EVALUATION OF ALTERNATIVES FOR IMPROVEMENTS IN CARRYING CAPACITY OF THE NORTH PLATTE RIVER AT NORTH PLATTE

FINAL TECHNICAL MEMORANDUM

July 12, 2011

BACKGROUND

The North Platte River channel and overbank areas at North Platte have limited capacity to carry flows through developed areas over several miles upstream of the CNPPID diversion dam which is located below the confluence of the North and South Platte Rivers. The Platte River Recovery Implementation Program office has requested an evaluation of alternatives for improving this capacity as part of meeting its water-related goals. The target improvement is to increase the carrying capacity of about 1,500 cfs at a 6.0 ft (gage datum is 2793.28 feet NAVD88, and NWS flood stage elevation is 2799.28 feet NAVD88) flood stage level at the DNR North Platte River at North Platte stream gauge, to 3,000 cfs without increasing the risk to properties on the floodplains, including Cody Park. Based on the current rating curve for the gage, it would be necessary to reduce the stage by 0.8 feet to achieve this capacity. The gauge is located on the right bank of the North Platte River 150 feet downstream of the U.S. Highway 83 Bridge. The gage was maintained by USGS until 1994, and is currently maintained by the Nebraska Department of Natural Resources (NDNR).

The National Weather Service (NWS) announced on 9 September 2002 that it was changing the previous 1994 flood stage of 6.0 ft (corresponding to a flow of around 3,800 cfs) to 5.7 ft (corresponding to a flow of around 1,980 cfs). Other updates have since occurred, including two in 2008, with the current (2010) stage being 6.0 ft, but corresponding to a flow rate of only 1,500 cfs.

Previous studies (Parsons 2003, FLO 1992) show that discharges corresponding to flood stages in this range fluctuate (in cycles) over time. From 1971 to about 1990, there appeared to be about a 9-year cyclic trend in stages for discharges around 2,000 cfs, with swings of 1- to 1.5-ft, but with no apparent gradual increase or decrease in stage. The cycling of the gauge heights prior to 1991 was moderately sinusoidal with a half period of about nine years. The rise that began about 1990 was at the same rate as earlier rises, but rather than peaking, it continued upward until around 2000 where it appeared to have flattened and remained steady until 2002.

FLO Engineering (1992) reported that stages at the North Platte gauge have a periodic 0.5 ft rise and fall with "no net change" during their observation period leading up to 1992, which corroborates the conclusion by Parsons that from 1971 to about the same time, conditions were relatively unchanging.

A total of 101 agency measurements of flows around 2,000 cfs near the U.S. Highway 83 Bridge from 1971 to 2000 show that around 1991 there were moderately-abrupt changes in the hydraulics, the cause of which is unclear. Water surface top widths and flow areas were stable until around 1991 when both experienced a quantum rise (not gradual) for the same flow rates and remained at the elevated values through 2000. Channel top widths at about this same time increased by about 200 ft (70 percent) and flow area increased by about 120 sq ft (20 percent). Similarly, average flow depths and average velocities for flows around 2000 cfs were steady until about the same time (circa 1991), then experienced respective "quantum" declines of 0.6 ft (30 percent) and 0.5 fps (20 percent).

As part of a field monitoring study during a short-term pulse release from Kingsley Dam, SEH (September 2009) evaluated the options for, and effectiveness of, removal of phragmites from the floodplains upstream and downstream of Highway 83. They reported that the stage at 3,000 cfs had increased about 1 ft since 1994, which corroborates Parsons' findings. They noted that the only change they observed was growth of Phragmites, but their modeling indicated that Phragmites accounts for only about 0.3 to 0.5 ft of this 1 ft increase. They attribute the difference to a bed elevation rise about 2000 ft downstream of Highway 83 plus a constriction at the east end of Cody Park. Further, they note that the reluctance by NWS to increase the flood stage discharge is related largely to problems at Cody Park rather than properties upstream of Highway 83. . Among their recommendations was a proposal to spray in-channel phragmites in both directions from the highway bridge. Another suggestion was to construct in-river "natural" spur dikes.

The new levels of these parameters had stayed about the same through 2002 (Parsons, 2003). Parsons' key finding was that the changes are definitely evident, and that they were episodic rather than gradual. Something that happened or began to happen around 1991 appears to have caused these changes.

From 2007 to 2009, the Program office implemented several improvements upstream of the Highway 83 Bridge, largely focusing on removal of Phragmites and Purple loosestrife by spraying and disking. Spraying and removal occurred downstream of the Highway 83 Bridge in 2010. This resulted in an apparent improvement of carrying capacity in 2009 to around 1,600 cfs, and based on NDNR discharge rating curve updates in 2010, the carrying capacity at a 6.0-ft stage is currently approximately 1,500 cfs.

This report provides a preliminary assessment of options for improvements downstream of Highway 83 and concludes with recommendations for three (3) of the most promising candidate actions for analysis in greater depth using computer hydraulic and sediment transport modeling.

THEORIES ON THE CAUSES OF THE CHANGES IN CARRYING CAPACITY

In order to evaluate alternatives that might increase carrying capacity, an understanding of causes of reductions in, or at least cycling of, carrying capacity is needed. Potential causes of change in carrying capacity and potential remediation options to increase carrying capacity were described by Parsons (2003), J.F. Sato (2005), and SEH (2009).

The studies focused largely on problems and potential improvements upstream of the Highway 83 Bridge. To some extent Sato's and SEH's were based on the assumption that channel capacity needed to be increased. Parsons' options primarily addressed improvements needed in the overbank areas. Thus, two general theories have evolved that either (1) the main channel has lost capacity, or (2) the floodplains have lost capacity.

1. The main channel capacity has decreased, causing overflow onto floodplains at lower flow rates. This position is held somewhat by J.F. Sato, who reported a "trend toward decreased conveyance in the [main] channel." SEH focused almost entirely on channel capacity as well, attributing about half of the 1-ft rise in stages to in-channel phragmites and the rest to a bar obstruction at Cody Park and a bed obstruction downstream. Causes of this condition could be the result of:

a. Narrowing of the main channel width or flow area due to reductions in peak and/or mean annual flows;
b. A rise in bed level (aggradation) due to sediment deposition caused by channel restrictions (see SEH report);
c. Lateral obstructions resulting from point bars;
d. A change in active channel morphology or roughness;
e. Vegetative encroachment in the channel and/or on the floodplains;
f. Armoring of the bed preventing bed mobilization and deepening during moderate flows;
g. Changes in morphology from braided to anabranched;
h. Deposition in the slack water zones upstream of flow restrictions (e.g., channel narrowing, bridges, or the Tri-County diversion dam);
i. Changes in size or numbers of point bars and transient macroforms;
j. Changes in arrival and departure rates of slow-moving macroforms creating temporarily-elevated stages; or
k. Other minor related causes mentioned in the reports.

2. The floodplain conveyance has decreased, resulting in higher stages on overbank areas for the same discharge rates. This primary cause was suggested by Parsons. Sato, and to a lesser extent SEH, both identify it at least as a contributing factor. Although Sato appeared to focus on declines in both channel and floodplain capacity, about 80 percent of their alternatives involved floodplain improvements. All of SEH's primary alternatives related to channel capacity problems. Causes of this condition could include:

    a. Aggradation on the floodplains due to flow restrictions (mostly vegetative encroachment);
    b. Sediment budget impacts of gravel mining;
    c. Slack water deposition effects of roadway embankments or elevated building foundations;
    d. Shortening of bridges;
    e. Vegetative encroachment in floodplain surfaces and chutes;
    f. Heavy development on the south overbank that forces greater portions of the total flow onto the north floodplain;
    g. Blockage of the historical natural chute leading to and downstream of the Hwy 83 box culvert;
    h. Blockage of flow paths by alteration of historical braids and drains on the floodplains, or
    i. Other minor related causes mentioned in the reports.

It is possible that both factors have resulted in the changed condition. Although evaluated by several agencies and consultants, no consensus of the primary cause of changes in carrying capacity has been reached. This makes it difficult to assure that proposed remedial measures focus on reversing or at least managing the causes. In the absence of full understanding of causes, analysts can only resort to recommending and then creating computer models of measures that are generally known to result in increased carrying capacities by either or both conveyance systems.

POTENTIAL REMEDIATION ACTIONS

The following six individual alternatives for improvements downstream of Highway 83, not listed in any particular priority order, are considered to be worthy of possible testing with computer modeling. The alternatives are separated into two categories involving (1) hydraulic improvements and (2) sedimentation management options.

Hydraulic Improvement Options

Wherever water surface profiles rise above normal depth in river reaches, water surface stages for moderate flow rates can possibly be lowered to normal depth by removing or altering impediments that create above-normal-depth backwater profiles. The impediments can be man-made or naturally-produced features such as point bars or river bends. If modeling shows that water levels at a flow rate of 3,000 cfs anywhere in the subject reach are above a normal flow profile, one or more of the following alternatives could possibly be implemented:

1. The addition of a box culvert north of the Highway 83 bridge crossing combined with vegetation removal and possibly enlargement of the overbank (floodplain) channels upstream and downstream of the culvert could provide relief to the overbank stages, which in turn could reduce overall stages. The floodplain channels would not be part of the active bed of the river and would carry relatively sediment-free flow, allowing the alternative to be evaluated with HEC-RAS. It is identified as Alternative 1.
2. Previous work by the Program upstream of the highway has improved flood stages upstream of the Highway 83 Bridge, leaving primarily the Cody Park area as the most vulnerable to flooding at 3,000 cfs. Flood-proofing of the park would remove this vulnerability by installation of an earthen "levee" along the bank at Cody Park with a crest elevation set with a freeboard of about 1 ft above the current stage at 3,000 cfs. It is assumed that NWS would respond by increasing the flood stage level, because no properties would be at risk at 3,000 cfs. The south-bank levee, probably about ½ mile long downstream of Highway 83, would not need to meet FEMA criteria for levees with floodways. It would only need to be of sufficient height to allow 3,000 cfs to pass by in the river without entry by the flow into the park facilities. Hydraulic modeling might prove that a levee as small as 1- to 3-ft tall above the existing bank level could double the carrying capacity from 1,500 to 3,000 cfs in this reach. Culverts with flap gates or stop logs might need to be installed in the levee to allow drainage back to the river of trapped local rainfall runoff or overtopping flows. Because the required elevation of the levee crest can be established using HEC-RAS, it is identified as a hydraulic improvement option. This is identified as Alternative 2.

None of the above options require mechanical or hydraulic alterations in the streambed sediments, and would likely make environmental permitting less cumbersome, although floodplain permitting requirements may increase compared to other options. Regardless, their potential can be evaluated using HEC-RAS as the primary tool.

Sediment Management Options

Because the SEH report hypothesizes that in-channel and at-bank sedimentation is the primary cause of stage changes, this set of options focuses on altering sediment transport processes through the downstream reach. Because each involves work below the ordinary high water mark (OHWM), environmental permitting would be more complex. The sediment management options include:

3. Re-activation of the former north bank main channel downstream of Highway 83 to and beyond the Cody Park restriction would have the effect of widening the active channel and increasing its carrying capacity. This channel was formerly part of the active bed but has experienced sediment deposition and vegetative establishment. The re-activation process would involve restoring the channel's upstream and downstream connection to the main channel, and restoring its invert to match the river's invert, allowing it to function as part of the main channel. Observations taken by Program staff in May 2011 indicate that this channel is at least partially connected to the main channel at flows of approximately 2,500 cfs. However, there is no clear reconnection of this channel back to the main channel, and a significant portion of water flowing through the channel either becomes ineffective flow area or floods the north bank floodplain. As a result, this alternative would focus on a) how to reconnect the channel at flows less than current flood stage, and b) how to route flow in the channel back to the main channel. This would need to be evaluated with the sediment transport model and is identified as Alternative 3.
4. A head cut at, and upstream of, the Tri-County diversion dam could be induced by opening the gates to sluice sediments or by increasing dredging upstream of the diversion dam. Dredging plus bypass of sediments needs to be in balance with sediment arriving at the diversion. If the modeling shows that there is an imbalance in transport, either overall or at various times of the year, recommendations for changes in gate operation or dredging could arise from the study. This would need to be evaluated with the sediment transport model and is identified as Alternative 4.
5. A moderately large sand deposit exists on the south bank, beginning just upstream of the UPRR Bridge and extending downstream past the existing sand pit lakes. The lakes are separated from the channel by a wide berm that extends into the channel and is part of this deposit. This "bulge" may be producing backwater in the 1,500 to 3,000 cfs range that may affect flood stage in the reach upstream and downstream of the Highway bridge. Mechanical removal of sufficient deposited sediments in this obstruction, with a partial realignment of the wide berm that separates the sand pit from the river, along a projection of the river's edge upstream and downstream of the deposit could provide significant additional channel capacity to accomplish the recovery goal. Coordination with UPRR and the owners of the sand pit ponds would be required. This sediment management option is identified as Alternative 5.
6. Alternative 2 (levee along Cody Park) would allow releases of pulse flows in the 3,000 cfs range without exceeding the upgraded NWS flood stages. This would have the potential of further re-activating the north channel (Alternative 3) by natural processes. A combination of construction of the Cody Park levee (Alternative 2) along with full re-activation of the north channel (Alternative 3) may provide better carrying capacities than either individual component. Sediment transport modeling of this combination is identified as Alternative 6.

Preliminary Evaluation and Ranking of Alternatives

The above list contains two options for hydraulic modifications and four involving sediment management options. The scope of work for the current investigation requires that three remediation alternatives be recommended as primary options to be analyzed with the hydraulic and sediment transport models.

A wide range of individual activities that might improve flood stages both upstream and downstream of Highway 83 were reviewed, and have been narrowed them down to the six options listed above.

To further narrow this list, a matrix evaluation method was applied to identify three for modeling. Criteria selected for the evaluation included considerations of the potential of each meeting the 3,000 cfs goal as well as permanence, impacts on others' properties, budgetary constraints, and permitting (both environmental and floodplain development).

The criteria are shown as columns in the attached table. The far right column shows how the alternatives ranked, with 1 being the highest and 6 the lowest. They are:

Rank, Highest to Lowest	Alternative No.	Description
1	6	Re-activate N. Bank Channel and Install Cody Park Levee
2	5	Remove S. Bank Deposit @ UPRR and Sandpit
3	3	Re-activate N-bank Channel
4	2	Floodproof-Type Berm at Cody Park
5	1	N. Overbank Floodplain Relief Channel and New Culvert
6	4	Induce Headcut at Diversion

It was recommended that the three top-ranked options would be carried forward for the modeling analysis. All three are sediment management options, although HEC-RAS can be used to evaluate fixed-bed versions of any of them.

North Platte Channel Choke Alternative Evaluation Matrix, May 6, 2011 Alternatives Ranked by HDR Team from 1 to 5, with 5 having highest potential in each category									
	Alternatives	**High Stand-Alone Potential to Allow Full 3,000 cfs?**	**High One-time Potential (Perma-nence)?**	**Potential to Eliminate Flooding at Cody Park at 3,000 cfs?**	**Small Impact on Other Public/Private Properties?**	**Relative Ease of Permitting?**	**Is Cost Feasible Within Program Budget?**	**Totals by Row**	**RANK**
Hydraulic Improvement Options									
1	N. Overbank Floodplain Relief Channel and New Culvert Channel U/S and D/S of 83, Including New Culvert	2	5	3	3	5	3	21	**5**
2	Floodproof-Type Berm at Cody Park from Hwy 83 to 1/2 mile D/S	3	5	5	3	3	4	23	**4**
Sediment Management Options									
3	Re-activate N-bank ch d/s of 83	3	4	3	5	4	5	24	**3**
4	Induce Headcut at Diversion	3	1	3	4	3	4	18	**6**
5	Remove S Bank Deposit @ UPRR and Sandpit	4	5	5	3	2	5	24	**2**
6	Combine Alts. 2 and 3 Above	5	5	5	4	3	3	25	**1**

ALTERNATIVE EVALUATION

General Comments on Analysis and Results

The three alternatives with the highest rank (Alternatives 6, 5, and 3; **Figure 1**) were evaluated for their effectiveness to increase the carrying capacity from the current discharge of 1,500 cfs to 3,000 cfs without increasing stage. Based on the current gage rating curve, this increase in capacity corresponds to a reduction of the 3,000 cfs stage from about 6.8 feet to 6.0 feet. The analysis was performed using the 1-D steady state hydraulic and sediment-transport models developed by the HDR and Tetra Tech team as part of the Program's 1-D modeling project (HDR and Tetra Tech, 2011). The models extend from about 5-miles upstream from the Highway 83 Bridge to the Tri-County Diversion Structure, a distance of about 10 miles (**Figure 2**).

