1-D Hydraulic Model

Draft Hydraulic Modeling Technical Memorandum

Prepared for

Platte River Recovery Implementation Program

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Table of Contents

List of Tables

List of Figures

List of Appendices

Platte River Recovery Implementation Program
1-D Hydraulic Modeling Technical Report

Keystone Diversion to North Platte (River Mile 370 – River Mile 310)
August 31st, 2011

1. Introduction and Background

Project Introduction

The Platte River Recovery Implementation Program (Program) was initiated on January 1, 2007 between Nebraska, Wyoming, and Colorado and the Department of the Interior to address endangered species issues in the central and lower Platte River basin. The species considered in the Program, referred to as "target species", are the whooping crane, piping plover, interior least tern, and pallid sturgeon. The Program would like to investigate physical processes within the Platte River System and how altering flow and sediment load might impact these processes. A hydraulic model of the North Platte River provides Program participants a tool to evaluate the relationship between streamflow and habitat of Program target bird species.

Statement of Work

The purpose of this project is to develop and calibrate a one-dimensional (1D) hydraulic model that will be used to estimate the attenuation of North Platte River flows, in particular, short duration high flows (SDHFs). SDHFs are one component of the Flow-Sediment-Mechanical (FSM) management strategy being considered by the Program to improve habitat for the Program target species. SDHFs would consist of Program flow releases to achieve flows of 5,000 to 8,000 cfs for 3 days in 2 out of 3 years at Overton. To aide this, a 1D steady state and unsteady model was developed from the Keystone Diversion to the Tri-County Diversion (North Platte).

A 1D steady-state hydraulic model was previously developed for the 10 mile reach of the North Platte River from the Tri-County Diversion upstream to about 5.5 miles upstream of the Highway 83 Bridge in North Platte (HDR et al. 2011). That model was not modified for the scope of work for this effort. A new steady-state model for the area upstream of North Platte to the Keystone Diversion was constructed for this scope of work, and was then combined with the existing steady-state model in the downstream reach near North Platte. A new 1D unsteady hydraulic model was constructed for this scope of work for the entire reach from the Keystone Diversion to the Tri-County Diversion, as there was no existing unsteady model in the downstream reach.

Site Description

Model Extents

The headwaters of the North Platte River originate in Colorado. The North Platte flows through Colorado, Wyoming and Nebraska. Lake McConaughy, an impounded lake on the North Platte River, provides storage and is used to monitor and control releases downstream through the Keystone Diversion. Due to the consistent measured flow record at this location, the Keystone Diversion was used as the upstream boundary condition. The confluence of the North Platte and South Platte Rivers is located near North Platte, Nebraska. The confluence occurs immediately upstream from the Tri-County Diversion. The Tri-County Diversion was constructed by Central Nebraska Public Power and Irrigation District (CNPPID) to allow for diversion of Platte River water into their canal system. CNPPID maintains stage, flow diversion, and flow bypass records at the Tri-County Diversion. In a previous modeling effort (HDR et al. 2011), a 1D steady state and unsteady hydraulic model was developed from the Tri-County

Diversion to Chapman. The Tri-County Diversion served as the upstream boundary condition for this model. Therefore, due to long term record and consistent point of reference between models, the Tri-County Diversion, located approximately at River Mile (RM) 310 was designated as the downstream extent of the model. As part of the model development effort from Tri-County Diversion to Chapman, a separate model of the North Platte River "Choke Point" reach, located in the vicinity of the City of North Platte, was also developed. The Choke Point model included an approximately 10-mile reach of the North Platte River from the Tri-County Diversion extending to about 5.5 miles upstream of the U.S. Highway 83 Bridge in North Platte. The geometry from the "Choke Point" model was incorporated in the current model (North Platte River model).

River System Characteristics

In the period from 1900 through 1938, the North Platte River channel maintained a predominantly braided form, although the width of the river decreased significantly. Braided river forms are characterized by a series of shallow, interconnected low flow channels within the overall channel. This form provides desirable riverine habitat (i.e., habitat occurring along a river) for whooping crane, interior least tern, and piping plover because there are wide areas of water with unobstructed sight distances and bare sandbars for roosting, nesting, and security from predators (BOR and USFWS, 2006).

Over time, reductions in flow volumes, peak flows, and sediment supply have shifted the river's form from a wide, braided channel to a channel consisting of multiple narrow and deep channels separated by vegetated islands (anastomosed). These changes have led to a decrease in desirable habitat for the target species (BOR, 2006).

Hydrologic Inputs and Outputs

In addition to numerous channel splits, there are several tributaries as well as irrigation and power canal diversions located within the modeled reach. Since these gains and losses can impact hydraulics, they were considered in the modeling effort. The location of these canals and diversions are summarized in Table 1.1, and are shown in Appendix A.

Table 1.1: Summary of Canals and Tributaries, RM 310 to RM 156.

Canal or Tributary Name	Approximate River Mile	Gage Number
Keith-Lincoln Canal Diversion	361	NDNR 76000
North Platte Canal Diversion	340	NDNR 114000
Paxton-Hershey Canal Diversion	340	NDNR 121000
Birdwood Creek	334	NDNR 6692000
Suburban Canal Diversion	333	NDNR 136000
Cody-Dillion Canal Diversion	324	NDNR 27000
Lincoln Co. Drain No. 1	317	NDNR 6692500

In addition to tributaries and canals, long-term reach gain/loss (RGL) affects streamflow. Long-term average monthly gains and losses were computed for gaged reaches between the Keystone Diversion, Sutherland, and North Platte gages, based on mean daily flow data from water years 1985-2008. These calculations are summarized in Table 1.2. In this table, positive values indicate a historical long-term gain to surface flows, and negative values would indicate a loss from surface flows.

Table 1.2: Summary of Average Monthly Long-Term Reach Gains/Losses, WY 1985-2008

	Reach Gain/Loss (cfs)	
	Keystone to Sutherland	Sutherland to North Platte
January	86	134
February	101	136
March	73	172
April	47	187
May	132	103
June	85	110
July	32	58
August	72	134
September	130	117
October	50	176
November	79	141
December	73	135

Significant Landmarks and Structures in Modeled Reach

As stated previously, the Keystone Diversion (located at RM 370) was designated as an appropriate upstream boundary for the North Platte Model. Downstream of Keystone, the Platte River typically has a main channel with occasional secondary channels and channel splits. The first canal diversion is the Keith-Lincoln Diversion, located approximately 9 miles downstream of the Keystone Diversion. The next two diversions, the North Platte Canal and Paxton-Hershey Canal diversions are located at approximate River Mile 340. These structures consist of a low head structure across the river, and a gated structure to regulate flow into the canal. Less than 3 miles downstream of these two diversions, the North Platte River at Sutherland gage is located on the Prairie Trace road bridge (RM 337). Three miles downstream of the Sutherland Gage, Birdwood Creek flows into the North Platte River. There is a gage on Birdwood Creek which monitors return flows. At the confluence, the river bifurcates. A control structure referred to as the North-side Suburban Canal diversion structure (located on the north channel split, just downstream of the channel bifurcation) is used to direct flow into the south channel. The Suburban Canal Diversion diverts water from the south channel approximately one mile downstream of the North-side Suburban Canal diversion structure. The last gaged diversion in the modeled reach, the Cody-Dillion Canal diversion, is located at River Mile 324. Aerial photographs indicate that there is no inline structure associated with the Cody-Dillion Canal diversion. Lincoln County Drain No. 1, a bank tributary near river mile 317, also has a gage that monitors return flows. The North Platte River at North Platte streamgage is located at the highway 83 bridge, at approximate River Mile 316. The Tri-County Diversion structure (RM 310) is the downstream extent of this model reach. See Appendix A, Sheets 1-10.

2. Steady State Hydraulic Model

Model Development

Stationline and Secondary Channels

The main channel stationline was delineated for the approximately 60-mile continuous path of the river was based on aerial photography, LiDAR topography and field observations. Nine hydraulically distinct, secondary flow paths ranging in length from about 0.5 to 4 miles that transport a significant amount of flow at higher discharges were also identified. The locations of several significant tributaries that primarily represent diversion return channels were also identified as point source inflows along the stationline. The entire model between the Keystone Diversion and the Tri-County Diversion, including the secondary flow paths, represents approximately 81 miles of river channel.

Cross-Section Alignment, Spacing, and Orientation

Model cross sections were laid out to extend across the active channel and floodplain and remain perpendicular to the direction of flow. The cross sections were spaced at 1,200- to 1,500-foot intervals, based on target criteria of three to four channel widths for the approximately 400-foot-wide main channel. Additional cross sections were placed at hydraulic structures (e.g., bridges and diversions) near reach junctions and in areas where ground survey data were available. In total, the model from the Keystone Diversion to the Tri-County Diversion contains 415 cross sections to characterize the main and secondary channels.

Topography

Topography for the model was developed primarily from LiDAR data provided by the Program. LiDAR data covering the entire project reach were collected March 19, 2009, and referenced the North American Datum of 1983 (NAD83) and North American Vertical Datum of 1988 (NAVD88). Horizontal accuracy was reported to be 1.88 feet, with a vertical accuracy of 0.28 feet. A limited amount of bathymetric survey data were also available and incorporated into the model in the region of the "Choke Point" reach. These data were collected between 1998 and 2009, and included the Program's Anchor Point and longitudinal profile surveys that were conducted for the Geomorphology Monitoring Program in 2009, surveys specifically for this project by The Flatwater Group (TFG), and surveys by the U.S. Geological Survey (USGS) and Bureau of Reclamation (BOR). Topographic data in each cross section required filtering to achieve the HEC-RAS cross section-point limit of 500. This was accomplished using a feature within RAS that removes points based on minimizing the change in area. A typical before-and-after cross section is shown in Figures 2.2 and 2.3). In the Choke-Point model reach, ground survey data was incorporated in certain locations to improve sub-aqueous channel representation within the model. No additional topographic data were collected for the work completed for this modeling effort.

Hydraulic Structures

The model includes 10 bridge structures. Of these structures, as-built bridge plans for seven were available and obtained from the Nebraska Department of Roads (NDOR) (Table 2.1). As-built plan sets for two of the bridges were obtained from Union Pacific Railroad (UPRR). However, as-built plan sets were not available for the N Sand Road Bridge. This bridge is located just upstream of the Suburban Canal Diversion and therefore the bridge is not expected to be a local hydraulic control. For this reason, bridge geometry was approximated based on aerial photography and LiDAR data. Expansion and contraction coefficients in the bounding cross sections to each bridge were set to 0.5 and 0.3, respectively. As-built plan sets for the diversion structures were not available from the canal owners. Information on the dimensions of the Keith-Lincoln Diversion structure (RM 261) was obtained from conversation with Patrick Thomas, the manager and secretary of the Keith-Lincoln County Irrigation District. Dimensions for the Paxton-Hershey, North Platte, North-side Suburban, and Suburban Diversion structures were obtained in the field by Platte Valley Irrigation District Manager, Secretary/Treasurer, and Ditch Rider Martin Fischer. Elevations of these structures were approximated based on photographs, field observations, and LiDAR data. All of these structures were incorporated as gated inline structures with a low head control section in the geometry. See Table 2.2 for a summary of inline structures.

Table 2.1: Bridges Included in Steady State Model

Bridge	Model Geometry Data Source	Reach	Station
Keystone-Roscoe Rd	NDOR	RM 370-358	1135192
UP RR Bridge, S Morrill Subdivision (North Platte Division)	UPRR	RM 356-355	1072625
Road East TN	NDOR	RM 354-352	1059039
Road East VN	NDOR	RM 350	1039643
N Prairie Trace Rd.	NDOR	RM 341-337	973416
N Sand Road	N/A	RM 334-331	948956
N Hershey Road	NDOR	RM 331-310	937892
Hwy 83	NDOR	RM 331-310	860316
RR Bridge, North Platte	UPRR	RM 331-310	848800
Hwy 30	NDOR	RM 331-310	844958

Table 2.2: Diversion Structures in Hydraulic Model

Structure	Reach	Station
Keith-Lincoln Diversion	RM 370-358	1099531
North Platte Diversion	RM 341-337	989431
Paxton-Hershey Diversion	RM 341-337	987431
Suburban North-side Diversion	RM 334-331 Split*	906831
Suburban Diversion	RM 334-331	948831

*Split indicates a bifurcated channel. In this reach (RM 334-331), two distinct channels are represented as river reaches in the model. To differentiate, one channel is named "RM 334-331" and the other is named "RM 334-331"

Hydraulic Roughness

Hydraulic roughness was incorporated into the model using Manning's roughness coefficients that vary horizontally along the cross section. Vegetation and land-use information from the Program's Vegetation Monitoring Program was used to develop polygons that represent different roughness zones. Roughness was based on 2005 land use/cover data, which was digitized by the USFWS in 2009. This was the same coverage used to develop the Tri-County to Chapman model (HDR et al., 2011). The original land-use polygons in this area provided an adequate density and required no adjustment to the density of the data. A section of the study area near Lake McConaughy (see Figure 2.1) required land use classification to be generated to fill a void in the land use/cover data. Since existing land cover data was created using 2005 data, 2005 CIR imagery was used as the basis for land use classification. The area was digitized by hand from the imagery and incorporated into the existing land use/cover shapefile (see Figure 2.1). HEC-GeoRAS was then used to determine the stationing of the roughness zones along the cross section (USACE, 2009). These roughness zones were then assigned a Manning's roughness coefficient based on the vegetation description, field observations, bed-material characteristics, past experience with similar streams, and published values for similar streams (Barnes, 1967; Hicks and Mason, 1991; Arcement and Schneider, 1989) (Table 2.3). Roughness values in the overbanks ranged from 0.020 for flat surfaces with no vegetation to 0.12 for densely vegetated areas with irregular topography. Roughness values for the vegetated area within the channel were then assigned by evaluating the aerial photography, topography and vegetation-type polygons. Main channel roughness coefficients ranged from 0.028 for the active, non-vegetated portion of the channel to 0.10 for densely vegetated mid-channel bars and islands. Because the resulting composite roughness values in the channel vary with depth, and because the assigned roughness values appear to result in good model calibration (discussed below), vertical variation in roughness was not used.

Table 2.3: HEC-RAS Model Roughness Values

Vegetation Type/Land Use	Manning's Roughness Coefficient
Agricultural	0.035
Bare ground/Sparse Veg	0.03
Canal/Drainage	0.02
Irrigation Reuse Pit	0.3
Mesic Wet Meadow	0.03
Riparian Shrubland	0.07
Riparian Woodland	0.11
River Channel	0.028
River Early Successional	0.1
River Shrubland	0.07
Roads	0.02
Rural Developed	0.02
Sand Pit	0.02
Unvegetated Sandbar	0.035
Upland Woodland	0.12
Warm-water Slough	0.08
Xeric Wet Meadow	0.03

Phragmites

The land use and vegetation shapefile provided by the Program did not indentify any specific areas of phragmites growth. Although phragmites were not explicitly identified, there are a number of near-channel areas that have roughness values of either 0.11 (Riparian Woodland) or 0.07 (Riparian or River Shrubland). These areas could potentially contain high-density phragmites growth. If these areas are identified to have high-density phragmites growth, the Manning's roughness could be changed to 0.101. The impact is not expected to significantly change model results, since phragmites growth areas have a similarly high Manning's roughness coefficient.

Ineffective Flow Areas, Blocked Obstructions, and Levees

Ineffective flow areas, blocked obstructions, and levees were used at each cross section to prevent the model from computing flow to areas that are either hydraulically disconnected from the river channel (i.e. sandpits, roadway ditches) or would not contribute to the area through which flow is passing. Blocked obstructions were used in areas that permanently are not considered as flow area. Levees were used in a similar fashion to eliminate conveyance behind permanent and contiguous features. Permanent ineffective flow areas were used to describe areas that may be connected to river flow, but will not contribute to the conveyance area over the range of modeled flows, while non-permanent ineffective flow areas were used to limit conveyance at low flows while allowing conveyance at higher flows.

Boundary Conditions

Reach Flow

As previously mentioned, the North Platte River bifurcates at several locations within the modeled reaches. The flow split optimization feature in HEC-RAS was used to determine the amount of flow in each of the two parallel channels by balancing the hydraulic energy at the flow split location (referred to as the upstream junction). The initial allocation of flow among parallel channels prior to flow optimization was estimated based on the size of the two split flow channels, and preserved the total river system flow at the upstream junction.

Downstream Boundary Conditions

The stage upstream from the Tri-County Diversion Structure is generally maintained at an elevation equal to the top of the Ogee Spillway (pers. comm., Cory Steinke/CNPPID), so a constant stage of 2770.04 feet NAVD 88 was used over the range of modeled flows.

Gate Openings at Inline Structures

There were several diversion structures mentioned in Landmarks and Structures. Each structure was modeled in HEC-RAS. For purposes of the steady-state model, all in-line gates were assumed to be fully open, in order to represent a condition in which no flow was diverted. This condition best represented a scenario in which no flow was diverted at these structures.

Model Calibration

Calibration Data

In general, the calibration objective for the steady-state model was to:

- Minimize the differences between measured and predicted water-surface elevations, with average differences of less than a few tenths of a foot;
- Eliminate any consistent trend of over- or under-predicting along the length of the project reach and;
- Reduce maximum differences to less than 1 foot.

The available calibration information included rating curves at the stream gages, water-surface elevations collected in conjunction with local cross-section surveys, and inferred water-surface elevations from the LiDAR data. Rating curves were available at 2 gages operated by NDNR in the reach between the Keystone Diversion and the Tri-County Diversion (Table 2.4). A water surface profile survey conducted by TFG in 2011 provided a number of locations where a water-surface elevation and approximate channel discharge could be correlated. A water-surface elevation profile was inferred from LiDAR data survey by assuming the survey did not penetrate the water surface, and was therefore, represented by the lowest measured elevation within the channel. The LiDAR data were collected between March 19, 2009, during which time the discharge in the river ranged from 0 (no measured release at the Keystone Diversion) to about 340 cfs at the North Platte River at North Platte gage.

Table 2.4: Rating Curves Available for Steady-State Model Calibration

Gage Identification	River Mile	Station
North Platte River near Sutherland (NDNR Gage No. 6691000)	337	72457
North Platte River at North Platte (NDNR Gage No. 6693000)	316	19297

Additionally, color-infrared and low-altitude photography was available for some sections of the project reach at several different high flows. These photos were qualitatively compared with predicted results to validate effective and ineffective flow areas specified in the model.

Calibration Methods

Calibration of the model was achieved by refining the cross-section roughness parameters and low-flow channel geometry. As discussed previously, the general horizontal distribution of the Manning's roughness coefficients was originally assigned using information from the Program's Vegetation Monitoring Program. The limits of these roughness zones were initially adjusted to better match the channel geometry and aerial photography. The zones along the banks were then slightly adjusted (up or down the bank) to improve calibration. In areas where no survey data were available, the channel bottom was adjusted to account for the area below the water surface in the LiDAR survey (Figures 2.2 and 2.3).

