Limited Reevaluation Report

Missouri River Levee System
Unit L142
Jefferson City, Missouri

June 2003

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INTRODUCTION

Purpose

This Limited Reevaluation Report (LRR) reevaluates the National Economic Development (NED) Plan identified in the April 2001 General Reevaluation Report (GRR) for the Missouri River Levee System, Unit L142. The reevaluation uses updated economic data, a new construction cost estimate, and water surface profiles from a new hydraulic model. Based on the results of the new studies, we have identified a new optimum top-of-levee profile for design. The recommended levee alignment, identified as alternative 10a in the GRR, is not a subject of this reevaluation. Figure 1 shows the location and vicinity of Unit L142.

Scope

The study area consists of approximately 40 river miles on the Missouri River, from River Mile (RM) 116 to RM 156. The hydrology and hydraulics section of this report documents the development of water surface profiles through the project reach. These profiles form the basis for setting the top of levee profile for Levee Unit L142. The top of levee profile corresponds to the top of levee elevation identified as the National Economic Development (NED) plan through the risk analysis and optimization procedure discussed in the economics section of this LRR.

HYDRAULIC REEVALUATION

We developed a new hydraulic model based on the most current floodplain maps, cross-sections, and hydrographic surveys. Updated hydrology from a separate, independent study completed in 2001 is also incorporated. The hydraulic computations are based on the Hydrologic Engineering Center River Analysis System (HEC-RAS) computer software program calibrated to produce water surface profiles for the Missouri River near Jefferson City, Missouri. We calibrated the new model to the 1993 flood event and verified with the surveyed water surface profile from June of 1995.

Stream and Valley Characteristics

The Missouri River natural valley flood plain near Jefferson City averages about 2 miles wide while the main channel averages about 1,200 feet wide through the project reach. Bank-full discharge is approximately 250,000 cubic feet per second (cfs), which corresponds to an event between the 50-percent chance of exceedance (2-year) event and the 20-percent chance of exceedance (5-year) event. The average bed slope through the project reach is on the order of 1 foot per mile. The drainage area of the Missouri River at Jefferson City is approximately 503,500 square miles.

An unrated stream gage is located on the Missouri River within the project reach. The Jefferson City Gage is on the downstream side of the Highway 63 Bridge. The datum of the gage is 520.13 feet above mean sea level (ft., msl), National Geodetic Vertical Datum of 1929 (NGVD 1929). Flood stage for the Jefferson City Gage is 23 feet, which corresponds to an elevation of 543.13 ft., msl.

In addition to the Jefferson City Gage data, we obtained data from two other gages on the Missouri River. The Boonville Gage is a rated gage, measuring stage and discharge, at Missouri River Mile 197.1. The Herman Gage measures stage and discharge at Missouri River Mile 97.9. Both of these gages have an equivalent record length of 70 years.

Hydrology

Two major studies, the *Missouri River Agricultural Levee Restudy Program* (circa 1962); and the *Upper Mississippi River System Flow Frequency Study*, 2001provide estimates of the overall hydrology and the consequent flow frequencies on the Missouri River in the Jefferson City area.

<u>Missouri River Agricultural Levee Restudy Program (1962)</u>
Hydrology for the Missouri River in the Kansas City District was developed and published in a Hydrology Report dated March 1962. The data presented in that report was used to estimate flood flows for subsequent flood control studies, Federal Emergency Management Agency (FEMA) Flood Insurance Studies, and similar purposes. Table 1 summarizes these flows applicable to the Missouri River near Jefferson City.

TABLE 1
DISCHARGE (CFS) FREQUENCY FROM MARCH 1962 HYDROLOGY REPORT

Missouri River Mile Range for which discharge applies	Frequency in percent-chance exceedance (Return Interval-years)					
	50-percent (2-year)	20-percent (5-year)	10-percent (10-year)	2-percent (50-year)	1-percent (100-year)	0.2-percent (500-year)
116.0 – 130.3	218,000	319,000	398,000	545,000	610,000	800,000
130.3 – 156.0	198,000	285,000	365,000	485,000	550,000	700,000

Upper Mississippi River System Flow Frequency Study (2001)

Recently, new discharge-frequency relationships on the Missouri River were produced from The Upper Mississippi River System Flow Frequency Study (UMRFFS 2001), a large, complex evaluation of the regulated and unregulated flows on the Mississippi, lower Illinois, and Missouri Rivers. This study produced a detailed analysis of the effects of reservoir regulation on the main stem of the Missouri River and in the Kansas and Osage River Basins. Table 2 summarizes the flow frequency estimates applicable to the Missouri River near Jefferson City.

TABLE 2
FLOW FREQUENCY DATA AS DEVELOPED IN UPPER MISSISSIPPI RIVER SYSTEM FLOW FREQUENCY STUDY AS APPLICABLE TO THE MISSOURI RIVER NEAR JEFFERSON CITY

Missouri River Mile Range for which discharge applies	Frequency in percent-chance exceedance (Return Interval-years)					
	50-percent (2-year)	20-percent (5-year)	10-percent (10-year)	2-percent (50-year)	1-percent (100-year)	0.2-percent (500-year)
116.0 – 130.3	241,000	348,000	424,000	607,000	690,000	903,000
130.3 – 138.0	210,000	299,000	364,000	521,000	593,000	778,000
138.0 – 156.0	208,000	297,000	361,000	516,000	588,000	772,000

We used these discharges to establish the existing conditions flow frequency data in this LRR. Since we use flood events above the 0.2-percent-chance exceedance (500-year) event in the economic analysis, the discharge-frequency curves were extended up to the 0.02-percent-chance exceedance (5000-year) event by graphing the discharge-frequency curves and performing a straight-line extrapolation. Table 3 summarizes the additional flood discharges used in this study. Figures 2 through 5 show the discharge-frequency curves.

TABLE 3
ADDITIONAL FLOOD DISCHARGES USED IN THIS ANALYSIS

Missouri River Mile Range for which discharge applies	Frequency in -chance of exceedance (Return-Interval years)	
	0.05-percent (2000-year)	0.02-percent (5000-year)
116.0 – 130.3	1,100,000	1,270,000
130.3 – 138.0	958,000	1,110,000
138.0 – 156.0	950,000	1,100,000

Hydrologic Uncertainty

Risk Based Analysis (RBA) methodology requires a characterization of the hydrologic uncertainty of flow estimates. The Monte Carlo algorithms built into the Risk and Uncertainty Analysis computer program developed by the Omaha District, Corps of Engineers recognize this uncertainty. The uncertainty bands used in the program reflect the effective record lengths used to develop the flow frequency estimates. . For this study, the effective record length is governed by the gage histories of the main stem Kansas River gages. These gage records apply not only to the Kansas River directly, but also to estimated regulated peak flows on the Missouri River for the period prior to the construction of the Kansas River basin reservoir system. Useable information from these gages goes back to 1920, therefore, in the UMRFFS study, the effective record length is 70 years. Table 4 summarizes the analytical hydrologic data.

TABLE 4
ANALYTICAL HYDROLOGIC DATA

Mean	5.3257
Standard Deviation	0.1763
Skew	0.2607
Equivalent Years	70

Hydraulics

A baseline (existing conditions) HEC-RAS model, HEC-RAS version 3.0.1, as developed by the Hydrologic Engineering Center, U.S. Army Corps of Engineers, was the basis for the hydraulic analysis. We calibrated this model to the flood event of 1993 and verified with the surveyed water surface profile from June of 1995. From the calibrated model, we obtained a series of steady flow water surface profiles based on the flood discharges listed above.

HEC-RAS Model Development

The first step in the model development was to enter all applicable geometric data, including a schematic of the river system, cross-section data, bridge data, and other geometric data.

River System Schematic and Cross-Section Data

The Missouri River centerline is based on the navigation sailing line and the 1960 adjusted river mileage. All of the mapping and cross section production work was part of the Mississippi Basin Modeling System (MBMS) project. The Missouri River mapping of the floodplain from bluff to bluff was accomplished under contracts with Horizons, Inc. The mapping is based on aerial photography from 1995 and 1998. Digital format for this mapping data is in the form of a Digital Terrain Model (DTM). Data files are in Universal Transverse Mercator (UTM) Zone 15, North American Datum (NAD) 83, NGVD 29, with units in feet. Mapping in the Jefferson City area is from the 1995 data.

