer Bridge #2		56900	11.3
Market Lane Bridge #1		34985	6.5
Alicel Bridge #1	Grande Ronde River	66200	NA
McKennon Lane Bridge		49950	NA
Hull Road Bridge		34700	NA
Striker Lane Bridge		29300	NA
Rinehart Lane Bridge		6800	NA
Booth Ln Bridge #2	State Ditch	17350	NA
Market Lane Bridge #2		9100	NA
Ruckman Lane Bridge		5220	NA

Model cross sections were delineated in HEC-GeoRAS along each field surveyed cross section location upstream and downstream of bridges and diversions. The cross section channel topographic information initially derived from the LiDAR terrain model was replaced in HEC-RAS within the channel by the surveyed information. Figure 5 shows an example of a surveyed cross section and its replacement in the HEC-RAS model. The differences between the terrain model and the surveyed cross sections are considered small outside of the channel, which verifies the methods used to develop the terrain model.

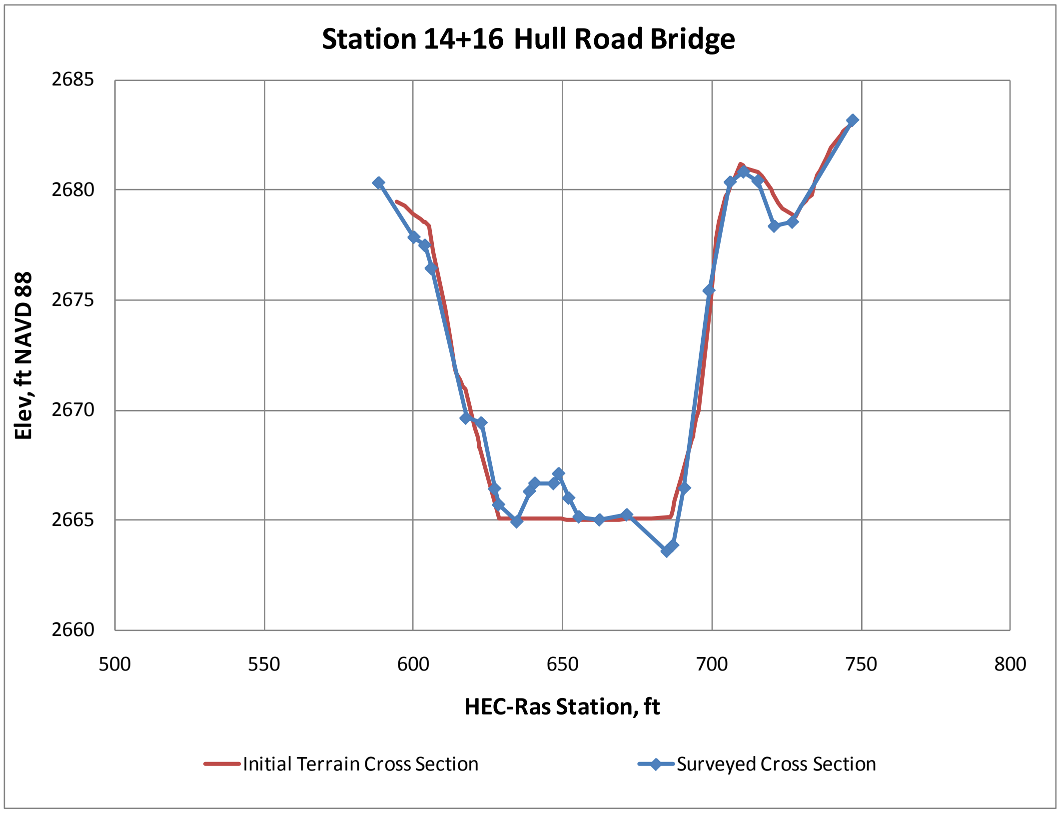

Figure 5. Example of channel cross section near bridge that was modified to include AP survey data.

Bridges

Twenty-nine bridges were included in the model. Information utilized in the model is documented in Table 2. In all cases, the energy equation was used for all flows. If bridges were skewed from the channel centerline, they were projected onto the upstream and downstream cross sections to account for the angle. The bridge opening and pier thickness in Table 2 are the projected values. All information was based on survey information, ground photographs, aerial photographs, and terrain models. In the adjacent upstream and downstream cross sections, ineffective flow areas were set where the road leading to and from the bridge was higher than the surrounding ground elevations. Ineffective flow areas are locations where water will likely pond and the velocity is zero in the downstream direction, as is the case with water being ponded behind a road embankment (Brunner 2008).

Table 2. Information used to incorporate bridge geometry in HEC-RAS model.

Bridge Name	Bridge Opening (ft)	Width (ft)	Piers	Pier Thickness (ft)	Top Deck Elevation (ft)	Low Chord Elevation (ft)
Brinker Creek Rd.	53	10	NA	NA	3,087.0	3,084.5
Hwy 203 #B1	58	20	NA	NA	3,042.3	3,039.8
Hwy 203 #B2	56	13	NA	NA	2,889.7	2,888.5
Highway 203 #B3	76	54.9	2	2	2,827.8	2,826.2
Bellwood	67	29.6	NA	NA	2,797.7	2,795.4
Main St.	46	63	NA	NA	2,792.9	2,789.5
5th St	69	28.2	NA	NA	2,782.4	2,780.2
10th St.	47	28	NA	NA	2,771.9	2,769.8
Pond Slough	38	14	NA	NA	2,722.1	2,719.7
Miller	98	31	NA	NA	2,717.5	2,714.3
Highway 203 #B4	92	41	2	2	2,711.3	2,709.4
Lower Davis	60	17	NA	NA	2,708.2	2,706.6
Woodruff	57	29.5	NA	NA	2,706.2	2,704.9
Wilkinson #1	88	28	1	1.5	2,703.3	2,701.7
Godley Lane #1	81	22	1	1.5	2,697.9	2,696.3
Gekeler #1	84	28	NA	NA	2,697.0	2,693.2
Highway 237 #1	131	42	2	3	2,698.0	2,695.8
Booth Lane #1	124	36	2	2	2,691.5	2,689.5
Elmer #1	130	21	NA	NA	2,694.6	2,692.0
Elmer #2	119	18	4	1	2,688.7	2,687.4
Market Lane #1	108	35	NA	NA	2,691.9	2,685.4
Alicel #1	147	33.5	1	2	2,695.3	2,692.3
McKennon Lane	192	29	1	2.5	2,695.9	2,691.2
Hull Road	169	36.22	1	3	2,694.6	2,689.6
Striker Lane	140	28.5	NA	NA	2,689.7	2,685.0
Rinehart Lane	194	32	1	3	2,688.3	2,687.7

Bridge Name	Bridge Opening (ft)	Width (ft)	Piers	Pier Thickness (ft)	Top Deck Elevation (ft)	Low Chord Elevation (ft)
Booth Ln #2	114	36	2	1.3	2,705.1	2,703.4
Market Lane #2	101	28	NA	NA	2,701.0	2,696.7
Ruckman Lane	120	28	NA	NA	2,698.4	2,694.1

Diversions

Nine diversion dams were included in the HEC-RAS model. Only the grade control portion of a diversion dam was included. For example, Medical Springs Diversion #D2 is a series of notched weirs (shown in Figure 6). To include this structure, the four adjacent cross sections were adjusted in a similar manner to the bridge cross sections. Then, the highest weir (typically the most upstream) elevations, width, and dimensions were input as an inline weir structure in HEC-RAS. The highest weir acts as a water surface control and the other weirs would have only a small local effect on the hydraulics.

Figure 6. Medical Springs #D2 Diversion (aka Catherine Creek Adult Collection facility, CCACF) looking upstream. Photo courtesy of AP, taken on November 16, 2010.

Three of the diversion dams (Elmer, Lower Davis, and Upper Davis) have boards placed to increase the backwater behind the dams during the irrigation season. The surveys of all three structures were conducted while boards were in place (see Figure 7). The usage of the boards, such as how many are in place and for what months of the year, is not well documented and highly variable. Therefore, the structures were all modeled assuming that no boards were in place and irrigation was not occurring. Additionally, the primary purpose of the model is to evaluate high flows, during which time the boards would likely be removed. Further refinement of the modeled structures can occur if the operating conditions of the dams are defined.

Figure 7. Photo of Diversion Structure located at station 67045 on Catherine Creek, illustrating the use of boards to increase backwater. Photo from Anderson Perry Surveys, September 28th 2010.

3.1.5. Model Discharges

Thirteen model discharges were developed for multiple recurrence intervals using available streamflow gage data. For details of the hydrology analysis performed to develop the discharges, refer to Appendix A. Table 3 below shows the flow change locations in the model and associated flow magnitudes for the 1.5-, 2-, 10-, 50-, and 100-year flood, which represent the discharges used in this modeling effort.

Table 3. Flow changes locations and discharges for various flood return intervals.

Hydraulic Model Station (ft)	RM (mi)	Description	Flood return interval (Q xx) discharges (ft^3/sec)				
			Q1.5	Q2	Q10	Q50	Q100
247207.5	46.7	Catherine Creek near Union, stream gage	645	760	1,228	1,628	1,796
209189.3	39.5	Catherine Creek at Union, stream gage	677	797	1,288	1,707	1,884
194813.4	36.9	Catherine Creek below Pyles Creek	941	1,109	1,791	2,374	2,619
189174	35.9	Catherine Creek below Little Creek	973	1,146	1,851	2,454	2,708
165306.4	31.4	Catherine Creek below Ladd Creek	1,325	1,562	2,522	3,344	3,689
153863.5	29.4	Catherine Creek below McAlister Slough	1,355	1,598	2,580	3,421	3,774
125641.6	24.1	Catherine Creek below Mill Creek	1,546	1,822	2,942	3,900	4,303
116101	22.5	Catherine Creek below Old Grande Ronde River Channel	1,632	1,924	3,107	4,119	4,544
80916.28	15.8	Catherine Creek below Eckesley Creek	1,763	2,078	3,356	4,450	4,909
66414.06	0	Grande Ronde River below Catherine Creek	4,456	5,376	9,547	13,672	15,564
12225.36	NA	Grande Ronde River below Willow Creek	4,779	5,757	10,162	14,488	16,464
28848.65	NA	State Ditch	2,692	3,297	6,190	9,222	10,655

3.1.6. Model Boundaries

The downstream boundary condition is located on the Grande Ronde River approximately 12.6 miles downstream of the confluence of Catherine Creek and State Ditch. No in-channel data were collected downstream of the Rinehart Bridge, which is located at about station 6800. As a result, the initial bed elevations used in the downstream-most 9 cross sections of the model were extracted directly from the LiDAR and were several feet higher than elevations surveyed in upstream cross sections. This led to a drastic change in slope at the downstream boundary and caused unrealistic backwater up to 6 miles upstream for low flows (approximately 96 ft^3/sec) (Figure 8). To remedy this problem, bed elevations within these 9 cross sections were dropped on average 2 feet to create a low flow channel with a slope of about 0.02%, which is consistent with localized slopes within the reach. An example of the modifications made to cross sections is shown in Figure 9. In the future, it is recommended that additional topographic data be collected in the channel for the downstream 6,800 feet of the model.

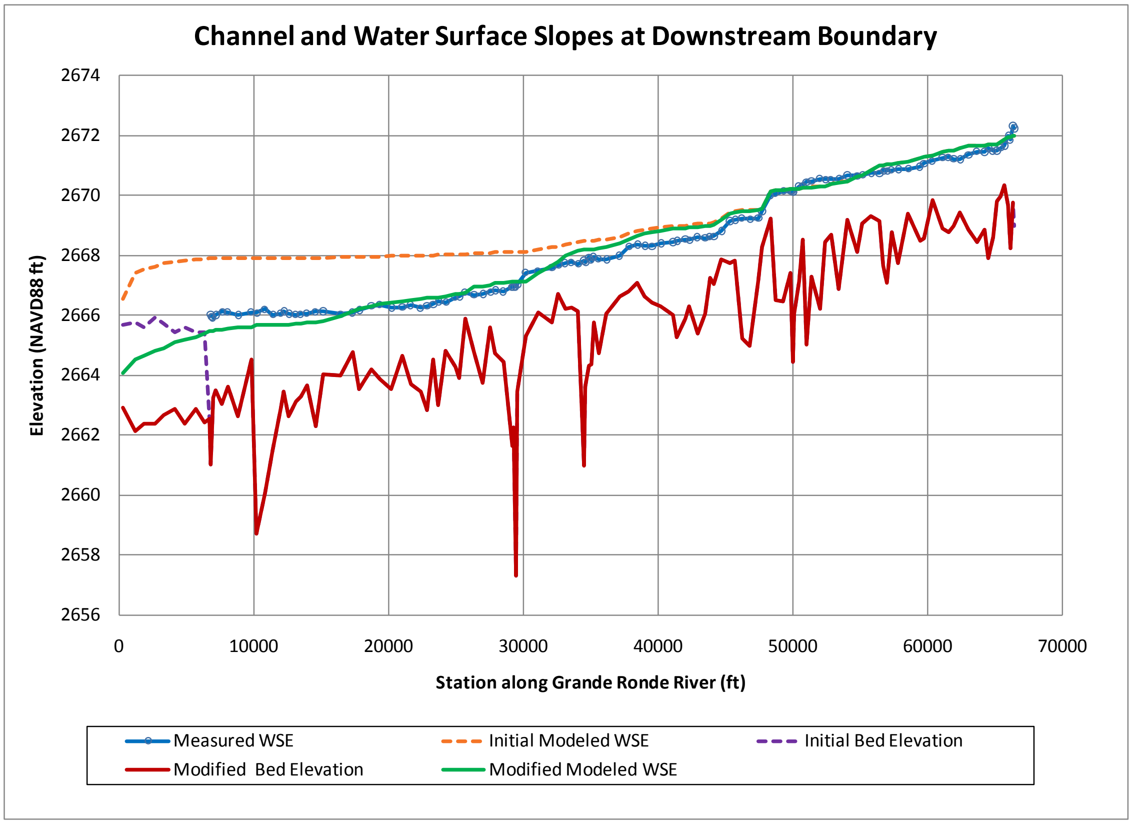

Figure 8. Illustration of changes to bed elevation at downstream end of model and resultant changes in water surface profile for a flow of 95 cfs.

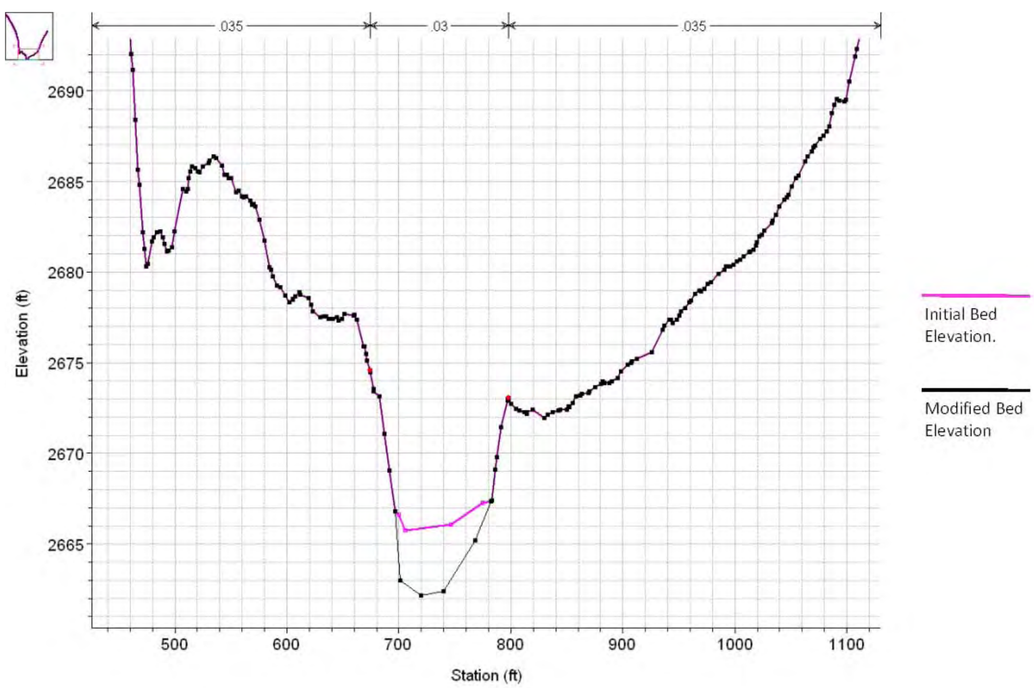

Figure 9. Example of channel bed modification to downstream-most 9 cross sections. Cross section shown is at station 1198.25.

No data regarding a stage/discharge relationship is currently available at the downstream end of the model; therefore, it was assumed that the boundary condition operates under normal depth conditions. The downstream boundary normal depth slope was varied for the June, 2010 discharge (see Section 3.2.1) to determine the slope that most closely simulates the high water marks. Figure 10 shows the results of several different slopes. Once the slope was greater than 0.3%, the water surface elevation values changed very little and only in the downstream most mile. Although the 0.3% slope does appear to be overestimating the water surface elevation, the modeled water surface elevations are within 1 foot of the measured high water marks. Therefore, a slope of 0.3% was used for all additional simulations. Assuming normal depth may not be an accurate assumption at the boundary; a rating curve would be ideal to capture the hydraulics at this location. If more data become available, this downstream boundary condition can be refined and extended.

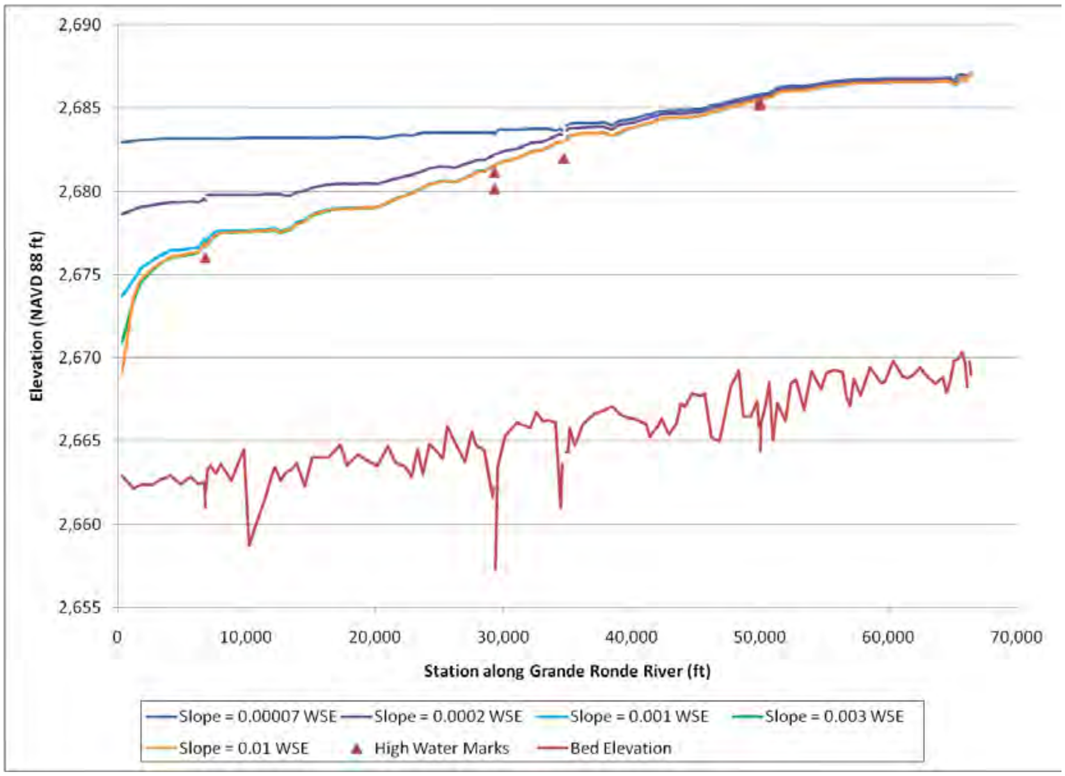

Figure 10. Comparison of different downstream boundary condition slopes for the June, 2010 discharge.

Based on the results of the boundary condition variation, the model appears highly sensitive to the downstream boundary condition. Using a slope of 0.007% results in backwater at the downstream boundary upstream to Grande Ronde River station 66,200 or approximately 500 feet downstream of the confluence of State Ditch and Catherine Creek. Although this slope is not recommended for input at the downstream model boundary, it illustrates that the boundary condition is not likely to impact model results in Catherine Creek or State Ditch.

3.1.7. Roughness Values

Roughness values were determined using a combination of pebble counts, vegetation, agricultural land use, and professional judgment. Pebble counts were collected by the Technical Service Center along State Ditch in November 2010 and by PNRO along Catherine Creek upstream of Pyles Creek confluence at RM 36.9 in summer 2010 (Rob McAffee, personal communication). The Manning's-Strickler equation was used to estimate grain roughness based on the pebble counts in these areas. The Manning's roughness values within the channel were then increased slightly to account for other components of form roughness present, such as vegetation along the channel banks and large woody debris within the channel. In modeled reaches where no bed material data were available other than visual observations, Manning's roughness coefficients were estimated based on similar values used in other rivers with similar bed material conditions. Roughness coefficients for floodplain areas were delineated based upon the presence of vegetation cover or agricultural land use. Both floodplain and in-channel roughness values were consistent with guidance presented by Chow (1959). Table 4 below summarizes the Manning's n values used.

Table 4. Hydraulic roughness values used in the HEC-RAS model.

River	Hydraulic Model Station (ft)		Manning'n value		
	From	To	Left	Channel	Right
Catherine Creek	247207.5	194387.8	0.075	0.045	0.075
Catherine Creek	194387.8	709.4	0.035	0.030	0.035
Grande Ronde	66414.1	302.5	0.035	0.030	0.035
State Ditch	28848.7	22875.8	0.035	0.030	0.035
State Ditch	22875.8	54.1	0.035	0.040	0.035

For the floodplain areas, except the upstream end of Catherine Creek, the roughness was determined assuming the area was agricultural land with crops such as mint. Figure 11 shows an example of the land adjacent to Catherine Creek.

Figure 11. Example of the floodplain along Catherine Creek. Photo taken April 27, 2010.

At the upstream end of Catherine Creek, agricultural areas are more sparse, and more trees and high relief areas are present (Figure 12). The roughness in the floodplain was increased for this area from 0.035 to 0.075.

Figure 12. Example of the floodplain along the upstream portion of Catherine Creek in Reach 4. Photo taken April 28, 2010.

Attempts were made to validate the in-channel portion of the model using high flow and low flow discharge and water surface information. Sensitivity analyses were performed to better understand how potential variability in Manning's roughness may impact model results (Section 3.2.7). In the future, aerial photography could be used to delineate land cover to capture variability in roughness across the floodplain. However, the level of effort to accomplish these tasks may be more efficiently performed at a more detailed level of investigation and input to a two dimensional (2D) model.