Sediment transport modeling results generally indicate that aggradation would occur throughout the 10-mile model reach for the baseline condition and for all of the alternatives considered (**Figure 3a**). This prediction is indicative of a system with greater sediment supply than sediment transport capacity, at least for the hydrology and sediment input used in the model (historical hydrology from Water Year (WY) 2002 to WY2009, and corresponding sediment rating curve). For this reason, the alternatives considered for increasing hydraulic capacity may have the potential to reduce aggradation relative to the baseline condition but not necessarily to increase hydraulic capacity over the long term unless there are either increases in sediment transport capacity (e.g., through higher flows), or decreases in sediment input.

In general, the alternatives that were selected for detailed evaluation were targeted to reduce backwater effects due to constrictions and increase the conveyance through overbank areas or along the main channel. The constriction points evaluated were the Cody Park constriction under Alternatives #3, #3a and #6, and the constriction at the UPRR Bridge under Alternative #5. Conveyance increase through overbank areas was evaluated as part of Alternatives #3, #3a and #6, and along the main channel as part of Alternative #5. Results from the hydraulic modeling indicate that, while the constrictions evaluated do cause a small decrease in velocities in the upstream "backwater" areas, these backwater effects are relatively minor and do not extend over large distances. As a result, the sediment-transport modeling results generally indicate that increasing the conveyance in the left overbank at the Cody Park restriction and widening the main channel at the constriction caused by the UPRR Bridge tends to exacerbate deposition at and upstream from those constrictions. This is due to the reduced velocities associated with the wider conveyance area (**Figure 3b**). Increasing the overbank flow conveyance in the vicinity of the Cody Park restriction reduces the flow and sediment-transport capacity in the main channel, thereby causing a slight increase to the deposition at and upstream from the entrance to the reactivated channel. However, downstream from the entrance to the reactivated channel, a significant reduction to the main channel deposition occurs due to increased storage of sediment in the overbank flow paths.

Model Background

The relative hydraulic effects of the various components of each alternative were initially evaluated by comparing the results of the with-project, steady-state models to those from the existing (baseline) conditions model. The steady-state modeling was carried out using the Corps of Engineers HEC-RAS (version 4.1.0) software (USACE, 2010). HEC-RAS is a one-dimensional (1-D) hydraulic model that can be used to perform steady-state, step-backwater computations, unsteady-flow hydrograph routing, and movable bed sediment-transport modeling. The effects of the alternatives on the long-term sediment-transport characteristics were evaluated by comparing the aggradation/degradation patterns and associated hydraulic conditions predicted by the with-project sediment-transport models to the values predicted by the baseline conditions sediment- transport model. Due to data output limitations, HEC-RAS version 4.1.0 was not used for sediment transport modeling for this project. Instead, the sediment-transport model was developed using HEC-6T (version 5.13.22 08p; MBH, 2010). HEC-6T is a sediment-transport program that uses algorithms similar to the Corps' HEC-6 and HEC-RAS (Version 4.1.0, USACE, 2010) sediment-transport module. The steady-state HEC-RAS modeling represents the effects of the alternatives under existing (short-term) conditions, while the sediment transport modeling that was conducted using an 8-year simulation period provides insight into the longer-term effects of the alternatives. In assessing the model results, it is important to note that predicted deposition or erosion at one location could affect the hydraulic and sediment-transport conditions at up- and downstream locations.

The original hydraulic model for the North Platte River included cross-sections spaced at approximately 1,600 feet (Figure 1). Channel width is typically approximately 300 to 400 feet, but narrows to approximately 170 feet at the "Cody Park restriction" approximately 0.5 miles downstream of the Highway 83 Bridge (approximate Station 8585+00 in Figure 1). The original hydraulic model did not include a cross-section at the Cody Park restriction, and predicted a velocity of approximately 4.7 feet per second at 3,000 cfs upstream of the restriction. A cross-section was added to the hydraulic model at the restriction to estimate the backwater effect upstream of the restriction. The additional cross-section resulted in velocity of approximately 3.5 feet per second upstream of the restriction. With the additional cross-section at the restriction, water surface elevation near Cody Park at 3,000 cfs increased by approximately 0.8 feet relative to the original hydraulic model (**Figure 4**). Decreased velocity and increased water surface elevation for the additional cross-section indicate that there is a backwater area upstream of the restriction where sedimentation may be occurring. Although the added cross-section indicates a backwater area upstream of the restriction, the effects on the hydraulics are not anticipated to be significant (based on good calibration to measured data over a wide range of discharges for the original hydraulic model). The addition of a cross-section at the restriction could also decrease the stability of the sediment transport model as a result of variations in cross-section spacing. For these reasons, the cross-section at the restriction was not included in the models used to estimate effects of the alternatives described in this memorandum.

Baseline Simulation

A baseline simulation was carried out using the calibrated HEC-6T model that was developed for the previous 1-D modeling project (HDR and Tetra Tech, 2011) to provide a basis for comparison with model results for alternatives considered to increase hydraulic capacity. Input for the baseline model calibration runs included upstream and tributary inflows based on measured data at the Highway 83 and Lincoln County Drain No. 1 gages over the 8-year period between WY 2002 through WY2009. The baseline simulation also included the 50-day "warm-up" period that was used in the original calibration.

After calibration, the hydrology was modified to include Short Duration High Flows (SDHFs), and the models were re-run to assess how the reach would have responded with implementation of SDHFs. Consistent with the Program's goals for implementing SDHFs, this was accomplished by inserting into the flow record a discharge at the upstream end of the reach of 3,000 cfs over the 3-day period between April 18 and April 20 in 2 out of every 3 years, resulting in six SDHFs (WY2002, WY2003, WY2005, WY2006, WY2008, and WY2009). Comparison of the change in mean bed elevation predicted by the calibration run without the SDHFs and the baseline run with SDHFs indicates that the SDHFs would generally result in slightly to moderately less aggradation in the reach. An exception to this general conclusion occurs in the 2.4-mile reach upstream from Highway 83, between Stations 8766+00 to 8639+00, where an additional approximately 0.5 feet of aggradation occurs (**Figure 5**). It should be noted that the calibration run used the existing geometry for the starting conditions; thus, the predicted bed elevation changes represent the changes that would likely occur over an 8-year period if the hydrologic conditions were similar to the WY2002 and WY2009 period.

Alternative #6 (Rank #1)

Alternative #6 consists of constructing an approximately 0.5-mile long levee along the South Bank downstream from Highway 83, and reconnecting the overbank channel along the north bank in the vicinity of Cody Park (Figure 1). The levee top elevation would be established to provide about 1-ft of freeboard above the water-surface elevation at 3,000 cfs.

Steady State Hydraulic Modeling and Analysis

South Bank Levee at Cody Park

Two potential levee alignments (**Figure 1**) were initially evaluated for this alternative by comparing the existing elevation along the proposed alignments with the predicted water-surface elevation at 3,000 cfs. The comparison indicates that the existing ground surface contains the 3,000 cfs water-surface along the upstream 2,000 feet of both alignments, and overtopping of either alignment only occurs for short distances in the downstream portion of the reach (**Figure 4**). It also appears that some flow could enter the downstream limit of Cody Park at 1,560 cfs near Station 8580+00.

Because the ground elevations behind the existing berm in Cody Park are well above the elevation where flow can overtop the berm (Elevation 2798 feet), it is unlikely that significant backwater associated with a flow breakout would affect Cody Park. However, based on observations by Program staff on May 16, 2011, minor overtopping of the high ground adjacent to the river occurred at an approximate discharge of 2,500 cfs (pers. Comm. Steve Smith, May 2011). As discussed above, the original hydraulic model did not include a cross-section at the Cody Park restriction, so a cross-section was added to the hydraulic model at the restriction to estimate the backwater effect upstream of the restriction. Results from this model indicate slightly elevated water-levels upstream from the added section, but that the 3,000 cfs water level would still only overtop the existing berms at the same locations indicated by the original model. Both models (with and without the Cody Park restriction cross-section) indicate that flows would be contained within the existing channel for flow up to at least 3,000 cfs, which contradicts the observed overtopping at about 2,500 cfs,. This indicates that channel topography may have changed between the survey and LiDAR data used in model calibration and the high flows when the overtopping was observed.

The model results indicate that increasing the height of the berm would probably not mitigate flooding in Cody Park. As a result, and because the resolution of the model would have to be significantly increased (i.e., cross section spacing of less than 100 feet) to evaluate the hydraulic conditions in the vicinity of the predicted overtopping, the increased levee heights associated with any improvements were not incorporated in the model geometry.

Reactivated North Bank Channel

The existing profile at the upstream end of the reactivated north bank channel contains high ground or a berm that will not allow low flows to enter the overbank. In addition, a majority of the overbank contains relatively heavy vegetation, as shown on the land use coverage and aerial photographs. Therefore, three scenarios were initially evaluated to assess the potential conveyance along the reactivated overbank channel along the north bank in the vicinity of Cody Park, including:

- Scenario 1: No mechanical excavation or vegetation clearing;
- Scenario 2: Excavated pilot channel through the berm, approximately the upstream most 2,300 feet with vegetation clearing along the excavated area only, and
- Scenario 3: Excavation at entrance to reactivated channel and significant vegetation clearing along the channel and the surface between the reactivated channel and main channel (**Figure 1**).

Scenario 1 was modeled in HEC-RAS by adding a split flow reach to the reactivated channel and eliminating the ineffective flow areas that were used in the original model along this flow path (Figure 1). For Scenario 2, a trapezoidal channel with a 40-foot bottom width and 2H:1V side slopes was inserted along the primary historic channel in the upstream most 2,300 feet of the split flow reach, with a profile set to approximate the slope in the better-defined downstream portion of the channel. The vegetation clearing associated with this scenario was modeled by setting the hydraulic roughness (Manning's *n*-value) in the excavated channel to 0.028, which is the same value as the main channel.

The hydraulic analysis for Scenario 3 included removal of the berm that currently blocks discharges above about 2,000 cfs from entering the split-flow channel. In addition, a trapezoidal channel with a 40' bottom width and 2H:1V side slopes along the upstream most 600 feet of the reactivated channel was inserted (**Figure 6**). The vegetation clearing associated with Scenario 3 (shown in **Figure 1**) was modeled by setting the n-value in the excavated channel to 0.028 and the n-value along the bar surface between the reactivated channel and the main channel to 0.035.

Results from the hydraulic analysis indicate that only about ~2% of the total flow at 3,000 cfs is carried in the reactivated channel under Scenario 1. More significant conveyance (~9% and 8% of the total flow, respectively, at 3,000 cfs) occurs under Scenarios 2 and 3 (**Table 1**). Since the additional conveyance under Scenarios 2 and 3 is similar, but costs and required permitting efforts for Scenario 3 would be substantially less than Scenario 2, the Scenario 3 configuration was selected for evaluation with the sediment-transport model.

Table 1. Summary of predicted split flows for various configurations of Alternative Number 3 (North Channel Reactivation).

Total Discharge (cfs)	Discharge Delivered to Reactivated North Channel (cfs)			Percentage Delivered to Reactivated North Channel		
	[1]Scenario 1	[2]Scenario 2	[3]Scenario 3	[1]Scenario 1	[2]Scenario 2	[3]Scenario 3
250	0	0	0	0%	0%	0%
500	0	4	3	0%	1%	1%
1000	0	31	25	0%	3%	3%
2000	2	78	62	0%	4%	3%
2500	25	196	181	1%	8%	7%
3000	51	271	248	2%	9%	8%

[1]Scenario 1: No mechanical excavation or vegetation clearing.

[2]Scenario 2: Excavated pilot channel along upstream 2,300 feet and vegetation clearing in excavated area only.

[3]Scenario 3: Excavation at entrance to reactivated channel with significant vegetation clearing.

Sediment-transport Modeling and Analysis

The modifications that were made to the hydraulic model for Alternative #6, Scenario 3, were incorporated into the calibrated HEC-6T sediment-transport model (HDR and Tetra Tech, 2011). Because the hydraulic modeling indicated that the south bank berm would not significantly affect hydraulic conditions, this component was not included in the sediment-transport model. The revised model was executed over the same 8-year hydrologic sequence used in the baseline simulation that included the 50-day warm-up period and the 6 annual SDHF flows of 3,000 cfs for 3 days.

Results from the Alternative #6 simulation indicate that the increase in mean bed elevation at the end of the simulation (relative to the end of the warm-up period) is somewhat less than that under baseline conditions along most of the reach between the Highway 83 Bridge and the UPRR Bridge, with reductions of between 0.3 and 0.4 feet lower than the baseline condition at the end of the simulation along most of this reach (**Figures 3a and 3b, Table 2**). However, the overall volume of aggradation in this reach is higher for Alternative #6 than for baseline conditions (**Figure 7**). The increased aggradation in the short reach just downstream from Highway 83 occurs because more flow is carried in the reactivated channel and overbanks, reducing the flow and sediment transport capacity in the main channel. This increase in aggradation causes a minor backwater effect that also causes additional aggradation for a short distance upstream of the Highway 83 Bridge (**Figures 3a, 3b and 7**). The additional sediment stored in this part of the reach reduces the downstream sediment supply, slightly reducing aggradation volumes in the downstream reaches.

Table 2. Summary of subreach limits used to evaluate the sediment-transport model results.					
Subreach	U/S Limit	D/S Limit	U/S Sta (ft)	D/S Sta (ft)	Length (ft)
Subreach 1	U/S Model Limit	US Hwy 83	8867+90	8603+90	26,400
Subreach 2	US Hwy 83	UPRR Bridge	8603+90	8489+10	11,480
Subreach 3	UPRR Bridge	US Hwy 30	8489+10	8450+10	3,900
Subreach 4	US Hwy 30	Tri-County Diversion	8450+10	8313+60	13,650

To evaluate the potential effects of this alternative on flood stage, the predicted model geometry at the end of the baseline and project conditions sediment-transport simulations was incorporated into the steady-state HEC-RAS model (HDR and Tetra Tech, 2011). The model was executed for a discharge of 3,000 cfs. This comparison indicates that the proposed changes under project conditions would lower the water-surface elevation by as much as 0.3 feet in the vicinity of the reactivated channel, increasing the in-channel capacity from about 1,700 cfs under baseline conditions to about 2,200 cfs under this alternative. This effect extends downstream to below U.S. Highway 30, with reductions in the water-surface elevations of about 0.1 feet (**Figure 8**). However, this alternative appears to provide essentially no benefit to flood stage or channel capacity at the Highway 83 gage (**Table 3**) in the vicinity of the bridge. Detailed evaluation of the model results indicates that a significant amount of the aggradation occurs during the extended period of high flows in 2002 (**Figure 9**), through further building of the mid-channel bar upstream from Highway 83 (**Figures 10 and 11**). Because this is the only extended period of high flows in the simulation, the deposition on the mid-channel bar persists, resulting in an increase in the 3,000 cfs water-surface elevation of up to 0.4 feet compared to baseline conditions in the approximately 3,600 feet reach upstream of Highway 83 (**Figure 8**).

