Greeley, CO
General Investigations Study

Feasibility Study
Hydraulics Analysis

U.S. Army Corps of Engineers
Omaha District
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Table of Contents

Appendix A Hydraulic Modeling

Appendix B Uncertainty Analysis

List of Tables

List of Plates

1. INTRODUCTION

This report describes the hydraulic analysis conducted for the Greeley, CO, G.I. Study. The primary goal of the study is to evaluate possible projects for a combination of flood damage reduction and environmental restoration on the Cache la Poudre River.

The bulk of the hydraulic analysis consisted of hydraulic modeling. Results of the hydraulic model were used to evaluate water surface elevations for existing conditions as well as a number of alternatives.

An economic analysis was conducted to determine the damages reduced by various alternatives as well as the cost associated with their implementation. Described in this report is the hydraulic input to the economic model and hydraulic data relative to levee elevations and channel improvements.

Flood boundaries for the 1-percent-annual-chance (100-year) event have been delineated using the water surface profiles computed with the hydraulic model for existing conditions and the selected alternative.

2. HYDRAULIC MODELING

Hydraulic modeling was completed to evaluate water surface elevations for existing conditions as well as for several alternatives. The computed water surface profiles were used for several purposes:

- Stage-discharge relationship used in economic modeling
- Sensitivity analyses to determine uncertainty for use in economic modeling
- Preliminary screening of various flood damage reduction alternatives
- Sizing of channel improvement features
- Determination of levee elevations

Three hydraulic models (one for the main channel and two split flow reaches) used in the 2003 Flood Insurance Study (FIS) were updated and combined into one model. Additional modifications were made to the FIS model to reflect existing conditions. The updated hydraulic model was used to compute water surface profiles from the mouth of the Cache la Poudre upstream to approximately Weld County Road 23, a reach length of approximately 20 miles. The alternative evaluation focused on the reach between the U.S. Highway 85 bypass and 25th Avenue with a reach length of just over 3.5 miles. This reach was selected because preliminary screening showed high damages over a short stream length, indicating the best possibility for reducing flood damages. This reach will hereafter be referred to as the high damage reach.

2.1 General Modeling Information

The 2003 FIS model is described in detail in Reference 6. Other data used consisted of 2-ft contour mapping provided by the City, bridge plans, sketches, and inspection reports.

River station (RS) zero coincides with the confluence of the Cache la Poudre River and the South Platte River. The upstream boundary of the model is at RS 105277 (stationing in feet from the mouth), near Weld County Road 23. A general location map is given on Plate 1. The stream centerline along with the cross section layout can be seen on Plates 2-7.

The 2003 FIS model was updated and used as the base model for this analysis. Several cross sections were added to the 2003 FIS model to better define the water surface elevation in the high damage reach. For more information about the FIS model, see Reference 6. For more information on updates to the FIS model, see Appendix A.

Discharges used in this analysis are given in Table 1.

Table 1 Study Discharges

		Computed **Probability Discharge (cfs) for recurrence interval equal to:**								
Location	**River Station**	**2-Year**	**5-Year**	**10-Year**	**20-Year**	**25-Year**	**50-Year**	**100-Year**	**200-Year**	**500-Year**
Mouth	0	1,470	2,440	3,460	4,810	5,340	7,330	10,000	13,600	20,200
Eaton Draw	36060	1,500	2,490	3,520	4,880	5,410	7,420	10,100	13,700	20,300
Coalbank Creek*	47860	1,780	2,940	4,100	5,610	6,190	8,360	11,200	14,900	21,700
Law Ditch	110950	2,070	3,370	4,670	6,360	7,000	9,390	12,500	16,500	23,800

Roughness in the model was simulated using Manning's "n" values. Roughness values are unchanged from the 2003 FIS model. Overbank roughness values ranged from 0.045 for open pasture or cropland to 0.085 for urban areas. A roughness value of 0.035 was used for the channel.

Contraction and expansion (C&E) coefficients were set to 0.1 and 0.3, respectively, through most of the reach. Near bridges, the C&E coefficients were increased to 0.3 and 0.5, respectively, to reflect the losses incurred at abrupt transitions typically associated with bridges.

Appendix A gives detailed information about the existing conditions model along with a comparison between the 2003 FIS model and the updated model used in this analysis.

2.2 Modeling of Alternatives

Several alternatives were modeled to determine their effectiveness in reducing flood stages and damages. As a starting point, bridge removal was evaluated to determine the effect of the bridges on the water surface profiles. Individual bridges were completely removed from the model and the impact on the water surface elevations was evaluated.

Next, channel improvements and levees were evaluated separately. In addition to the goal of the project of both reducing flood damages and providing ecosystem restoration, results of separate modeling efforts made it apparent that a combination of levees and channel improvements would be necessary. Using channel improvements only does not reduce water surface profiles enough to significantly reduce damages while levees alone cause induced stages requiring mitigation and provide only minimal, if any, ecosystem restoration opportunities.

A combination of channel improvements and levees was then modeled and the results compared to the base model. The analysis of flood damage reduction alternatives focused on the river reach between RS 28320 (near U.S. Hwy 85) and RS 40516 (approximately halfway between 11th Ave. and 25th Ave) because of the high damages computed for this reach for existing conditions (the "high damage reach").

2.2.1 Bridge Removal

Evaluation of bridge removal was completed as a substitute for evaluating a range of bridge replacement configurations. If complete removal of a bridge had little effect on water surface profiles, then replacing the bridge would not be likely to reduce the profiles.

In general, the removal of bridges did not significantly reduce the 100-year water surface profiles. Several of the bridges in the high damage reach have been replaced relatively recently using guidance from the Warzyn Report (Reference 13). The low chord of a number of the bridges in the high damage reach (8th Street, 6th Avenue, U.P. RR, and 11th Avenue) is above the 100-year water surface elevation for existing conditions.

The remaining bridges in the high damage reach are the Hwy 85 Bypass, 5th Street, 8th Avenue, and the pedestrian/horse bridge at Island Grove Park. When the Hwy 85 Bypass bridge was removed, water surfaces upstream of the bridge did decrease. However, this decrease upstream from the bridge caused less water to flow over the highway into the split flow reach north of East 8th Street (i.e., more water stayed in the channel). Therefore, water surface elevations downstream from the Hwy 85 bypass were higher than existing conditions.

Results of the bridge removal evaluation are summarized in Table 2:

Table 2 Bridge Removal Summary

Bridge Removed	Maximum Reduction in 100-year Water Surface Elevation	Approximate Length of Stream with Reduced 100-year Water Surface Elevation
	(ft)	(ft)
US Hwy 85 Bypass	0.06	1400
8th Street	0.02	600
5th Street	0.28	3100
6th Avenue	0.64	4100
UP Railroad	0.74	1900
8th Avenue	0.52	800
11th Avenue	0.65	800
Foot/horse bridge at Island Grove Park	0.21	400

2.2.2 Channel Improvements

Initially, channel improvements were modeled without levees to determine the decrease in stage that could be obtained with channel widening alone. The main constraint for the channel improvement option is that many of the city's bridges have been recently replaced and have a good portion of their design life remaining. It is undesirable to replace these bridges due to high costs. One issue with the bridge replacements is that the new construction was based on the Warzyn report (Reference 13). The elevation of the newly constructed bridges is relatively high, but the widths are based on the efficient trapezoidal channel proposed in the Warzyn report. Therefore, a consistent channel width could not be maintained throughout the high damage reach let alone the entire study reach due to the constrictions presented by the bridges.

Note that a combination of bridge removal/channel improvements was not evaluated. Evaluating both bridge removal and channel widening simultaneously would allow for a corridor with consistent width and would reduce flood stages further than either alternative by itself.

Another constraint imposed on this evaluation was the exclusion of channel stability in the scope of work. Because no channel stability analysis was to be conducted, simplifying assumptions were made. The existing channel was used up to a certain stage rather than evaluating a range of low-flow channel configurations.

Channel improvements were made at or above the stage of a certain frequency event. Above the 2-year stage, a "bench" was coded into the model. Below this stage, the existing channel bed was used. The width of the bench varied from cross section to cross section and was based on plan view considerations including typical expansion/contraction ratios, bridge widths, structure locations, and real estate availability. The channel width for this option varied from a typical 200 ft to a maximum of over 500 ft upstream from 11th Ave.

The bench was coded in with approximately a 2% slope and then a 4H:1V slope was extended upward to the natural ground elevation at the appropriate width.

Channel improvements made above the 2-year stage added more conveyance to the channel but the constraints imposed by the bridges and lack of channel stability analysis limited the reduction in water surface elevations. The reduction in stage on the improved reach averaged approximately 0.4 ft for the 10-year event to nearly 0.5 ft for the 100-year event. Maximum reduction in water surface was realized at the 50-year event with an average of 0.6 ft.

Because the channel improvements alone would provide minimal benefits to the community and would not remove residents and businesses from the 100-year flood plain, levees were also evaluated.

2.2.3 Levees

Levees were coded into the base model (without channel improvements) from RS 28320 (U.S. Hwy 85) to RS 40516 (approximately 17th Ave) on the right bank. The levees were located near the channel banks along the alignment of the existing spoil banks. Levee elevations were set artificially high to prevent stages from all events from exceeding the top of levee.

The average increase in stage due to the levees ranged from less than 0.1 ft for the 10-year event to over 1.8 ft for the 500-year event. The average increase for the 100-year event was approximately 0.9 ft with a maximum of over 3.5 ft.

In addition to the increase in stages, a flood damage reduction plan including only levees and no ecosystem restoration features makes this alternative undesirable. Therefore, a combination of channel improvements and levees was evaluated.

