US Army Corps
of Engineers
Kansas City District

MANHATTAN LOCAL PROTECTION PROJECT
FEASIBILITY STUDY
MANHATTAN, KANSAS
(Manhattan Levee)

(Section 216 Review of Completed Civil Works)

Engineering Appendix

August 2014 Final Feasibility Report

VOLUME 1 EXISTING CONDITIONS
ENGINEERING ANALYSIS

VOLUME 1 TABLE OF CONTENTS

PART 1 – INTRODUCTION
1.1 Engineering Appendix Purpose and Context 1-1
1.2 Project Authority.....1-1
1.3 Feasibility Study Overview.....1-1
1.4 Levee Unit Location and General Description.....1-2
1.5 Engineering Documentation.....1-3
1.6 Existing Conditions Analysis 1-3
1.7 Future Without Project Conditions Analysis 1-4
1.8 Future Without Project Conditions
– Project Alternatives Overview 1-5
1.9 Plan Evaluation, Comparison and Selection 1-7
1.10 Project Implementation 1-7

PART 2 – HYDROLOGY AND HYDRAULICS
2.1 Site Description.....2-1
2.2 Site Background.....2-7
2.3 Hydrology.....2-14
2.4 Hydraulics 2-33
2.5 Conclusions 2-46
2.6 References.....2-49

PART 3 – GEOTECHNICAL
3.1 Introduction 3-1
3.2 Description of Existing Levee Unit 3-1
3.3 Underseepage Analysis 3-4
3.4 Probabilistic Underseepage Analysis Results.....3-7
3.5 Slope Stability Analysis.....3-8
3.6 Summary.....3-13
3.7 References.....3-13

PART 4 – STRUCTURAL
4.1 Introduction 4-1
4.2 Engineers.....4-1
4.3 Deterministic Criteria.....4-2
4.4 Uncertainty Analysis 4-4
4.5 Risk Calculations 4-5
4.6 Structural Material Properties.....4-5
4.7 Soil Material Properties.....4-6
4.8 Structural Analysis 4-6
4.9 References.....4-15

PART 5 – CIVIL

5.1 Introduction 5-1
5.2 Description of Existing Levee Unit 5-1
5.3 Utility Uplift Analysis.....5-1
5.4 Above Ground Utilities and Features 5-6
5.5 Pump Station Assessment.....5-6
5.6 Summary.....5-7
5.7 Items of Consideration for Second Phase.....5-7
5.8 References.....5-9

Detailed drawings and calculations supporting this Appendix are available upon request.

PART 1 – Introduction to Engineering Appendix Volume 1

Table of Contents

1.1 ENGINEERING APP. VOL. 1 PURPOSE AND CONTEXT 1-1

1.2 PROJECT AUTHORITY 1-1
1.2.1 Original Levee Unit Authority 1-1
1.2.2 Feasibility Study Authority 1-1

1.3 FEASABILITY STUDY OVERVIEW 1-1

1.4 LEVEE UNIT LOCATION AND GENERAL DESCRIPTION 1-2

1.5 ENGINEERING DOCUMENTATION 1-3

1.6 EXISITING CONDTIONS ANAYLSIS 1-3

1.7 FUTURE WITHOUT PROJECT CONDTIONS ANAYLSIS.....1-4

1.1 ENGINEERING APPENDIX VOLUME 1 PURPOSE AND CONTEXT

This Feasibility Report Engineering Appendix Volume 1 documents the existing conditions engineering analysis and the development of the future without project conditions which support the Manhattan, KS Local Protection Project Feasibility Study. Specifically, this Appendix includes chapters detailing the Hydrology and Hydraulics, Geotechnical, Structural, and Civil Design analyses of the existing project. Information pertaining to the future with project analyses, and the evaluation of the proposed alternatives and project implementation, is found in the Engineering Appendix Volume 2.

1.2 PROJECT AUTHORITY

1.2.1 Original Levee Unit Authority

The Flood Control Act approved 3 September 1954 (Title II, Public Law 780, 83d Cong., 2d Sess., H.R. 9859) authorized the original Manhattan, Kansas, local flood protection project. Construction of the project began on 4 May 1961, and local interests accepted the completed project for operation and maintenance in July 1963.

1.2.2 Feasibility Study Authority

This feasibility study was initiated in 2005 under the authority of **Section 216** of the Flood Control Act of 1970, which allows the Corps to review previously constructed Civil Works projects and make recommendations for improvements. Section 216 reads as follows:

"The Secretary of the Army, acting through the Chief of Engineers, is authorized to review the operation of projects, the construction of which has been completed and which were constructed by the Corps of Engineers in the interest of navigation, flood control, water supply, and related purposes, when found advisable due to the significantly changed physical or economic conditions, and to report thereon to Congress with recommendations on the advisability of modifying structures or their operation, and for improving the quality of the environment in the overall public interest."

1.3 FEASIBILITY STUDY OVERVIEW

The existing Manhattan Kansas local protection project is comprised primarily of one levee unit and associated appurtenances and is locally known as the "Manhattan Levee". The U.S. Army Corps of Engineers Kansas City District (CENWK) and the local project sponsor (City of Manhattan, Kansas) are conducting this feasibility study. The Manhattan Levee is located along and near the confluence of the Big Blue and Kansas Rivers and protects a highly-developed area around downtown Manhattan, Kansas from river flooding. This is a single purpose study focusing on flood risk management. The feasibility study will update and verify data on the level of flood risk management provided by the existing project, and as warranted, will develop alternative plans for increasing the reliability of the existing project.

The existing levee withstood the 1993 Flood (60,000 cfs on Big Blue River and peak flows of 100,000 cfs on the Kansas River), but the 60,000 cfs release from Tuttle Creek Dam created a near overtopping situation at some Big Blue River locations along the levee. The 1993 event raised concerns that the levee may provide much less than the design level of performance. The Big Blue River tieback elevation was (originally) designed for a 110,000 cfs release from Tuttle Creek Lake plus 2 feet of freeboard (with 210,000 on Kansas). This potential for overtopping led to a request from the City of Manhattan for Corps assistance to study the problem, determine if the levee may provide less than the authorized benefits for which it was designed, and report out.

The City of Manhattan, Kansas owns and operates the Manhattan Kansas local protection project, and serves as the primary local point of contact for all community-related matters regarding this study. The City staff work with the Corps of Engineers study team members on a routine basis and ensure that City and local considerations are taken into account as the study progresses.

The main Feasibility Report contains the feasibility investigations, findings, alternative plan evaluation, and the recommended plan for reliability improvements to the existing project. Following approval, the Feasibility Report is accompanied by a Chief of Engineers Report transmittal to Congress. Congressional project authorization (normally provided under a “WRDA” – Water Resources Development Act) is then needed to move forward with any recommended construction project. An Environmental Assessment will accompany the Final Feasibility Report and provide the supporting environmental and NEPA documentation for any recommended Federal action.

1.4 LEVEE UNIT LOCATION AND GENERAL DESCRIPTION

The City of Manhattan is located in central Kansas, and lies at the confluence of the Big Blue River and the Kansas River. The Big Blue River is east of the downtown area and flows into the Kansas River which lies just south and southeast of the city. Both rivers have a history of repeated and sometimes catastrophic flooding. A number of Corps lake projects have provided some regulation of these rivers starting in the mid-twentieth century, resulting in some reduced flooding risks for the city.

The Manhattan levee unit is located generally west and north of the confluence of the Big Blue River and the Kansas Rivers, and is approximately 28,850 feet long. The levee was typically constructed with a 10-foot crown width and three horizontal to one vertical (3H: 1V) embankment slopes. The levee is predominantly an earthen unit -- with a small number of concrete and steel structural components. The land area within the levee unit is almost completely developed and exhibits intense commercial, governmental and light industrial uses accompanied by an residential population located generally along the fringe of the current protection.

The Corps of Engineers Tuttle Creek Lake is situated just to the north of Manhattan with the Big Blue River flowing into and out of Tuttle Creek Lake. Tuttle Creek is a major

lake in the Kansas River basin system of lakes. The system is critical to the Corps' flood risk management mission for both the Kansas and Missouri Rivers.

1.5 ENGINEERING DOCUMENTATION

Study procedures were conducted and the Engineering Appendix content was prepared and arranged in accordance with the engineering guidance described in ER 1105-2-100 Appendix G for feasibility studies. In general, this appendix falls in the category of important supporting documentation for the plan formulation and selection process.

1.6 EXISTING CONDITIONS ANALYSIS

Much of the existing conditions analysis was performed using data and observations from recent high water events (since the original project design), especially the 1993 Flood. Current Corps levee criteria were used. This provided a thoroughly updated engineering analysis of the project. The engineering evaluations, the economic conditions and flood damage analysis, and a summary baseline environmental review of the study area together form a picture of the existing conditions used in the study analysis.

During the existing conditions analysis, the team performed a comprehensive review of the existing hydraulic, geotechnical and structural performance characteristics of the levee unit for various flood conditions. Vol. 1 of the Engineering Appendix (and the accompanying mapbook) contains details of the engineering analysis which supports the identification and quantification of the existing levee risks. Fragility curves were calculated by geotechnical and structural engineers at several locations along the existing levee where vulnerabilities were identified. These separate fragility curves were then translated to "index points", using new hydraulic profiles generated during the study, and combined into a single probability of failure curve for each index point and associated reach.

The Corps HEC-FDA model (a certified economic flood damage analysis program used for Corps feasibility studies) includes the capability for geotechnical and structural failure curve inputs. These failure curves must be considered whenever the levee performance is in doubt, i.e. anytime the levee could fail prior to being overtopped. Including the geotechnical and structural characteristics in HEC-FDA ensures that the economic analysis results are indicative not only of overtopping risks, but also of the risks and potential for damages posed by other major limitations of the existing levee. Once the combined and translated probability of failure curves for each reach are input to the HEC-FDA program, the model uses the probability of failure curves in conjunction with water surface profiles to determine: existing expected annual damages, levee assurance statistics, and other economic-related characteristics of the existing (or any proposed) project and study area. Part 1 Hydrology & Hydraulics (H&H) of Engineering Appendix Vol. 2 contains additional details of the risk translation and use of index points.

A City of Manhattan effort (locally-funded) recently completed the construction of a minimal levee raise for a short segment along the Big Blue River as shown in Figure 1. This small raise addressed a critical low area in levee height and enabled the City to

receive a FEMA 100-year level of protection certification letter early in 2013. The Kansas City District Corps of Engineers reviewed the approved the modification under 33 USC 408 and worked closely with the City to ensure that the H&H modeling used for the design of the raise, and the associated FEMA certification process, was the same as the new modeling generated during this study. This small raise is included in the existing baseline conditions of the levee for this study -- except that this raise does not show on the older O&M Drawings in the mapbook.

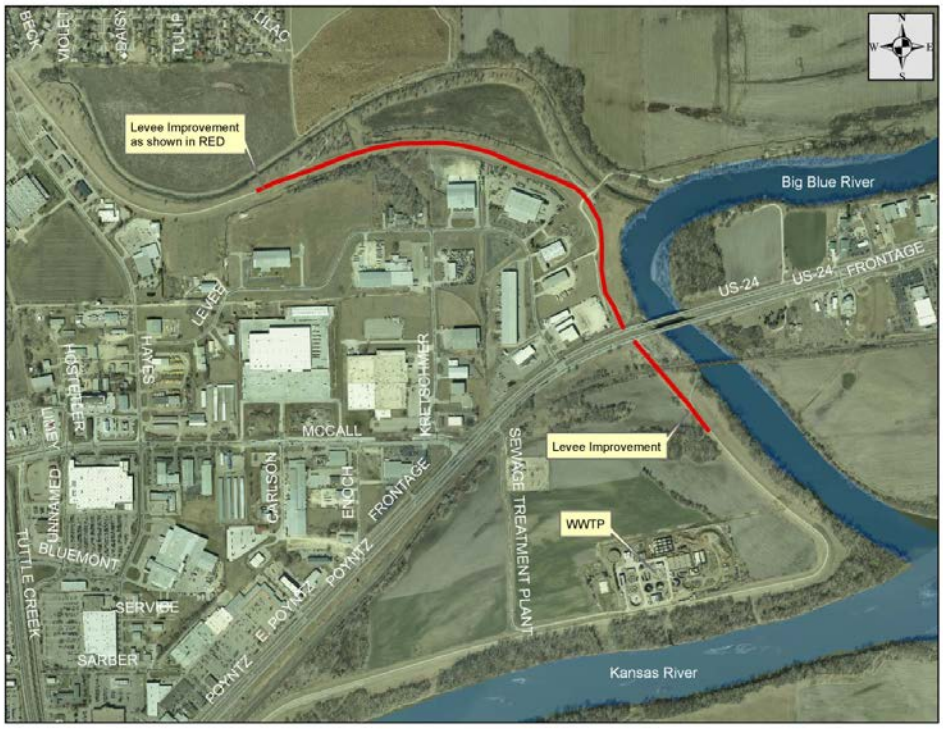

Figure 1
Local FEMA Levee Raise ("Levee Improvement") completed in 2012

1.7 FUTURE WITHOUT PROJECT CONDITIONS ANALYSIS

Establishing a basis for the comparison of project alternatives involves analysis and forecasting of the most likely future without project condition. The future without project condition for this study captures the prevailing future conditions if no major (Federal) action is taken towards solving the Manhattan levee performance problems. The future without project condition is the essentially the same as the "no action" alternative described in the National Environmental Policy Act (NEPA) regulations.

From a hydraulic engineering standpoint, the future without project condition for the study represents the: probable stage-discharge relationship at a selected future time (year 2060) based on the best available current data, the incorporation of any definite projects planned to be completed within the study reach, and any long-term natural river processes that may affect future stages. The future without project conditions analysis includes the following hydraulic characteristics:

- For the purposes of this study, the future without project hydraulic conditions are reasonably expected to be static and consistent with present conditions.
- No projects are planned for the Kansas River or the Big Blue River that will affect the future conditions water surface elevations in the Manhattan area.
- No stage trends are evident in the Big Blue nor Kansas River stretches under study.
- For the flood conditions under study, the operations of Tuttle Creek Lake and Dam, and the upstream Kansas River basin lakes will be consistent with current operational guidelines.

The geotechnical and structural condition of the levee will remain fairly constant over time provided that the City of Manhattan continues to perform adequate operation and maintenance. Based on recent annual inspections of completed works (a joint levee inspection performed by Kansas City District and the City of Manhattan), the City's operation and maintenance support are currently adequate. The City is committed to performing adequate maintenance into the period of feasibility analysis.

In the future, absent a Federal project, the study area will remain exposed to potential flood damages similar to those associated with the existing condition. The potential cost of future flood damages may increase as new development continues to occur within the protected area. Significantly, the land available for development within the City is somewhat constrained by institutional boundaries (Fort Riley and Kansas State University) and physical barriers (the Kansas River and the Big Blue River). Population growth and development within and around the general study area continues resulting in pressure for development of any available tracts of open land. Higher demand leading to re-development and higher values within the protected area might make our future flood damage projections conservative. Any new developments within the protected area that have reliable near-term development schedules are being considered during remaining feasibility efforts.

See the main feasibility report for a more detailed discussion of the future without project condition.

PART 2 – Hydrology and Hydraulics

Table of Contents

2.1 SITE DESCRIPTION 2-1
2.1.1 Study Area 2-1
2.1.2 Manhattan Levee Unit 2-1
2.1.3 Kansas River 2-3
2.1.4 Big Blue River 2-4
2.1.5 Tuttle Creek Lake 2-5

2.2 SITE BACKGROUND 2-6
2.2.1 Flood History 2-6
2.2.2 Flood of 1844 2-7
2.2.3 Flood of 1903 2-7
2.2.4 Flood of 1951 2-8
2.2.5 Flood of 1993 2-11

2.3 HYDROLOGY 2-12
2.3.1 Levee Design Discharges 2-12
2.3.2 General Considerations 2-12
2.3.3 Summary of Studies 2-12
2.3.4 Original Adopted Discharges 2-13
2.3.5 Design Profile for Protected Works 2-13
2.3.6 Model Discharges 2-14
2.3.7 Flow Frequency Analysis 2-14
2.3.8 Big Blue River 2-15
2.3.9 Kansas River Upstream of Big Blue River 2-20
2.3.10 Kansas River Downstream of Big Blue River 2-24
2.3.11 Hydrologic Uncertainty 2-25
2.3.12 Coincident Frequency Analysis 2-28
2.3.13 Hydrologic Period of Record 2014 Sensitivity Analysis 2-31

2.4 HYDRAULICS 2-34
2.4.1 Background 2-34
2.4.2 Model Development 2-34
2.4.3 Cross-Section Data 2-34
2.4.4 Kansas River Cross-Section Data 2-35
2.4.5 Big Blue River Cross-Section Data 2-36
2.4.6 Bridge Data 2-38
2.4.7 Other Geometric Data 2-38
2.4.8 Model Calibration 2-39
2.4.9 Existing Conditions Hydraulic Profiles 2-41
2.4.10 Hydraulic Uncertainty 2-42

2.5 CONCLUSIONS 2-46
2.5.1 River Discharges 2-46
2.5.2 River Stages 2-47

2.6 REFERENCES.....2-49

List of Tables

Table 2-1 Estimated Flow Duration and Mean Flow for USGS Gaging Stations at Fort Riley and Wamego on the Kansas River from Perry et al. 2004.....2-4
Table 2-2 Estimated Flow Duration and Mean Flow for USGS Gaging Station on the Big Blue River Near Manhattan from Perry et al. 2004.....2-5
Table 2-3 Discharge Capacity at Tuttle Creek Lake.....2-6
Table 2-4 Comparison of Maximum Daily Peak Streamflows for Floods of 1844, 1903, 1951, and 1993 at Selected USGE Streamflow-Gaging Stations.....2-7
Table 2-5 Monthly Rainfall Leading up to July 1951 Event.....2-8
Table 2-6 Natural and Modified Discharges – Vicinity of Manhattan, Kansas.....2-13
Table 2-7 Flow Frequency Results for the Big Blue River at Manhattan, Kansas.....2-17
Table 2-8 Flow Frequency Results for Kansas River Upstream of Big Blue River.....2-21
Table 2-9 Flow Frequency Results for Kansas River Downstream of Big Blue River.2-25
Table 2-10 Coincident Kansas River Flows for a Big Blue River Flood.....2-29
Table 2-11 Coincident Big Blue River Flows for a Kansas River Flood.....2-30
Table 2-12 Bridges Included in Manhattan Levee HEC-RAS Model.....2-38
Table 2-13 HEC-RAS Model Calibration to 1993 High Water Marks.....2-40

Tables of Figures

Figure 1 Manhattan Levee Unit.....2-2
Figure 2 Kansas River and Tributaries.....2-3
Figure 3 Isohyetal Map of Rainfall in June, 1951.....2-9
Figure 4 Isohyetal Map of Rainfall Resulting from Event Spanning July 9th to 12th.....2-10
Figure 5 Map of Area Flooded During July 1951 at Manhattan, Kansas from USGS Water Supply Paper 1139 (USGS 1952).....2-11
Figure 6 Big Blue River Flow Frequency.....2-18
Figure 7 Big Blue River Annual Peak Flow Relationships, Unregulated Versus Regulated Flows.....2-19
Figure 8 Kansas River Upstream of the Big Blue River Confluence Flow Freq.....2-22
Figure 9 Kansas River Upstream of the Big Blue River Confluence Annual Peak Flow Relationships, Unregulated Versus Regulated Flows.....2-23
Figure 10 Kansas River Downstream of the Big Blue River Confluence Flow Frequency.....2-26
Figure 11 Kansas River Downstream of the Big Blue River Confluence Annual Peak Flow Relationships, Unregulated Versus Regulated Flows.....2-27
Figure 12 Big Blue River and Kansas River Downstream of the Confluence, Coincidence of Regulated Peaks.....2-30

Figure 13 Kansas River Upstream of the Confluence and Kansas River Downstream of the Confluence, Coincidence of Regulated Peaks 2-31
Figure 14 Peak Streamflows Upstream and Downstream of Manhattan, KS 2-32
Figure 15 Graphical Results of Regulated and Unregulated Bulletin 17B POR extension.....2-33
Figure 16 Spatial Extents of Source Data for TIN.....2-35
Figure 17 Kansas River Cross Section Spacing through Study Area.....2-36
Figure 18 Big Blue River Cross Section Spacing through Study Area 2-37
Figure 19 HEC-RAS Model Calibration to the 1993 Flood Event and Check of 1951 Flood Event at the Manhattan Levee.....2-40
Figure 20 HEC-RAS Existing Condition Water Surface Profiles at Manhattan Levee 2-42
Figure 21 Comparison of Rating Curve #28 and Observed Measurements at Wamego Gage on the Kansas River 2-45
Figure 22 Comparison of Rating Curve #56 and Observed Measurements at Manhattan Gage on Big Blue River 2-45
Figure 23 Comparison of Kansas River Flood Profiles to 2005 FEMA FIS Profiles ...2-48
Figure 24 Comparison of Big Blue River Flood Profiles to 2005 FEMA FIS Profiles ..2-48

Acronyms

cfs	cubic feet per second
FEMA	Federal Emergency Management Agency
FIS	flood insurance study
km	kilometer
HEC-DSS	Hydrologic Engineering Center Data Storage System
HEC-RAS	Hydrologic Engineering Center River Analysis System
HEC-SSP	Hydrologic Engineering Center's Statistical Software Package
LP3	log-Pearson Type III
LPS	low point of steel
msl	mean sea level
R.M.	river mile
USACE	U.S. Army Corps of Engineers
USGS	United States Geological Survey

2.1 SITE DESCRIPTION

2.1.1 Study Area

The study area includes that portion of the Kansas River from the confluence with the Big Blue River west to Wildcat Creek, and the Big Blue River from the confluence with the Kansas River north to the outlet of Tuttle Creek Dam. Within this area are the City of Manhattan (primarily urban commercial and residential) and the unincorporated areas of Riley and Pottawatomie Counties (primarily agricultural and low-density residential) surrounding the Big Blue and Kansas River confluence.

The flood plain of the Kansas River in the vicinity of Manhattan is from one-half to one mile wide. Elevations in Manhattan vary from 1,005 feet in the southeast part to 1,040 feet in the northwest part. All elevation references are NAVD88 unless noted otherwise. Wildcat Creek, which is outside of the levee, is south of the city, flows east and enters the Kansas River at River Mile 148.4. The Big Blue River is east of the city and flows southward, entering the Kansas River immediately downstream from Manhattan at River Mile 145.15.

The urban Manhattan floodplain includes a centrally located downtown and commercial area (which includes a major regional shopping mall), neighborhoods generally to the west of the downtown area, and a light industrial area to the east. This area contains more than 1,500 homes and roughly 500 businesses and public facilities.

One bridge crosses the Kansas River in the study reach; the K-18/177 Bridge at Manhattan (R.M. 147.04). Four bridges cross the Big Blue River in the study reach; the two parallel K-24 Bridges (R.M. 0.8), the Union Pacific Railroad Bridge (R.M. 0.7), and the Dyer Road Bridge (R.M. 7.63).

2.1.2 Manhattan Levee Unit

The Manhattan levee unit is located in and around the City of Manhattan, in Riley County Kansas. As show in Figure 1, the levee unit provides flood risk management to Manhattan from floods on the Big Blue and Kansas Rivers. The existing flood risk management measures consist of 28,841 feet of levee, and 4,100 feet of channel improvement for the Kansas River, modification of the Rock Island (now Union Pacific) railroad bridge, six pressure relief wells, and two pumping plants (USACE 2004). Construction of the project was initiated on May 4, 1961 and the completed project was transferred to local interests for operation and maintenance in July 1963.

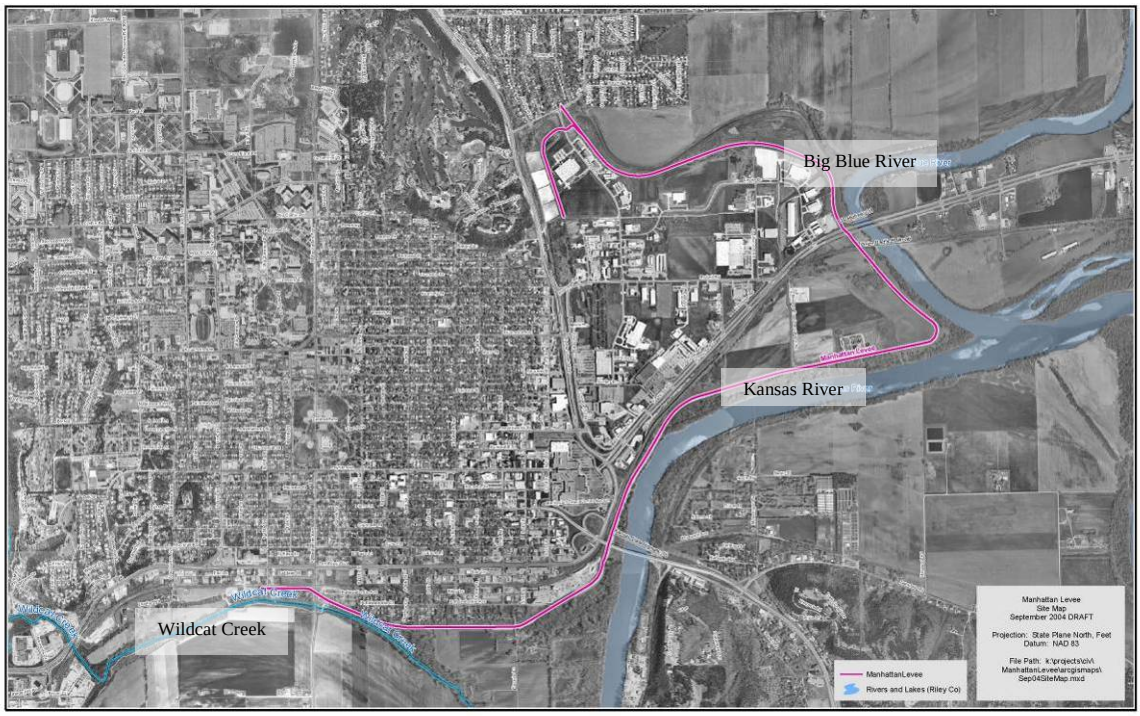

Figure 1. Manhattan Levee Unit

The Manhattan levee unit starts at high ground on the left bank of Wildcat Creek in the vicinity of 15th Street; continues downstream along a pronounced shelf in the river valley to the vicinity of 4th Street; then along the left bank of the Kansas River to the mouth of the Big Blue River where it continues along the right bank to where the Big Blue River turns east; and then along the left bank of the old abandoned Big Blue River channel and across it to high ground. (USACE 1980)

The levee unit was designed to operate with upstream reservoirs such as Tuttle Creek, Milford, Glen Elder, Wilson, and Kanopolis. Tuttle Creek Reservoir controls the Big Blue drainage area upstream of Manhattan. The drainage area above Manhattan on the Kansas River is 45,464 square miles, of which 39,643 square miles are controlled by upstream reservoirs, leaving an uncontrolled drainage area of 5,821 square miles upstream of Manhattan. The Kansas River portion of the project was designed using a balanced overtopping discharge of 335,000 cubic feet per second (cfs). This provided a freeboard of 2 feet above the design discharge (220,000 cfs) at the mouth of the Big Blue River. The Big Blue River tieback was designed using an 110,000 cfs release from Tuttle Creek Reservoir with 2 feet of freeboard. (USACE 1980).

2.1.3 Kansas River

The Kansas River Basin is located in northern Kansas, southern Nebraska, and eastern Colorado. Major tributaries include the Big Blue, the Republican, the Saline, the Solomon, and the Smoky Hill Rivers. Much of the river's watershed is dammed for flood control, but the river is generally free-flowing and has only minor obstructions, including diversion weirs and one low impact hydroelectric dam. The Kansas River is formed by the convergence of the Republican and Smoky Hill Rivers at Junction City, Kansas, about 20 miles upstream from Manhattan (see Figure 2). The river receives discharge from the Big Blue River about 2 miles downstream from Manhattan, where the normal discharge of each river is nearly equal (Perry 1993).

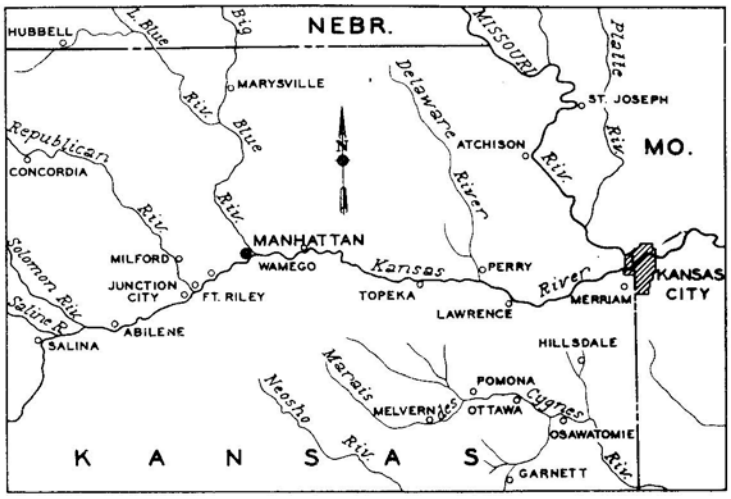

Figure 2. Kansas River and Tributaries

The Kansas River valley averages 2.6 miles (4.2 kilometer [km]) in width, with the widest points being downstream (east) of Manhattan between Wamego and Rossville, Kansas. In the 148-mile stretch between the gaging stations at Fort Riley and Bonner Springs, the river has an average gradient of approximately 2 feet per mile (Fader, 1974). Along this same stretch of the river, the floodplain has a gradient of nearly 2.5 feet per mile. The river follows a sinuous path but primarily flows on the south side of the valley, impinging on the south valley wall at numerous places between Junction City and Lawrence, Kansas (Lawrence et al. 1998). The river approaches the north valley wall at two places between Junction City and Manhattan, Ogden Hill just east of the confluence of the Republican and Smoky Hill Rivers and at Stagg Hill located south of Manhattan.

The average top width of the Kansas River between the confluence of the Smoky Hill and Republican Rivers (R.M. 170) and the Big Blue River is about 480 feet (Simons et al. 1984). Downstream the average top width increases to 740 feet between the Big Blue River confluence and Willard, Kansas (R.M. 100).

The variation in Kansas River discharge is related to climate, season, and location in the drainage basin. The highest discharge occurs in the summer months with the maximum monthly discharge in July. Estimated flow duration and mean flow values for United States Geological Survey (USGS) gaging stations at Fort Riley and Wamego on the Kansas River (from Perry et al. 2004 Table 5 and 6) are summarized in Table 2-1.