Table 3. Summary of predicted stage for 3,000 cfs and estimated capacity at NWS Flood Stage at the Highway 83 gage at the end of the simulations for Baseline Conditions and each alternative.		
Scenario	Predicted Stage at Gage, 3,000 cfs (ft, NAVD 88)	Estimated Capacity at Gage (cfs)
Baseline Conditions	2800.26	1610
Alts #3 and #6	2800.23	1610
Alt #5	2800.29	1560
Alt #3a	2800.15	1680

Alternative #5 (Rank #2)

Alternative #5 includes widening of the channel through the UPRR Bridge, and setting back the bank and sandpit levees up- and downstream from the bridge along an alignment that matches the main channel approaches to this existing channel constriction (Figure 1).

Steady State Hydraulic Modeling and Analysis

The geometry for this alternative was represented in the hydraulic model (HDR and Tetra Tech, 2011) by modifying the right overbank area through the widened reach (**Figures 12 and 13**). A 2H:1V sideslope was used for both the setback banks and levees, and the main channel roughness value (n=0.028) was extended to the revised top of bank. For the sections with the setback sandpit levees, a 30-foot top width was assumed for the levee crest, and the ineffective flow areas used to eliminate conveyance through the existing sandpits in the baseline model were modified to the extents of the revised sandpit riverbank geometry.

Results from the hydraulic model indicate that the channel widening under this alternative lowers the 3,000 cfs water surface by as much as 1.2 feet at the downstream face of the UPRR bridge, where the largest cut volume occurs (**Figure 14**). The lower water-surface elevations extend for a distance of about 3,800 feet upstream from the bridge, but have essentially no impact farther upstream.

Sediment-transport Modeling and Analysis

Although the steady-state hydraulic effects of the Alternative #5 channel widening only extend about 2,500 feet upstream from the improvements (3,800 feet upstream from the UPRR Bridge), it is possible that the associated changes to the aggradational trends could affect the sediment-transport conditions farther upstream. To evaluate this, the geometric modifications were incorporated into the sediment-transport model (HDR and Tetra Tech, 2011) and simulations were run using the 8-year flow record described above. The results indicate that Alternative #5 would cause increased aggradation in the reach between Highway 83 and the UPRR Bridge (Subreach 2 in **Figures 3a, 3b and 7**), because there is a substantial reduction in velocities through the widened reach due to backwater caused by the aggradation near the bridge. The most significant aggradation occurs along the right bank through the UPRR Bridge opening where the existing bar was formed (**Figure 15**). Compared to baseline conditions, the additional sediment trapped at and upstream from the channel widening reduces the downstream sediment supply and slightly lowers aggradation volumes in the downstream reaches. There is essentially no effect upstream from Highway 83.

The predicted geometry from the end of the sediment-transport simulation was incorporated into the steady state hydraulic model, and the modified model was executed for a discharge of 3,000 cfs. The results indicate that the water-surface elevations are similar to or slightly less than the baseline water-surface elevations through the widened reach. This indicates that, despite the aggradation discussed in the previous paragraph, the channel widening under this alternative still results in some benefit to channel capacity in this portion of the reach. The increased aggradation through most of the remainder of the reach between Highway 83 and the UPRR Bridge results in water-surface elevations that are as much as 0.4 feet higher than under baseline conditions, but there is no significant effect upstream from Highway 83 (**Figure 8**). At the gage, the 3,000 cfs stage increases slightly, resulting in a decrease in channel capacity from about 1,610 cfs under baseline conditions to 1,560 cfs under this alternative. Downstream from the widened reach, the water-surface elevation is slightly lower than under baseline conditions due to the lower aggradation levels in the downstream reaches.

In general, the rivers ability to transport sediment is related to grain size and river hydraulic characteristics, in particular slope and velocity. Higher velocity typically causes more sediment transport. As a result, in locations where the project actions would cause a decrease in the in-channel velocity, such as occurs when the split flow channel is activated and the vegetation is removed from the overbanks, reducing the flow in the main channel, the sediment transport rates will decrease and sediment will deposit on the bed. If of sufficient magnitude, the deposition may, in turn, reduce the flow area and increase the stage at high flows. Conversely, flow entering a narrower section will typically experience an increase in velocity, and an increase in sediment transport capacity. Thus, the river may have the ability to pick up more sediment than it is carrying, which can scour the bed, thereby increasing the flow area and reducing the stage.

Alternative #3 (Rank #3)

Alternative #3 consists of reactivation of the north bank channel between the Highway 83 Bridge and the Cody Park Restriction. This alternative is similar to Alternative #6 except the south bank levee that was proposed to protect Cody Park was eliminated. Because the proposed levee under Alternative #6 would have a negligible effect on the hydraulic and sediment-transport conditions, model results would be identical; thus no modeling was carried out for this alternative.

Modifications to Alternative #3

Because none of the selected alternatives appear to have the desired effect of significantly reduced flood stages for a target discharge of 3,000 cfs at the Highway 83 gage, a modification to Alternative #3 was evaluated to determine the potential benefits of additional vegetation clearing and minor earthwork along the right bank bar at the Cody Park restriction (**Figure 1**). The earthwork involved lowering the berm along the upstream side of the bar to a level that is similar to the elevation of the landward side of the bar at Station 8586+00. To evaluate this alternative (identified as Alternative #3a), it was assumed that the small berm along the upstream edge of this bar would be mechanically excavated to an elevation equal to the leeward side of the bar. In addition, the vegetation from the bar would be removed to increase the conveyance at lower discharges. Each of the other elements associated with the north bank channel reactivation under Alternatives #3 and #6 were also included in this alternative.

Alternative #3a was modeled by allowing conveyance over the bar Cross Sections 857526 and 855960 that was blocked-out in the baseline model, and the roughness value was changed from 0.11 to 0.035 to represent clearing of vegetation (**Figure 16**). It should be noted that similar effects would likely be achieved if the vegetation clearing were limited to an area much smaller than that shown in Figure 1, but the more significant clearing was incorporated into the model since this alternative was intended to represent the most extreme improvements in the Cody Park area. Results from the steady state hydraulic model indicate that this alternative would lower the 3,000 cfs water-surface elevation by as much as 0.4 feet between the restriction and Highway 83, but the water surface is only lowered by about 0.1 feet at the gage (**Figure 17**). The reduction in water-surface elevation would result in NWS flood stage capacity of approximately 2,200 cfs between the restriction and Highway 83, and approximately 1,700 cfs at the gage.

The modifications for this alternative were incorporated into the sediment-transport model, and the model was executed over the same 8-year hydrologic scenario. Under this alternative, slightly less change in mean bed elevation occurs in the reaches downstream from Highway 83 than under Alternatives 3 and 6; thus, this alternative appears to be more favorable, (**Figures 3a and 3b**). There would be more aggradation in the reach between Highway 83 and the UPRR Bridge compared to baseline conditions and also compared to Alternative #6, with the increase primarily along the bar at the Cody Park restriction (**Figure 7**). This increased sediment storage in the overbanks results in slightly lower aggradation volumes in the downstream reaches compared to baseline conditions. The 3,000 cfs water-surface profile using the end-of-simulation cross sections from the sediment routing model for this alternative is lower through the Cody Park reach than under Alternative #6. Compared to baseline conditions, the water-surface elevations are as much as 0.4 feet lower through the Cody Park reach, and are slightly higher (about 0.1 feet) in the downstream reaches (**Figure 8**).

Downstream from the UPRR Bridge, the higher water-surface elevations are due primarily to deposition on the mid-channel bars in this reach. This deposition occurs during the high flows early in both the baseline and Alternative #3a simulations, but the reduced deposition along the main channel under Alternative #3a results in less frequent conveyance and erosion of the bar surface later in the simulation. Similar to the results from the Alternative #6 simulation, the 3,000 cfs water-surface elevation upstream from Highway 83 is as much as 0.4 feet higher than under baseline conditions due to higher levels of deposition upstream from the reactivated channel entrance (**Figure 8**). This results in deposition on the mid-channel bar upstream from the highway (**Figure 11**).

Summary and Conclusions from the Alternative Evaluation

As discussed above, the alternatives that were selected for detailed evaluation involved reducing the backwater effects at constrictions and increasing the conveyance through overbank areas or along the main channel. Results from the hydraulic modeling indicate that, while the constrictions of interest do result in some reduced velocities in the upstream “backwater” areas, these backwater effects are relatively minor and do not extend over large distances. As a result, the sediment-transport modeling results generally indicate that increasing the width at the constrictions tends to exacerbate deposition at and upstream from the constriction due to the reduced velocities associated with the wider conveyance area. For similar reasons, increasing the conveyance along the main channel also results in higher levels of aggradation. However, increasing the overbank conveyance results in a significant reduction to deposition in the adjacent main channel due to increased storage of sediment in the overbank flow paths. The following specific conclusions may be drawn from the hydraulic and sediment-transport model results:

1. The proposed levee along the south bank in the vicinity of Cody Park (Alternative #6) will not have a significant effect on mitigating the impacts of flooding in this area at discharges up to 3,000 cfs because the existing ground surface is capable of containing the 3,000 cfs water-surface profile. Although some overtopping of this surface does occur near the downstream (east) end of

the park, backwater flooding associated with these locations is minimal since the ground elevation through most of Cody Park is above the water-surface elevation at the breakouts.

2. Reactivation of the channel in the north overbank below Highway 83 (Alternatives #3 and #6) could be achieved best through minimal earthwork and extensive vegetation clearing. This would result in relatively significant flow conveyance at discharges above 1,000 cfs, with moderate improvements at higher flow rates up to 3,000 cfs. The reactivated channel would reduce aggradation in the main channel, and would ultimately reduce the water-surface at 3,000 cfs by as much as 0.3 feet along the reach of the main channel that is adjacent to the reactivated channel. However, there would be minimal effects on water surface elevation at and near the North Platte River at North Platte Gage, where the 3,000 cfs flood stage is essentially the same at the end of the baseline and Alternative #6 simulations (**Table 3 and Figure 18**). Increased sediment storage in the overbank would also reduce the supply to downstream reaches, reducing downstream aggradation and flood stages. This alternative would likely involve some long term maintenance in the form of vegetation clearing, but some earthwork may also be required to re-grade the predicted aggradation in the reactivated channel.

3. Channel widening in the vicinity of the UPRR Bridge (Alternative #5) would result in decreased velocities that would exacerbate deposition at about the same location, the effects of which would extend upstream to about 4,000 feet below the Highway 83 Bridge. The increased aggradation is offset by the increased overbank conveyance which causes a net decrease in water-surface elevations at the target discharges directly upstream of the UPRR Bridge. However, higher aggradation levels farther upstream would result in 3,000 cfs water-surface elevations that are up to 0.4 feet higher (Station 8542+00) than under baseline conditions approximately halfway between Highway 83 and the UPRR Bridge. Results at the end of the simulation for this alternative indicate the 3,000 cfs flood stage would be slightly increased compared to the predicted stage at the end of the baseline simulation (**Table 3 and Figure 18**).

4. If Alternative #3, which is hydraulically the same as Alternative #6, was modified to include vegetation clearing and minor earthwork to improve conveyance over the south bank bar at the Cody Park restriction (Alternative #3a), both the levels of aggradation and the 3,000 cfs water-surface elevation would be more significantly reduced in this area (i.e., lowered by 0.1 to 0.4 feet between Highway 83 and the just upstream of the UPRR Bridge). Compared to baseline conditions, the net aggradation in the reach between Highway 83 and the UPRR Bridge would increase, but aggradation in the main channel would be reduced due to increased sediment storage along the reactivated overbank areas. The reduction in main channel aggradation lowers the 3,000 cfs water surface elevation at the gage by about 0.1 feet compared to baseline conditions (**Table 3 and Figure 18**).

Based on the model results, none of the evaluated alternatives cause the desired increase in channel carrying capacity from about 1,600 cfs to 3,000 cfs at a stage of 6.0 ft at the gage, or anywhere in the reach downstream of the Highway 83 Bridge. A reduction of approximately 0.8 feet for the water surface elevation at 3,000 cfs would be needed to achieve the objective of keeping 3,000 cfs below the NWS flood stage of 6.0 feet. Among the alternatives, the largest reduction in the 3,000 cfs stage at the Highway 83 gage would be about 0.1 feet under Alternative #3a, and the capacity at flood stage would only increase to about 1,680 cfs (**Table 3 and Figure 18**). Since the evaluated alternatives only include elements located below Highway 83, it is likely that implementing upstream measures that would reduce the sediment supply to the bridge (i.e., reactivation of overbank channels in the reach above the bridge) would be necessary to significantly reduce flood stages at the gage and possibly downstream near the Cody Park restriction. Based on the model results from the evaluated alternatives, reactivating overbank channels could result in increased sediment storage in the overbanks, thereby reducing the sediment supply to and associated aggradation in downstream reaches. It is therefore recommended that an evaluation of additional alternatives that include variations of these measures be carried out to assess the potential benefits on flood stage and carrying capacity. Potential areas for reactivating overbank channels include the left overbank channel between Station 8694+00 through 8845+00 and the right overbank channel between Station 8717+00 through 8760+00. In addition, it is recommended that a sensitivity analysis be carried out to assess the response of the system to hydrologic conditions that are different than those included in the simulations for this study, since each of the subreaches upstream from the Highway 30 Bridge appear to be most aggradational during sustained periods of high flow under baseline conditions (**Figure 19**). Finally, any change to the NWS flood stage that occurs as a result of the current (Spring and Summer, 2011) high magnitude, long duration flows should be evaluated to determine the effects of sustained large flows and to revise the flood carrying capacity goals.

BIBLIOGRAPHY

FLO Engineering, June 10, 1992. "North Platte River Channel Stability Investigation Downstream of Keystone Dam," Prepared for Twin Platte Natural Resources District, North Platte, NE.

HDR and Tetra Tech, 2011. 1-D Hydraulic and Sediment Transport Model. Prepared for Platte River Recovery Implementation Program, January 28, 151 p.

J.F. Sato, August 1, 2005. "Memorandum – Preliminary Findings NE Channel Capacity Study". Prepared for Western EcoSystems Technology, Inc.

Mobile Boundary Hydraulics, 2010. Sedimentation in Stream Networks (HEC-6T), User Manual, March 16.

Parsons Corp, January 2003. "Evaluation of Channel Capacity in the North Platte River at North Platte, NE," Report prepared for Central Nebraska Public Power and Irrigation District.

SEH, September 2009 and May 2010. "Current Conclusions and Recommendations from the April 2009 Short Duration High Flow," and "April 2009 High Flow Event Update Report," submitted to the Platte River Recovery Program.

Tetra Tech, 2010. Hydraulic and Sediment-transport Modeling for the Platte River Sediment Augmentation Feasibility Study, Nebraska. Prepared for The Flatwater Group, September 7, 92 p.

U.S. Army Corps of Engineers, 2010. HEC-RAS, River Analysis System, Users Manual, Version 4.1.0, Hydrologic Engineering Center, Davis, California.