Calibration Results

Keystone Diversion to the Start of the Choke Point Model

Model calibration for the Keystone diversion to the upstream extent of the "Choke Point" model Reach was performed using LiDAR data, flown on March 19, 2009 and current gage rating curves supplied by the NDNR.

The initial calibration involved iteratively adjusting subaqueous river channel bathymetry utilizing LiDAR data. Adjustments were made to achieve agreement between the modeled water surface and observed water surface. Calibration was performed so that computed and observed WSELs had agreement at most cross sections within 0.3 feet. Among cross sections, an average error of -0.04 ft and standard deviation of 0.17 ft was achieved. Figure 2.4 depicts these results.

Calibration was also performed at the one NDNR gage located in the modeled reach from the Keystone Diversion to approximately 5 miles upstream of North Platte, the North Platte River at Sutherland (No. 6691000). The rating curve for this gage was provided by the NDNR. Model runs for flows ranging from 500 to 6,000 cfs were conducted to develop a modeled rating curve. Bank stations, ineffective flow areas, and bridge hydraulics were adjusted to calibrate the modeled rating curve to the NDNR published rating curve. These changes were then reflected in the remainder of the model, so that consistent ineffective flow area, bank station assignment, Manning's roughness coefficient, and bridge hydraulic parameters were employed throughout the model.

Figure 2.5 shows the comparison of predicted and gaged WSELs at North Platte River near Sutherland Gage based on the NDNR rating curve (2011), on the upstream face of Prairie Trace Road. At the range of flows the model was calibrated to, the predicted WSEL matches within half a foot, with the greatest deviation from the published curve occurring at the highest flows (between 5,000 and 6,000 cfs). At these upper-end flows, the model predicts a WSEL less than 0.4 ft higher than the published rating curve.

Table 2.5 shows predicted water surface elevations at high flows (approximately 5,300 to 5,700 cfs based on gaged flow at the Keystone, Sutherland and North Platte gages) compared to surveyed high-water elevations at the observed flows in June of 2011. Comparison to both the calculated WSEL and EGL are shown to indicate a range of possible high water marks that may be expected given model results. The RAS-calculated WSEL assumes a cross-sectional average velocity. High-water elevations are surveyed at the bank or other solid structures where the velocity would be significantly less than the average channel velocity. The energy grade line (EGL), which accounts for both elevation and velocity head, was also used in the comparison to measured WSELs. The EGL represents an approximate upper-limit of the range of elevations that may occur at the given cross section if the velocity at the measured point was zero (for example, at an upstream face of a bridge embankment). Comparisons between measured high water elevation and predicted WSEL and EGL are summarized in Table 2.5. The results are consistent with other calibration results, and indicate reasonable reach-scale hydraulics are represented in the model.

Table 2.5: Comparison of Measured and Predicted Water Surface Elevations, June 7th -8th, 2011 – Elevations are in NAVD '88 feet.

Measurement Description	Measured Water Elevation	Predicted WSEL	Predicted EGL	Difference Between Measured WSEL and Predicted WSEL	Difference Between Measured WSEL and Predicted EGL
DS Keystone-Roscoe Rd	3093.30	3093.70	3094.05	-0.40	-0.75
US Keystone-Roscoe Rd	3094.02	3094.14	3094.38	-0.12	-0.36
DS Road E TN	3021.00	3020.70	3020.99	0.30	0.01
US Road E TN	3021.29	3021.27	3021.52	0.02	-0.23
DS Road E VN	3000.31	3000.15	3000.44	0.16	-0.13
US Road E VN	3000.81	3000.62	3000.79	0.19	0.02
DS Prairie Trace Rd	2929.00	2928.31	2928.59	0.69	0.41
US Prairie Trace Rd	2929.96	2929.41	2929.67	0.55	0.29
DS N Hershey Rd	2888.96	2888.97	2889.24	-0.01	-0.28
US N Hershey Rd	2889.68	2889.56	2889.78	0.12	-0.10
Average				**0.15**	**-0.11**

Choke Point Model Calibration

As was reported in HDR et al., 2011, predicted results from the North Platte "Choke Point" model also match the measured and gage rating curve information reasonably well (Figures 2.6 and 2.7, respectively). Although the most weight was given to the surveyed and gaged water surfaces, the calibration required optimizing the match between the predicted and various measured WSELs (Figure 2.6). For example, at the Highway 83 gage, the rating-curve comparison indicates the model over-predicts the water surface by about 0.2 feet at 1,200 cfs, but the model under-predicts the surveyed WSEL at survey section 861265 (located a short distance upstream) by about 0.5 feet. The model appears to calibrate reasonably well, since the predicted WSELs are generally within a few tenths of the WSEL inferred from the LiDAR data, with maximum differences of +0.5 and -0.8 feet, and no consistent over- or under-prediction. The predicted results also match the surveyed WSELs reasonably well. The model also appears to match the gage-rating curve fairly well, especially considering the scatter in the measured WSELs (Figure 2.7)

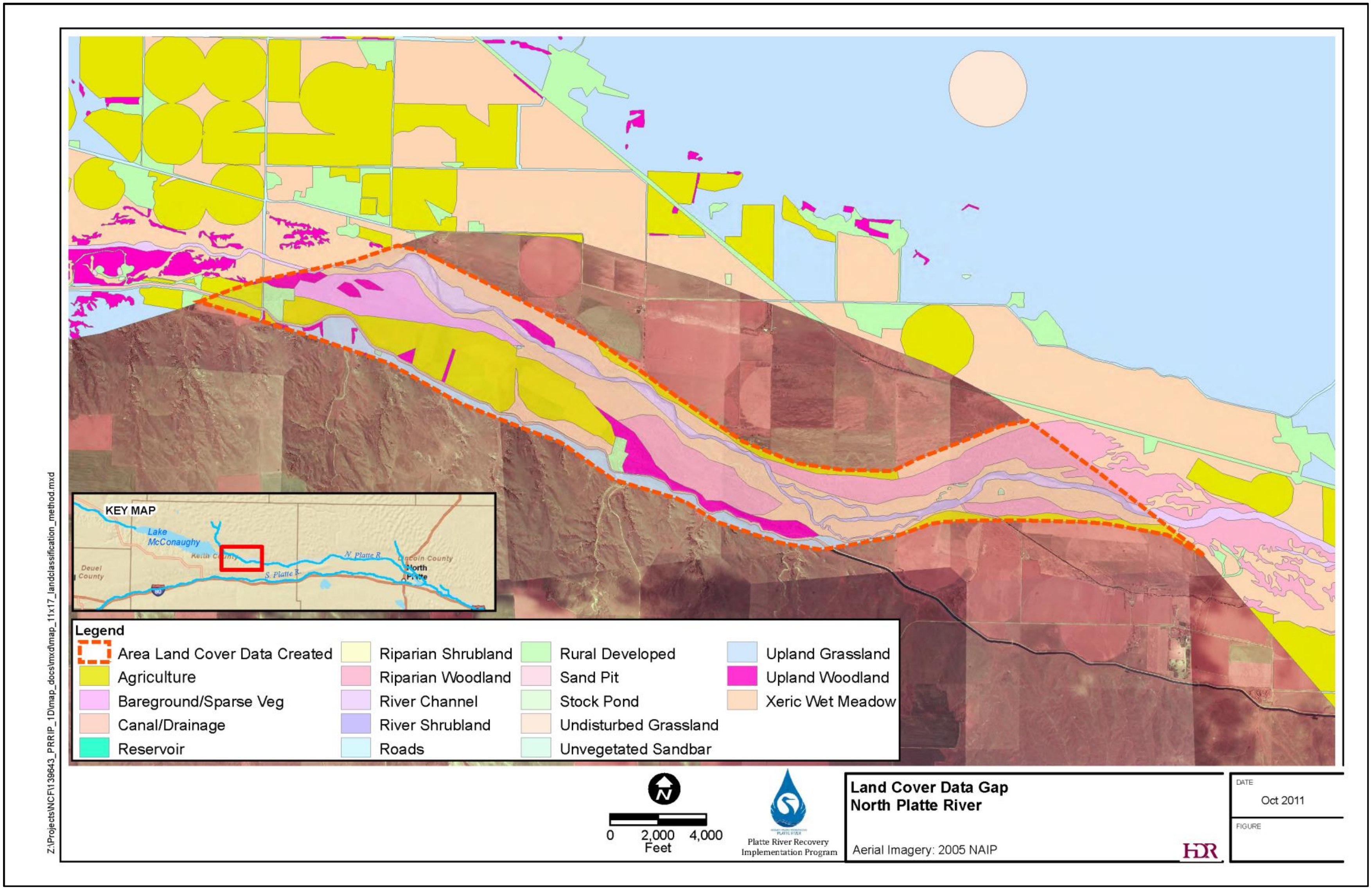


Figure 2.1: Land Use Polygon used to Assign Roughness Coefficients.

*The land use in the dashed red outlined area was created based on 2005 CIR Aerial photography obtained from TPNRD. Land use for the remainder of the model area was based on 2005 land use/cover data originally digitized by the USFWS in 2009.

Figure 2.2: Typical Cross Section Before Points Filtering and Subaqueous Channel Adjustment

Figure 2.3: Typical Cross Section After Points Filtering and Subaqueous Channel Adjustment

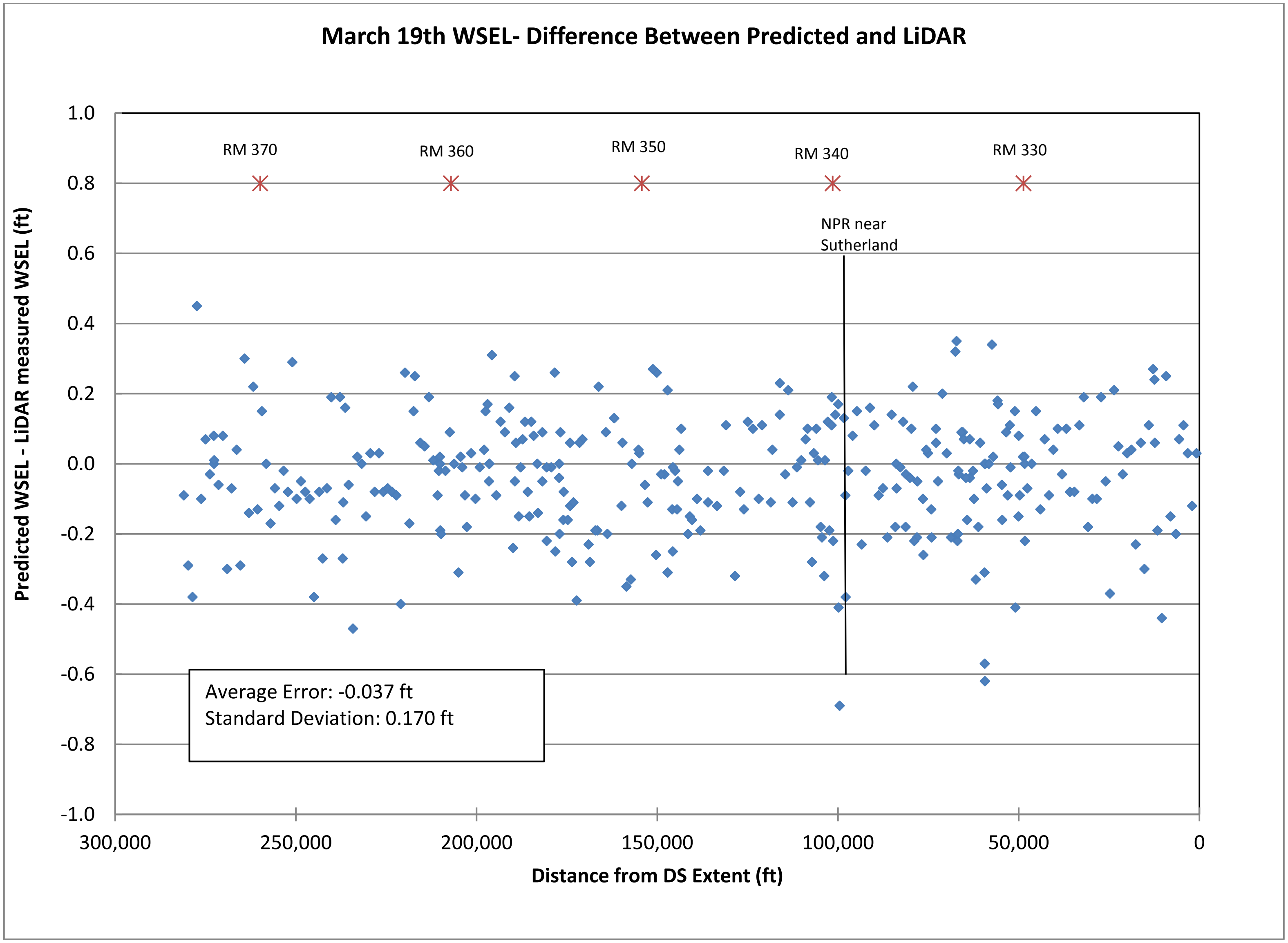


Figure 2.4: LiDAR flow calibration results, Extended Reach from approx. RM 370 to RM 321 (upstream of Choke-Point Model)

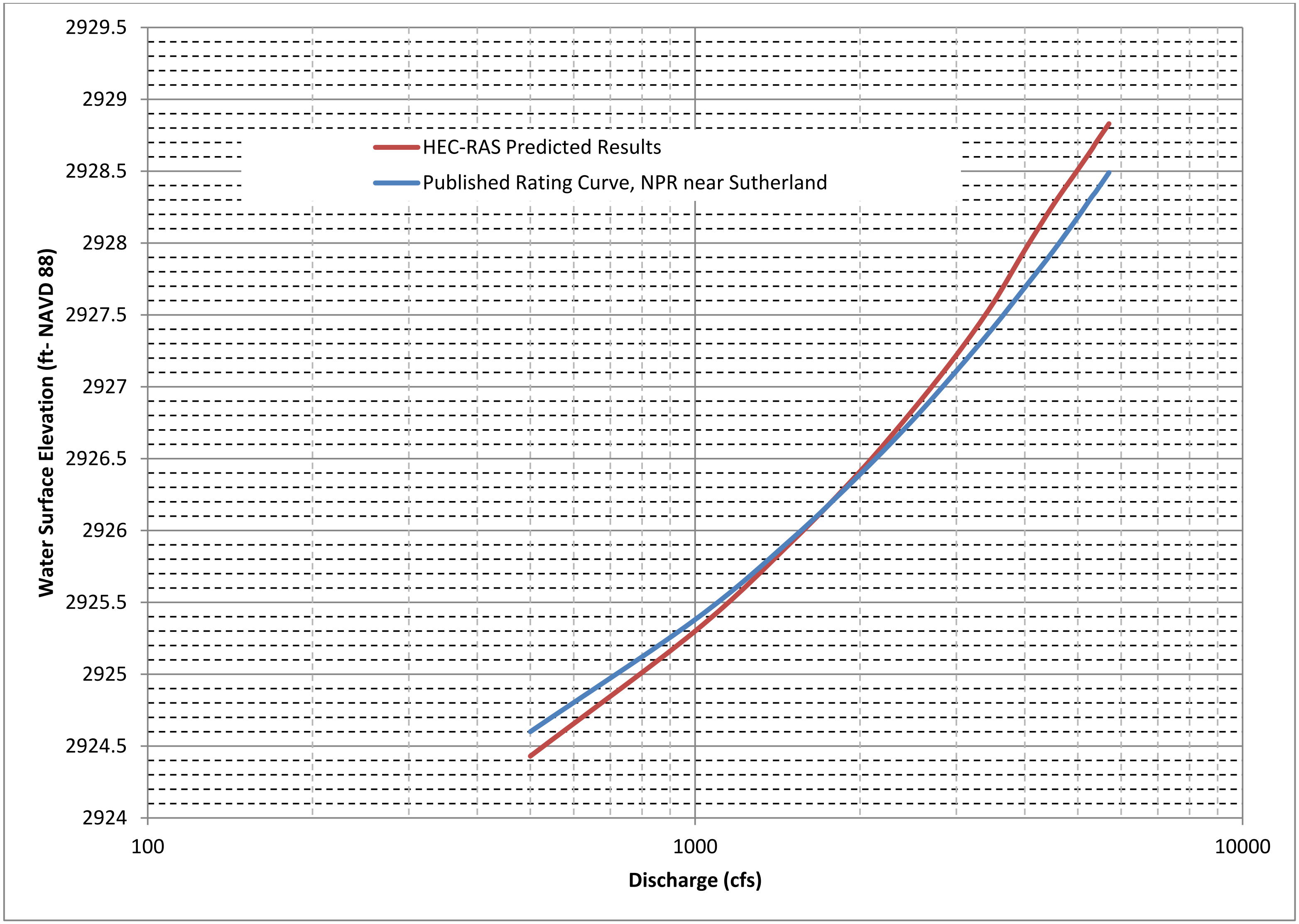


Figure 2.5: North Platte River near Sutherland, Rating Curve Comparison

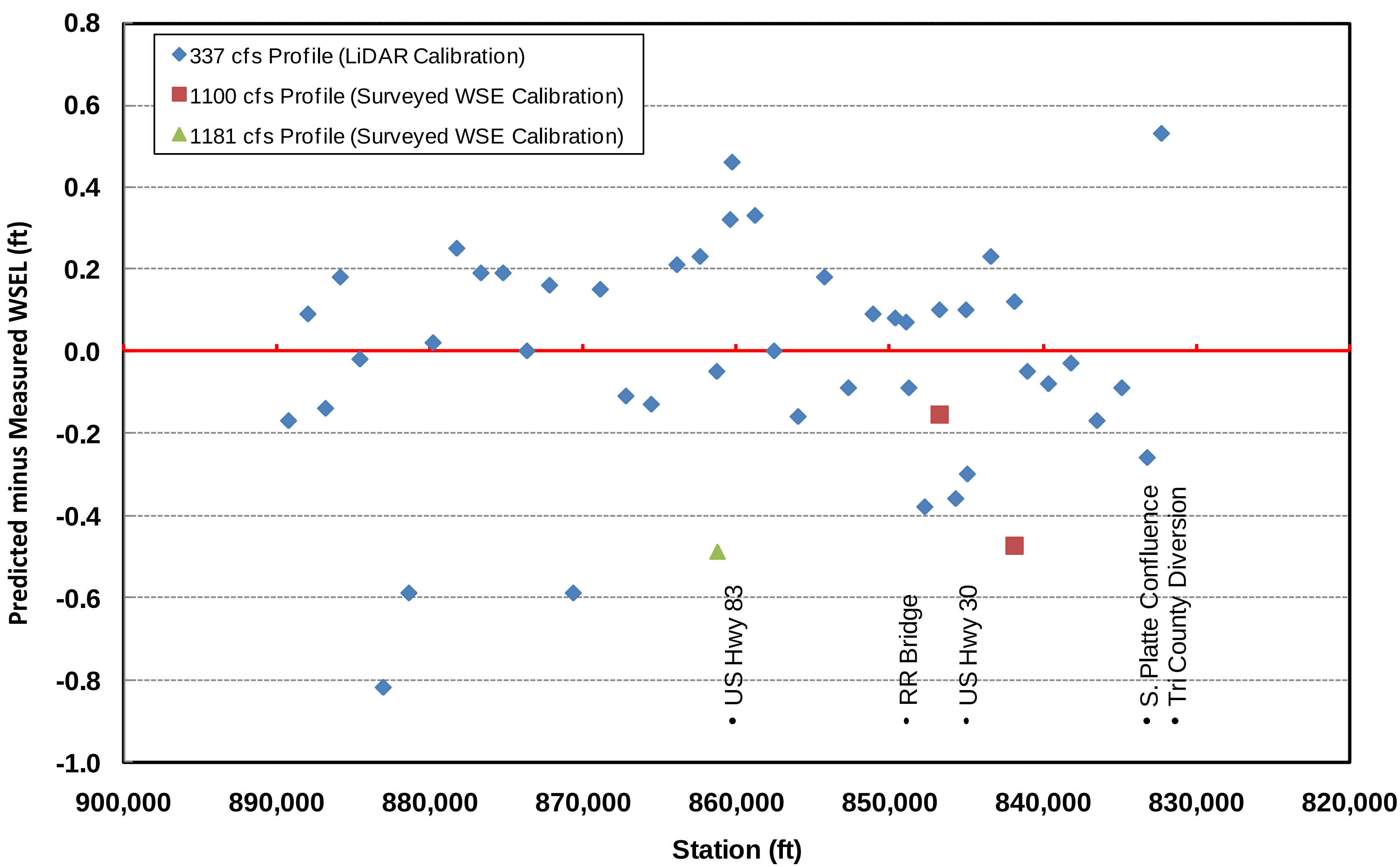


Figure 2.6: Comparison of WSELs predicted by the steady-state HEC-RAS model of the North Platte "Choke Point" reach and the observed (surveyed and LiDAR-inferred) WSELs.