Table 2-1. Estimated Flow Duration and Mean Flow for USGS Gaging Stations at Fort Riley and Wamego on the Kansas River from Perry et al. 2004

Gage Station	Stream Name	Estimated flow duration (cfs) for percentage of time flow equaled or exceeded					Mean streamflow 1961 to 2000 (cfs)
		90%	75%	50%	25%	10%	
06879100	Kansas River at Fort Riley, KS	408	637	1,350	3,310	7,450	3,010
06887500	Kansas River at Wamego, KS	846	1,360	2,720	6,340	13,600	5,650

2.1.4 Big Blue River

The Big Blue River basin comprises a total area of 9,628 square miles. Three-fourths of the basin is situated in Nebraska, and the remainder in Kansas. The source of the Big Blue River is near Grand Island, Nebraska. From there, the river flows generally eastward about 50 miles to Ulysses, Nebraska, and then southward over 300 miles to Tuttle Creek Lake (USACE 2004). The Big Blue River valley varies in width from 1,320 feet to nearly 1 mile, with an average of approximately 4,000 feet from Tuttle Creek dam northward to Seward, Nebraska (Weatherly 1994).

The portion of the Big Blue River within the study area (lower Big Blue) extends approximately 12.3 river miles downstream of Tuttle Creek Lake to its confluence with the Kansas River at R.M. 145. Below Tuttle Creek dam the presence of Rocky Ford dam just 1 mile downstream influences the tailwater elevation in the Tuttle Creek stilling basin and in River Pond (USACE 2002). Weatherly (1994) found that significant channel migration on the lower Big Blue occurred in response to flood events and that minimal lateral migration occurred between 1857 and 1989. Further, the channel path stabilized after the 1951 flood and since completion of the Tuttle Creek dam has not undergone significant lateral migration. Estimated flow duration and mean flow values for the USGS gaging station on the Big Blue River near Manhattan (from Perry et al. 2004 Table 5 and 6) are summarized in Table 2-2.

Table 2-2. Estimated Flow Duration and Mean Flow for USGS Gaging Station on the Big Blue River Near Manhattan from Perry et al. 2004

Gage Station	Stream Name	Estimated flow duration (cfs) for percentage of time flow equaled or exceeded					Mean streamflow 1961 to 2000 (cfs)
		90%	75%	50%	25%	10%	
06887000	Big Blue River near Manhattan	178	444	974	2,450	6,500	2,490

A study by Juracek (2001) assessing channel-bed elevation changes downstream from large reservoirs in Kansas found that the channel bed of the Big Blue River down-stream from Tuttle Creek Lake lowered 3.9 feet from 1962 (year of dam completion) to 1997. The channel bed lowered at an average rate of about 0.1 foot/year and showed no indication of slowing as of 1997.

2.1.5 Tuttle Creek Lake

Tuttle Creek Lake is located on the Big Blue River, 12.3 river miles upstream from the confluence of the Big Blue and Kansas Rivers and is 6 miles north of Manhattan. The lake's drainage area is 9,628 square miles. The lake provides flood control, recreation, fish and wildlife conservation, water quality control, and navigation supplementation. When at its multipurpose pool, elevation 1075.44 feet NAVD88, the lake covers about 12,350 acres, extends some 16 miles upstream from the dam, and spreads a mile wide, on average, over that length. At full flood control pool, elevation 1136.44 (NAVD88), the lake swells to cover about 53,700 acres and extends upstream 35 valley miles. (USACE 2004)

Discharges from Tuttle Creek Lake are controlled by the outlet works under both multi-purpose and flood control releases. The emergency spillway will be used under surcharge conditions or when surcharge operations are anticipated. The outlet works include a low flow outlet and a primary outlet works. The low flow outlet is two 24-inch pipes that have a discharge of approximately 100 cfs per pipe at pool elevation 1061.44 feet NAVD88. The primary outlet works consist of four 10 foot by 20 foot gates with one additional emergency gate. These gates control discharges to two 20 foot diameter horseshoe conduits that discharge to a stilling basin and outlet channel. The primary outlet works have a discharge capacity of 31,300 cfs at pool elevation 1075.44 feet and 45,900 cfs at pool elevation 1136.44 feet. The spillway is at a crest elevation of 1116.44 feet NAVD88 and is controlled with eighteen 40 foot by 20 foot tainter gates. The spillway has a maximum discharge of approximately 579,000 cfs at maximum water surface elevation of 1151.84 feet NAVD88. Table 2-3 summarizes the outlet work and spillway discharge capacity for Tuttle Creek Lake even though operational criteria will likely control actual releases from the lake.

Table 2-3. Discharge Capacity at Tuttle Creek Lake

Pool Elevation (NAVD88)	Low Flow Outlet Discharge (cfs)	Primary Outlet Works Discharge (cfs)	Spillway Discharge (cfs)
1030.44	0	0	0
1061.44	200	27,802	0
1075.44	290	31,300	0
1116.44	555	40,898	0
1124.44	606	42,802	50,000
1128.44	632	43,755	100,000
1132.44	658	44,707	150,000
1135.44	677	45,421	200,000
1136.44	684	45,900	233,500
1151.44	781	49,231	579,000

Note: Shaded cells are interpolated values based on flows taken from the Tuttle Creek Operation Manual. Non-shaded cells were taken directly from the manual.

2.2 SITE BACKGROUND

This study was initiated in response to events observed in Manhattan during the July 1993 flood. The Manhattan Kansas levee unit withstood the flood of 1993, but some elements of the system were seriously challenged as releases from Tuttle Creek Dam created a situation with approximately 3.5 feet of freeboard at some locations along the Big Blue River levee segment. Although the design documentation for the Manhattan levee describes a system designed for a significantly higher coincident flow than the July 1993 experience (USACE 2004), this event raised a concern that the levees may be performing below the intended design performance level. The following sections provide a discussion of the flood history of the City of Manhattan, including background information of the 1951 and 1993 flood events.

2.2.1 Flood History

Major floods on the Kansas River are usually caused by a series of short-duration, high-intensity storms following a protracted period of general rains which reduce the infiltration capacity of the soil to a minimum and cause a greater than normal flow in the stream channels. Many of the early floods in the Kansas River Basin are a matter of legend and tradition rather than actual historical record. Early floods affecting the area in the vicinity of Manhattan occurred in 1785, 1826, 1844, 1845, 1851, 1858, 1870, 1881, and 1886, but very little is known concerning these floods (USACE 1959). More recent flood events for which records are available occurred in 1903, 1904, 1908, 1915, 1935, 1941, 1945, 1947, 1948, 1950, 1951, and 1993. The 1844 flood appears to

have produced the maximum stages of record on the Kansas River followed in magnitude by the 1951, 1903, and 1993 stages.

A comparison of the relative flood depths by Juracek et al. (2001) showed that the 1844, 1951, and 1903 flood depths at Ogden, Kansas were about 10, 5.25, and 0.25 feet higher, respectively, than for the 1993 flood. The maximum daily peak streamflows for these four floods for the Kansas River at Ogden, Big Blue River near Manhattan, and Kansas River at Wamego are summarized in Table 2-4. A description of the four largest floods at Manhattan is given in the following subsections.

Table 2-4. Comparison of Maximum Daily Peak Streamflows for Floods of 1844, 1903, 1951 and 1993 at Selected USGS Streamflow-Gaging Stations

USGS Station	Station Name	Maximum daily peak streamflows (cfs) (date)			
		1844	1903	1951	1993
06879500	Kansas River at Ogden[2]	(1)	236,000	298,000	85,000
06887000	Big Blue River near Manhattan[3]	(1)	93,800 (May 31)	93,400 (July 12)	58,800 (July 23)
06887500	Kansas River at Wamego[3]	(1)	280,000 (May 30)	400,000 (July 13)	199,000 (July 26)

[1]Flow was greater than 1951 flood.
[2]From Juracek and others (2001).
[3]From Combs and Perry (2003).

2.2.2 Flood of 1844

A great flood is known to have occurred on the Kansas River in June 1844. Data are not available on the nature, extent, and magnitude of the storm rainfall, and only fragmentary data on flood stages are available. A stage of about 40 feet occurred on the Kansas River at Manhattan (USACE 1959).

2.2.3 Flood of 1903

The second largest flood for which reliable stage records are available occurred during the latter part of May and the first part of June 1903. This flood resulted from extraordinarily heavy rainfall the last five days in May preceded by above-normal rainfall throughout May. The rainfall at Abilene was estimated at 15 inches on May 28. Flood stage was exceeded approximately 16 days at Manhattan (USACE 1959).

2.2.4 Flood of 1951

The maximum flood of record occurred in July 1951 and resulted from the combined effects of well above average rainfall in May and June 1951 and a major event spanning four days (July 9 through 12) of extremely heavy precipitation over the lower portion of the Kansas River Basin. Table 2-5 also includes the average monthly rainfall and the standard deviation for each month.

Table 2-5. Monthly Rainfall Leading up to July 1951 Event

	Rainfall, inches		
	April	**May**	**June**
1951	2.8	10.3	11.1
Average	2.9	4.6	5.1
St Dev	1.7	2.7	3.0

An isohyetal map (connecting points having equal amounts of precipitation during a given time period or for a particular storm) for Kansas and western Missouri from the USGS report “Kansas-Missouri Floods of July 1951” (Figure 3) shows that about 40 percent of Kansas received more than 10 inches of rain during June 1951 (USGS 1952). This area included most of the Smokey Hill River Basin and portions of the Republican River Basin. The Big Blue River basin north to the Nebraska border had rainfall in excess of 12 inches, with one area exceeding 14 inches.

The event causing the 1951 flood peaks spanned from July 9th through 12th and occurred in three bursts, with the areas of highest total precipitation receiving nearly one-half of the total amount on July 11 (USGS 1952). The highest precipitation from July 9 through 12 was reported at the weather station in Alta Vista, Kansas 25 miles south of Manhattan. This event followed two months of above normal precipitation over the same area. Near record flows also occurred on the Nebraska headwaters of the Big Blue River during June (USGS 1952).

Figure 4 is an isohyetal map showing rainfall from this event (US Department of the Interior, 1951). This event, combined with the previous months’ rainfall, caused the 1951 flooding on the Kansas River. According to the US Weather Bureau’s Technical Paper 49, the 1% (1/100) ACE 4-day precipitation for these areas of Kansas range from 7 to 11 inches; therefore, this event alone surpasses a 1% (1/100) ACE event in magnitude of rainfall. Coupled with the effect of the two previous months’ above average rainfall, it produced record flooding on the Kansas River.

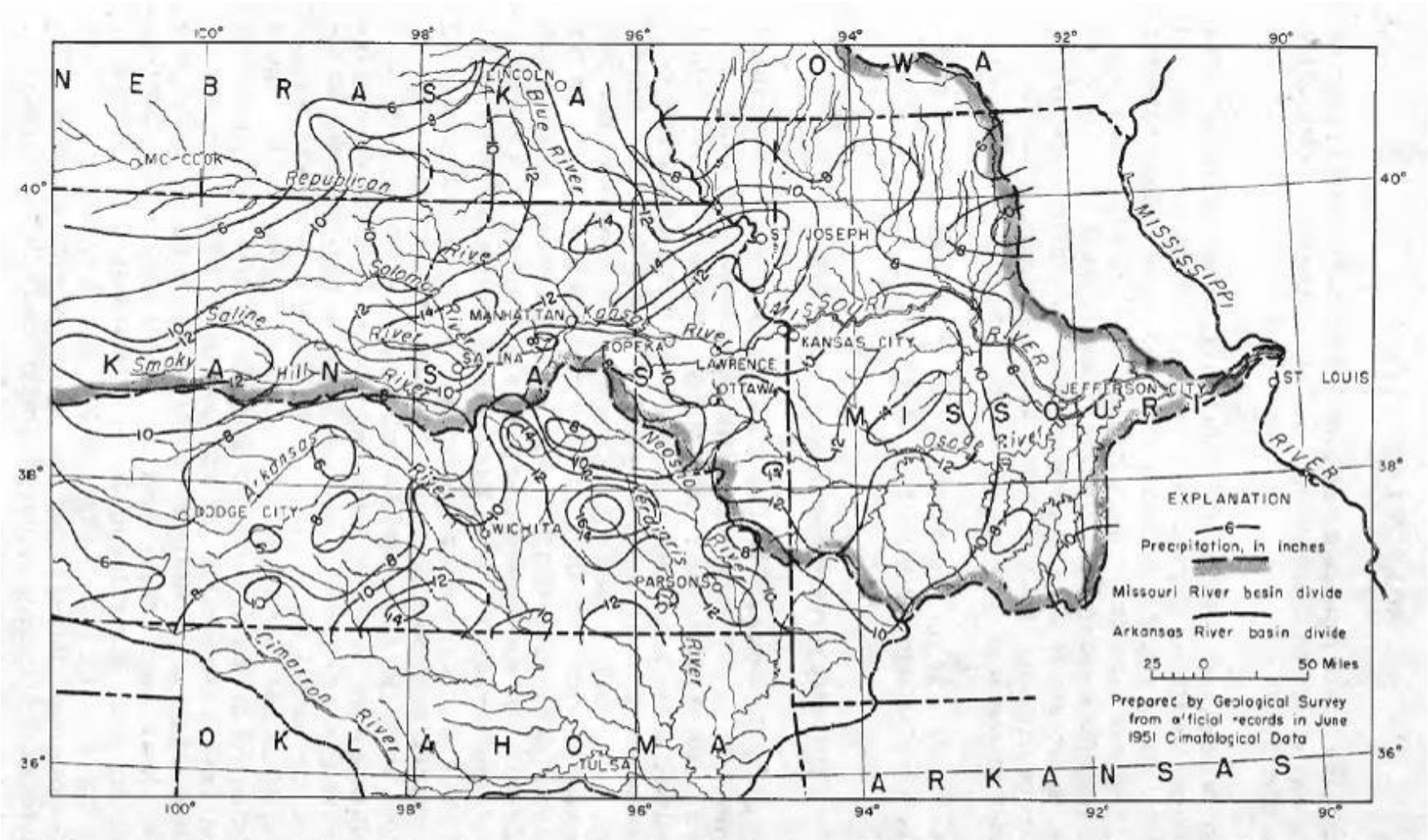

Figure 3. Isohyetal Map of Rainfall in June, 1951
Reference: U.S. Geological Survey (USGS), 1952, Kansas-Missouri floods of July 1951, USGS Water Supply Paper 1139

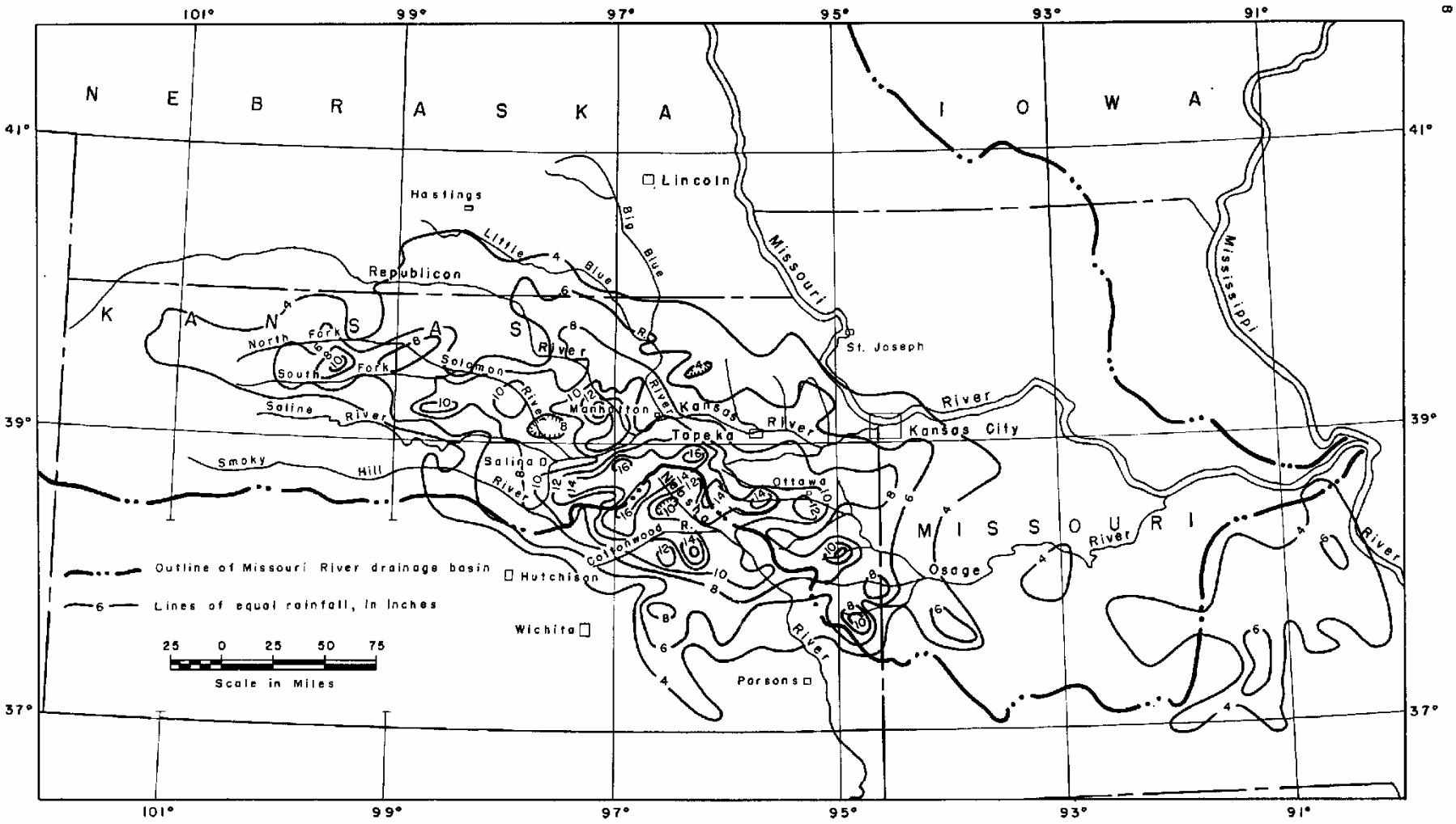

Figure 4. Isohyetal Map of Rainfall Resulting from the Event Spanning July 9th to 12th
Reference: U.S. Department of the Interior – Water Resources Division, Geological Survey Circular 151, October 1951

The Kansas River at Manhattan was above flood stage (17 feet) on June 4, June 7 through 19, and June 22 through July 24, 1951 for a total of 47 days. The maximum gage height of 33.40 feet occurred at 3:00 a.m., July 13, with an estimated discharge of 300,000 cfs in the Kansas River and about 98,000 cfs in the Big Blue River (USACE, Manhattan, Kansas Flood Protection Project Design Memorandum No. 1 - 1959). The USACE reported peak discharge for the Big Blue River near Manhattan in the 1959 report varies approximately 4600 cfs from the USGS peak discharge of 93,400 cfs recorded for the Big Blue River near Manhattan Gage in the 1951 flood.

Fort Riley and Manhattan were the first urban areas to be flooded by the Kansas River. Some of the barracks at Fort Riley were smashed or carried away by the flood. The Kansas River at Manhattan began overflowing on July 11 and was soon spread out from bluff to bluff along the entire valley. The main business district and about 1,600 homes were flooded. Depths of flooding in the main business section ranged from 6 to 8 feet. Swift currents through the parts of Manhattan located nearest the Kansas River demolished some homes and two large industrial buildings (USGS 1952). Figure 5 shows the area flooded during July at Manhattan.

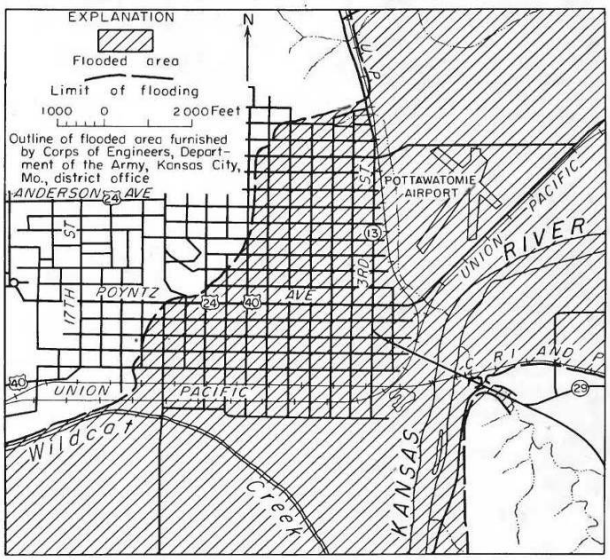

Figure 5. Map of Area Flooded During July 1951 at Manhattan, Kansas from USGS Water Supply Paper 1139 (USGS 1952)

2.2.5 Flood of 1993

The third largest flood for which reliable stage records are available occurred in July 1993. Moderate to major flooding occurred on the Kansas, Big Blue, Black Vermillion, Smoky Hill, Solomon, Saline, and Republican Rivers and their tributaries (Combs and Perry 2003). As with the flood of 1951, heavy rainstorms in July were preceded by several months of above average rainfall. Manhattan received more than 100 percent of the normal precipitation (1961 to 1990 average) for January through June 1993. May through July precipitation at Manhattan was 35.38 inches and was the

second wettest period in 104 years of record. July rainfall was 535 percent of the normal (Wahl et al. 1993).

A comparison of observed controlled and simulated uncontrolled daily mean discharge in the Blue River near Manhattan for July 1993 (Perry 1993) indicated that the storage of floodwaters in Tuttle Creek Lake reduced a potentially devastating flood of more than 95,000 cfs on July 5 on the Big Blue River near Manhattan to a much less destructive flood of 60,000 cfs on July 25. The water surface elevation in the Big Blue came within 3 to 4 feet of the top of the Big Blue River section of the Manhattan levee. Perry (1993) concluded that without the reservoir storage, the Big Blue River near Manhattan would have overtopped the Federal levee, and flooding downstream along the Kansas River would have been much more severe. High stages were most noticeable upstream and near the U.S. Hwy 24 and Union Pacific bridge crossings. The levee did not fail, but this incident prompted great concern within the local community about the levee.

2.3 HYDROLOGY

2.3.1 Levee Design Discharges

The original levee design discharges for the Manhattan levee unit as presented in Design Memorandum No. 1, Manhattan, Kansas Flood Protection Project, (USACE 1959) are discussed in the following subsections.

2.3.2 General Considerations

The development of a Standard Project Flood in accordance with the standard procedures at the time of original design was not practical for a stream such as the Kansas River at Manhattan. In establishing the design discharge for the local protection works at Manhattan, consideration was given to (1) confluence condition due to the Kansas and Big Blue Rivers; (2) the effects of existing, authorized and potential reservoirs in the basin above Manhattan; (3) hazards involved in the event of a flood occurrence with peak flows in excess of design discharges; (4) urban development in the areas to be protected; and (5) design discharges recommended in previous studies.

2.3.3 Summary of Studies

The July 1951 flood occurred with an estimated peak discharge of 298,000 cfs at Ogden, upstream of Manhattan, and 400,000 cfs at Wamego, downstream of Manhattan. Detailed studies were made following the July 1951 flood to establish hypothetical flood flows resulting from a transposition of the July 1951 storm center and to evaluate the effects of authorized and potential reservoirs on natural and hypothetical flows. The natural and modified discharges from these studies for Ogden and Wamego, located upstream and downstream from Manhattan, are shown in Table 2-6.

Table 2-6. Actual and Modified Discharges - Vicinity of Manhattan, Kansas (From Design Memorandum No.1)

	Ogden Mile 162.1	Wamego Mile 126.9
July 1951 flood	**(discharge in 1,000 cubic feet per second)**	
Actual discharge	298	400
Modified discharge (modified by existing and authorized reservoirs)	237	251
Modified discharge (modified by existing, authorized and potential reservoirs [a]	227	216
July 1951 transposition flood [b]		
Natural discharge	334	463
Modified discharge (modified by existing and authorized reservoirs)	270	277
Modified discharge (modified by existing, authorized and potential reservoirs) (a)	189	205

(a) Potential reservoirs considered were Sutphen Mills on Chapman Creek, Turkey Creek, Woodbine on Lyons Creek and Humboldt on Clark Creek.

(b) Transposition with storm center moved 40 miles northeast, rotated 25 degrees, counterclockwise, and no adjustment of rainfall amounts.

2.3.4 Original Adopted Discharge

The original design discharge was modified to 220,000 cfs on the Kansas River upstream of the Big Blue River during committee hearings in May 1954 and was authorized by the Flood Control Act approved 2 September 1954 (H.Doc. 642, 81st Cong., 2d sess.). “In reference to Table 3, protection height, based on a design discharge of 220,000 cfs, will provide for the July 1951 flood as modified by existing and authorized reservoirs, with some encroachment on the freeboard. Likewise, such a protection will provide adequate freeboard for a discharge based on the July 1951 transposed flood as modified by existing and authorized reservoirs and a reasonable number of the most effective potential reservoirs on the secondary tributaries of the Kansas River above Manhattan. Therefore, the authorized design discharge was considered adequate and was used in the project plan.”

2.3.5 Design Profile for Protected Works

The design profile along the Kansas River was based on backwater computations using the design discharge of 220,000 cfs with assumed “n” values of 0.025 and 0.05 for channel and overbank, respectively, which were verified from

calibration to known profiles of the Kansas River. Design flow is considered to be confined by left bank levees along the Kansas River. The elevation at the tieback along the Blue River was determined by assuming an outflow of approximately 110,000 cfs from Tuttle Creek Dam as resulting from a transposition of the July 9 to 13, 1951 storm above Milford and Tuttle Creek Dams and operation with antecedent storage which would have existed as of July 9. The top of levee profile was adopted which would provide balanced overtopping along the Kansas River for a discharge of 335,000 cfs. The revised top of levee profile provided a freeboard of 2 feet above the design discharge (220,000 cfs) at the mouth of the Big Blue River and approximately 3 feet of freeboard above the design discharge at the contraction due to the bridges over the Kansas River present at the time of design. Freeboard along the Blue River tieback levee was revised to 2 feet.

2.3.6 Model Discharges

River flow rates for both the Kansas and Big Blue Rivers for this evaluation were based on output from the Kansas River UNET model. The Kansas River UNET model was developed by the Kansas City District for unsteady flood routing of the daily flows over the period of 1929 to 2002. The daily flows were evaluated in two scenarios. The Unregulated scenario evaluated a synthetic period of record flows along the Kansas and Big Blue Rivers assuming that no reservoirs were in place within the Kansas River Basin for the entire period of record. The Regulated scenario developed a synthetic record assuming current Kansas River Basin reservoir operations were in place for the entirety of the period of record. This approach was developed using actual USGS daily gage records over the period of record. This is an approach similar to the methods used for the USACE Kansas River Hydrology Report (2002) and the USACE Upper Mississippi River System Flow Frequency Study – Appendix E (2003).

2.3.7 Flow Frequency Analysis

The flow frequency analysis completed for both the Big Blue and Kansas Rivers were conducted using a combined analysis of regulated and unregulated flow data. The presence of reservoirs upstream of the study reach for both rivers precludes a flow frequency analysis as defined in Bulletin 17B, as the presence of significant regulation will not produce valid analytical flow frequency results. The final method for flow frequency analysis used a hybrid analysis.

A graphical analysis of regulated data (following USACE Engineer Manual 1110-2-1415) for the most frequent flows is appropriate as the reservoirs provide complete regulation of the frequent flow events. A log Pearson type III (LP3) analysis of unregulated data (following USACE Engineer Manual 1110-2-1415 and Bulletin 17B) was conducted to provide an upper bound for the flow frequency analysis and estimate the most extreme events.

In the largest events, reservoirs will make releases for dam safety in lieu of flow regulation and outflows will tend to match unregulated releases. Between these two

flow frequencies there is a transition zone where the reservoir operations begin to release higher flows to maintain the flood control pool and avoid surcharge operations. In this transition zone an analysis relating unregulated flows to regulated flows was conducted and the relationship was used to convert the log Pearson III unregulated flow frequency to regulated discharges.

The recognition and exclusion of outliers played a significant role in this analysis. USACE Engineer Manual 1110-2-1415, Paragraph 3-2g defines an outlier as a data point which departs significantly from the trend of the remaining data. In a flow frequency analysis, an outlier will have a disproportionate effect on the frequency curve, skewing it either too high or too low against the trend of the rest of the data. For both reaches analyzed, low outliers were found. The Kansas River reach analysis also identified a high outlier. These are described in greater detail in the following sections.

2.3.8 Big Blue River

The final results of the Big Blue River flow frequency analysis for regulated conditions are shown in Table 2-7. Refer to Figure 6 for the flow frequency analysis of the Big Blue River below Tuttle Creek Reservoir. The regulated flow data were plotted and analyzed using a graphical analysis as defined in USACE Engineer Manual 1110-2-1415. The regulated data were assigned plotting positions and a curve was fit through the data. The data points above the 5% (1/20) annual chance exceedance event are from the 1993 and 1951 spillway discharge flood events and likely are not adequately assigned a probability based on the short period of record. Thus the graphical analysis of the regulated data was used only for the flow frequency analysis on the Big Blue River for flows up to the 5% (1/20) annual chance exceedance event.

In order to get a longer period of record than would be available at the dam using the Big Blue River at Manhattan, Kansas gage (pre-dam data started in 1950) and water management daily inflow records post dam (closure was in 1959), and to have a consistent data set, actual daily inflow records for Tuttle Creek Dam were not utilized in the computations. Inflow records in this hydrology study were simulated back to 1929 using upstream inflow gages with daily flows routed to Tuttle Creek Dam via Muskinghum-Cunge and the UNET algorithm for accounting for ungaged inflows to produce expected flow volumes at Manhattan, KS. In general, upstream gage streamflows were factored up using percentage comparisons to flow volumes at Manhattan for pre-dam overlapping data periods between streamflow records, where upstream gages appeared to capture over 80% of the streamflow upstream of Manhattan. Therefore, the daily values of the Tuttle Creek inflow records do not exactly match the simulated inflow record for this feasibility study.

The unregulated analysis was conducted using the procedures defined in USACE Engineer Manual 1110-2-1415 and Bulletin 17B. The unregulated flow frequency analysis was conducted using the Hydrologic Engineering Center's Statistical Software Package (HEC-SSP). The output has been included in Attachment A. The statistical analysis of the unregulated record on the Big Blue River did not identify any high outliers but did identify the 1934 annual peak flow as a low outlier. This data point

was removed from the log Pearson III analysis of the annual peak unregulated flows. The analytical analysis of the unregulated data will be the best estimate for the most extreme events as the reservoir operations will call for passing the incoming flows to protect the integrity of the dam.

The transitional zone of the flow frequency analysis was defined as the point where reservoir regulation of flows begins to be reduced by spillway releases at the lower end of the transition zone and the event where regulated flows are equal to unregulated flows at the high end of the transition zone. The flow frequencies of the transition zone and the point at which the reservoir no longer regulates flood flows must be determined using a comparison of regulated flows (reservoir outflows) and unregulated flows (reservoir inflow) data.