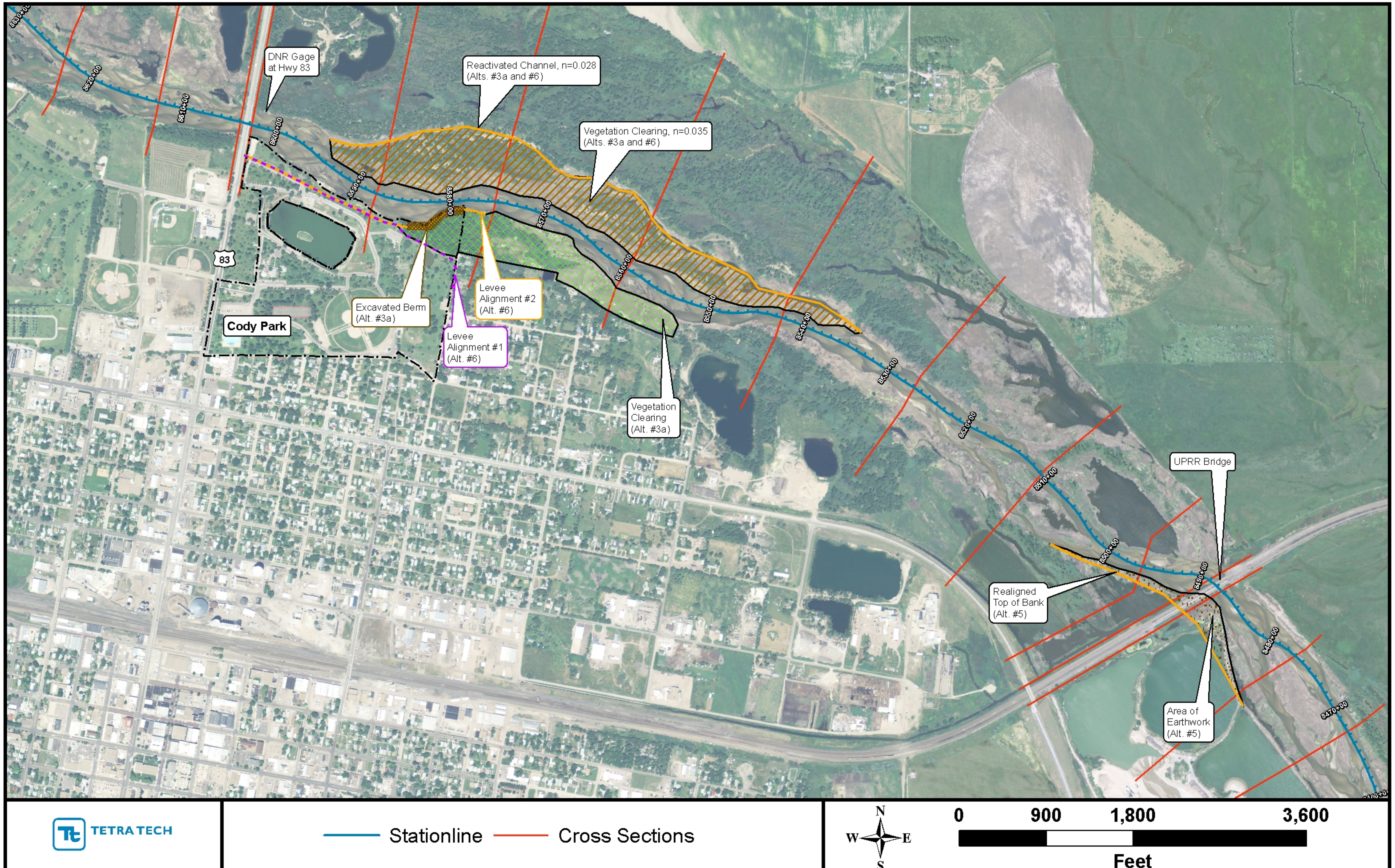

Figure 1. Aerial photograph showing the elements of the 3 alternatives selected for evaluation using the 1-D hydraulic and sediment-transport models.

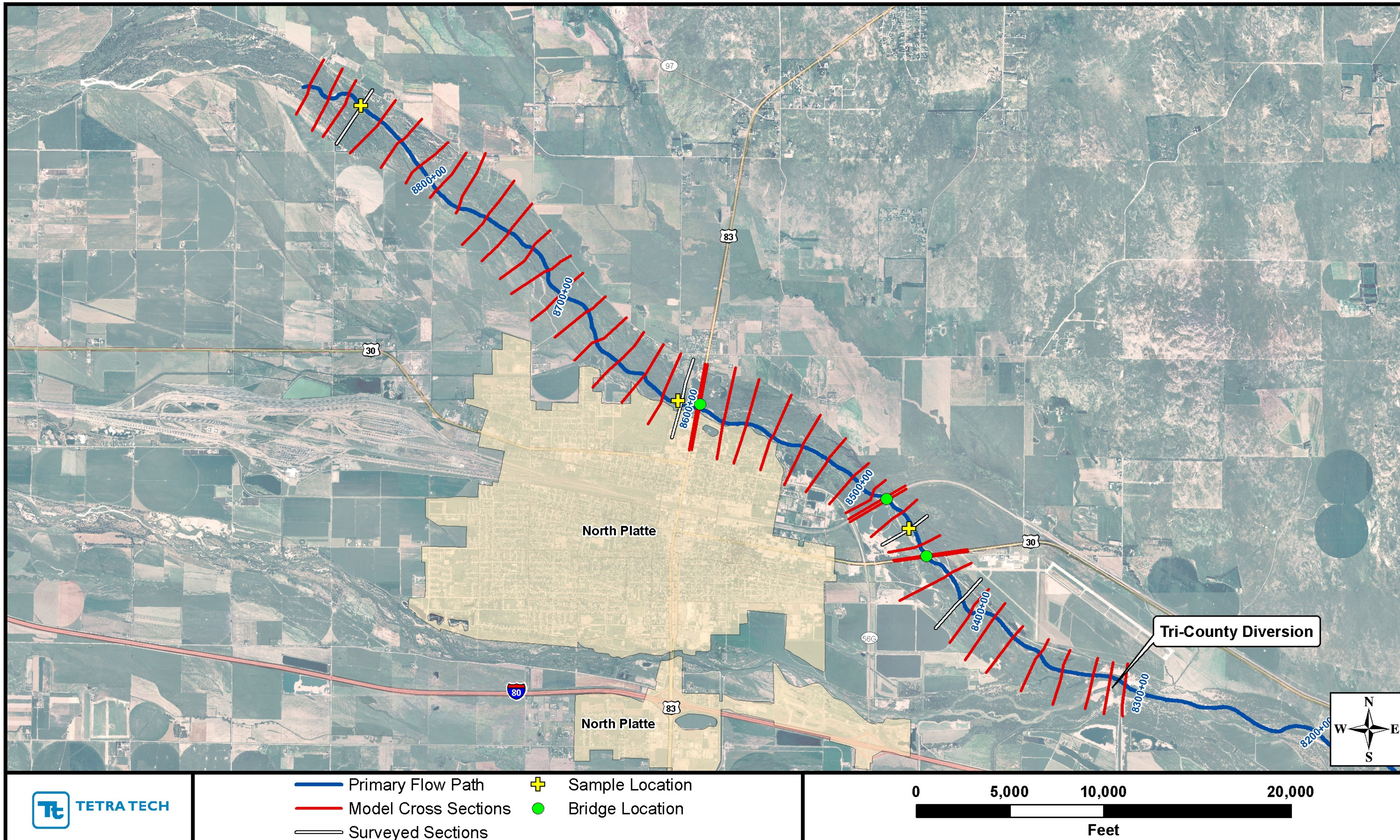

Figure 2. Aerial photograph showing the station line and location of the cross sections in the hydraulic and sediment-transport models. Also shown are the locations of surveyed cross sections and sediment samples used to develop the models.

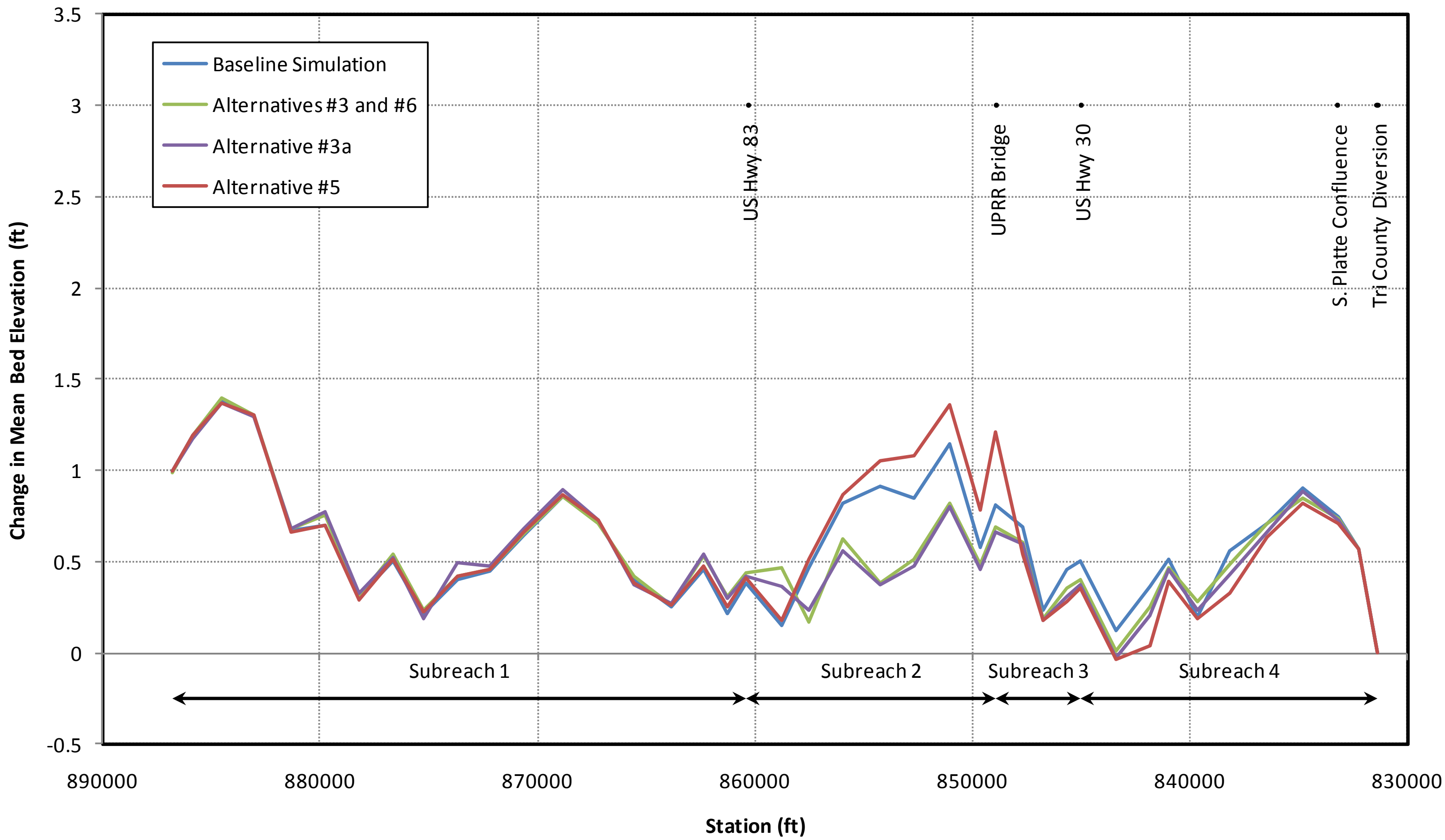

Figure 3a. Comparison of predicted change in mean bed elevation between the end of the warm-up period and the end of the simulation for the Baseline Simulation and the simulations for Alternatives #3, #3a, #5 and #6. Note that because the Cody Park Levee under Alternative #6 was not modeled, the results from Alternative #3 are the same as those under Alternative #6.

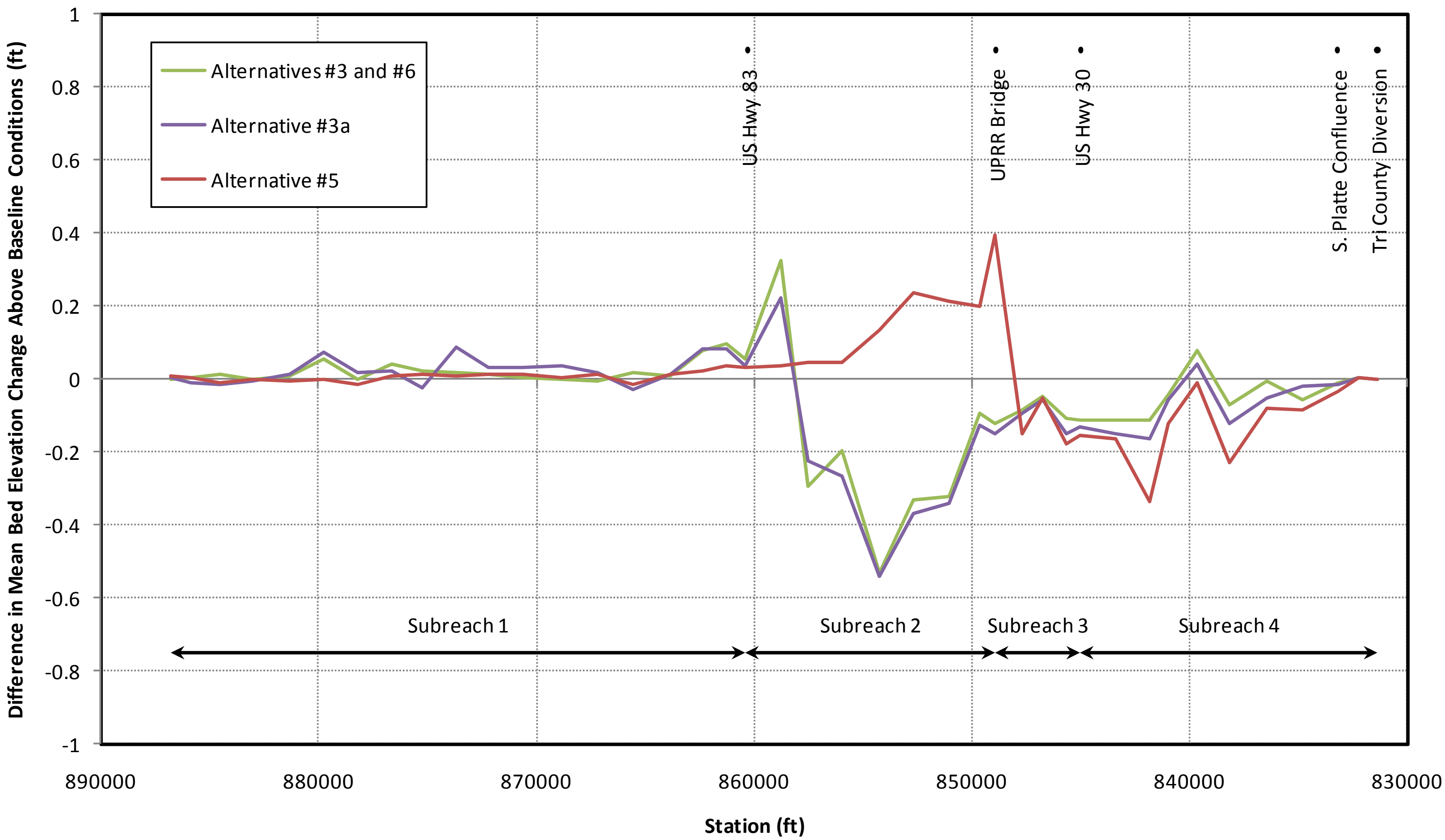

Figure 3b. Difference in predicted mean bed elevation change between the end of the warm-up period and the end of the simulation for the evaluated alternatives compared to the Baseline Simulation. Note that because the Cody Park Levee under Alternative #6 was not modeled, the results from Alternative #3 are the same as those under Alternative #6.

