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Managing Water in the West

Platte River in Central Nebraska Modeling of Pulse-Flow Release

Technical Report No. SRH-2008-2

by

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Bureau of Reclamation
Sedimentation and River Hydraulics Group
Technical Service Center
Denver Colorado

August 2008

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8/15/2008

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Table of Contents

List of Figures

List of Tables

ABSTRACT

Pulse-flow releases from Lake McConaughy, and other downstream reservoirs, are being considered to improve habitat conditions for threatened and endangered migratory birds along the Platte River in central Nebraska. The pulse-flow releases will be of short duration (3 to 5 days) and within the bankfull capacity of the river channel. These flows are expected to maintain open views for birds by scouring seedling vegetation from mechanically cleared areas of the river channel and to form sand bars for bird roosting and nesting.

An unsteady application of the HEC-RAS river hydraulics model was used to route the pulse-flow release along two reaches of the Platte River. The upstream reach is 71 river miles long between the cities of North Platte and Overton, Nebraska. The downstream reach is 82 river miles long between Overton and Chapman, Nebraska. The one-dimensional HEC-RAS model predicts the magnitude and duration of peak discharge and water-surface elevation as the discharge waves are routed downstream.

For discharge waves within the bankfull-channel capacity, the calibration process found that the HEC-RAS model must be supplemented with a bank storage model to accurately replicate the measured flow hydrographs of the Platte River. Therefore, the HEC-RAS model was supplemented with a internally-developed bank storage model, which is based on results from analytical and numerical groundwater models. The bank storage model simulates the flow of water from the river into the river banks, during the rising limb of the discharge hydrograph, and the release of bank storage water back to the river, during the falling limb of the hydrograph. The combined model procedure has been calibrated and verified against measured fluctuating-flow hydrographs from Overton, Nebraska (river mile 239.3) to Kearney, Nebraska (river mile 215.0) and continuing downstream to Grand Island, Nebraska (river mile 167.90). No data were available to calibrate the combined model in the upstream reach between North Platte and Overton, Nebraska.

1.0 Introduction

The headwaters of the North and South Platte Rivers both originate in Colorado. The South Platte River flows through Colorado while the North Platte River flows through Wyoming to Nebraska. These two tributary rivers join in western Nebraska to form the Platte River (Figure 1). The Platte River, flowing through central Nebraska (Figure 2), provides habitat for threatened and endangered migratory birds including whooping crane, interior least tern, and piping plover (U.S. Department of the Interior, 2006).

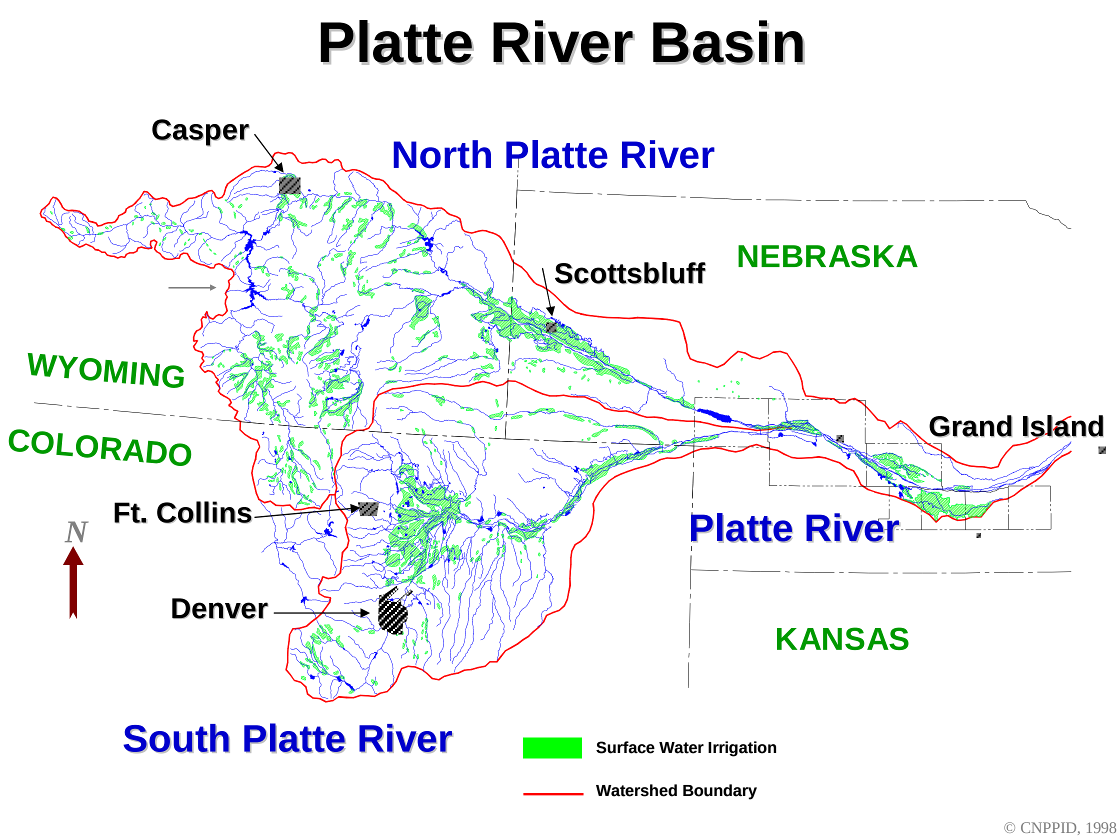

Figure 1. The Platte River watershed, upstream from Grand Island, Nebraska, includes portions of Colorado, Wyoming, and Nebraska.

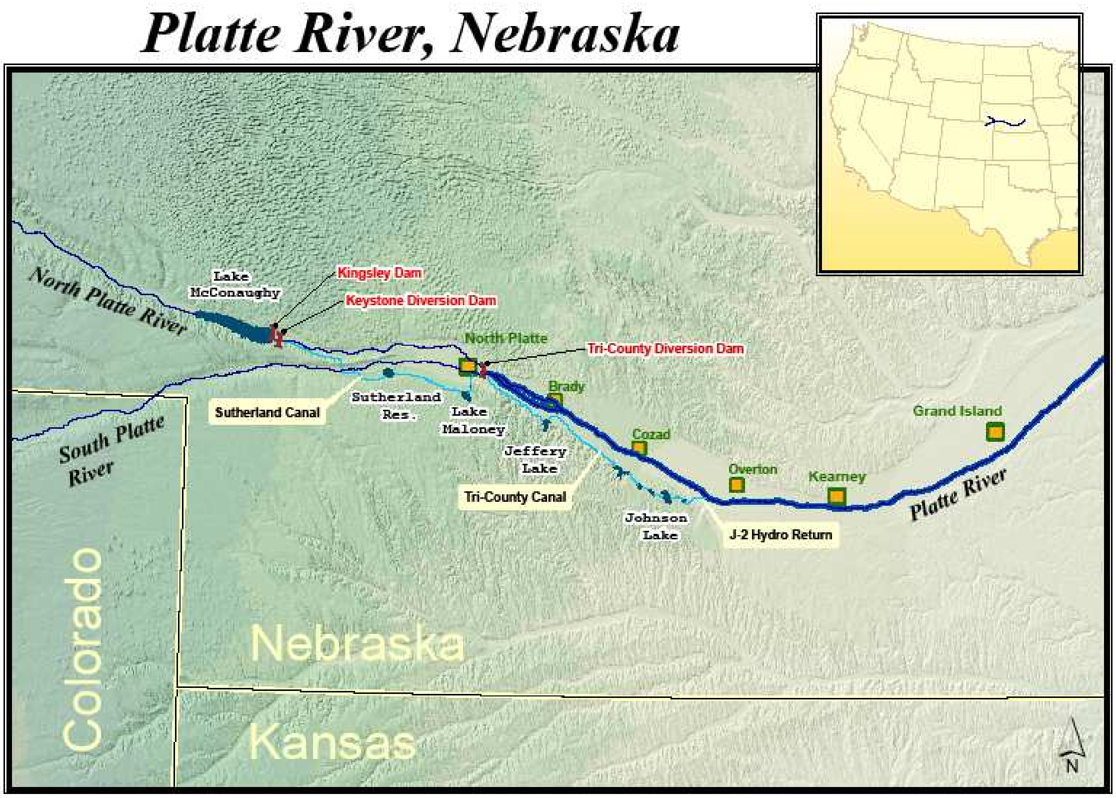

Figure 2. View of the North and South Platte River confluence and the Platte River through central Nebraska between North Platte and Grand Island, Nebraska. Orange squares represent city locations that are near stream-flow gaging stations.

Pulse-flow releases from Lake McConaughy (behind Kingsley Dam), and other downstream reservoirs, are being considered to improve habitat conditions for threatened and endangered migratory birds along the Platte River between Lexington (upstream from Overton) and Grand Island, Nebraska. These flows are expected to maintain open views for birds by scouring seedling vegetation from mechanically cleared areas of the river channel and to form sand bars for bird roosting and nesting. The pulse-flow releases will be of short duration (3 to 5 days) and within the bankfull capacity of the river channel.

1.1 Model Objectives

When the pulse flow is released from reservoirs, the duration of the peak flow needs to be long enough so that the peak flow rate does not significantly attenuate before reaching Grand Island. A predictive model was needed to simulate the downstream movement and attenuation of discharge waves from these pulse-flow releases. The objectives of the predictive models are listed below:

- Predict the travel time and attenuation of discharge waves along the Platte River.

- Predict the hydraulic properties at model cross sections along the river channel as a function of time.

The predictive model will provide a useful planning tool for the design and monitoring of pulse-flow releases. By applying the HEC-RAS and bank storage models to a range of peak-flow durations at an upstream location, the optimum peak duration can be determined so that the peak discharge will be achieved at the stream gage near Grand Island, Nebraska.

1.2 Unsteady Flow Examples

Unsteady flow releases from the Tri-County Canal, related to hydro-power generation, and from the Platte River upstream provide good examples of discharge waves that are within the bankfull-channel capacity (Figure 3 and Figure 4). These short duration discharge waves attenuated substantially in both peak and duration as they traveled downstream. The peak discharge of some waves actually increased with distance downstream due to storm runoff entering the Platte River between gaging stations.

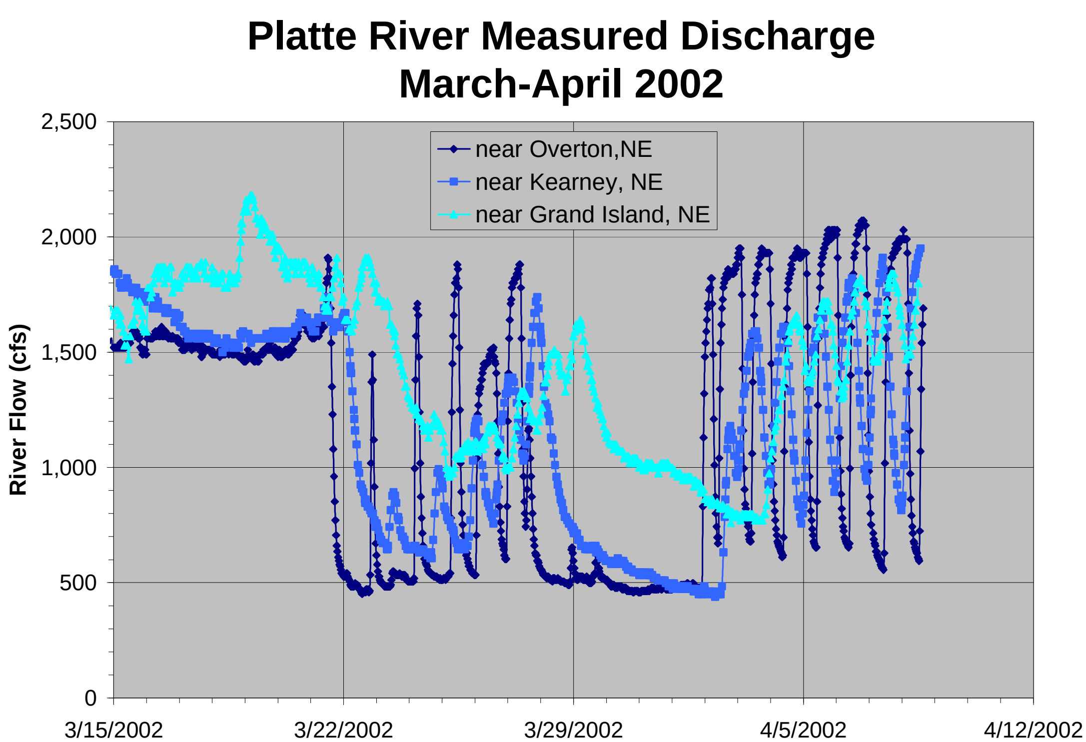

Figure 3. Measured Platte River discharge hydrographs at the USGS stream gages near Overton, Kearney, and Grand Island, Nebraska during March and April 2002.

The amount of attenuation varies depending on the duration of the upstream peak-discharge wave. In Figure 4, the duration of the first discharge wave at the Overton gage is short enough that there is substantial attenuation of this wave at the Kearney gage and by the Grand Island gage; the first discharge wave has combined with the second discharge wave. All the subsequent discharge waves are of long enough duration at the Overton gage that they can be tracked all the way downstream to the Grand Island gage. The peak discharges did not attenuate much between the stream gages near Kearney and Grand Island, Nebraska because of stream flow gains within this reach during April 2002.

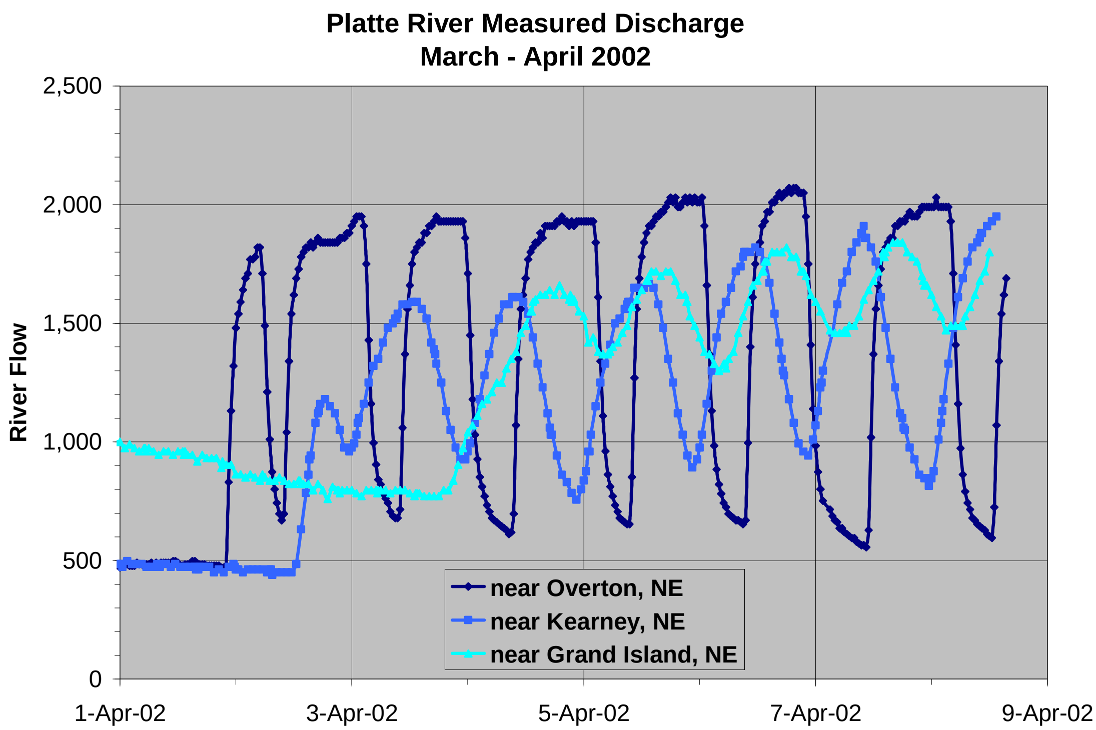

Figure 4. Measured Platte River discharge hydrographs at the USGS stream gages near Overton, Kearney, and Grand Island, Nebraska during the period April 1-9, 2002.

2.0 Unsteady Flow Modeling Strategy

At the time of this work, no unsteady, open-channel-flow models were available to route a discharge hydrograph through a river channel that also simulate the effects of bank storage and groundwater flow. The strategy for this effort was to add a new bank storage modeling procedure to an existing unsteady flow model. There are several unsteady flow models to choose from such as DAMBRK[1] (Fread, 1988), SRH-1D (Huang and Greimann, 2007), and HEC-RAS (U. S Army Corps of Engineers, 2005). The HEC-RAS model was chosen to route the hydrographs through the river channel because this model is widely used and accepted and the model has a very nice graphical user interface. Presently, the HEC-RAS model by itself does not consider the effects of bank storage.

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[1] BOSS International, BOSS DAMBRK Basic Version 3.5 Copyright 1988-94.

Analytical and numerical groundwater models were used to understand the bank storage flow process. An empirical, bank storage model procedure was then developed based on the parameterized results of the analytical and numerical groundwater models. The empirical model simulates the effects of bank storage and groundwater flow simultaneously with the unsteady, open-channel flow model.

3.0 Unsteady, Open-Channel Flow Model

The basic equations of unsteady, open channel flow are briefly described, followed by the application of the HEC-RAS model to the Platte River channel.

3.1 Unsteady Flow Equations

The basic equations for unsteady open channel flow include the equations for continuity (Eq. 1) and momentum (Eq. 2).

$$\frac{\partial Q}{\partial x} + \frac{\partial A}{\partial t} = 0$$

Eq. 1

Where Q = water discharge [L^3/T],
x = longitudinal distance along river channel [L],
A = cross-sectional area of river channel [L^2],
t = time [T].

$$S_f = S_o - \frac{\partial y}{\partial x} - \frac{1}{gA}\frac{\partial \left(Q^2 / A \right)}{\partial x} - \frac{1}{\partial A}\frac{\partial Q}{\partial t}$$

Eq. 2

Where S_f = friction slope,
S_o = channel bottom slope,
y = water surface elevation [L], and
g = acceleration of gravity [L/T^2].

Steady, uniform flow is defined by the first two slope terms of Eq. 2 (**Figure 5**). If the next two terms are added, the equation describes steady, non-uniform flow. Inclusion of the last term describes unsteady, non-uniform flow.

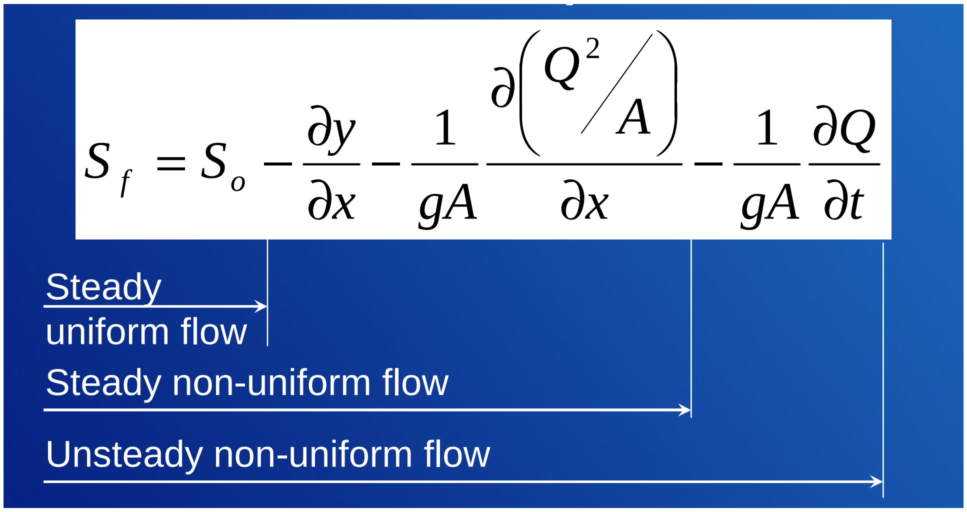

Figure 5. The various terms of the momentum equation account for steady-uniform flow, non-uniform flow, and unsteady flow.

The HEC-RAS model utilizes a linearized, implicit, finite difference scheme to solve the continuity and momentum equations (U.S. Army Corps of Engineers, 2005).

3.2 Platte River HEC-RAS Model Development

HEC-RAS models were assembled for two reaches of the Platte River (Figure 6):

1. From immediately downstream from the Tri-County Diversion Dam, near North Platte, Nebraska, downstream to the USGS stream gage near Overton, Nebraska (river miles 310.2 to 239.3).

2. From the USGS stream gage near Overton, Nebraska downstream to the highway Bridge near Chapman, Nebraska (river miles 239.3 to 157.2).

Figure 6. The confluence of the North and South Platte Rivers is at the Tri-County Diversion Dam. The Platte River begins at the diversion dam and continues about 160 river miles downstream to Grand Island and Chapman, Nebraska. The highway bridge near Chapman, Nebraska is about 10 river miles downstream from the USGS stream gage near Grand Island, Nebraska.

The data requirements for modeling unsteady open channel flow are listed below:

- river cross sections describing channel geometry
- Manning's *n* roughness coefficients for each channel segment between two cross sections
- initial water surface for each cross section at the beginning of the model simulation
- upstream boundary discharge hydrograph
- downstream boundary water surface elevation hydrograph or stage-discharge rating curve specified by the assumption of normal depth

Cross sections of the Platte River channel were measured during the following years:

- 1989: initial survey of 90 cross sections between North Platte and Grand Island.
- 1998: repeat survey of 30 cross sections between North Platte and Grand Island.
- 2002: repeat survey of 30 cross sections between Lexington and Overton

Some cross sections have been only measured once while others have been measured multiple times. Cross sections measured during 1989 were used to develop the HEC-RAS model for the Platte River reach between North Platte and Overton, Nebraska (Table 1). Cross sections measured during the most recent survey were used to develop the HEC-RAS model for the Platte River reach between Overton and Grand Island, Nebraska. Repeat surveys of selected cross section in 1998 between North Platte and Grand Island, Nebraska revealed only minor channel changes, except for degradation of 4 to 1 feet in the reach between Overton and Kearney, Nebraska. Cross sections in this degraded reach were resurveyed in 2002.

