Upper Des Plaines River Feasibility Study

Appendix A – Hydrology and Hydraulics

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Study Partnership
Illinois Department of Natural Resources (IDNR)
Southeastern Wisconsin Regional Planning Commission (SEWRPC)
Lake County Stormwater Management Commission (LCSMC)
Forest Preserve District Lake County (FPDLC)
Metropolitan Water Reclamation District of Greater Chicago (MWRDGC)
Cook County Highway Department (CCHD)
Forest Preserve District of Cook County (FPDCC)
U. S. Fish and Wildlife Service (USFWS)
U. S. Army Corps of Engineers (USACE)

US Army Corps
of Engineers
Chicago District

APPENDIX A Upper Des Plaines River Feasibility Study
Hydrology and Hydraulics

TABLE OF CONTENTS

LIST OF TABLES

LIST OF PLATES

LIST OF ATTACHMENTS

INTRODUCTION

WATERSHED DESCRIPTION

The Des Plaines River originates in Racine and Kenosha Counties in southeastern Wisconsin where the basin is primarily agricultural. The river enters Illinois in Lake County, flowing southward through Cook County to its confluence with Salt Creek in Riverside, Illinois where it turns sharply east and then curves to the southwest. The Des Plaines River flows southwest from Riverside to its confluence with the Chicago Sanitary and Ship Canal near Lockport, Illinois. The watershed is aligned primarily along a north-south axis with a length of 82 miles and an average width of 9 miles. The rise of the Des Plaines River from Lockport to its junction with the Root River in Wisconsin is about 130 feet, or an average of 1.3 feet per mile. The drainage area of the watershed at Riverside, including the tributary Salt Creek, is 630 square miles (See Main Report Plate 1).

Most of the southern half of the watershed is fully developed. The open areas remaining in that part of the watershed are primarily golf courses, forest preserves, parks, and cemeteries. The northern portion of the watershed continues to be developed as a primarily residential area with some commercial development.

PROJECT DESCRIPTION

This study is a continuation and extension of the Upper Des Plaines River Flood Damage Reduction Feasibility Study (Phase I Study) (USACE 1999). The Phase I Study was initiated to address severe overbank flooding along the Upper Des Plaines River. It investigated plans for urban flood risk management in the Upper Des Plaines River watershed and recommended six projects to reduce the main stem flooding. This Upper Des Plaines River and Tributaries Feasibility Study (Phase II Study) provides an opportunity to develop a more comprehensive solution to ongoing occurrences of flooding in the Upper Des Plaines River watershed in addition to the Phase I projects. The Phase II Study has three primary objectives: further reduction of mainstem flooding, reduction of tributary flooding, and environmental restoration of degraded ecosystems within the basin. Secondary objectives are improving water quality and enhancing recreational opportunities throughout the basin. The study will consider sites located within tributary watersheds and along the mainstem for both Flood Risk Management (FRM) and Ecosystem Restoration (ER) potential. The effects of flood-risk management sites within tributary watersheds on mainstem flooding will also be evaluated.

The study area for this project is the Upper Des Plaines River watershed and its tributaries upstream of the confluence with Salt Creek(See Main Report Plate 1). In support of this study, updated hydrologic and hydraulic modeling were developed for 15 tributaries: in Wisconsin, Unnamed Tributary No. 6, Salem Branch, Brighton Creek, Kilbourn Ditch, and Dutch Gap Canal; and in Illinois, Mill Creek, Newport Drainage Ditch, Bull Creek, Indian Creek, Buffalo-Wheeling Ditch, McDonald Creek, Weller Creek, Farmer-Prairie Creek, Willow-Higgins Creek, and Silver Creek. Various governmental agencies were responsible for hydrologic and hydraulic modeling of the tributaries of the Des Plaines River. The work contributed by each agency is summarized in Table 1 below. General information about the tributary models appears in this Appendix;

detailed reports on individual tributary models are cited in the References section.

Table 1. Tributary Model Completion Dates.

Tributary Name	Agency	Date Completed
Unnamed Trib. No. 6	SEWRPC	2005
Salem Branch	SEWRPC	2005
Brighton Creek	SEWRPC	2005
Kilbourn Road Ditch	SEWRPC	2005
Newport Drainage Ditch	LCSMC	2008
Dutch Gap Canal	SEWRPC	2005
Mill Creek	LCSMC	2008
Bull Creek	USACE	2005
Indian Creek	USACE	2007
Buffalo-Wheeling Ditch	IDNR	2006
McDonald Creek	USACE	2008
Weller Creek	USACE	2004
Farmer/Prairie Creek	IDNR	2005
Willow-Higgins Creek	CCHD	2005
Silver Creek	USACE	2007

Note: CCHD – Cook County Highway Department, IDNR – Illinois Department of Natural Resources, LCSMC – Lake County Stormwater Management Commission, SEWRPC – Southeastern Wisconsin Regional Planning Commission, USACE – US Army Corps of Engineers, Chicago District.

HYDROLOGY

MODEL DEVELOPMENT

The computer application HEC-1 was used for the Upper Des Plaines River model in the Phase I study. HEC-1 was also the preferred model for the Illinois tributaries. HEC-1 simulates the surface-runoff response of a watershed to precipitation. Simulations are limited to a single storm because the application does not include parameters for soil moisture recovery. This recommendation was based on the desire to eventually integrate these hydrology models for tributaries into the Phase I Des Plaines River watershed model, which was developed in HEC-1. More detailed information about the modeling effort on the Des Plaines River can be found in USACE (1999).

The tributaries in Wisconsin were modeled using the USEPA's Hydrological Simulation Program-Fortran, HSPF. This application performs continuous, long-term simulations of the hydrologic cycle in a watershed. Information on the hydrologic modeling for Wisconsin is summarized briefly in the next section; more detailed information on these tributaries can be found in SEWRPC (2003).

HSPF MODELING IN WISCONSIN

The Southeastern Wisconsin Regional Planning Commission (SEWRPC) used HSPF, a water resource simulation model, for the Des Plaines River watershed. It combined hydrologic, hydraulic, and water quality submodels in an effort to assess the need for an evaluation of existing and future flood control measures in the watershed. Plate A-1 shows the subbasin delineations in the Wisconsin portion of the Des Plaines River watershed.

The hydrologic submodel determines the volume and rate of runoff based on meteorological and land use inputs. Hydrologic Land Segment Types are assigned to areas of the watershed based on a combination of hydrologic soil groups, land cover, and proximity to meteorological station. Soil groups were determined by the National Resources Conservation Service (NRCS) regional soil survey. Land cover was determined by base maps and aerial photographs maintained by SEWRPC. A Theissen polygon network was constructed to subdivide the watershed into areas lying closest to each of three weather stations in Kenosha, WI, Union Grove, WI, and Antioch, IL. Large-scale topographic mapping, with 2-foot contours and a 1"=200' scale, was also used for the hydrologic submodel.

The first hydraulic submodel, HSPF accepts as inputs the surface runoff and groundwater discharges from the hydrologic submodel and routes it through the stream system. Reach routing is accomplished using a reservoir routing technique. HSPF outputs are discharge time series. Flow frequency analysis of the annual peak flows generates the various recurrence interval flood discharges that are then input to the second hydraulic submodel, HEC-2, for stage calculations.

Historical streamflow and flood stage data from USGS gages as well as Wisconsin DNR and SEWRPC were used to calibrate the models.

HEC-1 MODEL PARAMETERS

To develop an HEC-1 model of a watershed, various parameters need to be developed to describe it. These parameters include subbasin delineation, drainage area, curve numbers, unit hydrograph parameters such as time of concentration or lag time, base flow, and parameters for routing calculations. The derivation of these parameters is summarized in the following paragraphs.

Basin Delineation and Drainage Area

Drainage basins were re-delineated during the development of new hydrologic models. Plates A-2 through A-11 show the subbasin delineation for the tributaries. Many models used LIDAR data of the watershed from IDNR. The effort for Newport Ditch used 2-ft contours from LCSMC data. The subbasins in Buffalo-Wheeling Creek were delineated by hand.

The total drainage area of each stream was compared to the drainage area of the stream in the main stem Des Plaines HEC-1 Phase I model. The results are listed in Table 2 below. For all new tributary modeling (Phase II), the re-delineated drainage area is within +/-15% of the drainage area used in the Phase I Des Plaines River model.

Table 2. Drainage Area Comparison.

Stream	Drainage Area (mi^2) Mainstem Model	Tributary Model	Percent Difference
Brighton Creek	18.45	20.70	-12%
Center Creek (Root River)	9.82	NA	
Kilbourn Road Ditch	24.60	23.70	4%
Newport Drainage Ditch	7.59	7.85	-3%
Mill Creek	63.27	66.37	-5%
Gurnee Trib.	11.41	NA	
Bull Creek	12.06	11.27	7%
Indian Creek	37.61	37.82	-1%
Aptakisic Creek	13.18	NA	
Buffalo-Wheeling Creek	27.17	26.82	1%
McDonald Creek	10.64	10.23	4%
Feehanville Ditch	3.59	NA	
Weller Creek	17.95	18.67	-4%
Farmers'-Prairie Creek	4.74	4.35	8%
Willow-Higgins Creek	19.11	19.67	-3%
Crystal Creek	5.22	NA	
Silver Creek	11.36	12.98	-14%

The difference in delineated drainage area could be due to the availability of improved topographic data and GIS. No changes in drainage area were made after this assessment. The hydrologic modeling for Buffalo-Wheeling Creek was incorporated into the mainstem Des Plaines River HEC-1 model. It was found that baseflow parameters had to be adjusted in the new Buffalo-Wheeling Creek modeling to maintain the calibration of the Des Plaines hydrologic model. There are no plans to integrate any other tributary modeling into the mainstem HEC-1 model.