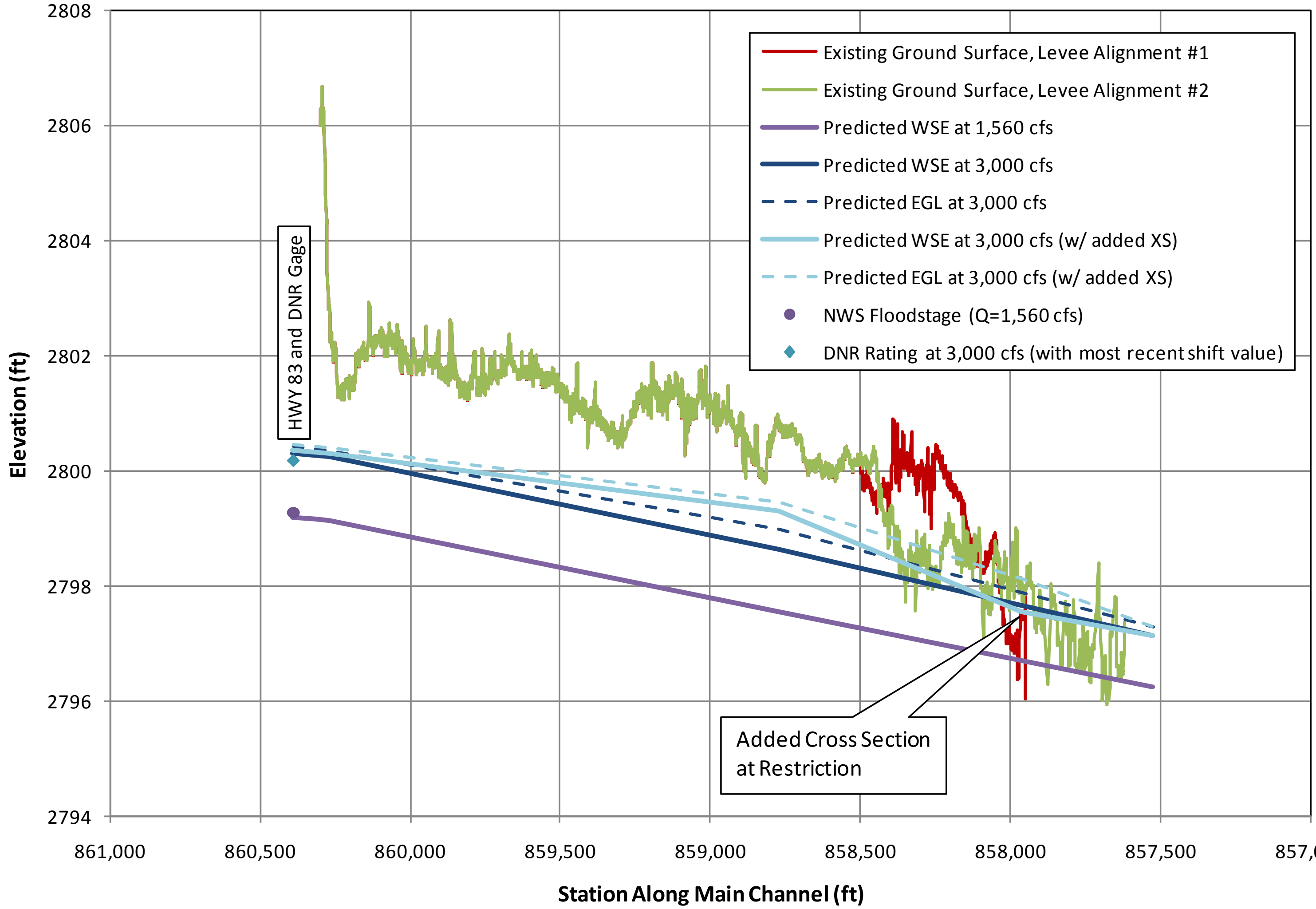

Figure 4. Comparison of the computed water-surface elevation at 1,560 cfs and 3,000 cfs with the existing “bankline” ground elevation along the two proposed levee alignments in the vicinity of Cody Park. Also shown are the water-surface elevations associated with the NWS flood stage of 6.0 feet (Q=1,560 cfs) and the DNR gage rating at 3,000 cfs (with the most recent shift value), and the computed water-surface elevation at 3,000 cfs based on the model with the added cross section at the restriction.

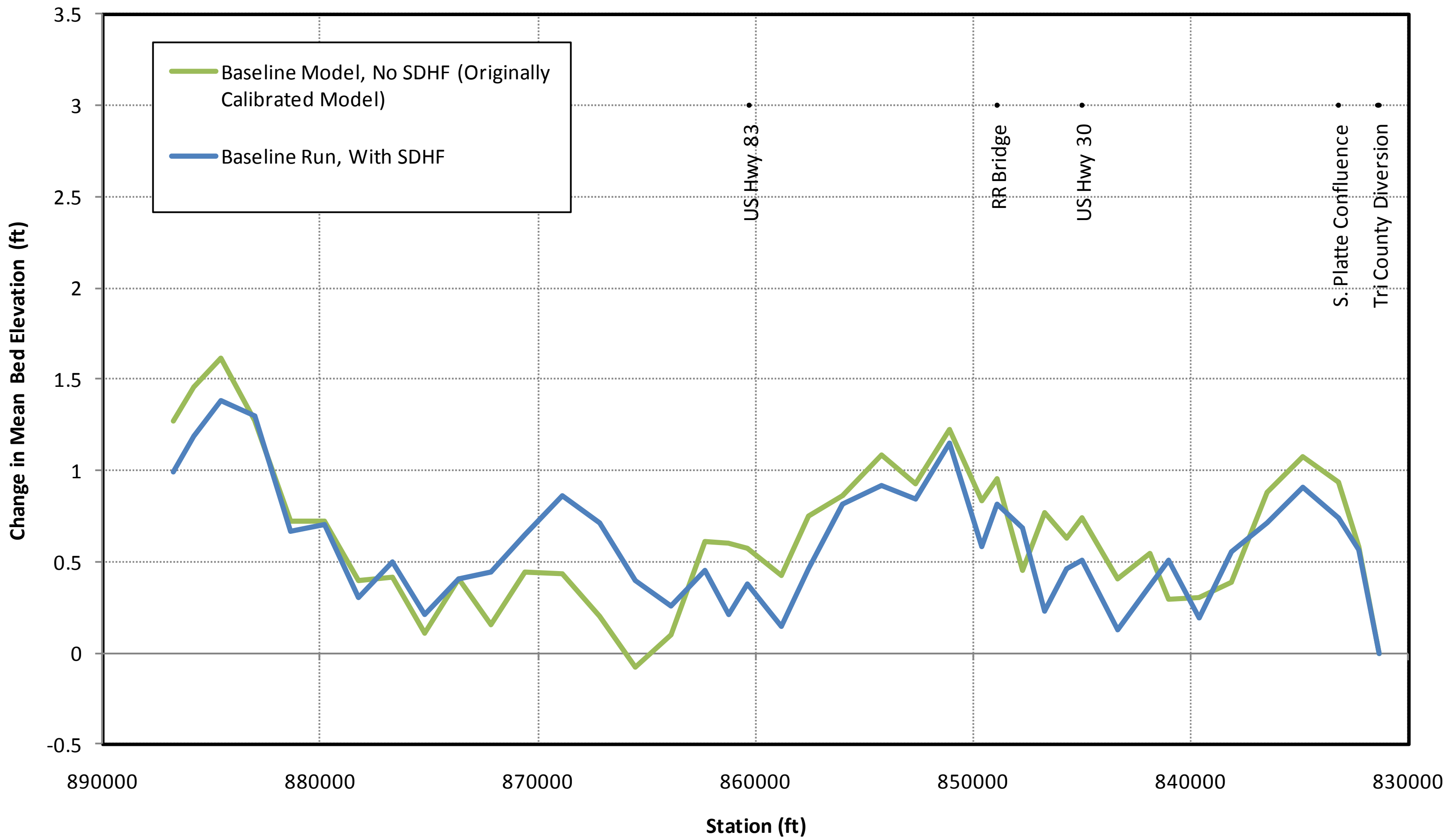

Figure 5. Comparison of predicted change in mean bed elevation from the with- and without SDHF sediment-transport simulations. The without SDHF simulation was the original simulation used to calibrate the model for the 1-D modeling project, and the with-SDHF simulation is the baseline simulation for this study.

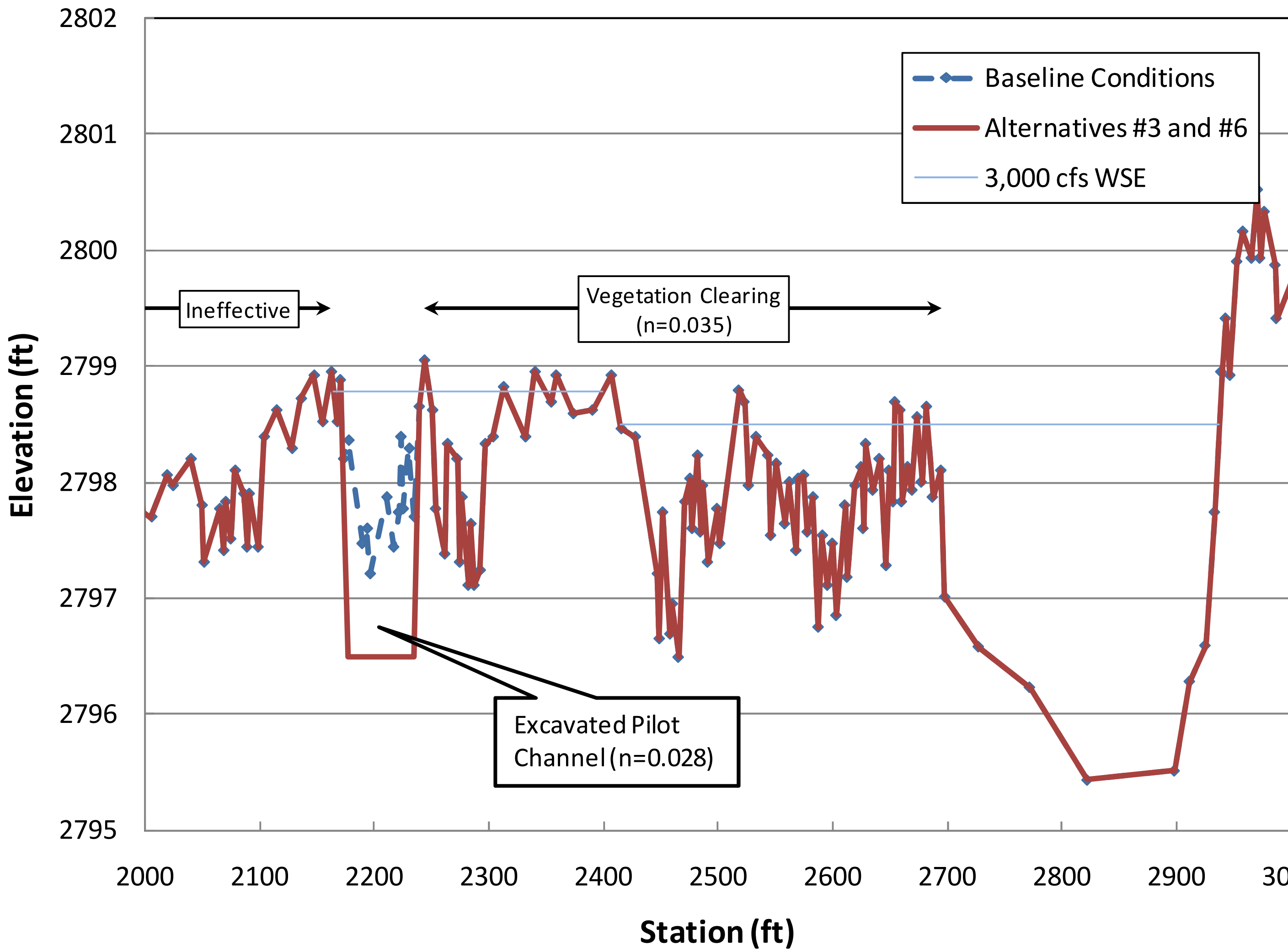

Figure 6. Cross sectional geometry at station 858771 near the entrance to the reactivated channel in the north overbank showing the existing (baseline) geometry and the assumed geometry with the excavated pilot channel under Scenario 3 of Alternative #6. Also shown is the extent of vegetation clearing assumed at this location.

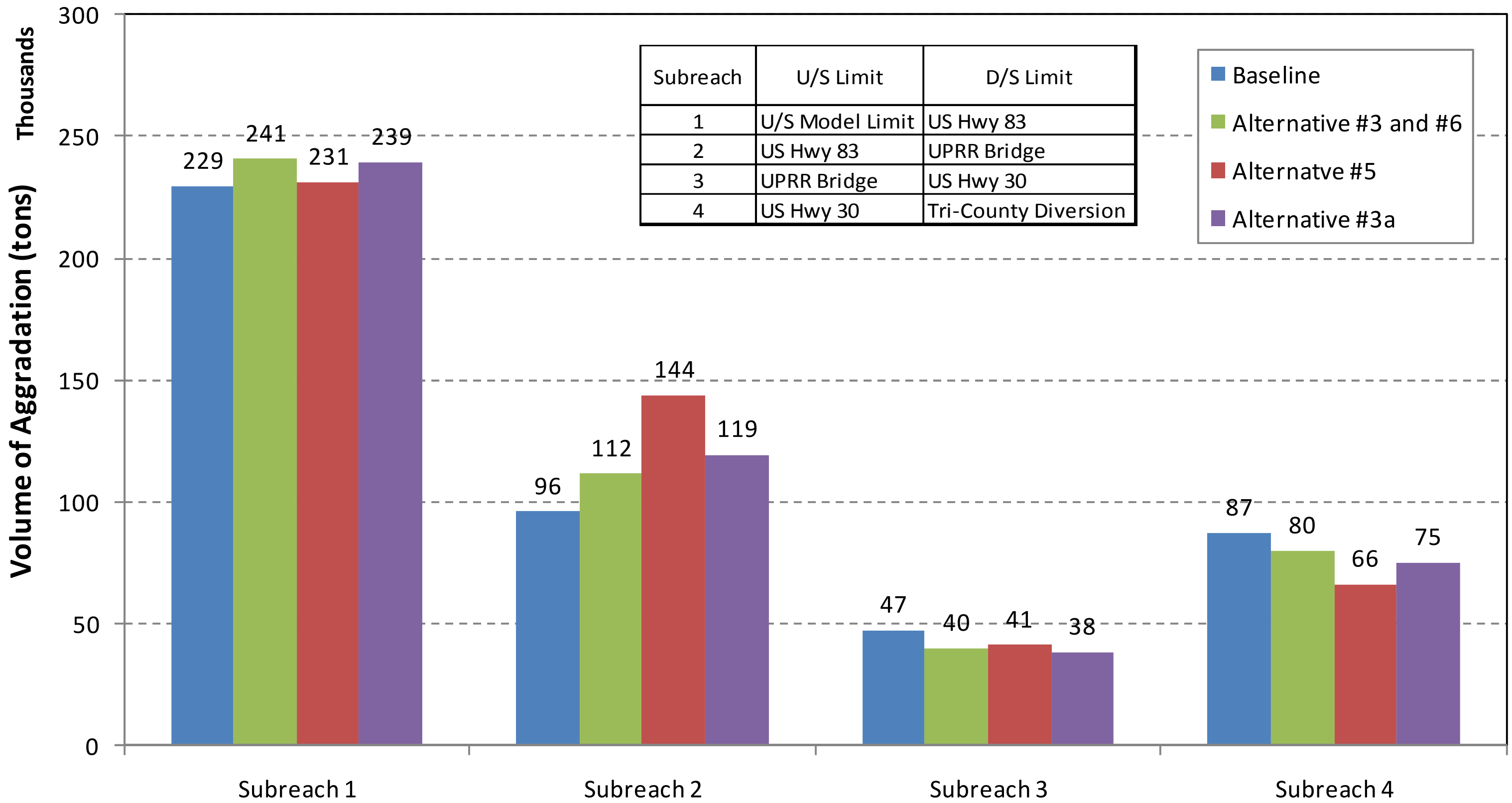

Figure 7. Volume of aggradation predicted in each of the subreaches under baseline conditions and with Alternatives #3, #6, #5, and #3a. Note that because the Cody Park Levee under Alternative #6 was not modeled, the results from Alternative #3 are the same as those under Alternative #6.