Figure 7 displays the comparison of the annual peak daily inflows to the annual peak daily outflows for a given year. It is evident that there is little correlation between the annual peak daily outflow and the annual peak daily inflow. This is evident by examining some of the extreme events seen on the Big Blue River. The unregulated daily peak flow for 1973 is greater than 100,000 cfs yet the regulated peak discharge for 1973 is only approximately 26,000 cfs, while the unregulated daily peak flow in 1993 was approximately 95,000 cfs and the regulated peak discharge was approximately 60,000 cfs.

A reservoir's outflow (regulated discharges) is dictated more by inflow volume as opposed to inflow daily peaks. A sustained inflow period is necessary to raise reservoir elevations to the point of releasing large outflows. This is evident in both the 1993 and 1951 flood events when large rainfall amounts were recorded over an extended period of time as discussed in Section 2.1.3. Figure 7 displays the relationship of the annual peak reservoir outflow to the annual peak average inflow volumes over 15-day, 30-day, 60-day, and 90-day periods. The correlation between inflow volumes and outflow is much better than that of daily inflow peaks. The peak annual 30-day inflow volume provided the best estimator of peak annual outflow as evidenced by the R^2 value for the trendline of the inflow-outflow relationships.

The 30-day average inflow volume was also chosen as it was the shortest average flow duration that provided a good predictor of outflow. Using the shortest period of inflow that provides a good estimation of outflow is preferable as it maintains the most resolution of inflow peaks from year to year without averaging away peaks for smaller events. A polynomial fit of the 30-day average inflow volumes (unregulated flows) and outflow (regulated flows) relationship was developed. A Log Pearson III frequency analysis of the peak annual 30-day average inflows was analyzed for the period of record.

The 30-day average inflow volumes from this frequency analysis were then converted to peak daily outflows using the polynomial fit. This method provided the outflows from the 4% (1/25) annual chance exceedance event to the 0.5% (1/200) annual chance exceedance event. This analysis also identified those reservoir inflow events greater in magnitude than the 0.5% (1/200) annual chance exceedance event

will most likely behave as unregulated and inflow will equal outflow. Thus the unregulated peak daily flow analysis results were used for the events equal to or greater than 0.33% (1/300) annual chance exceedance.

Table 2-7. Flow Frequency Results for the Big Blue River at Manhattan, Kansas

		Section 216 Feasibility Study Results			FEMA(1)
% Annual Chance Exceedance	Return Period	95% Confidence Limit	Frequency Curve	5% Confidence Limit	Estimated Peak Discharge (cfs)
0.133	750	119,400	179,400	269,700	NA
0.2	500	112,000	167,000	248,800	92,100
0.333	300	102,900	151,700	223,500	NA
0.5	200	81,000	115,800	165,400	NA
1	100	52,800	71,600	97,100	49,600
2	50	35,600	46,200	60,000	46,800
4	25	26,200	33,000	41,600	NA
10	10	21,800	27,000	33,500	36,300

1) FEMA. Revised Feb. 4, 2005. Flood Insurance Study Riley County, Kansas & Incorporated Areas.

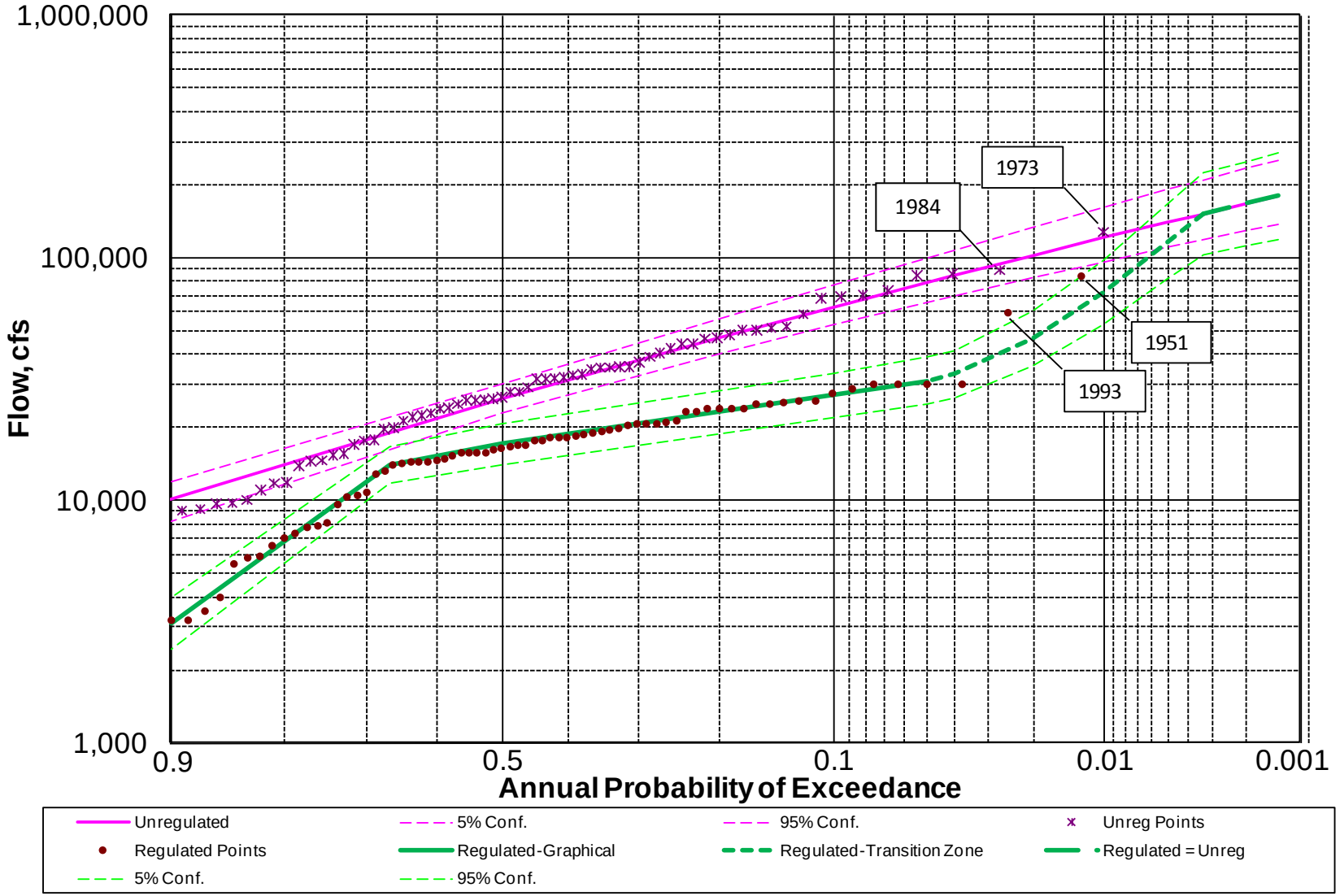

Figure 6. Big Blue River Flow Frequency

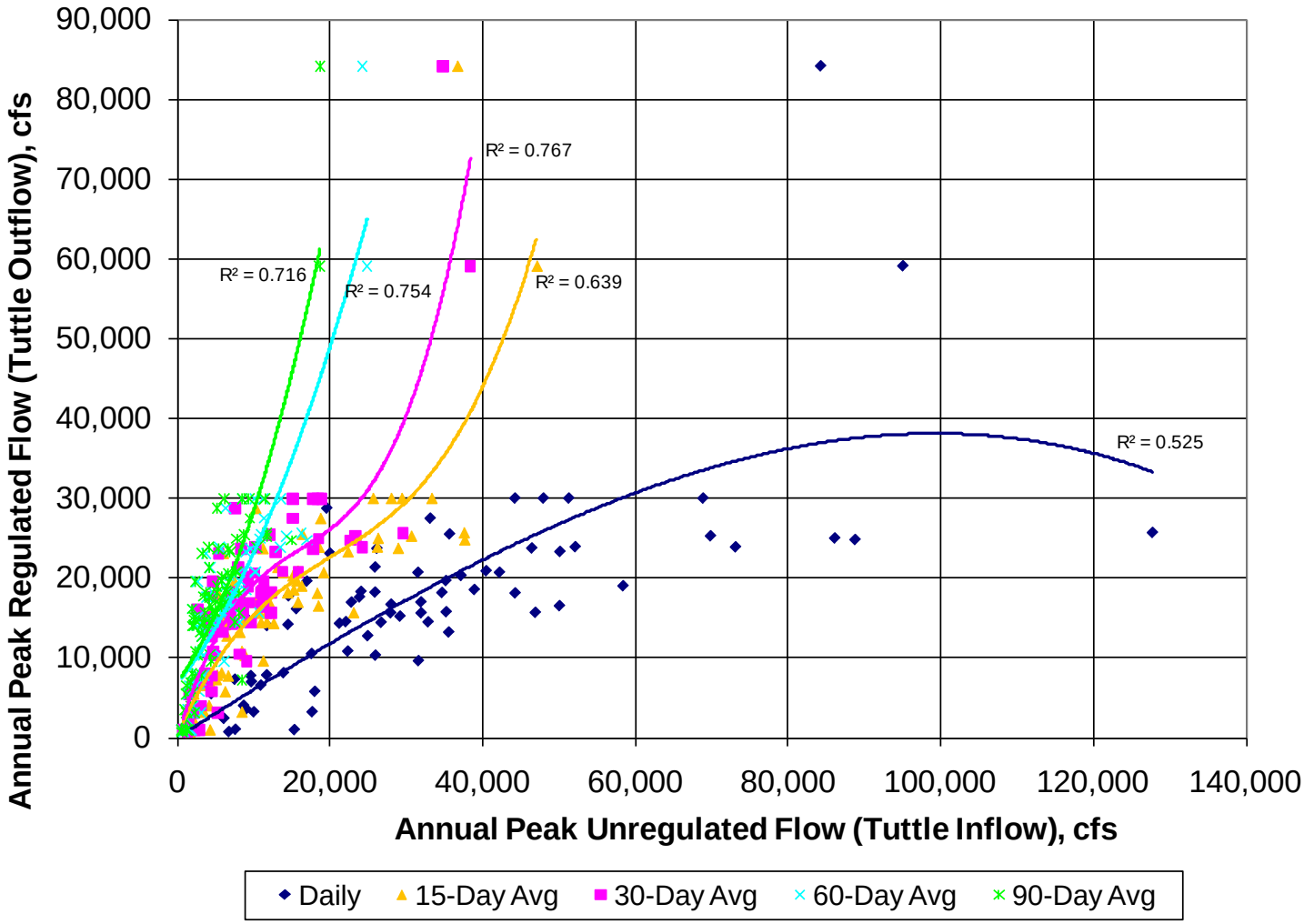

Figure 7. Big Blue River Annual Peak Flow Relationships Unregulated Versus Regulated Flows

2.3.9 Kansas River Upstream of Big Blue River

The final results of the Kansas River upstream of the Big Blue River confluence flow frequency analysis for regulated conditions are shown in Table 2-8. Refer to Figure 8 for the flow frequency analysis of the Kansas River upstream of the Big Blue River Confluence. The regulated flow data were plotted and analyzed using a graphical analysis. The data points above the 5% (1/20) annual chance exceedance event are from the 1993 and 1951 flood events and likely are not adequately assigned a probability based on the short period of record. Thus the graphical analysis of the regulated data was used only for the flow frequency analysis on the Kansas River for flows up to the 5% (1/20) annual chance exceedance event.

The unregulated analysis was conducted similar to the methods used for the Big Blue River. The output has been included in Attachment A. The statistical analysis of the unregulated record on the Kansas River did identify the 1951 peak flow as a high outlier and did not identify any low outliers. The 1951 peak unregulated flow was removed from the systematic record and treated as an historic flood event. The 1903 flood was the last large flood event on the Kansas River prior to the 1951 flood. The 1903 flood is listed as an historic flood event at the USGS Wamego gage on the Kansas River downstream of the confluence.

Thus the relevance of the 1951 flood could be extended to 1903, effectively identifying the 1951 flood event as the largest flood event over 100 years of record. This historic assignment allows for adjusting the log Pearson type III analysis by extending the period of record beyond the systematic record length. The analytical analysis of the unregulated data provides an upper bound for the regulated flow frequency and allows for the estimation of regulated frequency by applying regulation equations to unregulated flow frequencies, as discussed below.

The transitional zone of the flow frequency analysis was defined as the point where reservoir regulation of flows begins to be reduced by spillway releases at the lower end of the transition zone and the event where regulated flows are equal to unregulated flows at the high end of the transition zone. The flow frequencies of the transition zone and the point at which the reservoir no longer regulates flood flows must be determined using a comparison of regulated and unregulated data. Figure 9 displays the comparison of the annual peak daily unregulated flows and the time averaged annual peak unregulated flows to the annual peak daily regulated flows for a given year.

Unlike the Big Blue River flows, the Kansas River unregulated annual peak daily flows are correlated well to the regulated annual peak daily flows. Thus, the relationship between single daily peak flow unregulated and regulated values was used to estimate the regulated peak flows for events greater than the 5% (1/20) annual chance exceedance event. This result is different than the Big Blue River analysis that used annual peak 30-day average unregulated flows, but should be expected.

The Kansas River at Manhattan has significant non-regulated drainage areas and multiple reservoir systems contributing to peak flows. Thus no single reservoir discharge or inflow to a reservoir defines the peak on the Kansas River at Manhattan. Due to the reduced regulation influence on the Kansas River upstream of the confluence as compared to the heavily regulated Big Blue River, a single event that would produce a large unregulated daily peak flow is likely to produce a large regulated daily peak flow as well.

A polynomial fit of the daily peak unregulated flows and daily peak regulated flows was developed. The unregulated log Pearson type III analysis was then converted to the regulated analysis using the polynomial fit. This method provided the outflows from the 4% (1/25) annual chance exceedance event to the 0.133% (1/750) annual chance exceedance event. The unregulated/regulated relationship identified some small benefit of regulation even for the largest events analyzed in this study.

Table 2-8. Flow Frequency Results for Kansas River Upstream of Big Blue River

		Section 216 Feasibility Study Results			FEMA(1)
% Annual Chance Exceedance	Return Period	95% Confidence Limit	Frequency Curve	5% Confidence Limit	Estimated Peak Discharge (cfs)
0.133	750	185,000	277,400	415,900	NA
0.2	500	156,800	229,700	336,500	250,000
0.333	300	127,900	182,200	259,500	NA
0.5	200	108,300	150,800	210,000	NA
1	100	81,700	109,500	146,800	140,000
2	50	61,200	79,000	102,100	105,000
4	25	45,300	56,400	70,300	NA
10	10	27,000	32,000	37,900	46,000

1) FEMA. Revised February 4, 2005. Flood Insurance Study Riley County, Kansas & Incorporated Areas.

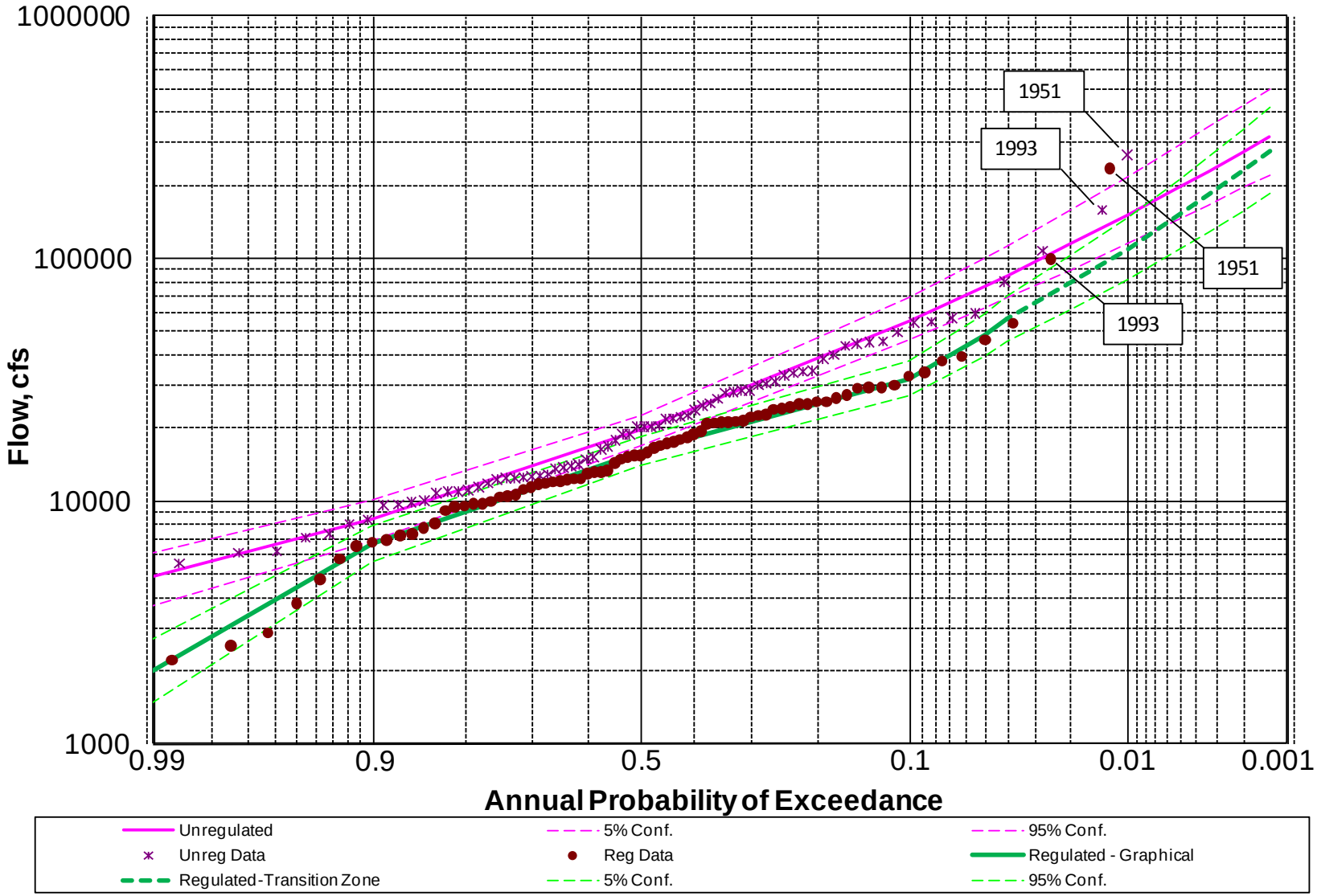

Figure 8. Kansas River Upstream of the Big Blue River Confluence Flow Frequency

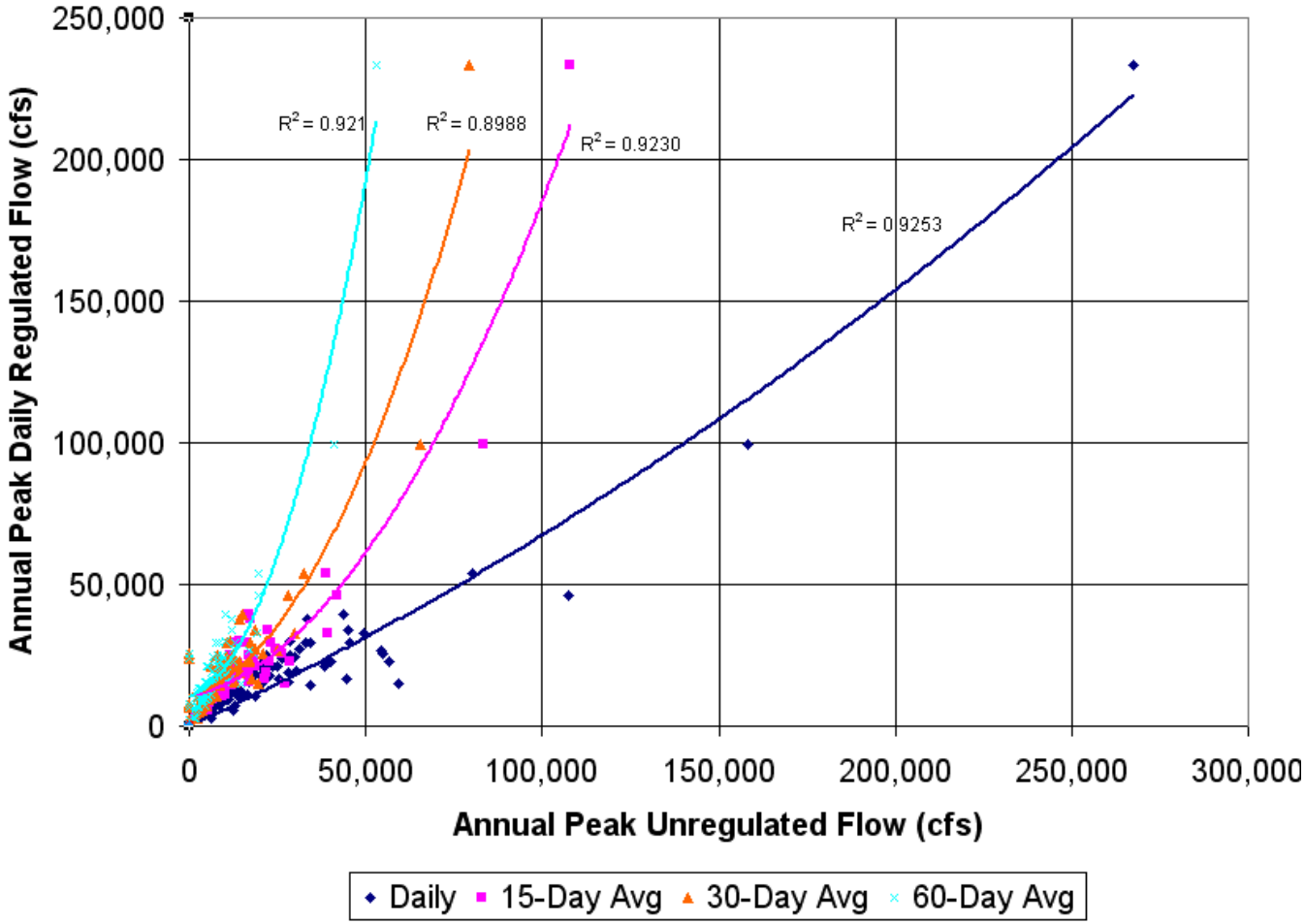

Figure 9. Kansas River Upstream of the Big Blue River Confluence Annual Peak Flow Relationships Unregulated Versus Regulated Flows

2.3.10 Kansas River Downstream of the Big Blue River

The final results of the Kansas River downstream of the Big Blue River Confluence flow frequency analysis for regulated conditions are shown in Table 2-9. Refer to Figure 10 for the flow frequency analysis of the Kansas River downstream of the Big Blue River Confluence. The regulated flow data were plotted and analyzed using a graphical analysis. The graphical analysis of the regulated data was used only for the flow frequency analysis on the Kansas River for flows up to the 5% (1/20) annual chance exceedance event.

The unregulated analysis was conducted similar to the methods used for the Big Blue River and the Kansas River upstream of the confluence. The output has been included in Attachment A. The statistical analysis of the unregulated record on the Kansas River did identify the 1951 peak flow as a high outlier and did not identify any low outliers. The 1951 peak unregulated flow was removed from the systematic record and treated as an historic flood event similar to the Kansas River above the confluence analysis.

The transitional zone of the flow frequency analysis was defined as the point where reservoir regulation of flows begins to be reduced by spillway releases at the lower end of the transition zone and the event where regulated flows are equal to unregulated flows at the high end of the transition zone. The flow frequencies of the transition zone and the point at which the reservoir no longer regulates flood flows must be determined using a comparison of regulated and unregulated data. Figure 11 displays the comparison of the annual peak daily unregulated flows and the time averaged annual peak unregulated flows to the annual peak daily regulated flows for a given year.

The annual peak 15-day average unregulated flows provide a much better correlation to regulated annual peak flows than the annual peak daily flows. The use of the 15-day average unregulated flows is reasonable. Due to the location immediately below the confluence of two major rivers, this is a hybrid between the Kansas River above the Confluence analysis which was governed by the daily peak unregulated flows and the Big Blue River analysis which was governed by the 30-day average unregulated peak flows. Thus, the relationship between unregulated 15-day average peak flows and regulated daily peak flows was used to estimate the regulated peak flows for events greater than the 5% (1/20) annual chance exceedance event. A polynomial fit of the 15-day average unregulated peak flows and daily peak regulated flows was developed. A Log Pearson III frequency analysis of the peak annual 15-day average unregulated flows was analyzed for the period of record. The 15-day average unregulated flows from this frequency analysis were then converted to peak daily regulated flows using the polynomial fit.

This method provided the outflows from the 4% (1/25) annual chance exceedance event to the 0.5% (1/200) annual chance exceedance event. This analysis also identified those events greater in magnitude than the 0.5% (1/200) annual chance exceedance event will most likely behave as unregulated and regulated flows will be equal to unregulated flows. Thus the unregulated peak daily flow analysis results were used for the events equal to or greater than 0.33% (1/300) annual chance exceedance.

Table 2-9. Flow Frequency Results for Kansas River Downstream of Big Blue River

		Section 216 Feasibility Study Results			FEMA(1)
% Annual Chance Exceedance	Return Period	95% Confidence Limit	Frequency Curve	5% Confidence Limit	Estimated Peak Discharge (cfs)
0.133	750	269,700	397200	585,000	NA
0.2	500	248,100	361200	525,700	290,000
0.333	300	222,500	319000	457,400	NA
0.5	200	185,000	258600	361,400	NA
1	100	132,700	177300	236,900	160,000
2	50	94,300	120500	154,000	120,000
4	25	58,000	70000	84,500	NA
10	10	40,100	46800	54,600	56,000

2.3.11 Hydrologic Uncertainty

Hydrologic uncertainties were calculated using the HEC-SSP program. The flow frequency curves, as determined by the methodology in Sections 3.3.1 – 3.3.3 for the three legs of the confluence, were each entered into HEC-SSP as a graphical analysis of data to allow computation of the uncertainty bands about the adopted frequency curve. The methodology employed by the program to calculate uncertainty for graphical analysis is consistent with USACE Technical Letter No. 1110-2-537 *Uncertainty Estimates for Nonanalytic Frequency* Curves (dated 1997). The hydrologic uncertainties for each leg of the confluence have been included in Tables 2-7 through 2-9 and Figures 6, 8, and 10.

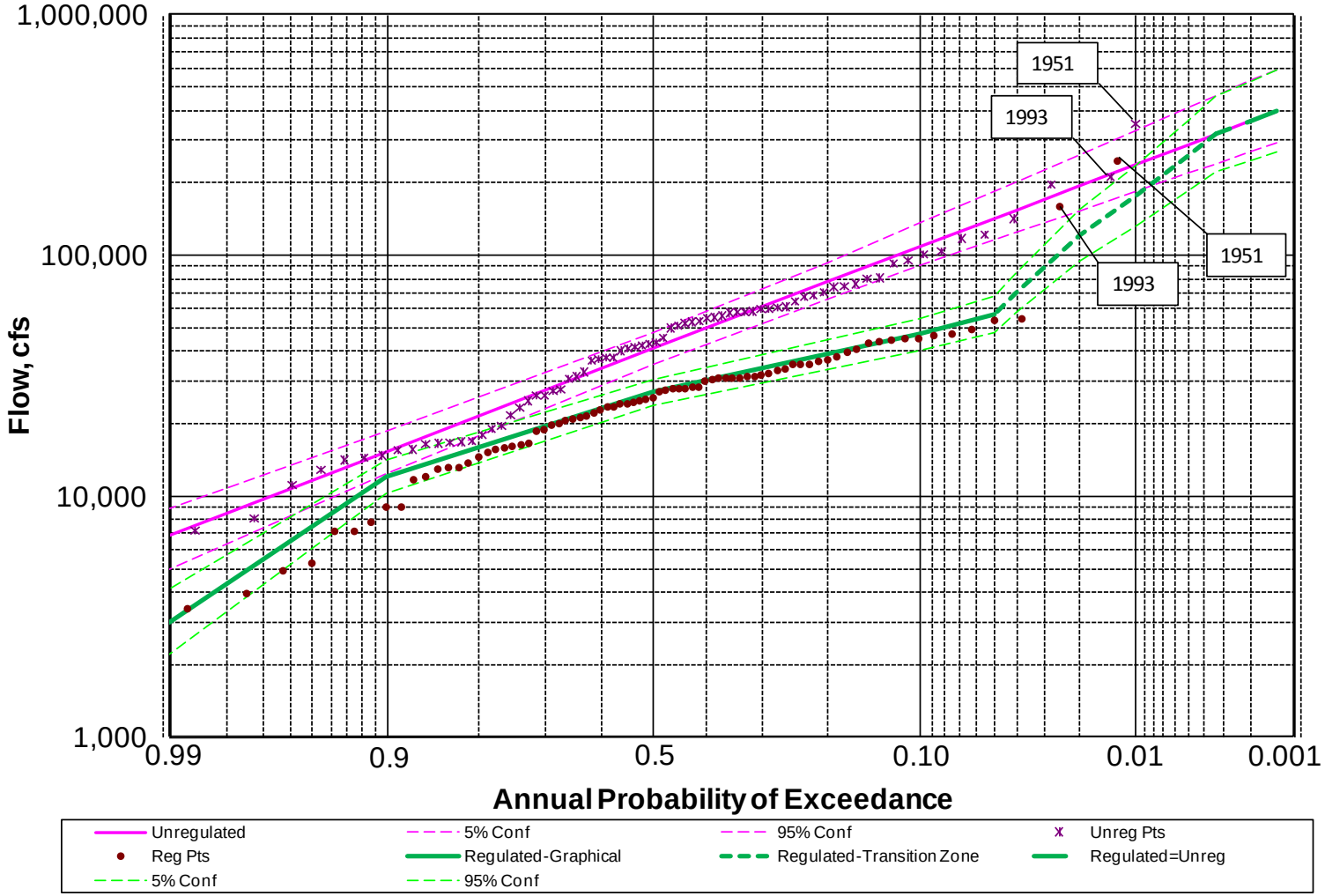

Figure 10. Kansas River Downstream of the Big Blue River Confluence Flow Frequency

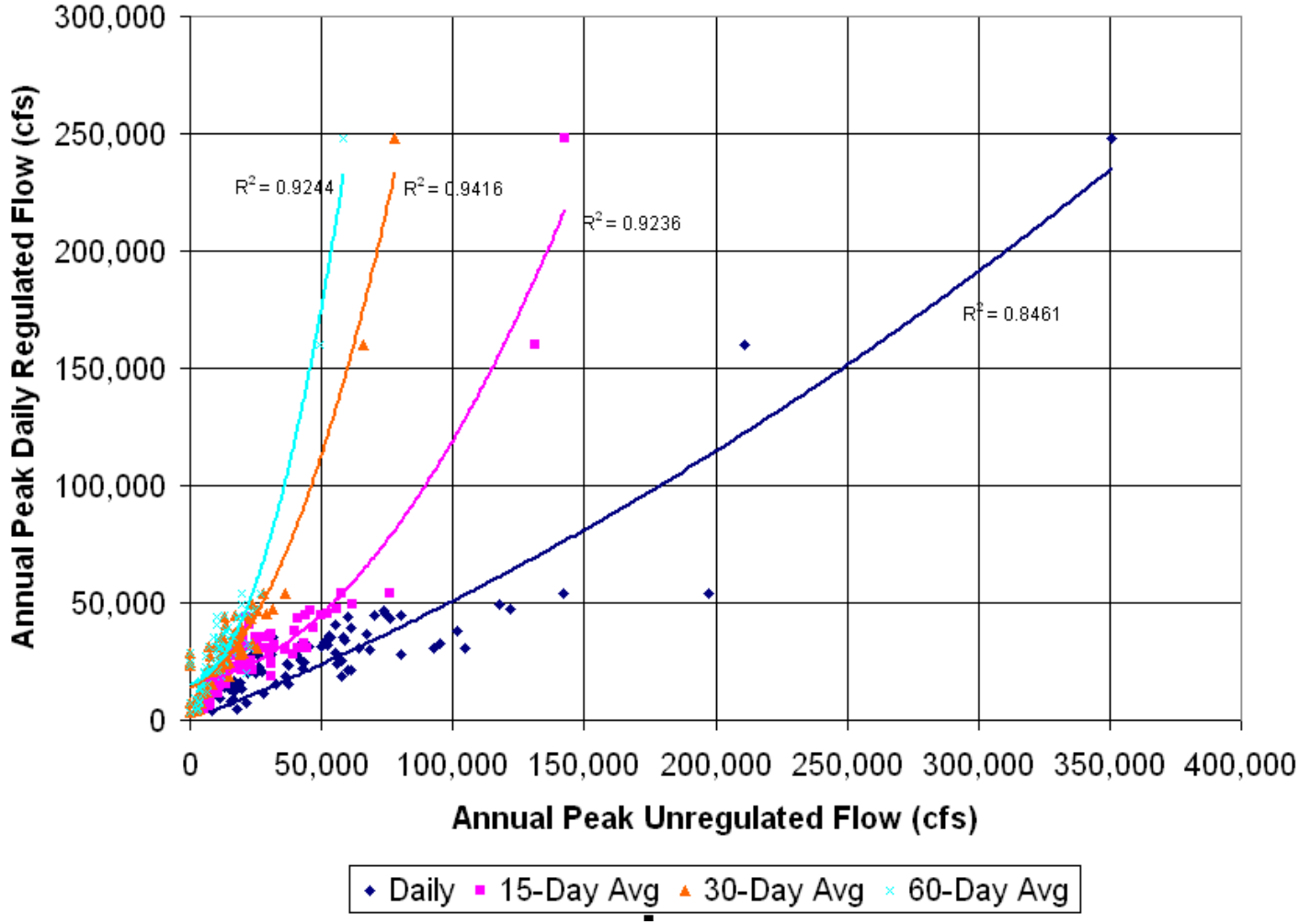

Figure 11. Kansas River Downstream of the Big Blue River Confluence Annual Peak Flow Relationships Unregulated Versus Regulated Flows

2.3.12 Coincident Frequency Analysis

The results of the flow frequency analyses performed in Sections 2.3.8 through 3.3.10 describe the individual frequencies of each of the three legs of the Kansas River and Big Blue River confluence. The individual flow frequency is looking at the historical flow records of each leg of the confluence, disregarding the flow of the remaining two legs of the confluence. However for the hydraulic analysis it is imperative to identify the most likely coincident flow on the remaining legs of the confluence given a flood event on the Big Blue River or the Kansas River.