The Platte River has multiple side channels in most reaches, which are often separated by vegetated islands (Murphy et al., 2004, Holburn et al., 2006, and U.S. Department of the Interior, 2006). The cross section surveys included the main channel and all principal side channels. The cross section alignment for all channels was field oriented to be perpendicular to the flow in the main channel. However, different water surface elevations were typically measured in the various multiple side channels of the cross section survey.

The model of the upstream reach includes the multiple channels of the Platte River, but not the channel south of Jeffery Island, which is between the Johnson-2 (J-2) hydro return and the stream gage near Overton. This south channel was not included because a dike, at the upstream end of Jeffery Island, blocks flow to the south channel. Clear-water releases from the Johnson-2 hydro return have caused the south channel to incise several feet (Murphy et al., 2004). The dike across the upstream entrance to the side channel causes all the Platte River to flow around the north side of Jeffery Island. If this dike were not present, the incised south channel would capture all the Platte River flow from the north channel. The dike has prevented head-cut erosion from progressing upstream from the south channel.

HEC-RAS is a one-dimensional model and can only simulate one water surface elevation at a given cross section and river discharge. Therefore, elevation leveling adjustments had to be applied to each separate side channel so that the average water-surface elevation of each side channel matched the measured water-surface elevation of the main channel. The leveling elevation adjustment was used to raise or lower all channel bank and bottom elevations of a given side channel. The leveling elevation adjustment was computed based on the difference in measured water-surface elevations between the side channel and main channel. If the water-surface elevation in a side channel was higher than the water-surface elevation in the main channel, then the side channel elevations were lowered by the leveling elevation adjustment. If the water-surface elevation in a side channel was lower than the water-surface elevation in main channel, then the side channel elevations were raised by the leveling elevation adjustment. An example of this procedure is shown in Figure 7, Figure 8, and Figure 9.

For a few cross sections, the measured water surface elevations of the various sub-channels had a linear slope in the cross-stream or lateral direction. For these cross sections, a linear slope adjustment was made so that water surface elevations were level.

Table 1. Summary of Platte River Cross-Section Surveys used in HECRAS Modeling.

Platte River between Grand Island and Overton, Nebraska

River Mile	Distinct Platte River Channel Survey Lines	Date of Survey 1	Date of Survey 2	Date of Survey 3	Datum Adjustment (feet) of 1989 survey to 1988 NAVD	Channel Survey Elevation Adjustments (feet) for a Level Water Surface Elevation	Special Notes
157.2	MC	1989			0.68		
162.2	MC	1989			0.69		
165.8	MC	1989			0.71		
165.85	MC	1989			0.71		
165.9	MC	1989			0.71		
166.9	MC	1989			0.71		
167.85	MC	1989	10/13/98				
167.9	MC	1989			0.71		
168.75	North	1989	10/13/98		0.99 for South Channel		
170.3	N, M, S	1989	10/13/98		0.82 for South Channel		
172.1	South	1989			0.73	Entire channel adjusted by +4.50	
172.4	South	1989			0.73		
172.6	N, NM, SM, S	1989	10/12/98		0.78	4 separate leveling adjustments	
172.7	South	1989			0.73		
172.8	South	1989			0.73	14.05 feet (11.2 feet for River slope and 2.85 feet for W.S Leveling)	
174.6	N, NM, SM, S	1989			0.73		
174.65	South	1989			0.73		
175.2	N, M, S	1989	10/13/98			Multiple leveling adjustments	
175.5	N, M, S	1989			0.74		

177.3	N, M, S	7/11/85	5/31/00		0.74		
178.4	N, M, S	7/10/85	6/2/00		0.75		
180.1	South	1989			0.75		
180.3	N & S	1989			0.75		
181.85	South	1989			0.76		
181.9	South	1989	10/12/98		0.42		
182.1	N & S	1989			0.76		
183.2	N & S	1989			0.76	South channel adjusted by -0.57 feet	
187.0	South	1989			0.51 for SM channel	North channel at RM 187.3 adjusted by -0.83 feet (+1.17 feet for water surface elevation and -2.0 feet because it was moved 0.3 mile downstream).	Survey data from the north channel at RM 187.3 were combined with survey data from the south channel at RM 187.0
187.3	N & S	1989	10/12/98			North channel adjusted by +1.17 feet	
187.4	N & S	1989			0.51 for S. Channel	North channel adjusted by -0.35 feet	
188.3	Multiple	1989			0.78		
189.3	Multiple	1989			0.78		
193.9	Multiple	1989			0.80		
194.9	Multiple	1989			0.80		
195.8	Multiple	1989	10/11/98				
197.4	Multiple	1989			0.81		
199.5	Multiple	1989	3/14/02				
201.2	N & S	1989					
202.2	N, M, S	1989	10/11/98			North channel adjusted by +3.29 feet	
203.3	N & S	1989	3/14/02				
206.6	South	7/17/85	10/6/98	2002			
207.9	N & S	1989	10/7/98			North channel adjusted by +3.5 feet	
208.6	N & S	1989		2002		North channel adjusted by +5.19 feet	
209.8	N & S	1989	10/11/98			North channel adjusted by +1.80 feet	
210.6	N & S	1989	3/16/02			North channel adjusted by +2.47 feet	
219.8	MC	1989	2002				
222.0	N & S	1989	2002			North channel adjusted by +0.74 feet	
224.0	MC	1989	10/8/98				

224.3	MC	1989	2002				
225.1	N & S	1989	3/18/02		0.89 for S. Channel	South channel adjusted by +0.84 feet	
228.7	MC	1989	3/19/02				
230.0	MC	1989	2002				
230.8	Multiple	1989	10/8/98	3/19/02			
231.5	Multiple	1989			0.91		
233.8	Multiple	1989	11/18/98	3/21/02			
237.2	Multiple	1989	11/18/98	3/21/02			
239.0	Multiple	1989	3/22/02				
239.3	MC	1989	10/8/98	3/22/02	0.73		**USGS stream gage near Overton, Nebraska**

Platte River between Overton and North Platte, Nebraska							
River Mile	Distinct Platte River Channel Survey Lines	Date of Survey 1	Date of Survey 2	Date of Survey 3	Datum Adjustment (feet) of 1989 survey to 1988 NAVD	Channel Survey Elevation Adjustments (feet) for a Level Water Surface Elevation	Special Notes
241.1	North				0.75	Survey at RM 239.9 was moved to represent RM 241.1	
244.0	North	1989	3/23/02		0.90		
246.5	North	1989	3/24/02		0.88	Left channel adjusted by -0.43 feet	
247.8	Multiple	2002			0.89	Linear cross-slope adjustments north and south of the main channel	
249.8	MC	1989	3/24/02		0.01		
250.5	Multiple	1989	10/10/98	3/13/02	0.94		
251.6	MC	1989	10/10/98		1.05		
254.4	MC	1989			0.91		
258.0	N & S	1989	10/10/98		0.74 for N, 0.95 for S. channel		
258.3	Multiple	1989			0.93	Linear cross-slope adjustments on right side of channel	
261.7	Multiple	1989			0.95		
266.7	MC	1989			0.98		
267.9	N & S	1989			0.98	Left side of south channel adjusted by -1.56 feet	
269.9	N & S	1989			0.99	South channel adjusted by +4.55 feet	
277.3	N & S	1989			1.04	North channel adjusted by -2.49 feet	
281.8	MC	1989			1.06	Linear cross-slope adjustment	
284.9	MC	1989			1.08	Linear cross-slope adjustment	
287.7	MC	1989			1.10		
288.1	N & S	1989			1.11	South channel adjusted by +2.24 feet	

297.0	MC	1989	10/9/98		1.23		
298.5	MC	1989			1.18		
302.0	MC	1989			1.21		
304.0	MC	1989	10/9/98		1.55		
305.4	MC	1989			1.23		
307.5	MC	1989			1.25		
309.0	MC	1989			1.26		
310.0	MC	1989	10/9/98		1.08		
310.2	MC	1989	10/9/98		1.12		
Legend:	MC = Main Channel (one survey line) Multiple = Multiple channels (one survey line) North = North Channel survey line					N & S = Distinct survey lines of the North and South Channels N, M, S = Distinct survey lines of the North, Middle, and South Channels N, NM, SM, S = Distinct survey lines of the North, North Middle, South Middle, and South Channels	

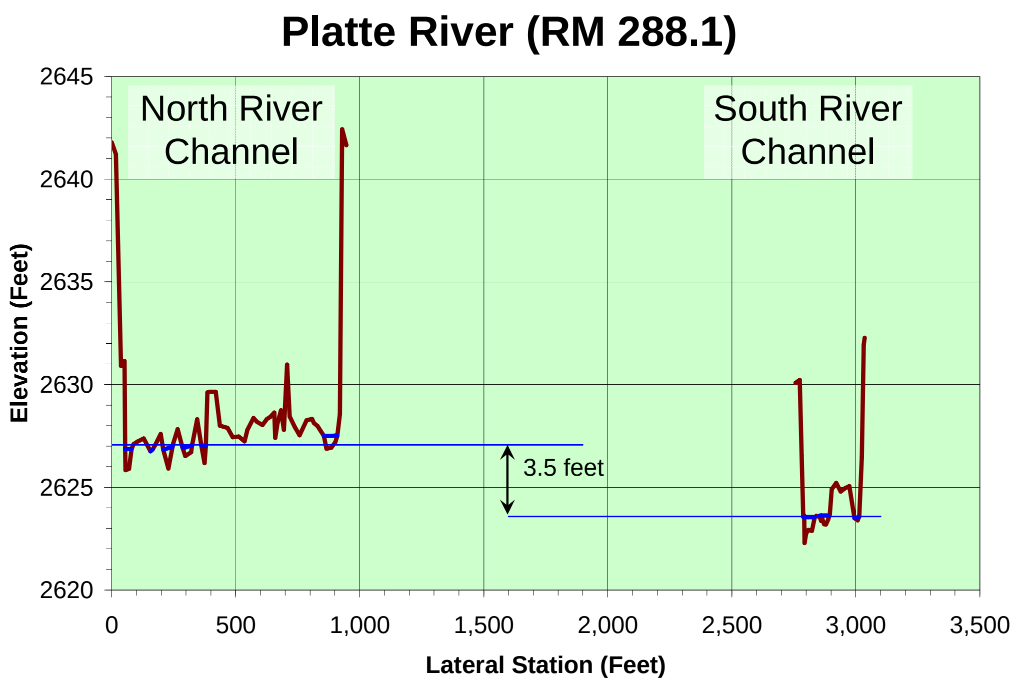

Figure 7. The difference in water surface elevations of the north and south channels of the Platte River at river mile 288.1 is due to incision of the south channel.

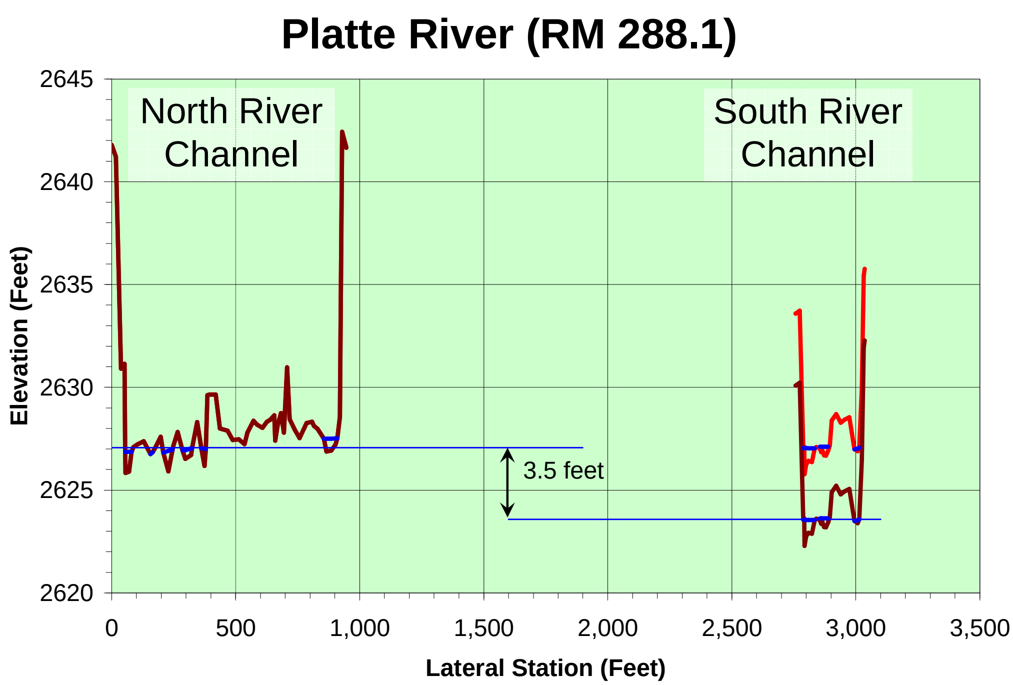

Figure 8. Platte River south channel at river mile 288.1 was raised 3.5 feet to account for the difference in water surface elevations between the north and south channels.

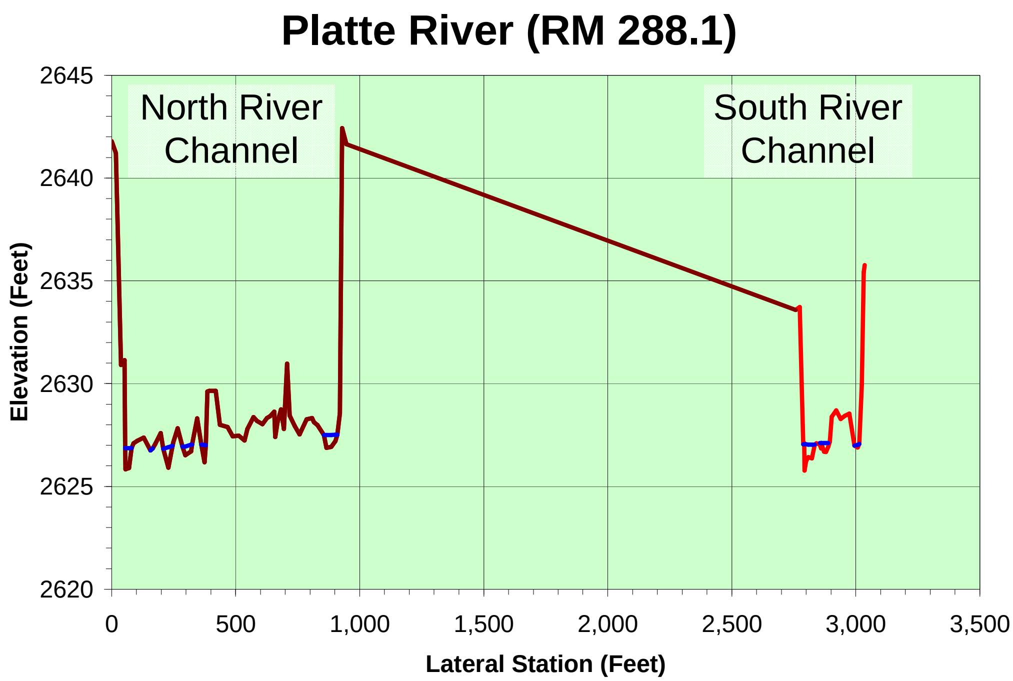

Figure 9. Platte River north and south channels at river mile 288.1 after a elevation leveling adjustment of 3.5 feet was applied to the south channel.

The longitudinal spacing between surveyed cross sections varied between 0.05 and 9.2 miles with an average spacing of 1.8 miles. This average longitudinal spacing is too large for an unsteady flow model to have a stable solution; therefore, the HEC-RAS model was used to interpolate river cross sections every 500 feet.

The initial water surface elevations for each cross section (at the beginning of the unsteady model simulation) were computed by the HEC-RAS model assuming that the river flow is initially steady and equal to the first flow of the modeled hydrograph.

The downstream boundary water surface elevation was computed by the HEC-RAS model assuming normal depth with an energy-grade line slope of 0.0012, which is the average channel slope based in the 1989 cross section surveys.

The Manning's n roughness coefficient for the river channels was calibrated by comparing the simulated and measured travel times of discharge waves for the reach between Overton and Grand Island, Nebraska. The measured discharge hydrograph (during the period April 1-9, 2002) was used in the calibration, which had peak discharges between 1,600 and 1,700 ft^3/s (Figure 10). The measured hydrograph at the stream gage near Overton, Nebraska was routed 71 miles downstream by the HEC-RAS model to the stream gage near Grand Island, Nebraska. Trial roughness coefficients in the HEC-RAS model ranged from 0.020 to 0.050 (Figure 11). A roughness coefficient of 0.024 for the river channel was found to provide the best match in the discharge wave travel time (determined by the start of rise in the discharge wave). The roughness coefficient of 0.024 is reasonable for a sand-bed channel. This roughness coefficient was calibrated by comparing the start of rise in the simulated discharge wave model with measurements at the USGS stream gage near Kearney, Nebraska (Figure 12). Peak discharge and hydrograph shape could not be used in the roughness calibration because the bank storage model is needed to fully simulate the discharge wave attenuation. A Manning's *n* roughness coefficient of 0.07 was used for the channel over banks and high river islands based on values reported for the FEMA flood insurance study (Peter Murphy, oral communication, 2000). These areas were not calibrated, but were also generally not inundated during the model runs described in this report.

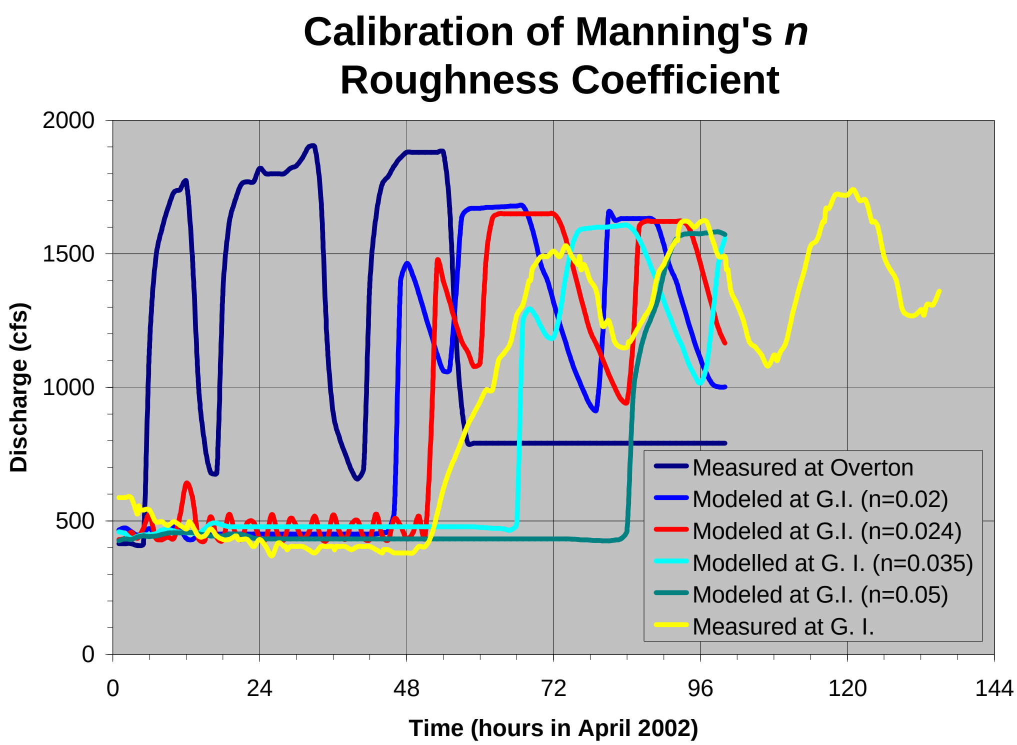

Figure 10. Comparison of measured and modeled hydrographs near Grand Island, Nebraska for various Manning's *n* roughness coefficients applied to a fluctuating flow period during April 1-9, 2002.

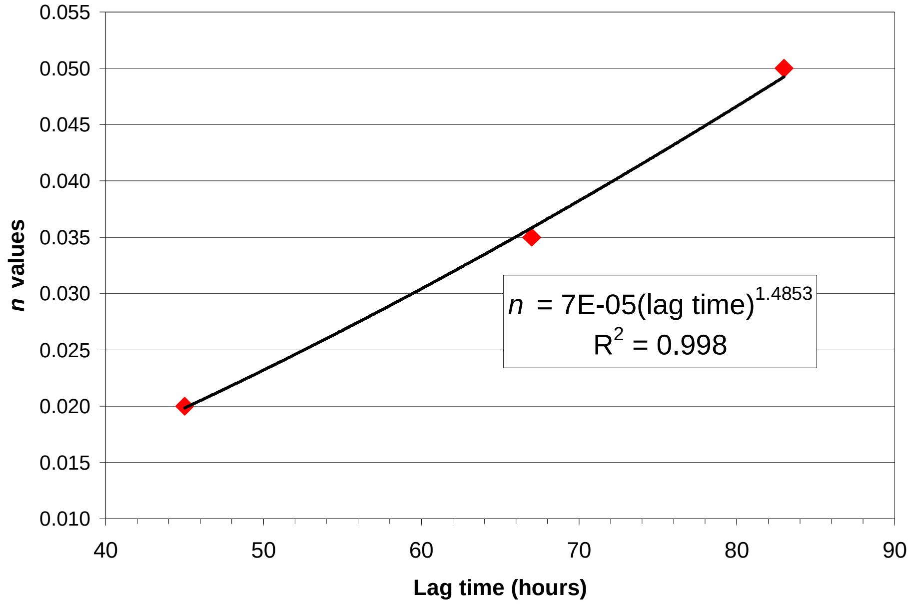

Figure 11. Channel roughness coefficient as a function of the discharge wave travel time (lag time) at the USGS stream gage near Grand Island, Nebraska.

