Papillion Creek and Tributaries Lakes, Nebraska

General Reevaluation Report

US Army Corps of Engineers®

Appendix B
Hydraulics

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Omaha District
Northwestern Division

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EXECUTIVE SUMMARY

The purpose of this general reevaluation report (GRR) study was to model flood risk management alternatives in the Papillion Creek watershed. The hydraulic appendix documents the hydraulic analysis completed for these evaluations.

This appendix focuses on the hydraulic modeling efforts to determine federal interest in various considered alternatives. Steady flow hydraulic modeling was used first to screen out alternatives in a time-efficient manner in accordance with the large scope of this study. Once economically justified alternatives were identified from the steady flow hydraulic analysis, these results were confirmed using more detailed and time-intensive unsteady flow hydraulic modeling. Alternatives that were still justified went through an optimization process to determine the final optimized plan. Table 1 provides a summary of all the alternatives considered and when in the process they were eliminated to determine the final optimized plan.

Table 1 Summary of Alternatives Considered

	Alternative	Location
Preliminary Screened Alternatives – Screened out prior to detailed economic evaluation based on evident disproportionate cost to benefits and environmental and/or societal concerns using steady flow hydraulic analysis	Dam Site 3C	Big Papillion, upstream of Pawnee Road
	Dam Site 7	Tributary to Big Papillion, downstream of Bennington Road
	Dam Site 8A	Tributary to Big Papillion, downstream of Bennington Road
	Dam Site 9A	Tributary to Big Papillion, downstream of Bennington Road
	Dam Site 12	West Papillion, upstream of West Maple Road
	Combination of Dam Sites 7, 8A, 9A, 10, 12, and 19	Throughout watershed
	Big Papillion Channel Widening, Steady Flow Damage Reaches BP4 through BP6	Blondo St to West Center Road
	West Papillion Channel Widening, Steady Flow Damage Reaches WP6 through WP7	Hillsdale Drive to RR crossing North of Giles Road
	Little Papillion Channel Widening, Steady Flow Damage Reaches LP2 through LP8	Grover St to West Center Rd and Dodge Street to Maplewood Boulevard
	South Papillion Channel Widening, Steady Flow Damage Reach SP3	Railroad crossing upstream of 144th Street to confluence with West Papillion

	Alternative	Location
Preliminary Screened Alternatives – Screened out prior to detailed economic evaluation based on evident disproportionate cost to benefits and environmental and/or societal concerns using steady flow hydraulic analysis (continued)	Big Papillion Levee Raise to the 0.2% AEP, Damage Reaches BP6 through BP8	L Street to the confluence with the West Papillion, including Little Papillion tie back from the Little Papillion confluence to L Street
	Big Papillion Levee Raise to the 1% AEP, Damage Reaches BP6 through BP8	L Street to the confluence with the West Papillion, including Little Papillion tie back from the Little Papillion confluence to L Street
	West Papillion Levee Raise to the 0.2% AEP, Damage Reach WP9	84th St to Big Papillion confluence
	West Papillion Levee Raise to the 1% AEP, Damage Reach BP9	84th St to Big Papillion confluence
	West Papillion W. Maple Rd Culvert Modification, Damage Reach WP1	West Maple Road
	UPRR Crossing, Damage Reach LP8	Union Pacific Railroad Crossing over Little Papillion, just downstream of I-80
	I-80 Crossing, Damage Reach LP8	I-80 Crossing over Little Papillion
	Pedestrian Crossing, upstream of Mercy Road, Damage Reach LP7	Pedestrian Crossing over Little Papillion, upstream of Mercy Road
	Pedestrian Crossing, west of the College of St Mary’s campus, Damage Reach LP7	Pedestrian Crossing over Little Papillion, west of College of St. Mary's campus
	Pedestrian Crossing, downstream of Pine Street, Damage Reach LP7	Pedestrian Crossing over Little Papillion, downstream of Pine Street
	Pedestrian Crossing, downstream of Pacific Street, Damage Reach LP7	Pedestrian Crossing over Little Papillion, downstream of Pacific Street
	Pacific Street Bridge, Damage Reach LP6	Pacific Street crossing over the Little Papillion
Screened out after detailed economic analysis based on steady flow hydraulic modeling	Little Papillion Levee/Floodwall to the 0.2% AEP, Damage Reaches LP5 through LP7	Mercy Road to Cass Street
	Little Papillion Levee/Floodwall to the 1% AEP, Damage Reaches LP5 through LP7	Mercy Road to Cass Street

	Alternative	Location
TSP after detailed economic analysis based on steady flow hydraulic modeling	Dam Site 10	Thomas Creek, upstream of Bennington Road
	Big Papillion Channel Widening, BP4 and BP5	Blondo St to 105th Street
	Big Papillion Levee Raise, Damage Reaches BP7 and BP8	I-80 to Harrison Street on the Big Papillion, Big Papillion confluence to I-80 on the Little Papillion
	Little Papillion Creek Levee/Floodwall in combination with Dam Site 10	Mercy Road to Cass Street
Carried into Unsteady Modeling despite BCR < 1	Dam Site 19	South Papillion Creek, upstream of 192nd Street
	West Papillion Creek Floodwall, Damage Reach WP6	Boxelder Cr to Millard Avenue
Screened out after detailed economic analysis using unsteady flow hydraulic modeling	Big Papillion Channel Widening	Blondo St to 102nd Street
	West Papillion Creek Floodwall	Boxelder Creek to Millard Avenue
	Big Papillion Creek Levee Raise	I-80 to Harrison Street on the Big Papillion, Big Papillion confluence to I-80 on the Little Papillion
Final Optimized Plan	Dam Site 10	Thomas Creek, upstream of Bennington Road
	Dam Site 19	South Papillion Creek, upstream of 192nd Street
	Little Papillion Creek Levee/Floodwall to the 1% EGL with 3 additional ft in combination with Dam Site 10	Mercy Road to Western Avenue

1. INTRODUCTION

This report describes the hydraulic analysis conducted in the Papillion Creek Basin for the Papillion Creek and Tributaries Lakes General Reevaluation Report (GRR). The primary goal of the project is to address flood and life safety risk issues in Douglas, Sarpy, and Washington counties in order to reduce flood and life safety risks in the Papillion Creek Basin. Note that this is a feasibility level assessment; further details will be refined during design.

The hydraulic analysis computes water-surface profiles and inundation mapping for the 50-, 20-, 10-, 4-, 2-, 1-, 0.5-, 0.2-percent annual exceedance probability (AEP) event discharges, commonly known as the 2-, 5-, 10-, 25-, 50-, 100-, 200-, and 500-year return intervals, for both existing and future basin conditions for with- and without- project alternatives . Refer to the Hydrology Appendix A for additional information regarding existing and future basin condition assumptions. Potential projects within the study area focused on the following tributaries: Papillion Creek, Big Papillion Creek, West Papillion Creek, Little Papillion Creek, South Papillion Creek, Thomas Creek, Cole Creek, and Saddle Creek, see Figure 1.

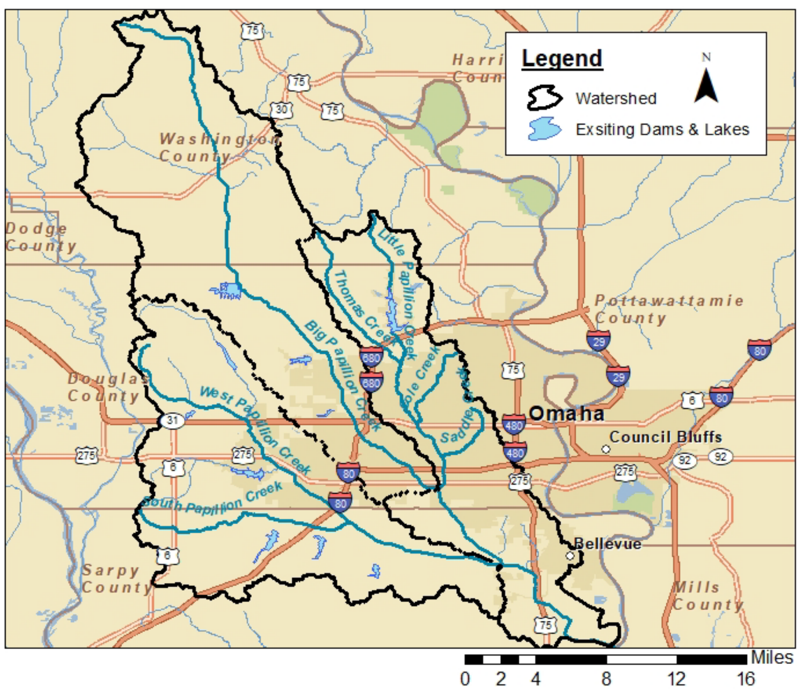


Figure 1. Study Area General Overview

2. 1D STEADY FLOW HYDRAULIC MODELING

The Hydrologic Engineering Center's River Analysis System version 5.0.5 (HEC-RAS) was used to perform one-dimensional (1D) steady flow hydraulic modeling to screen the proposed structural alternatives and determine the Tentatively Selected Plan (TSP). After the TSP milestone meeting, the hydraulic modeling was refined from 1D steady flow to 1D/2D unsteady

flow modeling which includes full hydrographs as an input. This modeling is detailed in Section 3.
The hydraulic analysis evaluated water surface elevations for existing without- and with-project conditions as well as future without-project conditions. The computed water surface elevations were used for economic modeling in the Hydrologic Engineering Center's Flood Damage Analysis software (HEC-FDA). Delineation of inundated floodplain was also developed as part of the hydraulic analysis.

To aid the economics analysis during the screening process, the 400 square mile Papillion Creek watershed was divided into 34 damage reaches. Table 2 below describes the breakdown of the damage reaches while Appendix B-P1 provides this information in map form.

Table 2. Damage Reach Breakdowns

Damage Reach	Upstream Boundary	Downstream Boundary
BP1	Upstream Extent	Military Road
BP2	Military Road	West Maple Road
BP3	West Maple Road	Blondo Street
BP4	Blondo Street	West Dodge Road
BP5	West Dodge Rd	105th Street
BP6	105th Street	Railroad Crossing, downstream of I-80
BP7	Railroad Crossing, downstream of I-80	Railroad Crossing, downstream of Q Street
BP8	Railroad Crossing, downstream of Q Street	36th Street
BP9	36th Street	Big Papillion/West Papillion Confluence
PC1	Big Papillion/West Papillion Confluence	Hwy 75, south of Offutt AFB
WP1	Upstream Extent	Old Lincoln Hwy, upstream of 192nd Street
WP2	Old Lincoln Hwy, upstream of 192nd Street	168th Street
WP3	168th Street	Pacific Street
WP4	Pacific Street	West Center Road
WP5	West Center Road	144th Street
WP6	144th Street	Millard Avenue
WP7	Millard Avenue	Giles Road
WP8	Giles Road	96th Street
WP9	96th Street	Big Papillion/West Papillion Confluence
LP1	Upstream Extent, downstream of Cunningham Lake	Blair High Road
LP2	Blair High Road	Maple Street
LP3	Maple Street	Blondo Street
LP4	Blondo Street	Western Avenue

Damage Reach	Upstream Boundary	Downstream Boundary
LP5	Western Avenue	Dodge Street
LP6	Dodge Street	Pacific Street
LP7	Pacific Street	Mercy Road
LP8	Mercy Road	Big Papillion/Little Papillion Confluence
SP1	Upstream Extent	173rd Street
SP2	173rd Street	Railroad Crossing, downstream of 156th Street
SP3	Railroad Crossing, downstream of 156th Street	West Papillion/South Papillion Confluence
TC1	Upstream Extent	Bennington Road
TC2	Bennington Road	Blair High Road
TC3	108th Street	Thomas Creek/Little Papillion Confluence
CC1	Upstream Extent	Cole Creek/Little Papillion Confluence
SC1	Upstream Extent	Saddle Creek/Little Papillion Confluence

All elevation data in this report references the North American Vertical Datum of 1988 (NAVD88) unless otherwise specified.

The horizontal coordinate system used for hydraulic modeling was NAD 1983 State Plane Nebraska FIPS 2600.

2.1 Existing Conditions Model Development

The following sections describe the data sources, methods, and assumptions used in developing the 1D HEC-RAS models for the Papillion Creek and Tributaries Lakes GRR.

2.1.1 Model Overview

Existing models created to update the Flood Insurance Study (FIS) and Flood Insurance Rate Map (FIRM) in 2016 were obtained from the Papio-Missouri River Natural Resources District (P-MRNRD) to expedite the project schedule. At the time of this study, remapping had not been made public. Table 3 describes the model reaches.

Table 3. River Reach Description

River	Reach Length (ft)	Downstream Cross-Section	Upstream Cross-Section	Number of Cross-sections	Number of Bridges
Big Papillion Creek/Papillion Creek	156430	500	162650	289	43
West Papillion Creek	116773	530.088	116773.1	309*	36
Little Papillion Creek	55543	441	55543	234	32

South Papillion Creek	48654	2224.846	52274.71	126*	18
Thomas Creek	37996	147	37996	159	14
Cole Creek	24533	40	24533	164	27
Saddle Creek	19962	812	20877	142	1

* Includes interpolated cross-sections

2.1.2 Hydrology

Recently approved hydrology provided in the *Papillion Creek Watershed Hydrologic Analysis,* see Reference 1, was reviewed by the United States Army Corps of Engineers (USACE) and used to update the peak flow inputs used in the steady flow modeling. The Hydrologic Engineering Center Hydrologic Modeling System (HEC-HMS) model used was provided by the P-MRNRD. More information on the HEC-HMS model and the previous analysis can be found in the Hydrology Appendix A.

As is typical in Federal Emergency Management Agency (FEMA) mapping, flow changes provided by the HMS model were applied at the upstream cross-section of each sub basin in the RAS model to account for inflow that enters the channel before the downstream end of the basin. This practice can result in conservative outputs from the hydraulic model and was kept consistent throughout steady flow modeling. Saddle Creek was not updated with the recently approved hydrology. However, no structural project alternatives exist on Saddle Creek and the modeling effort in this area focused on determining incurred damages due to backwater.

In some locations, HEC-HMS junctions showed an attenuation in flow. Flow attenuation present in without project conditions is significantly reduced by channel and levee projects which constrict flows. Therefore, in areas that show attenuation, attenuated flows were eliminated and the higher flows from upstream were carried through the section. This results in overstating damages in both the existing and future without project conditions, but significantly reduces the risk of under sizing any levee or channel widening alternatives. For this reason, some peak flows reported in the Hydrology A may be lower than the flows used in the HEC-RAS model.

One notable exception was the screening of P-MRNRD priority dam sites on tributaries to the Big Papillion Creek. By removing flow changes that resulted in flow attenuation in the existing model, analysis on the proposed dam alternatives in this area would not fully capture the reduction in peak flow provided by the reservoirs. Since the dam sites do not exist in the existing conditions model, it was difficult to determine whether the decrease in flow under with-project conditions result from the addition of a dam or the HEC-HMS calculated channel attenuation. Therefore, all flow changes provided by the reviewed HEC-HMS model were input into the Big Papillion Creek HEC-RAS model so that economic comparisons could be made. Using this assumption results in lower damages and potentially under sizing alternatives on the Big Papillion Creek during the screening process. Once a TSP was selected, an unsteady HEC-RAS model was used which computed its own attenuation. This modeling is described later in this report.

Original modeling referenced in the *Papillion Creek Watershed Hydrologic Analysis,* see Reference 1, reports a stretch of unsteady HEC-RAS modeling that was created for the Big Papillion Creek from F Street to the mouth of Papillion Creek. For the purposes of alternative screening, use of unsteady modeling was deferred until after TSP.

Additional information regarding the hydrologic analysis can be found in the Hydrology Appendix A.

Appendix B-P2 provides a summary of the study discharges used and shows the location of the HEC-HMS junctions and flow change locations in the HEC-RAS model.

2.1.3 Geometry

Original model geometry was reviewed to ensure that existing conditions were appropriately modeled. However, no additional surveys were conducted to confirm channel bathymetry for this study and station-elevation points in the model cross-sections were not updated. Cross-section naming was left unchanged from the original FIS models.

Generally, the reaches modeled are characterized by shallow channels with heavy vegetation on the overbanks in the upper regions of their respective watersheds. As they flow through the urban environment, many have been straightened. Several have become entrenched in areas with evidence of sloughing and bank scour. In some reaches, the built environment has encroached heavily into the overbanks while in others, development has been held back to provide grassed overbanks. Several sections already contain levees.

The Papillion Creek basin has experienced channel stability problems for years resulting from widespread urbanization which increases the volume and rate of flow into receiving channels. Typically, this results in vertical and lateral channel instability. Past projects to address channel instability include bank stabilization and vertical grade control structures. Some proposed alternatives in this report have the potential to increase instability, especially those that increase channel velocities. Alternatives considered in this analysis include channel widening and new levees or floodwalls. Channel velocities and possible adverse project impacts will be evaluated in the further design phases.

Roughness in the model was simulated using Manning's "n" values. Roughness values from the P-MRNRD obtained models were mostly left unchanged; however, some changes were made based on recent development and professional judgment. A summary of the roughness factors used is provided in Table 4.

Table 4. Roughness Values

River	Typical Channel Roughness Values	Typical Overbank Roughness Values
Big Papillion Creek	0.03 – 0.045	0.04 – 0.09
West Papillion Creek	0.028 – 0.055	0.04 – 0.08
Little Papillion Creek	0.020 – 0.055	0.030 – 0.450
South Papillion Creek	0.04 – 0.05	0.035 – 0.1
Thomas Creek	0.03 – 0.045	0.035 – 0.060
Cole Creek	0.030 – 0.045	0.040 – 0.060
Saddle Creek	0.015 – 0.023	0.023 – 0.045

Pictures supporting the use of these n-values are provided below in Figures 2 – 6.

Figure 2. South Papillion Creek at 126th and Giles Rd

Figure 3. Big Papillion at 86th and Fredrick

Figure 4. Big Papillion Creek at 108th and West Dodge Rd

Figure 5. West Papillion Creek at 143rd and L St

Figure 6. Little Papillion Creek at Maple St and Keystone Dr

Contraction and expansion (C&E) coefficients were set to 0.1 and 0.3, respectively, throughout most of the reach. Near bridges, the C&E coefficients were generally increased to 0.3 and 0.5, respectively, to reflect the losses incurred at abrupt transitions typically associated with bridges. At some bridges and culverts these values were increased based on engineering judgment. Values used for contraction and expansion coefficients did not exceed 0.6 and 0.8, respectively. No sensitivity was performed on the contraction and expansion coefficients.