The Kansas City District performed a hydrographic survey of the Missouri River channel in 1998/99 via channel soundings. Under contract with the Kansas City District, the firm of Bohannan Huston merged the channel soundings and the DTM data into one continuous DTM to represent the Missouri River and its floodplain from bluff to bluff.

Using the DTM, Bohannan Huston then produced geo-referenced, bluff-to-bluff cross sections based on cross section layouts from the Kansas City District. Cross-sections average about three per river mile with a higher density in the vicinity of bridges.

For the HEC-RAS model, we imported a "River System Schematic" using ArcView Geographic Information System (GIS) software. The River System Schematic is a single river reach from Missouri RM 116 to Missouri RM 156. The model extended far enough downstream (RM 116) to overcome uncertainties in starting water surface elevations. That is, any instabilities are eliminated before the water surface profile reaches the downstream end of the project reach (approximately RM 141.5). The upstream end extended far enough (RM 156) to assess project impacts upstream from the L-142 area. Figure 6 is a plan view HEC-RAS schematic of the modeled area.

Geometric Data

We entered bridge plans obtained from the Missouri Department of Transportation, along with the flood plain mapping, the bridge embankment, highway profile, and pier sizes and shapes into HEC-RAS. The only bridge input into the HEC-RAS model is the Missouri State Highway 54 bridge over the Missouri River at approximately RM 144. We modeled the two separate bridge structures as one composite structure 150 feet wide because the structures are within 75 feet of each other. The composite structure has the pier alignments and spacing of the newest structure. We used the standard step energy method within HEC-RAS to calculate the energy loss through the bridge. For high frequency flood events, impact from the bridge is limited to pier effects. However, for lower frequency flood events, pier effects, the Highway 54 embankment across the floodplain, and bridge low chord elevations impact the water surface profile calculations.

We entered ineffective flow areas into the HEC-RAS model to represent floodplain areas where the downstream movement of floodwater is typically negligible. The Highway 54 embankment produces large areas of ineffective flow on both the upstream and downstream sides. We initially determined the ineffective flow areas downstream of the bridge using the guidelines in the HEC-RAS Hydraulic Reference Manual. Table 5.1 on page 5-3 of this manual gives ranges for expansion ratios based on different parameters. Based on an estimated 0.3 for the b/B ratio (ratio of the bridge opening width to the total floodplain width), a bed slope of about 1 ft./mile, and an average overbank roughness to channel roughness ratio of 2, we estimated the expansion ratio at 2:1. We determined the ineffective flow areas upstream from the bridge mostly from physical features in the floodplain. Highway 63 is adjacent to the river from RM 147 to RM 145. The road surface is elevated approximately 10 feet from the floodplain and we used this feature to set the contraction ineffective flow areas. From the Highway 63 starting point, we adjusted the ineffective flow areas slightly during the calibration. Aerial photography and high water marks from the 1993 event also confirmed the placement of these ineffective flow areas. We determined all other ineffective flow area locations based on physical parameters of the cross-sections and calibration.

We entered contraction and expansion coefficients into the model according to the guidance distributed by HEC (HEC 2001). These parameters account for losses associated with flows expanding and contracting across the flood plain between cross-sections. Except for cross-sections at or near the bridge (about 1,500 feet upstream and

about 3,500 feet downstream), we assigned all cross-sections the default contraction and expansion coefficients of 0.1 and 0.3, respectively. Table 3.3 on page 3-20 of the Hydraulic Reference Manual recommends these values for gradual subcritical flow transitions between cross-sections. As the flow transitions move from gradual to more abrupt (upstream and downstream of the bridge), we increased the contraction and expansion coefficients to compute a greater energy loss between cross sections. We arrived values of 0.15 for the contraction coefficient and 0.4-0.5 for the expansion coefficient at through the calibration process. These values are within the recommended ranges given in Table 3.3 of the Reference Manual.

We entered initial Manning's n-values based on field investigation and aerial photography of land cover. We modified these roughness coefficients during the calibration.

Calibration

The flood event of July 1993 was a rare and unique flood event on the Missouri River that occurred in the recent past and presents a great opportunity for calibration. Reliable and readily available data exist for this event. We calibrated the HEC-RAS model to the 1993 flood event using relevant high water marks located near the channel.

The next step in the HEC-RAS model development was to enter the flow data specific to the 1993 flood event. On July 29th, 1993, at the peak of the flood event, the Boonville Gage (RM 197.1) measured discharge of 755,000 cubic feet per second (cfs). The peak flow downstream of the project area was on July 31st, 1993 and measured 750,000 cfs at the Herman Gage (RM 97.9). Because the increase in contributing drainage area between the Boonville Gage and Jefferson City is relatively minor (about 1,800 square miles), and the flow of the Osage River at RM 130.3 was essentially shut off during the 1993 flood event, we used 755,000 cfs as the 1993 calibration discharge in HEC-RAS for the entire modeled reach. This discharge is in agreement with the 1993 discharge used in the GRR.

Along with the discharges, we entered 1993 high water mark data taken from the 1993 Post-Flood Report (USACE 1994). USGS data obtained from the vicinity of the Highway 54 Bridge (USGS 1995) supplemented the 1993 data. Refer to Exhibit 1 for the high water mark locations near Highway 54. We used these high water marks during the calibration as appropriate based on their location with respect to the channel. All of the high water mark data were taken into account when setting the top-of-levee profile. We adjusted (interpolated) observed water surface elevations slightly to associate a high water mark with a HEC-RAS cross-section already entered in the model. We used the high water marks to set the starting water surface elevation within HEC-RAS. Interpolation of the two high water marks immediately upstream and downstream of RM 116 resulted in a starting water surface elevation of 534.06.

Once all of the geometric and flow data were entered into the model, we began an iterative calibration process. The goal of the calibration process is to produce a water surface profile that closely matches the observed water surface elevations. We varied Manning's n-values, coefficients, and ineffective flow areas to develop the profile that "passes through" or averages the appropriate high water marks. The resulting profile passes through the high water mark in some locations, and slightly over or under the observed high water mark in other locations.

The most difficult area of the river to calibrate was immediately upstream and downstream of the Highway 54 Bridge and embankment. This difficulty results from the inability of HEC-RAS to model flow that occurs in more than one direction. The highway embankment across the Missouri River floodplain had a damming effect on the river during the 1993 flood event. The high water mark patterns indicate that the highway embankment caused the river to flow perpendicular to the main channel (parallel to the highway embankment), thus creating a multidirectional flow condition.

Near the bluff-line, upstream from the highway embankment, the overbank flow became quiescent, having velocities at or near zero. Moving closer to the channel, upstream of the bridge embankment, decreasing high water mark elevations indicate an increase in velocity. The increase in velocity is caused by a decrease in conveyance area in the left overbank. The river is forced out of the left overbank and into the main channel just upstream of the bridge. This constriction of the river produces a drawdown effect just upstream and through the bridge.

The high water marks indicate a 5 to 6--foot drop in the water surface across the highway embankment in the left overbank near the bluff-line. However, away from the bluff and towards the Missouri River channel, the drop across the highway embankment is only 3-4 feet.

Another difficult area to model was near the confluence of the Osage River at Missouri River Mile (RM) 130.3. A number of cross-sections cut across the Missouri River floodplain, and they also include the mouth of the Osage River as it parallels the Missouri River for about 4 miles. The portion of the cross-section that represents the Osage River channel was blocked off as an ineffective flow area.

Table 5 compares the appropriate observed high water mark data to the computed water surface elevation for the 1993 flood event on the Missouri River in the vicinity of Jefferson City. Note that the USACE data are recorded to the nearest 0.01 foot while the USGS are only to the 0.1 foot. These data are also presented graphically on Figure 7.