A review of the records identify that the tributary rivers sometimes respond to a rainfall event that produces a pronounced peak in one hydrograph and only a weak response in the other tributary. At other times, both tributaries respond to the same event, but rarely do both hydrographs peak on the same day. The two largest events for the regulated record of both streams are the peak annual flows of 1993 and 1951. For this study, the streams are being classified as semi-independent. Since each basin has shown the capacity to influence the presence of flooding below the confluence, it has been decided to develop individual relationships between the flow coincidence from each basin and the Kansas River below the confluence.

To develop these relationships between each basin and the Kansas River below the confluence, the peak annual flow from each basin was extracted from the UNET regulated record and plotted with the corresponding flow below the confluence on the same day as the tributary peak flow. Figure 12 displays the plot relating the Big Blue annual peak regulated flows with the corresponding flow below the confluence. A typical method to develop a relationship through this scatter data would be to pass a best fit curve through this data.

However, there is a lot of uncertainty in fitting a curve through this data to predict coincidence out to the 0.133% (1/750) event. It is evident that from the 79 year record that the maximum data point recorded for evaluation is the 1951 event that would represent a recurrence interval between the 1% event and 0.5% event. To provide additional points of reference, for extending the relationship beyond the modeled period of record, the paired flow frequency events from the independent flow frequency analyses for both the Big Blue River and the Kansas River below the confluence were included on the graph. For example, the 0.2% flood event discharge from the Big Blue River was paired with the 0.2% flood event discharge from the lower Kansas River. A best fit curve of the period of record peak annual flow data would lie in line with the paired frequency data.

Thus it was determined that the best predictive relationship between the Big Blue and the Kansas River below the confluence is to utilize the paired flow frequency events. For example, any analysis of the 1% (1/100) annual chance event on the Big Blue River would assume a 1% (1/100) annual chance event on the Kansas River below the confluence. The difference between these two flows would be the coincident flow from Kansas River above the confluence. Table 2-10 shows the most likely coincident Kansas River flows for a Big Blue River flood event.

Table 2-10. Coincident Kansas River Flows for a Big Blue River Flood

% Annual Chance Exceedance	Return Period	Big Blue River Flow (cfs)	Most Likely Kansas River Below Confluence (cfs)	Coincident Kansas River Above Confluence (cfs)
0.133	750	179,400	397,200	217,800
0.2	500	167,000	361,200	194,200
0.333	300	151,700	319,000	167,300
0.5	200	115,800	258,600	142,800
1	100	71,600	177,300	105,700
2	50	46,200	120,500	74,300
4	25	33,000	70,000	37,000
10	10	27,000	46,800	19,800

Figure 13 displays the plot relating the Kansas River annual peak regulated flows with the corresponding flow below the confluence. The same method used in the Big Blue River coincidence analysis was utilized for the Kansas River above the confluence analysis. The Kansas River above the confluence peak flows plotted against the flow in the Kansas River below the confluence on the same day was in general agreement with the paired frequency data except for the 1951 data point. The 1951 data point indicates that for the peak regulated discharge seen above the confluence on the Kansas River (233,500 cfs), the flow below the confluence on the same day was only 248,100 cfs.

This indicates that a large event on the Kansas River above the confluence may have a reduced contribution from the Big Blue River. A best fit curve for this data lies below the paired data. The best fit curve equation was used to estimate the Kansas River Flows below the confluence for a given flow on the Kansas River above the confluence. The difference between these two flows would be the coincident flow from the Big Blue River. Table 2-11 shows the most likely coincident Big Blue River flows for a Kansas River flood event. The coincident frequency analysis results were used in the hydraulic analysis of the existing conditions as described in Section 2.3 of this report.

Table 2-11. Coincident Big Blue River Flows for a Kansas River Flood

% Annual Chance Exceedance	Return Period	Kansas River Above Confluence (cfs)	Most Likely Kansas River Below Confluence (cfs)	Coincident Big Blue River Flow (cfs)
0.133	750	277,400	387,700	110,300
0.2	500	229,700	345,100	115,400
0.333	300	182,200	283,200	101,000
0.5	200	150,800	235,200	84,400
1	100	109,500	167,200	57,700
2	50	79,000	115,900	36,900
4	25	56,400	70,000	13,600
10	10	32,000	39,800	7,800

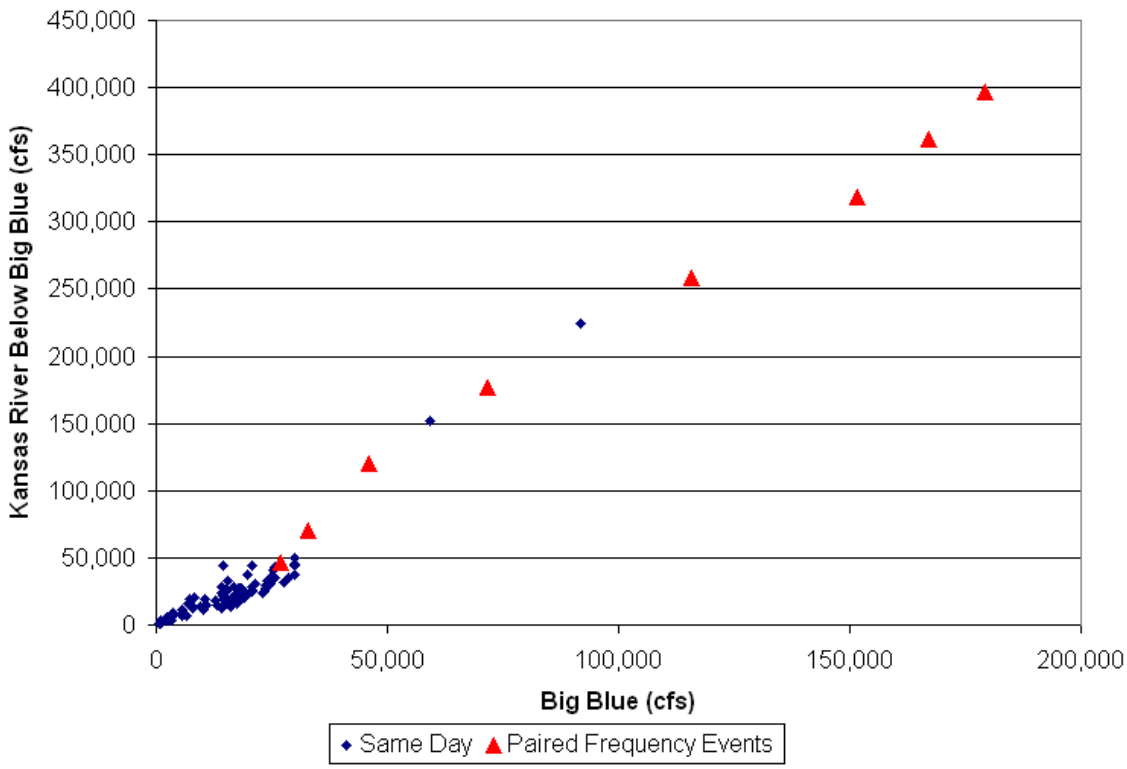

Figure 12. Big Blue River and Kansas River Downstream of the Confluence Coincidence of Regulated Peaks

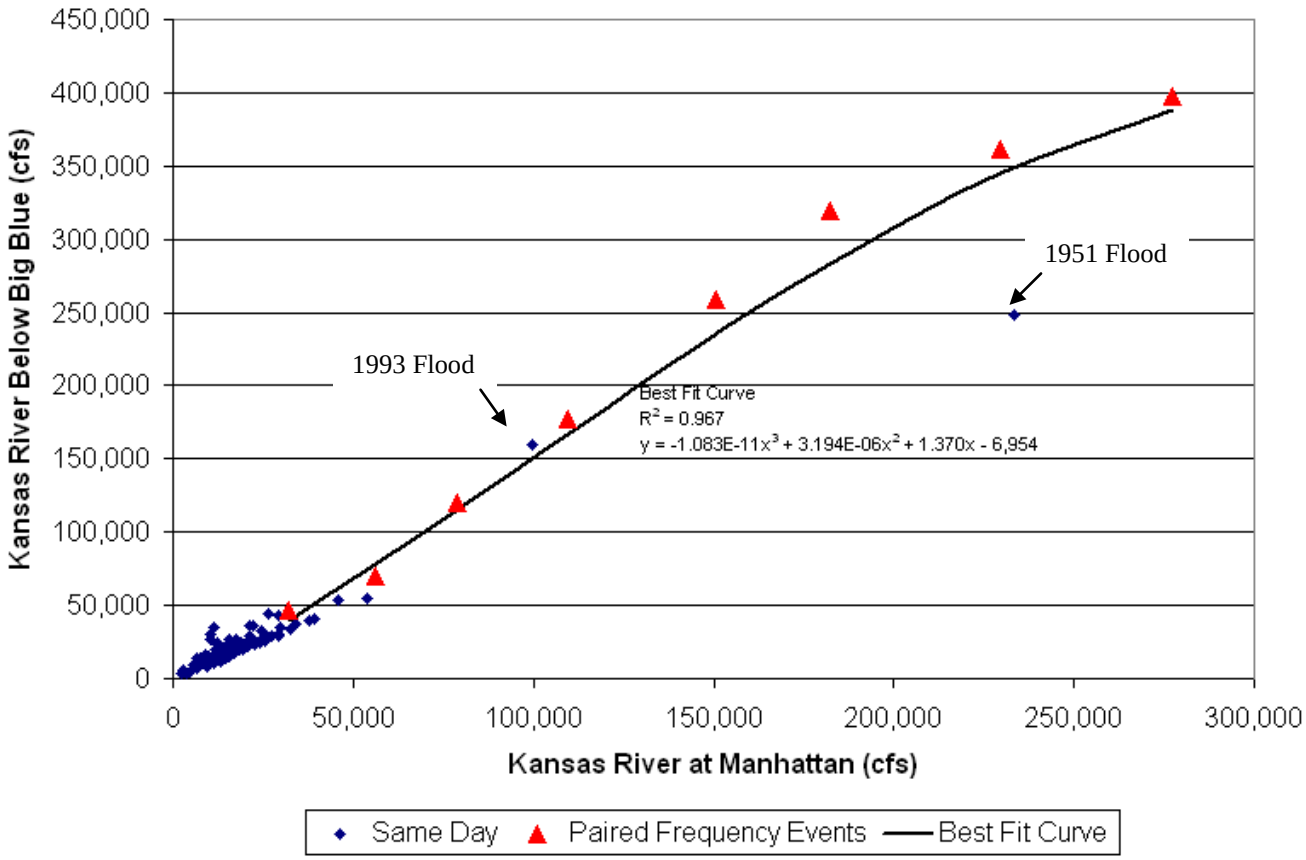

Figure 13. Kansas River Upstream of the Confluence and Kansas River Downstream of the Confluence Coincidence of Regulated Peaks

2.3.13 Hydrologic Period of Record 2014 Sensitivity Analysis

Twelve years of hydrologic data have been observed since the 2002 Kansas Hydrology Study was published. The additional data was evaluated to determine its potential impact to the accuracy of the 2002 hydrologic analysis. Sensitivity analysis was focused on the Big Blue River hydrology as this segment has the lowest existing conditions levee elevations. Figure 14 shows the peak streamflow data recorded by the USGS with the extended 12 years of data on the Big Blue River and the Kansas River. As seen in the figures, no major flood events have occurred since the study's Period of Record (POR) of 1929-2001 was evaluated.

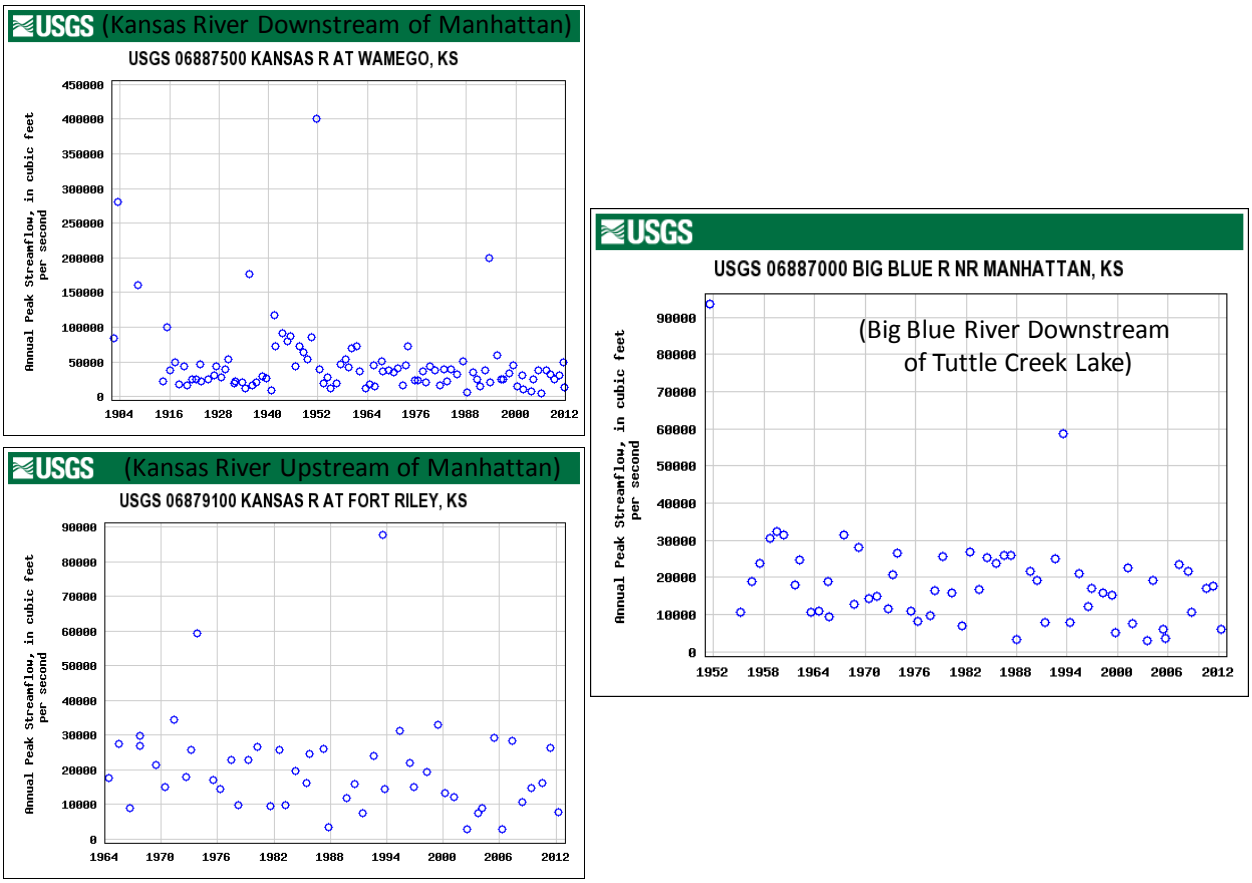

Figure 14. Peak Streamflows Upstream and Downstream of Manhattan, KS (note: Tuttle Creek Reservoir Closed in 1959)

The original study utilized hydrologic routings including reservoir simulations, ungaged inflow simulations along with volumetric statistics to develop a consistent unregulated dataset and convert the data to a consistent regulated dataset and adopted flow frequencies. A simplified approach was adopted to perform the sensitivity analysis. The addition of the unregulated POR on the Big Blue River was estimated with the computed daily inflows to Tuttle Creek Lake from the database maintained by the Kansas City District Water Management section. The 2002-2013 annual peak daily inflows were multiplied by a 10 percent peaking factor to estimate instantaneous unregulated peak streamflows. This extended dataset was incorporated with the original POR into a Bulletin 17B analysis with HEC-SSP software. The regulated peak streamflows at the USGS gage on the Big Blue River were also added to the original POR to evaluate a regulated Bulletin 17B analysis to visualize how the regulated frequency curve may shift with the additional data. Both analyses are presented in figure 15.

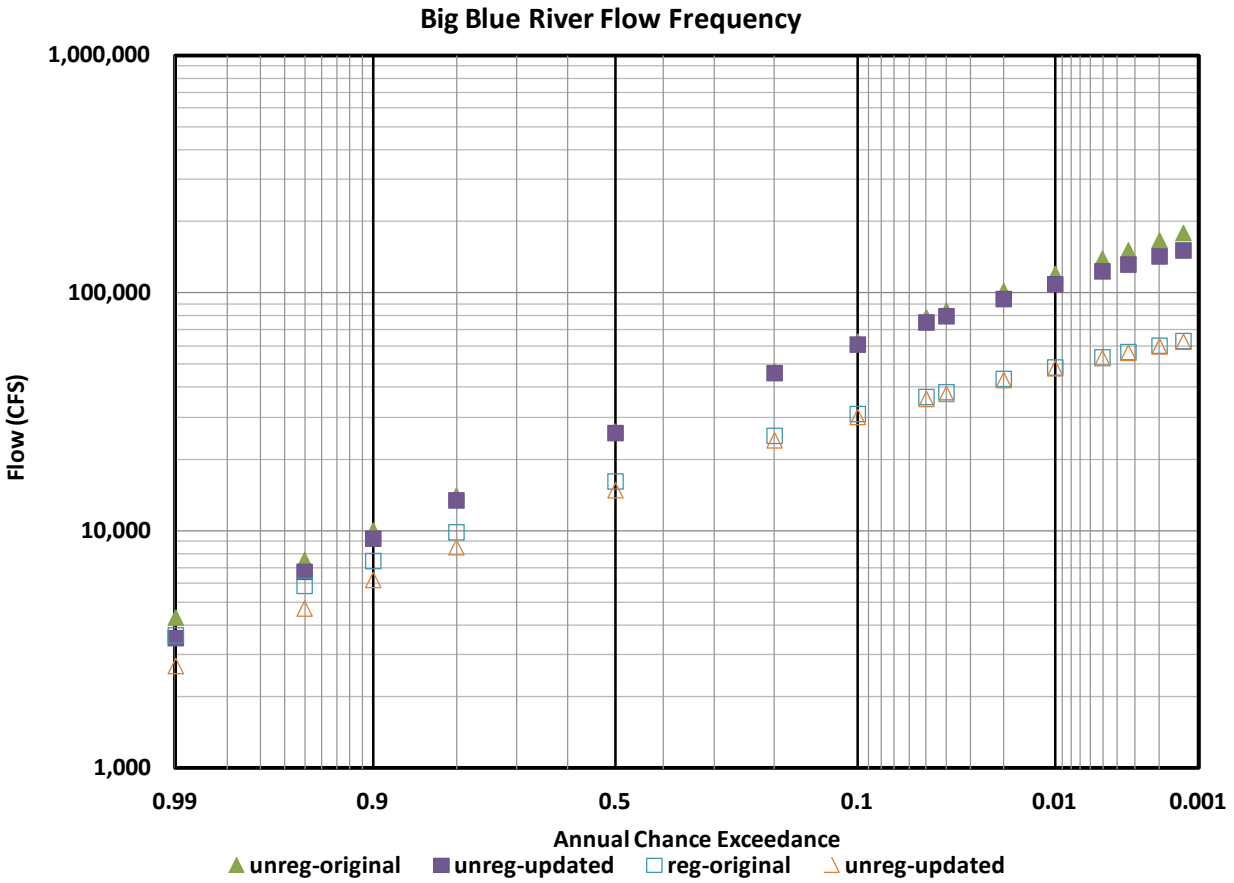

Figure 15. Graphical Results of Regulated and Unregulated Bulletin 17B POR extension

The impact of the POR extension to the hydrologic analysis included a reduction of the 0.33% flow frequency estimate by about 13% in the unregulated data analysis and a much smaller downward impact was observed in the regulated analysis. Regulated data showed a potential larger downward shift for events smaller than a 0.5 ACE. In review of Figure 6, the most likely scenario with the additional data, given that no major events occurred, would be a shift of the 1993 and 1951 data points originally treated as outliers closer to the adopted regulated frequency curve.

Though unregulated peaks do not directly translate to regulated peaks which are required for this study, they provide the apparent largest (downward) change to the hydrology and were considered for a hydraulic sensitivity analysis. After reducing 0.33% event flows by 12.8 percent from 151,700 cfs to 134,124 cfs in the hydraulic model along the Big Blue River based on the shift of the unregulated peak data, the 0.33% event profile was reduced by a maximum of 0.4-feet in the vicinity of the levee system on the Big Blue River. This reduction was considered minimal and did not warrant a major re-design for the study.

2.4 HYDRAULICS

2.4.1 Background

At the peak of the 1993 event, the Big Blue River downstream of Tuttle Creek Lake had flows of approximately 58,800 cfs, and the Kansas River had flows of approximately 100,000 cfs upstream of the Big Blue River. The water surface elevation in the Big Blue came within 3 to 4 feet of the top of the Big Blue River section of the Manhattan levee. These stages were higher than expected based on the previous modeling at the time of design. The concern associated with proximity of the 1993 event to the top of levee at Manhattan prompted this feasibility analysis. The hydraulic analysis of the water surface profiles for multiple frequency events will be critical to the findings of the feasibility study.

2.4.2 Model Development

The basis of the hydraulic analysis was the development of an existing conditions HEC-RAS model. HEC-RAS, version 4.1, as developed by the Hydrologic Engineering Center, U.S. Army Corps of Engineers, was used in this analysis. The model was calibrated to the flood event of 1993 from measured highwater marks and known discharges. The model was further verified with 1951 flood event data. Once the model was calibrated, a series of steady flow water surface profiles were created based on the flood discharges previously discussed in Section 2.3.

2.4.3 Cross-Section Data

In order to create a georeferenced data set for use in the HEC-RAS model, HEC-GeoRAS, an ArcGIS interface containing a set of functions used to process geospatial data, was used. HEC-GeoRAS requires a Triangulated Irregular Network (TIN), or digital representation of the earth's surface, to acquire elevation data. The TIN was created using a total of four sources; Western Air Maps spots and breaks from an aerial photo collection flown in 2001, 2006 LIDAR mapping acquired from Riley County, USGS Seamless 10 meter DEM data, and a field survey completed in July 2006. A map outlining the extents of each data set is shown in Figure 16. The cross sections cut from the TIN were modified manually within HEC-RAS to ensure that the channel cross sections and immediate overbank areas agreed with the field survey conducted in July 2006. The top of levee elevations came from the National Levee Database Top of Levee survey conducted in 2007.

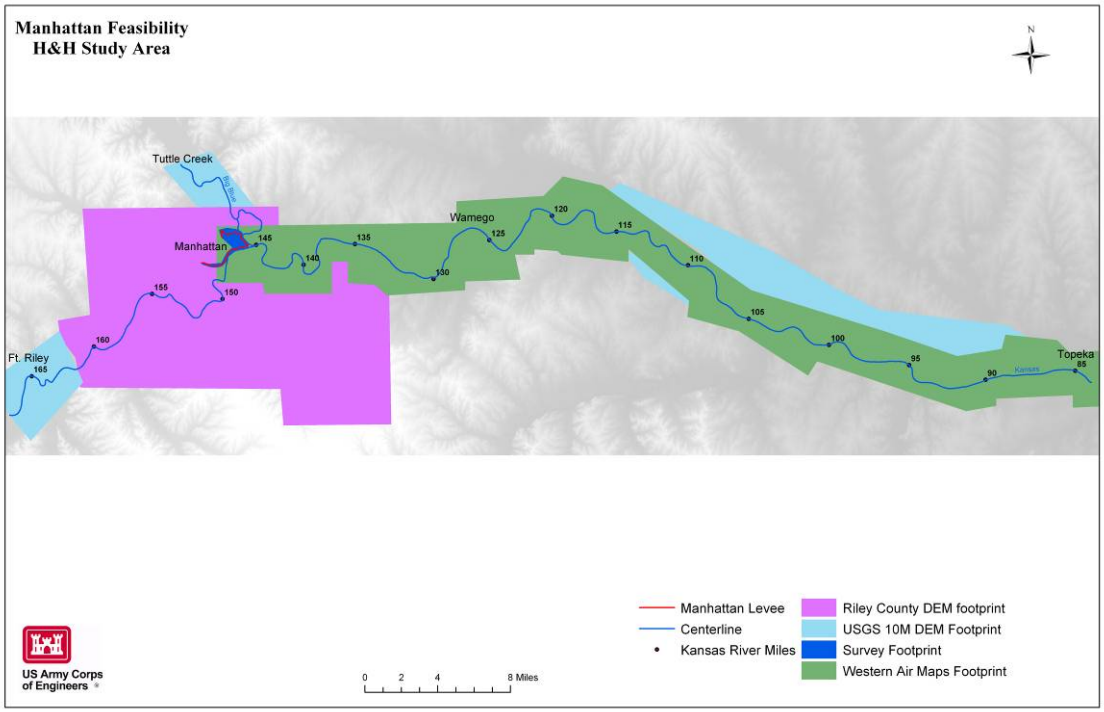

Figure 16. Spatial Extents of Source Data for TIN

River mileages were established for the Big Blue and Kansas Rivers by digitizing a centerline within HEC-GeoRAS with the aid of aerial photography acquired in 2005. River miles were established from the flow line created.

2.4.4 Kansas River Cross-Section Data

The study utilized cross-sections spaced approximately 3-5 miles apart from Topeka (RM 85.7) to Wamego (RM 126.64). The boundary conditions used the Rating Curve at the Topeka Gage (USGS Gage 06889000). The length of the model allowed for the water surface impacts from the boundary condition assumption to be calmed before reaching the study area. Cross-sections were located approximately 1 mile apart on the Kansas River from the Wamego Gage (USGS 06887500) near RM 126.64 to the confluence of the Big Blue River at RM 145.15. There are 12 cross sections on the Kansas River adjacent to the Manhattan Levee, for an average spacing of approximately 0.25 miles (Figure 17). Upstream of the Manhattan Levee the cross section spacing varies from 2 to 3 miles up to the Fort Riley Gage 06879100 location at RM 166.48.