There were 171 crossings in the P-MRNRD obtained models consisting of bridges, culverts, and aerial pipelines. Modeled crossing information is summarized in Appendix B-P3. For the purposes of this study, all crossings were assumed to be clear of debris and sediment.

These crossings were spot checked and mostly left unchanged. However, this review identified several discrepancies. These issues and their resolutions are summarized below in Table 5.

Table 5. Bridge Discrepancies

River	River Station	Description	Resolution
Big Papillion Creek	19829	A pedestrian bridge existed in the model but has since been removed.	Remove bridge from model.
West Papillion Creek	107329.85	A pedestrian bridge is missing from the model. The Q_{100} = 4,860 cfs at this location.	Note absence and acknowledge that it may influence computed water surface elevations.
West Papillion Creek	104060	The 204th St Bridge deck is at a much lower elevation in the model.	Note discrepancy. The lower modeled bridge deck may have a conservative effect on modeled water surface elevations. The actual 204th St bridge deck is significantly higher than modeled water surface elevations and the piers in the model are extended enough that their presence is being captured by the model.

River	River Station	Description	Resolution
West Papillion Creek	71824.61	A pedestrian bridge is missing from the model. The Q_{100} = 17,090 cfs at this location.	Note absence and acknowledge that it may influence computed water surface elevations
Little Papillion Creek	53403	Based on aerial photography, culvert appears to have been removed	Remove culvert from HEC-RAS model
Cole Creek	23515	A low-level pedestrian crossing is missing from the model.	This bridge should only impact lower flows. Ignore missing bridge and document as uncertainty.

Ineffective flow areas added to the geometry were also reviewed and modified in line with conveyance expansion and contraction guidelines.

Existing levees were modeled in the P-MRNRD provided models using levee points. However, for events that barely overtopped the levee system, large sections of inundation would be missing from the inundation output using this method. Therefore, these levee points were converted into ineffective flow areas. Modeling the levees with ineffective flow areas instead of levee points maintains the decrease in overbank conveyance that levees provide (up to top of levee) while still allowing the model output to inundate behind the levees according to the terrain. Inundation in overbank areas behind existing levees that remained disconnected from the channel were removed during post-processing using tools within ESRI ArcGIS 10.2.2. This is because these areas cannot be inundated until water surface elevations reached an elevation above the top of levee. Estimated levee elevations based on average overtopping depths were provided for the economic analysis at defined index points.

Blocked obstructions were used to account for structures that have the potential to block flood conveyance downstream. After reviewing the original models, these were left unchanged.

2.1.4 Boundary Conditions

Except for the Big Papillion Creek model, water surface elevations at each confluence were used as the downstream boundary conditions for each tributary assuming coincident events. Flows modeled to obtain the resultant water surface elevation were produced by the Papillion Creek HEC-HMS model. More information on this can be found in the Hydrology Appendix

A. Assuming coincident events is conservative and may overstate the damages in the existing and future without project conditions, leading to lower benefits realized. However, the effect of the downstream boundary condition does not propagate a significant distance upstream and did not affect the identification of the TSP. The water surface elevations used as downstream boundary conditions are summarized in Table 6. The downstream boundary condition used in the Big Papillion Creek/Papillion Creek model remained at normal depth s = 0.00134 which reflects the slope at the confluence with the Missouri River.

Table 6. Boundary Conditions – Existing Conditions

Annual Chance Exceedance	River					
	West Papillion Creek	Little Papillion Creek	South Papillion Creek	Thomas Creek	Cole Creek	Saddle Creek
50%	982.39’	995.86’	1018.54’	1070.74’	1027.88’	1006.77’
20%	986.79’	999.65’	1022.72’	1073.76’	1031.55’	1010.72’
10%	989.43’	1001.87’	1025.16’	1075.74’	1033.92’	1013.49’
4%	992.22’	1005.09’	1027.96’	1077.89’	1037.18’	1017.15’
2%	993.59’	1007.52’	1029.91’	1079.54’	1041.97’	1019.76’
1%	994.56’	1010.21’	1031.50’	1081.19’	1042.11’	1022.75’
0.5%	995.40’	1012.22’	1033.05’	1081.96’	1043.08’	1024.75’
0.2%	996.43’	1013.66’	1035.44’	1085.37’	1045.71’	1026.51’
Location	Big Papillion Confluence BP XS 43397	Big Papillion Confluence BP XS 71656	West Papillion Confluence WP XS 39898.36	Little Papillion Confluence LP XS 43674	Little Papillion Confluence LP XS 22999	Little Papillion Confluence LP XS 10832

2.2 Existing Conditions Results

Water surface profiles and inundation maps were developed for the 50-, 20-, 10-, 4-, 2-, 1-, 0.5-, and 0.2-percent AEP discharges (i.e., 2-, 5-, 10-, 25-, 50-, 100-, 200-, 500-year return intervals). Resulting inundation maps were filtered to remove hydraulically disconnected ponding areas. Filtered water surface grids were then sent to Economics for damage estimation.

2.2.1 Inundation Mapping

Inundation Mapping was completed using HEC-RAS RAS Mapper as well as ESRI ArcGIS 10.2.2 for Desktop. Terrain elevation data used for modeling was downloaded from the United States Geological Survey (USGS) and has a resolution of 1/9 Arc-Second (3.4 meters). Elevation data was published in January of 2012 and covers most of Douglas and Sarpy Counties in Nebraska.

Water surface grids and inundation boundaries for each creek were first created in HEC-RAS RAS Mapper. These were then exported as tiff files and shapefiles, respectively. In ArcMap, the shapefiles were used to trim hydraulically disconnected inundation from the water surface grids. The grids were then merged into a single grid prior to the econ analysis in HEC-FDA. Additional details can be found in the Economics Appendix F.

The final computed water-surface inundation boundaries for the 50% through the 0.2% AEP events for all modeled reaches are provided in Appendix B-P4.

It is important to note that current inundation mapping of the steady-state existing conditions shows inundation behind the existing Big Papillion Creek levees (from L Street to the West Papillion confluence) at the 4% AEP event despite the levees being designed to a larger design discharge along the Big Papillion Creek. This is due to backwater effects resulting from the recently updated hydrology increasing discharges at the Big Papillion/West Papillion confluence by three to four thousand cubic feet per second. This increase in flow at the junction causes an increase in water surface, at the 4% AEP event, by 1.2 feet at the confluence to 0.1 feet at L Street. Since this increase in water surfaces, caused by backwater effects, causes overtopping of the existing levee system at the 4% AEP event, inundation behind the levees is depicted in the mapping and accounted for in the economic modeling.

2.2.2 Water Surface Profiles

The final computed water-surface profiles for the 50% through the 0.2% AEP events for all modeled reaches are provided in Appendix B-P26.

2.3 Hydraulic Input to Economic Analysis

In accordance with current USACE guidelines, an uncertainty analysis was completed to better define flood damages. HEC-FDA version 1.4.2 was used.

The HEC-FDA model runs were executed by the USACE economist. Details pertaining to the HEC-FDA model can be found in the Economics Appendix F. Hydraulic input to the economic analysis is described herein.

2.3.1 Hydraulic Uncertainty

The stage-discharge function for each reach is based on the water surface profiles computed with the HEC-RAS model at the index station. Input to HEC-FDA requires the description of stage uncertainty of the computed water surface profiles.

Uncertainty in computed stage profiles reflects modeling assumptions, numerical errors, and parameter estimation. Model uncertainty was estimated for the entire study reach by performing sensitivity analyses with the 1D HEC-RAS model in accordance with the guidance found in EM 1110-2-1619, Risk-Based Analysis for Flood Damage Reduction Studies (USACE, 1996). Lacking observations of high-water marks to compare to simulated profiles, the standard deviation of the stage uncertainty for this study was estimated using the range in elevation between the "reasonable" upper and lower bounds on stage for a given discharge. The bounding stages were calculated by accounting for uncertainty in energy losses in the channel and floodplain through the varying of Manning's n-values in accordance with Figure 5-4 of EM 1110-2-1619.

The 1D HEC-RAS model was used to calculate water-surface elevations at each index location for the various combinations of n-values. The mean stage-discharge curves were computed based on the best-estimated n-values. For purposes of this study, the best-estimated n-values were based on the values found in the P-MRNRD supplied models. Estimated minimum and maximum n-values were then determined using Figure 5-4 in the above referenced EM. The upper and lower bounds of stages were computed using these estimated maximum and minimum n-values, respectively. Figure 5-4, provided in Appendix B-P5, lists the estimated minimum, normal, and estimated maximum n-values.

For this sensitivity analyses, the difference between the high and low water surface profiles was tabulated at each cross section. The differences were then averaged over each tributary and the standard deviation calculated using Equation 5-7 found in the above referenced EM.

$$S = E_{mean} / 4 \tag{1}$$

Where S = Standard Deviation
E_{mean} = mean stage difference between upper and lower limit water surface profiles

These average differences are summarized below in Table 7.

Table 7. HEC-RAS Sensitivity Analysis Summary

River	Average difference between high and low water surface elevations	Calculated Standard Deviation
Big Papillion Creek	3.40'	0.85'
Papillion Creek	-	0.75'*
West Papillion Creek	3.14'	0.78'
Little Papillion Creek	2.43'	0.61'
South Papillion Creek	3.00'	0.75'
Thomas Creek	1.71'	0.43'
Cole Creek	0.99'	0.25'**
Saddle Creek	0.10'	0.03'

*The standard deviation used on Papillion Creek is an average of the standard deviations on the Big Papillion, West Papillion, Little Papillion, and South Papillion Creeks.
**The calculated standard deviation on Cole Creek was not used in the economics analysis. Instead the Little Papillion Creek standard deviation of 0.61' was used.

For all reaches in the analysis, it was assumed that cross-sections were based on topographic mapping with 2-5' contours. Because the models initially obtained from the P-MRNRD are for the purpose of updating the FIS, it was also assumed that the Manning's n-Value reliability is good and has been calibrated to high water marks. Thus, based on the EM 1110-2-1619, it was determined that the minimum standard deviation error in stage should be 0.6-ft. The calculated standard deviation is less than this minimum in three instances: Thomas Creek, Cole Creek, and Saddle Creek. For the remainder of this analysis, 0.61' will be used on Cole Creek while 0.43' and 0.03' will be used on Thomas Creek and Saddle Creek, respectively. Although using a lower standard deviation underestimates damages, potentially lowering the calculated benefits of a project, no structural alternatives were evaluated on these reaches. No structural alternatives exist on Saddle Creek, and dam site 10 was justified based on impacts to the Little Papillion alone. Therefore, no alternative was determined unjustified due to the economic results of these reaches.

Additional information regarding the alternatives analysis can be found in the next section, Section 2.4.

During the alternatives analysis, described later in this report, levee and floodwall alternatives will be modeled using risk and uncertainty guidance found in ER 1105-2-101. To expedite alternative development over a large area of interest, a simplifying assumption that 3-ft for risk and uncertainty will be used during the initial screening prior to TSP. Based on the above sensitivity analysis, it can be reasonably assumed that 3-ft for risk and uncertainty will provide at least a 90% assurance of containing the design flood. After TSP, the remaining project alternatives will be refined to fully incorporate risk in all channel, levee, and floodwall projects.

2.4 Alternatives Analysis

Figure 7 shows the general overview of all the alternatives considered.

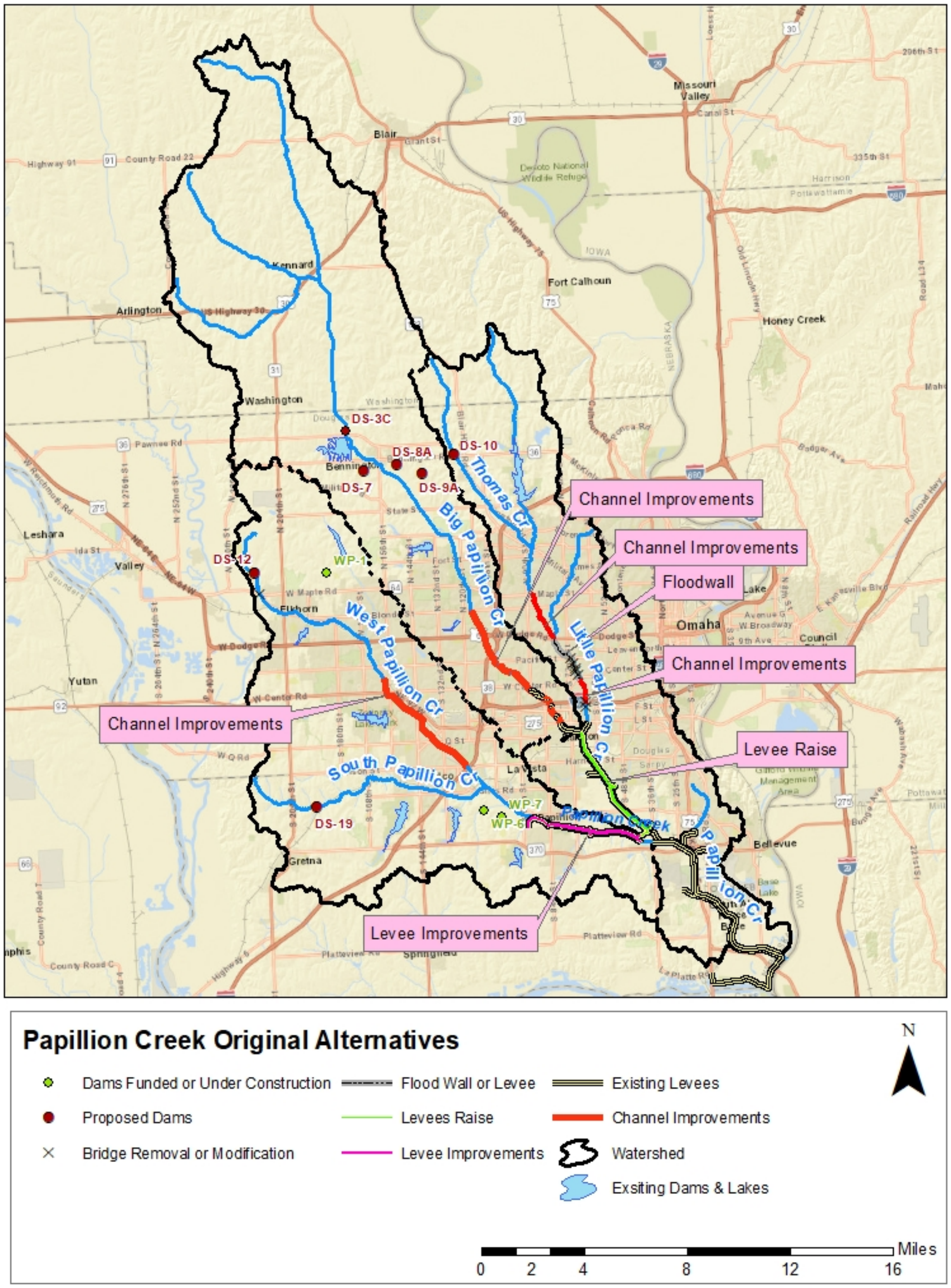


Figure 7. Overview of Original Alternatives Considered

2.4.1 Future without Project

The future without project assumes the same channel geometry, i.e. no further encroachment or river crossings were anticipated, and the same existing conditions geometry file was used during the future without project alternative modeling. Flow files were updated to account for full build-out conditions of the watershed and downstream boundary conditions were revised accordingly. See Hydrology Appendix A for the detailed hydrology analysis regarding future without project flows. The flows utilized in the hydraulic analysis for the future without project alternative are provided in Appendix B-P6. Updated Boundary Conditions are provided in Table 8. Inundation maps are provided in Appendix B-P15.

Table 8. Boundary Conditions – Future Without Project

Annual Chance Exceedance	River					
	West Papillion Creek	Little Papillion Creek	South Papillion Creek	Thomas Creek	Cole Creek	Saddle Creek
50%	982.39’	995.86’	1019.41’	1070.74’	1027.88’	1008.35’
20%	986.79’	999.65’	1023.61’	1073.76’	1031.55’	1012.25’
10%	989.43’	1001.87’	1025.96’	1075.74’	1033.92’	1014.91’
4%	992.22’	1005.09’	1028.79’	1077.89’	1037.18’	1018.12’
2%	993.59’	1007.52’	1030.46’	1079.54’	1041.97’	1020.66’
1%	994.56’	1010.21’	1032.01’	1081.19’	1042.11’	1023.16’
0.5%	995.4’	1012.22’	1034.68’	1081.96’	1043.08’	1025.17’
0.2%	996.43’	1013.65’	1035.66’	1085.37’	1045.71’	1026.75’
Location	Big Papillion Confluence BP XS 43397	Big Papillion Confluence BP XS 71656	West Papillion Confluence WP XS 39898.36	Little Papillion Confluence LP XS 43674	Little Papillion Confluence LP XS 22999	Little Papillion Confluence LP XS 10832

2.4.2 Preliminary Screened Alternatives

The intent of this section is to give a summary of previously considered alternatives and the reason they were not considered further. The following measures were screened out without a detailed economic evaluation based upon evident disproportionate costs to benefits and environmental and/or social concerns. Figure 8 displays the alternatives screened during the initial analysis.

