APPENDIX A
HYDROLOGY AND HYDRAULICS

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US Army Corps
Of Engineers

Fort Worth District, Water Resources Branch

DALLAS FLOODWAY FEASIBILITY STUDY

APPENDIX A

HYDROLOGY & HYDRAULICS

TRINITY RIVER BASIN, DALLAS, TEXAS

December 2014

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EXECUTIVE SUMMARY

The Dallas Floodway Feasibility Study was conducted to provide a full response to the Section 5141 of the Water Resources Development Act (WRDA) 2007 project authorization. Under that authority, the feasibility study developed a range of alternative plans for flood risk management, including levee safety, with potential costs and benefits associated with the plans; identified a National Economic Development (NED) Plan, which provides the highest excess of benefits over total costs of a plan; and, identified a plan that would address the life safety concerns in the study area. Following identification of the Flood Risk Management (FRM) plan, the Comprehensive Analysis phase was developed. The goals of the Comprehensive Analysis phase are (1) to determine on the basis of "technically sound" and "environmentally acceptable" the suitability of the City of Dallas' plans for constructing the Balanced Vision Plan (BVP), Interior Drainage Plan (IDP), and various Section 408 projects, including the Trinity Parkway, within the existing Dallas Floodway Project and (2) to ensure that those project features are compatible with the Dallas Floodway Project. The feasibility report also provides a recommendation on a "Modified Dallas Floodway Project" to be implemented under Section 5141 of WRDA 2007. See additional detail in the main report (Section 1.7) regarding the study overview and objectives of the Comprehensive Analysis.

As part of the feasibility study, a Risk Assessment was conducted to inform the decision-making process with respect to levee integrity and potential levee failure modes. During the Risk Assessment, a review of the Hydrologic and Hydraulic (H&H) modeling performed in support of the feasibility study was conducted. Several concerns arose from this review and an upper level review was performed. Based on this review three studies were identified to better define the existing conditions H&H: (1) a Regulated versus Unregulated Flow Study, (2) a Design Storm Study, and (3) an Unsteady Hydraulic Modeling Study. The report documenting the recommendations is enclosed with this appendix.

The results of the Regulated versus Unregulated Flow Study and the Design Storm Study were used to develop an updated frequency curve for the Trinity River at Dallas gage. When the Hydrology Committee reconvened in August 2012, the recommendation of the Hydrology Committee was to accept the composite frequency curve computed by the Fort Worth District. This composite curve utilized the historical Dallas gage annual peak flows from 1955 – 2011, the Design Storm study results, and the upper portion of the Regulated versus Unregulated study discharge frequency curve. The final existing conditions frequency curve at Dallas has a 100-year (1% Annual Chance Exceedance [ACE]) peak discharge of 114,000 cubic feet per second (cfs), and it indicates that the return period of the Standard Project Flood (SPF), which has a peak discharge of 269,300 cfs, is about 2,500 years (or 0.04% ACE).

The unsteady hydraulic modeling study was developed in order to better account for the effects of timing and flood volume resulting from a levee system overtopping flood event. The unsteady hydraulic analysis was performed for baseline and future without-project conditions to measure the performance of the existing Dallas levees against a range of levee overtopping flood events. The results from the unsteady flow analysis were then used as input into HEC-FDA and HEC-FIA to evaluate the economic and life safety consequences due to overtopping and/or breaching of the levees.

The final baseline hydraulic runs showed that while the East Levee would be the first to overtop and breach, the interior of the West Levee fills up faster due to the relative volumes available. The final results indicated that the East Levee could breach when the total Trinity River discharge equals or exceeds 255,000 cfs, and when it does breach, the average interior flooding elevations would vary between 415 and 420 feet. The final results also showed that the West Levee could breach when the river discharge equals or exceeds 273,000 cfs, and when it does breach, the average interior flooding elevations varied between 421 and 425 feet. However, the hydraulic sensitivity runs showed that the uncertainty in the breach assumptions could change the final flood elevations by +/- 6 feet. A levee internal erosion failure mode due to seepage leading to piping failure of the foundation of the levee was later modeled as a separate baseline condition in the hydraulic analysis. In the final results, the average interior flooding elevations varied between 405 and 420 feet on the East Levee and between 410 and 425 feet on the West Levee for internal erosion baseline conditions.

Unsteady hydraulic modeling was also used to evaluate the FRM alternatives for the feasibility study. Five different types of project alternatives were evaluated with unsteady HEC-RAS: (1) the AT&SF Bridge Modification, (2) Levee Height Modifications, (3) Levee Armoring, (4) Controlled Overtopping by Notching the Levee with armoring, and (5) Seepage Cut-off Walls to prevent breach for the levee internal erosion failure mode. These alternatives were developed to reduce the risks of interior flooding resulting from levee breach associated with overtopping and internal erosion, as identified in the Risk Assessment. For each of these alternatives, the HEC-RAS unsteady flow model was used to estimate the with-project inundation levels associated with the various floods events that were modeled and compare these to the without-project inundation levels. The economist then used HEC-FDA to estimate the with-project conditions reduction in economic damages resulting from the change in inundation levels.

At the conclusion of the alternative analysis, the plan that had the highest net benefits out of all of the analyzed alternatives was the 277,000 cfs (277k) levee raise with 3:1 levee side slopes along with the AT&SF Bridge modification. Therefore, the selected plan was the combination of the 277k levee raise with the AT&SF Bridge modification.

Following the selection of the FRM plan, the feasibility study moved into Comprehensive Analysis, which developed a plan to recommend as the Modified Dallas Floodway Project (MDFP) and ensured the projects would function on a system-wide basis and the combined features would not impact the functioning of the MDFP. The combinations of projects evaluated under the Comprehensive Analysis included the selected FRM plan, the City of Dallas' BVP, the IDP, and various local Section 408 projects, including the Trinity Parkway. The H&H analysis for the Comprehensive Analysis modeled several combinations of projects in HEC-RAS to determine if the overall project could achieve hydraulic neutrality and meet the H&H criteria defined in the Record of Decision (ROD) for the Trinity River (USACE 1988). The ROD and the H&H criteria used for this evaluation process are described in Section 6.1.1 of this appendix.

The results of the Comprehensive Analysis showed that the comprehensive plans for the BVP with and without Trinity Parkway did not meet the ROD criteria in terms of valley storage and water surface rise; however, the potential negative impacts are relatively insignificant. While additional design refinement efforts may be able to reduce the valley storage losses noted and/or reduce the water surface rises for the 1% ACE flood event within the Dallas Floodway on the main stem Trinity River, meeting the ROD criteria on every point is likely not achievable for such a large and complex combination of projects. Further reducing the negative impacts for valley storage loss to some extent may be achievable, but since these estimated impacts are relatively insignificant, efforts to further reduce them are not likely to be cost effective at this level of design. At the current level of design for the various project components considered, the level of compliance with regard to meeting the goals of the 1988 ROD criteria is estimated to be very near optimal. Further discussion of the 1988 ROD criteria and the application of the criteria to the analysis are provided in Section 6.1.

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1.0 INTRODUCTION

1.1 PURPOSE OF THE STUDY

The Dallas Floodway Feasibility Study is being conducted to provide a full response to Section 5141 of the Water Resources Development Act (WRDA) 2007. Under that authority, the feasibility study developed a range of alternative plans for flood risk management, including levee safety, with potential costs and benefits associated with the plans; identified a National Economic Development (NED) Plan, which provides the highest excess of benefits over total costs of a plan; and, identified a plan that would address the life safety concerns in the study area. Following the identification of the Flood Risk Management (FRM), a Comprehensive Analysis of all reasonably foreseeable proposed actions potentially impacting the Dallas Floodway was developed. The Comprehensive Analysis developed a plan to select a Modified Dallas Floodway Project (MDFP) to implement under Section 5141 of WRDA 2007 and ensured the projects would function on a system-wide basis and the combined features would not impact the functioning of the MDFP. See additional detail in the main report (Section 3.2) regarding the objectives of the Comprehensive Analysis. Primarily the Comprehensive Analysis addresses the acceptability of the City of Dallas' plans for constructing the Balanced Vision Plan (BVP), Interior Drainage Plan (IDP), Local Features and Section 408s, including the Trinity Parkway, within the existing Dallas Floodway Project and ensures those features are compatible with the existing system. At the end of the Comprehensive Analysis phase, the feasibility report recommends a plan that modifies the Dallas Floodway Project (the MDFP). Should the report be approved by the Assistant Secretary of the Army (Civil Works), the project may be constructed without additional authorization.

1.2 PROJECT LOCATION AND DESCRIPTION

The focal point in the study area is the Dallas Floodway Levee System comprised of the East and West Levees shown on Figure 1-1. The Dallas Floodway Levee System is a federally sponsored project currently maintained by the City of Dallas. The levee system extends along the Trinity River upstream from approximately the Atchison, Topeka, and Santa Fe (AT&SF) Railroad Bridge at Trinity River Mile 497.37, to the confluence of the West and Elm Forks at River Mile 505.50, thence upstream along the West Fork for approximately 2.2 miles and upstream along the Elm Fork approximately 4 miles. Of the 22.6 miles of levees within this reach, the East Levee is 11.7 miles in length and the West Levee is 10.9 miles in length, which includes a 1.5-mile segment along Mountain Creek. In addition to the levees, the Dallas Floodway includes a modified river channel, six pumping plants, seven pressure sewers, and three gravity sluices. The Dallas Floodway East Levee provides FRM benefits to areas that include the City of Dallas' Central Business District and the West Levee benefits West Dallas.

Immediately downstream of the Dallas Floodway Levees is the Dallas Floodway Extension project. The Dallas Floodway Extension project is comprised of three major structural FRM components. They are the Lamar Street Levee, the Cadillac Heights Levee and the Chain of Wetlands. The Dallas Floodway Extension is currently under construction, but for the purposes of the Dallas Floodway Feasibility study, the complete Dallas Floodway Extension project is considered part of existing conditions.

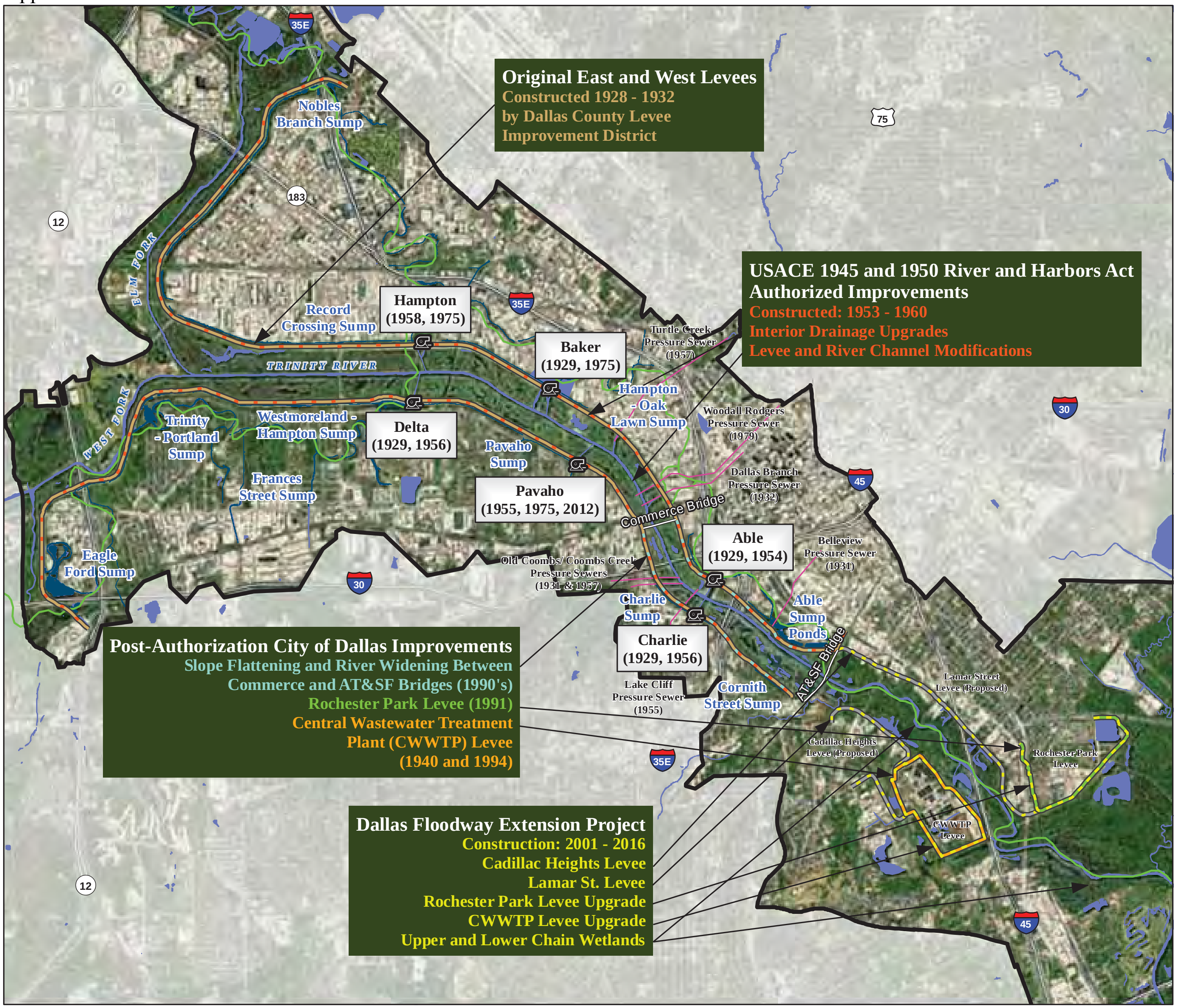
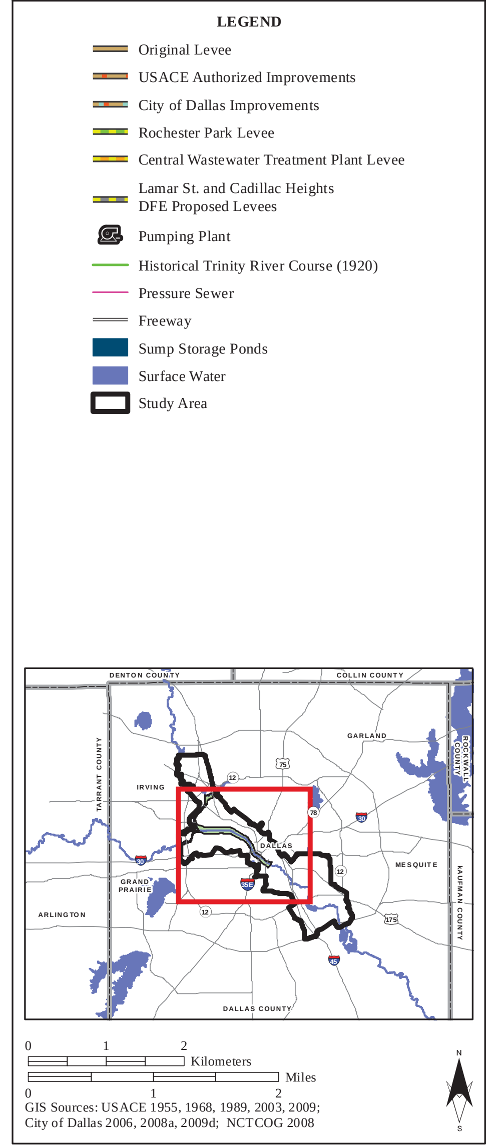


Figure 1-1
Map of the Dallas Floodway Levee System



1.3 PROJECT HISTORY

Major riverine flooding occurred in 1844, 1866, 1871, and 1908 in the Upper Trinity River watershed. Of note, in May 1908, the watershed experienced 10 to 15 inches of rainfall during a three-day period. The resulting flood killed several people and 4,000 others fled their homes to seek higher ground. Much of the downtown area and all of West Dallas was flooded. The Trinity River was nearly two-miles wide between west and downtown Dallas. The 1908 flood resulted in approximately $5 million in damage and was the impetus for initial efforts to control the Trinity River through the City of Dallas. The catastrophic 1908 flood led the City of Dallas to seek protection from Trinity River flooding. Between 1928 and 1932, the Dallas County Levee Improvement District (DCLID) constructed earthen levees to protect the City of Dallas from riverine flooding. The DCLID relocated the confluence of the West and Elm Forks, rerouted the Trinity River by constructing a channel within the floodway, and filled or set aside the original channel for interior drainage and sump storage. These original levees had a total length of 22.6 miles, an average crest width of 6 feet, an average height of 26 feet, and a maximum height of 37 feet.

In the mid-1940s, major storms, compounded by continued urbanization in the watershed, resulted in severe flooding in the area. To reduce the riverine flood risk within the City of Dallas, Congress authorized the flood control project commonly referred to as the Dallas Floodway, Dallas, Texas project, in 1945, and again in 1950. From August 1952 to June 1955, United States Army Corps of Engineers (USACE) produced six reports for design of the Dallas Floodway improvements to the original levees and interior drainage facilities. The improvements consisted of strengthening the 22.6 miles of the existing levees on both sides of the river, clearing the floodway channel, and improving the capabilities of the interior floodway drainage facilities. Levee strengthening included expanding the levee cross-section, flattening the levee side slopes, and increasing the crest width to 16 feet. Most of the additional fill material added to the levee slopes was placed on the riverside of the levees. Work was initiated on the authorized Dallas Floodway in July 1950 and construction was completed in April 1959. The Dallas Floodway Levee System "Levees" constructed under the 1945 and 1950 authorization are commonly referred to as the East Levee (left descending riverbank) and West Levee (right descending riverbank).

The Trinity River and Tributaries Regional Environmental Impact Statement (TREIS) (USACE 1987) was prepared by USACE Fort Worth District to address the proposed increases in floodplain development occurring in the Upper Trinity River basin during the Dallas-Fort Worth Metroplex development boom in the mid-1980s. Individually or cumulatively, these projects were considered to have the potential to affect existing flood risk management afforded to floodplain residents, and to impact wetlands and other natural resources. Two major conclusions were drawn from the TREIS:

1. A widespread lack of SPF protection existed.
2. Different USACE and local community permitting strategies have a significant impact on the extent of increase of this lack of SPF protection.

The Record of Decision (ROD) prepared for the TREIS specified criteria that USACE would use to evaluate future permit applications in the Trinity River Basin; specifically, projects located within the SPF floodplain of the Elm Fork, the West Fork, and the main stem of the Trinity River. The TREIS ROD established criteria for actions that require a USACE permit to address hydrologic and hydraulic impacts and mitigation of habitat losses. The findings in the TREIS provided the impetus for follow-on studies under the 1988 Upper Trinity River Study Authority (USACE 1988).

USACE initiated the Upper Trinity River Feasibility Study (UTRFS) in response to the authority contained in the U.S. Committee on Environment and Public Works Resolution dated April 22, 1988 and the findings of the 1990 Upper Trinity River Basin Reconnaissance Report. The UTRFS identified approximately 90 potential projects addressing flood risk management, ecosystem restoration, and recreation within the Upper Trinity River Basin. Of these 90 projects, three USACE projects were identified that had local sponsorship and were viewed as reasonably foreseeable, including modifications to the Dallas Floodway Project.

In May 1996, acting as the non-federal sponsor for the on-going UTRFS, the North Central Texas Council of Government (NCTCOG) coordinated with USACE and the City of Dallas to modify the UTRFS Cost Sharing Agreement to include an Interim Feasibility Study of the existing Dallas Floodway Levee System. The team assessed several flood risk management alternatives pursued under an Interim Feasibility Study.

An analysis was initiated in November 1998 to identify the NED Plan for resolving flood related problems and needs within the existing Dallas Floodway Levee System. The analysis considered various alternatives including No Action, non-structural measures including floodplain management, flood warning system, flood proofing, and relocation, as well as structural alternatives including channelization (i.e., widening the bottom of the river channel), and levee raises of 0, 1, 2, and 3 feet above the current SPF. In addition to considering ways to increase the level of flood risk management, USACE and the City of Dallas also developed additional environmental quality alternatives to benefit fish and wildlife habitat, improve water quality, and enhance visual resources while minimizing adverse impacts to existing cultural resources and flood risk management benefits. Out of this process in early 2000, the City of Dallas began development of what was to become the BVP.

Subsequently, Section 5141 of WRDA 2007 authorized USACE to review the City of Dallas BVP and IDP and to construct the project if the project components were determined to be technically sound and environmentally acceptable. This authorization superseded the need to continue development under the UTRFS and resulted in the current Dallas Floodway Feasibility Report and EIS preparation.

1.4 Watershed Description

The watershed of the Trinity River, from its headwaters to the confluence of Five Mile Creek, near the IH-20 Bridge in south Dallas, contributes to the hydrology of the Dallas Floodway and was evaluated during this analysis. This area, which is commonly referred to as the “Upper Trinity” watershed, covers about 6,275 square miles. It includes the majority of the Dallas-Fort Worth (DFW) Metroplex. Terrain in this watershed varies in elevation from about 1,200 feet National Geodetic Vertical Datum (NGVD) at the headwaters of the West Fork of the Trinity River just northeast of Olney, Texas, to about 380 feet NGVD at the confluence of Five Mile Creek.

Of the five USACE flood control reservoirs in the study area, three (Lakes Benbrook, Lewisville, and Grapevine) were impounded in the early 1950s. Impoundments in the other two USACE reservoirs (Lakes Joe Pool and Ray Roberts) were initiated in January 1986 and June 1987, respectively. Additional major USACE flood control projects in the study area include the Fort Worth Floodway and the existing Dallas Floodway levee/channel improvement systems.

The two largest non-federal lakes in the study area, both of which are situated on the West Fork of the Trinity River, are Lake Bridgeport and Eagle Mountain Lake. Lake Bridgeport is located just west of Bridgeport in Wise County. Eagle Mountain Lake is located in northwestern Tarrant County, just upstream from the much smaller Lake Worth, which is owned by the City of Fort Worth. Eagle Mountain Lake has two sets of outlet gates and an emergency spillway, but since it has no dedicated flood control storage, large releases are required during flooding periods. Smaller lakes within the Upper Trinity watershed include: Lake Amon Carter, located on Big Sandy Creek south of Bowie in southwestern Montague County; Lake Weatherford, located on the Clear Fork of the Trinity River northeast of Weatherford in Parker County; Lake Arlington, located on Village Creek in western Arlington in Tarrant County; and Mountain Creek Lake, located on its namesake in Grand Prairie in western Dallas County.

2.0 UPDATES TO THE H&H METHODOLOGIES FOR DALLAS FLOODWAY

As a part of the feasibility study, a Risk Assessment was conducted by the USACE to inform the decision-making process with respect to levee integrity and potential levee failure modes. The analysis performed included a review of the Hydrologic and Hydraulic (H&H) modeling performed by the Fort Worth District in support of the feasibility study. The majority of the H&H modeling was completed over a decade ago. Some concerns arose from the review including:

- The computed probability discharge frequency relationship was based on the period of record 1955 – 1990 and did not include the last 20 years of record.
- The numerical modeling used by the Fort Worth District to generate the standard frequency peak discharges introduced conservatism in the results.
- Methodology to determine the SPF was not in accordance with USACE policy.
- Hydraulic modeling was performed with steady flow analysis using a simplified levee overtopping methodology.

These concerns led to an upper level review by the USACE Hydrology Committee.

The Hydrology Committee provides consulting services on specific problems as requested by various elements of the USACE. In February and March 2012, the Hydrology Committee met with members of the Fort Worth District, the Risk Management Center (RMC), and Southwestern Division to discuss the H&H modeling for the Dallas Floodway. The following issues were discussed during the meetings:

1. Determination of the standard project flood hydrograph and peak discharge estimate.
2. Application of depth area duration relationships to frequency rainfall events.
3. Frequency analysis for the period of record data.
4. Selection of the peak discharge frequency relationship.
5. Estimation of the return period for the standard project flood peak discharge.
6. Determination of interior versus exterior inundation relationships for estimation of consequences.

Following those meetings, the Hydrology Committee produced a report titled, “Findings and Recommendations Regarding Hydrology for the Dallas Floodway Project”, dated May 2012. This report is included for reference following Section 7. In the Hydrology Committee report, several studies were recommended as updates to the H&H analyses for baseline conditions and are documented herein. The recommended studies were:

- Regulated versus Unregulated Flow Study
- Design Storm Study
- Unsteady Hydraulic Modeling Study

These three studies were conducted by the Fort Worth District, and the results of those studies are documented in Sections 3.1, 3.2 and 4.0 of this appendix. A study of the effects of urbanization on the watershed was also recommended but was not conducted due to funding and schedule constraints.

Sections 2.1 through 2.6 below provide more detail on the Hydrology Committee's recommendations regarding these studies.

2.1 Determination of the Standard Project Flood Hydrograph and Peak Discharge Estimate

USACE policy for determination of SPF estimates is provided by Engineering Regulation (ER) 1110-2-1464. For projects located east of the 105th meridian, such as the Dallas Floodway, the policy requires that the SPF estimate be developed using the procedures described in Engineering Manual (EM) 1110-2-1411. For projects located west of the 105th meridian, the SPF may be estimated as 50% of the probable maximum flood. The current SPF for the Dallas Floodway project has a peak discharge of about 269,000 cubic feet per second (cfs) at the Dallas gage located near Commerce Street (United States Geological Survey [USGS] gage 08057000). The current SPF is based on a total rainfall amount equal to 50% of the probable maximum precipitation determined in accordance with Hydrometeorological Reports 51 and 52. The Hydrology Committee reviewed the methodology that had previously been used to compute the SPF and recommended that the current SPF estimate based on 50% of the probable maximum precipitation with peak discharge of about 269,000 cfs be used for the Dallas Floodway project. Their recommendation for this approach is based on the fact that, the EM for SPF determinations is outdated and that probable maximum precipitation estimates are more current. It was determined that since the selected methodology for development of the SPF was not in compliance with ER 1110-2-1464, that a policy waiver should be requested for this study. The waiver request was forwarded to USACE Headquarters with expected concurrence by the USACE Southwestern Division and the USACE Regional Integration Team. The waiver was approved November 7, 2014.

2.2 Application of Depth Area Duration Relationships to Frequency Rainfall Events

Hydrologic models are used to simulate frequency based precipitation events to obtain a peak discharge estimate for a given frequency flood. Hydrologic models use point rainfall estimates, balanced hyetographs to obtain a temporal distribution, and depth area duration relationships to obtain frequency discharge estimates. One such depth area duration relationship is Figure 13-1 in EM 1110-2-1417, which is a reproduction of Figure 15 in Technical Paper 40 (TP40). This relationship is used by both HEC-1 and HEC-HMS (USACE hydrologic modeling software). Frequency based precipitation events simulated for the Dallas Floodway Feasibility Study using HEC-1 utilized the TP40 based depth area duration functions. These TP40 depth area duration functions are only valid for drainage areas up to approximately 400 square miles. The total drainage area of the Upper Trinity watershed above the Dallas gage is about 6100 square miles of which about 5000 square miles is influenced by USACE reservoirs located above the gage. About 1000 square miles of the watershed contributes to the flow at the Dallas gage. Therefore, the TP40 depth area duration functions used in HEC-1 for the simulation could overestimate the flows at the Dallas gage. The Hydrology Committee recommended that HMR-52 depth area duration relationships be applied to obtain the spatial and temporal distribution of rainfall for the frequency based precipitation events. In addition, the Hydrology Committee also supported initiating a new study to develop regional depth area duration relationships. Results of this study ensure that appropriate and consistent depth area duration relationships are available for the Dallas Floodway study and numerous other studies throughout the region. A sufficient number of storms of various sizes, durations, and magnitudes need to be evaluated to support adoption of regional depth area duration relationships.

2.3 Period of Record Frequency Analysis

It is common practice to use historic observations to inform selection of a peak discharge frequency relationship. Appropriate consideration must be given to issues such as record length, regulation effects, and homogeneity of the available data. Procedures for hydrologic frequency analysis are provided by EM 1110-2-1415. A frequency analysis was performed for the Dallas gage for the 1995 GRR using water years 1955-1992 and Bulletin 17B methodology. It was not updated for the current study on the Dallas Floodway. The Hydrology Committee developed an analytical peak discharge frequency relationship at the Dallas gage based on water years 1955 through 2011 (57 years of record) using Bulletin 17B methodology. The Hydrology Committee recommended that the full period of record of readily available data for water years 1955 through 2011 be used for the period of record frequency analysis.

In addition, the Hydrology Committee questioned the use of Bulletin 17B methodology in such a regulated watershed. The Fort Worth District believes that reservoir regulation does not significantly affect the shape of the peak discharge frequency relationship at the Dallas gage. The Hydrology Committee recommended that further analyses be undertaken to provide evidence in support of this claim. The Hydrology Committee recommended that the period of record data be adjusted to reflect unregulated conditions and that an unregulated peak discharge frequency relationship be developed. A regulated versus unregulated relationship should then be applied to obtain a regulated peak discharge frequency relationship.

The Upper Trinity watershed above the Dallas gage has experienced significant urbanization over the period of record of the gage. There are some uncertainties and unknowns regarding the potential impacts of urbanization on peak discharge frequency estimates. The Hydrology Committee supported initiating a new study to investigate urbanization impacts. Results of this study will be important to ensure that urbanization impacts on the peak discharge frequency relationships are appropriately addressed for the Dallas Floodway study and other studies throughout the region.

2.4 Peak Discharge Frequency Relationship

Both the period of record analysis, as well as frequency rainfall hydrologic modeling methods, were used to estimate the peak discharge frequency relationship for the Dallas Floodway study. The Hydrology Committee recommended that the analytical peak discharge frequency relationship for water years 1955 through 2011 with an adopted skew of 0.1 be used for the study. The Hydrology Committee also recommended that the analytical peak discharge frequency relationship for water years 1955 through 1992 be evaluated as a sensitivity case recognizing that this relationship may be somewhat conservative due to the limited record length. The Hydrology Committee recommended that selection of a final peak discharge frequency relationship for the Dallas Floodway project be deferred until completion of the unregulated frequency analysis, the study on regional depth area duration relationships, and the study on urbanization impacts.

2.5 Standard Project Flood Frequency Estimate

Prior to the USACE adopting explicit probability and uncertainty analysis methods in the mid-1990s, it was standard practice to communicate flood frequencies in terms of their expected probability. When the expected probability adjustment is applied to the frequency relationship using the period of record 1955 - 1990 computed for the Dallas Floodway, the expected annual chance exceedance probability for the SPF peak discharge of 269,000 would be about 0.00125 (800 year return period). To be consistent with current USACE policy, which requires explicit probability and uncertainty analysis methods, the Hydrology Committee recommended that the median estimate be used to characterize the return period for the SPF peak discharge estimate and that this estimate be used for purpose of HEC-FDA. The Hydrology Committee recommended a 4500 year return period estimate for the base condition based on the frequency analysis performed by the Hydrology Committee. A 2000 year return period estimate should be evaluated as a sensitivity case. Furthermore, the Hydrology Committee recommended that selection of a final estimate be deferred until selection of a final frequency relationship is made based on the results of the unregulated frequency analysis, the regional depth area duration relationship study, and the urbanization impact study. The final frequency curve results are discussed below in Section 3.4 and the selected return period for the SPF was 2500-year or 0.04 % AEP.

2.6 Inundation Depth for Estimating Consequences

Analyses conducted using HEC-FDA are based on levee overtopping and/or levee overtopping with breach and the results from inundation of the levee protected area. HEC-FDA does not explicitly distinguish between breach prior to overtopping, overtopping with breach, and overtopping without breach inundation scenarios. The peak water surface elevation for the resulting inundation of the levee protected area is determined by an interior versus exterior function provided by the user or by a default assumption that the resulting inundation elevation of the levee protected area is equal to the water surface elevation in the river. Consequences are then estimated using a peak water surface elevation versus a depth/damage function. A concern arose that the consequence estimates for the Dallas Floodway study conducted earlier using only HEC-RAS steady flow analysis may be overestimated and somewhat conservative due to the approach applied by the Fort Worth District. The Hydrology Committee also recommended that the Fort Worth District consult with the MMC Production Center to develop unsteady flow hydraulic models for use in developing interior versus exterior relationships. These relationships should be used with the HEC-FDA model for the final feasibility study analysis.