TABLE 5
COMPARISON OF 1993 HIGH WATER MARK ELEVATIONS AND COMPUTED WATER SURFACE ELEVATIONS (WSEL) MISSOURI RIVER

HEC-RAS River Station	High Water Mark Source*	Observed 1993 High Water Mark Elevation (ft)	Computed Water Surface Elevation (ft)	Difference: Observed WSEL vs. Computed WSEL (ft)
117.70	USACE	535.46	535.86	0.40
119.93	USACE	538.39	537.62	-0.77
124.47	USACE	541.50	541.77	0.27
129.29	USACE	546.33	545.95	-0.38
131.84	USACE	548.52	548.53	0.01
134.25	USACE	552.01	551.3	-0.71
139.20	USACE	554.48	554.37	-0.11
140.81	USGS	555.8	556.1	0.3
141.73	USGS	556.7	556.8	0.1
142.16	USGS	557.0	557.0	0.0
142.80**	USGS	557.5	557.9	0.4
143.14	USGS	558.2	558.0	-0.2
143.56	USGS	558.7	557.9	-0.8
143.818	USGS	559.1	558.6	-0.5
143.898	***USGS***	***559.3***	559.1	-0.2
144.04**	USGS	560.0	559.9	-0.1
145.57	USGS	562.9	563.3	0.4
149.30	USACE	564.66	565.11	0.45
154.40	USACE	568.52	568.72	0.20

*see references USACE – 1994 , USGS - 1995
**interpolated cross-section
USGS measured stage at Jefferson City Gage

All computed water surface elevations were within 1 foot of matching the observed water surface elevation and only 4 of the 19 locations did not come within 0.5 foot of the observed high water marks. The resulting range of roughness coefficients for the channel is 0.024-0.027. The overbank roughness coefficients range from 0.035 to 0.07.

After the model was calibrated, it was necessary to verify the results with a different flood event. Limited historic data exist for flood events of the magnitude of the July 1993 event. High water mark data were not found for the smaller flood event of May 1995. However, the Missouri River water surface profile was surveyed in June of 1995. Using these surveyed water surface elevations, we performed interpolations to generate an observed elevation at the closest HEC-RAS cross-sections. Using this profile, and discharge estimates from the GRR, we verified the model with this in-channel event. We set ineffective flow areas at or near the bank stations for most of the cross-sections to block out low-lying areas in the overbanks. Table 6 compares the observed profile to the computed water surface elevation for the 1995 event. These data are also presented graphically on Figure 8.

TABLE 6
COMPARISON OF 1995 WATER SURFACE PROFILE AND COMPUTED WATER SURFACE ELEVATIONS (WSEL) MISSOURI RIVER

HEC-RAS River Station	June 1995 Observed Water Surface Elevation (ft)	Computed Water Surface Elevation (ft)	Difference: Observed WSEL vs. Computed WSEL (ft)
120.56	526.23	526.62	0.39
125.12	529.07	529.86	0.79
129.29	532.23	532.51	0.28
131.84	534.25	534.22	-0.03
134.57*	536.26	536.15	-0.11
136.61	537.74	537.52	-0.22
139.20	539.51	539.59	0.08
143.56	543.56	542.72	-0.84
148.66	547.55	547.11	-0.44
151.74	550.36	549.75	-0.61

*interpolated cross-section

All computed water surface elevations were within 1 foot of matching the observed water surface elevation and only 3 of the 10 locations did not come within 0.5 foot of the observed water surface elevation. Since this HEC-RAS model utilizes current cross-section data, and no other significant out-of-bank flood events occurred in the recent past (1980 to present), no other flood events were analyzed.

Missouri River Profiles

Once we calibrated and verified the model, we made a copy of the geometry file and added the proposed levee to the cross-sections using the blocked obstruction option. We then generated eight existing condition water surface profiles along with eight with-project profiles. The starting water surface elevations for each of the profiles were taken from the rating curve at RM 116 produced from the Kansas City District Missouri River Backwater program. Exhibit 2 presents the HEC-RAS results. Figure 9 is a graphical representation of the existing condition profiles.

Missouri River Stage Trend

In addition to computing the existing conditions profiles, it was also necessary to estimate the future conditions stages. The April 2001 GRR presented an in-depth analysis of the stage trend forecast. The GRR forecast uses a combination of linear and ocular projections to estimate anticipated stage increases for different discharge ranges. The following Table 7 is an excerpt from Table B-2.11 of the GRR showing stage increases in the vicinity of Jefferson City.

TABLE 7
STAGE INCREASES AS REPORTED IN TABLE B-2.11 OF THE GRR

YEAR	Discharge, cfs				
	100,000	200,000	300,000	400,000	500,000
2025	0.5’	2.2’	2.1’	1.2’	1.6’
2050	0.9’	3.7’	3.4’	2.1’	2.8’

These values were modified to produce an estimated stage increase over time for each different flood event. Table 8 summarizes the results by event. Exhibit 3 provides supporting documentation for this analysis.

TABLE 8
MODIFIED STAGE INCREASES FOR VARIOUS FLOOD EVENTS

Exceedance Probability	Discharge cfs	2006 Stage Increase, ft	2031 Stage Increase, ft
0.5	208,000	0.80	2.59
0.2	297,000	0.77	2.41
0.1	361,000	0.73	2.26
0.02	516,000	0.58	1.85
0.01	588,000	0.48	1.64
0.002	772,000	0.13	1.01
0.0005	950,000	0	0.29
0.0002	1,100,000	0	0

Hydraulic Risk and Uncertainty

Uncertainties in computed water surface profiles are a result of imperfect knowledge and lack of appropriate data. Uncertainties in stage result from a number of physical factors such as bed forms, debris and other obstructions, channel scour or deposition, sediment transport, waves, and other physical factors. In hydraulic modeling, other factors such as hydraulic roughness variation with season, inexact geometry and loss coefficients, and error in setting high water marks result in errors in computed water surface elevations. Estimating these uncertainties in stage is based on sensitivity analyses, analytical studies of gauge readings, and interpretation of the success of model adjustments (EM 1110-2-1619).

We adopted stage uncertainty from the GRR for use in this study. Stage uncertainty is expressed in the Risk Based Analysis as a standard deviation (in feet). To obtain a total standard deviation, the GRR presents a thorough investigation of the gage data using the total period of record and only data from 1980 to the present. The GRR recommends using the latter as the source for calculating stage uncertainty for the different ranges of flood discharges. The resulting standard deviation used in the LRR Risk Analysis is 1.4 feet

Traditional Analysis vs. Risk Based Analysis

To account for uncertainties in discharge-frequency estimates, stage-discharge functions, and engineering parameters (geotechnical and structural), the traditional analysis allowed for "freeboard" to be added to the levee design as a safety factor above the estimated peak water surface elevation. For example, if 772,000 cfs were the design discharge, the levee would be designed to pass this flow with a certain degree of freeboard, typically 3 feet on Federal levees. When describing the project performance, or level of protection, under the traditional method, we would express the largest flood that could be accommodated by the project, with a high degree of assurance, as an average return period in years. Therefore, if the design discharge of 772,000 cfs had a frequency of 0.2-percent chance of exceedance, the levee could be characterized as a "500-year levee." The assurance or confidence relied on the freeboard to offset any and all other inaccuracies in the design.

Recently, the Corps of Engineers adopted a methodology called Risk Based Analysis (RBA) for formulating flood damage reduction projects. This method accounts for all the traditional parameters as quantified risks during the analysis in lieu of using freeboard to raise the level of confidence in project performance. Project performance is measured in terms of how a plan will function when exposed to a full range of floods that could occur. Under the RBA method, we express the level of protection as the average return period in years of the largest flood that can be accommodated by the plan under study, with a conditional non-exceedance probability of 90 percent. We no longer use the concept of freeboard in plan formulation.

For the L-142 Analysis, the top of levee was assigned a level of protection based on the RBA methodology rather than the traditional analysis. Adopting the new methodology requires a radical departure from the terminology associated with the more familiar way of designing, and talking about, levees. For example, what would have traditionally been described as a "100-year levee" would have had a top elevation 2 or 3 feet higher than the predicted peak elevation for a flood with a 1-percent chance of occurring in any year. Using the current methodology, the nearest counterpart to a "100-year levee" is described as a levee having a "90-percent chance of passing the 1-percent- chance exceedance flood event."

Recommended Top of Levee

Using the water surface profile data, stage and discharge uncertainty data, and the stage trend data as input, the Risk Analysis Program identified top of levee elevation of 561.5 at the index point (River Mile 142.8, ~ levee station 149+60) as the NED Plan. The economics section of this LRR has further discussion of the optimization process.