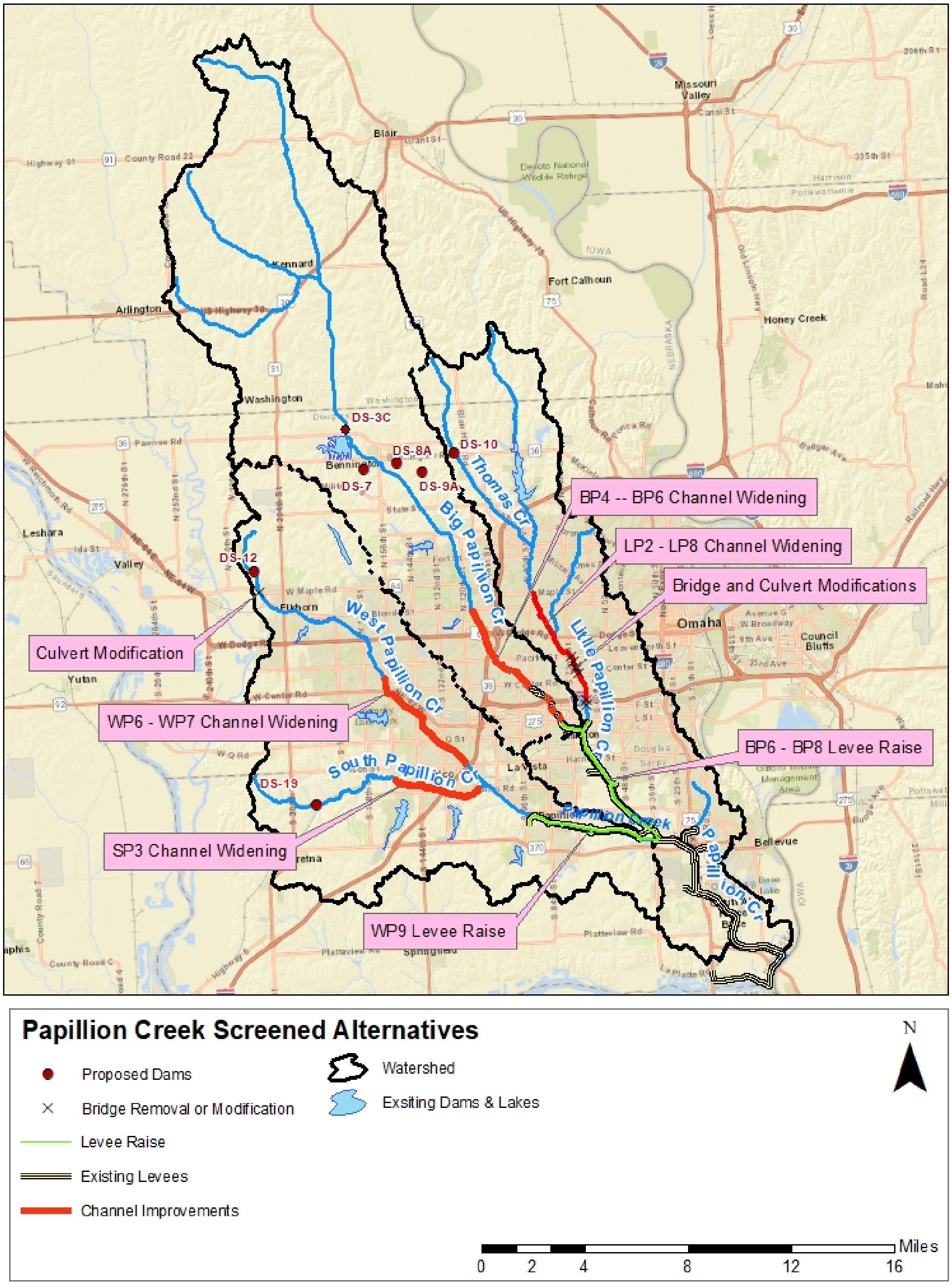


Figure 8. Overview of Screened Alternatives

2.4.2.1 Reservoirs

The evaluation of several reservoirs was requested by the local sponsor. These included dam sites: 7, 8A, 9A, 10, 12, and 19. Dam Site 3C near the border of Douglas and Washington counties was also evaluated. Additionally, an alternative that considered dam sites 7, 8A, 9A, 10, 12, and 19 in conjunction was also analyzed.

To quickly determine which reservoirs provided enough justification to carry forward into a more detailed feasibility study, reservoirs were first assumed to capture and hold all runoff for the eight events modeled. New flow files were created with this change in flow and the resulting boundary conditions on each river for each of the proposed reservoir alternatives. Study discharges for dam sites 3C, 7, 8A, 9A, 12, and a combination of 7, 8A, 9A, 10, 12, and 19 are provided in Appendices B-P7 through B-P12, respectively. These flows do not represent full build-out conditions. Updated downstream boundary conditions for all reservoir alternatives are provided in Appendix B-P13. Channel geometry was assumed to remain unchanged with no further encroachments or river crossings.

Water surface grids and inundation boundaries were created using HEC-RAS RAS Mapper and processed in ArcMap before being provided to the project economist for analysis in HEC-FDA. Calculated benefits were then compared to projected construction costs from previous studies to determine if further analysis was warranted.

All reservoir locations, apart from 10 and 19, were ruled out early in the analysis. For the screened reservoir alternatives, plates showing the resulting inundation boundaries are provided in Appendix B-P14. Inundation created behind dam locations are a result of the hydraulic model and do not accurately depict pool boundaries for the screened alternatives.

Dam Site 3C: Dam Site 3C is located on Big Papillion Creek about 0.2 miles north of Pawnee Road between 168th and 180th Street. Noticeable impacts from the addition of the dam extend downstream to approximately L Street. Despite the reduction in inundation upstream of L Street, the reduction in flow from the dam did not take much out of the resulting peak flow at the Little Papillion Creek/Big Papillion Creek confluence.

Dam Site 7: Dam Site 7 is located on a tributary to Big Papillion Creek about 0.4 miles south of Bennington Road between 156th and 168th Street. Slight reductions in inundation are visible downstream of the dam until Fort Street. The observed benefits are largely agricultural.

Dam Site 8A: Dam Site 8A is located on a tributary to Big Papillion Creek about 0.2 miles south of Bennington Road between 138th and 156th Street. Like Dam

Site 7, slight reductions in inundation are visible downstream of the dam until Fort Street. The observed benefits are largely agricultural.

Dam 9A: Dam Site 9A is located on a tributary to Big Papillion Creek about 0.1 miles east of 138th Street between Rainwood Road and Bennington Road. Like Dam Sites 7 and 8A, benefits are visible downstream of the dam until Fort Street and are largely agricultural.

Dam Site 12: Dam Site 12 is located on the West Papillion, just southwest of the N 216th Street and Fort Street intersection. Noticeable impacts from the addition of the dam extend to approximately 144th Street. However, the benefits are primarily contained to a few residences, commercial properties, agriculture, and open space. The impacts were not great enough to affect any other creeks of interest in this study.

Combination of Dam Sites 7, 8A, 9A, 10, 12, and 19: This alternative looked to collectively analyze reduction in inundation due to all of the sponsor requested dam sites working in conjunction without the effects of Dam Site 3C. Benefits are similar to how they are described above with additional slight reduction in inundation throughout the Papillion Creek system.

2.4.2.2 Channel Widening

Several channel widening alternatives were considered in the early part of the analysis. Alternatives were evaluated assuming a bench built between the 99.9% and 50% AEP water surface elevations. This bench elevation was selected based on past channel widening projects in the Papillion Creek watershed. Setting the bench elevation too low will result in sedimentation and a decrease in hydraulic capacity. Currently no study has been conducted to evaluate the degree of sedimentation on the existing channel widening projects. Further monitoring and evaluation should be done on existing channel widening projects to determine if the 99.9% to 50% AEP bench elevation is still an adequate design assumption. The width of the bench was designed to accommodate the 2% AEP within the channel, assuming no sedimentation, and tie back into existing grade with a 3(H):1(V) slope. See Figure 9 for a visual representation of the channel widening cross-section. Alternatives to optimize the channel alignment to avoid infrastructure impacts and minimize real estate needs were not pursued at this time. Furthermore, existing basin condition peak flows were utilized during this analysis and were not updated to capture potential increased flows downstream of the modified area that would result from the additional channel conveyance provided; however, this was partially mitigated by not using any attenuated flow values. The same downstream boundary conditions that were used for existing conditions were also used in the initial channel widening alternative analysis.

Water surface grids and inundation boundaries were created using HEC-RAS RASMapper and processed in ArcMap before being provided to the project economist for analysis in HEC-FDA. Calculated benefits were then compared to construction costs for the required excavation to determine if more analysis was needed.

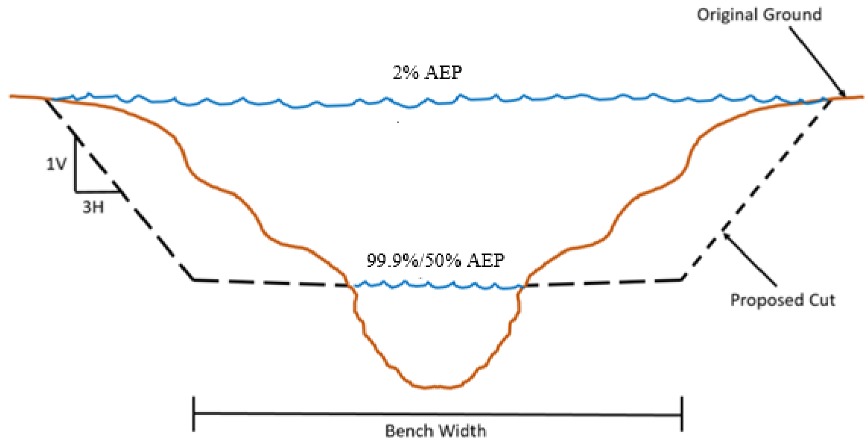


Figure 9. Channel Widening Cross-Section

For the screened channel widening alternatives, the resulting inundation boundaries are provided in Appendix B-P16.

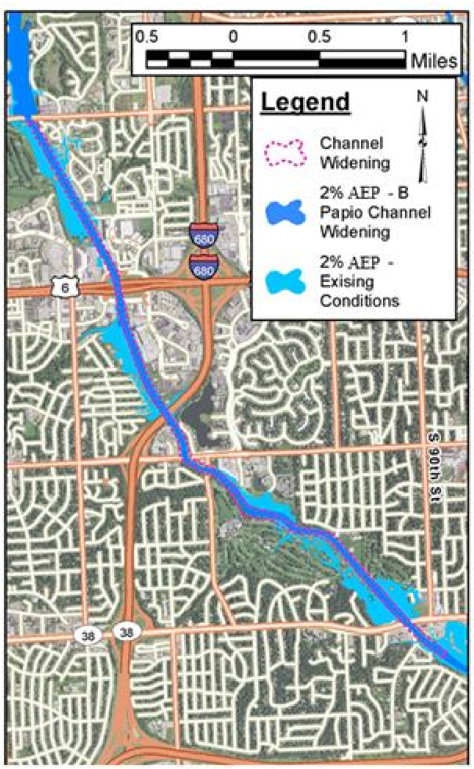


Figure 10 Screened Channel Widening on the Big Papillion Creek

Big Papillion, Damage Reaches BP4 through BP6: The impact of this alternative spans roughly six miles from Blondo Street to L Street. Channel widening would take place from Blondo Street to West Center Road with an approximate length of four miles, as shown in **Error! Reference source not found.**. The bench width is set to 120-ft at the 99% AEP water surface elevation and low berms would be required in locations of localized low overbank elevations. These berms would be less than three feet high. The total excavation volume required for this alternative is approximately 625,000 cubic yards. In addition to the excavation, the 105th Street Bridge would need to be widened with a similar bench width as this bridge currently serves as a choke point for flow. Downstream of West Center Road there exists six segments of non-federal levees. Due to this, it was assumed that in this two-mile stretch, these levees would be raised to contain the 2% AEP event. At most, the levees/existing ground would need to be elevated by three feet. The levee raises proposed for this alternative would need to tie into West Center Road and L Street.

West Papillion, Damage Reaches WP6 and WP7: The roughly 3.5 mile long channel widening alternative on the West Papillion Creek begins at the railroad crossing just north of Giles and extends upstream to approximately Hillsdale Drive, just southeast of the 144th Street crossing, see Figure *11*, and is intended to minimize damages in WP6. As such, containing the 2% AEP within the channel banks downstream of 132nd Street, where appropriate connected floodplain is present, was a low priority. Instead, the section of channel widening between the railroad bridge and 132nd Street was widened just enough to contribute to lower stages upstream. The alternative utilized a 150-ft channel bench located at the 99% AEP water surface elevation. This resulted in nearly 1.4 million cubic yards of excavation. In addition to the excavation requirements, bridge modifications at 132nd Street, Harrison Street, I-80, and the downstream railroad crossing would be needed to realize the full benefits of the alternative.

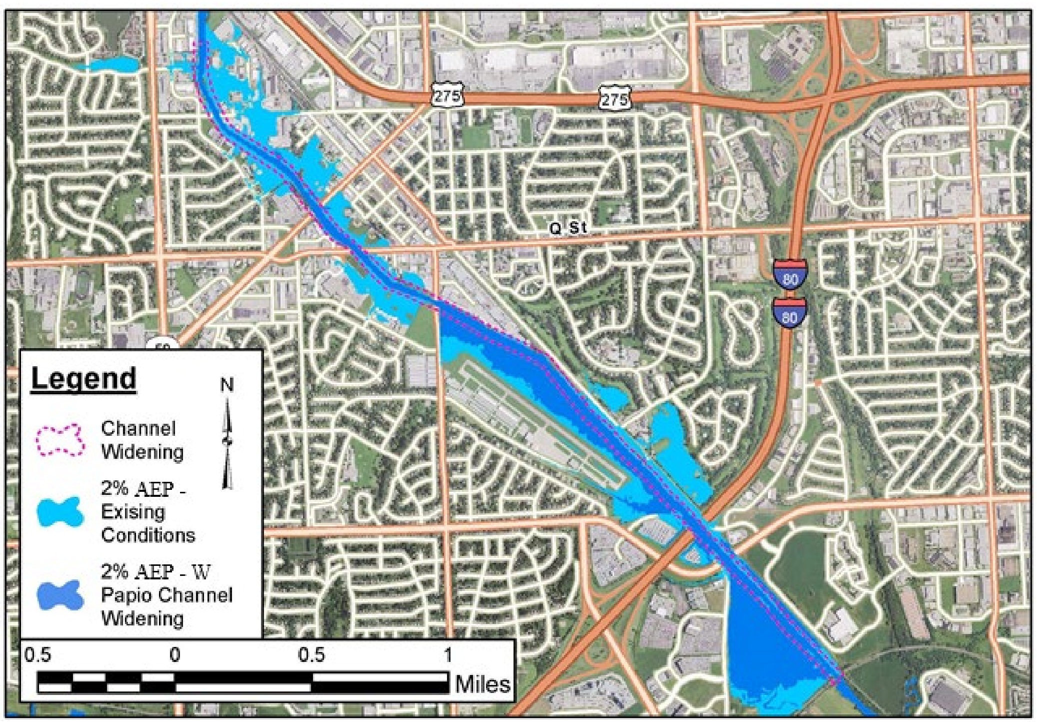


Figure 11. Screened Channel Widening on the West Papillion Creek

Little Papillion, Damage Reaches LP2 through LP8: Several channel widening projects have been completed in these damage reaches on the Little Papillion in the past. Therefore, the hydraulic analysis focused on sections of the Little Papillion where the 2% AEP event was not already contained within the channel

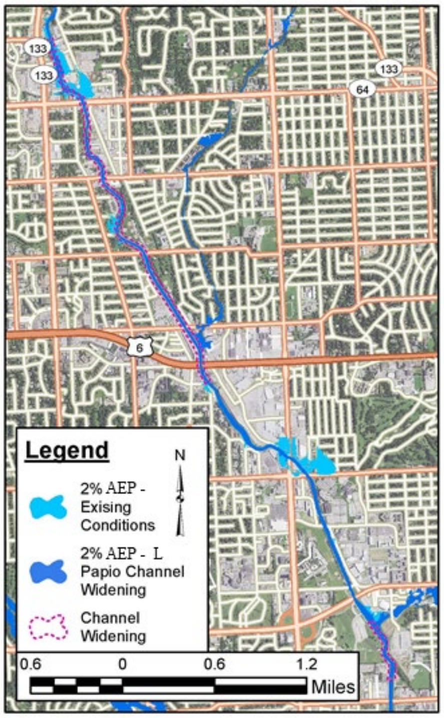


Figure 12. Screened Channel Widening on the Little Papillion Creek

banks. This included a half-mile section of channel between Grover Street and West Center Road, a 2.6-mile section of channel from Dodge Street to Maplewood Blvd, and a short section between Pacific and 72nd streets, see Figure 12 below. Channel widening between Grover Street and West Center Road and from Dodge Street to Maplewood Blvd resulted in a total excavation of approximately 600,000 cubic yards. Both sections utilized a 100-ft bench located at the 99% AEP water surface elevation. Although channel widening was shown to have minimal effect between the 72nd and Pacific Street bridges, modifying the Pacific Street Bridge contained the 2% AEP event within the channel cross-section. Additional information regarding bridge modification modeling is provided in Section 2.4.2.4.

South Papillion, Damage Reach SP3: The 3.5 mile long channel widening alternative on South Papillion Creek begins near the confluence with West Papillion Creek and extends upstream to the railroad crossing between 144th and 156th Street, see Figure 13 below. This alternative assumes a 50-ft channel bench located at the 99% AEP water surface elevation. Assuming this configuration, the total excavation volume is approximately 343,200 cubic yards with no bridge modifications required.

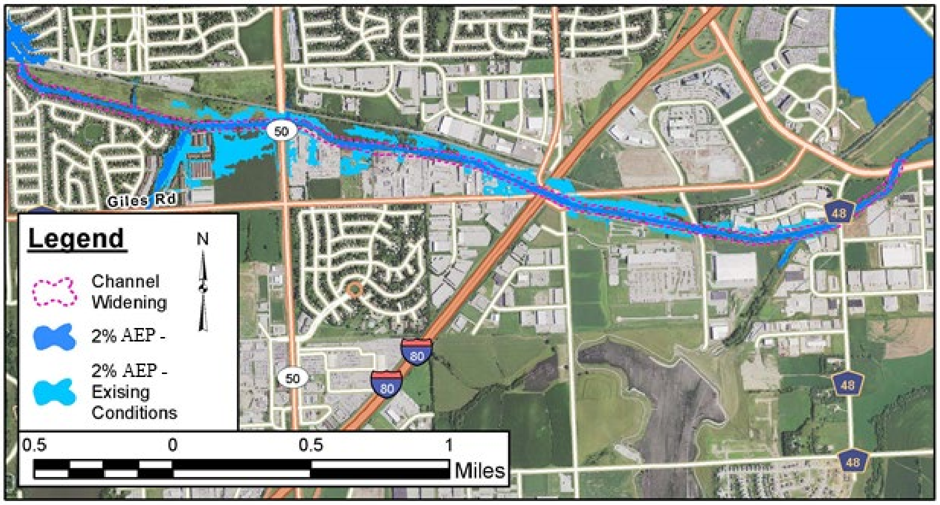


Figure 13. Screened Channel Widening on the South Papillion Creek

2.4.2.3 Levee/Floodwall Alternatives

Levee alternatives were also considered as part of this analysis. Levee alignments attempted to follow pre-existing levees and/or bike trails where possible. Where this was not possible, proposed levee alignments followed the floodway.