3.0 HYDROLOGIC ANALYSIS

3.1 Regulated versus Unregulated Study

3.1.1 Introduction and Purpose of Regulated versus Unregulated Study

The regulated versus unregulated relationship study was completed to estimate the discharge frequency curve for the Trinity River at Dallas gage as part of the Dallas Floodway Feasibility Study. The Dallas gage has five USACE reservoirs in the upper watershed that regulate the flow on the Trinity River. By developing the relationship between the regulated condition and the unregulated condition of the Upper Trinity River basin at the Dallas gage, a regulated watershed condition discharge frequency curve could be determined based on the unregulated watershed condition discharge frequency curve.

The unregulated discharge frequency curve was calculated by utilizing the Hydrologic Engineering Center – Statistical Software Package Bulletin 17B Frequency Analysis. The computed curve from the analysis was used in part to construct the regulated versus unregulated relationship. Within the study, a graph was built plotting the regulated and unregulated instantaneous flows on a log/log plot. The regulated versus unregulated graph provided the final connection for the regulated versus unregulated relationship. The resulting discharge frequency curve for the Dallas gage was constructed using the unregulated watershed condition discharge frequency curve and the regulated versus unregulated flow graph.

3.1.2 In Progress Review Team

During the early scoping phase of the regulated versus unregulated study, the Fort Worth District reached out to experts in the hydrologic engineering field to solicit their help in developing the scope of work. These experts also reviewed the progress of the work throughout the study via conference calls and webinars hosted by the Fort Worth District.

3.1.3 Regulated and Unregulated Peak Flow Development at the Trinity River at Dallas Gage

3.1.3.1 RiverWare Trinity River Model

The Trinity River Basin is modeled with the reservoir system simulation program RiverWare for the period of record 1940-2009. The model reflects the current conditions of the Trinity River Basin from the upper West Fork of the Trinity River through the Trinity River at Romayor, Texas. The RiverWare Trinity River model is calibrated to best match actual operations of USACE reservoirs in the basin. As part of this study, an additional calibration effort was undertaken to reproduce the historical peak daily flows at the Trinity River at Dallas gage.

The simulated daily flows at the Trinity River at Dallas gage were developed using this RiverWare model for both regulated and unregulated conditions. The unregulated conditions model removed the USACE reservoirs above the Dallas gage (Benbrook, Joe Pool, Grapevine, Ray Roberts, and Lewisville) and control point objects were inserted in their place in order to route the inflow hydrographs downstream. Both the regulated and unregulated period of record flows for the Dallas gage were exported to a dss file. Using HEC-DSSVUE math functions, the maximum annual daily flows were developed.

3.1.3.2 Instantaneous Peak Flows

To create the relationship between the regulated and unregulated flows at Dallas, the daily flows were converted to instantaneous peak flows. The daily flows calculated from RiverWare were factored by a ratio to obtain the instantaneous peak flows. A study was performed to determine whether an average ratio could be applied to all the simulated daily flows. Using the USGS daily and instantaneous flows at the Dallas gage for the period of 1955-2011, an average ratio of 0.17 was calculated. Figure 3-1 shows the USGS daily and instantaneous flows plotted. The three lines represent the ratios of 1:1, 1:1.17, and 1:1.47. The 1:1.47 is the ratio calculated for the May 26, 1957 flows.

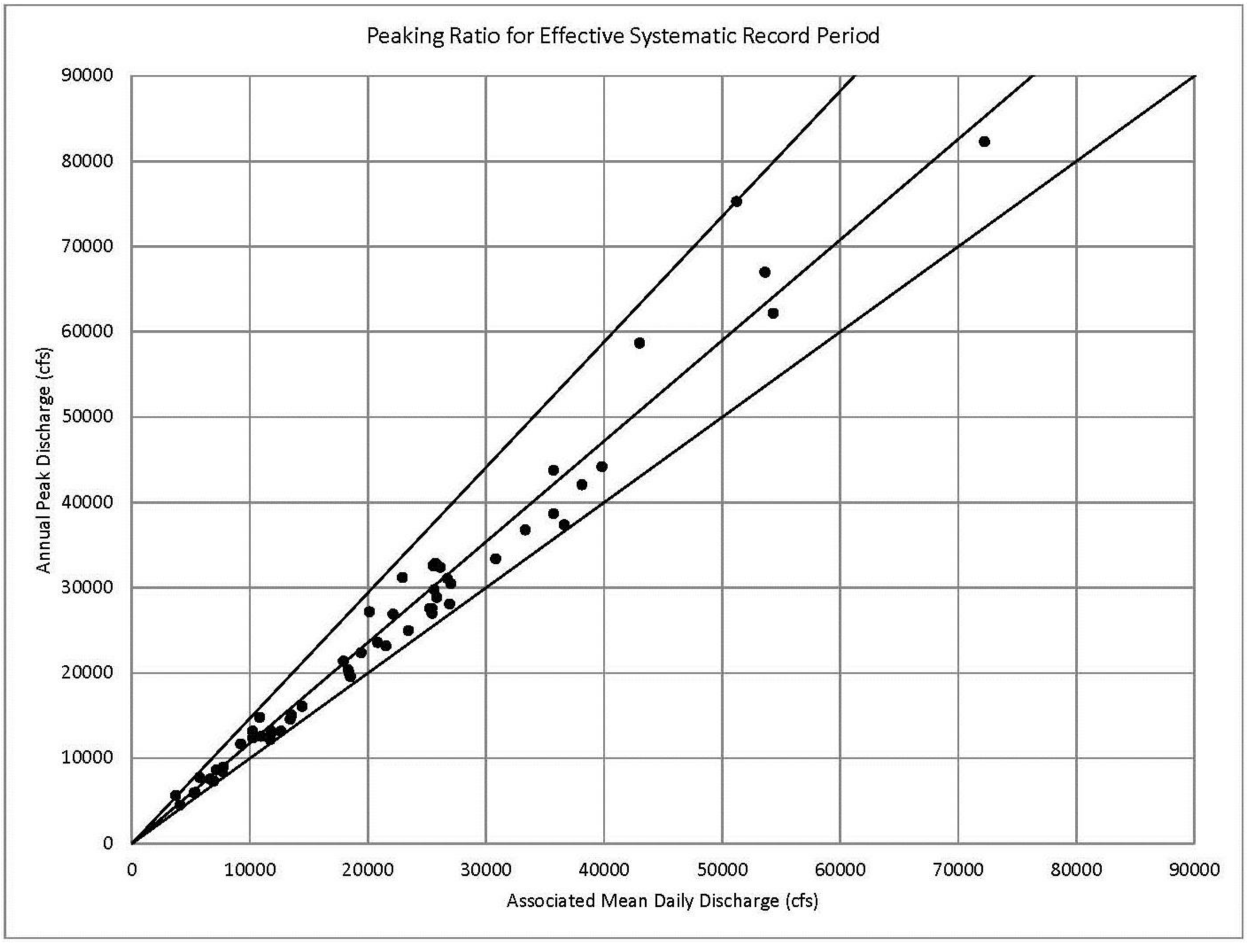


Figure 3-1 USGS Daily and Instantaneous Peak Flows from 1955-2011

Since there was USGS data for most years in the period of record, the ratios applied to the RiverWare simulated flows came from the actual observed USGS data rather than using the average ratio for the 1955-2011 period. For every maximum annual daily flow at the Dallas gage (both regulated and unregulated) research was completed to find the daily and instantaneous peak flows for that same event in the USGS historical records. The ratios calculated from the historical observed USGS data were applied to the calculated RiverWare flows at Dallas for that same event. USGS data was found for 68 of the 70 years for the RiverWare flows. For the years that did not have historical data, an average ratio was used.

3.1.4 Frequency Analysis

A Hydrologic Engineering Center – Statistical Software Package Bulletin 17B analysis was performed on the unregulated maximum annual instantaneous peak flows discussed in Section 3.1.3.2 of this report. Median plotting position and the default confidence limits (5% and 95%) were selected for the analysis. The analysis calculated the station skew, mean, and standard deviation for the computed curve as -0.362, 4.619, and 0.386, respectively. Table 3-1 shows the Bulletin 17B tabular results and Figure 3-2 shows the plotted curves.

Table 3-1 Bulletin 17B Tabular Results for Unregulated Flows at Dallas Gage

Frequency Curve for: Dallas INST-UNREG Flow-DALT2-FLOW				
Percent Chance Exceedance	Computed Curve Flow in cfs	Expected Prob. Flow in cfs	Confidence Limits Flow in cfs	
			0.05	0.95
0.2	365152.7	392103.5	545007.7	266864.7
0.5	303525.7	321088.6	441227.3	226080.2
1.0	258933.9	271121.8	368132.8	195950.9
2.0	216211.1	224140.8	299925.5	166488.6
5.0	162779.5	166780.5	217605.6	128600.0
10.0	124765.1	126753.6	161518.8	100705.9
20.0	88820.4	89596.9	110936.2	73306.4
50.0	43849.3	43849.3	52378.2	36787.7
80.0	20066.8	19842.1	24273.0	16113.3
90.0	12927.1	12638.6	16100.0	9905.8
95.0	8843.6	8524.9	11377.1	6461.8
99.0	4164.6	3830.1	5775.4	2739.1

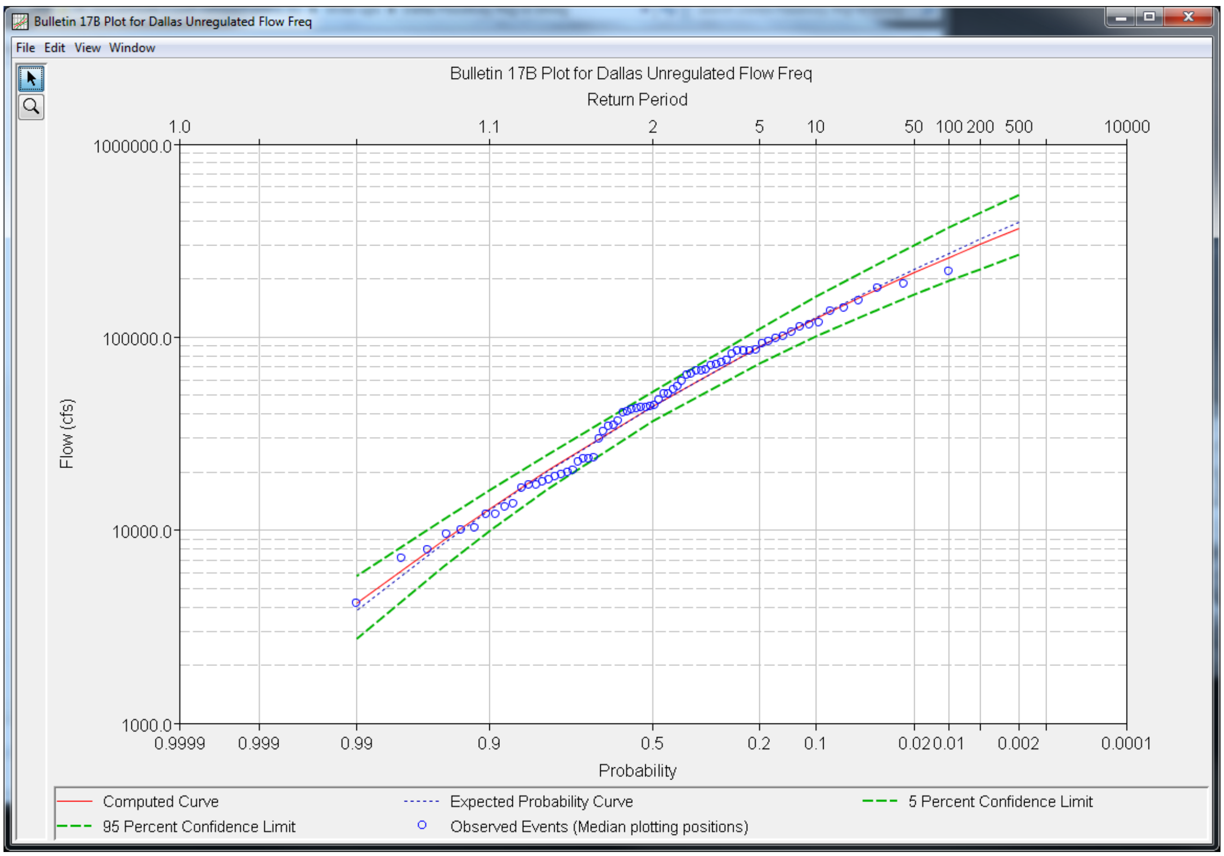


Figure 3-2 Bulletin 17B Plot of Unregulated Flows at Dallas Gage with Station Skew

The station skew calculated by Bulletin 17B for the unregulated flows at Dallas was -0.362. The regional skew for the Trinity River basin is 0.0 according to a study performed by Leo R. Beard and documented in the report titled “Generalized Skew Coefficients of Annual Maximum Streamflow Logarithms in Southwestern Division, Corps of Engineers”, dated March 1978. To evaluate the sensitivity of the final discharge frequency curve results to the skew applied to the Bulletin 17B computed curve, an additional

17B analysis was performed using the regional skew. All other parameters remaining the same, Figure 3-3 shows the computed curve utilizing the regional skew. The computed curve calculated using the regional skew had a weighted skew, mean and standard deviation of -0.273, 4.619, and 0.386, respectively.

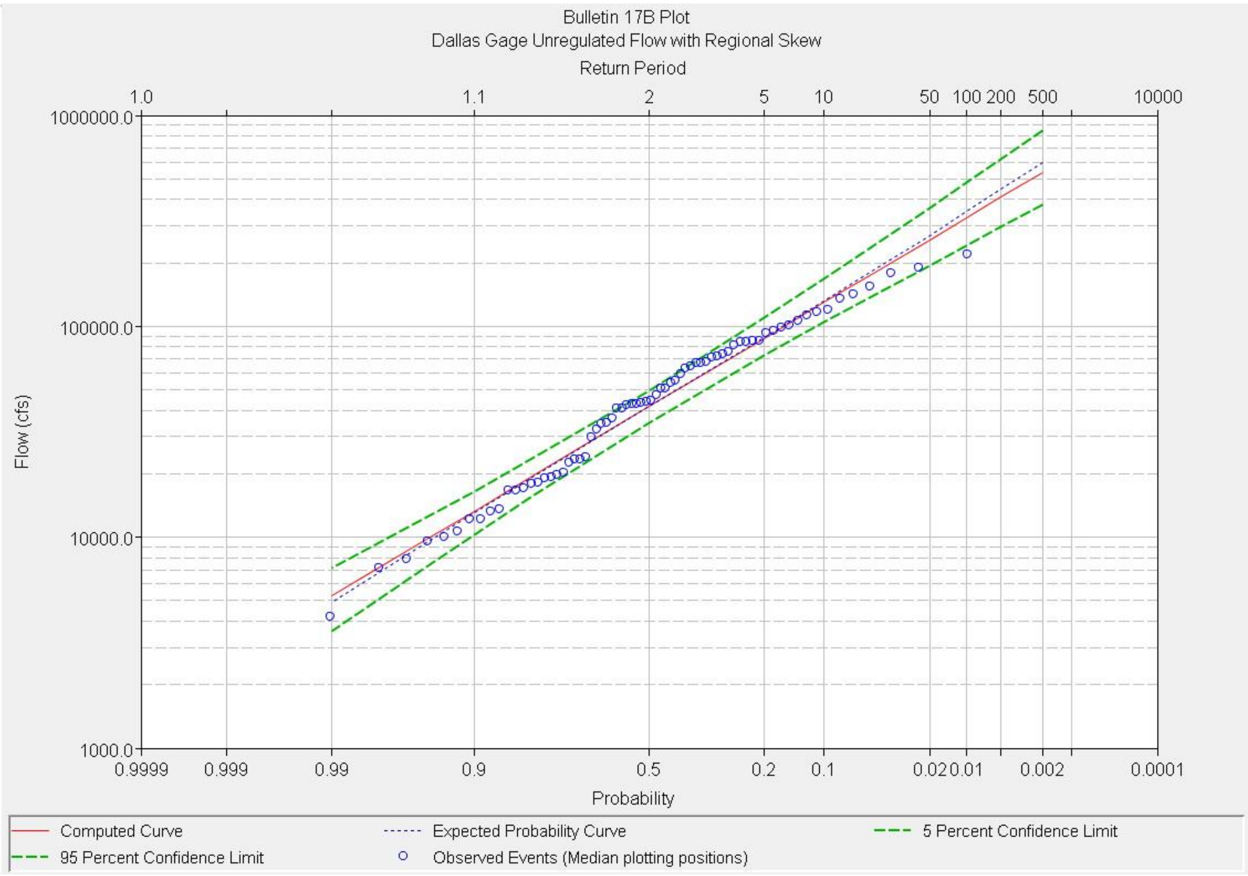


Figure 3-3 Bulletin 17B Plot of Unregulated Flows at Dallas Gage with Regional Skew

The computed curve for each analysis was re-created in Microsoft Excel in order to create a trendline that best fit the computed curves. The trendline equation for the computed curve based on the station skew was:

$$y = -4.66x^6 - 7.57x^5 + 294.61x^4 + 3{,}004.21x^3 + 14{,}746.84x^2 + 38{,}720.55x + 43{,}850.84$$

The trendline equation for the computed curve based on the regional skew was:

$$y = 41{,}572.03e^{0.89x}$$

Given a probability, the equations can be used to calculate the unregulated flow at the Dallas gage. The interpolated flow can then be utilized with the regulated versus unregulated relationship developed as part of this study.

3.1.5 Regulated versus Unregulated Relationship

3.1.5.1 RiverWare Period of Record Flows

Following the guidance of Chapter 3 of EM 1110-2-1415 a plot was developed to graphically relate the regulated and unregulated flows at the Dallas gage. The 70 years of regulated and unregulated RiverWare simulated flows were plotted on a log/log plot. Each point on the plot represents a flow that occurred during the same year, so that the regulated and unregulated flows for 1990 make up one point. Figure 3-4 shows the 1940-2009 RiverWare simulated flows that were peaked to instantaneous flows plotted on a log/log axis. The May 1990 storm is highlighted.

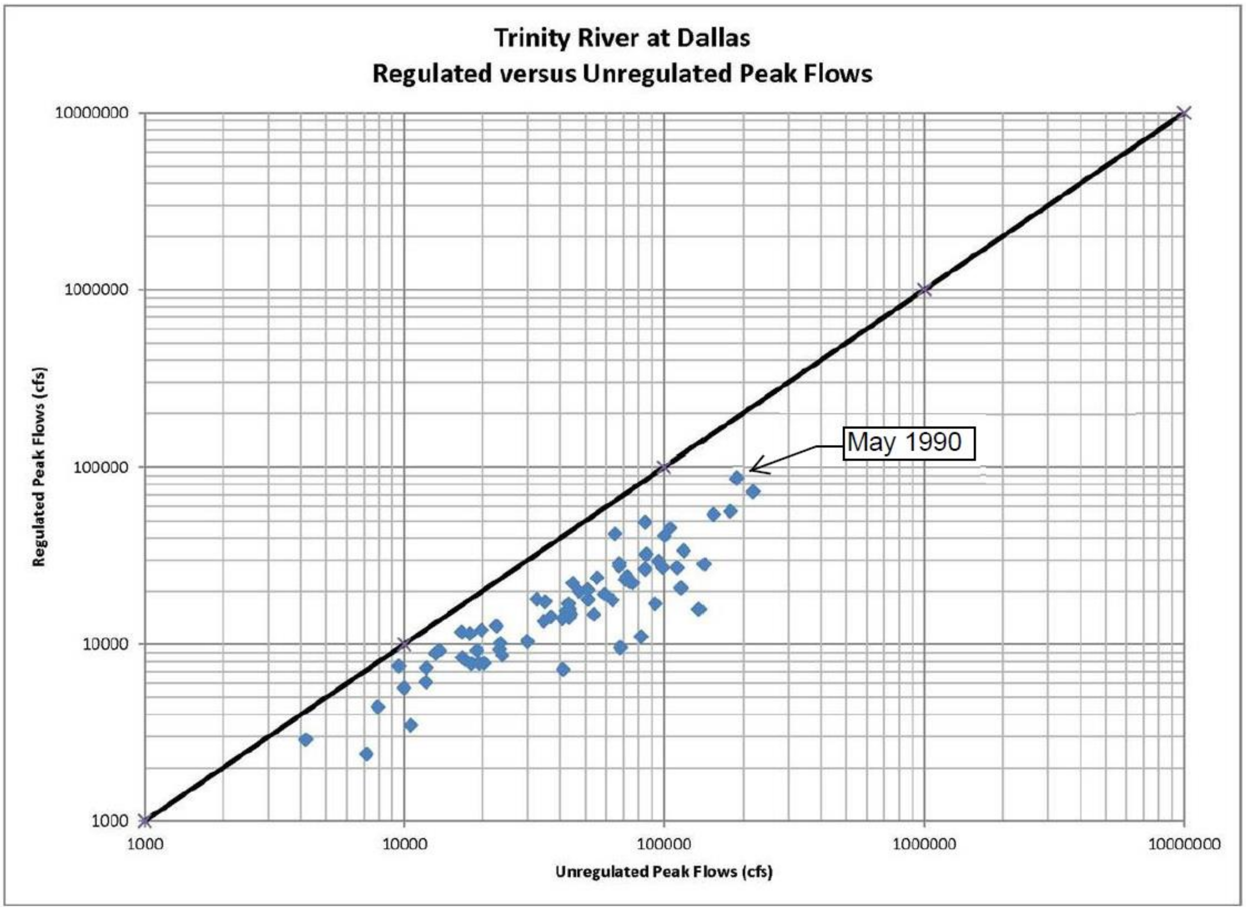


Figure 3-4 Regulated Versus Unregulated Dallas Gage Instantaneous Flows from RiverWare Simulations

A trendline through the points was required to complete the analysis. Due to the data being more scattered than uniformly plotted, the regulated and unregulated flows were each ranked from highest to lowest. The plot was re-created using the sorted flows. Figure 3-5 shows the regulated versus unregulated flows from the sorted data. Ranking the regulated and unregulated flows from highest to lowest suggested more of a relationship.

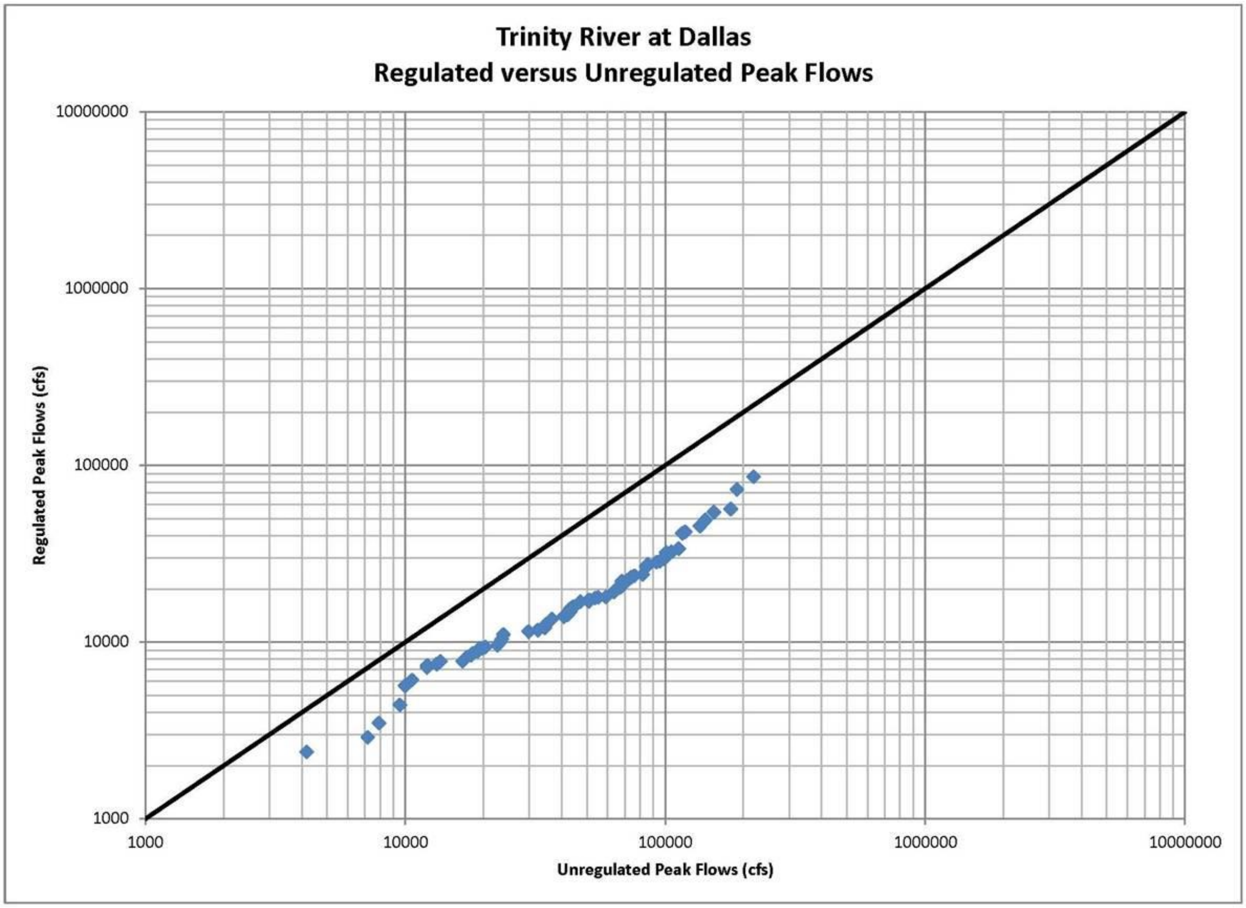


Figure 3-5 Regulated Versus Unregulated Dallas Gage Instantaneous Flows from RiverWare Simulations Sorted from Highest to Lowest

3.1.5.2 HEC-1 Synthetic Storm Events

The RiverWare period of record simulated data covers a limited range of flows leaving a gap in the regulated versus unregulated relationship. Calibrated HEC-1 models were used to develop synthetic storm events that produced greater peak flows at the Dallas gage than have occurred historically. The May 1990 and May 1989 calibrated HEC-1 models were selected for this analysis. The analysis increased the known rainfall for the historical storm events by various factors. Six synthetic storm events were simulated from the May 1989 and the May 1990 calibrated HEC-1 models. The May 1989 rainfall was increased by factors of 2, 3, 4, 5, 7.5, and 10. The May 1990 rainfall was increased by factors of 1.5, 1.75, 2, 3, 5, and 10. The new calculated flows were added to the period of record flows. The data was plotted again, both unsorted and sorted as previously explained. Figures 3-6 and 3-7 show the plots for the regulated versus unregulated flows with the HEC-1 results added to the period of record results.

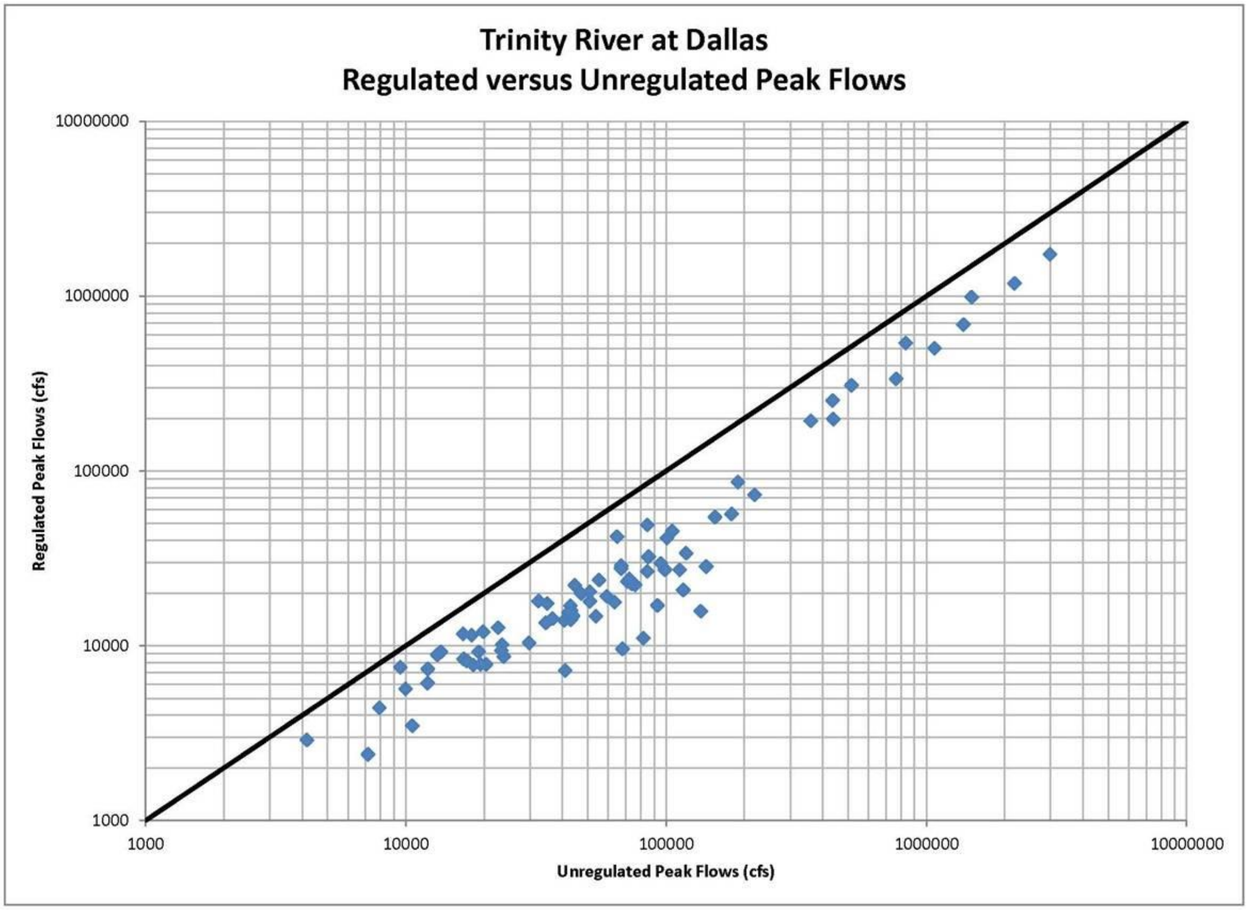


Figure 3-6 Regulated Versus Unregulated Dallas Gage Instantaneous Flows from RiverWare Simulation and 1989 and 1990 HEC-1 Synthetic Storms

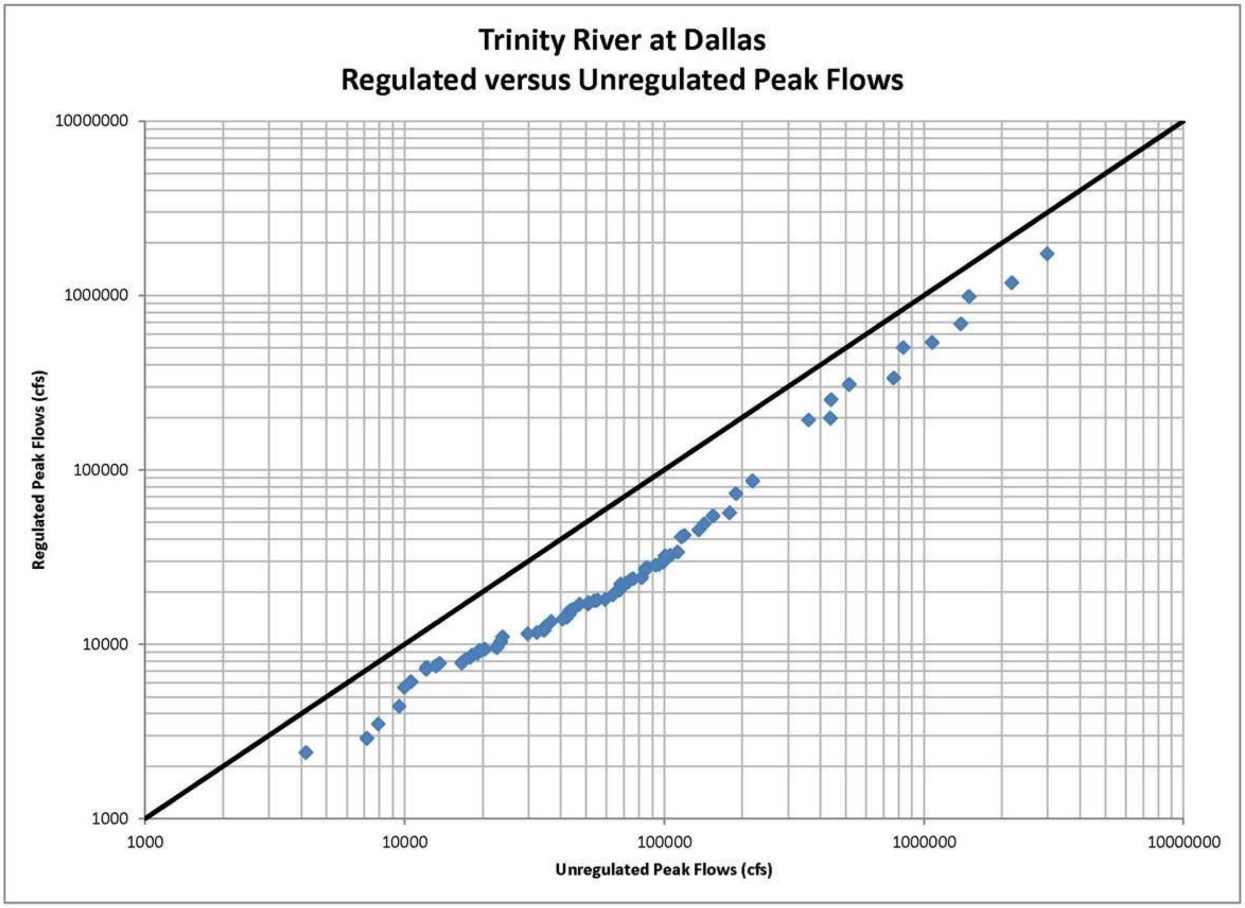


Figure 3-7 Regulated Versus Unregulated Dallas Gage Instantaneous Flows from RiverWare Simulated and 1989 and 1990 HEC-1 Synthetic Storms sorted from Highest to Lowest

3.1.5.3 Multiple Parameter Visualization Tool Synthetic Storms

The Multiple Parameter Visualization Tool was used to create 40 additional data points for the regulated versus unregulated flow plot. The Multiple Parameter Visualization Tool was used to create random storm events from the May 1990 storm. The 2-4 May 1990 rainfall file that contains the three day totals for 85 precipitation gages was used in Multiple Parameter Visualization Tool Synthetic to create a rainfall tin. Multiple Parameter Visualization Tool storm transposition tools were used to find a storm centering and rotation and increase the factored rainfall amount for each storm event.