Curve Number

Losses due to infiltration were represented with curve numbers, computed using the method described in TR-55 (NRCS 1986). The curve number method was developed to represent the effects of soil type, land use, and antecedent moisture conditions on a basin's capacity for infiltration. Land use data used in many of the tributary modeling came from the Northeastern Illinois Planning Commission (NIPC); the Buffalo Creek model used land use data from the State of Illinois. The land use in the Newport Ditch watershed was determined using local zoning maps. The soil types were obtained from NRCS. The soil moisture conditions were assumed to be AMC II. Curve numbers were adjusted for future conditions based on NIPC projections of population growth and known public works projects, such as O'Hare Airport expansion. Table 3 summarizes the range of curve number values used in the hydrologic models.

Table 3. Range of Curve Numbers in Hydrologic Models.

River Name	CN
Des Plaines River	46 – 96
Newport Drainage Ditch	72.07 – 93.58
Mill Creek	66 – 92
Bull Creek	77 – 88
Indian Creek	72 – 87
Buffalo-Wheeling Ditch	68 – 82
McDonald Creek	75.1 – 83
Weller Creek	74 – 91
Farmer/Prairie Creek	72 – 98
Willow-Higgins Creek	70 – 96
Silver Creek	76 – 92

Unit Hydrograph

The unit hydrograph is a method used to transform excess rainfall into a runoff hydrograph. Most of the modeling done for this project utilized the Clark unit hydrograph (Clark 1943), which is based on models of watershed storage. The parameters used in HEC-1 to describe the Clark unit hydrograph for each subbasin are the time of concentration, Tc, and a storage coefficient, R. Some models used the SCS dimensionless unit hydrograph (NRCS 1969), for which HEC-1 requires a lag time, tp. Table 4 below summarizes the unit hydrograph methods used in the hydrologic modeling, and it includes notes on references used to compute the unit hydrograph parameters when cited by the modelers.

Table 4. Unit Hydrograph Methods.

River Name	Unit Hydrograph		Reference
	Clark	SCS	
Des Plaines River	X	X	
Newport Drainage Ditch		X	
Mill Creek	X		Lake Co regression equations USGS WRI 82-22 (Graf et al. 1982)
Bull Creek	X		USGS Open File 96-474 (Melching and Marquardt 1997)
Indian Creek	X		USGS WRI 82-22
Buffalo-Wheeling Creek	X		USGS WRI 82-22, USGS Open File 96-474
McDonald Creek	X		USGS WRI 82-22
Weller Creek	X		USGS WRI 82-22
Farmer/Prairie Creek		X	
Willow-Higgins Creek		X	
Silver Creek	X		USGS WRI 82-22

The Willow-Higgins Creek HEC-1 model was designed with a customized SCS dimensionless unit hydrograph. The modeler used a non-standard peak rate factor that had been used in prior

studies. This necessitated pairing the model with a modified form of HEC-1 because the standard peak rate factor is a hard-coded value in that application. The Chicago District decided to run the HEC-1 input file for Willow-Higgins Creek with a standard version of HEC-1 to be consistent with the other models in the study. The issue of integration of a basin-wide hydrologic model is discussed in a separate paragraph at the end of the Hydrology section.
Base Flow

Base flow is the discharge in a stream during non-rainy periods. Base flow parameters were estimated through analysis of streamflow gage records, although not every tributary model included base flow. These stream models did account for base flow: Des Plaines River, Indian Creek, and Buffalo Creek.

These streams did not include baseflow in hydrologic modeling: Newport Ditch, Mill Creek, Bull Creek, McDonald Creek, Weller Creek, Farmer-Prairie Creek, Silver Creek, and Willow-Higgins Creek.

Routing Reaches

Hydrograph routing was performed using various methods depending on the conditions at the location in the watershed and the availability of appropriate data. Much of the channel routing in the hydrologic models developed for this study was performed using the Modified Puls method as described in the USACE Hydrologic Engineering Center's Training Document 30 (Bonner 1990). This method simulates the movement of the flood wave in a river by defining storage volume vs. elevation relationships for discrete river reaches. Other methods used for routing include Muskingum-Cunge, direct hydrograph lag, and normal depth channel routing. Channel routing was not included in the hydrologic models for Mill Creek and Newport Ditch; unsteady-state hydraulic models were developed for these streams so routing was not required in HEC-1.

Synthetic Storm Development

The precipitation depths for the various storm frequencies were obtained from Circular 172 (Huff and Angel 1989). The temporal distribution of the storms was obtained from Circular 173 (Huff 1990). Eight storm frequencies were used, with chance of occurrence in any given year of 99% (1-year), 50% (2-year), 20% (5-year), 10% (10-year), 4% (25-year), 2% (50-year), 1% (100-year) and .2% (500-year).

MODEL CALIBRATION

Calibration is a critical activity in hydrologic modeling: in order for a model to be useful the modeler must be confident that the model is representing the rainfall-runoff response appropriately. The calibration procedure involves selecting a rainfall event, entering precipitation data into HEC-1, and then comparing the computed hydrographs at one or more locations to measured discharges. Parameters such as curve number and time of concentration can be adjusted to cause the HEC-1 model to produce hydrographs with similar shape, peak flow, and total runoff volume as measured data.

The HEC-1 model for the Des Plaines River was calibrated using flow data from 16 streamflow gages throughout the watershed. Mill Creek, Buffalo Creek, McDonald Creek, and Weller Creek each had one streamflow gage in their respective watersheds that was used for calibration. The streamflow gage on Bull Creek had a period of record of only seven years; this model was calibrated using synthetic events and flow frequency analysis of gage records for a gage on McDonald Creek which had a period of record of nearly 50 years. Indian Creek modeling was calibrated both to historic events compared to an Indian Creek streamflow gage and synthetic events compared to frequency analysis of gages in similar watersheds.

These tributaries did not contain streamflow gages at the time of this study: Newport Ditch, Farmer-Prairie Creek, Willow-Higgins Creek and Silver Creek. The modeling for Willow-Higgins Creek was adjusted to match the results of a TR-20 model completed for a previous study. For the other streams, stage data was used to calibrate the HEC-1 and HEC-RAS models simultaneously.

CRITICAL DURATION ANALYSIS

The critical durations determined for the tributaries to the Des Plaines River in Illinois are summarized in Table 5 below.

Table 5. Critical Duration of Storms.

River Name	Critical Duration (hr)
Des Plaines River	240
Newport Drainage Ditch	12
Mill Creek	48
Bull Creek	24
Indian Creek	24
Buffalo-Wheeling Ditch	24
McDonald Creek	24
Weller Creek	24
Farmer/Prairie Creek	6
Willow-Higgins Creek	24
Silver Creek	24

A critical duration analysis was performed for each tributary. The analysis began with modeling the complete range of storm frequencies using various storm durations. Next the peak discharges were obtained for various locations along the tributary. The results were examined to determine whether a particular storm duration produced higher peak flows at those points. The critical duration storm was used to determine the structural damages along each tributary. It was noted that the critical duration could change based on changing watershed conditions and would need to be recalculated. It was also noted that transportation damages may not be calculated with the critical duration storm since the amount of traffic damage is a function of both peak discharge and duration of road inundation, and the duration (or "spread") of a hydrograph was not included in the critical duration analysis. Because a continuous simulation model was used for Wisconsin, critical duration analyses were not performed for those tributaries.

BASELINE AND FUTURE CONDITIONS

The Existing Conditions models were based on current land use conditions with only functional ecosystem, stormwater, or flood control projects included. The Baseline Conditions models used a base year of 2004. This year was selected based on the projected time frame for model completion. The year 2020 was selected as the Future Conditions year. At the time that the tributary modeling was initiated, 2020 was the latest year with predictions of land use and population as developed by the Northeastern Illinois Planning Commission. Future watershed hydrology was modeled by varying the loss rate function to reflect changes in population and land use. In practice this involved adjusting the curve numbers and unit hydrograph parameters.

BASIN-WIDE MODEL DISCUSSION

Except for Wisconsin, all models were developed using the same application, HEC-1. The Phase II study team developed a document, the Hydrology and Hydraulics User Manual, which listed assumptions, references, and recommendations to ensure that the models were developed in a consistent way. The Manual recommended the use of the HEC-1 application based on the desire to eventually integrate the hydrology models for tributaries into the Phase I Des Plaines River watershed model. The Buffalo-Wheeling Creek model was brought into the mainstem model. A white paper on the mainstem hydrologic and hydraulic modeling was developed in January 2008 and is included as Attachment A-1. The white paper outlined flow frequency analysis that confirmed that the Phase I hydrology model would still be appropriate for Phase II analysis of the mainstem Des Plaines. The study team considered incorporating updated hydrology into a basin-wide hydrology model but did not consider the work to be a high-priority task given the results of the white paper. Tributary models are valuable on a tributary scale because of the greater detail, and the mainstem HEC-1 is appropriate on a Des Plaines watershed scale.