All levee/floodwall alternatives were designed to the 1 or 0.2% AEP water surface elevation with an additional 3-ft to account for risk and uncertainty. Because steady state 1D hydraulic modeling was utilized and mapping was being done in HEC-RAS RAS Mapper, levee points were placed at an elevation relative to an overtopping event for each alternative. To maintain a realistic computed water surface elevation, ineffective flow areas were placed at that same location with an elevation equal to that of the respective water surface elevation plus the additional 3-ft to remove conveyance from behind the levees up to the design elevation. Existing condition peak flows were used during this evaluation. Downstream boundary conditions are provided in Appendix B-P17.

No other work was considered in conjunction with levee, floodwall, and levee raise alternatives. It is assumed that work necessary to account for induced stages will be nonstructural in nature or evaluated in the next phase. Resulting induced damages were included in the calculation of the Benefit Cost Ratio (BCR). More information on this topic can be found in the Economics Appendix F.

Water surface grids and inundation boundaries were created using HEC-RAS RAS Mapper and processed in ArcMap before being provided to the project economist for analysis in HEC-FDA. Calculated benefits were then compared to estimated construction costs of the proposed levee. For the screened levee/floodwall alternatives, the resulting inundation boundaries are provided in Appendix B-P18.

Big Papillion, levee raise to the 0.2% AEP, Damage Reaches BP6 through BP8: Non-Federal levees currently exist on Big Papillion Creek (and Little Papillion Creek near the Big/Little confluence) and extend downstream until they tie into the Federal R-613 system. The non-federal levees currently protect to an estimated event between the 4 and 2% AEP. This alternative assumes that the non-federal levees, including the tie-back on the Little Papillion, are raised to the 0.2% AEP water surface elevation with an additional 3-ft for risk and uncertainty, to assess economic benefit. The total length of this alternative would be around 6.75 miles and span from L Street to the Big Papillion/West Papillion Creek confluence. On average, this alternative raises the existing levee system by 10.5-ft including the additional 3-ft. The tie-back on the Little Papillion is approximately 0.4 miles and raises the existing levee approximately 10-ft. For this alternative, the levee raise is tied into high ground at the L Street embankment. In general, induced stages are produced from L Street to West Center Road and take place in residential, industrial, and commercial areas.

Big Papillion, levee raise to the 1% AEP, Damage Reaches BP6 through BP8: This alternative is similar to the Big Papillion Levee Raise to the 0.2% AEP alternative, but instead is only considering a raise to the 1% AEP with an additional 3-ft to account for risk and uncertainty. This results in an average raise of 5-ft, which includes the additional 3-ft, on the Big Papillion. For this alternative, the levee raise is tied into high ground at the L Street embankment. Additionally, the tie back on the Little Papillion will also be raised by approximately 1-ft. Using this assumption, overtopping occurs between the 0.5 and 0.2% AEP events. Like the raise to the 0.2% AEP alternative, induced stages exist between L Street and West Center Road and takes place in residential, industrial, and commercial areas.

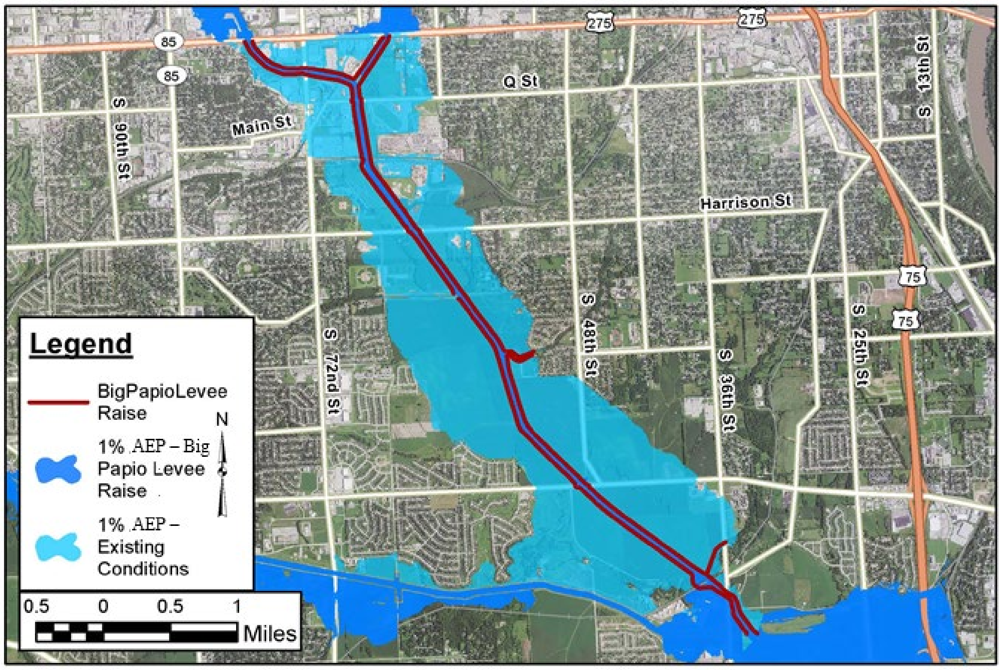


Figure 14. Screened Levee Raise on Big Papillion Creek

West Papillion, levee raise to the 0.2% AEP, Damage Reach WP9: Non-federal levees currently exist on the West Papillion starting at the confluence with the Big Papillion and extending upstream past the 84th Street Bridge. Most of this levee system is included in the PL84-99 program and protects to nearly the 0.5% AEP with minimal overtopping in various locations at the 1% and 0.5% AEP events. The sections from 42nd Street to the confluence with Big Papillion on the south bank and from 48th Street to the confluence on the north bank provide protection for less than the 10% AEP and therefore do not meet the requirements for minimal protection required to be included in the PL 84-99 program. Because so much of this levee system protects to nearly the 0.5% AEP it was decided to raise the levees to the 0.2% AEP water surface elevation with an additional 3-ft for risk and uncertainty to assess economic benefit. On average, this alternative raises the existing levee system 6.5-ft, however, the greatest increases in height are present near the confluence in the non-accredited sections of levee. In this area, the average increase in height is 11-ft, while the average raise in levee height in the accredited sections is 4.5-ft.

Because the existing levees offer such a high level of protection, most benefits are not realized until the 0.2% AEP. Induced stages are produced upstream of the levees; however, they are mostly confined to open space and agricultural areas.

West Papillion, levee raise to the 1% AEP, Damage Reach WP9: Being similar to the existing condition, raising the existing levee to the 1% AEP water surface elevation with 3-ft to account for risk and uncertainty provides protection to nearly the 0.5% AEP event. The average raise in levee height in the PL84-99 sections is 1-ft while the average raise in levee height in the non-compliant sections is 10-ft.

Damage reaches were broken out at the beginning of the study and do not fully capture the differences in levee height in the PL 84-99 sections versus the height in the non-compliant section. This shortcoming may prevent the project team from fully capturing the benefits of improving the non-compliant section of levee. In this area there are several commercial properties and apartment buildings that are shown to be inundated at the 1% and 0.5% AEP. However, no investigation into whether these properties were built out of the floodplain or not was undertaken by the hydraulics team at the time of this report.

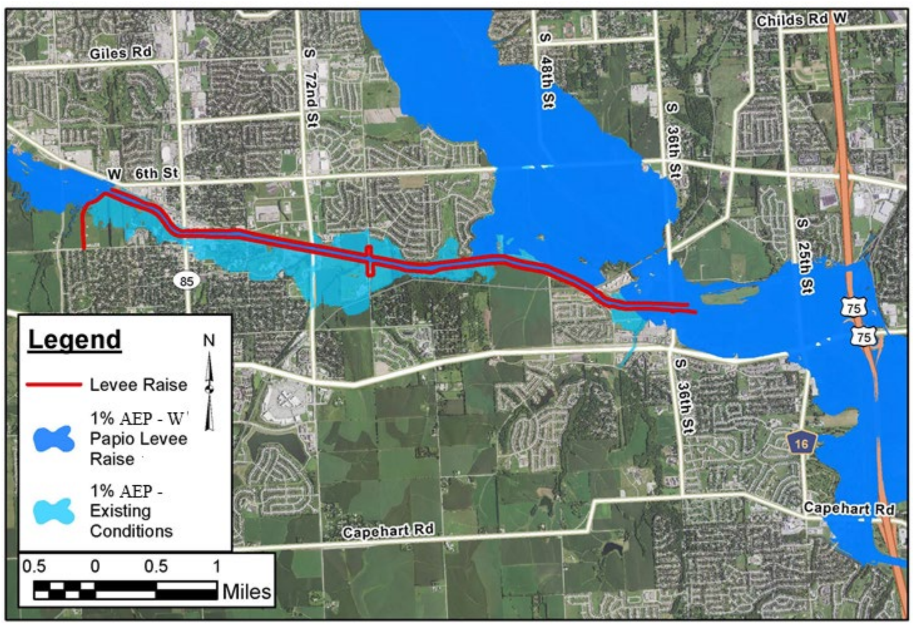


Figure 15. Screened Levee Raise on West Papillion Creek

2.4.2.4 Bridge and Culvert Modifications

Bridge and culvert modifications were considered at several locations where flow appeared to be impeded. In general, this analysis was done assuming no combination with other alternatives. Current existing basin condition peak flow was modeled in each case with current existing downstream boundary conditions.

For these bridge and culvert modification alternatives, plates showing the resulting inundation boundaries are provided in Appendix B-P19.

West Papillion, W Maple Road Culvert Modification, Damage Reach WP1: The existing culvert on the West Papillion at W Maple Road is a quad-8'(R) x

7'(S) Reinforced Concrete Box Culvert (RCBC) with flared wing walls. In the current condition, there is enough capacity for the 50% AEP, all other events pass over the roadway. To model the culvert modification, the culvert was fully removed from the channel. A bridge deck was added with a minimum low chord elevation 3-ft higher than the 1% AEP water surface elevation and 3 piers. At this elevation, the bridge has capacity for all modeled events. The bridge was modeled under subcritical and mixed flow routines. Regardless of the modifications to the crossing, the flow routine used, and changes to the cross-section, any impacts of this alternative were very localized, spanning less than 1,000-ft. Although raising the crossing to convey all events under the bridge deck would provide benefits to the roadway, very little benefits would be realized in the surrounding areas.

Little Papillion, Various Bridge Removals and Modifications: Seven bridge modifications and removals were analyzed on the Little Papillion. To quickly evaluate if these measures were feasible, bridges and their corresponding ineffective flow areas were removed in their entirety from the model geometry. If enough benefits could be realized, effort would be made to define specific design details for the modification

1. <u>UPRR Crossing, Damage Reach LP8</u> – The Union Pacific railroad crossing is located just downstream from the I-80 Bridge. It is a double track that is elevated significantly higher than all events modeled; the minimum low chord is approximately 70-ft higher than the 0.2% AEP water surface elevation. A previous modification at this bridge constructed a concrete flume beneath the crossing that widened the cross-section and provided increased conveyance through the section. Because this modification has already been done, it is assumed that any additional modifications in this location would need to be a full bridge removal and replacement.

   Removing the bridge from the project geometry in its entirety provided some minimal impacts at the 2% AEP at the Baxter Arena parking lot. Likewise, benefits were visible for the 1% AEP event as well, however they were mostly confined to parking areas. The 0.5% and 0.2% AEP provided additional benefits that included some residential, but the benefits were confined to downstream of West Center Road, in the case of the 0.5% AEP, and downstream of Grover Street, in the case of the 0.2% AEP.

2. <u>I-80 Crossing, Damage Reach LP8</u> – The I-80 crossing is a ten-lane bridge with 3 piers. It is elevated significantly higher than the hydraulic events modeled; the minimum low chord elevation is nearly

20-ft above the 0.2% AEP. Resulting benefits are shown for only the highest events modeled.

3. <u>Pedestrian Crossing, upstream of Mercy Road, Damage Reach LP7</u> – The crossing just upstream of Mercy Road is a single span pedestrian bridge that conveys up to the 2% AEP. Removing the bridge in its entirety does appear to provide benefits as far upstream as 72nd Street for the 1% AEP, however, very little impact can be seen in any other event.

4. <u>Pedestrian Crossing, west of the College of St Mary's campus, Damage Reach LP7</u> – The crossing west of the College of St. Mary's campus is a single span pedestrian bridge that conveys up to the 2% AEP. Removing the bridge in its entirety appears to provide little to no benefit.

5. <u>Pedestrian Crossing, downstream of Pine Street, Damage Reach LP7</u> – The crossing just downstream from Pine Street is a single span pedestrian bridge. It provides capacity up to the 2% AEP in its current geometry. Removing the bridge in its entirety provides some benefits in the direct vicinity of the bridge for the 1% and 0.5% AEP, however, these impacts appear localized and negligible.

   Since completing the hydraulic analysis, the city has removed this pedestrian bridge. This bridge was not removed in the hydraulic modeling due to time constraints. Since benefits were assumed negligible, this should have little to no impact on the hydraulic benefits.

6. <u>Pedestrian Crossing, downstream of Pacific Street, Damage Reach LP7</u> – The pedestrian bridge downstream of Pacific Street is a single span bridge and provides capacity for the 50% - 1% AEP. Removing the bridge and its corresponding ineffective flow areas had a minimal impact on water surface elevations in the area.

7. <u>Pacific Street Bridge, Damage Reach LP6</u> – The Pacific Street crossing is a four-lane bridge with 2 piers. It provides capacity for the 50% through 4% AEP in its current condition and has a backwater effect on the upstream water surface elevation starting at the 2% AEP. Removing the bridge and ineffective flow areas from the geometry provided benefits that extended slightly upstream of 72nd Street for the 2% and 1% AEP. The impacts appeared negligible for all other events. The mapping was provided as part of the Little Papillion channel widening alternative.

2.4.3 Evaluated Alternatives

The alternatives in the following section, shown in Figure 16, went through the same screening process as those outlined in the previous sections and were found to justify more detailed economic analysis. The sections below describe this additional analysis.

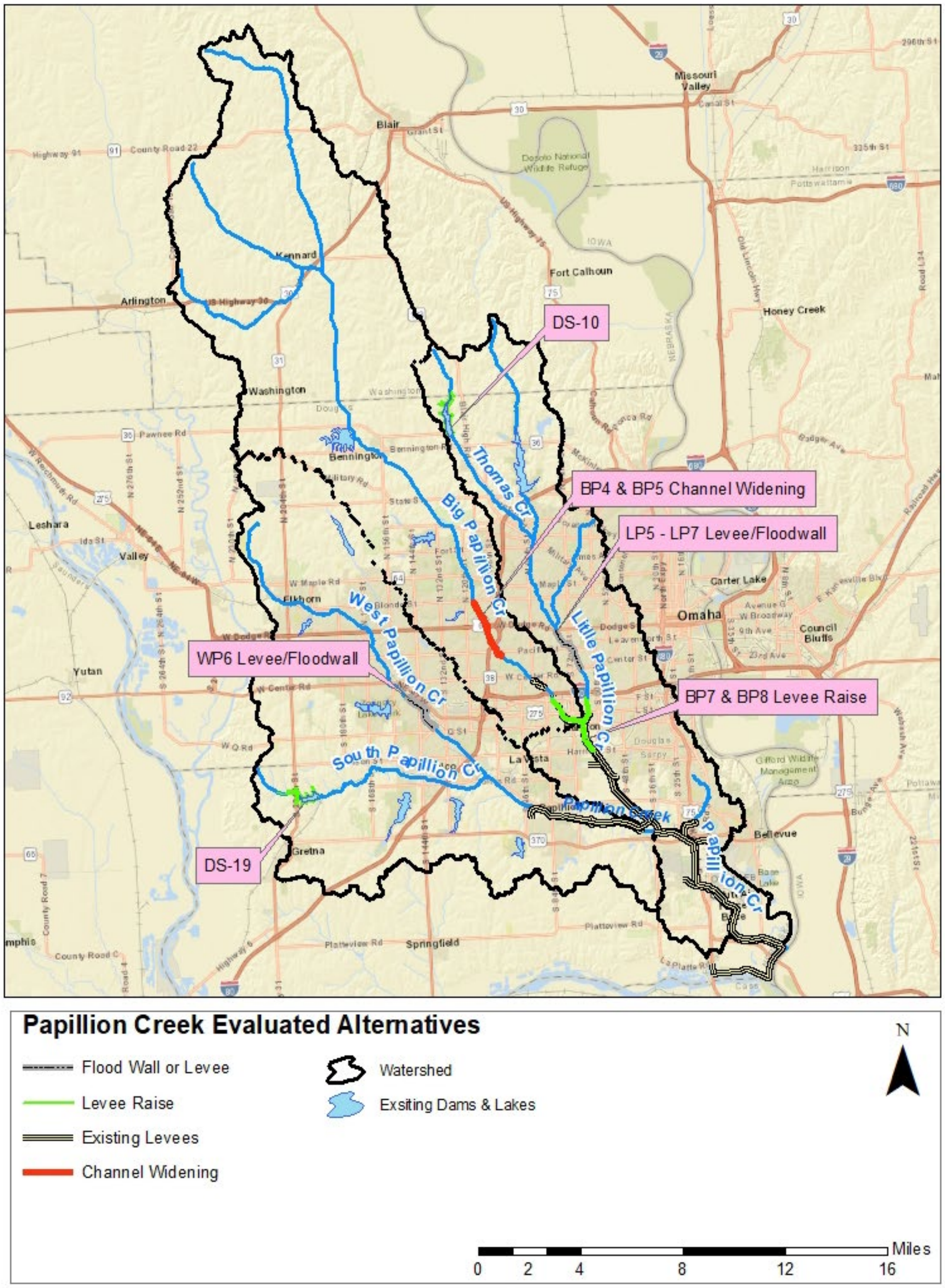


Figure 16. Overview of Evaluated Measures

2.4.3.1 Reservoirs Alternatives

As was stated previously, during steady flow analysis reservoir alternatives were assumed to capture and hold all runoff in its upstream watershed for the eight events modeled under peak flow conditions. New flow files were created with this change in flow and the resulting boundary conditions on each river for each of the proposed reservoir alternatives. Updated study discharges from the steady flow modeling are provided in Appendices B-P20 and B-P21, respectively. Updated downstream boundary conditions are provided in Appendix B-P13. Channel geometry was assumed to remain unchanged with no further encroachments or river crossings.