We determined the new levee design profile using the HEC-RAS model to produce a profile consistent with the RBA top of levee at the index point. The discharges in the model were increased until a computed water surface elevation reached 561.5 at the index point. Table 9 summarizes the discharges required to achieve this elevation.

TABLE 9
DISCHARGES USED TO SET TOP OF LEVEE PROFILE

Missouri River Mile Range for which discharge applies	Discharge Q (cfs)
116.0 – 130.3	1,055,000
130.3 – 138.0	907,000
138.0 – 156.0	900,000

Exhibit 4 shows HEC-RAS output for this profile. This profile formed the basis for setting the top of levee profile. Table 10 shows the design top of levee elevation. Exhibit 5 is a more detailed table of how we determined the top-of-levee elevations.

Main Stem Top of Levee (Levee Sta. 83+00 to 206+10)
For the portion of the levee that approximately parallels the Missouri River, approximately RM 144.1 (~levee station 83+00) to RM 141.73 (~levee station 206+10), the water surface profile was used to set the top of levee. To ensure that the levee would overtop near the downstream end, a superiority adjustment of up to 0.5 foot was added to the profile. The maximum profile increase of 0.5 foot extends from levee station 0+00 to levee station 131+50. After adding superiority to the profile, we used engineering judgment to "smooth" the profile for an acceptable level of design.

Upstream Tieback Top of Levee (Levee Sta. 0+00 to 83+00)
The 1993 flood showed water surface elevations considerably higher near the bluff than in the channel. To account for the higher elevations at the bluff, we used the energy grade elevation at RM 144.32 for the top of levee instead of the water surface elevation. This elevation of 567.6, including the adjustment for superiority, was used from levee station 0+00 to 60+00. Between levee stations 60+00 and 83+00, the top of levee elevation results from an interpolation between the energy grade elevation at RM 144.32 and the water surface elevation at RM 144.18.

Downstream Tieback Top of Levee (Levee Sta. 206+10 to 245+00)
Levee Station 206+10 (RM 141.73) is the approximate downstream location where the levee turns toward the bluff. The top of levee elevation at this point is based on the computed water surface elevation of 559.6 with no adjustments for superiority. This elevation is held constant for 400' to levee station 210+10. From station 210+10 to station 243+00 the top of levee transitions from 559.6 up to 561.0, which is the energy grade elevation at RM 141.54. The top of levee is at a constant elevation of 561.0 from station 243+00 to the tie-off at the bluff (station 248+00).

TABLE 10
DESIGN TOP OF LEVEE ELEVATIONS

HEC-RAS River Station	Equivalent Levee Station	Top of Levee Elevation (ft., msl)
X	0+00	567.6
144.32	60+00	567.6
144.18	79+00	565.1
X	83+00	564.6
144.04	85+60	564.3
143.918	90+90	563.8
143.898	92+30	563.8
143.86	94+30	563.8
143.818	96+40	563.0
143.56	109+50	562.7
143.48	113+20	562.6
143.39	116+70	562.5
143.31	120+80	562.4
143.22	126+00	562.3
143.14	131+50	562.1
142.8	149+60	561.9
142.58	161+50	561.7
142.16	182+00	560.7
141.73	206+10	559.6
X	210+10	559.6
X	243+00	561.0
141.54	248+00	561.0

X = No Cross section at this levee station

Note: Top of levee elevations may be increased slightly during the design phase to accommodate anticipated settlement during and after construction.

Hydraulic Effects

We also used HEC-RAS to analyze the effects to the water surface profiles as a result of the L142 Levee Unit. Table 11 summarizes the maximum difference in computed water surface elevations for each event comparing the existing conditions HEC-RAS plan to the with-project HEC-RAS plan. Changes in water surface profiles within the project reach (RM 141.5 – RM 144.16) range from 0 to 2.3 feet for the different events. Impacts to the water surface profiles upstream of the project reach range from 0 to 2.0 feet.

We developed profiles (10 percent and 1 percent) on Wears Creek (a small right bank tributary to the Missouri River) to assess increases in water surface elevations and induced damages in downtown Jefferson City as a result of the new levee. The profiles were developed from an existing HEC-RAS model that was used for a recent flood insurance study in Cole County/Jefferson City. We used Missouri River 0.2 percent (500yr) water surface elevations for the with/without project conditions for the years 2006 and 2031 (at Missouri RM 143.5) as starting water surface elevations for the Wears Creek model. The increase at the mouth of Wears Creek due to L-142 construction was

0.6' for this event. This increase gradually decreased to 0 approximately 1.8 miles upstream from the mouth. The changes in profiles were provided to economics to calculate the increase in damages due to the 0.6-foot increase in profiles.

TABLE 11
L142 EFFECTS TO COMPUTED WATER SURFACE ELEVATIONS (WSEL)

Exceedance Probability	Return Interval	Maximum Increase in Computed WSEL through Project Reach (ft) (RM 141.73-144.32)	Maximum Increase in Computed WSEL Upstream of Project Reach (ft) (RM 144.46-155.37)
0.5	2 year	0	0
0.2	5 year	0	0
0.1	10 year	0.3	0
0.02	50 year	0.4	0.4
0.01	100 year	0.5	0.5
0.002	500 year	0.8	0.9
0.0005	2000 year	1.7	1.6
0.0002	5000 year	2.3	2.0

ECONOMICS, RISK ANALYSIS AND OPTIMIZATION

The economic analyses are a further iteration of the same analytical process employed in the GRR. We continue to use the Omaha Risk program to analyze the composite flood risk of flood damage and determine the optimum project size with the maximum net economic benefits are. In addition to substituting the new hydrologic and hydraulic study outputs in the risk analysis, we field verified critical property values in the floodplain and refined the construction cost estimate based on developing knowledge of cost-sensitive features in the project design.

Economic Analysis

Background

The GRR for the L142 project was published in April 2001 and approved by HQUSACE in February 2002. The economics section of the GRR analyzed damages for existing conditions of 1999, base year conditions of 2006, and a future condition of 2031. Of the six competing levee alignments evaluated for the GRR, the NED plan was alternative 10a, which would protect all significant properties in the study area. The recommended top of levee elevation, optimized with risk analysis, was 562.0. (This elevation and all others in this discussion are relative to the economic index point, which is RM 142.8, about 1 mile downstream of the Highway 54 Missouri River bridge.) The plan had a first cost of $24.5 million in October 2000 prices and a benefit-cost ratio of 2.2. The engineering performance of the levee, based on the top of levee elevation, was expressed in terms of an estimated annual exceedance probability of 0.1 percent (1,000-year). The project schedule contained in the GRR assumed completion of the levee in 2006. The levee was assumed to have a project life of 50 years, consistent with Corps guidance on new levee construction.

Revised Missouri River flow data from the Upper Mississippi River System Flow Frequency Study (UMRFFS) became available subsequent to publication of the GRR. The new hydrologic data necessitated revised Missouri River water surface profiles. Because the economic damages and benefits are founded on a particular set of stage-frequency data, the need for revised profiles also implied the need to revisit and confirm the project's economic justification. In addition, although hydrologic or hydraulic changes do not affect the economic stage-damage relationship as such, we nevertheless decided to take the opportunity provided by the LRR to update the stage-damage relationship as well as the stage-frequency. Economic updates of authorized projects are required every 3 years, and although such an update is not required for L142 until next year (2004), the current revalidation of benefits and costs will obviate the need for another update until 2006 and hence allow design tasks to proceed with minimal interruption.

Methodology

The economic analysis for the LRR consisted of two major tasks. The first was to update the stage-damage relationship. The second was to conduct a new risk analysis to reoptimize the top of levee elevation and then produce revised benefit-cost data and engineering performance statistics for the optimal plan. Economic analysis in this study follows the procedures outlined in Economic and Environmental Principles and

Guidelines for Water and Related Resources Implementation Studies (P&G) in Engineering Regulation (ER) 1105-2-100, dated 22 April 2000, and in Risk-Based Analysis for Flood Damage Reduction Studies in Engineer Manual (EM) 1110-2-1619, dated 1 August 1996, and Benefit Determination Involving Existing Levees in Policy Guidance Letter No. 26, dated 23 December 1991.