2.2.4 Combination of Channel Improvements and Levees

A combination of channel improvements and levees was evaluated in order to mitigate flood damages and improve the riparian corridor for ecosystem functionality. The increase in stages caused by levees is at least partially offset by the additional conveyance provided by the channel improvements.

Numerous channel/levee configurations were evaluated with a number of variables including stream length improved/protected, width of channel improvements, and level of protection provided. Although a number of different alternatives were evaluated, three were selected for further investigation: Alternatives 1, 1a, and 5.

For all three selected alternatives, the channel improvements included benches cut just above the 2-year stage. All levees were coded in artificially high to prevent overtopping by any event. This results in higher stages for the 500-year event than would be expected. However, for the level of detail used in this evaluation, the increase in stage was deemed acceptable. Additionally, even when the levees are overtopped, the interior cells will most likely fill to the elevation of downstream confining features because of the large volume and long duration of the Cache la Poudre hydrographs. Considerable effort could be expended to accurately determine the elevation of the 500-year water surface. An unsteady flow model using additional geometry of confining features could be used to evaluate the 500-year water surface in interior areas. However, unsteady flow modeling is out of the scope of this phase of the study.

All three selected alternatives require the removal or relocation of the pedestrian/horse bridge at Island Grove Park. Further investigation will be necessary to determine the feasibility of either moving the bridge upstream to a point where it will not cause significant headlosses or raising and lengthening the bridge in its current location. Costs associated with either method should be reflected in the formulation of the benefit to cost ratio. If relocation or reconfiguration of the bridge is not feasible, the political ramifications of completely removing the bridge and rerouting foot/horse traffic may need to be investigated.

Seven alternatives are described in Appendix A. Only the three alternatives (1, 1a, and 5) that were evaluated in detail to determine the benefit to cost ratio are described here. Following the descriptions of the individual alternatives is a summary table that compares features of all alternatives (see Table 3).

2.2.4.1 Alternative 1

Alternative 1, the shortest alternative in length, focused on the two damage reaches with the highest expected annual damages (EAD) based on existing conditions, reaches 9R and 10R (see Section 3.1 for economic model setup). Reaches 9R and 10R (right bank of reaches 9 and 10) extend from RS 36780 at 8th Avenue to RS 40516 (near the east-west location of 17th Avenue).

Alternative 1 includes channel improvements from RS 30715 up to RS 40516. Levees are included on the right bank from RS 36660 to 40516 with tie-offs extending laterally from the river in order to prevent flanking. The channel cuts as well as the levee alignments can be seen on Plate 8.

2.2.4.2 Alternative 1a

Alternative 1a includes all alternative 1 features with the addition of a left bank levee from RS 36080 to RS 40516 in order to prevent induced damages from increased stages due to the levees. The alternative 1a configuration can be seen on Plate 9.

2.2.4.3 Alternative 5

Alternative 5 includes all features of Alternative 1a. Additionally, the right bank levee continues downstream to RS 28320. The levee was aligned such that it could be constructed in phases. Plate 10 shows the channel improvements and levee alignments for alternative 5.

Table 3 Comparison of Alternatives

	Bounding River Stations			Alternative 1		Alternative 1a		Alternative 2		Alternative 3		Alternative 3a		Alternative 4		Alternative 5	
Damage Reach	Downstream	Upstream	Stream Bank	Channel Cuts	Levee	Channel Cuts	Levee	Channel Cuts	Levee	Channel Cuts	Levee	Channel Cuts	Levee	Channel Cuts	Levee	Channel Cuts	Levee
5	28320	30654	Right								X		X				X
			Left							X		X				X	
6	30715	33834	Right						X		X		X				X
			Left	X		X		X		X		X				X	
7	33875	36049	Right						X		X		X				X
			Left	X		X		X		X		X				X	
8	36080	36659	Right		X		X		X		X		X		X		X
			Left				X						X		X		X
9	36780	37996	Right		X		X		X		X		X		X		X
			Left	X		X	X	X		X		X		X		X	X
10	38075	40516	Right		X		X		X		X		X		X		X
			Left	X		X	X	X		X		X		X		X	X

3. UNCERTAINTY ANALYSIS

In accordance with current USACE guidelines, (see Reference 14) an uncertainty analysis was completed to better define both existing conditions and with project flood damages. The Hydrologic Engineering Center's Flood Damage Reduction Analysis (HEC-FDA) version 1.2.3 dated June 2005 was used in the analysis.

The HEC-FDA model runs were executed by the USACE economist. However, input from the hydrologic and hydraulic sections of the USACE were used in model setup.

3.1 HEC-FDA Setup and Hydraulic Input

Setup of the HEC-FDA model is described in detail in Appendix B. Table 4 summarizes the damage reach configuration.

Hydraulic input to the HEC-FDA model consisted of water surface profiles and a standard deviation value representing hydraulic uncertainty. Water surface profiles were input for eight flood events (2, 5, 10, 20, 25, 50, 100, and 500-year). The 20-year event was used instead of the 200-year event in order to better define the stage-discharge relationship near the top of the existing spoil banks (which lies near the 20 to 25-year event range).

Table 4 HEC-FDA Setup

Stream Name	Damage Reach Name	Downstream Station	Upstream Station	Stream Bank	Index Station	Description
Cache la Poudre	1R	3464	14952	Right	10998	Confluence with S. Platte River to Fern Ave
Cache la Poudre	1L	3464	14952	Left	10998	Confluence with S. Platte River to Fern Ave
Cache la Poudre	2R	14996	26048	Right	19353	Fern Ave to Ash Ave
Cache la Poudre	2L	14996	26048	Left	19353	Fern Ave to Ash Ave
Cache la Poudre	3R	26087	28199	Right	27305	Ash Ave to U.S. Highway 85 Bypass
Cache la Poudre	3L	26087	28199	Left	27305	Ash Ave to U.S. Highway 85 Bypass
Split Left	4R	10	19	Right	17	Delwood Ave to U.S. Highway 85 Bypass
Split Left	4L	10	19	Left	17	Delwood Ave to U.S. Highway 85 Bypass
Cache la Poudre	5R	28320	30654	Right	29190	U.S. Highway 85 Bypass to 5th Street
Cache la Poudre	5L	28320	30654	Left	30029	U.S. Highway 85 Bypass to 5th Street
Cache la Poudre	6R	30715	33834	Right	32153	5th Street to 6th Ave
Cache la Poudre	6L	30715	33834	Left	32153	5th Street to 6th Ave
Cache la Poudre	7R	33875	36049	Right	35655	6th Ave to U.P. Railroad
Cache la Poudre	7L	33875	36049	Left	35655	6th Ave to U.P. Railroad
Cache la Poudre	8R	36080	36659	Right	36257	U.P. Railroad to 8th Ave (U.S. Hwy 85)
Cache la Poudre	8L	36080	36659	Left	36257	U.P. Railroad to 8th Ave (U.S. Hwy 85)
Cache la Poudre	9R	36780	37996	Right	37700	8th Ave (U.S. Hwy 85) to 11th Ave
Cache la Poudre	9L	36780	37996	Left	37700	8th Ave (U.S. Hwy 85) to 11th Ave
Cache la Poudre	10R	38075	40516	Right	40230	11th Ave to approximately 16th Ave
Cache la Poudre	10L	38075	40516	Left	40230	11th Ave to approximately 16th Ave
Cache la Poudre	11R	40516	47750	Right	45065	Approximately 16th Ave to 25th Ave
Cache la Poudre	11L	40516	47750	Left	45065	Approximately 16th Ave to 25th Ave
Cache la Poudre	12R1	47783	48457	Right	47783	25th Ave to 27th Ave
Cache la Poudre	12L1	47783	48457	Left	47783	25th Ave to 27th Ave
Cache la Poudre	12R	48457	51998	Right	48457	27th Ave to 35th Ave
Cache la Poudre	12L	48457	51998	Left	48457	27th Ave to 35th Ave
Cache la Poudre	13R	52121	52718	Right	52234	35th Ave to approximately 39th Ave
Cache la Poudre	13L	52121	52718	Left	52234	35th Ave to approximately 39th Ave
Cache la Poudre	13R1	52718	60043	Right	54064	Approximately 39th Ave to approximately 49th Ave
Cache la Poudre	13L1	52718	60043	Left	54064	Approximately 39th Ave to approximately 49th Ave
Cache la Poudre	13R2	60043	72136	Right	61694	Approximately 49th Ave to 59th Ave
Cache la Poudre	13L2	60043	72136	Left	72017	Approximately 49th Ave to 59th Ave
Split Right	14R	19	23	Right	19	35th Ave to 47th Ave
Cache la Poudre	15R	72191	84514	Right	80402	59th Ave to just downstream from 83rd Ave
Cache la Poudre	15L	72191	84514	Left	78718	59th Ave to just downstream from 83rd Ave

3.2 Hydraulic Uncertainty

The stage-discharge function for each reach is based on the water surface profiles computed with the HEC-RAS model at the index station. HEC-FDA input requires the description of stage uncertainty of the computed water surface profiles. Uncertainty in computed stage profiles reflects modeling assumptions, numerical errors, and parameter estimation. Uncertainty was estimated for the entire study reach by performing a sensitivity analysis with the HEC-RAS model and also by evaluating the variation in historic measured discharge values at the USGS gaging station.

Determination of the standard deviation is described in detail in Appendix B.

Within the FDA model the total standard deviation of 1.3 ft at the 100-year event was employed at all locations. Between the channel invert and the 100-year stage, standard deviation values are interpolated while the value of 1.3 ft is used for all stages above the 100-year event.