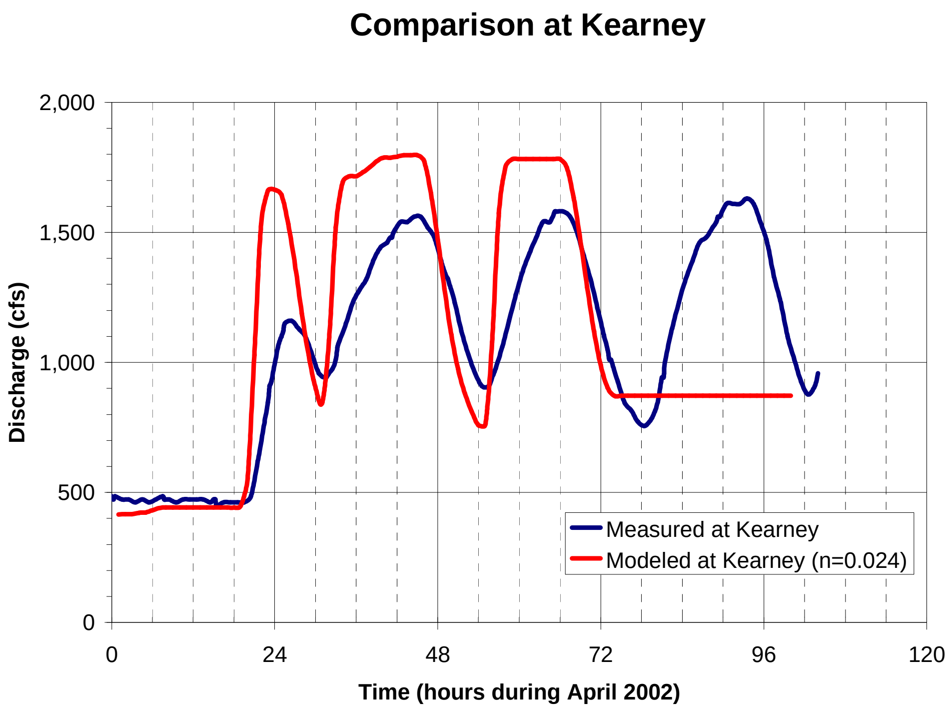

Figure 12. Comparison of measured and modeled hydrographs near Kearney, Nebraska applied to a fluctuating flow period during March and April 2002. The calibrated roughness coefficient of 0.024 was verified because the predicted start of rise matches the measured start of rise.

3.3 *Platte River HEC-RAS Model Results*

Discharge wave predictions from both HEC-RAS model reaches were tested against hydrographs measured at USGS stream gages. Model results from both reaches are presented below.

3.3.1 Model Reach between North Platte and Overton, Nebraska

The HEC-RAS model for the reach from North Platte to Overton, Nebraska was run for hydrographs with peak discharge values of at least 5,000 ft^3/s. Hydrographs measured during small floods in 1979 and 1987 were used for the HEC-RAS model testing (Figure 13 and Figure 14). The measured discharge hydrographs at the stream gages near Brady and Cozad were similar, but the discharge significantly increased by the stream gage near Overton, Nebraska. Most of the discharge increase between the gages near Cozad and Overton, Nebraska is likely due to additional flow releases through the Johnson-2 hydro return.

HEC-RAS model discharge hydrographs matched well with the measurements at the USGS stream gage near Cozad, Nebraska in June 1979 and June 1987 (Figure 15 and Figure 16). The HEC-RAS model for the reach between North Platte and Overton, Nebraska was not tested for hydrographs with peak discharges less than 5,000 ft^3/s because no such measured hydrographs were found.

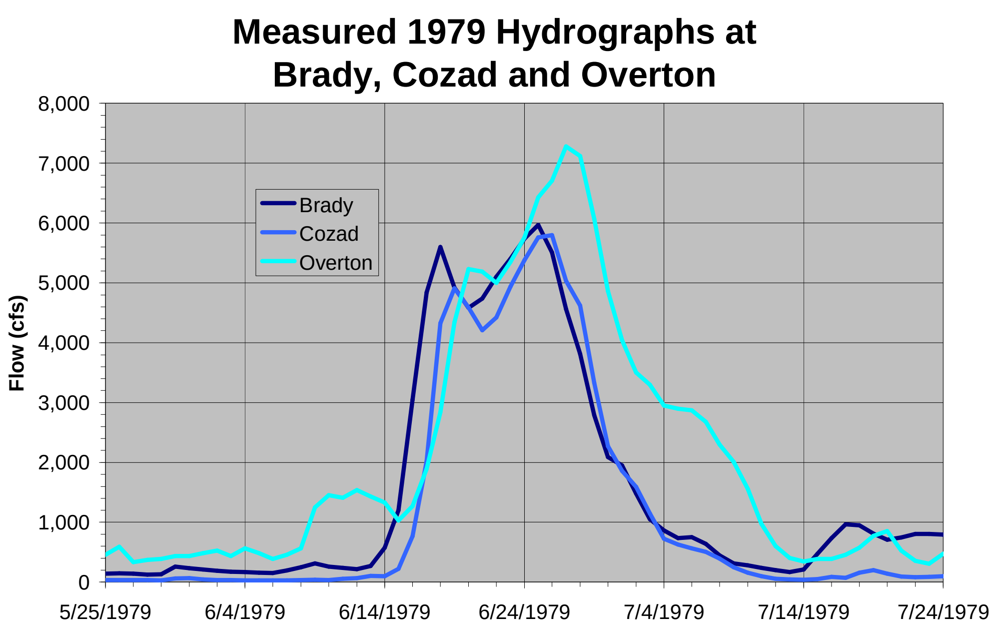

Figure 13. Comparison of discharge hydrographs measured at USGS stream gages near Brady, Cozad, and Overton, Nebraska for a flood in June 1979.

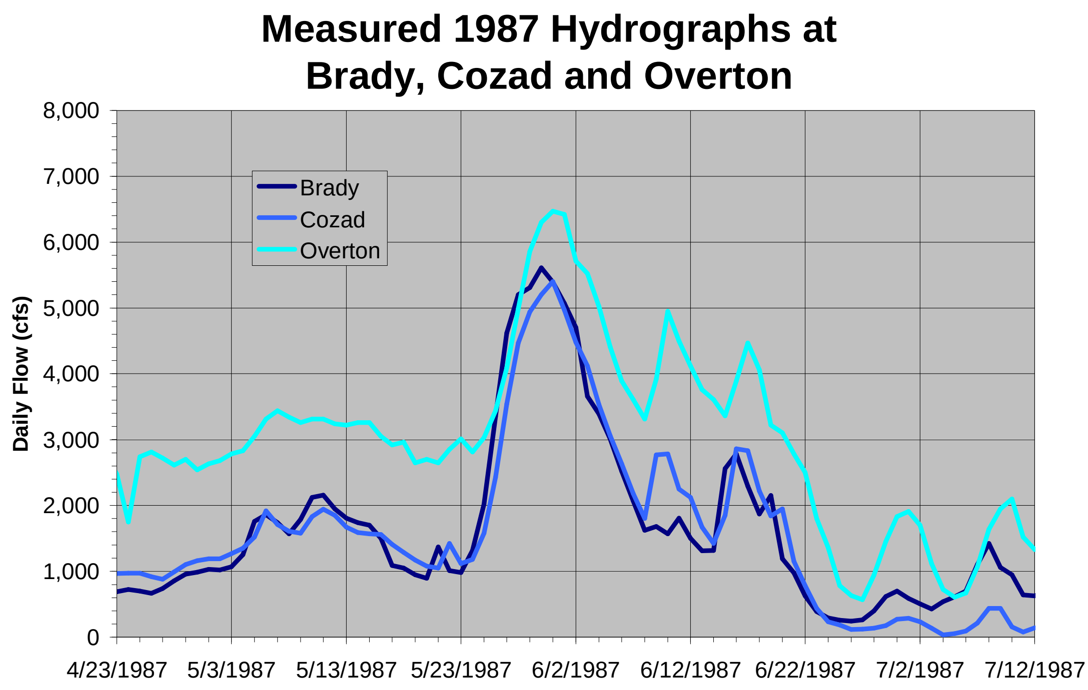

Figure 14. Comparison of discharge hydrographs measured at USGS stream gages near Brady, Cozad, and Overton, Nebraska for a flood in May 1987.

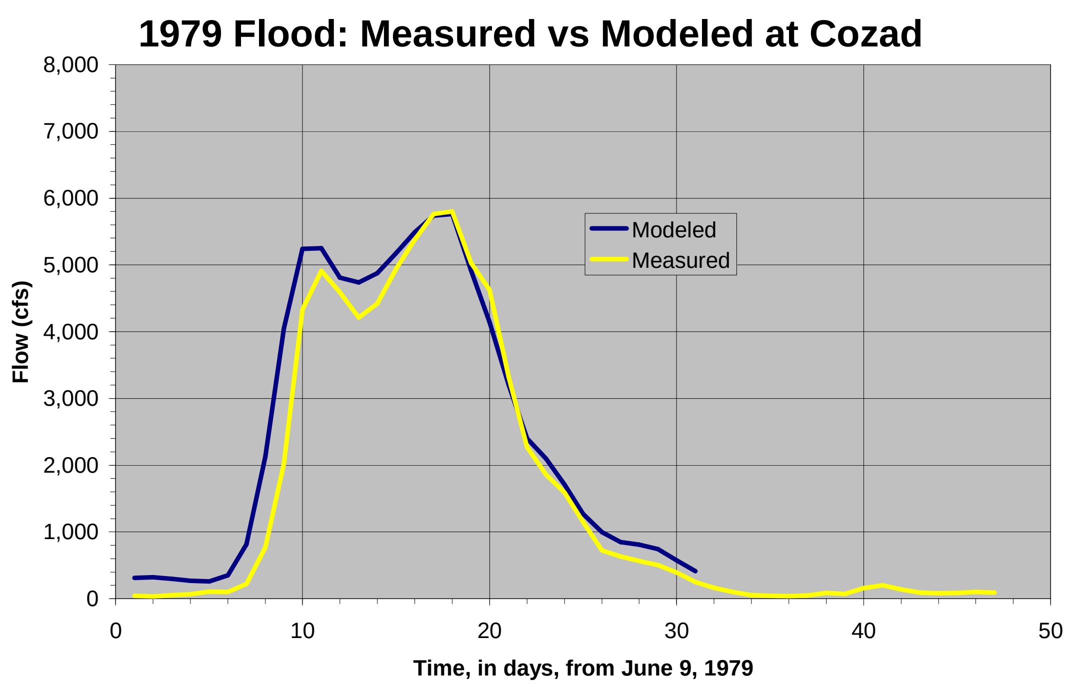

Figure 15. Predicted and measured discharge hydrographs are compared at the USGS stream gage near Cozad, Nebraska for a flood in June 1979.

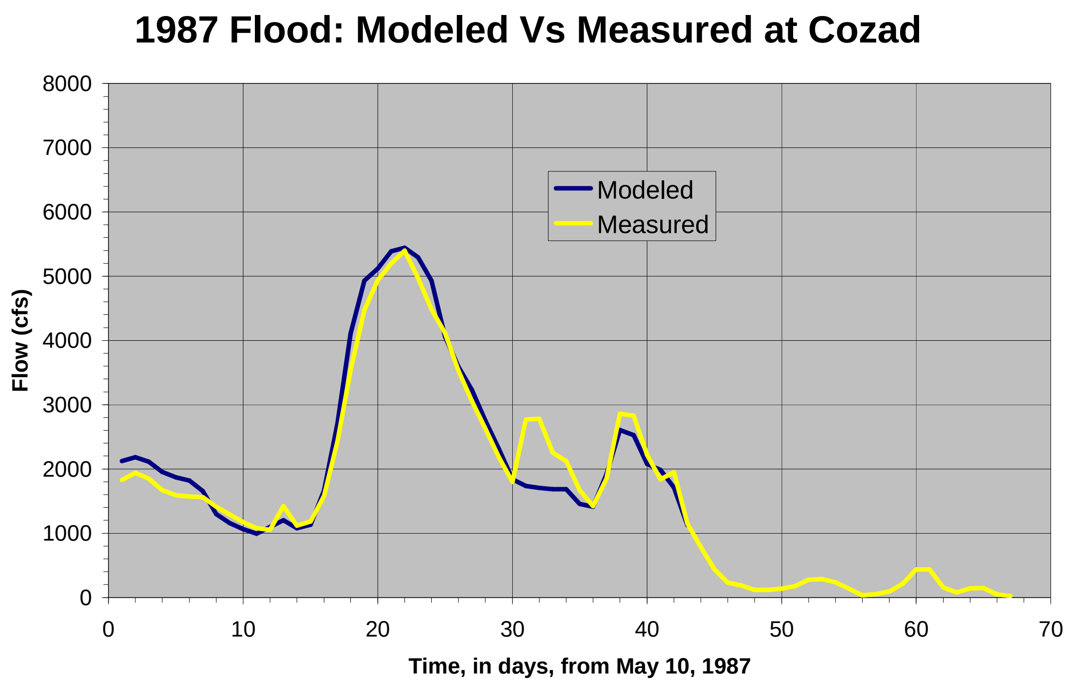

Figure 16. Predicted and measured discharge hydrographs are compared at the USGS stream gage near Cozad, Nebraska for a flood in May 1987.

3.3.2 Model Reach between Overton and Grand Island, Nebraska

For the HEC-RAS model reach from Overton to Grand Island, Nebraska, numerous unsteady flow hydrographs were available because of the fluctuating releases from the Johnson-2 hydro return. Fluctuating discharge hydrographs from the Johnson-2 hydro return are more similar to pulse-flow releases because they are within the bankfull-discharge capacity of the river channel and they are not generally affected by incremental storm runoff between gaging stations. Discharge hydrographs measured during fluctuating flows in 2002 and 2005 were used for model testing. The discharge hydrograph measured during April 2002 was first used to test the HEC-RAS model without considering the effects of bank storage (Figure 17). The measured peak discharge of the first wave was 500 ft^3/s less than the modeled peak discharge (30 percent less). This difference can only be accounted for by considering the bank storage process, which includes a loss of river flow to bank storage during periods of rising discharge and a lesser volume of flow gain back to the river from bank storage during periods of falling discharge. The losses to bank storage are associated with the start of rise of the first hydrograph wave, with the maximum loss occurring just before the peak of each discharge wave. The gains from bank storage seem to start after the river flow recedes and reaches a maximum rate at about 50 percent of the peak stage above the base flow.

By the time the measured discharge wave reaches the stream gage near Grand Island, attenuation and loss to bank storage has consumed most of the first discharge wave (Figure 18). Therefore, bank storage must be considered to more accurately model discharge hydrographs within the bankfull-channel capacity.

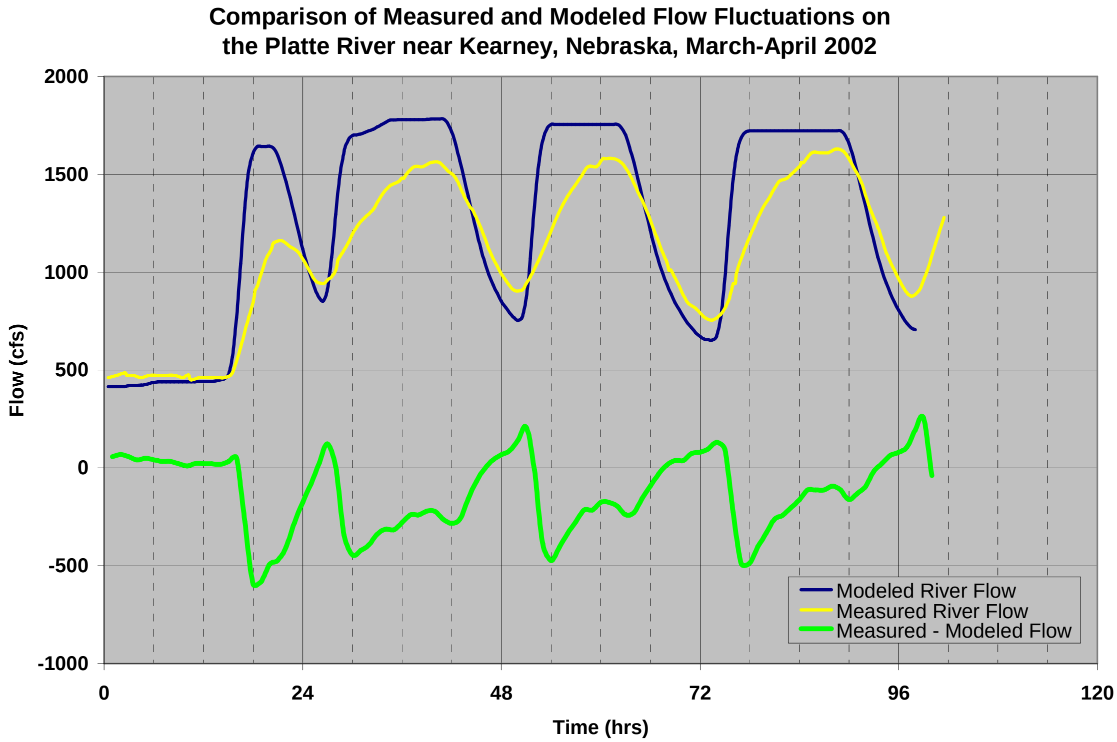

Figure 17. Predicted and measured discharge hydrographs are compared at the USGS stream gage near Kearney, Nebraska in April 2002.

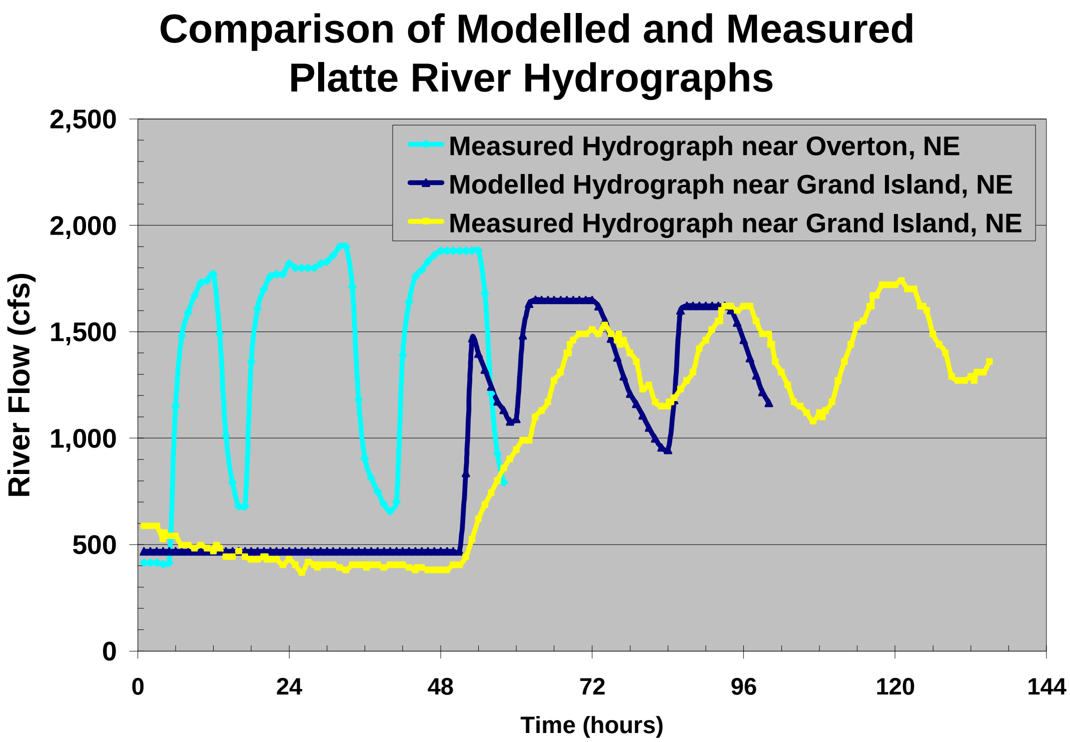

Figure 18. Predicted and measured and discharge hydrographs are compared at the USGS stream gage near Grand Island, Nebraska in April 2002.

4.0 Bank Storage models

The bank storage model developed for the Platte River is based on information and results of analytical and numerical models of groundwater bank storage. The analytical model provides an exact solution, but only for an idealized stage hydrograph defined by a sine function. The numerical groundwater model is applicable to a stage hydrograph of any shape, but the results are not an exact solution.

4.1 Analytical Bank Storage model

Cooper and Rorabaugh (1963) provide a very nice analytical model for the bank storage when the stage hydrograph can be defined by a sine function (Eq. 3).

$$\psi(t) = N\ h_0\ e^{\delta t}\left(1-\cos\omega t\right) \quad \text{when } 0 \le t \le \tau$$

Eq. 3

Where

$\psi(t)$	= river stage as a function of time [L],
h_o	= maximum stage rise [L],
t	= time since the beginning of the stage rise [T],
τ	= duration of the river stage hydrograph [T],
ω	$= 2\pi/\tau$, [1/T]
δ	$= \omega \cot(\omega t_c/2)$, [1/T], when $\delta = 0$, the stage hydrograph curve is sinusoidal,
t_c	= time of the stage crest [T], and
N	= a constant.

Eq. 4 and Eq. 5 predict the flow rate (Q) to and from the river bank as a function of time. Eq. 4 is applicable for times that are within the duration of the stage hydrograph.

$$Q_{t\le\tau} = \left(\frac{h_0\sqrt{\omega TS}}{2}\right)\sqrt{\frac{2}{\pi}}\left\{\frac{1}{2\omega t}\left[-\frac{(2\omega t)^2}{3}+\frac{(2\omega t)^4}{3\cdot 5\cdot 7}-\frac{(2\omega t)^6}{3\cdot 5\cdot 7\cdot 9\cdot 11}+\cdots\right]\right\}$$

Eq. 4

Where

Q	= time varying flow rate between the river channel and bank storage per unit length of river channel [L^2/T],
T	= transmissivity ($T = K\,B$) of the aquifer [L^2/T],
K	= hydraulic conductivity [L/T],
B	= aquiver thickness [L], and
S	= aquifer storage coefficient.