Hyetographs created from 24-hour gages were applied to the rainfall tin in the Multiple Parameter Visualization Tool and the hourly basin average precipitation amounts were calculated.

After the 40 storm events were created, the hourly basin average precipitation for each storm event was simulated in the regulated and unregulated HEC-1 models. The additional flows were added to the regulated and unregulated flow lists and plotted again both unsorted and sorted. Figures 3-8 and 3-9 show all the points for the regulated versus unregulated flows plotted for the unsorted and sorted data, respectively. With all of the data plotted, a trendline was fit to the data in Microsoft Excel. Because of the number of points, Excel calculated an equation that was approximately a straight line through the upper and lower points. This created a falsely high discharge for the 100-year recurrence interval. Two trendlines were created to fit the data from the low to the middle points and the high middle points. The trendlines intersected at the transition from the RiverWare simulated points to the synthetic storm points. The two equations for the trendlines are:

Lower Trendline: $y = (1E - 18x^4) – (2E - 12x^3) + (2E - 06x^2) + (0.144x) + 5000$

Upper Trendline: $y = (6E - 09x^2) + (0.6069x) - 43000$

These trendlines are plotted on the data in Figure 3-9.

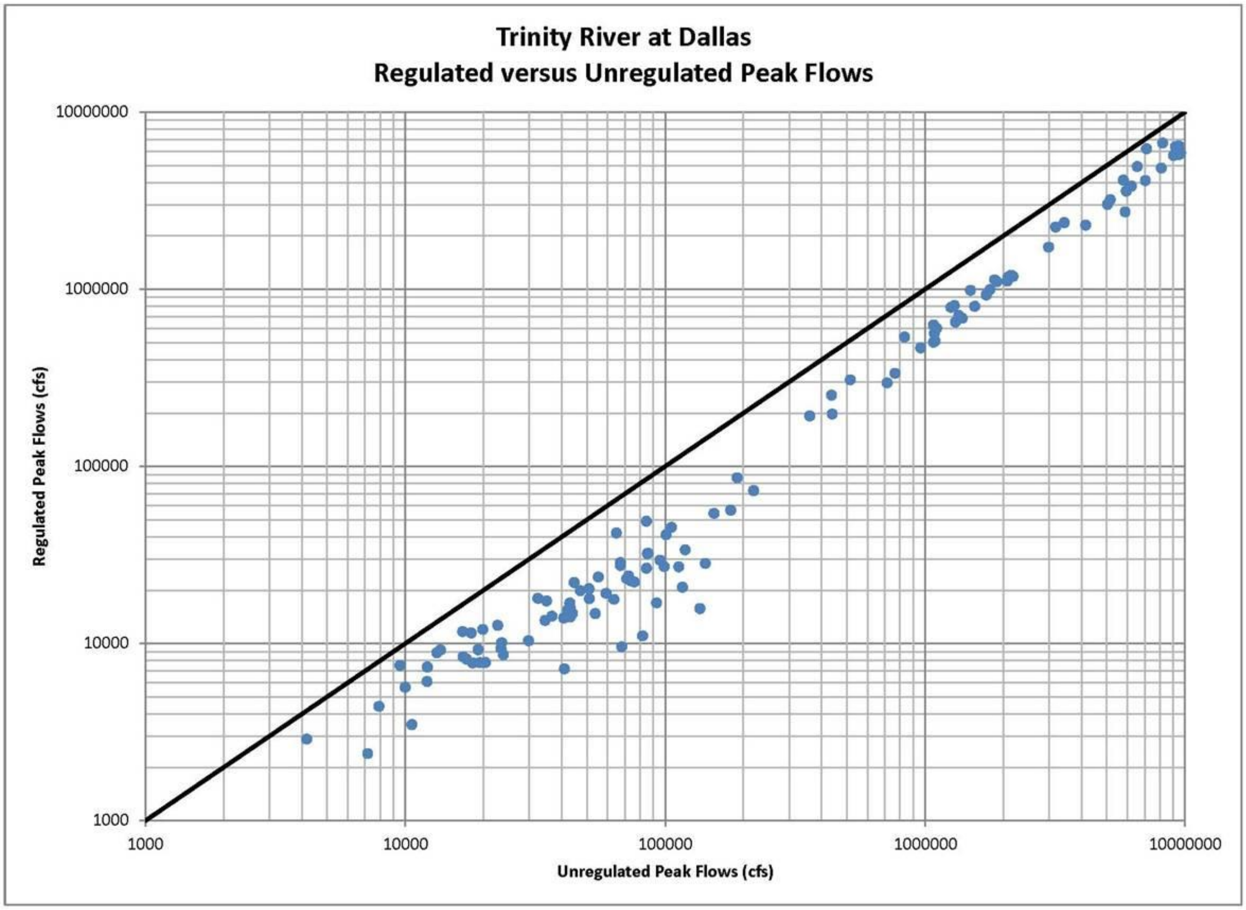


Figure 3-8 Regulated Versus Unregulated Dallas Gage Instantaneous Flows from RiverWare Simulation, 1989 and 1990 HEC-1, and Multiple Parameter Visualization Tool Synthetic Storm Events

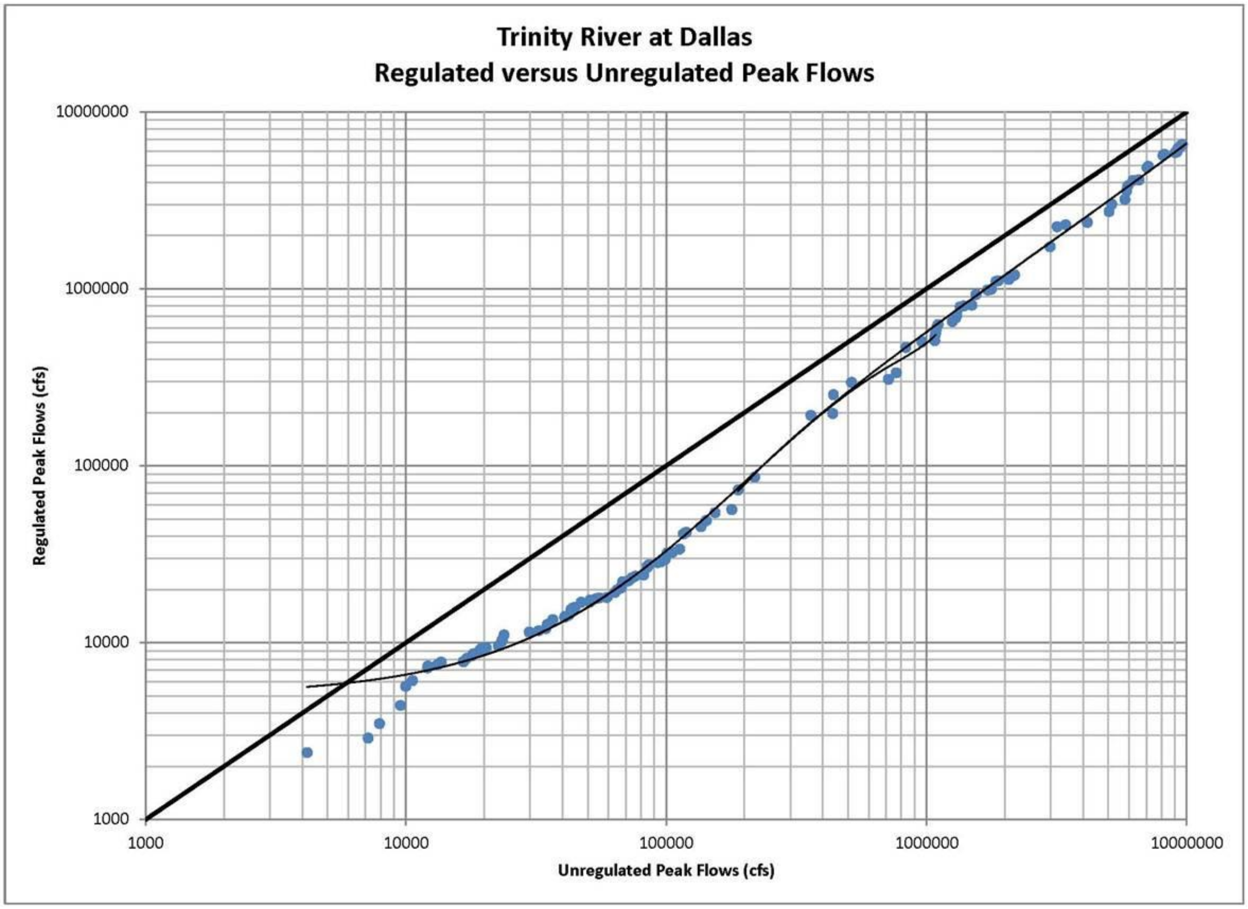


Figure 3-9 Regulated Versus Unregulated Dallas Gage Instantaneous Flows from RiverWare Simulation, 1989 and 1990 HEC-1, and Multiple Parameter Visualization Tool Synthetic Storm Events Sorted from Highest to Lowest

3.1.6 Regulated versus Unregulated Study Results

3.1.6.1 Trinity River at Dallas Gage Discharge Frequency Curve

The Trinity River at Dallas gage discharge frequency curve is a function of the regulated versus unregulated relationship. The regulated versus unregulated peak flows plot provides the connecting relationship to calculate a discharge frequency curve at the Dallas gage. The starting point in developing the discharge frequency curve begins with the Bulletin 17B unregulated flow computed curve. Using the equation for the computed curve based on the station skew in Section 3.1.4, the unregulated flows that are used to determine the regulated flow were calculated given a set of probabilities. The unregulated flows calculated from the computed curve were applied to the equations determined from the regulated versus unregulated peak flow graph. Working through the Bulletin 17B curve equation and the regulated versus unregulated peak flow equation, the equivalent regulated flow was calculated. This process builds the regulated versus unregulated relationship so that an unregulated flow for a given probability was related to a regulated flow for the same probability. Working through the set of probabilities, the Trinity River at Dallas gage discharge frequency curve was created. Figure 3-10 shows the discharge frequency curve and Table 3-2 lists the flows (rounded to the nearest thousand).

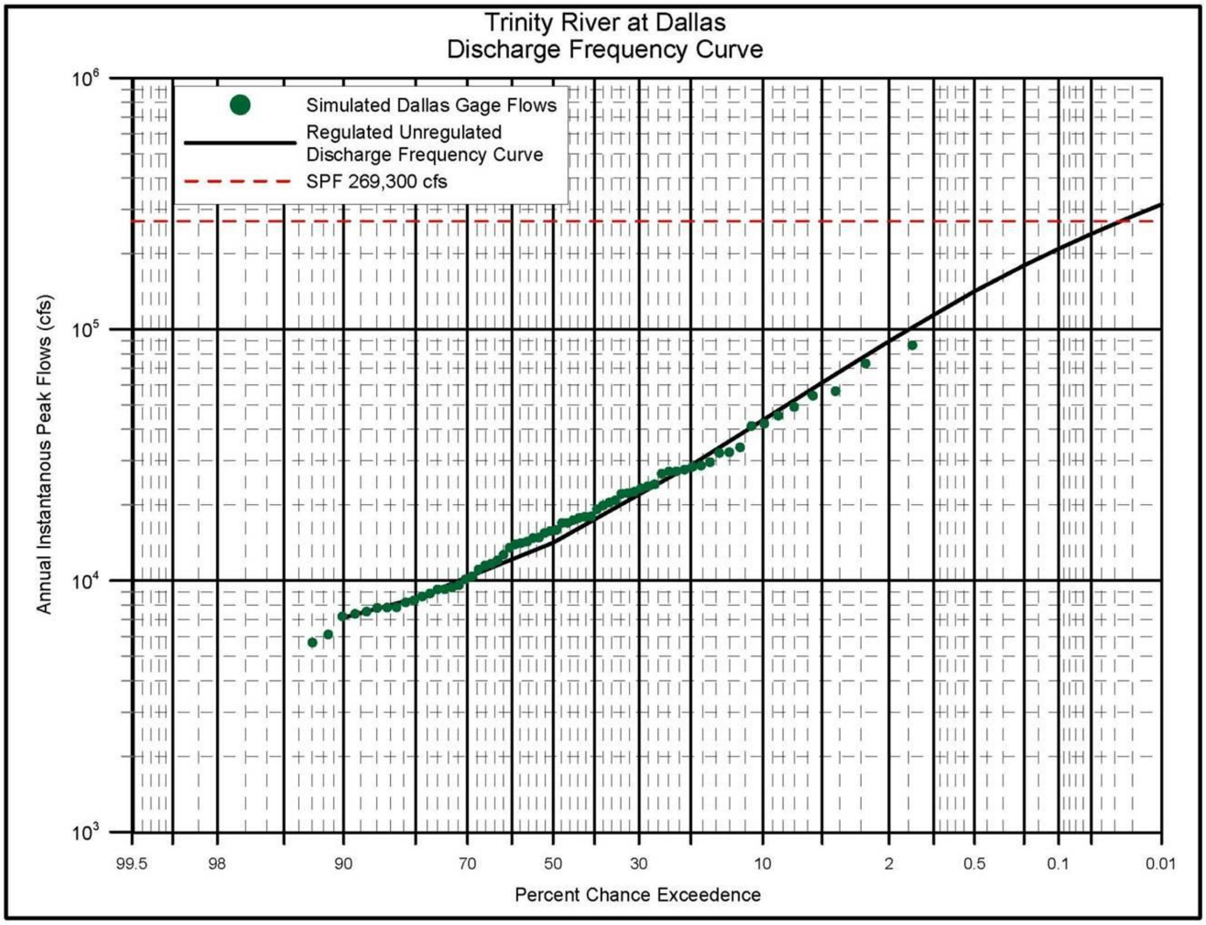


Figure 3-10 Trinity River at Dallas Gage Discharge Frequency Curve

Table 3-2 Trinity River at Dallas Gage Discharge Frequency Curve

Probability	*Flow(cfs)*
0.0001	314,000
0.0002	281,000
0.000261	269,300
0.0005	240,000
0.001	209,000
0.002	179,000
0.005	142,000
0.01	115,000
0.02	90,000
0.05	61,000
0.1	43,000
0.2	29,000
0.5	14,000
0.8	9,000
0.9	7,000

The discharge frequency curve determined through the regulated versus unregulated study fit the RiverWare simulated regulated instantaneous flows at the Dallas gage well. The historical Dallas gage instantaneous maximum annual flows and the Bulletin 17B computed curve calculated from those historical flows were plotted with the new discharge frequency curve in Figure 3-11. The extreme upper and lower flows match reasonably well, however, the historical flows tend to be higher than the RiverWare simulated flows. It should be noted again that the RiverWare model simulates current conditions of the watershed, which includes all USACE reservoirs built and the greater demands on water supply.

The results of the regulated versus unregulated study calculated the 100-year recurrence interval at approximately 115,000 cfs and the SPF flow of 269,300 cfs at slightly less than a 4000-year recurrence interval, 0.000261 probability.

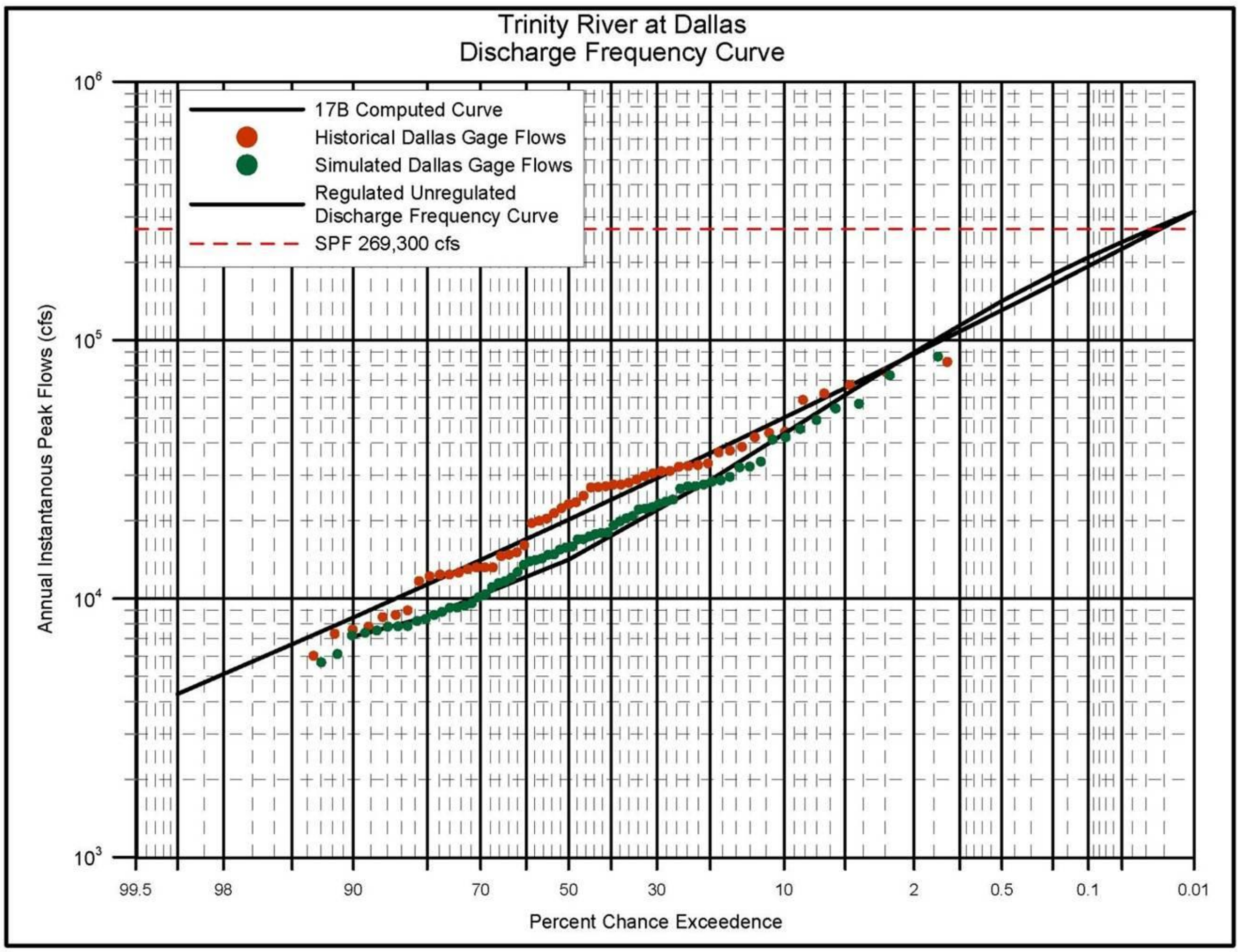


Figure 3-11 Trinity River at Dallas Gage Discharge Frequency Curve and Bulletin 17B Computed Curve for Historical Flows

3.1.6.2 Sensitivity Analysis

A sensitivity analysis was performed during this regulated versus unregulated study. A comparison of the Dallas gage discharge frequency curve results between the Bulletin 17B computed curves with the station skew and regional skew was conducted. Figure 3-12 shows the two different Dallas gage discharge frequency curves based on the different computed curves. Table 3-3 shows the actual values calculated for the range of probabilities. The discharge frequency curves differ greatly showing that the final discharge frequency curve for the Dallas gage is very sensitive to the skew used in the Bulletin 17B frequency analysis. The 100-year recurrence interval is 115,000 cfs for the computed curve calculated with the station skew and 157,000 cfs for the computed curve calculated with the regional skew. The Trinity River at Dallas gage SPF flow of 269,300 cfs has a recurrence interval of 3,831 for the station skew computed curve compared to a 425-year recurrence interval for the regional skew computed curve.

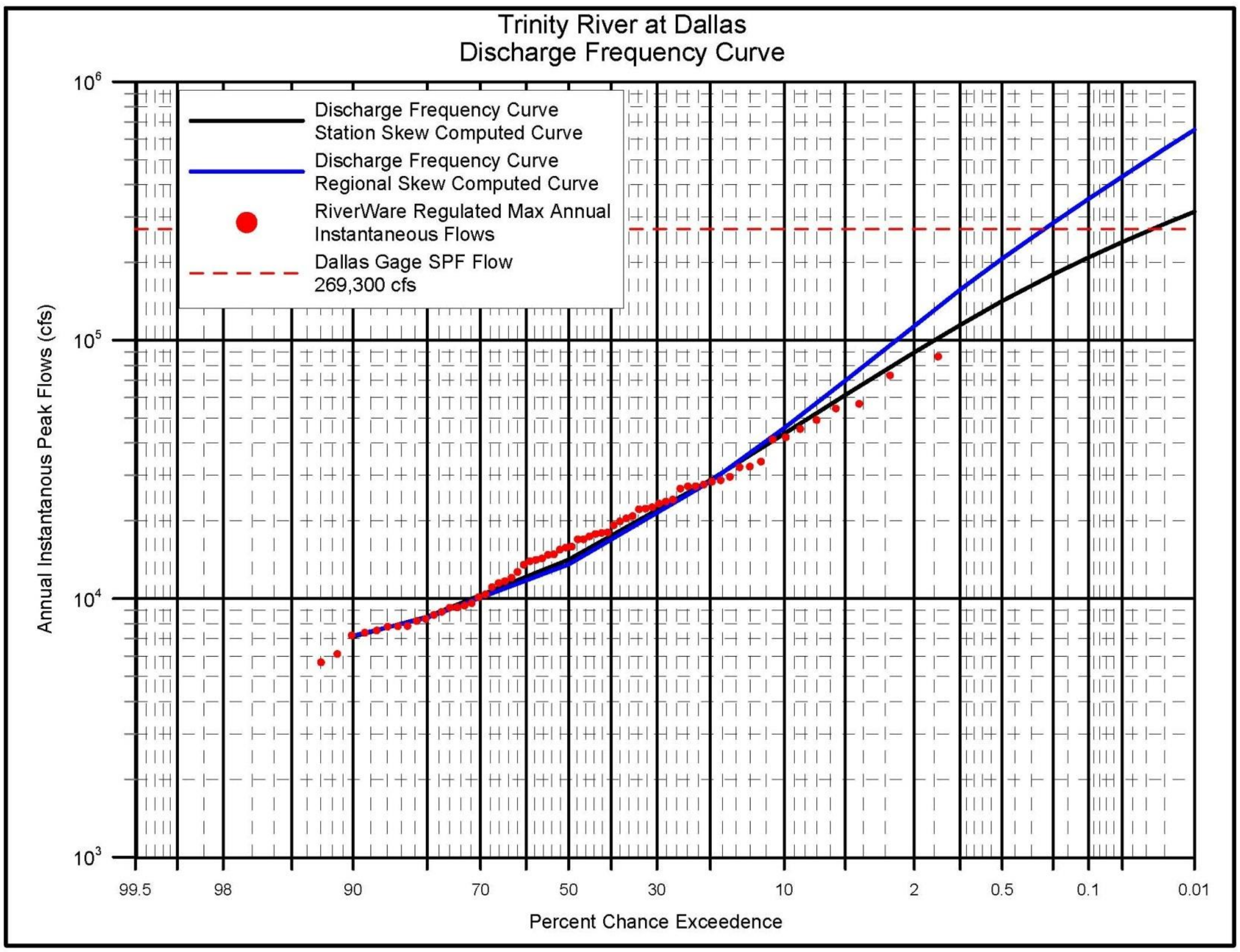


Figure 3-12 Trinity River at Dallas gage Discharge Frequency Curve Comparison of Bulletin 17B Computed Curves

Table 3-3 Trinity River at Dallas Gage Discharge Frequency Curve Comparison of Bulletin 17B Computed Curves

Probability	*Flow (cfs) (using station skew)*	*Flow (cfs) (using regional skew)*
0.0001	314,000	652,000
0.0002	281,000	549,000
0.000261	269,300	511,000
0.0005	240,000	430,000
0.001	209,000	353,000
0.002	179,000	284,000
0.00235	172,000	269,300
0.005	142,000	207,000
0.01	115,000	157,000
0.02	90,000	113,000
0.05	61,000	70,000
0.1	43,000	46,000
0.2	29,000	28,000
0.5	14,000	14,000
0.8	9,000	9,000
0.9	7,000	7,200

3.2 Design Storm Study

3.2.1 Introduction and Purpose of Design Storm Study

The Design Storm study was completed as part of the Dallas Floodway Feasibility Study in order to calculate the flow at the Trinity River at Dallas gage for design storms having recurrence intervals of 2, 5, 10, 25, 50, 100, and 500-years. Design storms for each recurrence interval were developed based on data from a collection of historical storms that have occurred in the Trinity River Basin region. The calculated discharges at the Dallas gage for each design storm would then be used in defining the Dallas gage discharge frequency curve.

As part of this analysis, the process to develop the design storms was developed. The parameters evaluated in constructing the design storms were the storm duration, depth-duration, depth-area relationship, and the spatial distribution and temporal distribution of the precipitation.

3.2.2 In-Progress Review Team

The Fort Worth District invited experts in hydrologic engineering to participate in the Design Storm Study from its inception. They provided input to the scope of work and remained engaged throughout the study via conference calls and webinars.

3.2.3 Storm Duration

Times of concentration were analyzed for reaches extending from the headwaters of the Trinity River to the Trinity River at Dallas gage. This analysis was performed to determine what the longest times of concentration were for the basin. There were no times of concentration greater than 24 hours. Based on this and the historical knowledge that the runoff hydrograph reaches the Dallas gage within 24 hours, a storm duration of 24 hours was used for the design storms.

3.2.4 Development of Depth-Duration Data

Twenty-four hour depth-duration data from National Weather Service Technical Paper 40 (TP40) was used for each of the design storms. Table 3-4 lists the point rainfall amounts that were used for each of the design storms.

Table 3-4 TP-40 Point Rainfall for each Design Storm Frequency Simulation

Return Period (year)	2	5	10	25	50	100
TP-40 (in)	4.0	5.33	6.43	7.54	8.55	9.55

3.2.5 Development of Depth-Area Relationships

3.2.5.1 Historical Storms

Available storm data for the Texas and Oklahoma region was collected. The focal point of the data collection was to find an appropriate generalized set of depth-area relationships for the region. The majority of the data collected came from a study by the USACE, War Department report, “Storm Rainfall in the United States – Depth-Area-Duration Data”, 1945. Other storm data was collected from the USACE Extreme Storm Team and from a study performed for the Tarrant Regional Water District (TRWD) by Applied Weather Associates, LLC, documented in the report, “Site-Specific Probable Maximum Precipitation Study for the Tarrant Regional Water District,” dated March 2011.

Depth-area relationships have been published by the National Weather Service in TP-40 based upon data from climatic gaging station networks ranging upwards to 400 square miles. Extrapolation of these relationships for areas far beyond 400 square miles has been questioned. Since about 1000 square miles of the effective drainage area for the Trinity River at the Dallas Floodway is uncontrolled, extended depth-area relationships were developed to cover areas significantly larger than 400 square miles.

3.2.5.2 Depth-Area Reduction Factors

In order to develop the rainfall file for the Multiple Parameter Visualization Tool, some additional depth-area calculations were needed. First, the depth-area reduction factors were calculated from the historical storms. The reduction factors represent the percent reduction of each historical storm’s precipitation amount from the point rainfall to each area. An example for the historical storm at Weatherford, TX, that occurred on 25 April 1922 is shown in Table 3-5.

Table 3-5 Example Reduction Factor Calculation

Weatherford Texas 25 April 1922 24 Hour Peak Precipitation Over a Given Area						
10 (mi^2)	100 (mi^2)	200 (mi^2)	500 (mi^2)	1,000 (mi^2)	5,000 (mi^2)	10,000 (mi^2)
8.9 in	8.3 in	8.2 in	8.0 in	7.6 in	5.1 in	4.0 in
Reduction Factor	0.933	0.921	0.899	0.854	0.573	0.449

This calculation was completed for all of the historical storms collected. The median, low, and high band reduction factors were calculated from 36 historical storms. Only storms that had point rainfall amounts in the range of the TP-40 24-hour, 2 through 100-year rainfall amounts were used to calculate the reduction factors. This created a precipitation range from 5 inches to 11 inches for the reduction factors.

The low and high bands of reduction factors are the collection of the lowest and highest reduction factors for each area. The low and high bands are not from one historical storm. Figure 3-13 displays the reduction factor bands for the 5-11 inch rainfall bands along with the HMR-52 reduction factors. Figure 3-14 shows the same bands clipped at the 1000 square mile point on the x-axis.