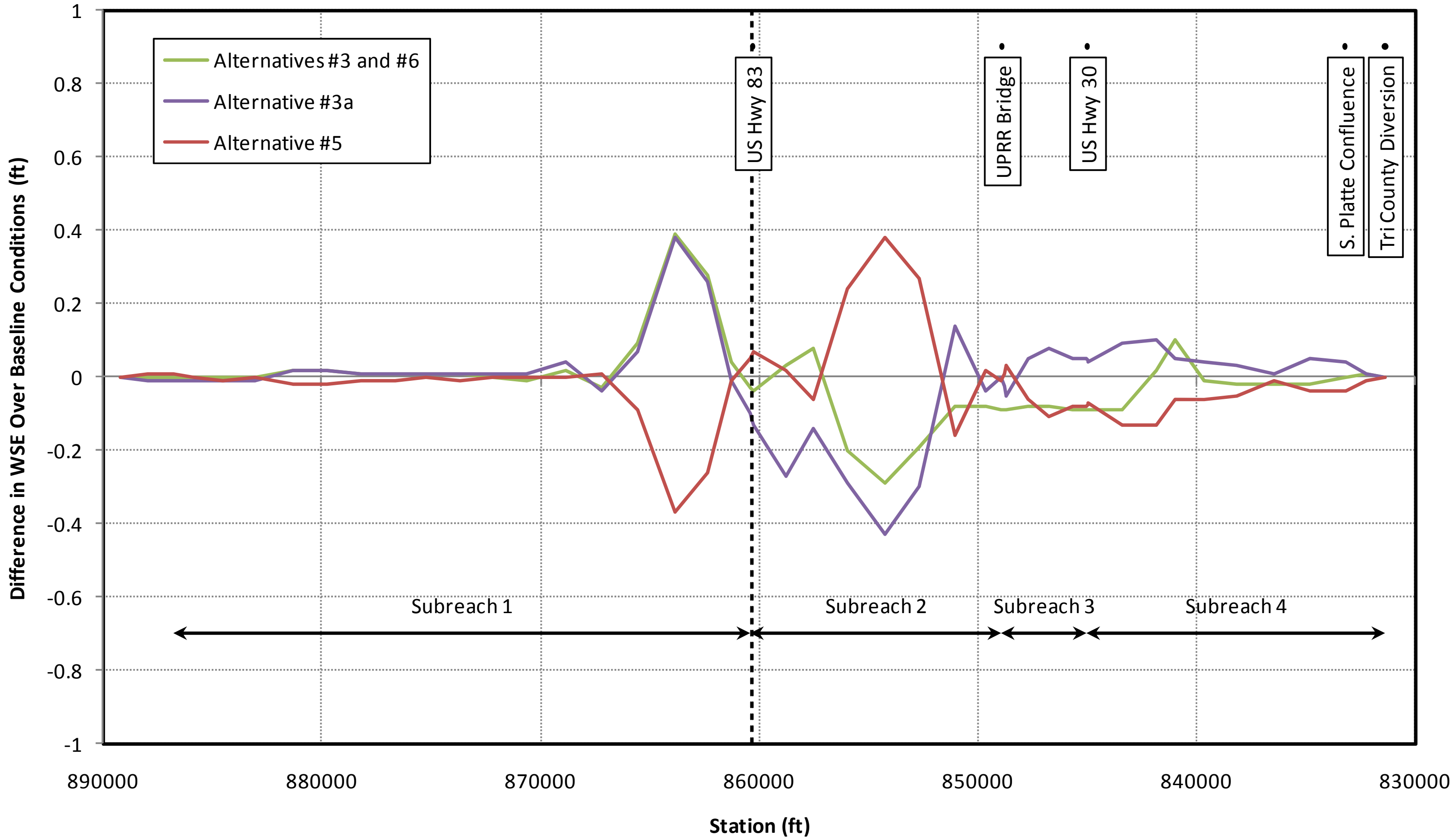

Figure 8. Computed difference in the 3,000 cfs water-surface elevation over baseline conditions, based on the HEC-RAS models with geometries that represent the end of the sediment-transport simulations.

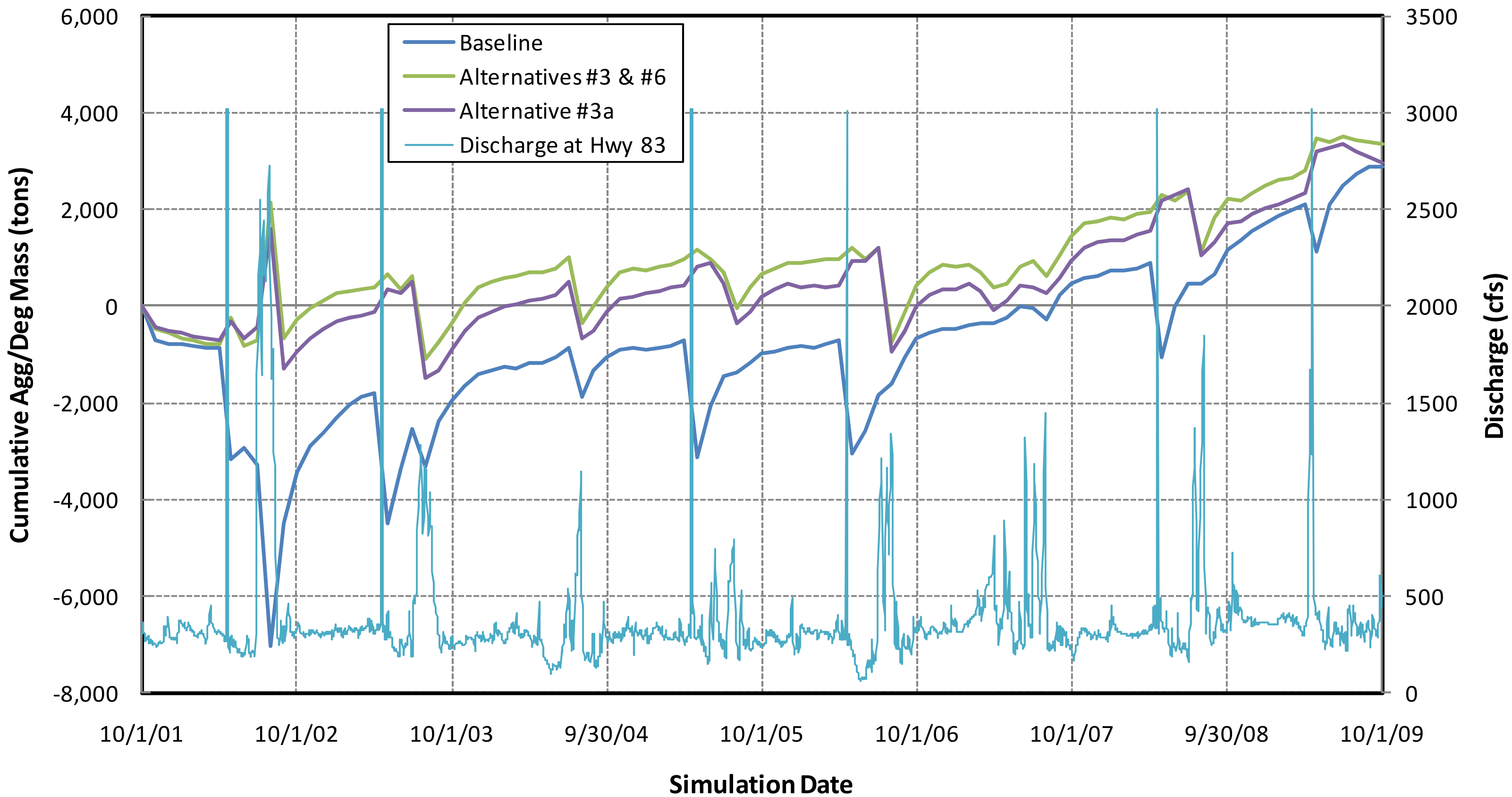

Figure 9. Predicted cumulative mass of aggradation or degradation at Cross Section 858771 located near the upstream limit of the reactivated channel for the 8-year simulation under baseline conditions and Alternatives #3 and #3a. Also shown is the discharge hydrograph at the Highway 83 gage.

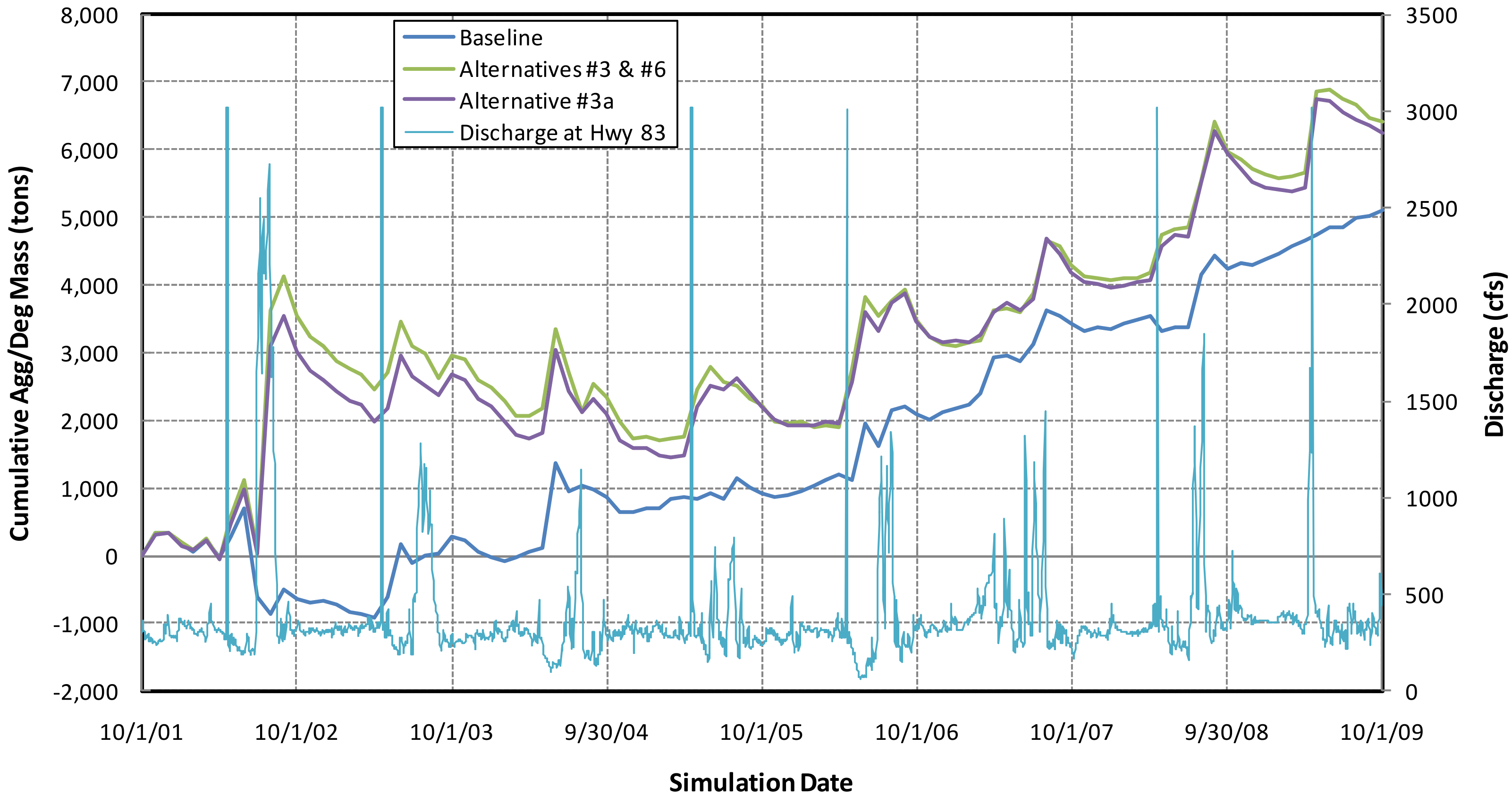

Figure 10. Predicted cumulative mass of aggradation or degradation at Cross Section 861265 located upstream from Highway 83 for the 8-year simulation under baseline conditions and Alternatives #3 and #3a. Also shown is the discharge hydrograph at the Highway 83 gage.

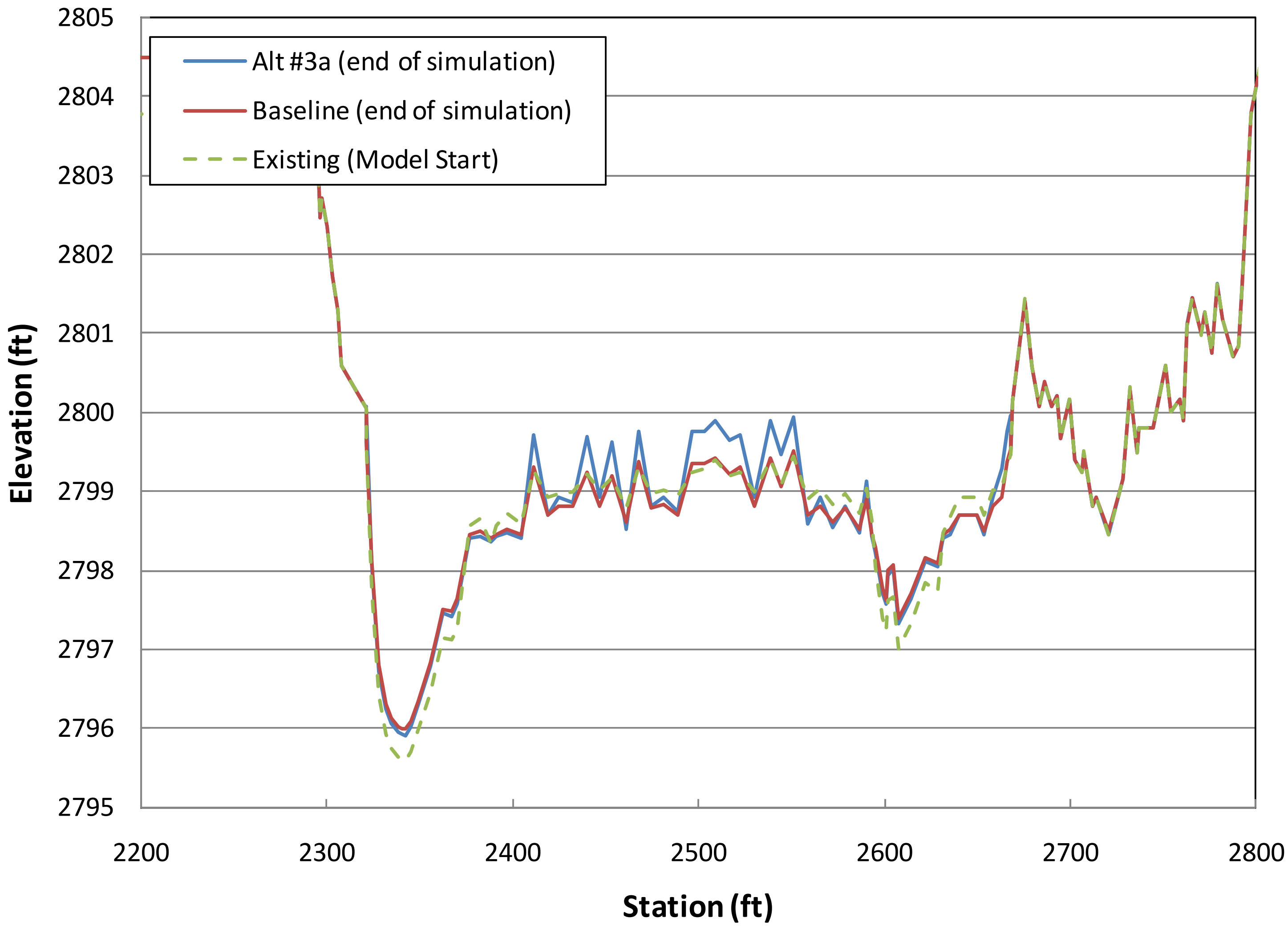

Figure 11. Comparison of predicted cross-sectional geometry at Cross Section 861265 (located upstream from Highway 83) at the end of the simulation for Alternative #3a and baseline conditions, and the geometry at the start of the simulation. It should be noted that this shows the aggradation at one cross-section location, and that aggradation everywhere in the study reach should not be inferred from this.