Eq. 5 is applicable for times that are beyond the duration of the stage hydrograph.

$$Q_{t>\tau} = \left(\frac{h_0\sqrt{\omega TS}}{2}\right)\sqrt{\frac{2}{\pi}}\left\{\begin{array}{l}\frac{1}{2\omega t}\left[-\frac{(2\omega t)^2}{3}+\frac{(2\omega t)^4}{3\cdot 5\cdot 7}-\frac{(2\omega t)^6}{3\cdot 5\cdot 7\cdot 9\cdot 11}+\cdots\right]- \\ \frac{1}{\sqrt{2\omega t-4\pi}}\left[\begin{array}{l}-\frac{(2\omega t-4\pi)^2}{3}+\frac{(2\omega t-4\pi)^4}{3\cdot 5\cdot 7}- \\ \frac{(2\omega t-4\pi)^6}{3\cdot 5\cdot 7\cdot 9\cdot 11}+\cdots\end{array}\right]\end{array}\right\}$$

Eq. 5

Eq. 6 and Eq. 7 predict the bank storage volume (V) as a function of time. Eq. 6 is applicable for times that are within the duration of the stage hydrograph.

$$V_{t\le\tau} = \left(\frac{h_0}{2}\sqrt{\frac{TS}{\omega}}\right)\sqrt{\frac{2}{\pi}}\sum_{n=1}^{\infty}\left[\frac{(-1)^{n-1}(2\omega t)^{(4n+1)/2}}{\prod_{m=1}^{2n}(2m+1)}\right]$$

Eq. 6

Where

V = time varying bank storage volume.

Eq. 7 is applicable for times periods beyond the duration of the stage hydrograph.

$$V_{t\ge\tau} = \left(\frac{h_0}{2}\sqrt{\frac{TS}{\omega}}\right)\sqrt{\frac{2}{\pi}}\sum_{n=1}^{\infty}\left[\frac{(-1)^{n-1}\left[(2\omega t)^{(4n+1)/2} - (2\omega t - 4\pi)^{(4n+1)/2}\right]}{\prod_{m=1}^{2n}(2m+1)}\right]$$

Eq. 7

The solutions to equations Eq. 3, Eq. 4, Eq. 5, Eq. 6, and Eq. 7 are presented graphically in Figure 19. The solution for river stage is made dimensionless by the maximum river stage (h_o). The solutions for bank storage flow rate and bank storage volume are made dimensionless by the term $\left(\frac{h_0}{2}\sqrt{\frac{TS}{\omega}}\right)$, which contain the maximum stage rise (h_o), the aquifer transmissivity (T) and storage (S), and the duration of the river stage hydrograph (τ).

The analytical solution indicates that the maximum flow rate from the river into bank storage occurs near, but just before, the time of maximum river stage. Flow from the river into bank storage stops when the river stage has receded to about one-half of the maximum stage. This also corresponds to the time of maximum bank storage volume. Not all of the bank storage volume is readily released back to the river. For example, at a time equal to 2.5 times the duration of the stage hydrograph, about 70 percent of the bank storage has been released back to the river. Even after the river stage has receded back to the base flow stage (0.0 in Figure 19), bank storage flow continues to return to the river channel for a long time.

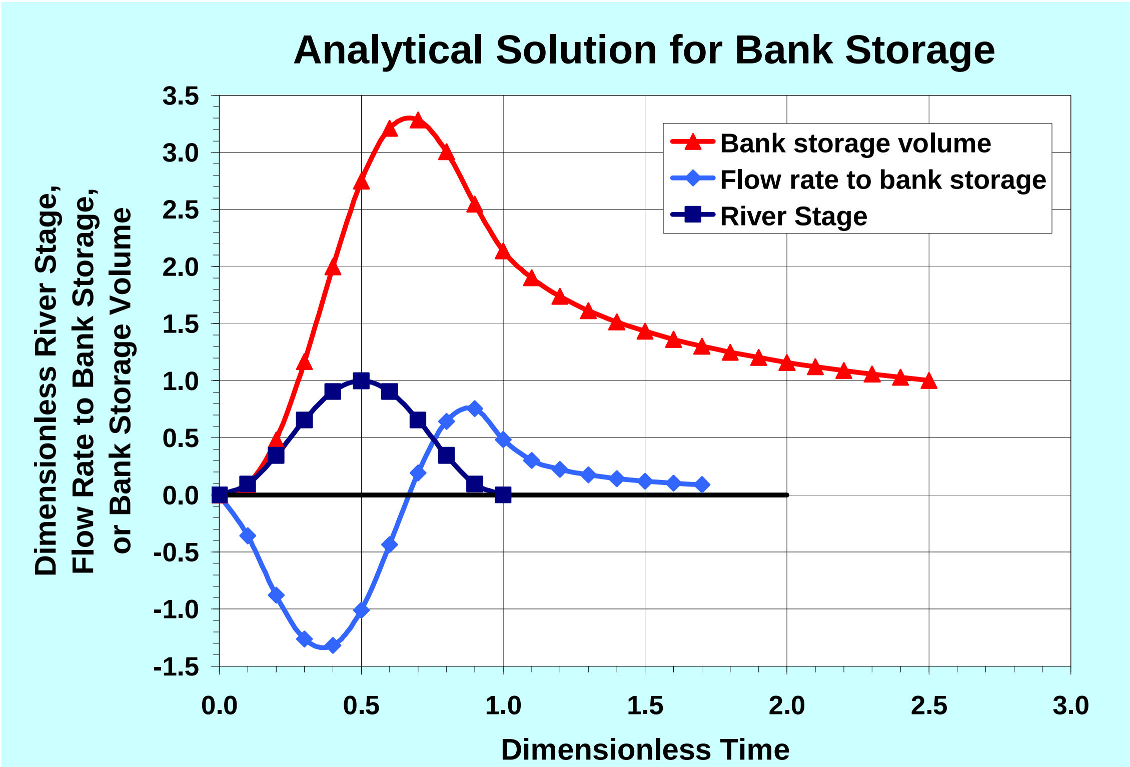

Figure 19. Analytical solutions for bank storage are represented in a series of dimensionless graphs. These graphs include river stage, bank storage flow rates, and bank storage volume.

4.2 Numerical Groundwater Model

An existing numerical groundwater model has been applied to predict flow from the river to bank storage and back from bank storage to the river due to the passage of a river-stage hydrograph of trapezoidal shape. The numerical groundwater model MODFLW96.3_3 was used for this purpose (Harbaugh and McDonald, 1996). MODFLOW is a three-dimensional finite-difference groundwater flow model. MODFLOW simulates steady and unsteady flow in an irregularly shaped flow system in which aquifer layers can be confined, unconfined, or a combination of confined and unconfined.

The MODFLOW model has been applied in a single, two-dimensional layer to a 1.0-mi^2 area of the Platte River with a straight channel through the middle of the area (Figure 20).

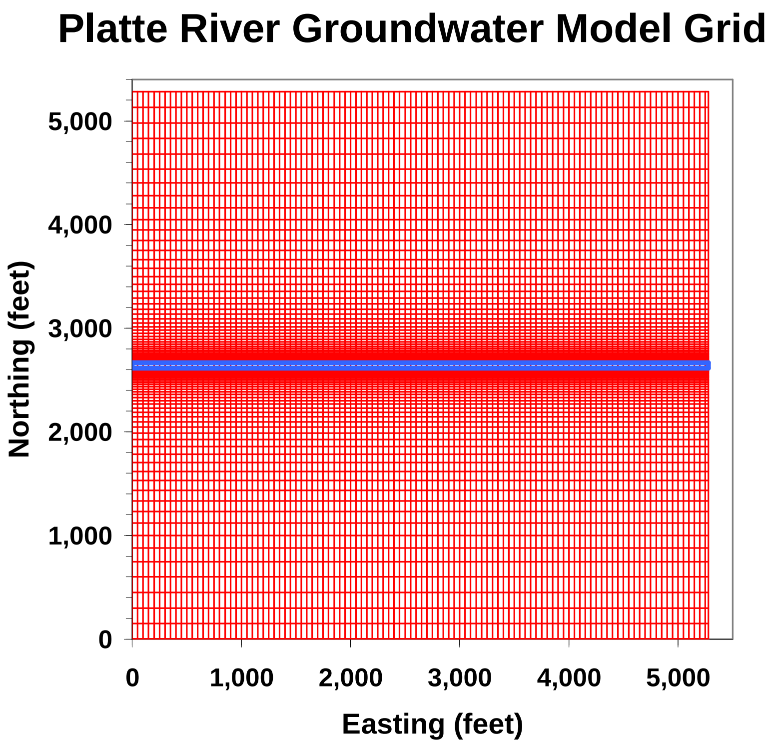

Figure 20. The two-dimensional model grid of an alluvial aquifer is presented for the case of a river channel crossing through an area of 1 mi^2. The model grid spacing is small close to the river channel and grid spacing steadily increases with distance away from the river channel.

Assumed numerical model input parameters are presented below:

- A unit length of river equal to 1.00 mile was modeled.
- Hydraulic conductivity, *K*, of the alluvial aquifer was set equal to 1.18 ft/hr.
- River-bed conductance was set equal to 20,000 ft^2/hr.
- Aquifer thickness, *B*, was set equal to 50 feet.
- Storage coefficient, *S*, was set equal to 0.10.

The assumed values for *K*, *B*, and *S* were based on reasonable values from Anderson and Woessner (2002). As will be shown later, the terms *K*, *B*, and *S* are multiplied together. Calibration of the bank storage model can only provide a value for the combined term (*KBS*).

The numerical groundwater model was used to simulate a variety of trapezoidal stage hydrographs. Figure 21 shows a typical river stage hydrograph with 2-foot stage rise and peak duration of 36 hours.

Plots of the bank storage water table (Figure 22) show how the water table varies with time as a result of the stage hydrograph presented in Figure 21. Water from the river channel spreads into the bank as the river stage increases and continues to spread into the bank until the river stage decreases. As the river stage decreases, a portion of the water in bank storage flows back to the river, but another portion of the bank storage water continues spreading farther into the bank. Therefore, groundwater stored in the banks, due to an increase in river stage, does not completely flow back to the river until long after the river stage has receded.

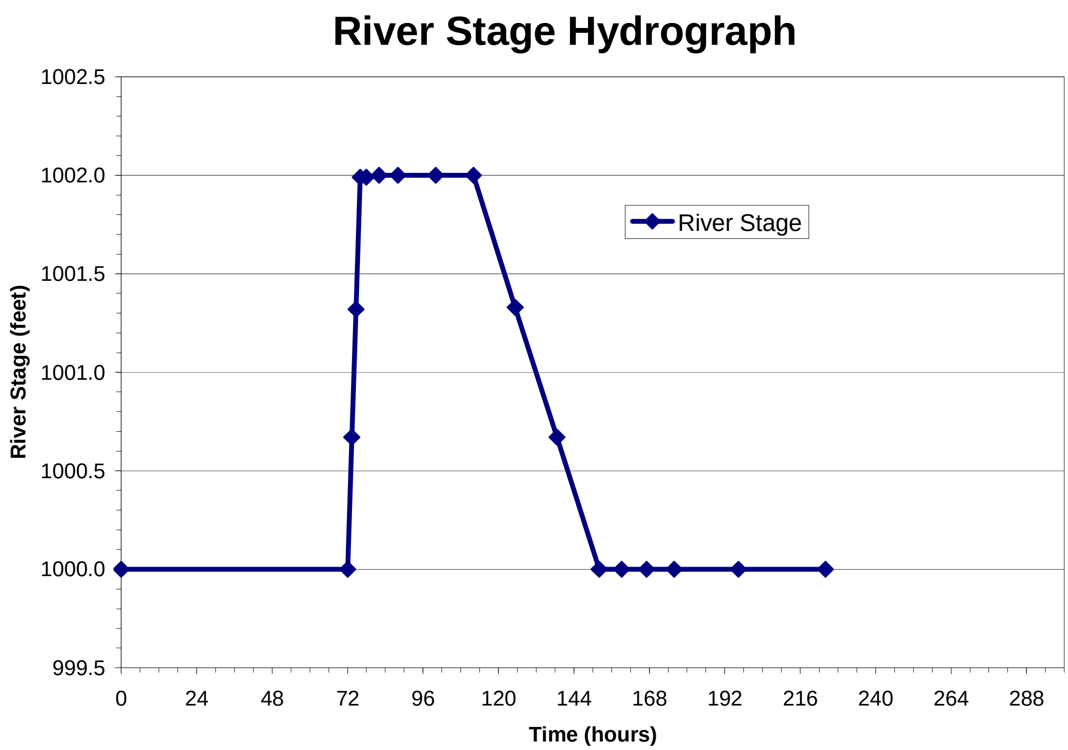

Figure 21. Assumed stage hydrograph where river stage is initially steady, then increase by 2.0 feet over a period of 4 hours (increasing at a rate of 0.5 ft/hr between hours 72 and 76), remains at the peak stage for a duration of 36 hours (hours 76 to 112), then decreases by 2.0 feet over a period 40 hours (decreasing at a rate of 0.05 ft/hr between hours 112 and 152).

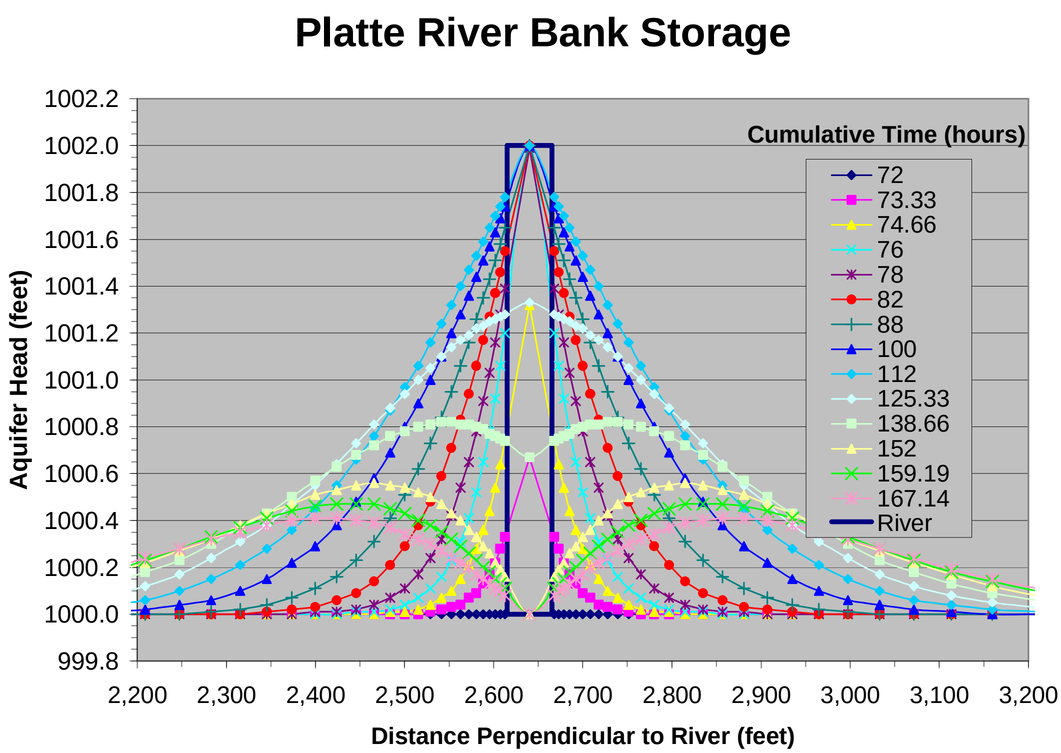

Figure 22. Aquifer and river cross-section plots of the water table at various times corresponding to the stage hydrograph plotted in Figure 21.

Figure 23 illustrates the model results for a trapezoidal stage hydrograph and can be compared to analytical model results presented in Figure 19 for a stage hydrograph based on a sine function. The two graphs are similar, except in the numerical-model example there is an exponential decay in the rate of flow into bank storage while the peak river stage remains constant.

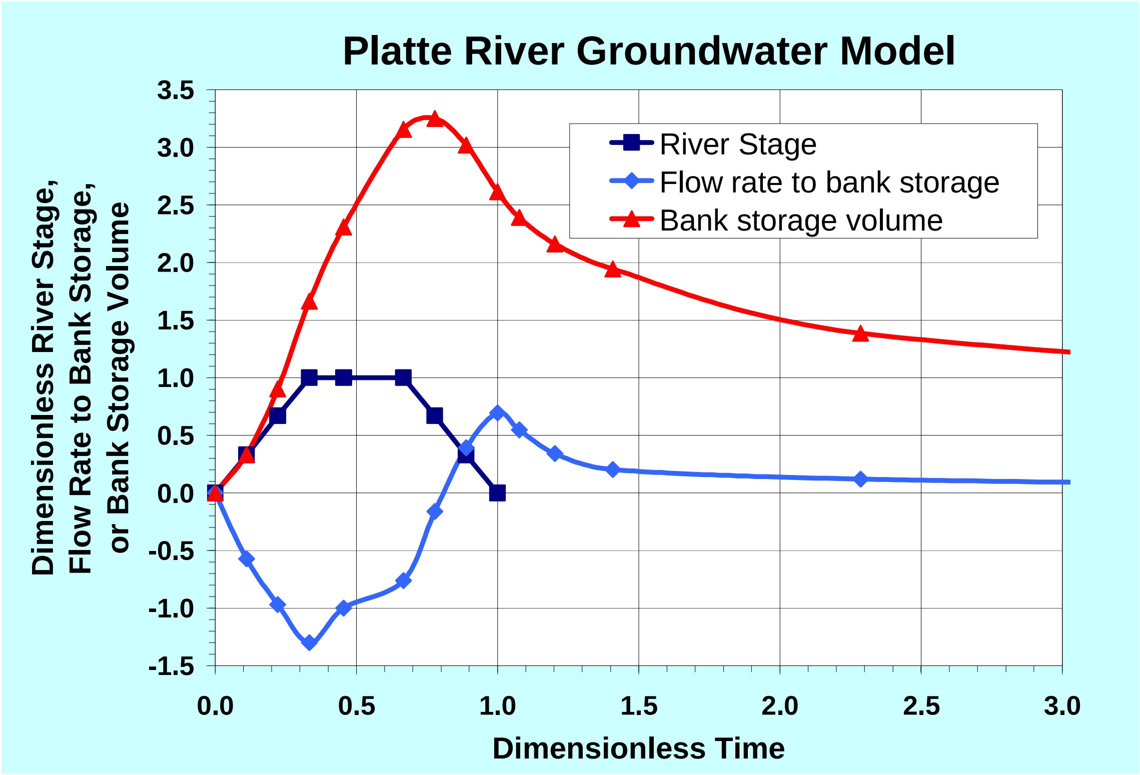

Figure 23. The numerical groundwater model results are represented in a series of dimensionless graphs of river stage, flow into and out of bank storage, and bank storage volume.

The numerical groundwater model was applied to several cases where the river stage was assumed to rise a certain height, *h*, (ranging from 0.5 to 2 feet) with an assumed duration of the peak stage, t_p, (ranging from 0.5 hour to 72 hours). A summary of the model simulations is presented in Table 2. For all simulations, the rate of river stage increase was set equal to 0.5 ft/hr while the rate of river stage decrease was set equal to 0.05 ft/hr.

Table 2. Summary of two-dimensional groundwater model simulation results.

Model scenario	Rise in river stage, *h* (feet)	Duration of river stage rise, t_r (hours)	Duration of peak river stage, t_p (hours)	Time, t (hours) from beginning of stage rise to end of peak stage
1	0.5	1.0	0.5	1.5
2	0.5	1.0	72	73
3	1.0	2.0	12	14
4	1.0	2.0	48	50
5	1.0	2.0	72	74
6	2.0	4.0	36	40
7	2.0	4.0	48	52
8	2.0	4.0	72	76

The results from these runs were analyzed to derive a relationship between the volume of bank storage (V) at time t and the parameter $h(tKBS)^{0.5}$, where t is the time from start of river stage increase to the end of the peak river stage. A good correlation ($R^2 = 0.9971$) was found between these two parameters and is presented in Figure 24. This relation is used to estimate volume of river flow loss to bank storage during the passage of a flood wave through a 1-mile reach of the river. This relationship can be converted to any length of river for which, K, B, and S can be considered constant. Conceptually this is the cross-sectional area of water in bank storage, at time t, for a given rise in river stage, h.

This relation is significant in terms of its general application to any alluvial river for estimating flow loss to bank storage during the passage of a flood wave through the river. The specific relationship derived for the Platte River depends on the assumed constant rate of stage rise (0.5 ft/hr). However, the model is not overly sensitive to the rate of rise for normal conditions on the Platte River. The relationship between bank storage volume and the parameter $h(tKBS)^{0.5}$ includes the time of stage rise and the duration of peak discharge. The duration of stage fall is not considered. Although a constant rate of rise was used for all simulations, a range of times were modeled for the duration of river stage rise (1 to 4 hours) and for the duration of the peak stage (0.5 to 72 hours).

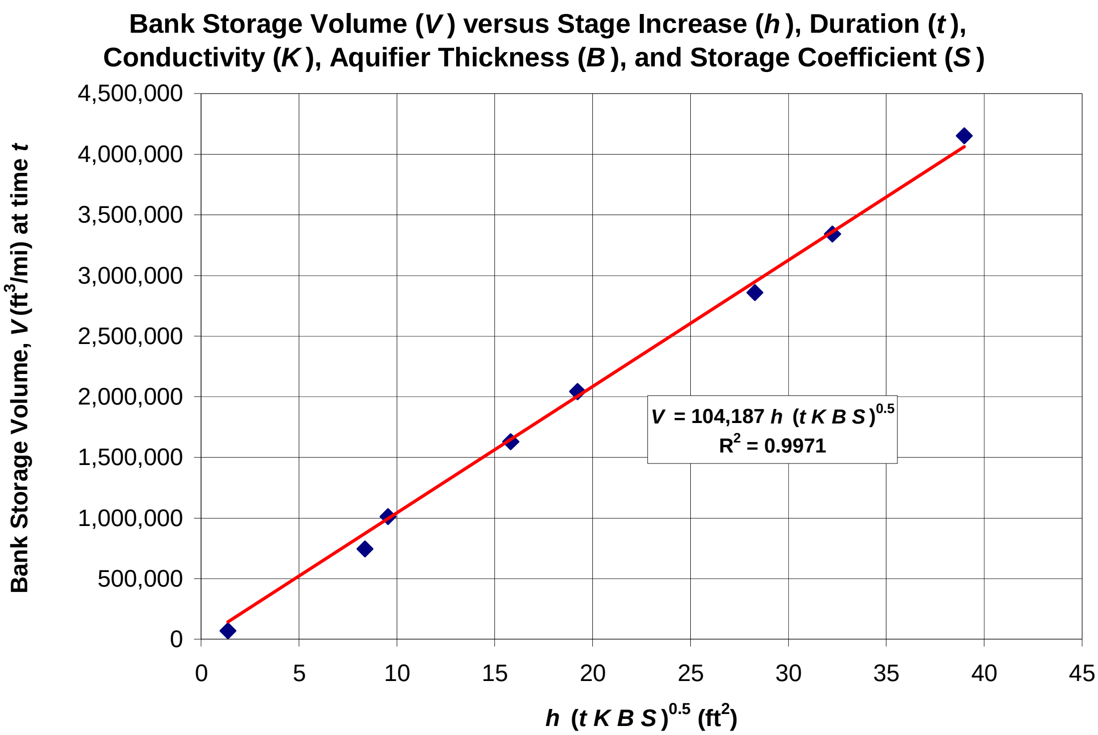

Figure 24. Bank storage volume as a function of the increase in river stage (*h*), duration of stage rise and peak stage (*t*), hydraulic conductivity (*K*), aquifer thickness (*B*), and storage coefficient (*S*).

