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 2 ALTERNATIVES ENGINEERING

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1.1 ENGINEERING APPENDIX VOLUME 2 PURPOSE AND CONTEXT

This Feasibility Report Engineering Appendix Volume 2 documents the engineering analysis and the development of the alternatives evaluated during the Manhattan, KS Local Protection Project Feasibility Study. Specifically, this Appendix includes chapters detailing the Hydrolgy and Hydraulics, Geotechnical, Structural, and Civil Design analyses of each of the proposed study alternatives. Information pertaining to the existing project conditions and future without project analyses, including discussions of project and study authority and location, are found in the Engineering Appendix Volume 1.

1.2 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.

1.3 FUTURE WITH PROJECT CONDITIONS - PROJECT ALTERNATIVES OVERVIEW

The initial future with project studies involved generating preliminary concepts and measures for levee reliability improvement, along with a reasonable set of early alternative plans using the findings from the existing conditions analysis. Potential nonstructural and structural solutions to flooding problems were evaluated using the latest hydrology and hydraulics modeling and the results were analyzed ("future with project" results). In the case of the existing Manhattan project, levee raises provided the most net benefits among the various potential solutions. This was followed by various sponsor and public discussions, alternatives screening, and various in-progress-reviews and refinement of the most promising solutions. See the main feasibility report for a more detailed discussion of the plan formulation and alternatives development and comparison process and details of the construction components making up the final array of alternatives summarized in the list below.

FINAL ARRAY OF ALTERNATIVES

1) Plan 1 -- No Federal Action

2) Plan 2 -- raising the current level of protection to pass the nominal 0.5% annual chance flood event with accompanying geotechnical and structural reliability improvements

3) Plan 3 -- raising the current level of protection to pass the nominal 0.33% annual chance flood event with accompanying geotechnical and structural reliability improvements

4) Plan 4 -- raising the current level of protection to pass the nominal 0.2% annual chance flood event with accompanying geotechnical and structural reliability improvements

5) Plan 5 -- raising the current level of protection with channel widening (CW) and bridge modifications on the Big Blue River for increased flood conveyance so as to pass the nominal 0.33% annual chance flood event with accompanying geotechnical and structural reliability improvements.

The alternatives are mapped and displayed in the accompanying Mapbook. Especially see sheet No. 10 in the Mapbook for a good comparison of the length/location of alternative levee raises and geotechnical features involved with each alternative.

Depending on the specific requirements of the alternative plan under development, appropriate engineering measures (consistent with the latest Corps design and safety guidance) were considered for:

- improved structural feature strength and stability,
- levee underseepage control,
- pump station performance and interior drainage characteristics,
- reliability against levee overtopping.

Certain engineering measures outside the immediate raised levee area are necessary so as to provide a levee unit meeting the Corps' current design criteria and factors of safety -- such as improved levee foundation, strengthened structural elements, and increased underseepage control.

The total project cost estimates for the final array of alternative plans roughly range between $20 million to $52 million (in 2014 dollars).

1.4 PLAN EVALUATION, COMPARISON AND SELECTION

The evaluation and comparison of the various alternative plans is described in the main Feasibility Report. The actual plan selection process is undertaken as one of the final steps in the planning process, and is performed in conjunction with sponsor input and HQUSACE reviews and approval. The selected plan is held to stringent standards. The plan must be technically viable, economically feasible and environmentally acceptable – all in accordance with governing agency regulations and associated Federal statutes.

All of the plans were evaluated and indicated positive net economic benefits and benefit to cost ratios, as described in the Feasibility Report. Based on the evaluation and comparison of the alternatives, the Plan 3, based on the nominal 0.33% annual chance flood event, is the National Economic Development Plan (NED) Plan and is the Recommended Plan.

1.5 ENVIRONMENTAL IMPACTS AND ENGINEERING

ER 1110-2-1150, Engineering and Design for Civil Works Projects, Paragraph 13.6.8 states that the project design shall seek to avoid and minimize adverse environmental impacts and when possible be in concert with the surrounding environment. Review and selection of the engineering measures and alternatives discussed in this appendix performance included a determination of their potential environmental impacts and a consideration of environmental engineering features that may be included in the design.

Details of the environmental impacts associated with each proposed alternative are discussed in the Environmental Assessment (EA) prepared in conjunction with this Feasibility Report. Section 4.0 of the EA presents a description of each plan's potential impact to soils, water quality, air quality, aquatic and terrestrial habitats, wildlife, threatened and endangered species, HTRW, and social and cultural resources. The proposed project would have none to minor adverse impacts to each of the resource categories, most impacts being short-term in nature (e.g. dust or noise from construction equipment). The project also would have no impacts to Federally listed threatened or endangered species or their designated critical habitat. As no wetlands or waters of the US are impacted, no Section 404 or 401 permitting will be necessary. The contractor would be required to obtain a land disturbance permit and develop the Stormwater Pollution Prevention Plan.

Potential impacts to terrestrial habitat and the need for environmental mitigation associated with each alternative were studied and coordinated with applicable resource agencies**.** The initial assumption for this impact analysis and alternative comparison was that all habitat within the construction easement would be destroyed or adversely impacted. This is a conservative estimate, as it is likely much of the habitat within the construction easement may be able to be avoided or the impacts minimized. Temporary construction easements as well as the permanent easements that are cleared during construction will be planted with native vegetation where possible following construction. During the design phase effort will be made to incorporate where practicable the use of native vegetation and to identify potential ways to enhance or expand existing riparian corridors. All trees at least 50 feet tall and/or greater than 24-inch dbh riverside of the levees should be avoided. These trees are potentially utilized as perching/roosting trees by the Bald Eagle. The contractor will be required to follow best management practices to avoid the introduction and spread of invasive species.

As presented in Section 4.2.2 of the EA, the Recommended Plan impacts to terrestrial habitats come from the lateral expansion of the levee footprint from the levee raise, underseepage berms, and landside toe embankment sand drains. It is also assumed that there will be disturbance to all the areas within the permanent and temporary construction easements. This would result in an impact of 6.23 acres of forested area, 0.67 acres of shrubland area, 17.50 of grassland most of which is mowed turfgrass, and 7.74 acres of cultivated cropland. In some cases these are

relatively small isolated patches of impacts, while in other areas the impacts can extend linearly for some distance along a forested area. This would decrease the width of the forested stands which may affect the habitat suitability for species that need larger blocks of habitat. With time, the minor impacts would be reduced as trees become reestablished within the construction easement area.

In accordance with the Migratory Bird Treaty Act clearing of vegetation should be avoided during the migratory bird nesting season, which in Kansas occurs during the period of January (owls, and hawks) through August (goldfinches). If vegetation clearing takes place during the nesting season, then the area to be cleared should be surveyed by a qualified biologist prior to clearing activity. Design and construction planning will take these requirements into consideration.

The primary materials required for construction of the recommended plan is earthen soil for the levee raise and underseepage berms. Approximately 158,000 cubic yards of borrow material which would be obtained from an approximately 20 acre location(s) identified on the map in Appendix I of the EA. The proposed borrow location is currently in agricultural row crop production. There is no other environmentally renewable material option available. There is not expected to be any spoil or other refuse from project construction and operation that can be put to beneficial reuse.

Implementation of the recommended plan will be generally limited to the already disturbed existing levee project location, maintaining the current ecological continuity in the project with the surrounding area and region. The existing project easement is already maintained by the non-Federal sponsor in accordance with USACE operations and maintenance guidelines to be free of excessive vegetation. There are little to no opportunities to design additional environmental attributes into the recommended plan within the existing project area. Future operations and management of the project following plan implementation will be essentially the same as current operations which already minimize environmental impact and energy consumption. No opportunities have been identified for enhancing environmentally beneficial operations or energy savings.

The study of alternative and determination of the recommended plan has identified no other indirect environmental costs or benefits and has considered the environmental sensitivity of the study area to project modification. All future project design and implementation phases will continue to adhere to all required environmental compliance measures and requirements.

The Recommended Plan has been determined to best meet the purpose and need of the project by providing for increased flood risk management with limited impacts to the environment in a cost effective manner. The Recommended Plan does not result in any significant long-term impacts to the human environment.

1.6 PROJECT IMPLEMENTATION

A 5-year construction phase beginning in 2018 is planned -- subject to authorization and availability of construction funding. The technical scope and magnitude of the project would be best served by a three construction contract package arrangement for the work item groups as shown below.

- Contract #1: Replace 2 gatewells (Big Blue River segment) prior to levee raise work. These are within the levee raise area.

- Contract #2: Entire levee raise, upstream berms, relief wells in levee raise area, and all utility relocations -- largest contract.

- Contract #3: Replace 3 gatewells (Kansas River levee segment) near conclusion of project. These are outside the levee raise area.

Significant complications arise with alternative plans which involve: 1) Big Blue River bridge modifications (either the Hwy 24 Bridge and/or the Union Pacific Railroad Bridge); 2) any work on the east side of the Big Blue River -- such as channel widening. Such work will likely require more construction contracts, and carefully sequenced construction phasing given the various Federal and State agencies involved in modifying any major transportation features. The project risks also increase – and new safety risks arise due to the need for maintaining vehicular traffic safety during bridge modifications.

During any modification of the existing levee, the construction contractor must maintain (at least) the current flood protection performance level of the existing levee features. Emergency response plans for the construction period must be coordinated and prepared, reviewed by the Corps, and then monitored and executed properly were any adverse flood conditions to occur. The Corps of Engineers Kansas City Area Construction Office is assigned to oversee any construction resulting from this study.

A levee raise requires expansion of the earthen footprint which requires additional real estate. It was determined early in the study that the footprint expansion will normally occur towards the landside (interior) of the levee so as to not constrict the already limited conveyance capacity of the existing Big Blue River channel, and to limit any disruptive environmental or adverse habitat effects to the reiverward riparian corridor. For the feasibility study, the Corps determines real estate requirements and the associated cost based on the selected plan footprint and a gross appraisal. More exact real estate requirements are developed during the design phase following completion of the feasibility study.

According to Federal law, the sponsor is responsible, with Corps guidance, to undertake (pay for) the required real estate actions. The overall project schedule anticipates timely real estate actions, and the sponsor is aware of their responsibilities in this regard. Real estate actions and acquisition required for the project construction will begin prior to the first construction contract award. The Manhattan levee has

relatively few utilities requiring relocation. The Corps evaluates the compensable or non-compensable nature of the various utility relocations, in conjunction with the sponsor. The Corps then assigns relocation costs when the utility ownership and real estate rights information is adequate for a compensability determination. Utility relocation design details are developed during the design phase.

It is difficult to predict the exact timing of the project funding allocations or whether such allocations will be forthcoming for construction. However, the schedule does assume reasonable periods for the design and construction process from the standpoint of Corps capabilities.

US Army Corps of Engineers
Kansas City District

Part 2
Hydrology and Hydraulics

Alternatives Engineering
Manhattan, KS

August 2014

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2 Hydrology & Hydraulics

2-1. Background

The Existing Conditions (EC) Hydraulics and Hydrology (H&H) Appendix of the Manhattan Levees Feasibility Study was complete in May of 2011. Since that report's completion, significant work has been accomplished to develop a Future Conditions (FC) alternatives analysis, inundation maps, hydraulic model improvements, and levee performance risk data for the economics analysis. The purpose of this report is to document the work that has been completed since the completion of the EC H&H appendix.

2-2. Hydraulic Analysis of Alternative Components

The alternatives analysis for the Manhattan Feasibility study was based on the calibrated existing conditions hydraulic model that is documented in the Hydraulics and Hydrology Existing Conditions Engineering Appendix.

2-2.1. Channel Widening and Levee Raises

Channel widening was evaluated for the alternatives Plan 5, and Early Formulation Plan B, which are described in Table 1 below. Approximately 1,200 feet of channel was widened on the Big Blue River to maintain a 250 feet wide bottom width with channel banks cut back to 2:1 horizontal:vertical slopes. Figure 1 below is a concept rendering of this alternative at the 24 Highway Westbound bridge. This analysis required adding a span to each bridge to increase conveyance area below the 24 Highway and Railroad bridges. An area-end method quantity was computed with the Channel Modification Tool in HEC-RAS to develop the quantities for costs associated with this alternative. This tool develops volumes by calculating changes in cross sectional area and estimates volume with the reach lengths between cross sections.

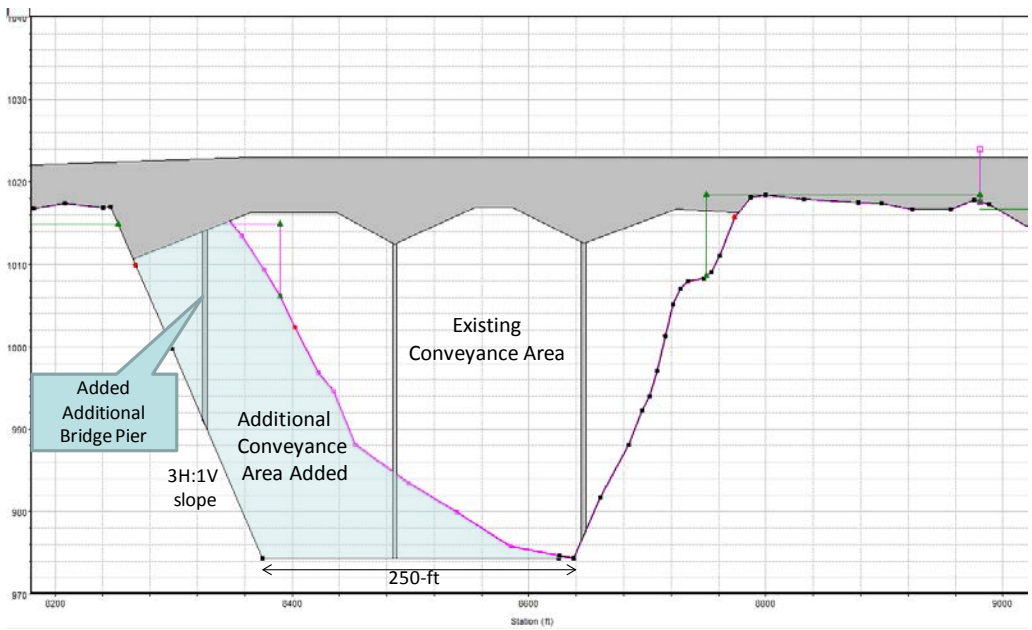


Figure 1: Concept rendering of channel widening under 24-HWY Westbound

The results of the channel widening in the hydraulic model are shown below in Figure 2. On the Big Blue River and Drainage Ditch segments of the levee, channel widening lowers the hydraulic profile of the 0.33% (1/300) Annual Chance Exceedance (ACE) by about 0.4 feet from the existing conditions profile and it is about 0.6 feet below the Plan 3 alternative profile. Closer to the confluence, the effects of channel widening raise the 0.33% (1/300) ACE flood profile above existing conditions by about 0.5 feet due to the increased channel conveyance around the 24 Highway and railroad Bridges. This additional conveyance, which previously backed water up behind the bridges, induces a backwater effect closer to the mouth of the Big Blue River. Additional considerations for channel widening in other alternatives were not pursued because of the minimal improvements channel widening makes to the hydraulic performance of the levee as well as the high costs incurred from modifying the three bridges that cross the Big Blue River.

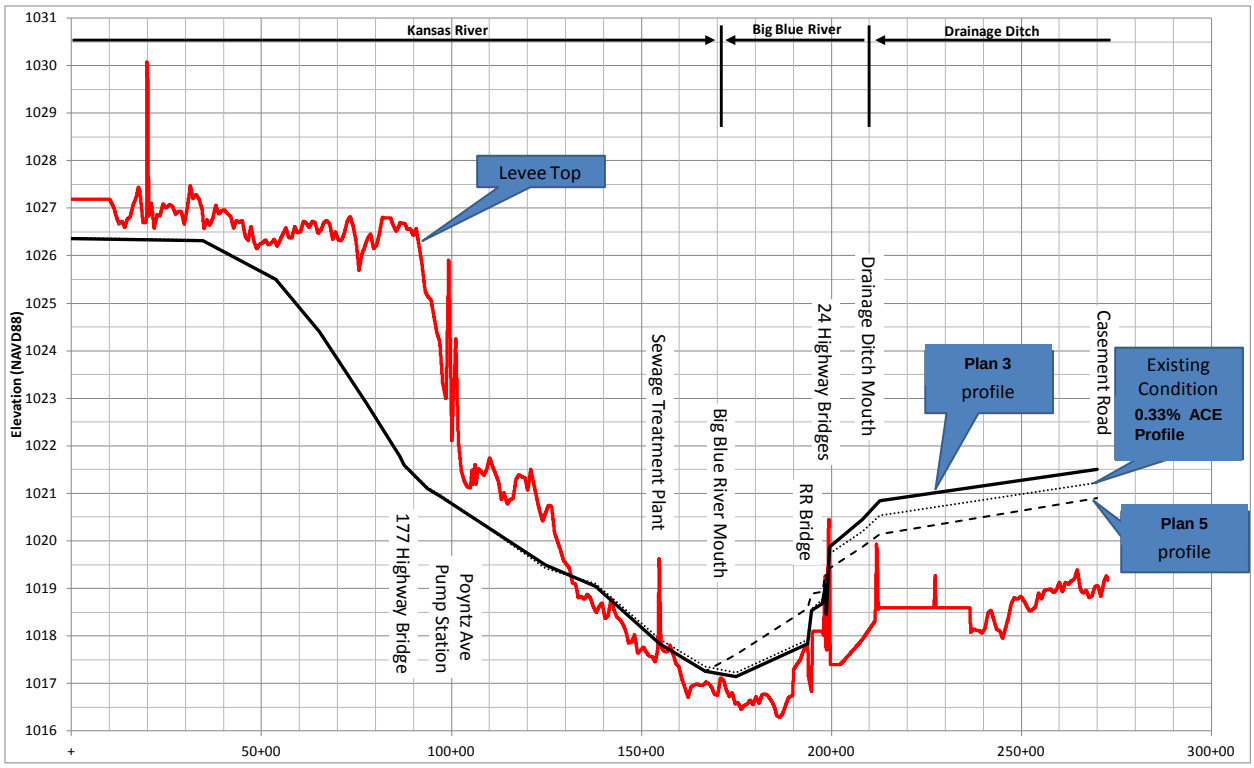


Figure 2: Hydraulic profiles of Plan 3, Plan 5, and existing conditions 0.33% (1/300) ACE floods

2-2.2. Levee Raises

Levee raises described in Table 1 were hydraulically evaluated for alternatives Plan 2, Plan 3, Plan 4, Plan 5, as well as Early Formulation Plans A and B,. Each raise includes the height above the corresponding existing top of levee that is required to account for the hydraulic impacts of raising the levee. However, the Plan 5 levee raise has the same discharge level of protection as the Plan 3 alternative profile shown in Figure 2 with an additional levee raise (about 0.5 feet) between the mouth of the Big Blue River and the bridges that cross the Big Blue River. Due to the complexity of the Big Blue River and the drainage ditch, a conservative approach was used to estimate the hydraulic profile at the upstream terminus of the levee, at approximate station 273+00. The energy grade line from the shortcut on the Big Blue River at river station 1838.18 in the hydraulic model was used to assign the profile elevation at approximate station 273+00 to appropriately account for the limitations of the one-dimensional hydraulic analysis in this reach. Table 1 shows the resulting levee raises for each of these alternatives.

Table 1: Levee Raises and Lengths to Existing Levee for Alternatives Array

Alternative	Description	Average Raise (ft)	Maximum Raise (ft)	Approximate Levee Raise Locations (stations)
Plan 1	No Federal Action	--	--	--

Plan 2	0.5% ACE Levee Raise	0.7	1.5	200+00 to 273+00*
Plan 3	0.33% ACE Levee Raise	1.5	3.3	131+00 to 273+00*
Plan 4	0.2% ACE Levee Raise	2.1	3.9	0+00 to 72+00 and 101+00 to 273+00*
Plan 5	0.33% ACE Levee Raise with channel widening	1.3	2.6	131+00 to 273+00*
Early Formulation Plan A	0.33% ACE Levee Raise with northern levee	1.5	3.3	131+00 to 273+00*
Early Formulation Plan B	0.33% ACE Levee Raise with northern levee and channel widening	1.5	3.9	131+00 to 273+00*

*Station 273+00 is approximate because there will be an extension to high ground for each alternative from the current levee terminus.

Hydraulic effects of levee raises along the existing levee were calculated. Nominal levee raises were estimated for the 0.5% (1/200) ACE, 0.33% (1/300) ACE, and the 0.2% (1/500) ACE floods. These raises were developed with the levee tool in the HEC-RAS geometry. The maximum increases to these three hydraulic profiles ranged from 0.2-0.6 feet. Discharges used to select top of levee heights for each alternative are shown in Table 2.

Table 2: Alternative Design Discharges for Levee Heights

Alternative	Big Blue River Flood*		Kansas River Flood*	
	Big Blue River	Kansas River	Big Blue River	Kansas River
Plan 2	115,800	142,800	84,400	150,800
Plan 3	151,700	167,300	101,000	182,200
Plan 4	167,000	194,200	115,400	229,700
Plan 5	151,700	167,300	101,000	182,200
*Levee height selection based on maximum water surface elevation resulting from these two floods				

Levee raise quantities were computed using the area end method for cost development of these alternatives. The Manhattan Levee Unit operation and maintenance manual was used to estimate the levee toe elevations, which were used for quantity calculations.

2-2.2.1. Plan 2

A levee raise for the 0.4% (1/200) ACE flood level of protection was considered along the existing alignment of the Manhattan Federal Levee Unit. Figure 3 shows the hydraulic profile of the existing conditions .4% and the hydraulic profile of the Plan 2 levee raise. This raise requires about 7,300 feet of levee between Stations 200+00 to 273+00, with a 200-feet extension up Casement Road (via a road raise) on the Big Blue River segment that is needed to meet high ground.

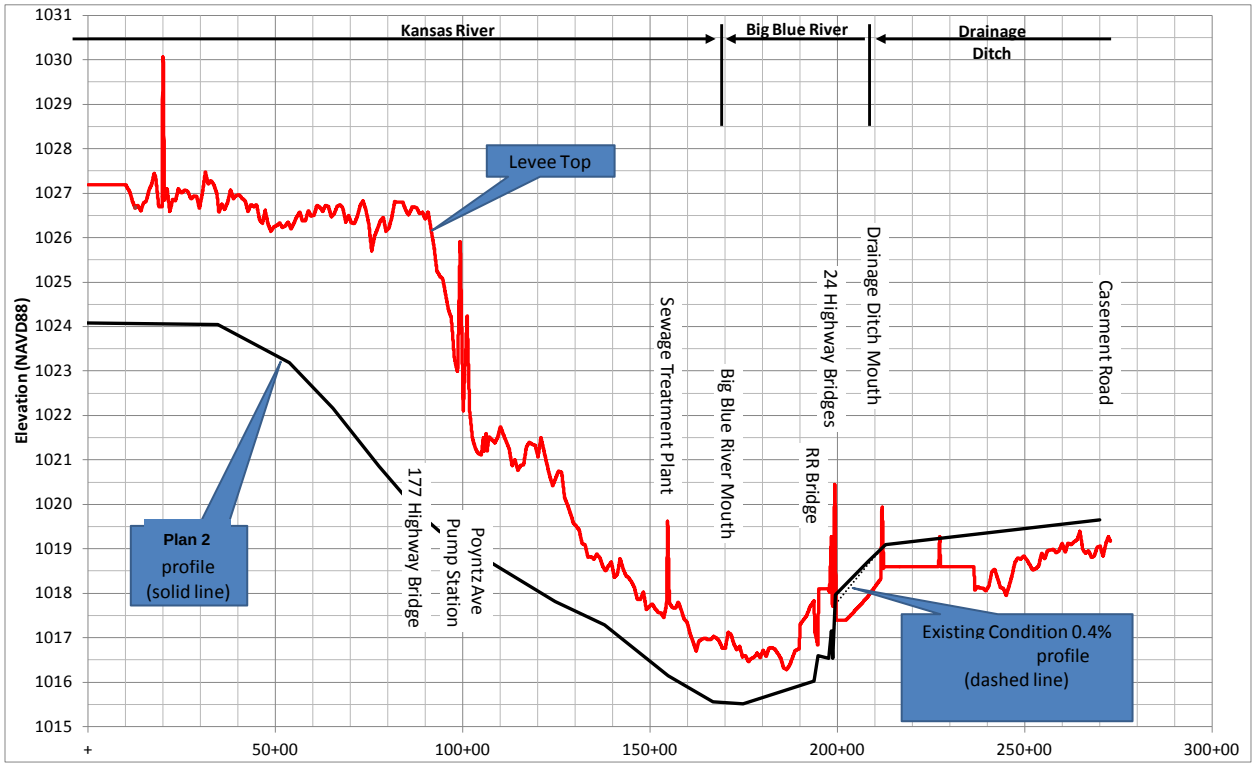


Figure 3: Plan 2 Levee Raise Hydraulic Profile

2-2.2.2. Plan 3

A levee raise for the 0.33% (1/300) ACE flood level of protection was considered along the existing alignment of the Manhattan Federal Levee Unit. Figure 2 shows the hydraulic profile of the existing conditions .33% (1/300) ACE and the hydraulic profile of the Plan 3 levee raise. This raise requires about 14,200 feet of levee between Stations 131+00 to 273+00 with a levee extension to high ground in the viscinity of Casement Road on the Big Blue River segment.