In completing the initial task, the purpose of the economics revision for the LRR was not to carry out a comprehensive reevaluation of the economic analysis contained in the previously approved GRR. The more limited aim was to identify and account for any major changes in the area's economic base and flood damage potential since the 2001 GRR. This required contacting the major businesses and facilities in the L142 area to obtain current values for buildings, equipment, and inventory. About 94percent of the total investment in the area is accounted for by the city's airport and treatment plant plus two businesses (A.B.B., or Asea Brown Boveri, a major transformer plant, and M.F.A. Agri Services), so this task was clearly delineated.

We previously had conducted extensive on-site interviews at each of these facilities during preparation of the GRR. The interviews included personal inspection of plant and office layout and produced a structure of relevant data for each facility including depth-damage estimates prepared on-site in cooperation with key corporate or city staffers. These depth-damage relationships reflect information also obtained during the interviews concerning flood avoidance measures typically undertaken by the company or facility. The key facilities dominating the area's property base are very experienced in flood avoidance measures, which they undertake at least once almost every year, but there are sharp limits to pre-flood evacuation or removal at these large facilities.

In November 2002, we made return visits to the businesses and the city government. We obtained current depreciated values for buildings and equipment as well as average replacement values for inventory. Depth-damage relationships generally did not require revision for the LRR analysis since the assumptions on which they were based had not changed significantly. We changed property elevations only where inaccuracies were discovered subsequent to the GRR.

At A.B.B., the largest company in the area, we toured a major new addition to the plant constructed since the GRR interviews. We obtained current information on airport fixed-base operators while following the GRR analysis in assuming that almost all airplanes would be evacuated prior to a flood event. (Operational losses at the Jefferson City Airport are also substantial, but are regarded as RED or regional economic development losses rather than NED damages because locally-based users generally can meet their short-term, temporary needs by taking their flights to the regional airport in Columbia.) In addition to visiting the major facilities, we also surveyed the remainder of the developed floodplain and noted any changes in occupancy or land use.

We entered the resulting updated field data into a risk analysis program for computation of a revised stage-damage relationship. The initial portion of the risk analysis program produces a stage-damage function. In this portion of the analysis, uncertainty factors over which it is believed a variable could range are estimated for each of four key economic variables: ground elevation, first floor elevation, property value, and depth-damage percentages. These uncertainty factors are converted into standard deviations (in

terms of feet for elevations, and in percentages for property values and depth-damage) and entered into the risk program to be applied to the field data in flood simulations. The economic risk parameters used in the LRR analysis were held constant from the GRR analysis. These parameters included standard deviations of 0.3 foot for ground elevations and 0.1 foot for first floor elevations (based on use of floor plans for the larger enterprises and topographical maps with two-foot contours for other properties) and standard deviations of 15 percent for property values and 20 percent for depth-damage relationships.

The stage-damage analysis also assumed the presence of the existing non-Federal Capitol View levee in the study area. A proposed Federal levee is not credited with benefits for preventing damages that are already prevented by an existing levee. For the risk analysis, the performance of the existing levee was evaluated using a risk-based technique summarized in Policy Guidance Letter no. 26. This guidance calls for identification of a probable failure point (PFP) and a probable non-failure point (PNP) for the existing levee. The probability of failure would range from zero below the PNP to 100 percent above the PFP. This function was entered into the risk analysis program and is reflected in the average annual damage totals for without-project conditions.

Production of the stage-damage relationship with risk led into the second major task, which was to combine the updated economics data with the revised hydrologic and hydraulic data, as well as a new round of cost estimates, in a new risk analysis. The purpose was to determine the economically optimal top of levee elevation in light of the new data. We used a risk program developed by the Corps of Engineers Omaha District in the LRR analysis because it has been used throughout the previous planning stages of the project. Although newer risk programs such as HEC-FDA increasingly have become standard throughout the Corps, these programs were not available during the initial GRR preparation phases. We used the Omaha program throughout preparation of the approved economic analysis contained in the GRR, and we therefore wanted to maintain consistency in the programs used so that the LRR results could be compared directly to earlier economic analyses with validity.

The basic plan selection from the GRR was not reopened for the LRR analysis. The economic analysis for the GRR concluded that the NED (National Economic Development) plan - defined as the plan maximizing net benefits over costs - was alternative 10a. While we have no mandate to reformulate the accepted plan at the present advanced stage of design, we nevertheless considered the implications of the new data for the GRR plan selection process. Based on a close assessment of the GRR plan selection process, we find no indication that the revised data would have produced a different NED plan alignment

All six alternatives evaluated for the GRR were levee plans, differing only in how much property was protected at the extreme upstream and downstream ends of the study area. The downstream end is essentially undeveloped and is planted in crop acreage, resulting in very limited economic damage potential and little influence on the damage potential of the larger study area given the large industrial facilities upstream. The upstream end of the study area, just west of Highway 54, includes a few businesses whose overall value is again dwarfed by the facilities east of the highway.

The change in profiles affecting the upstream area theoretically could raise issues concerning plan selection, since the higher GRR profiles, now superceded, could have overstated damages upstream of the highway and resulted in awarding economic advantage to alignments that included the full upstream area. Yet, the second- and third-ranking alignments in the GRR economic screening analysis differed from the selected plan, 10a, only in the inclusion or non-inclusion of the downstream crop acreage. Of the six alternatives evaluated during the GRR screening process, the two that involved less protection upstream of the highway came in fourth and fifth, some 10- to 15-percent behind the others in net benefits. For these reasons, the selection of Alternative 10a is not compromised by the revised profiles and damage data. The LRR analysis aims only to determine the economically optimal top of levee elevation in the context of the revised data supporting the new risk analysis.

We developed updated cost estimates for four points encompassing a nine-foot range in levee height at the economic index point (RM 142.8): 556.0, 559.0, 562.0, and 565.0. We annualized these costs using the current Federal interest rate of 5.875 percent. and a 50-year project life. All cost estimates include interest during construction and annual operation and maintenance costs. We calculated Interest during construction (IDC) based on an approximately 5-year implementation period from FY 03 to project completion in FY 08. We used the same time period for all sizes of levees considered, based on consultation with our project structural engineers who indicated that duration of construction would not differ significantly within the range of levee sizes being evaluated. IDC amounted to a little over 13 percent of total first costs for all annual cost totals calculated. We included annual operation and maintenance costs of $20,000 to $25,000 depending on the size of the levee. These costs would vary little between different sizes of levee in the range evaluated, according to project structural engineers. We then developed a cost curve from the annual costs for each of the four points. We interpolated annual costs for other elevations to be tested between the four points.

The risk analysis tested top of levee elevations throughout the 9-foot range from 556.0 to 565. That is to say, elevations were tested beginning 6 feet below the GRR plan's top of levee elevation of 562.0 and ending 3 feet above it. Within the ranges from 556.0 to 559.0 and from 562.0 to 565.0, top of levee elevations, we tested at half-foot intervals. Within the range from 559.0 to 562.0, we tested each tenth of a foot. Each potential top of levee elevation was evaluated for both base year (2006) and future year (2031) conditions. (Existing conditions of 1999 were not reevaluated for this analysis, which mainly focuses on the with-project condition.) The property base reflected in the future condition is identical to base year conditions; i.e., we assumed no additional investment for the future condition. The only variable separating base and future conditions in the risk analysis is the rise in Missouri River water surface profiles.

Without and with-condition estimates of average annual damages produced in the risk analysis ultimately were taken outside the risk program for completion of the benefit-cost estimates. For computation of the benefits, base year and future condition benefits were compared at each elevation tested. The growth from base to future was computed, then discounted back to the base year and added to the base year benefits. The total benefits for each nominated elevation therefore represent the base year benefits plus the discounted present worth of the growth in benefits from base year to future condition.

The total benefits were then matched with the appropriate annual cost for the top of levee elevation, resulting in benefit-cost ratios and net benefits for each point tested.

Damage analysis

The revised water surface profiles serving as the basis for the damage analysis are summarized in Table 12. For both the base year (2006) and future condition (2031), the current profiles for the 50-percent- and 20-percent-chance events have increased more than a foot over the GRR estimates. The 10-percent-chance event elevations are unchanged in both conditions. In the larger events, the LRR profiles are slightly lower than the GRR estimates.