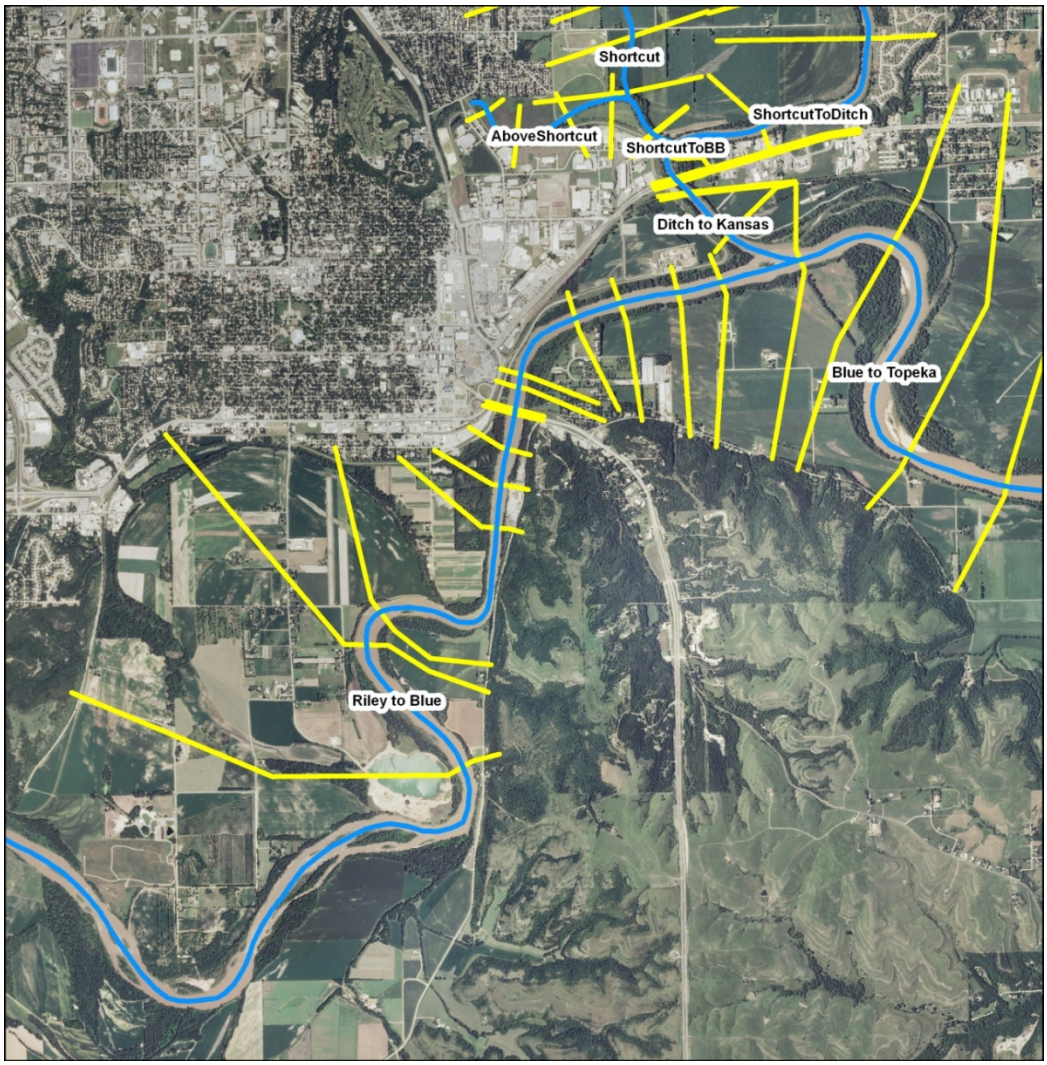

Figure 17. Kansas River Cross Section Spacing through Study Area

2.4.5 Big Blue River Cross-Section Data

The Big Blue River cross sections were also cut from the TIN and in-channel elevation data was used below the river's water surface. The elevation data extracted from the TIN in this reach was developed with 2006 LIDAR based DEMs. These two sources were used to refine the cross-sections on this segment. Degradation ranges used to analyze and track changes in the channel below Tuttle Creek dam are periodically surveyed during Dam Safety Inspections. Station and elevation data from 3 degradation ranges surveyed in 2005 were adapted for some of the model's in-channel data. The field survey from 2006 was also used where available on the Big Blue River. The cross-section layout for the Big Blue River is more complex than the Kansas River segment particularly with the horseshoe bend immediately upstream of the Manhattan Levee.

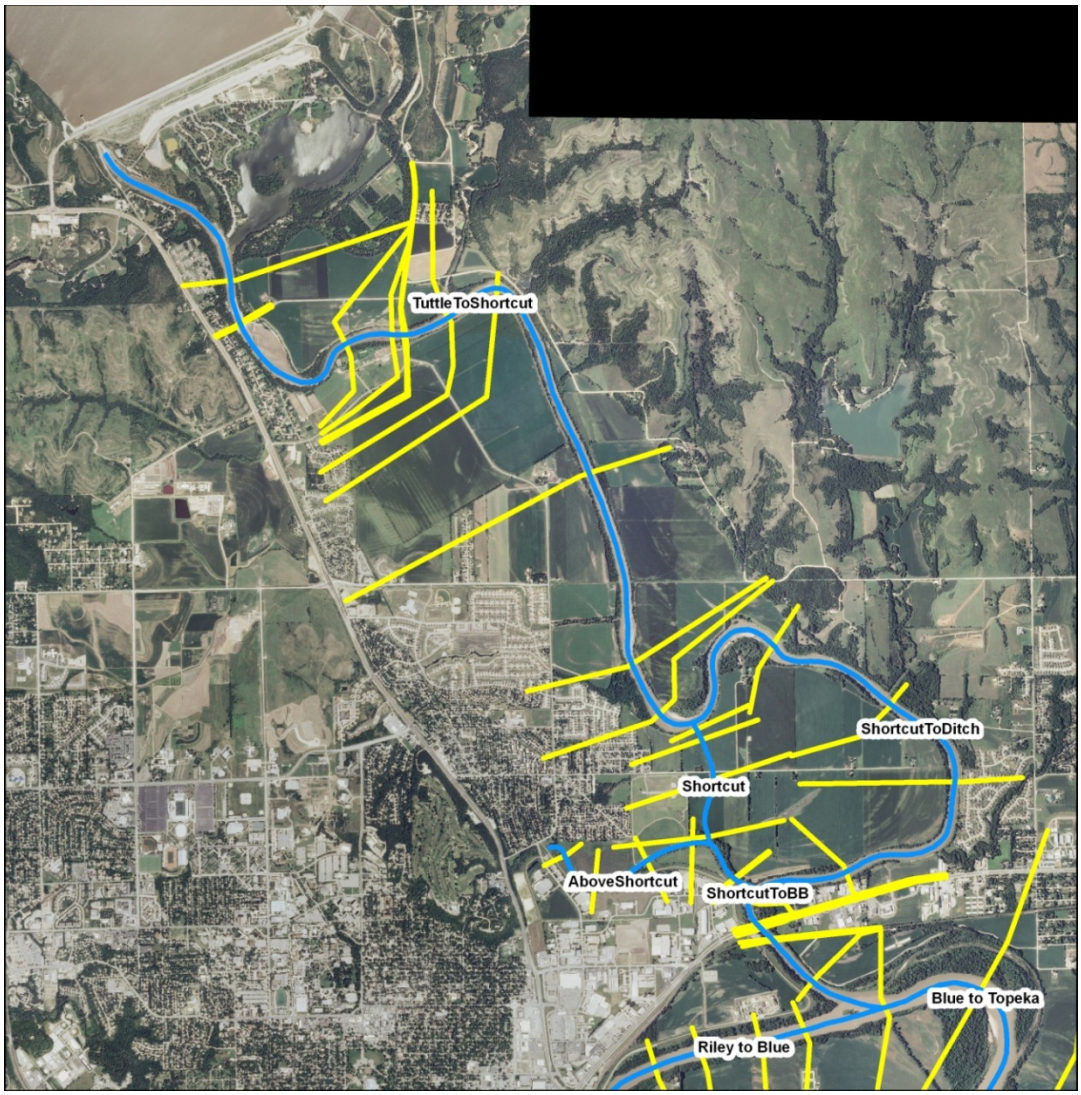

Figure 18. Big Blue River Cross Section Spacing through Study Area

As seen in Figure 18, there are two river reaches in addition to the mainstem that were added to the Big Blue River segment. The drainage ditch shown on the figure just north of the levee was included to better model the water surface on the tieback portion of the levee. Similarly, a shortcut was added at the upstream end of the horseshoe bend extending down to the drainage ditch.

This segment was added to the model after reviewing aerial photographs taken during the 1993 flood. From the photos it is evident that the Big Blue River, during a large flood event, begins spilling into the overbank at this location forming a secondary channel that ties into the drainage ditch and eventually meets back at the main Big Blue River channel.

The Big Blue River shortcut was modeled as a junction on each end of the shortcut and Split Flow Optimization was utilized from within HEC-RAS to balance the flows through the shortcut and around the horseshoe bend on the Big Blue River. For the profiles modeled in the Feasibility study, the 10% (1/10) and 4% (1/25) annual chance events did not produce

shortcut flows while all flows from the 2% (1/50) annual chance event and larger did utilize the shortcut.

2.4.6 Bridge Data

Bridges within the area of Manhattan Kansas were included in the model. A summary of bridges modeled in this study is included in Table 2-12.

Table 2-12. Bridges Included in Manhattan Levee HEC-RAS Model

Bridge	Location	Data Source	Cross-sections
K-18/177	Kansas River	KDOT/HNTB	147.06, 147.03
Union Pacific Railroad	Big Blue River	USACE	0.71, 0.70
U.S.- 24 Eastbound	Big Blue River	KDOT/USACE	0.80, 0.79
U.S.- 24 Westbound	Big Blue River	KDOT/USACE	0.82, 0.81
Dyer Road	Big Blue River	City of Manhattan	7.64, 7.62

The original levee design included the C.R.I.&P. (Chicago Rock Island and Pacific) Railroad bridge over the Kansas River near levee station 83+24 and the State Highway 13 Bridge at levee station 89+40. This railroad bridge has been removed since the construction of the levee. The existing Highway 18/177 Bridge over the Kansas River has replaced the Highway 13 bridge present at the time of original design.

Data for most of the bridges in Table 2-12 were compiled from drawings obtained from the Kansas Department of Transportation along with supplemental survey data collected by the Corps of Engineers survey crew during the course of the levee survey. Data for the bridge at Dyer Road was adapted from the City of Manhattan's Big Blue Predictive Model.

2.4.7 Other Geometric Data

The Manning's n-values used throughout the model were initially assigned based on aerial photography and engineering judgment. The n-values were then modified slightly to calibrate to the known 1993 water surface profile. In the vicinity of the Manhattan Levee, the n-values along the Kansas River channel were typically 0.030, as compared to the original design n-value of 0.025. The overbank n-values along the Kansas River ranged, due to vegetation and land use, from 0.060 to 0.130 for heavily forested areas, as compared to the original design n-value of 0.050. The Big Blue River channel n-values were typically 0.035, as compared to the original design n-value of 0.025, with the overbanks ranging similar to those of the Kansas River. These updated n-values resulted in updated hydraulic profiles that supersede the original design profiles. Ineffective flow areas were entered into the HEC-RAS model to account for areas of quiescent water that do not contribute to the discharge calculations.

Contraction and expansion coefficients were entered into the model according to the guidance distributed by HEC (HEC-RAS 2010). These parameters account for losses

associated with flows expanding and contracting across the flood plain. For areas with gradual transitions between cross sections, contraction and expansion coefficients were set at 0.1 and 0.3, respectively. In the vicinity of bridges, contraction and expansion coefficients were increased to 0.3 and 0.5, respectively. However, due to the close proximity of the bridges adjacent to the levee on the Blue River and the assumption that flow would not fully expand and contract between the bridges, contraction and expansion coefficients were set at 0.1 and 0.3 for the cross sections between these bridges. Contraction and expansion coefficients were increased to 0.3 and 0.5 at the cross section upstream of the westbound Highway 24 Bridge and the cross section downstream of the railroad bridge.

The top of levee elevations used in the model are from the National Levee Database top of levee survey completed in 2007. The interior protected area was modeled as ineffective and not allowed to provide conveyance for flood events large enough to produce overtopping of the levee. This assumption was made based on the uncertainty of levee breach locations and sizes in an overtopping event.

2.4.8 Model Calibration

The HEC-RAS model was calibrated to high water marks recorded during the 1993 flood event. High water marks (HWM) along both the Kansas River and Big Blue River were recorded at each of the freeboard gages along the Manhattan Levee. The freeboard gage measurements came from hourly field recordings from July 25 through July 27, 1993. The model was also calibrated to HWM's recorded by the City of Manhattan at the intersections of Beck Street and Casement Road and also the intersection of Parker Drive and Casement Road.

The Manning's n-values and ineffective flow areas were adjusted to provide the best matching profile in HEC-RAS to the 1993 HWM's (Figure 19 and Table 2-13). It should be noted that the geometric data for the existing Highway 18/177 bridge of the Kansas River was used in the 1993 calibration. The Highway 13 bridge geometry, present at the time of the 1993 flood, was not available for inclusion in the model.

The flows used for the 1993 event calibration were 58,800 cfs in the Big Blue River at Manhattan and 100,000 cfs in the Kansas River at Manhattan. The Big Blue River flow was recorded at the USGS Manhattan Gage. The Kansas River flow was taken from the UNET Regulated flow modeling for the 1993 flood event.

Table 2-13. HEC-RAS Model Calibration to 1993 High Water Marks

	River Mile	Manhattan Levee Station (ft)	1993 HWM Elev (NAVD88 ft)	1993 HEC-RAS Elev (NAVD88 ft)	Delta (ft)
Kansas River	147.51	62+20	1018.5	1018.3	-0.2
	147.15	81+25	1017.3	1016.8	-0.5
	147.01	89+83	1015.9	1016.4	0.5
	146.76	101+26	1015.8	1015.5	-0.3
	146.44	120+00	1014.2	1014.5	0.3
	146.06	140+00	1013.7	1013.4	-0.3
	145.67	163+00	1012.4	1012.4	0.0
Blue River	0.40	175+00	1011.9	1012.0	0.1
	1.00	211+00	1013.6	1013.3	-0.3
Ditch	N/A	234+00	1014.7	1013.9	-0.8
	N/A	269+50	1014.3	1015.2	0.9
Blue River	4.64	N/A	1017.6	1016.9	-0.7
	7.63	N/A	1022.0	1021.9	-0.1

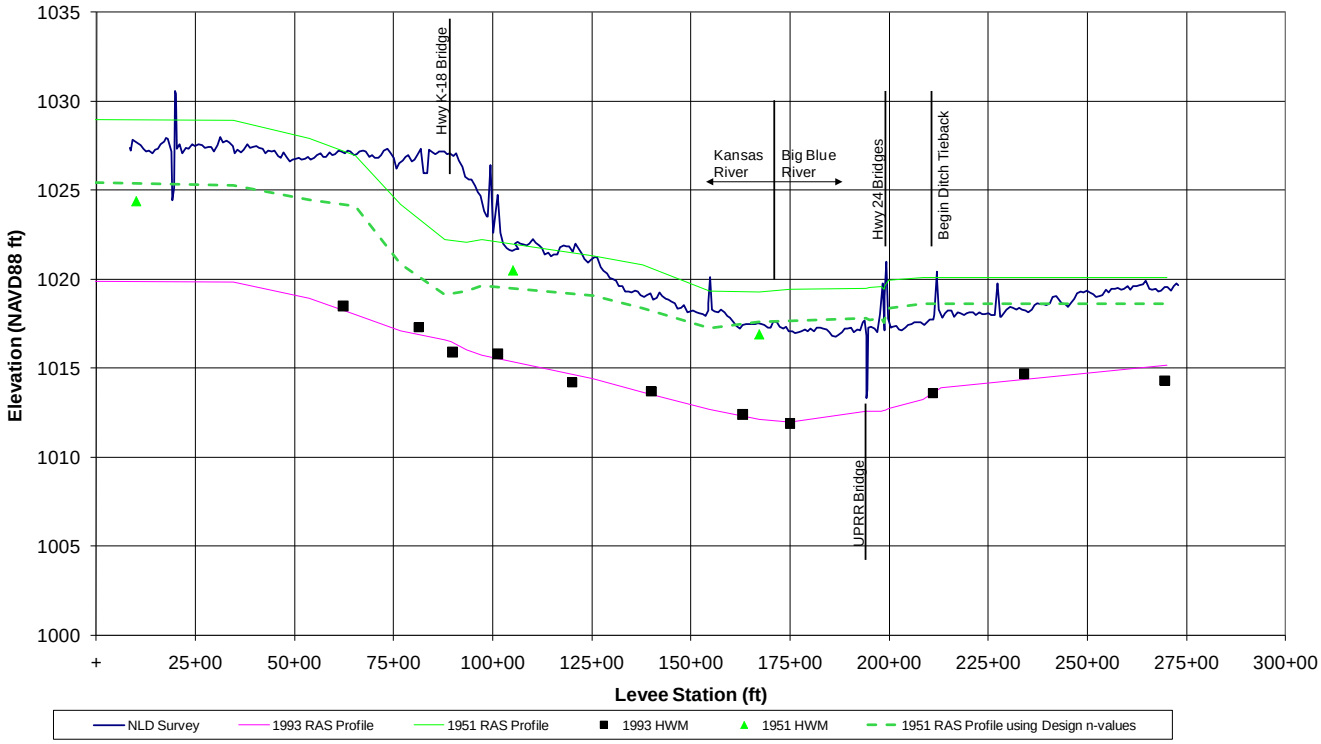

Figure 19. HEC-RAS Model Calibration to the 1993 Flood Event and Check of 1951 Flood Event at the Manhattan Levee

The 1951 flood profile was also modeled to compare the current model to the flood of record profiles. High water marks taken from the Manhattan Gage and Manhattan Levee Design Memorandum were used as comparison points (Figure 19). The 1951 flows were taken from USGS records. This Kansas River flow of 298,000 cfs recorded at the Ogden Gage was taken from the USGS Fact Sheet 041-01, *The 1951 Floods in Kansas Revisited*. The Big Blue River flow of 93,400 cfs was recorded at the USGS Manhattan Gage. The 1951 flood event was modeled with the Manhattan Levee removed from the Existing Conditions model because the levee was not present at the time of the 1951 flood. At the USGS Manhattan Big Blue River gage, the modeled stage matched the 1951 high water mark within 1 foot. The 1951 event was not used for calibration but for a check of the validity of the model for extreme flood events.

The current HEC-RAS model profile is above the 1951 high water marks in all locations, ranging from 2.4 feet above at the downstream end of the levee to 4.5 feet above at the upstream end of the unit. It is expected that the existing conditions model would over estimate the 1951 water surface elevations. The hydraulic analysis in Design Memorandum 1-A, dated 1960, the design Manning's n-values used were 0.025 for the channel and 0.050 for the overbank areas throughout the study reach.

This would indicate that at the time of the 1951 flood event the overbanks and channel were mostly free of woody vegetation. The overbanks along the Kansas River adjacent to and opposite of the Manhattan Levee are now covered with mature tree growth, resulting in a much higher Manning's n-values for the existing conditions. This additional roughness causes the existing conditions model to gradually diverge from the 1951 high water marks from downstream to upstream along the study area.

Figure 19 displays the 1951 water surface profile using current geometry and the original design n-values. The 1951 water surface profile computed using design n-values is within 0.9-feet of the Kansas River high water marks along the Manhattan Levee reach. This would indicate that the changed channel and overbank conditions since the 1951 flood have contributed to increased water surface profiles. Thus, the model seems to provide reasonable results when checking 1951 high water marks to the HEC-RAS estimate of the 1951 flood using existing conditions geometry and n-values.

2.4.9 Existing Conditions Hydraulic Profiles

Once the model was calibrated, existing conditions water surface profiles were generated for the 10% (1/10), 4% (1/25), 2% (1/50), 1% (1/100), 0.5% (1/200), 0.33% (1/300), 0.2% (1/500), and 0.133% (1/750) chance of exceedance flood events. The starting water surface elevations for each of the profiles were calculated using the rating curve at Topeka.

Two scenarios were investigated for each flood event profile. First, a Big Blue River controlled flood was modeled where the Big Blue River was experiencing flooding as analyzed in Section 3.3.1. The coincident flow on the Kansas River for each flood

event would be as determined in the Coincident Frequency Analysis discussed in section 3.3.5 and tabulated in Table 2-10.

The second scenario is a Kansas River controlled flood where the Kansas River was experiencing flooding as analyzed in Section 3.3.2. The coincident flow on the Big Blue River for each flood event would be as determined in the Coincident Frequency Analysis discussed in section 3.3.5 and tabulated in Table 2-11. The results of each of these two scenarios were compared at each cross-section, with the higher water surface elevation being selected as the controlling elevation for a given frequency event. This approach ensures that the range of flow distributions are accounted for and the most critical scenario is evaluated at the confluence. Figure 20 displays the Existing Conditions hydraulic profiles for the studied events plotted against the top of levee elevation. Figure 20 also includes the existing condition profile for the original Manhattan Levee design flows of 220,000 cfs and 110,000 cfs on the Kansas River and Big Blue River, respectively. Attachment B contains the HEC-RAS output for the study reach. Refer to Section 2.5 for conclusions based on this analysis.

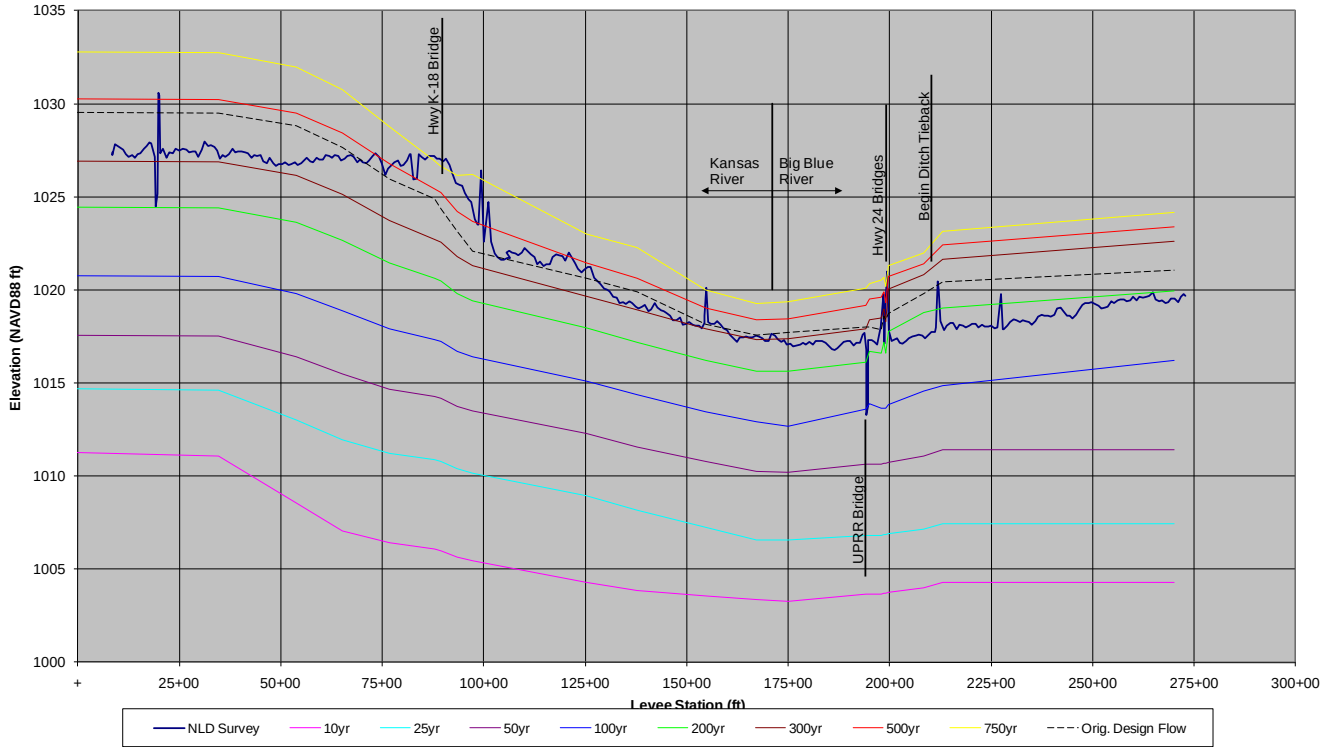

Figure 20. HEC-RAS Existing Conditions Water Surface Profiles at Manhattan Levee

2.4.10 Hydraulic 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, and waves. 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 gage readings, and interpretation of the success of model adjustments following traditional procedures presented in USACE Engineering Manual (EM) No. 1110-2-1619 (EM 1996).

Stage uncertainty is expressed in the Risk Based Analysis as a standard deviation (in feet). To obtain a total standard deviation the formula below was applied.

$$\text{Total Standard Deviation} = \text{SQRT}\left((S_{natural})^2 + (S_{model})^2\right)$$

where
$S_{natural}$ = standard deviation based on historical data and gage readings
S_{model} = standard deviation based on mapping detail and reliability of estimating Manning's n values

$S_{natural}$, is calculated by comparing observed data with the latest rating curve at the gage in the project reach. There were no gages available within the project reach, so the Wamego gage was chosen to represent the Kansas River and the Manhattan gage was chosen to represent the Big Blue River. To avoid potential problems due to shifts in the rating curve over time, only observed data going back to 1990 were used. Figure 21 shows this observed data compared to the current rating curve at the Wamego Gage on the Kansas River. Figure 22 shows this observed data compared to the current rating curve at the Manhattan Gage on the Big Blue River. Only data for bank full discharges and greater were analyzed. The formula below is used to calculate $S_{natural}$.

$$S_{natural} = \text{SQRT}\left((\Sigma(X-M)^2)/(N-1)\right)$$

where
X = stage corresponding to measured Q
M = best fit curve estimate of stage corresponding to Q
N = number of stage-discharge observations in the range being analyzed

The standard deviation based on historical data and gage readings, $S_{natural}$, was computed as 0.4 feet at the Wamego Gage and 1.1 feet at the Manhattan Gage. The Wamego Gage standard deviation was applied to the computed water surface elevations on the Kansas River both above and below the confluence. The Manhattan Gage standard deviation was applied to the computed water surface elevations on the Big Blue River.

The second component in quantifying standard deviation is S_{model}. S_{model} is obtained by estimating the confidence in the cross-section data from topographic mapping efforts and in estimating the reliability of the Manning's n-value. Table 5-2 in EM 1110-2-1619 quantifies a S_{model}, based on these two factors. A standard deviation

of 0.7 foot was chosen since the cross-sections were based on current aerial mapping (refer to Figure 16) and the Manning's n-values were assumed to be "fairly" reliable. It is noted that the most upstream portion of the model was based on larger scale DEM terrain data as opposed to aerial mapping, but since these areas are not the primary focus of this study the standard deviation in the study area was used throughout.

Once $S_{natural}$ and S_{model} are known, a total standard deviation can be computed. Following traditional procedures presented in EM 1110-2-1619, a total standard deviation of 0.8 feet was computed for the Kansas River and a total standard deviation of 1.3 feet was computed for the Big Blue River. It is recommended that the calculated total standard deviations be increased to 1.0 feet and 1.5 feet for use in risk analysis for the Kansas River and Big Blue River, respectively. These recommended increases account for the fact that the hydraulic uncertainties were calculated for USGS gage locations that were near to, but not immediately within the Manhattan Levee study area.

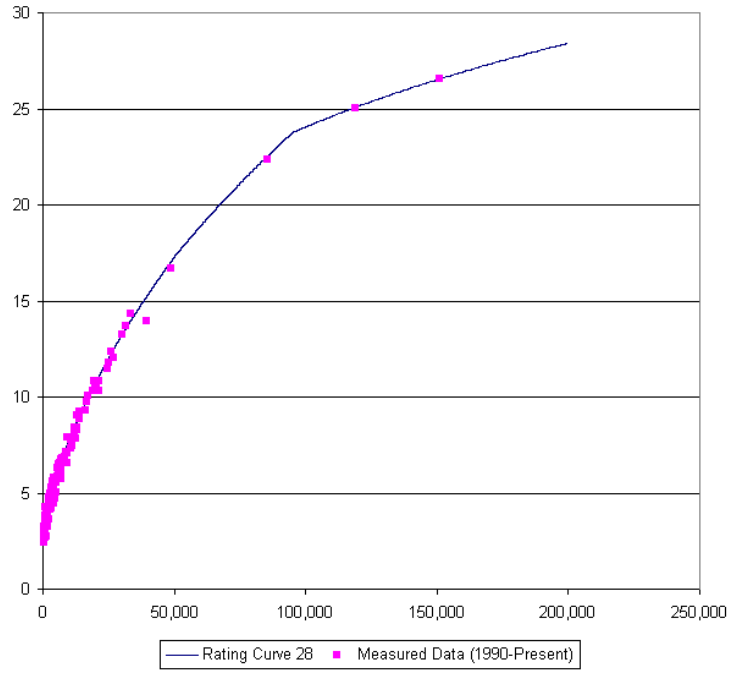

Figure 21. Comparison of Rating Curve #28 and Observed Measurements at Wamego Gage on the Kansas River

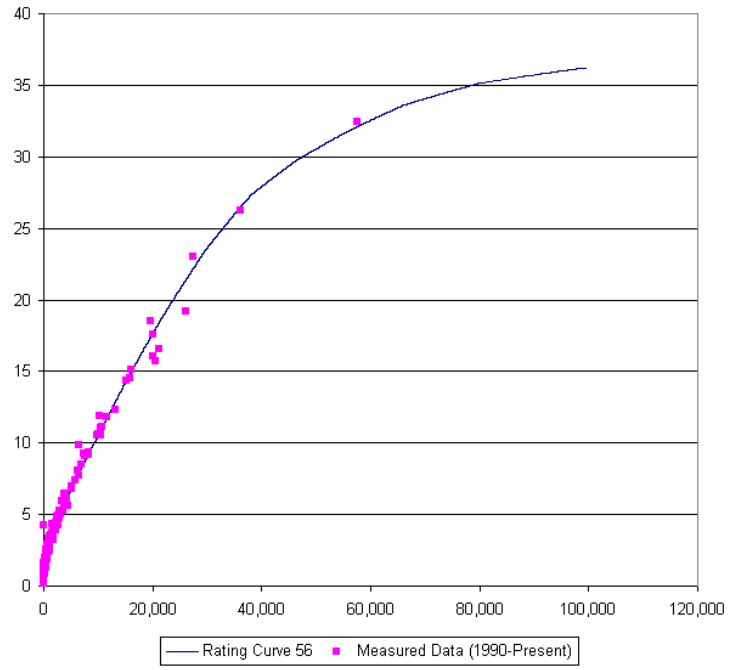

Figure 22. Comparison of Rating Curve #56 and Observed Measurements at Manhattan Gage on Big Blue River

2.5 CONCLUSIONS

2.5.1 River Discharges

This evaluation uses output from a calibrated UNET model of the Kansas River and Big Blue River near Manhattan Kansas. The UNET model, a one-dimensional unsteady river model, was run for a period of record that spanned from Water Year 1929 to Water Year 2002. Between years 2002-2012, no significantly large discharges occurred that would affect the flood flow frequency results less remote than the 10% (1/10) annual chance exceedance event. The UNET model output included simulated daily peak discharges for the period of record under two conditions:

1) Synthetic natural period of record (Unregulated)

2) Synthetic regulated period of record (Regulated)

The simulated discharge results for both rivers were used to develop the maximum yearly discharge over the period of record. The maximum yearly discharges were then used to develop the flow frequency relations for both the unregulated and regulated discharges. Tables 2-7 and 2-8 summarize the regulated discharge results compared to existing FEMA FIS discharges for the Kansas River and Big Blue River, respectively.

For the Kansas River, the flow frequency results indicate that:

1) The 1% (1/100) exceedance event was significantly less than the FEMA FIS 1% (1/100) discharge.

2) The 0.2% (1/500) exceedance event the flow frequency results were less than the FEMA FIS 0.2% (1/500) discharge.

3) The peak flow for the 1993 event on the Kansas River at Manhattan (~100,000 cfs) is estimated as a 1.3% (1/75) exceedance event.