2-2.2.3. Plan 4

A levee raise for the 0.2% (1/500) ACE flood level of protection was considered along the existing alignment of the Manhattan Federal Levee Unit. This raise requires about 24,400 feet of levee from Stations 0+00 to 72+00 and from Stations 101+00 to 273+00. There will also be a 1,700ft extension along the Kansas River segment that runs parallel to the railroad to reach high ground with a levee extension to high ground in the viscinity of Casement Road on the Big Blue River segment. Figure 4 shows the hydraulic profile of the existing conditions 0.2% (1/500) ACE and the hydraulic profile of the Plan 4 levee raise.

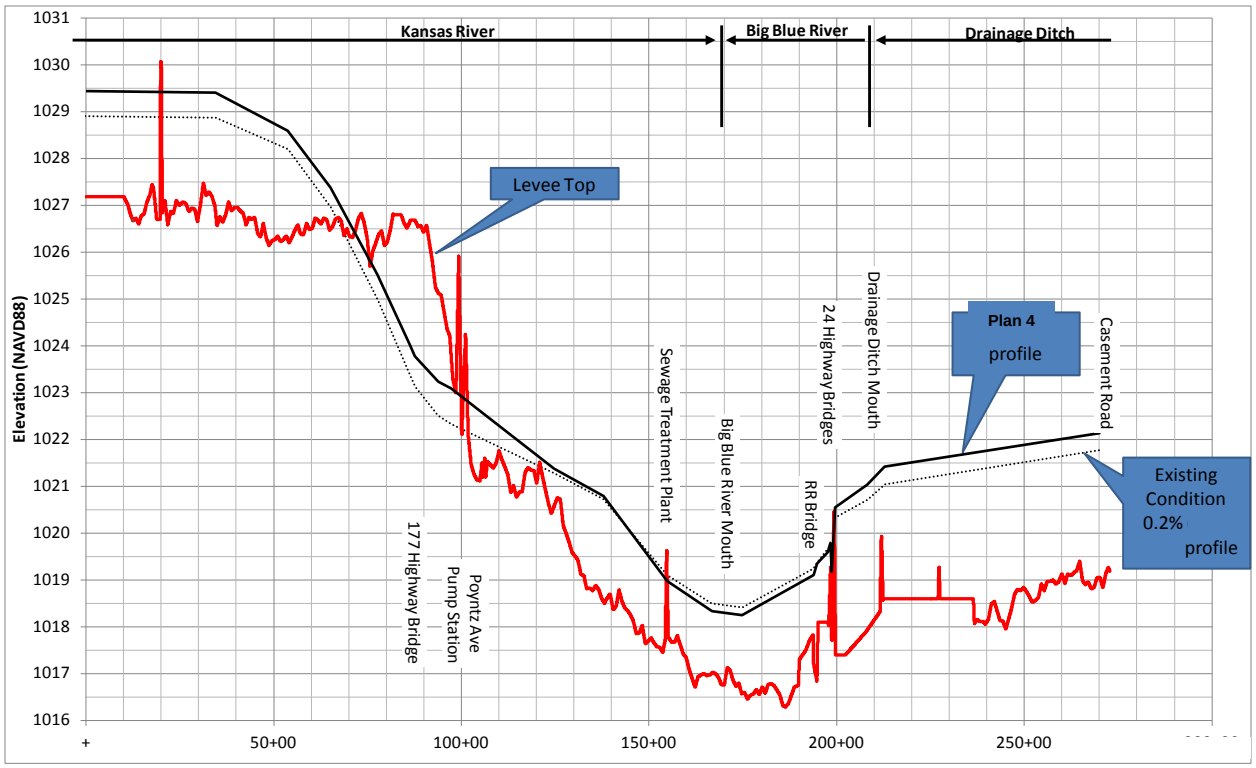


Figure 4: Plan 4 Levee Raise Hydraulic Profile

2-2.3. Considered but Screened Out Northern Levee

A new northern levee was considered and screened out for the alternatives Early Formulation Plans A and B. A preliminary 3.3 mile alignment was developed and the levee was hydraulically evaluated for cost development. The average levee height is 7.9 feet with a maximum height of 30.5 feet.

The hydraulics of a new levee were computed by modeling a levee in HEC-RAS with the levee tool in the model's geometry. The concept-level new northern levee (NNL) is on the right bank of the Big Blue River upstream of the existing Federal levee. The levee height was designed to the energy grade line of a 0.3% (1/300) ACE flood level of protection based on the future condition hydraulic profile computed in this HEC-RAS simulation.

Compacted fill quantities for the NNL were developed with the average end area method for cost development of the alternative. Quantities were based on the hydraulic model's output and the existing ground surface elevations along the levee's alignment, both of which are shown in Figure 5 below. Locations were also identified where structures would likely be required for interior drainage. Calculations assumed no topographic variation perpendicular to the levee's alignment. Other assumptions include a 10-feet wide top width with 3:1 horizontal to vertical side slopes. The existing ground surface elevations along the preliminary alignment were extracted from the 2006 LiDAR elevation dataset. The energy grade line was used to design levee heights to

account for the stages that would be observed in the overbank conveyance area where channel velocities are significantly less than those in the channel.

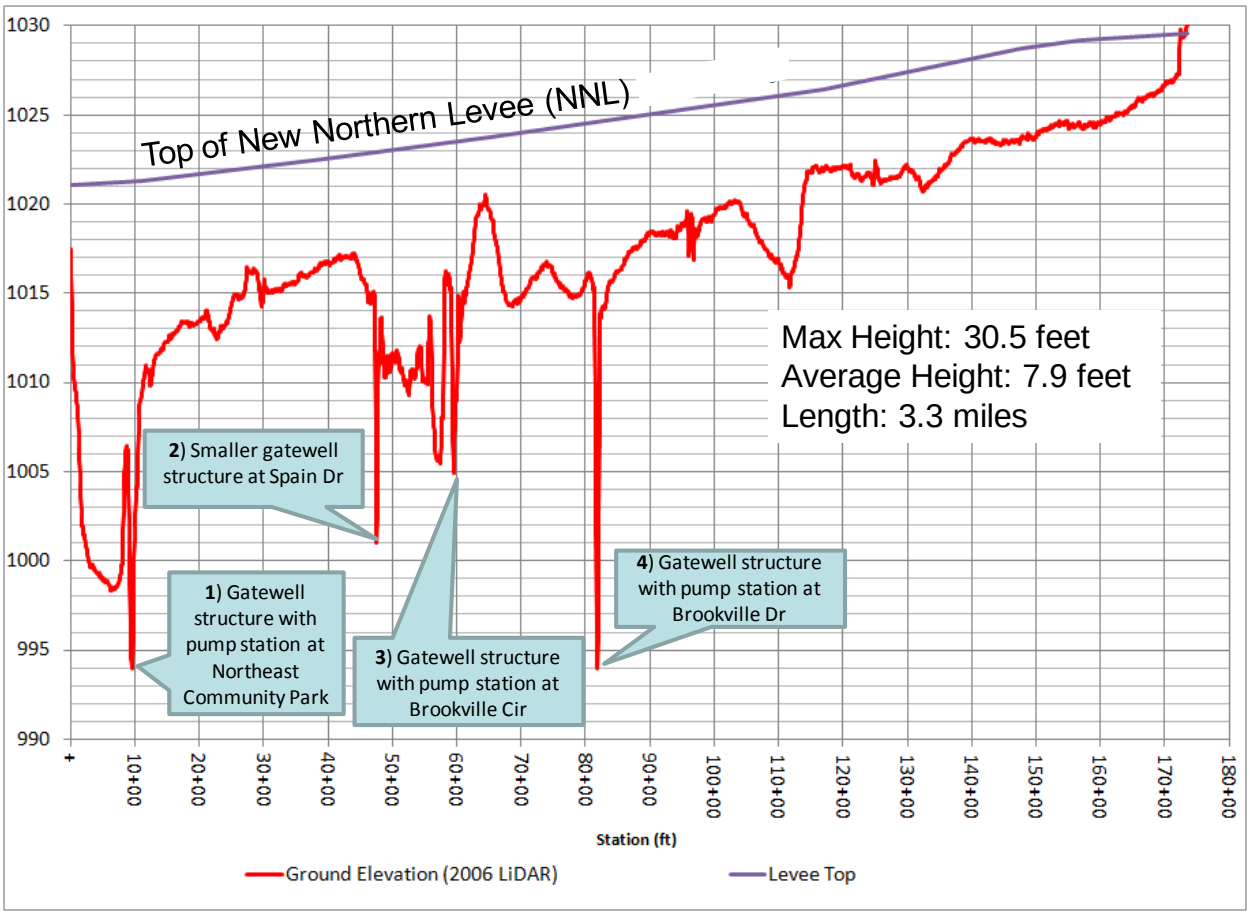


Figure 5: Profile of Screened Out New Northern Levee

2-3. Inundation Mapping

2-3.1. Existing Conditions Mapping

Inundation mapping was developed in the protected area of the Manhattan Levee unit as well as the unprotected areas near to the levee unit. Since the levee is near the confluence of the Big Blue River and the Kansas River, a blended inundation mapping approach was used to account for coincident flooding risk. For example, two simulations were made for the 1% (1/100) annual chance exceedance (ACE) flood to simulate both a Kansas River controlled flood and a Big Blue River controlled flood. Inundation maps were produced for the two scenarios and then blended to show the maximum inundation area produced by either river's control flood. Figure 6 shows the results of the 1% (1/100) ACE flood. Existing Conditions inundation maps were developed with similar methods for several flood events including the 4% (1/25) ACE, 2% (1/50) ACE, 1% (1/100) ACE, 0.5% (1/200) ACE, 0.33% (1/300) ACE, 0.2% (1/500) ACE, 0.13% (1/750) ACE events as well as the calibrated flood event of July, 1993. Enclosure 1 includes figures of the results of these other mapped flood events.

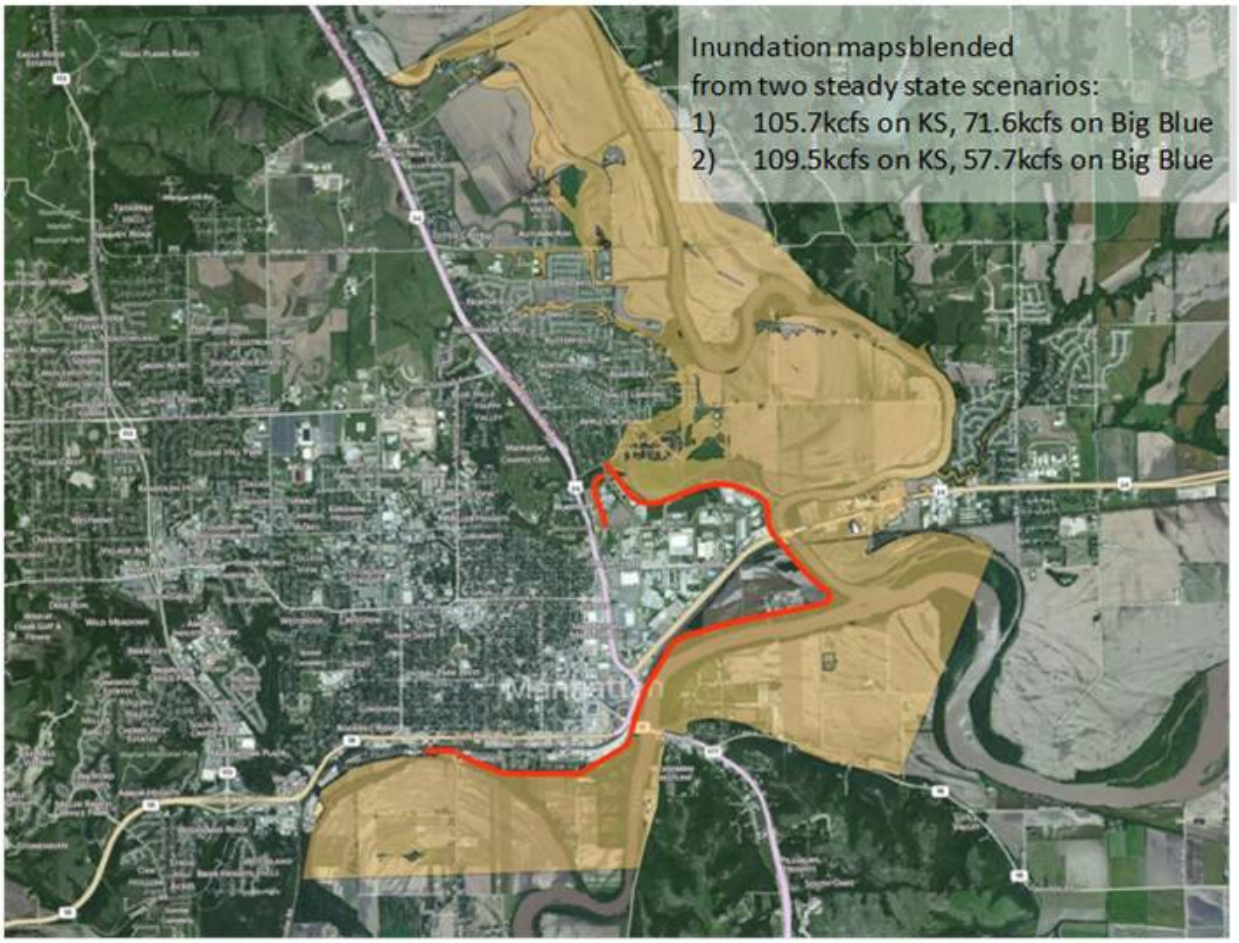


Figure 6: Inundation Extents of the EC 1% ACE flood

2-3.2. Future Conditions Mapping and Induced Damages

Detailed FC inundation maps shown in the mapbook section were developed with HEC-RAS Ras-Mapper Software. Each alternative has very small differences from the EC inundation maps except for the leveed area, which will have less inundation upon implementation of the preferred alternative. Since the recommended plan (Plan 3) alternative is a levee raise with no other hydraulic modifications to the channel or bridges, the EC inundation maps will accurately depict FC inundation maps at and below the existing level of protection, which is about 1 foot below the 0.5% (1/200) ACE flood and about 3 feet above the 1% (1/100) ACE flood.

North of the existing levee in Manhattan, there is a large area of residential development within the existing FEMA 0.2% (1/500) ACE floodplain, currently unprotected by any kind of structural flood protection project. Hydraulic analysis determined that the area north of the existing levee would likely experience slightly raised water surface profiles, if the current levee is raised. With the Plan 3 alternative, the greatest change in water surface elevation in this area would be less than 5 inches, which does not appear to significantly increase the inundated area, but rather increase

flood depths. This increase of 4.8 inches also occurs in an area that is largely undeveloped land. The bulk of the developed residential area would actually experience an estimated 1-2 inch increase in profile under the Plan 3 alternative as shown in Figure 7. The economic analysis appendix includes the economic effects of induced damages of each alternative. If the preferred alternative is a levee raise, induced damages are not expected below the EC level of protection because the hydraulic response during floods below the EC level of protection should not change.

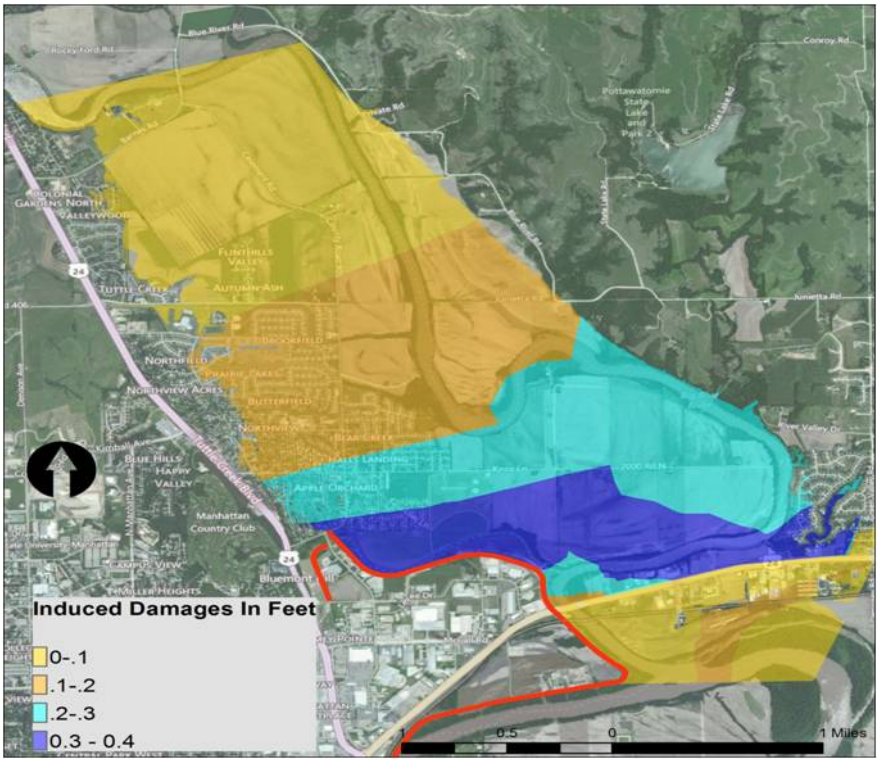


Figure 7: Modeled induced damages of the Plan 3 alternative

2-3.3. Leveed Area

Inundated areas were estimated behind the levee for modeled floods that overtop the levee. This was conducted with hydraulic model output, engineering judgment and high definition LiDAR elevation imagery in the protected area. HEC-RAS version 4.1 steady modeling was used to develop the Manhattan Feasibility hydraulic analysis and this model is based on steady-state calculations that do not account for volume and hydrologic routing if the levee is overtopped.

A modified backwater approach was combined with a “bathtub effect” approximation to develop inundation areas behind the levee. The RAS Mapper module of HEC-RAS version 4.1 was first used to estimate the inundated area behind the levee based on the hydraulic profiles shown in Figure 8. Then the inundated area was modified to include all areas in the leveed area below the maximum water surface elevation on the Big Blue River, which is near the levee’s terminus at Casement Road. Contours based on LiDAR imagery were developed and traced with ArcGIS software to

modify the modeled inundated area. Figures in Enclosure 1 show the inundation areas behind the levee in flooding events that overtop the levee.

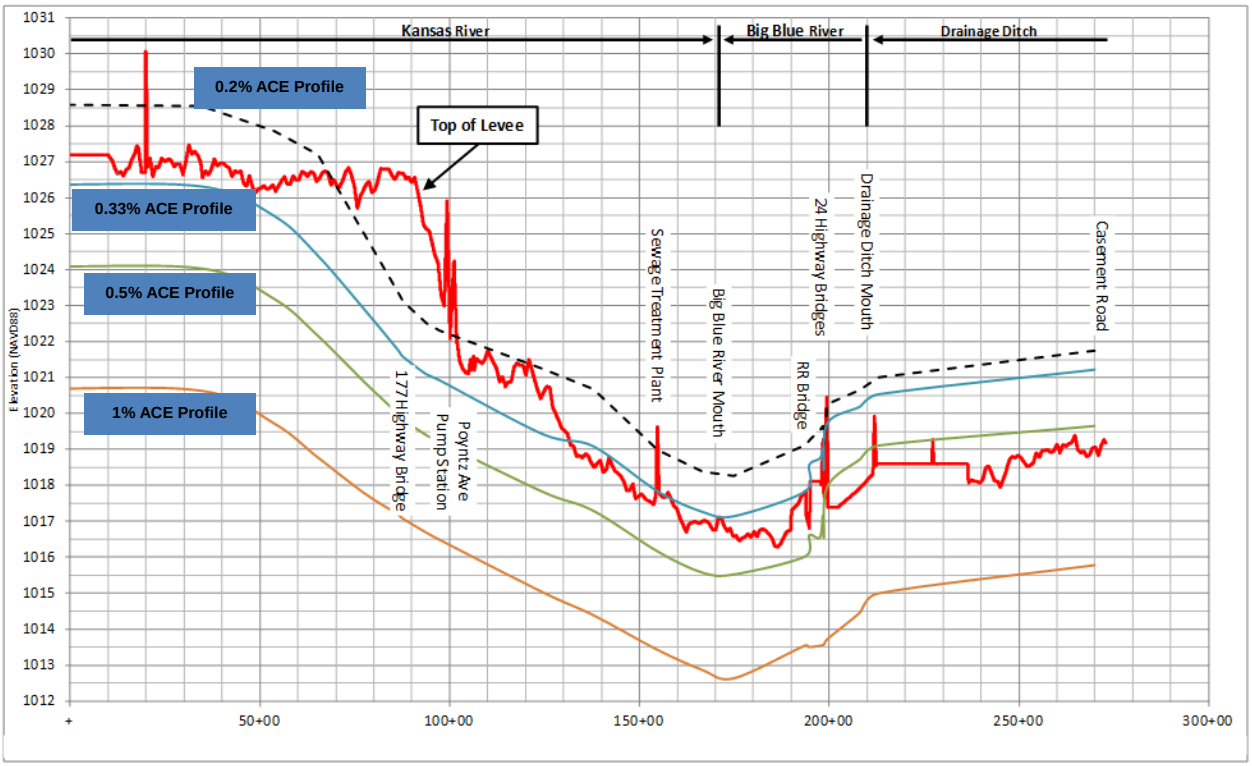


Figure 8: Hydraulic profiles at existing conditions

2-3.4. Anticipated Changes in the FEMA Regulated Flood Plain

The existing FEMA published Flood Insurance Rate Maps (FIRM) were developed in 1981 with legacy hydrologic information dating back to the 1960s. Updated hydrology used in the EC Feasibility hydraulic model incorporated over 40 years of additional hydrologic information about the basin than that used during the old FEMA flood insurance study. This analysis produced results that significantly changed the 1% (1/100) ACE flood on the Big Blue River, which are most substantial in the unleveed area north of the existing levee shown in Figure 9. Along the Big Blue River in Riley County, up to 317 structures are estimated to be added to the 1% ACE flood plain. This analysis is based on a calculation with ESRI software and 2010 census data, although this is only a preliminary estimate and revisions to the FIRM will be finalized by FEMA. It is important to recognize that Pottowatomie County is defined as unincorporated on the left bank, which is why there are not more boundaries on the left bank shown in Figure 9.

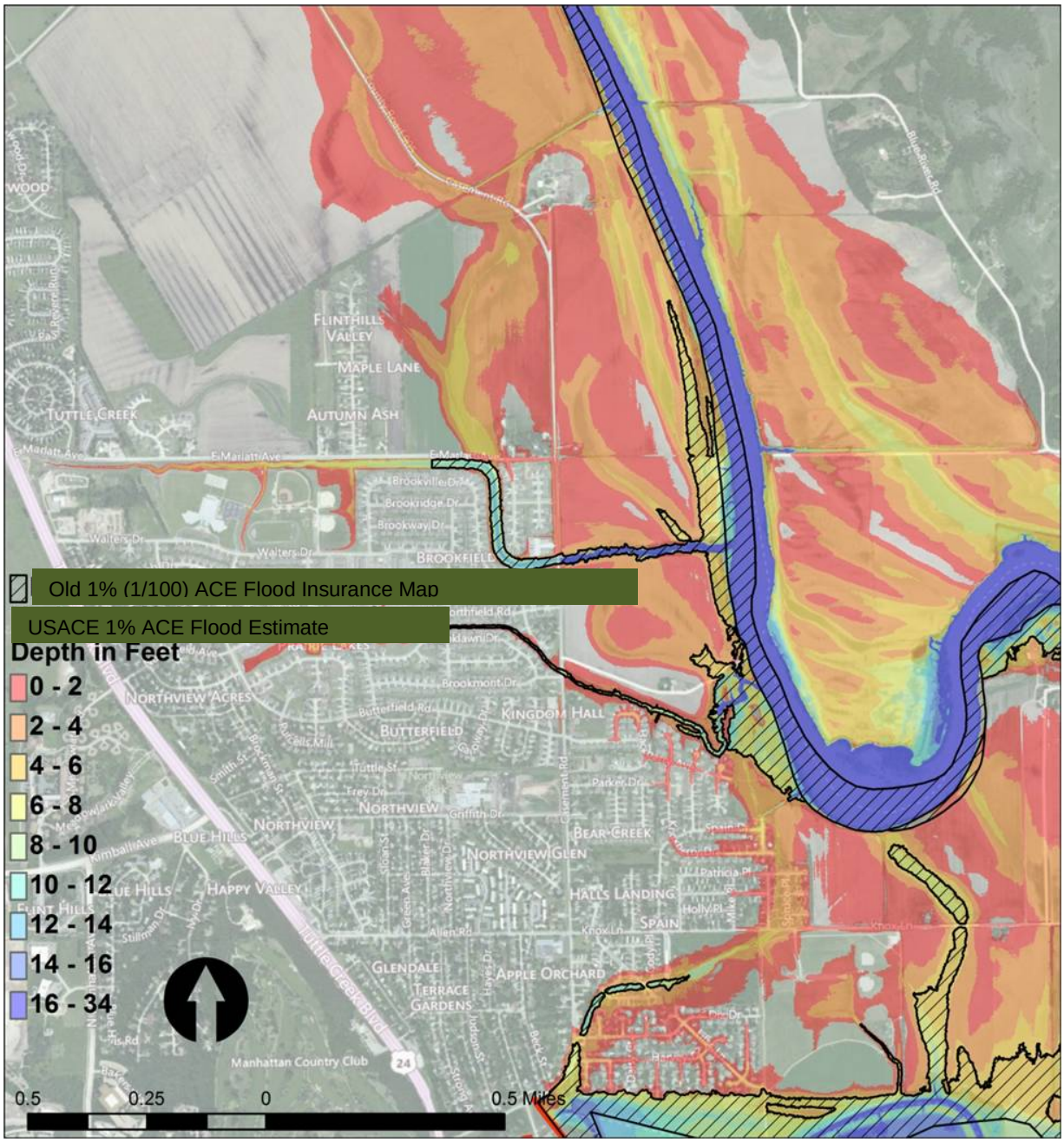


Figure 9: Estimated Changes to the 1% (1/100) ACE Flood Plain on the Big Blue River

2-4. Levee Modeling Approach

After developing the set of alternatives, updates were made to the existing conditions hydraulic model to allow better comparision of these alternatives and to improve inundation mapping at extreme events. Cross sections were extended across the leveed area to the bluff with permanent ineffective flow areas placed in the leveed area at an expected exit elevation at a low elevation point in levee near the confluence to account for the additional conveyance in each extended cross section. This exercise reduced the stages of simulated floods near the levee that exceeded the 0.5% (1/200) ACE flood.