Total investment for the L-142 area, estimated at $134,256,000 in 2001, has increased by a modest 4.5 percent to $140,311,000. (The 2001 GRR's published estimate of total investment actually was almost $200 million, but this total included several dozen aircraft, which would be evacuated prior to a flood. The current totals do not include aircraft.) The 2003 study area includes two major public facilities, three other public or institutional buildings, nine businesses, one residence, and one farm set. Table 13 summarizes the changes in the study area's investment from the GRR to the current estimates. Values shown for highways, streets, and crop acreage, representing only a very small portion of overall investment, were not changed for this analysis.

TABLE 12
CHANGES IN WATER SURFACE PROFILES, GRR TO LRR

Percent chance exceedance	BASE YEAR (2006)			FUTURE YEAR (2031)		
	GRR	LRR	Change	GRR	LRR	Change
50	542.6	543.9	+1.3	544.2	545.6	+1.4
20	547.9	549.6	+1.7	549.6	551.2	+1.6
10	551.8	551.8	---	553.4	553.4	---
2	555.0	554.6	-0.4	556.6	555.9	-0.7
1	556.3	556.0	-0.3	558.0	557.1	-0.9
0.2	558.8	559.0	+0.2	560.4	559.8	-0.6
Elevations refer to the Economic Index Point, RM 142.8, with the project in place						

TABLE 13
TOTAL INVESTMENT BY CATEGORY, GRR TO LRR

In thousands of dollars				
	GRR (2001)	LRR (2003)	change	change (percent)
Public facilities	$44,457.0	$40,284.0	-$4,173.0	-9.4%
Businesses	$83,393.0	$93,991.0	+$10,598.0	+12.7%
Residences	$433.5	$63.5	-$370.0	-85.4%
Highways & streets	$4,940.0	$4,940.0	---	---
Crop acreage	$1,033.0	$1,033.0	--	---
TOTAL	$134,256.4	$140,311.4	$6,055.0	+4.5%

Table 14 summarizes the revised stage-damage results. The LRR totals show increases over the GRR for the most frequent events (50- and 10-percent-chance), with damages for the 10-percent-chance event increasing by 49 percent. The damage estimates for the larger events are lower. The 2-percent-chance-event estimated damages are 54 percent lower than in the GRR, and the 1-percent-chance-event damages have dropped by 41 percent. These increases or decreases, though they appear to have a certain amount of significance, are somewhat misleading because the stages associated with each event have changed since the GRR. For example, as displayed in Table 12, the 50-percent-chance event stage has increased 1.3 feet from GRR to LRR, making the 307-percent increase in damages much less surprising. The 54-percent decrease in damages for the 2-percent-chance event is in large part caused by the 0.4-foot drop in the stage for that event.

TABLE 14
STAGE-DAMAGE CHANGES, GRR TO LRR

Percent chance exceedance	GRR	LRR	Change	Change (percent)
	In thousands			
50	$8.4	$34.2	+$25.8	+307
10	$5,998.5	$8.922.9	+$2,924.4	+49
2	$33,521.6	$15,384.0	-$18,137.6	-54
1	$66,059.3	$38,736.4	-$27,322.9	-41
0.2	$103,754.3	$84,921.4	-$18,832.9	-18
0.05	$112,361.8	$101,520.3	-$10,841.5	-10
0.02	$114,150.3	$109,166.4	-$4,983.9	-4

Average annual damages combine stage-damage results with engineering inputs, including the revised hydrology and hydraulics. Current estimates of average annual damages, as computed in the risk analysis for both base and future conditions, have fallen slightly relative to the GRR totals. Average annual damages for the base year conditions of 2006 total $3,361,000, a 3.6-percent reduction from the GRR estimate of $3,486,000. For future conditions of 2031, total average annual damages have fallen from $5,892,000 to $5,336,000, a reduction of 9.4 percent. The average annual damage total representing both base and future conditions is $4,329,000. This total includes base year damages plus the present worth of future growth in damages.

The LRR plan

The optimal plan, in economic terms, is the plan with the highest net benefits (excess of annual benefits over annualized costs). The outcome of the top of levee reoptimization, summarized in detail in Table 15, is a new top of levee elevation of 561.5. This top of levee elevation is one-half foot lower than the GRR plan at the economic index point (RM 142.8). All top of levee elevations tested displayed strong economic justification, and the optimal elevation has only marginal superiority over a large range of potential top of levee elevations. All elevations tested between 559.7 and 562.0, for example, have net benefits within 1percent of the optimal elevation.

Benefits and costs of the LRR plan

The estimated first cost of the LRR plan is $26,909,000, and the annualized cost is $1,924,000. Set against the plan's annual benefits of $4,159,000, the result is a benefit-cost ratio of 2.2. Net benefits total $2,235,000. All benefits and costs are expressed in October 2002 prices and reflect the current Federal interest rate of 5.875 percent. Table 16 summarizes benefit-cost data for the LRR and GRR.

TABLE 15
TOP OF LEVEE OPTIMIZATION

October 2002 prices; interest rate = 5.875%: Dollar values in thousands
Description of columns:

(1) Top of levee elevation
(2) Residual average annual damages – base year
(3) Benefits – base year (assumes total average annual damages of $3,361.4)
(4) Residual average annual damages – future condition
(5) Benefits – future condition (assumed total average annual damages of $5,336.3)
(6) Growth in benefits, base year to future condition
(7) Discounted growth in benefits (present worth)
(8) Total benefits – base + future (assumes reduction of $10.2 in residual damages from interior drainage and addition of $0.4 for minor benefit categories)
(9) Annual costs
(10) Benefit-cost ratio
(11) Net benefits

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
556.0	$857.4	$2,503.9	$1,671.9	$3,664.4	$1,160.5	$574.6	$3,068.7	$1,603.0	1.9	$1,465.7
556.5	$746.4	$2,614.9	$1,409.6	$3,926.7	$1,311.8	$647.3	$3,252.4	$1,627.0	2.0	$1,625.4
557.0	$636.0	$2,725.4	$1,179.6	$4,156.7	$1,431.3	$704.8	$3,420.3	$1,651.0	2.1	$1,769.3
557.5	$532.9	$2,828.5	$948.6	$4,387.7	$1,559.2	$766.0	$3,584.7	$1,675.0	2.1	$1,909.7
558.0	$448.2	$2,913.2	$776.6	$4,559.6	$1,646.5	$808.0	$3,711.3	$1,699.0	2.2	$2,012.3
558.5	$383.0	$2,978.4	$612.8	$4,723.4	$1,745.1	$854.9	$3,823.4	$1,723.0	2.2	$2,100.4
559.0	$326.3	$3,035.1	$487.5	$4,848.8	$1,813.7	$887.6	$3,912.9	$1,747.0	2.2	$2,165.9
559.1	$316.0	$3,045.3	$469.5	$4,866.8	$1,821.4	$891.4	$3,926.9	$1,754.1	2.2	$2,172.8
559.2	$307.0	$3,054.4	$457.8	$4,878.4	$1,824.1	$892.7	$3,937.3	$1,761.1	2.2	$2,176.2
559.3	$297.5	$3,063.8	$435.7	$4,900.5	$1,836.7	$898.7	$3,952.8	$1,768.2	2.2	$2,184.6
559.4	$276.9	$3,084.5	$420.3	$4,915.9	$1,831.5	$896.6	$3,971.3	$1,775.3	2.2	$2,196.0
559.5	$268.7	$3,092.7	$386.9	$4,949.4	$1,856.7	$908.5	$3,991.3	$1,782.3	2.2	$2,209.0
559.6	$261.6	$3,099.7	$376.0	$4,960.3	$1,860.6	$910.4	$4,000.3	$1,789.4	2.2	$2,210.9
559.7	$253.5	$3,107.9	$365.9	$4,970.4	$1,862.5	$911.4	$4,009.5	$1,796.5	2.2	$2,213.0
559.8	$250.7	$3,110.6	$352.3	$4,984.0	$1,873.4	$916.5	$4,017.3	$1,803.5	2.2	$2,213.7
559.9	$246.5	$3,114.8	$342.6	$4,993.7	$1,878.9	$919.1	$4,024.1	$1,810.6	2.2	$2,213.5
560.0	$240.5	$3,120.9	$331.4	$5,004.8	$1,884.0	$921.5	$4,032.6	$1,817.7	2.2	$2,214.9
560.1	$234.8	$3,126.6	$315.8	$5,020.5	$1,893.9	$926.3	$4,043.0	$1,824.7	2.2	$2,218.3
560.2	$224.9	$3,136.5	$299.6	$5,036.6	$1,900.2	$929.3	$4,056.0	$1,831.8	2.2	$2,224.2
560.3	$213.0	$3,148.3	$287.9	$5,048.4	$1,900.0	$929.4	$4,068.0	$1,838.9	2.2	$2,229.1