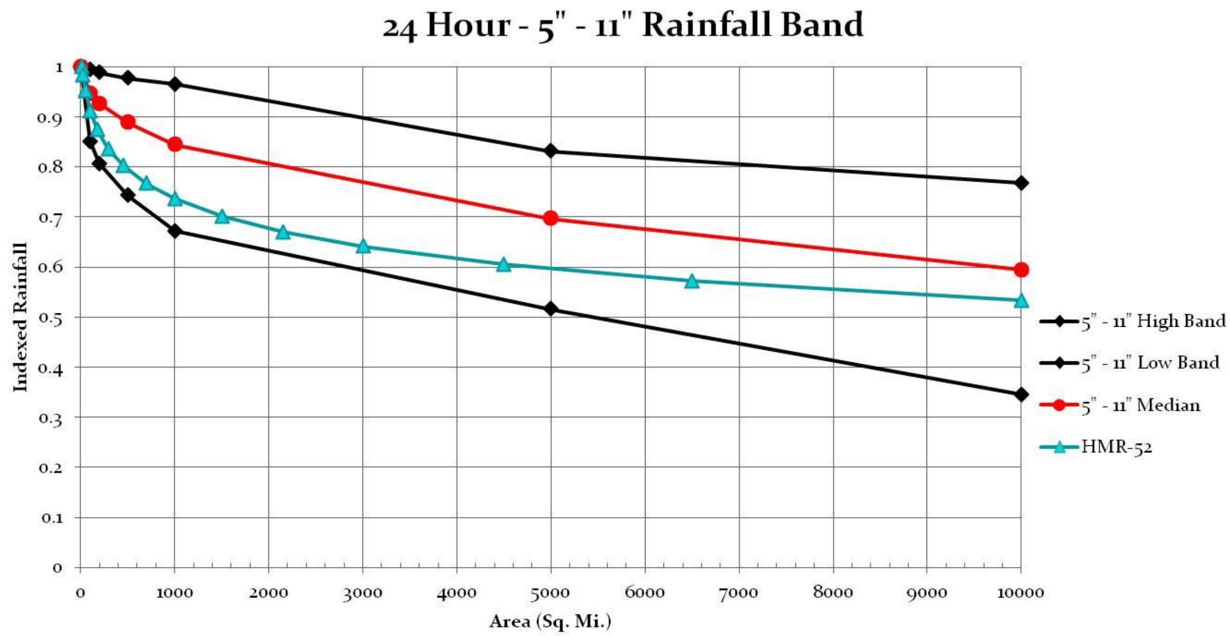


Figure 3-13 Historical Storm Depth-Area Reduction Factors for 5-11 Inch Bands (0-10,000 sq mi)

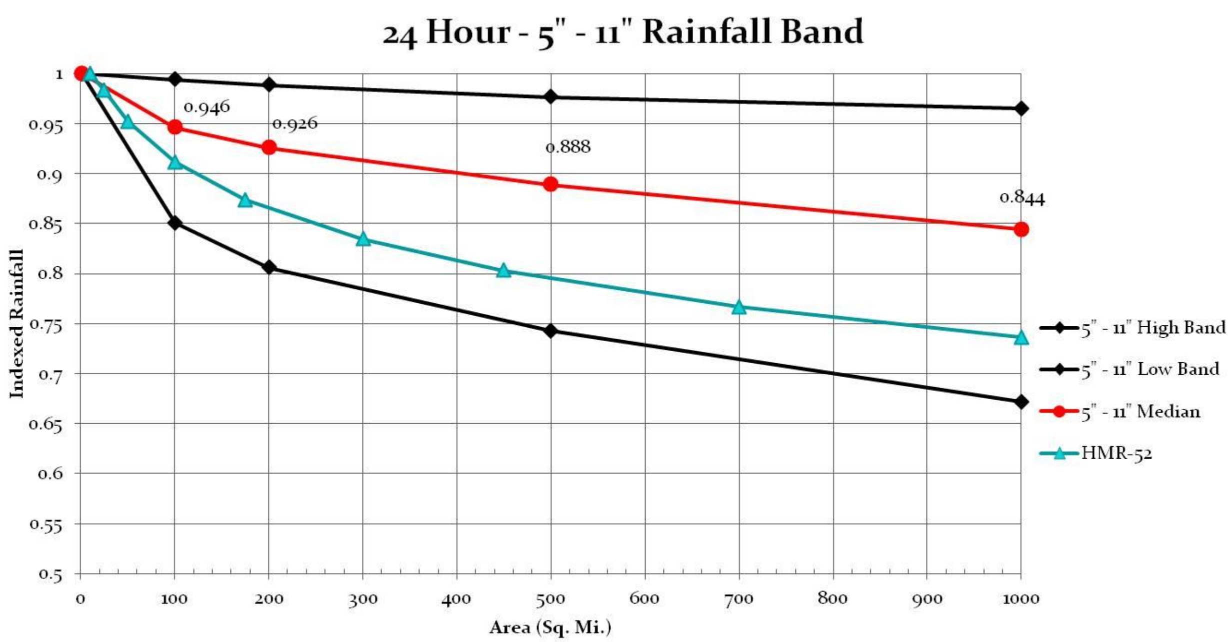


Figure 3-14 Historical Storm Depth-Area Reduction Factors for 5-11 Inch Bands (-1000 sq mi)

3.2.5.3 Isohyetal Rainfall Depths

The first step in determining rainfall depths for each Isohyetal was to apply the depth-area reduction factors to the TP-40 24-hour point rainfall to get the total depth in inches for each area. These depths were then converted to a total volume for each area. The incremental rainfall depths were then calculated by taking the difference in total volumes between two areas and dividing that volume by the incremental area of the two areas. With these calculations completed, the values for the isohyetal lines could be determined. The individual isohyetal rainfall values were calculated by using an average of the incremental area and incremental rainfall. With the isohyetal lines assigned rainfall values in inches, a rainfall tin was developed in the Multiple Parameter Visualization Tool. The temporal distribution was applied to this rainfall file to create the hourly subbasin rainfall DSS file. Figure 3-15 is an outline of the isohyetal line calculations from the total area isohyetal polygons. Table 3-6 shows the rainfall values calculated and assigned to the isohyetal lines.

Example: Calculation for the 10, 25, and 50 sq mi HMR-52 isohyetals.

TP40-100 Year Point Rain = 9.55 in

Total Rainfall Depth and Volume

The 10 sq mi has a reduction factor of 1.0, a total rainfall for the area of 9.55 in, and a total volume for the area of 95.5 sq-mi-in.

The 25 sq mi has a reduction factor of 0.991, a total rainfall for the area of 9.46 in, and a total volume for the area of 236.6 sq-mi-in.

The 50 sq mi has a reduction factor of 0.965, a total rainfall for the area of 9.22 in, and a total volume for the area of 460.8 sq-mi-in.

Incremental Rainfall Depths

1st incremental rainfall depth for an area --- (95.5 – 0.0) sq-mi-in = 95.5 sq-mi-in

95.5 sq-mi-in / 10 sq-mi = 9.55 in

(The first incremental area has no change as the TP-40 point rain is assumed to be for the 10 sq-mi area.)

2nd incremental rainfall depth --- (236.6 – 95.5) sq-mi-in = 141.1 sq-mi-in

141.1 sq-mi-in / (25 – 10) sq-mi = 9.41 in

3rd incremental rainfall depth --- (460.8 – 236.6) sq-mi-in = 224.2 sq-mi-in

224.2 sq-mi-in / (50 – 25) sq-mi = 8.97 in

Isohyetal Line Rainfall Values

10 sq-mi Iso Line --- (As this is a flat plain, the rainfall depth for the first isohyetal line is 9.55 in)

25 sq-mi Iso Line ---

[9.41 in * (25 – 10) sq-mi] + [8.97 in * (50 – 25) sq-mi] / [(50 – 25) sq-mi + (25 – 10) sq-mi] = 9.13 in

50 sq-mi Iso Line ---

[8.97 in * (50 – 25) sq-mi] + [8.85 in * (100 – 50) sq-mi] / [(100 – 50) sq-mi + (50 – 25) sq-mi] = 8.89 in

Table 3-6 Calculated Rainfall Amounts in Inches for Each Isohyetal Line for the 100 Year Median Band

Area of Isohyetal	Point Rainfall	10	25	50	100	175	300	450	700	1000	1500	2150	3000	4500	6500
Reduction Factor	Median Band	1.0	.991	.965	.946	.931	.913	.895	.868	.844	.814	.783	.753	.709	.665
Reduced TP-40 Rainfall		9.55	9.46	9.22	9.03	8.89	8.72	8.55	8.29	8.06	7.77	7.48	7.19	6.77	6.35
Isohyetal Rainfall		9.55	9.13	8.89	8.76	8.65	8.37	8.03	7.66	7.36	6.98	6.67	6.21	5.69	5.04

This process was continued until each of the HMR-52 isohyetals was assigned a rainfall depth. The Multiple Parameter Visualization Tool program took the shape file with the assigned isohyetal lines and created a tin that solves for every given point between the isohyetal lines to calculate the volume for each concentric isohyetal donut (polygon). Mathematical checks were completed to verify that the expected incremental volumes as well as the cumulative volumes were being calculated. Table 3-7 shows a comparison between the expected volumes of each incremental area and the cumulative areas compared to the volumes calculated by the Multiple Parameter Visualization Tool given the isohyetal lines assigned rainfall depths.

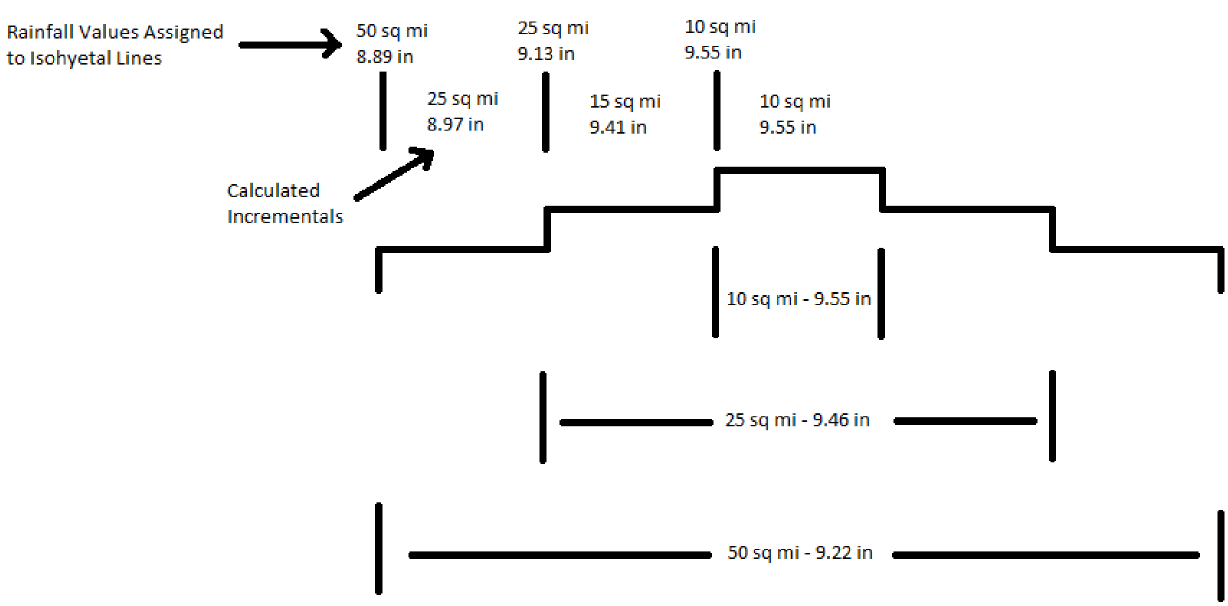


Figure 3-15 Isohyetal Line Calculation from Isohyetal Polygons

Table 3-7 TP-40 100 Year 24-Hour Median Depth-Area Reduction Factors

100 Year-24 Hour Median Depth-Area Reduction						
Area	MPVT Rainfall Volume		Expected Rainfall Volume		% Difference	
	Incremental	Cumulative	Incremental	Cumulative	Incremental	Cumulative
10	5,093	5,093	5,093	5,093	**0.01%**	**0.01%**
25	7,536	12,629	7,525	12,619	**-0.14%**	**-0.08%**
50	12,133	24,762	12,236	24,854	**0.85%**	**0.37%**
100	23,541	48,303	23,324	48,178	**-0.92%**	**-0.26%**
175	34,692	82,995	34,795	82,974	**0.30%**	**-0.03%**
300	56,529	139,524	56,619	139,592	**0.16%**	**0.05%**
450	65,706	205,230	65,558	205,151	**-0.22%**	**-0.04%**
700	105,029	310,259	105,397	310,548	**0.35%**	**0.09%**
1000	121,904	432,163	119,425	429,973	**-2.03%**	**-0.51%**
1500	198,500	630,663	200,913	630,886	**1.22%**	**0.04%**
2150	247,072	877,735	247,161	878,048	**0.04%**	**0.04%**
3000	296,397	1,174,132	299,287	1,177,335	**0.98%**	**0.27%**
4500	467,537	1,641,669	462,010	1,639,345	**-1.18%**	**-0.14%**
6500	554,152	2,195,821	565,768	2,205,113	**2.10%**	**0.42%**
Total	**2,195,829**		**2,205,113**		**0.42%**	

The same rainfall values described above were assigned to both the elliptical and circular isohyetal patterns. As a sensitivity analysis, both elliptical and circular isohyetal patterns were tested to determine the effect on the peak flow at the Dallas gage. Table 3-8 shows the differences in flow using both patterns. The difference is small and historical research would be needed to accept the use of the circular isohyetal pattern.

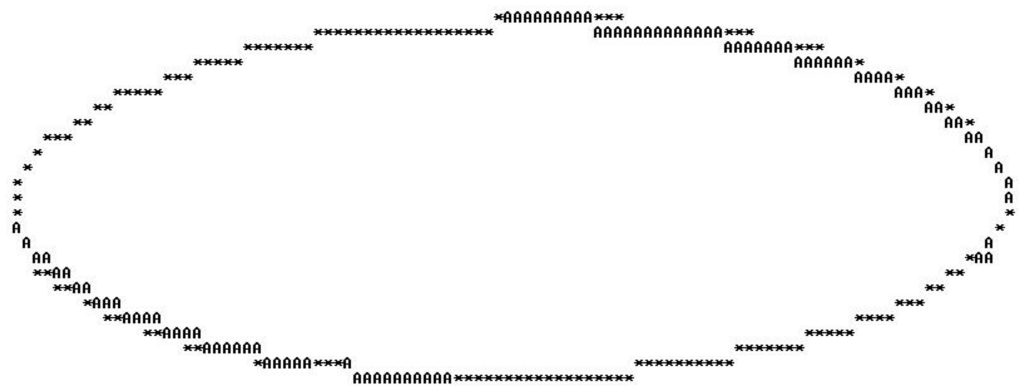
Table 3-8 Resulting Peak Flows at Dallas Gage for the Different Isohyetals, Depth-Area Curves, and Temporal Distributions for the 100-year event

TP-40 24 Hour 100 Year Rainfall									
Circular Storm					***Elliptical Storm***				
Center Location		***-97°13'30" 32°49'00"***			***Center Location***		***-97°14'30" 32°49'30" at 30°***		
Temporal Distribution	Area Depth Curves	High Band	Median	Low Band HMR-42	Temporal Distribution	Area Depth Curves	High Band	Median	Low Band HMR-52
Front Load		126,777	106,235	90,416	Front Load		126,792	106,904	92,686
Balanced		130,823	108,908	92,330	Balanced		130,368	109,294	93,600
Back Load		133,000	110,562	93,423	Back Load		132,631	110,810	95,293

3.2.5.4 HMR-52 Reduction Factors

HMR-52 reduction factors were developed for two main purposes. First, by calculating the HMR-52 reduction factors by the depth-area procedures, the HMR-52 expected volumes could be verified to insure accurate calculations were being performed with the procedure. Second, the HMR-52 reduction factors were used for the lowest estimates for the design storms rather than the 5 – 11 inch low band reduction factors from the historical storms.

Calculating the HMR-52 reduction factors could be completed by several methods. The method that was selected was to run the HMR-52 program using concentric ellipses of various sizes as the subbasins in the program. Within this output file the reduction factors were calculated using two sets of data. The 24-hour precipitation depth for each incremental isohyetal was extracted from the output file. This column remained the same for any isohyetal area. This list was used to directly calculate the incremental reduction factors for each incremental area. On Figure 3-16 the isohyetal 24-hour column was the incremental isohyetal precipitation depths that were extracted from the HMR-52 output file. The second set of data extracted was the total average 24-hour precipitation depth for each isohyetal area. On Figure 3-16, the 33.98 average precipitation of the 24-hour column was the first value on this total depth list. Scrolling through the HMR-52 output file, the total average 24-hour precipitation depth for each isohyetal area was extracted into a list. These precipitation depths represent the cumulative precipitation up through the area. With this list of numbers the incremental isohyetal reduction factors were calculated using the procedure described in the above sections. The final reduction factors do not calculate the exact same values, however, they are well within reason.



TIME INTERVAL = 60. MINUTES
1-HR TO 6-HR RATIO FOR ISOHYET A AT 20000 SQ. MI. = 0.300

DEPTH VS. DURATION

ISOHYET	5MIN	10MIN	15MIN	30MIN	1-HR	2-HR	3-HR	6-HR	12-HR	18-HR	24-HR	30-HR	36-HR	42-HR	48-HR	54-HR	60-HR	66-HR	72-HR
A	0.82	1.63	2.42	4.66	8.00	12.36	16.23	23.96	29.36	32.13	33.99	35.43	36.60	37.58	38.44	39.19	39.86	40.47	41.02
B	0.76	1.51	2.25	4.33	7.44	11.53	15.16	22.46	27.68	30.41	32.27	33.71	34.88	35.87	36.72	37.47	38.14	38.75	39.30
C	0.71	1.41	2.10	4.04	6.94	10.77	14.17	21.04	26.10	28.81	30.67	32.10	33.27	34.26	35.11	35.86	36.53	37.14	37.69
D	0.65	1.30	1.93	3.72	6.40	9.95	13.11	19.54	24.45	27.13	28.99	30.43	31.60	32.58	33.44	34.19	34.86	35.46	36.02
E	0.60	1.19	1.77	3.41	5.87	9.15	12.05	18.04	22.85	25.52	27.38	28.82	29.99	30.97	31.82	32.58	33.25	33.85	34.41
F	0.55	1.09	1.62	3.12	5.39	8.40	11.07	16.62	21.34	24.00	25.86	27.30	28.47	29.45	30.30	31.06	31.73	32.33	32.89
G	0.51	1.01	1.51	2.90	5.01	7.81	10.30	15.52	20.14	22.78	24.64	26.08	27.25	28.23	29.09	29.84	30.51	31.12	31.67
H	0.46	0.92	1.37	2.64	4.57	7.15	9.42	14.26	18.78	21.42	23.28	24.72	25.89	26.87	27.73	28.48	29.15	29.76	30.31
I	0.43	0.85	1.26	2.42	4.19	6.57	8.66	13.16	17.60	20.23	22.09	23.53	24.70	25.68	26.54	27.29	27.96	28.57	29.12
J	0.17	0.35	0.52	1.04	2.08	4.08	5.95	10.11	13.80	16.08	17.68	18.92	19.93	20.78	21.51	22.16	22.74	23.26	23.74
K	0.12	0.25	0.37	0.75	1.49	2.92	4.26	7.34	10.39	12.24	13.54	14.54	15.35	16.04	16.63	17.15	17.62	18.04	18.43
L	0.09	0.18	0.27	0.53	1.05	2.07	3.02	5.29	7.75	9.25	10.29	11.10	11.76	12.32	12.80	13.22	13.60	13.94	14.25
M	0.06	0.11	0.17	0.34	0.68	1.33	1.95	3.47	5.31	6.50	7.33	7.98	8.50	8.94	9.32	9.66	9.96	10.23	10.48
N	0.03	0.07	0.10	0.20	0.40	0.78	1.14	2.05	3.24	4.11	4.71	5.17	5.55	5.87	6.14	6.38	6.60	6.80	6.98
O	0.01	0.03	0.04	0.08	0.16	0.32	0.47	0.87	1.50	2.00	2.35	2.62	2.84	3.03	3.19	3.33	3.46	3.57	3.68
P	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Q	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
R	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
S	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
AVERAGE	0.82	1.63	2.42	4.66	8.00	12.35	16.22	23.95	29.35	32.12	33.98	35.42	36.58	37.57	38.42	39.17	39.85	40.45	41.01

Figure 3-16 HMR-52 Output File Example – 10 SM Isohyetal

3.2.6 Development of Spatial Distribution

3.2.6.1 Isohyetal Patterns

Both elliptical and circular isohyetal patterns were developed for the spatial distribution of the TP-40 24-hour point rainfall amounts. The elliptical isohyetal pattern has a ratio of major to minor axis of 2.5 to 1 and is the pattern used in standard engineering practice. The circular isohyetal was shaped so that the elliptical and circular isohyetals had equal areas. The isohyetals were created by using ArcMap 10 to create shapefiles that had polylines for each isohyetal. In addition, polygon shapefiles for each spatial distribution were created from the polyline shapefiles. These were later used to calculate the volume of each rainfall file to insure the correct volume was being routed through the HEC-1 model. Figures 3-17 and 3-18 are the rainfall images created from the circular and elliptical isohyetal patterns.

3.2.6.2 Storm Transposition

Simulations were run over multiple areas over the unregulated drainage areas upstream of the Dallas gage to determine the storm centering. The rainfall tin created from the TP-40 100-year 24-hour point rainfall was used as the basis during this process. After finding the centering that produced the highest peak at the Dallas gage from these general area runs, the optimal centering for the circular storm was found by combing over the area in a grid, running a simulation at every minute mark. Again, once the centering that produced the highest peak at the Dallas gage in the grid was found, simulations were then run at every half-minute to fine tune the centering. The starting point for finding the optimal storm center for the elliptical storm was the circular storm's optimal centering location. For the elliptical pattern, simulations were run at every half-minute in a grid around the starting point while additionally rotating the elliptical pattern in increments of 15 degrees between 0 and 90 degrees at every point.

With the centering and rotation of the elliptical isohyetal pattern found using the rainfall tin created from the TP-40 100-year 24-hour point rainfall, additional testing was completed to determine if there would be different storm centering for the other frequency storm events. Using the elliptical storm centering and rotation as the starting point, the different frequency rainfall amounts were tested over a grid to determine if a higher flow could be calculated at the Dallas gage. For the 25, 50, 100, and 500-year recurrence intervals, the centering and rotation of the elliptical pattern were the same. For the 2, 5, and 10-year recurrence interval the storm centering was slightly closer to the Dallas gage along the West Fork of the Trinity River. However, because the greatest difference in the peak flow that was found was less than 3%, the storm centering and rotation of the 100-year rainfall amount was used for all design storms.

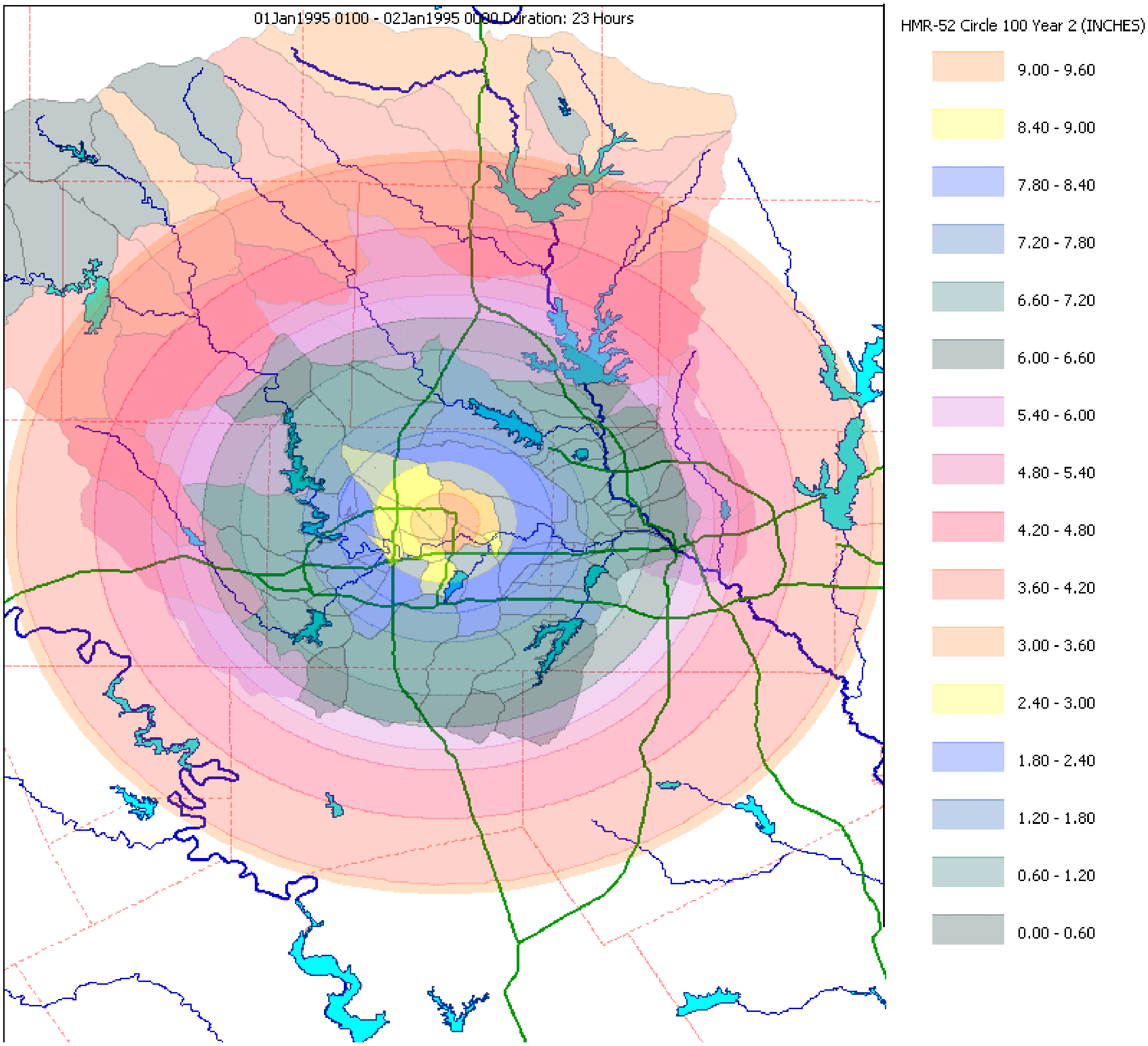


Figure 3-17 Circular Isohyetal Centering for Dallas Peak Flow

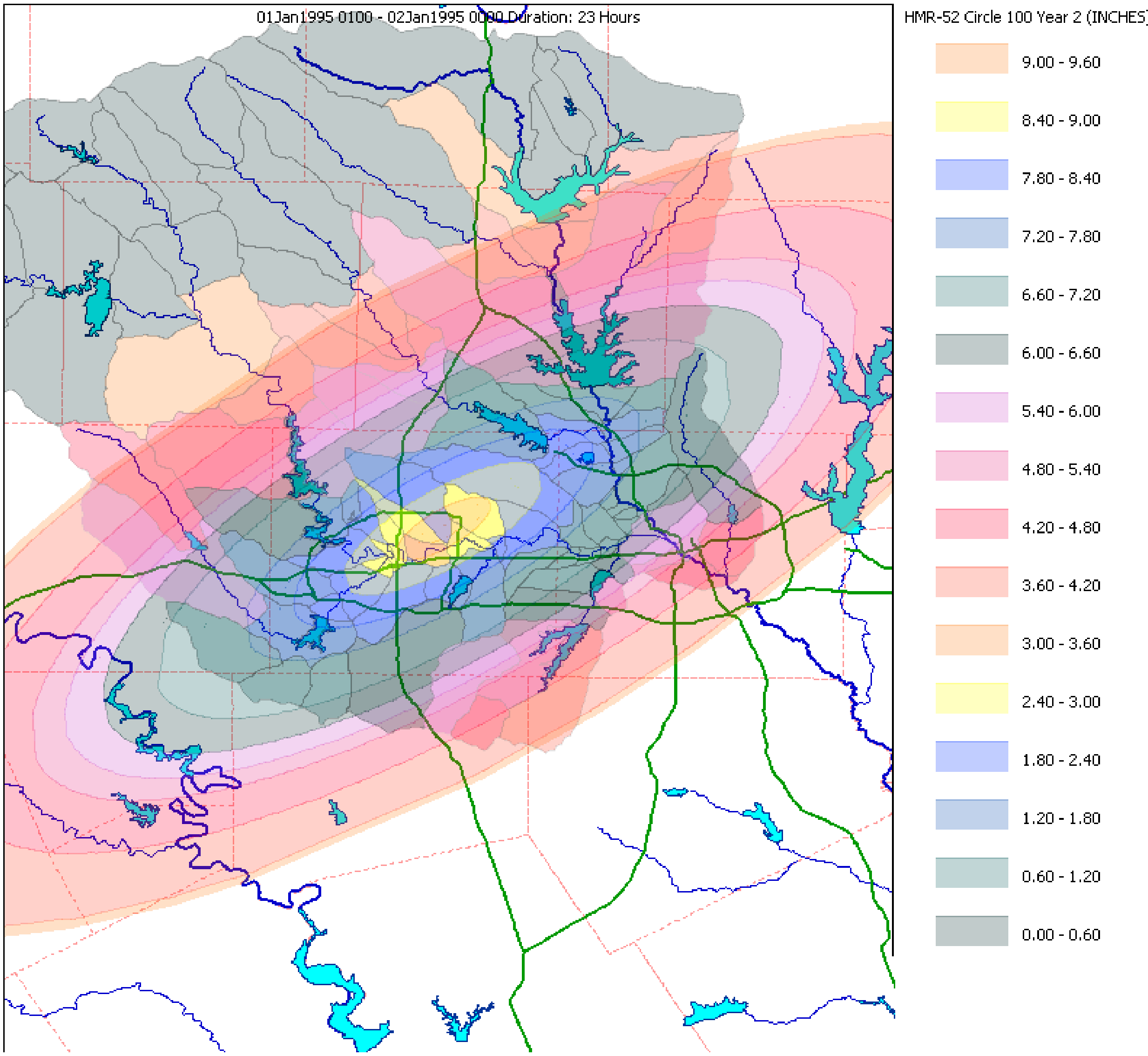


Figure 3-18 Elliptical Isohyetal Centering for Dallas Peak Flow

3.2.7 Development of Temporal Distribution

A 24-hour storm duration was used based on analysis of the time of concentrations within the basin. The hourly distribution of the storm was needed to develop the basin average precipitations from the rainfall tin in the Multiple Parameter Visualization Tool. The standard distribution used in the study was developed based on the Standard Balanced distribution in HEC-1. This provided an hourly distribution over the 24 hours that centered the peak 6 hours around the 12th hour. In addition, two other distributions were created to test the sensitivity of the final results to the temporal distribution. The two other distributions were a rearrangement of the Standard Balanced distribution. The hourly distributions were shifted forward and back to create a front and back loaded distribution where the peak was at the 9th hour and the 17th hour. Figure 3-19 shows the three temporal distributions tested in this study. Table 3-8 shows the differences in the peak flows at Dallas between the different distributions.