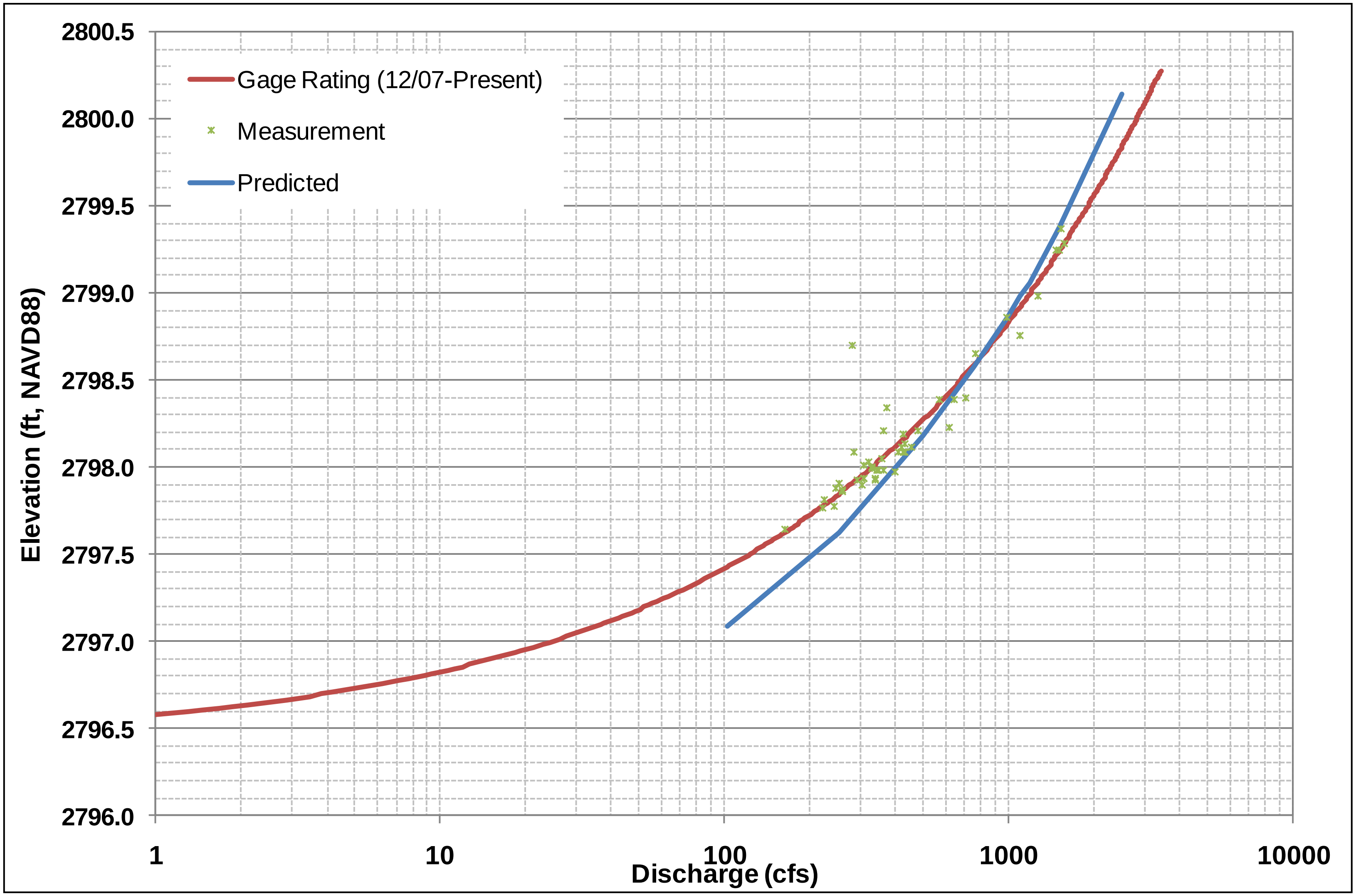


Figure 2.7: Comparison of the predicted WSEL and the North Platte River at North Platte DNR gage rating curve and associated measurements.

3. Unsteady Model

Steady-State Model Conversion

The calibrated steady-state model, including cross-section geometry, roughness parameters, hydraulic structures and ineffective flow areas, was used as the basis for the unsteady model. A variety of modifications were made to the model geometry and other parameters to achieve and maintain computational stability, typically the greatest challenge in creating a useable unsteady model. Unlike the standard-step backwater algorithm used in the steady- state model, unsteady modeling algorithms are very sensitive to certain physical parameters (USACE, 2010). As a result, it is commonly necessary to modify the original geometry and other data parameters, with the objective of increasing model stability while maintaining model reliability. Causes of instability include channel network complexity, cross-section spacing and critical-flow conditions at low-flow areas that cause numerical instabilities that prevent the model from finding a valid solution.

Some secondary channels are not active at all flows in the modeled hydrograph; however, the HEC-RAS unsteady algorithm requires that all channels that are incorporated into the model carry at least some flow. The HEC-RAS pilot channel functionality was used to designate small slots in secondary flow channels that allow them to carry a very small amount of flow that permits numerical stability, but does not significantly affect conditions in the primary flow paths. These pilot channels are ignored once flow levels rise to the point where the channels can flow normally.

Additionally, at junction locations, it was often necessary to copy the bounding cross sections to reduce the length over which junction approximations are made. When this was done, the channel depths in the copied cross sections were adjusted to match the local channel slope.

In general, low flows are the most problematic in achieving model stability. Based on the anticipated use of the model to route SDHF releases and other hydrographs, the model was refined to permit routing of a target minimum flow of 180 cfs at the upstream boundary.

Model Calibration

The model calibration approach for the reach from Keystone Diversion to Tri-County Canal model was consistent with the calibration approach used for the Tri-County to Chapman (Central Platte) model. (HDR et al., 2011)

Calibration and Validation Events and Data

The Program designated three events as calibration events for the unsteady modeling effort. The criteria used to identify calibration events in the Keystone Diversion to North Platte Reach were: peak flow magnitude as close as possible to planned SDHFs (1,000cfs to 3,500 cfs), similar vegetation condition as those embedded in the model (i.e., 2005 land use/cover), and data availability. The Program suggested the following calibration events based on a review of historical hydrographs: April 2009, June - August 2008, and July-August 2001. Complete 30-minute flow and stage data at the stream gages and gaged canal diversions were available for the 2009 and 2008 event. For the 2001 event, 30 minute data were only available for the gaged canal diversions and Lincoln Co. Drain No. 1. For this event, mean daily gauged flow data were used to provide model inputs and comparison data at the Sutherland and North Platte gages where 30-minute data were not available.

April 2009 Flow Routing Test

The April 2009 event featured a SDHF test release from Lake McConaughy. This event was selected because it was anticipated that the Program would use the model to predict the flow translation and attenuation of similar events. The Keystone Diversion gage measured flows above 1,200 cfs between

April 10th, 2009 at 12:30am and April 15, 2009 at 10:30pm. The peak flow measurement of 1,540 cfs occurred on April 13, 2009 at 5:30pm. The peak of this hydrograph increases as the event propagates downstream (approximately 1,630 cfs at Sutherland and 1,713 cfs at North Platte), due to ungaged gains and the inflow of approximately 160 cfs at Birdwood Creek (between the Sutherland and North Platte Gage). Hydrograph translation time was approximately 1 to 2 days between the Keystone and Sutherland gages, and approximately 2 days between the Sutherland and North Platte gages. No diversions were made from the North Platte River during this event, as a result of coordination with irrigation and power entities for the SDHF test release. See Figure 3.1 for the event hydrographs.

2008 Natural High Flow Event

A natural high flow event occurred between June and August 2008. This period experienced two peak flows with distinct rising and falling limbs. At the Keystone gage, these peaks occurred on July 14th and August 3rd, with magnitudes of approximately 2,000 cfs and 2,300 cfs, respectively. As the event moved through the system, the measured peaks decreased. This was likely due to the increased flow diversion for irrigation and smaller ungaged reach gains than the historical averages applied that occur during this event. Hydrograph translation time was approximately 1 day between the Keystone and Sutherland gages, and approximately 1 day between the Sutherland and North Platte gages. See Figure 3.2 for the event hydrographs.

2001 Natural High Flow Event

The August 2001 event, at the North Platte River at Keystone gage, has a peak flow of approximately 3,300 cfs, and has a duration of approximately 11 days above 1,500 cfs (from August 3 to August 14). The measured peaks at Sutherland and North Platte are approximately 2,950 cfs and 3,200 cfs respectively. Hydrograph translation time was approximately 2 days between the Keystone and Sutherland gages, and approximately 1 day between the Sutherland and North Platte gages. The 30-minute data from the Keystone, Sutherland, North Platte, and Birdwood Creek gages were not available from the Nebraska DNR. For model boundary conditions and calibration at these locations, mean daily flows were applied. See Figure 3.3 for the event hydrographs.

Historical Reach Gains and Losses

In addition to gaged inflows and diversions, incorporating historical reach gains and losses aided in the prediction of streamflow in many of the modeled reaches. The values used were calculated average monthly reach gains/losses calculated from water year (WY) 1985-WY 2008. Reach gains and losses (Table 1.2) were incorporated utilizing a uniform lateral inflow boundary condition. The calculated gain/loss rate was distributed, based on the fraction of distance between gages each model reach represented. For lengths where multiple channels exist, equal gain or loss was distributed among the two channels. In certain cases, the long-term historical gain or loss was not supported by event specific data (for instance, Sutherland to North Platte, summer 2008). To maintain a consistent modeling methodology, the historical reach gain/loss was applied.

Ungaged Inputs and Outputs

Gage data demonstrate, on an event and reach- specific basis, some discrepancies between upstream and downstream gaged volumes that are not easily reconciled by incorporating historical reach gain or loss. Such discrepancies can either be attributed to variability in gaged measurements, or an ungaged input or output. Ungaged inputs may be a result of significant contributions from ungaged tributaries (point input) or runoff from a local high-intensity storm event. In cases where ungaged inflows or outflows exist, calibration efforts were focused on matching timing and trends rather than absolute discharge measurements.

An example of ungaged inflow occurred during the 2009 event. During this SDHF test, a significant rainfall event occurred in western and central Nebraska. Daily rainfall totals at two regional rainfall measurement stations in western and central Nebraska during this event are shown below in Table 3.1 (High Plains Regional Climate Center).;

Table 3.1: Daily Regional Rainfall Totals for Regional Weather Stations (High Plains Regional Climate Center)

Date	Station Location	
	Hershey (RM 330)	**North Platte (RM 315)**
April 11, 2009	0	0
April 12, 2009	0	0
April 13, 2009	0.17	0.13
April 14, 2009	0	0
April 15, 2009	0	0
April 16, 2009	0	0
April 17, 2009	0.61	0.37
April 18, 2009	1.04	0.9
April 19, 2009	0.01	0
April 20, 2009	0	0
April 21, 2009	0	0

The effect of this event was most apparent in the gage hydrograph at the North Platte River at North Platte Gage. During the falling limb of this event, a second smaller rising limb, peak, and falling limb occur between April 18th and 20th. Rainfall data indicated a significant rain event occurred in North Platte on these dates, potentially explaining the additional "peak" that was not observed at the Keystone gage and not very apparent in the Sutherland gage.

Bank Storage Approach

Studies modeling high flow events in the central Platte River determined that the inclusion of bank storage effects is necessary to accurately predict the hydrograph peak and shape of SDHFs (Randle and Samad, 2008, HDR et al., 2011). Bank storage is the phenomena of a finite volume of water being temporarily stored in the porous near-bank terrain during an event. This is manifested as a volume of water entering this storage during the rising limb of an event and exiting the storage during the falling limb of the same event. The conceptual model developed by Randle and Samad approximates the bank storage response as a function of the hydrograph (Figure 3.4). Based on their conceptual model, a qualitatively similar approach was implemented in estimating bank storage effects in this model. The approach assumes a rapid linear increase of water entering the bank on the rising limb of the hydrograph. Once the hydrograph reaches it peak, that rate decreases exponentially. Similarly, on the falling limb of the hydrograph, a volume of water returns to the channel with a rate that decreases again over time.

Based on this response, a conceptual bank storage hydrograph was developed, shown in Figure 3.5. In this conceptual hydrograph, water is removed from the channel flow beginning at t_o. The rate of flow removed from the channel increases linearly from zero to its "peak" (Q_a) at t_a. After t_a, the flow removed decreases as an exponential decay function. The removal of flow from the river channel begins with the rising limb of the hydrograph, with t_a occurring during the rising limb. As the hydrograph reaches its falling limb, flow is then returned to the channel, starting at t_b. Flow is returned to the channel in a like manner- increasing linearly from zero to a "peak" (Q_c) at t_c, then decreasing as an exponential decay function. The April 2009 event is the only event modeled in which bank storage effects are considered and is most representative of a SDHF event. This event had steep rising and falling limbs. The model predicted relatively fast downstream translation and little change in hydrograph shape in comparison to gaged data.

A limitation of this approach is that flow is added and removed uniformly (utilizing a lateral inflow hydrograph boundary condition) throughout each model reach. In cases where there is a different discharge at the upstream and downstream ends of a reach (beginning of a sudden pulse), flow into bank storage would vary significantly. Modeling this as a uniform lateral flow can result in a brief period of spatial misrepresentation of flow into bank storage. This effect manifested as an occasional oscillation in the modeled discharge (see Figure 3.6, April 10th for an example). During the calibration, the timing of bank storage hydrographs was adjusted to minimize the presence of these oscillations.

The general concept of a bank storage hydrograph, used to represent the volume of water lost on the rising limb of the hydrograph and returned to the river channel after the falling limb was adopted as the a calibration tool for the April 2009 SDHF event. Bank storage hydrographs were developed as part of the calibration and implemented to produce similar pulse celerity and peak attenuation as that observed in gage data. Bank storage relationships were developed for the gaged reaches, and distributed among the corresponding model reaches. The uniform lateral inflow hydrograph boundary condition was utilized to apply the bank storage hydrograph throughout the model at the scale of individual model reaches.

Initial model runs for natural high flow events (e.g., the 2008 and 2001 calibration events used to calibrate the unsteady model) predicted reasonable hydrograph translation and attenuation without accounting for bank storage when compared to gaged reaches (Figures 3. 8 through 3.11). Bank storage effects were likely masked by the gradual nature of the rising and falling limbs and duration of the event. For this reason, bank storage was not considered during calibration of the long duration natural high flow events (2008 and 2001). This observation was consistent with other unsteady modeling efforts on the central Platte River (Randle and Samad, 2008). Based on the events evaluated, it is recommended that bank storage be considered when simulating short-term events (i.e., shorter than 1 week in duration), but not for longer duration events.

Calibration Methods

The unsteady hydraulic model was calibrated and validated by routing three events (2009, 2008, and 2001), using hydrologic inputs and outputs as the flow boundary conditions, and making bank storage adjustments based on comparison to gage data and overall model performance. In the calibration of the model, some isolated adjustments were made to improve model performance. Such adjustments included slight changes to subaqueous channel bathymetry, additional or eliminated cross sections, and appropriate adjustments to ineffective flow areas. Roughness coefficients were not adjusted in calibration of the unsteady model. These adjustments often improved model stability and minimized error associated with HEC-RAS hydraulic calculations. As previously stated natural high flow events (i.e., 2008 and 2001) did not warrant adjustment to account for bank storage.

For the 2009 event, a SDHF pulse release, bank storage hydrographs were adjusted to achieve improved calibration at main-channel gage stations. Calibration involved adjusting the bank storage hydrograph timing parameters (t_o, t_a,t_b, t_c) and magnitude parameters (Q_a, Q_c, k_1, k_2) (Figure 3.5) to replicate pulse magnitude, timing, and attenuation throughout the modeled river system. One bank storage hydrograph was developed per river section between gages, and was distributed throughout the length of river between the two gages.

Calibration was performed by comparing predicted hydrographs with measured discharge data provided by the NDNR. The model was considered to be adequately calibrated when computed hydrographs resulted in similar shapes, peaks, and temporal trends as measured hydrographs at the gages, considering the ungaged gains and losses. This was initially qualitatively assessed and later quantified using a Nash-Sutcliffe efficiency coefficient. The Nash-Sutcliffe efficiency coefficient is a normalized statistic comparing discrepancy between predicted and observed flow to variation of flow over an event. Values vary from 1.0 to -∞, where 1.0 indicates a perfect match and 0.0 indicates that a mean observed value matches the observed (half-hourly for instance) time-series data as well as model predictions. A target Nash-Sutcliffe coefficient was not explicitly established at the beginning of calibration, but this was used to aide modelers in quantifying incremental improvements in model performance. Generally, a Nash-Sutcliffe coefficient in the 0.7-0.8 range demonstrated acceptable model performance, but calibration efforts involved improving model performance beyond this range when possible.