Changes in the existing conditions hydraulic model were made to improve accuracy of flood inundation in the levee's protected area and to develop a way to evaluate induced damages caused by a preliminary levee raise. Figure 10 shows a plan of the previous existing conditions cross section locations with cross section extensions as dashed lines.

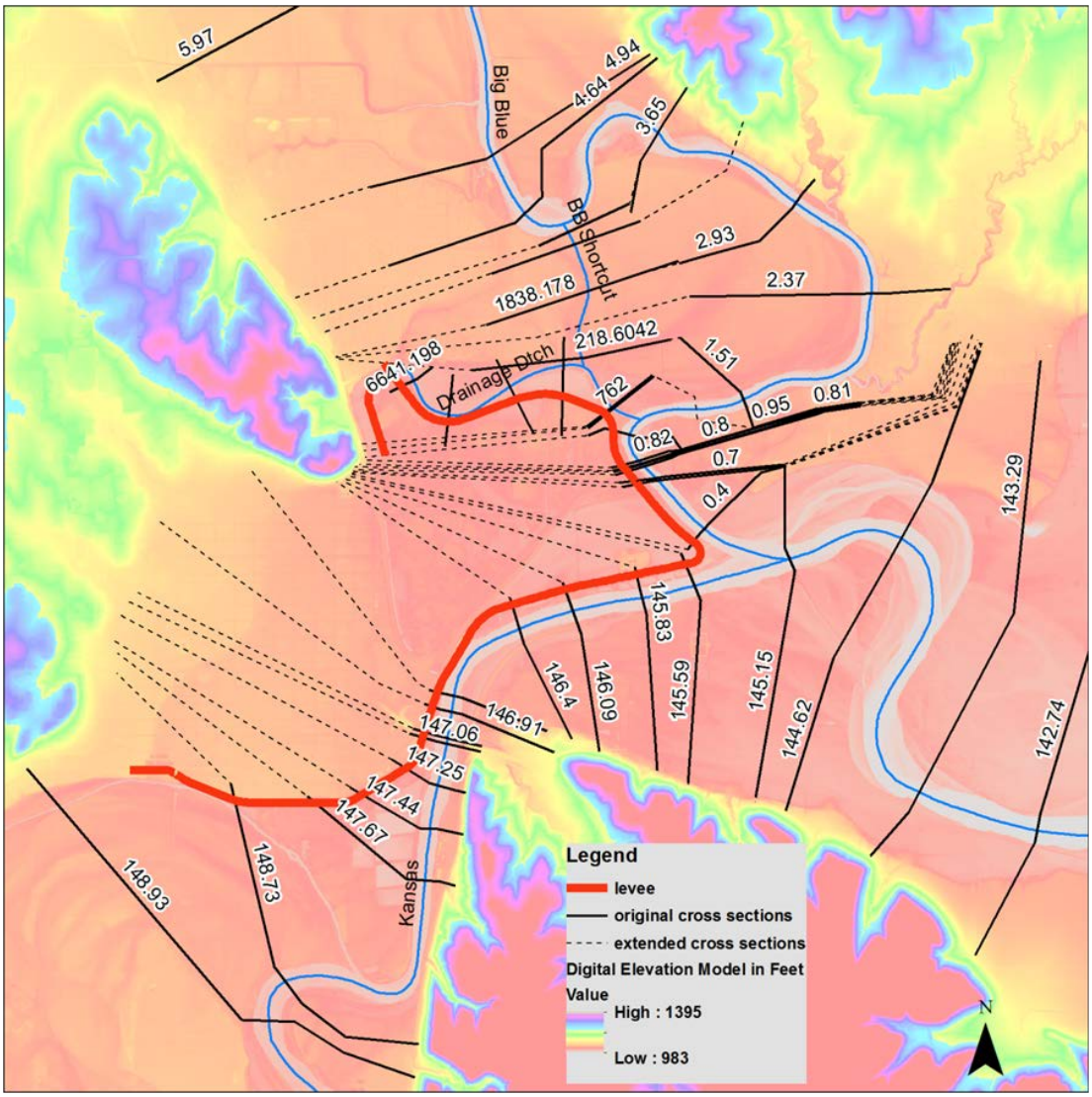


Figure 10: HEC-RAS cross section extensions and additions with LIDAR imagery in background

High ground in the leveed area on the Kansas River and Big Blue River does not occur perpendicular to the direction of flow near the confluence of the Kansas and Big Blue Rivers. This issue affects the one-dimensional assumption in HEC-RAS that cross sections are located along a river run perpendicular to the river centerline and flowpaths. The violation of this assumption in the model over-accounts conveyance area near the confluence. In these instances, permanent ineffective flow areas were applied in the leveed area at the edge of each affected cross section. Figure 11 illustrates this decision.

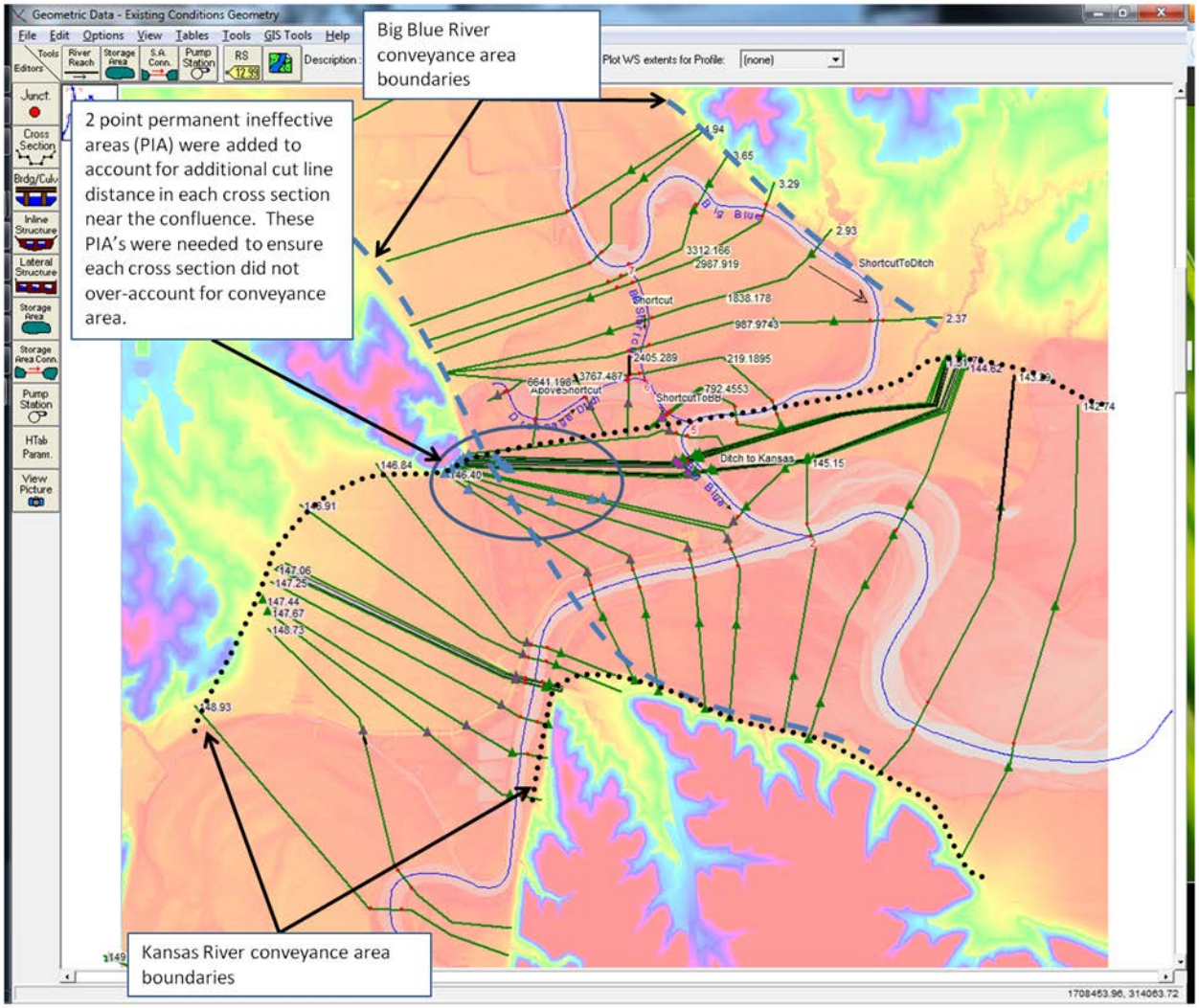


Figure 11: Plan of HEC-RAS Geometry with conveyance area boundaries

The methodology to choose where to add the permanent ineffective areas in the leveed area involved identifying cut lines that are longer than the distance approximately perpendicular with the conveyance area boundaries shown in Figure 11. The difference from the conveyance area boundaries and cross section length was measured and permanent ineffective areas in the leveed area were placed at that measured distance.

The existing condition model was updated with the results of this analysis to use as a baseline from which future condition alternative analyses were conducted.

2-5. Fragility Curve Analysis

Structural and geotechnical fragility curves were developed at several locations along the levee where vulnerabilities exist in the existing levee as shown on Figure 12 below. Green lines show underseepage fragility curve locations, black squares represent structural fragility curve locations, blue lines represent index point locations,

and the top of levee is shown in red. These fragility curves were translated to economic index points and cumulated for the economic analysis.

Slope stability fragility curves were not included in the study because they added unrealistically high probabilities of failure. The assumption of steady state phreatic surface gradients through the levee that would lead to slope stability failure require several weeks of saturation with water surfaces at the levee top for several months. Historical records suggest a relatively short flood durations on the Big Blue River. In July 1993, the flood of record since the levee's construction, exceeded the bank full discharge of 35,000 cfs for 16 days at the USGS gage 06887000 located on Casement Road.

The methodology to combine fragility curves involved translation of each fragility curve along the existing condition hydraulic profiles to the economic index points shown in Figure 12. Fragility curves were translated to index points on the Kansas River to two index points at river mile 146.84 and 147.67 and on the Big Blue River to two index points at river miles 0.95 and 0.82. Curves originating from the Kansas River were only translated to index points on the Kansas River while curves originating from the Big Blue were only assigned to index points on the Big Blue River. In an exception at the confluence of an underseepage fragility curve between stations 165+12 to 173+63, the curve was assigned to both Big Blue River and Kansas River index points. The four index points are cross sections in the Feasibility hydraulic model. The cumulative probability was then computed with the translated fragility curves at each index point. At each index point, probabilities of failure were combined with the cumulative probability equation known as the Unimodal Bounds Theorem (USBR 2011) in Equation 1. Independent fragility curves required interpolation of probability between calculated ordinates in order to calculate cumulative probability with other fragility curves at the same elevation.

$$P_{cumulative\ at\ elev\ i} = 1 - (1 - P_{1\ at\ elev\ i}) \times (1 - P_{2\ at\ elev\ i}) \times (1 - P_{3\ at\ elev\ i}) \times \ldots \times (1 - P_{n\ at\ elev\ i}) \quad \text{(Eq 1)}$$

The resulting cumulative probability of failure versus elevation curves are shown in Enclosure 2. The economic analysis then input the cumulative fragility curve to the Hydrologic Engineering Center Flood Damages Analysis (HEC-FDA) computer program model.

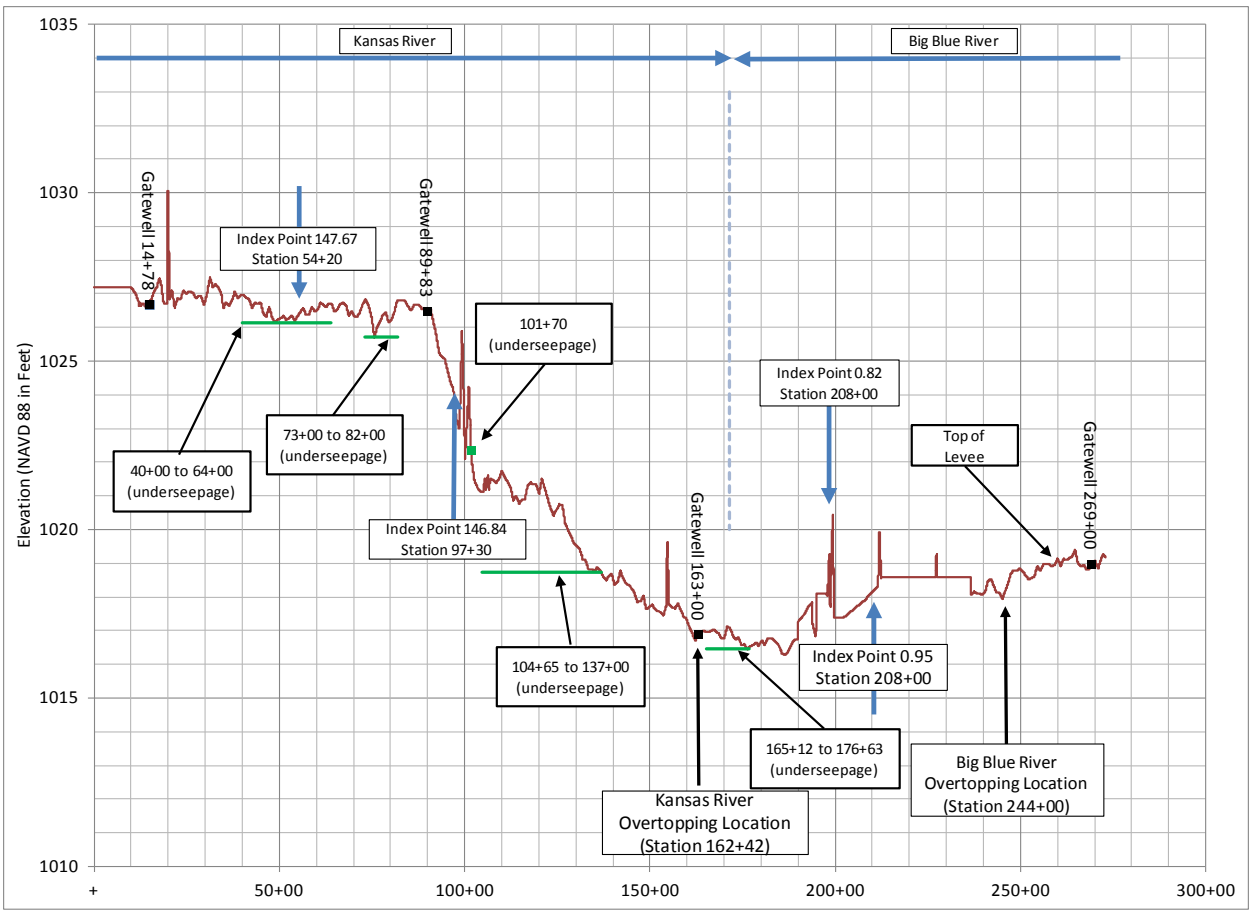


Figure 12: Levee Profile with Fragility Curves, Index Points, and Expected Overtopping Locations

2-6. Future Without Project Conditions (Baseline)

Overbank and in-channel vegetation in the floodway near the levee system is not expected to significantly change in future conditions. The community has been progressive in mapping flood risks along its streams and is currently working to develop floodplain management plans with engagement of local governments through the Corps Silver Jackets Program. Accordingly, any new hydraulic flow obstructions or conditions in the floodway such as buildings and woody vegetation are assumed to either be minimal or to not cause a raise of the hydraulic profiles along the levee. An analysis was conducted that assumed heavy forested overbanks along the left bank of the Big Blue River in the proximity of the levee, which resulted in a raise of the 0.33% ACE profile by a maximum of 0.8ft. Although no plans are currently known to exist, future changes to the existing highway and railroad bridges or highway causeways in the floodplain should be identified as a risk to the levee in the floodplain management plan and treated with caution.

The Manhattan levee system was originally designed for vegetation along the overbanks to be represented by an n-value of 0.05, which is typical of scattered brush and mature field crops. A maintenance measure of the original design was for the

removal of wild growth and drift deposits in the floodway. Currently, the overbank n-values assigned in the existing conditions hydraulic model range from 0.04, reflective of row-crop fields, to 0.13 to represent heavy stands of timber. An in-channel n-value of 0.035 was used in the existing conditions hydraulic model instead of the original design channel n-value of 0.025. Areas difficult or infeasible for the levee district to maintain due to land ownership and right of way, including areas across the river from the levee, appear to have already grown into dense forest as reflected in the existing conditions model. Areas of the floodplain near and downstream of the levee system with active agricultural fields are expected to continue to be maintained in crop production.

The future conditions without project represents the probable stage-discharge relationship at a selected future date 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. For the purposes of this study, the future conditions have been defined as conditions reasonably expected to be present in 2060. A critical assumption in the future conditions analysis is that hydrologic conditions along the Kansas River and the Big Blue River are relatively static and that flows used in the existing conditions study will be used in the future conditions analysis. Several factors exist that may influence future condition hydrologic conditions on the Big Blue River and include urbanization in Manhattan, Kansas as well as any unforeseen modification to regulation manual for the Tuttle Creek Reservoir. The future hydrologic conditions on the Kansas River will not likely change significantly due to its watershed's large size and the regulation it experiences from several reservoirs upstream of Manhattan, Kansas. Additional information on expected hydrologic trends can be found in section 1.7, Climate Change Considerations.

The Kansas River has been subject to many natural processes that have affected river stages for both low and high frequency flood events. These long-term changes in the stage-discharge relationship of the river are referred to as stage trends. Since the period of record is small at the Kansas River gage at Manhattan (USGS 06879820), the Wamego river gage, which is approximately 19 river miles downstream of Manhattan, was used for this analysis. The stage trends on the Kansas River at Wamego, Kansas are shown on Figure 13 for annual peak flows between 1914 and 2008 that range from 5,000 cfs to 400,000 cfs.

The period of record used in Figure 13 includes instantaneous annual peak discharges from 1914 to 2008. Unregulated discharges were recorded at the Wamego gage from 1914 until Tuttle Creek Dam was constructed in 1963. When discharge is less than 73,000 cfs the stage trend decreases about 1.6 feet between 1914-1950 and 1951-2008. The higher flow data series (greater than 73,000 cfs) generally reflect data collected during the floods of 1935, 1951, and 1993. These floods are the only floods in recent history to produce significantly higher discharges than other flooding events in the period of record and there is not enough evidence to suggest a stage trend occurs at large discharges. Because there is insufficient evidence of stage trend at high discharges, significant hydraulic uncertainty exists when predicting the future rating curve for the gage in Wamego, Kansas.

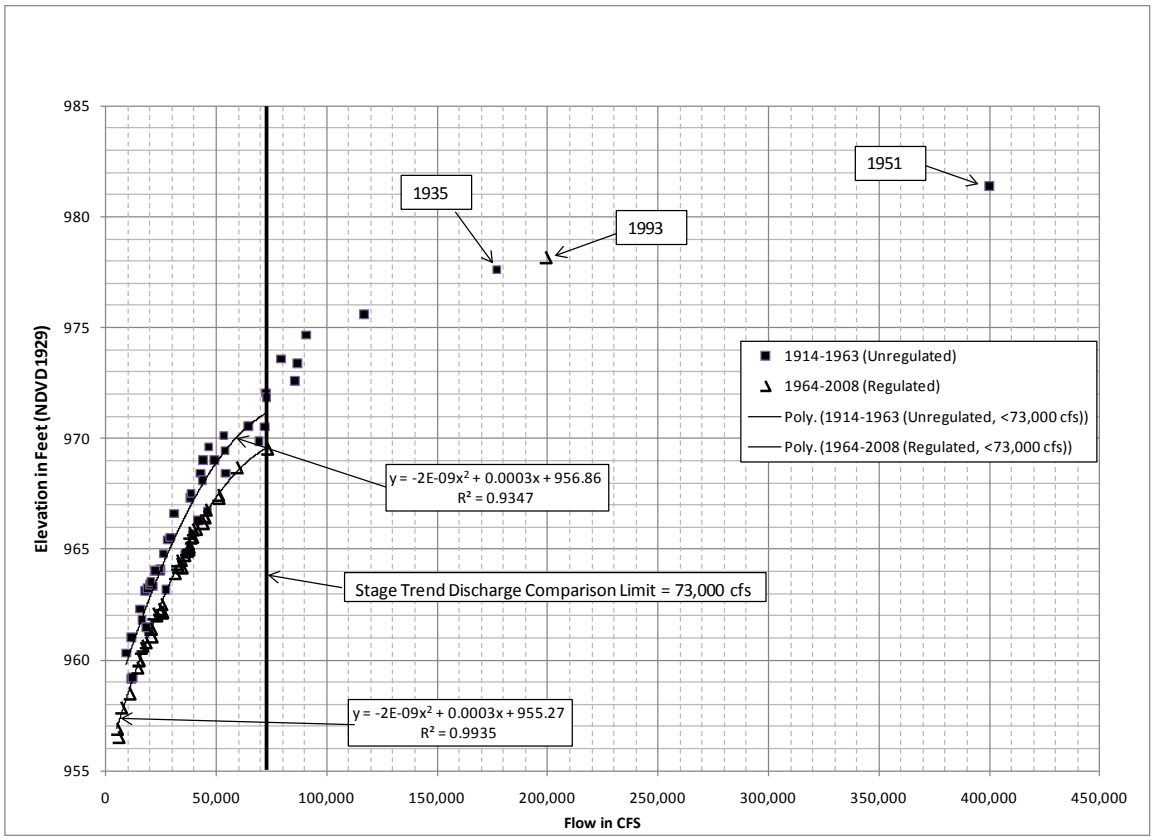


Figure 13: Stage Trends on the Kansas River at Wamego, Kansas

Therefore, based on available information, evaluation of trends in climate, stage trends, and expected floodplain management, the existing conditions model was assumed to be adequately representative of the future without project conditions.

2-7. Climate Change Considerations

The climate of northeast Kansas near Manhattan, Kansas trends toward a continental weather pattern of cold winters and hot, humid summers. The average temperature in 2013 at Topeka, KS (which represents the northeast portion of Kansas) was 60 degrees. The average high temperature was 73 and average low temperature was 47. The average yearly precipitation was about 37 inches of moisture.

A model of future conditions for the central plains of the United States was created by the NOAA National Environmental Satellite, Data and Information Service in a report issued in January 2013. This report is an assessment of Climate Trends and Scenarios into the next 50 to 100 years. The report cites that over the past period of record for the region of northeast Kansas, both temperature and precipitation has trended above normal, especially over the last 50 years. To account for climate change in the meteorological conditions of northeast Kansas, the future forecast of conditions in the region takes into consideration the past temperature and precipitation records, and then considers future modeled conditions in the area through 2070. According to the

NESDIS report, a warming trend of about 3-5 degrees F and a precipitation trend very slighty toward wetter conditions can be expected through the next 50 years although significant uncertainty is expected with these estimates.

Huntington (2006) notes that a proportionality has been observed in recent meteorological models between temperature and precipitation where precipitation increases by about 4.3% per degree Kelvin. In an effort to evaluate the hydraulic effects of this relationship during the 0.33% ACE event in 50 years, it was considered that in the worst case scenario, flood discharges may increase by no more than 15% if temperatures increase by 3-5 degrees F (1.7-2.8 degrees K). The analysis also assumes that all additional precipitation becomes direct runoff. When these flows were hydraulically modeled at the levee, the 0.33% event profile raised by an average of 1.0 feet and a maximum of 1.8ft.

An additional local level analysis was performed to do a first order detection of any changes in floods that have been occurring over the observed record period. For the period of record from 22 July 1959 through 21 January 2010 daily observations of discharge for inflow into Tuttle Creek Lake were analyzed in two ways. The first was to identify the annual maximum daily discharge from the record and perform a linear regression on the data to determine if there was a statistically significant slope (Figure 14). The second was to identify the largest annual three-day maximum discharge and perform a linear regression on the data to determine if there was a statistically significant slope (Figure 15). Both analyses resulted in a relatively small but statistically significant trend towards smaller annual maximum daily discharges as well as annual maximum three-day average discharges.