HYDRAULICS

MODEL DEVELOPMENT

The computer application HEC-2 was used for the hydraulic analysis in the Phase I study. HEC-RAS, run in steady state, was the preferred method for hydraulic analysis of the tributaries. The hydraulic models for two streams, Newport Ditch and Mill Creek, used unsteady HEC-RAS but steady-state runs were also performed using the resultant peak flows to be consistent with the other tributary models in the study. The models for Mill Creek, Buffalo-Wheeling Creek, and Silver Creek used the NAVD88 vertical datum; the remainder of the hydraulic models used NGVD29. HEC-RAS schematics for the tributaries in Illinois can be found in Plates A-12 through A-21. More information on the HEC-2 modeling of the Des Plaines River can be found in USACE (1999).

The tributaries in Wisconsin were modeled using HEC-2. Historical streamflow and flood stage data from USGS gages as well as Wisconsin DNR were used to calibrate those models. More information on the hydraulic modeling for the Wisconsin tributaries can be found in SEWRPC (2003).

MODEL PARAMETERS

The hydraulic models HEC-2 and HEC-RAS require three categories of input data: physical characteristics of the stream, discharge data, and boundary conditions. The physical characteristics include the geometry of cross sections and structures, reach lengths, and surface roughness. The discharge data used in this study were peak flows computed in HEC-1 at various locations in the watershed for eight synthetic storms. Since the majority of the modeling was performed assuming steady flows and the Des Plaines watershed is in a relatively flat part of the country, most models only required one boundary condition, at the downstream end. The physical characteristics and boundary conditions are described further in the following paragraphs.

Cross Section Geometry

Cross section geometry data and reach lengths were obtained from field surveys, USGS topographic maps, LIDAR data for the watershed, and previous hydraulic studies. In many models, detailed survey data defined the channel geometry, and other topographic data sources were used to extend the cross section into the floodplain.

Channel Roughness

Channel roughness, represented by Manning's n, was generally determined using observations gathered from site visits, site photography, and aerial photography. Manning's n values from prior studies of the streams were also used. Newport Ditch, Mill Creek, Bull Creek, and Silver Creek used the method described in Cowan (1956) to assign Manning's n values. Buffalo Creek used the method described in Chow (1959). Table 6 summarizes the range of Manning's n values used in the hydraulic modeling of the Des Plaines River and its tributaries in Illinois.

Table 6. Range of Manning's n Values in Hydraulic Models.

River Name	Channel "n"	Overbank "n"
Des Plaines River	0.019 – 0.065	0.07 – 0.19
Newport Drainage Ditch	0.017 – 0.055	0.017 – 0.075
Mill Creek	0.03 – 0.10	0.04 – 0.10
Bull Creek	0.034 – 0.040	0.04 – 0.19
Indian Creek	0.01 – 0.058	0.035 – 0.10
Buffalo-Wheeling Ditch	0.035 – 0.0495	0.030 – 0.138
McDonald Creek	0.03 – 0.04	0.035 – 0.055
Weller Creek	0.035 – 0.07	0.035 – 0.15
Farmer/Prairie Creek	0.025 – 0.04	0.03 – 0.08
Willow-Higgins Creek	0.02 – 0.065	0.03 – 0.085
Silver Creek	0.034 – 0.04	0.04 – 0.19

Downstream Boundary Conditions

The downstream boundary condition for the Upper Des Plaines River was a rating curve at Hoffman Dam, at River Mile 44.451.

The downstream boundary conditions of most of the tributary models were based on modeled flood stages in the Des Plaines River. The Office of Water Resources performed coincident frequency analysis on stream gages along Weller, McDonald, and Buffalo Creeks in conjunction with gages along the Des Plaines River. It is more typical that a precipitation event will occur over only a small portion of the basin than over the whole basin. A 100-year precipitation event over a tributary, in general, would not typically produce a 100-year coincident stage on the main stem of the Des Plaines River.

Analysis of the Weller and McDonald Creek gages showed that large tributary floods occur at times when the Des Plaines River is at a 6-year recurrence interval or less. The analysis of the Buffalo Creek gage showed that large floods in that stream occur when the Des Plaines River is experiencing a 2-year or smaller storm. Based on this analysis, the downstream boundary condition for most tributaries was set to a 5-year flood stage on the Des Plaines River.
The Corps of Engineers provided the water surface elevations at tributary confluences to the responsible agency performing a hydraulic analysis. The downstream boundary conditions used in the Illinois tributary models are summarized in Table 7. The downstream boundary condition for Newport Drainage Ditch varied with discharge. At the mouth of Indian Creek, the normal depth boundary condition was used with an energy slope of 0.0002. Information about tributaries in Wisconsin can be found in SEWRPC (2003).

It should be noted that flood damages along a tributary caused by main stem tailwater are not ignored: the main stem flood damages in the 1% chance (100-year) Des Plaines tail water areas are still taken into account by the 1% chance (100-year) flood profiles on the Des Plaines River.

Table 7. Downstream Boundary Conditions.

Tributary Name	Downstream B.C.
Newport Drainage Ditch	VARIES
Mill Creek	665.79
Bull Creek	656.00
Indian Creek	N/A
Buffalo-Wheeling Ditch	637.65
McDonald Creek	635.08
Weller Creek	632.31
Farmer/Prairie Creek	629.79
Willow-Higgins Creek	625.70
Silver Creek	620.22

Other Boundary Conditions

Other boundary conditions were used in some tributary hydraulic models. The unsteady-state runs of Newport Ditch and Mill Creek required initial conditions and an upstream boundary condition. The initial conditions were defined flows, and the upstream boundary conditions were flow hydrographs. While most of the hydraulic models were limited to subcritical flow conditions, three streams (Bull Creek, Indian Creek, and Silver Creek) were run under mixed flow conditions. The mixed-flow option in HEC-RAS allows for supercritical flow conditions. Since supercritical flow is upstream-controlled, the model requires an upstream boundary condition in case supercritical flow is computed at the upstream end of the model. This boundary condition is only used when necessary; otherwise, the stage at the upstream end is computed as part of a backwater curve. Bull Creek and Silver Creek used the normal depth assumption as the upstream boundary condition, while Indian Creek used critical depth.

MODEL CALIBRATION

The hydraulic models developed for this study were calibrated against rating curves, high water marks, or stage records at gages. The procedure involved inputting peak flow data from HEC-1 simulations of historic events and then comparing the computed stages to measured data. The parameters that could be adjusted in a hydraulic model during calibration are Manning's n, effective flow limits, changing bridge data, and rating curves.

The Des Plaines River hydraulic model was calibrated using rating curves at four streamflow gages and high water marks taken at various locations. Other models that used high water marks in calibration were Newport Ditch, Farmer-Prairie Creek, and Silver Creek. The Weller Creek model was calibrated using a rating curve at a streamflow gage. Stage records at streamflow gages were used to calibrate hydraulic models of Indian Creek, Buffalo Creek, and McDonald Creek.

As noted in the Hydrology section, a few streams were ungaged, so the entire calibration was based on stage measurements. These streams are Newport Ditch, Farmer-Prairie Creek, and Silver Creek. The hydraulic models for Bull Creek and Willow-Higgins Creek did not undergo calibration due to lack of stage measurements.

HISTORICAL STORMS AND FLOODS

Severe floods have occurred in the Upper Des Plaines River basin over the past several decades resulting in millions of dollars in damages. Two major floods that occurred in 1986 and 1987 in and around the Upper Des Plaines River basin (FEMA declarations #776 and #798 respectively) together caused more than $100 million in damages to more than 10,000 residential, commercial and public structures as well as damages attributed to traffic impacts. More than 15,000 residents were evacuated during the 1986 flood alone. Over 40 river crossings and numerous roads running parallel to the Des Plaines River flooded, causing traffic delays, prolonged detouring, and physical damage to the roadways. More recently, the Des Plaines River has seen large events in May 2004, August 2007, September 2008, December 2008, and April 2013 which resulted in significant flood damages and disaster declarations.

EXISTING FLOOD CONTROL MEASURES

Six flood risk management projects within the Upper Des Plaines River watershed were authorized as a result of the Phase I Study, and include:

- Van Patton Woods Lateral Storage in Wadsworth and Russell, IL
- North Fork Mill Creek Dam Modification in Old Mill Creek, IL
- Buffalo Creek Reservoir Expansion in Buffalo Grove, IL
- Big Bend Lake Reservoir Expansion in Des Plaines, IL

- Levee 37 in Prospect Heights and Mount Prospect, IL
- Levee 50 in Des Plaines, IL

The Van Patton Woods lateral storage area, North Fork Mill Creek Dam Modification, and Buffalo Creek Reservoir expansion are on hold due to landowner considerations, therefore the 25% reduction in flood damages had not been realized. Initial designs have been prepared for Big Bend Lake Reservoir expansion and the Van Patton Woods lateral storage area and are being coordinated with the non-Federal sponsors. Levee 37 is under construction and Levee 50 is complete.

A levee for flood risk management at North Libertyville Estates was constructed as authorized under Section 205 of the Continuing Authorities Program. North Libertyville Estates is a residential subdivision located on the east bank of the Des Plaines River in southern Lake County, approximately 2 miles northeast of Libertyville, Illinois. The project included construction of 5,500 linear feet of earthen levee, 150 linear feet of steel sheetpile floodwall, realignment of an existing drainage ditch, and implementation of an interior drainage plan and a flood warning system. The levee encircles the subdivision and ties into Buckley Road on the east and west sides of the subdivision. Interior drainage is provided by pipes through the levee with flexible check valves to prevent backflow into the subdivision. Additional drainage is provided by a permanent 2,000 gpm pump station and portable pumps used on an as-needed basis. A mitigation plan is being implemented to mitigate for the loss of habitat for the levee construction.