3.1.8. Other computational parameters

Coefficients of expansion and contraction of 0.3 and 0.1, respectively, were used at all sections except upstream and downstream of the bridges. For these cross sections 0.5 was used for the coefficient of expansion and 0.3 for the coefficient of contraction. For the bridges, the weir coefficient used varied between 2.6 and 3.05. A weir coefficient of 3.05 was used for all of the diversion structures. These values could be further calibrated in the future if more measured water surface elevation data become available.

3.2. Model Comparison

Two basic data sets were available to calibrate the model: high water marks were available from a flood that occurred in June 2010, and a water surface profile was measured in October 2010.

3.2.5. June 2010 Water Surface Elevations

In June 2010, a flood occurred in which discharge measurements were collected after the peak, but PNRO placed 21 high water marks at 11 bridges on Catherine Creek, 6 high water marks at 4 bridges on Grande Ronde River and 1 mark on a bridge on State Ditch. The marks were all placed on June 4th and June 5th. Discharge data corresponding to these dates were extracted from the three Oregon Water Resources Department (OWRD) stream gages: Catherine Creek near Union, Catherine Creek at Union, and Grande Ronde River near Perry. The highest mean daily flow occurred on June 4th (see Table 5).

Table 5. Discharge values at OWRD gages on June 4, 2010.

Gage Description	**Hydraulic Model Station**	**Mean daily flow on June 4, 2010 (ft^3/sec)**
Catherine Creek near Union	247,208	1,230
Catherine Creek at Union	209,189	1,290
Grande Ronde River near Perry	NA (upstream on State Ditch)	4,180

Three discharge measurements were collected on June 7th after the peak, and three water surface elevation markers from June 7th were surveyed by Anderson Perry. The discharge measurements were collected at bridge locations: Market Lane Bridge on State Ditch, Godley Road Bridge on Catherine Creek and Booth Lane on Catherine Creek. The measurements were collected using a Teledyne RDI StreamPro Acoustic Doppler Current Profiler (ADCP). At least six transects were collected at each location, processed using WinRiver software, and averaged to calculate the flow. The discharge measurements confirmed that the gage discharge data were reasonable (Table 6), however there were not enough water surface elevation markers collected to use the June 7th discharge data independently.

Table 6. Measured discharge values collected on June 7, 2010.

Discharge Measurement location	Hydraulic Model Station	Flow Measured on June 7, 2010 (ft^3/sec)
Godley Road Bridge on Catherine Creek	15,220	821
Booth Lane Bridge on Catherine Creek	78,063	1,060
Market Lane at State Ditch	8,897	2,600

Appendix A contains area and precipitation volume ratios developed for the tributaries. Using the gages and the ratios developed, the discharges for June 7, 2010 in Table 7 were calculated. These computed discharges were used to simulate the water surface elevations resulting from the June 2010 flood.

Table 7.Discharge values used in HEC-RAS model for June, 2010 simulation.

Hydraulic Model Station (ft)	RM (mi)	Description	June, 2010 Peak Discharge (ft3/sec)
247207.5	46.7	Catherine Creek near Union, stream gage	1,230
209189.3	39.5	Catherine Creek at Union, stream gage	1,290
194813.4	36.9	Catherine Creek below Pyles Creek	1,794
189174	35.9	Catherine Creek below Little Creek	1,854
165306.4	31.4	Catherine Creek below Ladd Creek	2,526
153863.5	29.4	Catherine Creek below McAlister Slough	2,584
125641.6	24.1	Catherine Creek below Mill Creek	2,947
116101	22.5	Catherine Creek below Old Grande Ronde River Channel	3,112
12225.36	NA	Grande Ronde River below Willow Creek	7,292
28848.65	NA	State Ditch	4,180

Comparisons of the measured high water marks versus modeled water surface elevations on Catherine Creek are presented in Figure 13 through Figure 16. Results suggest that the model provides conservative estimates of water surface elevations based upon the input data. All but one location have the modeled water surface elevations within 2 feet of the measured high water marks. Five of the eleven locations have modeled water surface elevations within 1 foot of the measured high water marks.

In an attempt to better match the high water marks, Manning's n values were reduced in the lower section of Catherine Creek (station 709.4 to 194387.8) to 0.031 on the floodplain and 0.026 in the channel. An example of the changes resulting from the reduced Manning's values is shown in Figure 13. From this analysis, it was determined that the disagreement between the high water marks and the modeled elevations is not a function of roughnesss alone. To match just a few of the high water marks exactly, roughness coefficients would need to be reduced to unreasonable values. The differences are more likely due to (1) uncertainty of the high water marks relative to discharge values, (2) the simplified representation of in-channel topography, or (3) lack of representation of localized hydraulics at bridges (where the high water marks were collected). Based on professional judgment, the Manning's n values were not adjusted from the values in Section 3.1.7 for the remainder of the analysis. Sensitivity analyses were performed to better understand how potential variability in Manning's roughness may impact model results (Section 3.2.7).
The Grande Ronde River comparison was shown above in Figure 10 and the modeled water surface elevations are within 1 foot of the measured high water marks. For State Ditch, one high water mark was collected at Booth Lane Bridge and had an elevation of 2,694.3. The modeled water surface elevation at this station was 2,694.2 or within a tenth of a foot of the high water mark.

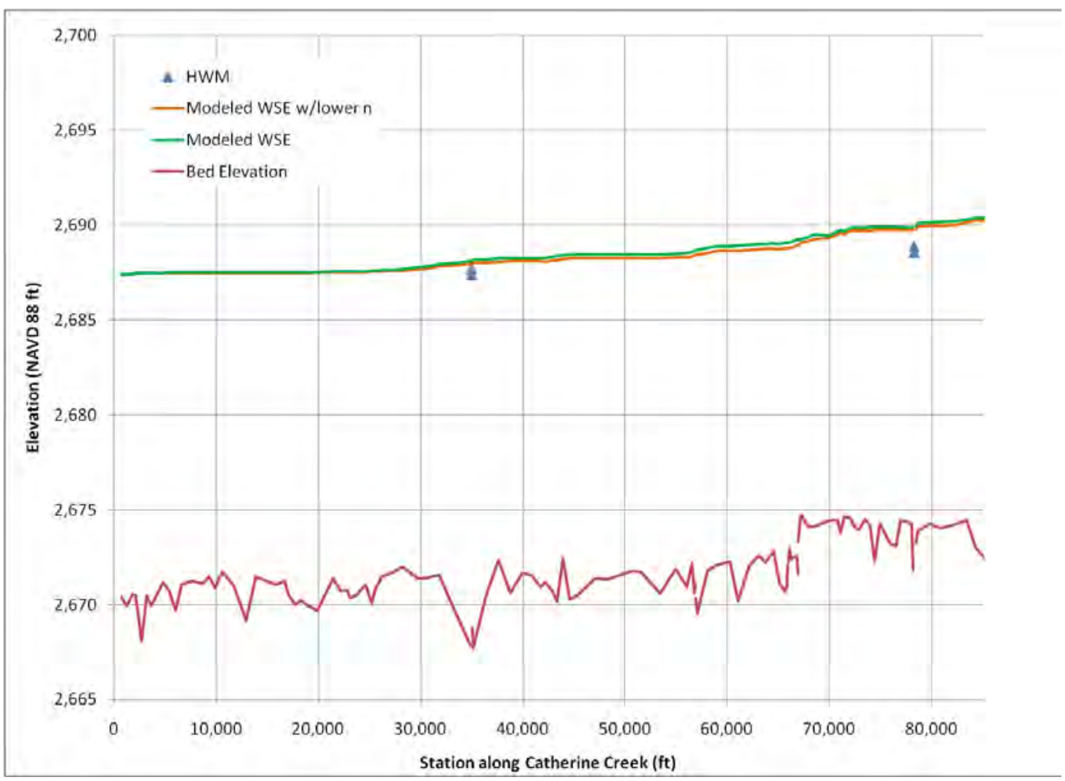

Figure 13. Modeled water surface elevations versus measured high water marks for Catherine Creek (RM 0 to 16.3).

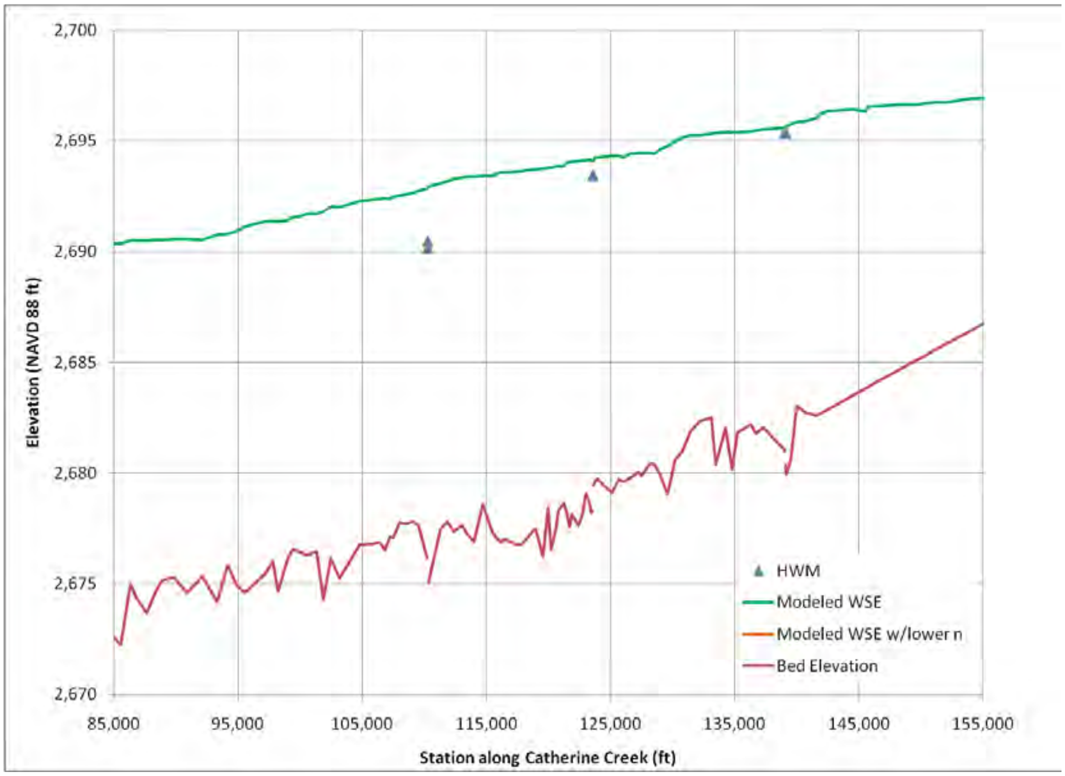

Figure 14. Modeled water surface elevations versus measured high water marks for Catherine Creek (RM 16.3 to 29.6).

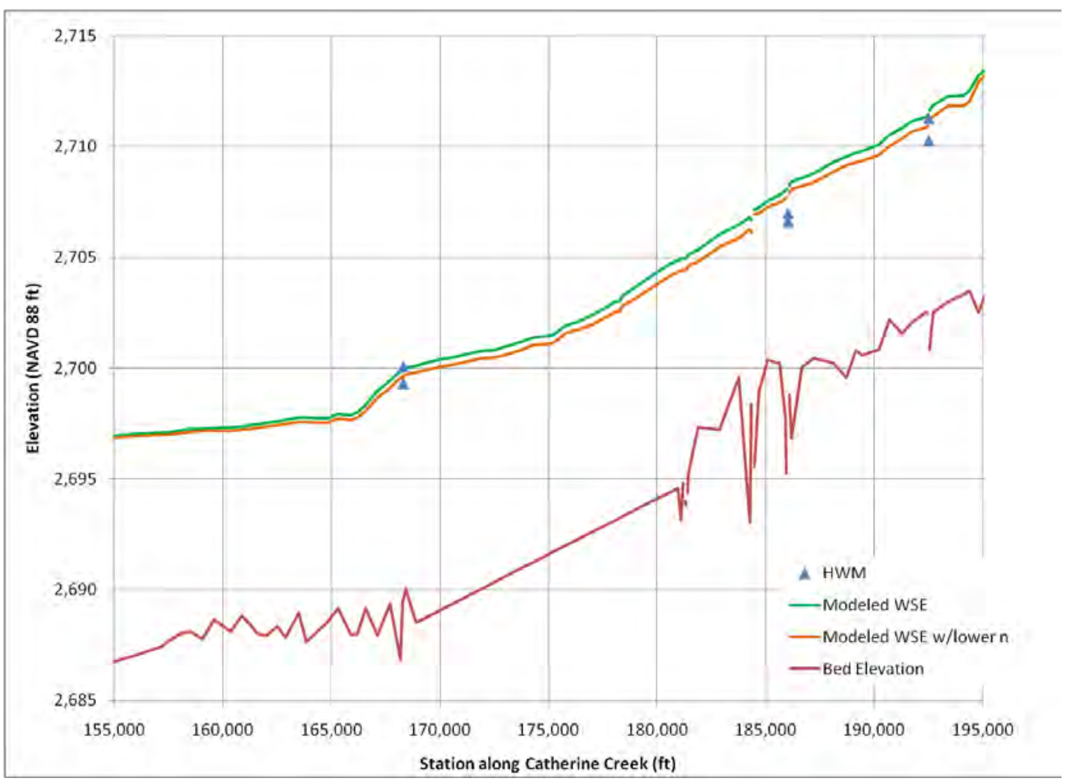

Figure 15. Modeled water surface elevations versus measured high water marks for Catherine Creek (RM 29.6 to 36.5).

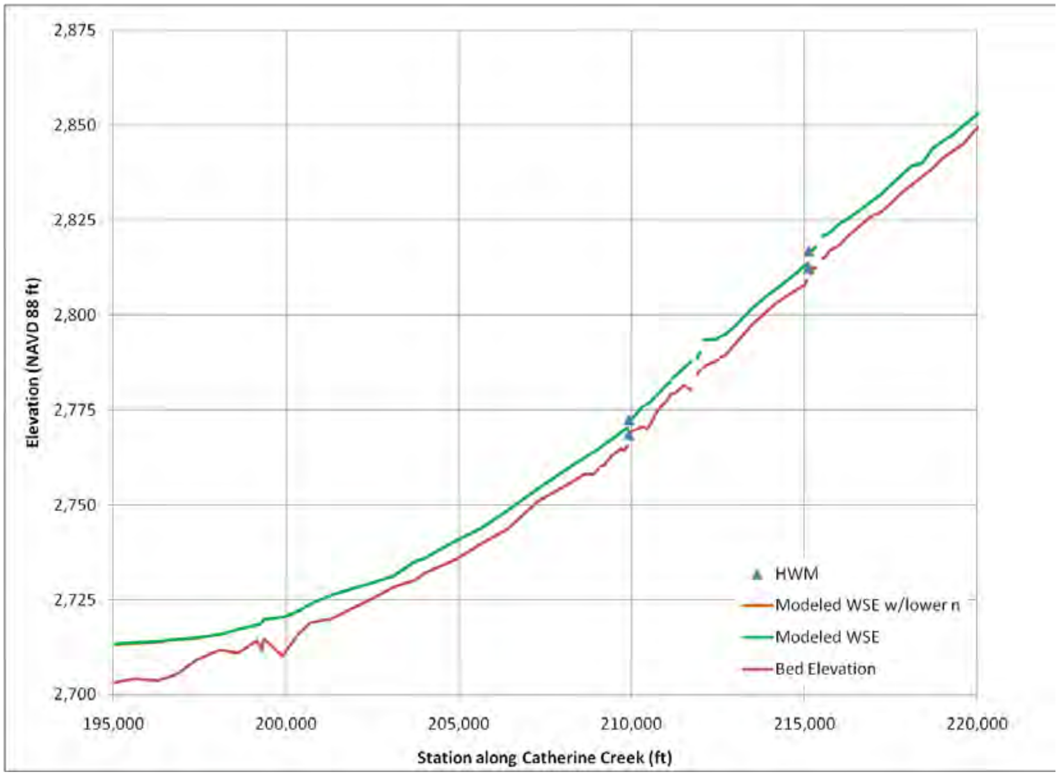

Figure 16. Modeled water surface elevations versus measured high water marks for Catherine Creek (RM 36.5 to 46.6).

3.2.6. October 2010 Water Surface Elevations

Although the model produced for this Tributary Assessment is intended to be a tool for investigating high flows, uncertainties were associated with the discharges at which the high water marks were collected as described in Section 3.1.2. In addition, high water marks within a close proximity sometimes varied by over 1 foot in elevation. A comparison of the model results to measured low flow water surface elevations collected during October and November 2010 was performed to evaluate the validity of the in-channel Manning's roughness coefficient and the downstream boundary conditions.

Water surface profiles were collected by boat between October 29th and November 2nd, 2010. Discharge data corresponding to these dates were extracted from the three OWRD stream gages: Catherine Creek near Union, Catherine Creek at Union, and Grande Ronde River near Perry. Discharges and surveyed reaches are shown by date in Table 8. Aside from discharge, all model parameters as described in Section 3.1.8 remained the same.

Table 8. Discharges and water surface elevation survey extent by day.

River	Upstream RS	Downstream RS	River Miles	Date Surveyed	Discharge (cfs)
Catherine Creek	192460	157500	36.5 - 30.0	10/29/2010	33
Catherine Creek	140685	84300	27-16.3	10/30/2010	33
Catherine Creek	84300	0	16.3-0	11/2/2010	44
State Ditch	29072	0	NA	10/31/2010	46
Grande Ronde	66103	6791	NA	11/1/2010	96

As discharge in a channel increases, the roughness associated with the channel generally decreases. Therefore, the Manning's roughness used to model high flows is generally not going to be the same as the Manning's roughness that will best match water surface elevations at low flows. Manning's roughness was not modified to match low flows in this model since its primary function is to predict high flow inundation patterns. However, the comparison is useful in understanding the model limitations and in determining additional data needs and model changes for future evaluation of lower flows.

In addition, multiple diversion dams within the system alter the hydraulics of the channel at low flows, particularly when stop logs or boards are in place to prevent flow over the weir, as was the case when the low flow surveys were collected. The model was developed with the consideration that no boards were in place because the operations of the diversion structures will likely change during high flows, when stop logs are less necessary to create the needed head for diversion.

Comparisons of the modeled versus measured water surface elevations are presented in Figure 17 through Figure 21. Within the Grande Ronde River from the downstream model boundary to the confluence with State Ditch and Catherine Creek, measured and modeled water surface elevations are very close and are generally within 0.3 feet at a discharge of 96 ft^3/sec (Figure 17). One reason for similarities between the measured and modeled water surface elevation was the adjustment of the in channel bed elevations near the downstream boundary (Grande Ronde station 0 to 6800). The comparison between measured and simulated water surfaces at low flow show that the major hydraulic controls in the reach are captured by the topographic information.

Within State Ditch, the comparison illustrates that the modeled water surface elevations are lower than the measured water surface elevations, typically by less than 0.5 feet but up to 1 foot in some localized areas (Figure 18). Increasing the Manning's n for lower flows will improve the agreement to the low flow data, but also results in greater discrepancies at high flow. Despite the underestimation of water surface elevations at low flows, the model is within 1 foot of the measured values for the majority of the reach.

In the downstream 12.5 miles of Catherine Creek, the match between the measured and modeled water surfaces is excellent, with maximum differences of 0.3 feet (Figure 19). However, the diversion structure located at stations 67045 (Elmer Diversion Dam, Figure 7) had boards in place that were not included in the model. Therefore, simulated water surfaces at the diversion and in the upstream backwater, which extends nearly 8 miles upstream, do not match the measured. Upstream of the backwater of Elmer Dam, the model is consistent with the measured water surface elevation up to station 130806 (RM 24.9), where a beaver dam was present during the data collection trip (Figure 22). Upstream of the backwater influence of the beaver dam, the modeled water surface is slightly higher than the measured water surface from approximately station 158000 to 167700, a distance of 1.8 miles. The maximum difference in this section is approximately 0.8 feet. Some of the difference may be attributable to flow diversions at two upstream dams; however, no gage records or measured discharges are available for verification.

On October 29th, a 2.5 mile section of river from station 167700 to 181000 was not surveyed due to access constraints (Figure 21). Within this channelized reach, no data are available for comparison and major hydraulic controls may not be adequately represented in the topography. At the upstream end of the surveyed data on October 29th, two diversion structures were present: Lower and Upper Davis Diversion Dams. They were both surveyed by Anderson Perry and included in the model. Each of these structures had boards in place that were not present in the model, and therefore comparison in this section of the model is not applicable. However, the modeled water surfaces are substantially lower, with differences exceeding 5 feet at the Upper Davis Dam diversion structure. This may indicate that downstream controls were not surveyed that could be impacting upstream water surface elevations within the vicinity of Lower Davis Dam. More topographic data are necessary to verify the accuracy of model results in this area.

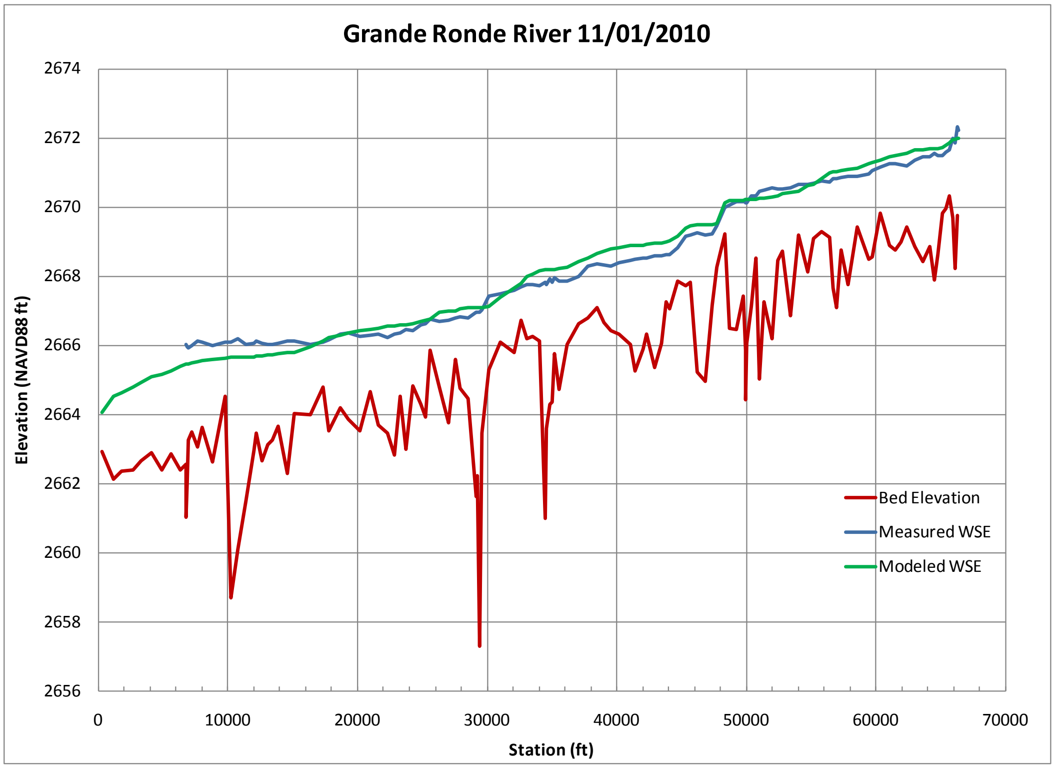

Figure 17. Measured versus modeled water surface elevations for Grande Ronde River downstream of State Ditch, measured on 11/01/2010.