Impacts associated with changes in flood frequencies to the project are directly related to whether the floods would be increasing or decreasing. Increases in flood magnitudes may increase the benefits of the proposed project in that there would be greater benefit achieved by reducing overall flood risk. However, increases in flood magnitudes would both alter the project performance in terms of whether it is satisfying the 1% Conditional Non-exceedance Probability or chosen performance criteria, as well as increase maintenance costs or repairs associated with potentially more frequent overtopping events then were originally assumed. Decreases in flood magnitudes may decrease assumed project benefits as the project would not be protecting against the greater floods assumed during the study. However, project costs in terms of maintenance or repairs may be reduced as the project will not be subjected to the level of floods assumed during this study.

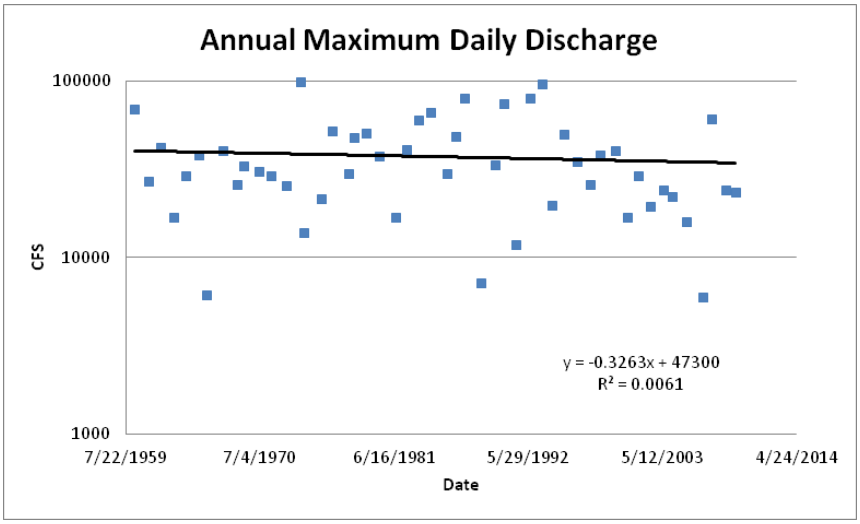


Figure 14: First order trend detection on annual maximum daily inflows into Tuttle Creek Lake. A negative slope is determined to be statistically significant at the p<0.05 level.

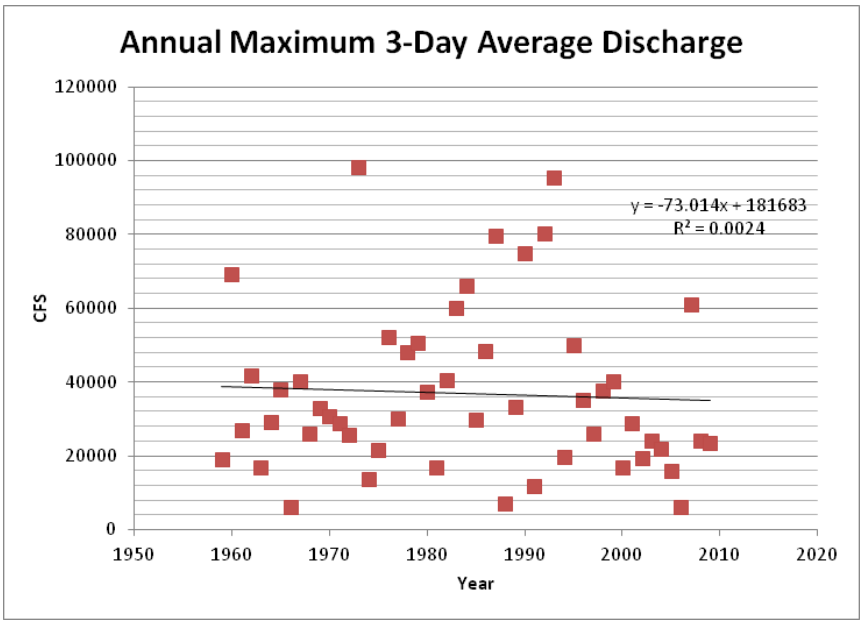


Figure 15: First order trend detection on annual three-day maximum daily inflows into Tuttle Creek Lake. A negative slope is determined to be statistically significant at the p<0.05 level.

The objective of the hydrologic analysis conducted for the USACE Manhattan Feasibility Study is to update estimates for frequency flooding events, which were last investigated with a period of record that ended in the 1960s. Since that time, flooding events have occurred that include 1993, the flood of record since significant flood control projects have been operational in the basin. This new hydrologic information has contributed to improved estimates for extreme flood flow frequencies. As time progresses and additional flood events occur, these estimates may need future updates. It is expected that climate change in the next 50 years may increase

frequency flows over the study basin, but these increases are being treated in this evaluation to be retained within the bands of uncertainty in the Existing Condition Feasibility hydrologic analysis.

2-8. Flood Stage Forecasting During Gatewell Construction

The levee will require emergency closure structures (ECS) to adequately protect against flooding during gatewell replacement construction because of openings in the levee during construction. The probability of occurrence of an extreme flood during construction is expected to be very low because of the relatively short time of vulnerability during the gatewell replacements. However, from a hydrologic perspective, there is a remote but highly consequential risk that a large flood may occur during construction. If earthen materials with riverside erosion protection are immediately available at the construction site, an emergency closure structure (ECS) of the levee will require about twelve hours to meet a level of protection equal to a 1% (1/100) ACE flood. Table 1 shows the required ECS height at each gatewell that will be replaced.

Table 3: Gatewell Replacements

Gatewell Station	**269+50**	**163+00**	**89+83**	**62+20**	**14+78**
Levee Segment	Big Blue	Kansas	Kansas	Kansas	Kansas
Levee Toe*	995.9	1002.9	1007.1	1008.0	1011.8
1% (1/100) ACE Elevation	1015.8	1013.0	1016.9	1019.0	1020.6
Levee Top**	1018.9	1016.9	1026.4	1026.6	1026.8
Emergency Closure Structure Height (ft)	19.9	10.2	9.8	11.0	8.8
Adjacent Levee Height (ft)	23.1	14.1	19.3	18.6	15.0

*Levee toe based on O&M drawings existing elevation centerline at centerline of levee and conversion to NAVD88 vertical datum with CorpsCon 6.1
**Levee top based on National Levee Database data and 2012 FEMA certification levee raise as-built data

2-8.1. Kansas River Forecasting

The Kansas River hydrologic response to rainfall is expected to be slow because the Kansas river basin at Manhattan, Kansas is significantly regulated and very large (45,288 square miles). The upstream portion of the basin that is regulated with flood control projects of Manhattan is about 39,740 square miles, or about 88 percent of the basin. Evidence of basin response during the 1993 flood at Fort Riley USGS 06879100 gauge on the Kansas River is shown in Figure 16. Discharge is not measured at the Manhattan gauging station but the observed discharge at the Fort Riley gauge is shown in Figure 16 to provide a glimpse of the hydrologic behavior nearby in the basin.

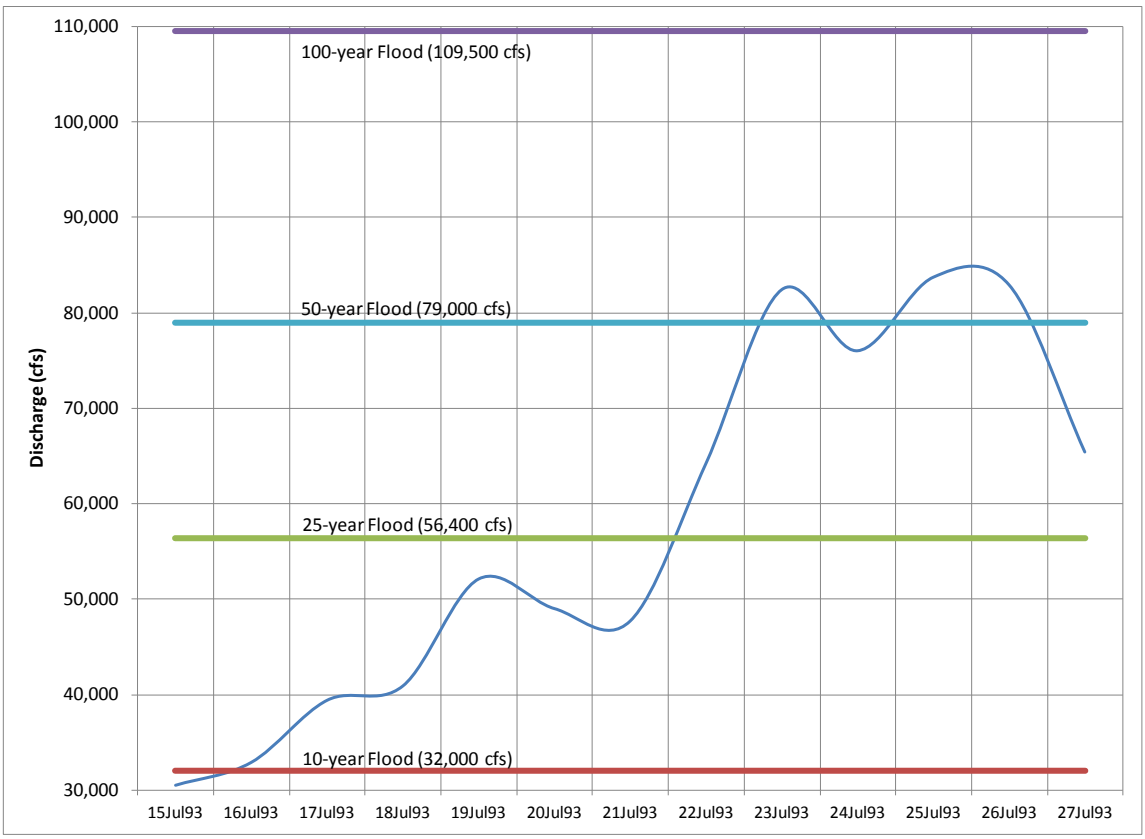


Figure 16: 1993 Flood Hydrograph on the Kansas River at Fort Riley, Kansas

The National Weather Service's Advanced Hydrologic Prediction Service (NWS AHPS) forecasts are available on the Kansas River both at Manhattan and Fort Riley, which is about 17.5 river miles upstream of Manhattan. Hydraulic analysis indicates the average channel velocity between Fort Riley and Manhattan is about 4.5 miles per hour during flooding conditions. In these conditions, the arrival time for flood waters from Fort Riley to Manhattan is about 4 hours. Local runoff area between Fort Riley and Manhattan is about 418 square miles, which is about 1 percent of the total basin area at Manhattan.

The NWS AHPS forecasts at Fort Riley and Manhattan, which are 3-4 days in advance, would provide enough time to build ECSs at gatewells under construction. The construction schedule should only allow one gatewell to be under construction at a time so multiple ECSs would not have to be made at the same time in the event of a flood. On the Kansas segment of the levee, the decision to stop construction and build a ECS to prepare the levee for flood protection would be made when the NWS AHPS forecast reaches flood stage either at Fort Riley or Manhattan.

2-8.2. Big Blue River Forecasting In Normal Conditions

The Big Blue River hydrologic response to rainfall is expected to be very predictable in normal flood control operations because of the levee's close proximity of Tuttle Creek Lake, which heavily regulates discharges on the Big Blue River at

Manhattan. On the Big Blue River, 3-5 day pool level forecasts at Tuttle Creek Lake are available from the Kansas City District Water Management Section that would provide enough time to construct ECS. The decision to construct ECSs should be made when the Tuttle Creek Lake pool elevation is forecasted to reach elevation 1125, which is the elevation at which the critical level of surveillance for Tuttle Creek Dam is underway. This is an appropriate threshold to seal the levee for active flood protection because there are 11 feet of remaining flood control pool, which is necessary for additional construction time if unfavorable wet conditions inhibit rapid closure material placement. At elevation 1136, when surcharge operations begin to govern the release schedule, much less time for downstream readiness will be available and the ECSs will already be sealed before that scenario occurs.

The hydrologic response during the 1993 flood at Manhattan on the Big Blue River is shown in Figure 17. Tuttle Creek Lake entered surcharge operations during the 1993 flood event, which led to unprecedented discharges on the Big Blue River. If an event similar to the 1993 flood occurs again during gatewell replacement construction, it is assumed that the ECSs would be constructed before surcharge operations at the lake occurred.

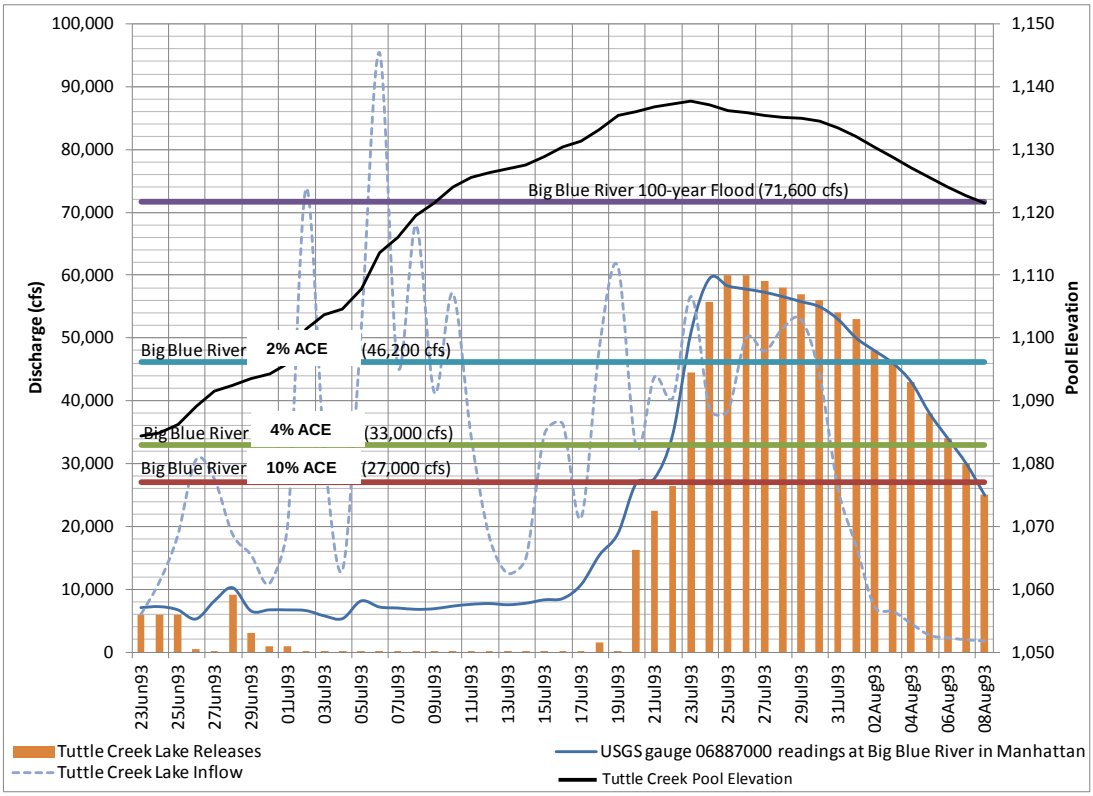


Figure 17: 1993 Flood Hydrograph and on the Big Blue River at Manhattan, Kansas Overlaid with Tuttle Creek Inflow and Pool Level

2-8.3. Big Blue River Forecasting In Surcharge Conditions at Tuttle Creek Lake

2-8.3.1. Background

Risk warning time at Manhattan, Kansas is largely dependent on releases from Tuttle Creek Lake, which is the upstream flood control project on the Big Blue River. The time for discharge releases from Tuttle Creek Lake to travel to Manhattan, Kansas is about 1-2 hours because the dam is less than 10 river miles from Manhattan. Releases that would cause significant downstream damages will occur in surcharge operations. These releases are primarily dependant on inflow amounts into the lake because there is no available space in the flood control pool to detain inflow. Surcharge releases are based on the last 1 or 2 hour inflow calculation, whichever is greater, and the gates are set to that release amount. The gates are set at that point or go higher if inflow increases based on the subsequent 1-2 hour inflow forecast computed hourly during the surcharge release period. Due to the relatively short time that is available to forecast inflows to the lake and the short travel time for water to travel from Tuttle Creek Dam to Manhattan, the actual risk warning time to residents within the leveed area is limited. In summary, risk warning time was estimated to be approximately six hours between the time at which Tuttle Creek Lake personnel are aware of upcoming spillway releases that resemble the 0.33% (1/300) ACE flood on the Big Blue River and the time at which the leveed area is completely inundated. Accordingly, egress readiness will be of particular importance if entering into surcharge operations.

2-8.3.2. Leveed Area Inundation Timing

Existing conditions hydraulic modeling of the 0.33% (1/300) ACE flood indicates there will be about 7,300 feet of levee (between stations 200+00 and 273+00) on the Big Blue River levee segment that will overtop by a representative height of 2.5 feet. In the most conservative case, the leveed area would be inundated by the 0.33% (1/300) ACE flood up to about elevation 1021.5. Figure 18 shows the overtopping height along the levee in the event of a 0.33% (1/300) ACE flood. The total volume landside of the levee that could be filled with water up to elevation 1021.5 was determined using ESRI GIS software and 2010 Light Detection And Ranging (LiDAR) elevations. Figure 19 shows the leveed area storage volume and inundated acreage as elevation increases.

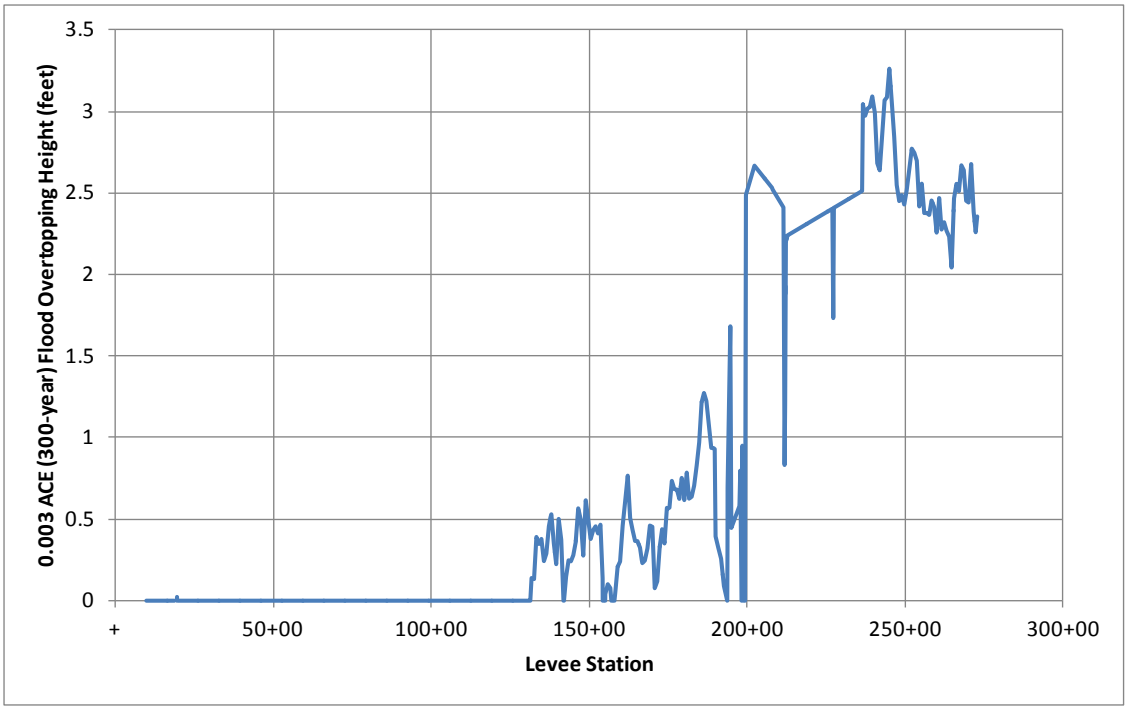


Figure 18: Levee Overtopping Height During the 0.33% (1/300) ACE Flood

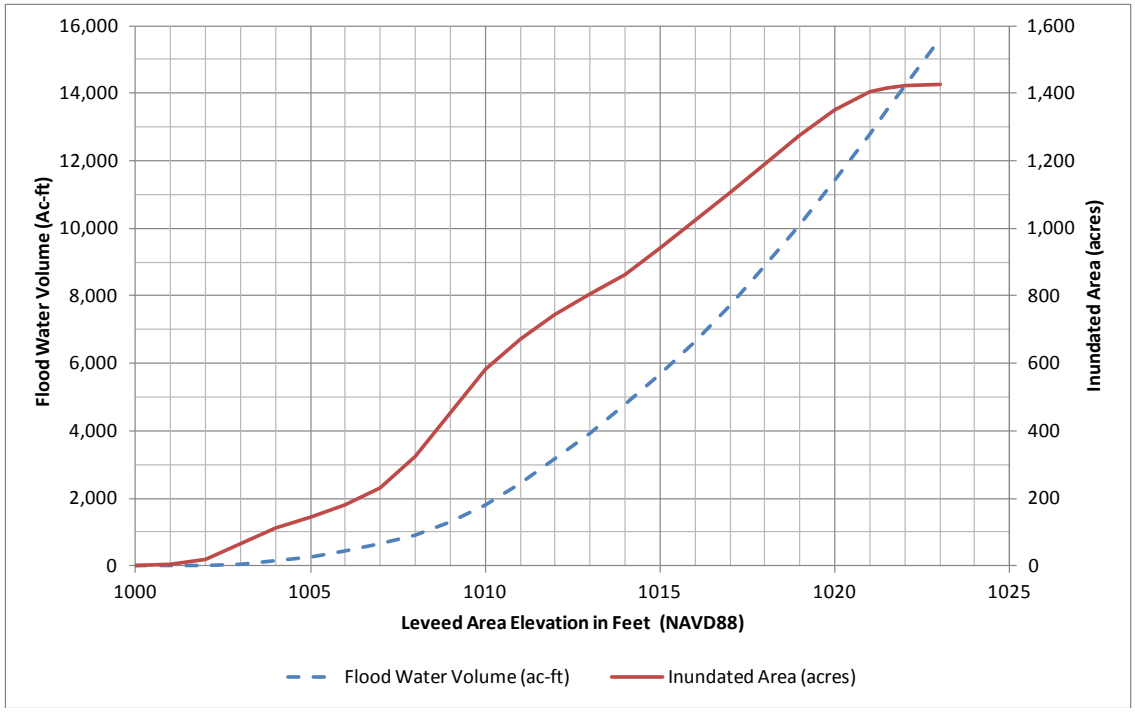


Figure 19: Leveed Area Storage Volume and Area versus Interior Elevation Curves

Hydrologic information from the Manhattan Levee Feasibility Study focused on determining flow frequencies, and accordingly, design hydrographs are not available for each return period flow to route flows from the dam to Manhattan and into the leveed

area. Accordingly, a weir equation was utilized to calculate potential peak flow rates into the leveed area in the event a 0.33% (1/300) ACE flood occurs. Assumptions of the weir equation include a constant flow depth over the levee, no levee erosion or breach, no headloss over the top of levee, an approximate rectangular shape, and a flat overflow section. A weir coefficient of 3.33 was used because it was suggested by Mays (2005). As seen in Figure 18, additional levee overtopping would occur between stations 125+00 and 200+00, however this was assumed to be minor in comparison to stations 200+00 to 275+00, and was not included in the weir calculations to provide a somewhat conservative estimate of peak flow rates into the leveed area. If the 0.33% (1/300) ACE flood occurs, about 95,000 cfs would overtop the levee and the inundated elevation of 1021.5 would be reached in about 2 hours. Approximately 13,500 acre-feet of flood water volume would be expected to fill the leveed area in this event, which would inundate about 1,420 acres.

The time to inundate the leveed area was estimated by dividing the interior volume by the estimated peak overtopping discharge computed with the standard weir equation. This simplified approach assumes a flat hydraulic profile behind the levee. It does not account for the interior drainage pumping plants to return water back to the river and it also does not account for local rainfall that would contribute to interior drainage. Additionally, a levee breach analysis that would likely decrease the time to fill the leveed area was not considered.

2-8.3.3. Egress Readiness

In the event of a surcharge release, when the pool elevation is within about 10 feet of the top of gates (in the 1026-1036 range), it is expected that there would be a heightened level of awareness in the downstream community and the City of Manhattan staff would be readying egress preparedness on the Big Blue floodplain. This would likely contribute to downstream evacuation planning and efficiency but it will not reduce the risk warning time.

2-9. Upstream Reservoir Operation Modification Alternatives

2-9.1. Background

Several upstream flood storage reservoirs exist upstream of the Manhattan Federal Levee that could have been considered for additional flood risk reduction at the Manhattan Levee. However, due to its near proximity to the Manhattan Levee, Tuttle Creek Lake was first considered for analysis. Only a minimal amount of unregulated basin area between the lake and the levee exists on the Big Blue River, and per the existing conditions analysis, the levee height is the most deficient along this segment. Accordingly, it was expected that affecting operations at Tuttle Creek would provide the most benefit for the levee of all the upstream flood control reservoirs. Figure 20 shows the flood control reservoirs that exist upstream of the levee in the Kansas City District.