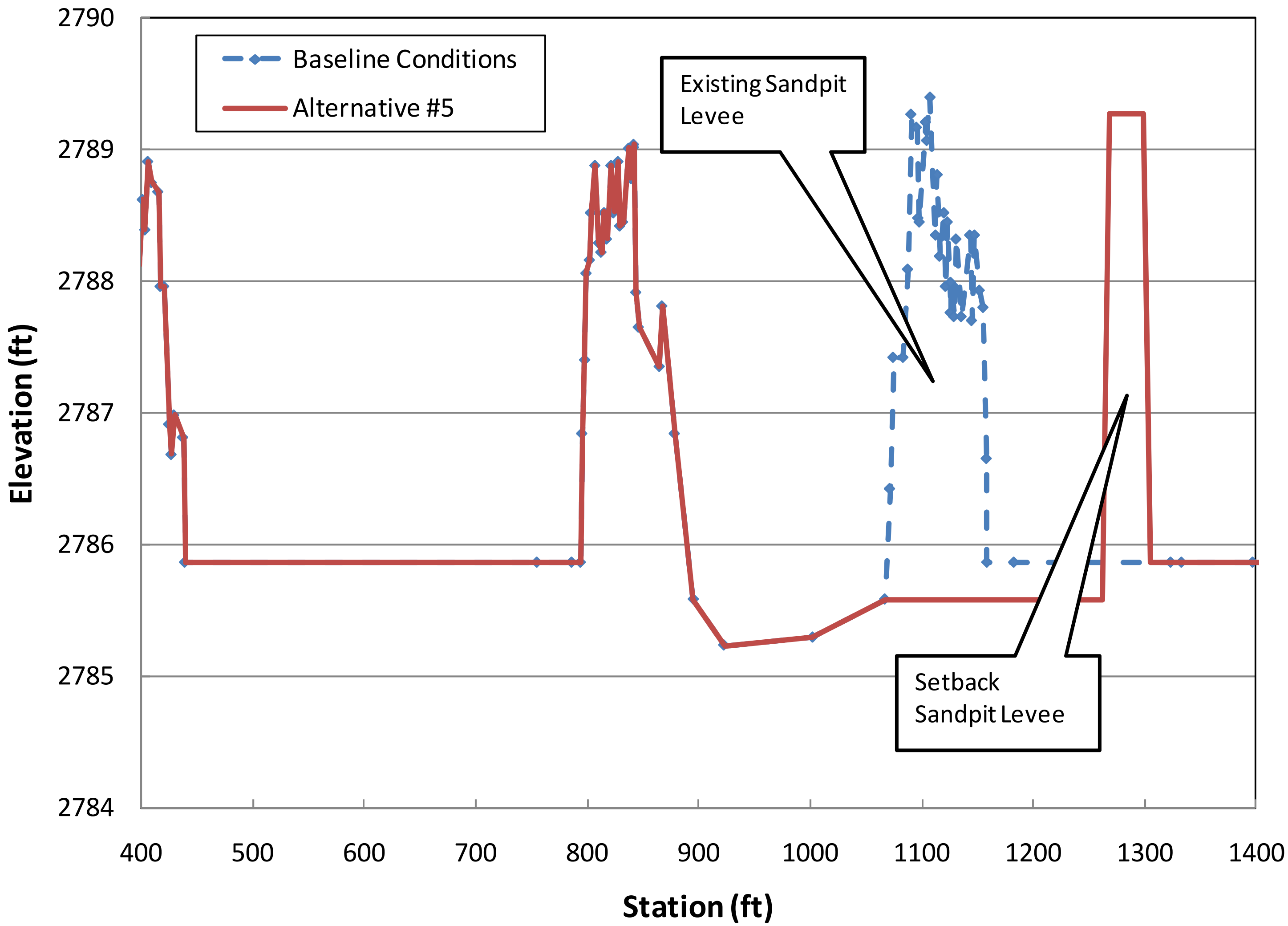

Figure 12. Example of modified cross section (XS 849264) with the setback sandpit levee under Alternative #5.

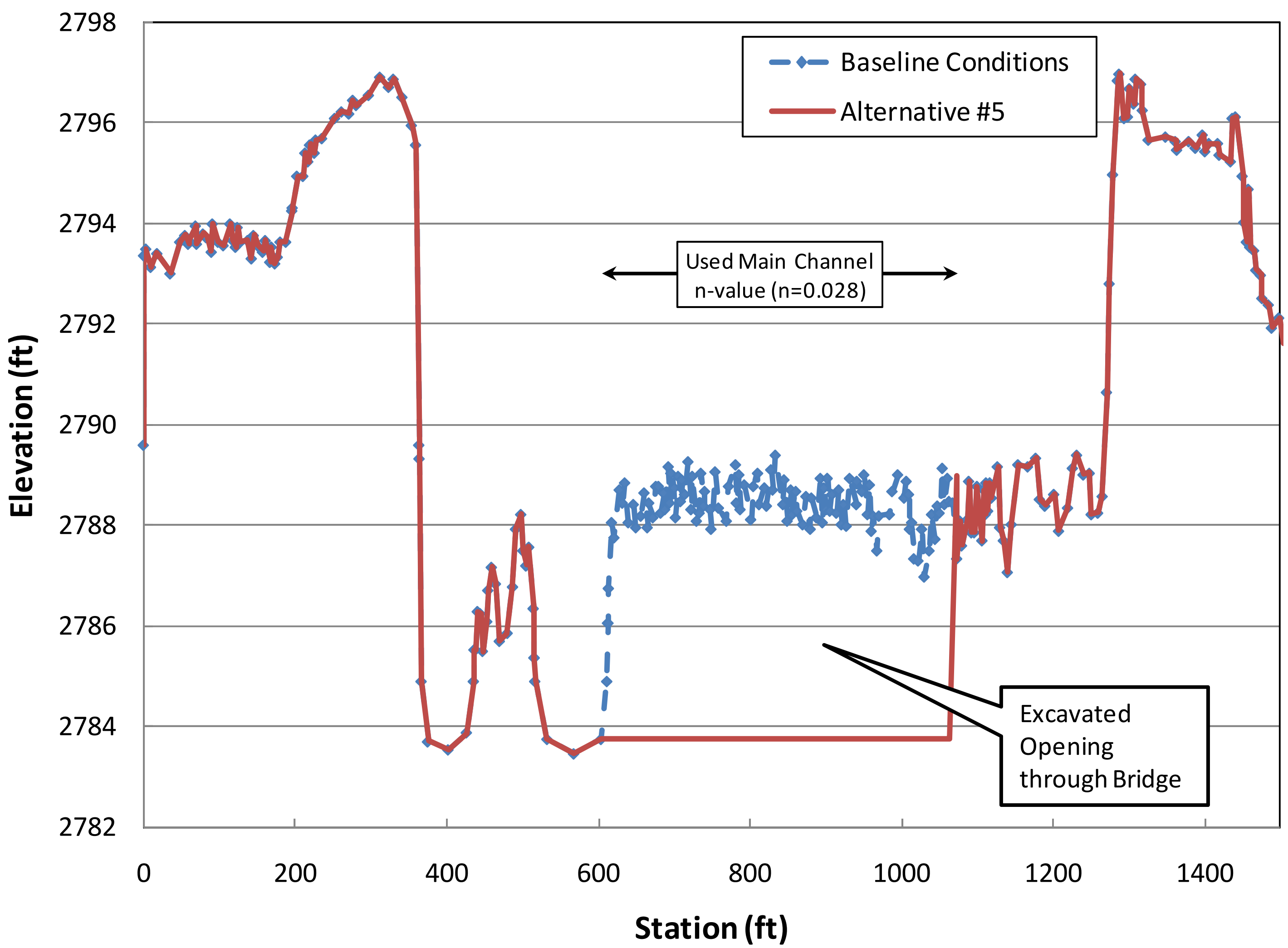

Figure 13. Example of modified cross section (XS 848735) with excavated area to match the proposed alignment for the right top of bank under Alternative #5.

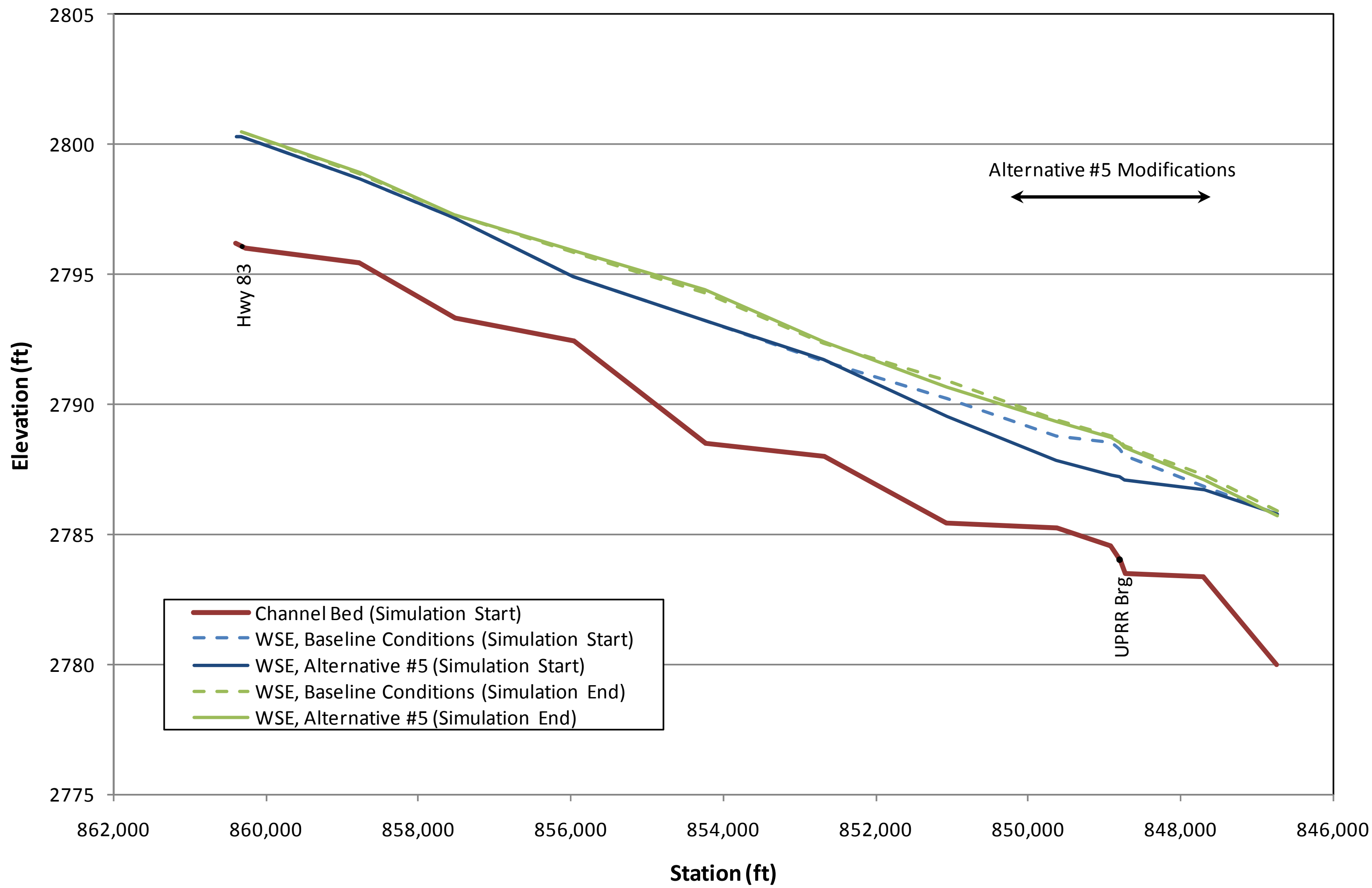

Figure 14. Comparison of predicted water-surface profiles from the hydraulic models for baseline conditions and Alternative #5 at a discharge of 3,000 cfs.

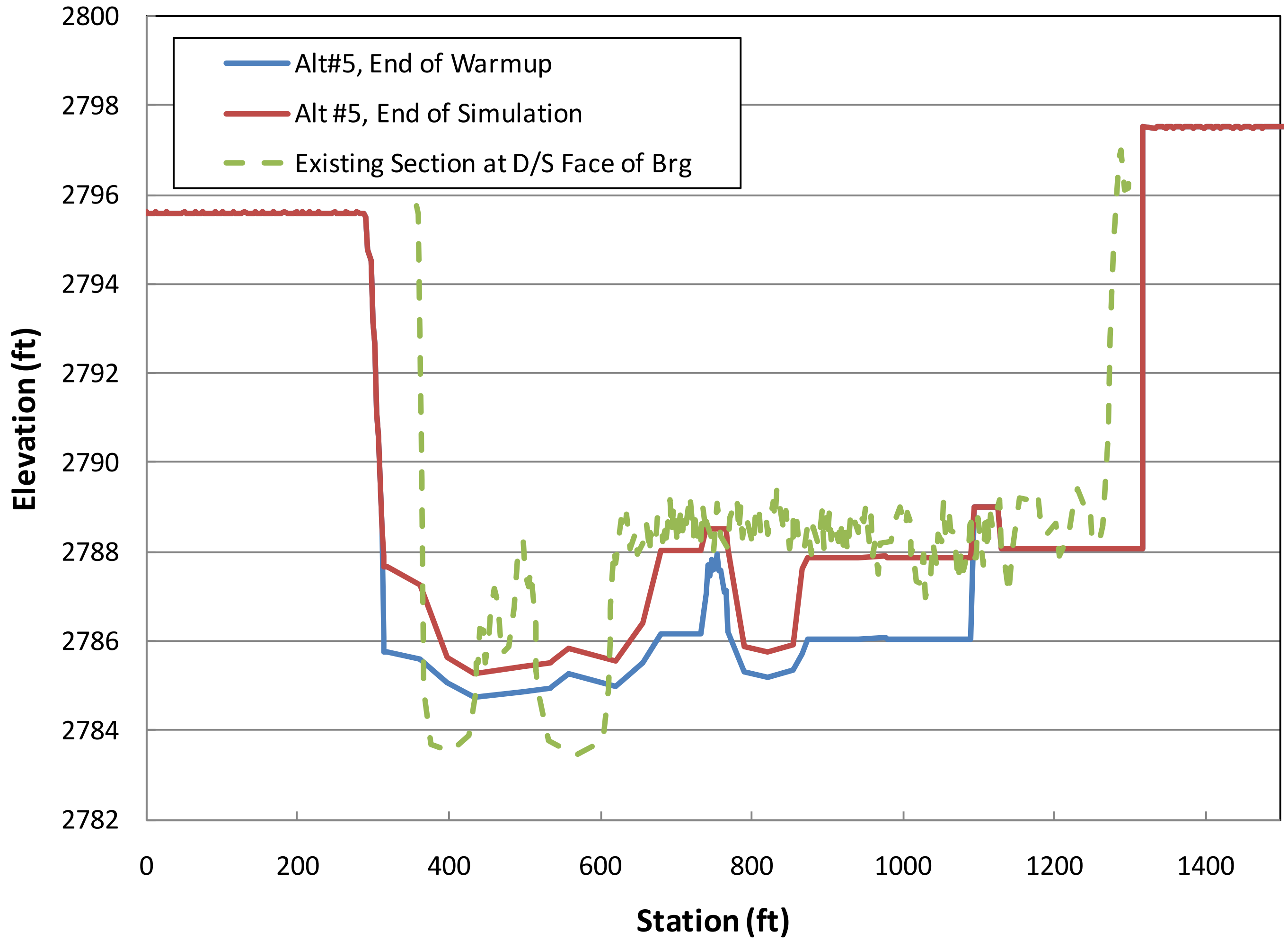

Figure 15. Comparison of the predicted geometry at the upstream face of the UPRR Bridge (XS 848912) from the Alternative #5 simulation at the end of warm-up period and at the end of the simulation. Also shown is the existing geometry at the downstream face of the bridge, showing aggradation trends that are similar to that predicted by the model.