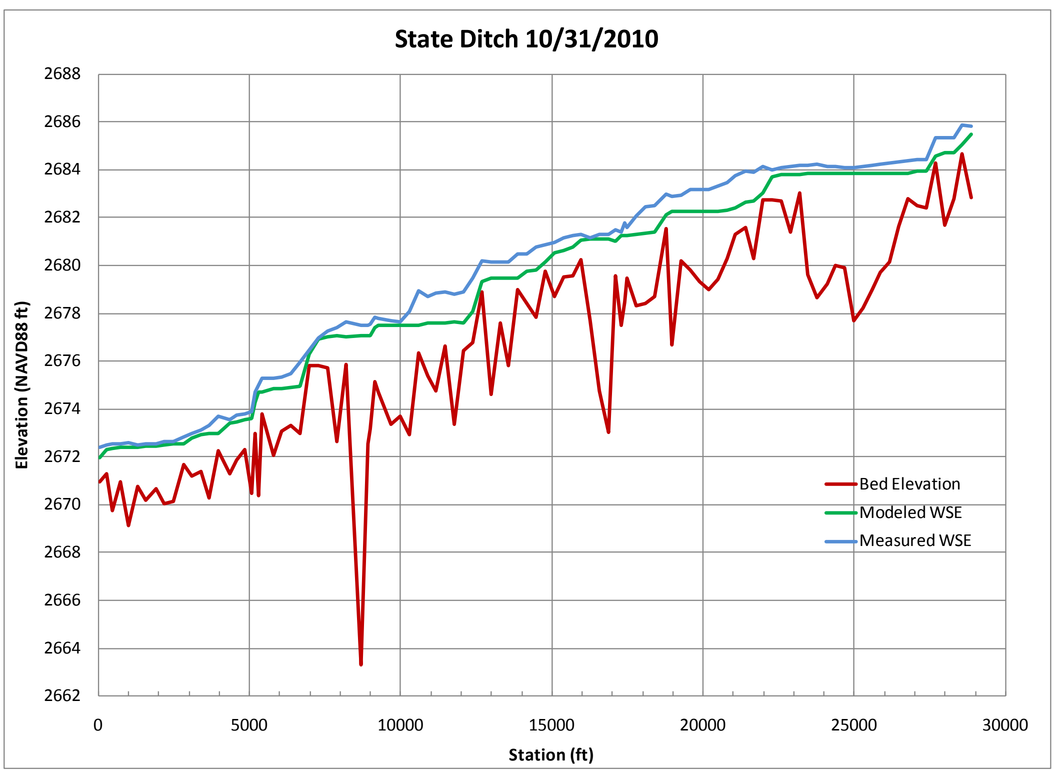

Figure 18. Measured versus modeled water surface elevations along State Ditch, measured on 10/31/2010.

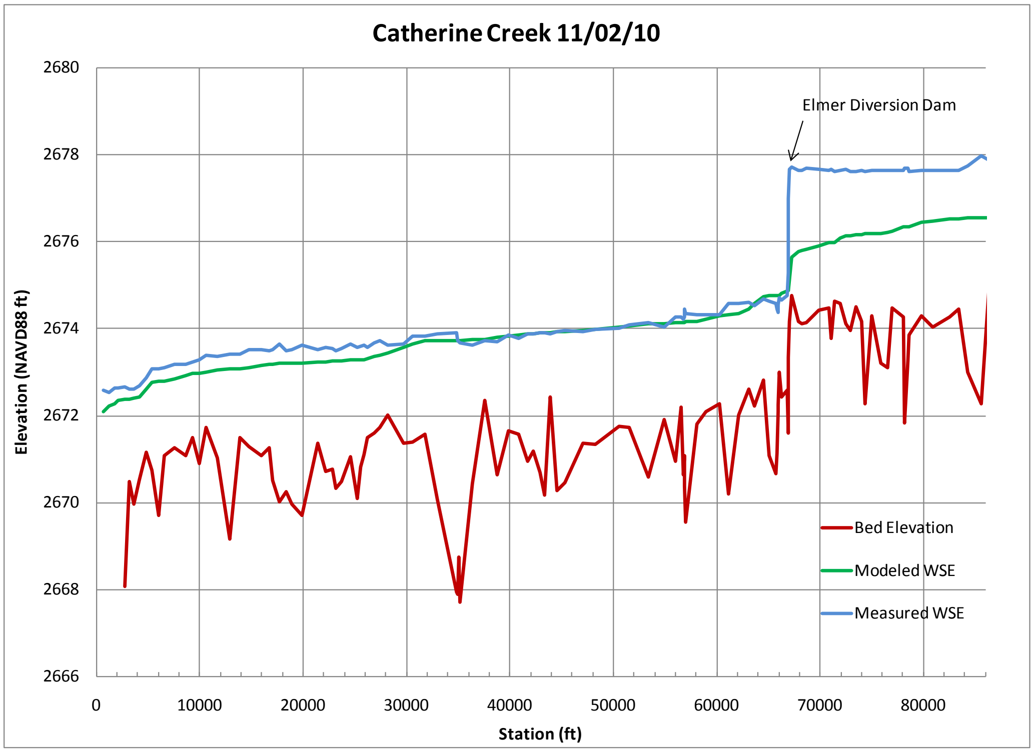

Figure 19. Measured versus modeled water surface elevations for Catherine Creek RM 0 to RM 16.3, measured on 11/02/2010.

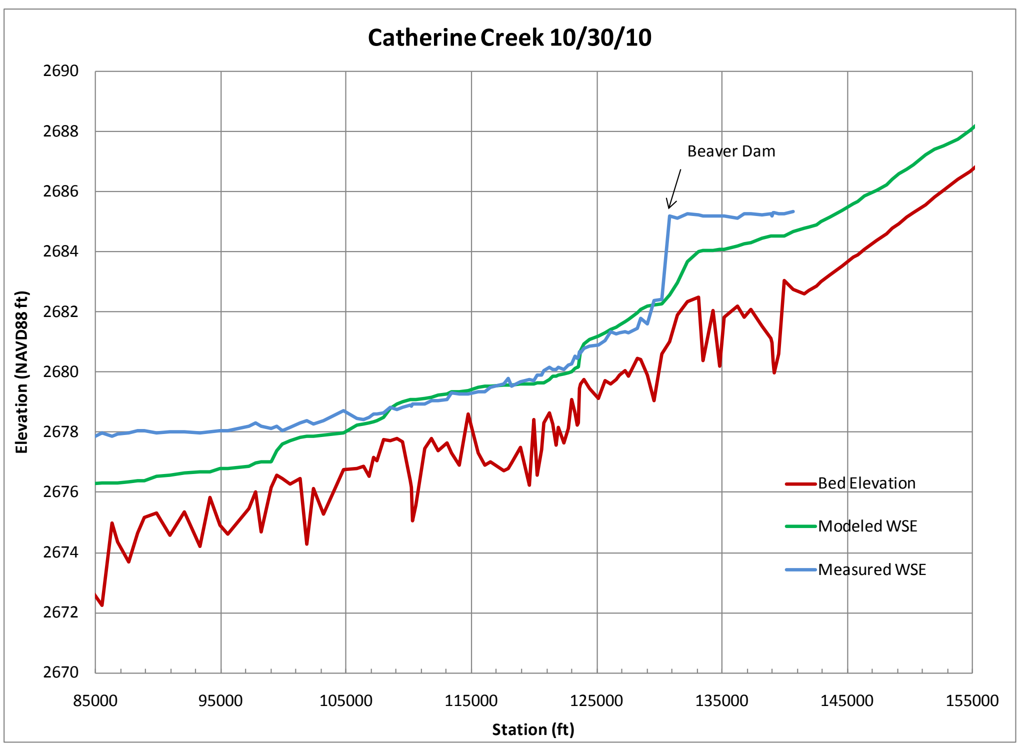

Figure 20. Measured versus modeled water surface elevations for Catherine Creek RM 16.3 to RM 29.6, measured on 10/30/2010.

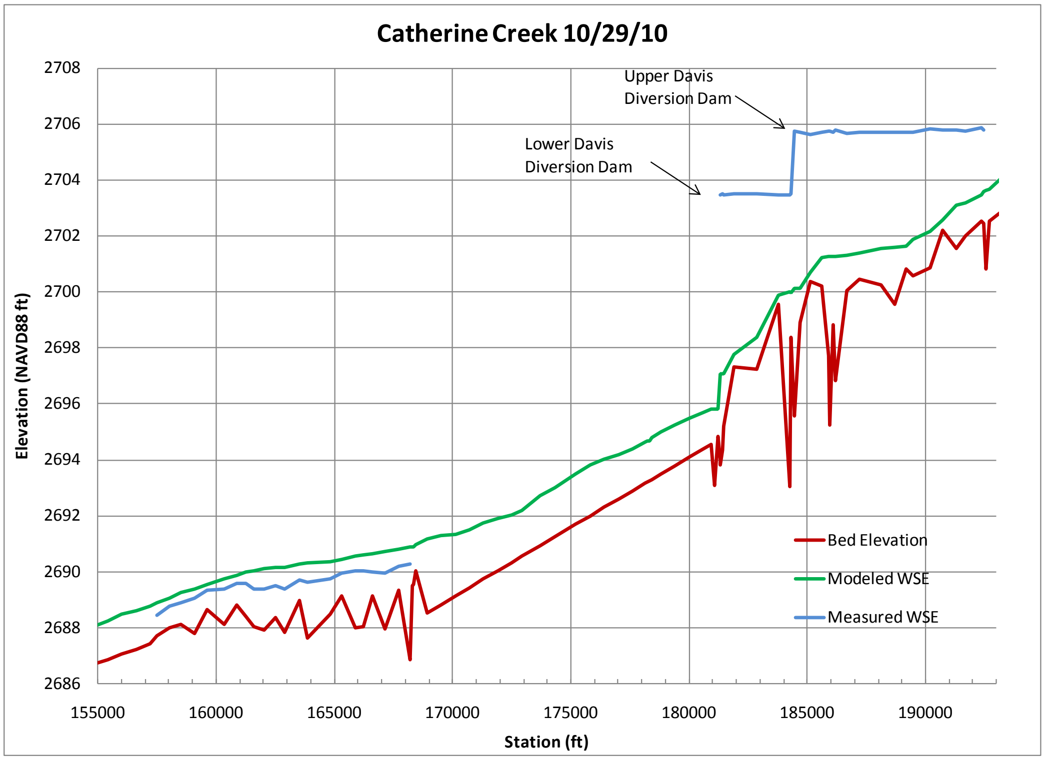

Figure 21. Measured versus modeled water surface elevations for Catherine Creek RM 29.6 to RM 36.5, measured on 10/29/2010.

Figure 22. Photograph of Beaver Dam located at RM 24.9 taken October 30, 2010.

3.2.7. Hydraulic Roughness Sensitivity

Within this Tributary Assessment, Manning’s n values were selected based upon field observations, professional judgment, and surveyed high water marks as discussed in Section 3.1.7 and 3.2.1. However, some uncertainties are associated with the high water marks and with the simulation of the water surface elevations. For example, high water marks within a close proximity sometimes varied by over 1 foot in elevation. In addition, it is difficult to confirm that the high water marks do represent high water since they were placed during the flooding and in some locations, based on field observations, appeared to potentially be lower than the true high water mark. Hydraulic roughness sensitivity analyses were conducted using the 1.5-year and 10-year discharges. These discharges were selected to represent changes associated with in-channel roughness values and floodplain roughness values. For the 1.5-year discharge, the majority of flow was conveyed within the channel for most cross-sections. Using the 10-year discharge, floodplain flows dominated in areas where overbank flooding is not precluded by topographic influences (high levees or terraces), and most cross sections were capable of containing flows based upon available terrain data. During the sensitivity runs, the Manning’s roughness values of the channel and floodplain were increased and decreased by 0.005 to investigate potential modifications to the results from differing hydraulic roughness coefficients.

The sensitivity simulations predicted high sensitivity to Manning’s n for reaches with low slopes, including Grande Ronde River, State Ditch, Catherine Creek Reach 1 and the downstream 10-miles of Reach 2. Within these reaches, increases or decreases of 0.005 in the in-channel or overbank roughness values result in differences of +/- 1 foot in the water surface for the 1.5-year discharge and differences of +/- 0.5 to 0.75 feet for the 10-year discharge. In reaches 3 and 4 of Catherine Creek, the sensitivity to changes in Manning’s n is greatly reduced. In reach 3, modifications to the roughness results in a maximum increase or decrease of 0.5 feet for the 1.5-year discharge and 0.25 feet for the 10-year discharge. Within Reach 4, maximum differences in water surface are +/- 0.25 feet for the 10-year discharge. Variations in Manning’s n values appear to impart less influence in areas of steeper slopes and as flows increase. Plots of the water surface elevations resulting from the variations in Manning’s n for Reaches 1 and 2 of Catherine Creek are shown below (Figure 23 to Figure 26).

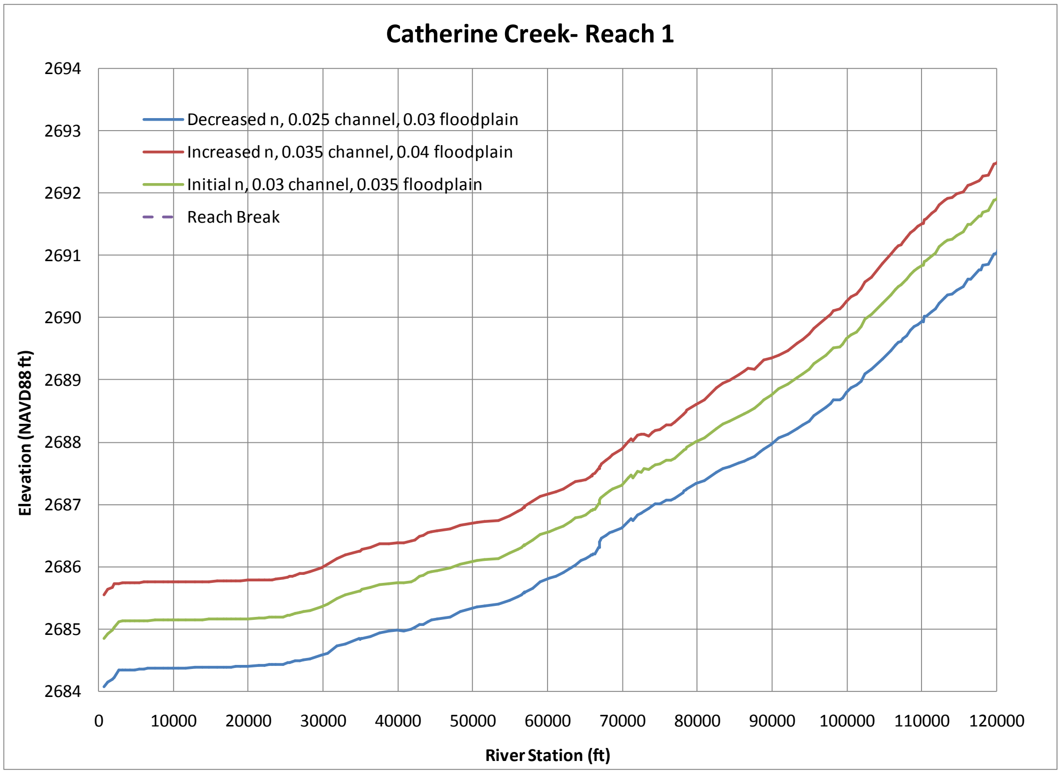

Figure 23. Sensitivity to Manning's n at a 1.5 year discharge for Reach 1 of Catherine Creek.

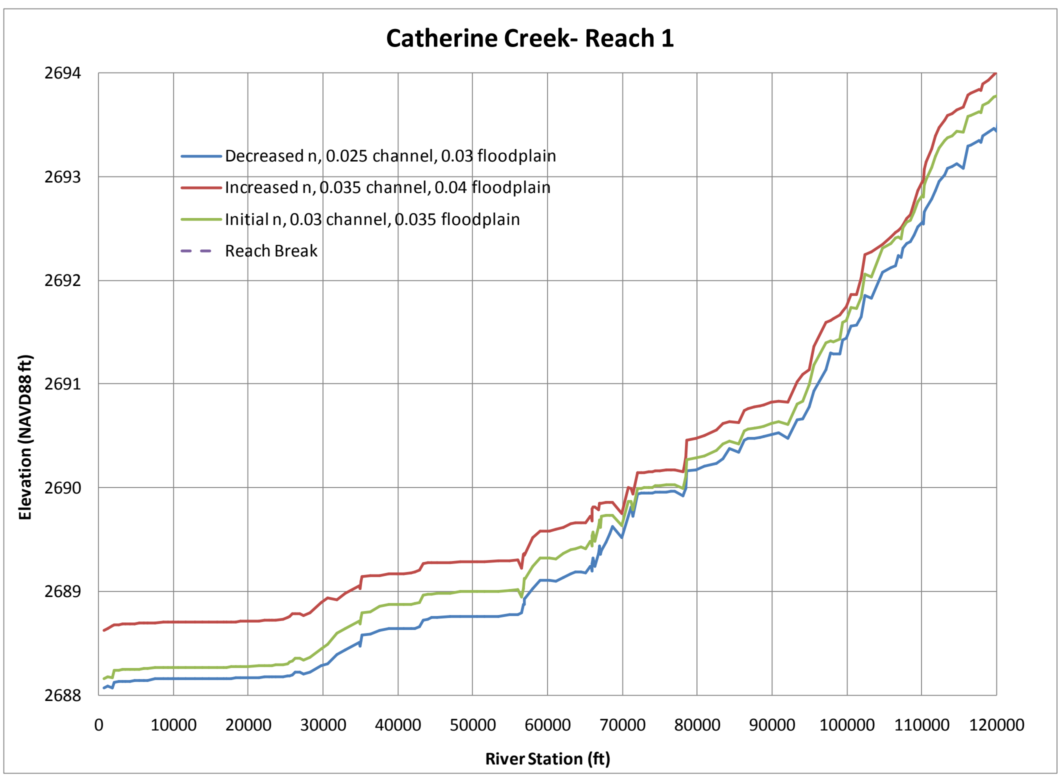

Figure 24. Sensitivity to Manning's n at a 10- year discharge for Reach 1 of Catherine Creek.

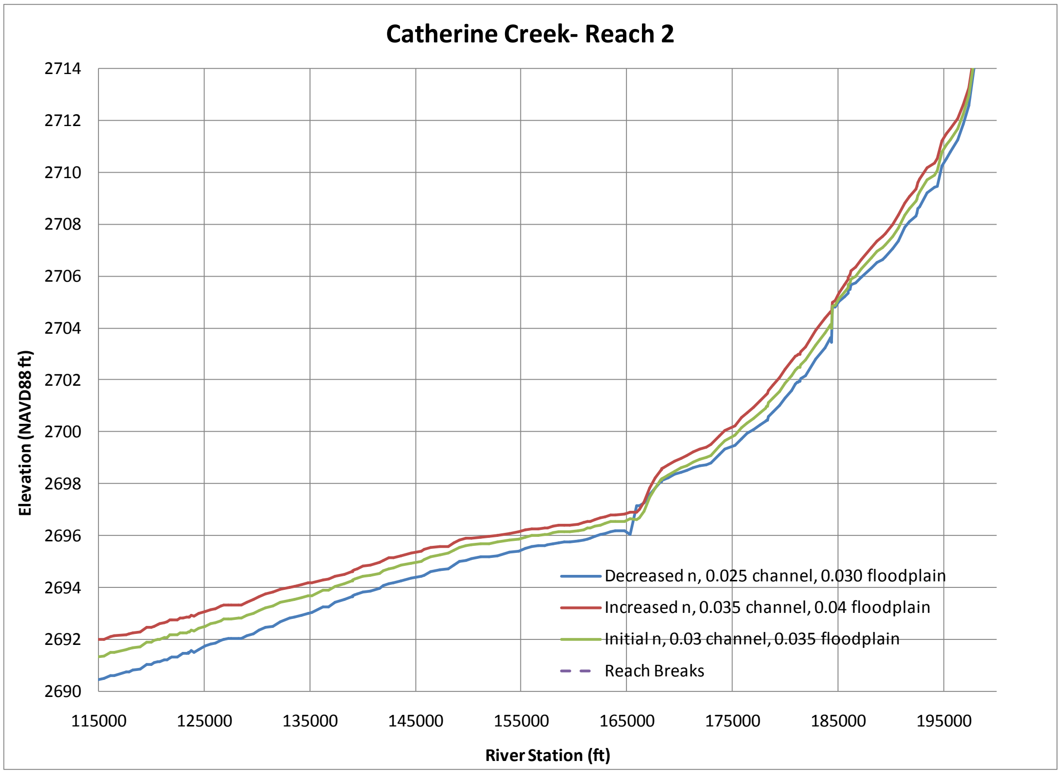

Figure 25. Sensitivity to Manning's n at a 1.5-year discharge for Reach 2 of Catherine Creek.