4.3 Empirical Bank Storage Model

Once the bank storage volume is determined from the parameter $h(tKBS)^{0.5}$, the volume is distributed along the timeline of the bank storage flow hydrograph. A conceptual model of the water-flow hydrograph between the river channel and bank storage was developed (Figure 25) based on the results from the analytical and numerical groundwater models. Once the river stage begins to increase (at time t_0), the flow rate from the river channel to bank storage (negative flow) is assumed to decrease at a linear rate (from time t_o to t_a) to the minimum flow rate (Q_a), which occurs just before the maximum river stage is reached. During the period of maximum river stage (from time t_a to t_b), the flow rate into bank storage is approximated by two exponential functions as depicted in Figure 26. Once the river stage begins to recede (at time t_b), the flow rate into the bank (negative flow) is assumed to increase at a linear rate until water begins flowing back from the bank (at time, t_x). The flow rate from bank storage to the river channel (positive flow) is assumed to continue increasing at the same linear rate until a maximum flow rate from the bank (Q_c) is achieved (at time t_c). After the time of maximum flow rate from the bank (from time t_c to t_d), the flow rate is approximated by two exponential functions as shown in Figure 27. The flow rate from bank storage back to the river channel decreases exponentially until the river stage begins increasing again.

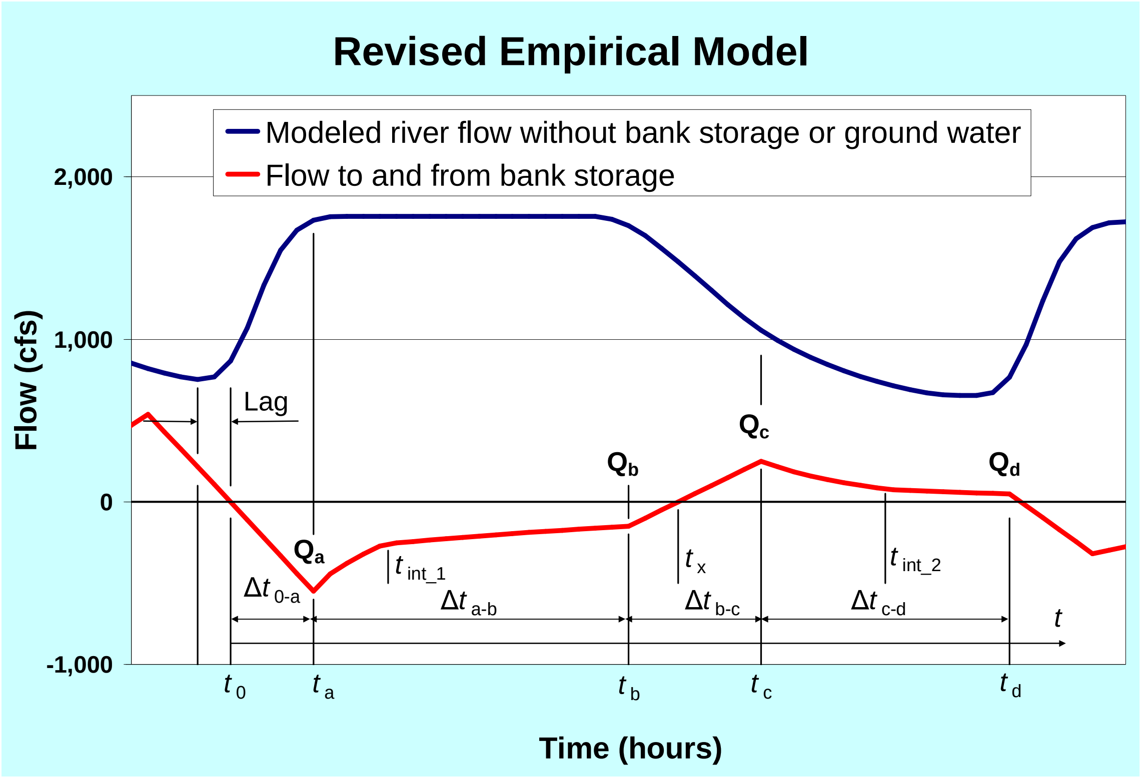

Figure 25. Conceptual hydrograph of bank storage flow rates from the river to bank storage (-) and flow rates from bank storage to the river (+).

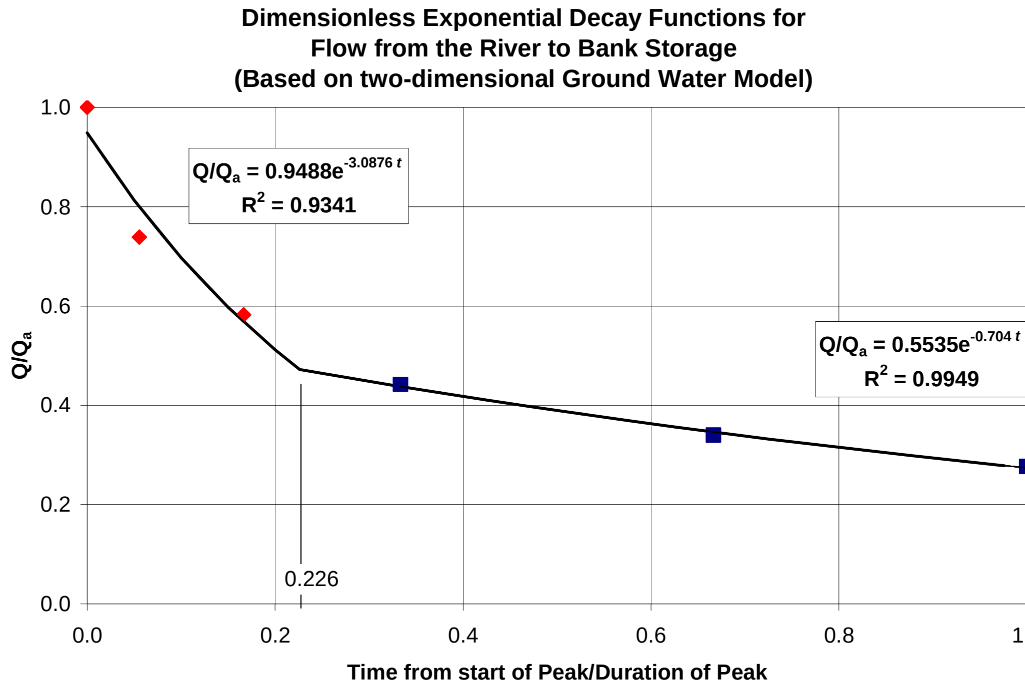

Figure 26. Dimensionless, exponential-decay functions are presented for river flow into bank storage.

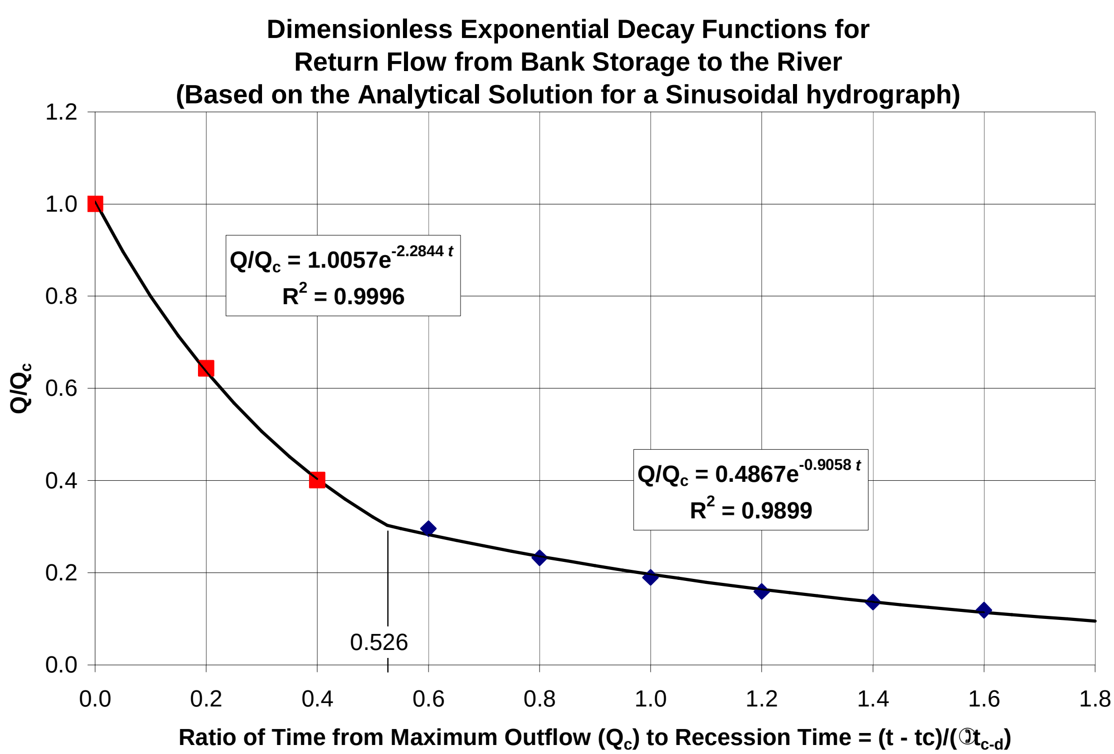

Figure 27. Dimensionless, exponential-decay functions are presented for return flow from bank storage to the river.

The times t_a, t_b, and t_c have been determined empirically through the combination of analytical and numerical model results:

- The time of maximum flow rate from the river to the bank (t_a) was found to occur at a certain percentage of the maximum stage increase (time corresponding to stages between 0.75 *h* and 0.99 *h*).
- The time when the maximum river stage is considered to have ended (t_b) is when the river stage has again decreased to a certain percentage of the initial stage increase (time corresponding to stages between 0.99 *h* and 0.50 *h*).
- The time of maximum flow rate from the bank back to the river (t_c) was found to occur when the river stage has decreased to a certain percentage of the maximum stage increase (time corresponding to stages between 0.50 *h* and 0.05 *h*) or when the river stage begins to rise again, whichever occurs first.

The volume of bank storage is defined by the equation in Figure 24. This same volume can also be computed by the integration of the bank-flow hydrograph presented in Figure 25. This curve is a function of the peak flow rates into and out of the bank (Q_a and Q_c) and the time durations of the stage rise and stage peak. The negative area under the bank flow hydrograph represents the bank storage volume into the bank from the river. The positive area under the bank flow hydrograph represents the volume of water flowing back to the river from bank storage. Eq. 8 is the integration of the negative portion of the bank-flow hydrograph (Figure 25), which represents the bank storage volume (V_{BS}) per unit length [1 mile] of river. The volume of water stored in the bank is predicted as a function of the peak flow rate into the bank (Q_a), attenuation of this flow rate (Q_b), and the time durations of the stage rise and stage peak.

$$V_{BS} = \frac{Q_a(t_0 - t_a)}{2} + \int_{t_a}^{t_{\text{int\_1}}} a_1 \exp(b_1 t)\,dt + \int_{t_{\text{int\_1}}}^{t_b} a_2 \exp(b_2 t)\,dt + \int_{t_b}^{t_x} (mt + Q_b)\,dt$$

Eq. 8

Where

V_{BS} = volume of bank storage per mile of river length [L^2] for the time period from t_0 to t_x,

t = time [T],

Q_a = maximum flow rate (minimum negative number) from the river channel to bank storage per unit length of river [L^2/T],

Q_b = attenuated flow rate from the river channel to bank storage per unit length of river [L^2/T] corresponding to the end of maximum river stage,

t_0 = time at start of river stage rise [T],

t_a = time at maximum flow rate from the river to bank storage [T],

t_b = time when maximum river stage has begun to recede (receded to a percentage of its increase) [T],

t_x = time when flow into the bank from the river channel has stopped [T],

t_{int_1} = time during the maximum river stage when there is a change in the exponential decay in the flow rate into the bank [T],

a_1 = 0.9488
a_2 = 0.5535
b_1 = -3.0876
b_2 = -0.7040

$$m = \frac{(Q_c - Q_b)}{t_c - t_b}$$

Eq. 9

Where

m = linear slope of the discharge line in Figure 25 between times t_c and t_b [L^2/T^2],
Q_c = maximum flow rate from bank storage to the river channel per unit length of river [L^2/T], and
t_c = time at maximum flow rate from the bank storage to the river [T],

Eq. 10 is the integration of the positive portion of the bank-flow hydrograph (Figure 25), which represents the volume of bank storage (V_{BS}) returning to the river channel per unit length (1 mile) of river. Over an infinite duration of time, the volume of water returning from bank storage is theoretically equal to the bank storage volume. However, the volume of bank storage returning to the river over the time scale of the discharge wave is much less. The coefficient c ($0 < c < 1$) is applied to the bank storage volume to predict the volume that will return to the river over the time scale of interest.

$$cV_{BS} = \int_{t_x}^{t_c} (mt + Q_b)\,dt + \int_{t_c}^{t_{\mathrm{int\_2}}} a_3 \exp(b_3 t)\,dt + \int_{t_{\mathrm{int\_2}}}^{t_d} a_4 \exp(b_4 t)\,dt$$

Eq. 10

Where

V_{BS} = volume of bank storage per mile of river length [L^2] returning to the river channel over the time period from t_x to t_d,
c = portion of bank storage volume returning to the river over the time period from t_x to t_d ($0 < c < 1$),
Q_d = attenuated flow rate from bank storage to the river channel per unit length of river [L^2/T], corresponding to when the river stage begins to rise again or the last time of interest,
t_d = time when maximum stage has receded to s percentage of its increase [T],
t_{int_2} = time during the recession in river stage when there is a change in the exponential decay (Figure 27) in the flow rate from bank storage [T],
a_3 = 1.0057
a_4 = 0.4867
b_3 = -2.2844

b_4 = -0.9058

The coefficients (a_1 and a_2) and exponents (b_1 and b_2) are for the dimensionless exponential decay curve (Figure 26) for flow from the river channel into bank storage.

The coefficients (a_3 and a_4) and exponents (b_3 and b_4) are for the dimensionless exponential decay curve (Figure 27) for flow from bank storage to the river channel.

The integrals (Eq. 8 and Eq. 10) can be solved with Eq. 9 to produce the following algebraic equations (Eq. 11 and Eq. 12).

$$V_{BS} = \frac{Q_a\left(t_0 - t_a\right)}{2} + \left[\frac{a_1}{b_1}\left\{\exp\left(b_1 t_{\text{int\_1}}\right) - \exp\left(b_1 t_a\right)\right\}\right] +$$
$$\left[\frac{a_2}{b_2}\left\{\exp\left(b_2 t_b\right) - \exp\left(b_2 t_{\text{int\_1}}\right)\right\}\right] + \left[\frac{m}{2}\left(t_x^2 - t_b^2\right) - Q_b\left(t_x - t_b\right)\right]$$

Eq. 11

$$cV_{BS} = \left[\frac{m}{2}\left(t_c^2 - t_x^2\right) + Q_b\left(t_c - t_x\right)\right] +$$
$$\left[\frac{a_3}{b_3}\left\{\exp\left(b_3 t_{\text{int\_2}}\right) - \exp\left(b_3 t_c\right)\right\}\right] + \left[\frac{a_4}{b_4}\left\{\exp\left(b_4 t_c\right) - \exp\left(b_4 t_{\text{int\_2}}\right)\right\}\right]$$

Eq. 12

These two equations can be further simplified by introducing the dimensionless exponential decay functions, shown in Figure 26 and Figure 27, to produce equations Eq. 13, Eq. 14, Eq. 15 and Eq. 16.

$$V_{BS} = \frac{Q_a\left(t_0 - t_a\right)}{2} +$$

$$Q_a(36hrs)\left[\frac{a_1}{b_1}\left\{\exp\left(b_1 t_{r1}\right) - 1\right\} + \frac{a_2}{b_2}\left\{\exp\left(b_2 t_{r1}\right) - \exp\left(b_2 0.226\right)\right\}\right] +$$

$$\left[\frac{m}{2}\left(t_x^2 - t_b^2\right) - Q_b\left(t_x - t_b\right)\right]$$

Eq. 13

$$t_{r1} = \frac{\left(t_b - t_a\right)}{36hrs}$$

Eq. 14

$$t_{r2} = \frac{\left(t_d - t_c\right)}{40hrs}$$

Eq. 15

$$m = \left[\frac{Q_c - Q_a\left\{a_2 \exp\left(b_2 t_{r1}\right)\right\}}{\left(t_c - t_b\right)}\right]$$

Eq. 16

Where

a_1 = 0.9488
b_1 = -3.0876
a_2 = 0.5535
b_2 = -0.7040
a_3 = 1.0057
b_3 = -2.2844
a_4 = 0.4867
b_2 = -0.9058

In addition to bank storage, the empirical model can also include any groundwater gains or losses. Groundwater gains might be supplied by tributary drainages. Groundwater losses might occur due to a low aquifer table. Groundwater gains or losses can be specified in the empirical model as a constant steady flow or as a time-varying hydrograph.

4.4 Model Calibration

A FORTRAN computer program was written to apply the empirical bank storage model to the hydrographs routed using the HEC-RAS model. Fluctuating flow hydrographs, measured during April 2002 and during February 2005 were used to calibrate the bank storage model. Hydrographs from these two periods were routed from Overton, Nebraska downstream to Kearney, Nebraska and continuing downstream to Grand Island, Nebraska using the HEC-RAS model. The empirical bank storage model was applied to these routed hydrographs just upstream of the stream gages near Kearney and Grand Island, Nebraska. After adjustment for bank storage and any net groundwater gains or losses, the resulting hydrographs were compared with the measured hydrographs at Kearney and Grand Island, Nebraska.

The aquifer thickness (B) was assumed to be equal to 50 feet. The aquifer storage coefficient (S) was set equal to 0.1. The empirical bank storage model was calibrated by adjusting the following parameters (Table 3):

- Hydraulic conductivity (K).
- Time positions of points a, b, and c (Figure 25). The time positions of these points were specified as the ratio of river stage increase to the maximum stage increase ($\Delta h/\Delta h_{max}$).
- Fraction of the bank storage volume returning to the river (c in Eq. 10).

Table 3. Calibrated parameters for the empirical bank storage model.

	Time period of model simulation			
	Overton to Kearney, Nebraska		Kearney to Grand Island, Nebraska	
Calibrated Model Parameter	Apr 2002	Feb 2005	Apr 2002	Feb 2005
Hydraulic conductivity, K (ft/hr)	1.00	1.00	4.0	4.0
Aquifer thickness, B (ft)	50	50	50	50
Aquifer storage coefficient, S	0.1	0.1	0.1	0.1
River stage ratio for time t_a	0.841	0.841	0.841	0.841
River stage ratio for time t_b	0.741	0.741	0.741	0.741
River stage ratio for time t_c	0.25	0.50	0.25	0.50
Fraction of bank storage volume returning to the river (c) during the time period from t_0 to t_d	0.15	0.25	0.10 Note[2]	0.25

The calibrated values for the hydraulic conductivity and the assumed values for the aquifer thickness and aquifer storage coefficient are considered to be within the values reported in the literature for sand (Anderson and Woessner, 2002).

The calibrated time positions for t_a, t_b, and t_c (Figure 25) and the fraction of flow

[2] A coefficient of 0.10 was used for the first three discharge waves, but the coefficient was increased to 0.15 for the fourth discharge wave.

returning from bank storage compare well to the results from the analytical and two-dimensional bank storage models.

Calibrated model hydrograph results compared well with measurements at the stream gage near Kearney, Nebraska in 2002 (Figure 28) and in 2005 (Figure 29). Calibrated, model hydrograph results also compared well with measurements at the stream gage near Grand Island in 2002 (Figure 30) and in 2005 (Figure 31). A constant groundwater gain had to be applied to the hydrograph near Kearney, Nebraska in 2005. An increasing groundwater or tributary-flow gain had to be applied to the hydrograph near Grand Island, Nebraska in 2002. Groundwater gains were set to zero for the other two hydrographs near Kearney in 2002 and Grand Island in 2005. Groundwater or tributary-flow gains had to be applied for the two of the four cases to match measured conditions. For the 2002 hydrograph near Grand Island, Nebraska, the measured peak discharge increased for each successive wave, which indicates that groundwater or tributary-flow gains increased with time.