PRINCIPLE FLOOD PROBLEMS

This Phase II study builds upon the results of the Phase I Study and considers sites located both within tributary watersheds and along the mainstem to address flood damages across the watershed.

Many damage areas reported in the Phase I Study are located at the mouth of tributaries (e.g., Farmer- Prairie Creek at mile 63.7, Aptakistic Creek at mile 75.5). However, these damages are calculated solely based on the flood stages on the mainstem Des Plaines River. In addition to damages from stages on the mainstem Des Plaines River, this Phase II Study includes estimated damages caused by flood stages along the entire length of the tributaries.

In addition to results from the Phase I Study, previous estimates of average annual flood damages (AAD) on several tributaries over the past 40 years were compiled. Average annual damage estimates were escalated using the Bureau of Labor Statistics historical Universal Consumer Price Indices (CPI-U). Sources of flood damages in these estimates include residential and non-residential structures, their contents, and traffic impacts.

Impacts to the road network were estimated based on increases in vehicle delay and distance traveled caused by flood induced detours. Simulation of flood induced detours on vehicles traveling the area transportation network were obtained through Visual Interactive System for Transport Algorithms (VISTA) Transportation modeling.

Flood hydrographs, showing modeled flood stages and durations, were created for each major roadway section susceptible to overbank flooding. Low-point elevations on the roadways, reviewed and confirmed by local transportation agencies, were used to determine the timing, duration, and depth of flooding. Roads crossing the mainstem and tributaries along with parallel roads were included in the inventory.

MODEL RESULTS

The without-project water-surface profiles for the Des Plaines River and its tributaries in Illinois can be found in Plates A-22a through A-42. Hydraulic model results for Wisconsin tributaries can be found in SEWRPC (2003).

PLAN FORMULATION

Three analyses were performed for plan formulation. First, hydrologic and hydraulic data were collected to aid in the determination of transportation damages for the baseline conditions. Then various proposed flood risk managmenet projects were evaluated. The major types of structural measures included in this analysis were reservoirs and levees. The resulting water-surface profiles were provided to Planning for further assessment. First-added and last-added economic analysis used modeling results to ensure that the system of proposed reservoirs was economically justified.

STRUCTURE DAMAGE ANALYSIS

Structural Damages were estimated using the Hydrologic Engineering Center Flood Damage Assessment (HEC-FDA) model. Structures within the 1% and 0.2% annual chance of exceedance (100-year and 500-year) floodplain of the Upper Des Plaines River and the modeled tributaries were included in the analysis. A preliminary assessment of potential structural flood damages was done for the entire watershed using GIS. Plate 11 in the Main Report, shows the existing 1% chance (100-year) floodplain in the study area. In Illinois, existing floodplains were extracted from FEMA digital flood insurance rate maps (DFIRMs) across the watershed. In Wisconsin, a detailed mapping of the floodplain was performed by Southeastern Wisconsin Regional Planning Commission (SEWRPC).

A structure inventory was compiled consisting of specific information for individual structures within the floodplain including location, use, elevation, and value. The 1% chance floodplain, FEMA hazard data (HAZUS), and block information from the 2000 Census were used to determine the number of structures located within the 1% chance floodplain by structure category. A buffer of 250 feet was added to capture any additional structures that may be impacted. Over 10,000 structures and vehicles are included in the inventory.

Structures are grouped in six categories: apartment (multi-unit residential), commercial, industrial, public (tax-exempt structures in the public ownership), residential, and automobiles. Building structure types were determined using local tax assessor category information for individual properties. First floor and low entry point elevations for all structures within the 1% chance floodplain were surveyed. Data previously collected for the Phase I Study by the Chicago District and for other local studies by IDNR and others were used where available. Surveys were conducted by MWRDGC in Cook County, IDNR in Lake County, and SEWRPC in Kenosha County for the remaining structures. For structures within the 0.2% chance floodplain but not captured by the survey an offset was applied to available Light Detection and Ranging

(LIDAR) land surface data. Further discussion of this procedure is included in Appendix E (Economic Analysis).

BASELINE TRANSPORTATION DAMAGES ANALYSIS

The purpose of this analysis was to determine road segments that cross a stream or run parallel to it and are affected by flooding in the Des Plaines River watershed. The duration and schedule of road closures were computed for eight synthetic storm events of varying frequency. The final product of the analysis was a database of flood depth and duration information that was used by VISTA, a transportation systems model. Depth-damage curves were developed from the baseline conditions and were used in the with-project damages analysis.

VISTA, or Visual Interactive System for Transport Algorithms, is a route-based traffic simulator that was developed at Northwestern University. It simulates vehicles traveling on the major roads in a region based on measured traffic levels. Traffic is affected by controls such as stop signs and signals. Closures due to flooding can also be entered as controls in the simulation. Road closures during a flood event would force the simulated vehicles to find alternate routes. The impacts of flooding on traffic can be reported by VISTA in terms of extra travel time, extra mileage, or extra emissions, for example. The results of the VISTA model are discussed in the Economics Appendix; what follows is a description of the hydrologic analysis used to compute input data for VISTA.

Overview

Geographic, hydrologic, and hydraulic data were compiled in order to compute the duration and depth of flooding on roads in the portion of the Des Plaines River watershed in Illinois. The geographic data included the VISTA transportation network, road maps, and topographic data. The hydrologic data included hydrographs generated by HEC-1 simulation of synthetic storm events. The hydraulic data included rating curves from HEC-2 or HEC-RAS modeling, rating curves from FEMA flood insurance studies FIS, and bridge cross sections. The methodology used in this analysis is described below. Plate A-43 is a graphical depiction of the depth and duration calculations.

Locate Flooded Roads

First, the VISTA network was compared to the stream alignments in GIS. A new layer was generated that consisted of the points of intersection between these two layers. The location of links in the VISTA network did not necessarily correspond to a road’s actual location. Because of this, the next step was to compare the new point layer to a road map to ensure that each point actually marked a road crossing. Points were moved to the actual location of the crossing, or deleted if the actual road location did not cross the stream.

Flooding could occur on roads that run along a stream but do not cross it. These “parallel” roads were manually added to the list of potentially flooded locations by comparing the 1% chance (100-year) floodplains defined by FEMA to the VISTA network in GIS.

Find Minimum Roadway Elevation

It was assumed that a road would be closed due to flooding once water inundated the lowest point on the top of the road. Road elevation data came from three sources: hydraulic models, FIS, and GIS. Where detailed hydraulic modeling was available, bridge cross section information was used to obtain the road elevation. If a stream was outside the range of the available hydraulic models, then the FIS was checked to see whether the road was noted on the flood profile plots. Roads are often indicated on those plots with a vertical line or "I" shape. The top of the line was used as the top of road. If the road did not appear either in a hydraulic model or FIS, then digital elevation data was used. GIS was used to estimate the road elevation for all parallel roads. The road elevations obtained from these sources by the Chicago District were provided to state and county transportation departments for their review. The agencies that reviewed the data were the Illinois Department of Transportation (IDOT), Illinois Tollway Authority, Cook County Highway Department (CCHD), Lake County Department of Transportation (LCDOT), and Wheeling Township. Those agencies provided updated road elevations where data and time allowed.

Select Appropriate Flood Hydrograph

Hydrographs were taken from HEC-1 models of the streams in the Des Plaines watershed. The hydrographs represent the discharge over time resulting from storms of various magnitudes. Eight storms were simulated, representing precipitation with 99% (1-year), 50% (2-year), 20% (5-year), 10% (10-year), 4% (25-year), 2% (50-year), 1% (100-year) and .2% (500-year) chance of recurrence (return periods). The storm duration varied by model. During the development of each model, an investigation into the critical duration was made. The critical duration is the storm duration that would produce the highest peak flows throughout the watershed. Those storm durations were maintained for this analysis. The critical duration for the main stem of the Des Plaines River is 10 days, and it was generally 24 hours for the tributaries, although it ranged from 6 to 48 hours. Notes in the HEC-1 input file and GIS layers of subbasin delineations, where available, were used to assign a hydrograph to each potentially flooded point. Some smaller tributaries in the Des Plaines model, such as Gurnee Tributary, were located in areas that were only coarsely subdivided into subbasins. The nearest hydrographs to points along that tributary seemed to greatly overestimate peak flows. For those points, the peak flows in the modeled hydrographs were compared to peak flows in the FIS. The flows in these hydrographs were multiplied by a factor that would bring the peak flow closer to the FIS value.

Select or Generate Rating Curve

Rating curves used in this analysis came from two sources: hydraulic models or FIS. If detailed hydraulic modeling existed for a location, then the curve computed by HEC-2 or HEC-RAS was used. If the stream was a smaller tributary that was not modeled hydraulically, then profiles and discharges listed in the county FIS were used. Parallel roads were assigned rating curves that corresponded to the stream cross section that came closest to intersecting the point that represented the parallel road.

Compute Flood Duration

Combining the data in each points hydrographs (flow versus time) and rating curve (stage versus flow), curves of stage versus time were constructed. Comparing the stage versus time curves for each storm to the "trigger" road elevation determined the duration and depth of flooding for each storm event. The start time and end time of the road closures were also included in the results. The flood depths and durations were computed within DSS using code written by the Chicago District. The flood depth, duration, and start time information for points in Wisconsin were provided to the Chicago District by SEWRPC. The data was reformatted and combined with the Illinois data.