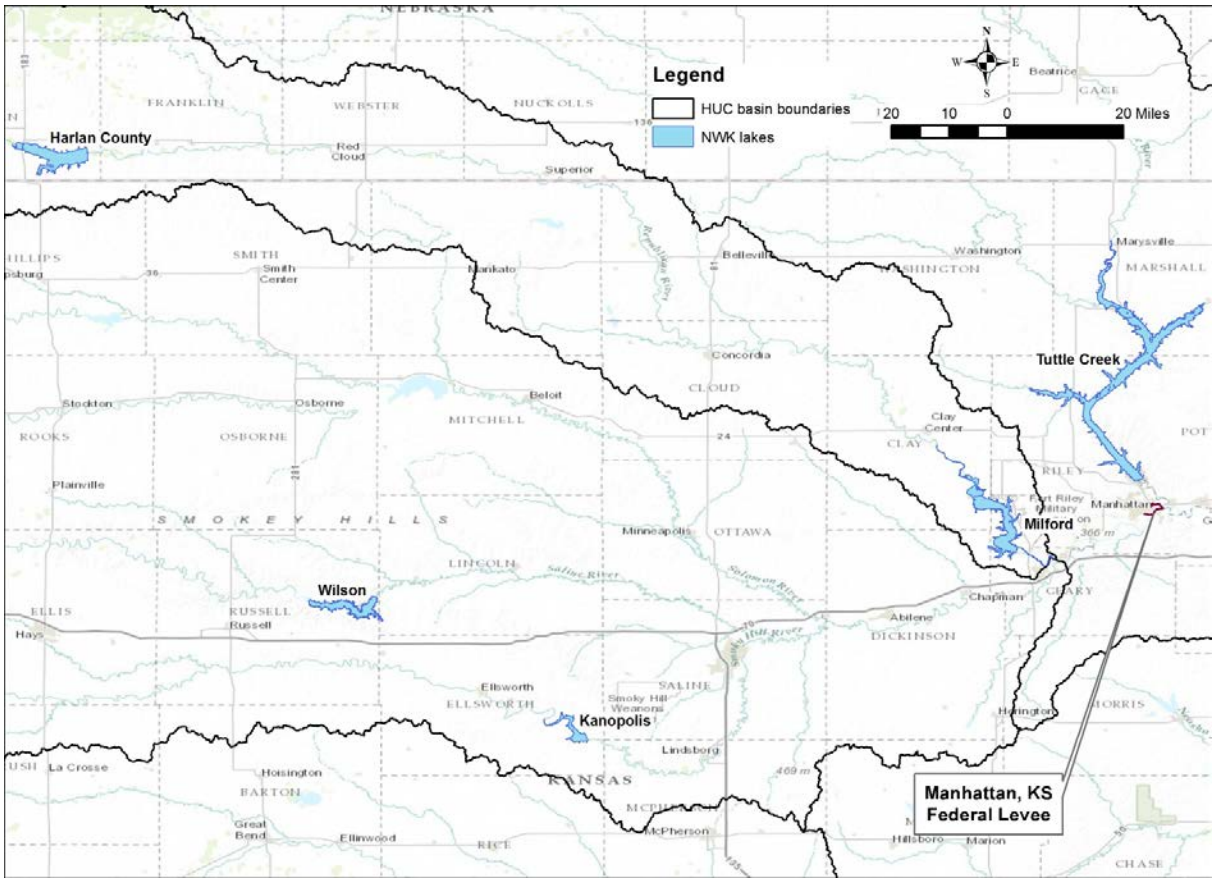
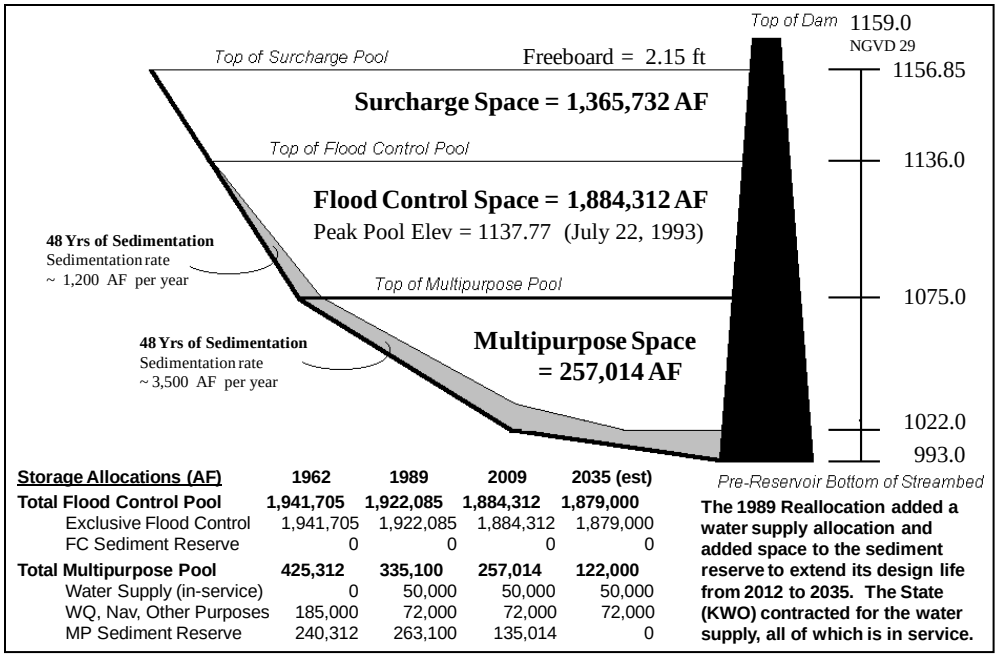


Figure 20: Flood control projects upstream of the Manhattan, KS Federal Levee

Tuttle Creek Lake was originally authorized for flood control and multipurpose pool benefits. The multipurpose pool was designed to meet the authorized and operating purposes of water quality, recreation, fish and wildlife, and navigation flow supplementation. In 1989, the multipurpose pool was reallocated to add an allocation for water supply and to increase the sediment reserve. The increase in the sediment reserve was needed to extend the design life of the pool from 50 years to 100 years. The water quality purpose includes support of minimum releases, normally 100 cfs although the releases can be reduced to 25 cfs during extreme drought conditions (para 8-12, Lake Regulation Manual). When hydrologic conditions permit, the minimum releases are set at 200 cfs, a convenient value for the project personnel operating the gates. Tuttle Creek Lake is also operated as a system with Milford and Perry lakes to support minimum flow criteria on the Kansas River at Topeka and DeSoto. The navigation function consists in providing supplemental releases for support of navigation flow targets on the Missouri River. This is also a system operation in conjunction with Milford and Perry lakes. Figure 21 below shows current storage allocations for Tuttle Creek Lake.



Storage Allocations (AF)	1962	1989	2009	2035 (est)
Total Flood Control Pool	**1,941,705**	**1,922,085**	**1,884,312**	**1,879,000**
Exclusive Flood Control	1,941,705	1,922,085	1,884,312	1,879,000
FC Sediment Reserve	0	0	0	0
Total Multipurpose Pool	**425,312**	**335,100**	**257,014**	**122,000**
Water Supply (in-service)	0	50,000	50,000	50,000
WQ, Nav, Other Purposes	185,000	72,000	72,000	72,000
MP Sediment Reserve	240,312	263,100	135,014	0

The 1989 Reallocation added a water supply allocation and added space to the sediment reserve to extend its design life from 2012 to 2035. The State (KWO) contracted for the water supply, all of which is in service.

Figure 21: Tuttle Creek Lake Storage Allocations

A revised Multipurpose Regulation section in the lake regulation manuals for all three lakes was formally approved in 1995, although elements of the plan were implemented as early as 1990. Downstream targets extend downstream along the Kansas River to the mouth, and then down the Missouri River to Waverly, Missouri. The plan limits the allowable drawdowns for supplemental navigation releases, sets criteria for the Kansas River minimum flow targets, and establishes system operation procedures for the three lakes in support of the downstream flow requirements. The revised multipurpose plan is supplemented with provisions in the Kansas River Master Manual and the Missouri River Master Manual.

Since construction of the dam in 1960, several potentially damaging floods have been prevented by Tuttle Creek Dam. Large inflows since 1960 are shown in Figure 22 and Figure 23 that were largely attenuated by Tuttle Creek Lake. In 1993, when the downstream flood of record occurred, the peak inflow to Tuttle Creek Lake was 95,400 cfs on July 6 and the peak release was 60,000 cfs on July 25.

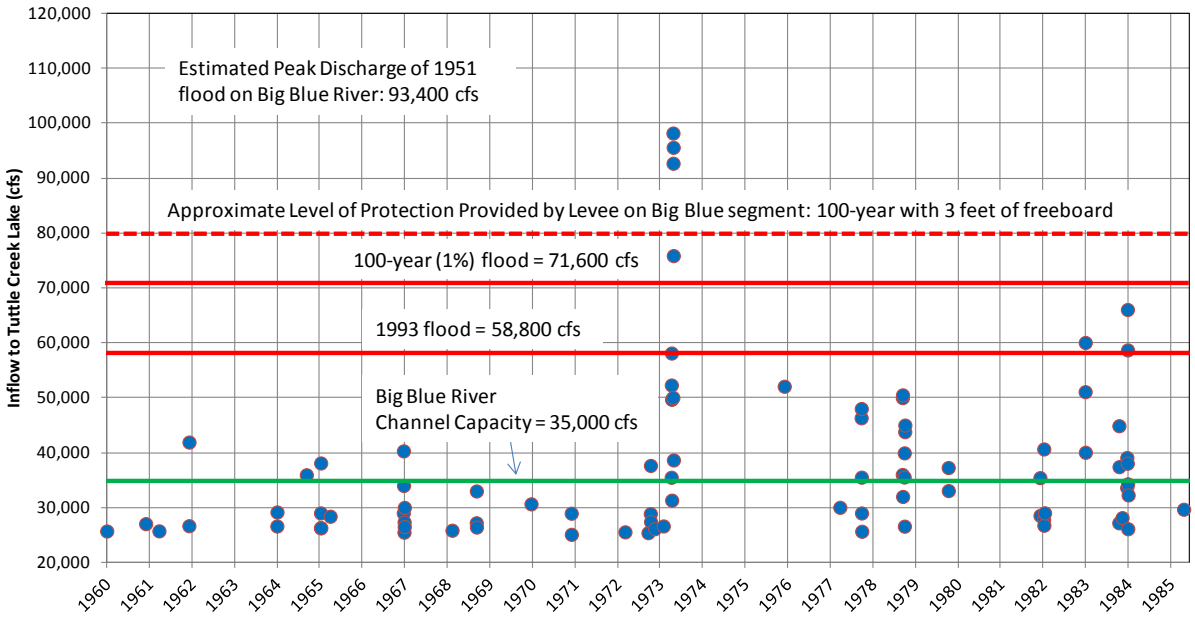


Figure 22: Large Inflows to Tuttle Creek Lake from 1960-1985

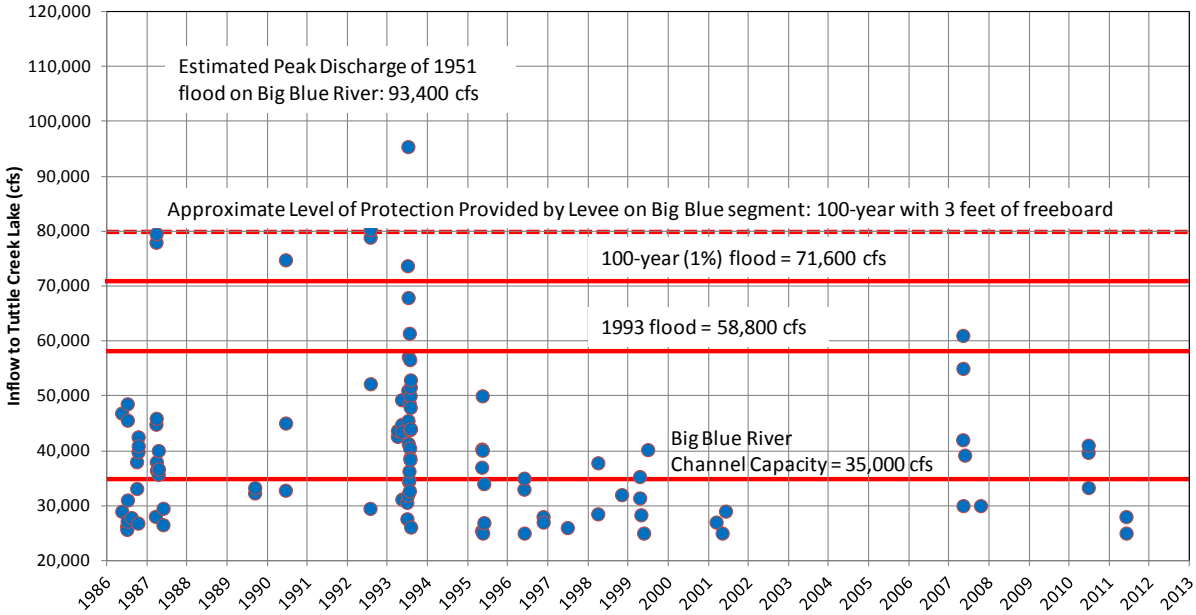


Figure 23: Large Inflows to Tuttle Creek Lake from 1986-2013

2-9.2. Tuttle Creek Lake Alternatives for Consideration

Modifications to the current operating procedures and purposes at Tuttle Creek Lake are technically feasible but are expected to provide marginal improved flood control benefits at high costs. Alternatives to increase downstream flood protection would likely require reducing benefits from other authorized purposes. Two alternatives for consideration involve storage reallocations to increase flood control space and modifying phased release schedules for flood control operations.

2-9.3. Multi-purpose and Flood Control Pool Reallocation Alternative

An analysis was conducted to determine the effects of reallocating all multi - purpose pool storage for the sole purpose of flood control. Tuttle Creek would essentially act as a dry detention facility in normal and drier than normal conditions. After reserving space for sediment through the design life of the reservoir, this would add 0.122 million acre-feet (MAF), to the existing 1.88 MAF of flood control storage allocation and 1.37 MAF of surcharge storage, resulting in an additional 6.5% storage at top of flood control and 3.8% additional storage at the top of surcharge. These small gains in flood control space would not be anticipated to provide any significant flood control benefits. If the 1993 flood reoccurs with all of the multipurpose pool allocation converted to flood control, the additional storage volume represents less than 2 days of releases, compared to 14 days of releases at or above the phase 3 maximum allowable release from flood control pool of 35,000 cfs. During the 1993 event, phase 3 releases were exceeded for 14 days and approximately 500,000 acre-feet of additional storage would have been needed to prevent surcharge releases in excess of 35,000 cfs. This additional storage requirement is about 4 times the available storage in the existing multi-purpose pool allocation. Effects of a single purpose flood control allocation would be at the expense of water supply during drought for urban areas such as Topeka, Lawrence, and Johnson County. This would also be at the expense of water quality on the KS river during drought. Furthermore this would eliminate essentially all water based recreation and fishery benefits on the lake, and eliminate a substantial portion of the supplemental flow source for navigation support on the Missouri River during drought.

Similar gains in flood control space could be realized through other modifications such as a dam raise. If an increase to the flood control pool allocation is considered to prevent surcharge releases during a reoccurance of the 1993 flood, the top of the flood control pool would need to be raised from elevation 1136 to about elevation 1145 to increase the existing storage capacity so it contains an additional 500,000 acre-ft. This would require the spillway gates to be raised about 9 feet above their current elevation to prevent gates from overtopping. The top of dam would also need to be raised in order to maintain both the required surcharge storage for the spillway design flood and existing 2.15 feet of freeboard for wind/wave concerns. Based on extrapolation of the 2009 storage-elevation curves in the Tuttle Creek Lake Regulation Manual, this would require a new top of surcharge at elevation 1163 and a new top of dam about elevation 1165, which is approximately 6 feet above the existing top of dam elevation 1159. Based on extrapolation of the 2009 area-elevation curves in the Tuttle Creek Lake Regulation Manual, approximately 8,000 acres of additional real estate would be effected by inundation around the perimeter of the lake during the spillway design flood and upstream levee systems at Maryville, Blue Rapids, and Frankfort, Kansas would likely require improvements and raises to mitigate a dam of flood control pool raise. Frankfort, Kansas is on the Black Vermillion River and at present does not appear to be significantly affected by backwater from Tuttle Creek. However, the downstream end of the levee at Frankfort, Kansas is at elevation 1146.5. Added pool elevation combined

with some coincident flows would increase the overtopping frequency of this Federal Levee. Additionally, the Blue Rapids Levee was built as part of the Tuttle Creek Dam Project to mitigate impacts of the flood pool. The levee is located in Marshall County, Kansas near the confluence of the Little Blue and Blue Rivers. Top of levee is set at elevation 1147, and a 9-ft raise in top of control would increase frequency of overtopping this levee. Lastly, a non-Federal Levee exists at Marysville, Kansas. This levee likely could also need to be raised to mitigate more frequent high pools. Accordingly, adding flood control space in Tuttle Creek as a measure to avoid raising the Federal Levee at Manhattan, Kansas would induce a need to raise at least two other federally built levees upstream of Tuttle Creek, and potentially a third non-federally built (but in the Corps non-federal PL-84-99 program) levee.

2-9.4. Phased Release Schedule Modification

An alternative was also considered to reduce the frequency at which Tuttle Creek Lake enters surcharge operations. This would require higher releases at lower pool elevations in an effort to lower the pool faster during flood control operations. By doing this, portions of the flood plain at downstream flow targets would be more frequently inundated in an effort to reduce the risk of inundating and damaging larger portions of the flood plain. This alternative would require intentional flooding without full assurance that surcharge operations would be avoided.

In the example of the 1993 flood, releases were held at Tuttle Creek Lake because downstream flooding was occurring at Waverly, Missouri on the Missouri River, not because of phase discharge limits at Tuttle Creek Lake. Figure 24 shows the 1993 flood hydrograph at the Missouri River at Waverly, Missouri and the inflow, outflow, and pool conditions at Tuttle Creek Lake.

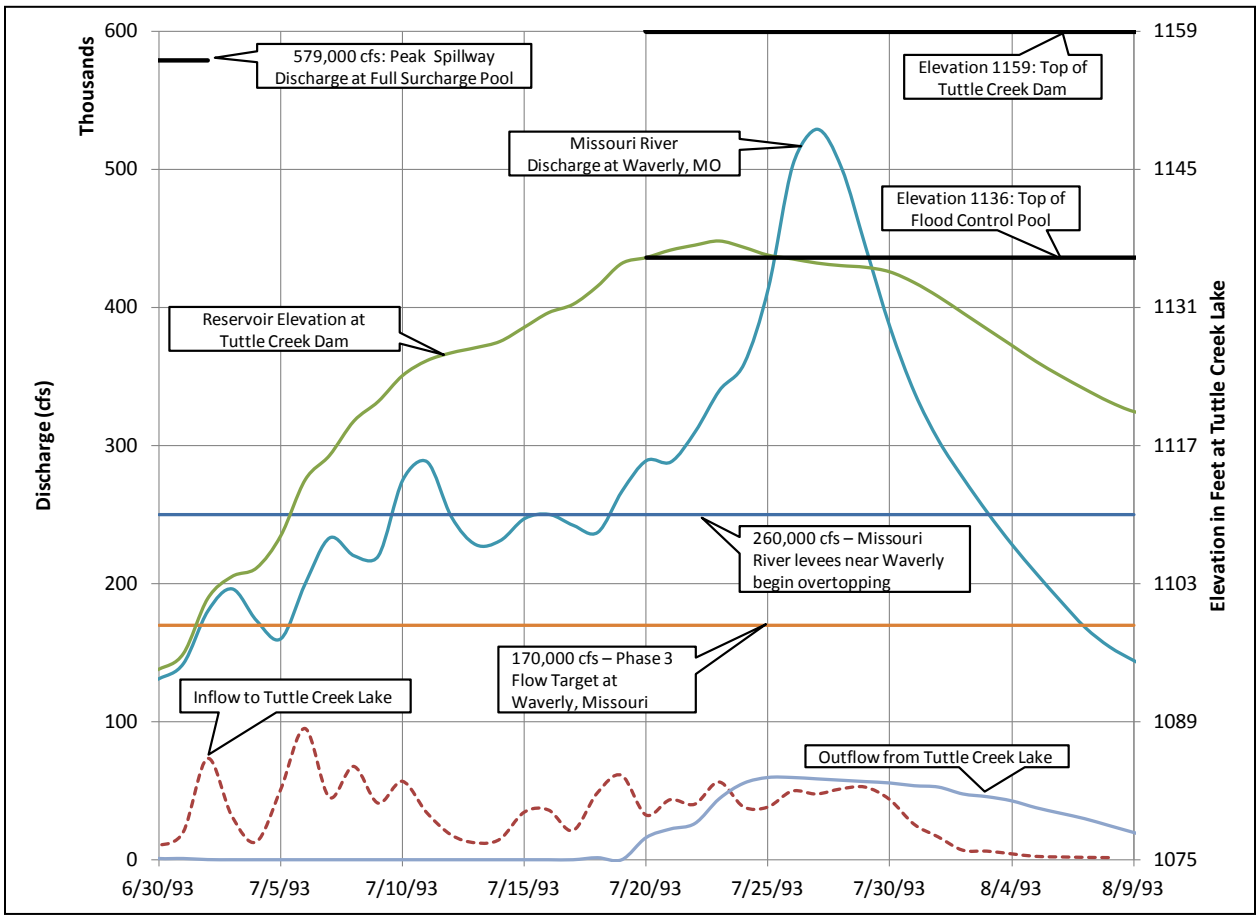


Figure 24: 1993 Flood Hydrographs and Tuttle Creek Pool Conditions

During the flood, Tuttle Creek Lake did not begin evacuating the pool until July 20th, when the pool reached the top of flood control elevation 1136. Before that day, Tuttle Creek Lake was filling its flood control pool to reduce downstream flood damages. Several levee systems were loaded along the Missouri River and additional releases at Tuttle Creek Lake would have likely increased the downstream flood damage risk without the guarantee of preventing surcharge releases from the dam. However, if the maximum phase releases (see Table 4) had been made earlier in the flood event with no consideration to reduce downstream flooding, surcharge conditions may have been prevented.

Currently, the Tuttle Creek Lake Regulation Manual (Aug 1973, pg IX-5) designates three levels of regulated discharges shown in Figure 25 as Phase I, Phase II, and Phase III with flow targets shown in Table 4. The Phase II level corresponds generally to zero damage but an attempt is made to meet flood stage, which is determined by the National Weather Service (NWS). Phase I defines a lower protection level which has been set as 80% of Phase II in the first reach below the dam and 60% of Phase II in all remaining reaches downstream. At the Phase III level, an attempt is made to meet moderate flood stage, which is determined by the NWS. At this point,

damage will be experienced in those areas where banks are below average in height and agricultural encroachment onto adjacent low bottom lands has been effected.