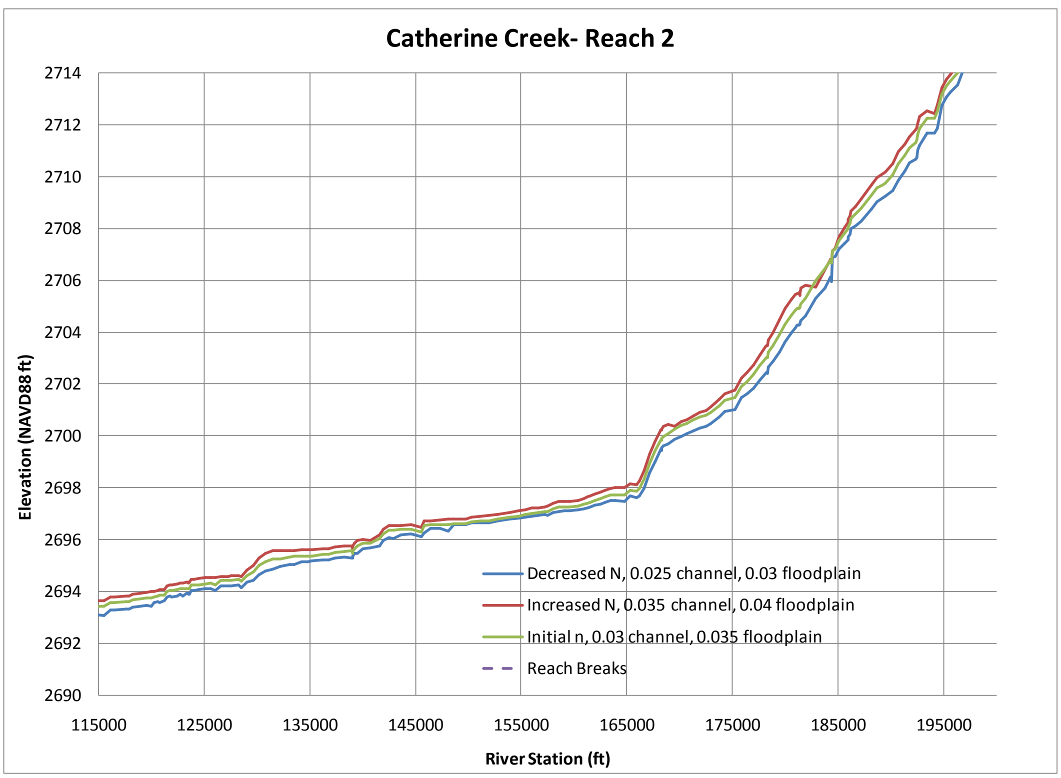

Figure 26. Sensitivity to Manning's n at a 10- year discharge for Reach 2 of Catherine Creek.

4. Present Conditions based on Model Results

In Appendix C, channel reach breaks along Catherine Creek were delineated based on the common geomorphic characteristics. There were seven reach breaks defined, four of which are included (reaches 1-4) in this hydraulic model. Only these four Catherine Creek sections are included in the present conditions analysis of the hydraulic model presented below. These reach breaks could be further refined based on the model results, which often had high variability within a reach. As an example, there is a slope break in Reach 2 at Ladd Creek. In addition there are other changes to channel capacity and velocity characteristics at this location. The refining of reach breaks may be useful for future analyses. Table 9 below briefly describes each reach. Figure 27 shows the longitudinal location of the reach breaks along Catherine Creek.

Table 9. Catherine Creek Reach break description.

Reach	River Miles (RM)	Model Station (ft)	Description
1	0 – 22.5	0 – 116514.9	Historically the Grande Ronde River which included Catherine Creek. Reach break occurs at historic confluence with the “Old” Grande Ronde River.
2	22.5 – 37.2	116514.9 – 196810.7	Reach break occurs at the lower end of an alluvial fan where the substrate,bank material, and valley slope change.
3	37.2 – 40.8	196810.7 – 216006.4	Encompasses the Catherine Creek alluvial fan and all of the Town of Union, OR.
4	40.8 – 45.8	216006.4 – 242735.5	Upstream of Union, OR in narrow valley reach with floodplain and steeper channel slope.

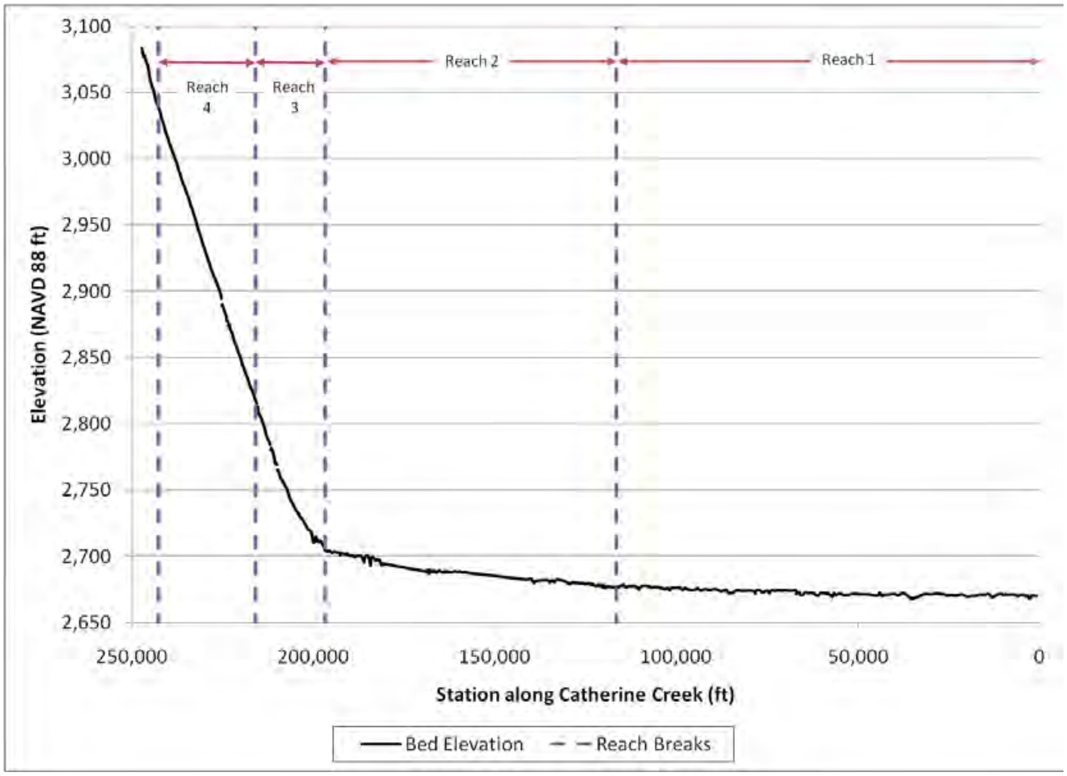

Figure 27. Reach break delineation for Catherine Creek.

4.1. Channel Slope and Water Surface Profiles

Results for computed water surface elevations at the 1.5-, 10-, and 100-year discharges are presented in Figure 28 through Figure 31. The average bed slope increases from Reach 1 to Reach 4. This is expected as you move from the valley up into the mountains. Figure 28 shows the results from Reach 1. The bed slope in this area can be divided into three sections. The slope of the bed is fairly constant at 0.004% until Elmer Dam at RM 13.1. There is a flat slope section behind Elmer Dam until Booth Lane which is likely due to sediment deposition upstream of the dam. From Booth Lane until the Old Grande Ronde River confluence (RM 22.5) the slope is constant at 0.01%, but steeper than in the other two sections. Elmer Dam provides a major hydraulic control at all flows simulated. The bridges in this reach appear to exert local hydraulic control at the 100-year flood, but not typically at the lower floods. Old Grande Ronde River is a slope break between the reaches. The slope steepens upstream of the confluence.

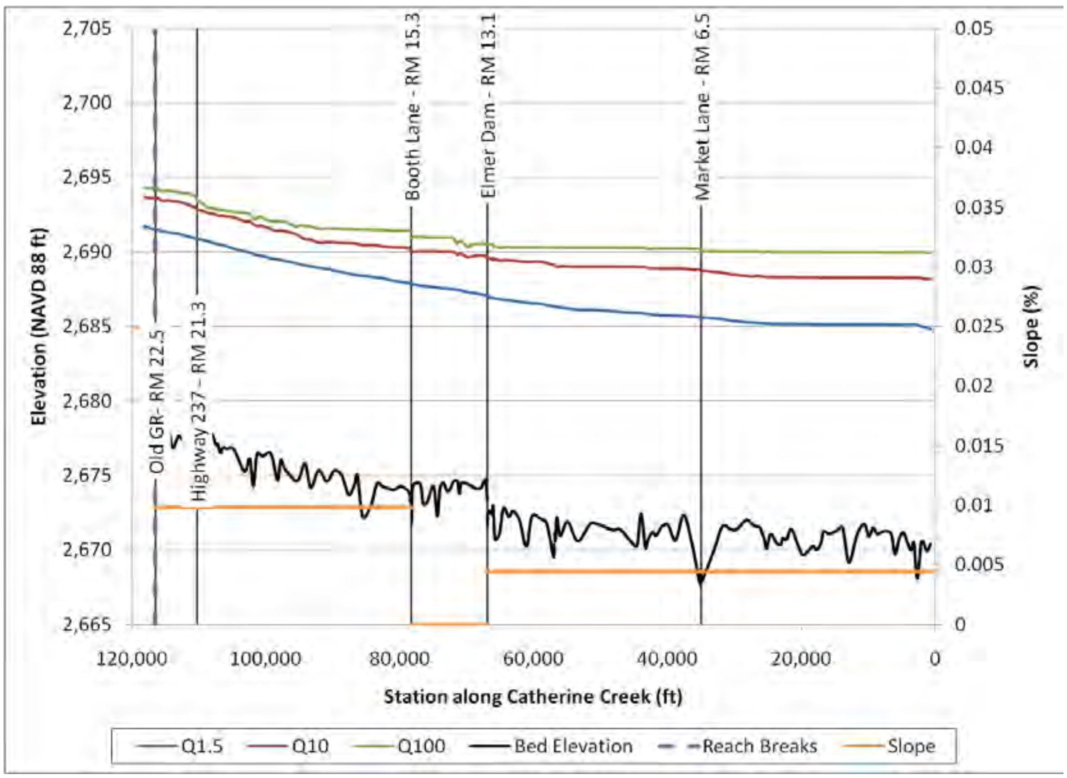

Figure 28. Computed water surface elevation for reach one on Catherine Creek.

Reach 2 is shown in Figure 29. The bed slope is fairly constant below Ladd Creek, 0.02%, although a portion of channel bottom had to be interpolated in this section and there could be hydraulic controls in this area, such as McAlister Slough, that are not included in the bed profile. Upstream of Wilkinson Lane Bridge, the bed slope steepens until Reach 3 to 0.05%. The bridges in Reach 2 act similarly to the bridges in Reach 1; they exert local hydraulic control on the river at higher flood flows. A beaver dam at RM 24.9 (station 130806) also acts as a hydraulic control. The influence can be seen in the 10- and 100-year profiles. Sediment deposition upstream of the dam is notable in the bed profile.

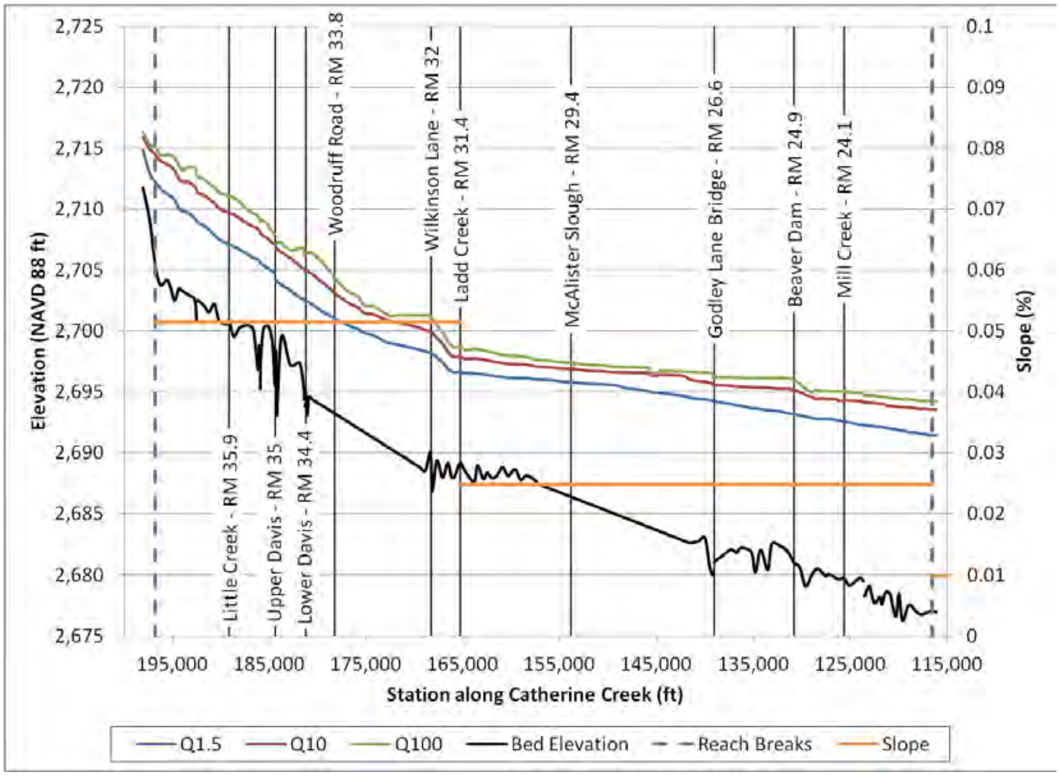

Figure 29. Computed water surface elevation for reach two on Catherine Creek.

Reach 3 is shown in Figure 30 and is the most upstream section of Grande Ronde Valley. The average slope in this reach, 0.59%, is steeper than in Reach 1 or Reach 2. However, variation in the slope throughout the reach is visible. Several of the bridges exert hydraulic control on flood flows. For example, Main Street Bridge at RM 40 raises water surface elevations at the two-year flood. There are two locations where the 100-year water surface elevation is lower than the 10-year water surface elevation: stations 203705 and 201306.4. At station 201306.4, the 100-year water surface elevation is at critical depth. For station 203705, the reason for the water surface elevation discrepancies between the 10-year and 100-year discharge is unclear, but could be related to levee overtopping. This could be investigated further in the future.

Reach 4 is shown in Figure 31 and is the upstream end of Union, OR. The slope in this reach, 0.83%, is steeper than in Reaches 1, 2 or 3. The CCACF diversion acts as a control on the water surface, causing an increase of approximately half a foot in water surface elevation at all flood flows. State Diversion also increases the water surface, ranging from a three to six inch increase depending on the flood.

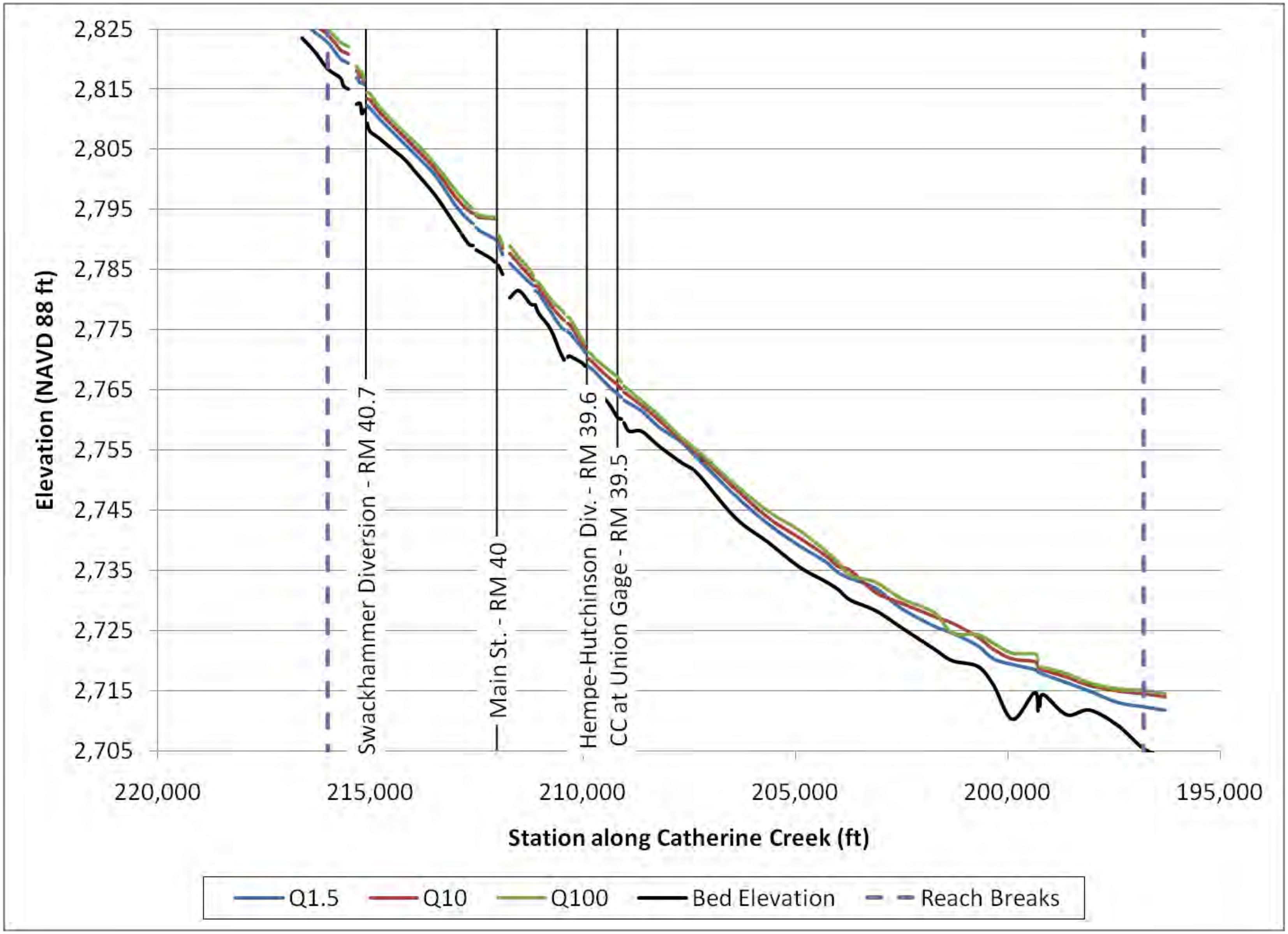

Figure 30. Computed water surface elevation for reach three on Catherine Creek.

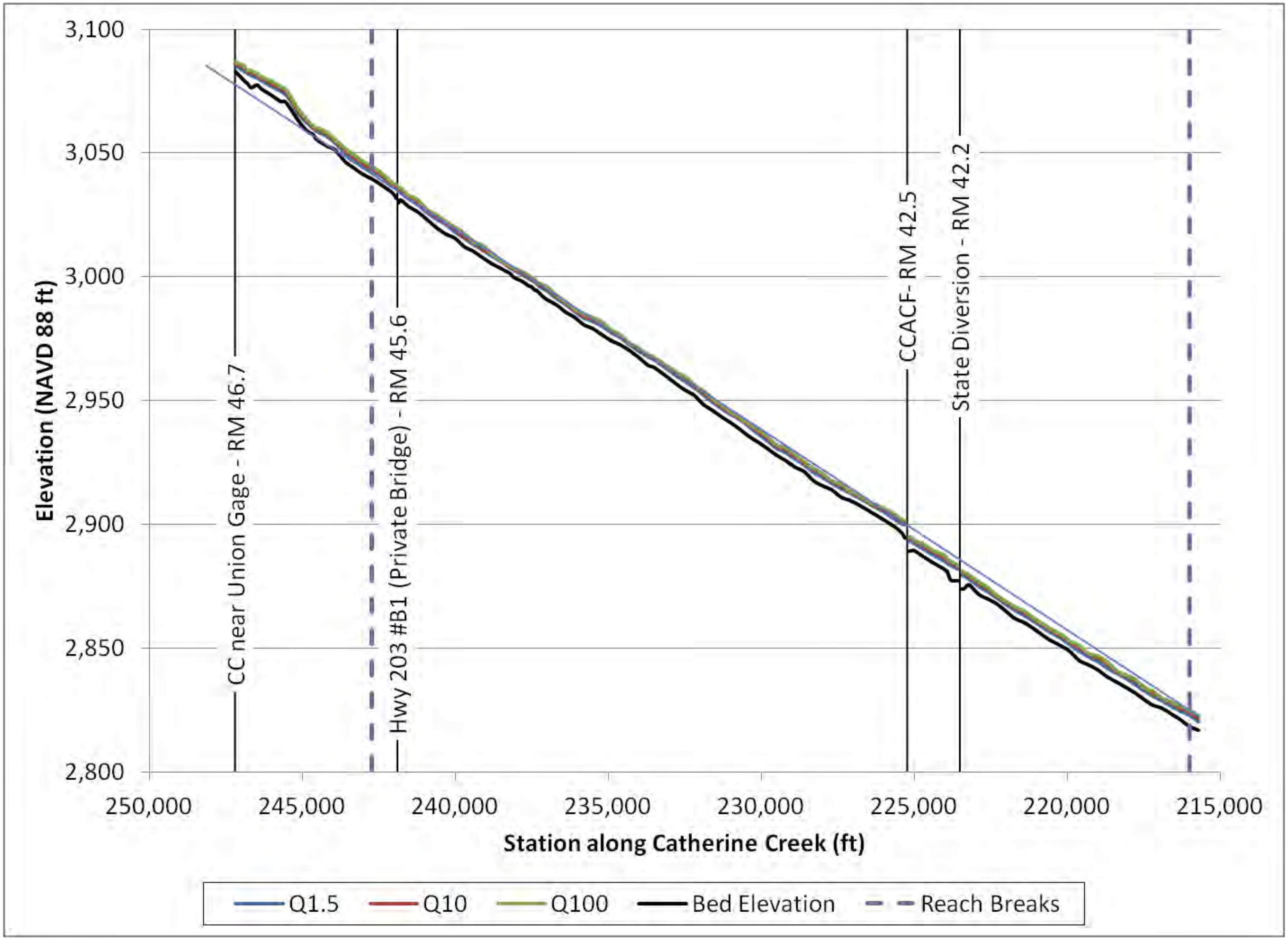

Figure 31. Computed water surface elevation for reach four on Catherine Creek.

4.2. Channel and Levee Capacity

4.2.5. Channel Capacity

Channel capacity was determined for each reach based upon the flow that overtops the channel banks as they are defined in the HEC-RAS model. Comparisons of channel bank elevations and water surface elevations for each reach are illustrated in Figure 32 to Figure 35. Using all high flow discharges evaluated, a histogram of the return period of the flood that overtops the bank elevation was used to define the most frequent channel capacity (Figure 36 to Figure 39). Within each reach, differences in capacity are expected due to the local topography of the cross section, the morphology of the reach (pool or riffle), and the user-defined bank and levee points. However, the reach-averaged conditions are useful in defining the discharge required to overtop most of the channel banks within the reach. Due to the length of the downstream reaches, some localized areas of higher or lower capacity may not be apparent in the reach-averaged conditions. More detailed analyses of the lower reach breaks could increase the resolution of the channel capacity.