Set Road Closure Schedule

The start times were adjusted to fit in with the mainstem schedule. This was done by comparing the time to peak at each tributary mouth as modeled in the mainstem HEC-1 model to the time to peak at the mouth in each tributary HEC-1 model. Then every start time in the tributary dataset would be adjusted by the difference between the peak times in the two models. The schedule shifts were determined using Excel. Attachment A-2 includes the road closure schedule used in VISTA for the eight synthetic storm events under baseline and future conditions. For the transportation analysis, the base year was set to 2010 and the future year was set to 2020.

PROJECTS CONSIDERED AND MODELING TECHNIQUES

Two main types of structural flood-damage reduction measures were included in the alternatives analysis: reservoirs and levees. The proposed reservoirs were divided up into three categories: off-line or excavated reservoirs, in-line reservoirs, and expansions of existing reservoirs.

Off-Line / Excavated Reservoirs

The concept for off-line reservoirs used in this study is similar to the concept used in the Phase I study, which in turn was similar to off-line reservoirs constructed along the North Branch Chicago River. At those sites, channel side-drop spillways direct water into the excavated offline reservoir when river elevations exceed the spillway crest. The diversion of flood waters to the facility stops when the reservoir is at capacity; after this point all flows are passed downstream. At the point when the reservoir is full and the water surface in the river is the same as that in the full reservoir, the area acts as if no reservoir were present and the surface of the reservoir becomes part of the floodplain. An excavated design also minimizes the potential for failure. While the reservoirs along the North Branch Chicago River are sized to control a large percentage of the design event volume, the potential reservoir volumes along the Des Plaines River are small compared to design flood volumes. If storage volume is to be preserved for high-damage events, water from the Des Plaines may not be diverted into the reservoir until a relatively high target flow is reached. This type of system may require a berm around the reservoir site in order to accomplish the desired response.

Off-line reservoirs were modeled as diversions in HEC-1. A relationship between river flow and flow diverted to the reservoir was developed, and the volume of diverted flow was limited to the storage capacity of the reservoir.

In-Line Reservoirs

An in-line reservoir would be placed directly on a stream or existing flow path. A berm and weir would be constructed to create additional retention time for the flow passing through the storage area in accordance with EM 1110-8-2(FR). In-line reservoirs were modeled as routing reaches in HEC-1, and a storage-discharge relationship was developed or adjusted as needed based on the configuration of the berm.

Expansions of Existing Reservoirs

Several existing reservoirs were investigated to see whether increasing storage volume would provide any reduction in flood damages. The expansions were based on the availability of adjacent open space. If open space was available, then the reservoir was expanded using the existing depth. If no open space was available, then the reservoir was excavated further. The reservoir expansions were assessed assuming that the existing inlet and outlet structures would be maintained.

Reservoir expansions were modeled in HEC-1 by adjusting the elevation-storage-outflow relationships that represented the existing structures in the model.

Levees

Levees or floodwalls remove areas from the floodplain by holding back floodwaters. The proposed levee locations were set based on the baseline damage analysis and the regulatory floodplain extents.

Levees were modeled in the hydraulic models by setting encroachment elevations in HEC-2 or by setting levee elevations in HEC-RAS. A levee height equal to 2 ft above the 1% chance (100-year) flood profile was used. This was based on Risk & Uncertainty analysis that indicated the two foot increase would provide the 95% certainty needed for FEMA certification. Because the presence of levees changes the available floodplain area, new channel routing reaches were computed and entered into the hydrologic model as well.

Greenway

A modified riparian greenway was investigated to evaluate whether clearing snags and other vegetation would improve conveyance. This was modeled by decreasing channel roughness in HEC-2.

Bridge Modification / Road Raise

Bridge modifications and road raises raise the height of a roadway to reduce damages due to road closures. The proposed bridge modification and road raise locations were set based on the baseline transportation damages analysis. Proposed bridge modification project DPBM01 was modeled by the Illinois Department of Natural Resources (IDNR) as described in the October 2009 Groveland Avenue Limited Strategic Study, which is attached to Appendix B. The analysis used a combination of HEC-RAS and HEC-2 to assess changes in bridge piers to provide smoother flow transitions through the structure. Bridge modifications/road raises DPBM04 (First Avenue Bridge Modification and DPBM13 (Route 120, Belvidere Road Bridge Modification) were also modeled with HEC-2 and checked with HEC-RAS.

Other Measures: Channel Improvements

Other site-specific measures were considered that did not fit into any other category. Other drainage improvements that were considered involved increasing the cross-sectional area at a culvert or in a reach of a stream. These were modeled by changing the geometry in HEC-RAS. The other measures are described in more detail in the Results and Discussion section of this report.

Table 8 is a summary of the projects considered and modeling techniques used.

Table 8. Summary of Project Categories.

Projects Considered	Modeling Techniques
Off-line reservoir	Diversion in HEC-1
In-line reservoir	Routing in HEC-1
Expansion of reservoir	Reservoir routing in HEC-1
Levees	Encroachment (HEC-2) or levee (HEC-RAS), adjust channel routing in HEC-1 as needed
Greenway	Decrease channel roughness in HEC-2
Bridge modiication / road raise	Model changes in structure in HEC-RAS (DPBM01) or none (all others)
Channel improvements	Increase cross-sectional area in HEC-RAS, adjust channel routing in HEC-1 as needed

EVALUATION OF ALTERNATIVES

This section summarizes the procedure developed to evaluate flood-damage reduction measures in the Illinois portion of the Upper Des Plaines River watershed. A more detailed discussion of the plan formulation procedure can be found in Volume 2 of this report.
For the most part, the study team decided to assess the Wisconsin portion of the Des Plaines River watershed separately from the Illinois portion. Analysis described in SEWRPC (2003) and in Attachment A-3 (SEWRPC, March 2009) indicated that floodwater storage would not be an effective mitigation measure to affect flood problems in Wisconsin. Additionally, any decreases in peak flows or stages that would travel to Illinois would be small. This was attributed to the flow-attenuating affect of a large floodplain wetland complex located along the eight-mile reach of the river extending upstream from the Illinois-Wisconsin state line. In light of these factors, the study team decided to focus the assessment of structural measures in Illinois.
The first step in the evaluation of structural measures in Illinois was site identification, which assessed whether site conditions were compatible with proposed measures. The second step was site screening, which used hydrologic and hydraulic modeling to examine whether proposed sites would provide reduction in flood damages. The third step was site evaluation, which refined the modeling of the most promising sites to improve the estimate of flood risk management.

Site Identification

The site identification step used four criteria to evaluate sites. They were field verification, existing compatibility, neighboring compatibility, and environmental compatibility. These criteria are described in detail in Section 4 of the Main Report. No technical analyses were performed during this step. At the beginning of this step, 200 potential storage sites and 23 potential levees were identified. The site identification process eliminated 130 reservoir sites.

Site Screening

The site screening step was a preliminary assessment of project feasibility for the sites that made it through the initial identification process. The modeling techniques used in this step were developed to provide a relatively rapid assessment of a project's impact on flood flows and stages. Projects located along tributaries to the Des Plaines River were modeled with the tributary models and with the mainstem models in order to see whether the projects impacted flood stages on the Des Plaines River. Once the with-project water surface profiles were generated, they were provided to Planning for further analysis using HEC-FDA.

Site screening of levee projects is described in Section 2.2 of Appendix B. The baseline water-surface profiles were combined with the levee locations within HEC-FDA to determine the areas removed from the floodplain for varying levels of protection. No additional hydraulic modeling was used in this step.

Reservoirs were modeled in the hydrologic model for a stream. Storage capacities were estimated based on the area of each site. For new reservoirs, a relationship between site area and storage volume was developed based on CDM (2004), which studied potential storage sites along Buffalo Creek. In that study, it was assumed that all excavated material would be stored on-site and that the reservoirs would be 10 ft deep with 4H:1V side slopes. Multiplying the site area, in acres, by 4.41 provided a volume, in ac-ft, that was in line with the proposed projects in that study. The concept of the diversion relationship was to "cut-off" the peak of the hydrograph and to divert it into the potential reservoir. Essentially, a volume of water equal to the storage capacity would be removed at the time of highest flows during the storm events. Three design storms were used: the 4% (25-), 2% (50-), and 1% (100-year) chance storm events.

No in-line reservoirs were modeled during the site screening step.

For reservoir expansions such as SCME01 and SCME02, storage was increased by expanding the size of the reservoir in an available direction. The existing depth was maintained in the expansion when adjacent open space was available. If no open space was available then the reservoir was made deeper. The inlet and outlet structures remained unchanged. New elevation-storage-discharge relationships were computed for the expanded reservoirs and were incorporated into HEC-1 for the with-project conditions.

Proposed greenway project DPOT02 was modeled by decreasing Manning's n by 0.001 between River Miles 46.01 and 73.095. The technique was based on analysis described in a June 2002 memorandum titled "Des Plaines Levee 37, Hydraulic Analysis for Tree Trimming to Mitigate for Project Induced Stage Increases Beyond State Regulatory Limits." The memorandum is included in Attachment A-4.