Calibration Results

Keystone to North Platte (RM 370 to RM 310)

As previously stated, the April 2009 event featured a short duration high flow release from Lake McConaughy. It was selected because it was anticipated that the Program would use the model to predict the flow translation and attenuation of similar events. The bank storage relationship developed for the reach from Tri-County Diversion to Chapman, (similar to that developed by Randle and Samad, 2008), was incorporated for this reach. The timing and magnitude parameters used to calibrate the model are explained in the next section title “Bank Storage Prediction Tool”. Uniform lateral inflow hydrographs were applied to account for water entering bank storage on the rising-end of the event and re-entering channel flow on the falling end of the hydrograph. Using this method to account for bank storage, improved agreement in hydrograph timing and shape occurred (Figures 3.6 and 3.7). A review of the historic reach gains and losses corresponded well with the reach gain and loss experienced during this event. An increase in flow occurred during the falling-limb of the hydrograph at North Platte (Figure 3.7) which likely resulted from rainfall runoff as local weather stations at North Platte and Hershey indicated 1.25-2 inches of rainfall precipitation occurred from April 16-18. No attempt was made to model runoff to the North Platte River associated with this precipitation event. A comparison of peak flow magnitude, timing, and Nash-Sutcliffe Coefficients is shown in Table 3.2.

The 2008 event was also used to calibrate the model. Similar to the 2009 event, the historic reach gain (based on gage records) was applied for both the Keystone to Sutherland and Sutherland to North Platte reaches. Average historical June, July, and August gains and losses were used to match the time of year for this June through August 2008 calibration event. Bank storage effects were not considered for the 2008 event because, as discussed in the previously-modeled reaches, bank-storage was only anticipated to be vitally-important for predicting the magnitude and timing of short duration events and was not expected impact the hydrograph for long-duration events. This resulted in good agreement between predicted and gaged hydrographs at the North Platte River near Sutherland gage (see Figure 3.8). The predicted hydrograph at North Platte over predicted the gaged flow on both rising limbs, but seemed to predict the falling limb accurately. However, a favorable Nash-Sutcliffe Coefficient of 0.664 indicated that the model predicted the timing and magnitude of this event with some accuracy (see Figure 3.9). A comparison of peak flow magnitude, timing, and Nash-Sutcliffe Coefficients is made in Table 3.2.

Overall, the 2008 event unsteady calibration results are satisfactory. Timing and magnitude of the hydrograph agree well at Sutherland (Figure 3.8). The one instance where there was consistent discrepancy between the predicted and gaged hydrograph is on the rising limb of both hydrographs at the North Platte gage (Figure 3.9). To address this, attention was given to all model inputs and gage data. The gage data indicate a gain between Keystone and Sutherland (agrees with historical trend) and a loss between Sutherland and North Platte (contrary to historical trend). One possible explanation for this trend would be the Sutherland gage being biased, recording higher discharges than may have actually occurred. The gains in this reach are inherently variable, since a number of canal returns (treated as ungaged gains) can either return flow back to the North Platte in this reach or the South Platte River. Given the known level of variability in ungaged inputs, the predicted hydrographs match reasonably well with gaged records. Figures 3.10 and 3.11 show all historical observed gage data as well as the calculated 30-minute gain/loss for the Keystone to Sutherland and Sutherland to North Platte reaches.

Data used to complete the 2001 Event comparison was limited due to the lack of available 30-minute data for Birdwood Creek as well as the Keystone, Sutherland, and North Platte gages on the North Platte River. Mean-daily data was used at Keystone and Birdwood Creek to provide model inputs and at Sutherland and North Platte to provide a basis for comparison between gaged and predicted hydrographs. Predicted hydrographs at Sutherland and North Platte were similar in peak discharge, shape and total volume to gaged records, though the pulse appeared to travel faster through the system than observed. See Figures 3.12 and 3.13 for hydrograph comparisons. This demonstrates the ability of the model to predict these characteristics accurately for events similar to those observed in July and August of 2001. Relative to using 30-minute data for model inputs, use of daily-average flows for model inputs has a smoothing effect on the hydrograph, providing few perturbations and no subdaily fluctuations in flow. Therefore, this calibration did not evaluate the model's ability to solve for the attenuation of any sub-daily flow characteristics originating at the Lake McConaughy. For the purposes of predicting the timing and magnitude of an event such as the 2001 event, daily flow data appears to be adequate. However, this event may not be as stringent an assessor of model performance as the 2008 or 2010 events, which utilized 30-minute data as both a model input and calibration metric. A quantitative comparison of peak flow magnitude, timing, and Nash-Sutcliffe Coefficients is made in Table 3.2.

Table 3.2: Results of 2009, 2008, and 2001 Calibration Simulations

Event	Gage	Peak Magnitude, Gaged	Peak Magnitude, Predicted	Peak Time, Gage	Peak Time, Predicted	Nash-Sutcliffe
2009	Sutherland	1649 cfs	1537 cfs	4/14/2009 17:00	4/14/2009 9:30	0.979
	North Platte	1747 cfs	1726 cfs	4/16/2009 17:30	4/15/2009 2:00	0.825
2008	Sutherland	2322 cfs	1919 cfs	8/6/2008 17:00	8/6/2008 12:30	0.873
	North Platte	1866 cfs	2089 cfs	8/7/2008 7:00	8/7/2008 3:00	0.634
2001	Sutherland	3120 cfs	2959 cfs	8/13/2001	8/12/2001 0:30	0.886
	North Platte	3230 cfs	3227 cfs	8/14/2001	8/13/2001 7:30	0.941

Model Sensitivity Notes

In addition to calibration simulations, two sensitivity analyses were performed to investigate the sensitivity of the predicted hydrograph to the in-channel Manning's roughness coefficient and the implementation of bank storage on a natural high flow event. Since the 2008 event demonstrated the largest difference between observed and predicted, sensitivity analyses were performed using this event as a way of bracketing the impact on modeled WSELs from changes to Manning's roughness and accounting for bank storage.

Manning's Roughness Coefficient Sensitivity

In-channel Manning's roughness coefficients were changed from 0.028 to 0.02 and 0.035. Figures 3.14 and 3.15 illustrate the sensitivity of the 2008 event to changes in the bed roughness. Results indicate that the translation and attenuation of this event is not extremely sensitive to changes in bed roughness. At the Sutherland gage station, the difference in timing between the low-roughness (0.02) and high-roughness (0.035) simulations is approximately 14 hours for the first peak (~1,600 cfs) and approximately 20 hours for the second peak (~1,950 cfs). At the North Platte gage station, the difference in hydrograph translation between low-roughness and high-roughness coefficient simulations is approximately one day at both the first and second event peaks.

Sensitivity to Bank Storage Application for a Natural High Flow Event with a Gradually-Rising Hydrograph

Also investigated was the sensitivity of bank-storage effects to a natural event. Again, the 2008 event was used as a test case. Figures 3.16 and 3.17 demonstrate the in inclusion of bank storage effects on the 2008 event, which was a natural, gradually-rising high flow event. The figures demonstrate that inclusion of the bank-storage effects can improve model performance, but it is not vitally important to predicting translation and attenuation, as is the case for short duration high flow events.

General Note on Model Stability

Model stability was a significant and continual issue in the development of the unsteady model. All calibration runs were successfully executed with a minimal number of computational warnings. As noted in the HEC-RAS User's Manual (USACE, 2010), unsteady-flow results are, by definition, approximations, and accuracy and stability become more difficult to achieve as model complexity increases. The development process detailed above focused primarily on finalizing the geometric parameters that best characterize the routing behavior of the river system for the hydrographs that were available for calibration. Because the routing behavior can change with differing antecedent and vegetation conditions and the magnitude and duration of the flows, it may be necessary to adjust these parameters in future model runs. The hydrologic inputs and computational parameters specified by the user can significantly affect the stability of the model runs. USACE (2010) provides guidance for adjustment of input values, including computational time steps, Theta weighting factor, water-surface calculation tolerances and maximum number of iterations that can improve stability and accuracy for a specific set of hydrologic input conditions. From our experience, the computational time step has a significant impact on model stability. If instabilities in the model arise, the timestep should be considered. If the timestep cannot be adjusted to stabilize the simulation, the most probable cause of instability may likely be related to having too little flow in the channel, from either input hydrology or a bank storage hydrograph.

Model Suitability

Based on calibration results, some inferences can be made about the suitability of model application for different sorts of events. Model calibration results indicated the best agreement between measured and predicted results occurred during the 2009 event. This can be attributed to the use of bank storage as a calibration metric. Additionally, the fact that this event occurs in April when diversion flows are not active eliminated some uncertainty with regards to the gains and losses in the system. During summer months when irrigation canals divert more flow, greater uncertainty in the amount of water returned, where it is returned, and at what rate can lead to inconsistent ungaged gain/loss over a reach. Given the calibration of bank storage effects to a short duration high flow event and better control of gains and losses within the reach, the model and modeling approach used is best-suited for predicting the timing of short duration high flow events (such as pulse-releases from Lake McCounaughy) performed prior to irrigation season. For longer duration events and events that feature a greater potential for variation in hydrology, the model has demonstrated an ability to predict hydrograph peak, shape and attenuation and can be used for this purpose. Additionally the model can be used to incorporate known diversion and return flows and aide in analyzing anomalies in reach-scale hydrology.

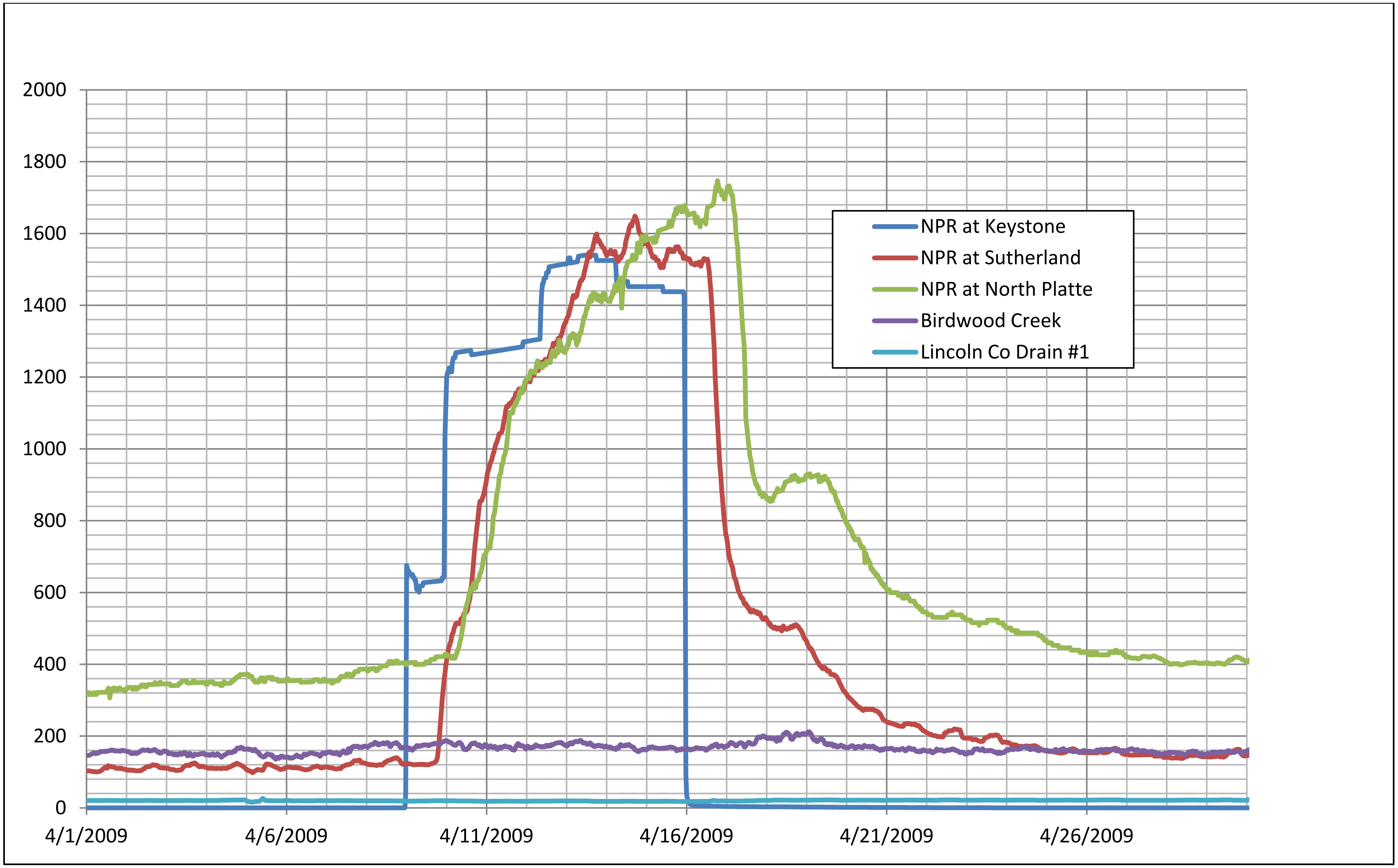


Figure 3.1. Gaged Hydrogrpahs for 2009 Event. Gage Records are instantaneous 30-minute flows. During this period, no flow was diverted from the NPR.

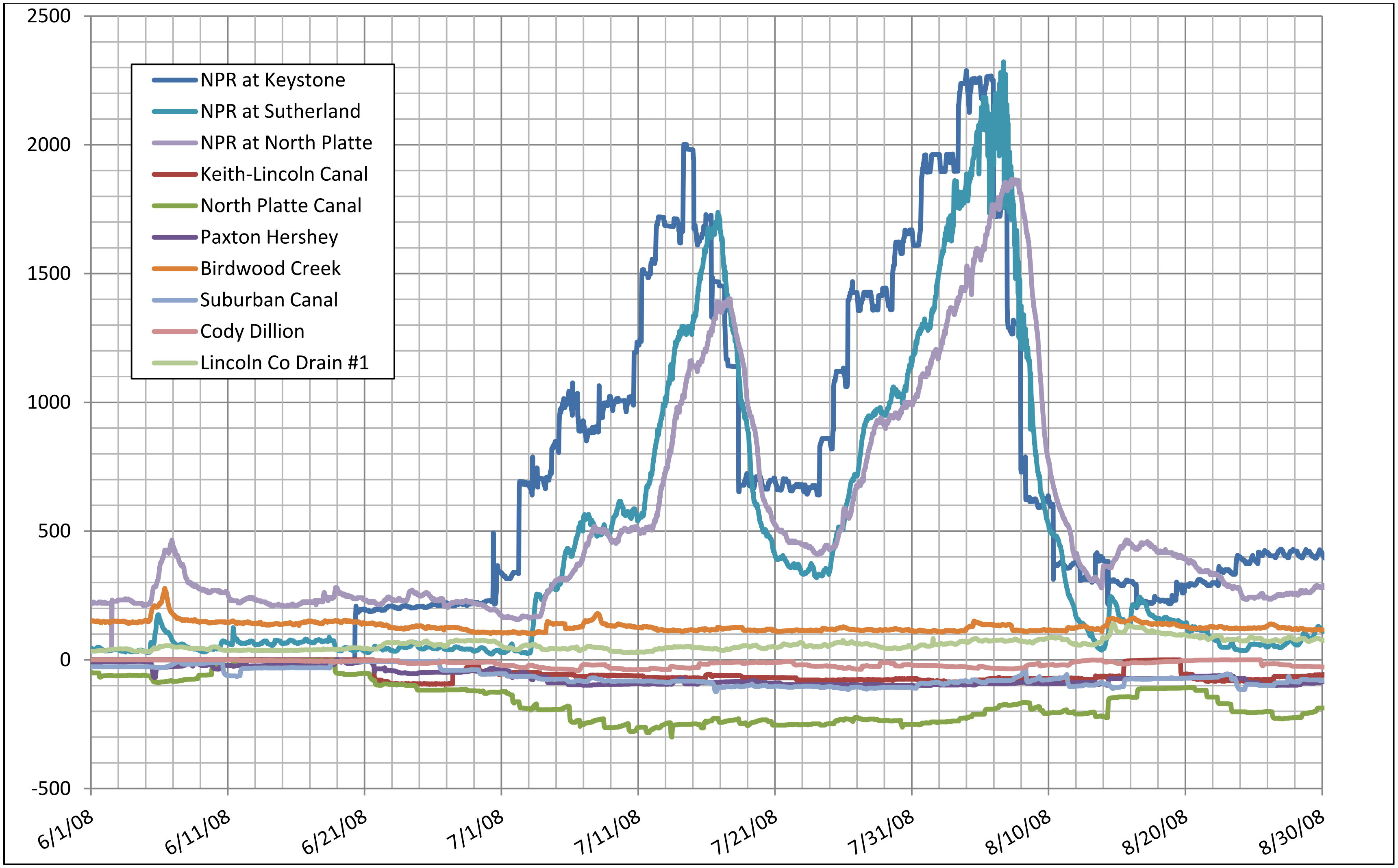


Figure 3.2. Gaged Hydrogrpahs for 2008 Event. Gage Records are instantaneous 30-minute flows. Negative values indicate flow diverted from the NPR.