For the Big Blue River, the flow frequency results indicate that:

1) The 1% (1/100) exceedance event was significantly higher than the FEMA FIS 1% (1/100) discharge.

2) The 0.2% (1/500) exceedance event was significantly higher than the FEMA FIS 0.2% (1/500) discharge.

3) The 1993 event on the Big Blue River (59,500 cfs) is estimated as a 1.5% (1/67) exceedance event.

2.5.2 River Stages

The calibrated HEC-RAS model of the Kansas River and Big Blue River near Manhattan, Kansas was used to model existing conditions water surface profiles. The model results displayed in Figure 20 indicate that in the existing conditions without freeboard considerations:

1) The Kansas River portion of the Manhattan Levee is approximately at the 0.33% (1/300) exceedance event.

2) The Big Blue River portion of the levee is overtopped between the 1% (1/100) and 0.5% (1/200) exceedance event.

3) The Manhattan Levee unit is overtopped by the original design flow event.

Figures 23 and 24 show the stage frequency water surface profiles compared to the 2005 FEMA FIS profiles for the 10% (1/10), 2% (1/50), 1% (1/100), and 0.2% (1/500) exceedance events without freeboard considerations. The comparison of the existing conditions HEC-RAS model and the 2005 FEMA FIS profiles reveals that for the Kansas River:

1) The profile for the 1% (1/100) exceedance event is approximately 3 feet higher than the FIS 1% (1/100) profile near the confluence.

2) The profile for the 1% (1/100) exceedance event reduces to only 1 foot above the FIS profile from river mile 146.5 to the upstream end of the levee unit.

The comparison of the existing conditions HEC-RAS model and the 2005 FEMA FIS profiles reveals that for the Big Blue River:

1) The profile for the 1% (1/100) exceedance event ranges from 3 to 4 feet higher than the FIS 1% (1/100) profile adjacent to the Manhattan Levee.

2) The profile for the 1% (1/100) exceedance event spreads from 4 feet higher than the FIS profile at the upstream end of the Manhattan Levee to approximately 6.5 feet above the FIS profile at Hackberry Avenue.

3) Upstream of Hackberry Avenue the 1% (1/100) exceedance event profile is approximately 5 to 6 feet above the published FIS 1% (1/100) profile.

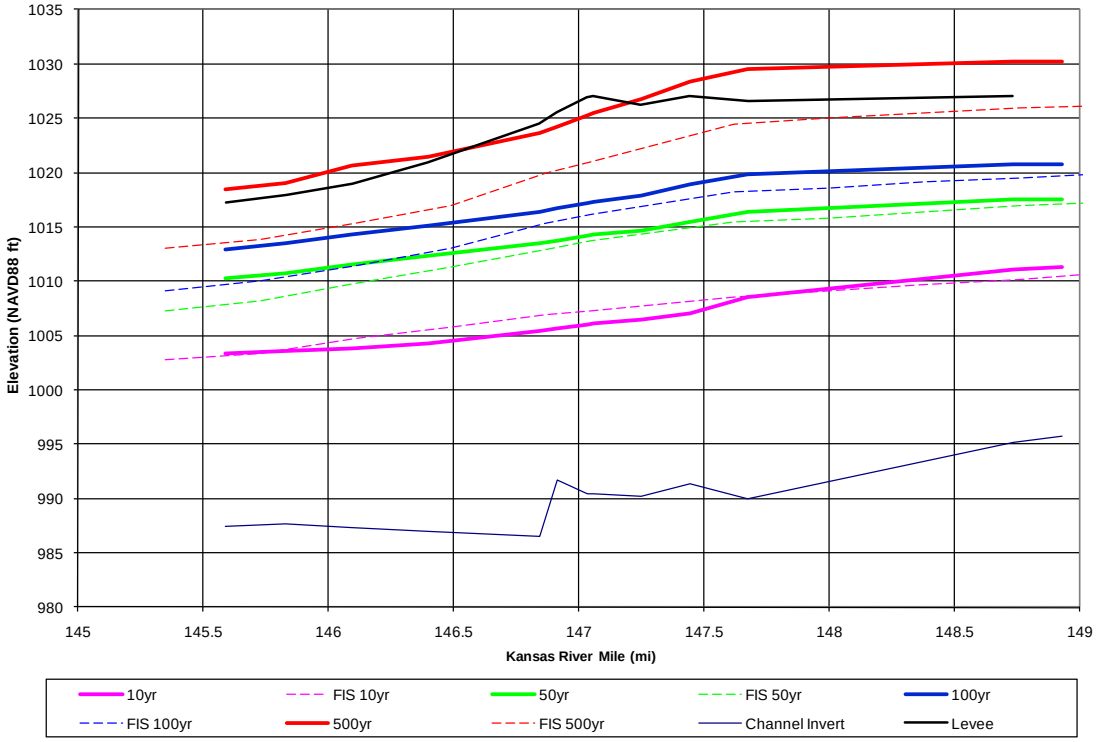

Figure 23. Comparison of Kansas River Flood Profiles to 2005 FEMA FIS Profiles

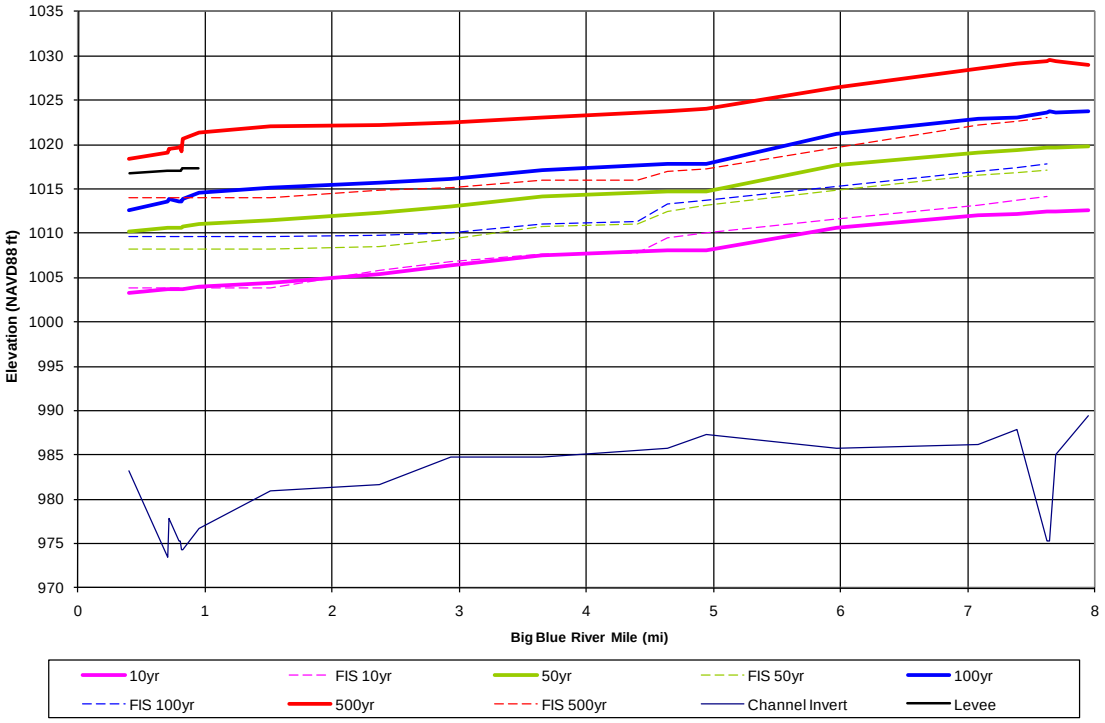

Figure 24. Comparison of Big Blue River Flood Profiles to 2005 FEMA FIS Profiles

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PART 3 – Geotechnical

Table of Contents

3.1 INTRODUCTION 3-1

3.2 DESCRIPTION OF EXISTING LEVEE UNIT 3-1
3.2.1 Levee Description.....3-1
3.2.2 Historical Flood Events.....3-2
3.2.3 Regional Geology.....3-2
3.2.4 Site Geology.....3-3
3.2.5 Subsurface Conditions 3-3

3.3 UNDERSEEPAGE ANALYSIS.....3-4
3.3.1 Methodology.....3-4
3.3.2 Deterministic Analysis 3-5
3.3.3 Probabilistic Analysis.....3-5

3.4 PROBABILISTICUNDERSEEPAGE ANALYSIS RESULTS.....3-7

3.5 SLOPE STABILITY ANALYSIS 3-8
3.5.1 Probabilistic Slope Stability Analysis 3-9
3.5.2 Probabilistic Stability Results.....3-10

3.6 SUMMARY 3-13

3.7 REFERENCES 3-13

List of Tables

Table 3-1 Permeability Ratios for Blanket Material Based on Material Type 3-4
Table 3-2 Observations of Seepage Conditions during 1952 Flooding on the Missouri River 3-6
Table 3-3 Underseepage Parameters Coefficients of Variation (COV).....3-7
Table 3-4 Underseepage Reliability Results 3-8
Table 3-5 Material Properties for Slope Stability Analyses.....3-10
Table 3-6 Steady State Slope Stability Analysis Results.....3-12
Table 3-7 Transient Seepage Analysis Results, 1993 Flood, and Tuttle Creek Emergency Release.....3-12

3.1 INTRODUCTION

This Appendix presents the results of geotechnical evaluation of the Manhattan levee in Manhattan, Kansas. The purpose of the geotechnical evaluation was to determine the reliability of the existing Manhattan levee unit for economic modeling. The evaluations were performed in general accordance with the USACE Engineering Technical Letter (ETL) 1110-2-556 "Risk-Based Analysis in Geotechnical Engineering for Support of Planning Studies." The results of the analyses are only intended for use in economic modeling. This is the first phase of the feasibility study where the existing condition of the levee unit is documented. The second phase of the feasibility study will evaluate ways to improve levee reliability with respect to all failure modes.

The evaluation started with a thorough review of Record Drawings, design memorandums, O&M Manuals, and recent investigations for levee accreditation. From the document review, the subsurface conditions and levee geometry were established. All elevations used in the geotechnical feasibility study are NGVD 1929 to be consistent with historical levee documents.

Geotechnical evaluation of the Manhattan levee consisted of underseepage and slope stability analysis for steady state flood conditions because they are the most likely controlling failure modes. Initial analyses were typically performed deterministically to identify critical areas of the levee. Final analyses were performed probabilistically to develop probability of failure relations with river elevation. The geotechnical probability of failure relations, or hazard curves, will be combined with other failure modes (structural, hydrologic, etc) for economic modeling.

The AE firm CDM was contracted by the Corps to define impending failure as it relates to the Manhattan levee and perform probability of failure analysis at critical levee reaches identified by the different failure modes. This study refines the work initiated by CDM using new subsurface information from borings drilled along the Manhattan levee alignment by AMEC as part of FEMA levee accreditation.

3.2 DESCRIPTION OF EXISTING LEVEE UNIT

3.2.1 Levee Description

The Manhattan levee is located west and north of the confluence of the Big Blue River and the Kansas River in Manhattan, Kansas. The levee consists of protection along Wildcat Creek, the Kansas River, and the Big Blue River. A plan view of the levee system is shown in **Appendix A.1**. The levee embankment begins at 8+50 and ends at station 272+85. The levee starts north of Wildcat Creek and is roughly aligned with 15th Street in Manhattan, Kansas. The levee follows the alignment of Wildcat Creek from Station 8+50 to Station 35+00, where it begins to parallel Pottawatomie Avenue to station 60+00. The levee alignment then turns to the northeast and turns north at 80+00 to align with the Kansas River. The alignment with the Kansas River continues to the confluence with the Big Blue River at approximately Station 173+00.

From the confluence, the levee turns towards the northwest, aligning with the Big Blue River until Station 209+00, where it turns further to the west and splits off from the Big Blue River. The levee continues in a west-northwest direction and aligns parallel to an existing drainage channel to its end at Station 272+85.

The Manhattan levee is approximately 28,850 feet long. The levee was typically constructed with a 10-foot crown width and three horizontal to one vertical (3H: 1V) embankment slopes. The embankment is a zoned embankment. The landside portion consists of a large “random fill” section. The random fill was obtained from channel improvement excavations and other local borrow areas. The random fill ranges from silty sands to lean clays. The riverside section is a minimum 3-foot-thick “impervious fill” section that is assumed to consist of locally available low-permeability lean and/or fat clays. The minimum Factor of Safety (FS) used in the original design for underseepage and stability was 1.1 (with water at top of levee) and 1.5 (with 3 ft of freeboard), respectively. Underseepage control was provided where needed by underseepage berms and relief wells. Six relief wells were originally installed at the Poyntz Avenue pump station inlet ditch. However, only 4 relief wells currently exist along the inlet ditch. On the other hand, stability berms were constructed in five levee reaches. Stability berms were placed in the following reaches: 1) between Station 77+00 and 81+25, 2) between Station 112+50 to 140+00, 3) between Station 160+50 and 179+00, 4) between Station 194+20 and 269+80, and 5) between Station 269+80 and 272+00.

The original design water surface for the Manhattan levee is 3 feet below the levee crest.

3.2.2 Historical Flood Events

The City of Manhattan has a long history of floods during periods of heavy sustained precipitation. The largest recorded floods occurred in 1844, 1903, 1908, 1951, and 1993. The great flood of 1951 and the Federal Flood Control Act of 1954 led to the construction of the Manhattan Levee in the 1960s. The largest recorded flood since levee construction is the 1993 flood. How the levee performed, from a geotechnical perspective is not well documented for the 1993 flood. There have been verbal accounts from memory by the sponsor indicating seepage and pin boils were present between stations 130+00 and 150+00. Aerial photography from the 1993 flood also indicates water ponded on the landside of the levee in the Kansas River-Big Blue River confluence. It is assumed that at least a portion of this is seepage water. Generally speaking, the 1993 flood performance is not documented well enough to provide much information that could be used to calibrate any analyses for this study.

3.2.3 Regional Geology

Almost all of Kansas was under a shallow inland sea during its geologic history. Therefore, most of the rock found in the state was formed in a sedimentary environment with near-horizontal bedding. The topography rises from east to west, exposing or approaching the older rock members including the Pennsylvanian and Mississippian

age deposits. The rock exposed at the surface in the Manhattan area is of Permian Age, with younger Jurassic and Cretaceous age rocks outcropping further to the west.

Toward the end of the Cretaceous times, the uplift of the Rocky Mountains raised the region containing Kansas above sea level. The Tertiary geologic period followed, depositing vast amounts of sand and gravel from surface water runoff, and from river and stream deposits. Alluvial, glacial and dune deposits are the youngest deposits in the region. They overlie the rock and are located near the western boundary of the state. **Figure 1** in **Appendix A.1** presents a generalized topographic and profile views of regional geology across Kansas and the general location of the Manhattan Levee project in Riley County. A detailed description of soil stratum in Riley County is located in **Figure 2** in **Appendix A.1**.

3.2.4 Site Geology

The city of Manhattan within Riley County is located on an outcrop of the Permian formation in a physiographic province referred to as the Flint Hills region. Unlike many of the formations in Kansas, the Manhattan area does not contain layers of sandstone because it was far from the shore of the inland seas that created the sandstone sediments. The Manhattan area is typical of the overall geology of Kansas consisting primarily of alternating layers of limestone (deposited in relatively shallow water) and shale typically deposited in deep water. Shale has weathered to form lean and fat clays in much of Kansas. However, flood plain alluvial deposits dominate the Manhattan, Kansas area.

3.2.5 Subsurface Conditions

The subsurface conditions for the Manhattan levee were mainly derived from historical borings logs from the Record Drawings and Design Memorandum No. 1. Historical boring logs from original design were supplemented by borings drilled by AMEC in 2011. AMEC is performing levee accreditation for the City of Manhattan. The AMEC borings were transferred to the plan/profile sheets containing historical boring logs. Based on all available boring logs, the foundation soils were delineated into two main layers – a natural blanket overlying a sand aquifer - along the levee alignment. Blanket materials consist of silts, sandy clays, lean clays, and fat clay (from Station 269+50 to 272+00). The blanket thickness ranges from 8 to 24 feet. The blanket bottom elevation is shown on the plan/profile sheets included in **Appendix A.1**. The sand aquifer consists of sands and gravels, generally grading coarser with depth. The aquifer thickness ranges from 24 feet to 47 feet. Below the sand aquifer is shale bedrock. Material strength parameters for the embankment and foundation soils were estimated based on information in Design Memorandum (DM) No. 1 and laboratory testing performed by AMEC on samples collected from their 2011 field exploration program for levee accreditation.

3.3 UNDERSEEPAGE ANALYSIS

Underseepage in the levee unit was first analyzed deterministically with water elevation at the crest of the levee to identify critical reaches. Levee reaches were determined based on levee geometry, blanket thickness, blanket material, and river entrance conditions. Critical levee sections were selected if the hydraulic gradient factor of safety is less than or equal to a minimum factor of safety of 1.1. For each of critical levee segment, the probability of failure as a function of water elevation was evaluated to generate the economic hazard curve.

3.3.1 Methodology

Underseepage analyses were performed in general accordance with Engineering Manual (EM) EM 1110-2-1913 – Levee Design and Construction. Local Kansas City District 1) uses permeability ratios instead of blanket and aquifer permeability (**Table 3-1**), 2) assumes an infinite landside blanket (Case 7 in EM 1110-2-1913), and 3) does not transform the blanket but uses a representative permeability ratio for existing materials. This Kansas City practice is based on the findings made at a 1962 Missouri River Division Conference held in Omaha, Nebraska. The conference findings are based on experience during the flood event in 1952 along the Missouri River.

Table 3-1. Permeability Ratios for Blanket Material Based on Material Type

Blanket Material	Assumed Permeability Ratio
SM	100
ML	200-400
ML-CL	400
CL	400-600
CH	800-1000

The analysis methods in EM 1110-2-1913 were modified near the Kansas River-Big Blue River confluence to take into account the sharp levee alignment change. The confluence zone was analyzed using the Intersection Angle Method discussed in **Appendix A.5**. The procedure is considered adequate for feasibility study use, but should be verified with 2-D seepage analysis during design phases.

The relief wells in the Poyntz Avenue pump station inlet ditch were analyzed in general accordance with EM 1110-2-1914 (Design, Construction, and Maintenance of Relief Wells).

As seen in **Figure 5-3** in EM 1110-2-1914, variations from EM 1110-2-1914 used in the analysis are:

1. The excess head is calculated by subtracting the drawdown from the hydraulic grade line assuming no wells. This was done because the procedure outlined in **Figure 5-3** assumes an impervious blanket. However, a semi-pervious blanket was assumed for the underseepage calculations.

2. An efficiency reduction factor of 70% was applied to the expected well flows based on pump tests of the wells provided by the City of Manhattan. This was done to account for the reduction in efficiency with time of the relief wells. An efficiency factor of 0.8 is typically used in the design of new wells as suggested in EM 1110-2-1914 "Design Construction and Maintenance of Relief Wells."

3.3.2 Deterministic Analysis

To identify critical underseepage reaches, a deterministic underseepage analysis was performed along the entire levee unit with water at the levee crest. For underseepage analysis the levee was divided into reaches of similar protection height, blanket thickness, blanket composition, seepage entrance conditions, location of relief wells, and river confluence effects. The factor of safety with respect to hydraulic gradient through the natural blanket was calculated for each of these reaches at the levee landside toe or landside ditch (if present). Calculations were performed assuming landside ditches were full and empty for thoroughness. **Tables 1** through **3** in **Appendix A.2** tabulate the calculations of factor of safety with respect to hydraulic gradient for the entire Manhattan levee unit.

Based on the results of the deterministic analysis, five critical reaches were identified for probabilistic underseepage analysis. These areas are Station 40+00 to 60+00 (deep landside ditch), 73+00 to 82+00, 101+70 (Poyntz pump station inlet ditch), 104+65 to 137+00, and 165+12 to 176+63 (Kansas-Big Blue confluence). Probabilistic analysis and results are discussed below.

3.3.3 Probabilistic Analysis

In probabilistic analysis, when the excess head at the landside levee toe is greater than zero and the blanket is thicker than one-fourth the levee height, the probability of failure can be calculated using the Taylor Series method described in ETL 1110-2-556. The exit gradient (i) was assumed to be log-normally distributed with probabilistic logarithmic moments: expected mean, E[ln i], and standard deviation, $\sigma_{\ln i}$. The limit state for underseepage failure would then be the natural log of the failure gradient (ln [i_f]) with the boundaries for the probability of failure being:

$$P_f = P(\ln i > \ln i_f) \qquad \text{Equation 2-1}$$

The probability of the ln[i] being greater than the ln[i_f] is determined by using the standard normalized variate (z), which is also analogous to the reliability index β The standard normalized variate is calculated as:

$$z = \beta \frac{\ln i_f - E[\ln i]}{\sigma_{\ln i}} = \frac{\ln\left[\frac{i_f * \sqrt{1 + COV[i]^2}}{E[i]}\right]}{\sqrt{\ln(1 + COV[i]^2)}} \qquad \text{Equation 2-2}$$

Where, *E*[i] is the expected value (mean) of the hydraulic gradient and *COV [i]* is the coefficient of variation of the hydraulic gradient.

The underseepage failure limit state, or the actual conditions indicative of an underseepage failure, are highly speculative. The underseepage analysis included in ETL 1110-2-556 (Risk-Based Analysis in Geotechnical Engineering for Support of Planning Studies) uses a threshold value of gradient factor of safety of 1.0 to define failure. A gradient factor of safety of 1.0 reflects a condition where floatation of particles theoretically begins and undesirable seepage and boils can first physically occur, however it is not necessarily a condition indicative of having certain levee failure. Observations during the Flood of 1952 on the Missouri River are shown in **Table 3-2**. The table shows the relation between observed field performance and calculated factors of safety. From the observations it can be seen that somewhere between a factor of safety of 0.55 and 0.80, undesirable seepage reaches a point where a failure could occur without outside intervention (flood fighting). In an effort to define a condition more representative of actual levee underseepage failure for this study, a gradient safety factor of 0.70 was utilized as a threshold value for when certain levee failure is likely to occur without heroic flood flight efforts. The chosen threshold value of gradient factor of safety of 0.70 falls within the "transition" zone in **Table 3-2** between tolerable seepage and objectionable seepage. In the probabilistic underseepage analyses a failure gradient (i_f) was calculated as:

$$i_f = \frac{i_c}{FS} = \frac{0.84}{0.70} = 1.23 \qquad \text{Equation 2-3}$$

where i_c is the critical gradient and *FS* is the gradient safety factor.

Table 3-2. Observations of Seepage Conditions during 1952 Flooding on the Missouri River

Computed Safety Factor at Flood Crest	Seepage Conditions During Flood Crest
Less than 0.55	Objectionable seepage: major flood fight; boils requiring sandbagging
0.55 to 0.80	Transition zone
Greater than 0.80	Tolerable seepage: distributed seepage, pin boils

Five random variables were used in the probabilistic analysis: blanket thickness, permeability ratio, aquifer thickness, entrance distance, and critical gradient (blanket unit weight). The Coefficients of Variation (COV) for the five parameters are as shown in **Table 3-3**.

Table 3-3. Underseepage Parameters Coefficients of Variation (COV)

Parameter	COV[3]
Blanket thickness[1]	25
Permeability ratio[4]	40
Aquifer thickness[1]	15
Critical gradient[2]	15
River entrance length[4]	50

[1]COV based on Engineering Judgment.
[2]COV based on COV for unit weight in ETL 1110-2-556.
[3]COV is defined as the ratio of standard deviation and expected value.
[4]COV based on ETL 1110-2-556.

Underseepage analyses was performed using the expected values of the random variables and plus and minus one standard deviations at different river levels. Using the log-normal distribution and the limit state function for underseepage, a probability of failure was developed for varying river elevations at each critical reach.

3.4 PROBABILISTIC UNDERSEEPAGE ANALYSIS RESULTS

The deterministic underseepage analysis characterized the whole levee unit based on blanket thickness, levee geometry, river entrance distance, presence of landside levee toe ditch and relief wells, and river confluence. Furthermore, locations with landside levee toe ditch were analyzed for two scenarios: ditch empty and full. Critical levee segments were determined relative to the minimum Factor of Safety (FS) of 1.1 used in the original design. Levee reaches with calculated piping FS less than or equal to 1.1 were identified as critical.

At the critical sections identified from deterministic analysis, the probability of failure was calculated for river elevations between the levee crest and where the probability of failure is zero. A summary of the results with water at the levee crest and with three feet of freeboard for the critical reaches are shown in **Table 3-4**. Of the five problem areas in **Table 3-4**, the deficiency at Station 101+70 is the responsibility of City of Manhattan. This is because four of the existing wells are operating below 80% efficiency (operation and maintenance issue) and two relief wells from the original design are missing due to modification of the Rhode Island/UPRR tracks without Corps approval. **Appendix A-3** contains results of the reliability analysis and the Probability of Failure (PoF) versus water elevation curves.

Table 3-4. Underseepage Reliability Results

Reach Stationing		Water at Levee Crest		Water 3 ft below Crest		
From	**To**	**Calculated Minimum FS**	**PoF (%)**	**Calculated Minimum FS**	**PoF (%)**	**Remarks**
40+00[2]	64+00	0.6	71	0.7	55	Landside ditch considered to be empty in analysis
40+00	64+00	3.2	3.3	5.0	0.9	Landside ditch considered to be full in analysis
73+00	82+00	1.1	7	1.3	1.6	
101+70[1]		0.54	86	0.62	66	Ponding elevation in ditch assumed to be 997 ft (O&M Manual)
104+65	137+00	0.8	36	1.5	18	
165+12	176+63	0.9	32	1.1	12	Confluence area

FS = Factor of Safety (FS = $i/[i_f = 1.23]$)
PoF = Probability of Failure
[1]Four relief wells located along landside ponding area. Two relief wells are missing from original design due to unapproved removal during expansion of Rock Island/UPRR tracks expansion about 30 years ago. Fix is responsibility of City of Manhattan.
[2]Fragility curve used in economics.

3.5 SLOPE STABILITY ANALYSIS

Slope stability analyses were performed in general accordance with EM 1110-2-1902 “Slope Stability.” Stability analysis was only performed for the landside slope under steady state seepage conditions. All analysis was performed with the Geostudio Slope/w software program using Spencer’s Method.

The embankment was assumed to be homogenous and impervious, even though it is comprised of impervious and random zones. This was done to simplify the analysis and because the random material is mostly comprised of impervious material based on AMEC borings through the centerline. The steady state piezometric surface through the levee section was assumed linear between the water surface-riverside slope intersection and the landside toe. Foundation seepage conditions were modeled in Geostudio Seep/W for use in Slope/w stability analysis. Boundary conditions were changed in the Seep/W model until the excess foundation pressures at the levee toe matched the results of the deterministic underseepage analysis performed using EM 11102-1913 “Design and Construction of Levees” methodology. This ensured pore pressures in the foundation blanket and sand during flood events are adequately modeled.

Critical sections for slope stability analysis were chosen using engineering judgment after considering underseepage analysis results, levee height, foundation materials, and other levee features. Slope stability was analyzed for five levee sections:

1) Deep landward inlet ditch at Poyntz Avenue pump station (Station 97+00 to 102+30)
2) Tallest embankment without stability berm (Station 94+00 top 97+00)
3) Levee segment with highest underseepage gradient (Station 113+50 to 130+00)
4) Levee section at the Kansas-Big Blue River confluence (Station 165+12 to 176+63)
5) Levee reach with weak fat clay blanket material (Station 245+00 to 265+70)

3.5.1 Probabilistic Slope Stability Analysis

Material parameters for slope stability were estimated from Design Memorandum No. 1 and geotechnical laboratory testing performed by AMEC on samples from their 2011 levee accreditation exploration of the Manhattan levee. The properties are considered mean, or expected, values and should only be used for probabilistic analysis. The values shown are likely larger than those typically used for design, and may result in unconservative designs. For probabilistic slope stability analysis, the expected material properties were assumed to vary according to a log-normal distribution. Coefficients of variation (COV) for each parameter were established based on engineering judgment and published values. A summary of the soil parameters used is shown in **Table 3-5**. Mean parameters with COV are used for probabilistic analysis for existing conditions. The seepage conditions were not a variable in the slope stability probabilistic analysis.

A threshold value of stability factor of safety of 1.0 to define a slope failure is by definition the critical state. To quantify the probability of failure, the frequency of FS < 1.0 is calculated using Slope/w with parameter variability. In a Log-normal distribution, the probability of failure is the area in FS distribution between FS = 0 and FS = 1.0. The GeoStudio software SLOPE/W implements this procedure using the Monte Carlo technique by using the uncertainty in soil parameters to evaluate the slope stability probability of failure. At a given water surface elevation, the analysis was iterated 10,000 times using Monte Carlo simulation to quantify the probability of failure. Slope/w results give the expected factor of safety and probability of failure.

Table 3-5. Material Properties for Slope Stability Analyses

Probabilistic Analysis				
Material	**Mean Friction Angle (deg.)**	**Coefficient of Variation (COV)**	**Moist unit weight (pcf)**	**Coefficient of Variation (COV)**
Embankment	32[1]	15.63[4]	115[3]	7.5[4]
Blanket (ML/CL)	32[1]	15.63[4]	115[3]	7.5[4]
Blanket (CH)	19[2]	15.63[4]	115[3]	7.5[4]
Sand (Foundation)	32[1]	15.63[4]	120[3]	7.5[4]
Design Analysis				
Embankment	26.5	N/A	115[3]	N/A
Blanket (ML/CL)	26.5	N/A	115[3]	N/A
Blanket (CH)	17	N/A	115[3]	N/A
Sand (Foundation)	30	N/A	120[3]	N/A

[1]Based on summary of AMEC 2011 exploration laboratory direct shear testing for levee accreditation (See Appendix A.5)
[2]Based on Design Memorandum (DM No. 1): Flood Protection Project, Kansas River Basin, Manhattan, Kansas, August 1959
[3]Assumed unit weights
[4]In accordance with ETL 1110-2-561: Reliability Analysis and Risk Assessment For Seepage and Slope Stability Failure Modes For Embankment Dams
N/A = Not Applicable

The slope stability analyses assumed the failure surface should be of significant magnitude to remove a major portion of the levee allowing the interior of the levee unit to flood. Utilizing the slope stability program Slope/W, failure was forced to originate at the water surface-riverside slope intersection. Using this constraint, the failure would be of significant magnitude to inundate the levee interior instead of assuming a progressive slope failure from the landward levee toe.