Proposed greenway projects DPOT02-A and DPOT02-B were modeled by decreasing Manning's n in the overbank from 0.19 to 0.08, and in the channel by an increment of 0.001. These reductions were made in areas both owned by the Forest Preserve and inundated by the 50% chance (2-year) storm. The technique is described in an August 2010 memorandum titled, "Des Plaines Levee 37, Hydraulic Analysis for Tree Trimming to Mitigate for Project Induced Stage Increases Beyond State Regulatory Limits," which is also included in Attachment A-4. Proposed channel improvement project SCCI01 was modeled by increasing the cross-sectional area of culverts in the hydraulic model. Proposed project BWCI01 was modeled as a diversion near the confluence of Buffalo Creek and the Des Plaines River as a screening-level estimate of performance because the site had been modeled as an offsite reservoir previously during development of the Feasibility Scoping Meeting materials (USACE 2007).

Proposed bridge modification project DPBM01 was modeled by the Illinois Department of Natural Resources (IDNR) as described in the October 2009 Groveland Avenue Limited Strategic Study, which is attached to Appendix B. This project extends each of the bridge piers upstream and downstream to reduce their effective width and provide smoother flow transitions.

Structure modification project FPCI01 was modeled by the Illinois Department of Natural Resources (IDNR) as described in the September 2009 Farmers/Prairie Creek Strategic Planning Study, which is attached to Appendix B. This project would lower flood stages on Lake Mary Anne by adding a 10cfs pump station and routing the discharge under Golf Road to Dude Ranch Pond.

Site Evaluation

The results of the site screening were used in HEC-FDA to determine preliminary benefit-cost ratios for the proposed projects. Projects with a preliminary BCR greater than 1 were carried through to the next step, site evaluation. In this step, a feasibility-level design was developed for each site, and more detailed hydraulic modeling could be implemented. The resulting water-surface profiles were provided to Planning in order to refine the flood risk management computations. During this step, 9 reservoirs and 6 levees were evaluated.

Four of the 9 reservoirs were eliminated before modeling took place, for reasons summarized in Table 12. Another four of the 9 reservoirs were modeled as new, off-line reservoirs. After the initial run of off-line reservoir sites, an additional step was taken to refine the diversion relationship based on the actual reservoir characteristics and inlet structure with a focus on reducing peak flows for the design storm that provided the greatest reduction in flood damages. Each reservoir was designed to receive the diverted flow by means of a lateral overflow weir running parallel to the bank of the river. Once the river stage exceeds the elevation of the weir, water flows over the weir and into the reservoir. First, a representative cross section near the reservoir diversion was chosen, and the corresponding HEC-2 or HEC-RAS rating curve from the without project conditions was used to determine the initial weir elevation. The site topography was examined to determine if the weir elevation was feasible for the site. Based on this elevation, a preliminary weir length was chosen by matching the preliminary diversion relationship (used in the site screening) with the diversion relationship determined from the use of the cross section rating curve and the equation for a broad-crested weir. This new diversion relationship was then inserted into the HEC-1 model and was run iteratively to optimize the weir configuration (elevation and length) and corresponding diversion. After finding a configuration optimizing peak flow reduction and reservoir storage capacity, the diversion relationship was inserted into the HEC-1 model and run as in the first evaluation to obtain the HEC-FDA input files for the proposed project.

One reservoir, BCRS02 in the Bull Creek watershed, was modeled as an inline structure. This concept was chosen for this site because of its location. It was upstream of a defined channel and the topography suggested multiple drainage paths that converged at a culvert. It was modeled as a routing reach in HEC-1. A berm and weir would be constructed to create additional retention time for the flow passing through the storage area and to decrease the outflow to the culvert. This site was modeled as a routing reach in HEC-1, and a storage-discharge relationship was developed or adjusted as needed based on the configuration of the berm.

Preliminary estimates of compensatory storage and interior drainage requirements were initiated for the levees that passed site screening and are described in the Additional Levee Considerations section of this report.

No additional modeling was done for reservoir expansions during this step.

Results and Discussion

Site identification

No technical analyses were performed during this step. Detailed descriptions of the results of this step can be found in Section 4 of the Main Report.

Site screening

Water-surface profiles of the with-project conditions were provided to Planning. The hydraulic modeling results were used along with damage and cost information to compute a benefit-cost ratio for each proposed project. Detailed descriptions of the results of this step can be found in the Volume 2 of this report. The sites that passed the site screening and moved on to the site evaluation step are listed in Table 9, Table 10, and Table 11.

Table 9. Reservoirs that Passed Site screening.

Site	Stream	Volume (ac-ft)	Drainage Area (mi^2)
ACRS03	Aptakistic Creek	248	3.90
ACRS08	Aptakistic Creek	418	3.90
BCRS02	Bull Creek	243	5.25
BWRS31	Buffalo-Wheeling Creek	383	2.51
DPRS07	Des Plaines River	1,000	438.48
DPRS23	Des Plaines River	330	357.57
FDRS01	Feehanville Ditch	2,000	373.29
FDRS03	Feehanville Ditch	24	2.17
WHRS06	Willow-Higgins Creek	586	7.02

Table 10. Levees that Passed Site screening.

Site	Stream	Length (ft)
DPLV01	Des Plaines	2,098
DPLV07	Des Plaines	1,722
DPLV09	Des Plaines	4,018
DPLV15	Des Plaines	1,793
SCLV02	Silver Creek	6,007
SCLV03	Silver Creek	4,901

Table 11. Modification of Existing Structures that Passed Site Evaluation

Project	Stream	Description
DPBM01	Des Plaines	Realign bridge piers to run parallel with flow
DPOT02-A	Des Plaines	Replace trees with greenway from RM 50.46 – 51.62
DPOT02-B	Des Plaines	Replace trees with greenway from RM 53.83 – 55.35
DPOT02	Des Plaines	Replace trees with greenway along 30-mile stretch of river
FPCI01	Farmer-Prairie Creek	Modify pump and provide a connection at Lake Mary Anne to Dude Ranch Pond
SCCI01	Silver Creek	Expand culvert at 31st St.
SCME01	Silver Creek	Expand structure 106
SCME02	Silver Creek	Expand structure 102

Site Evaluation

When initiating this step, some of the proposed sites that appeared to provide flood-damage reduction benefits due to their location in the Des Plaines River watershed were found to be infeasible. Four proposed reservoirs were eliminated based on factors other than modeling results. The sites that were eliminated are summarized in Table 12.

Table 12. Reservoirs Eliminated in Site Evaluation.

Site	Reason for Elimination
ACRS03	20-ft hill adjacent to stream, lower ground >650 ft away
BWRS31	Half of site now developed, other half has 15-20 ft hills
DPRS07	Eliminated Due to Poor Soil Conditions
FDRS01	Eliminated due to BCR<1
FDRS03	Small volume of 24 ac-ft
WHRS06	Eliminated Due to Detailed Analysis of H&H Profile Corrected H&H demonstrated BCR<1

Site ACRS03 was eliminated because constructing an excavated reservoir on very high ground would require excessive excavation and the resulting spoil may have to be hauled off-site, increasing costs. Site BWRS31 was eliminated due to unavailability of real estate and difficult terrain. Site FDRS03 was eliminated at this stage because of the small storage volume.

The reservoirs that were modeled during the site evaluation are summarized in Table 13. Preliminary designs for each proposed project can be found in the Civil Appendix, Appendix D.

Volume-storage curves for each of the proposed new reservoirs listed in **Table 13**, as well as for the reservoir expansion SCME02, can be found in these plates: Plate A-44 (BCRS02), Plate A-45 (ACRS08), Plate A-46 (DPRS23), and Plate A-47 (SCME02).

Table 13. Reservoirs Modeled in Site Evaluation and Considered for Further Evaluation.

Site	Design Storm (year)	Notes
ACRS08	100	Pump to and from site (far away and uphill)
BCRS02	-	Modeled as inline storage
DPRS23	25	Not justified individually, only as comp. storage for DPLV07
DPRS15	100	Compensatory storage for DPLV09

While the initial concept for the reservoirs was a channel side-drop spillway, this was only used to model DPRS23. Conditions at the two other sites suggested different design concepts. Site ACRS08 was more than 250 ft away from Aptakisic Creek and located on high ground. A pump station would convey water to and from the site. Site BCRS02 in the Bull Creek watershed was modeled as an inline structure. This concept was chosen for this site because it was upstream of a defined channel and the topography suggested multiple drainage paths that converged at a culvert. A berm and weir would be constructed to create additional retention time for the flow passing through the storage area and to decrease the outflow to the culvert.

Table 14. Levees Modeled in Site Evaluation and Considered for Further Evaluation.

Levee	Design Storm (year)	Notes
DPLV01	45	Modeled by IDNR
DPLV07	500	Paired with DPRS23 for compensatory storage
DPLV09	100	Created DPRS15 for compensatory storage
DPLV15	10	No compensatory storage necessary
SCLV02	25	Levee on both banks. Paired with SCME02 for comp. storage.
SCLV03	10	Levee on both banks. Paired with SCME02 for comp. storage.

The design storms for the levees were determined through economic analysis as described in Section 2.2 of Appendix B.

No additional modeling was done for DPOT02 or BWCI01 after site screening. To refine the modeling of SCCI01, a routing reach was adjusted in the hydrologic model to account for the change in the hydraulics at the project area. The resulting adjusted flows were input to HECRAS to obtain revised flood profiles.