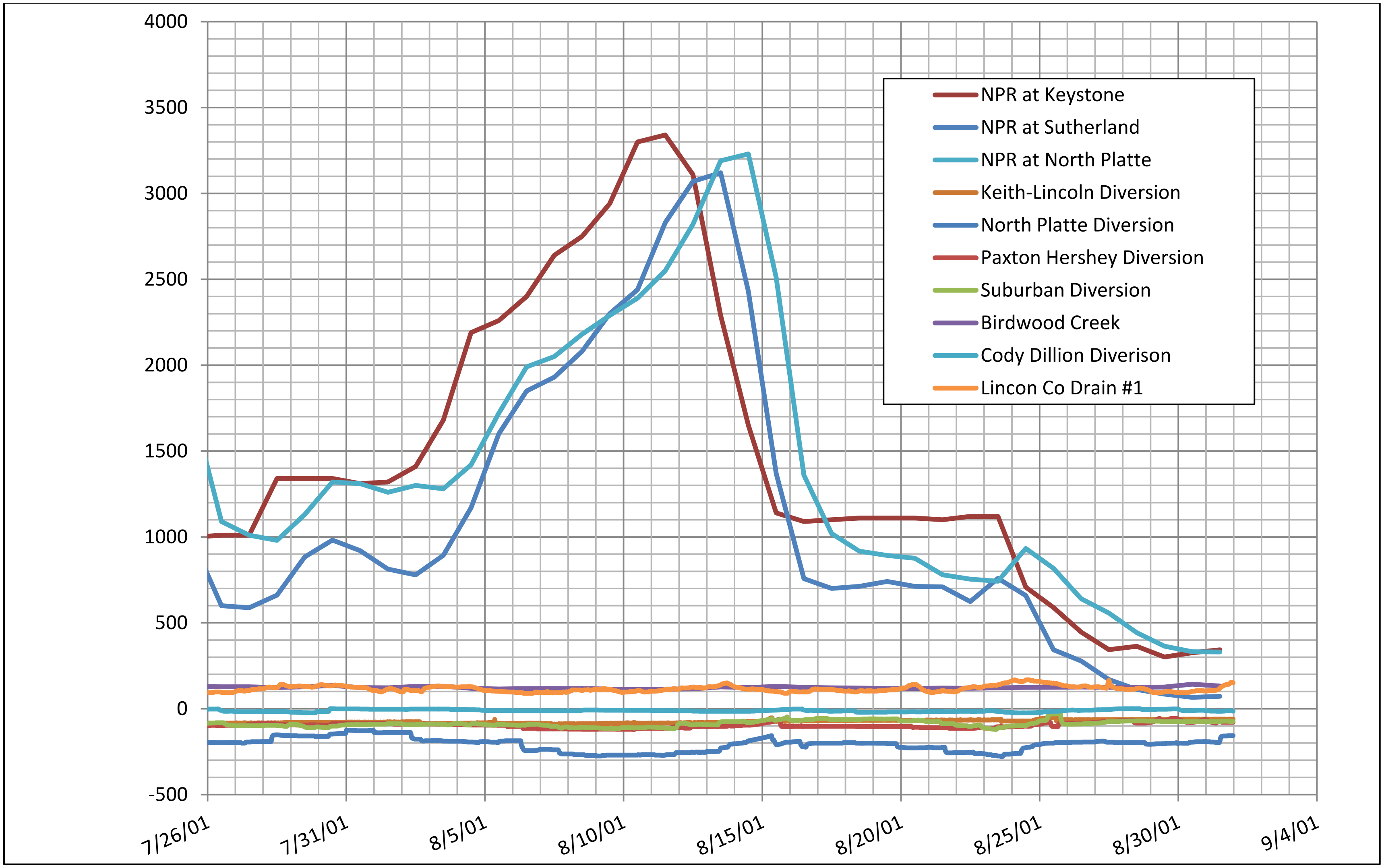


Figure 3.3. Gaged Hydrogrpahs for 2001 Event. Gage Records for NPR and Birdwood Creek Gages are mean daily flows. Negative values indicate flow diverted from the NPR.

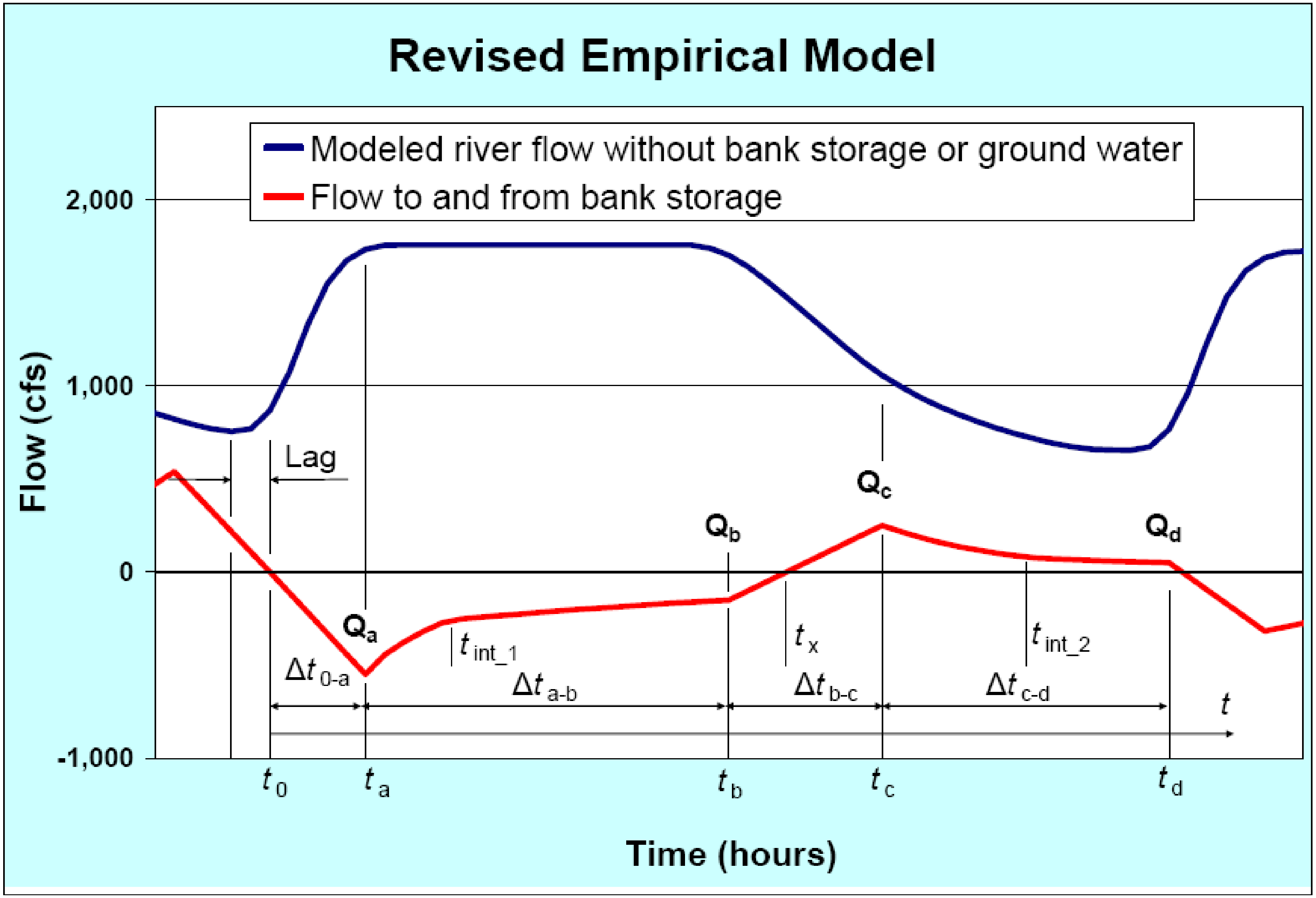


Figure 3.4: General Figure of Bank Storage relationship developed by Randle and Samad (2008)

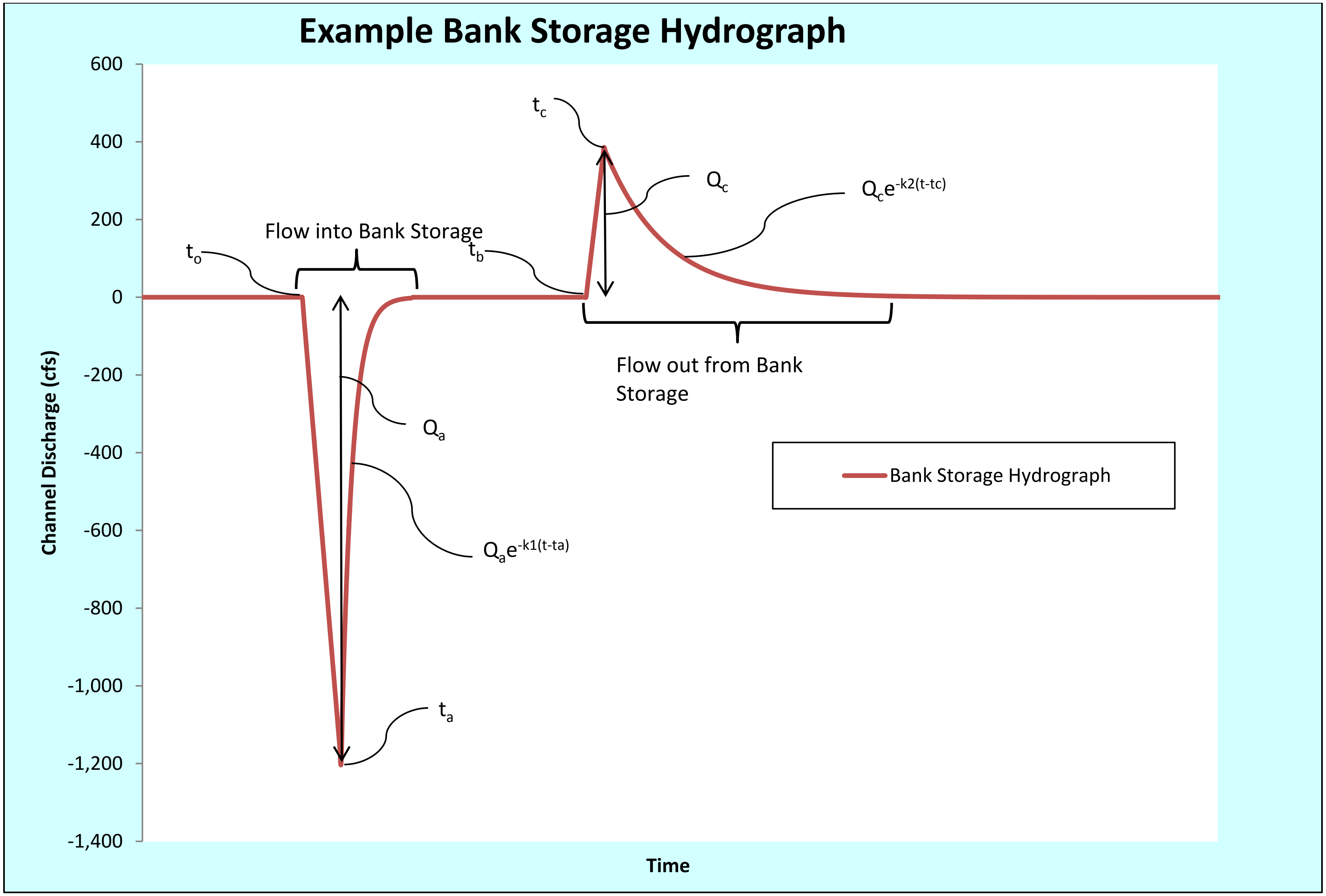


Figure 3.5: Example of Conceptual Bank Storage Hydrograph

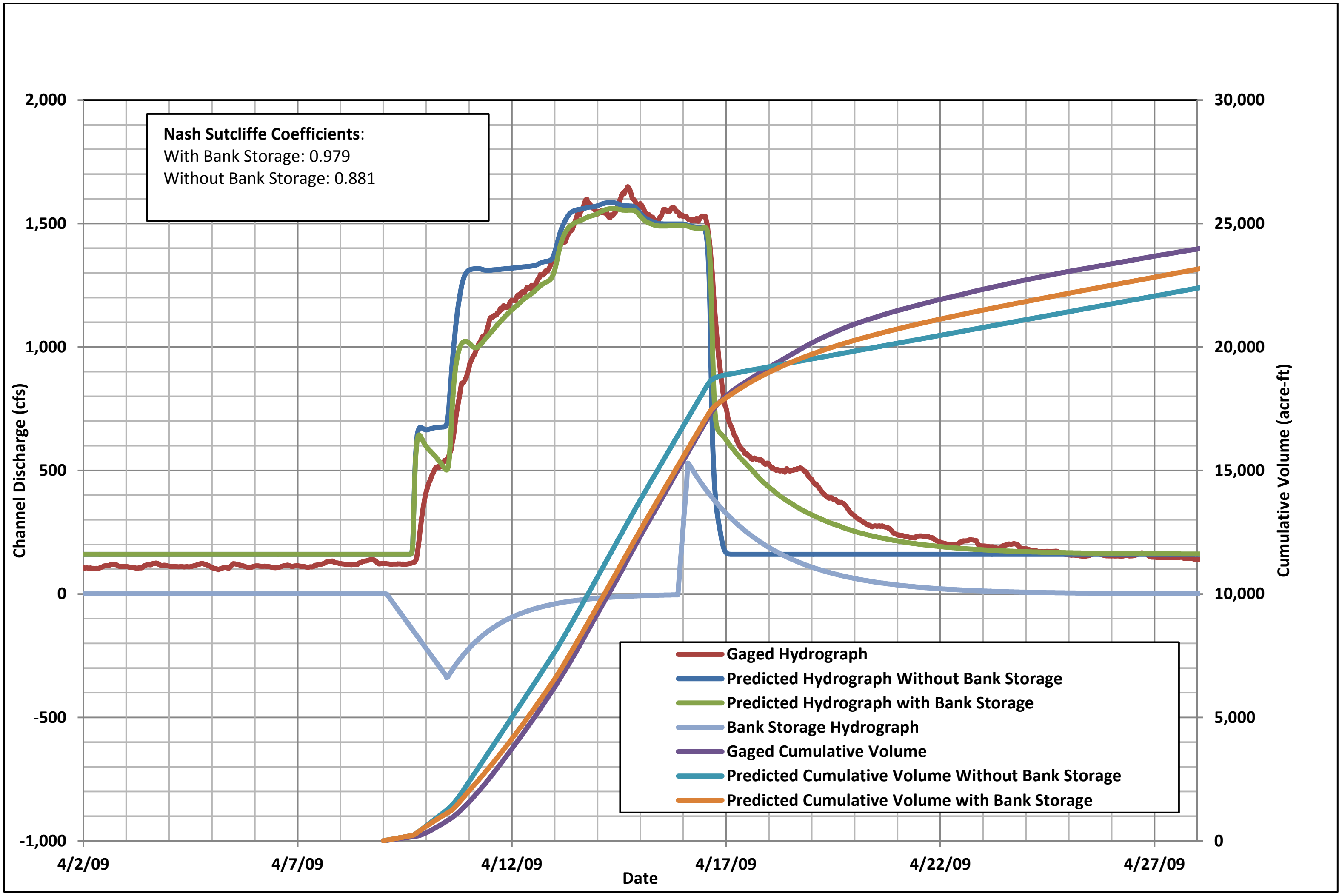


Figure 3.6: Predicted vs Gaged Hydrograph and Cumulative Volume, North Platte River at Sutherland, 2009 Event

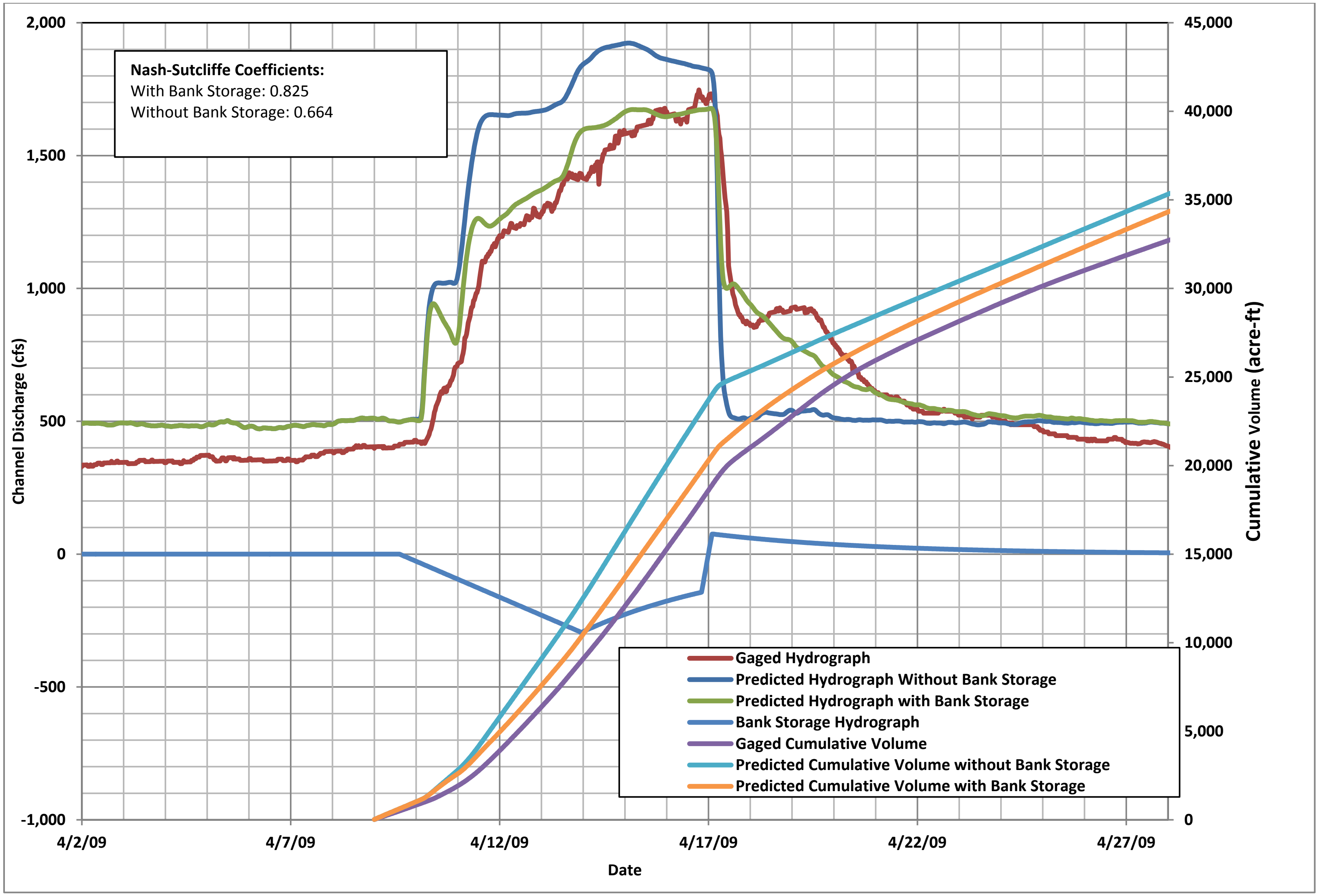


Figure 3.7: Predicted vs Gaged Hydrograph and Cumulative Volume, North Platte River at North Platte, 2009 Event.