Within Reaches 1 and 2, 63% and 55% of the cross sections have channel capacity equal to or below the 2-year discharge, respectively (Figure 36 and Figure 37). Within these reaches there are some spatial variations. The downstream five miles of Reach 1 appear to exceed the bankfull discharge on a more frequent basis than other portions of the reach. The upstream end of Reach 2 (upstream of Ladd Creek) generally has a higher capacity than the downstream end.

Within Reach 3, channel capacity at most cross sections is equal to or exceeds the 100-year discharge. Approximately 40% have a channel capacity between a 1-year and 50-year discharge and less than 30% of the cross sections have a channel capacity between a 1-year and 10-year discharge (Figure 38). A large portion of this reach is highly confined between artificial levees and high banks. In addition, the channel banks are coincident with the tops of levees in many of these cross sections, resulting in similarities between the channel and levee capacity.

Reach 4 channel capacity is most frequently between a 5-year and 10-year discharge. Sixty-five percent of cross sections have a channel capacity at or below the 10-year discharge, 42% of the cross sections have a capacity between the 5 or 10-year discharge(Figure 39).

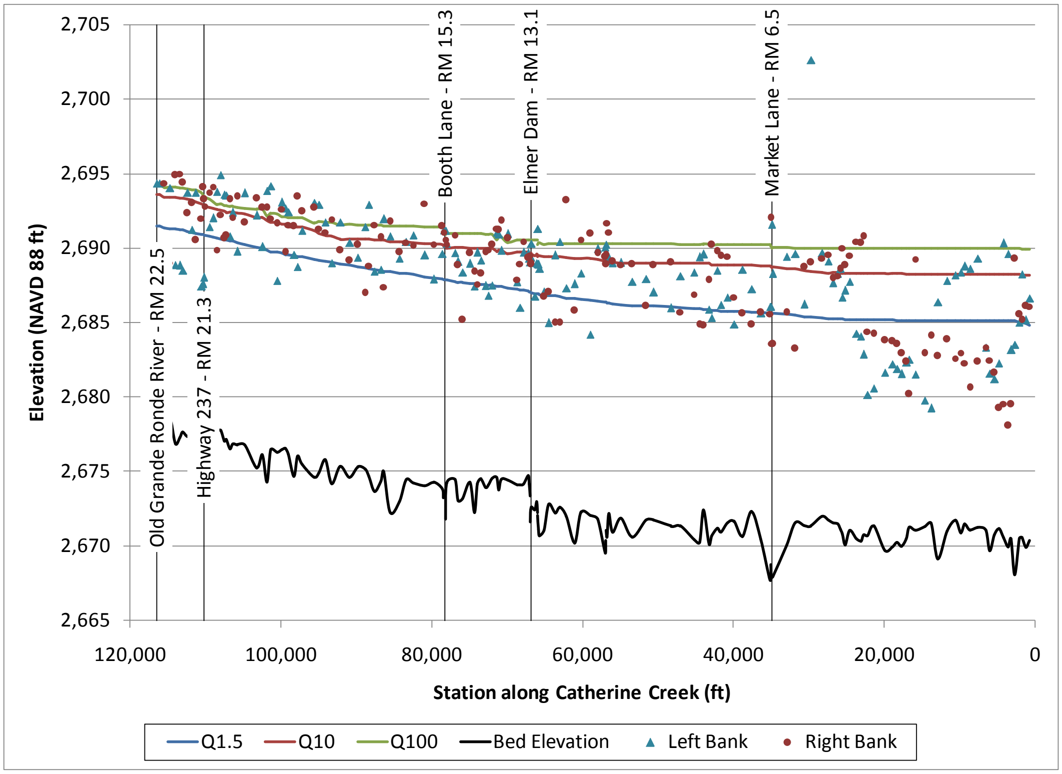

Figure 32. Comparison of bank elevations with flood discharges for Reach 1 of Catherine Creek.

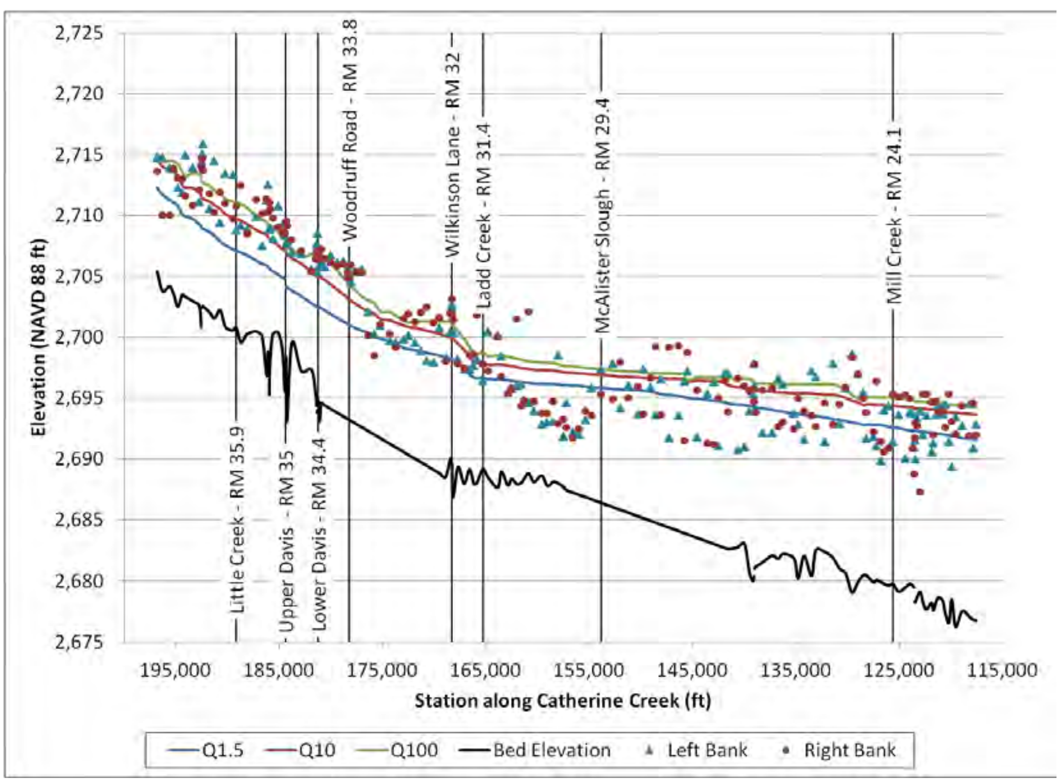

Figure 33. Comparison of bank elevations with flood discharges for Reach 2 of Catherine Creek.

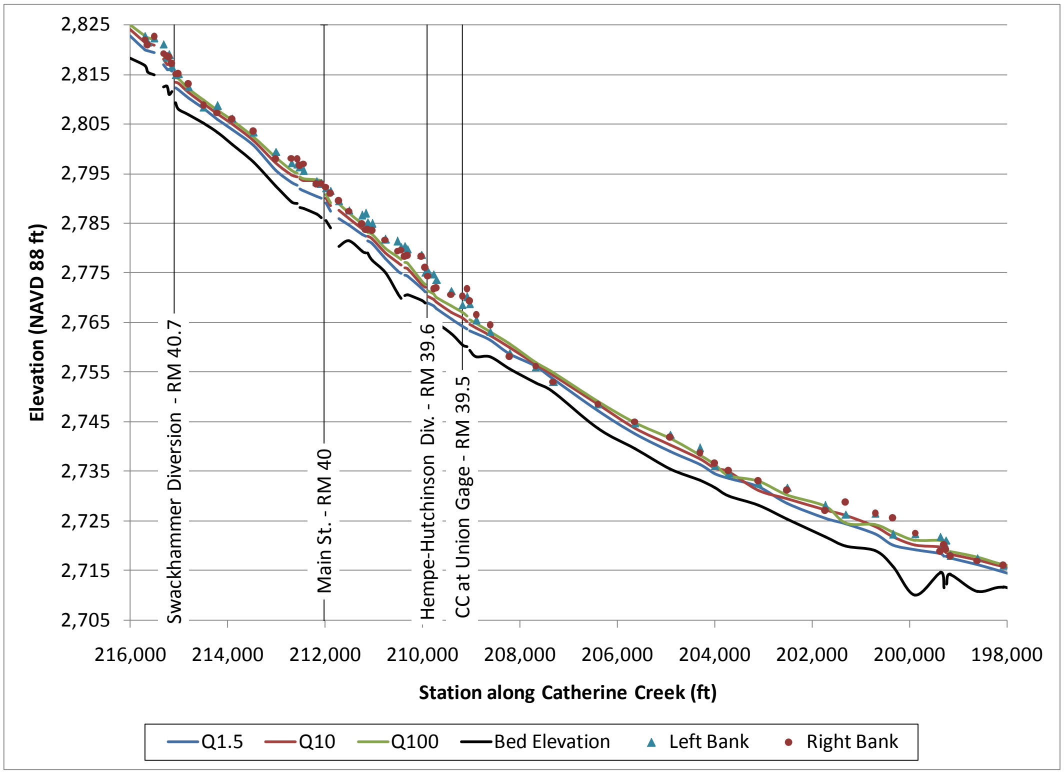

Figure 34. Comparison of bank elevations with flood discharges for Reach 3 of Catherine Creek.

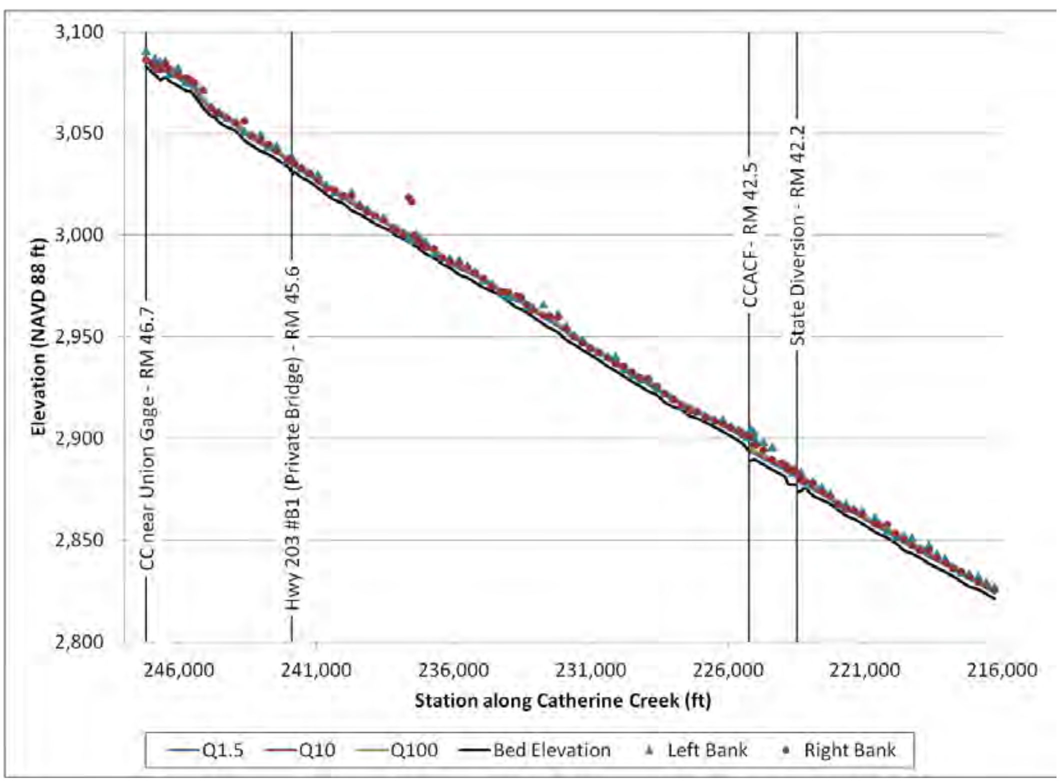

Figure 35. Comparison of bank elevations with flood discharges for Reach 4 of Catherine Creek.

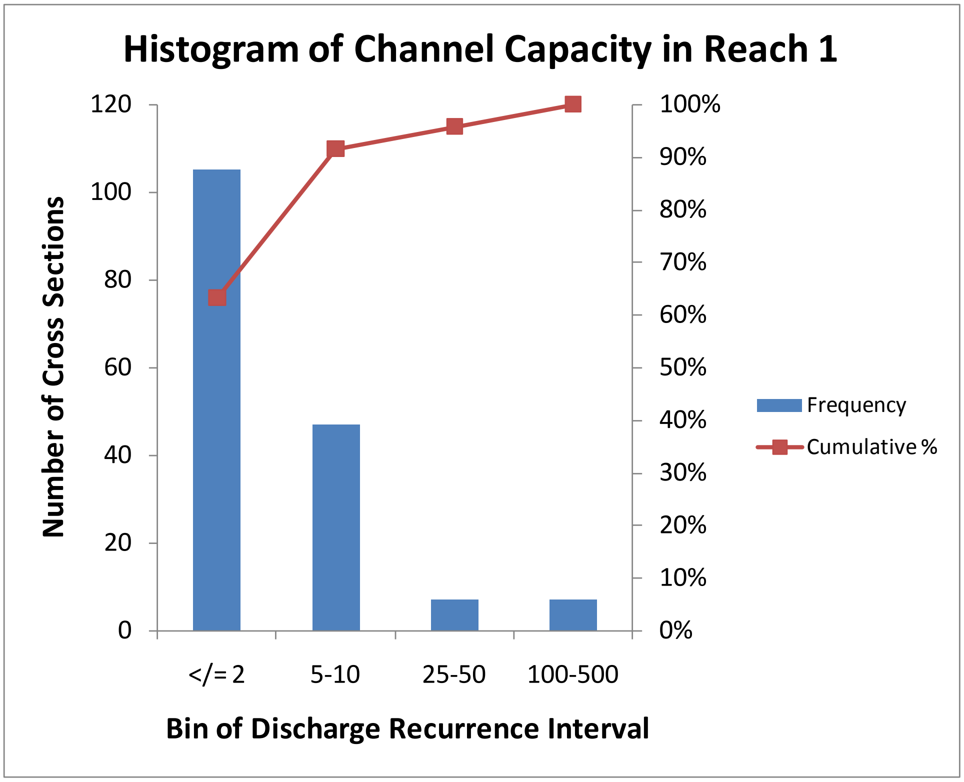

Figure 36. Distribution of channel capacity recurrence intervals for cross sections in Reach 1.

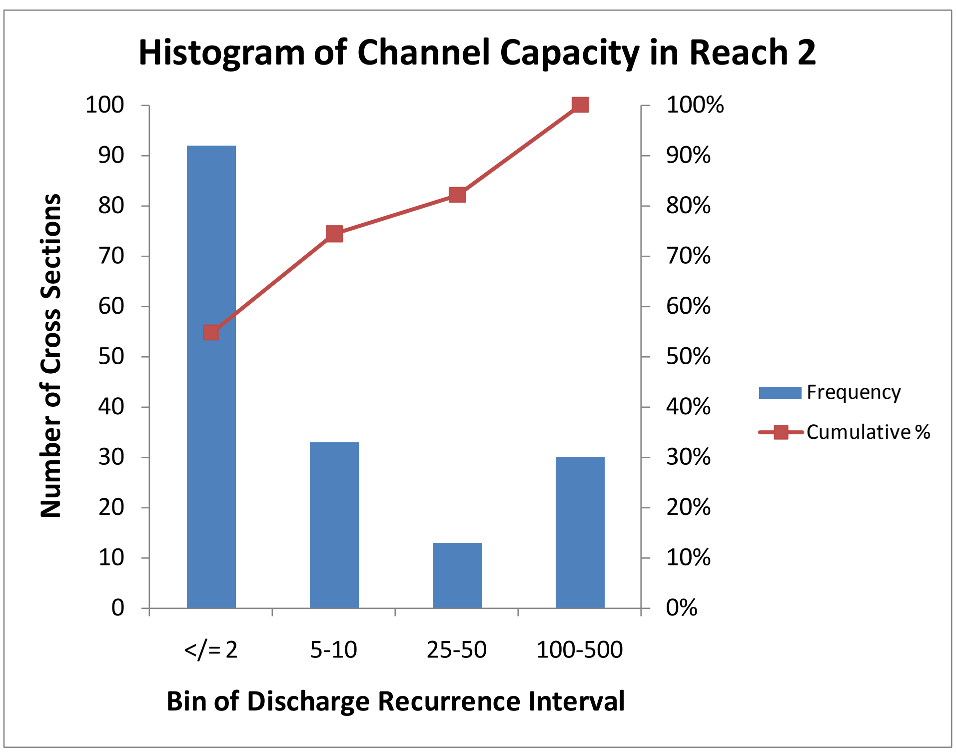

Figure 37. Distribution of channel capacity recurrence intervals for cross sections in Reach 2.

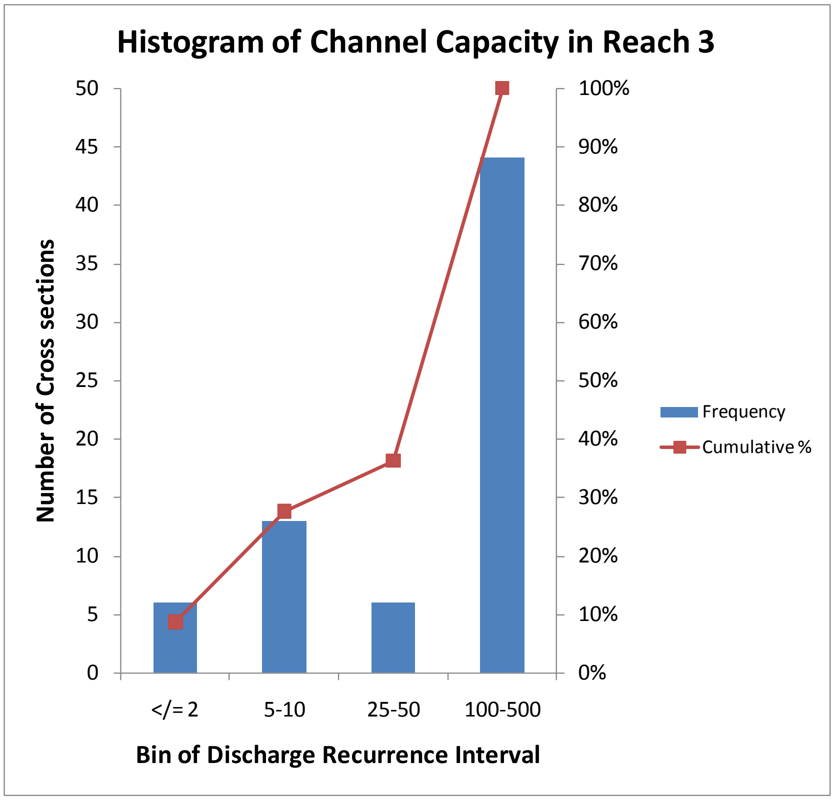

Figure 38. Distribution of channel capacity recurrence intervals for cross sections in Reach 3.

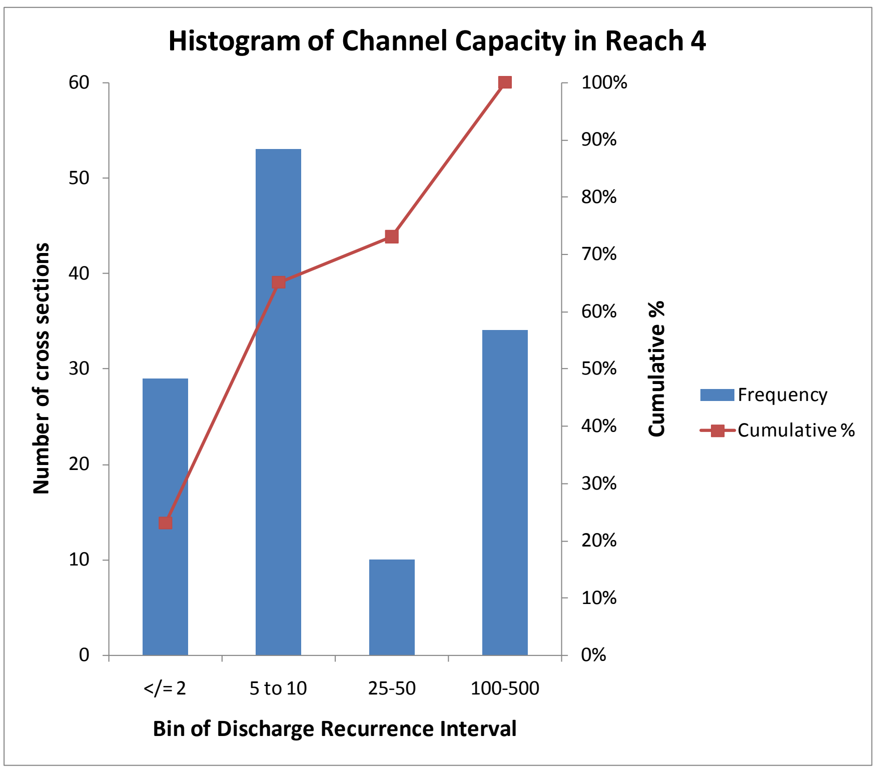

Figure 39. Distribution of channel capacity recurrence intervals for cross sections in Reach 4.

Bankfull discharge is similar to the channel capacity because it is a commonly used as a measure of the flow at which discharge begins to overtop the channel banks. However, bankfull stage is often determined based upon surveyed elevations of visual observations in the field indicating the top of bank, many of which are not purely topographic in nature. In this analysis, surveyed elevations of the bankfull elevation are not available, and instead channel bank elevations as determined in each cross section are utilized as a surrogate to the surveyed bankfull elevations. This surrogate allows for a rough approximation of expected bankfull conditions, but is not representative of the true bankfull stage as defined by Dunne and Leopold (1978).

Within Reaches 1 and 2, a more detailed investigation into the histograms for channel capacity was performed for flows with recurrence intervals ranging between 1.05 and 2.33 years (example shown in Figure 40). The distribution of discharges overtopping the channel banks within these reaches is sorted. High variability exists within the reaches and may warrant additional reach breaks at refined levels of analysis. For example, the downstream five miles of Reach 1 appear to exceed the bankfull discharge on a more frequent basis than other portions of the reach. While confidently describing Reaches 1 and 2 as having a specific discharge associated with the bankfull condition is difficult due to high variability, results indicate that over half of the cross sections in each reach have water surface elevations consistent with the channel banks at a 1.5-year discharge. Reach 3 and 4 bankfull discharge appears coincident with the channel capacity as described previously.