3.3 Levee Assurance

One item of interest related to hydraulics is the computed conditional non-exceedance probability (CNP). The CNP, also referred to as assurance, is a measure of the levee's ability to contain a specified event while considering uncertainty. In order to certify a levee as giving 100-year protection, 90% CNP must be demonstrated (see reference 14).

Preliminary HEC-FDA model executions indicated that 90% (or greater) CNP could be attained for the 100-year event with levee top elevations at three feet above the 100-year water surface elevation (i.e. the model indicates that three feet of "freeboard" is sufficient to allow levee certification).

In determining earthwork quantities (see Geotechnical appendix) for the alternative evaluation, levee top elevations were set to three feet above the 100-year water surface elevation in most cases. However, inclusion of the left bank levee in alternatives 1a and 5 induces damages when the levee is only three feet above the 100-year water surface elevation. Therefore, in order to eliminate induced damages, the left bank levee in the vicinity of the beef plant was increased to four feet above the 100-year water surface elevation. The levee top then tapers down to three feet above the 100-year water elevation upstream from 8th Avenue.

4. EVALUATION OF ALTERNATIVES

Three alternatives were evaluated for economic feasibility: Alternative 1, Alternative 1a, and Alternative 5 (as described in section 2.5.4). The result of the evaluation is a benefit to cost (B/C) ratio for each alternative. For more information on the evaluation, see the Geotechnical, Cost Engineering, and Economics appendices. The B/C ratios for the selected alternatives fall between 1.01 and 1.10 (see Table 5). Although the B/C ratios are low, they are above unity indicating a federal interest in the project.

Table 5 Benefit to Cost Ratio

Benefit /Cost Ratio			
	Alternative 1	Alternative 1a	Alternative 5
Total Annual Project Benefits	$432,374	$548,009	$685,852
Total Annual Project Costs	$427,940	$497,529	$672,649
Net Annual Benefits	$4,434	$50,480	$13,203
Benefit /Cost Ratio	1.01	1.10	1.02

5. CONCEPTUAL PHASE 2 STUDY ALTERNATIVES

Several potential flood damage reduction and ecosystem restoration ideas are outlined in this section. The evaluation of these ideas was outside of the scope of the Phase 1 analysis. However, their implementation could further reduce flood damages as well as provide ecosystem benefits.

5.1 Bypass Channel

Discussions of a bypass channel in the vicinity of the U.S. Highway 85 Bypass/Wastewater Treatment Plant began during a site visit to Greeley during September 2006. Although the exact alignment of a bypass channel is open for discussion, the channel would, in general, branch off from the main channel of the Cache la Poudre somewhere between 6th Avenue and the U.S. Highway 85 bypass, flow under the bypass (a bridge or culverts would be required) and join back up to the Cache la Poudre somewhere downstream from the wastewater treatment plant.

The intent of the bypass channel is twofold: to circumvent the hydraulic constriction caused by the wastewater treatment plant and to provide a continuous riparian corridor for both ecosystem and recreational purposes. A conceptual layout of possible alignments is given in Plate 11.

Design of the bypass channel would focus on reducing flood flows that currently leave the main channel of the Cache la Poudre between 5th Street and 8th Street and flow over the U.S. Highway 85 bypass on the north side of 8th Street. Most likely, some sort of diversion structure would be required to control flow into the bypass channel. One possibility would be to have a small notch at a lower elevation to allow baseflow through the bypass channel along with a longer weir at a higher elevation to allow higher flow during flood events. The lateral structure could be constructed of rock riprap and be unregulated or could include automatic or manually operated gates to regulate the flow through the bypass. Numerous possibilities exist for the configuration of the diversion structure.

The potential to use what appear to be former Cache la Poudre meander paths makes the bypass channel an attractive alternative. The old meander paths can be seen on Conceptual Alignment 1 in Plate 11.

Currently, the location of the wastewater treatment plant and its proximity to the Cache la Poudre creates a discontinuity in the corridor. In addition to reducing flood damages, the bypass channel would provide a continuous riparian corridor and the opportunity for a continuous recreational trail.

5.2 Channel Improvements

A more detailed evaluation of channel improvements has the potential to demonstrate further reduction in flood stages.

Because no channel stability analysis was completed in Phase 1, the channel below the 2-year stage was left unchanged as a simplifying assumption. A more hydraulically efficient low flow channel would add conveyance and reduce flood stages.

Additionally, a combination of channel improvements and bridge removal/replacement was not evaluated. As noted in section 2.2, bridges throughout the high damage reach pose a constraint on channel widening. Although numerous combinations of bridge removal/channel improvement configurations are possible, it is likely that only a select few, if any, would be desirable for the sponsor.

6. RESULTS

6.1 Water Surface Profiles

Tables of the existing condition water surface profiles are presented on Plates 12-14.

Existing conditions water surface profiles are plotted on Plates 15-18 (main channel), Plate 19 (left split) and Plate 20 (right split). Plates 15-18 include eight profiles (2-, 5-, 10-, 20-, 25-, 50-, 100-, and 500-year events). Plate 19 only shows the 20-, 25-, 50-, 100-, and 500-year profiles because the main channel profiles indicate that events below the 20-year flood do not exceed the U.S. Hwy 85 bypass elevations. Plate 20 shows only the 50-, 100-, and 500-year profiles based on similar reasoning.

The water surface profiles given on Plates 12-20 were computed assuming the uncertified spoil bank levees provide no protection.

A separate model was created utilizing the levee feature within HEC-RAS to compute "confined" water surface profiles (not included in this ITR). The levees were set artificially high such that the water surface profiles were always contained within the banks. These "confined" profiles were then compared against the toe of the spoil banks to determine the approximate discharge (and corresponding recurrence interval) when the base of the spoil bank would be reached. The recurrence interval determined as described above was used only for the economics evaluation. Any damages computed below the spoil bank toe recurrence interval were eliminated from the final damage total using the levee feature within HEC-FDA.

Economics used the unconfined water surface profiles to compute damages to structures based on the assumption that once the water surface exceeds the base of the spoil banks, the banks will breach and erode to natural ground.

Water surface profiles for the three selected alternatives are given in tabular format in Plates 21-29. The 100-year water surface profiles for the three selected alternatives are compared with the existing conditions 100-year water surface profile on Plate 30 for the high damage reach only (profiles are identical upstream and downstream).

6.2 Flood Boundaries

Flood boundaries for the 1% chance exceedance event (100-year flood) for two conditions have been plotted. Plates 31-36 shows the boundaries for existing conditions and for Alternative 1a.

Note: the flood boundaries on Plates 31-36 are taken as-is from the automated flood boundary plotting using the MicroStation/InRoads software package. The flood boundaries were not fine-tuned or adjusted and there may be situations where flood boundaries cross contour lines. Additionally, any "islands" or "potholes" located throughout the digital terrain model (DTM) supplied by the sponsor were not cleaned up. The flood boundaries are not suitable for a larger scale map than that on Plates 31-36, a scale of 1 inch equals 1000 feet.

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13. **Warzyn Engineering Inc.** "Final Design Report: Final Engineering for the Master Floodway Plan for the Greeley Reach of the Cache la Poudre River, Greeley, Colorado." May 1986.

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Appendix A Hydraulic Modeling

The Hydrologic Engineering Center-River Analysis System (HEC-RAS) version 4.0 Beta was used for hydraulic modeling. HEC-RAS is a one-dimensional hydraulic model that was developed to calculate water surface profiles for a uniform, steady state flow by the standard step method. The standard step method computational procedure is based on the solution of the one-dimensional energy equation and friction loss evaluated with Manning's equation.

The main input data requirement for HEC-RAS is stream and flood plain geometry in the form of cross sections taken perpendicular to the direction of flow. HEC-RAS has the ability to model various obstructions such as bridges, culverts, and structures.

A-1 ASSUMPTIONS AND LIMITATIONS

Based on the sponsor's desire for a minimum effort, least cost study, existing data was to be used to the maximum extent possible. For this reason, the HEC-2 model developed for the 2003 FIS was the base model for this study. The HEC-2 model was converted to HEC-RAS and updated to reflect existing conditions.

The HEC-RAS model water surface profiles are based on the assumption that the uncertified spoil bank levees on the Cache la Poudre provide no protection. As described in Appendix B, economic damages were computed with these profiles.

A-2 BASE HYDRAULIC MODEL

The FIS model is described in detail in Reference 6. Other data used consisted of 2-ft contour mapping provided by the City, bridge plans, sketches, and inspection reports.

River station (RS) zero coincides with the confluence of the Cache la Poudre River and the South Platte River. The upstream boundary of the model is at RS 105277 (stationing in feet from the mouth), near Weld County Road 23. A general location map is given on Plate 1. The stream centerline along with the cross section layout can be seen on Plates 2-7.