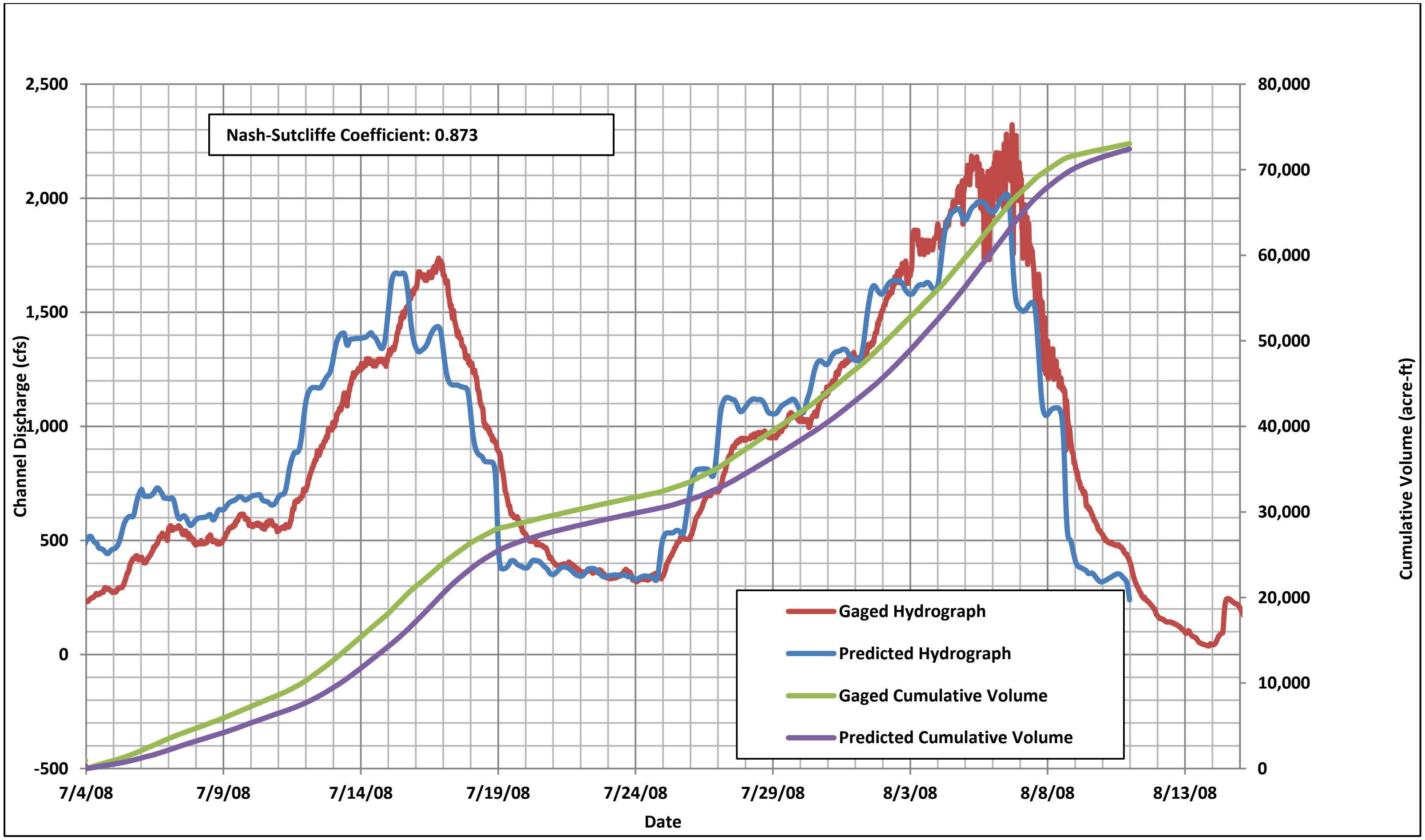


Figure 3.8: Predicted vs Gaged Hydrograph and Cumulative Volume, North Platte River at Sutherland, 2008 Event

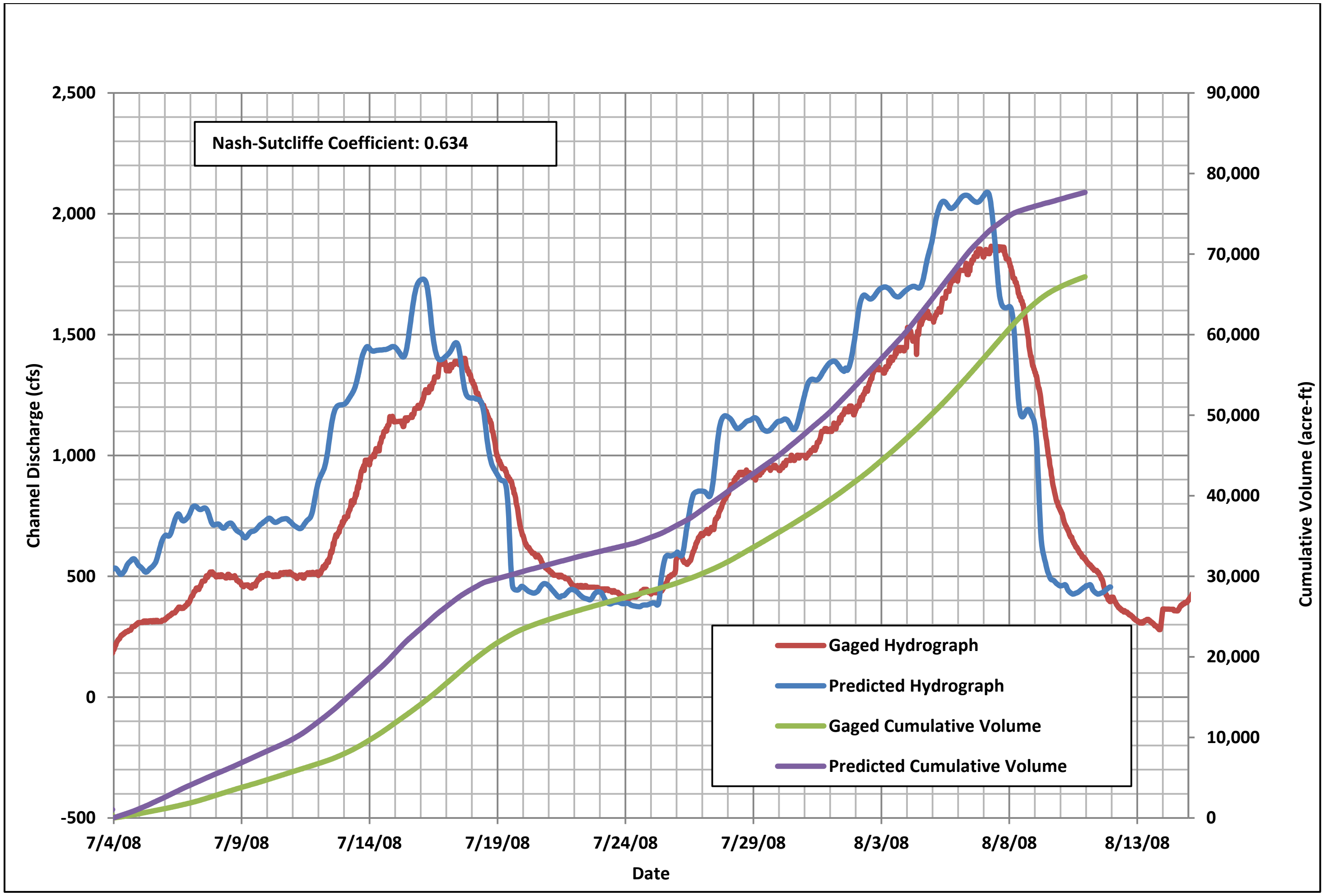


Figure 3.9: Predicted vs Gaged Hydrograph and Cumulative Volume, North Platte River at North Platte, 2008 Event.

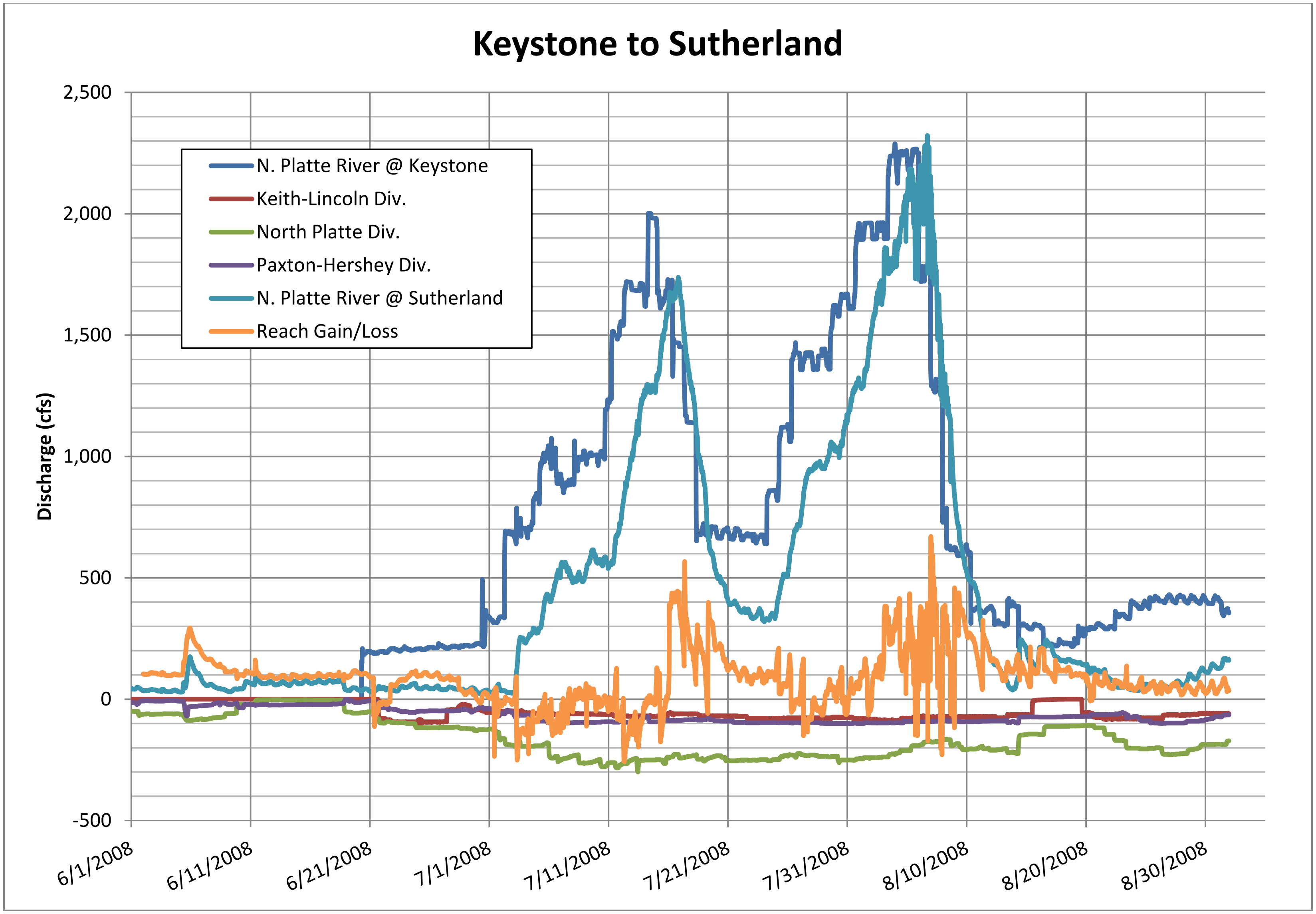


Figure 3.10: Gaged Flows and Calculated Gain/Loss for the 2009 Event, Keystone to Sutherland

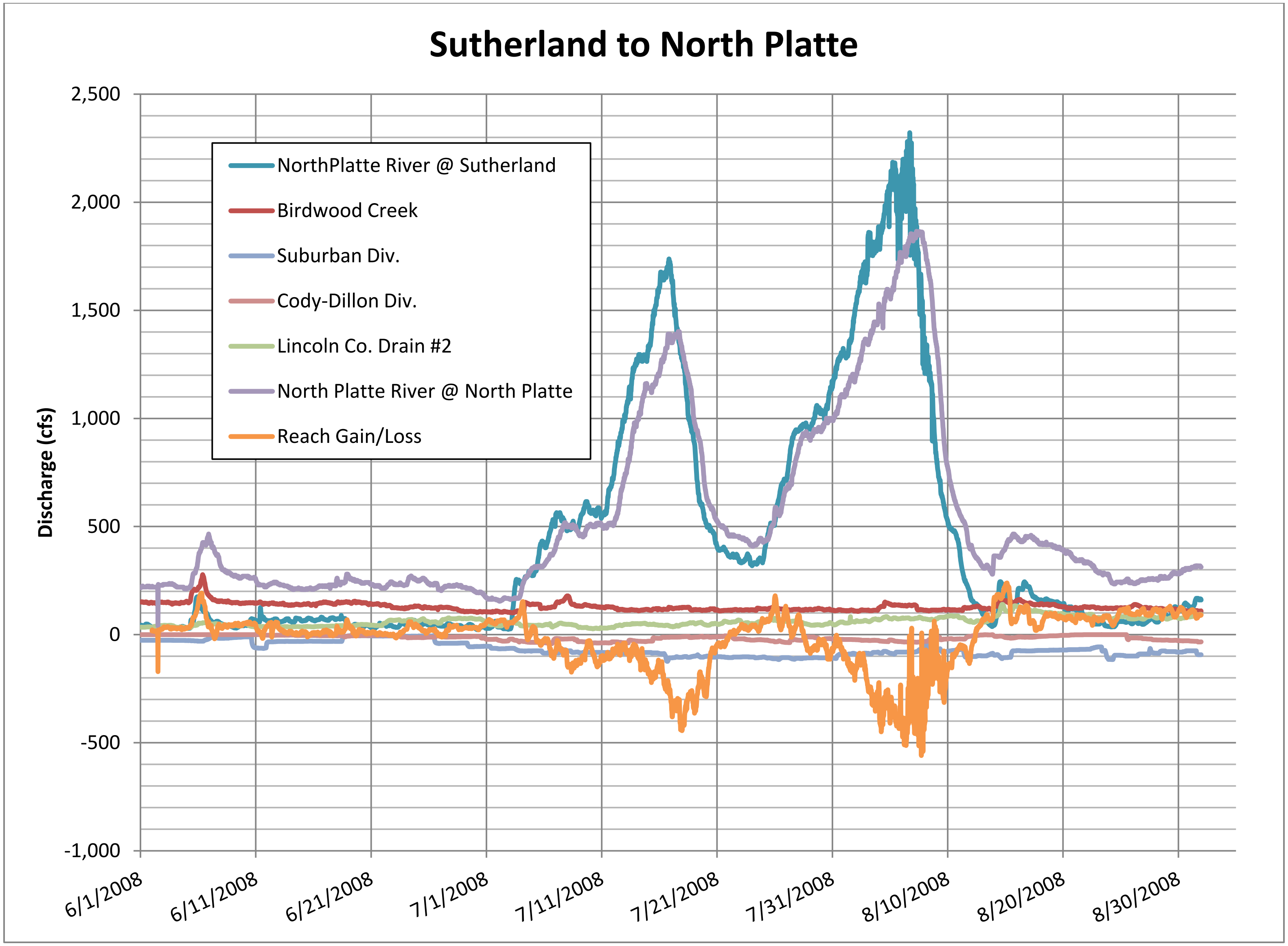


Figure 3.11: Gaged Flows and Calculated Gain/Loss for the 2009 Event, Sutherland to North Platte

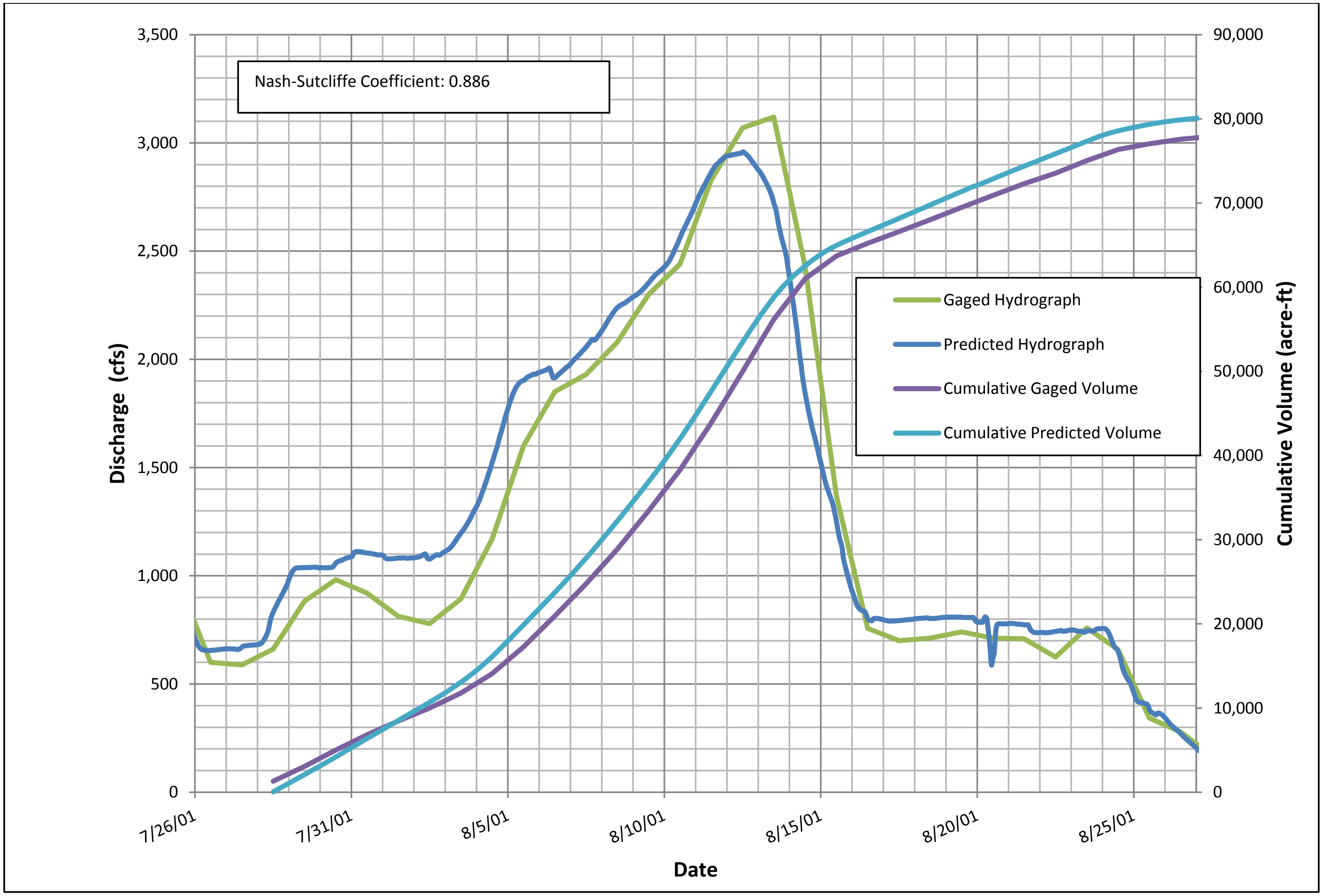


Figure 3.12: Predicted vs Gaged Hydrograph and Cumulative Volume, North Platte River at Sutherland, 2001 Event