3.5.2 Probabilistic Stability Results

A summary of results with water at the levee crest and with 3ft of free board are shown in **Table 3-6**. With water at the levee crest, the mean FS is significantly lower than the minimum FS stated in the Design Memorandum (FS = 1.5) and EM 1110-2-1913 (FS = 1.4). However, with 3 ft of freeboard the mean FS meets or exceeds the Design Memorandum (DM) minimum FS at all critical segments except between station 113+50 to 130+00 and station 165+12 to 176+63, FS of 1.3 and 1.4, respectively. The DM indicates the steady state stability analyses were performed with three feet of freeboard during design. Subsequently, results with water 3 ft below the crest are a better comparison with the DM and also indicate failure is very unlikely under the original design assumptions. Similarly, the mean FS meets or exceeds the EM 1110-2-1913 minimum FS at all critical reaches except between Station 165+12 and 176+63. However, since mean strengths were used the reaches may not meet criteria with design strengths because design strengths are normally lower than mean strengths.

One of the more significant assumptions for the reliability analysis is the development of a steady state seepage condition that is used in the analysis. The steady state seepage condition takes time to develop and does not occur instantaneously. If the steady state does not occur, then the stability analysis is

conservative and overstates the probability of failure. A transient analysis was completed to better assess the likelihood of a fully developed steady state condition occurring.

The time required to reach steady state was analyzed using GeoStudio software SEEP/W by performing transient seep analysis. The initial water table was assumed at the base of the embankment since slope stability failure is initiated by a driving hydraulic head above the riverside toe. An average permeability of 3.3×10^{-6} ft/sec (Holtz and Kovacs) was assumed representative of the entire embankment material. Summary of the transient seep analysis results with water at the levee crest and 3 ft below the crest is shown in **Table 3-7**. Typical slope stability and transient seep analysis outputs for the critical levee sections are found in **Appendix A.4**. A typical release from Tuttle Creek Dam may last 60 days. Flooding duration on the Kansas River, since it is largely unregulated by reservoirs, is expected to be approximately 15 days.

As seen in **Table 3-7**, the flood duration required to saturate the embankment when the levee is loaded to the crest and with 3 ft of freeboard is generally greater than the expected flood duration from either flood source. The shortest time for saturating the embankment (Station 245+00 to 265+70) at top of levee and with 3 ft freeboard was calculated to be 36 and 45 days, respectively. However, there are many uncertainties that make this analysis approximate. These uncertainties include rainfall impact, wave overwash for river levels approaching top of levee, the assumed permeabilities, simplification of embankment zoning, and the unknown character of the random fill zone which could vary significantly along the levee alignment because of the specification that allowed a wide variety of acceptable soils.

It is unlikely that steady state seepage conditions will develop for a top of levee loading on the Big Blue River or the Kansas River reach. Steady state seepage developing at lower river elevations, however, is possible. However, the risk associated with steady state seepage at lower river elevations is not significant.

The slope stability probabilistic results were not used for economic modeling since the likelihood of developing steady state conditions is relatively remote for river levels where the calculated risk is high. Additionally, underseepage seems to dominate the levee reliability. However, the slope stability reliability concerns under steady state condition will be considered for improvements during design of proposed raises to meet USACE stability criteria for this failure mode. Embankment drainage can easily be incorporated into areas of proposed levee raises to ensure USACE stability criteria is met. However, areas outside proposed raise areas will be more closely evaluated to determine design loading durations and anticipated seepage conditions for stability analysis in design. If areas outside proposed raises do not meet criteria, but do not pose considerable risk to the project, they will be documented and not remediated in the design phase.

Table 3-6. Steady State Slope Stability Analysis Results

Station		Toe Pressure Head (ft)	Blanket Thickness (ft)	River Water Elevation (ft)	Water at Levee Crest		Water 3ft below Crest	
From	To				Mean Factor of Safety (FS)	Probability of Failure (%)[1]	Mean Factor of Safety (FS)	Probability of Failure (%)[1]
94+00	97+00	11.8	20	1025.8	0.99	53.0	1.53	0.2
97+00[2]	102+30	8.9	28	1021.5	1.28	0.5	2.02	0.1
97+00[3]	102+30	6.6	28	1021.5	1.17	1.1	1.55	0.2
113+50	130+00	8.3	8	1021.0	0.66	100.0	1.30	0.3
165+12	176+63	13.4	14	1016.0	0.44	100.0	1.40	0.2
245+00	265+70	4.3	15	1016.5	1.02	33.7	1.82	0.2

[1]Probability of failure corresponds to ratio of the number of FS less than unity divided by the total number of Monte Carlo iterations
[2]Failure surface between riverside slope and landward toe of levee
[3]Failure surface between riverside slope and landward ditch toe

Table 3-7. Transient Seepage analysis results, 1993 Flood, and Tuttle Creek Emergency Release

Station		1993 Flood[1]	Tuttle Creek Emergency Release[2]	Water at Levee Crest	Water 3 ft below Levee Crest
From	To	Max Water Elevation Duration (days)	Release Duration (days)	Saturation Duration (days)[3]	Saturation Duration (days)[3]
94+00	97+00	11	N/A	88	106
97+00	102+30	11	N/A	36	64
113+50	130+00	11	N/A	128	150
165+12	176+63	11	60	87	125
245+00	265+70	11	60	26	45

[1]Duration of maximum water elevation during 1993 flood event corresponding approximately 3 ft freeboard (greater than 50-year and less than 100-year return period).
[2]Duration of Tuttle Creek Emergency release corresponding to roughly 4 ft freeboard (greater than 500-year-return period).
[3]Duration to saturate levee embankment.

3.6 SUMMARY

The geotechnical existing conditions analysis was performed to identify the critical sections from a geotechnical perspective and determine their probability of failure for economic modeling. The probabilistic analyses performed for this study were modeled with guidance given in ETL 1110-2-556 “Risk-Based Analysis in Geotechnical Engineering for Support of Planning Studies” (28 May 1999). The two failure modes considered at various river stages were underseepage and landside slope stability under steady state seepage condition.

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PART 4 – Structures

Table of Contents

4.1 INTRODUCTION.....4-1

4.2 ENGINEERS.....4-1
4.2.1 Team Leader.....4-1
4.2.2 Manhattan Avenue Pump Station 20+13.....4-1
4.2.3 Poyntz Avenue Pump Station 101+26.....4-1
4.2.4 Old By-Pass 98+80.....4-1
4.2.5 Gatewells.....4-1
4.2.6 Former Closure Structure Station 83+24.....4-2

4.3 DETERMINISTIC CRITERIA.....4-2
4.3.1 Assumptions.....4-2
4.3.2 Stability Requirements.....4-3
4.3.3 Strength Requirements.....4-3

4.4 UNCERTAINTY ANALYSIS.....4-4

4.5 RISK CALCULATIONS.....4-5
4.5.1 Strength.....4-5
4.5.2 Stability.....4-5
4.5.3 Structural Failure.....4-5

4.6 STRUCTURAL MATERIAL PROPERTIES.....4-5

4.7 SOIL MATERIAL PROPERTIES.....4-6

4.8 STRUCTURAL ANALYSIS.....4-6
4.8.1 Manhattan Avenue Pump Station.....4-7
4.8.2 Poyntz Avenue Pump Station.....4-8
4.8.3 Old By-Pass Pump Station.....4-9
4.8.4 Gatewells.....4-10
4.8.5 Former Closure Structure Station.....4-11

4.9 REFERENCES.....4-15

4.1 INTRODUCTION

The structural Flood Risk Management features of the Manhattan Levee Unit analyzed as part of this study consist of pump stations and gatewells. The structural analysis involved an assessment of the existing condition of the structures based on available construction plans, detailed engineering analysis, and engineering judgment. The method of analysis began with a deterministic analysis of critical components (without any load or resistant factors) to establish an expected Factor of Safety (FS). The analysis does not consider any defects, or incremental or secondary load effects, that may have been caused by previous floods.

Components of these structures were analyzed, without factors of safety and with consistent assumptions, in order to evaluate the relative risk and consequences for economic and risk-informed decision-making purposes. Risk and Reliability studies do not replace Load and Resistance Factored Design (LRFD) analysis, nor do such studies confirm that the structure and its components satisfy any design criteria, past or present. The studies simply provide information about the possible performance of the structure for the loads under consideration. The results of the study are provided to the decision-maker to aid in making a risk-informed decision. In this feasibility study, these results were input into the HEC-FDA model to develop the overall levee unit failure probability.

4.2 DETERMINISTIC CRITERIA (EXISTING CONDITIONS)

A series of screening criteria are used to determine if a probabilistic analysis is necessary for a given structure. Summarized below are the assumptions used to analyze structural components as well as the strength and stability criterion from the current design standards. If the analysis shows the existing structural component meets the below criteria, it is assumed reliable and a 99.8% reliability is assigned. If the structural component does meet the criteria, a reliability analysis is performed.

4.2.1 Assumptions

The following lists major assumptions in the feasibility study.

i. Some structural components were not analyzed. Only components judged to be critical based upon engineering experience were analyzed for feasibility.
ii. The pump station components were analyzed based on dimensions, quantities, and conditions represented by record drawings. Deviations from plans cannot be verified per scope and budget. This is a consistent assumption for relative risk and reliability assessment for levee feasibility studies.
iii. Parts of the components being analyzed that were not evaluated with this analysis include, but may not be limited to, minimum rebar embedment lengths, structure capacity at rebar cutoff locations, etc
iv. Any parts of the components that may have been damaged during a flood were repaired to minimum USACE criteria.
v. Construction practices are considered good, and all specifications noted on

plans satisfied.

vi. Materials, such as reinforcing bar, are in good condition.

vii. Soil is adequately compacted and fill type and strength parameters supplied by Geotechnical Section is correct.

viii. Concrete compression zone is without voids.

ix. Other assumptions were made for the uncertainty and risk analysis.

4.2.2 Stability Requirements

Structural stability criterion can be seen in Table 4-1. It is based on Engineering Manual (EM) 1110-2-2100 Stability Analysis of Concrete Structures, dated 1 December 2005. The structures in this study are being analyzed for extreme loading conditions.

Table 4-1. Required Factors of Safety for Flotation

Load Condition Category	Return Period	Factor of Safety
Usual	10 years	1.3
Unusual	300 years	1.2
Extreme	Top of Levee	1.1

4.2.3 Strength Requirements

Typical Strength reduction factors and load factors were not used in the analysis of these structures. Load factors and reduced strengths are used in design, but are not applicable to a probability of failure analysis. If an existing structure has a calculated factor of safety of less than 1.0 (Capacity/Demand), then it implies failure of that structure.

Factors for new design: For new structures designed with the Strength Design Method, loads are increased by multiplying service loads by appropriate load factors and nominal strengths are decreased by corresponding strength reduction factors. Load factors required by EM 1110-2-2104, Strength Design for Reinforced –Concrete Hydraulic Structures include a dead and live load factor (LF) of 1.7 and a hydraulic load factor (HF) of 1.3.

Combining these gives a total load factor (TF) of 2.21. The strength reduction factor for flexure (ϕ), the typical controlling failure mechanism, is 0.90. Dividing the load factor by the strength reduction factor gives an overall factor of safety of about 2.45 for a new design.

Factor for existing structures: A high enough Factor of Safety (FS) in the strength analysis will provide 99.8% reliability because any variance in coefficients is too low to overcome the safety factor. There is a limit where the FS is still above 1, yet the probability of failure (POF) will begin to increase due to statistical possibilities presented by the coefficient of variance. To prevent unnecessary POF analyses, it is desirable to determine this FS threshold. Two reasons are given to set this FS threshold at 1.5.

First, it is possible to calculate the maximum range of FS based upon the coefficients of variation used in the analysis. This is performed by likening the analysis to measurement and instrumentation. Coefficients of Variation (COV) are then treated like uncertainty of a measurement (FS) based upon the mean values. For a system with N^{th} order of uncertainty, a 95% confidence estimate of total uncertainty can be computed by the square root of the sum of the squares of each coefficient of variation.

Considering the Coefficient of Variation for concrete compressive strength, steel yield strength, unit weight of soil, seepage pressures, and the angle of internal friction yields a probable maximum range in FS of +/- .28. Failure is not attained until FS < 1. Therefore, by this method, the FS should not reduce the POF unless the FS is near or below 1.28. A FS threshold of 1.5 would guarantee capturing any change to the POF.

A second reason why a threshold FS of 1.5 is sufficient is based upon historical results. Historical results have shown that for a POF analysis with FS above 1.3, the reliability results were still the maximum (99.8% Reliability). Historical analyses have also shown that POF results didn't vary appreciably unless the FS was lower than 1.2. This was largely because the Standard Variation used in analysis was small compared to 0.5, and there were only two variables in the majority of the analyses. Currently, six variables are considered to substantially contribute to changes in FS. The additional variables also have small variations when compared to 0.5. Using FS threshold of 1.5 has been shown reliable, theoretically and historically.

4.3 UNCERTAINTY ANALYSIS

For structural features not meeting deterministic strength and stability criteria, a risk and uncertainty analysis was performed. The method adopted for calculating a probability of failure is that outlined for geotechnical engineering in “Factors of Safety and Reliability in Geotechnical Engineering”, by J. Michael Duncan, published in the Journal of Geotechnical and Geoenvironmental Engineering, April 2000. The use of this method provides consistency between the structural and geotechnical analyses.

To produce a probability of failure curve, the critical section of each feature not meeting criteria was analyzed (factor of safety determined) using mean material strengths and/or mean soil properties. Next, the parameters were varied to plus and minus one standard deviation from the mean one at a time and the factor of safety was recomputed. The reliability index equation from EM 1110-2-547 was used to determine the reliability of the feature not meeting the factor of safety. Assuming the feature started as 100% reliable, the probability of failure was determined by subtracting the reliability from the starting reliability. A 2% probability of failure was used as an appropriate non-failure threshold. If a probability of failure greater than 2% resulted, then the water elevation was lowered in 1-foot increments and the feature was reanalyzed until the probability of failure obtained was less than 2%.

The methods used are appropriate when data is normally distributed, when parameters display a linear relationship, and when degradation over time is not a

consideration. Because of the limited availability of data and with no information to suggest otherwise, an assumption of normal distributions for input data is reasonable and consistent with guidance provided in Engineering Technical Letter (ETL) 1110-2-547 (paragraph B-6.c). Examples of degradation over time would include scour around piles, reactive concrete, sliding movement, and deteriorating drainage systems that affect uplift. All available historic data, site inspections, and engineering judgment do not show time dependent deterioration of structures to be a concern for these Manhattan pump stations.

4.4 RISK CALCULATIONS

4.4.1 Strength

For strength calculations, uncertainty is measured by applying a mean and standard deviation to concrete compressive strength, steel yield strength, unit weight of soil, effective depth, the angle of internal friction, and seepage. The selected mean and normal standard deviation are based on engineering judgment and information published in both “Reliability Based Design in Civil Engineering” by Milton E. Harr, and ETL 1110-2-556.

4.4.2 Stability

For stability calculations, uncertainty is considered by applying a mean and standard deviation to the soil unit weight, shear strength, and varying seepage pressures based on values provided by the geotechnical engineers. The uncertainty inherent in determining the soil parameters provides a means to find a probability of failure. From experience on the Missouri River Levee Project L-142 Criteria Study (KCD-COE), it was determined through analysis that the unit weight and the soil shear strength have a noticeable effect on a floodwall’s factor of safety. Varying the concrete density has only a minor effect on the factor of safety.

4.4.3 Structural Failure

Failure is defined as the capacity to demand ratio (factor of safety) less than 1.0, or in other words when the demand (loads) exceed the capacity (structural or geotechnical).

4.5 STRUCTURAL MATERIAL PROPERTIES

For the screening portion of the Manhattan Pump Stations Feasibility Study the following structural properties were used. The American Concrete Institute recommended the use of a 3,000 psi concrete strength around the 1940’s through 1960’s, the typical timeframe of construction for the Manhattan Avenue pump station structure. The new Poyntz Avenue pump station was constructed in 2002 and a value of 4,000 psi concrete strength was used based on the As-Built drawings, Sheet S1.

Knowing the time period of construction (~1940’s – 1960’s) and based upon the Portland Cement Association’s pamphlet Engineered Concrete Structures, 1997, an assumed reinforcing steel design yield strength, Fy, of 40 ksi is used for the Manhattan

Avenue pump station structure. The newer Poyntz Avenue pump station was designed with Grade 60 ksi reinforcing, based on the As-Built drawings, Sheet S1.

Based on FEMA 310, the mean strength (or expected strength) for Risk and Uncertainty calculations shall be taken as 125% of the design strength. For reinforced concrete structures Harr suggests a 14% coefficient of variation.

i. Concrete Strength Variation (14%)
1. 1940's-1950's: $\mu - \sigma = 3225$, $\mu = 3750$, $\mu + \sigma = 4275$ (3000 psi min)
2. 1900's-1920's: $\mu - \sigma = 2150$, $\mu = 2500$, $\mu + \sigma = 2850$ (2000 psi min)

ii. Steel Strength Variation (14%)
1. 1940's-1950's: $\mu - \sigma = 43$, $\mu = 50$, $\mu + \sigma = 57$ (40 ksi min)
2. 1900's-1920's: $\mu - \sigma = 35.5$, $\mu = 41.25$, $\mu + \sigma = 47.0$ (33 ksi min)

4.6 SOIL MATERIAL PROPERTIES

The soil properties used to compute loads on structures for the Manhattan Pump Stations Feasibility Study are located in the geotechnical portion of the report. The values posted were obtained from the *Manhattan Levee Feasibility Study,* dated August 2006, in consultation with the geotechnical team members. These simplified values, shown in Table 4-2, were generalized conservatively for use in typical structural calculations.

Table 4-2. Soil Properties

Pump Station	Friction Angle (°)	Soil Unit Weight (lb/ft^3)
Manhattan Avenue 20+13	17	115
Poyntz Avenue 101+26	17	115
Old By-Pass 98+80	17	115
Gatewells 14+78, 34+62, 62+20, 81+25, 89+83, 98+80, 99+15, 105+05, 163+00, 234+00 269+50	17	120
Former Stop Log Gap	26	115

4.7 STRUCTURAL ANALYSIS

The structural evaluation of pump stations focused on floatation stability along with foundation wall and floor strengths. The potential for pump station uplift was computed according to EM 1110-2-2100, Stability Analysis of Concrete Structures taking into consideration site specific hydraulic grade lines supplied by Geotechnical project team members. Foundation wall and floor capacities were calculated using MathCAD worksheets. The CASE project program CORTCUL was used for analyzing the concrete walls on the reinforced concrete box for the Poyntz Avenue Pump Station.

For pump stations not meeting strength and floatation factors of safety, reliability calculations were performed.

4.7.1 Manhattan Avenue Pump Station

General: The Record "as-built" drawings dated August 1963 show that this pump station was constructed around February of 1961. It is located near the Tieback Levee portion of the Manhattan Kansas Levee System. The entire pump plant sits on an area of approximately 10.5 ft by 22 ft. A 72" Reinforced Concrete Pipe (RCP) extends to the face of the North and South Wet Well Walls. The West wall is bumped out (approximately 4 ft by 8 ft high) to go around the above mentioned RCP. The soil elevation is approximately the same around all four sides of the pump station.

Summary of the Analysis of Foundation Walls and Floor Strengths, and Uplift: Assumptions were made in analysis. It was also assumed that the water was at the top of the levee. It was determined after looking at the difference in the size and reinforcement of the walls, that the North and South Walls directly underneath the pump house (Wall 1: North and South Long Walls) could be analyzed as if they were identical.

The North and South Short Walls (Wall 3 or the walls that house the trash rack and sluice gate) could also be analyzed as if they were identical because of the size of the walls and the reinforcement. The East Wall (Wall 2) would be analyzed separately from the West Wall (Wall 4). Wall 4 is the bottom portion of the West Wall. The top portion of the west wall mimics Wall 2. The center wall will not be analyzed as a part of this feasibility report.

The walls were analyzed with applied soil, water and thrust loads. A plate analysis was done to determine the moment reaction coefficients for walls 1, 2, and 3. These moment reactions are for two-way bending and therefore, both horizontal and vertical steel must be checked. The moment reactions were used to determine the moment applied to the wall. A RAM Elements Model was used to determine the moments for Wall 4. These moments were compared to the wall and steel capacities and a factor of safety was computed based on the ratio of strength design wall capacity divided by computed wall moments.

After completing the plate analysis and the RAM Model, and putting the results into the spreadsheet to reanalyze, all of the walls met the factors of safety required for moment and for shear. The uplift forces on this structure were also calculated, and the results can be found in Table 4-3 below. It was determined that the pump station does meet the factor of safety requirements for uplift. The base slab of the pump station was also checked and found to be adequate.

Corrective measures will NOT be required for this pump plant because all the Factors of safety for strength were greater than 1.5 and the uplift factor of safety was greater than 1.1. Detailed calculations are available upon request.

4.7.2 Poyntz Avenue Pump Station

General: The exiting pump station is a fairly new structure that was built in approximately 2002 to replace the original pump station. The new pump station is located to the south of the old station and has a main interior space with plan dimensions of 16' x 16'. Drilled piers form two sides of the pump station and were used as form work during construction and probably used as form work to shore and dewater the area. The thicknesses of the drilled piers were neglected in the thickness of the pump station walls.

A unique aspect of this pump station is that the soil elevation varies depending on which wall is being analyzed. The soil elevation on the east side of the pump station (levee side) was raised up during construction and is higher than the soil elevation on other pump station walls. This causes the hydraulic grade line to be lower than the soil elevation on the east wall but higher than the soil elevation of the remaining walls.

Summary of the Analysis of Foundation Walls and Floor Strengths, and Uplift: Moment coefficients were taken from Plate tables (per Moody, 1963) to determine the wall moments in each direction. These moments are for two-way bending and therefore, both horizontal steel and vertical steel must be checked. These moments were compared to the wall and steel capacities and a factor of safety was computed based on the ratio of strength design wall capacity divided by computed wall moments. All factors of safety are greater than 1.5, and therefore 99.8% reliability is assigned. Uplift on the new pump station was also checked.

Even though the new pump station is doweled into and connected to the existing box conduit at several locations, uplift was checked for the new pump station separately based on the minimal dowels and connections. The uplift factor of safety was found to be greater than the required 1.1 factor of safety. The slab of the pump station was also checked and found to be adequate. Corrective measures will NOT be required for this pump plant because all the Factors of safety for strength were greater than 1.5 and the uplift factor of safety was greater than 1.1. Calculations are available upon request.

Summary of Reinforced Concrete Box Conduit Analysis: The reinforced concrete box conduit was analyzed with CORTCUL, a CASE program designed by the US Army Corps of Engineers. The conduit dimensions, steel reinforcement, concrete and reinforcement strength, reinforcement cover, soil data, elevations of the conduit and ground water, etc. were input into the program. Based on the age of the structure, concrete strength is assumed to be 3000 psi and the steel yield strength is assumed to 40,000 psi.

The conduit is analyzed at its worst case scenario, which is directly below the levee. The amount of soil and potential water over the conduit is at its greatest. The conduit is analyzed with no water in the conduit. A point load on the levee due to a vehicle load of 32 kip axle weight based on the design truck per ASHTO was also

assumed.

The program CORTCUL analyzes the four wall members of the conduit in bending, axial, and shear. The output gives factors of safety analyzed at each end and the centerline of each wall. All factors of safety for the conduit are greater than 1.5 and are therefore adequate for this feasibility study.

4.7.3 Old By-Pass Pump Station Sta. 98+80

General: Construction of this pump station is unknown and is presumed to be completed by the City of Manhattan, KS (City) prior to the construction of the current Federal levees in the early 1960's. The Old-By-Pass Pump Station was originally used by the City to pump effluent into the Kansas River from a now abandoned sewage treatment plant. The pump station is currently used as a back-up to the New Poyntz Ave pump station located at Sta. 101+26 just directly to the north-east which is fed by the nearby drainage canal via gravity flow.

A 24" CIP extends from the Old-By-Pass pump station through the levee. On 5/9/2011 AMEC, an A/E contracted by the City to conduct levee certification, conducted a video inspection of the 24" CIP. The video inspection was only able to see from the outlet to the levee. At the levee, the pipe makes a sharp bend upward towards a valve box at the top of levee. The pipe from the levee to the pump station was not inspected as the only way to inspect would be to cut a hole into pipe or disassemble pipe in pump station. According to the Manhattan Periodic Inspection Report #3, the pipe was rated as unacceptable because it was not completely inspected.

Summary of the Analysis of Foundation Walls and Floor Strengths, and Uplift: Old By-Pass was analyzed similarly to Manhattan and Poyntz pump station. It should be noted that reinforcing bar sizes, and wall and slab thicknesses are based on the use of ground penetrating radar and leaves some uncertainty. Moments are for two-way bending and both horizontal and vertical steel must be checked.

All factors of safety are greater than 1.5, and therefore 99.8% reliability is applied. Uplift on the pump station was also checked. The uplift factor of safety was found to be inadequate for water to top of levee (0.87). Per the criteria mentioned above in Section 4.3, a probability of failure (POF) was conducted to determine the reliability of the structure by varying the pressures on the base of the structure.

The POF curve was determined by calculating the POF due to uplift with water at 1022.6’ (top of levee), 1015.5’ (50 year event), and 1008’ (no water on levee) see Figure 4.1 below. Calculations are available upon request.

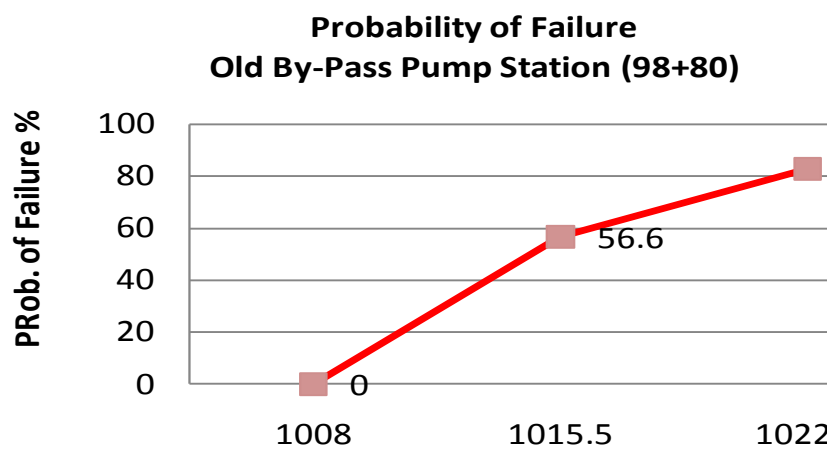

Figure 4.1 Probability of Failure

Considering the unknowns and the estimated risks associated with this pump station and because this is a back-up pump station to the new Poyntz Avenue pump station, it is recommended that water within the Old-By-Pass pump station be maintained at 5'-10" during pumping operations. 5'-10" is the amount of water needed to achieve an uplift factor of safety of 1.1 required by USACE criteria. The Old-By-Pass Pump Station is not part of the Federal project; however since it falls within the levee critical zone, the Sponsor has been made aware of the possible deficiency, and recommendations have been made to address in separate official correspondence.

4.7.4 Gatewells

General: The eleven Manhattan Flood Protection gatewells were analyzed with water to top of levee (worst case) only to determine their reliability for strength and uplift. The results of this analysis are shown in Table 4-6.

Summary of the Analysis of Foundation Walls and Floor Strengths, and Uplift: Five of the eleven gatewells showed inadequate Factors of Safety (1.5 or less) for strength at the walls, except for the Sta. 62+20 gatewell. Upon first failing to meet the deterministic criteria, Sta. 62+20 was considered 100% reliable after conducting a probability of failure analysis. However, considering the age of the structure and based on engineering judgment, it was determined for feasibility purposes that this structure should be assumed to be fully replaced. Per the criteria mentioned above in Section 4.3, a probability of failure (POF) analysis was conducted to determine the reliability of the structure by varying the pressures on the gatewell as a function of dropping the water surface elevation on the structure from top of levee (TOL), 3-feet, and 6-feet down, respectively. See results in Table 4-6. The POF graph(s) below Table 4-6 will be used by economics for assessing economic impact. There were no concerns with uplift on the structures.

Alternatives to improve structural reliability include strengthening walls with W-sections and bracing or adding to the concrete thickness of the walls. Full pipe

replacements will be made for Sta. 269+50 and 163+00 since these pipes will be subject to induced loading caused by the raise on the Big Blue levee segment. The remainder of the pipes on the Kansas River segment will remain in place except for the first approximately 10-feet upstream and downstream of the gatewell replacements.

4.7.5 Former Closure Structure

General: A field visit was conducted on December 18, 2010, in order to obtain measurements of the stop log gap since no as-built drawings were available as mentioned above. An excavator provided by the City of Manhattan was used to excavate at two locations (1) riverside (RS) from the centerline of the stoplog gap and (2) next to the right RS abutment facing landside.

It was impractical to excavate on the landside (LS) as the excavator would not have enough clearance on top of levee to excavate and would compromise fully compacted and settled in-situ soils. Once riverside footing was exposed, measurements were taken. The RS footing width is 5.5' from stem wall and the heel depth exposed to 3' (did not go all the way down).

After the excavations and further research in ED-DS archives, a design computation from September 1961 was found showing the concrete sill and adjacent abutment/floodwall design. The verifiable dimensions matched the measurements and the other dimensions from the computations were assumed to be correct.

Summary of the Analysis: The structure was analyzed for sliding, overturning, and strength at the abutments. The structure is **ADEQUATE** for sliding and overturning with moist soil conditions. However, the stop-log abutments are **NOT ADEQUATE** for strength as a result of the loading being reversed since its original design. The 3-#4's at the landside face of the abutment (now in tension) were not designed to retain land side soil. A recommendation was made for the Sponsor to place USACE approved select levee fill material (equivalent to landside) on the riverside to further enhance the structural stability of the structure and off-set soil loading on landside. In 2012, the Sponsor added the approved fill on the landside of wall as recommended.

Table 4-3. Pump Stations analyzed with water at top of levee

Station	Station Name	Uplift Factor of Safety (> 1.1 Req'd)	Water Req'd to meet 1.1 Uplift Factor of Safety (ft)	Water Available (ft)	Comments
20+13	Manhattan Avenue	1.24 (Wet) 1.13 (Dry)	0	3.09	**No Corrective Measures Necessary**
101+26	Poyntz Avenue	1.18 (Wet) 1.10 (Dry)	0.28	3.18	**No Corrective Measures Necessary**
98+80	Old By-Pass	0.87 (Wet) 0.78 (Dry)	5.8	3	Verification prior to or during PED that no heel exists. IF heel exist then uplift FS will improve. If no heel present then corrective measures will be necessary. Measures may consist of rock or helical anchors, adding a concrete heel, etc.