The available bridge data consisted of plans for 4 of the 17 bridges and sketches, inspection reports, or no data for the remainder. See Table A-1 for a summary of the available bridge data:

Table A-1 Available Bridge Data

Bridge #	RS	Description	Have bridge plans	Have sketches	Have inspection report	Have nothing
1	14894	Weld Co. Rd 45 (Fern Avenue)		X	X	
2	26068	Ash Avenue		X		
3	28260	U.S. Hwy 85 Bypass				X
4	29155	East 8th Street				X
5	30685	5th Street				X
6	33855	6th Avenue				X
7	36065	UP Railroad				X
8	36720	U.S. Hwy 85				X
9	38036	11th Ave	X			
10	39420	Foot/horse bridge at Island Grove Park				X
11	47767	25th Ave		X	X	
12	52029	Weld Co. Rd 35 (N. 35th Ave)		X	X	
13	60033	Railroad bridge		X		
14	72164	Weld Co. Rd 31 (N. 59th Ave)	X	X		
15	78846	Weld Co. Rd 29 (N. 71st Ave)	X	X	X	
16	89328	Weld Co. Rd 27 (83rd Ave)	X	X	X	
17	97209	Weld Co. Rd 25			X	

A-2.1 Conversion from HEC-2 to HEC-RAS

In order to get a running, georeferenced HEC-RAS model from the FIS HEC-2 model, the following steps were completed:

1. The base model was imported into HEC-RAS using the automated import routine.
2. The cross sections were then georeferenced using the following procedure: used Arc shape file depicting stream centerline and cross section cut lines, exported to dxf, created surface using InRoads, then exported ASCII with x-y coordinates.
3. Switched coordinates on the cross section schematic so that all cross sections were oriented from left to right looking downstream.
4. Used the cross section points filter to remove duplicate points in station-elevation data.
5. Fixed bridge width-first increased reach lengths of xsects just upstream of bridges by 1 ft, then renamed river stations just below bridges (minus 1 ft). Then adjusted the “Upstream Distance” in the bridges to 0.5 ft so that there is 0.5 ft upstream and downstream of the bridges.
6. Renamed profiles and flow file.
7. Fixed bridge piers. Some bridges were imported with the piers defined both as pier obstructions and in the deck/roadway data. Piers entered as deck station/elevations were deleted and entered only as piers.
8. Imported left split flow area and added lateral structures on left bank of the Cache la Poudre. Changed some reach lengths based on GIS measurements so that the lateral structures would fit.
9. Added lateral structures on the right bank of the left flow split reach to return flow to the main channel.

10. Connected left split flow reach back to the Cache la Poudre with a junction.
11. Imported right split flow reach and connected back to Poudre main channel with a junction.
12. Coded ineffective areas and blocked obstructions according to base HEC-2 models.

A-2.2 HEC-2 Model vs. Converted HEC-RAS Model

The HEC-RAS model was first adjusted to match the FIS water surface profiles as closely as possible.

Table A-2 compares the 100-year and 500-year flow splits determined in the FIS model to those computed using the converted HEC-RAS model.

The 100-year water surface profiles from the FIS model and the converted HEC-RAS model are compared in tabular format on Plates 37-39.

The following conclusions pertaining to the 100-year water surface elevation comparison are made from Plates 37-39:

- At most cross sections, the WS elevations are within 0.2 ft (99 out of 114)
- 106 out of 114 cross sections are within 0.5 ft
- Of the eight cross sections not within 0.5 ft, seven of them are directly upstream from RS 37996 and RS 38075. The placement of blocked obstructions in these two sections block out half of the channel in the FIS model (see Plate 40). The blocked obstructions were moved to the correct station in the updated model.
- There are several other cross sections that are off between 0.2 and 0.5 ft. All are just upstream of bridges. It is assumed that the computational differences between HEC-2 and HEC-RAS account for the difference in water surface elevations.

Table A-2 Flow Splits: HEC-2 vs. HEC-RAS

Reach	River Sta	Profile	Q US	Q Leaving Total	Q DS	Perc Q Leaving		Totals	**FIS Splits**	Difference	Percent difference
			(cfs)	(cfs)	(cfs)			(cfs)	**(cfs)**	(cfs)	
Upper	61000	100-year	12000	1633	10067	14	Right	1633	**1705**	72	4.2
Middle	31520	100-year	10700	0	10600	0	Left	3388	**3230**	-158	4.9
Middle	30653	100-year	10600	0	10600	0					
Middle	30213	100-year	10600	0	10600	0					
Middle	30028	100-year	10600	0	10600	0					
Middle	29809	100-year	10600	84	10516	1					
Middle	29443	100-year	10516	2005	8514	19					
Middle	29118	100-year	8514	983	7533	12					
Middle	28771	100-year	7533	316	7218	4					
Upper	61000	500-year	24650	10491	13683	43	Right	10491	**10705**	214	2.0
Middle	31520	500-year	22700	2086	20701	9	Left	13647	**12870**	-777	6.0
Middle	30653	500-year	20701	445	20261	2					
Middle	30213	500-year	20261	408	19868	2					
Middle	30028	500-year	19868	526	19357	3					
Middle	29809	500-year	19357	1673	17728	8					
Middle	29443	500-year	17728	4960	12849	28					
Middle	29118	500-year	12849	2157	10727	17					
Middle	28771	500-year	10727	1392	9381	13					

A-2.3 Changes from Converted to Existing Conditions HEC-RAS

Once the FIS water surface profiles were matched, the HEC-RAS model was updated further to reflect existing conditions and methodology. The following are changes made to the FIS model resulting in differences in the computed water surface elevations:

A-2.3.1 General Changes

- Encroachments in the form of ineffective flow areas were used in the HEC-RAS model to reflect areas of minimal to zero conveyance. Near bridges, ineffective flow areas were placed as needed such that the active top width for the 100-year discharge maintains smooth transitions (plan view). Where possible, a 1:1 contraction ratio and 1:2 expansion ratio was maintained.
- Definition of bridges and bridge piers was adjusted for consistency and to reflect updated HEC-RAS methodology. In HEC-2, the piers of some bridges were defined both as a pier obstruction in the special bridge record and in the deck geometry. The updated HEC-RAS model uses piers instead of added points in the definition of the deck.
- The FIS model used the special bridge routine for 13 of the 17 bridges. The special bridge routine in HEC-2 uses the Yarnell equation for Class A low flow (subcritical). According to the HEC-RAS Hydraulic Reference Manual, the Yarnell equation should only be used at bridges where the majority of energy losses are associated with piers. The updated HEC-RAS model uses the energy equation for all bridges.
- The 2003 FIS model utilized expected probability discharges from 1988 hydrology (see Reference 12). The current study has adopted computed probability discharges from the 2006 updated hydrology (see hydrology appendix). The discharges from the two studies are compared in Table A-3.

A-2.3.2 Location Specific Changes

- Ineffective flow areas were adjusted at the following river stations in order to prevent profile crossings:
    - 14959
    - 14996
    - 33834
    - 33875
    - 36659
    - 36780
    - 52059
    - 52159

- 60023
- 60043

- Contraction and expansion coefficients were increased from 0.1 and 0.3 to 0.2 and 0.4, respectively, at river stations 39406, 39434, and 39814. Values of 0.2 and 0.4 were used instead of the typical values of 0.3 and 0.5 at bridges because the bridge at 39420 is a pedestrian bridge with no piers. The channel cross section in the vicinity of this bridge does not change abruptly.

A-3 GENERAL MODELING INFORMATION

The general modeling information presented in this section applies to both the existing conditions analysis and the evaluation of alternatives. Information specific to either existing conditions or alternatives is presented in subsequent sections (A-4 and A-5).

A-3.1 Discharges

Study discharges were computed by hydrology section during the 2006 hydrologic analysis. The discharges are summarized in Table A-3. Computed probability discharges were adopted for this study. Discharges used in the 2003 FIS are also given in Table A-3 for comparison. For more information on study discharges, refer to the hydrology appendix.

Table A-3 Study Discharges vs. Other Discharges

THESE DISCHARGES ADOPTED FOR USE IN THIS STUDY										
		Computed Probability Discharge (cfs) for recurrence interval equal to:								
Location	**River Station**	**2-Year**	**5-Year**	**10-Year**	**20-Year**	**25-Year**	**50-Year**	**100-Year**	**200-Year**	**500-Year**
Mouth	0	1,470	2,440	3,460	4,810	5,340	7,330	10,000	13,600	20,200
Eaton Draw	36060	1,500	2,490	3,520	4,880	5,410	7,420	10,100	13,700	20,300
Coalbank Creek*	47860	1,780	2,940	4,100	5,610	6,190	8,360	11,200	14,900	21,700
Law Ditch	110950	2,070	3,370	4,670	6,360	7,000	9,390	12,500	16,500	23,800

These discharges were developed in the 2006 hydrology update and are given for comparison purposes only.										
		Expected Probability Discharge (cfs) for recurrence interval equal to:								
Location	**River Station**	**2-Year**	**5-Year**	**10-Year**	**20-Year**	**25-Year**	**50-Year**	**100-Year**	**200-Year**	**500-Year**
Mouth	0	1,470	2,460	3,490	4,900	5,460	7,590	10,500	14,500	22,200
Eaton Draw	36060	1,500	2,500	3,550	4,970	5,530	7,680	10,600	14,600	22,300
Coalbank Creek*	47860	1,780	2,950	4,140	5,710	6,320	8,630	11,700	15,900	23,600
Law Ditch	110950	2,070	3,390	4,720	6,470	7,150	9,690	13,100	17,600	26,000

These discharges were developed in the 1988 hydrologic analysis and are given for comparison purposes only.					
		Expected Probability Discharge (cfs) for recurrence interval equal to:			
Location	**River Station**	**10-year**	**50-year**	**100-year**	**500-year**
Mouth	0	3310	7550	10600	22800
At Eaton Draw	36060	3400	7660	10700	22700
Downstream of Coalbank Creek*	47860	3900	8540	11800	24400
Upstream of Coalbank Creek*	47860	3870	8470	11700	24200
Downstream of Law Ditch	110950	4620	9720	13200	26300
Upstream of Law Ditch	110950	4590	9640	13100	26100

*Coalbank Creek is shown on most maps as Graham Seep.