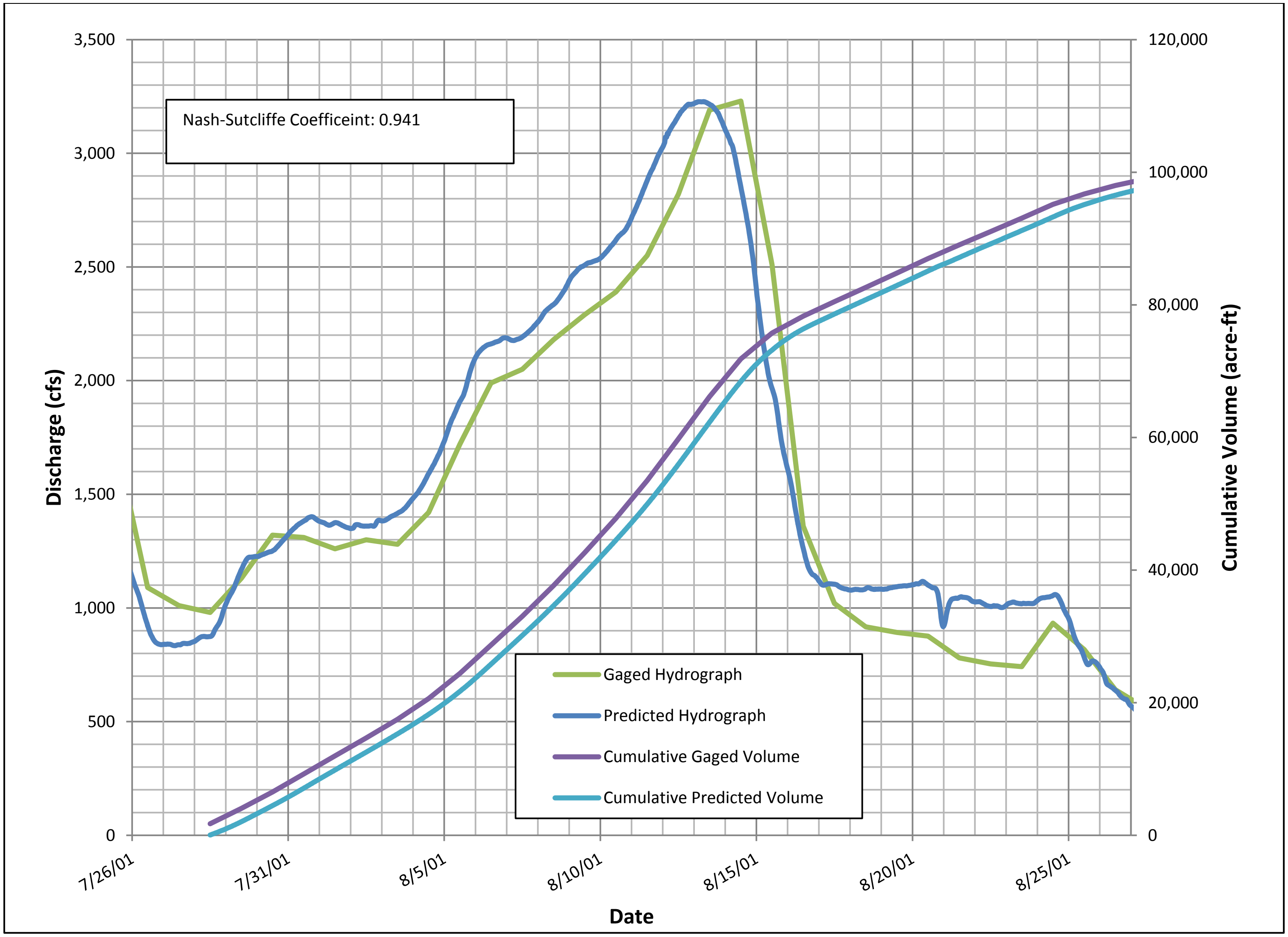


Figure 3.13: Predicted vs Gaged Hydrograph and Cumulative Volume, North Platte River at North Platte, 2001 Event.

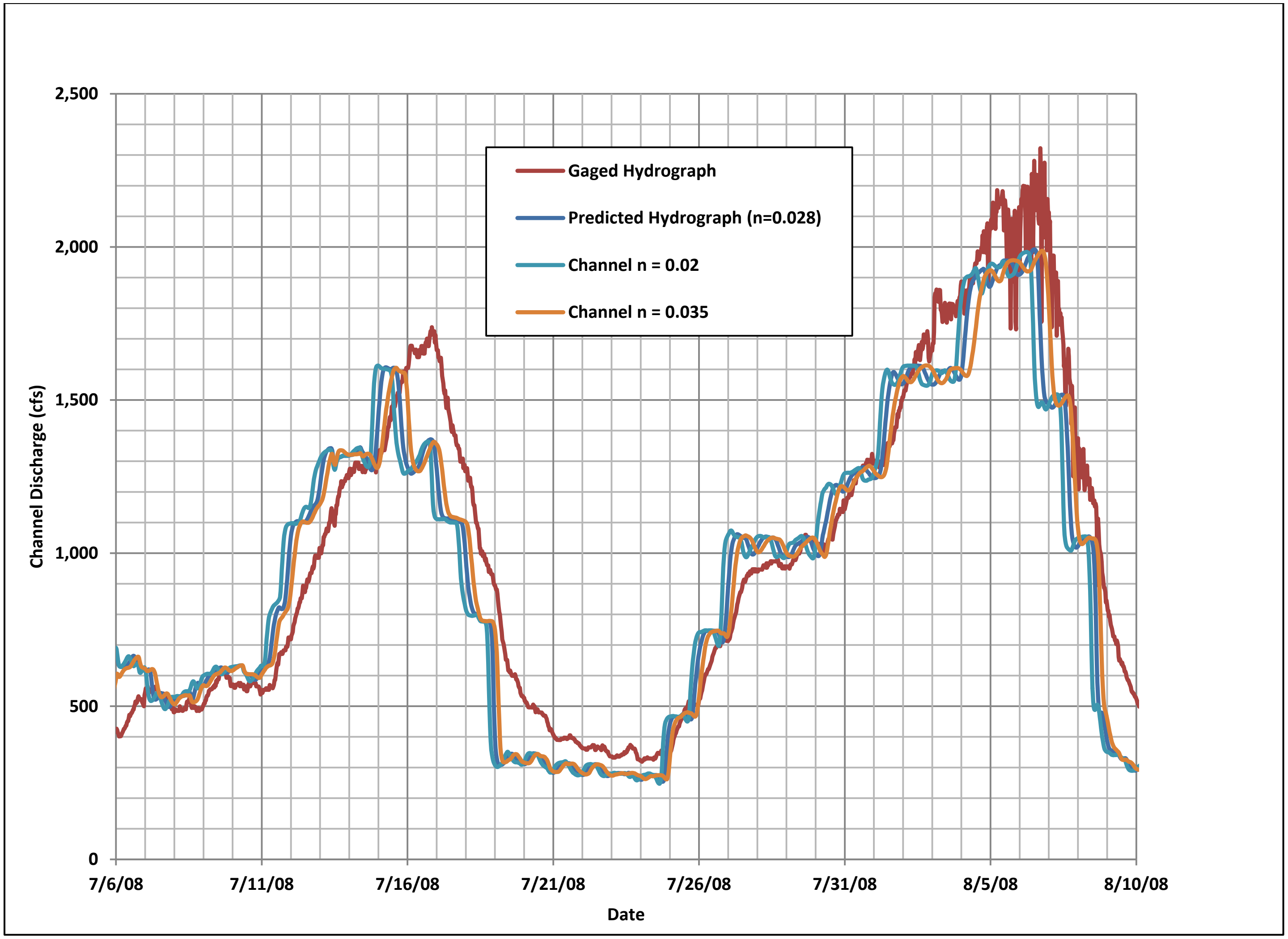


Figure 3.14: Predicted vs Gaged Hydrographs at North Platte River near Sutherland- Manning's Roughness Sensitivity Analysis.

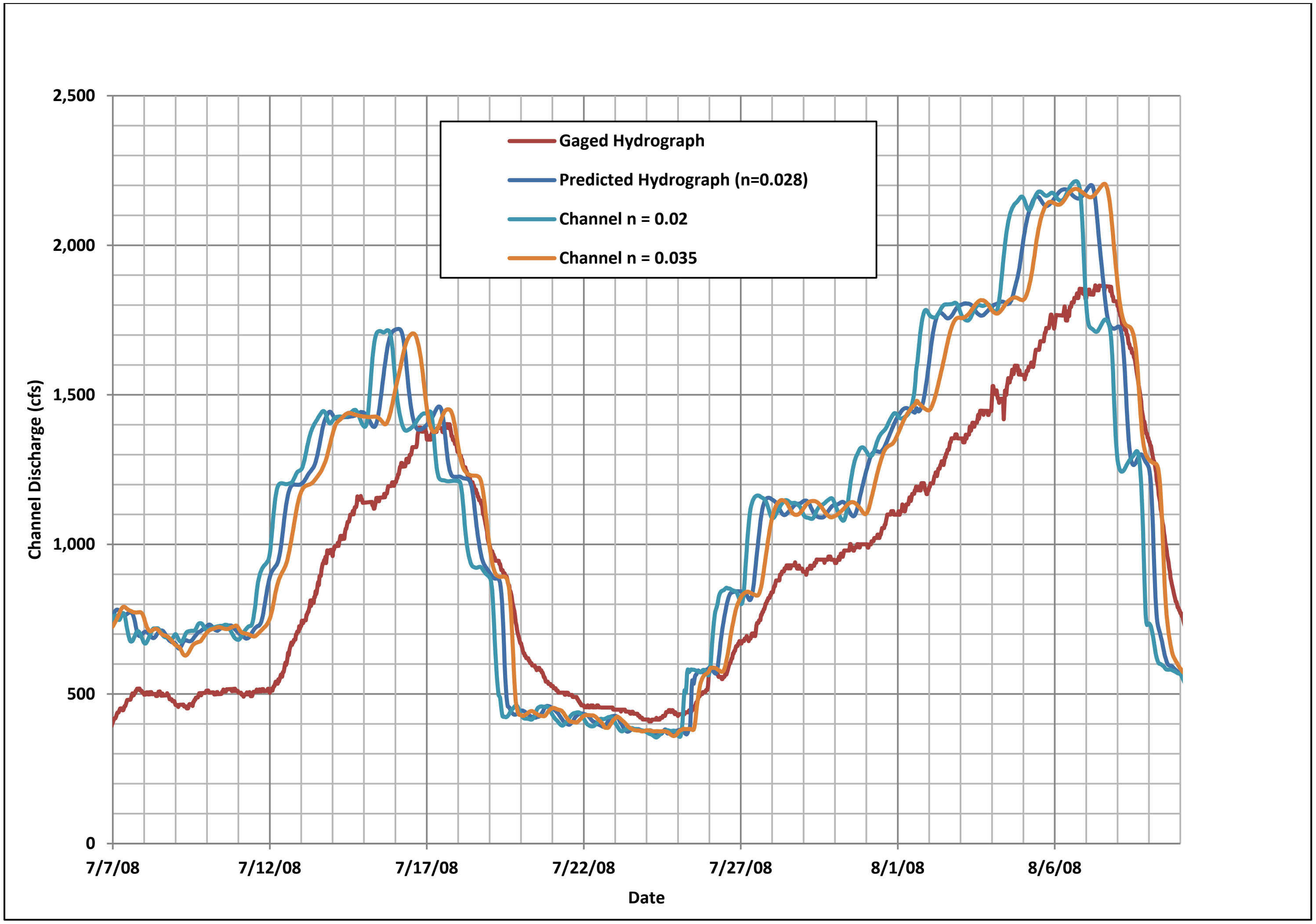


Figure 3.15: Predicted vs Gaged Hydrographs at North Platte River At North Platte- Manning's Roughness Sensitivity Analysis.

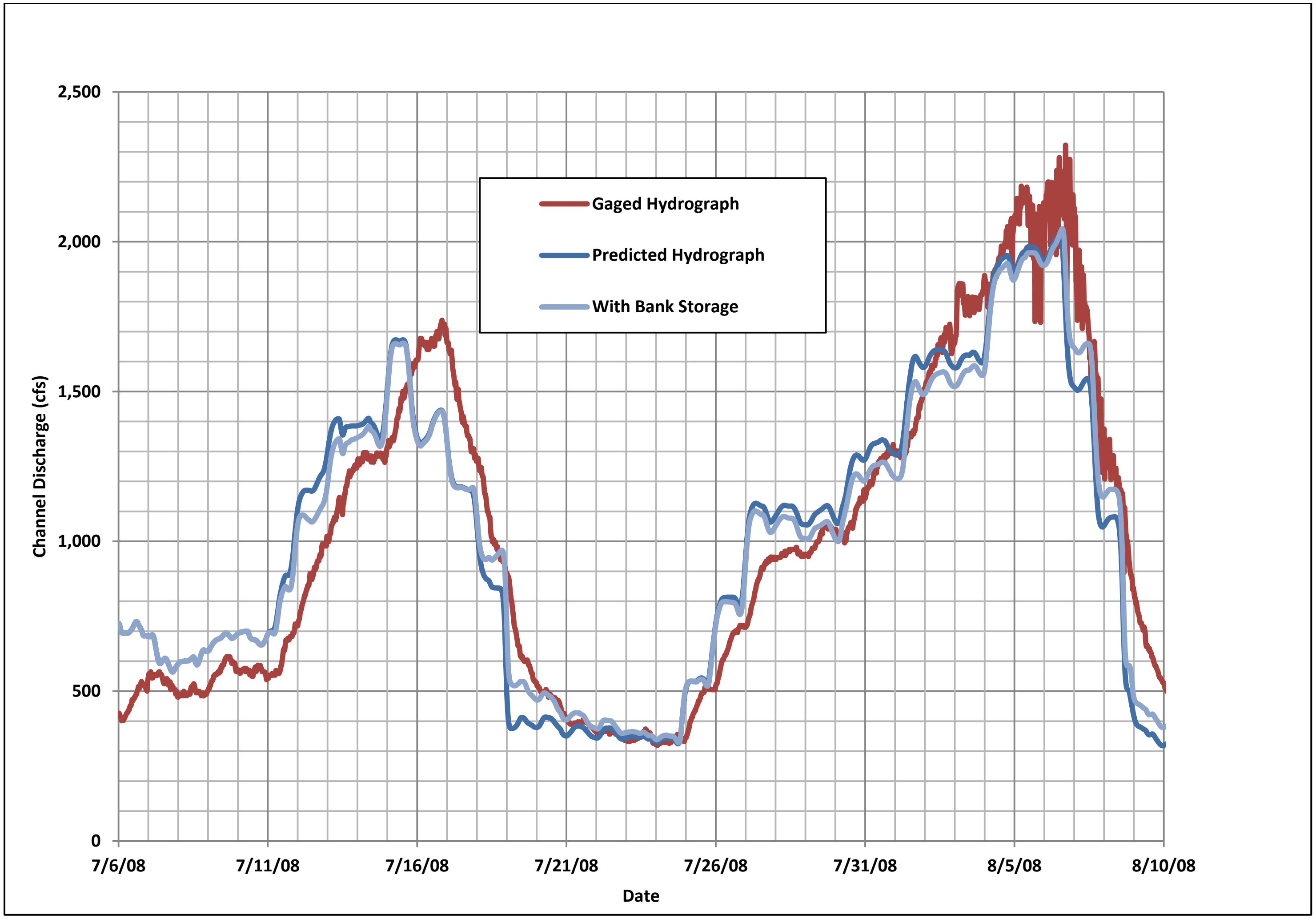


Figure 3.16: Predicted vs Gaged Hydrographs at North Platte River near Sutherland- Bank Storage Effects Sensitivity Analysis.

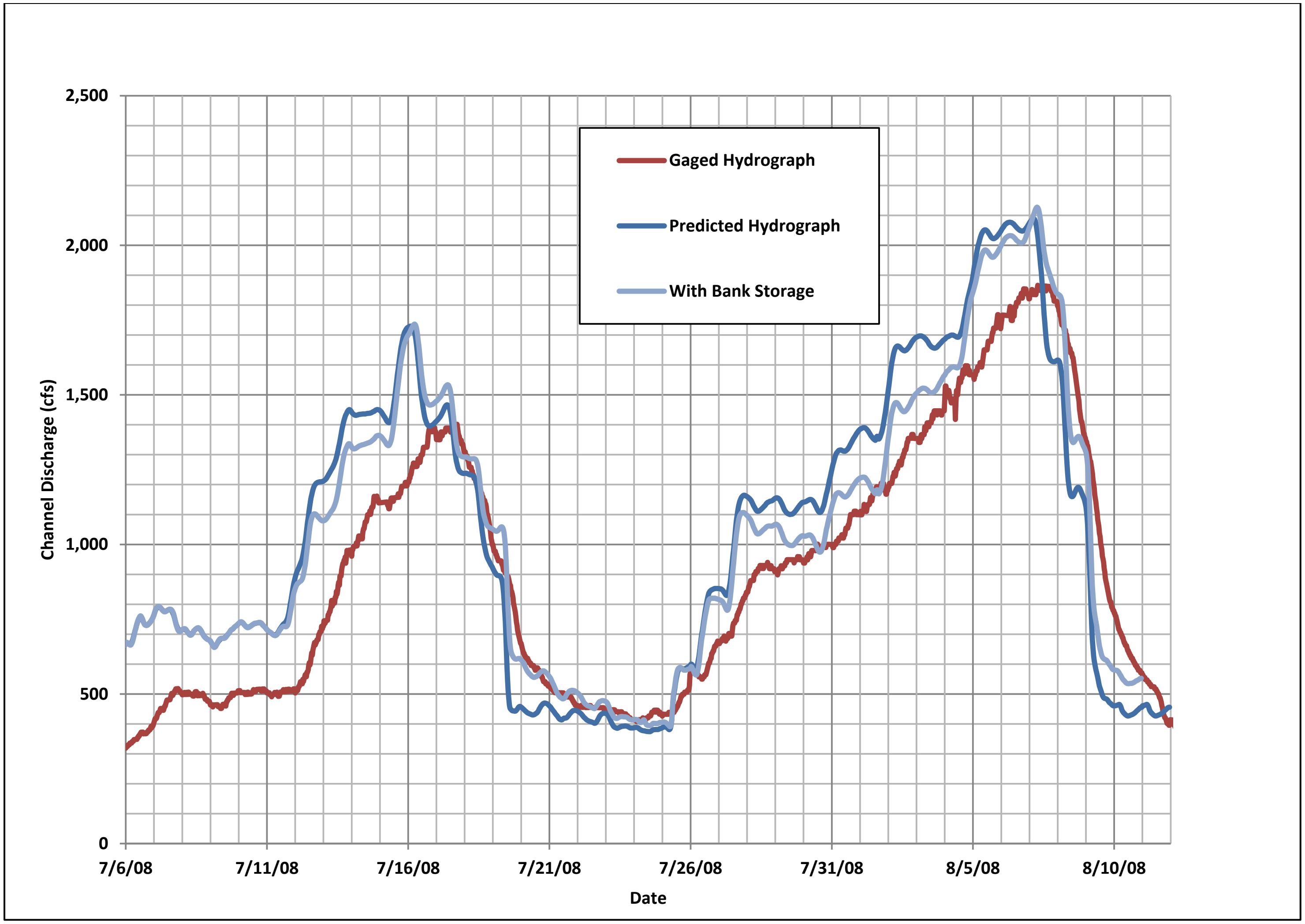


Figure 3.17: Predicted vs Gaged Hydrographs at North Platte River at North Platte- Bank Storage Effects Sensitivity Analysis.