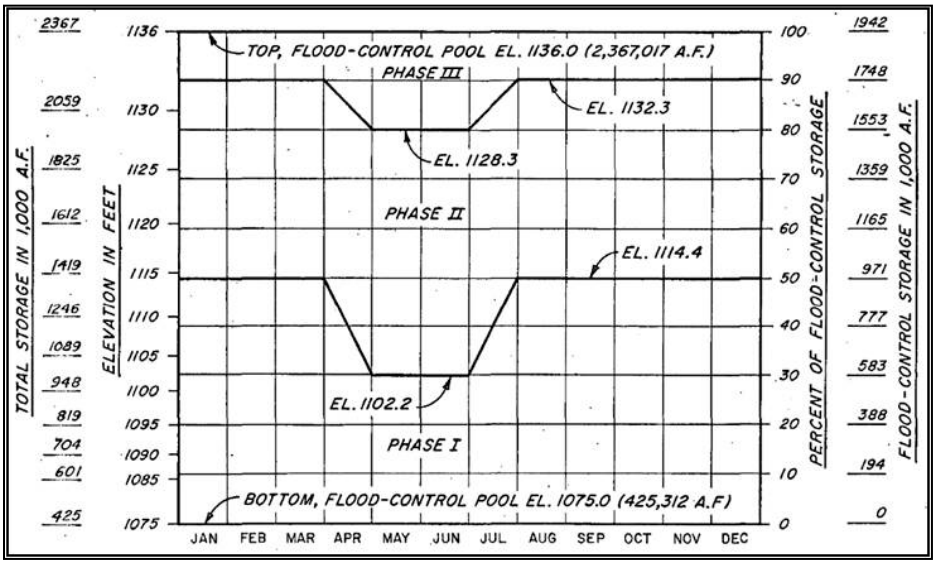
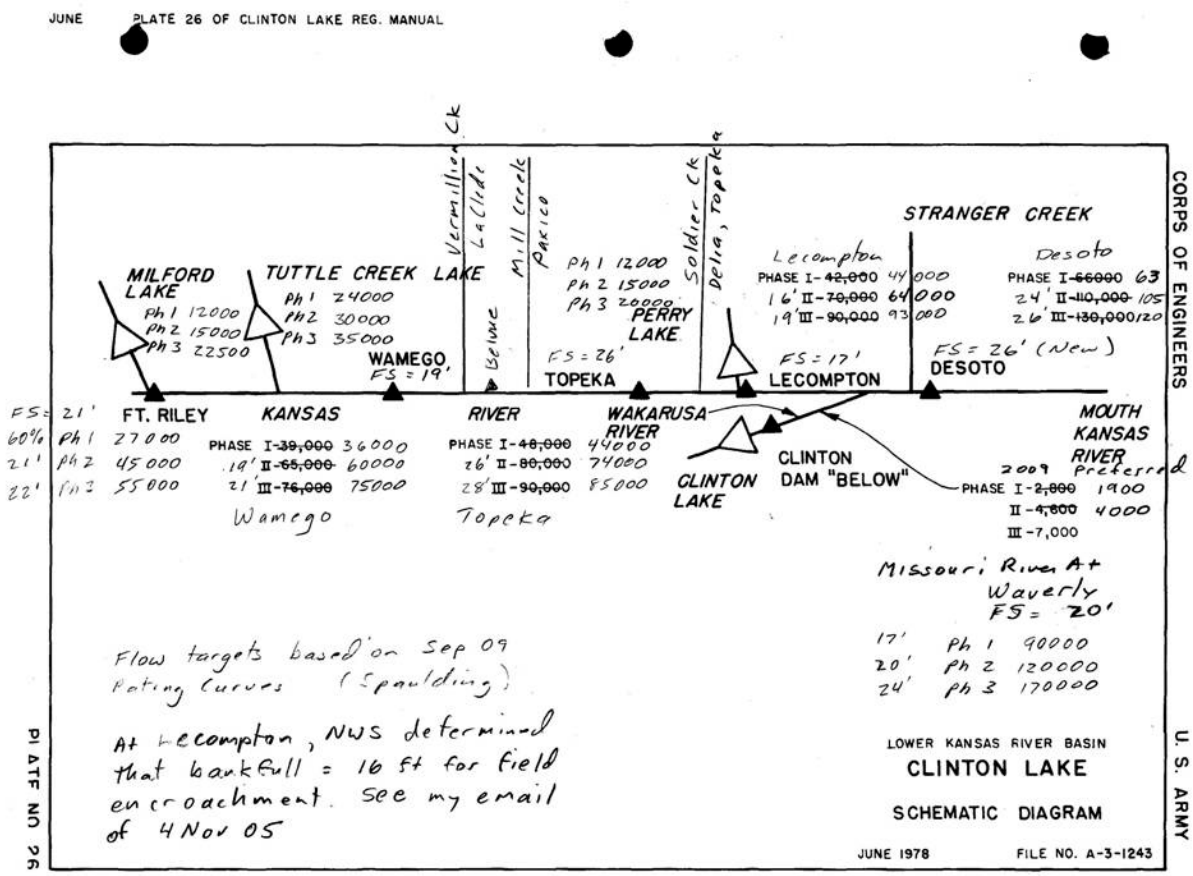


Figure 25: Seasonal Guidelines for Phase Determination (WCM, Plate 37)

Table 4: Regulating Discharges on the Kansas River from Phased Releases



2-9.5. Recommendation

Changes to the allocations and operational phased releases are not recommended to provide flood safety assurance for the City of Manhattan. Changes to storage allocations in Tuttle Creek Lake are not expected to significantly improve downstream flood protection. If phased releases were adjusted for a more aggressive release schedule earlier in a flooding event, it is likely that this would intentionally increase minor and moderate flooding without providing full assurance that surcharge operations would be avoided. In addition, in the example of 1993, this type of earlier evacuation would have increased streamflows by approximately 10% at Waverly, Missouri at a time when Missouri River levees were already substantially loaded and there was not certainty that additional rain would fall at that time. Upstream regulation modifications were analyzed at Tuttle Creek Lake and provided only marginal improvements at the Manhattan Levee. Additional measures at reservoirs located farther upstream are not expected to provide substantial added benefits at Manhattan, KS any more than those evaluated from Tuttle Creek Lake in part because their effects are expected to be attenuated due to their proximity to Manhattan, KS.

2-10. References

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12. United States Bureau of Reclamation "Combining and Portraying Risks", March 14, 2011. http://www.usbr.gov/ssle/damsafety/Risk/BestPractices/28-CombiningAndPortrayingRisks20110326.pdf