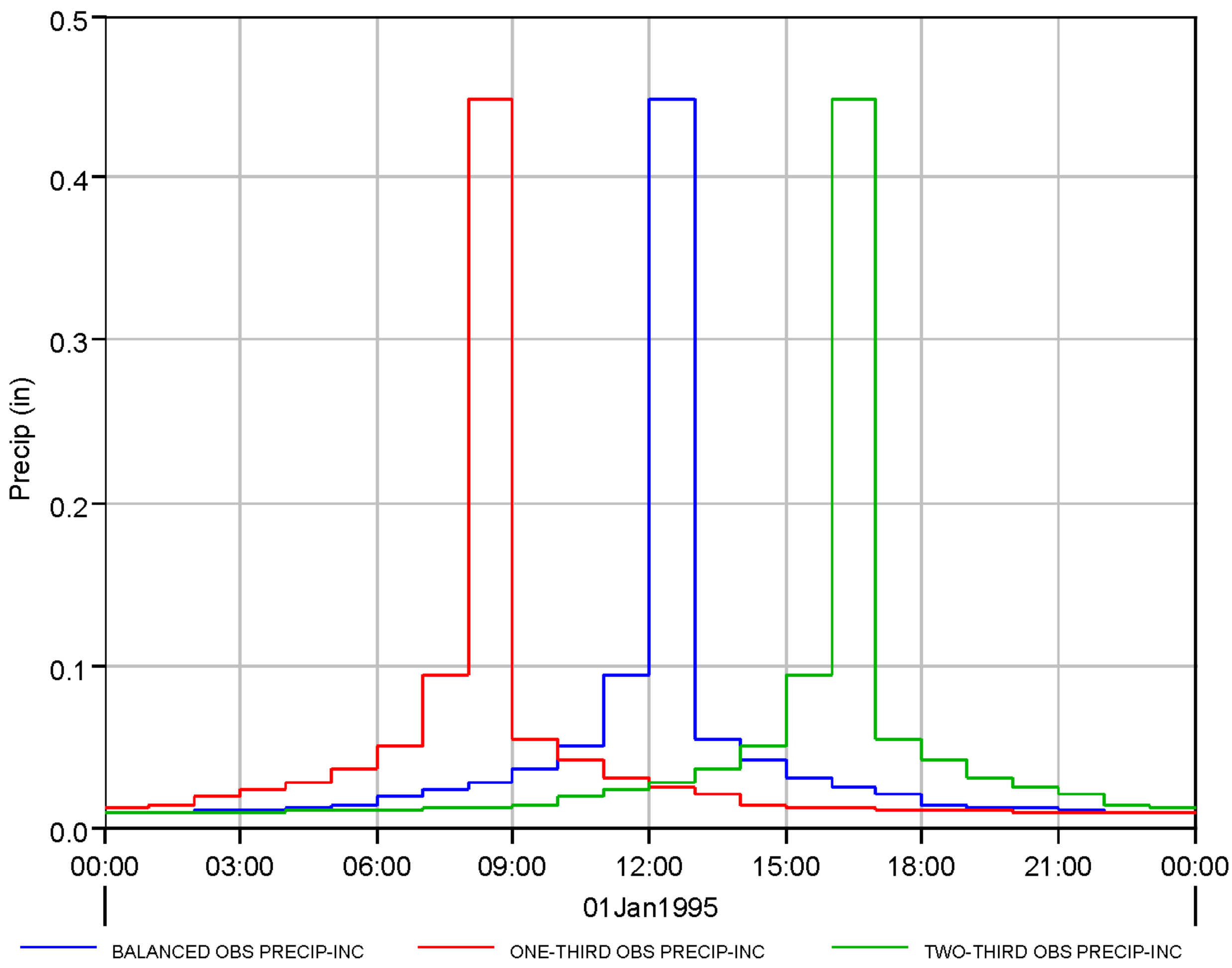


Figure 3-19 HEC-1 Temporal Distribution for a Balanced, Front, and Back Loaded Distribution

3.2.8 Sensitivity Testing and Results

3.2.8.1 Sensitivity Testing

Multiple assumptions were made throughout the design storm study. It was prudent to test each of the parameters where possible to evaluate the sensitivity a parameter had on the discharge frequency curve at the Dallas gage. The parameters tested were the spatial distribution, temporal distribution, and depth-area reductions. Table 3-8 shows the Dallas gage peak flows for the TP-40 24-hour 100-year precipitation. The table shows the range in flow from the sensitivity testing of these three parameters. The spatial distribution patterns resulted in a very small variance of the flow with an approximate range of 0.12 - 2.79% difference. The temporal distribution resulted in a small difference of flow with an approximate difference of 1.29 – 2.83% from the balance distribution given the elliptical shape. The depth-area reduction factors had a large variance in the flows calculated. The discharge frequency curve was sensitive to this parameter. Therefore, the high band and low band depth-area reduction factors were used as confidence limits for the final Dallas gage discharge frequency curve. The HMR-52 depth-area reduction factor was adopted as the low band reduction factor. Table 3-9 shows the Dallas gage flows for each series of design storm frequencies for the three bands of depth-area reduction factors. Given the elliptical shape, the depth-area parameter had an approximate difference of 13.3 – 19.7% from the median band of reduction factors.

At the conclusion of the sensitivity testing with the TP-40 24-hour 100-year rainfall the other recurrence interval design storms (2, 5, 10, 25, 50, and 500) were developed using the Standard Balanced temporal distribution, HMR-52 elliptical isohyetal pattern, and the three depth-area reduction factors. Table 3-9 shows the Dallas gage flows for each design storm applying the three bands of reduction factors.

Table 3-9 Peak Flows at Trinity River at Dallas Gage

	Depth-Area Reduction Factors		
Return Period (year)	*High Band*	*Median Band*	*Low Band HMR-52*
500	218,951	185,835	151,383
100	130,368	109,294	93,600
50	105,217	91,227	75,266
25	86,242	70,930	57,237
10	56,136	47,442	39,448
5	42,979	35,408	29,334
2	26,485	22,891	19,910

3.2.8.2 Discharge Frequency Curve

Figure 3-20 shows the discharge frequency curve for the historical Dallas maximum annual instantaneous flows, RiverWare simulated maximum annual instantaneous flows, the original Bulletin 17B computed curve with the 0.1 skew, and the new Regulated versus Unregulated Study discharge frequency curve based on the weighted skew.

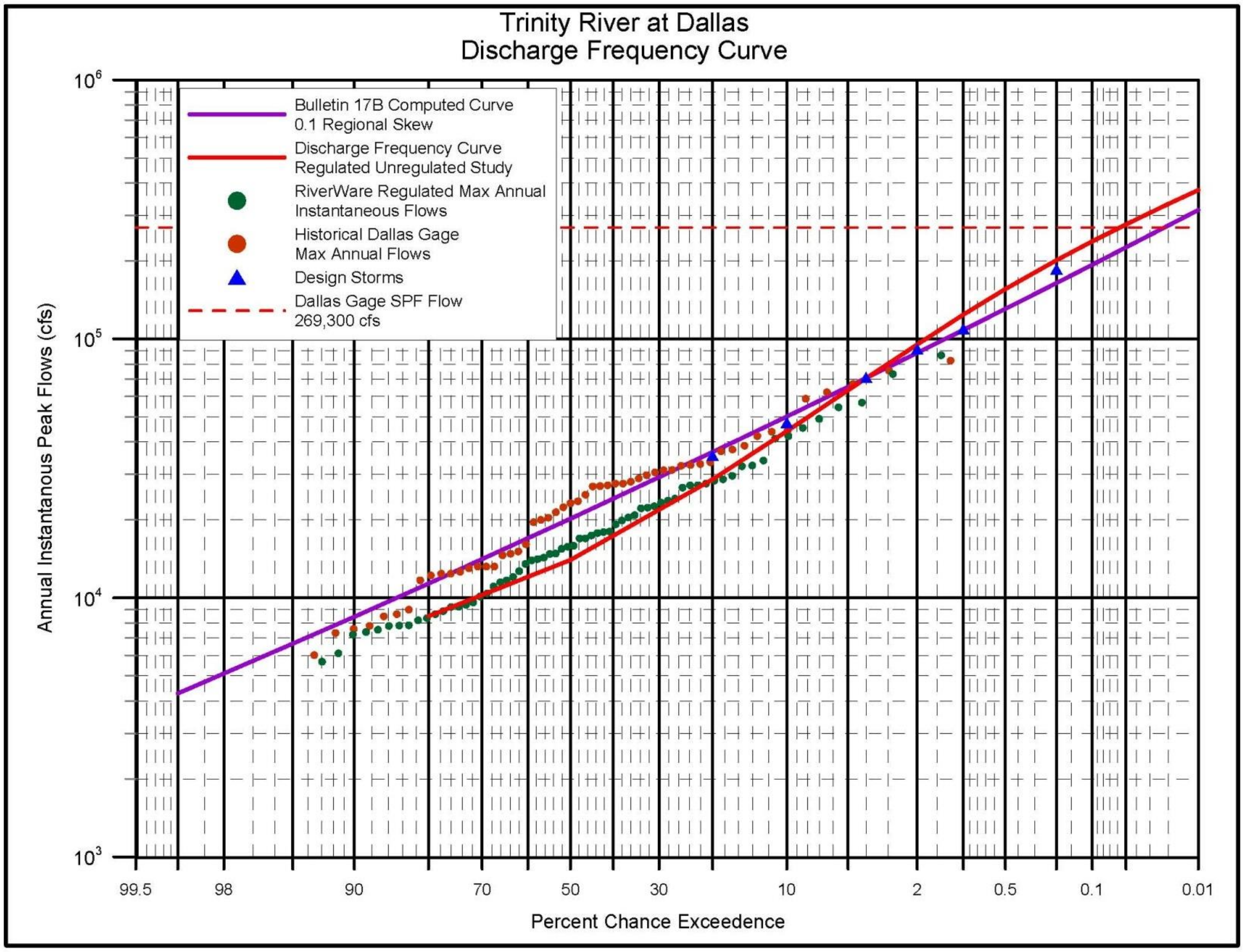


Figure 3-20 Trinity River at Dallas gage Discharge Frequency Curve with historical flows, RiverWare simulated flows, Bulletin 17B computed curve with 0.1 skew, and Regulated versus Unregulated frequency curve based on weighted skew

The discharge frequency curve shown in Figure 3-21 has removed the RiverWare simulated flows and added the new Bulletin 17B computed curve with a 0.2 skew. This was developed to fit a curve to the Design Storms and the Regulated versus Unregulated study discharge frequency curve. Using a computed curve that fits all the data rather than attempting to construct a composite curve was determined to be a better solution. The Design Storm based on the high band and HMR-52 depth-area reduction factors were added as confidence limits on the discharge frequency curve.

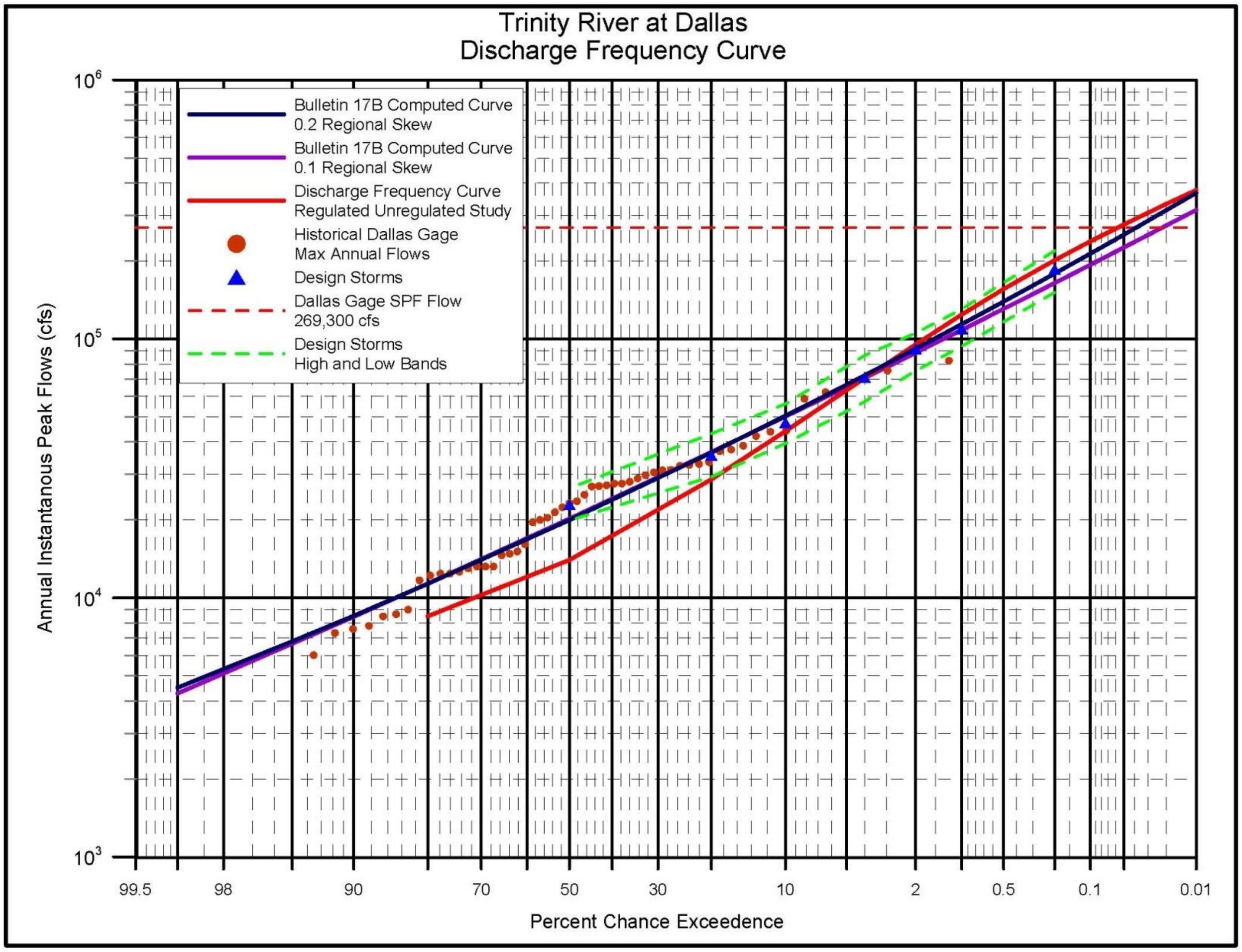


Figure 3-21 Trinity River at Dallas Gage Bulletin 17B frequency curve with 0.2 skew

Figure 3-22 shows the final Dallas gage discharge frequency curve with the upper and lower design storms plotted for confidence limits, the Design Storm points, and the historical flows at the Dallas gage. Table 3-10 shows the final probabilities and flows for this discharge frequency curve.

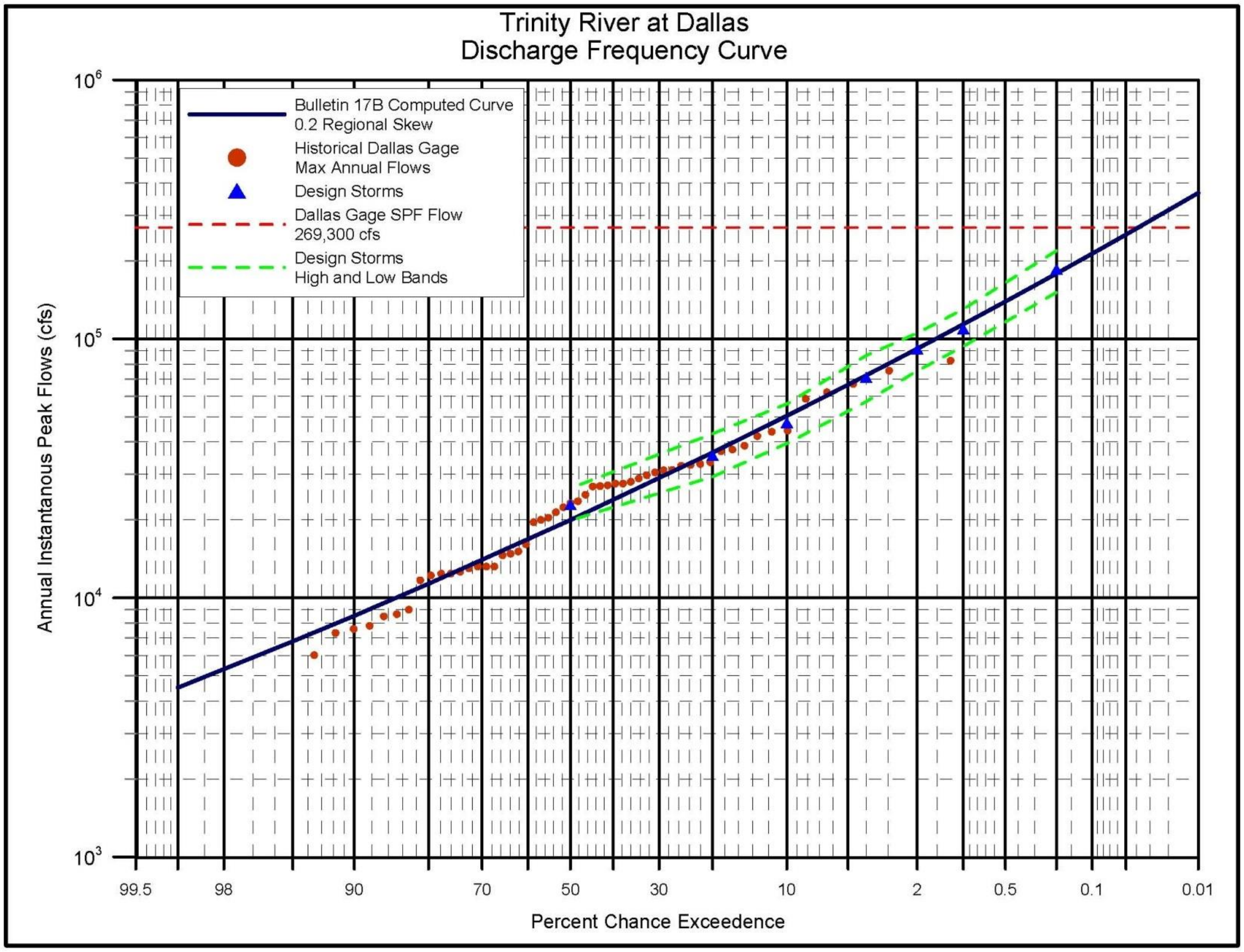


Figure 3-22 Trinity River at Dallas Gage adopted Frequency Curve

Table 3-10 Trinity River at Dallas Gage Adopted Frequency Curve for Existing Conditions

Probability	*Return Period (year)*	*Flow (cfs)*
0.0004	2500	269,300
0.002	500	179,000
0.01	100	114,000
0.02	50	92,000
0.05	20	67,000
0.1	10	50,000
0.2	5	36,000
0.5	2	26,485

3.3 Urbanization Study

The USACE Hydrology Committee recommended in their report "Findings and Recommendations Regarding Hydrology for the Dallas Floodway Project", dated May 2012, that the Fort Worth District conduct a study of the Trinity River Basin, in the vicinity of the Dallas Floodway, to determine the effects of urbanization on the frequency peak discharge relationships for gages within the basin. Further, it was recommended that this study be conducted in accordance with the guidelines published in Engineering Manual 1110-2-1415. The Hydrology Committee recognized that average annual discharges have doubled in magnitude over time due to changes within the watershed; however, the committee believes that urbanization impacts will be less significant for infrequent floods such as those used for project design.

The Fort Worth District agreed that a study of the effects of urbanization should be undertaken; however, schedule and funding constraints are such that it was not feasible to conduct this study at this time. Additional work performed by the district for the Corridor Development Certificate (CDC) update has shown that the computed probability relationships for the majority of gages within the urbanized portion of watershed are showing a significant negative skew. The district has undertaken a larger scale look at computed probability relationships within the hydrologic region to determine possible causes for these significant negative skews and to update the 1978 regional skew study by Leo Beard, "Generalized Skew Coefficients of Annual Maximum Streamflow Logarithms in Southwestern Division, Corps of Engineers". This study is not complete but may show that urbanization may be the cause of these negative skews. Significant drought and wet cycles may also contribute to the significant negative skews.

The methods within Engineering Manual 1110-2-1415 require an extensive look at historical floods in addition to preparing numerical models reflecting development and channelization conditions for different points in time. Further complicating this analysis is the construction of several USACE reservoirs in the late 1980's.

The district intends to pursue this study to determine the impacts of urbanization of peak discharges in the future when funding can be identified and resources dedicated to performing the work.

3.4 Final Frequency Curve Results

Results from the Regulated versus Unregulated Flow Study and the Design Storm Study were presented to the USACE Hydrology Committee via webinar/conference call on 30 July 2012. On 15 August 2012, the Hydrology Committee reconvened to make a final recommendation on the Trinity River at Dallas frequency curve. The recommendation of the Hydrology Committee was to accept the composite frequency curve computed by the Fort Worth District, the Bulletin 17B Log Pearson III computed curve with a 0.2 skew. This composite curve utilized the historical Dallas gage annual peak flows from 1955 – 2011, the Design Storm study results, and the upper portion of the Regulated versus Unregulated study discharge frequency curve.

3.4.1 Existing Watershed Conditions

The adopted frequency curve for existing conditions on the Trinity River at Dallas indicates the 1% Annual Exceedance Probability (AEP), also known as the 100-year flood event, has a peak discharge of 114,000 cfs. The return period of the peak Standard Project Flood (SPF) discharge of 269,300 cfs was calculated at 2,500 years with an AEP of 0.04%. The full range of final frequency flows for existing conditions are shown below in Figure 3-23 and in Table 3-11.

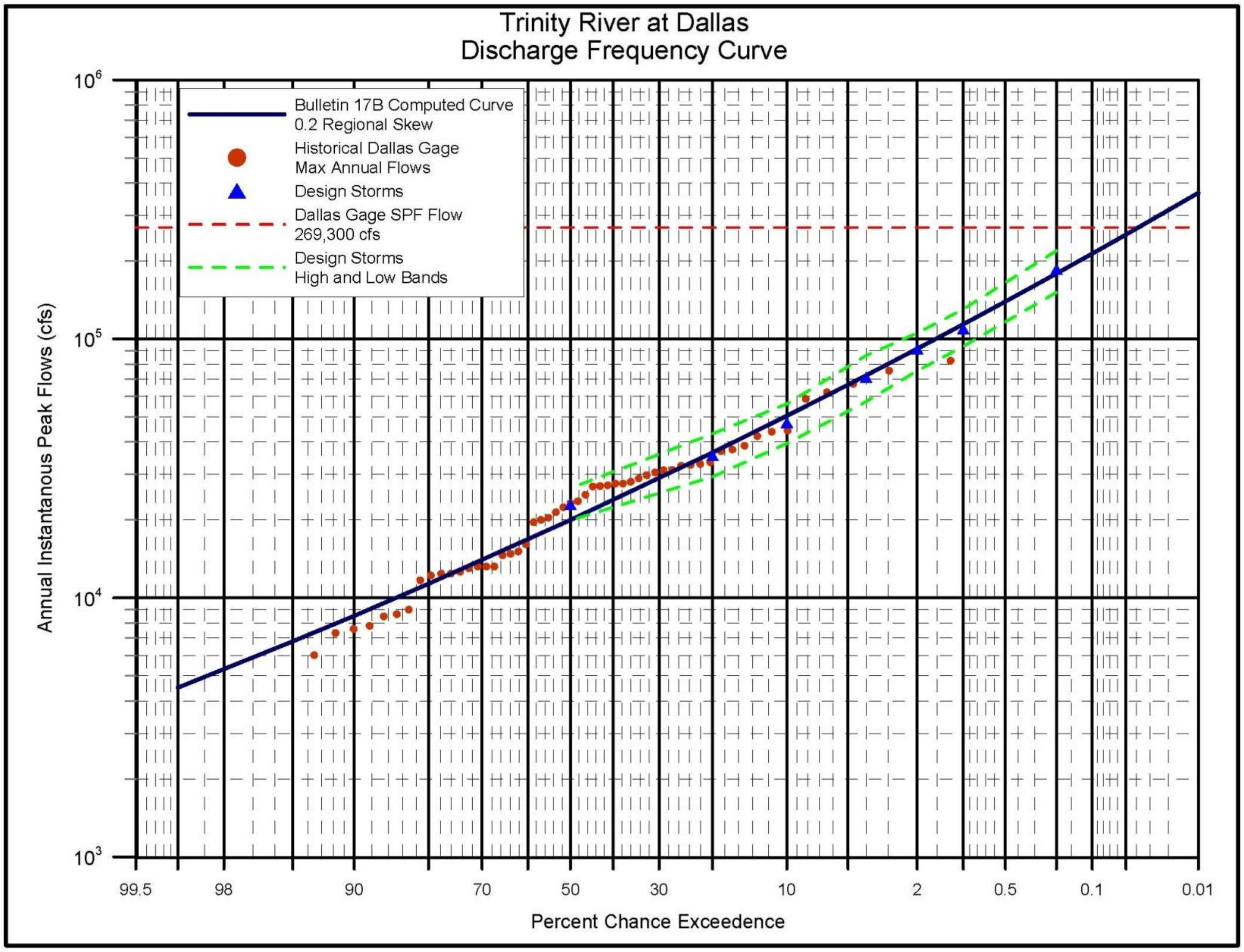


Figure 3-23 Final Frequency Curve at Dallas for Existing Conditions

3.4.2 Future Watershed Conditions

In order to account for the effects of future urbanization on the Trinity watershed, projections were made about future land use. During the reconnaissance phase of the Upper Trinity study, the Corps of Engineers requested information from the North Texas Council of Governments (NTCOG) regarding future land use for a 50-year period of analysis. Thirty-two cities in the Dallas-Fort Worth Metroplex responded to that request with varying degrees of detail on projected future development. That information was used to estimate the future land use and urbanization percentage of each subarea, and a watershed runoff model was used to estimate future peak flows (USACE-Fort Worth District, 1990). This estimate was assumed valid for watershed conditions near the end of a 50-year period of analysis. Table 3-11 shows the final frequency flows at Dallas for existing and future conditions.

Table 3-11 Final Frequency Flows at Dallas for Existing and Future Conditions

Annual Exceedance Probability	*Return Period (years)*	*Existing Conditions Peak Flow (cfs)*	*Future Conditions Peak Flow (cfs)*
0.0004	2500	269,300 (SPF)	277,000 (SPF)
0.002	500	179,000	184,000
0.01	100	114,000	119,000
0.02	50	92,000	96,000
0.05	20	67,000	72,000
0.1	10	50,000	55,000
0.2	5	36,000	41,000
0.5	2	26,485	30,000

4.0 UNSTEADY HYDRAULIC ANALYSIS FOR BASELINE CONDITIONS

4.1 INTRODUCTION AND PURPOSE OF UNSTEADY MODELING

Previous hydraulic analyses for the Dallas Floodway Feasibility Study were performed with calibrated steady flow HEC-RAS models. These steady flow HEC-RAS models were primarily used in this study for the Comprehensive Analysis phase but were also used to develop the Unsteady Flow models for levee overtopping analysis. Further discussion of the baseline steady flow models including the calibration and stage–frequency uncertainty estimates and description of data sources are provided in Section 6.2 for the Comprehensive Analysis phase. The decision to switch to an unsteady flow analysis for levee overtopping was made in order to better account for the effects of timing and flood volume during a levee system capacity exceedence or levee breach. Unsteady hydraulic modeling using HEC-RAS was completed in support of the Dallas Floodway Feasibility Study. This analysis was performed for baseline and future without-project conditions to measure the performance of the existing Dallas Levees against a range of flood events. The unsteady flow analysis was used as input into HEC-FDA and HEC-FIA to evaluate both the economic consequences and life safety consequences due to the overtopping and/or breaching of the levees. Dallas Floodway Feasibility Study project alternatives were also analyzed with this unsteady flow model following the formulation of the final array of alternatives.

4.2 IN-PROGRESS REVIEW TEAM

4.2.1 Purpose of Review Team

Measuring the risk of failure for existing levees has become a focus of national attention ever since the failure of the New Orleans levees during Hurricane Katrina. However, there is a limited amount of existing research or studies that have been completed on the topic of levee failure estimation, and USACE is still in the process of formulating its guidance on how to model levee failures and estimate levee breach sizes. Therefore, in order to make the best hydraulic estimates possible for the Dallas Floodway Feasibility Study, the Fort Worth District reached out to various experts within the Corps and assembled an in-progress review team for the unsteady hydraulic modeling. The purpose of this in-progress review team was to review the district's levee failure methodologies and assumptions as they were being modeled and to make recommendations on ways to improve the model's estimates.

4.2.2 Review Team

During the early scoping phase of the unsteady modeling effort, the Fort Worth District reached out to various USACE experts in unsteady hydraulic modeling, levee safety, risk, and consequence estimation and asked them to be a part of this in-progress review team. The final team included members of the USACE HEC, RMC, and MMC production centers. The District's H&H, economic, planning, and project management team members were also invited to participate. Bi-weekly conference calls were held with the review team during the duration of the unsteady modeling effort. Meeting minutes of discussions held regarding the selection of modeling parameters are included in the Dallas Floodway Feasibility Study project file.

4.3 HEC-RAS Model Development

4.3.1 The Risk Assessment Unsteady Hydraulic Model

At the beginning of this modeling effort, an unsteady hydraulic model was developed for the Dallas Floodway Risk Assessment. The unsteady flow model used for the Dallas Floodway Feasibility Study uses the Risk Assessment model for its base HEC-RAS geometry. The objective of the Dallas Floodway Risk Assessment was to provide hydrologic information relative to the Risk Assessment for the Dallas Floodway project. The Risk Assessment Unsteady Hydraulic Model was used to predict the timing and depths of inundation of the protected areas for a variety of levee breach and overtopping scenarios, which was used as input for consequence assessment in terms of loss of life. The Risk Assessment model considered only the "base condition" and made no attempt to consider future projects such as the Dallas Floodway Extension Project. The modeling effort focused on the East and West Levee reaches and did not consider other related nearby projects such as the Rochester Park Levee and the Central Wastewater Treatment Plant Levee.

The Risk Assessment unsteady model includes approximately 23 miles of the Dallas Floodway East and West Levees along both sides of the main stem Trinity River through the City of Dallas and along one side of the Elm Fork and the West Fork, as shown in Figure 4-1. The existing Dallas Levees are modeled as "lateral weirs" with the protected areas behind the levees modeled as a series of interconnected storage areas. The top of levee elevations in the model represent existing conditions and were taken from a 2003 ground survey as represented in the Dallas Floodway 2007 Periodic Inspection Report. This survey includes levee crest elevations taken every 100 feet along the levees.

In the unsteady hydraulic HEC-RAS model, the areas behind the East and West Levees were divided into multiple storage areas in order the model the effects of flood volume and timing during a levee failure. Ten storage areas were modeled behind the East Levee, and seven storage areas were modeled behind the West Levee, as shown in Figure 4-1. However, the unsteady hydraulic modeling results showed that for each levee breach that was analyzed, all 10 storage areas behind the East Levee filled up to the same maximum water surface elevation (within 0.01 foot). Similarly, the storage areas behind the West Levee all filled up to the same maximum water surface elevation (within 0.01 foot) for a given levee breach. This phenomenon of the storage areas filling to a single elevation, occurred consistently for any levee breach that was analyzed, whether it be from internal erosion, overtopping, an upstream location, or a downstream location. The flood inundation behind each levee was consistently represented by a single elevation. Therefore, after reviewing the results of the hydraulic model, it was decided that the HEC-FDA economic model would be divided into one reach for the East Levee and one reach for the West Levee.