As before, water surface grids and inundation boundaries were created using HEC-RAS RAS Mapper and processed in ArcGIS before being provided to the project economist for analysis in HEC-FDA. Project designs and real estate costs were modified to comply with ER 1110-8-2 (FR) *Inflow Design Floods for Dams and Reservoirs (USACE, 1991).* Calculated benefits were then compared to projected construction costs. See Hydrology, Economics, and Cost Appendices (Appendices A, F, and E, respectively) for more detailed information.

Resulting inundation maps are provided in Appendix B-P22 for the steady flow analysis. Inundation created behind dam locations are a result of the hydraulic model and do not accurately depict pool boundaries. For images of the pool extents, refer to the Real Estate Appendix J.

Dam Site 10: Dam Site 10 is located on Thomas Creek, just northwest of the Bennington Road and Blair High Road intersection. Noticeable benefits extend as far downstream as West Center Road on the Little Papillion. The benefits range from primarily agricultural on Thomas Creek to primarily commercial, industrial, and residential properties on the Little Papillion.

Dam Site 19: Dam Site 19 is located on South Papillion Creek between 192nd Street and 204th Street. Noticeable benefits extend downstream of the dam until the South Papillion/West Papillion Creek confluence. The benefit areas are residential, industrial, and commercial.

2.4.3.2 Channel Widening Alternatives

The evaluated channel widening alternative was modeled in a similar fashion to the screened channel widening alternatives, however the extents were refined to focus on a specific damage area to minimize initial costs. The evaluated channel widening alternative similarly uses existing basin condition peak flows and downstream boundary conditions.

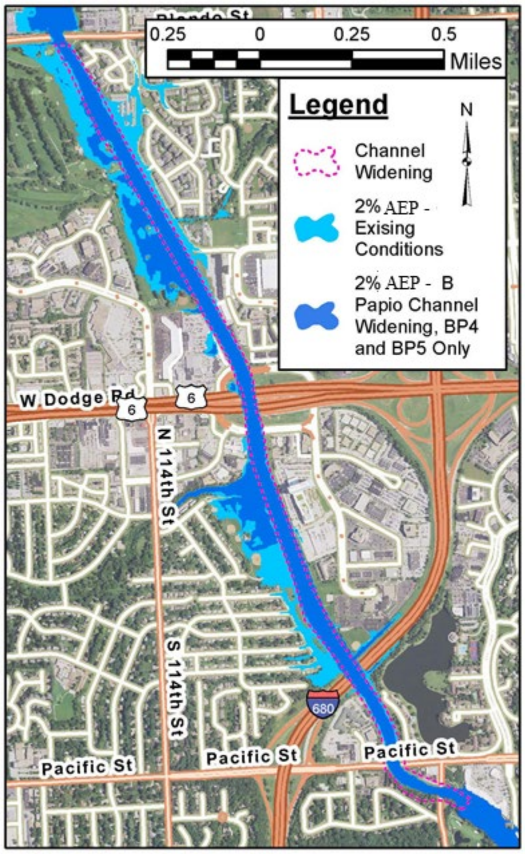


Figure 17. Evaluated Channel Widening in BP4 and BP5

Big Papillion Creek, Damage Reaches BP4 and BP5: Channel widening projects have been completed on Big Papillion Creek in the past, so this analysis is focused upstream of the existing projects where the cost/benefits appeared to be most justifiable. This proposed widening alternative takes place beginning slightly downstream of 105th Street and extending upstream until Blondo Street, see Figure 17. The proposed bench width was determined to maximize benefits while minimizing the amount of bridges that would need to be modified. The proposed bench width is 120-ft between the 99 and 50% AEP water surface elevations. This bench elevation was selected based on past channel widening projects in the Papillion Creek watershed. Setting the bench elevation too low will result in sedimentation and a decrease in hydraulic capacity. Currently no study has been conducted to evaluate the degree of sedimentation on the existing channel widening projects. Further monitoring and evaluation should be done on existing channel widening projects to determine if the 99.9% to 50% AEP bench elevation is still an adequate design assumption. Total excavation volume would be approximately 201,250 cubic yards. For this alternative to be effective, the 105th Street Bridge would need to be widened with an approximate 120-ft bench width between the 99 and 50% AEP water surface elevations.

2.4.3.3 Levee/Floodwall Alternatives

As with the screened levee/floodwall alternatives, levee and floodwall alignments attempted to follow pre-existing levees and/or bike trails where possible. Where this was not possible, proposed levee alignments followed the floodway.

All levee/floodwall alternatives again incorporated an additional 3-ft to account for risk and uncertainty. Because steady state 1D hydraulic modeling was utilized and mapping was being done in HEC-RAS RAS Mapper, levee points were placed at an elevation relative to an overtopping event for each alternative. To maintain a realistic computed water surface elevation, ineffective flow areas were placed at the levee top elevation in the same location as the levee points to remove conveyance from behind the levees up to the design elevation. Existing condition peak flows were utilized during this evaluation as well.

No other work was considered in conjunction with levee, floodwall, and levee raise alternatives. It is assumed that work necessary to account for induced stages will be nonstructural in nature or evaluated in the next phase. Again, the induced damages were included in the calculation of the BCR. More information is provided in the Economics Appendix F.

Water surface grids and inundation boundaries were created using HEC-RAS RAS Mapper and processed in ArcGIS before being provided to the project economist for analysis in HEC-FDA. Calculated benefits were then compared to construction costs which included the cost of building the levee, real estate, and closure structures. Closure structure requirements are summarized in Appendix B-P25.

Inundation maps are provided in Appendix B-P24 for the resulting analysis.

Figure 18. Evaluated Levee Raise on Big Papillion Creek in BP7 and BP8

Big Papillion Creek Levee Raise, Damage Reaches BP7 and B8: This alternative is a proposed levee raise beginning at Harrison Street and extending upstream to the railroad embankment just south of Interstate 80. On the Little Papillion Creek, the levee raise also extends upstream of the confluence to the railroad embankment. The total length of this alternative is 4.5 miles, see Figure 18. The height of raise was set based on the 1% AEP water surface elevation with an additional 3-ft of height to account for risk and uncertainty. On average, the levee is raised by 4.2-ft. Using this raise configuration, this levee alternative overtops approximately at the 0.5% AEP. Closure structures would be required at the railroad crossing between Harrison Street and Q Street on the Big Papillion, and on the pedestrian bridge just upstream of the confluence and L Street on the Little Papillion. Both the Big Papillion closure structure and the pedestrian bridge closure structure are within the additional 3-ft and can therefore be assumed to be HESCO barriers. The closure structures on the Little Papillion are outside the additional 3-ft. This alternative ties into the currently existing high ground of the railroad and Harrison Street embankments. It would be possible to add parallel levees to these embankments, but at an additional cost. For this analysis, adding levees parallel to the road and railroad embankments were not considered.

Benefits are realized as early as the 2% AEP event. Induced stages due to this alternative exist primarily from the upstream railroad embankment to West Center Road. Induced stages are also seen through the whole alternative reach at the 0.5 and 0.2% AEP.

West Papillion Creek Floodwall, Damage Reach WP6: The floodwall alternative evaluated through Damage Reach 6 begins at Millard Avenue and extends upstream to Boxelder Creek, see Figure 19 below. This alternative includes 3.5-miles of floodwall and is designed to the 1% AEP water surface elevation with an additional 3-ft to account for risk and uncertainty. At this height, the floodwall also contains the 0.5% AEP, and is overtopped by the 0.2% AEP. The alternative requires three closure structures; however, these closure structures are located within the additional 3-ft for risk and uncertainty zone. Therefore, it is assumed that HESCOS will suffice in these areas.

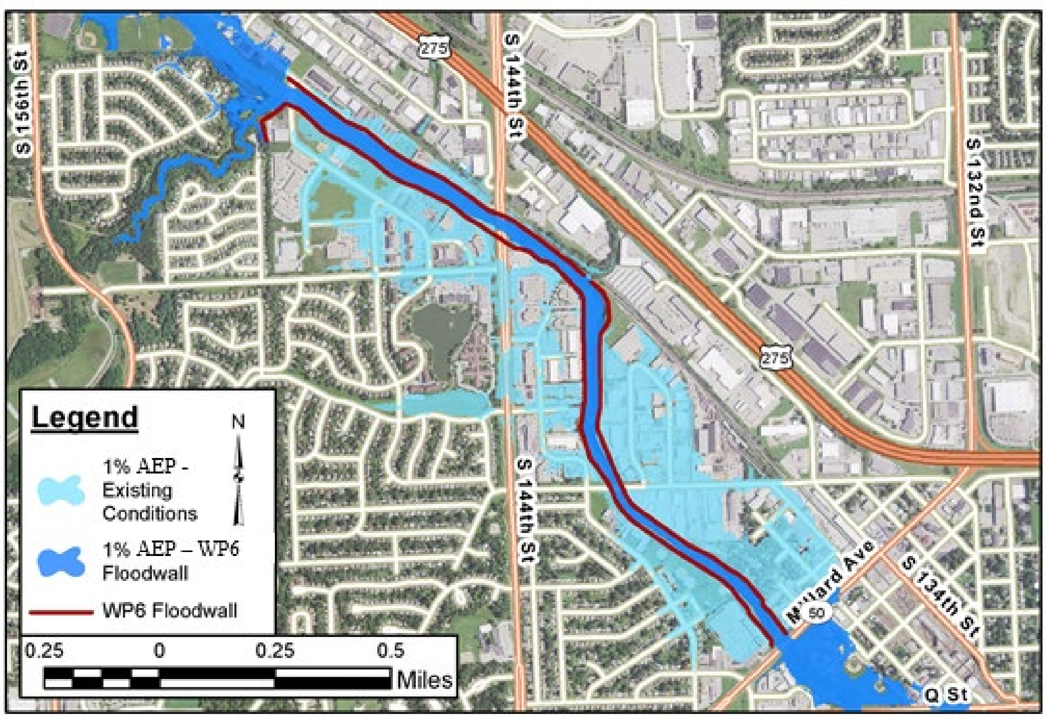


Figure 19. Evaluated Levee on West Papillion Creek in Damage Reach WP6

Benefits are realized as early as the 4% AEP.

Because the floodwall alternative does not tie into high ground on the downstream end, at Millard Avenue, it acts as a trailing levee. During the 1% and 0.5% AEP, backwater on the left bank may overtop Millard Avenue, causing damages not captured by the hydraulic and economic analysis.

Little Papillion Creek Levee/Floodwall to the 0.2% AEP, Damage Reaches LP5 through LP7: This levee/floodwall alternative extended from the Saddle Creek/Little Papillion Confluence upstream to the Cole Creek/Little Papillion confluence. This is just downstream of Mercy Road to approximately Cass Street. In the existing geometry, levee points were added at an elevation equal to the 0.2% AEP water surface elevation plus an additional 3-ft.

On average, the height of the new levee or floodwall is approximately 11.5-ft with a maximum height of around 15-ft. In addition to constructing a levee or floodwall to the specified height in this area, closure structures would be needed at twelve bridges. The minimum height needed for these closures structures is 3.27-ft for a span of 59-ft at Mercy Road. The maximum height needed is 16.7-ft for a span of 114-ft at 72nd Street. Cass Street is not used as an upstream tie-in as it is lower than the surrounding terrain. Modeling assumptions were made which assumed that a levee was made along Cass Street and used a closure structure across the street until the new levee tied into existing high ground.

This levee alternative offers flood risk management for the 2% - 0.2% AEP for the surrounding properties, which range from commercial to residential. However, the alternative also causes induced stages upstream from the levee, from Cass Street to nearly Blondo Street. The induced stages range from approximately zero to over 7-ft just upstream of 72nd Street for the 0.2% AEP. This significant increase in water surface elevation would be difficult to minimize or mitigate during alternative refinement.

Little Papillion Creek Levee/Floodwall to the 1% AEP, Damage Reaches LP5 through LP7: Similar to the 0.2% AEP levee/floodwall alternative, this alternative extends from the Saddle Creek/Little Papillion Confluence upstream to the Cole Creek/Little Papillion confluence, see Figure 20. Protection was provided to properties behind the levees for an elevation equal to the 1% AEP plus 3-ft.

On average, the height of the new levee or floodwall is approximately 4.5-ft. Additionally, closure structures would be needed at 11 crossings. It is assumed that HESCOS will be permitted in lieu of constructed closure structures in areas where the closure needed is within the additional 3-ft. That reduces the number of closure structures needed to seven. The maximum height needed is 7.2-ft for a span of 10-ft. The minimum height needed is 3.05-ft for a span of 10-ft. Cass Street is not used as an upstream tie-in as it is lower than the surrounding terrain. Modeling assumptions were made which assumed that a levee was made along Cass Street and used a closure structure across the street until the new levee tied into existing high ground.

This levee alternative provides similar impacts to those described in the above section; however the 0.5% AEP overtops the levee/floodwall at several locations. Additionally, this alternative does induce stages upstream of the levee. These induced stages range from approximately zero to as much as 4-ft for the 0.2% AEP event and extend from Cass Street to nearly Blondo Street. After public review, this alternative will be refined to minimize stage increases.

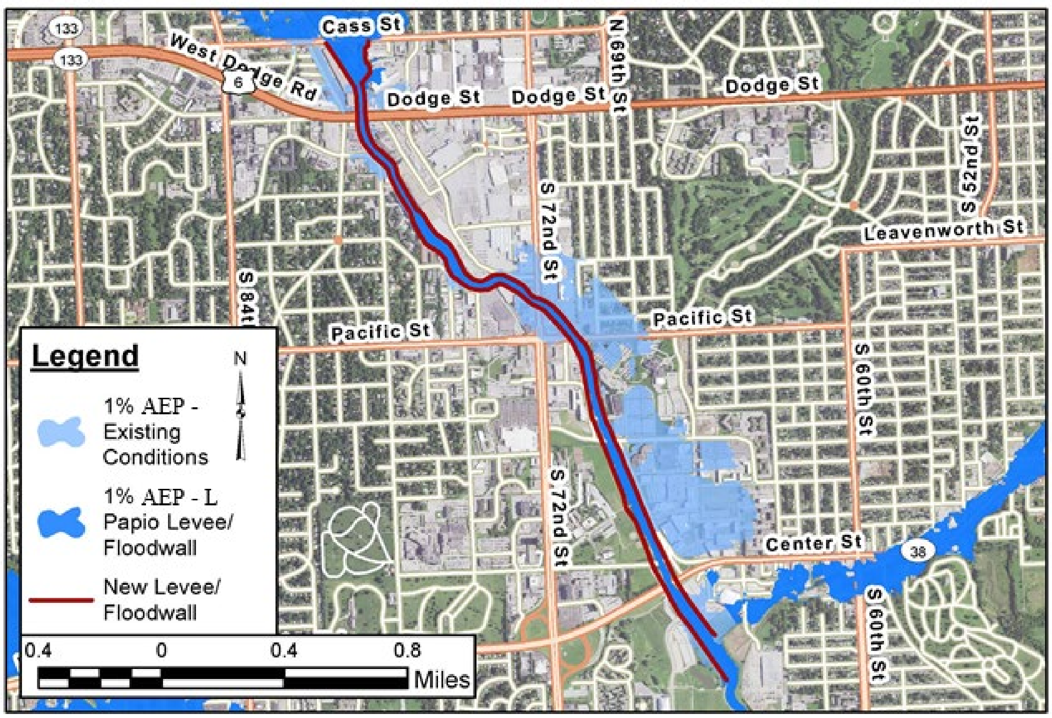


Figure 20. Evaluated Levee on Little Papillion Creek in Damage Reaches LP5 – LP7

2.4.3.4 Combination Alternative

The following combination of alternatives was not explicitly modeled hydraulically. Refer to the Economics Appendix F for more information.

Little Papillion Creek Levee/Floodwall to the 1% AEP in combination with Dam Site 10: To determine if the combination of Dam Site 10 with the addition of levees and/or floodwalls through damage reaches LP5 through LP7 provided more economic justification than either as a standalone alternative, the resulting water surface grids from the Dam Site 10 analysis were provided to the project economist. In addition to the water surface grids, levee elevations for each index point within damage reaches LP5-LP7 were provided. The levee elevation was equal to the resulting 1% AEP water surface, form the Dam Site 10 analysis, with an additional 3-ft for risk and uncertainty. This resulted in a levee and/or floodwall that was, on average, approximately 2.2-ft tall. Additionally, closure structures would be needed at 7 crossings. It is assumed that HESCOS will be permitted in lieu of constructed closure structures in areas where the closure needed is in the risk and uncertainty zone. That reduces the number of closure structures needed to 2. The minimum height needed is 4.05-ft for a span of 10-ft. The maximum height needed is 4.82-ft for a span of 114-ft. The extents of the levee and/or floodwall are roughly the same as the previously mentioned levee/floodwall alternative on the Little Papillion.

2.5 Tentatively Selected Plan Based on Steady Flow Modeling

At the completion of the steady flow hydraulic modeling level of analysis, the hydraulic features included in the TSP were dam site 10, channel widening from Blondo Street to 105th Street (BP4-BP5), a new levee and/or floodwall from Cass Street to Mercy Road on the Little Papillion (LP5-LP8), and a levee raise from I-80 to Harrison Street on the Big Papillion (BP7-BP8). These alternatives were carried forward in the study and their analyses refined using more site-specific hydrologic updates, unsteady hydraulic modeling, and more detailed economic modeling to confirm results. This detailed analysis was also completed on DS19 and the new levee/floodwall from Boxelder Creek to Millard Avenue on the West Papillion Creek (WP5-WP6) because these two alternatives were close to being justified and might become justified with more detailed analysis. All alternatives carried forward into the unsteady hydraulic modeling analysis are shown in Figure 21.