A-3.2 Cross Section Geometry

Cross section geometry for the majority of the model was taken from the 2003 FIS model. The topography used to create the cross sections is based on a number of sources, including:

- Aerial photogrammetry (May 1987)
- Ground surveys (unknown date)
- Aerial photogrammetry (December 2001)

Five cross sections were added in the high damage reach. The cross sections were added to better define channel improvements in reaches where the cross sectional density was inadequate. The added cross sections are at RS 32931, 33508, 35655, 36257, and 36467. The additional cross sections allow for channel improvements within stream reaches that would have otherwise shown no benefits due to the constraint imposed by not evaluating bridge modifications. Geometry for the five new cross sections was taken from 2005 aerial photogrammetry data supplied by the City of Greeley in the form of a MicroStation/InRoads compatible digital terrain model (DTM).

Several hydraulic models were combined to create the 2003 FIS model:

- A model was developed by a consultant in 1997 for the reach between Ash Avenue at River Station (RS) 26087 and 25th Avenue (RS 47783).
- Two HEC-2 models were developed in 1998 for a Technical Assistance Study.
  - One model extended from the mouth of the Cache la Poudre at RS 0 to Ash Avenue (RS 26087).
  - The second model extended from 25th Avenue (RS 47783) upstream to the downstream limit of the Windsor FIS detailed study described in Reference 10, approximate RS 105277.
- In 2001, the City of Greeley decided to update the cross sections and bridge data in the reach between Ash Avenue and 25th Street. The aerial photogrammetry developed in 2001 was used to update this reach.

A-3.3 Boundary Conditions

Starting water surface elevations were set based upon normal depth at the downstream section. An energy slope of 0.005 ft/ft (Platte River slope in the confluence area) was used in the computations, consistent with the 2003 FIS model.

A-3.4 Roughness Values

Roughness in the model was simulated using Manning's "n" values. Roughness values are unchanged from the 2003 FIS model.

Channel roughness values were set to 0.035 consistent with the 2003 FIS model and previous studies.

The overbank roughness values vary from 0.045 for open pasture or cropland to 0.085 for urban areas.

A-3.5 Contraction and Expansion Coefficients

Contraction and expansion (C&E) coefficients were set to 0.1 and 0.3, respectively, through most of the reach. Near bridges, the C&E coefficients were increased to 0.3 and 0.5, respectively, to reflect the losses incurred at abrupt transitions typically associated with bridges.

Consistent with the FIS model, C&E coefficients were set at 0.3 and 0.5 between the Ash Avenue bridge (RS 26068) and the 5th Street bridge (RS 30685) because of the high density of bridges in the reach (4 bridges over approximately 4600 ft).

A-3.6 Bridges

Bridges in HEC-RAS were modeled according to available data. As noted in section A-2 and Table A-1, available bridge data varied from nothing to bridge plans. Where nothing was available, the bridges were left unchanged from the 2003 FIS model. Where data was available, the bridges were spot checked for accuracy.

All bridges in the HEC-RAS model use the energy method for both high and low flows.

Table A-4 summarizes bridge data as modeled.

Table A-4 Bridge Parameters

Bridge #	River Station	Description	Span Length	Width	Number of Piers	Pier Width	Minimum Low Chord Elevation	Minimum Top of Road Elevation	Computed 100-yr Water Surface
			(ft)	(ft)		(ft)	(ft	(ft	(ft NGVD29)
1	14894	Weld Co Rd 45 (Fern Ave)	150	36	2	1.5	4613.4	4616.4	4616.23
2	26068	Ash Ave	144	38	1	2.0	4637.4	4640.4	4632.01
3	28260	U.S. Hwy 85 Bypass	250	120	4	2.0	4636.7	4640.4	4638.25
4	29155	East 8th St	193	70	2	1.0	4639.0	4642.1	4638.86
5	30685	5th St	165	60	1	2.0	4639.3	4644.3	4640.12
6	33855	6th Ave	262	40	1	3.0	4641.3	4647.0	4641.3
7	36065	U.P. Railroad	260	30	14	2.0	4648.0	4651.7	4645.97
8	36720	U.S. Hwy 85	286	120	3	2.0	4645.7	4649.7	4646.58
9	38036	11th Ave	250	78	2	2.5	4650.4	4656.0	4648.57
10	39420	Foot/horse bridge at Island Grove Park	98	20	0	-	4650.0	4652.0	4651.44
11	47767	25th Ave	160	32	2	2.0	4662.3	4665.0	4663.68
12	52029	Weld Co Rd 35 (N. 35th Ave)	166	60	1	1.5	4671.0	4673.4	4669.59
13	60033	Railroad bridge	261	10	14	1.0	4682.3	4684.9	4682.79
14	72164	Weld Co Rd 31 (N. 59th Ave)	306	54	4	2.8	4698.7	4701.2	4693.81
15	78846	Weld Co Rd 29 (N. 71st Ave)	197	34	1	2.0	4692.2	4697.7	4699.56
16	89328	Weld Co Rd 27 (N. 83rd Ave)	105	36	1	1.7	4711.1	4714.4	4710.37
17	97209	Weld Co Rd 25	100	32	2	2.0	4721.6	4722.6	4719.99

A-3.7 Sensitivity Analyses

A-3.7.1 Starting Water Surface

The sensitivity of the model to the starting water surface elevations was analyzed. As an upper bound, the 100-year water surface elevation on the South Platte River was used as the starting water surface elevation for all Cache la Poudre River events. All water surface profiles essentially converged (within 0.1 ft) with the base model profiles by river station 12,901, well downstream from the main reach of interest (river station 26,000-45,000) and 2000 feet downstream from the USGS gage. If a project extending below RS 12901 is considered, a more detailed coincident analysis will be necessary.

A-3.7.2 Roughness Values

A sensitivity analysis was conducted to determine impacts of higher or lower roughness values on computed results. A range of Manning's n values was evaluated. The range was determined using Figure 5-4 of Reference 4 with an estimated n value of 0.035 corresponding to a standard deviation of 0.010. The average n value was varied by ±1 standard deviation. Channel n values were varied between 0.025 and 0.045. The overbank values were varied ±0.010 relative to the channel roughness values.

The water surface profiles of the geometries utilizing the highest and lowest "n" values were compared. For the 10-percent-annual-chance (10-year) event, the elevation difference between the lowest and highest "n" value water surface elevations ranged from 0.05 feet to 2.37 feet and had an average of 1.23 feet. For the five hundred year flow event, the elevation difference between the lowest and highest "n" value water surface elevations ranged from 0.00 feet to 2.96 feet and had an average of 1.05 feet. The sensitivity analysis showed that the model was moderately sensitive to n values.

A-3.7.3 Contraction and Expansion Coefficients

Sensitivity of the model to contraction and expansion (C&E) coefficients was analyzed. The C&E coefficients were varied by ±0.1 near bridges and the resultant water surface elevations were compared.

The water surface profiles of the geometries utilizing the high and low C&E coefficients were compared. For the ten year flow event, the elevation difference between the lowest and highest C&E coefficients water surface elevations ranged from 0.00 feet to 0.40 feet and had an average of 0.07 feet.

For the five hundred year flow event, the elevation difference between the lowest and highest C&E coefficients water surface elevations ranged from 0.00 feet to 0.70 ft and had an average of 0.17 feet. The sensitivity analysis showed that the model was slightly sensitive to C&E coefficients.

A-4 EXISTING CONDITIONS MODELING

A-4.1 Calibration

Available calibration data was minimal. The HEC-RAS generated rating curve at RS 14959 was compared to data from USGS gage #06752500. This gage is near the downstream end of the reach so additional calibration data was sought.

A stage gage at the wastewater treatment plant (≈ RS 27,000-28,000) was put into service in 2002. However, a stage-discharge rating curve is still being developed for this gage so the provisional data was not used. Additionally, no major flood events have occurred since the gage was installed.

No surveyed high water marks from past flood events were readily available. Five photographs from the 1999 flood (≈ 20-year event) were used to estimate the water's edge at the time the photographs were taken. The flood boundary estimated from the photographs was then surveyed at the five points and the results were compared to the HEC-RAS generated water surface profiles.

A-4.1.1 USGS Gage 06752500

Data from USGS gage 06752500 (Cache la Poudre River near Greeley, CO) was used to compare the stage-discharge relationship. The gage is located on the downstream side of the Fern Avenue bridge (RS 14894) and has records from 1903 to present.

The gage datum, based on information from the Colorado Division of Water Resources, is 4604.81 ft NGVD29 (compared to the value of 4610.00 ft NGVD29 given on the gage website).

Data from the Fern Ave. gage consisted mainly of annual peaks from 1961 to 1998. Additional data from this site consisted of measured discharges obtained from the Colorado Division of Water Resources. Measured discharges were received by facsimile dating from 1960 to the present.

The HEC-RAS generated rating curve from RS 14959 is compared to the annual peaks and measured data on Plate 41. For the period 1960-1998, only measured discharge values greater than 500 cfs were plotted. For the period 2000-present, all values are included.

A-4.1.2 High Water Marks

A search was completed by the sponsor for other available calibration data. Aerial photographs from the 1983 flood were available but were of limited use because they were not orthorectified and may or may not have been taken during the peak. An attempt was made to plot flood boundaries from the photographs but the resulting flood boundaries are of poor quality. Additionally, several new bridges have been installed since the 1983 flood. Limited calibration to the 1983 flood could be accomplished by coding the geometry of the old bridges into the model, but once again, this calibration would serve little use for the present study.

Several photos taken from the ground are available from the 1999 flood and have been used to estimate high water marks (HWMs-see Plates 42-46). However, the timing of the photos relative to the peak flow is unknown. During a site visit in September 2006, the water's edge, as estimated from the photographs, was located and marked. City of Greeley staff then surveyed the marked locations. Table A-5 lists the approximate location of the photographs, the corresponding river station, and the surveyed elevations. Plate 47 shows the approximate locations of the photographs.