Table 4-4. Pump Stations Moment Factor of Safety (> 1.5 Req'd)

Station	Station Name	Wall 1	Wall 2	Wall 3	Wall 4	Base Slab	Comments
20+13	Manhattan Avenue	1.52	2.13	1.73	2.24	1.59	**No Corrective Measures Necessary**
101+26	Poyntz Avenue	1.79	2.23	1.82	N/A	2.12	**No Corrective Measures Necessary**
98+80	Old By-Pass	2.46	4.62	N/A	N/A	1.70	**No Corrective Measures Necessary**

Table 4-5. Pump Stations Shear Factor of Safety (> 1.5 Req'd)

Station	Station Name	Wall 1	Wall 2	Wall 3	Wall 4	Base Slab	Comments
20+13	Manhattan Avenue	1.73	2.70	3.35	2.41	4.67	No Corrective Measures Necessary
101+26	Poyntz Avenue	2.49	3.06	2.62	N/A	2.12	No Corrective Measures Necessary
98+80	Old By-Pass	2.89	2.38	N/A	N/A	2.25	No Corrective Measures Necessary

Table 4-6. Gatewells Existing Conditions Analyzed with Water TOL

Station	Uplift Factor of Safety HGL (>1.1 Req'd)	Controlling Strength Factor of Safety (>1.5 Req'd)	Controlling Structural Mechanism	Assigned Reliability (%)	Comment
14+78	1.2	0.97	Mid-Span Wall Bending	94.8	Wall AD mid-span bending ,shear, and base slab fail to meet the minimum Factor of Safety of 1.5 required per our guidance. Consider strengthening walls and slab of gatewell
34+62	1.5	2.5			
62+20	1.7	1.3	End Span Wall Bending	100	Wal 1 negative end span moment meets relibaility after conducting POF and varying concrete and steel strengths. Based on engineering judgement assume full replacement.
81+25	1.8	2.2			
89+83	1.3	1.1	End Span Bending	97.21	Consider strengthening walls of gatewell. Nearly meets criteria.
98+80	1.5	4.4			
99+15	1.4	2.8			
105+05	1.5	2.3			
163+00	1.6	0.95	Mid-Span Bending	62.1	Wall AD mid-span bending , fails to meet the minimum Factor of Safety of 1.5 required per our guidance. Consider strengthening walls of gatewell
234+00	2.2	2.8			
269+50	1.6	0.76	Mid-Span Bending	0	Wall AD fails to meet criteria and is recommended to strengthen walls in PED.

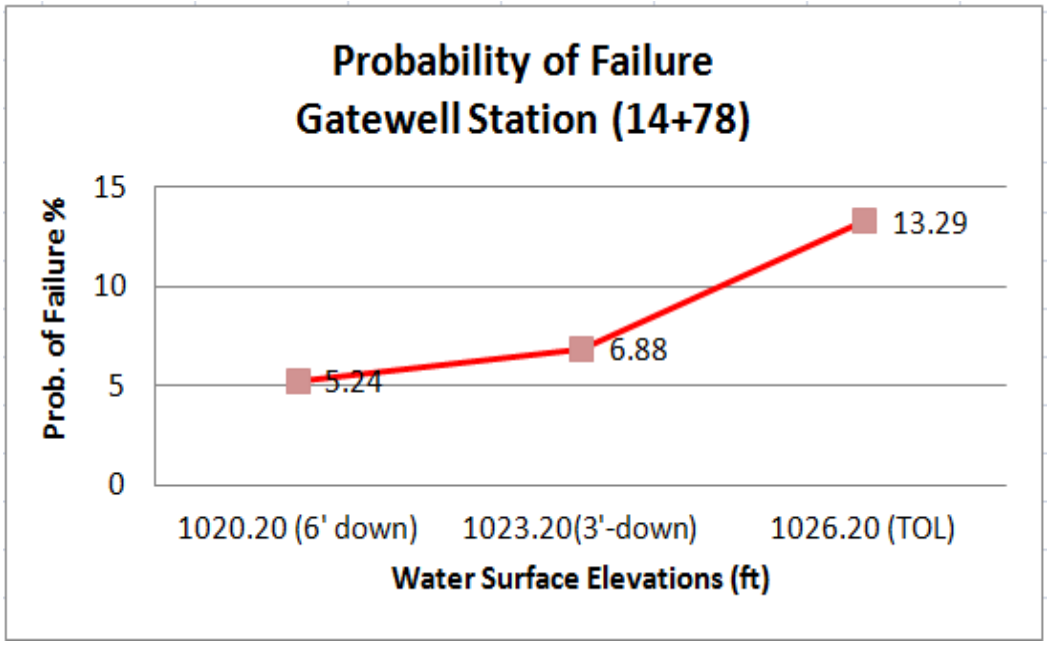

Figure 4.2 Probability of Failure versus Water Surface Elevation for Economics (Sta. 14+78)

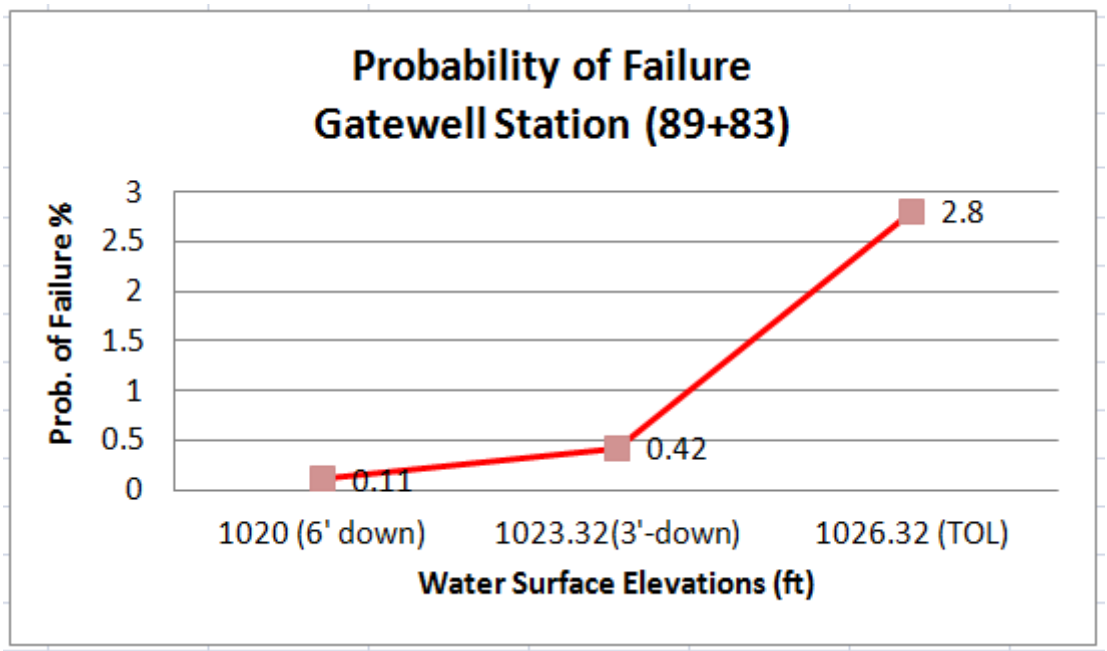

Figure 4.3 Probability of Failure versus Water Surface Elevation for Economics (Sta. 89+83)

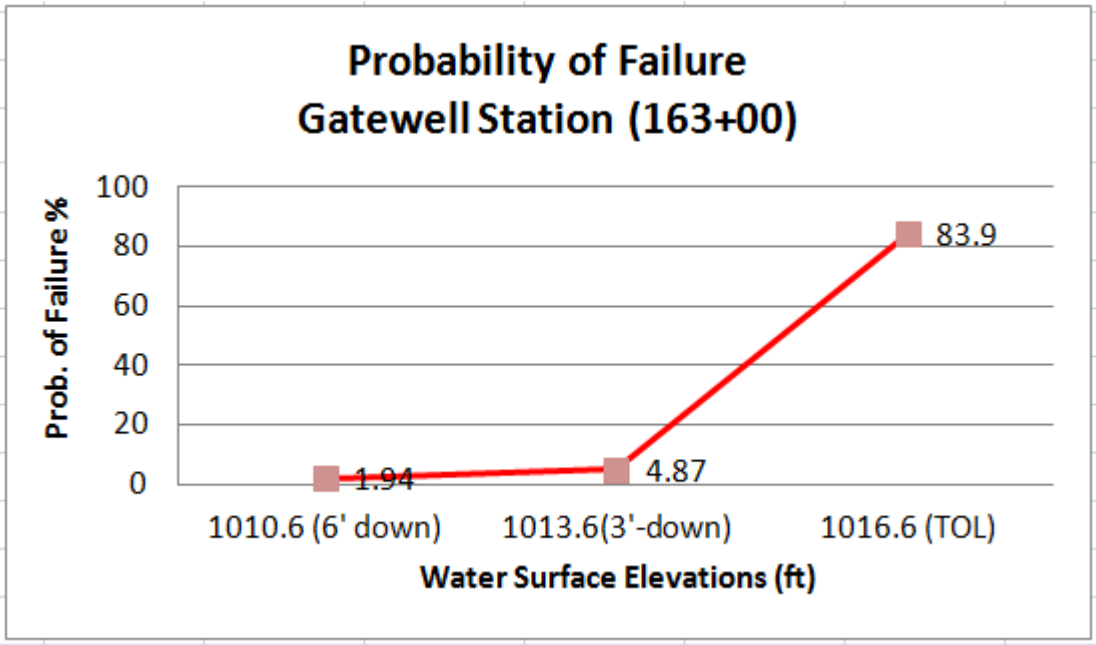

Figure 4.4 Probability of Failure versus Water Surface Elevation for Economics (Sta. 163+00)

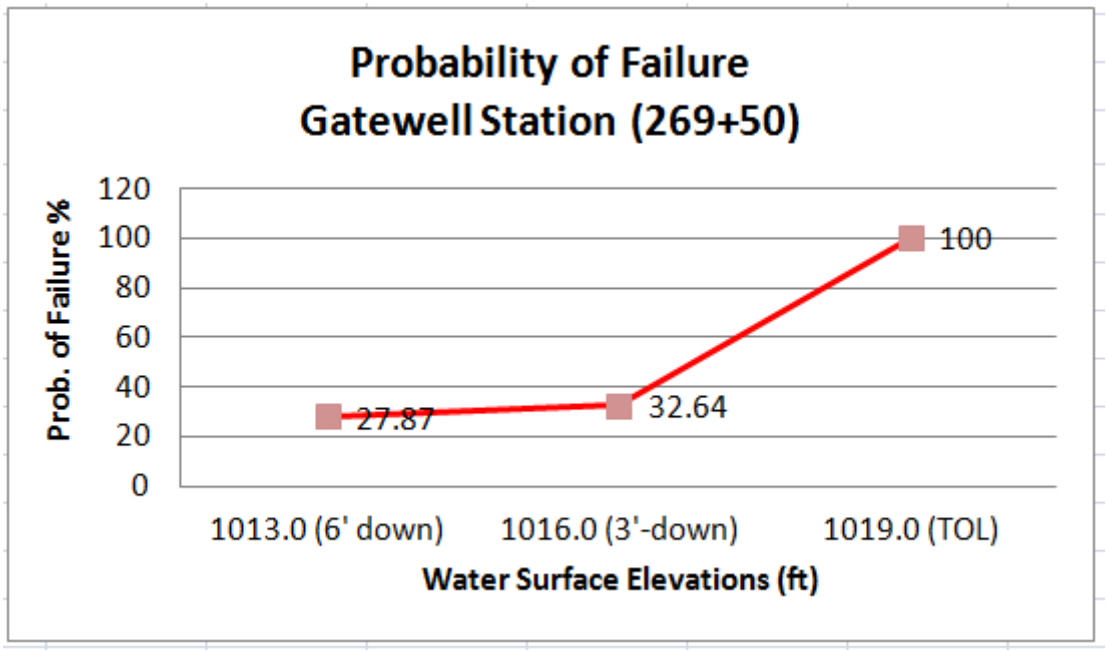

Figure 4.5 Probability of Failure versus Water Surface Elevation for Economics (Sta. 269+50)

4.8 REFERENCES

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PART 5 – Civil

Table of Contents

5.1 INTRODUCTION.....5-1

5.2 DESCRIPTION OF EXISTING LEVEE UNIT 5-1
5.2.1 Levee Description 5-1

5.3 UTILITY UPLIFT ANALYSIS 5-1
5.3.1 Methodology 5-1
5.3.2 Results.....5-3
5.3.2.1 Utilities Crossing the Levee.....5-3
5.3.2.2 Utilities in the Critical Zone.....5-4

5.4 ABOVE GROUND UTILITIES AND FEATURES 5-6

5.5 PUMP STATION ASSESSMENT.....5-6

5.6 SUMMARY 5-7

5.7 ITEMS OF CONSIDERATION FOR ALTERNATIVES.....5-7
5.7.1 Vicinity of Station 110+00 to 150+00.....5-8
5.7.2 Vicinity of Station 255+00 5-8
5.7.3 Other Utilities 5-8
5.7.4 Borrow Quantities and Source of Materials 5-8

5.8 REFERENCES.....5-9

5.1 INTRODUCTION

This document presents results of civil / utilities evaluation of the Manhattan levee in Manhattan, Kansas to include utility uplift and limited hydraulic assessment of pump stations under existing conditions.

This is the first phase of the feasibility study where the existing condition of the levee unit is documented. The second phase of the feasibility study considers alternatives to improve levee reliability with respect to all failure modes, including improvement of the factor of safety of utilities constructed as part of the Federal project found to be deficient, and is presented in Volume 2.

5.2 DESCRIPTION OF EXISTING LEVEE UNIT

5.2.1 Levee Description

The Manhattan levee protects the City of Manhattan, Kansas from high water along Wildcat Creek, the Kansas River, and the Big Blue River. The levee is approximately 5-½ miles long and generally has a 10-foot wide crown and three horizontal to one vertical side slopes. Construction of the levee was completed in the early 1960s. The levee was designed with freeboard, e.g. the original design water surface for the Manhattan levee is 3 feet below the levee crest. Note that, while the original design included 3 feet of freeboard, this analysis considers water to the top of the existing levee.

5.3 UTILITY UPLIFT ANALYSIS

Uplift is the force acting on buried structures (e.g. utility lines) due to excess hydrostatic pressure during times of high river stage. Excessive uplift forces can literally “float” a structure or utility out of the ground, causing a situation that is difficult to flood-fight. Since excessive pressures could occur at any location along the levee, all utilities crossing the levee or within the critical zone (300 feet riverward to 500 feet landward of the levee) were inventoried and analyzed. .

5.3.1 Methodology

A variety of information sources were considered. Riley County supplied GIS data sets for water and sewer lines during the reconnaissance phase. This information, along with information obtained from discussions with Kansas Gas Service representatives, was used for the utilities analysis. Selected operational and maintenance manual (O&MM) drawings for the Manhattan levee were also reviewed and used for utility information.

Utilities were broken into two groups; buried utilities analyzed for uplift, and above ground utilities which may pose clearance problems for proposed construction. Buried utilities analyzed for uplift were further broken into two groups; those that cross the levee and those located in the critical zone.

The geotechnical data used for uplift analysis was provided by district geotechnical engineers, and was the same data used for seepage and stability computations.

In cases where pipe materials are unknown, steel is assumed for gas lines; ductile iron for other pressurized lines (e.g. water and sewage force main), and reinforced concrete for gravity sewer lines.

Depths of utilities used in the analysis are to the invert. Actual reported or surveyed depths were used when available. When actual depths were not available, sanitary and storm sewers up to 30 inches diameter were estimated to be 10 feet deep. Depths for 36 inch and larger sewers were estimated at 15 feet. Depths for pressurized lines (water, gas, sewage force main) were estimated to provide approximately 4 feet of cover, e.g. analysis depth for 6 inch water would be 4-1/2 feet; analysis depth for 30 inch water would be 6-1/2 feet.

Wall thickness was neglected for relatively thin materials, including ductile iron, steel, and vitrified clay. Wall thickness for reinforced concrete utilities was estimated as the nominal diameter in feet, plus one inch, e.g. for a 4 foot diameter reinforced concrete pipe (RCP) the wall thickness would be estimated at 4+1 = 5 inches, resulting in an outside diameter for analysis of 48+5+5 = 58 inches.

Representative unit weights for various pipe materials were taken from a variety of sources. American Society for Testing and Materials (ASTM) C76 “Standard Specification for Reinforced Concrete Culvert, Storm Drain, and Sewer Pipe” was used to estimate weight per lineal foot, assuming wall type “B”. ASTM C700 “Standard Specification for Vitrified Clay Pipe, Extra Strength, Standard Strength, and Perforated” was used to estimate weight of clay pipe. Steel pipe weights were estimated for schedule 40 pipe. Cast and ductile iron pipe weights were estimated for the lowest pressure of class readily available in a particular size, e.g. class 150 for 30 inch diameter.

While the original design included 3 feet of freeboard, the head pressures used in the uplift calculations are based upon water to the top of the existing levee, as well as subsurface information as determined by Kansas City District geotechnical engineers. In some cases, head pressures for a given range vary due to the existence of landside ditches. In these cases, the most conservative values (e.g. highest excess head pressures) were used in the utility uplift calculations. Structures in the area formed by the ponding levee were not analyzed since they are outside of the critical zone of the primary levee system.

With the above information in hand, each utility was analyzed for uplift according to Kansas City District Geotechnical Design and Dam Safety Section guidance for uplift. A “spot check” comparison was made per EM 1110-2-2100: Stability Analysis of Concrete Structures with results generally consistent, with the EM being slightly more conservative (e.g. producing a slightly lower factor of safety).

The analysis was conducted in three steps, with the first step considering buried pipes subject to uplift forces, and neglecting similar hydrostatic forces acting down on the pipe. This was a “screening level” analysis intended to identify areas where due diligence would dictate a closer look at utilities. The second step looked at those utilities identified as potentially problematic in step 1, but considered all of the hydrostatic forces acting on the pipe, including those forces acting downward. The third step considered manholes associated with the utilities. Unlike buried pipes, manholes would not experience downward-acting hydrostatic forces since they would typically extend to or above the ground surface. In other words, if potential utility problems are identified in an area, they would likely be most serious at manholes.

5.3.2 Results

Screening level analysis was performed on all utilities crossing or in the critical zone of the levee. Utilities considered to be questionable or problematic based on screening level analysis are discussed below. In each case, a factor of safety of 1.1 (corresponding with the extreme loading condition for flotation per EM 1110-2-2100) or greater is considered to be acceptable under existing conditions.

5.3.2.1 Utilities Crossing the Levee

<u>Station 62+20, ID 11xng</u>

This drainage structure is a 48-inch RCP. At the line of protection the structure invert is 20 feet below grade. Screening level uplift analysis shows this line to have acceptable factors of safety when full; unacceptable when empty. Further detailed analysis, considering all of the hydrostatic forces acting on the pipe, results in acceptable factors of safety at all locations. Analysis of the gatewell structure is presented in the structural portion of the existing conditions document.

<u>Station 81+25, ID 12xng</u>

This drainage structure is a 24-inch RCP. At the line of protection the structure invert is 22 feet below grade. Screening level uplift analysis shows this line to have an unacceptable factor of safety, near the landside levee toe. Further detailed analysis, considering all of the hydrostatic forces acting on the pipe, results in acceptable factors of safety at all locations. Analysis of the gatewell structure is presented in the structural portion of the existing conditions document.

<u>Station 163+00, ID 18xng</u>

This drainage structure is an 84-inch RCP. At the line of protection the structure invert is 20 feet below grade. Screening level uplift analysis shows this line to have acceptable factors of safety when full; unacceptable when empty. Further detailed

analysis, considering all of the hydrostatic forces acting on the pipe, results in acceptable factors of safety at all locations. Analysis of the gatewell structure is presented in the structural portion of the existing conditions document.

Station 234+00, ID 21xng

This drainage structure is an 84-inch RCP. At the line of protection the structure invert is 16 feet below grade. Screening level uplift analysis shows this line to have acceptable factors of safety when full; unacceptable when empty. Further detailed analysis, considering all of the hydrostatic forces acting on the pipe, results in acceptable factors of safety at all locations. Analysis of the gatewell structure is presented in the structural portion of the existing conditions document.

Station 269+50, ID 22xng

This drainage structure is a 72-inch RCP. At the line of protection the structure invert is 21 feet below grade. While this pipe would likely be full during a high water event due to uncontrolled seepage and impeded drainage, screening level uplift analysis shows an unacceptable factor of safety, near the landside levee toe. Further detailed analysis, considering all of the hydrostatic forces acting on the pipe, results in acceptable factors of safety at all locations. Analysis of the gatewell structure is presented in the structural portion of the existing conditions document.

5.3.2.2 Utilities in the Critical Zone

Station 35+00 – 45+00, ID 5cz, 6-inch Water within 220 feet of levee

Screening level uplift analysis shows this water line to have unacceptable factors of safety only when empty, and only for a portion of its length (Station 40+00 to 45+00) due to unusually high residual head pressures. Further detailed analysis, considering all of the hydrostatic forces acting on the pipe, results in acceptable factors of safety at all locations.

Station 70+00 – 100+00, ID 12cz, 24-inch Water within 125 feet of levee

Uplift analysis shows this water line to have unacceptable factors of safety only when empty, and only for a portion of its length (Station 73+00 to 87+00 and 94+00 to 97+00) due to unusually high residual head pressures. Further detailed analysis, considering all of the hydrostatic forces acting on the pipe, results in acceptable factors of safety at all locations.

Station 110+00 – 115+00, ID 17cz, 42-inch Sewer within 110 feet of levee

Uplift analysis shows this sanitary sewer line to have unacceptable factors of safety at some locations when empty, and also at some locations when full. Further detailed analysis, considering all of the hydrostatic forces acting on the pipe, results in acceptable factors of safety at all locations *on the pipe*. Further detailed analysis of the manholes, assuming 6 foot diameter typical precast structures, results in unacceptable factors of safety.

While these manhole structures are not part of the Federal project and therefore not used in economic calculations for levee evaluations, they do present a potential means of failure under conditions of high water and corresponding high excess head. The Sponsor should further evaluate these structures and consider means of reducing excess head pressures, adding weight, or anchoring the structures.

Station 112+00 – 114+00, ID 21cz, 10-inch Sewer within 200 feet of levee

Uplift analysis shows this sanitary sewer line to have unacceptable factors of safety at some locations (Station 113+00 – 114+00) when empty. Closer examination shows that the sewer is farther from the levee (approx. 300 feet) within this area of exceptionally high residual head. At that greater distance, the factor of safety is acceptable.

Station 115+00, ID 22cz, 36-inch Sewer within 110 feet of levee

Uplift analysis shows this sanitary sewer line to have unacceptable factors of safety when empty or filled. Further detailed analysis, considering all of the hydrostatic forces acting on the pipe, results in acceptable factors of safety at all locations *on the pipe*. Further detailed analysis of the manholes, assuming 6 foot diameter typical precast structures, results in unacceptable factors of safety.

While these manhole structures are not part of the Federal project and therefore not used in economic calculations for levee evaluations, they do present a potential means of failure under conditions of high water and corresponding high excess head. The Sponsor should further evaluate these structures and consider means of reducing excess head pressures, adding weight, or anchoring the structures.

Station 115+00 – 151+00, ID 23cz, 54-inch Sewer within 50 feet of levee

Screening level uplift analysis shows this sanitary sewer line to have unacceptable factors of safety at some locations when empty, and also at some locations when full. Further detailed analysis, considering all of the hydrostatic forces acting on the pipe, results in acceptable factors of safety at all locations *on the pipe*. Further detailed analysis of the manholes, assuming 8 foot diameter typical precast structures, results in unacceptable factors of safety.

While these manhole structures are not part of the Federal project and therefore not used in economic calculations for levee evaluations, the do present a potential means of failure under conditions of high water and corresponding high excess head. The Sponsor should further evaluate these structures and consider means of reducing excess head pressures, adding weight, or anchoring the structures.

Station 194+00 – 200+00, ID 26cz, 16-inch Water within 0 feet of levee

Uplift analysis shows this water line to have unacceptable factors of safety only when empty. Further detailed analysis, considering all of the hydrostatic forces acting on the pipe, results in acceptable factors of safety at all locations.

Station 200+00 – 213+00, ID 28cz, 20-inch Water within 0 feet of levee

Uplift analysis shows this water line to have unacceptable factors of safety only when empty. Further detailed analysis, considering all of the hydrostatic forces acting on the pipe, results in acceptable factors of safety at all locations.

Station 255+00 – 272+00, ID 34cz, 36-inch Sewer within 175 feet of levee

Uplift analysis shows this sanitary sewer line to have unacceptable factors of safety when empty within a limited range (Station 265+70 – 269+50). Since this is a main line, it would be expected to contain water at all times, with the depth of water varying with the time of day. With the pipe half full, analysis still results in an unacceptable factor of safety. Further detailed analysis, considering all of the hydrostatic forces acting on the pipe, results in acceptable factors of safety at all locations *on the pipe*. Further detailed analysis of the manholes, assuming 6 foot diameter typical precast structures, results in unacceptable factors of safety.

While these manhole structures are not part of the Federal project and therefore not used in economic calculations for levee evaluations, they do present a potential means of failure under conditions of high water and corresponding high excess head. The Sponsor should further evaluate these structures and consider means of reducing excess head pressures, adding weight, or anchoring the structures.

Station 255+00 – 272+00, ID 36cz, 12-inch Water within 125 feet of levee

Uplift analysis shows this water line to have unacceptable factors of safety only when empty, due to unusually high residual head pressures. Further detailed analysis, considering all of the hydrostatic forces acting on the pipe, results in acceptable factors of safety.

5.4 ABOVE GROUND UTILITIES AND FEATURES

Above ground features such as bridges and overhead utilities will be considered with regard to prospective levee raises, and how those levee raises might affect clearance. Since potential levee raises are very small, slight reductions in clearance are not expected to be problematic.

5.5 PUMP STATION ASSESSMENT

The Manhattan levee was designed and constructed with two pump stations; the Manhattan Avenue Pump Plant and the Poyntz Avenue Pump Plant, both of which were originally constructed as part of the Federal project. The Poyntz Avenue plant was demolished and a new plant constructed at the same location in 2003 by local interests. A third facility known as the Bypass Pump Plant is also used on occasion, but was not designed as part of the original levee system.

Information on the Manhattan Avenue Pump Plant was obtained from 1961 record drawings and the operations & maintenance manual. This station was designed to handle interior drainage and limited relief well flow. Since interior drainage is a local

responsibility and several of the relief wells have been abandoned, this station was not analyzed for hydraulic capacity. Capacity requirements for any proposed improvements which would result in concentrated seepage flow (e.g. relief wells) to these stations should be addressed in future conditions analysis. Structural evaluation of the Manhattan Avenue Pump Station is provided in Part 4 Structures.

Information on the original Poyntz Avenue Pump station was obtained from 1961 record drawings. Local interests demolished the original Poyntz Avenue station and constructed a new / larger station in 2003. Record drawings dated January 2003 were provided by the local Sponsor. Since this station handles only interior drainage, which is a local responsibility, the hydraulic capacity of this station was not analyzed. Structural evaluation of the Poyntz Avenue Pump Station is provided in Part 4 Structures.

No detailed information such as as-built drawings or pump curves was found on the Old Bypass Pump Station. This facility was reportedly constructed by the City of Manhattan prior to construction of the Federal levee in the early 1960s and originally used to pump effluent into Kansas River from a now abandoned sewage treatment plant. While this plant is not part of the Federal system, it is currently used as a backup to the new Poyntz Avenue station during high rainfall events concurrent with high river stages. Since this station handles only interior drainage, which is a local responsibility, the hydraulic capacity of this station was not analyzed. Structural evaluation of the Old Bypass Pump Station is provided in Part 4 Structures.

5.6 SUMMARY

The civil / utilities existing conditions analysis was performed to identify utilities that could pose a risk of failure during high water events, to broadly assess the hydraulic capacity of existing pump stations as they relate to levee features, and to identify above ground utilities or bridges that could affect implementation of future improvements (e.g. a levee raise). Several utilities raise concern, as noted in the following section.

5.7 ITEMS OF CONSIDERATION FOR ALTERNATIVES TO IMPROVE RELIABILITY OF UTILITIES FOR RESISTING UPLIFT AND PROPOSED LEVEE

As the study moved into alternatives examination, it became necessary to determine what improvements would be required to address the deficiencies noted under existing conditions and to support potential levee raises averaging 1.5 feet with some levee segments needing up to a three foot raise depending, on location and existing topography. These potential improvements would likely include the levee raise, berms and / or relief wells, improvements to existing structures, and incidental utility relocations and other associated work items.

5.7.1 Vicinity of Station 110+00 to 150+00, ID 17cz, 22cz, 23cz

This area is characterized by an interceptor sewer transitioning from 42-inch to 54-inch as it picks up flow from a 36-inch trunk, in an area where the Kansas River is particularly close to the levee, and excess heads are high under extreme flooding conditions. The manhole with the most uplift force under extreme conditions would require approximately 75,000 pounds of downward force to prevent flotation. This downward force could be provided by anchors, heel extensions, or simply by adding weight. For purposes of estimating potential costs, a 20 foot diameter x 3 foot thick concrete ring is assumed for each of eleven manholes in this area. If a berm were to be proposed in this area, these manholes would also need to be raised accordingly.

5.7.2 Vicinity of Station 255+00 – 272+00, ID 34CZ

This sewer is located in an area where excess heads under conditions of high river stage could cause unacceptable uplift forces on manholes. These manholes will not experience the same level of uplift as those discussed above, however for purposes of estimating potential costs, a 20 foot diameter x 3 foot thick concrete ring is assumed for each of three manholes in this area.

5.7.3 Other Utilities

Thirty power poles from Station 104+00 to 140+00 may need to be raised to accommodate any proposed berms in this area.

An 8-inch diameter gas line at Station 227+00 would need to be raised to accommodate any proposed levee raise in this area.

A 36-inch diameter water line at Station 232+28 would need to be raised to accommodate any proposed levee raise in this area.

5.7.4 Borrow Quantities and Source of Material

Borrow quantities and potential sources of material are discussed in Volume 2 of the Engineering Appendix.

5.8 REFERENCES

US Army Corps of Engineers. Operation and Maintenance Manual: Flood Protection Project, Kansas River Basin, Manhattan, Kansas, revised June 1980

Kansas City District Geotechnical Design and Dam Safety Section Guidance for Uplift, January 2003, http://www.nwk.usace.army.mil/geotech/pdf/uplift.pdf

US Army Corps of Engineers, EM 1110-2-2100: Stability Analysis of Concrete Structures

US Army Corps of Engineers. Construction Plans for Levees and Appurtenances: Flood Protection Project, Kansas River, Manhattan, Kansas, February 1961

US Army Corps of Engineers. EM 1110-2-1913: Design and Construction of Levees, 30 April 2000

American Society for Testing and Materials (ASTM). C76: Standard Specification for Reinforced Concrete Culvert, Storm Drain, and Sewer Pipe

American Society for Testing and Materials (ASTM). C700: Standard Specification for Vitrified Clay Pipe, Extra Strength, Standard Strength, and Perforated

Flood Protection Project Manhattan, Kansas, Supplemental Design Memorandum No. 1-C (1960) was used in the development of Paragraphs; Design Storm.

Riley County GIS website http://gis.rileycountyks.gov/