October 2002 prices; interest rate = 5.875%: Dollar values in thousands

Description of columns:

(1) Top of levee elevation
(2) Residual average annual damages – base year
(3) Benefits – base year (assumes total average annual damages of $3,361.4)
(4) Residual average annual damages – future condition
(5) Benefits – future condition (assumed total average annual damages of $5,336.3)
(6) Growth in benefits, base year to future condition
(7) Discounted growth in benefits (present worth)
(8) Total benefits – base + future (assumes reduction of $10.2 in residual damages from interior drainage and addition of $0.4 for minor benefit categories)
(9) Annual costs
(10) Benefit-cost ratio
(11) Net benefits

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
560.4	$208.5	$3,152.9	$276.1	$5,060.2	$1,907.3	$932.9	$4,076.0	$1,845.9	2.2	$2,230.0
560.5	$200.5	$3,160.9	$268.6	$5,067.6	$1,906.8	$932.7	$4,083.8	$1,853.0	2.2	$2,230.8
560.6	$194.3	$3,167.1	$256.2	$5,080.1	$1,913.0	$935.7	$4,093.0	$1,860.1	2.2	$2,232.9
560.7	$186.7	$3,174.6	$247.3	$5,088.9	$1,914.3	$936.4	$4,101.3	$1,867.1	2.2	$2,234.1
560.8	$176.0	$3,185.3	$244.3	$5,091.9	$1,906.6	$933.0	$4,108.5	$1,874.2	2.2	$2,234.3
560.9	$176.0	$3,185.3	$241.2	$5,095.1	$1,909.7	$934.5	$4,110.0	$1,881.3	2.2	$2,228.7
561.0	$167.8	$3,193.5	$229.3	$5,106.9	$1,913.4	$936.3	$4,120.0	$1,888.3	2.2	$2,231.7
561.1	$164.3	$3,197.1	$222.2	$5,114.1	$1,917.0	$938.0	$4,125.3	$1,895.4	2.2	$2,229.9
561.2	$154.9	$3,206.5	$210.9	$5,125.4	$1,918.9	$939.1	$4,135.7	$1,902.5	2.2	$2,233.3
561.3	$149.8	$3,211.5	$200.7	$5,135.6	$1,924.1	$941.5	$4,143.2	$1,909.5	2.2	$2,233.7
561.4	$140.8	$3,220.6	$197.6	$5,138.7	$1,918.1	$938.9	$4,149.7	$1,916.6	2.2	$2,233.0
561.5	136.0	$3,225.4	$183.3	$5,153.0	$1,927.7	$943.4	$4,158.9	$1,923.7	2.2	$2,235.3
561.6	$132.6	$3,228.7	$180.2	$5,156.0	$1,927.3	$943.3	$4,162.2	$1,930.7	2.2	$2,231.5
561.7	$130.8	$3,230.5	$171.7	$5,164.6	$1,934.1	$946.5	$4,167.2	$1,937.8	2.2	$2,229.4
561.8	$129.4	$3,231.9	$163.7	$5,172.6	$1,940.7	$949.5	$4,171.7	$1,944.9	2.1	$2,226.8
561.9	$124.3	$3,237.1	$153.4	$5,182.9	$1,945.8	$952.0	$4,179.3	$1,951.9	2.1	$2,227.3
562.0	$122.3	$3,239.1	$150.1	$5,186.1	$1,947.1	$952.6	$4,181.9	$1,959.0	2.1	$2,222.9
562.5	$101.2	$3,260.1	$130.2	$5,206.1	$1,946.0	$952.4	$4,202.7	$2,003.3	2.1	$2,199.4
563.0	$80.4	$3,280.9	$103.3	$5,233.0	$1,952.0	$955.5	$4,226.7	$2,047.7	2.1	$2,179.0
563.5	$71.3	$3,290.1	$83.6	$5,252.6	$1,962.5	$960.5	$4,240.8	$2,092.0	2.0	$2,148.8
564.0	$54.9	$3,306.5	$70.7	$5,265.6	$1,959.1	$959.2	$4,255.9	$2,136.3	2.0	$2,119.5
564.5	$51.0	$3,310.3	$52.4	$5,283.9	$1,973.5	$966.0	$4,266.5	$2,180.7	2.0	$2,085.8
565.0	$47.1	$3,314.3	$50.6	$5,285.7	$1,971.4	$965.0	$4,269.5	$2,225.0	1.9	$2,044.5

Benefit totals include about $400 in flood insurance benefits in addition to the physical inundation damages prevented. An estimated $170,000 in average annual residual damages would continue to occur in the with-project condition.

TABLE 16
REVISED BENEFIT-COST DATA

For LRR plan with top of levee = 561.5 GRR data in thousands of Oct. 2000 dollars; interest rate = 6.125% LRR data in thousands of Oct. 2002 dollars; interest rate = 5.875%			
	GRR	**LRR**	**Percent Change**
First cost	$24,508.0	$26,909.0	+9.8%
Annual benefits	$4,442.0	$4,159.0	-6.4%
Annual costs	$1,979.0	$1,924.0	-2.8%
Benefit-cost ratio	2.2	2.2	
Net benefits	$2,463.0	$2,235	

Non-Federal sponsor and cost-sharing

Funding for the L142 project will be guided by the general policy of a 75/25 split between Federal and non-Federal obligations. (The 25 percent non-Federal share is a minimum that can be increased by LERRD costs.) The City of Jefferson City is the local sponsor for the L142 project. The City currently has plans in place for submitting a local improvements tax to voters. Proceeds from this tax would cover the local sponsor's share of the total project cost, which is currently estimated at $6,727,000. This computation is summarized in Table 17. A thorough evaluation of the sponsor's ability to finance the project will be completed in conjunction with preparation of the project cooperation agreement (PCA), but it can be said that the City has cooperated with the Corps on previous studies and has met its obligations satisfactorily. Based on ongoing discussions with the City concerning their plans for financing the L142 project, we do not at this time foresee any fundamental obstacles affecting its participation in a PCA.

TABLE 17
ESTIMATED NON-FEDERAL FUNDING OBLIGATION

First cost	$26,909,000
Parameters	
Minimum 25 percent local share	$6,727,000
Minimum 5 percent local cash	$1,346,000
LERRD	
Lands & damages	$2,432,000
Relocations	$2,458,000
Total LERRD	$4,800,000
Cash balance	$1,927,000
Total local share	$6,727,000
Total Federal share	$20,182,000

Engineering performance of the LRR plan

In addition to computing economic performance data, the risk analysis program evaluates engineering performance or reliability by performing numerous simulations of flood events and comparing the number of overtopping events to the total number of simulations. The raw data from the risk simulations can be expressed in terms of a variety of statistics, as summarized in Table 18.

A key result is that the selected levee plan would have a 90-percent chance of containing a 0.26-percent-chance event. The corresponding return interval is 380 years. The 90-percent confidence statistic is important because the Corps standard for identifying the nominal levee performance is the 90-percent confidence level. Moreover, a levee must have a 90-percent or better chance of containing a 1-percent-chance event to gain certification from the Federal Emergency Management Agency (FEMA). The LRR plan, which has an estimated 99.7-percent-chance of containing a 1-percent-chance event, is certifiable under FEMA requirements.

Another important result for Jefferson City is the 88.9-percent confidence in containing an event equivalent to the record 1993 Missouri River flood. A slightly larger 0.2-percent-chance (500-year) event would be contained with 84.8 percent confidence. The levee would have 95-percent confidence in containing the 0.37-percent-chance (270-year) event. The 50-percent confidence point, at which the levee would have a 50-50 chance of containing the flood, is a 0.071-percent (1,410-year) event. The annual exceedance probability of the levee, defined as the annual chance of experiencing any overtopping event of whatever magnitude, is 0.12 percent.