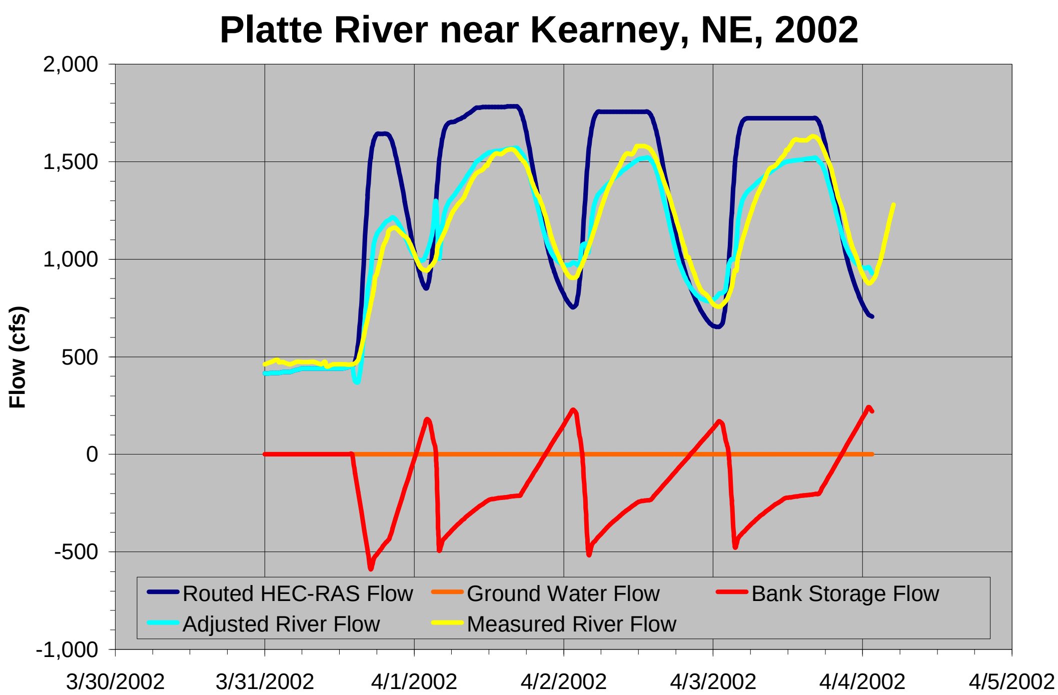

Figure 28. Calibrated model results for the Platte River reach between Overton and Kearney, Nebraska during unsteady flow in March and April 2002.

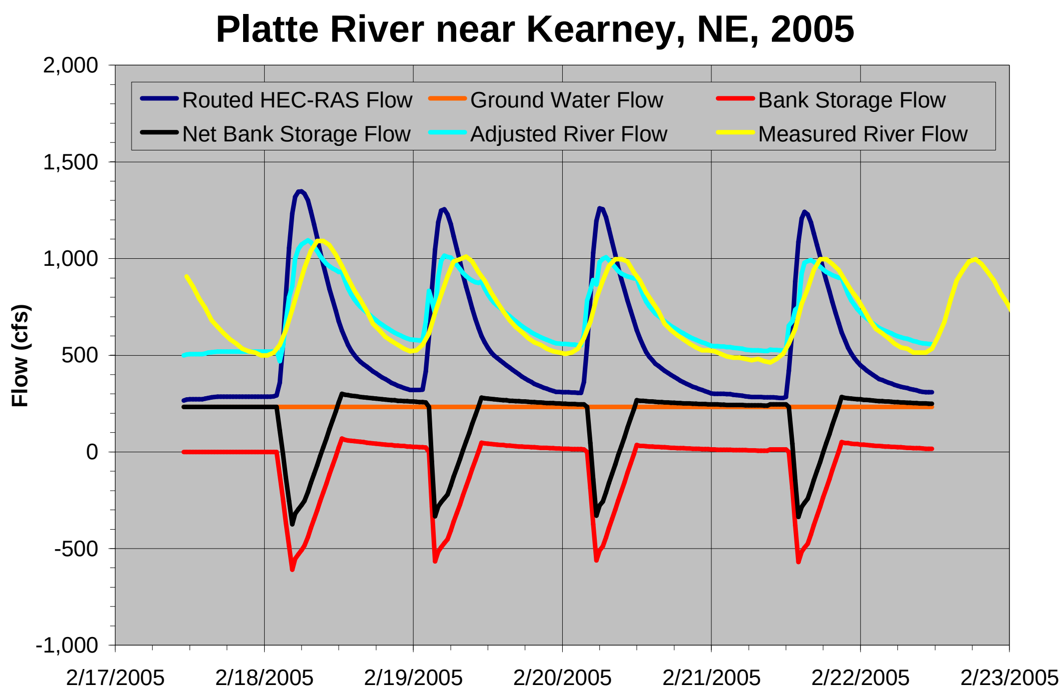

Figure 29. Calibrated model results for the Platte River reach between Overton and Kearney, Nebraska during unsteady flow in February 2005.

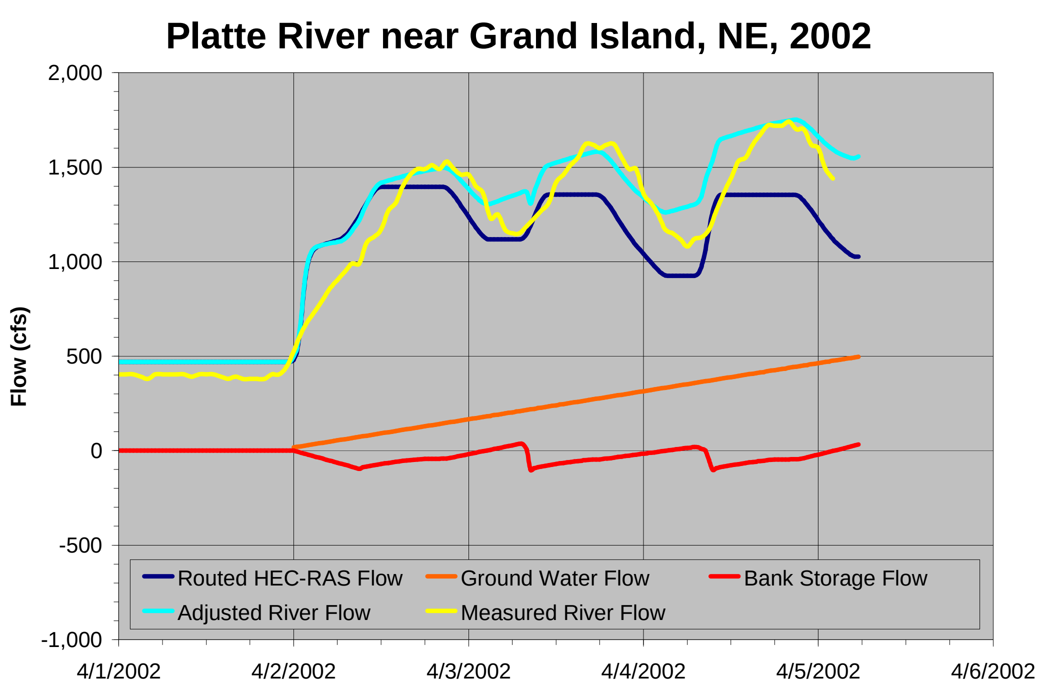

Figure 30. Calibrated model results for the Platte River reach between Kearney and Grand Island, Nebraska during unsteady flow in April 2002.

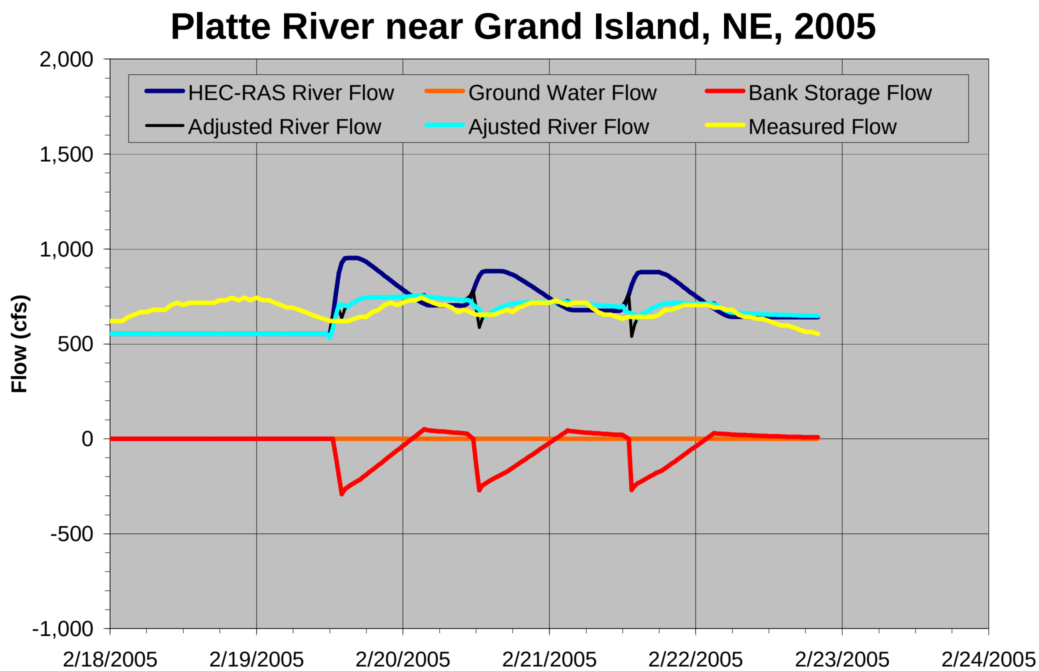

Figure 31. Calibrated model results for the Platte River reach between Kearney and Grand Island, Nebraska during unsteady flow in February 2005.

4.5 Comparison of Model Results

A proposed pulse-flow release hydrograph was routed with the HEC-RAS model from the stream gage near Overton, Nebraska downstream to stream gage near Kearney, Nebraska (Figure 32). The empirical bank storage model was applied to this hydrograph and the predicted results were compared to the results from the two-dimensional groundwater model (MODFLOW).

MODFLOW model predictions of flow to and from bank storage and bank storage volume versus time are presented in Figure 33. The empirical bank storage model predictions of flow to and from bank storage and the adjustment of the HEC-RAS river discharge hydrograph, which accounts for bank storage, are presented in Figure 34. Direct comparison of the river discharge hydrographs, adjusted for bank storage, is presented in Figure 35 based on both the empirical bank storage model and the two-dimensional groundwater model. Minor differences between the two sets of model results can be observed, but the hydrograph shapes and peak discharge are very similar.

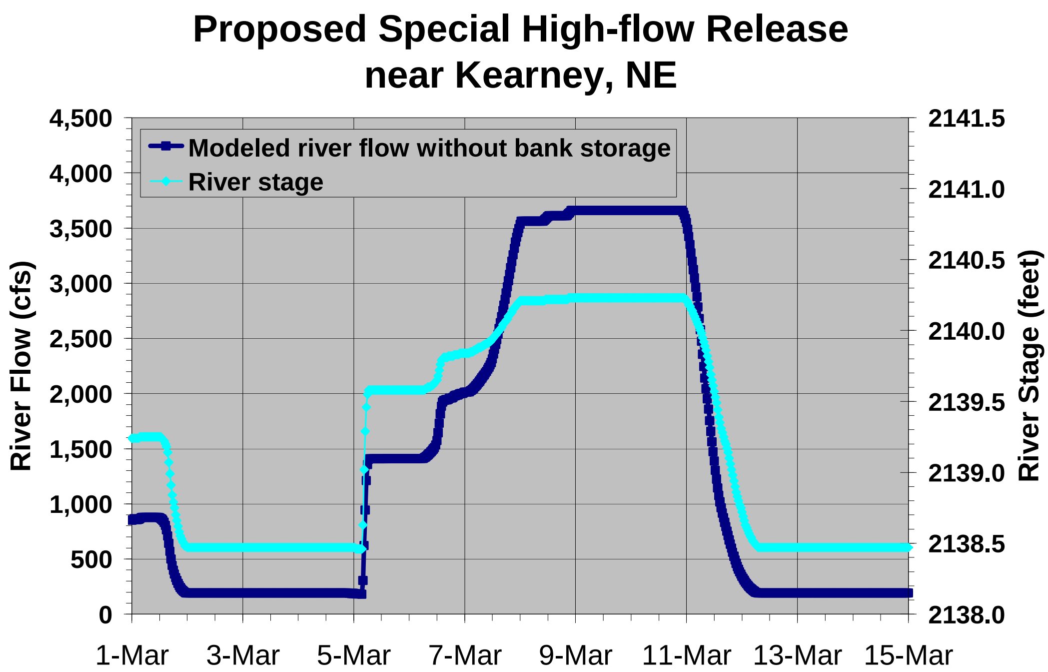

Figure 32. Pulse-flow release hydrograph near Kearney, Nebraska used for model verification.

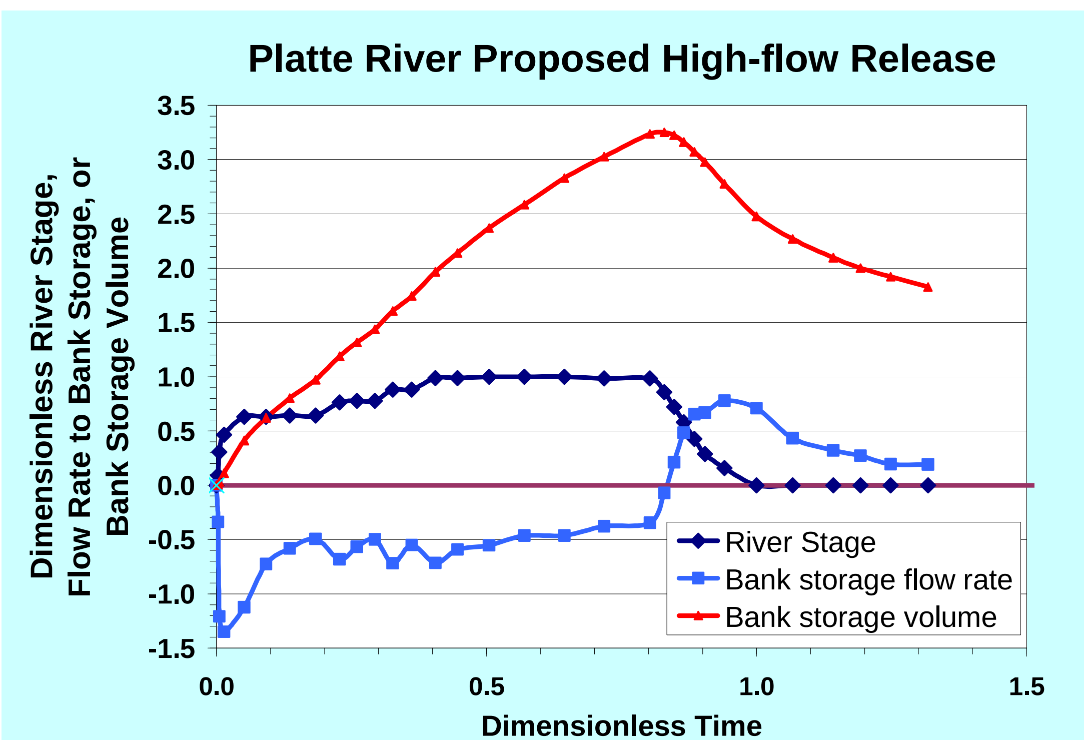

Figure 33. Dimensionless predictions of the bank storage flow rate and bank storage volume is presented based on the two-dimensional groundwater model.

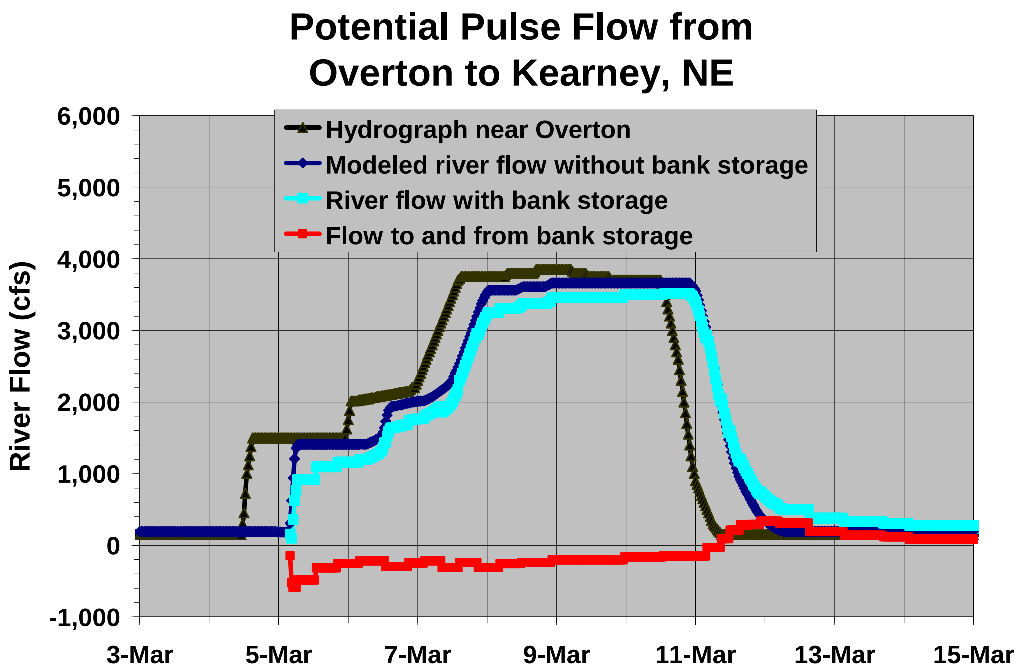

Figure 34. Empirical bank storage model results are presented for a pulse-flow release at the stream gage near Kearney, Nebraska.

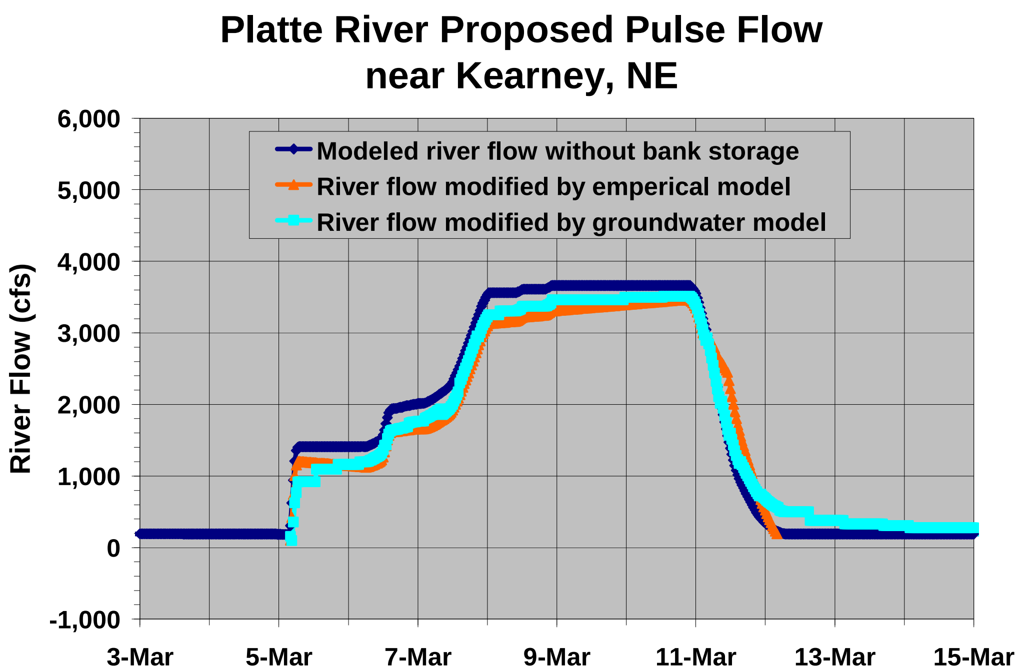

Figure 35. Discharge hydrographs are compared based on predictions from the empirical and numerical bank storage models.

5.0 Platte River Flow Losses and Gains

Platte River flows can experience losses or gains, with distance downstream, depending on the season and year. The U.S. Fish and Wildlife Service (Don Anderson, written communication, 2006) computed the monthly losses and gains in river flow between the stream gages near Overton and Kearney, Nebraska and between Kearney and Grand Island, Nebraska for the period 1970 to 2004. Data were sorted by month and a probability analysis was performed to determine the gain or loss in stream flow corresponding to the following cumulative probabilities 10, 25, 50, 75, and 90 percent (Figure 36 and Figure 37).

These figures can be used as a planning guide for estimating the range of stream flow losses or gains by river reach and by month. For the reach between Overton and Kearney, Nebraska, stream flow gains have at least a 50% probability of occurring during March and April, while stream flow losses are most likely during September and October. For the reach between Kearney and Grand Island, Nebraska, stream flow gains have at least a 50% probability of occurring during the period of February through July and in October, while stream flow losses are likely during September, November, December, and January.

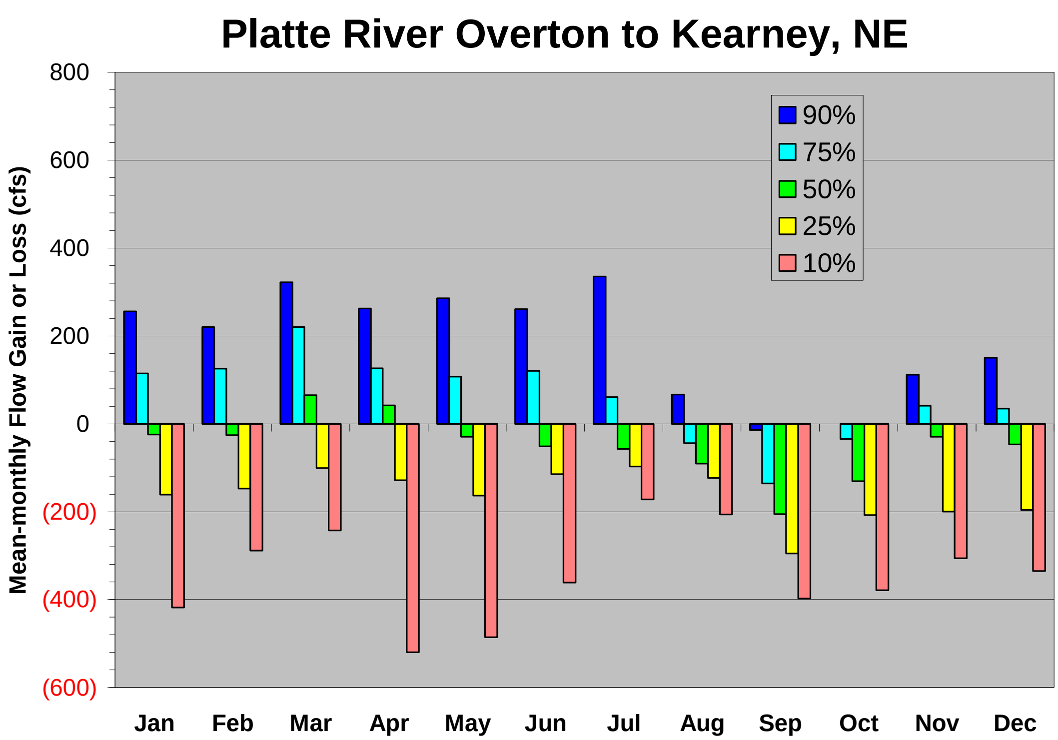

Figure 36. Monthly variation in the cumulative probability of net gain or loss to river flow is presented for the Platte River reach from Overton to Kearney, Nebraska.