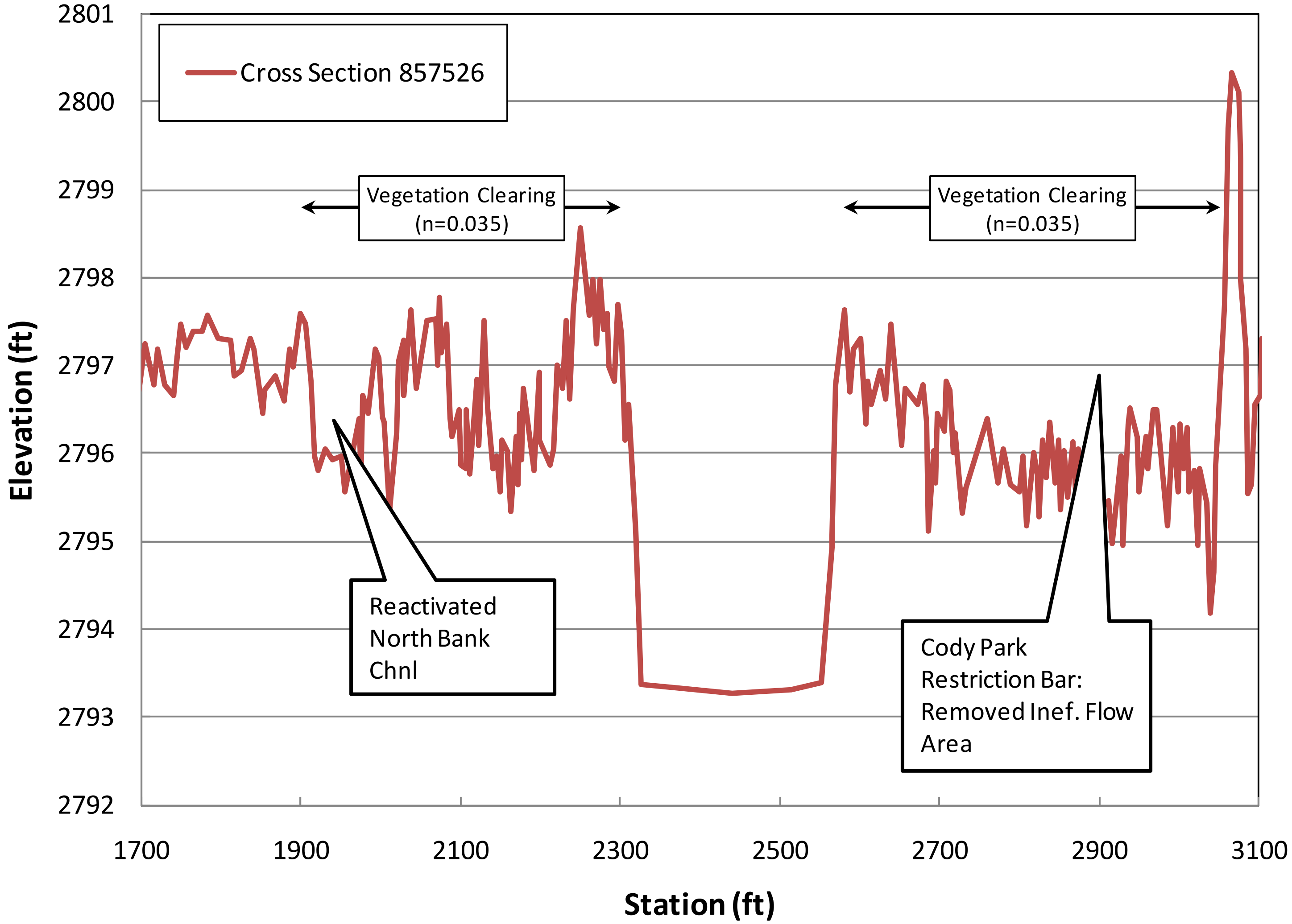

Figure 16. Cross Section 857526 showing the location of the reactivated north bank channel and extent of vegetation clearing in the left overbank and the re-opened conveyance area and extents of vegetation clearing along the bar at the Cody Park Restriction under Alternative #3a.

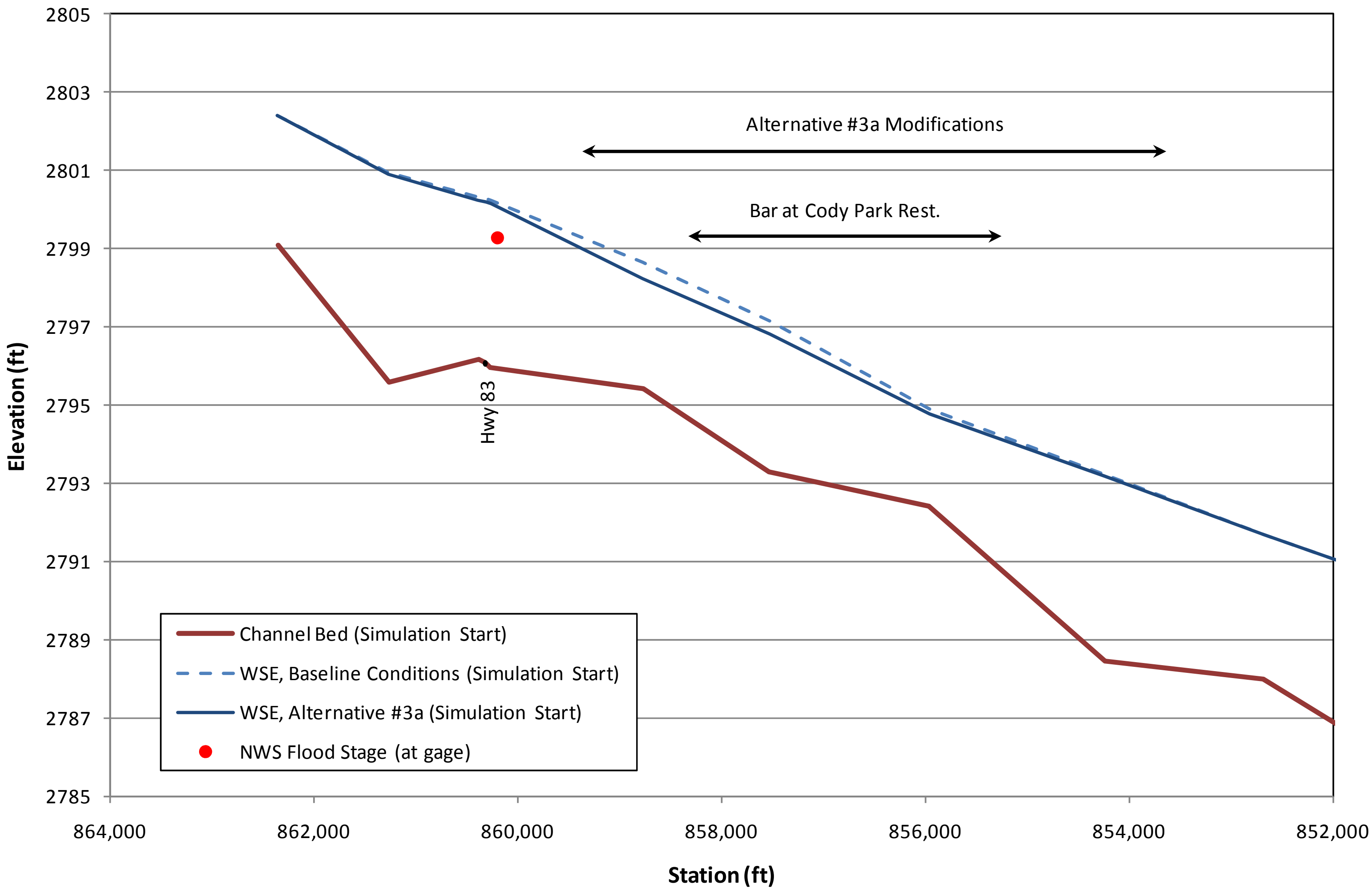

Figure 17. Comparison of water-surface profiles at 3,000 cfs under Alternative #3a and baseline conditions.

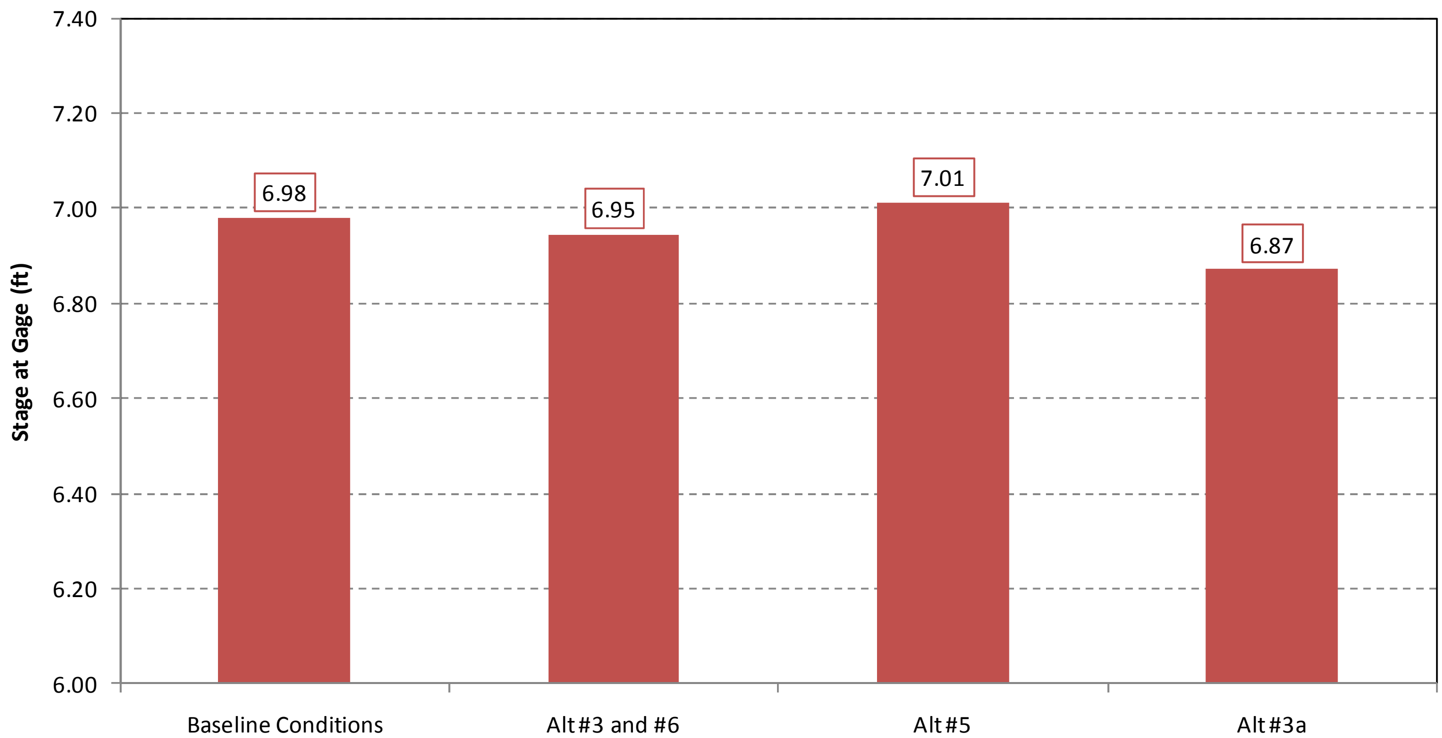

Figure 18. Comparison of the predicted 3,000 cfs flood stage at the end of the sediment-transport simulations for baseline conditions and under Alternatives #3, #3a, #5 and #6.

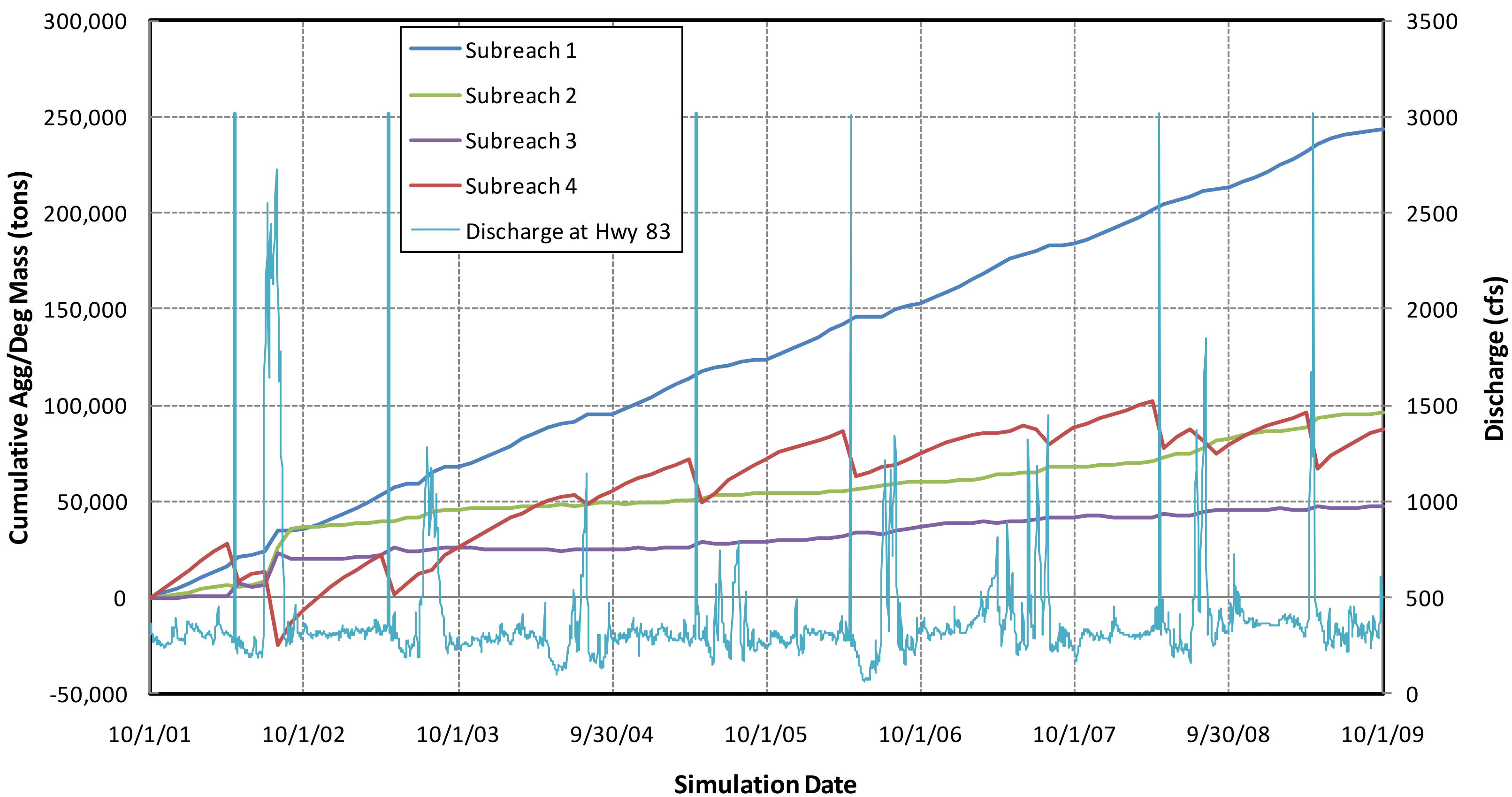

Figure 19. Predicted cumulative volume of aggradation or degradation in each of the four subreaches for baseline conditions.