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2-11. Enclosures

2-11.1. Enclosure 1: Existing Conditions Inundation Maps

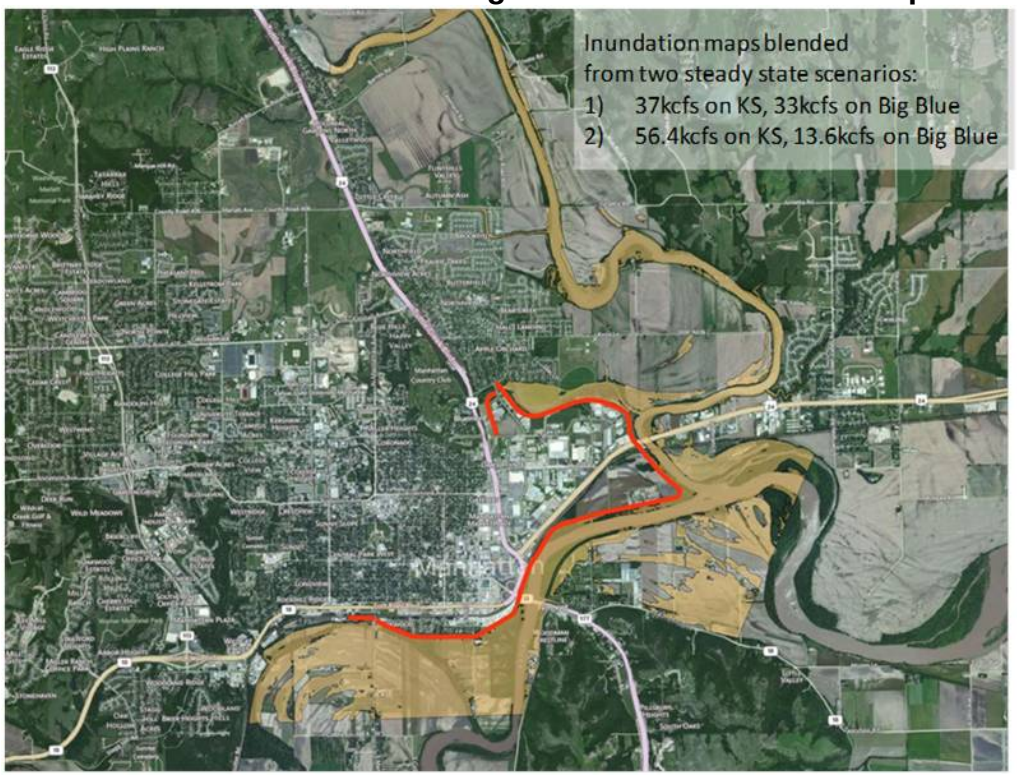

Figure 26: Inundation map of the EC 4% (1/25) ACE flood

Figure 27: Inundation map of the EC 2% (1/50) ACE flood

Figure 28: 1993 Flood Calibrated Results at EC

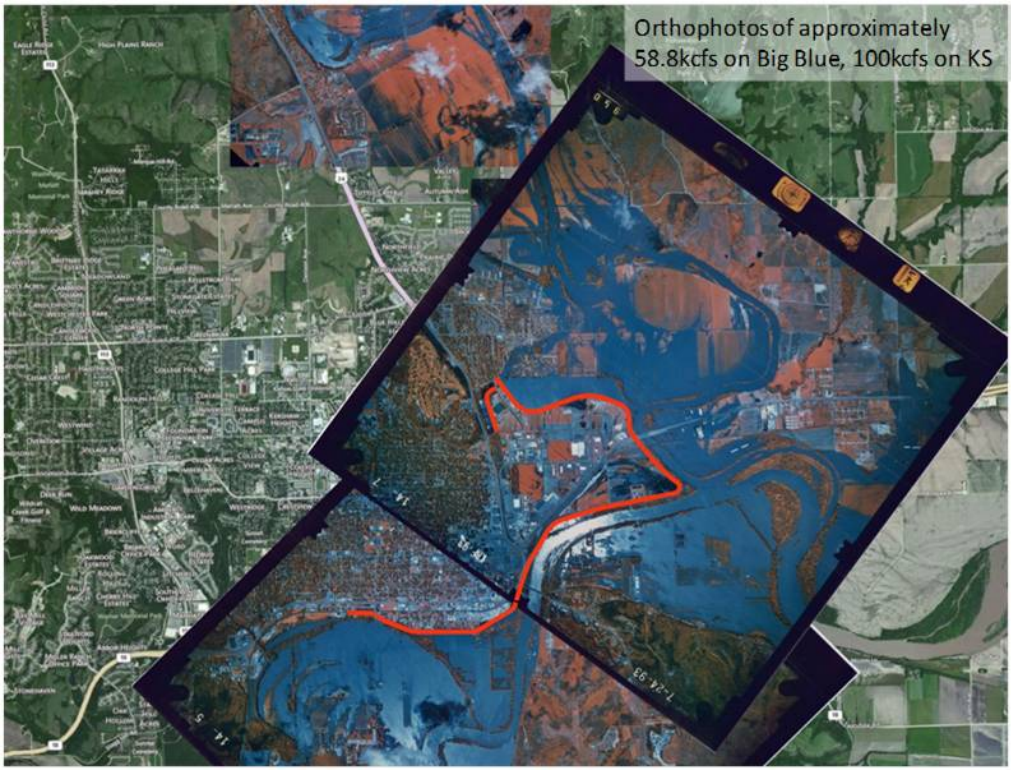

Figure 29: Orthography of the 1993 flood

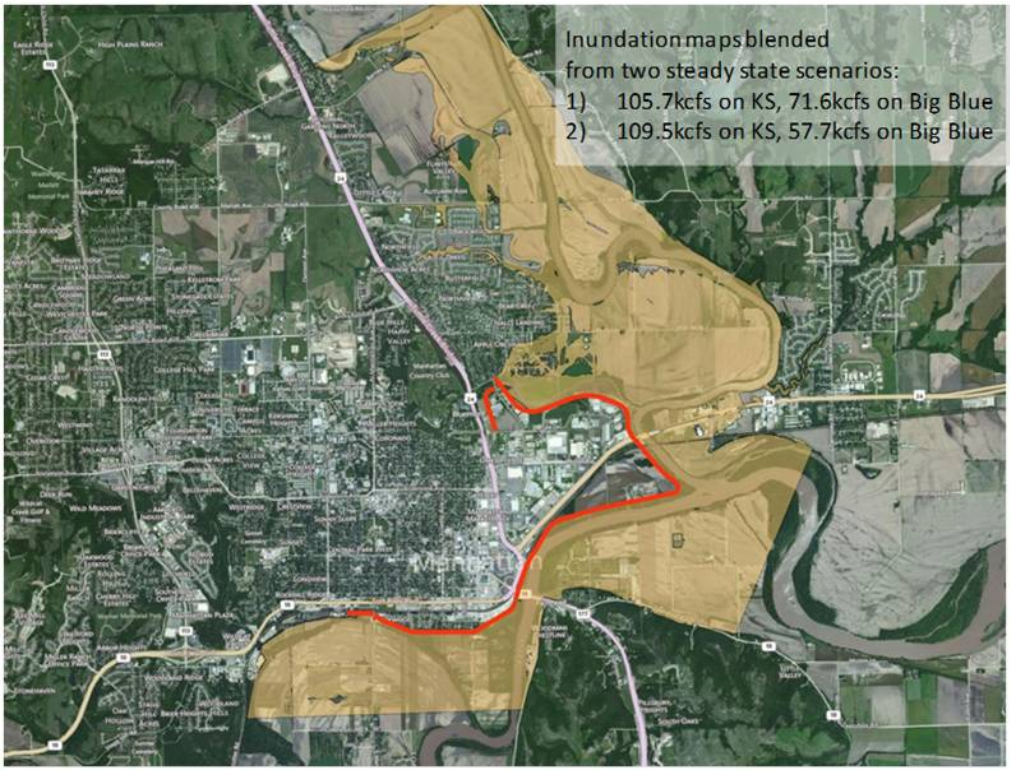

Figure 30: Inundation map of the EC 1% (1/100) ACE flood

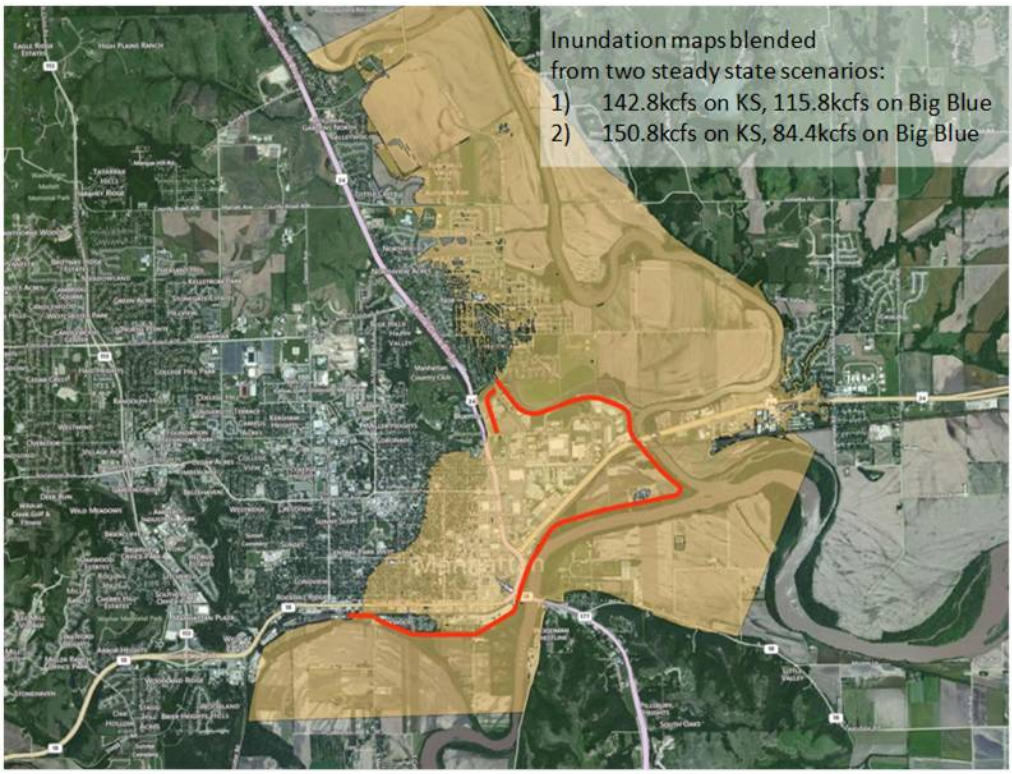

Figure 31: Inundation map of the EC 0.5% (1/200) ACE flood

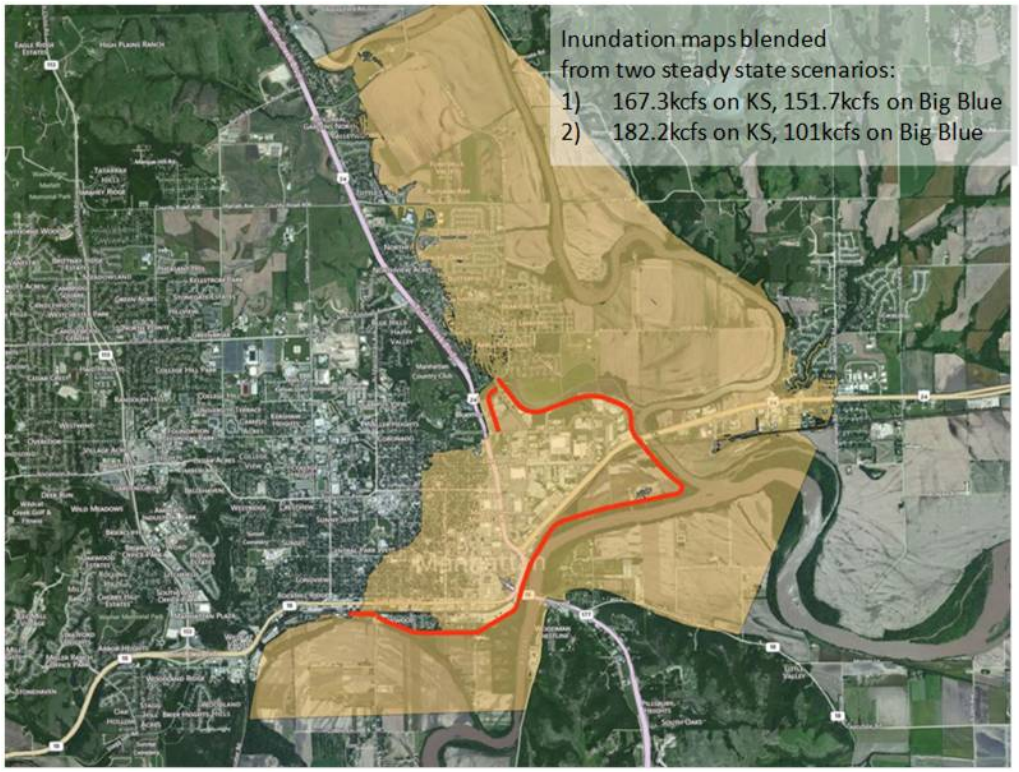

Figure 32: Inundation map of the EC 0.33% (1/300) ACE flood

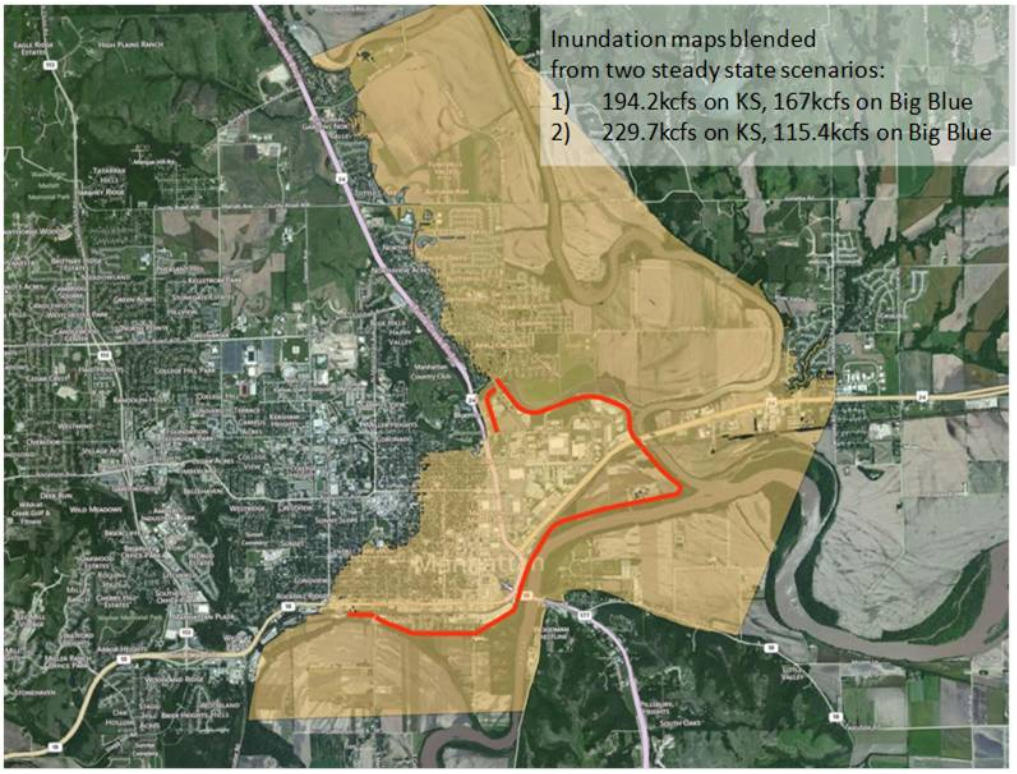

Figure 33: Inundation map of the EC 0.2% (1/500) ACE flood

Figure 34: Inundation map of the EC 0.13% (1/750) ACE flood

2-11.2. Enclosure 2: Fragility Curve Combinations

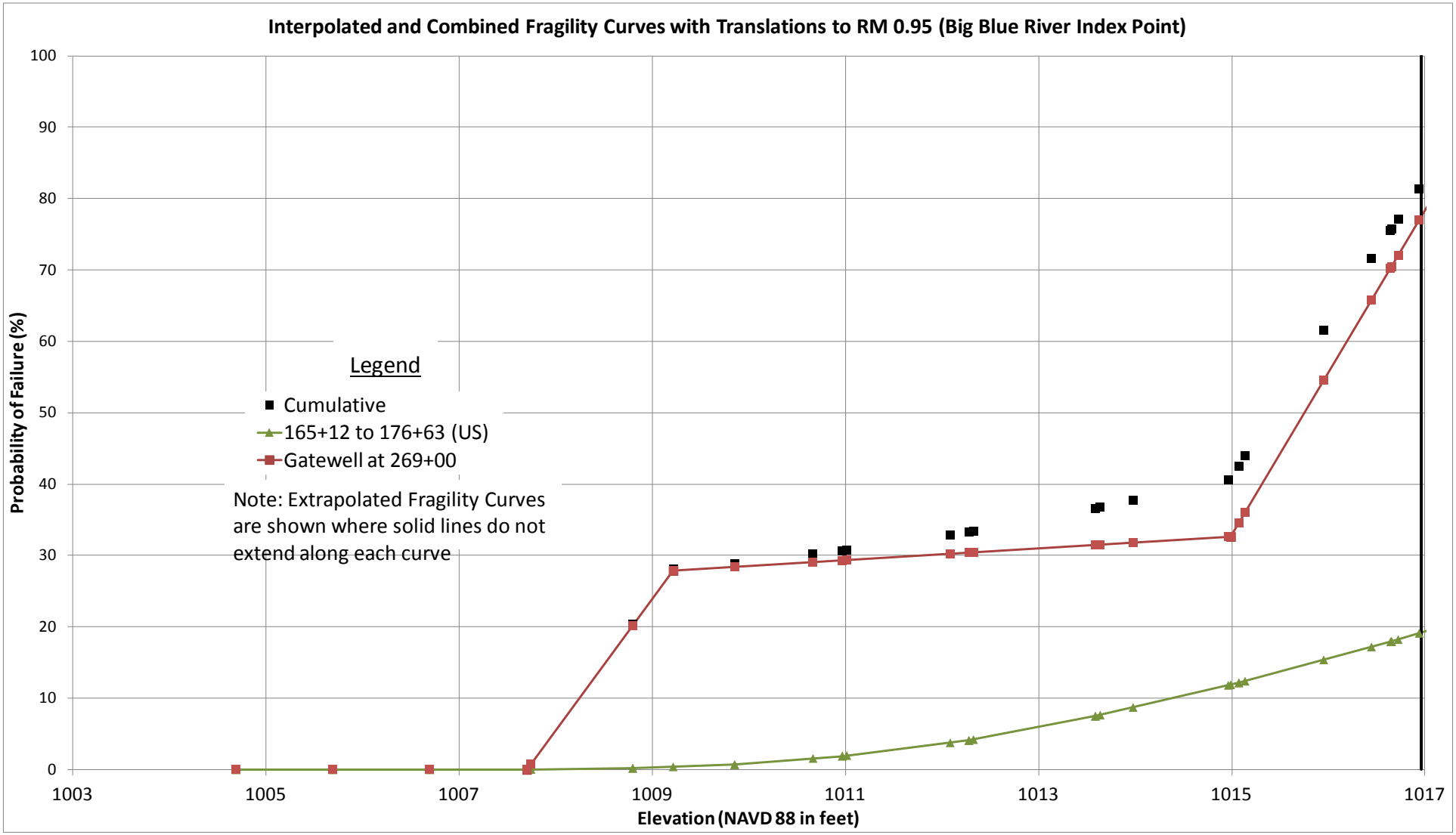

Figure 35: Cumulative Probability Plot at Index Point 0.95 on Big Blue River

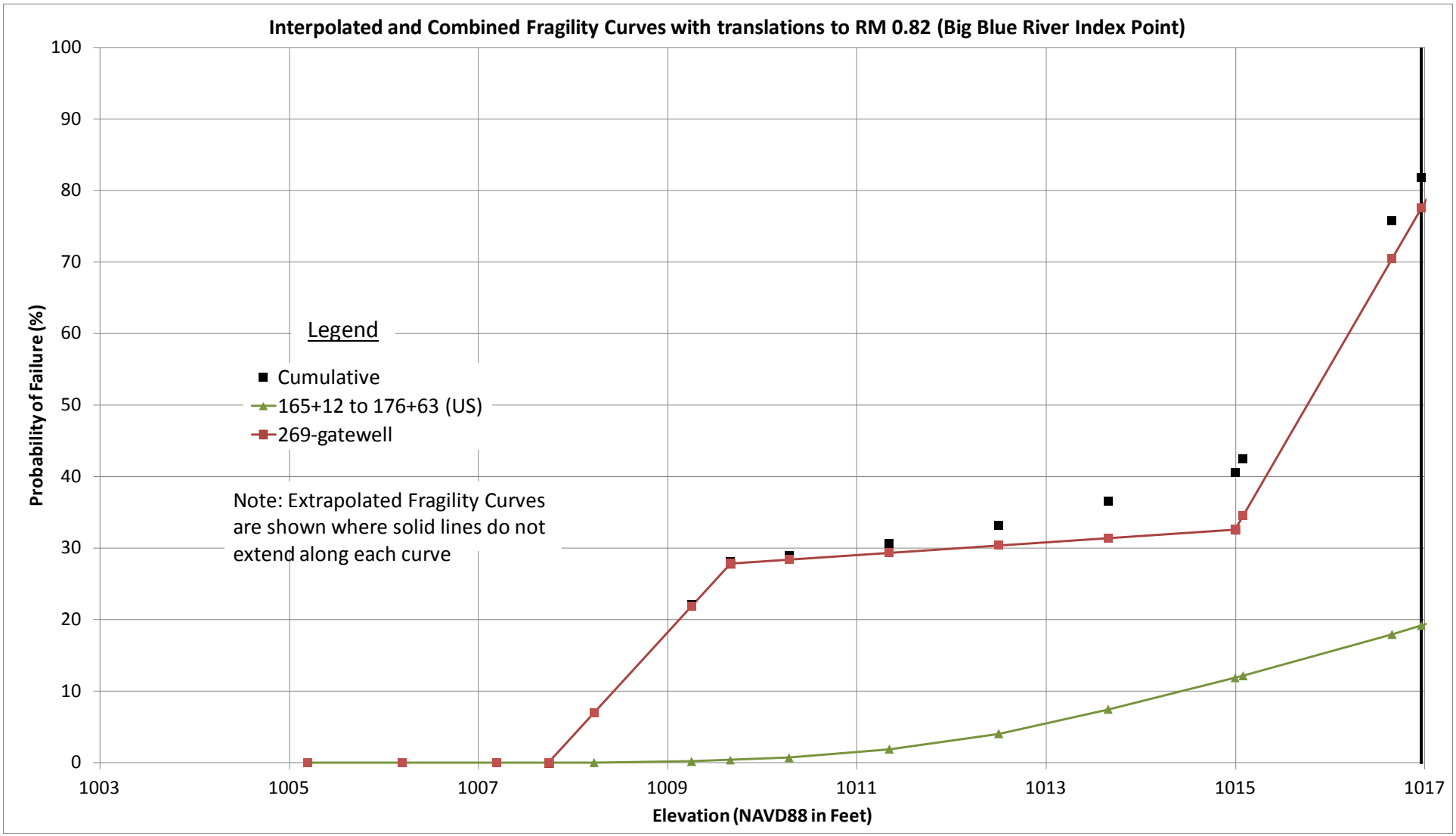

Figure 36: Cumulative Probability Plot at Index Point 0.82 on Big Blue River

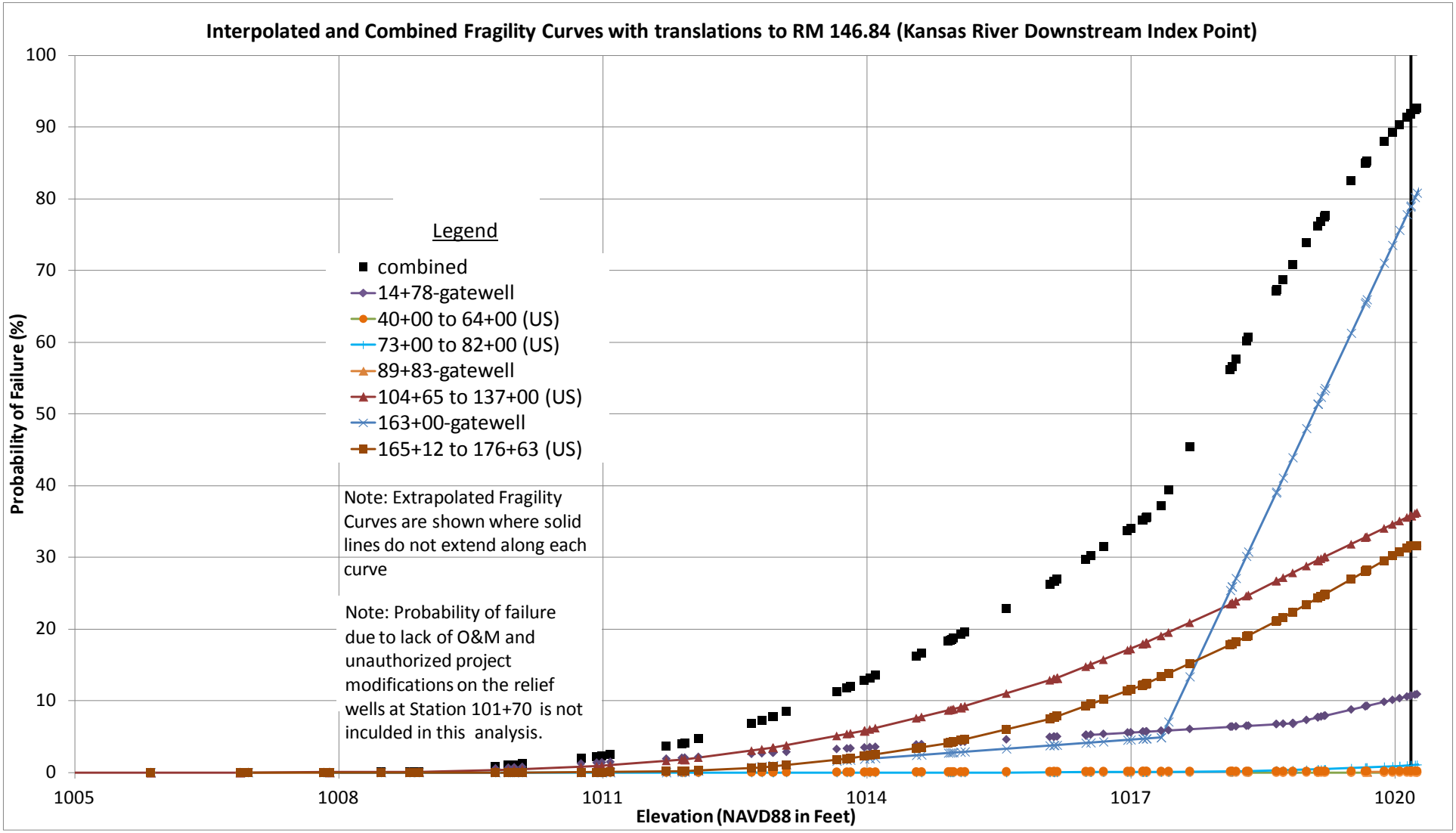

Figure 37: Cumulative Probability Plot at Index Point 146.84 on Kansas River

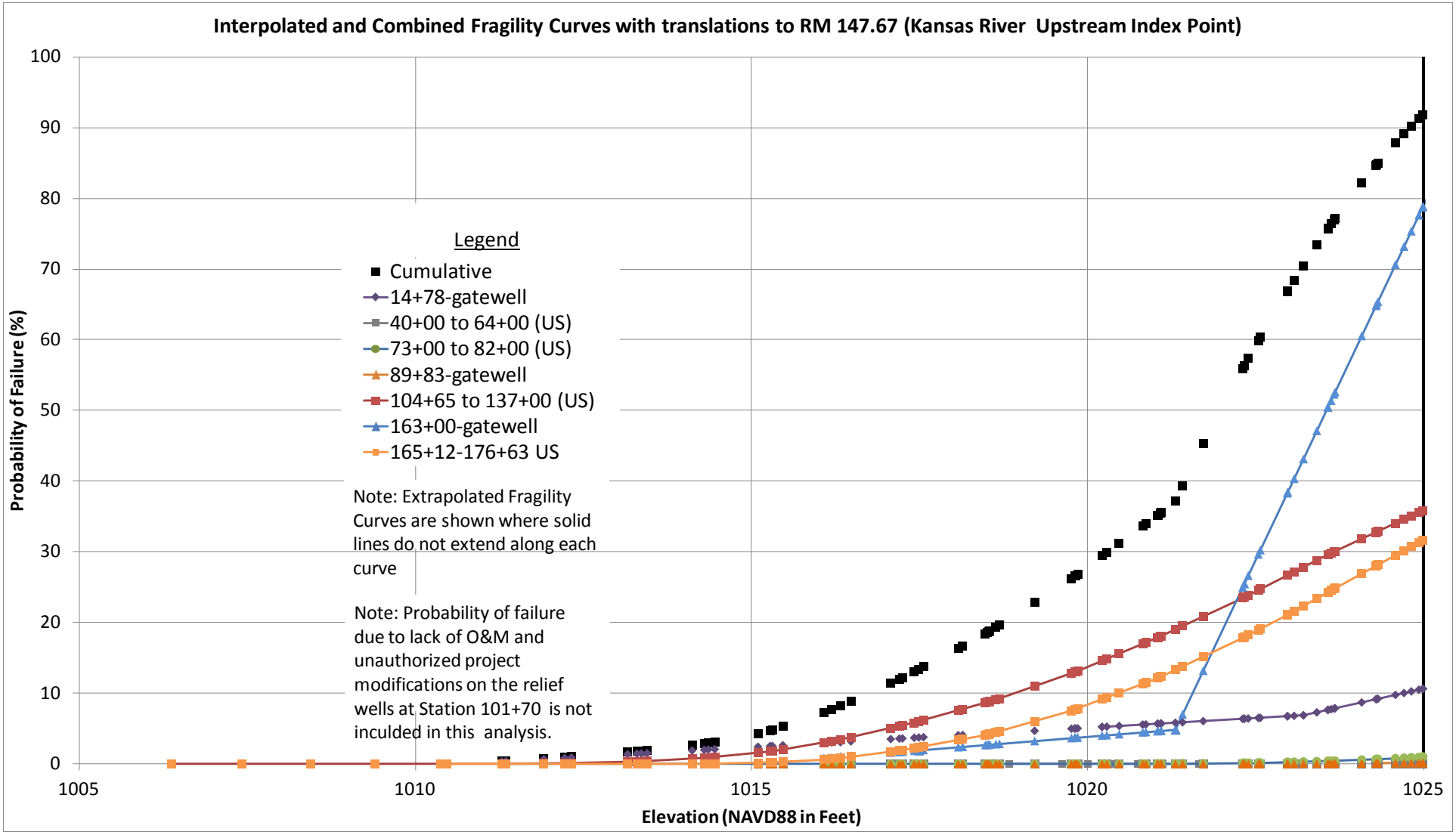

Figure 38: Cumulative Probability Plot at Index Point 147.67 on Kansas River

US Army Corps
of Engineers
Kansas City District

Part 3
Geotechnical
Levee Raise Alternatives and Underseepage and Slope Stability Improvements

Alternatives Engineering
Manhattan, KS

August 2014

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Enclosure A: Preliminary Underseepage Analysis, Underseepage Berm Design, and Relief Well Design
Enclosure B: Schematics of Potential Solutions to Slope Stability and Underseepage Concerns
Enclosure C: Preliminary Slope Stability Analysis with Landside Stability Berm and Filter Drain

3.1. Introduction

A Feasibility study of the Manhattan Levee Unit was initiated after the 1993 flood event to investigate protection adequacy, see Phase I of the Manhattan Feasibility Study. The feasibility identified high risk areas for underseepage and landside slope stability failure modes for the existing levee. Details of the analysis are documented in the Phase I report.

Phase 2 of this study was to evaluate features needed to address identified risk areas with respect to underseepage and landside slope stability for array of alternatives. Three proposed levee raise alternatives were evaluated: 0.50%, 0.33%, and 0.20%. The proposed alternatives were determined based on hydrologic models that account for historical performance of the levee unit during flood events. Details of the hydrology models are in the Hydrology chapter of this study.

During the 1993 flood event, most of the water loading was along the Big Blue River levee reach which had about 4 ft of freeboard. As a result all three levee raise options involve raising the levee unit along the Big Blue River. The 0.50% chance of exceedance improvement alternative is limited along the Big Blue River, the 0.33% chance of exceedance improvement alternative extends to the Kansas River between Station 130+00 and 169+00 (Confluence), and the 0.20% chance of exceedance improvement alternative covers the whole levee unit except between Station 70+00 and 100+00. The proposed alternatives involve levee raises along the Big Blue River. The average levee raise for each of the proposed alternative is 0.75, 1.5, and 2.1 ft, respectively. Due to minimal differences in the levee raise between the 0.50 and 0.33% chance of exceedance alternatives, an analysis was only performed for the 0.33% chance of exceedance and the results used for the 0.50% chance of exceedance.

Details on the levee unit, levee features, foundation conditions, and other pertinent information relevant to this study are found in the Phase I report. The following sections discuss underseepage and stability improvements needed along the levee raises based on current criteria, for the raise alternatives.

3.2. Underseepage Analysis Methodology and Criteria

For each alternative, sections of levee raise were analyzed deterministically to identify all areas not meeting current criteria under existing conditions, see **Enclosure A**. Underseepage performance was assessed in general accordance with Engineering Manual (EM) 1110-2-1913 *Design and Construction of Levees*. The methodology used in the underseepage analyses follows EM 1110-2-1913; however, the Kansas City

District uses the following variations: **1**) permeability ratios are used instead of blanket and aquifer permeabilities (see **Table 3-1**), **2**) an infinite landside blanket is assumed (Case 7 in EM 1110-2-1913), and **3**) no blanket transformations are performed and a representative permeability ratio is used.

The design of future alternatives was based on a minimum Factor of Safety (FS) of 1.6 at the levee toe with water at top of levee. Levee segments with FS equal to or greater than 1.6 are adequate and do not need underseepage control measures. However, levee reaches with FS less than 1.6 require underseepage control measures such as underseepage berms or relief wells. It is recognized the design FS of 1.6 is not consistent with the current EM 1110-2-1913 (2000), which specifies criteria based on gradient. However, the design FS of 1.6 is consistent with ETL 1110-2-569 (expired 2010). A design FS of 1.6 was also recommended for use in NWK in a 2011 Memorandum for Records on seepage criteria for ongoing NWK levee design projects on high consequences levees. The NWK 2011 criterion was established considering ETL 1110-2-569 and Draft EM 1110-2-1913 (2013). While the Manhattan Levee is not called out in the memorandum, it is similar to the levees specifically mentioned. Additionally, coordination has recently taken place with the USACE team working on revisiting EM 1110-2-1913 and a design FS of 1.6 at the levee toe is the likely guidance that will be in the revised document. The design FS of 1.6 selected for Manhattan ensures long term performance and is consistent with current USACE underseepage considerations.

Areas identified in Phase 1 that represent underseepage risk or that do not meet the selected design underseepage criteria outside the proposed raise reaches are recommended for underseepage improvements. This is to ensure high confidence in performance of the entire levee during flood events and to match the high reliability modeled in the economic benefit calculation for the future condition alternatives for water loadings up to overtopping.

Underseepage berm design was performed in accordance with the Kansas City District guidelines for underseepage berm shown in **Table 3-2**. The relief well systems were designed in general accordance with EM 1110-2-1914 *Design, Construction, and Maintenance of Relief Wells* with the following variations:

1) The excess head computed at the landside toe was used as the net head on the system of wells instead of full driving head. This was done because the procedure outlined in Figure 5-3 in EM 1110-2-1914 assumes an impervious blanket. However, a semi-impervious blanket was assumed for the underseepage calculations.
2) An efficiency reduction factor of 0.8 was applied to the expected well flows. This was done to account for the reduction in efficiency with time of the relief wells. An efficiency of 0.8 was chosen as EM 1110-2-1914 requires remedial action once a loss of 20% in specific capacity of a well is observed from pumping test.

Design details are found in **Enclosure A**. Schematics of the underseepage and slope stability solutions are found in **Enclosure B**.

Table 3-1: Permeability Ratios for Blanket Material Based on Material Type Used in Underseepage Analysis

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

Table 3-2: Kansas City District Underseepage Guidelines for Berm Toe

NWK Guidelines (2011)				
Distance From Landside levee toe (ft)				
[1]100	200	300	400	500
FS_i = 1.5	FS_i = 1.4	FS_i = 1.3	FS_i = 1.2	FS_i = 1.1
[1]100 feet is the proposed minimum berm width from the landside levee toe				

3.3. Slope Stability Analysis Methodology and Criteria

In this study, preliminary steady state slope stability analyses were performed in general accordance with EM 1110-2-1913 *Design and Construction of Levees*. Stability analysis was only performed for the landside slope with water at top of levee. All analysis was performed using the SLOPE/W package in the Geo-Slope software using Spencer's method. The Spencer's method is generally used because it satisfies both force and moment equilibrium. 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.

The analyses were conducted to meet the requirements of EM 1110-2-1913 summarized in **Table 3-3**. End-of-Construction and Rapid drawdown performance were not analyzed due to lack of laboratory testing. Detailed analysis of the End-of-Construction and Rapid drawdown conditions in this project needs to be assessed during the Project Engineering Design (PED). Design strengths used for the steady state analysis were based on the Design Memorandum (DM), boring logs drilled by AMEC in 2010 for the levee certification, and engineering assumptions based on

experience with similar projects, see **Table 3-4**. The embankment and blanket friction angles were assumed similar since the boring logs revealed similar strata.

In this analysis, 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. The steady state piezometric surface through the levee section was assumed approximately linear between the water surface-riverside slope intersection and the landside toe.

Phase 1 indicated that most of the levee does not likely meet required landside steady state seepage stability factors of safety with water at the levee top. The original stability design was for steady state seepage conditions for the “design water surface” (levee top minus 3ft freeboard). As discussed in Phase 1, the likelihood of having steady state seepage through the embankment with water at the levee top is extremely low. As such, areas where there is no levee raise proposed for each alternative will not be recommended for stability improvements such as internal drainage or stability berms. Instead, these areas will be evaluated in detail during the design phase to ensure that there is not considerable risk remaining after the proposed projects are complete. Even though areas where the levee will be raised are at low risk of experiencing steady state seepage with water at the levee top, a raise can easily accommodate internal embankment drainage or stability berms as part of the already planned earthwork, with minor cost impacts. This will ensure that areas being raised will meet criteria and that there are no unacceptable slope stability risks remaining in unraised areas at project completion.

A slope stability analysis was performed on a representative levee segment to determine what stability modifications will be needed for the raised reaches. The representative levee segment was Station 97+00 to 102+30 was analyzed to determine adequate slope stability solutions to address potential future concerns during flood loading. The levee segment between Station 97+00 and 102+30 is of average height (10 ft) and has representative underseepage characteristics.

Table 3-3: Minimum Factors of Safety

Loading Conditions	Recommended Factor of Safety
End-of-Construction	1.3
Steady State	1.4
Rapid Drawdown	1.0 to 1.2[a]
[a]Lower factors of safety may be appropriate when consequences of failure in terms of safety, environmental damage, and economic losses are small.	

Table 3-4: Slope Stability Design Strengths

Material	Friction Angle (ϕ) Degrees	Unit Weight (pcf)[1]
Embankment	26	115
Blanket[2]	26	115
Foundation	32[3]	120
Berm Sand	32[3]	120
Riprap	40[1]	140

[1]Assumed based on experience.

[2]Assummed similar to embankment fill based on 2010 AMEC certification boring logs.

[3]Assummed based on average blow count of 10 reported in 2010 AMEC certification boring logs.

3.4. 0.50% and 0.33% Chance of Exceedance of Levee Raise

This levee raise alternative is between Station 131+00 and 272+00 with an average raise of approximately 1.5 ft. The levee reach between Station 130+00 and 169+00 is along the Kansas River. The levee raise is proposed to be constructed to the landside with exception between Station 149+00 and 163+00 where the levee raise will be on the riverside due to sanitary treatment plant constraint.

3.5. 0.20% Chance of Exceedance of Levee Raise

This levee raise applies between Station 8+00 and 272+00 except between Station 72+00 and 101+00. The levee raise is proposed to be constructed to the landside with an average raise of approximately 2.1 ft. The exception is between Station 40+00 and 64+00 and between Station 149+00 and 163+00 where the levee raise will be on the riverside due to the presence of private properties and a sanitary treatment plant, respectively. Between Station 149+00 and 163+00 the levee raise will require lengthening of the outlet pipe at approximately Station 156+00 and raising of the gatewell at roughly Station 163+00.

3.6. Underseepage Control: 0.50%, 0.33%, and 0.20% Chance of Exceedance

There is not a levee performance report available for the levee from the 1993 flood. However, recollections of field observations by the sponsor indicated that there may have been significant seepage and possible pin boils observed between Station 130+00 and 150+00. While the flood performance documentation is extremely limited, this information does help to lend confidence in the underseepage deficiencies identified in this study between Station 104+95 and 137+00. This study identified poor underseepage performance at Station 101+70 due to poor relief well maintenance and an unauthorized removal of two relief wells along the Poyntz Avenue pump station ditch. The removal is an unauthorized modification to the project, and improving the performance of this area cannot be performed under the authorized study. For the rest of the levee alignment, the following underseepage control features are recommended for each alternative:

a. 0.50% and 0.33% Chance of Exceedance of Levee Raise

Station 40+00 to 64+00: This levee reach has a landside ditch which does not meet criteria when it is empty. Private property lines are adjacent to the ditch and modification will require costly real estate purchases. It is recommended that a minimum ponding elevation of 1015 ft be maintained during flood events to maintain criteria.

Station 64+00 to 97+00: This levee reach is a reach of a generally thin blanket. Private properties are located near the landside toe which excludes underseepage berm as a solution. Installation13 fully penetrating relief wells fully with 12 inch diameters spaced between 200 and 300 ft.

Station 110+00 to 120+00: This levee reach is a reach of a generally thin blanket. Railroad tracks constraint real estate on the landside. Relief well design requires installation of 12 fully penetrating relief wells spaced 100 ft apart.

Station 120+00 to 137+00: This levee reach is a reach of a generally thin blanket. A 100 ft wide underseepage berm with a minimum thickness of 3 ft at the berm toe is recommended to meet criteria.

Station 165+12 to 173+50: This levee reach is a reach of a generally thin blanket. This levee reach is along the Big Blue and Kansas River confluence. A farm is located on the protected side which allows for construction of a 300 ft wide underseepage berm with a minimum thickness of 3 ft at the berm toe is required to meet criteria.

Station 265+70 to 272+00: This levee reach is a reach of a generally thin blanket. The landside contains business buildings adjacent to the levee toe which excludes the construction of underseepage berms. Relief well design requires the installation of 4 fully penetrating relief wells with 12 inch diameters spaced 200 ft apart.

b. <u>0.20% Chance of Exceedance of Levee Raise</u>

Station 18+00 to 23+00: This levee reach has a ponding area located approximately 75 ft from the landside toe. Analysis revealed that it does not meet criteria. Due to the presence of railroad tracks on the landside, relief wells were recommended instead of underseepage berms. The relief well design requires the installation of 6 fully penetrating relief wells with 12 inch diameters spaced 100 ft apart.

Station 40+00 to 64+00: This levee reach has a landside ditch which does not meet criteria when it is empty. Private property lines are adjacent to the ditch and modification will require costly real estate purchases. It is recommended that a minimum ponding elevation of 1015 ft be maintained during flood events to maintain criteria.

Station 64+00 to 97+00: This levee reach is a reach of a generally thin blanket. Private properties are located near the landside toe which excludes underseepage berm as a solution. Installation 13 fully penetrating relief wells fully with 12 inch diameters spaced between 200 and 300 ft. This levee reach will not be raised; as a result, the solution is identical to the 0.33% chance of exceedance.

Station 110+00 to 120+00: This levee reach is a reach of a generally thin blanket. Railroad tracks constraint real estate on the landside. Relief well design requires installation of 12 fully penetrating relief wells spaced 100 ft apart. This solution is identical to the 0.33% chance of exceedance because the levee raise is minimal.

Station 120+00 to 137+00: This levee reach is a reach of a generally thin blanket. A 200 ft wide underseepage berm with a minimum thickness of 3 ft at the berm toe is recommended to meet criteria.

Station 165+12 to 173+50: This levee reach is a reach of a generally thin blanket. This levee reach is along the Big Blue and Kansas River confluence. A farm is located on the protected side which allows for construction of a 300 ft wide underseepage berm with a minimum thickness of 3 ft at the berm toe is required to meet criteria. The berm width is identical to the 0.33% chance of exceedance because the levee raise difference is approximately 1 ft.

Station 190+00 to 210+00: This levee reach is a reach of a generally thin blanket. The landside contains business buildings, water wells, Highway 24, and railroad tracks adjacent to the levee toe which excludes the construction of

underseepage berms. Relief well design requires the installation of 10 fully penetrating relief wells with 12 inch diameters spaced 400 ft apart. The spacing of the relief wells will have to be evaluated in detail due to the heavy presence of infrastructure on the landside.

Station 265+70 to 272+00: This levee reach is a reach of a generally thin blanket. The landside contains business buildings adjacent to the levee toe which excludes the construction of underseepage berms. Relief well design requires the installation of 4 fully penetrating relief wells with 12 inch diameters spaced 200 ft apart. The solution is identical to the 0.33% chance of exceedance because the levee raise difference is about 0.5 ft.

3.7. Slope Stability Control

Evaluation of the representative levee segment indicated a need to construct internal embankment drainage near the landside toe as part of proposed raises. The proposed drain extends from the new landside toe to a point equivalent to the landside crest shoulder projected landward at a 1V:1H slope. The required drain thickness is 3 feet. For a typical section, the drain will be 28 feet wide; the 0.33% chance of exceedance quantities were estimated using 76 ft^3/ft of material, see details in Enclosure C. Potential borrow areas are discussed in the Civil portion of this report.

3.8. Expected Settlement of Design Features

No calculations were performed to determine expected settlement resulting from the 0.33 and 0.20% chance of exceedance levee raises. In this study it is assumed that since the levee raise is minimal it will not lead to significant foundation consolidation since the foundation materials have already consolidated due to the existing levee.

3.9. Recommendations for PED Phase

The following recommendations should be considered in the PED phase of this project:

1) Perform subsurface exploration at locations with sparse boring logs to refine underseepage blanket elevations and facilitate laboratory testing of "undisturbed" samples to determine/refine strength parameters for steady state, end-of-construction, rapid drawdown, and settlement analysis.