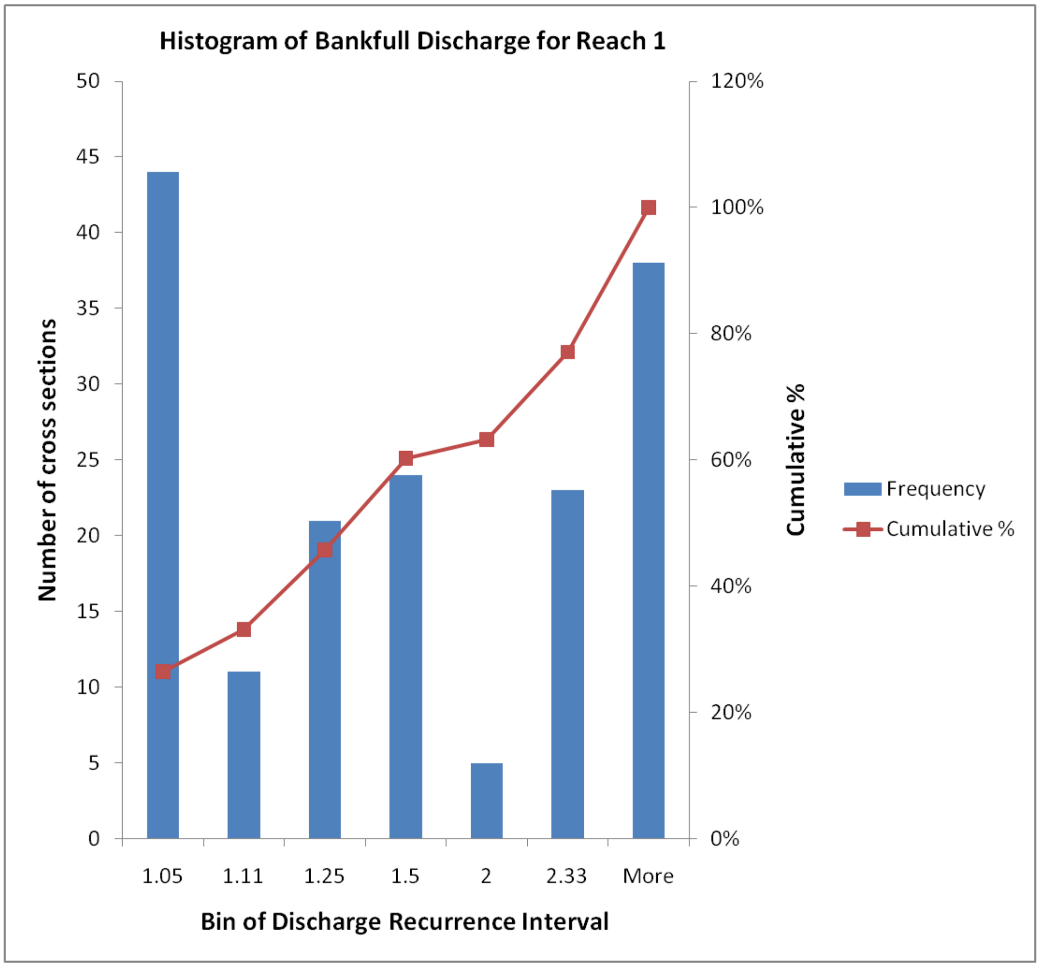

Figure 40. Plot of the distribution of bankfull discharges for cross sections in Reach 1.

4.2.6. Levee Capacity

Levee capacity was determined in each cross section by analyzing the flow that overtops the lower of the left or right levee. Cross sections had levees assigned where topographic features were present that prevented flows from accessing floodplain areas. In cross sections where a defined levee was not present, levee elements were often assigned in HEC-RAS to keep flow from accessing lower elevation floodplain areas without first filling the channel to capacity. The analysis of levee capacity includes all levees as assigned in HEC-RAS.

Each reach of Catherine Creek was investigated individually. Histograms indicating the distribution of cross sections at which flows begin to overtop the levees are provided in Figure 41 and Figure 42. Levees in Reach 1 are overtopped at the greatest frequency, with more than 80% of cross sections experiencing levee overtopping at discharges of 10-years or less. Levees at the downstream end tend to be overtopped on a less frequent basis than levees at the upstream end. A comparison of the water surface profiles and levees elevations for the left and right banks is shown in Figure 43 and Figure 44. Levees that correspond with the bank elevations are notable in the plots.

Levees in Reach 2 tend to be overtopped at less frequent recurrence intervals than Reach 1. Less than 40% of cross sections levee are overtopped at flows equal to or less than the 10-year discharge. Nearly 50% of cross sections indicate that levees are not overtopped until flows exceeding the 100-year discharge are experienced. In general, cross sections upstream of Ladd Creek (RM 31.4) require smaller discharges to overtop the levees. Comparisons of the levee elevations and the water surface profiles for Reach 2 are shown in Figure 45 and Figure 46.

Within Reach 3 and 4, levees are typically not overtopped at flows less than 50-year discharge. More than 70% of cross sections in each reach do not experience levee overtopping at flows less than the 500-year discharge. A comparison of Reach 3 levee elevations and water surface elevations is shown in Figure 47. Localized areas of more frequent overtopping may be present, but more detailed investigations of individual reaches is necessary to identify specific locations of levee overtopping.

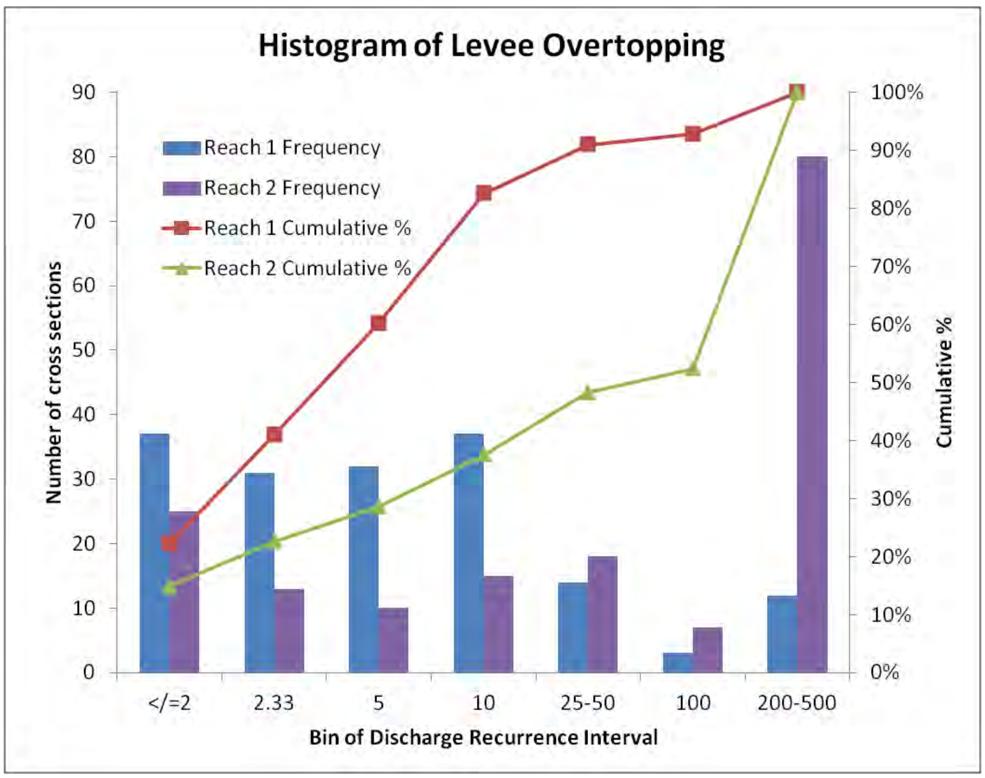

Figure 41. Distribution of the number of cross sections overtopping levees at specific recurrence interval discharges within Reaches 1 and 2.

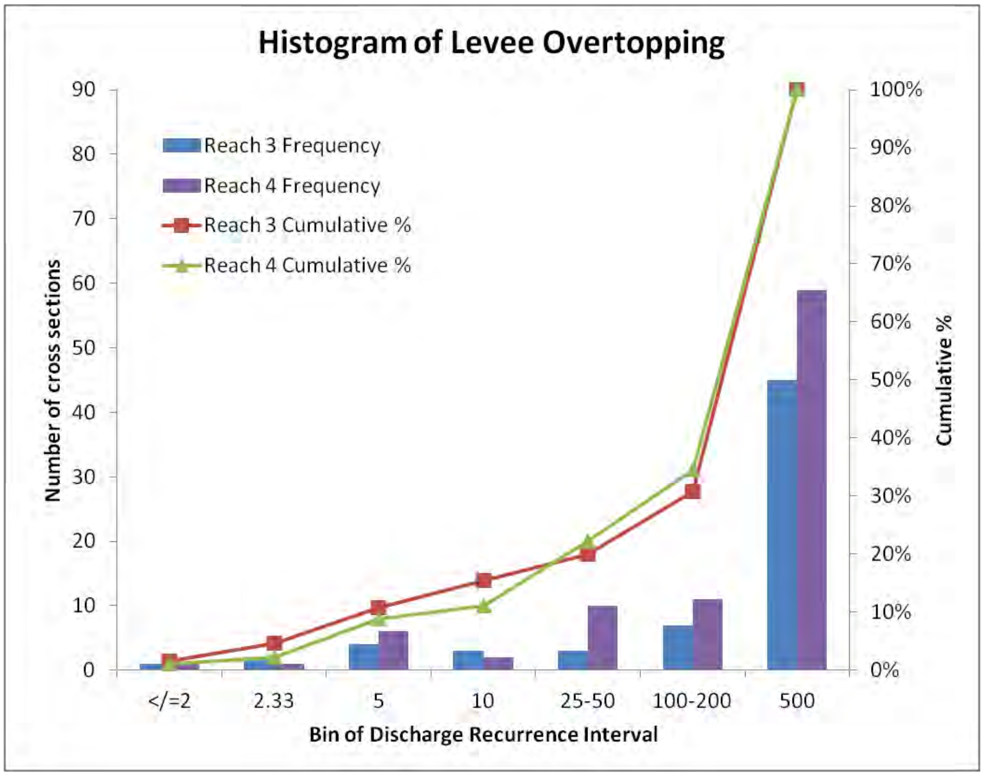

Figure 42. Distribution of cross sections overtopping levees at specific recurrence interval discharges within Reaches 3 and 4.

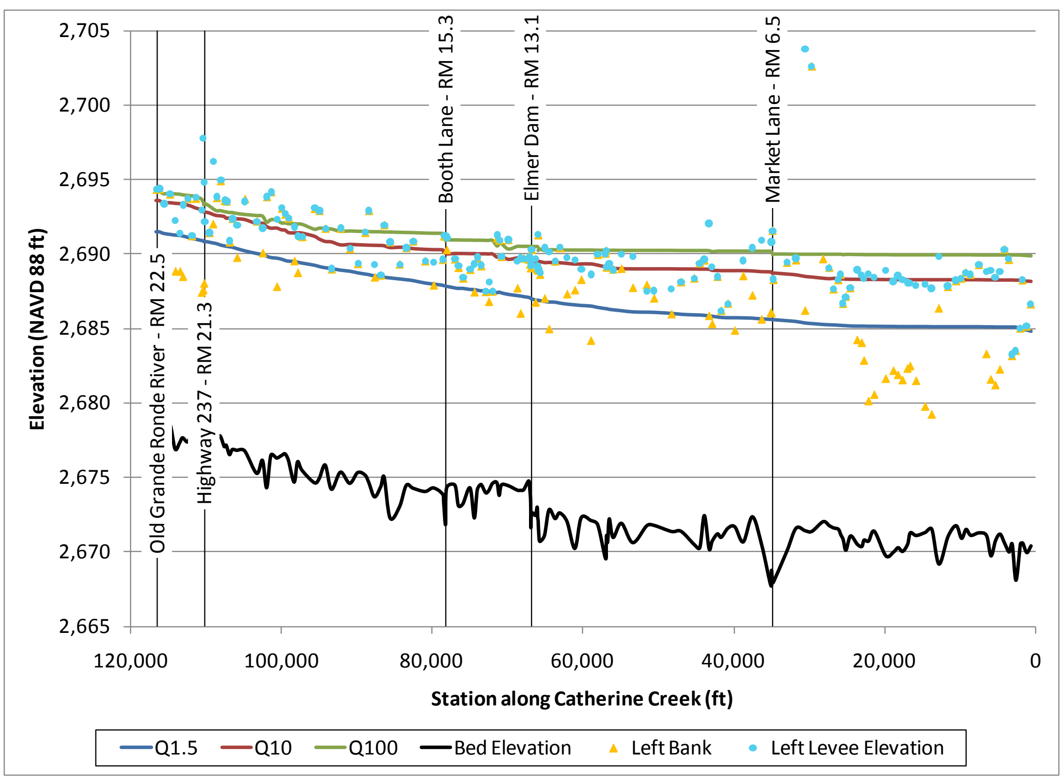

Figure 43. Reach 1 water surface profiles compared with levee and bank heights along the left bank.

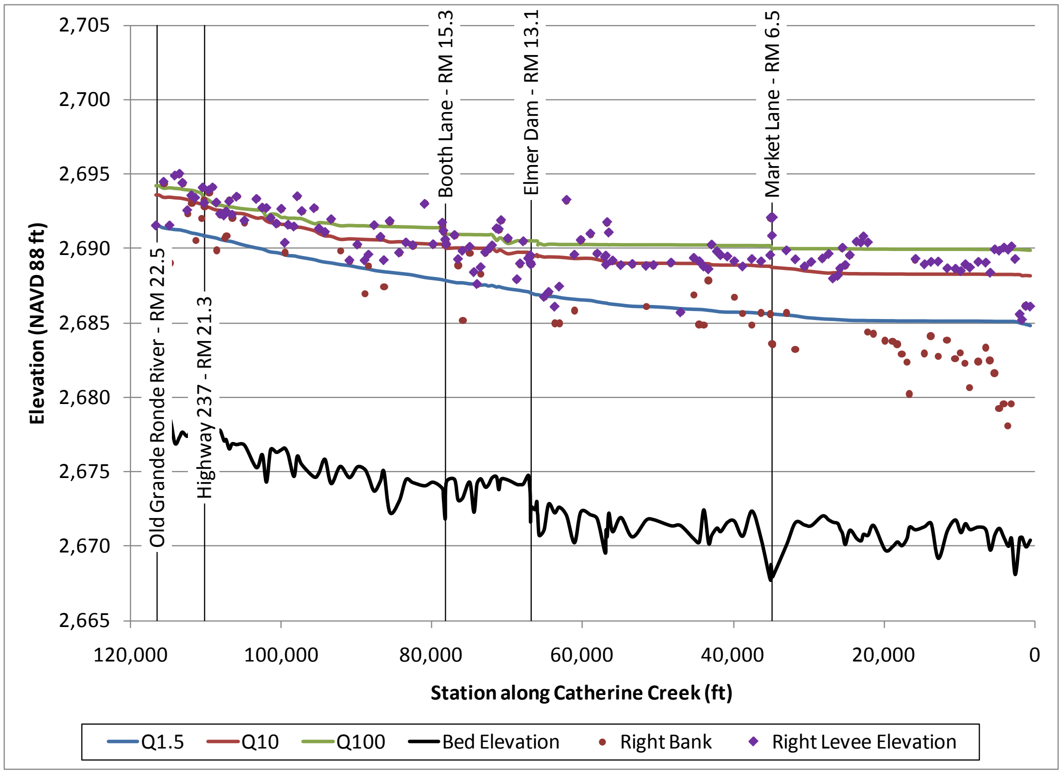

Figure 44. Reach 1 water surface profiles compared with levee and bank heights along the right bank.

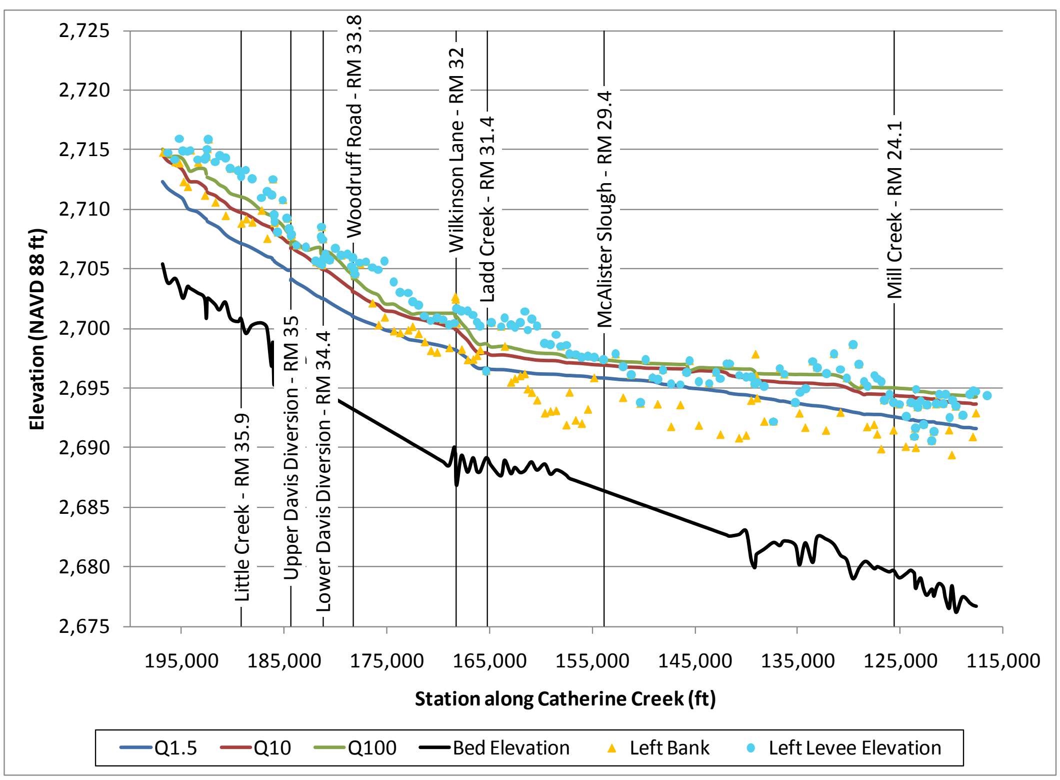

Figure 45. Reach 1 water surface profiles compared with levee and bank heights along the left bank.

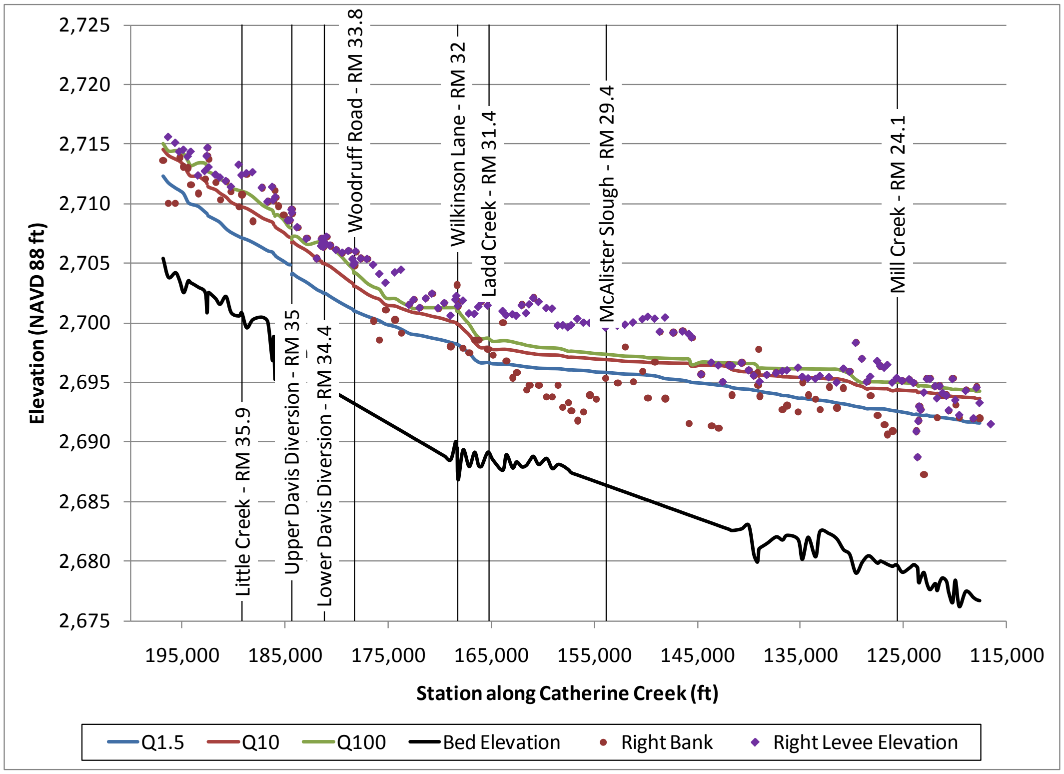

Figure 46. Reach 2 water surface profiles compared with levee and bank heights along the right bank.

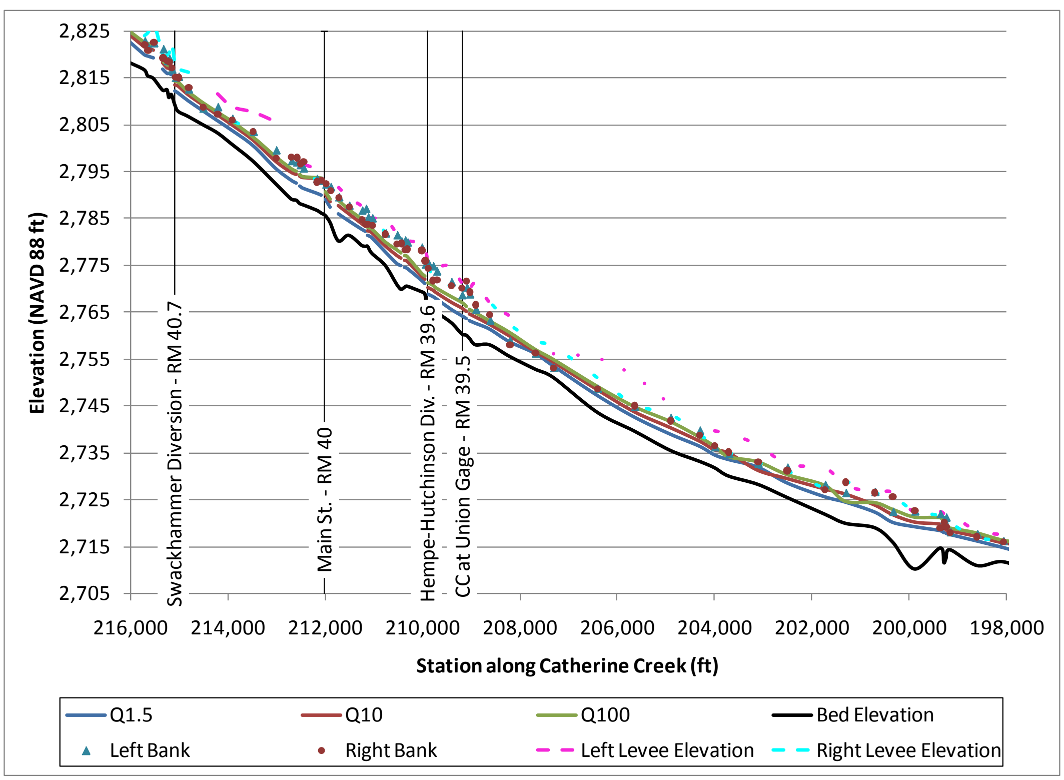

Figure 47. Reach 3 water surface profile compared with levee elevations.