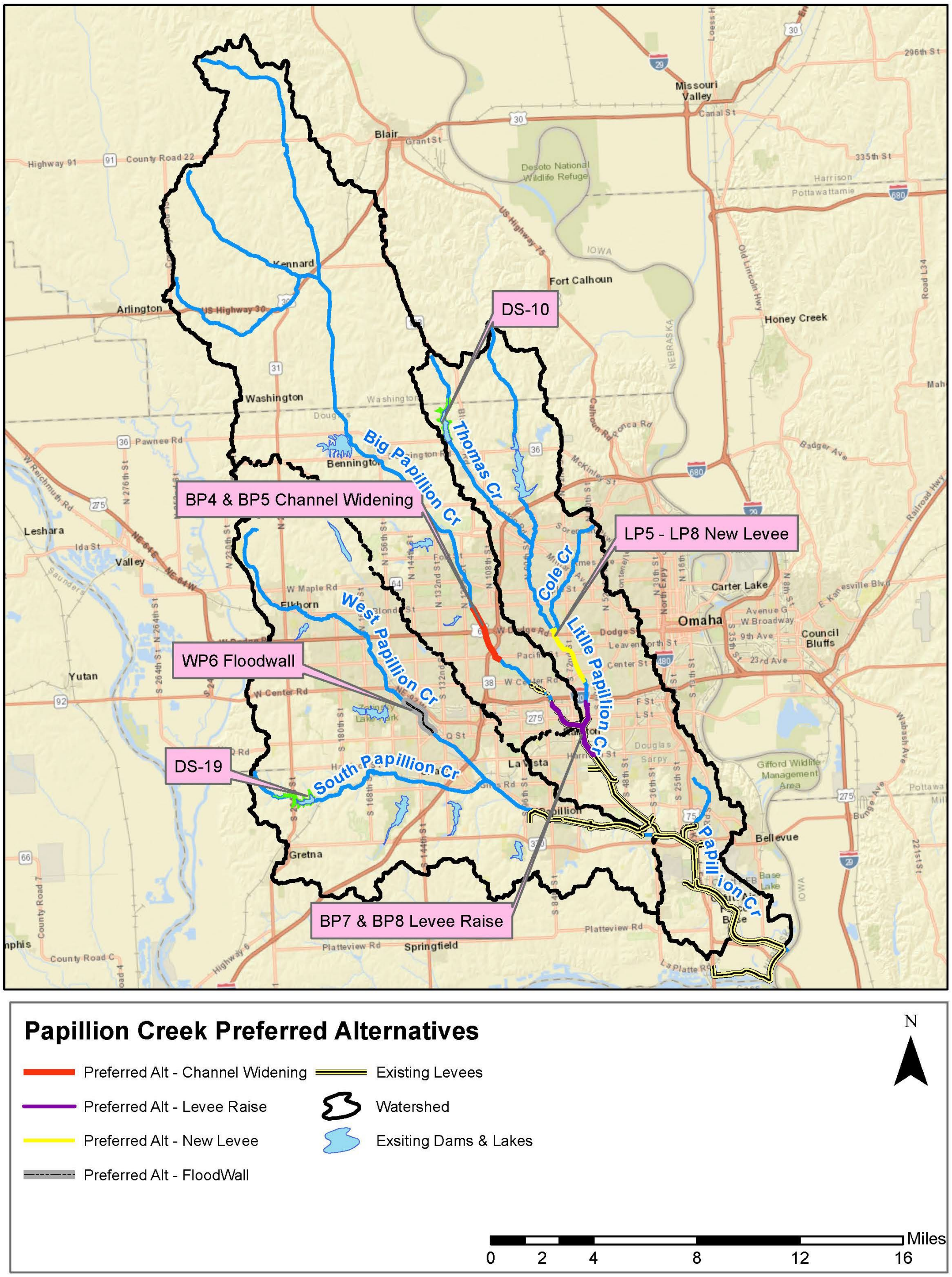


Figure 21. Overview of Hydraulic Features Carried Forward into Unsteady Hydraulic Modeling

3. 1D/2D UNSTEADY FLOW HYDRAULIC MODELING

After the TSP milestone meeting, the hydraulic modeling was refined from 1D steady state to 1D/2D unsteady which includes full hydrographs as input. This modeling is more detailed and time-intensive than steady flow modeling so it was implemented only after alternatives had been reduced. Hydraulic modeling during this phase of the analysis was completed using the HEC-RAS version 5.0.7. The resulting water surface elevations from both the existing and future with- and without-project conditions were utilized in the economic analysis using HEC-FDA to determine project justification and optimized levels of protection.

The 34 damage reaches previously used were further refined to better capture the impacts of the alternatives studied during the unsteady modeling effort. Appendix B-P1 has been updated to include these new reaches.

3.1 Existing Conditions Model Development

The separate 1D steady flow hydraulic models used in the previous analysis were combined into a single unsteady flow hydraulic model for use in the remainder of the study. The following sections detail the process, methods, and assumptions used in developing the unsteady flow model for the Papillion Creek and Tributaries Lakes GRR.

3.1.1 Geometry

The georeferenced 1D steady flow hydraulic models obtained from the P-MNRD for the first phase of the study were joined into one combined model using stream junctions.

Lateral structures were constructed in locations of existing and proposed levees. Elevation data from the National Levee Database (NLD) were input for the existing levee weir elevations while elevation data for areas of proposed levees followed the ground line. All lateral structures computed overflow using the weir equation to avoid a known issue in HEC-RAS 5.0.7 that allows water to leave 2D areas within the bridge computational zone when the normal 2D equations are used to calculate overflow. Weir coefficients were assigned to each levee in accordance with Table 3-1 from the HEC-RAS Two-Dimensional Modeling User's Manual, shown in *Figure 22*. In general, lateral structures with elevations that followed un-elevated ground were given a lateral weir coefficient equal to 0.5. Meanwhile, lateral structures that followed existing levees and raised roadway embankments were given lateral weir coefficients of 2.0.

Table 3-1. Lateral Weir Coefficients

What is being modeled with the Lateral Structure	Description	Range of Weir Coefficients
Levee/Roadway – 3ft or higher above natural ground	Broad crested weir shape, flow over levee/road acts like weir flow	**1.5** to **2.6** (2.0 default) SI Units: 0.83 to 1.43
Levee/Roadway – 1 to 3 ft elevated above ground	Broad crested weir shape, flow over levee/road acts like weir flow, but becomes submerged easily.	**1.0** to **2.0** SI Units: 0.55 to 1.1
Natural high ground barrier – 1 to 3 ft high	Does not really act like a weir, but water must flow over high ground to get into 2D flow area.	**0.5** to **1.0** SI Units: 0.28 to 0.55
Non elevated overbank terrain. Lat Structure not elevated above ground	Overland flow escaping the main river.	**0.2** to **0.5** SI Units: 0.11 to 0.28

Figure 22 Lateral Weir Coefficients

Two-dimensional (2D) areas were constructed with 200ft by 200ft computational cells on the landward side of each lateral structure. Cross-sections that extended beyond the lateral structures and into the 2D areas were trimmed, and blocked obstructions were placed in any remaining overlap. Terrain elevation data that was downloaded from the USGS and used for mapping purposes in the previous phase of this study was used to create the terrain model for the 2D areas. Gridded land use information was obtained from the USGS (NLCD 2011) and imported for use in the model. A user entered Manning's n-value was assigned to each land cover type as shown in Table 9 below.

Table 9 Manning's n Values per Land Cover

Land Cover Type	Manning's n Value
Barren Land Rock/Sand Clay	0.03
Cultivated Crops	0.045
Deciduous Forest	0.12
Developed, High Intensity	0.15
Developed, Low Intensity	0.06
Developed, Medium Intensity	0.08
Developed, Open Space	0.045
Emergent Herbaceous Wetlands	0.07
Evergreen Forest	0.12
Grassland/Herbaceous	0.045
Mixed Forest	0.12
Open Water	0.035
Pasture/Hay	0.035

Land Cover Type	Manning's n Value
Shrub/Scrub	0.08
Woody Wetlands	0.07

Breaklines were then added to 2D areas in locations where there was a barrier to flow, such as natural high ground and elevated roadway embankments.

Figure 23 *Unsteady Hydraulic Model Geometry*below provides an overview of the hydraulic model geometry.

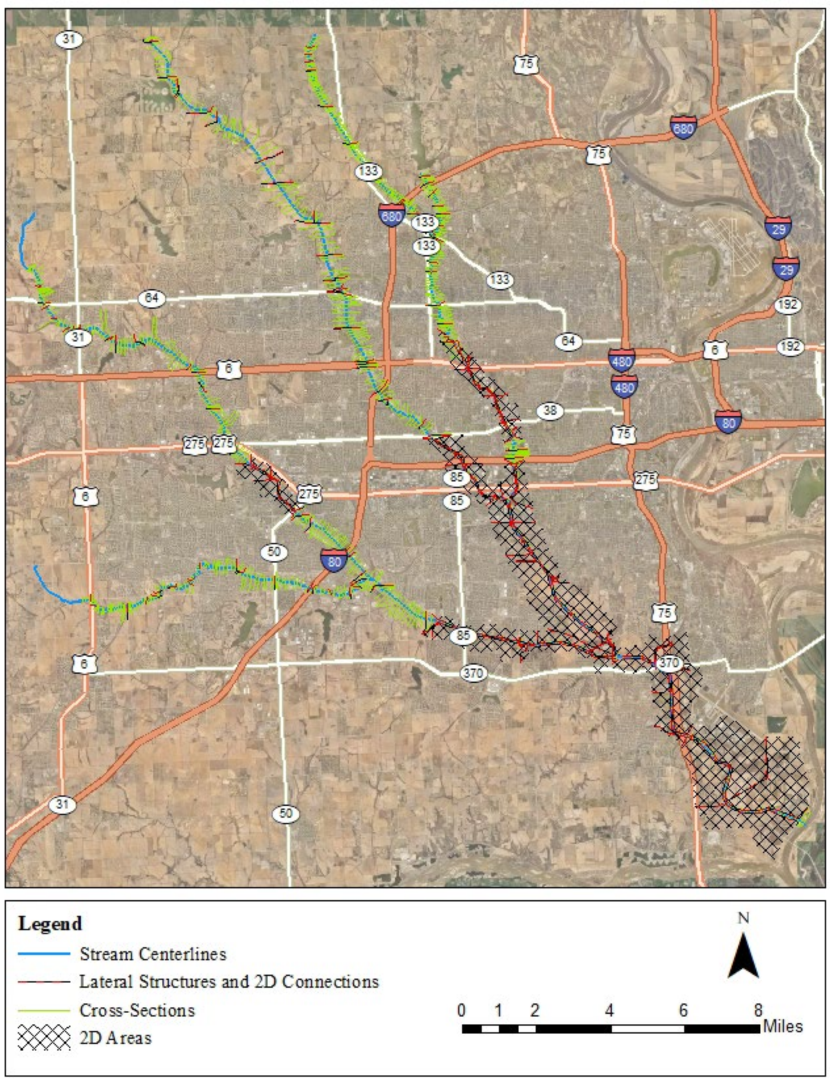


Figure 23 Unsteady Hydraulic Model Geometry

3.1.2 Hydrology

The same HMS model used to produce peak flows for the steady-state analysis before TSP was used to produce runoff hydrographs as input to the unsteady RAS model. Runoff hydrographs were created and provided as DSS flow files by the project hydrologist for each sub basin within the Papillion Watershed for an array of different storm sizes. During the steady-state analysis, the storm size used to produce the flow at each junction of the RAS model was determined based on the cumulative drainage area to that specific inflow point. However, when an alternative like a levee is sized, all sub basins to that point in the channel should have the same storm area which produces a consistent intensity of rainfall over the full contributing drainage area to the proposed levee. To appropriately size the alternatives while still being able to compare with- and without-project results consistently, changes were made to the sub basin storm sizes as shown *Figure **24*** below.

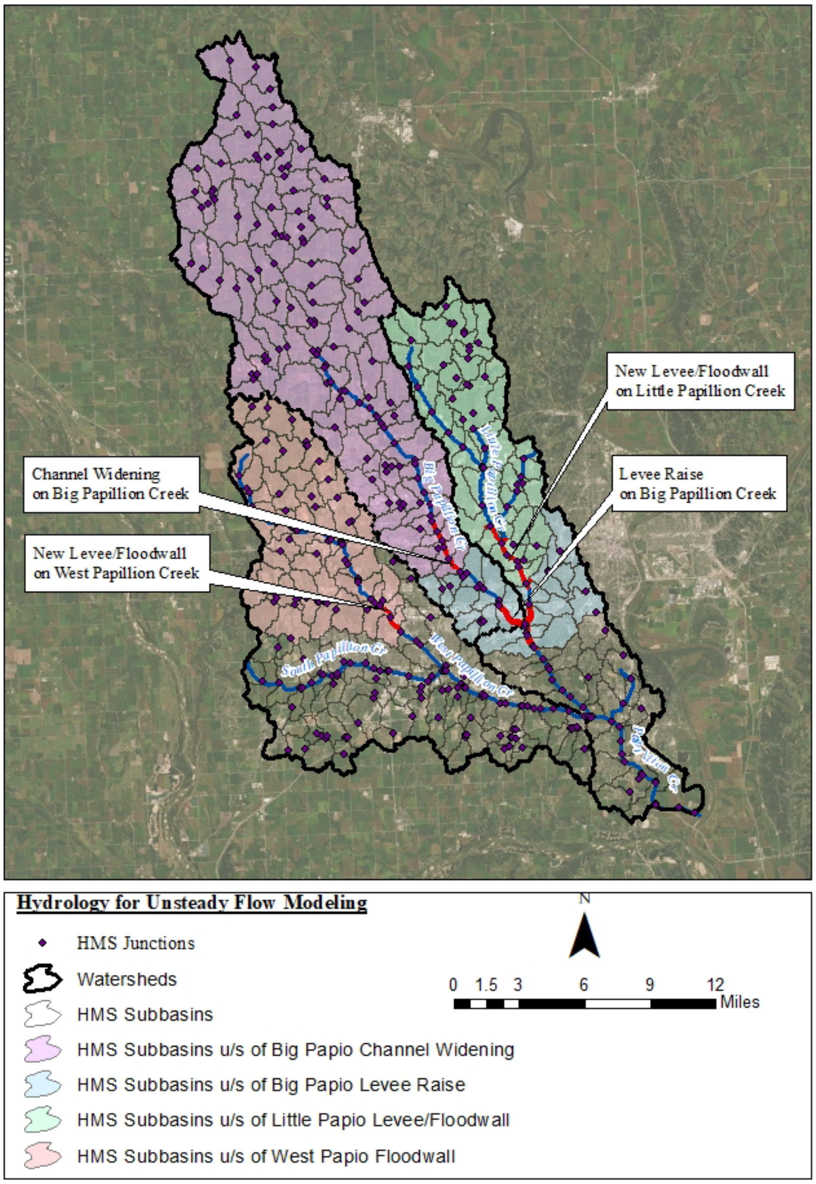


Figure 24 Storm Size Adjustment for Unsteady Flow Alternatives Analysis

To provide model stability, a minimum flow of 200 cfs was applied at the upstream extent of each reach. Because event peak flows in modeled reaches exceeded 200 cfs, this initial flow assumption does not impact peak model results.

Appendix B-P27 shows where HMS elements are applied in the model.

3.1.3 Model Simulations

Unsteady model simulation was evaluated over a 5-day time period. Default values were retained in the computation options with a few exceptions. Warm-up time steps were adjusted to provide model stability at the onset of the simulation. Likewise, the lateral structure flow stability factor and weir flow submergence decay exponent were both updated to 3.0. This provides additional model stability where lateral structures connect 1D and 2D areas with little impact to model results.

Computational interval sensitivity found that a 10 second time step was appropriate with output mapping set to 15 minutes.

3.1.4 Existing Conditions Model Results

Inundation mapping and water surface grids were prepared and provided in the same manner as the earlier phase of this study. Resulting inundation maps and water surface profiles are provided in Appendix B-P29. In most locations, due to the change in hydraulic modeling approach and more detailed hydrology inputs, the water surface elevations resulting from the unsteady modeling are lower than those resulting from the steady modeling. This will likely result in a decrease in without-project damages and therefore potentially lower the with-project benefit cost ratios (BCRs).

3.2 Alternatives Analysis and Optimization

Unsteady flow modeling was used to confirm and optimize all alternatives identified in the TSP during steady flow modeling to identify the designs that yielded the maximum benefit for the least cost. Additionally, two alternatives were carried forward and analyzed using unsteady flow modeling despite having a BCR slightly below unity: DS10 and the West Papillion floodwall. To appropriately analyze these alternatives, the future without project condition also had to be modeled using unsteady hydraulic modeling. Resulting water surface grids were provided to the project economist to use in the HEC-FDA analysis.

3.2.1 Future Without Project

The future without project condition assumes the same channel geometry, i.e. no further encroachment or river crossings were anticipated, and the same existing conditions geometry file was used with the exception of the 36th Street bridge over the West Papillion. Existing condition hydrology was also updated to reflect full build-out conditions throughout the basin. The following sections go into more details. Resulting inundation maps for Future Without Project conditions are in Appendix B-P30.

3.2.1.1 Geometry

At the time of this report, the City of Bellevue was currently working on a project to widen 36th Street from Highway 370 to Cornhusker Road. This section of roadway will be widened to a four-lane divided section from a two-lane section. The project will also include raising the roadway profile to accommodate a 2% AEP event and 1% AEP event on the West Papillion Creek Bridge. The roadway profile is planned to tie into the exiting bridge over the Big Papillion Creek, which will remain in place.

The most current bridge plans at the time were obtained from Felsburg, Holt, & Ullevig and used to modify the 36th Street bridge over the West Papillion Creek in the model. This consisted of not only raising the bridge profile, but also widening the bridge deck and bridge opening. Internal 2D connections were added to the 2D area to simulate a raise and widening of 36th St between Highway 370 and Cornhusker Rd.