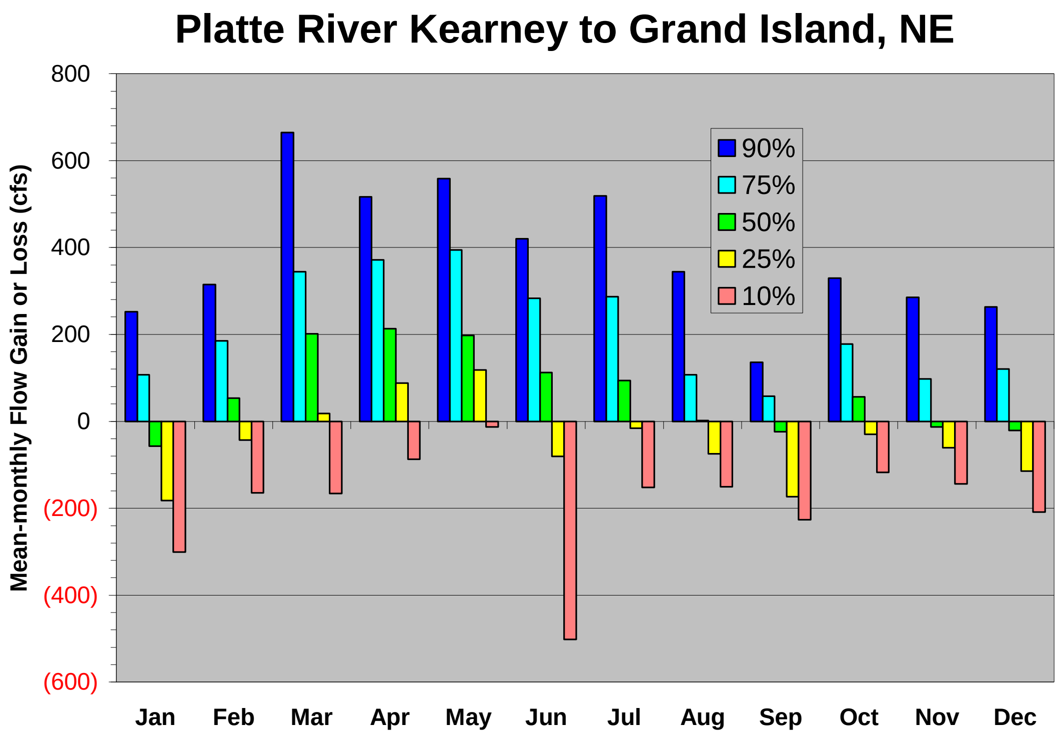

Figure 37. Monthly variation in the cumulative probability of net gain or loss to river flow is presented for the Platte River reach from Kearney to Grand Island, Nebraska.

6.0 Application of HEC-RAS and Bank Storage models

The HEC-RAS and empirical bank storage models were applied to two possible, pulse-flow scenarios:

1. The first hydrograph scenario builds to a peak discharge of about 5,000 ft^3/s at Overton, Nebraska over a four-day period and the peak is sustained for more than two days.

2. The second hydrograph scenario builds to a peak discharge of about 4,000 ft^3/s at Overton, Nebraska over a three-day period and the peak is sustained for less than two days.

The model results from these two scenarios are presented in sections 6.1 and 6.2. The step-by-step procedure on how to execute the Platte River HEC-RAS model is described Appendix A. The step-by-step procedure on how to apply the empirical bank storage model is presented in Appendix B.

6.1 Pulse-Flow Scenario 1

The bank storage and stream-flow hydrographs for flow scenario 1 are presented in Figure 38 for the stream gage near Kearney, Nebraska and in Figure 39 for the stream gage near Grand Island, Nebraska. The duration of the peak discharge is long enough (more than two days) that the peak discharges only attenuated from 4,800 ft^3/s near Overton to 4,600 ft^3/s near Grand Island, Nebraska (4 percent per stream gage). After adjustments for bank storage, the predicted river-flow hydrographs were adjusted by ± 10 ft^3/s per mile to account for potential stream-flow gains or losses.

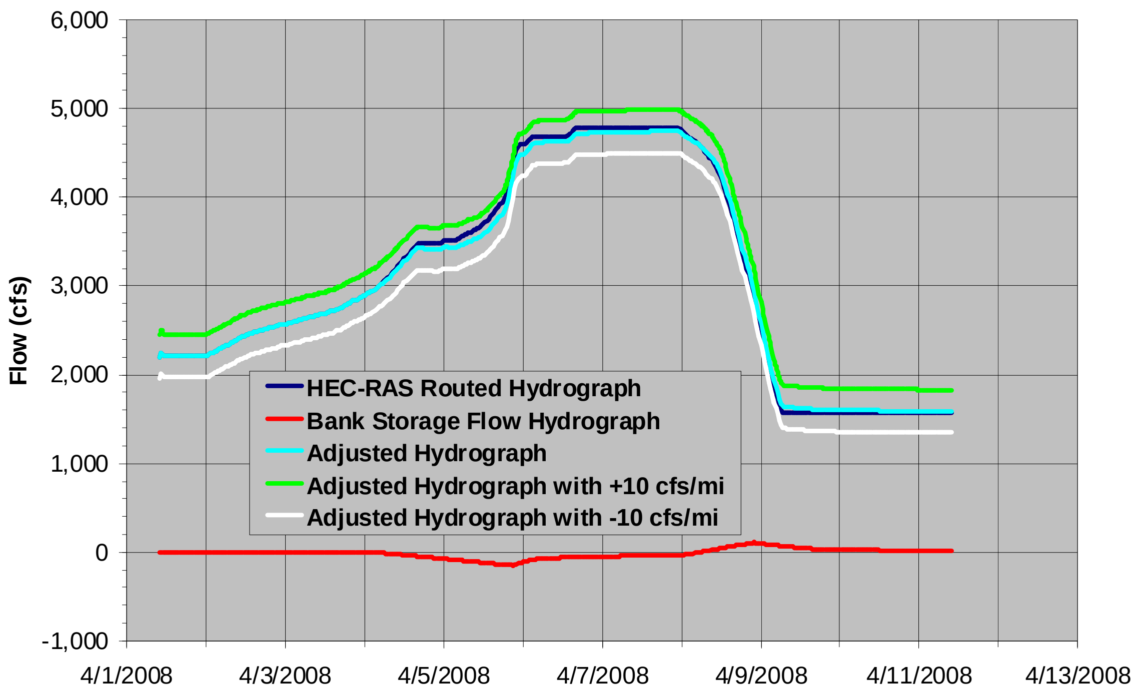

Figure 38. Possible, pulse-flow scenario 1 routed to the stream gage near Kearney, Nebraska.

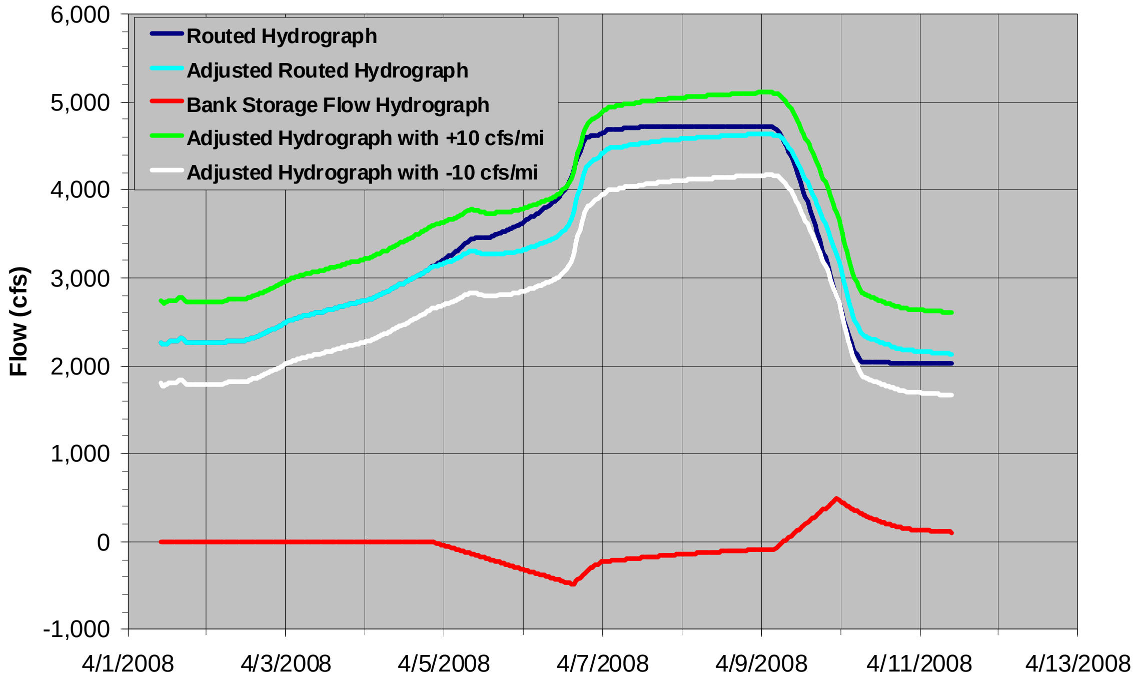

Figure 39. Possible, pulse-flow scenario 1 routed to the stream gage near Grand Island, Nebraska.

6.2 Pulse Flow Scenario 2

The bank storage and stream-flow hydrographs for flow scenario 2 are presented in Figure 40 for the stream gage near Kearney, Nebraska and in Figure 41 for the stream gage near Grand Island, Nebraska. The duration of the peak discharge is shorter (less than two days) and the peak discharges attenuated from 3,500 ft^3/s near Overton to 3,050 ft^3/s near Grand Island, Nebraska (13 percent per stream gage). After adjustments for bank storage, the predicted river flow hydrographs were adjusted by ± 10 ft^3/s per mile to account for potential stream-flow gains or losses.

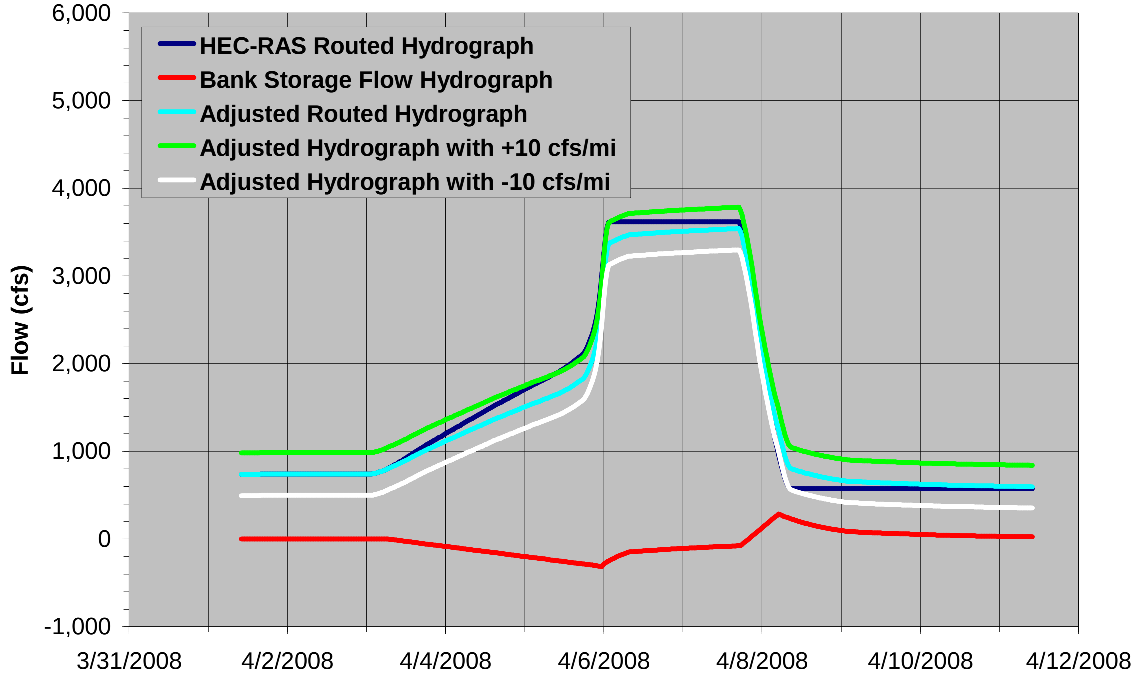

Figure 40. Possible, pulse-flow scenario 2 routed to the stream gage near Kearney, Nebraska.

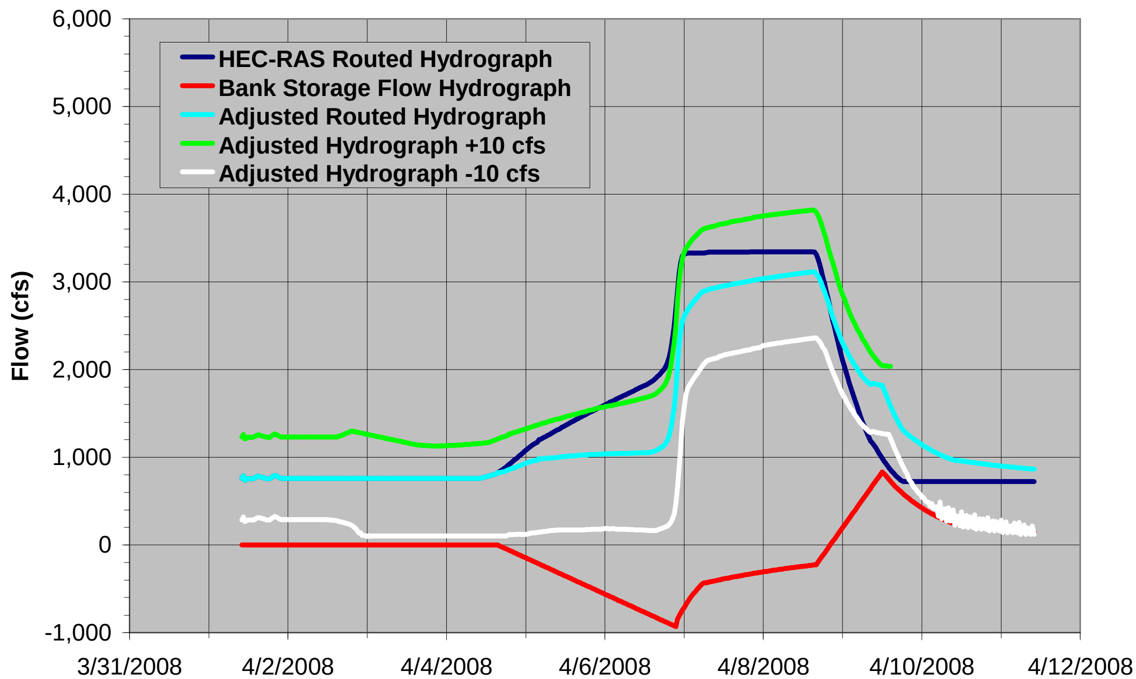

Figure 41. Possible, pulse-flow scenario 2 routed to the stream gage near Grand Island, Nebraska.

7.0 Conclusions

Discharge waves have a tendency to attenuate as they travel downstream. For the Platte River, bank storage can significantly accelerate the rate of peak-discharge attenuation, especially when flows are less than the bankfull discharge and when the moisture content in the river banks is low.

HEC-RAS models have been developed for two reaches of the Platte River from North Platte to Overton, Nebraska and from Overton to Chapman, Nebraska. In the future the model could be expanded to include the North Platte River between the Keystone Diversion Dam and North Platte, Nebraska. The model could also be expanded as a network to include the Sutherland and Tri-County Canals. Sufficient cross sections of the North Platte River are not presently available for accurate model simulations.

The HEC-RAS model (unsteady river hydraulics) is adequate to simulate natural floods when the duration of the peak is at least several days, when rainfall has increased the soil moisture content of the river banks, or when the peak discharge overtops the river banks. Predicted discharge hydrographs, from the HEC-RAS model, agreed well with measurements of floods in 1979 and 1987 (Figure 15 and Figure 16). These floods were simulated in the Platte River reach between North Platte and Overton, Nebraska where the peak discharges were greater than 5,000 ft^3/s for more than four days. However, the HEC-RAS model, by itself, cannot match measured discharge hydrographs when smaller discharge waves (within the bankfull-channel capacity) are released into the river when the banks are relatively dry. HEC-RAS model predictions were 30 percent too high at Kearney and 100 percent too high at Grand Island (Figure 17 and Figure 18).

An empirical bank storage model was developed to supplement the HEC-RAS model to improve predictive capability in peak-flow attenuation, which in turn effects hydraulic computations of depth, velocity, and travel time. The bank storage model has only been tested against measured data on the Platte River reach between Overton and Grand Island, Nebraska. Data from a discharge wave with a peak flow rate of less than 5,000 ft^3/s and peak duration of about two days would be needed to test the bank storage model in the Platte River reach between North Platte and Overton, Nebraska.

The empirical bank storage model conceptually agrees with results from the analytical and two-dimensional groundwater models. When the empirical bank storage model is applied, in conjunction with the unsteady HEC-RAS river hydraulics model, the predicted downstream hydrographs agree reasonably well with measurements in the Platte River reach between Overton and Grand Island, Nebraska (Figure 28, Figure 29, Figure 30, and Figure 31).

The empirical bank storage model should be used in conjunction with the HEC-RAS model when simulating pulse-flow releases because they are generally of short duration and the peak discharge is not to exceed the bankfull-channel capacity. The two-dimensional groundwater model should be used to simulate the bank storage if the shape of the river discharge hydrograph is highly complex or of unusual shape.

The empirical-bank storage model is ready to be validated with data from an actual pulse-flow release whenever that should occur. Data necessary for model validation would include 15-minute-interval measurements of river stage and discharge at the Platte River stream gages at North Platte, near Cozad, near Brady, near Overton, near Kearney, and near Grand Island. Predicted river depths from the HEC-RAS model could also be validated with peak river stage measurements at intervening locations. Predictions of river depth would be helpful for determining the bankfull-discharge capacity. Model predictions, prior to an actual release, could be used to predict the travel times of the discharge waves to facilitate monitoring activities.

Peak river discharge will tend to attenuate less with distance downstream if the duration of peak river discharge is increased or if the river banks are already saturated or nearly saturated. The river banks are likely to be more saturated during, or just after, winter or spring storms because these storms tend to be larger in area than more isolated, summer-thunder storms. Also, there is a greater probability for river discharge gains from groundwater and tributaries during larger storms. Pulse-flow releases scheduled during winter or spring are likely to have less attenuation of the peak discharge than if they are scheduled during the summer or fall. However, special high flow releases scheduled during large storms have a greater risk of exceeding the bankfull-discharge capacity.

8.0 References

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Murphy, P.J.; Randle, T.J.; Fotherby, L.M.; and Daraio, J.A. (2004). *The Platte River Channel: History and Restoration*, U.S. Department of the Interior, Bureau of Reclamation, Technical Service Center, Sedimentation and River Hydraulics Group, Denver, Colorado, April 2004

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Appendix A: Platte River HEC-RAS Model Execution Instructions

1. Create a new directory on your computer and save the previously provided project file, geometry file, plan file, and the unsteady flow file in this new directory.

HEC-RAS model files names for two Platte River reaches.			
Platte River reach from North Platte to Overton, Nebraska		Platte River reach from Overton to Grand Island, Nebraska	
Project file	NPOverton.prj	Project file	2005A.prj
Geometry file	NPOverton .g01	Geometry file	2005A.g01
Plan file	NPOverton.p05	Plan file	2005A.p04
Unsteady flow file	NPOverton.u03	Unsteady flow file	2005A.u01

2. Double click on the HEC-RAS icon and the HEC-RAS model window will appear.

3. Click on 'File' and then click on 'Open Project'.

4. Browse to the directory where the files in Step1 above have been saved and select the project file (either 'NPOverton.prj' or '2005A.prj') and then click on 'OK'. This will load the four files in the HEC-RAS model for the selected reach.

5. To run the simulation, go to the HEC-RAS main window and select the Unsteady Flow Analysis (click the 7th icon from the left). Check the box labeled 'post processor' and then click on 'compute' and the program will start running.

6. A 'close' button will appear after the computation has finished. From the main HEC-RAS window, you can view the unsteady model results by clicking appropriate icons on the tool bar.

   - Click the discharge hydrograph icon (14th icon from the left) to view the river stage and discharge hydrographs. For selected model cross sections, you can view the stage and discharge hydrograph, a table of the values, and a plot of the stage-discharge rating curve. You can view all three of these outputs for selected model cross sections by clicking the ↓ ↑ keys that are just right of the 'River Sta:.' view window.

   - Click the cross-section icon (9th from the left) to view plots of river cross sections. By default, the maximum water surface predicted by the model is plotted on the cross section. HEC-RAS can plot the water surface elevation associated with certain times of the hydrograph (click options, then profiles, then double click the hydrograph times of interest). You can view plots of each model cross section by clicking the ↓ ↑ keys that are just right of the river station. Cross sections that are plotted with a gray color are interpolated by the model, while cross sections that are plotted in black are the cross sections input by the user (typically the measured cross sections).

- Click the profile icon (10th from the left) to view a longitudinal profile plots of model reach. By default, the maximum water surface predicted by the model is plotted along with the lowest point (thalweg) of each cross section. HEC-RAS can plot the water surface elevation associated with certain times of the hydrograph (click options, then profiles, then double click the hydrograph times of interest). You can zoom in on the profile plot by clicking 'options' and then click 'zoom in.' Use the computer mouse to draw a box around the area you would like to zoom in on and then let go of the mouse.