Table A-5 High Water Marks

Plate #	Photograph	Location	Approximate RS	Elevation
				(ft NGVD29)
13	HWM #1	6th Avenue and A Street	33875	4641.1
14	HWM #2	6th Avenue and 4th Street	32153	4639.0
15	HWM #3	5th Avenue and 5th Street	30715	4639.9
16	HWM #4	5th Street Bridge	30654	4639.8
17	HWM #5	Hwy 85 and 8th Street	29119	4636.0

The high water marks are plotted on Plate 48 against two HEC-RAS generated profiles. The two HEC-RAS profiles (with and without levee) are based on the measured discharge at the Fern Ave. gage on the same day (4590 cfs at the gage, 4690 cfs at the HWMs). The measured discharge at the gage is prorated moving upstream relative to the ratio of the 100-year discharge given by hydrology at selected locations.

From Plate 48, it can be seen that the steady flow HEC-RAS model does not replicate the high water marks. Based on local accounts of the 1999 flood, several issues may explain why the HEC-RAS model does not match the HWMs.

The high water marks out in the overbank may not reflect the water surface or energy grade elevation in the corresponding channel location. Because of the highly irregular spoil bank elevation, it is difficult to pinpoint breach or overtopping locations. City of Greeley staff have indicated that water was able to leave the channel in several locations, flow parallel to the spoil banks on the landward side, fill in low lying elevations, and return to the channel at unspecified locations. If future evaluation warrants a more detailed existing conditions analysis, an unsteady flow model is recommended to account for flood volume and storage. At this time, it is not anticipated that an unsteady flow model will be necessary because any projects that are evaluated will supersede the inconsistent level of protection now offered by the spoil bank levees.

A-5 MODELING OF ALTERNATIVES

The hydraulic evaluation of alternatives for flood damage reduction was completed with HEC-RAS version 4.0 Beta.

As a starting point, bridge removal was evaluated to determine the effect of the bridges on the water surface profiles. Individual bridges were completely removed from the model and the impact on the water surface elevations was evaluated.

Next, channel improvements and levees were evaluated separately. In addition to the goal of the project of both reducing flood damages and providing ecosystem restoration, results of the separate modeling made it apparent that a combination of levees and channel improvements would be necessary. Using channel improvements only does not reduce water surface profiles enough to significantly reduce damages while levees alone cause induced stages requiring mitigation and do not provide any ecosystem restoration opportunities.

A combination of channel improvements and levees was then modeled and the results compared to the base model. The analysis of flood damage reduction alternatives focused on the river reach between RS 28320 (near U.S. Hwy 85) and RS 40516 (approximately halfway between 11th Ave. and 25th Ave. because of the high damages computed for this reach for existing conditions (hereafter referred to as the high damage reach).

A-5.1 Bridge Removal

Evaluation of bridge removal was completed as a substitute for evaluating a range of bridge replacement configurations. If complete removal of a bridge had little effect on water surface profiles, then replacing the bridge would not be likely to reduce the profiles either.

Bridge removal was evaluated by removing the bridge from the model and adjusting the upstream and downstream cross sections to reflect the without-bridge conditions. Bridges were removed one at a time to determine the impact to the water surface profiles from individual bridges. The focus of the evaluation was on the 100-year water surface profiles.

In general, the removal of bridges did not significantly reduce the 100-year water surface profiles. Several of the bridges in the high damage reach have been replaced relatively recently using guidance from the Warzyn Report (Reference 13). The low chord of a number of the bridges in the high damage reach (8th Street, 6th Avenue, U.P. RR, and 11th Avenue) is above the 100-year water surface elevation for existing conditions.

The remaining bridges in the high damage reach are the Hwy 85 Bypass, 5th Street, 8th Avenue, and the pedestrian/horse bridge at Island Grove Park. When the Hwy 85 Bypass bridge was removed, water surfaces upstream of the bridge did decrease. However, this decrease upstream from the bridge caused less water to flow over the highway into the split flow reach north of East 8th Street (i.e., more water stayed in the channel). Therefore, water surface elevations downstream from the Hwy 85 bypass were higher than existing conditions.

Table A-6 summarizes the results of the bridge removal evaluation.

Table A-6 Bridge Removal Summary

Bridge Removed	Maximum Reduction in 100-year Water Surface Elevation	Approximate Length of Stream with Reduced 100-year Water Surface Elevation
	(ft)	(ft)
US Hwy 85 Bypass	0.06	1400
8th Street	0.02	600
5th Street	0.28	3100
6th Avenue	0.64	4100
UP Railroad	0.74	1900
8th Avenue	0.52	800
11th Avenue	0.65	800
Foot/horse bridge at Island Grove Park	0.21	400

A-5.2 Channel Improvements

Initially, channel improvements were modeled without levees to determine the decrease in stage that could be obtained with channel widening alone. The main constraint for the channel improvement option is that many of the city's bridges have been recently replaced and have a good portion of their design life remaining. It is undesirable to replace these bridges due to high costs. One issue with the bridge replacements is that the new construction was based on the Warzyn report (Reference 13). The elevation of the newly constructed bridges is relatively high, but the widths are based on the efficient trapezoidal channel proposed in the Warzyn report. Therefore, a consistent channel width could not be maintained throughout the high damage reach let alone the entire study reach due to the constrictions presented by the bridges.

Note that a combination of bridge removal/channel improvements was not evaluated. Evaluating both bridge removal and channel widening simultaneously would allow for a corridor with consistent width and would reduce flood stages further than either alternative by itself.

Another constraint imposed on this evaluation was the exclusion of channel stability in the scope of work. Because no channel stability analysis was to be conducted, simplifying assumptions were made. The existing channel was used up to a certain stage rather than evaluating a range of low-flow channel.

Channel improvements were made at or above the stage of a certain frequency event. Initially, the 20% chance exceedance event (5-year) stage was selected. Above the 5-year stage, a "bench" was coded into the model. Below this stage, the existing channel bed was used. The width of the bench varied from cross section to cross section and was based on plan view considerations including typical expansion/contraction ratios, bridge widths, structure locations, and real estate availability. The channel width for this option varied from a typical 200 ft to a maximum of over 500 ft upstream from 11th Ave.

The bench was coded in with approximately a 2% slope and then a 4H:1V slope was extended upward to the natural ground elevation at the appropriate width.

Because the 5-year event uses a relatively large portion of the channel capacity, adding benches above the 5-year stage adds little conveyance to the channel. Several different reach lengths were modeled with benches above the 5-year stage and the impact to the water surface profiles was evaluated. The decrease in stage averaged less than 0.5 ft through the improved reach for all events. For the 10-year event, the decrease in stage was less than 0.2 ft while for the 100-year event, the decrease in stage averaged less than 0.35 ft.

Because the reduction in stage was relatively low for the channel improvements that were cut above the 5-year stage, channel improvements were made above the 2-year stage using similar methodology to that described above.

Channel improvements made above the 2-year stage added more conveyance to the channel but the constraints imposed by the bridges and lack of channel stability analysis still limited the reduction in water surface elevations. The reduction in stage on the improved reach averaged approximately 0.4 ft for the 10-year event to nearly 0.5 ft for the 100-year event. A maximum reduction in water surface was realized at the 50-year event with an average of 0.6 ft.

Because the channel improvements alone would provide minimal benefits to the community and would not remove residents and businesses from the 100-year flood plain, levees were also evaluated.

A-5.3 Levees

Levees were coded into the base model (without channel improvements) from RS 28320 (U.S. Hwy 85) to RS 40516 (approximately 17th Ave) on the right bank. The levees were located near the channel banks along the alignment of the existing spoil banks. Levee elevations were set artificially high to prevent stages from all events from exceeding the top of levee.

The average increase in stage due to the levees ranged from less than 0.1 ft for the 10-year event to over 1.8 ft for the 500-year event. The average increase for the 100-year event was approximately 0.9 ft with a maximum of over 3.5 ft.

In addition to the increase in stages, a flood damage reduction plan including only levees and no ecosystem restoration features makes this alternative undesirable. Therefore, a combination of channel improvements and levees was evaluated.

A-5.4 Combination of Channel Improvements and Levees

A combination of channel improvements and levees was evaluated in order to mitigate flood damages and improve the riparian corridor for ecosystem functionality. The increase in stages caused by levees is at least partially offset by the additional conveyance provided by the channel improvements.

Numerous channel/levee configurations were evaluated with a number of variables including stream length improved/protected, width of channel improvements, and level of protection provided. Although a number of different alternatives were evaluated, three were selected for further investigation: Alternatives 1, 1a, and 5.

For all three selected alternatives, the channel improvements included benches cut just above the 2-year stage. All levees were coded in artificially high to prevent overtopping by any event. This results in higher stages for the 500-year event than would be expected. However, for the level of detail used in this evaluation, the increase in stage was deemed acceptable. Additionally, even when the levees are overtopped, the interior cells will most likely fill to the elevation of downstream confining features because of the large volume and long duration of the Cache la Poudre hydrographs. Considerable effort could be expended to accurately determine the elevation of the 500-year water surface. An unsteady flow model using additional geometry of confining features could be used to evaluate the 500-year water surface in interior areas. However, unsteady flow modeling is out of the scope of this phase of the planning process.

All three selected alternatives require the removal or relocation of the pedestrian/horse bridge at Island Grove Park. Further investigation will be necessary to determine the feasibility of either moving the bridge upstream to a point where it will not cause significant headlosses or raising and lengthening the bridge in its current location. Costs associated with either method should be reflected in the formulation of the benefit to cost ratio. If relocation or reconfiguration of the bridge is not feasible, the political ramifications of completely removing the bridge and rerouting foot/horse traffic may need to be investigated.