Station	Investigation Type	Location	Depth	Proposed Laboratory Testing
95+00	Boring	Centerline	40 ft	[a]R-bar /Index
100+00	Boring	Centerline	40 ft	[a]R-bar /Index
110+00	Boring	Centerline	40 ft	[a]R-bar /Index
113+00	Boring	Centerline	40 ft	[b]Index
116+00	Boring	Centerline	40 ft	[b]Index
119+00	Boring	Centerline	Refusal	[a]R-bar /Index
122+00	Boring	Centerline	40 ft	[b]Index
125+00	Boring	Centerline	Refusal	[a]R-bar /Index
128+00	Boring	Centerline	40 ft	[b]Index
131+30	Boring	Centerline	Refusal	[a]R-bar/Index
134+00	Boring	Centerline	40 ft	[b]Index
137+00	Boring	Centerline	40 ft	[b]Index
140+00	Boring	Centerline	40 ft	[a]R-bar/Index
[d]200+00	Boring	Centerline	40 ft	[c]Consolidation
[d]220+00	Boring	Centerline	40 ft	[c]Consolidation
[d]240+00	Boring	Centerline	40 ft	[c]Consolidation
[d]260+00	Boring	Centerline	40 ft	[c]Consolidation

[a] Used for Underseepage analysis and "undisturbed sampling for steady state, end-of-consolidation, and rapid drawdown slope stability analysis.
[b]Used for Underseepage analysis.
[c]Used for settlement analysis along levee raise.
[d]Borings along Big Blue River Levee Reach.

2) Dig test pits along landside levee toe in locations of proposed sand drains to confirm landside toe material type.

Station	Investigation Type	Location	Depth	Information to be Used For
[a]95+00	Test pit	Landside toe/slope	6 ft or until impervious material is encountered	Determine random material for underseepage analysis
[a]100+00	Test pit	Landside toe/slope	6 ft or until impervious material is encountered	Determine random material for underseepage analysis
[a]130+00	Test pit	Landside toe/slope/Riverside toe	6 ft or until impervious material is encountered	Determine random material for underseepage analysis
[a]140+00	Test pit	Landside toe/slope	6 ft or until impervious material is encountered	Determine random material for underseepage analysis
[a]150+00	Test pit	Landside toe/slope	6 ft or until impervious material is encountered	Determine random material for underseepage analysis
[a]163+00	Test pit	Landside toe/slope	6 ft or until impervious material is encountered	Determine landside random material for stability and underseepage analysis
[a]165+00	Test pit	Landside toe/slope	6 ft or until impervious material is encountered	Determine random material for stability and underseepage analysis
[b]170+00	Test pit	Landside toe/slope/Riverside toe	6 ft or until impervious material is encountered	Determine random material for stability and underseepage analysis
180+00	Test pit	Landside toe/slope	6 ft or until impervious material is encountered	Determine random material for stability analysis
230+00	Test pit	Landside toe/slope	6 ft or until impervious material is encountered	Determine random material for stability analysis
240+00	Test pit	Landside toe/slope/Riverside toe	6 ft or until impervious material is encountered	Determine random material for stability analysis
250+00	Test pit	Landside toe/slope	6 ft or until impervious material is encountered	Determine random material for stability analysis
260+00	Test pit	Landside toe/slope	6 ft or until impervious material is encountered	Determine random material for stability and underseepage analysis
270+00	Test pit	Landside toe/slope/Riverside toe	6 ft or until impervious material is encountered	Determine random material for stability and underseepage analysis

[a]Levee reach along Kansas River.
[b]Levee reach at confluence of Kansas and Big Blue Rivers.

3) Perform settlement analysis to determine levee overbuild requirements in levee raise sections.

4) Detailed analysis of landside slope stability in areas outside proposed levee raises based on anticipated development of embankment through seepage and underseepage for design flood events.

5) Finalize design of relief wells and flow collection systems based on pilot holes boring logs.

US Army Corps of Engineers
Kansas City District

Part 4
Structural Engineering

Alternatives Engineering
Manhattan, KS

August 2014

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4.1 Background

The structural features considered for the Manhattan Levee feasibility study alternative plans (future with project conditions) consist mainly of pump stations and gatewell replacements, additions, or strengthening. The alternative plans examined and documented within Part 4 are listed below:

- Plan 2 raising current level of protection to satisfy passing the .5% flood
- Plan 3 raising current level of protection to satisfy passing the .33% flood
- Plan 4 raising current level of protection to satisfy passing the .2% flood
- Plan 5 +CW raising levee to pass .33% flood and widening of the Big Blue River channel

Structural analysis of the alternatives involved using the previous assessment of structural existing conditions found in the reference below, experience with similar levee improvement projects within the Kansas City District, and engineering judgment. The recommended structural measures for each of the final alternatives are listed below. This analysis was conducted at a level suitable for costing at this stage of the feasibility study. Additional refinements to the structural measures and cost are expected as the plan alternatives are compared and a final alternative selected.

Note that in the existing conditions analysis (Volume 1, Part 4); the components of the existing levee 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 existing condition analysis simply provides information about the possible performance of the structure for the loads under consideration.

4.2 References

1. Volume 1 Existing Conditions Engineering Analysis, Part 4

4.3 Structural Features for Alternative Plans

4.3.1 Plan 2

Structural Features - This alternative primarily consists of an average levee raise of .75 feet which will add additional loads to the already non-conforming structures (as detailed in the reference). This would entail replacing of five gatewells and having an Emergency Closure Structure (ECS) of fill and plastic sheathing available if flooding occurs during construction as detailed in the Hydrology and Hydraulics section of the Engineering Appendix Vol. 2.

Costs - Were determined by taking recent information regarding the construction of gatewells for Dodson Flood Protection Project in Kansas City, MO in 2011 with

contingencies added. A potential exists for modifying the existing structures to increase strength verses full replacement that will be investigated during the next phase of work. Structural modifications if proven adequate could lower the overall cost of the project verses full replacement.

4.3.2 Plan 3

Structural Features - This alternative consists of an average levee raise of 1.5 feet which will add additional loads to the already non-conforming structures (as detailed in the reference). This would entail replacing five gatewells and providing the ECS mentioned above.

Costs - Were determined by taking recent information regarding the construction of gatewells in 2011 for Dodson Flood Protection Project in Kansas City, MO with contingencies added.

4.3.3 Plan 4

Structural Features - This alternative consists of an average levee raise of 2.1 feet which will add additional loads to the already non-conforming structures (as detailed in the reference). This would entail replacing thirteen gatewells, raising one gatewell, strengthening the walls and floor slabs of one pump station, and abandonment of the Old By-Pass pump station near Poyntz Ave and adding a Stoplog Gap at Manhattan Avenue and ECS mentioned above.

Costs - Were determined by taking recent information regarding the construction of gatewells in 2011 for Dodson Flood Protection Project in Kansas City, MO. Pump station and stoplog costs were obtained using parametric cost data from cost engineering with contingencies added.

4.3.4 Plan 5+CW

Structural Features - This alternative consists of an average levee raise of 1.5 feet which will add additional loads to the already non-conforming structures (as detailed in the reference). This would entail replacing five gatewells and providing an ECS as mentioned above. This alternative also includes expansions of the Highway 24 and Union Pacific railroad (UPRR) bridges. This expansion would entail extensive rehabilitation of the existing Highway 24 Bridge by extending the eastbound and westbound lanes by approximately 160 feet. The Union Pacific Railroad Bridge, is estimated to be approximately 80+ years-old, and assumed to be a total replacement for cost estimating purposes.

Costs - Were determined by taking recent information regarding the construction of gatewells in 2011 for Dodson Flood Protection Project in Kansas City, MO. with contingencies added. Bridge cost information was taken from the MoDOT preliminary design manual tables. Bridge costs breakdown is as follows:

Highway 24 Bridge Expansion Estimate for Manhattan Feasibility Study								
$ 250.00	Avg. price per SQFT of deck major river crossing per MoDOT reference provided							
	by USACE structural engineering section member with DOT bridge experience.							
160	New total length of bridge (ft) (includes replacement of existing span on left bank)							
32	Width (ft)							
$ 1,280,000								
$ 2,560,000	Two Bridges(East and West), each 2 lanes							
	100% contingency due to unknowns.							
$ 5,120,000	Total Cost							
say								
$5,000,000								

New UP RR Bridge								
$ 405.00	per SQFT	http://on.dot.wi.gov/dtid_bos/e			per SQFT			
$ 9,160.00	per LF	http://www.dot.state.mn.us/pa			per LF			
600	New total length of bridge (ft)							
	(existing is 440 lf plus 160 foot extension from CW)							
20	Width (ft)							
$ -	by SF cost							
$ 5,496,000	by LF cost	Assume higher cost and apply contingency						
	100% contingency due to unknowns:							
$ 10,992,000	Total Cost							
say								
$11,000,000								

Another significant concern with this alternative is on the west side (the City side) of the Highway 24 and UPRR bridges, as there is a highly complex situation involving aesthetic, historic, and recreational aspects. This location is near the main entrance of the Linear Park Trail, a main entrance for hiking and biking. The trail goes under the bridge, and it is fairly evident that it was selected to go through this area in part due to the scenic and historic nature of this particular area. The UPPR Bridge appears to have been constructed during the 1920's or 1930's, and may be eligible for the national registry of historic bridges. Based on these factors, the channel widening alternative could potentially be highly problematic and costly as a result of the many public interests associated with this area.

4.4 Structural Feature Overview

4.4.1 Drainage Structures

Drainage structures associated with the Manhattan Levee include: gated through levee drainage structures and a conduit over the levee. The project has 29 drainage structures with two located in the ponding levee. The project contains 16 gated main embankment structures and two gated ponding levee structures. Table 4-1 below provides details of these structures as well as for other project structures.

4.4.2 Closure Structures

Closure structures associated with the Manhattan Levee originally included one stoplog gap and two sandbag gaps. The stoplog gap at Station 83+24 and the sandbag gap at Station 194+48 were constructed to accommodate railroad lines that intersected the flood protection system. The railroad line that used the stoplog gap has since been abandoned. The stoplog gap has been permanently closed with the construction of a floodwall spanning between the stoplog slots. It is unclear exactly when this wall was constructed. The railroad constructed the original stoplog gap and no plans are available of that construction. In addition, it is unclear who constructed the floodwall to close the structure, and no plans are available for this replacement (see appendix, photo # 1). An analysis was conducted of the in-fill wall for strength during this phase of work and can be found in reference 1. The in-fill wall was found to be unacceptable for strength with water to top-of-wall. Per the recommendations in reference 1, the Sponsor has since added approved fill to the riverside of the structure to equalize loading. The sandbag gap at Station 194+48 has a concrete sill and was constructed with the rest of the levee project in 1963. The third, a sandbag gap at Manhattan Avenue, was constructed to accommodate the main levee flood protection system (see appendix, photo # 4). This sandbag gap simply consists of the roadway and shoulders and was constructed with the rest of the levee project in 1963.

4.4.3 Pumping Plant Structures

There are three structures associated with the Manhattan Levee. The Manhattan Avenue Pumping Plant is located at Station 20+13, Poyntz Avenue Pumping Plant located at Station 101+26 reconstructed in 2002, and the Old-By-Pass pump station located at Station 98+80. In addition to the pump wells and the associated gatewell structures, each of these plants has building structures that protect the pumps and equipment. The Manhattan Avenue and Old-By-Pass plants have a metal panel building enclosing its pumps. The Poyntz Avenue plant has modular block retaining walls surrounding a block wall building enclosing its pumps (see appendix, photos # 2, 3, 4).

Table 4-1: Structures and Sandbag Gaps

TABLE 3.1 Structures and Sandbag Gaps													
Station	**Element in which Structure Located**	**Pipe ϕ (inch)**	**Type**	**Qty**	**Pipe Invert (ft) (at the Outlet)**	**Entrance/Exit Structure**		**Gate Type(s)**			**Drawing Year**	**Recorded In**	**Remarks**
						Inlet	Outlet	GW	FG	SG			
14+78	Tieback Levee	84	RCP CMP	1	1000.88	ST	CS	X	X	X	1963	ABD	2 Gatewells Plate Nos.14, 18 & 20
20+00	Tieback Levee	Sandbag Gap at Manhattan Ave.									1963	ABD	Plate No. 3
20+13	Tieback Levee	72	RCP	1		ST	CS	X	X	X	1963/ 2003	ABD	Manhattan Ave Pumping Plant and GW Plate Nos. 23-25, 30-35, 37-38
34+62	Tieback Levee	24	RCP	1	1007.72	ST	CS	X	X	X	1963	ABD	Plate Nos. 15, 19 &21
40+00	Tieback Levee	66	RCP	1	*	*	*	X	*	*	*	2010 AI	Structure construction noted in 2010 AI photos
62+20	Tieback Levee	48	RCP	1	1002.9	CS	CS	X	X	X	1963	ABD	Plate Nos. 15 & 20
69+00	Tieback Levee	60	RCP	1	*	*	*	X	*	*	*	2009 Letter	Structure construction approved in 2009 ED-G Letter
81+25	Tieback Levee	24	RCP	1	1005.5	CS	CS	X	X	X	1963	ABD	Plate Nos. 15, 19 & 21
83+24	Main Levee	Stoplog Gap at UPRR. Gap constructed by RR. Gap later closed with floodwall (After 1977 PI). ED-DS is planning to make a site visit to obtain field measurements of the floodwall prior to PI.									1963	ABD	Plate No.5 and info. from Feasibility Study
89+83	Main Levee	72	RCP	1	995.7	ST	CS	X		X	1963	ABD	Plate Nos. 15 & 19
98+80	Main Levee	14	CIP	1	999.7	*	CS	GV	X		1963	ABD	Pressurized Flow Plate Nos. 16, 19 & 21
99+15	Main Levee	24	RCP	1	991.34	ST	CS	X	X	X	1963	ABD	Plate Nos. 16, 19 & 21
99+29	Main Levee	24	CIP	1	*	*	CS	*		*	*	PI 1	Recorded in 1971 PI 1, Page III-4-8
101+26	Main Levee	10' x10'	RCB	1	989.3	CS	CS	X	X	X	1963	ABD	Poyntz Ave Pumping Plant and GW Plate Nos. 26-38
105+05	Main Levee	24	CIP	1	1000.4	*	CS	X	X	X	1963	ABD	Plate Nos. 16, 19 & 21
155+00	Main Levee	*	*	1	*	*	*	X	*	X	*	PI 2	Recorded in 1977 PI 2, Page III-2-3-2
163+00	Main Levee	84	RCP	1	992.8	CS	CS	X		X	1963	ABD	Plate Nos. 53, 55 & 56
194+48	Main Levee	Sandbag Gap at UPRR.									1963	ABD	Plate No. 54
234+00	Main Levee	24	RCP	1	1000.0	CS	CS	X	X	X	1963	ABD	Plate Nos. 53, 54 & 56
269+50	Main Levee	72	RCP	1	996.9	CS	CS	X	X	X	1963	ABD	Plate Nos. 53, 55 & 56
West Crossing of Fairlane	Man. Ave. Ponding Area	36	CMP	1	1010.0						1963	ABD	Plate No. 3

TABLE 3.1 Structures and Sandbag Gaps

Station	Element in which Structure Located	Pipe ф (inch)	Type	Qty	Pipe Invert (ft) (at the Outlet)	Entrance/Exit Structure		Gate Type(s)			Drawing Year	Recorded In	Remarks
						Inlet	Outlet	GW	FG	SG			
East Crossing of Fairlane	Man. Ave. Ponding Area	36	CMP	1	1010.0						1963	ABD	Plate No. 3
Between Southern Ponding Areas	Man. Ave. Ponding Area	48	CMP	1	1010.0						1963	ABD	Plate No. 3 & 14
14+50	Man. Ave. Ponding Area	12	CMP	1		CS					1963	ABD	Plate Nos. 3 & 10
18+25	Man. Ave. Ponding Area	12	CMP	1		CS					1963	ABD	Plate Nos. 3 & 10
10+00	Poyntz Ponding Levee	18	*	1	*	*	*		X		1963	ABD/ PI 1	Plate No. 48 in ABD and 1971 PI 1, Page III-4-11
20+73	Poyntz Ponding Levee	18	*	1	*	*	*		X		1963	ABD/ PI 1	Plate No. 48 in ABD and 1971 PI 1, Page III-4-11
22+00	Landside Drainage Ditch	24	CMP	1	1016.2						1963	ABD	Plate No. 3
44+95	Landside Drainage Ditch	24	CMP	1	1009.2						1963	ABD	Plate Nos. 4 & 10
47+75	Landside Drainage Ditch	24	CMP	1	1008.5						1963	ABD	Plate Nos. 4 & 10
56+25	Landside Drainage Ditch	36	CMP	1	1006.4						1963	ABD	Plate Nos. 4 & 10
61+12	Landside Drainage Ditch	42	CMP	1	1005.2						1963	ABD	Plate Nos. 4 & 10

4.5 Enclosures

4.5.1 Photos of Structures

Figure 1: Riverside face of former stop log gap Sta. 83+24 prior to placement of fill.

Figure 2: Sta. 101+26 Poyntz Ave. pump station in foreground adjacent to levee. Notice Old By-Pass pump station in the background near white GSA vehicle.

Figure 3: New Poyntz Ave pump station. Sta. 101+26

Figure 4: Sand Bag Gap at Sta. 20+00 Standing on right abutment (if viewed from landside). Gap extends across Manhattan Ave. to left abutment across street. Manhattan Ave. pump station in background.

US Army Corps of Engineers
Kansas City District

Part 5
Civil

Alternatives Engineering
Manhattan, KS

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5-1. Introduction

This document presents results of utility uplift and limited hydraulic consideration of pump stations. Plans considered include:

-Plan 1 – No Federal action.

-Plan 2 – 0.5% plan – raising the current levee to pass the nominal 0.5% chance flood event profile with accompanying geotechnical and structural reliability improvements.

-Plan 3 - 0.33% plan – raising the current levee to pass the nominal 0.33% chance flood event profile with accompanying geotechnical and structural reliability improvements.

-Plan 4 – 0.2% plan – raising the current levee to pass the nominal 0.2% chance flood event profile with accompanying geotechnical and structural reliability improvements.

-Plan 5 – 0.33%+CW plan – raising the current levee according to plan 3 with the addition of channel widening (CW) and bridge modifications on the Big Blue River for increased flood conveyance.

Plan 3 is the recommend plan. This alternative entails a levee raise averaging 1.5 feet, with a maximum raise of approximately 3.3 feet. The other plans were considered, but only to sufficient detail to allow comparison and evaluation. Please refer to the existing conditions analysis for full calculations and figures to support the analysis.

5-2. Utility Uplift Anaylsis

Please refer to the existing conditions analysis and the summary section of this document for results of uplift analysis and recommendations for addressing problematic areas.

5-3. Above Ground Utilities and Features

Above ground features such as bridges and overhead utilities were considered with regard to prospective levee raises, and how those levee raises might affect clearance. No bridges were in the range of the plan 3 alternative raise. Levee raises associated with the plan 3 alternative average 1.5 feet, with a maximum raise of 3.3 feet, resulting in a decrease in clearance for overhead electric lines at two locations. Limited field efforts determined that these lines probably would still have sufficient clearance, even with the levee raise, however cost contingencies are included to cover the unlikely possibility that these lines would need to be raised.

5-4. Pump Station Assessment

The plan 3 alternative considers relief wells, all of which would discharge within the drainage area of the Poyntz Avenue pump station. The Manhattan Avenue Pump Plant would be unaffected. The plan 2 alternative would entail somewhat fewer relief wells, but at similar locations.

The original Poyntz Avenue station was demolished and replaced with a new / larger station in 2003 by local interests. While design documentation was unavailable, this larger station was most likely required as a result of extensive development within the drainage area and possibly some filling of the original ponding area. Adding relief well flow would result in this station becoming a “dual purpose” facility, e.g. handling runoff from an intense local storm coincident with river stages just high enough to require pumping (design storm); and handling runoff from a somewhat smaller coincident storm coincident with high river stages, resulting in maximum relief well flow (design river stage). The “design storm” case would be expected to control nearly every time, particularly in this case, where the discharge is up and over the levee and therefore pumping capacity is unaffected by changes in river stage. So while the hydraulic effect of additional relief wells was not calculated, it is assumed to be negligible for the reasons noted above and will be confirmed during the design phase. The possibility for needing to increase pumping capacity at Poyntz, while believed to be very small, was captured as a line item in the cost and schedule risk analysis.

The plan 4 raise would entail extensive numbers of relief wells throughout the system, resulting in the need for at least one new pump station. This was accounted for as a line item in the plan 4 cost estimate.

5-5. Road Raises

The plan 3 levee raise originally considered a 500 foot road raise at the Casement Road tie-in. This road raise would also extend through the intersection at Hayes Drive and terminate near Judson Street. No utility relocations were identified at this location. Cost estimate quantities were considered for pavement removal, fill for the tie in and for transitions, and re-paving. No other road raises were anticipated for plan 3.

The PDT was later asked to consider alternatives to the road raise, and developed preliminary costs for extending the levee southwest along Casement Road, terminating near Judson Street, and leaving a 90 foot long x 2 foot high sandbag gap across Hayes Drive. While this option was found to be feasible, and at less cost than the road raise, it would require the Sponsor to seal off the sandbag gap during times of high flow. This option was cooridinated with the Sponsor, who preferred this type of arrangement to the road raise. As such, the cost estimate was updated to include this levee extension with sandbag gap in lieu of the road raise.

5-6. Items of Consideration to Improve Reliability of Utilities for Resisting Uplift And Proposed Levee Raises

The PDT was asked to consider what improvements would be required to address deficiciencies noted under existing conditions and to provide various magnitudes of levee raises. These improvements include a various levee raises, berms and / or relief wells, improvements to existing structures, and incidental utility relocations and other work.

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. Any potential alternatives involving berm construction in this area would require top slabs of these manholes to be raised accordingly to match the final ground surface elevation.

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.

Other Utilities

Thirty power poles within the area of the proposed berm from Station 104+00 to 140+00 would need to be raised an average of 8 feet.

An 8-inch diameter gas line at Station 227+00 currently passes up and over the existing levee. This gas line will need to be raised approximately 3 feet to accommodate the proposed levee raise in this area, and new isolation valves installed.

A 36-inch diameter water line at Station 232+28 currently passes up and over the existing levee. The water line will need to be raised approximately 3 feet to accommodate a proposed levee raise in this area, and new isolation and air release valves installed.

Five gate well structures are proposed for replacement, as described in the structural section of the report. Three of the gate wells are located in areas in which the levee is not being raised. In these cases, the inlet and outlet pipes to be replaced are assumed to be very short; the minimum length so as to allow the existing gate wells to be demolished and new structures built. Two of the gate wells (Sta 163+00 and Sta 269+50) are located in areas in which the levee is being raised. Since these inlet and outlet pipes will experience higher loads as compared to original designs, full pipe replacement is recommended and included in the cost estimate.

Borrow Quantities and Source of Material

The total fill quantity was estimated at 200,000 bank cubic yards (BCY), based on early evaluation of likely levee raise and berm locations. This amounts to an area approximately 13 acres by 10 feet deep. Later refinement resulting in a lesser required borrow amount, however the original 200,000 BCY figure was retained for purposes of identifying prospective borrow areas. There is a large open area approximately one mile north of the Big Blue confluence that was originally considered for purposes of generating estimated costs. “On highway" vehicles (e.g. excavate / load onto trucks, travel on public roads) would be used to transport this material on Knox Lane, west to Casement Road, south to the levee, and then traveling along the levee, for a total one way haul distance of approximately 3 miles. This area was later determined to be unavailable, prompting Corps staff to conduct a limited sensitivity analysis of haul distance effect on estimated construction cost. This effort showed that, even with a doubling of the haul distance (6 miles), cost increases were small (less than 5%) relative to the overall project. Cost increases to the earthwork components of the project (levee raise and berms), however, are more significant. Therefore, the cost estimate includes costs for borrow material purchased as a provision of the construction contract, and also contains contingencies to capture the potential costs associated with a longer haul distance.

5-7. Alternative Plans

The PDT was asked to consider alternate plans to address deficiencies noted under existing conditions and to provide various combinations of features and levels of raise. These alternatives were not developed to the level of the plan 3 preliminary key concept, but contained sufficient detail so as to allow preliminary comparison and evaluation. The alternatives included plan 2, plan 4, and plan 3 with channel widening. Earthwork quantities are included with the cost estimating information, and while they vary between alternatives, tend to be driven largely by fill requirements underseepage berms which are needed for all proposed levee raises. Utility relocations are also provided in the cost estimating information, and are pro-rated based upon results from the plan 3 alternative. Other uplift and utility concerns are the same for all other alternatives as for plan 3, that is, they need to be remedied, as discussed in part 4.6, regardless of the level of levee raise.

5-8. Summary Of Survey Work

Survey information was obtained from a variety of sources, and is discussed in a series of memos in the project file. Most of the recent information is in NAD 83 (H) and NAVD 88 (V) datums. Some of the older information such as Corps of Engineers record drawings is in NAD 27 (H) and NGVD 29 (V) datums. The PDT is aware of the differing survey datums, and adjusts information as required to ensure consistency in the results presented.

The survey information currently available is believed to be adequate for feasibility-level efforts, but not for detailed design. When design efforts commence, a new survey will be conducted.

5-9. 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/