7. Edit the upstream boundary discharge hydrograph by clicking on the 'enter/edit unsteady flow' icon (5th from the left). Click on the 'Flow Hydrograph' button under the heading 'Boundary Condition Types.' This will bring up the window where you can edit or enter hydrograph data.

Appendix B: Platte River Bank Storage model Execution Instructions

Introduction

The bank storage model (PULSE_ADJUST2.exe) predicts a discharge hydrograph of flow from the river channel into bank storage (negative discharge) and flow back from bank storage to the river channel (positive discharge). The primary input to the bank storage model is the discharge hydrograph, predicted by the Platte River HEC-RAS model, at a location of interest. The bank storage hydrograph, with negative and positive discharge, is then provided as a lateral flow input to the HEC-RAS model so that simulations can proceed downstream.

Execution Steps

1. Run the Platte River HEC-RAS Model for the reach of interest (Appendix A).

2. After the HEC-RAS simulation is complete, click the discharge hydrograph icon (14^{th} icon from the left) from the main window to view the river stage and discharge hydrographs.
   - Click the ↓ ↑ keys that are just right of the 'River Sta:.' view window to find the hydrograph for the cross section of interest. For example, RS: 215.012 (an interpolated cross section at the Kearney gaging station).
   - Click the 'Table' button to view the stage and discharge hydrograph numbers.
   - Click the upper left corner cell (blank) to highlight the entire hydrograph table.
   - Click the 'File' button of the 'Stage and Flow Hydrographs' window.
   - Click 'Copy to Clipboard with Headers.'

3. Open Excel and paste the hydrograph table into the worksheet and plot the hydrograph data (Table 4 and Figure 42).
   - If necessary, edit the hydrograph table in Excel to smooth out any small spikes in the flow and stage values. Such inconsistencies in the HEC-RAS model output may be present during the initial periods of low flow.
   - If necessary, extend the hydrograph data to complete the last discharge wave.

4. Get the Excel data table ready for export to the bank storage model.
   - Insert 3 blank rows below the first heading line of hydrograph table (the hydrograph discharge and stage data should begin on the 8^{th} row of the file).
   - Adjust the columns spacing of the first four columns (A, B, C, and D) to a width of 13, 15, 10, and 10 spaces (click the Excel column heading then click 'Format — Column — Width').
   - Save the edited file as a 'Formatted Text (Space delimited) (*.prn)' file (Table 4).

Table 4. Example hydrograph file from HEC-RAS with minor edits for the bank storage model.

River: Platte River Reach: Bigbend RS: 215.012*

Date		Stage INST-VAL FEET	Flow INST-VAL CFS	
117Feb2005	1100	2138.60	266.31	
217Feb2005	1130	2138.60	270.84	
317Feb2005	1200	2138.61	273.30	
417Feb2005	1230	2138.60	272.31	
517Feb2005	1300	2138.60	272.24	
617Feb2005	1330	2138.60	272.02	
717Feb2005	1400	2138.60	272.43	
817Feb2005	1430	2138.61	275.88	
917Feb2005	1500	2138.62	280.28	
1017Feb2005	1530	2138.62	282.81	
1117Feb2005	1600	2138.62	284.18	
1217Feb2005	1630	2138.63	285.31	
1317Feb2005	1700	2138.63	285.43	
1417Feb2005	1730	2138.63	285.45	
1517Feb2005	1800	2138.63	285.45	
1617Feb2005	1830	2138.63	285.58	
1717Feb2005	1900	2138.63	285.85	
1817Feb2005	1930	2138.63	285.85	
1917Feb2005	2000	2138.63	285.85	
2017Feb2005	2030	2138.63	285.85	
2117Feb2005	2100	2138.63	285.86	
2217Feb2005	2130	2138.63	285.82	
2317Feb2005	2200	2138.63	285.82	
2417Feb2005	2230	2138.63	285.86	
30323Feb2005	1800	2139.37	1045.48	
30423Feb2005	1830	2139.33	989.56	
30523Feb2005	1900	2139.29	932.85	
30623Feb2005	1930	2139.26	878.08	
30723Feb2005	2000	2139.22	824.14	
30823Feb2005	2030	2139.18	770.82	
30923Feb2005	2100	2139.15	717.06	
31023Feb2005	2130	2139.10	663.42	
31123Feb2005	2200	2139.04	614.06	
31223Feb2005	2230	2138.98	564.70	
31323Feb2005	2300	2138.92	515.34	
31423Feb2005	2330	2138.86	465.98	***extrapolated***
31524Feb2005	0000	2138.80	416.62	***values***
31624Feb2005	0030	2138.74	367.26	
31724Feb2005	0100	2138.68	317.90	

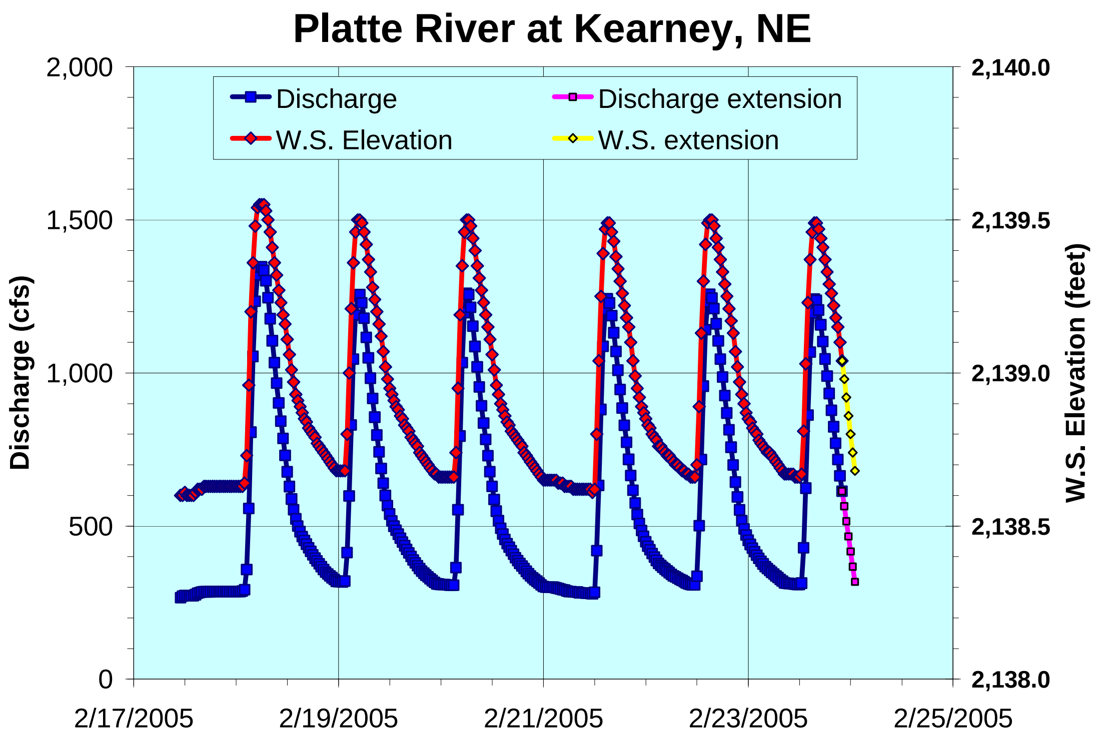

Figure 42. Excel Plot of HEC-RAS discharge and stage hydrographs.

5. Edit the Pulse-Flow Data file for relevant parameters specific to the Platte River reach. Most parameters can remain the same as the example in Table 5. The parameters that need to be changed are listed below:

   - The calibrated hydraulic conductivities are 1.0 ft/hr for the Platte River reach from Overton to Kearney and 4.0 ft/hr for the reach from Kearney to Grand Island.

   - The number of river miles may have to be changed to reflect the longitudinal distance between the cross section where the bank storage hydrograph is to be computed and the pervious upstream location, which could be the upstream boundary of the model reach.

   - The number of discharge waves may have to be adjusted to reflect the actual hydrograph.

   - The net gain or loss of river discharge from or to groundwater may have to be changed to reflect seasonal or year-to-year variations (Figure 36 and Figure 37). This can be entered as a constant gain or loss, but values for each time step can be entered as well. If the first value is not zero (positive or negative) and no other values are provided, then the first value will be treated as a constant gain or loss for the entire simulation.

Table 5. Example Pulse-Flow Data file.

```
Platte River between Overton and Kearney, Nebraska
C     Read hydraulic conductivity (H_K) [ft/hr], aquifer
C     thickness (B) [ft], and the aquifer storage coefficient (S).
      1.     50.     0.1
C  Read the rise and fall coefficients:
C     Qrise   is the rate of rise tolerance for a rising discharge
C     Qfall   is the rate of fall tolerance for a falling discharge
C     LagRF   is the number of consecutive discharge values used to
C             determine if the flow is rising, steady, or falling.
C     LAG     is the number of time steps past the time d
      0.005  -0.005  5  2
C  Read the Loss/gain storage coefficients:
C     BSL_PCT is the percentage of river stage increase (relative to the change
C             in river stage between the base flow and maximum stage) where the
C             loss rate to bank storage is at a maximum (at time Ta).
C     BSF_PCT is the percentage of river stage increase (relative to the change
C             in river stage between the base flow and maximum stage) where the
C             stage is falling enough that loss rate to bank storage is about
C             to end (at time Tb).
C     BSG_PCT is the percentage increment of river stage (relative to the change
C             in river stage between the base flow and maximum stage) during the
C             falling limb where the gain rate from bank storage is at a maximum
C             at time Tc).
    0.841  0.741  0.19
Read the number of discharge waves
    6
Read the calibrated bank storage volume coefficients
    1.000  1.000  1.000  1.000  1.000  1.000
Read the calibrated bank storage efficiency
     0.50  0.50   0.50   0.50   0.50   0.50
Read the number of river miles
    24.30
Read the net loss or gain in flow (cfs), constants for the simulation.
 25.
```

6. Edit the file named “file name.txt” to make sure it contains the exact file name of the Pulse-Flow Data file created in step 5 above (table B-3).

Table 6. Example of the 'file name.txt' file.

```
RM 215 HEC-RAS output.prn
(13X,I2,A3,I4,F4.0,I2,1X,F10.0,F10.0)
```

7. Make sure all three input files (“file name.txt”, Pulse-Flow Data file, and the HEC-RAS hydrograph file) are all in the same directory as the bank storage model executable file (PULSE_ADJUST2.exe).

8. Open windows explorer and navigate to the directory containing the three files described in step 7.

9. Run the bank storage model by double clicking the file named: PULSE_ADJUST2.exe from windows explorer. The bank storage program will create an output file called PULSE.out.

- Use Excel to open the PULSE.out file as ‘Fixed width’. Adjust the field widths (column breaks) so that the date and time (mm/dd/yyyy hh:mm) import into a single column. This time and date column is useful for creating Excel plots. The first part of the PULSE.out file is shifted to the right and contains summary information for each discharge wave (table B-4). The second part of the file contains the discharge hydrograph data (table B-5).

- Enter a formula in column I of the spreadsheet to numerically add the Net Flow Adjustment (column F) to the HEC-RAS model hydrograph (column H). Copy this formula for each time step in the spreadsheet.

- Plot the hydrograph routed by HEC-RAS model, the bank storage hydrograph, and the river discharge hydrograph after adjustment for bank storage (Figure 43).

- Change the file name and save the data as an Excel file.

- If the adjusted hydrograph appears reasonable, then proceed to step 10. If not, then carefully check the HEC-RAS model discharge and stage hydrographs and the input values in the Pulse-Flow Data file.

Table 7. Example bank storage model output part 1: Time index and discharge for each discharge wave K.

K	Nstart	Q	Ta	Q	Npeak	Q	Tb	Q	Tc	Q	Td	Q
1	31	291.86	36	1,234.23	39	1,346.21	40	1,334.97	64	394.84	79	413.30
2	81	830.29	82	1,045.22	85	1,256.52	86	1,229.09	110	389.28	130	364.22
3	132	793.95	134	1,197.12	135	1,260.52	136	1,255.81	159	387.18	195	283.79
4	197	632.64	199	1,087.18	201	1,241.93	202	1,228.44	224	387.49	243	335.72
5	245	717.05	247	1,140.87	249	1,256.70	250	1,244.77	272	394.13	292	313.36
6	294	623.55	296	1,068.12	298	1,240.84	299	1,237.30	316	367.26	317	317.90
7	317	317.90										

Table 8. Example bank storage model output part 2: Discharge hydrograph values for each time step.

Index	Date	Decimal Date	Bank Storage (cfs)	Ground-Water Gain or Loss (cfs)	Net Flow Adjust (cfs)	River Rise, Fall, or Steady (+1,0,-1)	HEC-RAS Model Output (cfs)	Adjusted River Hydrograph (cfs)
1	2/17/2005 11:00	732,009.458	0.00	25.00	25.00	0	266.30	291.3
2	2/17/2005 11:30	732,009.479	0.00	25.00	25.00	0	270.80	295.8
3	2/17/2005 12:00	732,009.500	0.00	25.00	25.00	0	273.30	298.3
4	2/17/2005 12:30	732,009.521	0.00	25.00	25.00	0	272.30	297.3
⋮	⋮	⋮	⋮	⋮	⋮	⋮	⋮	⋮
27	2/18/2005 00:00	732,010.000	0.00	25.00	25.00	0	285.80	310.8
28	2/18/2005 00:30	732,010.021	0.00	25.00	25.00	0	285.80	310.8
29	2/18/2005 01:00	732,010.042	0.00	25.00	25.00	0	285.90	310.9
30	2/18/2005 01:30	732,010.063	0.00	25.00	25.00	0	287.80	312.8
31	2/18/2005 02:00	732,010.083	0.00	25.00	25.00	1	291.90	316.9
32	2/18/2005 02:30	732,010.104	-96.72	25.00	-71.72	1	358.00	286.3
33	2/18/2005 03:00	732,010.125	-193.45	25.00	-168.45	1	557.40	389.0
34	2/18/2005 03:30	732,010.146	-290.17	25.00	-265.17	1	806.40	541.2

35	2/18/2005 04:00	732,010.167	-386.90	25.00	-361.90	1	1,054.10	692.2
36	2/18/2005 04:30	732,010.188	-483.62	25.00	-458.62	1	1,234.20	775.6
37	2/18/2005 05:00	732,010.208	-439.60	25.00	-414.60	1	1,319.80	905.2
38	2/18/2005 05:30	732,010.229	-421.14	25.00	-396.14	1	1,345.50	949.4
39	2/18/2005 06:00	732,010.250	-403.47	25.00	-378.47	0	1,346.20	967.7
40	2/18/2005 06:30	732,010.271	-386.53	25.00	-361.53	-1	1,335.00	973.5
41	2/18/2005 07:00	732,010.292	-363.92	25.00	-338.92	-1	1,301.40	962.5
42	2/18/2005 07:30	732,010.313	-341.31	25.00	-316.31	-1	1,245.80	929.5
43	2/18/2005 08:00	732,010.333	-318.69	25.00	-293.69	-1	1,177.30	883.6
44	2/18/2005 08:30	732,010.354	-296.08	25.00	-271.08	-1	1,105.40	834.3
45	2/18/2005 09:00	732,010.375	-273.47	25.00	-248.47	-1	1,034.10	785.6
46	2/18/2005 09:30	732,010.396	-250.86	25.00	-225.86	-1	966.30	740.4
47	2/18/2005 10:00	732,010.417	-228.25	25.00	-203.25	-1	902.20	699.0
48	2/18/2005 10:30	732,010.438	-205.63	25.00	-180.63	-1	842.50	661.9
49	2/18/2005 11:00	732,010.458	-183.02	25.00	-158.02	-1	786.30	628.3
50	2/18/2005 11:30	732,010.479	-160.41	25.00	-135.41	-1	730.90	595.5
51	2/18/2005 12:00	732,010.500	-137.80	25.00	-112.80	-1	676.90	564.1
52	2/18/2005 12:30	732,010.521	-115.19	25.00	-90.19	-1	628.90	538.7
53	2/18/2005 13:00	732,010.542	-92.57	25.00	-67.57	-1	588.30	520.7
54	2/18/2005 13:30	732,010.563	-69.96	25.00	-44.96	-1	553.00	508.0
55	2/18/2005 14:00	732,010.583	-47.35	25.00	-22.35	-1	523.40	501.1
56	2/18/2005 14:30	732,010.604	-24.74	25.00	0.26	-1	500.00	500.3
57	2/18/2005 15:00	732,010.625	-2.13	25.00	22.87	-1	481.50	504.4
58	2/18/2005 15:30	732,010.646	20.48	25.00	45.48	-1	465.20	510.7
59	2/18/2005 16:00	732,010.667	43.10	25.00	68.10	-1	452.80	520.9
60	2/18/2005 16:30	732,010.688	65.71	25.00	90.71	-1	440.60	531.3
316	2/24/2005 00:30	732,016.021	314.41	25.00	339.41	-1	367.30	706.7
317	2/24/2005 01:00	732,016.042	307.30	25.00	332.30	-1	317.90	650.2

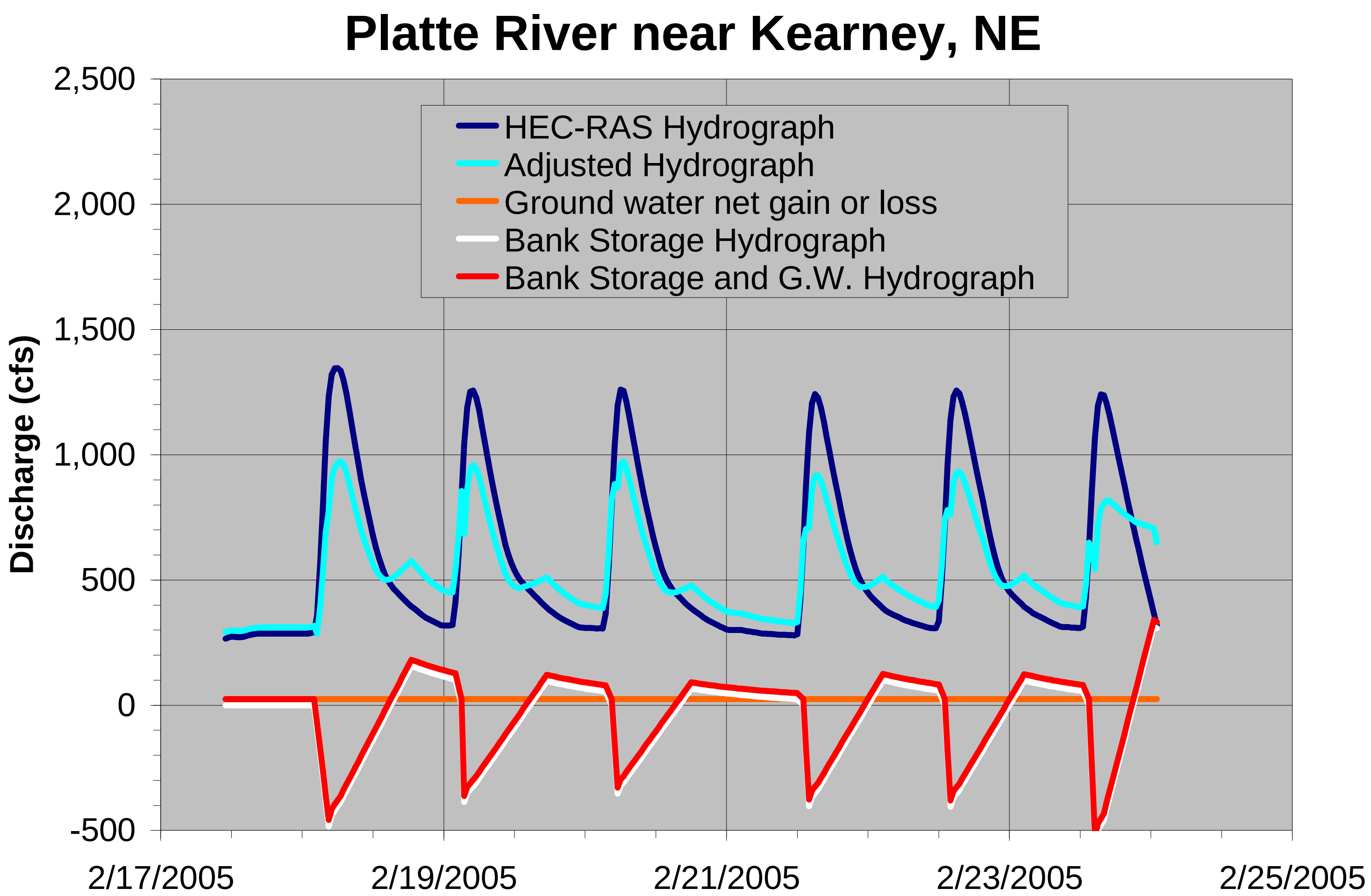

Figure 43. Example hydrograph output is presented from the bank storage model.

10. Insert the bank storage hydrograph in the HEC-RAS model as a lateral-inflow hydrograph. In the HEC-RAS model, the "Lateral Inflow Hydrograph" is used as an internal boundary condition. This option allows the user to bring in flow at a specific point along the stream. The user attaches this boundary condition to the river station of the cross section just upstream of where the lateral inflow will come in. The actual change in flow will not show up until the next cross section downstream from this inflow hydrograph. Implement the following steps to input the lateral inflow hydrograph into the HEC-RAS model:

    - Click the 'enter/edit unsteady flow' icon (5th from the left) to open the hydrograph input window.

    - Select the desired river mile from the 'River Sta.:' window.

    - Click the 'Add a Boundary Condition Location' button and then click the 'Lateral Inflow Hydr.' button.

    - Change the data time interval to 30 minutes.

    - Click the 'No. Ordinates' button and change the value to the number of time steps from the hydrograph adjustment file. The 'No. Ordinates' button is the left most button just above the green 'Hydrograph Data' cell.

    - From the Excel file, copy the cells of hydrograph adjustment column (column F), then highlight all the cells in the HEC-RAS 'Lateral Inflow' column and press [control-V] to paste in the data. Click the 'Plot Data' button to verify the results. Close the plot window and then click 'OK.'

    - Click the 'Apply Data' button and then close the unsteady flow input window.

    - Run the HEC-RAS unsteady flow model (Appendix A).