Seven alternatives are described below. Only three alternatives (1, 1a, and 5) were evaluated in detail to determine the benefit to cost ratio. Following the descriptions of the individual alternatives is a summary table that compares all alternatives (see Table A-7).

A-5.4.1 Alternative 1

Alternative 1, the shortest alternative in length, focused on the two damage reaches with the highest expected annual damages (EAD) based on existing conditions, reaches 9R and 10R (see Section 3.1 or Appendix B for economic model setup). Reaches 9R and 10R (right bank of reaches 9 and 10) extend from RS 36780 at 8th Avenue to RS 40516 (near the east-west location of 17th Avenue).

Alternative 1 includes channel improvements from RS 30715 up to RS 40516. Levees are included on the right bank from RS 36660 to 40516 with tie-offs extending laterally from the river in order to prevent flanking. The channel cuts as well as the levee alignments can be seen on Plate 8.

A-5.4.2 Alternative 1a

Alternative 1a includes all alternative 1 features with the addition of a left bank levee from RS 36080 to RS 40516 in order to prevent induced damages from increased stages due to the levees. The Alternative 1a configuration can be seen on Plate 9.

A-5.4.3 Alternative 2

Alternative 2 includes all features from alternative 1 but the levee extends further downstream to RS 30715 to include damage reaches 6R, 7R, and 8R.

Preliminary runs with the economic model indicated that the damages prevented with the alternative 2 levee alignment would not likely justify the increase in cost. Therefore, alternative 2 was not evaluated further.

A-5.4.4 Alternative 3

Alternative 3 includes all features from alternative 2 but the channel improvements and levee extends downstream to RS 28320 to include damage reach 5R.

Following a conference call with the sponsor on 16 October 2007, a plan similar to alternative 3 was suggested by the sponsor. The sponsor-suggested plan would protect the same areas as alternative 3, but would allow for constructing the plan with a phased approach.

Because of the similarities between alternative 3 and the sponsor-suggested plan (alternative 5) and because of the unmitigated induced stages on the left bank, alternative 3 was not evaluated further.

A-5.4.5 Alternative 3b

Alternative 3b was identical to alternative 3 with the addition of a left bank levee for damage reach 8 (RS 36080-36660). The left bank levee required a long tieback to the north on the upstream side in order to prevent flanking. Alternative 5 includes a left bank levee so alternative 3b was not evaluated further because it is similar to alternative 5.

A-5.4.6 Alternative 4

Alternative 4 was a combination of alternatives 1-3 in an effort to obtain the maximum benefit to cost ratio.

Channel improvements were considered only in reaches 9 and 10 (RS 36780-40516). Other features included in alternative 4 were the right bank levee used in alternative 1 and the left bank levee from alternative 3b.

Channel improvements were not included in damage reaches 6-8 mainly because the methodology being used to evaluate damages caused an increase in damages when the channel improvements were included. The reason damages increased for the improved channel is that when evaluating existing conditions, credit was given to the spoil banks to half their height, but when evaluating the improved channel, the lack of spoil banks caused damages to begin at more frequent events.

A decision was made near the end of October 2007 to not give any credit to the spoil banks for existing conditions because of their poor condition and because the materials used to construct the spoil banks is unknown. This change in methodology changes the approach used for reducing damages.

Because of the change in methodology, similarities to other alternatives, and relatively small stream reach benefitting from this alternative, it was not evaluated further.

A-5.4.7 Alternative 5

Alternative 5 is similar to alternative 3b except that the right bank levee alignment could be constructed in phases and the left bank levee extends upstream to RS 40516. Plate 10 shows the channel improvements and levee alignments for alternative 5.

Table A-7 Comparison of Alternatives

	Bounding River Stations			Alternative 1		Alternative 1a		Alternative 2		Alternative 3		Alternative 3a		Alternative 4		Alternative 5	
Damage Reach	Downstream	Upstream	Stream Bank	Channel Cuts	Levee	Channel Cuts	Levee	Channel Cuts	Levee	Channel Cuts	Levee	Channel Cuts	Levee	Channel Cuts	Levee	Channel Cuts	Levee
5	28320	30654	Right								X		X				X
			Left							X		X				X	
6	30715	33834	Right						X		X		X				X
			Left	X		X		X		X		X				X	
7	33875	36049	Right						X		X		X				X
			Left	X		X		X		X		X				X	
8	36080	36659	Right		X		X		X		X		X		X		X
			Left				X						X		X		X
9	36780	37996	Right		X		X		X		X		X		X		X
			Left	X		X	X	X		X		X		X		X	X
10	38075	40516	Right		X		X		X		X		X		X		X
			Left	X		X	X	X		X		X		X		X	X

A-5.5 Induced Stages

Because the modeled levees confine flow to a relatively narrow floodplain, certain areas have higher stages with the alternatives in place. The induced stages were mitigated in most cases by using levees to prevent floodwaters from leaving the immediate channel area. However, areas still exist with higher flood stages for certain events. On the upstream end of the project, the alternative water surface profiles tie in at or below the existing conditions profiles due to the channel improvements in the area and because of the removal of the pedestrian/horse bridge at Island Grove Park. Table A-8 compares the high damage reach existing conditions 100-year water surface profiles to each of the three selected alternatives.

Table A-8 Induced Stages

	Difference in 100-year Water Surface Elevation from Existing Conditons (ft) *		
River Station	Alternative 1	Alternative 1a	Alternative 5
47085	0	0	0
45065	-0.01	-0.01	-0.01
42851	-0.09	-0.11	-0.11
41441	-0.19	-0.24	-0.23
40516	-0.27	-0.04	-0.02
40230	-0.23	-0.18	-0.16
39814	-0.12	-0.02	0.02
39434	-0.08	0.05	0.09
39406	0.6	0.73	0.78
38896	1.01	1.16	1.22
38604	1.06	1.22	1.27
38293	1.07	1.25	1.32
38075	0.94	1.17	1.25
37996	1.58	1.85	1.94
37700	1.83	2.14	2.24
37153	2.02	2.36	2.47
36780	2.01	2.21	2.33
36659	1.15	1.45	1.56
36467	1.1	0.91	1.05
36257	0.41	0.34	0.53
36080	0.52	0.52	0.62
36049	0.65	0.65	0.37
35655	0.18	0.18	0.02
34185	-0.45	-0.45	0.09
33875	-0.08	-0.08	0.68
33834	-0.1	-0.1	0.58
33508	-0.03	-0.03	0.38
32931	-0.02	-0.02	0.34
32153	-0.01	-0.01	0.34
31521	-0.01	-0.01	0.34
30715	0	0	0.28
30654	0	0	0.07
30214	0	0	0.24
30029	0	0	0.29
29810	0	0	0.16
29444	0	0	-0.03
29190	0	0	-0.01
29119	0	0	-0.01
28772	0	0	0.01
* Negative value reflects lower water surface for the alternative			

Appendix B Uncertainty Analysis

In accordance with current USACE guidelines, (see Reference 14) an uncertainty analysis was completed to better define both existing conditions and with project flood damages. The Hydrologic Engineering Center's Flood Damage Reduction Analysis (HEC-FDA) version 1.2.3 dated June 2005 was used in the analysis.

The HEC-FDA model runs were executed by the USACE economist. However, input from the hydrologic and hydraulic sections of the USACE were used in model setup.

B-1 HEC-FDA SETUP

Setup of HEC-FDA for the majority of the reaches followed standard practices (i.e. discharge-probability, stage-discharge, and stage-damage curves were input). However, for the two split flow reaches (damage reaches 4L, 4R, and 14R) another method was used. The reason for using another method is that the split flow reaches do not receive flow during all of the eight required events. The uncertainty bands on the discharge-probability curves when there is zero or very low flow for the frequent events is unreasonably large. Reach 4 does not receive flow until the U.S. Hwy 85 Bypass is overtopped which occurs at approximately the 20-year event. Reach 14 does not receive flow until a small road is overtopped which occurs at approximately the 50-year event. For this reason, another method was selected to evaluate the split flow reaches with HEC-FDA as described in section B-1.2.

Damage reaches were delineated based on physical features (mainly roads) and where water surface profiles had break points in slope. Table B-1 summarizes the damage reach naming convention, boundaries, and index stations.

Hydraulic input to the HEC-FDA model consisted of water surface profiles and a standard deviation value representing hydraulic uncertainty. Water surface profiles were input for eight flood events (2, 5, 10, 20, 25, 50, 100, and 500-year). The 20-year event was used instead of the 200-year event in order to better define the stage-discharge relationship near the top of the existing spoil banks (which lies near the 20 to 25-year event range).