Appendix B-P28 provides the current bridge plans at the time of this study.

3.2.1.2 Hydrology

Model hydrology was updated to account for full build-out conditions. More information can be found in the Hydrology Appendix A.

3.2.2 Reservoir Alternatives

Both DS10 and 19 were modeled first as dry dams to determine flood risk benefit before being modeled as wet dams and including recreational benefits. Public opposition has led the sponsor to support modeling DS10 as a dry dam only.

Under steady flow modeling, DS10 proved to have the highest BCR when modeled in combination with a new levee on the Little Papillion Creek spanning from Cole Creek to Saddle Creek. Therefore, DS10 was only analyzed as part of this combination. The following sections describe the modeling process for both dam site alternatives, however, results for DS10 are only shown in combination with a new levee on the Little Papillion later in the report.

3.2.2.1 Rating Curves

In the previous phase of the analysis, all sub basins upstream of proposed dam sites were removed from the HEC-HMS model to quickly and conservatively gage effectiveness. During the unsteady modeling phase of the analysis, outlet and spillway rating curves were developed for the two remaining reservoir alternatives and utilized in HEC-HMS to develop realistic outflow hydrographs. These hydrographs could then be used in the unsteady hydraulic models.

Dry Dam Outlet Rating Curves:

Outlet works for the DS10 and 19 dry dam configurations consist of a reinforced concrete box with a minimum size of 6’(R) by 4’(S). Adhering to this minimum size allows for easier maintenance with a skid loader or Bob Cat. Outlet rating curves were developed using the relationship

$$Q = C_o A \sqrt{\frac{2gh}{k}}$$

where: C_o = orifice coefficient
A = culvert area, ft^2
g = acceleration due to gravity, 32.2 ft/s^2

<u>under inlet control:</u>
h = distance between upstream water surface and centroid of flow, ft
k = 1

<u>under outlet control</u>
h = distance between upstream and downstream water surface, ft
$k = 1.5 + \left[\frac{29Ln^2}{R^{1.33}}\right]$, where: L = culvert length, ft
n = culvert Manning's n value
R = hydraulic radius, ft

Rating curves were developed for various culvert sizes with the target of a max flow near the 50% AEP event. Flow under both inlet and outlet control was calculated for each headwater elevation, the minimum value being the controlling value.

An additional DS19 dry dam configuration was required to determine cost allocation and demonstrate that DS19 met economic justification for flood risk management without recreation benefits prior to moving forward with the DS19 wet dam analysis.

Initially, following a similar outlet works design as the DS10 dry dam outlet works, the DS19 wet dam showed more benefits than the DS19 dry dam. This is because the outlet works of the wet dam configuration restricted high occurrence event flows from the dam more significantly than the dry dam configuration. Therefore, the goal of the dry dam analysis became to more closely match downstream benefits to those of the wet dam configuration and more accurately identify the costs and benefits associated with creating a permanent pool. The configuration of the dry dam outlet works was modified to mirror the outlet works of the wet dam configuration (see next section for more information) with the addition of small openings the length of the intake structure, much like a perforated riser pipe, to allow discharge at any pool elevation. The openings along the intake structure varied in size and placement, which was dictated by what was needed to closely match the wet dam downstream discharges at all eight modeled AEP events. Openings had diameters ranging from 4 to 14 inches. Figure 25 provides a concept drawing of this structure. It is not drawn to scale nor are the holes placed in the same location as was used in the analysis. Over time, this structure is expected to fill into with sediment until only the larger openings near the top convey flow. At this point, the dam would have the same real estate requirements as the wet dam design.

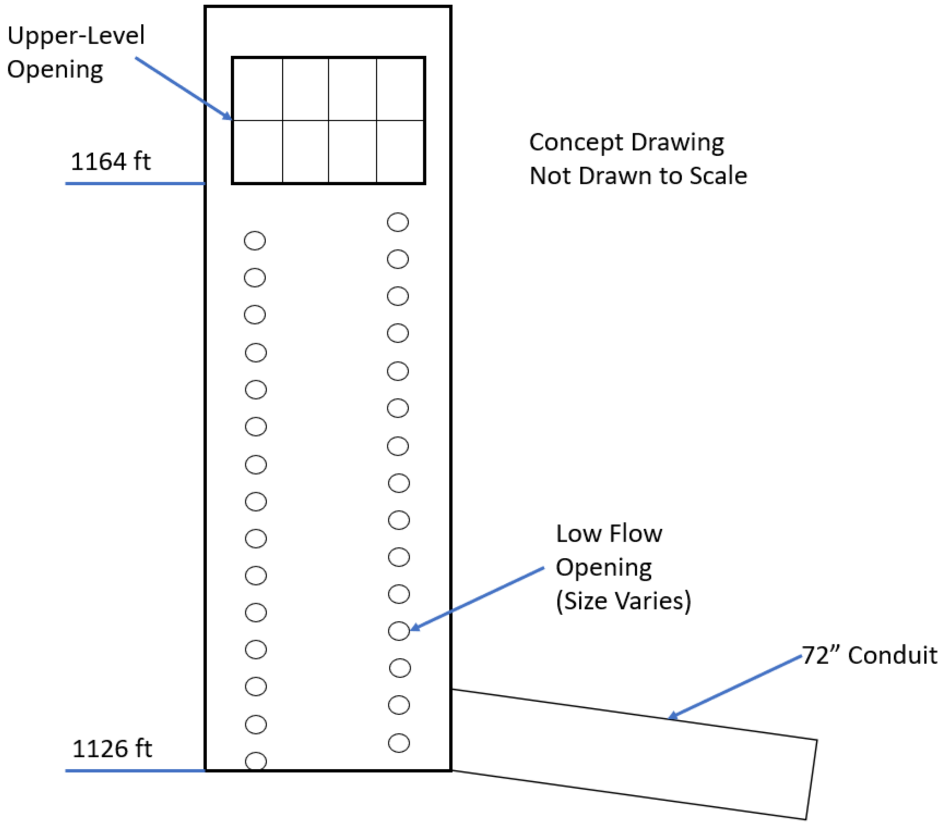


Figure 25 DS19 Dry Dam Outlet Structure

All dry dam rating curves were provided to the hydrologic engineer for HMS routing to determine pool stages and resulting reservoir outflow and are provided in Appendix B-P33.

Wet Dam Outlet Rating Curves:
The outlet works configuration for wet dam DS19 follows a similar design to what currently exist at the other Papillion Creek dams. A final inner pipe diameter of 72” was selected with two low flow openings of 6’ width by 5.5’ height set at elevation 1164 ft NAVD88. The final curve was produced by taking the lowest flow, at each selected elevation, of the weir, orifice, and head loss equations. The pool elevation for pipe flow was backcalculated by determining the total head losses due to friction, entrance, exit, and bend headlosses and then adding them to the initial assumed pool elevation. Those equations are summarized below. The wet dam outlet rating curve can be found in Appendix B-P33.

The Weir equation is defined as:

$$Q = CLH^{1.5}$$

Where: Q = Discharge in CFS
C = Weir Coefficient
L = Length of Weir in Feet
H = Hydraulic Head in Feet

The Orifice equation is defined as:

$$Q = C_d A \sqrt{2gH}$$

Where: Q = Discharge in Cubic Feet per Second
C_d = Orifice Coefficient
A = Flow Area through Orifice in Square Feet
g = Acceleration due to Gravity, 32.2 ft/s^2
H = Hydraulic Head in reference to center of Orifice in Feet

The Head Loss equation is defined as:

$$H = \frac{CLv^2}{2Dg}$$

Where: H = Headloss
C = Headloss Coefficient (I.E. f for friction, etc.)
L = Length of Pipe Flow in Feet
V = Flow Velocity in Feet per Second
D = Inner Pipe Diameter in Feet
g = Acceleration due to Gravity, 32.2 ft/s^2

Spillway Rating Curves:
Spillway rating curves were developed by creating a one-dimensional steady-state hydraulic model focused on the spillway of each dam. A trapezoidal channel was created from a range of bottom widths and 1V on 3H side slopes which were then cut into the existing terrain. Flows were then routed down this channel and an elevation was taken at the most upstream cross-section which was assumed to represent the pool elevation. Cross-sections were far enough upstream of the spillway channel that the computed water surface elevation was roughly equal to the energy grade line (elevation at which no velocity is present, i.e. pool). The final elevations used to create the spillway rating curves were the energy grade line elevations. The Manning's "n" coefficient used for each spillway curve was assumed to be 0.025 which is what was used in the design of the other Papillion Creek dams. The downstream extent of the 1D models extended far enough that changing the downstream boundary conditions did not affect the pool rating curve.

A range of curves were produced based on varying bottom widths. These curves were then provided to Hydrology to route and determine the final selected alternative width. The final selected spillway rating curves can be found in Appendix B-P33.

3.2.2.2 Hydraulic Model

To simulate the addition of a reservoir, changes were made to both the model geometry and hydrology. All cross-sections upstream of the reservoir outlet works were removed from the model. In the DS10 model geometry, this resulted in removing 35 cross-sections and five bridges from the most upstream segment of Thomas Creek. In the DS19 model geometry, this resulted in removing 22 cross-sections and two bridges from the most upstream segment on the South Papillion Creek.

Boundary conditions that applied flow to the model upstream of the dam outlet work locations were also removed. DSS files from HMS that provided the resulting outlet flow hydrographs for each event for both existing and future conditions were then applied at the most upstream cross-section for each model.

Dam Site 10: The hydrographs provided for DS10 modeling resulted from routing reservoir flows in HMS through a dry dam utilizing a 7(R)'x 8(S)' concrete box culvert.

Dam Site 19: Dam Site 19 was analyzed both as a dry dam and a wet dam. The hydrographs provided for the dry DS19 modeling resulted from routing reservoir flows in HMS utilizing the dry dam outlet structure previously described (box culvert). The hydrographs provided for the wet DS19 modeling resulted from routing reservoir flows in HMS utilizing the wet dam outlet rating curve previously described.

3.2.2.3 Results

As expected, the addition of DS10 and DS19 reduced downstream water surface elevations. The influence of DS19 reaches downstream to the West Papillion – South Papillion confluence. While decreased discharges are seen all the way to the Papillion Creek – Missouri River confluence, the benefits are less noticeable downstream of the West Papillion – South Papillion confluence. The wet dam alternative produces more benefits than the dry dam alternative for DS19 because the dry dam continues to have much larger releases at lower pool elevations. Because these higher releases occur at more frequent pools, they have a larger impact on the economic benefits calculations.

The influence of DS10 reaches downstream past the confluence with the Little Papillion, as far downstream as Mercy Road. Because DS10 was not carried forward after Agency Decision Milestone (ADM) as an individual alternative, but rather in combination with a levee on the Little Papillion, this model geometry and hydrology will be used in optimizing the proposed levee/floodwall alternative on the Little Papillion and is discussed further in section 3.2.4.3 below.

3.2.2.4 Post-Analysis Design Changes

After a semi-quantitative risk assessment was undertaken by the project team it was decided to move the conduit outlet invert elevations from 1126 ft to 1139.4 ft and from 1151 ft to 1154 ft for DS19 and DS10, respectively. This was done to elevate the outlet works into more stable geology. The elevation of the upper level invert elevation at DS19 remained unchanged at 1164 feet, NAVD88. Sensitivity testing determined that pool volume was not significantly affected by these changes. The change in conduit outlet elevation raised the top of flood control pool by 0.4 feet for DS10, however, it did not produce a perceptible increase in the PMF pool elevation. At DS19, the updated elevation of the outlet works resulted in increases to the peak pool elevations created by the frequency events by 0.1 feet or less and a decrease in the standard project flood (SPF) from 0.97*SPF to 0.96*SPF. Therefore, no other design changes were needed. See the Hydrology Appendix A for additional information. Additionally, updated outlet rating curves show less outflow at each headwater elevation, suggesting that previously modeled output from using the original elevations would provide conservative results. Therefore, this late design change was not incorporated into the unsteady hydraulic models.

3.2.3 Channel Widening Alternatives - Big Papillion Creek Channel Widening

Figure 26 1% AEP Unsteady Flow Modeling Results for Channel Widening on the Big Papillion

The Big Papillion Creek channel widening alternative extends from Blondo Street at the upstream end down to 102nd Street near the vicinity of Pacific Street. See Figure 9 in Section 2.4.22 for a typical cross-section. Three different bench widths were considered for this alternative to determine which, if any, were economically justifiable. The three bottom bench widths chosen were 150 feet, 170 feet, and 200 feet. An additional multi-width alternative was analyzed which had the goal of minimizing real estate takings. The same flows were used for each of these alternatives. The economic analysis determined the BCR for this alternative was less than unity, resulting in it no longer being considered.

3.2.4 Levee/Floodwall Alternatives

Three levee/floodwall alternatives were carried forward: a levee raise on the Big Papillion, a new levee or floodwall on the West Papillion, and a new levee or floodwall on the Little Papillion. Because weir flow is based on the elevation of the energy grade line (EGL) rather than the water surface elevation in HEC-RAS, each of these alternatives was initially modeled by setting the top of the levee/floodwall equal to the elevation of the 1% AEP energy grade line elevation. Two additional heights were modeled to aid the economic optimization analysis. One used the 1% AEP energy grade line elevation with three additional feet of height and the second used the 1% AEP energy grade line elevation with five additional feet of height. For each alternative, construction costs were quantified, real estate needs were determined, and flood damage reduction was evaluated to determine the optimal design.

3.2.4.1 West Papillion Creek Floodwall

At the TSP, the West Papillion levee/floodwall alternative modeled with steady flow hydraulic modeling had a BCR close to, but under, unity. Unsteady flow hydraulic modeling was completed on this alternative to determine if a more detailed analysis would provide justification. After establishing exact tie off locations, this alternative is approximately 1.75 miles and extends from Boxelder Creek to Millard Avenue, see Figure 27.

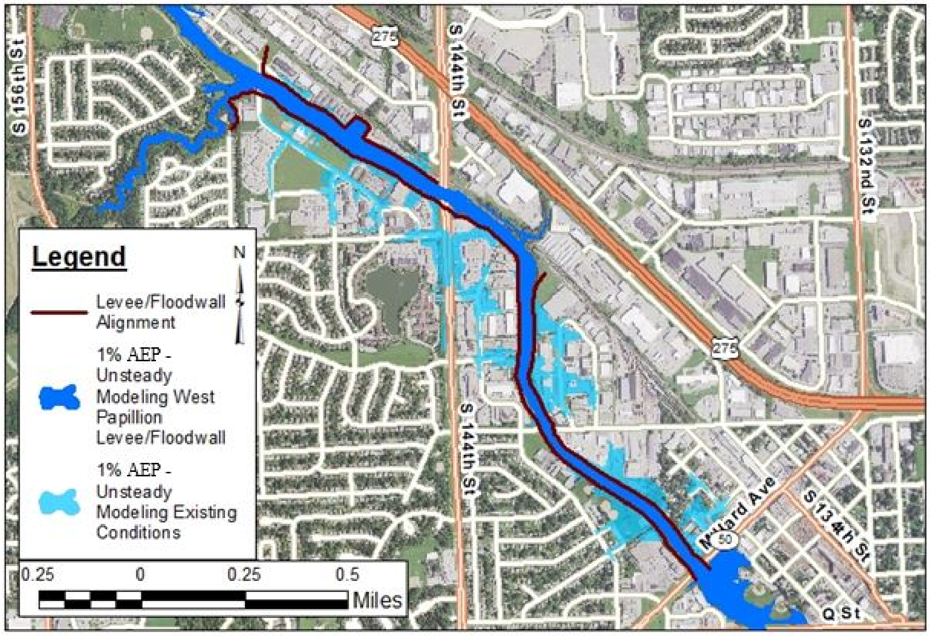
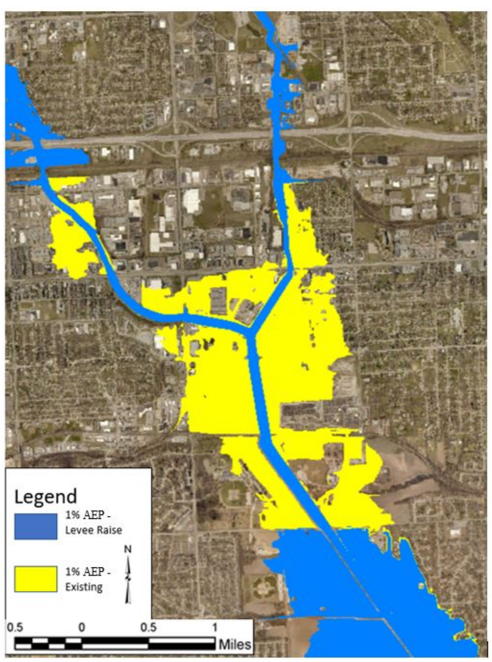


Figure 27 1% AEP Unsteady Flow Modeling Results for the Levee/Floodwall on West Papillion

Although some benefits were realized as early as the 4% AEP event, the analysis concluded that a BCR greater than 1 could not be achieved. Therefore, this alternative is no longer being considered and was not taken through optimization.

3.2.4.2 Big Papillion Creek Levee Raise



The Big Papillion Creek levee raise alternative started on both the Little and Big Papillion Creeks at the railroad embankment just downstream of Interstate 80 and had a downstream boundary at Harrison Street. Although benefits were seen with this alternative, the ensuing economic analysis showed this alternative to be unjustified at any height. Therefore, this alternative is no longer being considered.

Figure 28 1% AEP Unsteady Flow Modeling Results for the Levee Raise on Big Papillion

3.2.4.3 Little Papillion Creek Levee in Combination with DS10

After using unsteady flow hydraulic modeling to inform exact tie-off locations, the new levee or floodwall alignment on the Little Papillion is approximately 3.5 miles long and extends from Cole Creek to Saddle Creek on the left bank and from Charles Street to Spring Street on the right bank. See Figure 29.