The above statistics assume conditions of 2006, the base year in which the levee would become operational and also the key reference year for assessing levee performance. However, a future condition, 25 years after the levee becomes operational (reference year 2031), also was evaluated, as shown in Table 18. Rising Missouri River stage trends would lessen the levee's degree of performance somewhat by 2031, but the levee would retain a robust margin of protection.

TABLE 18
ENGINEERING PERFORMANCE OF SELECTED PLAN

	2006 conditions	**2031 conditions**
Largest event contained with 90 percent confidence Return interval	0.26 percent 380 years	0.35 percent 290 years
Largest event contained with 95 percent confidence Return interval	0.37 percent 270 years	0.50 percent 200 years
Largest event contained with 50 percent confidence Return interval	0.071 percent 1410 years	0.095 percent 1050 years
Annual probability of overtopping Return interval	0.12 percent 810 years	0.17 percent 590 years
	2006 conditions	2031 conditions
Chance of containing 1 percent (100-year) event Height that levee exceeds 1 percent event at the index point, RM 142.8	99.7 percent 5.1 feet	99.2 percent 4.0 feet
Chance of containing 1993 event Height that levee exceeds 93 event at the index point, RM 142.8	88.9 percent 3.1 feet	82.3 percent 1.9
Chance of containing 0.2 percent (500-year) event Height that levee exceeds 0.2 percent event at the index point, RM 142.8	84.8 percent 1.9 feet	76.5 percent 1.1 feet
Note: Exceedance heights in this table are relative to the index point of RM 142.8; they will vary along the levee profile.		

Induced damages potential

Construction of the L142 levee presents at least the theoretical potential for higher flood elevations in areas immediately upstream, downstream, or across the river from the levee. Particularly at issue is the downtown/Capital Complex area of Jefferson City, which is directly across the river from the levee. Portions of this area, including several large hotels and office buildings, are in the Wears Creek floodplain, which is affected by Missouri River backwater during large flood events on the river. We developed a set of Wears Creek water surface profiles, without and with the L142 levee in place, which assumed a 0.2-percent-chance event on the Missouri River together with either a 10-percent-or 1-percent-chance event on Wears Creek. (Note that the backwater profiles assume a Missouri River flood event that is even larger than the 1993 flood.) These profiles were entered into a traditional spreadsheet damage program along with field data for major properties in the Wears Creek basin. GRR and LRR study funding did not allow for a comprehensive right bank survey to include all properties potentially affected in Jefferson City, but several attenuated surveys of the area were made in 2001 and 2002 to account for the largest properties with the most critical damage potential. Updated GIS-based contour mapping also was obtained from the City for estimation of property elevations.

The right bank damage analysis evaluated without and with condition damages for both base year and future conditions. The Wears Creek water surface profiles reflecting backwater from major Missouri River flood events indicate that construction of the L142 levee could raise stages about a half-foot in the urban Jefferson City floodplain. However, this stage increase assumes an extremely rare event on the Missouri River occurring simultaneously with a significant (e.g., uncommon) event on Wears Creek. As

a result, any additional average annual damages due to effects of Levee Unit L142 would be minimal. The LRR analysis of potential induced damages in Jefferson City found only about a $1,000 increase in average annual damages due to the levee. This result is similar to the findings of a similar analysis done for the GRR. We have no indication that construction of the L142 levee would significantly augment flood damage along Wears Creek during Missouri River flooding.

Several factors explain the relatively small economic induced effects: (1) The Jefferson City government has bought out many damageable properties in the Wears Creek floodplain over the past 30 years, and only a small number of damageable properties remain; (2) Remaining properties are relatively new and are built high relative to the floodplain (These two factors account for the failure of several studies of the potential for structural flood reduction measures in the Wears Creek floodplain to find economic justification.); and (3) The induced effects occur only in extremely large and infrequent Missouri River events, which limits the size of the effects when the damages are expressed in annual terms. We have no indication that construction of the L142 levee would significantly augment flood damage along Wears Creek during Missouri River flooding.

ENVIRONMENTAL CONSIDERATIONS

We prepared the L142 Environmental Assessment (EA) and published it in the GRR in April 2001. The GRR and the mitigative FONSI were both signed on 26 April 2001. We determined by the mitigative FONSI that there are no significant impacts "…with wetland mitigation, as proposed, in place."

The GRR assumed a "worst case" scenario. As the top-of-levee elevation proposed in the LRR has decreased, we reviewed existing NEPA compliance (found in the GRR) for the proposed project and determined that the existing EA/FONSI is adequate. However, several important environmental measures must be included in the plans & specifications, including:

a. The 42-acre borrow site in the northeast corner of the proposed project must be developed into a permanent wetland. The wetland must be designed to avoid large areas of open water (an airport requirement);

b. Final design and construction will use an unnamed tributary to Turkey Creek to provide a reliable water source and a potential seed source for the created wetland. Using that tributary to supply the water may require breaching or modifying a small agricultural levee on the tributary's bank;

c. Project activities must not affect the significant cultural resources site identified as "23CY236";

d. The project must incorporate the State of Missouri's "Americans with Disabilities Act" (ADA) requirements at the intersection with the KATY trail;

e. The project must incorporate the other environmental protection measures discussed in the EA.

And, as stated in the FONSI, " this determination may be reevaluated if warranted by later developments."

CONCLUSION

We performed a hydraulic analysis on the Missouri River near Jefferson City, Missouri to revalidate the NED plan top-of-levee elevation for Missouri River Levee Unit L142 and to identify the top-of-levee profile. The hydraulic computations are based on a HEC-RAS computer software program calibrated to the 1993 event and verified with the 1995 observed water surface profile. The model incorporates new discharges from a comprehensive study of the Lower Missouri and Kansas River Basins completed in 2001. The optimal top of levee elevation at the economic index point is 561.5 ft., msl. The levee with a top elevation profile consistent with the optimal top elevation at the index point would have a 90-percent chance of containing a 0.26-percent-chance event and would exceed the 1-percent-chance event elevation by 5.1 feet. The benefit-cost ratio of 2.2 revalidates this project's continued strong economic justification. Table 19 compares some significant values for the currently recommended levee and the levee design described in the GRR.

TABLE 19
COMPARISON OF GRR AND LRR RECOMMENDED PROJECTS

Item	GRR	LRR
1-percent-chance flood elevation at RM 142.8 with levee in 2006	556.3 ft., msl	556.6 ft., msl
1-percent-chance flood elevation at RM 142.8 with levee in 2031	557.9 ft., msl	558.2 ft., msl
Levee Reliability in the 1-percent chance flood in 2006	99.4 percent	99.7 percent
Levee Reliability in the 1-percent-chance flood in 2031	97.5 percent	99.2 percent
Increase in the 1-percent-chance flood profile at RM 142.8 with the recommended levee in place in 2006	Less than 0.2 foot	Not more than 0.5 foot
Increase in the 0.2-percent-chance flood profile at RM 142.8 with the recommended levee in place in 2006	0.2 to 0.6 foot	Not more than 0.8 foot
Increase in the 1-percent-chance flood profile at RM 142.8 with the recommended levee in place in 2031	Less than 0.2 foot	Not more than 0.5 foot
Increase in the 0.2-percent-chance flood profile at RM 142.8 with the recommended levee in place in 2031	0.2 to 0.6 foot	Not more than 0.8 foot
Estimated Benefit-Cost Ratio	2.2	2.2
Estimated Construction Cost	$24,508,000	$26,909,000

RECOMMENDATION

I recommend that design of Missouri River Levee Unit L142 proceed based on the finding that the NED Plan is a levee on Alignment 10a, as identified in the April 2001 General Reevaluation Report, except that the levee design will be in accordance with the findings in this report for a top-of-levee profile consistent with elevation 561.5 at the index point, Missouri River Mile 142.8, at an estimated construction cost of $26,909,000, and a benefit-cost ratio of 2.2.

DONALD R. CURTIS, JR.
Colonel, EN
Commanding

REFERENCES

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4. USACE 1994 – The Great Flood of 1993 Post Flood Report, Lower Missouri River Basin, Appendix E, US Army Corps of Engineers Kansas City District, September 1994.

5. USGS 1995 – Hydrologic Atlas HA 735-A, Delineation of Flooding Within the Upper Mississippi River Basin – Flood of July 30, 1993, in Jefferson City and Vicinity, Missouri, 1995.