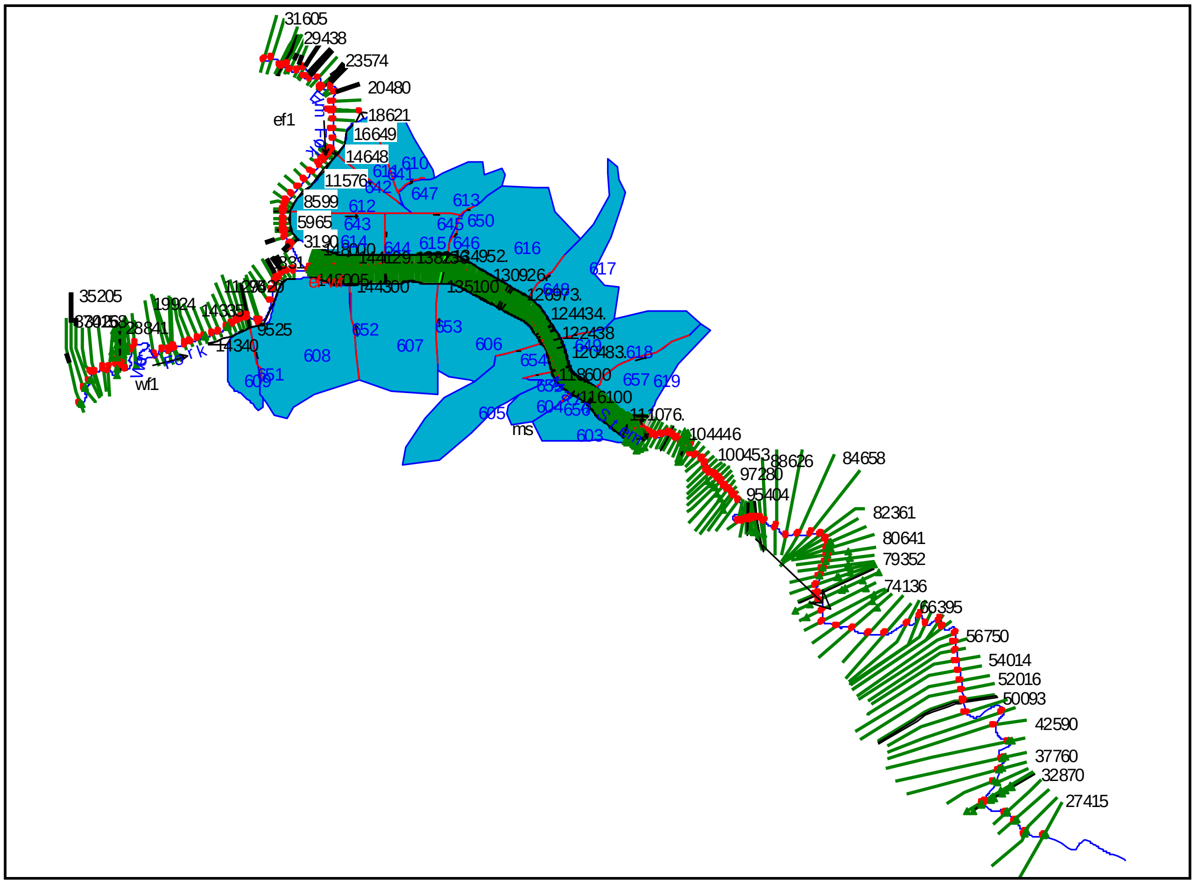


Figure 4-1 Risk Assessment HEC-RAS Geometry

The Risk Assessment HEC-RAS unsteady flow model for the Dallas Floodway was developed from data taken from the Dallas Floodway Feasibility Study steady flow model. The Dallas Floodway Feasibility Study steady flow model was developed specifically for this study with cross section and bridge geometry information used in the current version of the Upper Trinity River CDC model. The Dallas Floodway Feasibility Study steady flow model is a geo-referenced model that extends from near Hutchins, TX at the downstream end up to near the interstate 35E crossing on the Elm Fork and Grand Prairie, TX on the West Fork. The CDC model is a non geo-referenced model that extends from near Hutchins, TX at the downstream end up to the Lewisville Lake dam on the Elm Fork Trinity River and the Lake Worth dam on the West Fork Trinity River. The downstream boundary condition for the Risk Assessment HEC-RAS unsteady flow model is about 15 river miles downstream of the Dallas Floodway and was defined with a rating curve taken from the Dallas Floodway Feasibility Study model. Further discussion of the baseline steady flow models including the calibration and stage–frequency uncertainty estimates and description of data sources are provided in Section 6.2 for the Comprehensive Analysis phase.

The Risk Assessment HEC-RAS unsteady flow model required the addition of the existing levee profile as lateral structures and the addition of a series of interconnected storage areas to model the accumulation of flow throughout the levee protected floodplain. The Dallas Floodway Feasibility Study steady flow model cross sections terminate at the crest of the levees such that all flow is confined to the floodway between the levee crests. The Risk Assessment model was developed with the lateral structures at the crest of the levees using a 2003 levee crest survey that provides elevations every 100-feet of levee stationing. According to the Risk Assessment report, the 2003 survey was compared to a later 2010 survey and the differences were found to be minimal.

The current levee crest profile has significant variability compared to the 1950s levee design grade. The original design grade (circa 1952) of the levee has been altered due to a combination of settlement, sloughing, local crest restoration projects, and construction tolerances.

Several other relatively minor edits to the geometry were made in order to convert the model to unsteady flow and to calibrate the model to the results from the steady flow model. The Risk Assessment unsteady model generally matched the Dallas Floodway Feasibility Study model within ±0.5 foot. Although not an all-inclusive list, a general summary of those edits that were required to develop the Risk Assessment model are shown below.

- Defined the HTAB parameters for all cross sections and structures.
- Adjusted some unsteady flow expansion and contraction coefficients (note that these are separate from the steady flow expansion and contraction coefficients, which are not used in unsteady HEC-RAS).
- Some adjustments to roughness parameters.
- Addition of one cross section near the confluence of the Elm Fork and West Fork to allow for placement of the lateral structure representing the levee in this area.
- Removal of one low bridge due to unsteady constraint regarding number of cross sections between bridges.

The Risk Assessment model was developed using the Texas North Central State Plane (feet) coordinate system, to remain consistent with the Dallas Floodway Feasibility Study steady flow model. The vertical datum used was NGVD29. Within the limits of the model the difference between NGVD29 and NAVD88 is less than one inch (USACE-RMC 2012).

4.3.2 HEC-RAS Geometry for the Dallas Floodway Feasibility Study FRM Unsteady Flow Model

The Risk Assessment unsteady flow model geometry was used as the basis for the development of the HEC-RAS unsteady flow model geometry for the Dallas Floodway Feasibility Study FRM analysis. Fort Worth District personnel reviewed the Risk Assessment HEC-RAS geometry for appropriateness for the Dallas Floodway Feasibility Study and documented any changes. This included a review of the current calibration of the model with emphasis on flows that are near the levee overtopping levels.

One significant difference between the Risk Assessment geometry and the Dallas Floodway Feasibility Study geometry was how the existing East Levee floodwall was modeled. The East Levee floodwall is located near the AT&SF Railroad Bridge and the DART Rail Line Bridge. This lateral extension of the East Levee is comprised of both an earthen levee section and a concrete floodwall section and ties the East Levee to high ground. The concrete floodwall portion of the existing East Levee was constructed as part of the original Dallas Floodway Levees around 1930 and was retained as a functional part of the East Levee when the Corps of Engineers constructed the upgrades to the levee system in the 1950s. The concrete floodwall portion of the East Levee has an original design crest elevation of 423.0 feet and was not raised or modified when the levee system was upgraded by the USACE in the 1950s.

The authorized Dallas Floodway Extension Project is located immediately downstream from the existing Dallas Floodway levees and the design is comprised of three major structural FRM components. They are the Lamar Street Levee, the Cadillac Heights Levee and the Chain of Wetlands. The Chain of Wetlands has been divided into the Upper Chain of Wetlands and the Lower Chain of Wetlands construction segments and only the Lower Chain of Wetlands component of the Dallas Floodway Extension project has been constructed to date.

The proposed Lamar Street Levee portion of the Dallas Floodway Extension project when constructed will tie directly to the earthen levee portion of the East Levee at the DART Rail Line Bridge crossing, linking the existing East Levee with the existing Rochester Park Levee downstream. The Dallas Floodway East Levee, the proposed Lamar Street Levee, and the existing Rochester Park Levee will then function as a continuous levee system thus forming a complete “extension” of the Dallas Floodway. When the Lamar Street Levee is constructed, the existing earthen levee and the concrete floodwall portion of the East Levee beneath the DART Rail Line Bridge would become isolated from the floodway and would no longer be considered a levee overtopping risk.

The Dallas Floodway Feasibility Study considers the Dallas Floodway Extension project as a complete project for baseline conditions. The planned levee modifications associated with the Dallas Floodway Extension project will block flooding access to the existing floodwall, such that overtopping would not occur at that location. The Risk Assessment geometry, on the other hand, represents existing conditions and does not include the unconstructed components of the Dallas Floodway Extension project. Therefore, some changes were made to the Risk Assessment geometry to account for the effects of the proposed Dallas Floodway Extension on the floodway. Specifically, the portion of the downstream left bank lateral structure below elevation 426 feet, representing the East Levee Floodwall at the downstream end of the levee, was removed so that levee overtopping cannot occur where the new levee as part of the Dallas Floodway Extension project is to be built to a higher elevation. Modeling of the Dallas Floodway Extension project components for Dallas Floodway Extension levee overtopping analysis is not included in this Dallas Floodway Feasibility Study Unsteady flow model. However, the downstream hydraulic impacts of the Dallas Floodway Extension project are included in the analysis.

The decision to include the authorized Dallas Floodway Extension project as a completed project for baseline conditions for the Dallas Floodway Feasibility Study was documented in a Memorandum for Record following an In-Progress Review that was held on January 11-12, 2012. The Dallas Floodway Feasibility Study team would manage any potential future changes to the Dallas Floodway Extension project during design. In the worst case, if the remaining components of the Dallas Floodway Extension project were canceled, the Dallas Floodway team would consider additional improvements to the floodwall at the downstream end of the East Levee to compensate for the lack of a Dallas Floodway Extension levee downstream.

4.3.3 Calibration

Additional calibration was performed on the Dallas Floodway Feasibility Study unsteady HEC-RAS model to better match the Dallas Floodway Feasibility Study steady flow model profiles. The calibration was performed by adding constant hydrographs in unsteady RAS so that the discharges at each cross section matched the Dallas Floodway Feasibility Study steady flow model. The unsteady model had a simulation time that was long enough for the computed water surface profiles to stabilize. The resulting profile was then compared to the CDC model. Parameters were adjusted until the unsteady water surface elevations came within an acceptable tolerance of the steady flow profiles.

The following items were adjusted as part of the calibration effort.

- **Unsteady contraction and expansion coefficients**. – Typical values of (0.1, 0.3) were added to a number of cross sections, while maximum values of (0.5, 0.8) were used for the most severe contractions.

- **Bridge HTAB parameters -** The maximum headwater was adjusted to only a few feet (less than 5 feet) above highest profile in order to improve definition of the curve while capturing the highest profile.
- **Manning's roughness parameters -** The roughness values on the West Fork were increased 0.003, while the values on the Elm Fork were adjusted 0.005. The Main Stem values were not adjusted.
- **Junction method** – The junction method was modified from "Force Equal WS Elevations" to an "Energy Balance Method," and reach lengths were added to the downstream cross sections of the Elm Fork and West Fork to allow HEC-RAS to calculate head loss through the junction.

The resulting calibrated water surface profiles differed by less than 0.1 feet at every cross section from the confluence of the Elm Fork and the West Fork (River Station 148136) to the Commerce Street Gage (River Station 120729). From the Commerce Street Gage to the downstream end of the Dallas Levees, the Trinity River profiles differed by less than 0.3 feet. On the Elm Fork and West Forks, the unsteady and steady flow profiles generally differed by less than 0.1 and 0.3 feet, respectively.

4.3.4 Inflow Hydrographs

The hydrology (specifically, the discharge-frequency relationship) for the Dallas Floodway was finalized simultaneously with this unsteady hydraulic analysis. Therefore, the unsteady hydraulic modeling was completed by scaling existing unsteady flow hydrographs up and down to produce a range of peak discharges. The actual frequencies of the modeled peak discharges for baseline and Future Without-Project Conditions will be assigned with the final discharge-frequency relationship during economic analysis.

The shape of the inflow hydrographs were taken from the future SPF hydrograph for the Trinity River at Dallas (Commerce Street) Gage, which peaks at 277,000 cfs. The SPF hydrograph likely has a similar shape to the scale of floods that would overtop the Dallas Floodway levee system, resulting in damage to the protected areas. A series of multiplication factors were applied to the hourly flows of this hydrograph to scale the hydrograph up or down to the desired peak discharge at the Commerce Street gage. Figure 4-2 illustrates the shape of the SPF hydrograph as well as the proportionate contributions of the Elm Fork and the West Fork. These proportions were maintained in the scaled hydrographs. Table 4-1 contains the final eight inflow hydrographs and their associated multiplication factors. In the HEC-RAS flow file, these inflow hydrographs were entered at the upstream ends of the Elm Fork and the West Fork reaches.

Table 4-1 Dallas Floodway Feasibility Study Inflow Hydrographs Scaled from the SPF

Event #	*SPF Hydrograph Multiplier*	*Peak Q at Commerce (cfs)*	*Peak WS Elev at Commerce (feet)*	*Peak Corresponds with:*
1	0.47	120,000	416.5	100-year Future Peak Q
2	0.85	217,400	424.5	2 feet below Threshold
3	0.96	245,000	426.6	Threshold for Overtopping the East Levee
4	1.018	259,900	427.6	1 feet over Threshold
5	1.053	269,300	428.2	SPF Existing Peak Q
6	1.083	277,100	428.7	SPF Future Peak Q
7	1.132	289,200	429.6	SPF + 1 foot
8	1.184	302,100	430.6	SPF + 2 feet

Notes: Commerce location is the upstream side of Commerce Street Bridge (RS 120765). For this analysis, the lateral structure computations were turned off so that no spill would occur into the storage areas.

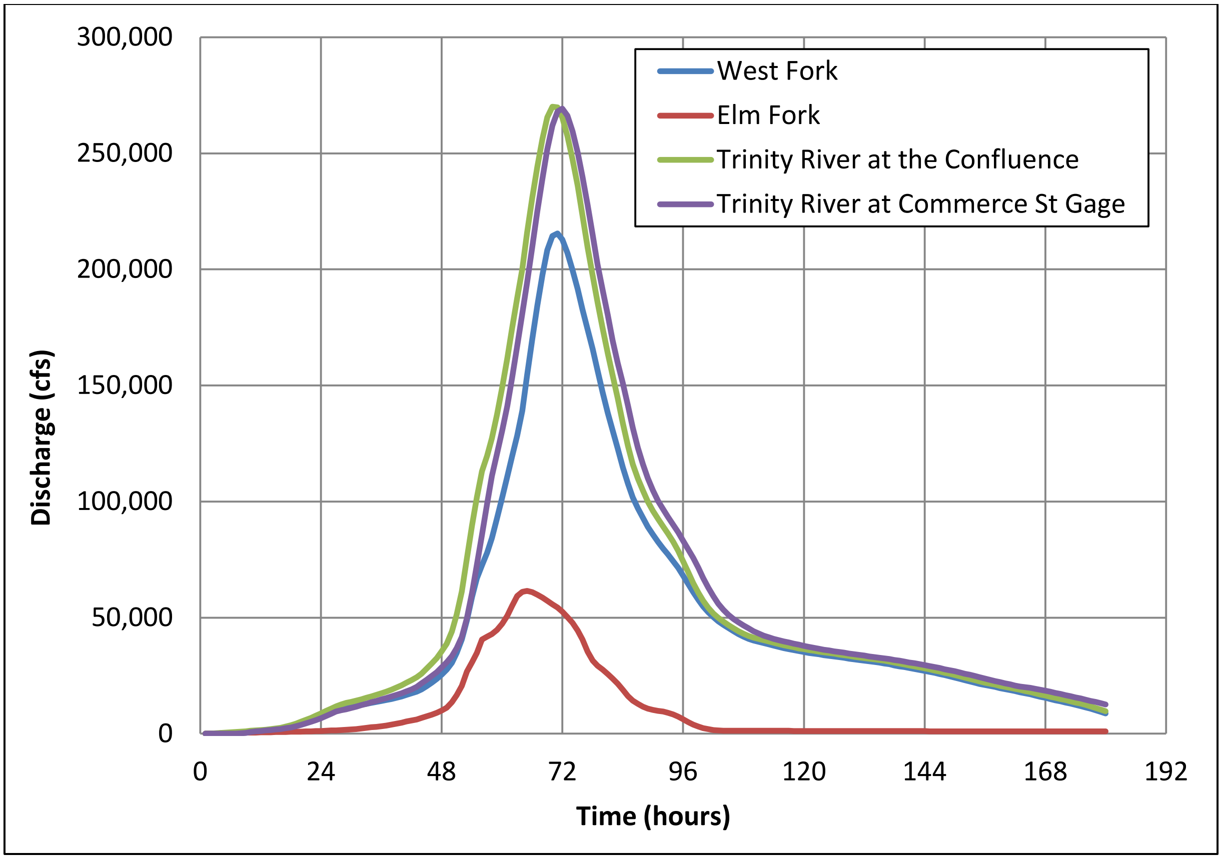


Figure 4-2 SPF Hydrographs

4.4 Levee Breach Assumptions

4.4.1 Existing Guidance

Assumptions that are made regarding the potential failure of the levees have a significant effect on interior flooding depths, as well as the resulting estimates of economic damage and life loss. However, there is very little existing guidance for levee breach analysis. Very few studies or research efforts in this area have been completed, and published breach regression equations, such as the ones used by the MMC, were developed primarily for application to dam breach analysis. It is commonly accepted that there is a high degree of uncertainty with using the regression equations for dam breach analysis (Wahl 2004), and thus their application to levee breach would be questionable as well. The USACE is in the process of formulating its guidance on how to model levee failures and estimate levee breach sizes.

To provide additional guidance, the Dallas Floodway Feasibility Study unsteady flow model was developed with close coordination between team members from throughout the USACE. Numerous USACE team members reviewed the levee failure methodologies and assumptions as they were being modeled and made recommendations on improvements.

4.4.2 Assumptions in the Risk Assessment Model

The Risk Assessment analyzed 11 potential failure locations (3 overtopping, 8 piping) that were identified in the Probable Failure Mode Analysis. The 2007 flood hydrograph was used as the base hydrograph that was scaled to produce five additional hydrographs that were used in the analysis. The Risk Assessment assumed that initial overtopping would occur at the east downstream floodwall. Table 4-2 below shows the resulting Risk Assessment inflow hydrographs. It was noted that the scenarios do not produce exactly the ½, ¾, or full levee load at all of the levee sections due to the variability of levee height.

Table 4-2 Risk Assessment Inflow Hydrographs

Event	*Hydrograph Multiplier*	*Peak Discharge (cfs)*
June 2007 Flood	NA	35,700
1/2 Levee	3.3	117,810
3/4 Levee	5.5	196,350
Threshold	6.7	232,050
Full Levee/Overtopping A	7.9	282,030
Overtopping B	9	321,300

Levee breach dimensions and formation time in the existing Risk Assessment analysis were based on WinDAMB results and expert geotechnical elicitation. WinDAMB is a program designed for dam overtopping, but the Risk Assessment assumed that the erosion mechanism would be similar for levees, although the hydraulic conditions may be significantly different for a dam. Key parameters for the WinDAMB model are the inflow hydrograph, total unit weight for the soil, erodibility index (kd), undrained shear strength, plasticity index, and particle diameter in inches. From Table 14 in the Risk Assessment report, the computed breach formation times ranged from 5.6 hours to 14.9 hours. The breach widths ranged from 118 feet to 167 feet. Table 4-3 presents the breach parameters that were used in the Risk Assessment HEC-RAS model.

Table 4-3 Breach Parameters Used in the Risk Assessment Model

Event	*Breach Width (feet)*	*Formation Time (hours)*
1/2 Levee	150	26
3/4 Levee	150	6
Threshold	150	6
Overtopping A	150	13
Overtopping B	150	13

For a more detailed explanation of the assumptions made in the Risk Assessment analysis, please refer to the Risk Assessment report (USACE-RMC 2012).

4.4.3 Breach Triggers: Overtopping versus Piping

Two potential failure modes for the levee system were considered: internal erosion (piping) failures and levee overtopping resulting in a breach. Initially the focus of the Dallas Floodway Feasibility Study was only on the overtopping with breach failure mode. However, the internal erosion failure mode was added to the economic analysis following the determination that the life safety risk could potentially be reduced by measures to address internal erosion.

The breach progressions for these two failure modes are entirely different and independent from one another. Therefore, the evaluation of these two failure modes required two different analyses for baseline conditions. The remainder of this section (Chapter 4) describes the assumptions and analysis for baseline conditions for the overtopping failure mode only. The internal erosion failure mode was later analyzed as a separate baseline condition, and its analysis is discussed in Section 5.6 of this appendix.

For the overtopping with breach failure mode, the base conditions hydraulic analysis assumed that the East and West levees were allowed to breach together within the same HEC-RAS run. Breach initiation was triggered when the water surface reached 0.5 foot of depth over the levee (Elevations 430.77 feet for the East levee and 432.88 for the West levee). It was assumed that this depth would produce erosive velocities on the levee sufficient to initiate a breach in the levee.

4.4.4 Breach Locations

The breach locations were selected at the location where overtopping would first occur on the existing levee crest. The breach location for the East levee was at river station 134952. The breach location for the West levee was at river station 139920, as shown in Figure 4-3 below.

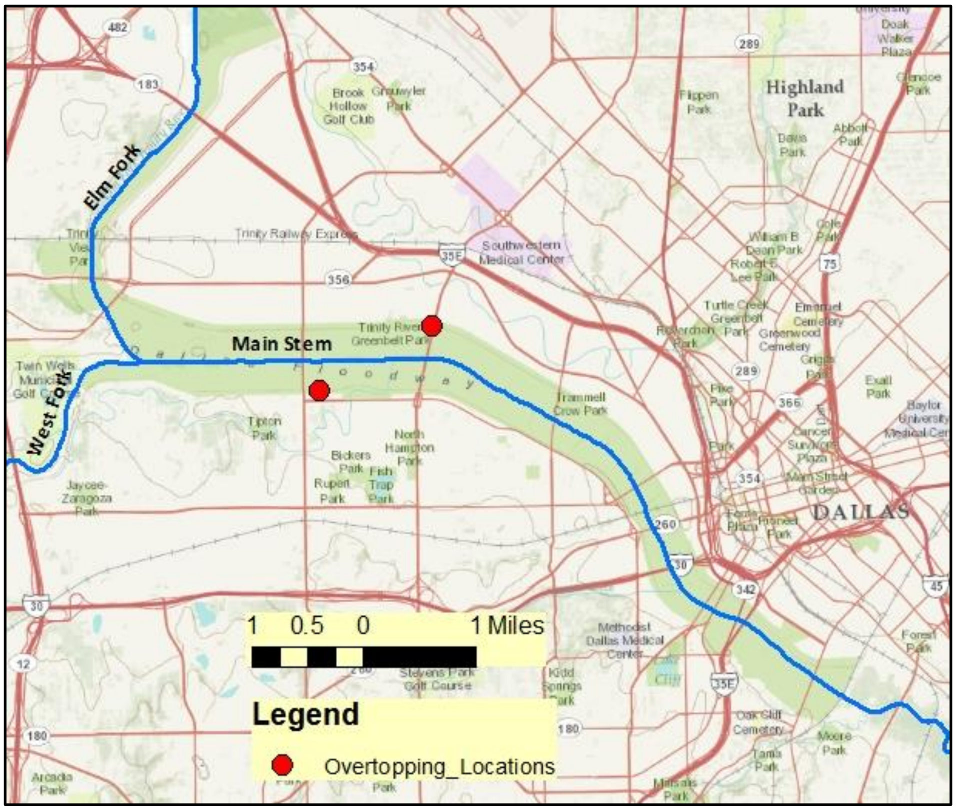


Figure 4-3 Modeled Breach Locations for Overtopping

4.4.5 Initial Sensitivity Tests on Breach Assumptions

Most of the breach methods within HEC-RAS ask the user to input the final breach dimensions and formation times, but there is a lot of uncertainty regarding these factors. Therefore, during the course of this analysis, a wide variety of breach assumptions and dimensions were tested and their results analyzed in order to determine the reasonableness of the assumptions and the sensitivity of the flood depths toward differing assumptions. Initial sensitivity tests used HEC-RAS version 4.1 with user specified breach dimensions and formation times. Within those tests, breach bottom widths were varied from 80 feet to 1,300 feet. Breach formation times were varied from 1.5 hours to 30 hours. The effects of moving the breach location upstream or downstream were tested, as well as the effects of breaching the East and West Levees separately or together. A summary of the effects of these assumptions on interior flood depths is given in Figures 4-4 and 4-5 below.

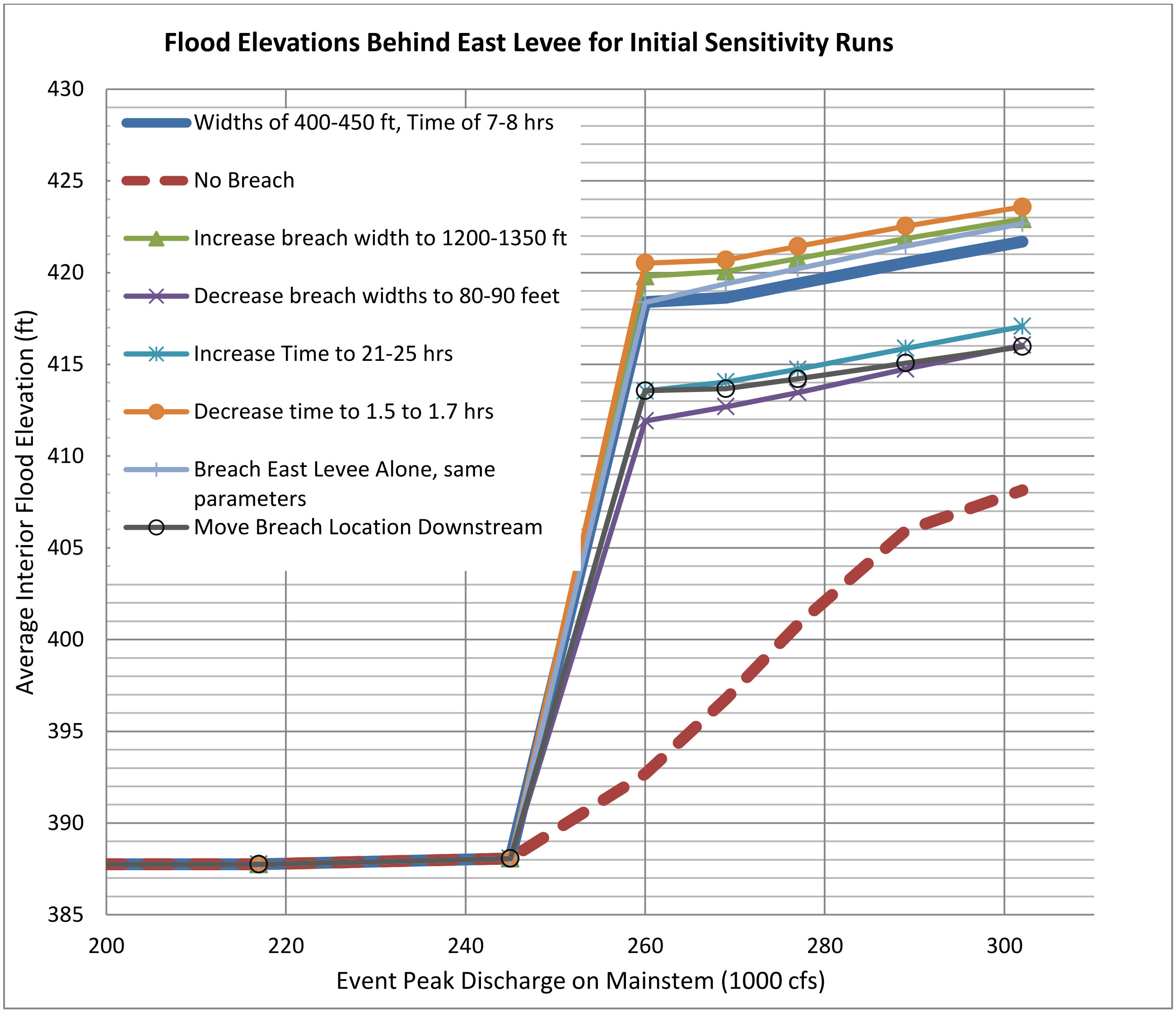


Figure 4-4 Initial Sensitivity Runs on the East Levee

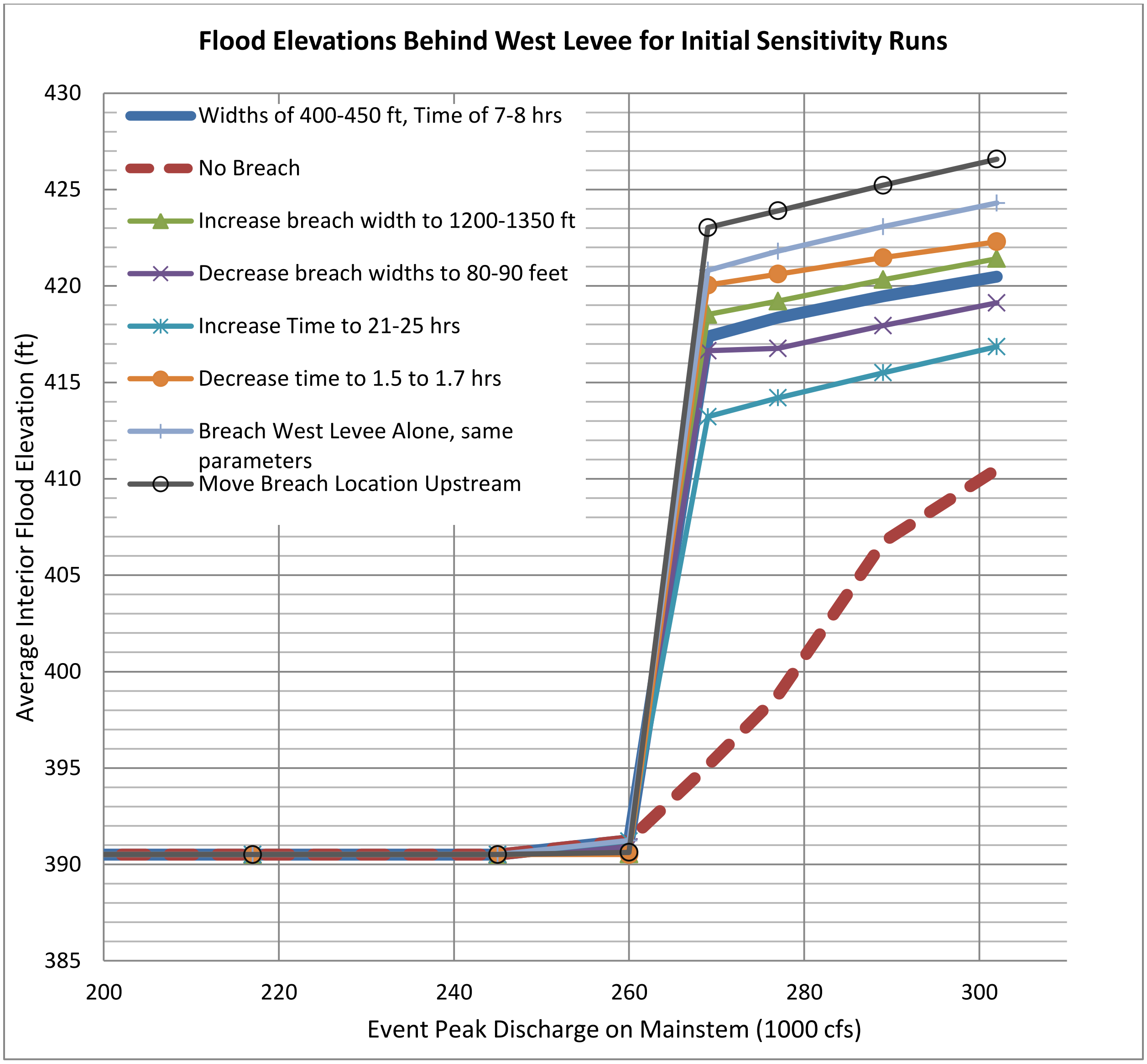


Figure 4-5 Initial Sensitivity Runs on the West Levee

After these initial tests, the results were examined and the in-progress-review team was asked for input on narrowing what the expected range of breach widths and formation times should be. After some discussion, the group came to a consensus on using a minimum breach width of 130 feet and a maximum of 400 feet. This decision was based on information from the velocity and flow hydrographs from HEC-RAS, the expected soil conditions of the levees, and engineering judgment. For formation time, the team decided to use the velocity through the breach as a guide. A velocity of 8 feet/sec was considered the threshold for erosion of the levee. Once velocities dropped below that threshold, the breach formation would be assumed to be complete. Formation times were calculated by varying the formation time until it matched the time when the velocities dropped below 8 feet/sec.