4. References

Arcement, G.K. and Schneider, V.R., 1989. Guide for Selecting Manning's Roughness Coefficients for Natural Channels and Floodplains. U.S. Geological Survey Water Supply Paper 2339.

Barnes, H.H., 1967. Roughness characteristics of natural channels. U.S. Geological Survey Water-Supply Paper 1849.

Bureau of Reclamation, 2006. Trends of Aggradation and Degradation Along the Central Platte River: 1985-2005. Report prepared by Holburn, E.R, Fotherby, L.M., Randle, T.J., Carlson, D.E. and reviewed by Ferrari, R.L., February, 180 p.

HDR, TetraTech, and The Flatwater Group, January 2011. Platte River Recovery Implementation Program 1-D Hydraulic Modeling Technical Report, Keystone Diversion to North Platte (River Mile 370 – River Mile 310) Technical Memorandum.

Hicks, D.M. and Mason, P.D., 1991. Roughness Characteristics of New Zealand Rivers. Water Resources Survey, DSIR Marine and Freshwater, Wellington, New Zealand.

Randle, T.J. and Samad, M.A., 2008. Platte River in Central Nebraska; Modeling of Pulse-Flow Release. Bureau of Reclamation Technical Report No. SRH-2008-2.67p.

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, 2009. HEC-GeoRAS, GIS Tools for Support of HEC-RAS using ArcGIS, Users Manual, Version 4.2.93, Hydrologic Engineering Center, Davis, California

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

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

Appendix A: Map Book of Modeled North Platte River System: Keystone to North Platte

Appendix B: Bank Storage Prediction Tool

Hydrograph Routing Prediction

The routing characteristics of the reach during short duration, high flow releases appear to be strongly affected by the gains and losses in flow associated with bank storage. The HEC-RAS software includes a relatively simple algorithm based on Darcy's Law that is intended to provide a means of accounting for the interaction between the surface and groundwater along the reach. The study team initially believed that this algorithm could be used for this project. Extensive testing early in the model development phase, however, demonstrated that the controlling processes in the study reach of the Platte River cannot be adequately accounted for using this simple algorithm, primarily because the level of the near-river groundwater table varies with time during the passage of the hydrograph, and the HEC-RAS software only allows the user to specify a constant groundwater table. After evaluating a range of potential methods for accounting for the groundwater interaction, it was determined that the most practical approach was to model the process implicitly by applying a boundary condition hydrograph that approximated the flow out of and back into the flows in the river following the general form of the relationship suggested by Randle and Samad (2008). As discussed above, distributed hydrographs representing the flow losses and gains from bank storage that had the same general shape as the Randle and Samad (2008) hydrographs were developed during model calibration for the 2009 event. The hydrographs were then used as the basis to develop an empirical relationship between the input hydrograph and the parameters that define the specific hydrograph shape. The conceptual bank storage hydrographs are defined by four magnitude parameters (Qa, Qc, k1 and k2) and four timing parameters (To, Ta, Tb and Tc) (Figure B.1). A series of rating curves were developed for the reaches bounded by the available gages that relate channel discharge to the average stage in the reach (FigureB.2). The values of these parameters were selected based on the set of values that best matched the observed behavior of the 2009 hydrograph in each reach (Table B.1). The selected coefficients reflect the transmissivity of the bank material and are assumed to be constant for a given antecedent condition. The timing parameters were then related to the corresponding points in the input hydrograph (Trise, Tpeak, Tfall and Tbase) by the following relationships:

$$To = (A + Trise)$$
$$Ta = To + B * (Tpeak - Trise)$$
$$Tb = A + Tfall + C * (Tfall - Tbase)$$
$$Tc = Tb + D * (Tfall - Tbase)$$

Table B.1: Bank storage parameter values that best matched the observed behavior of the 2009 hydrograph in each reach.*

Reach	k1	k2	A	B	C	D
Keystone to Sutherland	-0.850	-0.550	0.120	0.400	0.600	1.000
Sutherland to North Platte	-0.250	-0.250	0.000	0.950	0.000	0.050

*See text for explanation.

Bank Storage Hydrograph Prediction Tool

A Microsoft Excel spreadsheet was compiled to apply the above relationships to enable the user to develop the distributed hydrographs necessary to model future scenarios and further refine the empirical relationships as necessary. The Excel file consists of two spreadsheets corresponding to the segments of the model between successive gages. Each sheet contains a series of color-coded cells that contain the information necessary to calculate an approximate bank storage hydrograph for each reach (Figure B.3). The yellow cells define the averaged stage-discharge relationship for each reach. These cells are fixed geometric characteristics of the reach and should not be changed unless additional bathymetry and steady-state modeling indicates that the relationship should be updated. These cells have been locked in the current version of the tool. Grey cells indicate values determined from empirical analysis of the available calibration data, including the decay coefficients k1 and k2, the hydrograph timing variables To, Ta, Tb and Tc as well as the two rating curves defining the relationships between the magnitudes of Qa and Qc and change in river stage. Though not intended to vary, these values may be refined as more data become available, and are therefore, not locked in the current version of the tool. Additionally, a threshold discharge can be defined below which bank storage does not occur. Orange cells represent user input for the upstream boundary condition hydrograph, as well as the timing parameters Tr, Tp, Tf and Tb. Once the inputs are entered, the user clicks the "Execute" button and the calculated results will be written in the "Calculated Results" cells. The hydrograph produced by the Excel file represents an estimation of the total flow of water into and out of the banks within each reach. This hydrograph can be distributed across the various subreaches within the reach by applying a percentage of the total flow into each of the uniform lateral inflow boundary conditions as indicated in the HEC-RAS flow file.

Empirical Relationship Adjustment

At the time of this report, calibration data were limited to only the 2009 event. While the empirical parameters developed for this study are believed to be reasonable, the groundwater interaction behavior of the reach will likely vary with the specific hydrograph and antecedent conditions. As additional calibration data become available, particularly in the range of the anticipated SDHF's, these parameters should be checked and adjusted, as appropriate. The decay coefficients, k1 and k2, are likely affected by antecedent conditions that are wetter or drier than the 2009 event. Under drier conditions, one would expect flow into the bank to decay less quickly because the potential groundwater storage is likely larger. As more short duration high flow calibration events become available, the relationship between antecedent conditions and empirical parameters describing the bank storage response can be refined and implemented to improve the model's ability to predict short duration high flow release hydrographs.

Conditions for Incorporation of the Bank Storage Relationship

As seen in the 2009 event simulation, incorporating the volume fluxes resulting from bank storage is a vitally important part of predicting timing and shape of short duration high flow hydrographs with antecedent conditions similar to those observed in 2009. As the rise in hydrograph becomes more gradual and events have greater volume, the rate at which water enters the near-bank storage and the total volume of bank storage anticipated would impact shape and timing of the overall event less. Additionally, a wet-weather event upstream can often change the antecedent soil conditions in the modeled reach, impacting the volume of bank storage. Sensitivity analyses indicated that the inclusion of bank storage effects can improve the model predictions of longer-duration events such as the 2008 event, but not as drastically as the 2009 event. For the modeled events, bank storage was only considered for events initiating from a sudden change of discharge (e.g. opening gates at Lake McCounaghy Dam). According to data from the North Platte River at Keystone gage, flow increased from 0 cfs to nearly 1300 cfs in roughly 1.5 days. The 2008 and 2001 events occurred due to wet-weather events in the basin and rising hydrograph limbs lasting several days. Additionally, the 2008 and 2001 events were likely subject to larger influx of ungaged flow, which might mask the effect bank storage would have on hydrograph shape. For these reasons, bank storage effects were not considered in the 2008 and 2001 event simulations. Based on this report, it is suggested that bank storage effects be considered only for pulse hydrographs that have an increase in discharge over no more than about four days. As more calibration hydrographs become available it may be possible to refine and improve the bank storage parameters used in these reaches. Additionally, more calibration events may help determine the characteristics of hydrographs most affected by bank storage.

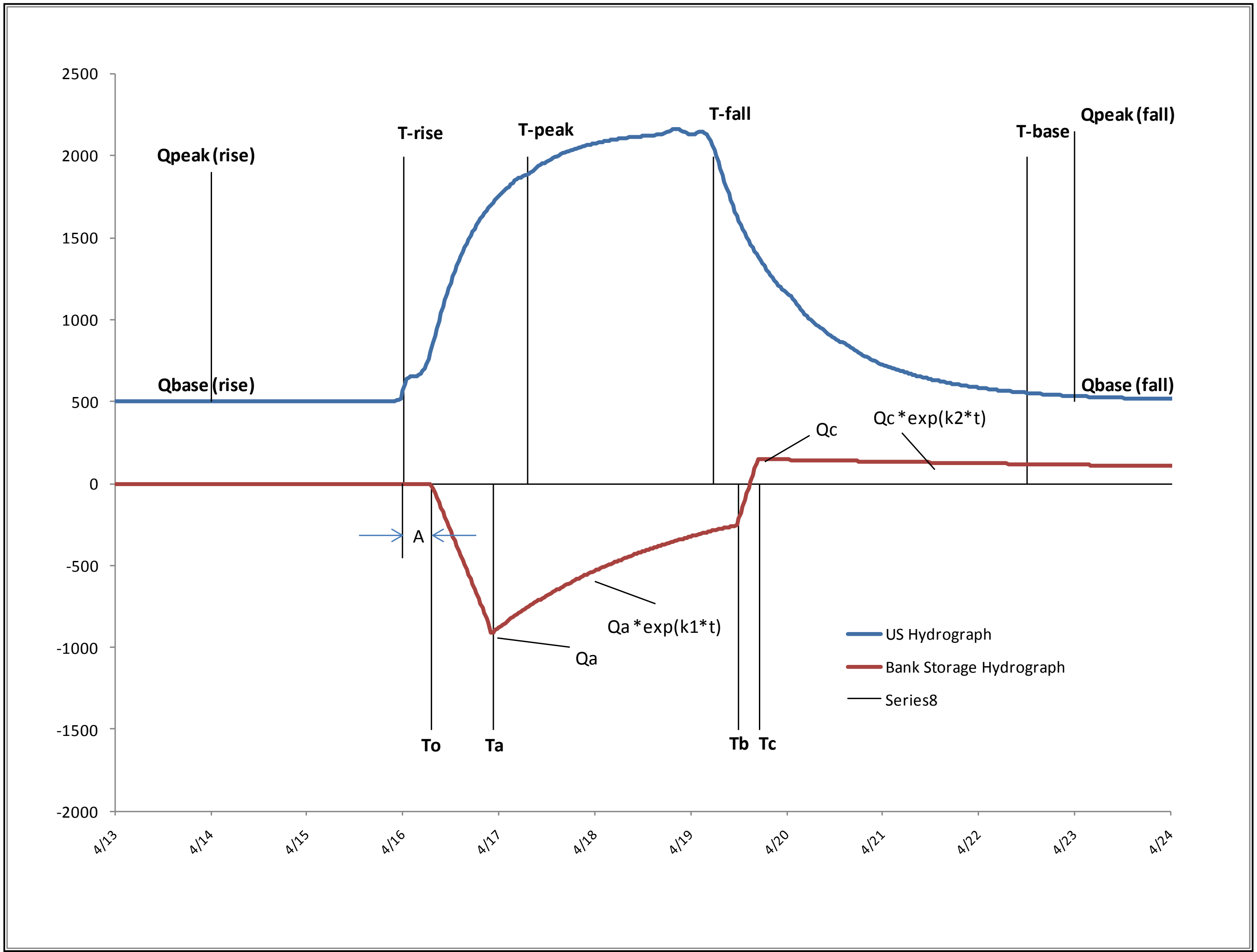


Figure B.1: Example river flow and bank storage hydrographs and the parameters used in to describe bank storage relationship.

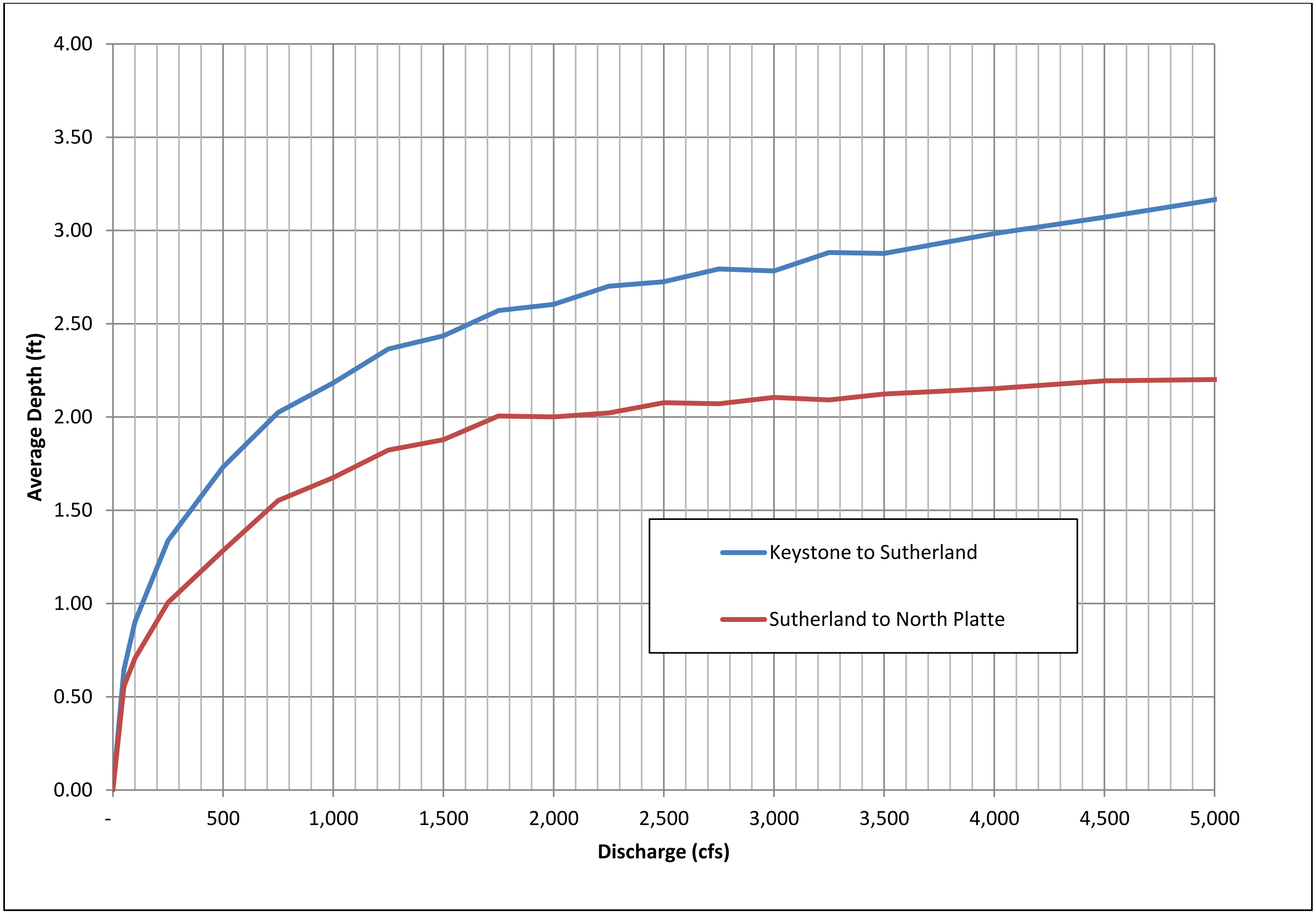


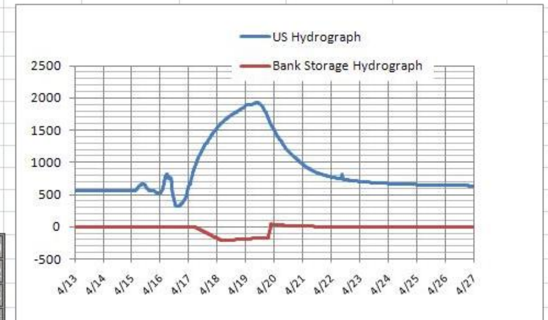
Figure B.2: Reach-averaged stage versus discharge rating curves for each of the bank storage reaches

	Input Conditions	
	Timing	Magnitude
Qbase (rise)	4/16/2009 23:00	553
Qpeak (rise)	4/19/2009 5:30	1900
Qpeak (fall)	4/19/2009 12:00	1903
Qbase (fall)	4/21/2009 11:00	850

	Imposed Conditions
K1	-0.158
K2	-1.500
To	4/17/2009 4:45
Ta	4/18/2009 4:11
Tb	4/19/2009 19:10
Tc	4/19/2009 21:59
Threshold (cfs)	-

User Input
Bank Storage Parameters
Fixed Geometric Relationship
Calculated Result

Execute



Date	US Hydrograph	Bank Storage Hydrograph
4/1/2009 0:00	1000	0
4/1/2009 0:30	1000	0
4/1/2009 1:00	1000	0
4/1/2009 1:30	1000	0
4/1/2009 2:00	1000	0
4/1/2009 2:30	1000	0
4/1/2009 3:00	1000	0
4/1/2009 3:30	1000	0
4/1/2009 4:00	1000	0
4/1/2009 4:30	1000	0
4/1/2009 5:00	1000	0
4/1/2009 5:30	1000	0
4/1/2009 6:00	1000	0
4/1/2009 6:30	1000	0
4/1/2009 7:00	1000	0
4/1/2009 7:30	1000	0
4/1/2009 8:00	1000	0
4/1/2009 8:30	1000	0
4/1/2009 9:00	1000	0
4/1/2009 9:30	1000	0
4/1/2009 10:00	1000	0
4/1/2009 10:30	1000	0
4/1/2009 11:00	1000	0
4/1/2009 11:30	1000	0
4/1/2009 12:00	1000	0

Rating Curve	
Q (cfs)	Stage (ft)
100	0.77
200	1.16
500	1.76
750	2.11
1,000	2.37
1,500	2.78
2,000	3.15
2,500	3.47
3,000	3.78
3,500	4.07
4,000	4.33
4,500	4.58
5,000	4.81
6,000	5.20
7,000	5.56
8,000	5.88
9,000	6.18
10,000	6.46
12,000	6.96
15,000	7.61

Stage Change (ft)	Qa (cfs)	Qc (cfs)
0	0	0
0.25	-43	12
0.5	-87	23
0.75	-130	35
1	-174	46
1.25	-217	58
1.5	-261	69
1.75	-304	81
2	-348	93
2.25	-391	104
2.5	-435	116
2.75	-478	127
3	-521	139
3.25	-565	150
3.5	-608	162
3.75	-652	174
4	-695	185
4.25	-739	197
4.5	-782	208
4.75	-826	220
5	-869	231
5.25	-913	243
5.5	-956	255
5.75	-999	266

Figure B.3: Example of the bank storage tool that was developed to calculate the bank storage hydrograph for each reach.