ADDITIONAL LEVEE CONSIDERATIONS

Compensatory Storage

Because levees remove areas from the floodplain, they change the area available for active conveyance and hydrograph routing, which often induce backwater effects that increase flood stages upstream. Compensatory storage requirements were investigated for the 5 levees that passed site screening. The goal was to provide a rough estimate of the size of a reservoir that could be paired with a levee to offset any adverse impacts. To quickly provide order-of magnitude values, the volume of floodplain that would be cut off by the levee was estimated. Hydraulic model results were used to compute the volume of water in the proposed levee reaches for different flood events, and these volumes were compared to the baseline conditions. The change in floodplain volume as computed by the hydraulic modeling is shown in Table 15.

Table 15. Change in Reach Volume during Levee Model Runs.

Levee	Bank	Floodplain Change (ac-ft)				
		10-yr	25-yr	50-yr	100-yr	500-yr
DPLV07	R	0	-5	-13	-28	-44
DPLV15	L	0	-4	-15	-26	-36
SCLV02	R+L	-20	-51	-141	-173	-192
SCLV03	R+L	-2	-17	-90	-143	-186
DPLV04	R	-1	-11	-7	-16	-44
DPLV05	R	-3	-15	-40	-57	-165
DPLV09	R	-2	-21	-52	-64	-180

A more refined estimate of compensatory storage was initiated, and it was completed for four of the seven levees, DPLV07, DPLV04, DPLV05, and DPLV09 (and also the combined DPLV04, DPLV05, and DPLV09 Levee). The channel routing was updated in the HEC-1 models to account for the reduction in flow area caused by each levee. Constant flows were run through the HEC-2 model to determine the modified volume-storage relationship of the modeled cross sections as described in Bonner (1990). These adjusted relationships were then incorporated into the HEC-1 model, and the modeling process repeated.

Compensatory storage sites would be limited to those reservoirs being considered for construction on their own merit. The levees DPLV07 and DPLV09 were paired with proposed reservoirs in an attempt to eliminate significant upstream stage increases caused by the levee. The reservoirs were designed independently of the levees, so the levee-reservoir pairings had varying levels of success. It was found that DPRS23 could be configured to eliminate upstream impacts of some events, but more analysis will be completed in a later stage of this project to finalize the levee-reservoir pairings. Since the volume of DPRS23 is 330 ac-ft, this suggests that the storage estimates listed in Table 15 should be multiplied by a factor of 2 or 3 to obtain a better estimate of required storage.

The mitigation reservoirs were investigated to find stage mitigation that would satisfy Illinois Department of Natural Resources – Office of Water Management (IDNR-OWR), construction in a floodway permit requirements. No stage impacts are allowed for the one percent chance exceedence flood event (100 year recurrence interval), and all the flood events more frequent than the one percent event. IDNR-OWR interprets no stage impacts as 0.0 ft. (0.044 ft stage increase and less is rounded to zero as a courtesy). Also stage impacts that do not impact structures can be mitigated with flowage easements. The baseline condition models were used to determine stage differences for permitting for these analyses.

Interior Drainage

While a levee protects an area from flooding due to a rising river, the structure can block natural drainage paths and cause the backup of water on the landward side of the levee. The purpose of interior drainage analysis is to identify the nature of interior flooding and to formulate alternatives to reduce it. Measures such as pumping stations, gravity outlets, or interior storage areas could be developed to address this issue. The interior drainage calculations are included as Attachment A-5.

An interior drainage analysis for levees was performed using HEC-RAS version 4.1.0 and HEC-HMS version 3.4. HEC-HMS was used to develop an interior hydrograph for a period of record between 1969 and 2009 using hourly rainfall from the rainfall gage in McHenry, Illinois. The drainage basin delineation was assumed to follow existing topography. Land use and NRCS soils data were used to develop the basin model inputs. A Clark unit hydrograph was used along with the Green-Ampt loss method. Table 16 summarizes the interior basin model inputs.

Table 16. DPLV15 Interior Basin Model Inputs.

Parameter	Value
Drainage Area	0.83 mi2
Time of concentration, Tc	1.1 hr
Storage coefficient, R	1.7 hr
Initial loss	0.40 in
Moisture deficit	0.20
Suction	18.3 in
Conductivity	0.2 in/hr
Percent Impervious	20%

The interior hydrographs were developed using a period of record analysis, as well as the 1% chance (100-year) synthetic storm. The exterior stages were developed using flow data from the Gurnee gage and the unsteady HEC-RAS model of a short reach of the Des Plaines River. The unsteady HEC-RAS results were compared to stage measurements at Gurnee to verify the model.

The interior drainage analysis showed that interior drainage features would be required for levees. Interior drainage features would include both gravity drainage and a pump station for the minimum required facility.

Conditional Non-Exceedance Probability

Overtopping is not a risk for greenway projects or bridge modifications. The proposed reservoirs are excavated, which minimizes the potential for failure and causes the site to simply revert to the without-project conditions when the storage capacity is reached. Overtopping of levees, however, can lead to significant landside erosion of the levee or even be the mechanism for complete breach. Table 17 below summarizes the non-exceedance probabilities for each of the proposed levees. According to Engineering Manual EM 1110-2-1619, a levee can be said to provide a certain level of protection if the non-exceedance probability is greater than 95%.

Table 17. Conditional Non-Exceedance Probability Levees.

Levee	Description	Non-Exceedance Probablility at Event:				
		10-yr	25-yr	50-yr	100-yr	500-yr
DPLV01	Tie back existing Riverside levee	1.00	1.00	0.99	0.99	0.99
DPLV04	6,400 ft floodwall and levee	1.00	1.00	1.00	1.00	0.99
DPLV05	6,000 ft floodwall and levee	1.00	1.00	1.00	1.00	0.99
DPLV09	11,000 ft floodwall and levee	1.00	1.00	1.00	1.00	0.99

FIRST / LAST ADDED ANALYSIS OF DETENTION COMPONENTS

A first-added analysis and a series of last added analyses were performed in an effort to determine the performance of the potential detention components individually and together. Once the final array of potential NED detention alternatives was considered a first-added analysis was performed to justify each component individually. The HEC-FDA runs completed during the primary design analysis were used for this purpose. For each run, one NED detention component was tested and the resulting residual damages were compared against those computed by HEC-FDA for the without-project conditions. This shows the effects that a particular NED component would have if it were to be the first component incorporated. The difference between the residual damages of the baseline conditions and the residual damages of the with-project conditions is equal to the benefit of the proposed project. The potential projects were ranked by net benefits.

Once the first fixed component was determined, the first round of the last-added analysis could begin. The remaining potential reservoirs were added individually to the NED plan, consisting only of DPBM04 for the first round. That is, Site ACRS08 was added to DPBM04 and the hydrologic, hydraulic, and economic models were run to determine the reduction in damages resulting from the combination of the two sites. This process was repeated for the two other proposed reservoirs, DPRS15 and BCRS02. Net benefits were computed for each of the three pairs of projects in Round 1, and the pairs were ranked from highest to lowest net benefits. The pair of projects with the highest net benefits in Round 1 was then used to begin Round 2 of the last-added analysis. Because there were only three reservoirs, only three rounds of last-added analysis were necessary. The results of the last-added analysis are shown in Table 18.

The first added analysis and round 1 of last added analysis included the project DPBM04, the modification of the bridge at First Avenue, as shown in Plate 24 of the Main Report. Round 2 of last added analysis showed that the proposed project DPBM04 paired with the combined DPLV04, DPLV05, DPLV09 levee and ACRS08 produced the highest net benefits. This became the fixed component of last added Round 3. The remaining potential projects were added individually. This process was repeated for the additional project: DPLV01. Each round, the highest ranking project would be selected to serve as the base for the subsequent round. After Round 3, projects evaluated for economic benefits produced no hydraulic impacts, so the last added Round 3 hydraulics were used. The results of the last added analysis are shown in Table 18.

Table 18. First Added and Last Added Analysis Results Summary.

Round	Project with Rank = #1
Last Added 1	DPBM04
Last Added 2	DPBM04 + DPLV04 + DPLV05 + DPLV09 + ACRS08
Last Added 3	DPBM04 + DPLV04 + DPLV05 + DPLV09 + ACRS08 + DPLV01

All projects included in the last added analysis provided incremental benefits to the system as they were added. A full description of the economic component of the first-and last added analyses, and more detailed results, can be found in Volume 2 of this report.

RISK AND UNCERTAINTY

A risk analysis was performed for this study using HEC-FDA. This program incorporates a Monte Carlo simulation to sample the interaction among the various hydrologic, hydraulic, and economic uncertainties. Uncertainties in the hydrology and hydraulics include the uncertainties associated with the discharge-frequency curve and the stage-discharge curve. Both of these relationships have statistical confidence bands that define the uncertainty of the relationships at various target frequencies. The Monte Carlo simulation routine randomly samples within these confidence bands over a range of frequencies until a representative sample is developed. Reliability statistics are based on the results of the Monte Carlo random sampling.

The uncertainty in the discharge probability was determined using the graphical method in HEC-FDA. The uncertainty in flood stages was based on the uncertainty in Manning's n. Manning's n was varied by +15% and -15% in the hydraulic models. To capture the maximum error at each particular cross section, the maximum value was selected to represent the stage error for that cross section. A single stage error value was then generated by taking the average stage errors over all cross sections for each particular model. A detailed discussion of the risk and reliability analyses can be found in the Economics Appendix.