4.3. Flood Depth Maps

Maps were developed to illustrate depths of potential flooding within the bounds of the modeled cross sections for the 100-year discharge. Figure 48 shows an example of an area where the flow was out of bank. A full suite of maps illustrating depths within the cross section boundaries are shown in Attachment A.

The process used to create the maps included creating a TIN of the water surface elevations derived from the HEC-RAS model for the 100-year discharge, subtracting the water surface TIN from the terrain models, and manually adjusting the wetted areas to account for the effects of levees or other high points in the terrain that would prevent water from reaching certain overbank areas. Areas that were not directly connected to the channel, i.e. “islands” of water, were removed from the inundation mapping since there was no direct pathway for the water to reach these locations. In addition, wetted areas outside of the cross section extents were removed since the accuracy of the inundation in these areas is uncertain.

The impacts of the levees were investigated in more detail. If a levee was overtopped by at least one foot of water, the area behind the levee was not manipulated and was permitted to remain flooded. Flow was allowed to inundate the area both upstream and downstream of the point of overtopping to the extent of the cross section unless another feature was present within the cross section that prevented inundation. As a result, water surface elevations are likely overestimated since the volume of water available to inundate an area was assumed to be infinite. The area behind a levee was not included as a possible inundation area if a levee parallel to the river was not overtopped by at least one foot, and an upstream and downstream road or levee perpendicular to the river was not overtopped by one foot (essentially enclosing an area). Levees directly adjacent to the channel and further out in the floodplain were considered. Although the valley is very flat, a 1-foot criterion was selected for overtopping because the volume of water required to submerge the areas behind the breached levees is large. Actual depths of inundation are highly uncertain behind levees because the model is a steady state 1D model, and flood storage impacts within the floodplain are not simulated.

Several limitations apply to the depth maps. First, the spatial extent of the flooding was restricted to the extent of each cross section. However, actual inundated areas during a 100-year discharge may extend several miles beyond the length of the cross section. Therefore, these maps may not represent the likely extent of a100-year flood. Backwater areas included in the maps may not have water at the location and depth indicated. Although attempts were made to capture hydraulic controls in selecting placement of the cross sections, the model can not accurately predict levee overtopping between cross sections. The wetted areas presented in the maps provide potential depths of flow as constrained by the 1D modeling effort limitations. To evaluate inundation outside of levees for various discharges, additional data are needed. First, topographic data beyond the terrain models could be collected extending to the valley walls. This would allow for a separate model with cross sections extending across the entire valley and spaced at a greatly reduced frequency from the present model. Finally, an unsteady flow model would be needed to evaluate flood storage impacts, and unsteady two-dimensional models would improve understanding of lateral flood processes and flow patterns between cross sections.

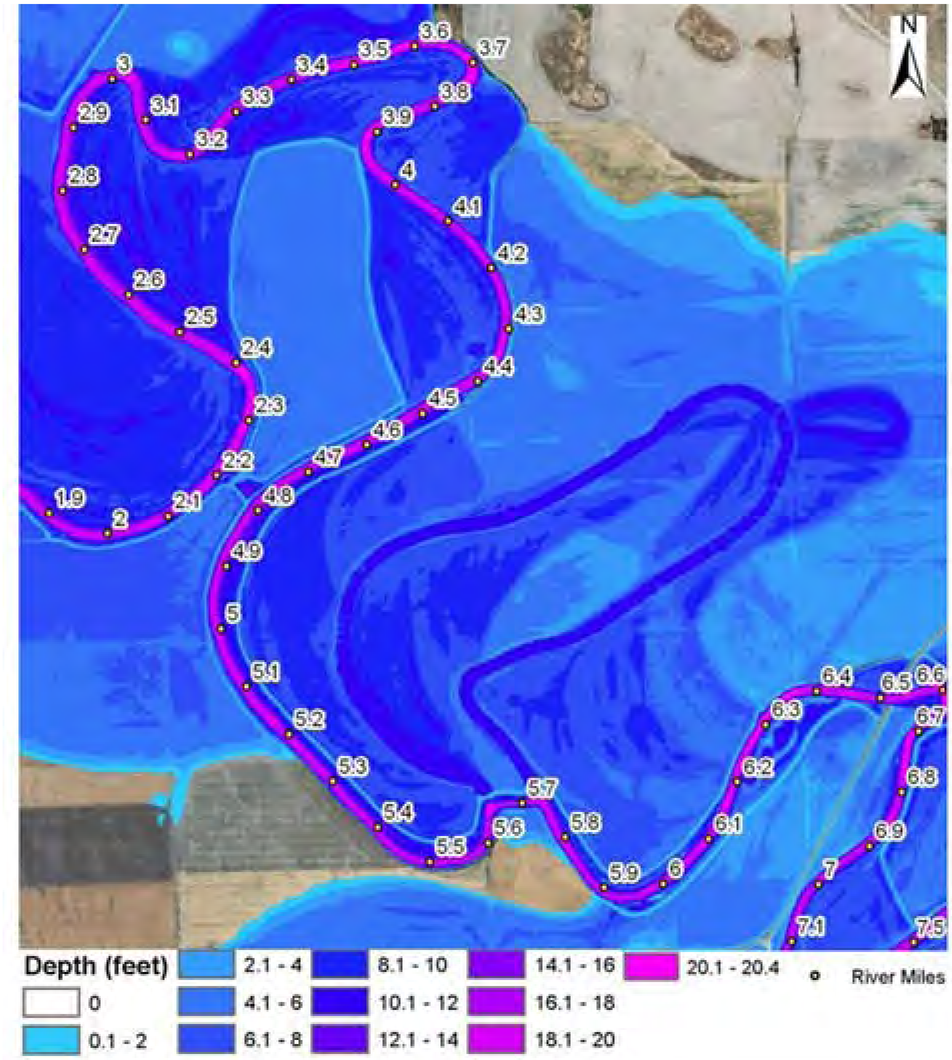

Figure 48. Potential flooding depths within the bounds of the modeled cross sections for the 100-year discharge in a portion of Catherine Creek Reach 1.

4.3.5. Oxbow Inundation

Eighteen disconnected oxbows were delineated on Catherine Creek between RM 0 and RM 39. An oxbow was selected for delineation if it appeared cut-off from the main channel but frequently inundated. For each oxbow, the closest upstream cross section was examined. In that cross section, the levee elevation was determined and the flood frequency discharge at which the levee was first overtopped was recorded. Although the levee elevation in the upstream cross section may not be the same elevation as the levee right at the location of the oxbow, it was assumed that if flow overtopped a levee just upstream, it was likely to inundate the oxbow as well. Table 10 below provides a location of each oxbow and the stage and discharge that overtopped the closest upstream cross section.

Table 10. Description of disconnected oxbows and the discharge recurrence interval that causes overtopping.

River Mile	Bank	Discharge recurrence interval (yr)	Water surface elevation of upstream cross section at overtopping flow
5.7	Right	25	2689.2
6.6	Right	25	2689.4
8.5	Right	100	2690.2
10.2	Right	1.5	2686.1
13.2	Left	10	2689.7
13.7	Left	10	2689.9
14	Left	1.5	2687.5
14.8	Right	5	2689.9
16.3	Left	2.33	2689.5
17.5	Right	2	2689.5
19.6	Right	25	2692.0
23.4	Left	1.25	2691.4
23.6	Left	5	2693.9
25	Left	No overtopping	NA
26.7	Left	25	2696.1
27.1	Left	No overtopping	NA
37	Right	No overtopping	NA
38.1	Left	No overtopping	NA

There are five oxbows (RM 10.2, 14, 16.3, 17.5, and 23.4) where there is likely to be overtopping at less than a five year flood. Oxbows at RM 16.3 and RM17.5 are shown in Figure 49. Figure 50 shows the oxbow at RM 14. Four of these oxbows are located in Reach 1, and one is in Reach 2. These oxbows are of the greatest concern for fish stranding since they are most frequently overtopped. The greatest frequency at which a flood overtops an oxbow is 1.25 years. Based on this result, the delineated oxbows are not overtopped by non-flood flows, and flooding of the oxbows may only occur once or twice per year. However, the entrance and exit conditions of the oxbow connections were not closely evaluated to determine if fish passage into or out of the oxbows by means other than overtopping is possible.

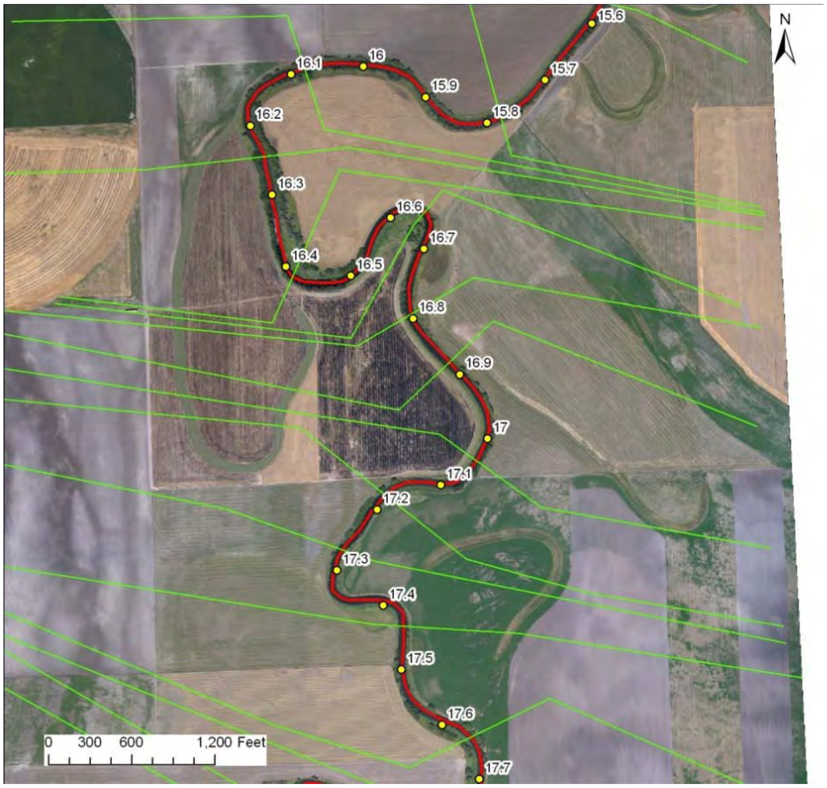

Figure 49. Oxbows at RM 16.3 and RM 17.4 that are inundated by the 2.33 and 2 year floods respectively.

Figure 50. Oxbow at RM 14 that is inundated by the 1.5- year flood.

The Old Grande Ronde River confluence is also an area that can be overtopped, experience backwater conditions, and provide a potential fish stranding concern. A levee exists where the Old Grande Ronde River was disconnected from Catherine Creek. In addition, there is a levee that extends along the eastern side of the Old Grande Ronde River (Figure 51). Based on the levee elevation surrounding the Old Grande Ronde River, the levee would be overtopped with a 1.5-year flood. The Old Grande Ronde river channel geometry and confluence were not included in the HEC-RAS model. Therefore, information regarding storage or extent of flow backwatering into the Old Grande Ronde River is not available.

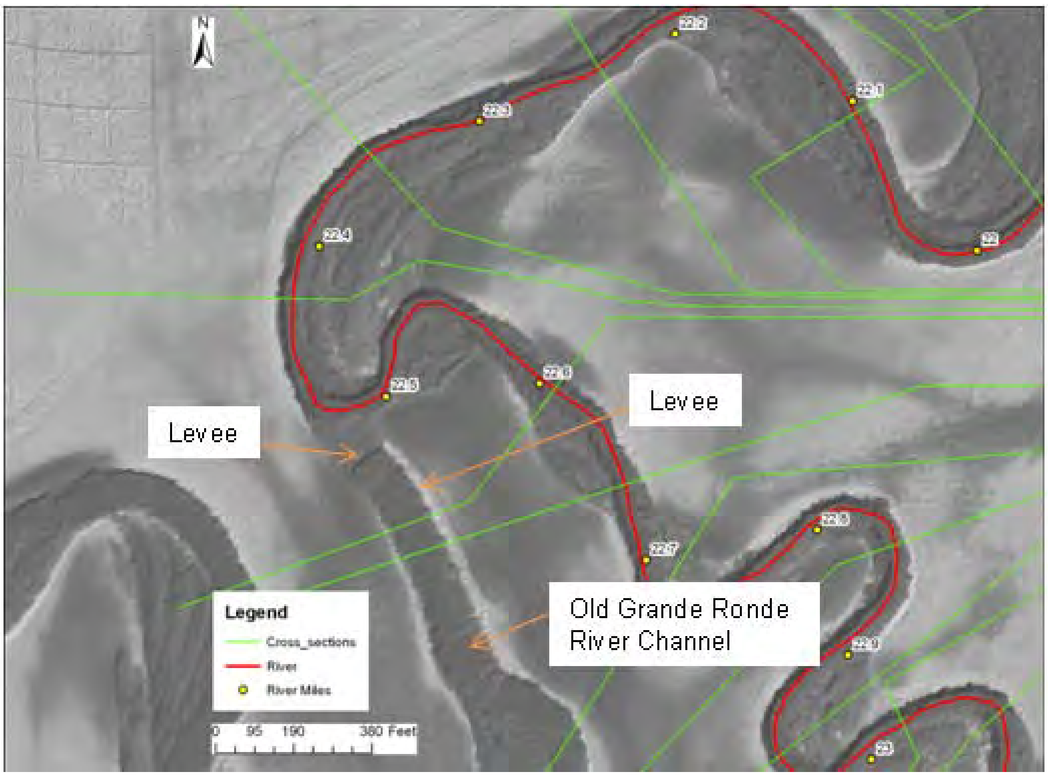

Figure 51. Existing elevation at Old Grande Ronde River confluence with Catherine Creek.

4.4. Velocity

Results for computed cross-section averaged in-channel velocities at the 1.5-, 10-, and 100-year discharges are presented in Figure 52 through Figure 55. In Reach 1 (Figure 52), velocities are fairly constant across all flood flows presented (approximately 1.3 ft/sec). This is likely because flows get out of bank at low elevations, and therefore at higher floods, the velocity does not increase because the amount of flow in the channel is not greater. Reach 2 (Figure 53) shows similar results of velocities being fairly constant (approximately 1.7 ft/sec) with increasing discharge until Ladd Creek. The velocities become more stratified (higher velocities for greater discharges) upstream of Ladd Creek. The reach-averaged velocity increases from approximately 2.7 ft/sec for the 1.5-year flood to 3.3 ft/sec for the 100-year flood.

In Reach 3 (Figure 54), the channel velocity increases to approximately 4.6 ft/sec for the 1.5 year flood to 6.6 ft/sec for the 100-year flood. In addition, the flow is staying in the channel at greater discharges so the velocities are increasing with greater discharges. Velocities in Reach 4 (Figure 55) act similarly to velocities in Reach 3 whereby velocity increases with discharge. The reach-averaged velocity is slightly higher in Reach 4 at approximately 4.8 ft/sec for the 1.5 year flood to 6.7 ft/sec for the 100-year flood.

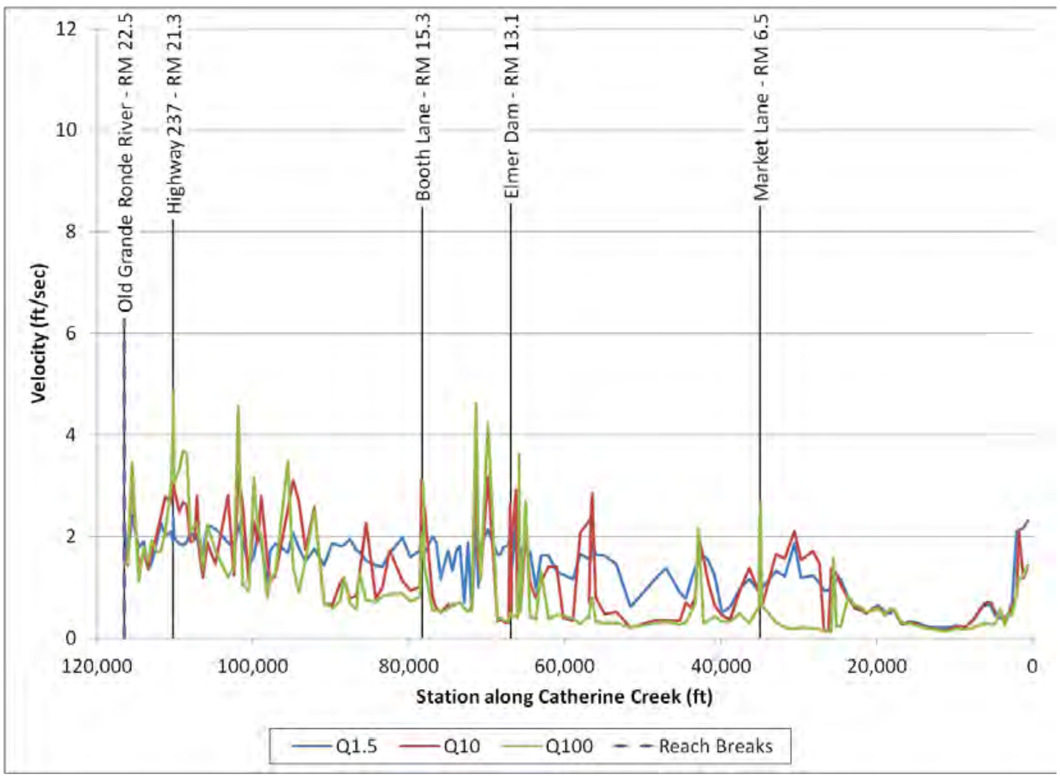

Figure 52. Computed cross-section average in-channel velocity for Reach 1 on Catherine Creek.

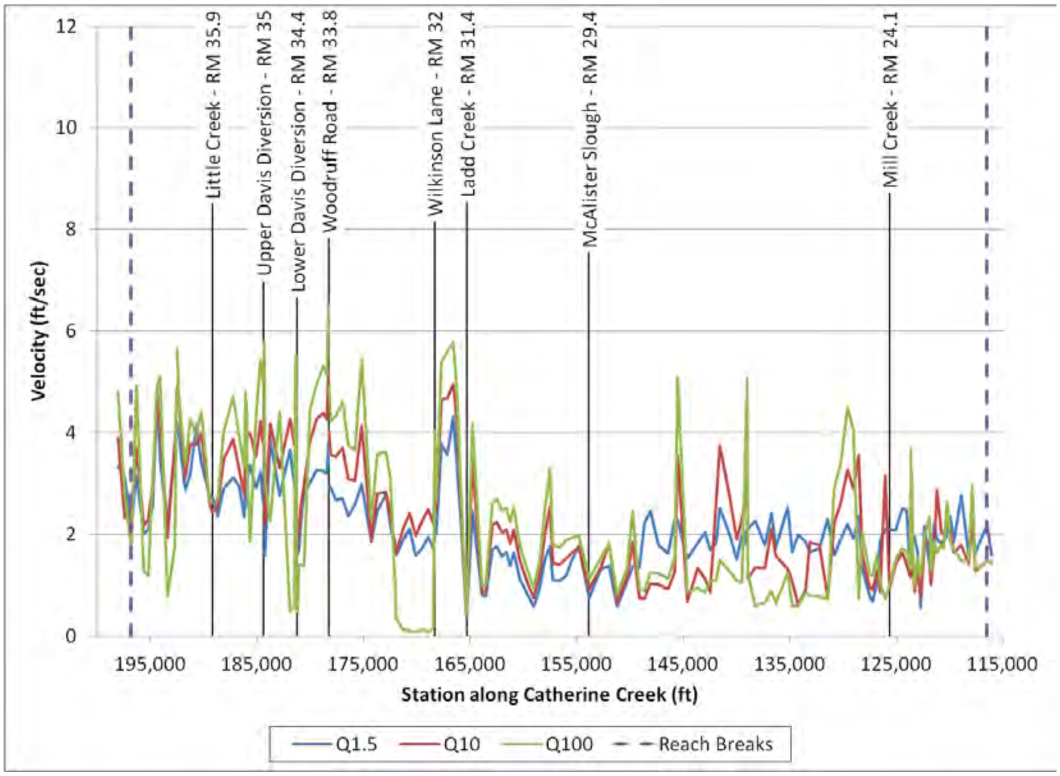

Figure 53. Computed cross-section average in-channel velocity for Reach 2 on Catherine Creek.

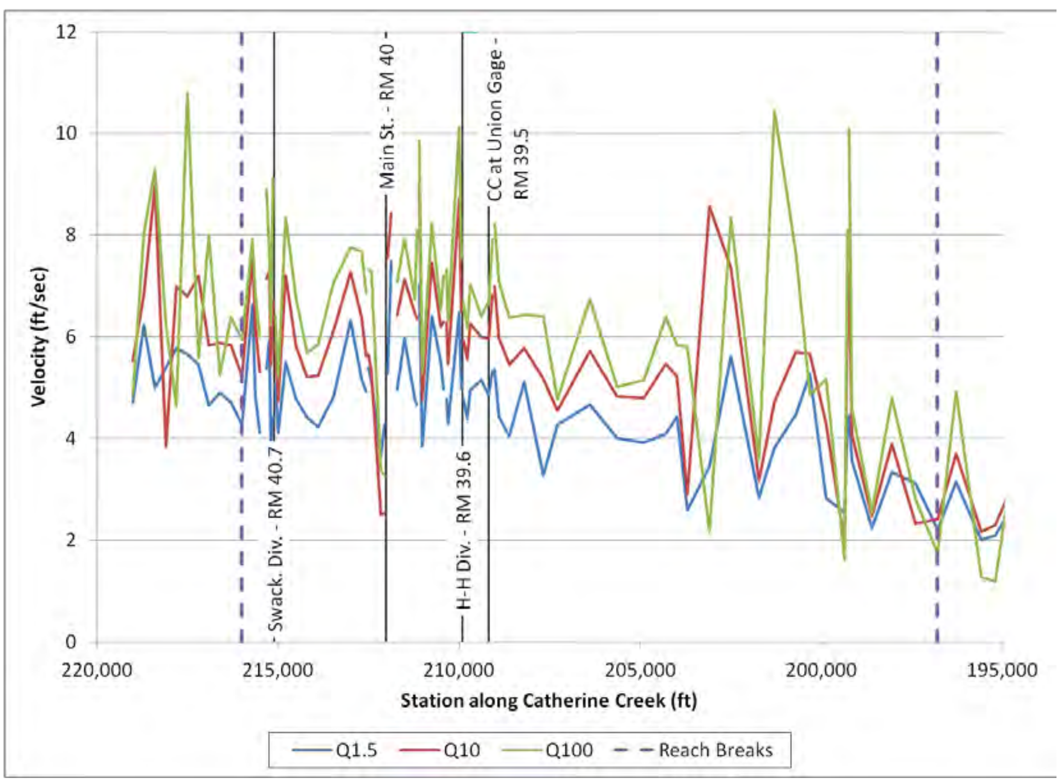

Figure 54. Computed cross-sectional average channel velocity for Reach 3 on Catherine Creek.