4.4.6 Breach Modeling Options in HEC-RAS 4.2

Following a proposal by the team, HEC-RAS version 4.2b (July 19, 2012) was used to complete the final analysis. This version was proposed by team members from the HEC, RMC, and MMC as being an improvement over the current 4.1 version to model levee breaches. HEC-RAS 4.2b was also used in the Risk Assessment. The current 4.1 version of HEC-RAS is limited to the user specifying the final breach width and breach formation time. HEC-RAS 4.1 also does not have a way to differentiate the horizontal formation time from the vertical formation time. For example, if a 20-foot tall levee is modeled to breach to a 200-foot bottom width with a 10-hour formation time, it would basically result in a 2 feet/hour vertical downcutting rate and a horizontal rate of 20 feet/hour. In reality, the breach would erode to the toe within a relatively short time period, and then continue to widen after that. The HEC-RAS 4.1 method would result in a slower breach growth than is likely in the vertical direction, thus underestimating the consequences of the levee breach to the protected area.

HEC-RAS 4.2b offers two additional methods to modeling levee breaches. One method is referred to as the “User Entered Data”. This method is most similar to the 4.1 method in that it requires the user to specify the breach width and formation time. The main difference is that it has added the option to differentiate between the vertical and horizontal breach progression by specifying a Vertical/Horizontal Growth Ratio. From the above example, if the user believes in a 10 hour horizontal formation time and a 2 hour vertical formation time, they could specify a Ratio of 0.2 (2/10=0.2).

Another method within HEC-RAS 4.2b is the “Simplified Physical.” This is the method that was used for the final results of this analysis. This method requires the user to specify an erosion rate (feet/hour) by inputting a table of erosion rates relative to various velocities. HEC-RAS will then look at the velocity through the breach at each computational time step, compare it to the erosion rate versus velocity table, and then compute how much and how fast the levee breach will grow. The breach will continue to grow until it reaches the user entered maximum values or until the velocities are no longer high enough to cause erosion. Both of these methods were tested with the range of breach widths (130 to 400 feet) that were recommended by the review team.

4.4.7 Testing the User Specified Breach Method in HEC-RAS 4.2b

The first method used what is referred to as the “User Specified” method for breaching in HEC-RAS 4.2b. Using this method, the user is responsible for specifying the final breach bottom widths and breach formation times that are used for the breach analysis, similar to the version 4.1 analysis. The breach widths were entered from the agreed upon range of values of 130 feet for the 260k lowest overtopping event and 400 feet for the 302k highest overtopping event. The values for the intermediate events were interpolated based on the maximum non-fail depths of water over the levee. The breach formation time was then adjusted until it matched the time when the average velocity through the breach fell below 8 feet/sec, from which it was assumed the breach would not continue to grow. The vertical/horizontal growth ratio was adjusted so that the breach would reach the toe in approximately 2 hours. The results from using the user specified breach method are in Table 4-4.

The results in Table 4-4 show that the West Levee has much shorter breach formation times than the East Levee (8-10 hours versus 20-30 hours). This is caused by the fact that there is less storage volume behind the west levee, so it fills up faster and slows velocities sooner. Because of this phenomenon, one would also expect the West Levee to have smaller breach widths than the East Levee, but these results do not reflect any difference in widths between the levees.

Table 4-4 Results of the User Specified 130 to 400 foot Breach Widths in HEC-RAS 4.2b

Flow Event:	**East Levee**			**West Levee**		
	Fixed Bottom Width (feet)	*Calculated Formation Time (hrs)*	*Avg WS Elev Behind Levee (feet)*	*Fixed Bottom Width (feet)*	*Calculated Formation Time (hrs)*	*Avg WS Elev Behind Levee (feet)*
245k	-	-	-	-	-	-
260k	130	30	412	-	-	-
269k	230	28	414.9	-	-	-
277k	300	26	416.9	300	10	422
289k	360	23	419	360	9	423.8
302k	400	21	420.5	400	8.5	425.1

4.4.8 Testing the Simplified Physical Breach Method in HEC-RAS 4.2b

The second method that was tested is referred to as the “Simplified Physical” method, and it uses an estimate of erosion rates caused by water flowing through a breach. Using this method, the user is responsible for specifying the rates of erosion by entering a table for velocity versus erosion rate (both downcutting and widening). Ideally, these erosion rates would be determined with input from Geotechnical Engineers. In lieu of actual erosion rates from Geotechnical Engineers, two different sets of erosion rates were back calculated based on assumed range of bottom widths for the East levee. It should be noted that these rates would be different if the West Levee was used to back calculate the erosion rate, since its formation time is significantly different from the East Levee. However, the East levee is expected to experience the greatest range of breach widths since it has more available volume behind it.

Based on the specified erosion rates, HEC-RAS will then compute the breach width and the breach formation time. This method improves the consistency of the results as both levees will have the same rates of erosion, but will have different breach widths due to the different time periods that each levee will experience erosive velocities (above 8 feet/sec).

Erosion Rate 1 was back calculated from a 400-foot bottom width for the 302k Event, and it represents the maximum expected rate of erosion. A velocity of 8 feet/sec was assumed as the threshold velocity below which no significant widening would occur. The rate was derived by iteratively modifying the erosion rate until the final width on the East Levee reached 400 feet during the 302k inflow event. The final rate was then used for the East and West Levee for all five overtopping events (260k, 269k, 277k, 289k, 302k).

Similarly, Erosion Rate 2 was back calculated from a 130-foot bottom width for the 260k Event, and it represents the minimum expected rate of erosion. A velocity of 8 feet/sec was assumed as the threshold velocity below which no significant widening would occur. The rate was derived by iteratively modifying the erosion rate until the final width on the East Levee reached 130 feet during the 260k inflow event. The final rate was then used for the East and West Levee for all five overtopping events (260k, 269k, 277k, 289k, 302k).

Table 4-5 shows the final erosion rate versus velocity tables that were calculated for Erosion Rates 1 and 2. The resulting widths and formation times from Erosion Rate 1 are in the Table 4-6, and the results of Erosion Rate 2 are shown in Table 4-7.

Table 4-5 Calculated Erosion Rates for Expected Maximum/Minimum Breach Widths

Velocity (feet/sec)	**Erosion Rate 1 (Maximum)**		**Erosion Rate 2 (Minimum)**	
	Downcutting (feet/hour)	*Widening (feet/hour)*	*Downcutting (feet/hour)*	*Widening (feet/hour)*
0	0	0	0	0
8	0	0	0	0
11	**21**	**21**	**7.1**	**7.1**
20	85	85	28.4	28.4

Table 4-6 Results from Erosion Rate 1 (Faster Rate of 21 feet/hour)

Flow Event:	**East Levee**			**West Levee**		
	Calculated Bottom Width (feet)	*Calculated Formation Time (hours)*	*Avg WS Elev Behind Levee (feet)*	*Calculated Bottom Width (feet)*	*Calculated Formation Time (hours)*	*Avg WS Elev Behind Levee (feet)*
245k	-	-	-	-	-	-
260k	345	19	417.5	-	-	-
269k	367	18	418.7	-	-	-
277k	381	17	419.5	0	0	-
289k	384	19	419.9	259	9	422.5
302k	402	19	421.1	274	10	423.7

Table 4-7 Results from Erosion Rate 2 (Slower Rate of 7 feet/hour)

Flow Event:	**East Levee**			**West Levee**		
	Calculated Bottom Width (feet)	*Calculated Formation Time (hours)*	*Avg WS Elev Behind Levee (feet)*	*Calculated Bottom Width (feet)*	*Calculated Formation Time (hours)*	*Avg WS Elev Behind Levee (feet)*
245k	-	-	-	-	-	-
260k	129	30	412.8	0	0	
269k	148	30	414	0	0	
277k	149	33	414.6	114	16	419.1
289k	166	31	416.1	126	16	420.8
302k	179	30	417.5	133	16	422.1

Both erosion rates calculated a narrow band of breach widths for the five inflow events. With the faster rate of 21 feet/hour, the breach widths on the East Levee varied between 350 and 400 feet, and with the slower rate of 7 feet/hour, they ranged between 130 and 180 feet. This supports the idea that the hydraulic head over the levee doesn't vary much for the range of overtopping events being modeled; therefore, their resulting breach widths also have little variance from each other.

The Erosion Rate method also takes into account the time it takes to fill the available volume behind the East versus West levee, so it calculated shorter formation times and smaller breach widths for the West Levee. Based on these results, the team decided that the Simplified Physical method gave the most consistent results for the levee breaches, and it should be the method used for the final results. The question then became what was the appropriate erosion rate to use as the best estimate for the levee breaches.

4.4.9 Breach Erosion Rates

Ideally, the erosion rates should be estimated by Geotechnical Engineers based on the specific soil conditions of the levees at the expected overtopping locations. H&H reached out to the district's Geotechnical Engineering department in order to obtain those estimates. The district geotechnical engineer then, in turn, contacted geotechnical experts at RMC and ERDC. No standard or published rates could be found from these contacts. The experts at ERDC recommended using the USDA's program WINDAMB to estimate erosion rates as the best available tool. Based on these recommendations, a member of the district's geotechnical department could run WINDAMB with soil characteristics based on the district's borings. It is expected that these soil characteristics will show considerably more sand than was accounted for in the Risk Assessment WINDAMB analysis. The WINDAMB analysis is not expected to produce an exact estimate of erosion rates, but an expected range that could be used to verify or modify the results of this analysis.

In the meantime, H&H was still tasked with submitting a best estimate of breaching for baseline conditions with the information that is currently available. The results of Erosion Rates 1 and 2 were discussed with the in-progress review team. During this discussion, logical arguments could be made in favor of both the faster (21 feet/hour) and slower (7 feet/hour) rates, but it was ultimately left to the district to make the final judgment call. In the end, district H&H staff decided to use an "average" erosion rate of 14 feet/sec for the final baseline runs. The final erosion rates used in this analysis are shown in Table 4-8 below.

Table 4-8 Final Breach Erosion Rates for Baseline Conditions

Velocity (feet/sec)	**Final Erosion Rates**	
	Downcutting (feet/hour)	*Widening (feet/hour)*
0	0	0
8	0	0
11	14	14
20	56	56

4.4.10 Results from Final Breach Assumptions

The final baseline hydraulic runs used the "Simplified Physical" method using an "average" erosion rate of 14 feet/hour. The results of this erosion rate are shown in Table 4-9 below and in the example hydrographs in Figures 4-6 and 4-7. The breach invert for these final runs was assumed to be at the toe of each levee (Elevation 399 feet for the West levee, and 405 feet for the East Levee). Final breach widths and formation times were calculated within the program based on the erosion rate versus velocity table that was input into the model (see Table 4-8 above). In Table 4-9, cells with a dash line did not reach the breach trigger and thus did not breach. Note that the flow events in this table are designated by their total peak flow at the Commerce Street Gage (in 1000 cfs).

Table 4-9 Results from the Final Breach Erosion Rates

Flow Event:	**East Levee**			**West Levee**		
	Calculated Bottom Width (feet)	*Calculated Formation Time (hours)*	*Avg WS Elev Behind Levee (feet)*	*Calculated Bottom Width (feet)*	*Calculated Formation Time (hours)*	*Avg WS Elev Behind Levee (feet)*
245k	-	-	-	-	-	-
260k	253	23	416	-	-	-
269k	272	22	417.1	-	-	-
277k	285	21	418	0	0	-
289k	291	23	418.7	205	12	422
302k	307	23	419.9	215	12	423.2

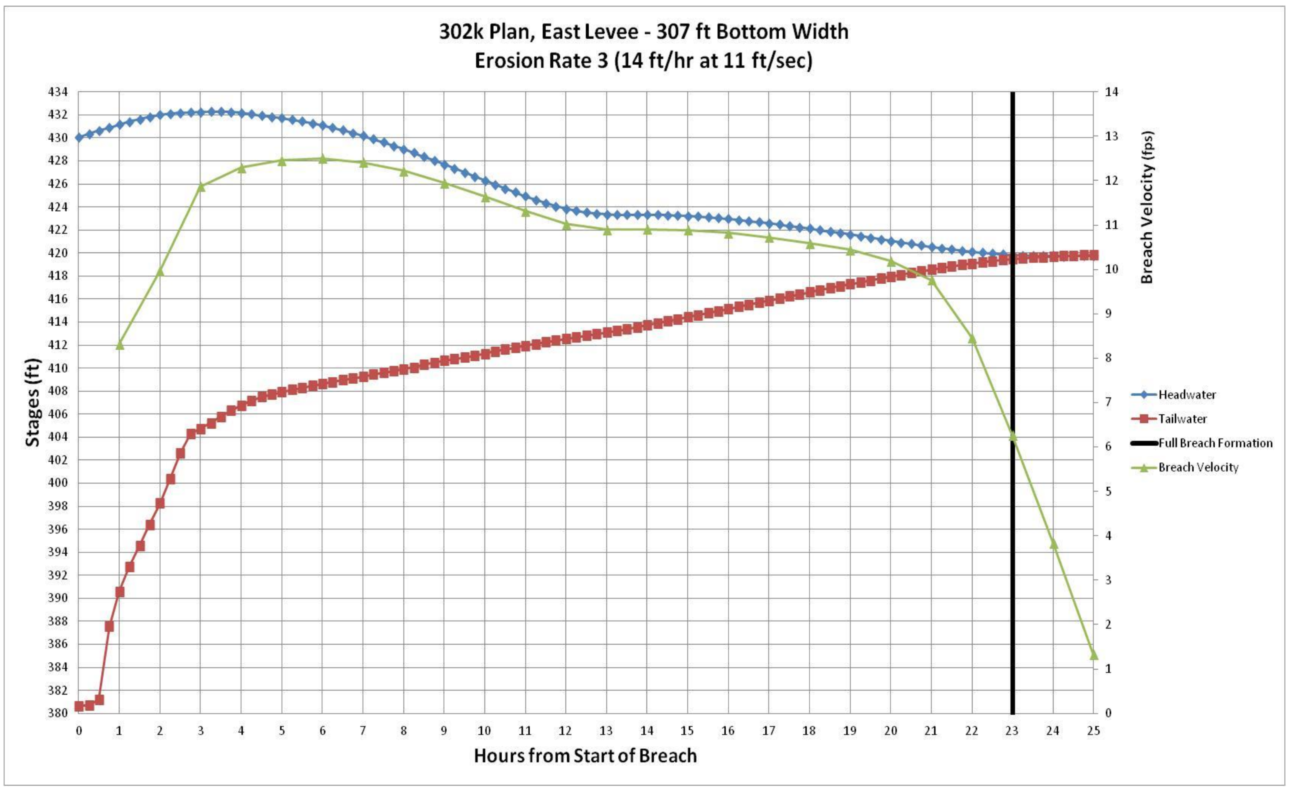


Figure 4-6 Final Breach Hydrographs on the East Levee for the 302k Overtopping Event

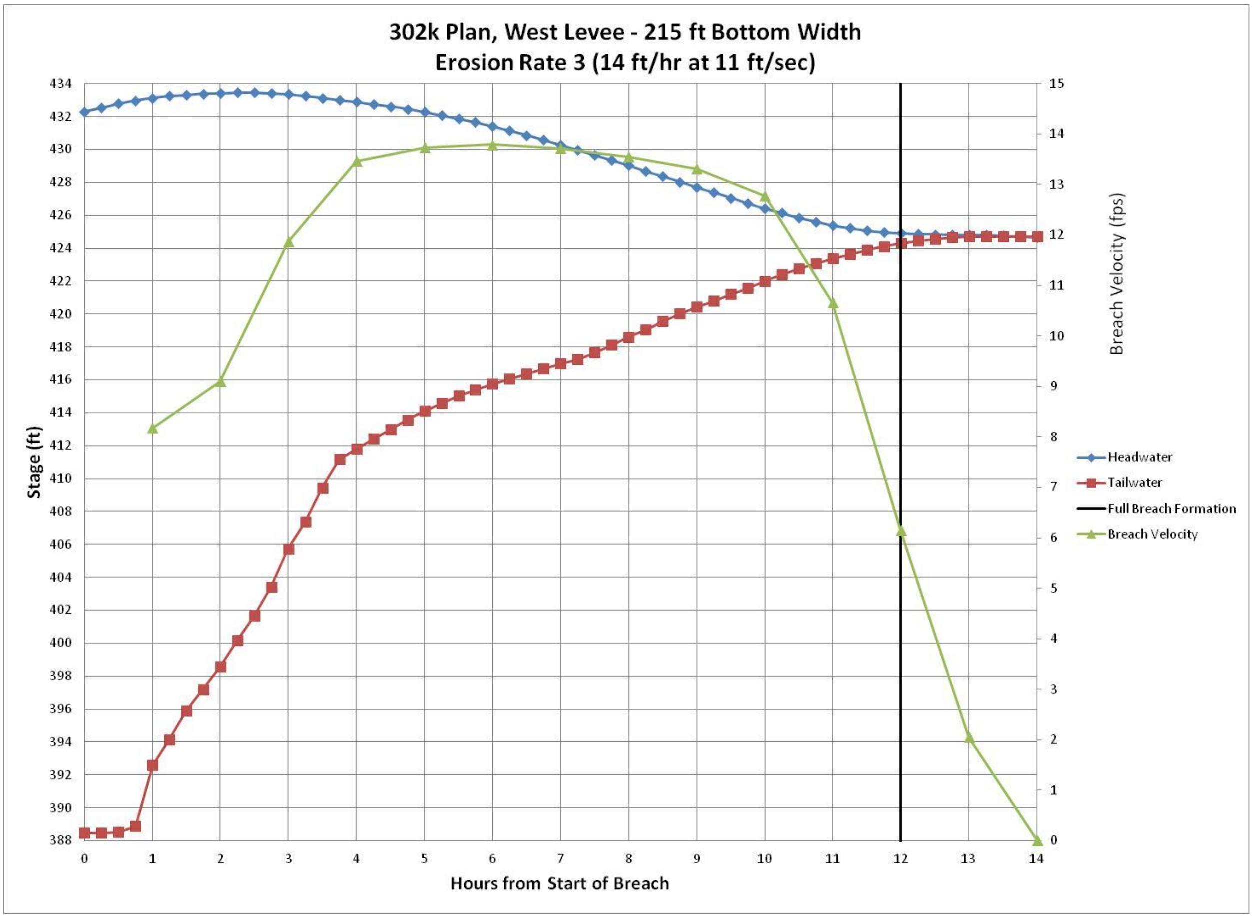


Figure 4-7 Final Breach Hydrographs on the West Levee for the 302k Overtopping Event

4.5 FINAL HYDRAULIC RESULTS FOR BASELINE CONDITIONS

4.5.1 Flooding Depths for With and Without Breach Conditions

Final HEC-RAS runs were made for both the nonfailure (no breach) and failure (with breach) conditions of the overtopping events. The maximum depth of flooding from the without breach nonfail scenarios was 27 feet for both the East and West Levee. The maximum depth of flooding from scenarios with the final breach assumptions was 40 feet for both the East and West Levee. The average flooding elevations within the protected areas with and without levee breach are shown in Figure 4-8, and the flooding elevations for each HEC-RAS storage area are shown in Tables 4-10 through 4-13.

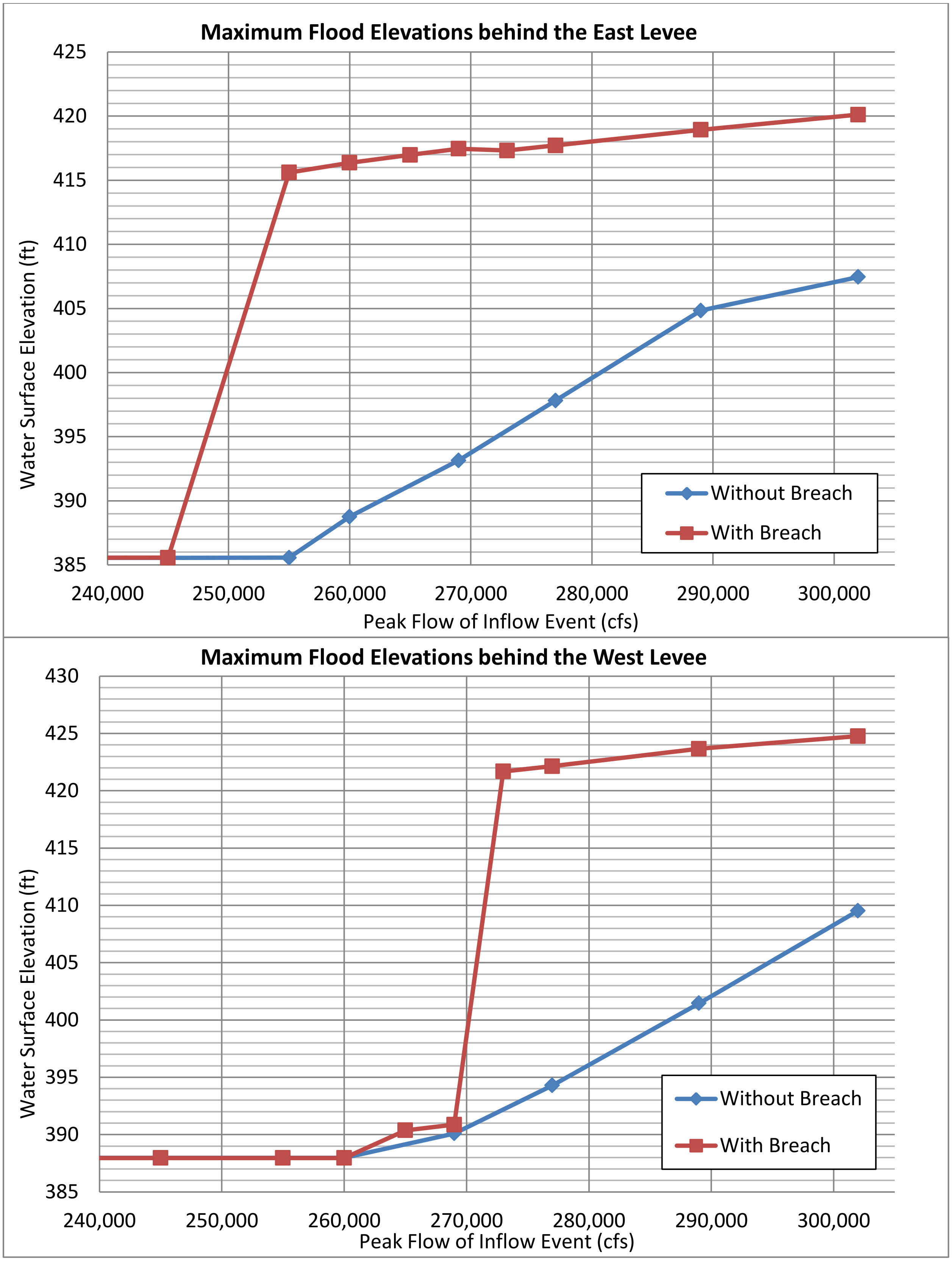


Figure 4-8 Interior Flood Elevations With and Without Breach for Baseline Conditions

Table 4-10 East Levee Storage Area Elevations Without Breach for Baseline Conditions

Event	*120k*	*217k*	*245k*	*260k*	*269k*	*277k*	*289k*	*302k*
SA ID	Storage Area Elevations (feet)							
610	389.7	389.7	389.7	389.7	398.0	402.1	405.3	407.6
611	390.1	390.1	390.1	390.1	390.1	398.5	405.8	407.7
612	391.4	391.4	391.4	391.4	391.4	398.8	405.8	408.1
613	385.4	385.4	385.4	389.7	398.0	402.1	405.3	407.6
614	385.8	385.8	385.8	385.8	385.8	385.8	404.9	407.6
615	380.7	380.7	380.9	389.8	398.0	402.1	405.3	407.6
616	383.0	383.0	383.0	389.8	398.3	402.1	405.3	407.6
617	389.2	389.2	389.2	393.2	398.9	402.1	405.3	407.6
618	379.9	379.9	379.9	384.8	386.5	392.3	402.7	406.6
619	380.3	380.3	380.3	383.3	386.5	392.3	402.7	406.6

Table 4-11 East Levee Storage Area Elevations With Breach for Baseline Conditions

Event	*245k*	*255k*	*260k*	*265k*	*269k*	*273k*	*277k*	*289k*	*302k*
SA ID	Storage Area Elevations (feet)								
610	389.68	415.61	416.36	416.98	417.46	417.32	417.71	418.93	420.12
611	390.1	415.61	416.36	416.98	417.46	417.32	417.71	418.93	420.12
612	391.39	415.61	416.36	416.98	417.46	417.32	417.71	418.93	420.12
613	385.43	415.61	416.36	416.98	417.46	417.32	417.71	418.93	420.12
614	385.77	415.61	416.36	416.98	417.46	417.32	417.71	418.93	420.12
615	380.91	415.62	416.36	416.98	417.46	417.32	417.71	418.93	420.12
616	382.99	415.6	416.36	416.98	417.46	417.32	417.71	418.93	420.12
617	389.18	415.6	416.36	416.98	417.45	417.32	417.71	418.93	420.12
618	379.91	415.6	416.36	416.97	417.45	417.32	417.71	418.93	420.12
619	380.25	415.6	416.36	416.97	417.45	417.32	417.71	418.93	420.12

Table 4-12 West Levee Storage Area Elevations Without Breach for Baseline Conditions

Event	*120k*	*217k*	*245k*	*260k*	*269k*	*277k*	*289k*	*302k*
SA ID	Storage Area Elevations (feet)							
603	381.6	381.6	381.6	381.6	384.6	387.9	395.7	408.6
604	384.0	384.0	384.0	384.0	384.0	387.9	395.7	408.6
605	388.7	388.7	388.7	388.7	388.7	388.7	395.7	408.6
606	384.3	384.3	384.3	384.3	393.1	401.3	406.8	409.8
607	388.5	388.5	388.5	388.5	391.6	397.5	405.3	409.8
608	391.7	391.7	391.7	391.7	391.7	397.5	405.3	409.8
609	397.0	397.0	397.0	397.0	397.0	399.2	405.7	411.5

Table 4-13 West Levee Storage Area Elevations With Breach for Baseline Conditions

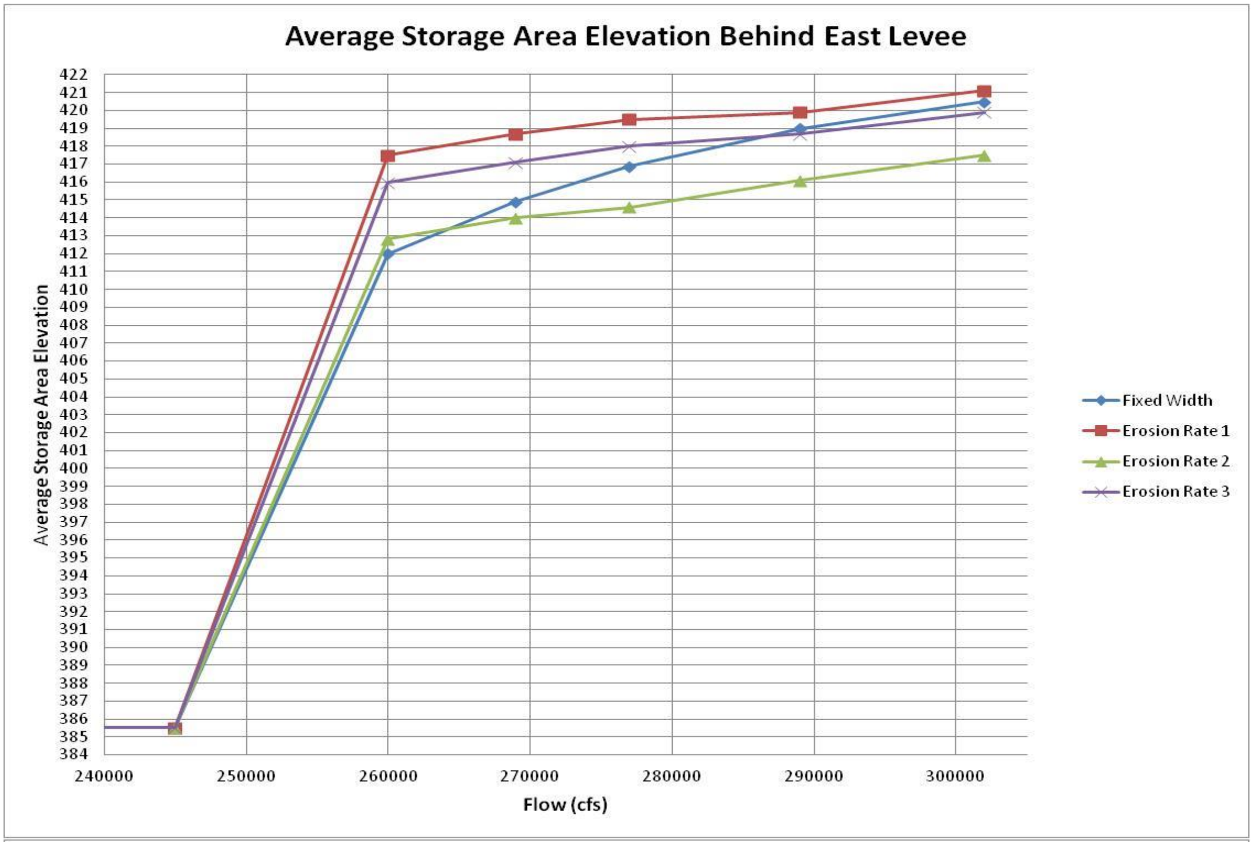
Event	*245k*	*255k*	*260k*	*265k*	*269k*	*273k*	*277k*	*289k*	*302k*
SA ID	Storage Area Elevations (feet)								
603	381.6	381.6	381.6	381.6	381.6	418.46	418.86	420.17	421.32
604	384	384	384	384	384	418.46	418.86	420.17	421.32
605	388.66	388.66	388.66	388.66	388.66	418.46	418.86	420.17	421.32
606	384.28	384.28	384.28	384.28	385.77	421.68	422.13	423.66	424.76
607	388.46	388.46	388.46	388.53	389.02	421.69	422.14	423.67	424.77
608	391.73	391.73	391.73	391.73	391.73	421.69	422.14	423.67	424.77
609	396.95	396.95	396.95	396.95	396.95	421.68	422.14	423.67	424.77

4.5.2 Uncertainty in the Results

As previously mentioned there is a large degree of uncertainty involved with levee breach analysis. Many assumptions need to be made in the process of the analysis, and there is very little existing research on which to base those assumptions. Below is a list of some of the assumptions that were made in this analysis and how those assumptions could affect the results.

- This analysis assumed a 100% probability of failure with 0.5 foot of levee overtopping. In reality, the levees may not always breach if overtopped. The difference in results between the breach and non-breach conditions was shown in Figure 4-8 to be as much as 25 feet of flood depth.
- This analysis assumed only one location for the breach on each levee at the location where overtopping first occurs. If the first overtopping location holds, the levees could breach at another location. If piping failure is considered, there could be an endless number of potential failure locations and elevations. The effects of moving the location of the breach were shown on Figures 4-4 and 4-5 to be as much as 6 feet of flood depth.
- This analysis assumed a single breach per levee. It is possible the levee could fail in multiple locations along the levee, in which case the flood depths would approach the maximum elevations shown in the sensitivity tests.
- This analysis assumed that both levees would fail together if over 0.5 foot of overtopping was reached. It is possible that only one of the levees would fail. Figure 4-5 showed that failing the levees independently could increase flood depths by as much as 4 feet.
- This analysis assumed an erosion rate (14 feet/hour at 11 feet/sec) and threshold velocity (8 feet/sec) which resulted in the computed breach widths and formation times. A different threshold velocity and/or erosion rate would produce different breach widths and formation times which could produce significantly different results. A comparison of the results for different possible erosion rates is given in Figure 4-9. Erosion Rate 1 = 21 feet/hour. Erosion Rate 2 = 7 feet/hour, and Erosion Rate 3 = 14 feet/hour.

Bi-weekly conference calls with review team members took place during the duration of this analysis to maintain transparency and accountability in the assumptions that were being made. These assumptions represent the USACE' best estimates of levee failures given the information and time available to complete the study; however the above limitations and uncertainties should be considered and understood when applying the results.