DISCUSSION OF FLOOD DAMAGE ANALYSIS

Flood damages were calculated through the HEC-FDA program, and a detailed discussion can be found in Appendix E. In a watershed characterized by relatively flat terrain, slight changes in stage potentially result in dramatic reductions in damages. That was the case in this study. The stage reductions for more frequent events (1-year through 25-year return periods) provided the highest damage reduction because the storms occur more often: approximately 72% of the damage reduction was claimed in the smaller-magnitude events. Larger storms (50-year through 500-year return periods) incur higher damages, but because they occur less frequently they have a lower weight in the expected valuation calculation.

SYNTHETIC EVENT MODELING OF THE NED PLAN

Plates A-48 through A-55 show the with-project conditions on the Des Plaines River and Silver Creek. The plates include both water-surface profiles and the change in flood stage from the without-project conditions.

DESCRIPTION OF SELECTED RESERVOIRS

The reservoirs in the NED plan are shown on the Reservoirs Location Map, Plate D-0 in Appendix D. Section 1 of Appendix D includes several paragraphs on the design for each proposed reservoir describing the location, top and bottom area, volumetric storage capacity, weir elevation, and culverts. All proposed reservoirs are lateral storage areas except for BCRS02, in the Bull Creek watershed, which was modeled as an inline structure.

Bull Creek Reservoir BCRS02

The Bull Creek Reservoir site is located in the city of Mundelein in Lake County, Illinois. The proposed reservoir is designed for a 4% chance (25-year) to 1% chance (100-year) flood event and has a storage volume of 176.7 ac-ft. The top area of the reservoir is 53 ac, and the bottom area is 47.5 ac. The reservoir is 4 ft deep from the toe of the levee with an approximate bottom elevation of 754 ft (without topsoil and seeding). Storm water flows into the reservoir through a proposed concrete reinforced pipe with inlet elevation of 758.5 ft. Water flows out of the reservoir through a proposed concrete weir. The weir is 10 ft wide with a top elevation of 758.0 ft. This reservoir will meet EM 1110-8-2(FR) design criteria and will be classified as a Standard 3 dam.

Aptakisic Creek Reservoir ACRS08

Site ACRS08 is located in Lake County just north of Aptakisic Creek, a tributary of the Des Plaines River. The proposed reservoir is designed for a 4% chance (25-year) to 1% chance (100-year) flood event and has a storage volume of 405.8 ac-ft. The top area of the reservoir is 42 ac, and the bottom area is 40.2 ac. The reservoir is 4.5 ft deep from the toe of the levee with bottom elevations varying between 676 ft to 682 ft. Storm water is pumped into the reservoir from Aptakisic Creek with an inflow rate of 200 cfs. The pumping duration to fill the reservoir is assumed to be 2 days. After an event, the reservoir will be drained through the proposed sluice gate and by pumping. A sluice gate is located at the existing ground level with an approximate invert elevation of
678.5 ft.

Aptakisic Creek Reservoir ACRS08 (expanded to 550 Acre-feet for stage mitigation)

The originally economically justified Reservoir ACRS08 was expanded to provide stage mitigation for the combined DPLV04, DPLV05, and DPLV09 levee for compliance with IDNR-OWR construction in a floodway permit requirements. For more information see attachment A-6, "Levee 4 and Levee 5 Analysis Memorandum". The proposed reservoir is designed for a 25-year to 100-yr flood event and has a storage volume of 550 AC-FT. The top area of the reservoir is 45.4 AC, and bottom area is 40 AC. The reservoir is 2 feet deep from the toe of the levee with bottom elevations varying from 678 to 684. The bottom elevation does not consider the reservoir seeding over 6 inches of topsoil. Storm water is pumped into the reservoir from Aptakisic Creek with an inflow rate of 200 cfs. Approximately 870 LF of 60-inch reinforced concrete pipe (RCP) will collect storm water from the creek through the pump station and outlet through two 36-inch RCP to the reservoir. Two, 100 CFS pumps are assumed with an additional pump and 36-inch discharge for redundancy. The pumping duration to fill the reservoir is assumed to be 2 days. After an event, the reservoir will be drained through the proposed sluice gate and by pumping. A sluice gate is located at the existing ground level with an approximate invert elevation of 678.5. The sluice gate will allow above grade water to drain into the existing ditch. Remaining water below grade in the reservoir will be pumped out through the pump station and into the ditch. One of the 36-inch RCP pipe and pump can be used to drain the reservoir.

DESCRIPTION OF SELECTED LEVEES

<u>Des Plaines Levee DPLV01</u>

Plate D-19 in Appendix D shows the location and alignment of DPLV01. Site DPLV01 is a levee along the north side of Park Pl that is approximately 365 ft long and a floodwall along the west side of West Ave that is approximately 693 ft long. The crest of the levee and the top of floodwall are at elevation 616.0 ft. No temporary easements were laid out for this site. The whole area is heavily developed, but there is a school with a large athletic field just east of the site that could be used. A survey of the existing levee crest along Groveland Ave should be performed in the next phase of this project to ensure it maintains an elevation of 616.0 ft for the entire length. One road closure structure will be necessary at Forest Ave to complete the line of flood protection.

<u>Des Plaines Levee DPLV09 and Reservoir DPRS15</u>

Plates D-19A thru D-19C in Appendix D shows the location and alignment of DPLV09. Site DPLV09 is a levee/floodwall combination from Ashland Avenue to Fargo Avenue running along the west side of the Des Plaines River. The levee has a total length of 1,900 ft with a crest width of 10 ft. The floodwall has a total length of 9,100 ft. The crest of the levee and the top of the floodwall is stair stepped and includes 2 feet above the 1% chance flood elevation, which gives a 95% chance that a 1% chance (100-year) flood would not overtop the levee. The stage mitigation reservoir DPRS15 is shown on plates D-02 and D-03 and has a flood storage capacity of 205 acre-ft. DPLV09 is located between river miles 62.88 and 64.60 in the Mainstem Des Plaines River HEC-2 model.

Note that while Reservoir DPRS15 provided adequate compensatory storage for DPLV09 as a stand alone levee, it did not provide adequate storage for the final plan that included the combination levee DPLV09, DPLV04 and DPLV05.

<u>Des Plaines Levee DPLV05</u>

DPLV05 is a 6,000 foot levee and floodwall along the west bank of the Des Plaines River in Schiller Park. The structure will protect homes and businesses along the mainstem Des Plaines River from Belmont to Irving Park Road. The crest elevation is two feet above the 1% annual chance of exceedance flood elevation. The probability that this levee will not be overtopped during the 1% annual chance of exceedance flood event will be greater than 95%. DPLV05 is located between river miles 55.99 and 56.93 in the Mainstem Des Plaines River HEC-2 model.

Des Plaines Levee DPLV04

DPLV04 is a 6,400 foot levee and floodwall along the west bank of the Des Plaines River in River Grove. The structure will protect homes and businesses along the mainstem Des Plaines River from the Palmer Street and Fifth Avenue along Fifth Avenue and River Road to the Canadian North Railroad. The crest elevation is two feet above the 1% annual chance of exceedance flood elevation. The probability that this levee will not be overtopped during the 1% annual chance of exceedance flood event will be greater than 95%. DPLV04 is located between river miles 54.29 and 55.27 in the Mainstem Des Plaines River HEC-2 model.

DESCRIPTION OF OTHER SELECTED PROJECTS

DPBM04 - 1st Avenue Bridge Raise

Plate D-24 presents the 1st Avenue Bridge raise. The bridge was raised to 631.84 along with the approach ramps and connecting roadways. This puts the roads above the 1% annual chance of exceedance flood elevation. The site will be designed to prevent adverse impacts to surrounding structures by extending the bridge length, providing greater conveyance capacity under the roadway.

OPERATIONS AND MAINTENANCE ESTIMATES

Operations and maintenance requirements were identified for each of the proposed projects for cost estimation purposes only. Design-level O&M plans have not been developed during the Feasibility Study phase. In general, O&M requirements for reservoirs will include: Inspection, Mowing, Fill/Repair, Debris Removal, and Tree and Brush Trimming. In general, O&M requirements for pump stations will include: Semi-Annual Reporting, Inspection, quarterly Oil & Grease, Trash Rack Equipment Maintenance, Electrical Consumption, and Mechanical Reconditioning, Rehabilitation and Replacement. Gates will require Inspection, Cleaning/Lube, Debris Removal, and Repair/Replacement. A summary of O&M costs for each of the evaluated projects is shown in Appendix B.

OFFSITE IMPACTS DISCUSSION

In general, levee projects are accompanied by the risk of upstream and downstream stage increases. Bridge pier modifications can also potentially hold back water and affect stages upstream. The greenway projects all increase conveyance by reducing roughness in the channel and overbank, which decreases hydrograph attenuation and can potentially increase stages downstream. These risks were considered in the NED plan formulation process. In accordance with Illinois State Law, which requires that construction will not reduce floodway conveyance or storage, and will not increase velocities and flood heights, NED projects were designed to cause stage increases no greater than 0.04 ft.

SUMMARY

Updated hydrologic and hydraulic models were developed for the major tributaries to the Upper Des Plaines River. The modeling was used to develop relationships between flood depth and transportation damages for major roads that could be impacted by flooding in the watershed. Potential structural flood-damage reduction measures such as reservoirs and levees were identified and screened. Preliminary designs were developed for proposed projects that passed the screening phase. The resulting water-surface profiles were provided to Planning for further assessment. Preliminary estimates of compensatory storage and interior drainage requirements for the levees were developed. First-added and last-added analyses provided economic justification for the system of detention components.

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