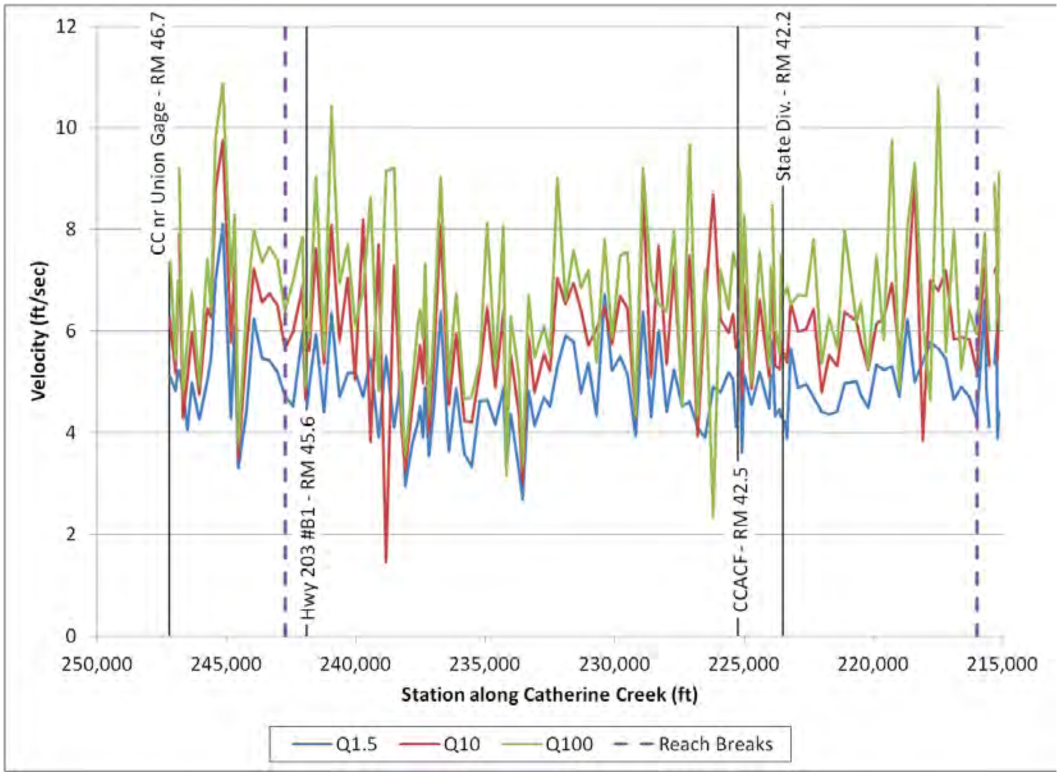

Figure 55. Computed cross-section average in-channel velocity for Reach 4 on Catherine Creek.

4.5. Shear Stress

In Figure 56 the in-channel shear stress was averaged for each reach on Catherine Creek. The shear stress for Reaches 1 shows no significant change with discharge. This is likely due to flow getting out of bank at low flood frequencies. There is a slight increase with discharge in Reach 2. Reaches 3 and 4 do show larger increases in shear stress for increases in discharge. This is caused by more flow staying in the channel at greater discharges. The magnitude of the in-channel shear stress much smaller in Reaches 1 and 2 than in Reaches 3 and 4. Although the reach-averaged shear stress provides an overview of what is happening in channel, high variability is present within the reaches. As an example, Figure 57 shows the variability of shear stress among cross sections in Reach 4.

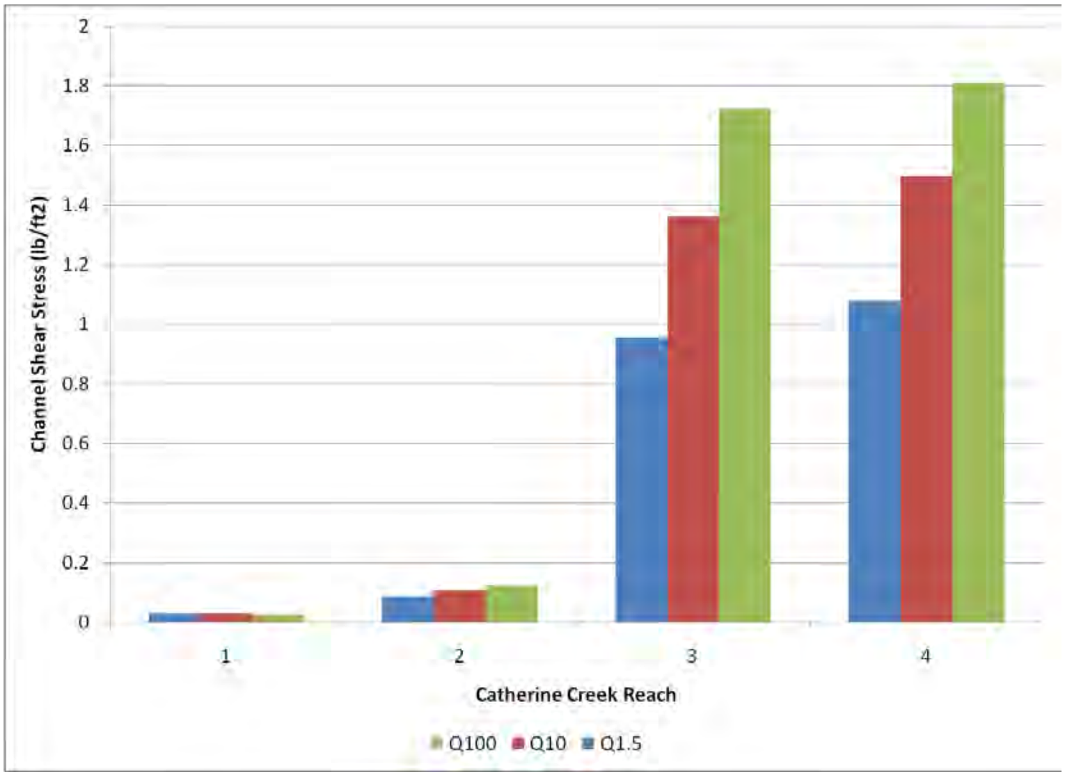

Figure 56. Reach averaged channel shear stress on Catherine Creek.

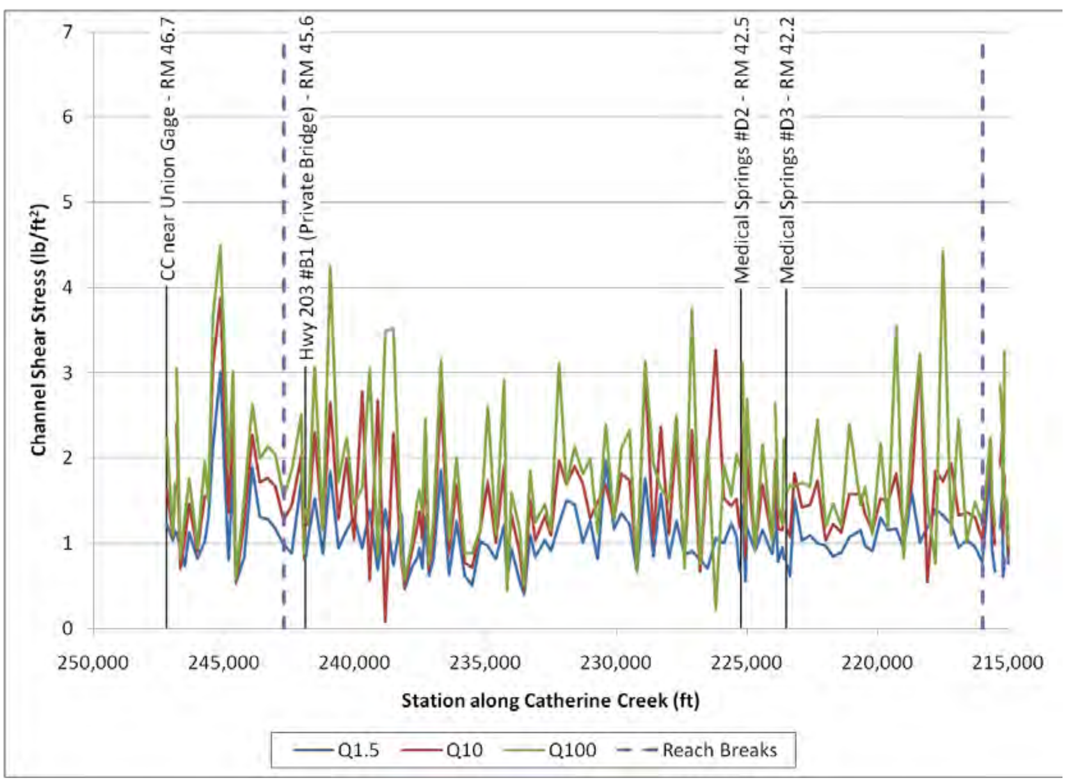

Figure 57. Channel shear stresses in Reach 4 on Catherine Creek.

5. Discussion and Summary

Hydraulic modeling conducted for this assessment provides a large-scale evaluation of impacts to flood processes along the Grande Ronde River and Catherine Creek. The information presented in this report documents current reach-averaged channel and levee capacities, velocities, and shear stresses that are experienced during floods on Catherine Creek. Results from the modeling effort can be used to verify hypotheses related to other disciplines, such as biology, geomorphology, and vegetation, and can be integrated with other disciplines to form conclusions related to flooding potential and resultant impacts to habitat.

5.1. Hydraulics Related to Flooding and Potential Habitat Impacts

Patterns in hydraulic properties of Catherine Creek help evaluate flood processes, including the potential for inundation of lands adjacent to channel. From upstream to downstream, in-channel velocities and shear stresses tend to decrease under current conditions. This corresponds to changes in valley confinement, channel slope, and sediment sizes. Upstream Reach 4 is situated in a narrow valley with a slope of approximately 0.83%, while the downstream-most reach of Catherine Creek, Reach 1, is in an expansive and flat valley floor with an average channel slope of 0.006%. Because of the wide, flat valley and channel, the natural potential for flooding of adjacent lands is much greater in Reaches 1 and 2 than in Reaches 3 and 4. With the installation of levees, the potential for flooding changes from pre-settlement conditions in association with the hydraulics between the levee bounds. Hydraulic modeling predicts that levee overtopping typically does not occur in Reaches 3 and 4 at discharges less than the 50-year recurrence interval and in most locations requires discharges near the 500-year recurrence interval to occur. On the contrary, Reaches 1 and 2 experience substantial levee overtopping at flows coincident with the 10-year recurrence interval or less. Levees at the downstream end of Reach 1 tend to be overtopped on a less frequent basis than levees at the upstream end of Reach 1. Within each reach, however, localized areas may experience levee overtopping at much smaller discharges than modeled either due to a short section of lower levees or due to overtopping of upstream levees.

Evaluation of the hydraulic modeling results indicates that the Grande Ronde River and Reaches 1 and 2 of Catherine Creek have experienced substantial impacts to flood processes over time. Conversion of floodplain to agricultural land use has resulted in greatly reduced access to high flow habitat, including inundated floodplains and side channels. Constriction of flows between levees has also likely resulted in increased velocities within the channel banks and reduced high flow refugia along the channel margins during more frequent discharges. Overbank areas that do remain accessible between the levees are expected to have reduced complexity compared with unimpaired conditions. The overbank areas are hypothesized to have been wetlands pre-settlement, but now are primarily agricultural areas with little diversity. Fish may become stranded where levees are overtopped and access to the channel on the receding limb of a flood is prevented. This occurs during floods with greater than 10-year recurrence intervals. Stranding may also occur in disconnected oxbows that become inundated during high flows but lack a point of exit once flows recede.

Within Reaches 3 and 4 of Catherine Creek, the model illustrates that the presence of low-head diversion structures and bridges impact current river processes and impact localized hydraulic controls on water surface elevations. However, the impacts of the structures on floodplain access are less severe since the floodplain extent is much narrower when compared with downstream reaches. Addition topographic features and anthropogenic activities, such as clearing of large wood and river channelization, may impact hydraulics and resultant habitat, but additional hydraulic modeling of historical conditions coupled with analysis of historical photography and channel geometries and slopes would be needed to verify this.

5.2. Summary of Reach-averaged Hydraulics

A summary of the notable hydraulic characteristics of each reach is provided below:

Reach 1 can be described as a wide, unconfined valley with an average slope of approximately 0.006%. The channel capacity of the reach is highly variable, with most locations exhibiting bankfull conditions at flows between the 1.5- to 2-year discharges. Average in-channel velocities are very low and are typically around 1.3 ft/s at discharges with recurrence intervals between 1.5 and 100 years. Similarly, shear stresses are very low, indicating the potential to transport only sand size sediment under flood conditions. Levees are present along most of the reach, limiting floodplain access. In most locations, levees are overtopped at flows equal to or less than the 10-year discharge. There are four disconnected oxbows (RM 10.2, 14, 16.3, and 17.5) in this reach where the levee is overtopped at less than a five year flood. The most notable hydraulic controls in this reach are Elmer Dam at RM 13.1 and the Old Grande Ronde River, which is located in the upstream extent of the reach at RM 22.5. A change in slope occurs at the Old Grande Ronde River. Bridges within the reach, including Booth Lane, Market Lane, and Highway 237, exert local controls at flows exceeding the 100-year discharge but do not appear significant at lower discharges.

Reach 2 is also a wide, unconfined valley with an average slope of approximately 0.04%. A noteworthy break in slope occurs at the confluence of Ladd Creek near RM 31.4, which coincides with changes in hydraulic properties. Channel capacity throughout the reach is variable, with bankfull conditions occurring in most cross sections around 1.5 to 2-year discharges. In-channel velocities below Ladd Creek are generally around 1.7 ft/s. Upstream from Ladd Creek, velocity increases with discharge and averages 3.1 ft/s. Shear stresses in Reach 2 are slightly higher than those in Reach 1, with reach averages ranging from approximately 0.10 to 0.17 lb/ft^2 for discharges between the 1.5- and 100-year recurrence intervals. Levees within Reach 2 are overtopped less frequently than Reach 1 and only 50% of the cross section levees are overtopped at the 100-year discharge. Notable hydraulic controls in this reach include Upper and Lower Davis Dams, Ladd Creek, Wilkinson Lane Bridge, and a Beaver Dam located at RM 24.9. Similar to Reach 1, most bridges in the reach impart some hydraulic control at the 100-year discharge, but their influence appears to be localized.

The reach break between Reach 2 and Reach 3 is a transition zone at the base of the Catherine Creek alluvial fan that results in hydraulics changes. The confinement of the valley within Reach 3 increases from downstream to upstream. Average bed slope within this reach is 0.59%. Channel capacity in this reach is high compared to downstream Reach 1 and 2 and also compared with upstream Reach 4. Over 60% of cross sections require a flow of 100-year recurrence interval or greater to exceed the channel banks. Reach-averaged channel velocities range from 4.6 ft/sec for the 1.5 year flood to 6.6 ft/sec for the 100-year flood. Shear stresses in the reach range from about 1 lb/ft^2 for a 1.5-year discharge to 1.75 lb/ft^2 for a 100-year discharge, indicating some potential to transport gravels at higher discharges. Less than 30% of cross sections with levees indicate levee overtopping for flows less than a 500-year discharge. In other words, an extreme event is necessary for levee overtopping to occur in most locations within the reach. Four of the bridges on Catherine Creek exert hydraulic control, greater than half a foot, on floods that are more frequent than the 100-year event. For Reach 3, Main Street Bridge exerts control for a 2-year event and Pond Slough and Hwy 203 #B3 exert control for a 25-year event. Hwy 203 #B2 exerts control for a 50-year event in Reach 4.

Reach 4 is a confined valley reach with an average channel slope of 0.83%. The channel capacity at most locations is between the 5 and 10-year discharge. The reach averaged velocity in Reach 4 is approximately 4.8 ft/sec for the 1.5year discharge and 6.7 ft/sec for the 100-year discharge. Average in-channel shear stresses in the reach range between 1.1 lb/ft^2 for a 1.5-year discharge to about 1.8 lb/ft^2 for a 100-year discharge. Similar to Reach 3, levees present in Reach 4 typically require a discharge of 500-year recurrence interval to overtop. Some localized overtopping of less formidable levees may occur during more frequent floods. The most significant hydraulic control within the reach is the CCACF diversion structure.

5.3. Limitations

Several limitations exist with the current 1D model. In some cases, these limitations result from 2D and three dimensional (3D) processes that are not possible to capture with a 1D model. The 1D model cannot capture complex floodplain hydraulics, which is important for a basin where there is often water outside the channel. The model can also not represent the effects an upstream cross section has on a downstream cross section, especially in the case of levee breaching. Additional information could be collected and applied to improve the 1D model results, but additional data will not impact the ability of the 1D model to replicate 2D and 3D processes. More in channel bathymetry, topographic data extending to the valley walls, and information regarding oxbow connections to the main channel could be collected to improve the model results. This section details the limitations of the current model used for this hydraulic analysis and describes additional data that could be collected to improve the model results.

5.3.5. One-dimensional model

Numerical modeling, such as done here with HEC-RAS, provides a useful tool for analyzing hydraulics in a channel resulting from channel geometry, flow rate, and the presence of structures (weirs, culverts, bridges). The objectives of each modeling effort help determine the type of model used to investigate significant flow patterns and represent the important processes. One-dimensional models are capable of simulating longitudinal changes in hydraulics while neglecting vertical and lateral variation. 2D models incorporate lateral differences in velocity and water surface elevation, and 3D models add the vertical components of velocity non-parallel to the stream bed. Interpretation of channel hydraulics with lower dimensional methods requires understanding the limitations of the model results.

A 1D model was selected to represent large-scale high flow inundation patterns of 60 river miles of Catherine Creek, State Ditch, and the Grande Ronde River. While State Ditch is fairly uniform in channel dimensions and levees widths, about 50 miles of the modeled reaches of Catherine Creek and Grande Ronde River are highly sinuous with broad floodplains and a myriad of disconnected oxbows, abandoned or breached levees, and more distant formidable levees. With complex floodplain hydraulics, a 2D model is necessary to capture lateral variations in water surface and velocities. However, the 1D model can provide useful information as to initial levee overtopping and approximate flows at which disconnected oxbows are inundated. The 1D model results for this analysis will be valid for discharges that result in flows remaining between the levees. However, once flows overtop the levees, the velocities and shear stresses within the channel and floodplain lose validity. Several limitations apply to the depth maps; these are discussed in detail in Section 4.3.

5.3.6. Levee Overtopping

Within HEC-RAS, one levee element can be used to contain flow on each side of the channel. The program does not allow flow to access areas outside of the defined levee element until a water surface elevation is reached that overtops the elevation set for the levee. In the model developed here, levee elements were used to represent the closest visibly unbreached levee on each side of the channel. In the absence of a levee, levee elements were often placed adjacent to the channel bank within the cross section to keep flow from accessing lower swales, side channels, or floodplain areas without overtopping the channel banks first. The 1D model cannot represent the effects that the upstream section may have on the downstream section. For example, an upstream cross section may show levee overtopping at a 10-year discharge. However, the next downstream cross section could result in all flow being conveyed between the levees with no levee overtopping. Once flow overtops the levee in an upstream cross section, flow will be conveyed outside of the levee in downstream cross sections unless there is a mechanism for flow to be conveyed back to the channel. These complex flow patterns require a 2D model to adequately capture processes occurring between the cross sections. Once levees are overtopped, hydraulics outside of the levees may not be accurate. Because of this model limitation, reach-averaged hydraulics are applied to define the extent of inundation and cross-section averaged hydraulic parameters (water surface elevations, velocities, and shear stresses). Caution should be used in interpreting the results to represent absolute values for each hydraulic parameter.

5.3.7. Low Flow Channel

In-channel topography data were collected in reaches where access was granted. In areas where access was not possible, LiDAR was often used to estimate the low flow channel elevations. Modified LiDAR data were utilized to represent the bed surface in the downstream-most 1 mile of the Grande Ronde River just above Rhinehart Gap and also upstream of RM 37.9. Within these areas, the modeled bed elevation is likely different than the true bed elevation. In addition, two short sections of Catherine Creek from RM 32 to 34.5 and from RM 27 to RM 30 could not be accessed for surveys. Within these reaches, the bed elevations were linearly interpolated from upstream and downstream bed elevations. Model results within the areas where modified LiDAR or linear interpolation was performed to develop the in-channel surface are not likely providing accurate estimates of flows less than the 2-year discharge. However, once a high percentage of flows are conveyed overbank, the percentage of discharge conveyed in low flow channel becomes less important. Therefore, model results of higher discharges are likely minimally affected by the bed elevations. Additional data could be collected in these reaches to improve model results for more frequent floods. Sensitivity tests could also be run to evaluate the impacts of the low flow channel bed elevations on model results.

5.3.8. LiDAR Extent

Another limitation of the model results from the limited extent of LIDAR data. In many cross sections along the Grande Ronde River, State Ditch and Catherine Creek, the LiDAR does not extend far enough across the floodplain to capture all of the high flow discharges evaluated. In a few cross sections, the model does not contain 10-year discharge. A greater number of cross sections do not contain discharges in excess of a 50 to 100 year recurrence interval. At these locations, the model creates vertical walls along the boundaries of the cross sections. In reality, flows would likely extend much farther across the channel floodplain until a true topographic feature were encountered that limits the inundation extent. The model likely overestimates the water surface elevations for discharges exceeding the lateral cross section extent. Additional data, such as a USGS DEM data, could be utilized and cross sections elongated to contain all of the discharges. This would be a considerable effort, and may only be warranted in areas where more detailed investigations are needed.

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