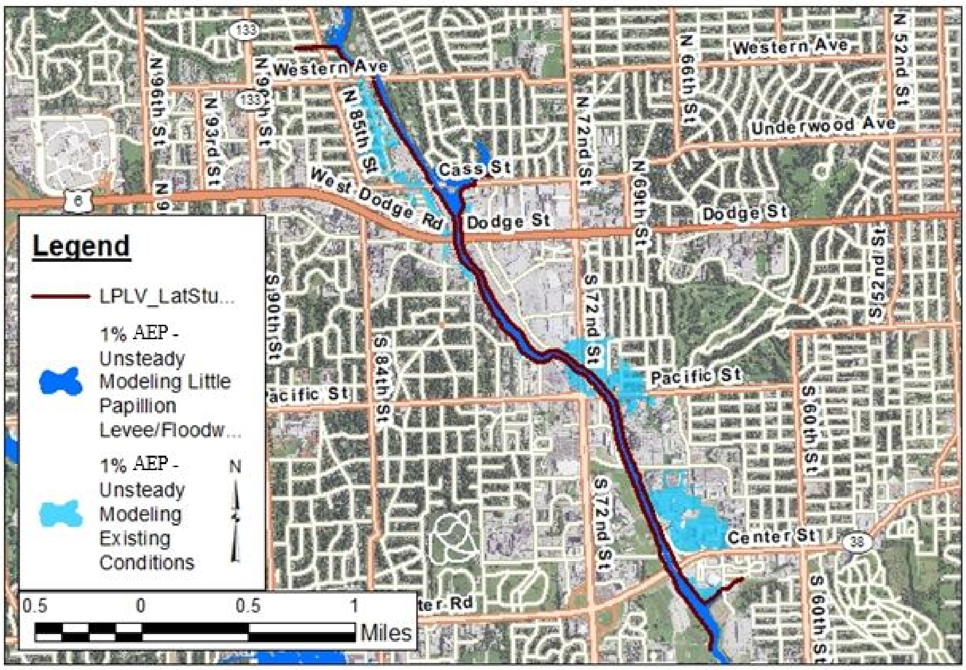


Figure 29 1% AEP Unsteady Flow Modeling Results for the Levee/Floodwall on Little Papillion

The economic analysis determined that the optimized height was equal to the elevation of the 1% AEP energy grade line with an additional 3ft. Benefits are realized as early as the 2% AEP event, and of the 8 events modeled for both existing and future flow conditions, only the 0.2% AEP event was shown to overtop the proposed levee at this height. See the structural Appendix D for information regarding updated information for closure structures.

3.3 Erosion Protection and Grade Stabilization

There have been several flood mitigations projects within the Papillion Creek watershed. Because these projects have performed well over the years, riprap requirements for the proposed features were modeled after them. It was assumed that riprap protection would be needed at drainage structure outlets, through bridges, and in locations of active erosion. However, areas that have been included in previous federal projects that fall under previous operation and management (O&M) requirements were excluded.

The proposed project on the Little Papillion Creek is entirely contained within a previous federal channel widening and realignment project. It was assumed that rock had been placed and continues to be maintained at all existing outfalls, through bridges, and in areas of current or expected erosion as detailed in the O&M manual. It was also assumed that any new outfalls or modifications to bridges since the federal project's construction would have designed with adequate protection. Therefore, the only areas requiring riprap protection in conjunction with the proposed project are in sections where there is an expected increase in high velocities. Figure 30 *Riprap Bank Protection Detail* shows the riprap bank protection detail used in areas of increased velocities to determine needed riprap quantities.

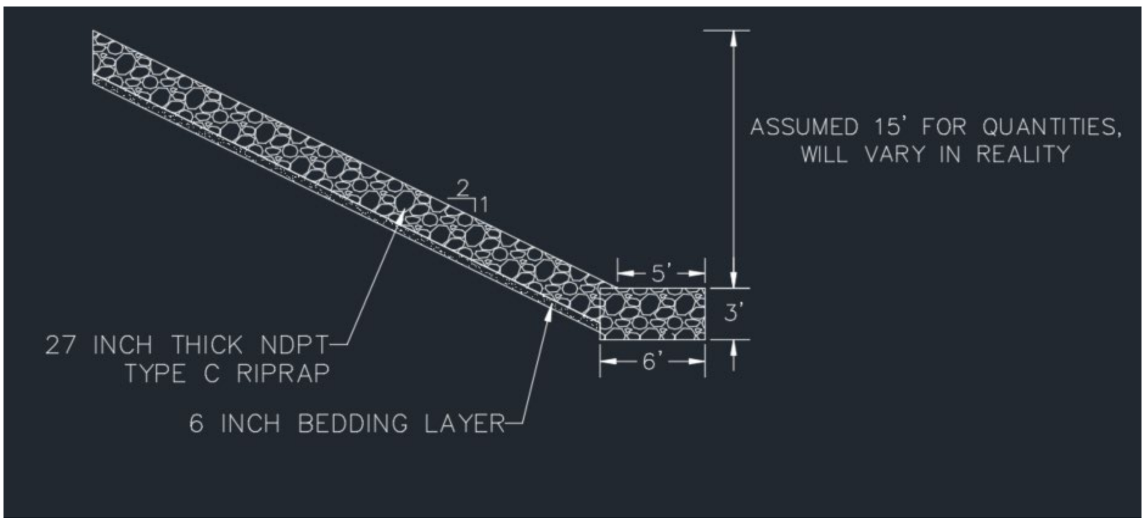


Figure 30 Riprap Bank Protection Detail

Both proposed projects on the Big Papillion overlap previous federal levee and channel improvement projects. As with the past project on the Little Papillion, O&M manuals state that riprap protection was placed at all existing outfalls, through bridges, and in areas of current or expected erosion and that these areas must be maintained by the local project sponsor. Additionally, the channel widening project is anticipated to improve channel conditions, making erosion protection along these banks unnecessary. Therefore, riprap protection has only been included at the modified 105th St bridge. Riprap needs were assumed to be satisfied by the detail shown in **Error! Reference source not found.**30.

Table 10 provides project quantities for riprap protection for each remaining alternative.

Table 10 Project Riprap Quantities

Alternative	Riprap Quantities (TN)	Bedding Quantities (TN)
Little Papillion Levee/Floodwall	4,537	887
Big Papillion Levee Raise	0	0

Big Papillion Channel Widening	5,566	1,043

Grade stabilization was also considered at this time. Areas downstream of the proposed projects were investigated for regions of active downcutting. Less than half a mile (0.4 miles) downstream from the proposed levee/floodwall on the Little Papillion Creek exists a concrete flume below the Union Pacific Railroad bridge. This flume was constructed during the Little Papillion federal channel improvement project. The concrete flume is 266 feet long and includes sheet pile cutoff walls on the upstream and downstream ends. It is anticipated that this structure will continue to stabilize the grade below the proposed project and, therefore, grade stabilization was determined unnecessary for its design.

Grade control was also considered for the proposed projects on the Big Papillion Creek. A shapefile was provided by USACE Omaha District River and Reservoir Engineering Section depicting areas of known headcutting in the Omaha metro. None of the indicated areas were near or assumed to impact the proposed projects.

3.4 Final Optimized Plan

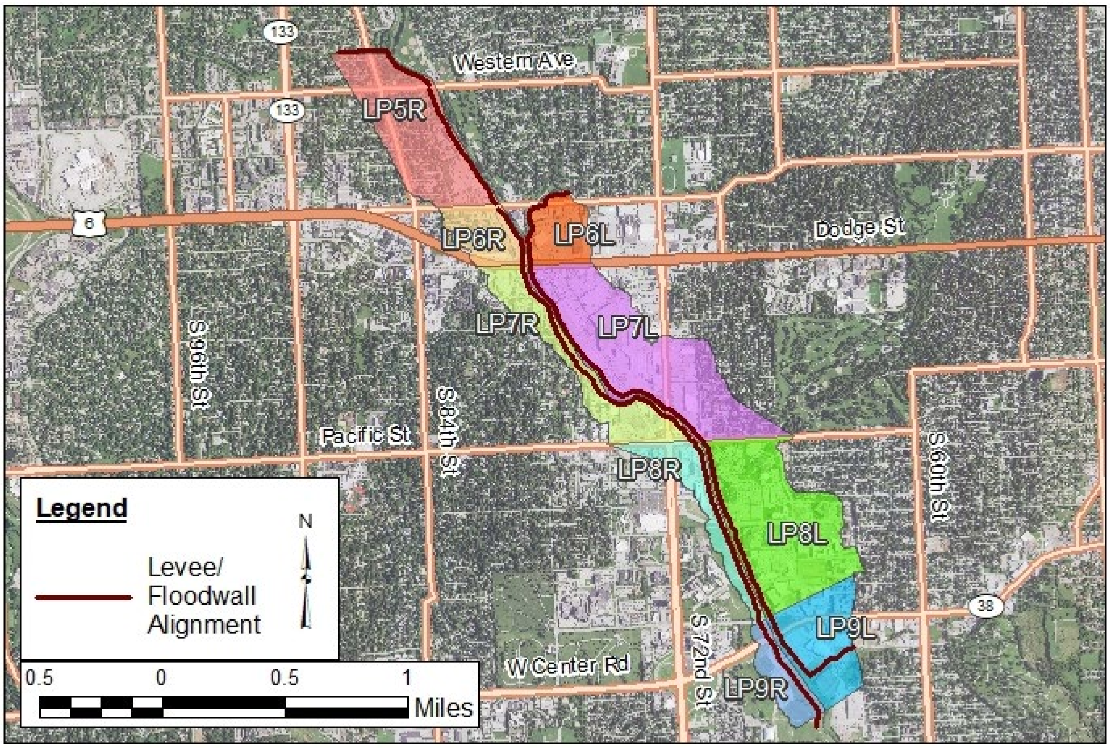
Once unsteady modeling and the corresponding economic analysis was completed for each alternative considered, the final optimized alternatives were modeled as one final plan. This included the DS19 wet dam, the DS10 dry dam, and the levee/floodwall on the Little Papillion with a top elevation equal to the 1% AEP energy grade line with an additional 3 feet. Resulting inundation boundaries and water surface profiles can be found in Appendix B-P31 and B-P32. Conditional Non-Exceedance Probabilities (CNP) per damage reach can be found in Table 11 *Conditional Non-Exceedance Probabilities for the Final Optimized Plan*

Table 11 Conditional Non-Exceedance Probabilities for the Final Optimized Plan

Damage Reach*	CNP									
	Existing Hydrology					Future Build-out Hydrology				
	10%	4%	2%	1%	0.2%	10%	4%	2%	1%	0.2%
LP5R	1.0000	0.9965	0.9776	0.9337	0.7987	1.0000	0.9971	0.9779	0.9312	0.7569
LP6R	1.0000	0.9841	0.9188	0.7941	0.4698	1.0000	0.9844	0.9161	0.7765	0.4228
LP6L	1.0000	0.9841	0.9188	0.7941	0.4698	1.0000	0.9844	0.9161	0.7765	0.4228
LP7R	1.0000	0.9954	0.9765	0.9406	0.8608	1.0000	0.9937	0.97	0.9265	0.8266
LP7L	0.9999	0.9842	0.9202	0.799	0.5305	0.9999	0.9775	0.8994	0.7621	0.4649
LP8R	0.9999	0.9868	0.9358	0.8383	0.6303	0.9999	0.9795	0.9104	0.7954	0.5613
LP8L	1.0000	0.9969	0.9839	0.9587	0.8986	1.0000	0.9952	0.9772	0.9457	0.8797
LP9R	1.0000	0.9983	0.9876	0.9621	0.8666	1.0000	0.9911	0.9552	0.8867	0.7301
LP9L	1.0000	0.9975	0.9842	0.9542	0.8552	1.0000	0.991	0.9549	0.8862	0.7299

* Note: the damage reaches listed in this table have been updated for use in the economic analysis that references unsteady flow hydraulic modeling. See Appendix B-P1 for more information on how the damage reaches were modified between steady flow to unsteady flow hydraulic analyses.

Figure 31 on the next page provides the corresponding damage reaches reported in the CNP table. Damage reaches LP7L and LP7R as well as LP8L and LP8R are using different locations for index points. This may, in part, be causing the discrepancy in CNP values seen on each bank of LP7 and LP8. See the economic appendix for additional information on the determination of index points and the CNP analysis.



*Note: the damage reaches displayed in this figure have been updated for use in the economic analysis that references unsteady flow hydraulic modeling. See Appendix B-P1 for more information on how the damage reaches were modified between steady flow to unsteady flow hydraulic analyses.

Figure 31 Damage Reaches for the Optimized Levee/Floodwall Alternative on the Little Papillion

4. RISK AND UNCERTAINTY

There are risks and uncertainties that exist due to the assumptions that were needed based on the project constraints. These are summarized in Table 12.

Table 12. Risk and Uncertainties

Risk	Potential Outcome
Natural uncertainty was not included in the stage-discharge uncertainty provided for the economic analysis	Economic analysis may underestimate damages. However, because this was kept consistent across the existing and alternatives conditions, this should be sufficient for selection of a TSP.
During steady flow modeling, damage reach WP9 extended from the confluence with the Big Papillion to the upstream extent of the levee system on the West Papillion.	Because the levee system offers significantly more protection in the upstream section, treating as one damage reach may not fully capture benefits provided in the shorter, non-federal section that offers much less protection.
During steady flow modeling, all alternatives used existing hydrology as opposed to future build-out hydrology.	Economic analysis may underestimate damages and alternatives may be under-designed, affecting which alternatives were carried forward into unsteady flow analysis. However, because this was kept consistent across all alternatives and outputs were compared to that from the existing conditions, this should be sufficient for selection of a TSP.
During unsteady modeling, hydrology upstream of each planned alternative was adjusted so that each upstream sub basin had the same storm area as the alternative being modeled.	Although this is necessary to accurately size each alternative, resulting without project damages would be reduced, affecting how many benefits each alternative can claim.

5. SUMMARY AND CONCLUSIONS

This report describes the hydraulic analysis completed to evaluate flooding in Sarpy, Douglas, and Washington Counties, NE, from the Papillion Creek and its Tributaries. Existing conditions were established, then without- and with-project alternatives were evaluated. This evaluation was originally completed using steady flow hydraulic modeling to screen out alternatives and determine a TSP. Alternatives included in this plan were then confirmed using unsteady flow hydraulic modeling during the ensuing optimization analysis. Results of the hydraulic modeling were used in the economic analysis to determine the configuration with the maximum net benefit. After optimization was concluded, it was determined that the optimal plan included DS19 as a wet dam, DS10 as a dry dam, and the levee/floodwall alternative on the Little

Papillion creek with a levee top elevation equal to the 1% AEP energy grade line elevation with an additional 3 feet, see Figure 32.

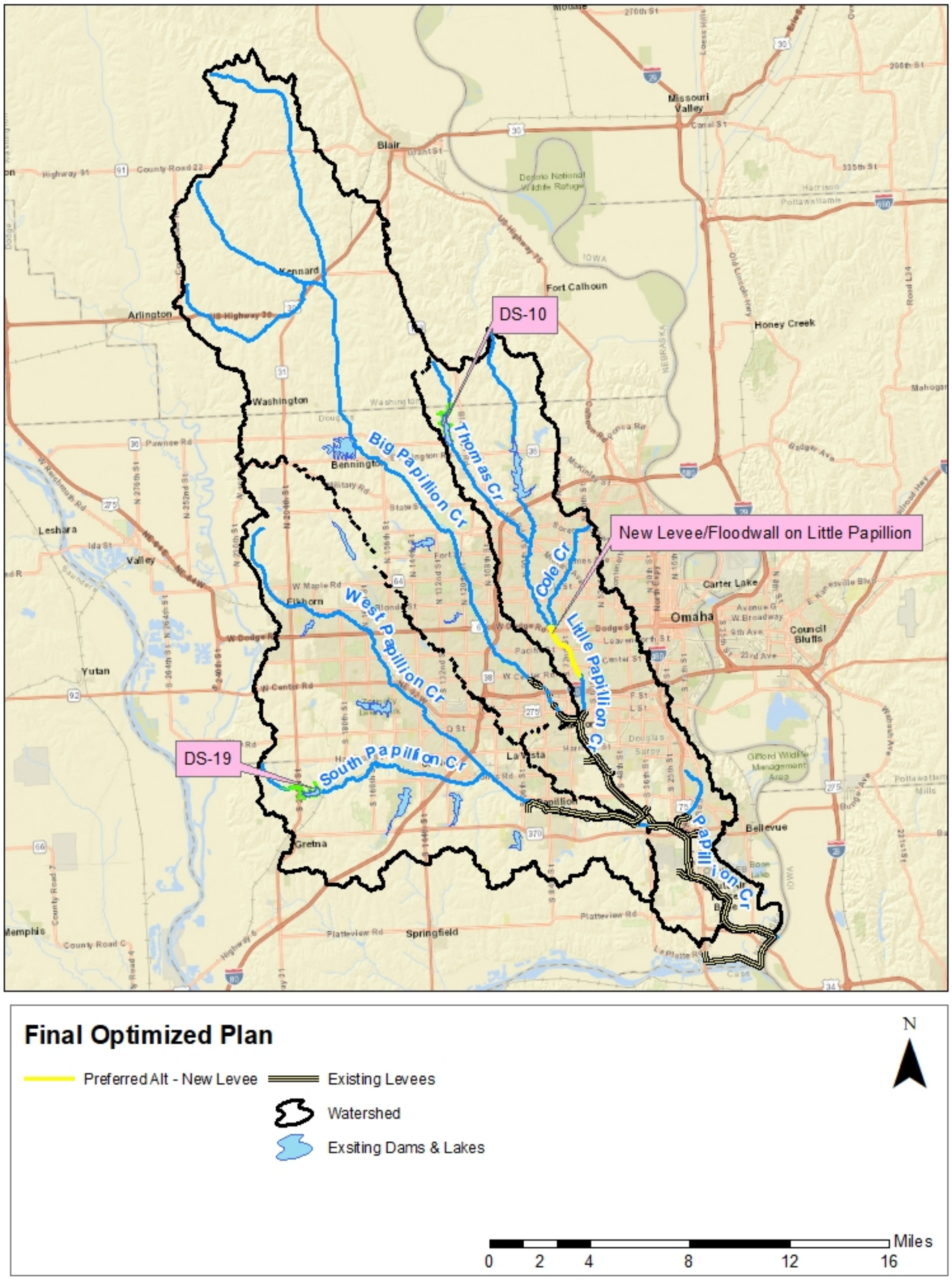


Figure 32 Final Optimized Plan

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