Table B-1 HEC-FDA Setup

Stream Name	Damage Reach Name	Downstream Station	Upstream Station	Stream Bank	Index Station	Description
Cache la Poudre	1R	3464	14952	Right	10998	Confluence with S. Platte River to Fern Ave
Cache la Poudre	1L	3464	14952	Left	10998	Confluence with S. Platte River to Fern Ave
Cache la Poudre	2R	14996	26048	Right	19353	Fern Ave to Ash Ave
Cache la Poudre	2L	14996	26048	Left	19353	Fern Ave to Ash Ave
Cache la Poudre	3R	26087	28199	Right	27305	Ash Ave to U.S. Highway 85 Bypass
Cache la Poudre	3L	26087	28199	Left	27305	Ash Ave to U.S. Highway 85 Bypass
Split Left	4R	10	19	Right	17	Delwood Ave to U.S. Highway 85 Bypass
Split Left	4L	10	19	Left	17	Delwood Ave to U.S. Highway 85 Bypass
Cache la Poudre	5R	28320	30654	Right	29190	U.S. Highway 85 Bypass to 5th Street
Cache la Poudre	5L	28320	30654	Left	30029	U.S. Highway 85 Bypass to 5th Street
Cache la Poudre	6R	30715	33834	Right	32153	5th Street to 6th Ave
Cache la Poudre	6L	30715	33834	Left	32153	5th Street to 6th Ave
Cache la Poudre	7R	33875	36049	Right	35655	6th Ave to U.P. Railroad
Cache la Poudre	7L	33875	36049	Left	35655	6th Ave to U.P. Railroad
Cache la Poudre	8R	36080	36659	Right	36257	U.P. Railroad to 8th Ave (U.S. Hwy 85)
Cache la Poudre	8L	36080	36659	Left	36257	U.P. Railroad to 8th Ave (U.S. Hwy 85)
Cache la Poudre	9R	36780	37996	Right	37700	8th Ave (U.S. Hwy 85) to 11th Ave
Cache la Poudre	9L	36780	37996	Left	37700	8th Ave (U.S. Hwy 85) to 11th Ave
Cache la Poudre	10R	38075	40516	Right	40230	11th Ave to approximately 16th Ave
Cache la Poudre	10L	38075	40516	Left	40230	11th Ave to approximately 16th Ave
Cache la Poudre	11R	40516	47750	Right	45065	Approximately 16th Ave to 25th Ave
Cache la Poudre	11L	40516	47750	Left	45065	Approximately 16th Ave to 25th Ave
Cache la Poudre	12R1	47783	48457	Right	47783	25th Ave to 27th Ave
Cache la Poudre	12L1	47783	48457	Left	47783	25th Ave to 27th Ave
Cache la Poudre	12R	48457	51998	Right	48457	27th Ave to 35th Ave
Cache la Poudre	12L	48457	51998	Left	48457	27th Ave to 35th Ave
Cache la Poudre	13R	52121	52718	Right	52234	35th Ave to approximately 39th Ave
Cache la Poudre	13L	52121	52718	Left	52234	35th Ave to approximately 39th Ave
Cache la Poudre	13R1	52718	60043	Right	54064	Approximately 39th Ave to approximately 49th Ave
Cache la Poudre	13L1	52718	60043	Left	54064	Approximately 39th Ave to approximately 49th Ave
Cache la Poudre	13R2	60043	72136	Right	61694	Approximately 49th Ave to 59th Ave
Cache la Poudre	13L2	60043	72136	Left	72017	Approximately 49th Ave to 59th Ave
Split Right	14R	19	23	Right	19	35th Ave to 47th Ave
Cache la Poudre	15R	72191	84514	Right	80402	59th Ave to just downstream from 83rd Ave
Cache la Poudre	15L	72191	84514	Left	78718	59th Ave to just downstream from 83rd Ave

B-1.1 Damage Reach Setup-Typical

The majority of the damage reaches were set up using discharge-exceedance, stage-discharge, and stage-damage relationships as described below.

B-1.1.1 Hydrologic Uncertainty

The discharge-exceedance probability uncertainty functions used in the analysis were computed using analytical procedures within HEC-FDA. The parameters used in the Log-Pearson Type III statistical evaluation are summarized in Table B-2. Note that damage reaches 4L, 4R, and 14R are not included in Table B-2. These three damage reaches are located on split flow reaches and were analyzed differently, as described in section B-1.2.

Table B-2 HEC-FDA Hydrologic Data

	User-entered Statistics			Computed Log-Pearson Type III Log Statistics			
Reaches	2-Year Event (50% Probability of Exceedance)	10-Year Event (10% Probability of Exceedance)	100-Year Event (1% Probability of Exceedance)	Mean (M)	Standard Deviation (S)	Skew (G)	Equivalent Record Length (N)
	(cfs)	(cfs)	(cfs)				(years)
1L, 1R, 5L, 6L, 6R, 7L, 7R,	1500	3520	10100	3.2284	0.2392	1.3555	124
2L, 2R	1500	3520	9626	3.2227	0.2425	1.1797	124
3L, 3R	1500	3520	6740	3.1700	0.2947	-0.1240	124
5R	1500	3520	8039	3.1987	0.2617	0.5207	124
8L, 8R, 9L, 9R, 10L, 10R, 11L, 11R, 12L1, 12R1	1780	4100	11200	3.2985	0.2357	1.2578	124
12L, 12R, 13L, 13R, 13L2, 13R2, 15L, 15R	2070	4670	12500	3.3634	0.2295	1.2756	124
13L1, 13R1	2070	4670	10115	3.3354	0.2517	0.4637	124

B-1.1.2 Hydraulic Uncertainty

The stage-discharge function for each reach is based on the water surface profiles computed with the HEC-RAS model at the index station. HEC-FDA input requires the description of stage uncertainty of the computed water surface profiles. Uncertainty in computed stage profiles reflects modeling assumptions, numerical errors, and parameter estimation. Uncertainty was estimated for the entire study reach by performing a sensitivity analysis with the HEC-RAS model and also by evaluating the variation in historic measured discharge values at the USGS gaging station.

The HEC-RAS sensitivity analysis consisted of varying the roughness coefficients by 0.10 within the HEC-RAS model (see section A-3.7.2). An average difference was computed for the models reflecting the maximum and minimum roughness conditions. The maximum difference between the profiles was just under 3.0 ft during the 500-year event. For the 100-year event, the maximum difference in profiles was approximately 2.7 ft. A value of 3.0 ft was used to compute the standard deviation.

Another method employed to estimate the standard deviation of error in stage uses measured discharge values at a gage. For Greeley, USGS gage 06752500 (Cache la Poudre River near Greeley, CO) was used. Several ranges of discharges were analyzed: 0-2000 cfs, 2000-3000 cfs, 3000-4200 cfs, and 4200-6360 cfs. Additionally, the entire range of discharges was analyzed. The analysis used equation 5-3 from Reference 4. Results of the analysis show that for all discharge ranges, the standard deviation is below 0.5 ft. A value of 0.5 ft was used.

The minimum standard deviation of error for modeling was determined based on survey data error and Manning's n value reliability estimates. All methods described above were considered and used the establish uncertainty in the computed stage values. Results are summarized in Table B-3.

Table B-3 HEC-FDA Hydraulic Uncertainty

Estimating Standard Deviation of Error in Stage			
Method	Parameter	Standard Deviation	Computation
Measured Gage Data	USGS Gage at Greeley, CO	0.5	$S=\sqrt{\frac{\sum_{i=1}^{N}(X_i-M)^2}{N-1}}$ (EM 1110-2-1619, Eq. 5-3)
HEC-RAS Sensitivity (max. and min. profiles)	3.0 max. difference, use 100-year elevation for constant	0.75	$S_{model} = E_{mean} / 4$ (EM 1110-2-1619, eq. 5-7)
Minimum Standard Deviation of Error	2 foot contour interval, fair to poor Manning n reliability	1.2	EM 1110-2-1619, Table 5-2

The standard deviation of the total uncertainty is determined from:

$$S_t = \sqrt{S_{natural}^2 + S_{model}^2} \quad \text{(EM 1110-2-1619, eq. 5-6)}$$

Using the estimated values of 0.5 ft for $S_{natural}$ and 1.2 ft for S_{model} (overrides 0.75 ft determined from sensitivity analysis) with the above equation, the total standard deviation S_t is 1.3 ft.

Within the FDA model the total standard deviation of 1.3 ft at the 100-year event was employed at all locations. Between the invert and the 100-year stage, standard deviation values are interpolated while the value of 1.3 ft is used for all stages above the 100-year event.

B-1.2 Damage Reach Setup-Split Flow Reaches

Because the split flow reaches (damage reaches 4L, 4R, and 14R) do not carry flow during all events, a different approach was taken for the uncertainty analysis. Differences from the standard damage reach setup, outlined below, apply only to damage reaches 4L, 4R, and 14R:

- The Exceedance Probability Function was input as a graphical stage-probability (rather than discharge-probability) function with the stages computed at the index station of the *main channel reach* from which flow enters the split flow reach
- The HEC-FDA levee features with an exterior/interior relationship were employed to link the split flow reach to the main channel. The top of levee stage corresponds to the elevation at which flow can overtop the respective road and enter the split flow reach. The exterior stage within the levee features corresponds to the main channel index station while the interior stage is tied to the index station of the split flow reach.

Using this approach, HEC-FDA first assumes a stage for the damage reach based on the range provided in the stage-exceedance probability function. This stage is compared to the top of levee stage to determine whether or not the reach is inundated. If the stage is higher than the top of levee, HEC-FDA enters the exterior/interior relationship table and determines the stage at the index station of the split flow reach. Damages are then computed using the aggregation profiles based on the split flow reach. The use of the exterior/interior relationship circumvents the unrealistically high uncertainty bounds computed when a discharge-exceedance probability function with zero discharge at the more frequent events is used.

Hydrologic and hydraulic uncertainty for the split flow reaches was entered similar to the typical reaches as described in sections B-1.1.1 and B-1.1.2.

B-2 HEC-FDA MODEL RESULTS

Results of the HEC-FDA modeling are presented in the Economics appendix to the feasibility report. However, one item of interest related to hydraulics is the computed conditional non-exceedance probability (CNP). The CNP, also referred to as assurance, is a measure of the levee's ability to contain a specified event while considering uncertainty. In order to certify a levee as giving 100-year protection, 90% CNP of the 1%-chance-exceedance event must be demonstrated (see reference 14).

Phase 1 HEC-FDA model executions indicate that greater than 90% CNP is attained for the 100-year event with levee top elevations at three feet above the 100-year water surface elevation (i.e. the model indicates that three feet of "freeboard" is sufficient to allow levee certification) for all three alternatives. Refer to the Economics appendix for more details on levee performance.