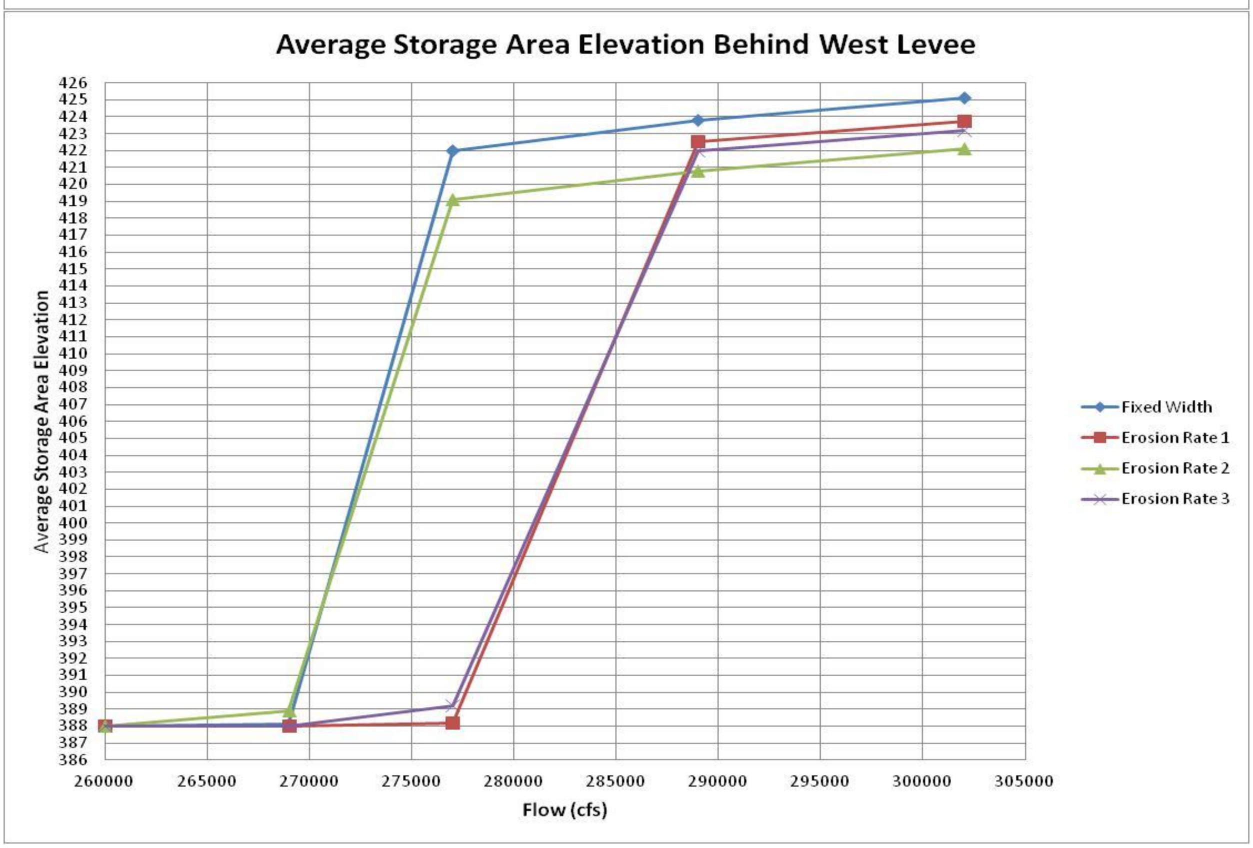


Figure 4-9 Range of Uncertainty in Flood Elevations Due to Different Erosion Rates

4.5.3 Flow – Frequency Uncertainty for HEC-FDA

Within HEC-FDA, an Equivalent Record Length of 40 years was applied. This composite estimate is based upon consideration of both the local USGS stream flow gauging record period (at the Commerce Street Gauge) and the relative level of confidence, considering the methodologies used to define the general trajectory of the extended discharge frequency curve. For watershed runoff condition homogeneity purposes, the annual peak series data at the Commerce Street Gauge was truncated at the historic point in time when the majority of the upstream flood control reservoirs had initially risen to their conservation pool levels and had begun to be operated normally. After that point in time, the annual peaks have been effectively dominated by localized runoff hydrograph peaks alone. That milestone was reached in the mid 1950s. However, rather than simply adopt an Equivalent Record Length of about 60 years (i.e. 2015 - 1955), consideration was also given to the fact that the upper end of the discharge frequency curve was subsequently, significantly adjusted, in response to collaborative guidance and recommendations from the USACE Hydrology Committee. While the lower half of the discharge frequency curve would have a relative level of confidence potentially exceeding that of a 60-year equivalent record length, the trajectory of its upper half is substantially less certain. Based upon hydrologic engineering judgment, a more moderate 40-year equivalent record length was applied.

4.5.4 Steady Flow HEC-RAS Stage Uncertainty

While the computed design water surface profile must be used as a guide for establishing the design crest profile for proposed levees, the design water surface is not an absolute, but is regarded as a most likely value derived from the best estimates of key factors, parameters and data components, which have some inherent variability or uncertainty. This most likely value for the water surface elevations was used in the risk analysis along with probability distributions representing a range of values.

The Hydrologic Engineering Center-Flood Damage Analysis (HEC-FDA) program incorporates risk analysis to compute estimated flood damages, which requires estimates for the uncertainties involved in computing the H&H input. The uncertainty estimate for computed water surface profiles was determined by estimating a standard deviation of error for input to the HEC-FDA model. This estimate was determined initially for the Dallas Floodway Extension project study, which preceded the current Dallas Floodway Feasibility Study. The Dallas Floodway Extension project reach is immediately downstream of the Dallas Floodway and has widely varying land uses within the Dallas Floodway Extension reach thus requiring more varied roughness coefficients for modeling. This is primarily due to the extensive forested floodplain areas that are within the Dallas Floodway Extension reach but are not represented in the Dallas Floodway reach. Because of the more significant variability of roughness coefficient estimates used in the modeling process and the likelihood for a wider range of water surface profile uncertainty in the Dallas Floodway Extension reach, the uncertainty estimates used for the Dallas Floodway Extension study were carried forward for use in the current Dallas Floodway study. These uncertainty estimates were obtained by developing a sensitivity analysis with the HEC-RAS existing conditions model by varying the roughness coefficients assigned to the varying land uses. Varying the roughness coefficients from the most likely values was a technique used to determine the likely range of potential water surface elevation values that may deviate from the most likely water surface profile.

A sensitivity analysis to estimate the upper and lower limits for a range of flood events was performed by adjusting the Manning's roughness coefficients for each of the land use types identified in the floodplain. The upper and lower limits of the estimated roughness coefficients were obtained primarily by using guidance from Manning’s roughness coefficient guides such as: “Open Channel Hydraulics” (Chow, 1959) and the “Guide for Selecting Manning’s Roughness Coefficients for Natural Channels and Flood

Plains" (U.S.G.S, 1989). The result was about one foot elevation difference between the upper and lower limits in the computed profiles for flood events up to the 2% AEP (50-year) flood event. This range was estimated to encompass 95% of the entire population of measured gage reading data points that could be expected for the Below Dallas Gage at Loop 12. The maximum discharge recorded at the gage is about a 2% AEP flood discharge. The difference between the upper and lower limits in the computed profiles for flood events between the 2% chance flood event and the SPF event within the project reach ranges from 1.5 to 2.0 feet. Based on these results, a standard deviation of 0.5 feet for flood events in the range of the SPF flood event was adopted since flood events of this magnitude was of primary interest in the analysis.

5.0 UNSTEADY FLOW HYDRAULIC ANALYSES FOR THE FLOOD RISK MANAGEMENT ALTERNATIVES

5.1 DESCRIPTION OF THE FLOOD RISK MANAGEMENT ALTERNATIVES

The Flood Risk Management Plan and measures to address levee-overtopping risk were developed with consideration for requirements outlined in both ER 1105-2-101 and the levee design guidance in ETL 1110-2-299. The ER states that levee design should include analysis to assure safe and predictable performance and to formulate features to manage capacity exceedance at potential least damaging locations. Also, it states that levee designs may include levee superiority at critical infrastructure such as pumping stations. The Engineering Technical Letter (ETL) 1110-2 299 (1986) entitled "Overtopping of Flood Control Levees and Floodwalls" provides guidance on designing levee systems to reduce the negative impacts of overtopping of levees since prevention of overtopping can never be absolutely assured. Some considerations for good overtopping design for flood risk management outlined in the ETL are the following:

- May allow initial overtopping to occur in a reach having the least negative impacts.
- Control the initial overtopping to reduce the impact of sudden overtopping failure or breach.
- Potentially reduce the chance of overtopping in areas of critical infrastructure.
- Potentially reduce project maintenance and replacement costs.
- Potentially reduce the risk associated with flow velocity resulting from overtopping inundation.
- Potentially reduce the risk to life loss due to altering the timing of flood inundation.

Types of measures that may be considered to address these goals are the following:

- Localize the initial overtopping in areas that have the least negative impacts.
- Design levee crest for overtopping to reduce risk of levee breaching using armoring at the crest and interior slopes.
- Use levee superiority design to control the initial overtopping location.
- Use levee flattening or similar methods to reduce the risk of levee breaching.
- Use interior area dikes or similar methods to reduce the rate of flood spreading.
- Improve levee access for flood fighting.

The following measures were developed to address the levee overtopping risk identified in the Risk Assessment. Measures that lower water surface elevations for high flood events provide benefits by reducing the frequency of levee overtopping. Measures that raise the levee crest height also reduce the frequency of overtopping but may also provide benefits to the protected area by altering the rate of overtopping flow. Measures that armor the levees provide benefits by preventing or delaying the initiation of a levee breach. Evaluation of these measures is expected to show reduction in damages and loss of life estimates under "with-project" conditions.

From these general measures, specific project alternatives were developed and evaluated to address the levee overtopping with breach failure mode. The types of project alternatives evaluated for levee overtopping included (1) Modification of the AT&SF Bridge, (2) Levee Height Modifications, (3) Armoring, and (4) Controlled Overtopping by Notching the Levee.

Internal erosion due to seepage at the levee foundation was also identified in the Risk Assessment as a risk near to exceeding the tolerable risk guideline. Therefore, another project alternative, (5) seepage cut-off walls and clay lining on the river side of the East and West Levees, was evaluated for reduction in risk for that failure mode.

For all of the proposed alternatives, the Dallas Floodway Feasibility Study HEC-RAS Unsteady Flow model was used to estimate the inundation levels associated with the various overtopping and internal erosion flood events that were evaluated. The HEC-FDA model was then used to estimate the with-project conditions reduction in economic damages resulting from the change in inundation levels.

It should be noted that this plan formulation analysis for the FRM alternatives to determine the NED Plan is based on the impacts of interior levee flooding resulting from levee overtopping and subsequent breach or levee internal erosion failure caused by riverine flooding only. This analysis is independent of the Interior Drainage Plan for either the Without-Project conditions or the With-Project conditions and does not include any operational benefits or costs that may be associated with the IDP.

5.2 AT&SF Bridge Modification Plan

The abandoned AT&SF Railroad Bridge spans the main stem of the Trinity River and is located toward the downstream end of the Dallas Floodway. The bridge was taken out of service and abandoned as a railroad bridge in the 1990s when the Dallas Area Rapid Transit (DART) system purchased the bridge and right-of way and constructed a new light rail system bridge parallel to the old bridge. At that time, the rails and cross ties were removed from the bridge deck and the remainder of the structure was abandoned in place.

The current AT&SF Bridge structure consists of, from left to right looking downstream (east to west), a 1,414 foot wood pile trestle, a 208 foot steel truss (over the river channel), a 180 foot steel pile trestle, a 453 foot earth embankment, a 660 foot section of concrete piers and deck spans, and another 385 foot earth embankment. Total length of the floodway crossing is 3,300 feet. The river channel segment through the bridge is concrete lined on the side slopes. The wood trestle pier bents have been modeled as 1.3 feet in width with a typical 14-foot spacing. Piers supporting the steel truss span have been modeled as 8 feet in width. Concrete bridge piers have been modeled as 2.0 feet in width with 33 foot spacing. Ineffective flow areas have been used in both the upstream and downstream cross sections to model the effects of the earth embankments. Figure 5-1 shows the current AT&SF Railroad Bridge as modeled in the baseline conditions HEC-RAS geometry. This figure includes the modifications to the bridge as a result of construction for the Santa Fe Trestle Trail Project.

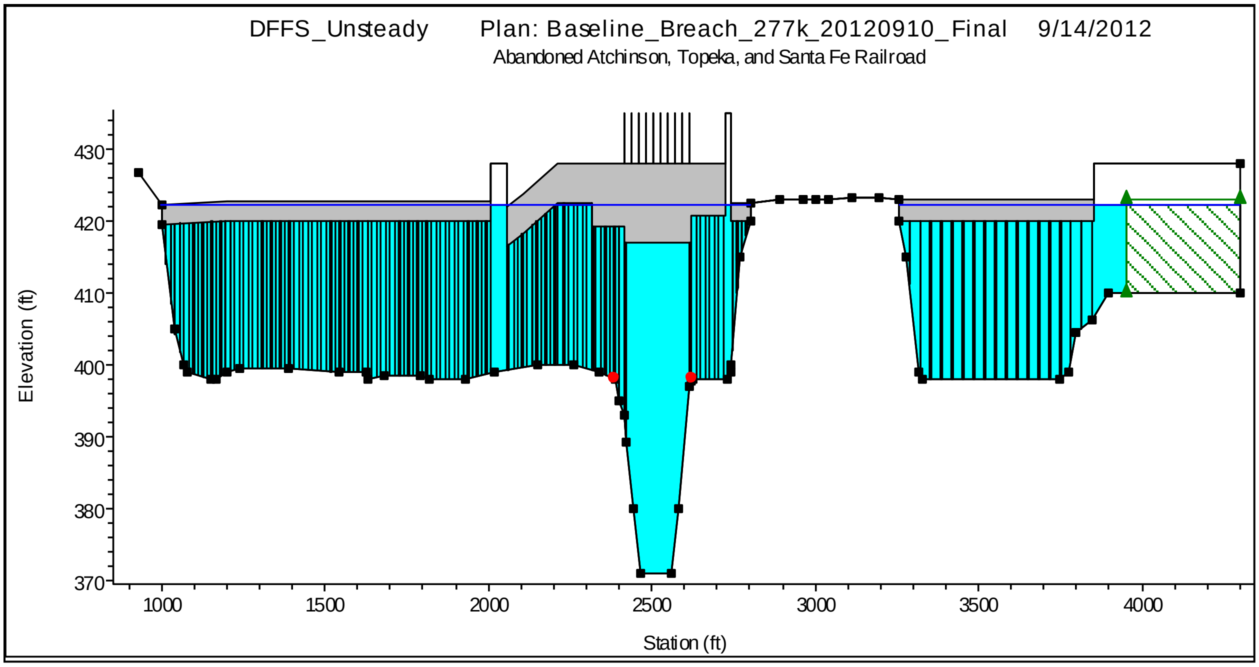


Figure 5-1 AT&SF Railroad Bridge in Baseline HEC-RAS Geometry

A proposed FRM alternative is for the removal of major portions of the abandoned AT&SF Railroad Bridge referred to as the AT&SF Bridge Modification Plan. The modification plan is not a complete removal of the original AT&SF Bridge components but includes the removal of portions of the original bridge that are not part of the Santa Fe Trestle Trail project. The following is a description of AT&SF Bridge Modification Plan from the left end of the bridge to the right looking downstream. In this plan, a 1,050-foot section of the wood trestle bridge in the left overbank is proposed to be removed. A 350-foot section of the wood trestle bridge adjacent to the steel truss bridge section on the left overbank, the steel single span truss bridge over the river channel with its large supporting piers, and a 100 foot section of the steel trestle bridge section on the right bank side of the steel truss bridge section are to remain as components of the Santa Fe Trestle Trail. The remaining 60-foot section of the existing steel trestle bridge on the right overbank, a 570-foot section of embankment, the 660-foot section of concrete railroad bridge, and a temporary earth embankment upstream of the DART Bridge are proposed to be removed. The temporary earth embankment upstream of the DART Bridge was placed during the construction of the Santa Fe Trestle Trail project to achieve hydraulic neutrality upon removal of the upper portion of the extreme right bank AT&SF bridge embankment as part of the Santa Fe Trestle Trail project. The lower portion of the old AT&SF railroad embankment at the right bank is to remain as part of the Santa Fe Trestle Trail.

The Modification Plan for the AT&SF Bridge lowers the water surface elevations for high flood events compared to the existing bridge. The reduction in the water surface elevation due to the removal of portions of the bridge affords flood damage reduction benefits for the Dallas Floodway while preserving much of the historic and beneficial aspects of the abandoned bridge by retaining the portions of the bridge that are part of the Santa Fe Trestle Trail project. The partial removal of the AT&SF Bridge results in lowering the 277,000 cfs flood event by approximately 1 foot upstream of the bridge. This change is without consideration for debris impacts. The bridge in its current condition poses a more significant risk due to the potential for debris accumulation in a major flood event. While the floodway has not experienced flood events sufficient to overtop the bridge, most significant observed flood events result in accumulation of large woody debris on the structure that require removal by maintenance crews.

5.2.1 Modeling Methodology

The AT&SF Bridge Modification Plan was modeled by removing portions of the bridge structure (BR 108287) from the HEC-RAS geometry for the Dallas Floodway Feasibility Study. Figure 5-2 shows the ATS&F Bridge as modeled in the with-project conditions HEC-RAS geometry. This figure includes both the portions of the existing bridge that are retained as part of the Santa Fe Trestle Trail project as well as the modification features of the Santa Fe Trestle Trail project. The same range of unsteady flow events was run with the modified HEC-RAS geometry, and the breach settings were also left the same as in baseline conditions (i.e. if the water surface rose 0.5 feet or more above the top of the levee, then a breach would be initiated).

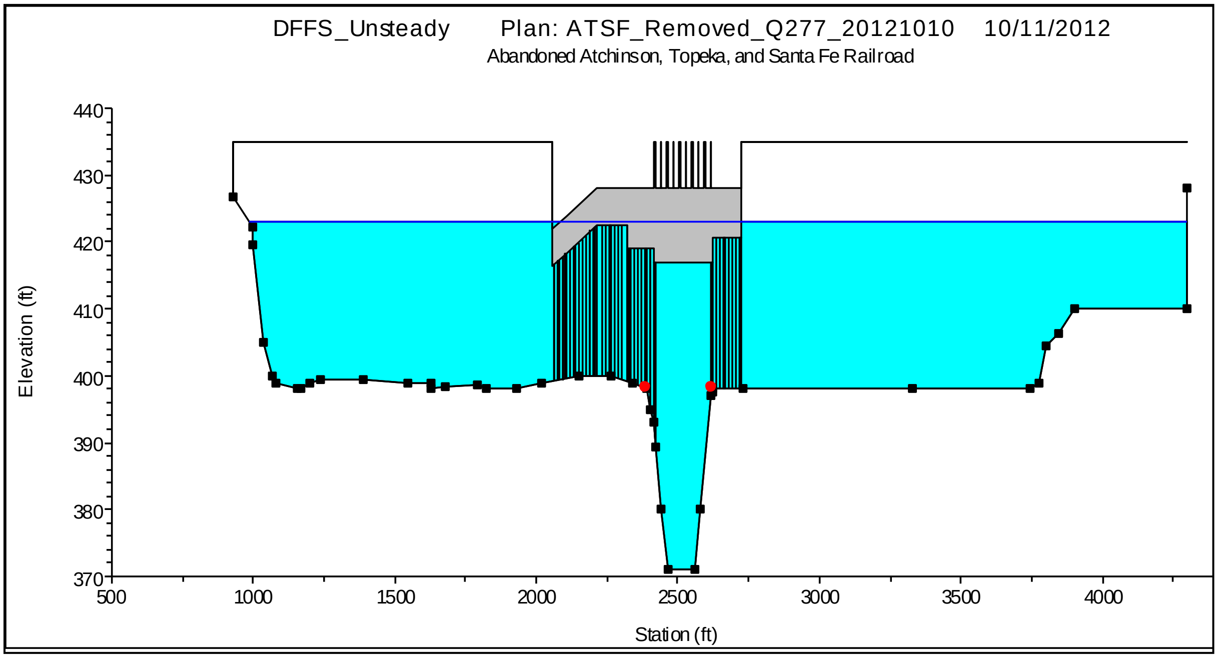


Figure 5-2 AT&SF Railroad Bridge in the With-Project HEC-RAS Geometry

5.2.2 Modeling Results

After running the eight flood events with the AT&SF Bridge removal geometry, the resulting water surface elevations on the interior of the levees were compared with baseline conditions. The results showed that the removal of the AT&SF Bridge did lower water surface elevations on the river, but the effects were relatively small at the location of the breach for overtopping. Just upstream of the bridge, the maximum water surface was lowered by 0.6 to 0.75 foot, but at the location of the overtopping breach, which is over 5 miles upstream of the bridge, the river's water surface elevations were only lowered by 0.1 to 0.2 feet. The resulting water surface elevations on the interior of the levees were lowered by less than half a foot in most cases, as shown in Table 5-1.

Table 5-1 Maximum Water Surface Elevations Behind the Levees (feet) for the AT&SF Bridge Removal Plan (Assuming No Debris Accumulation)

	Baseline Conditions		With AT&SF Bridge Removed		Difference With Project	
Event (1000 cfs)	*East Levee WS Elev (feet)*	*West Levee WS Elev (feet)*	*East Levee WS Elev (feet)*	*West Levee WS Elev (feet)*	*East Levee WS Elev (feet)*	*West Levee WS Elev (feet)*
217	385.54	387.95	385.54	387.95	0.00	0.00
245	385.56	387.95	385.54	387.95	-0.03	0.00
255	415.61	387.95	415.00	387.95	-0.61	0.00
260	416.36	387.95	415.93	387.95	-0.43	0.00
265	416.98	390.37	416.58	390.36	-0.40	-0.02
269	417.46	390.87	417.08	390.38	-0.38	-0.49
273	417.32	421.69	416.98	420.11	-0.34	-1.58
277	417.71	422.14	417.35	421.70	-0.36	-0.44
289	418.93	423.67	418.59	423.43	-0.34	-0.24
302	420.12	424.77	419.77	424.55	-0.35	-0.22

5.2.3 Possible Effects of Debris Accumulation on the AT&SF Bridge

The water surface elevations in Table 5-1 do not account for possible debris accumulation on the existing bridge structure. In its current configuration, the AT&SF Bridge represents a further risk to the levee system due to possible debris impoundment on the structure in the event of a major flood. The bridge's closely spaced piers with cross bracing have the potential to accumulate large amounts of debris during flood events. Debris accumulation during a major flood event could result in higher flood levels upstream of the bridge than those computed without debris impacts and result in greater levee overtopping into the protected areas behind the levees. Debris blockage on the bridge structure would also increase the likelihood of a levee breach due to overtopping. Additional flood damages expected due to debris accumulation were not accounted for in the previously described hydraulic results for the AT&SF Bridge removal plan.

An additional hydraulic analysis to determine potential debris accumulation impacts for the baseline AT&SF Bridge was developed to support economic justification for the AT&SF Bridge Modification Plan. Significant debris accumulation on the AT&SF Bridge has been observed following most of the higher flood events on record but was most notable after the May 1990 flood event. This flood event is the flood of record following the construction of the flood control reservoirs upstream. Since some flood high watermarks were available for the 1990 flood event both upstream and downstream of the bridge, a calibration of the bridge debris impact for this flood event was performed using the current Dallas Floodway Existing Conditions model. The recorded peak flow of 82,300 cfs for the 1990 flood event was used with the high watermarks to approximate the debris impact on the bridge. The water surface profile was computed for the river reach from downstream of the AT&SF Bridge, upstream to the USGS Gage at the Commerce Street Bridge. Two high watermarks from the 1990 flood event plus the peak flow recorded stage at the USGS Gage were used to perform the calibration for the debris impact at the bridge. Since conditions downstream of the AT&SF bridge have changed since 1990 and are reflected in the current Dallas Floodway Existing Conditions HEC-RAS model, a "known water surface" elevation was input to the HEC-RAS debris analysis model to reflect a high watermark from the 1990 flood event that was located just upstream of the Martin Luther King (MLK) Jr. Blvd Bridge which is located a short distance downstream of the AT&SF bridge. The downstream high watermark at elevation 413.1 feet was input to the HEC-RAS model at cross section 107551 and is representative of downstream tailwater conditions that existed during the 1990 flood event. Another high watermark located just downstream of the Corinth Street bridge was at elevation 414.0 and the peak flow recorded stage at the USGS gage near the Commerce Street bridge was 47.1 feet, which converts to elevation 415.1 feet. The HEC-RAS calibration model to approximate the debris impact for the 1990 flood event was developed using the floating debris method that is available in HEC-RAS version 4.1.0. This method basically widens each bridge pier that the user selects up to a specified width and depth. The specified depth of the pier widening is applied from the computed water surface elevation down to an elevation that matches the user-selected depth of debris accumulation. HEC-RAS has the option to input different bridge pier sizing and locations at the upstream and downstream sides of the bridge model. However, the pier widening for the floating debris method is only applied to the upstream side of the bridge piers for the computations.

Several debris blockage model settings were tested, and a pier widening setting of 10 feet width and 10 feet depth appeared to match the high watermarks adequately. The computed water surface at the downstream side of the Corinth Street bridge was 414.07 (HWM elev. 414.0) and the computed water surface at the USGS gage location at the downstream side of the Commerce Street Bridge was 415.41 (Gage reading elev. 415.10).

The purpose of this debris analysis was to determine if debris impacts would have economic feasibility to support the bridge modification plan. For this analysis, the same assumption of debris accumulation for the 1990 flood event was used to determine the debris impacts for the levee overtopping flood events. No assumption of debris accumulation escalated for extreme flood events above that determined for the 1990 flood event was used. Therefore, the same HEC-RAS bridge pier widening setting at the AT&SF Bridge for the 1990 flood event debris impact was used in the steady flow analysis of high flow events and also for the unsteady flow analysis for the levee overtopping flood events to determine the economic benefits of the AT&SF Bridge Modification Plan for comparison to the existing AT&SF Bridge with debris impacts. The AT&SF Bridge as modeled with possible debris blockage using the floating debris technique is shown in Figure 5-3.

The results of the HEC-RAS steady flow analysis for the floating debris impacts at the AT&SF bridge indicated that for the 277,000 cfs flood event, the water surface rise upstream from the bridge was approximately 1.1 feet higher than for the existing bridge without debris. At the location of initial levee overtopping on the East Levee, the rise for the debris impact for the 277k cfs event was 0.84 foot. Comparing the 277k cfs computed water surface elevation upstream from the bridge with debris and the water surface elevation with the AT&SF Bridge Modification Plan included was a difference of approximately 1.88 feet.

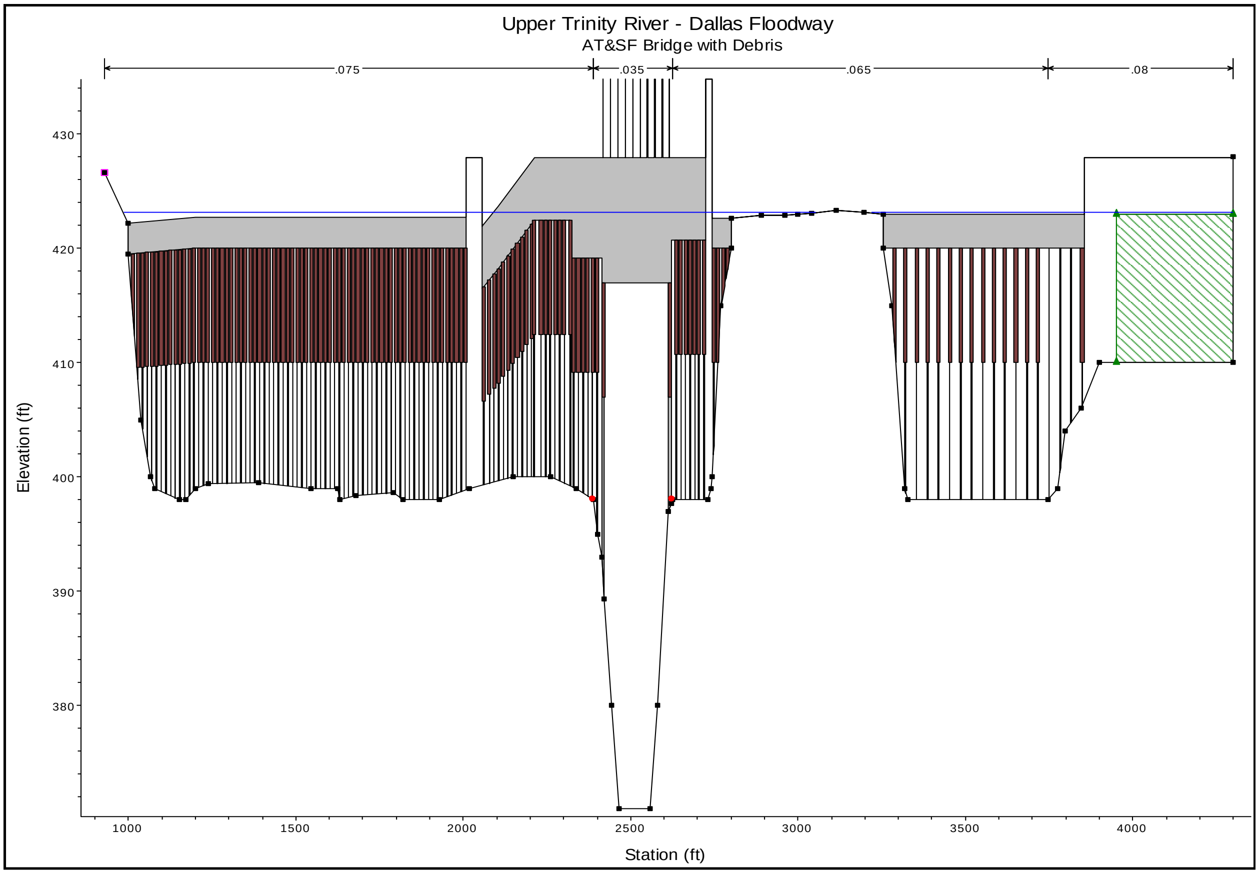


Figure 5-3 AT&SF Railroad Bridge HEC-RAS Geometry with Debris Accumulation

To determine the economic results for the debris impact compared to the AT&SF Bridge Modification Plan, the same HEC-RAS debris setting obtained in the 1990 flood debris calibration was used in the HEC-RAS unsteady flow model. The results of the unsteady hydraulic analysis indicated that debris accumulation could cause an additional rise in the water surface just upstream of the bridge of approximately 0.85 feet for the future SPF (277,000 cfs) event. This rise is compared to the existing bridge configuration, so it is in addition to the rise associated with the bridge with no debris. The rise in water surface associated with debris accumulation also caused the levees to overtop and breach sooner in the array of flood events. For example, it changed the incipient overtopping peak flow for the East Levee to 225,000 cfs, compared to 245,000 cfs without the debris impact.

Figures 5-4 and 5-5 and Table 5-2 show the impact of debris accumulation at the bridge on the East and West Levees in terms of the maximum water surface elevations in the protected areas behind each levee. From these figures, one can see that the debris would cause the levees to breach during lower flood events than were previously accounted for and that the peak water surface elevations in the protected areas would be about a foot higher than they would without the debris. The additional flood damages caused by potential debris accumulation give compelling reason for including the AT&SF Bridge Modification Plan as a first added measure in the FRM plan formulation.

This analysis of the bridge with the floating debris blockage resulted in a B/C ratio of 6.7 for the AT&SF Bridge Modification Plan in the economic analysis. Therefore, no further analysis is needed to confirm the decision that the Bridge Modification Plan is economically justified as a first added FRM measure.

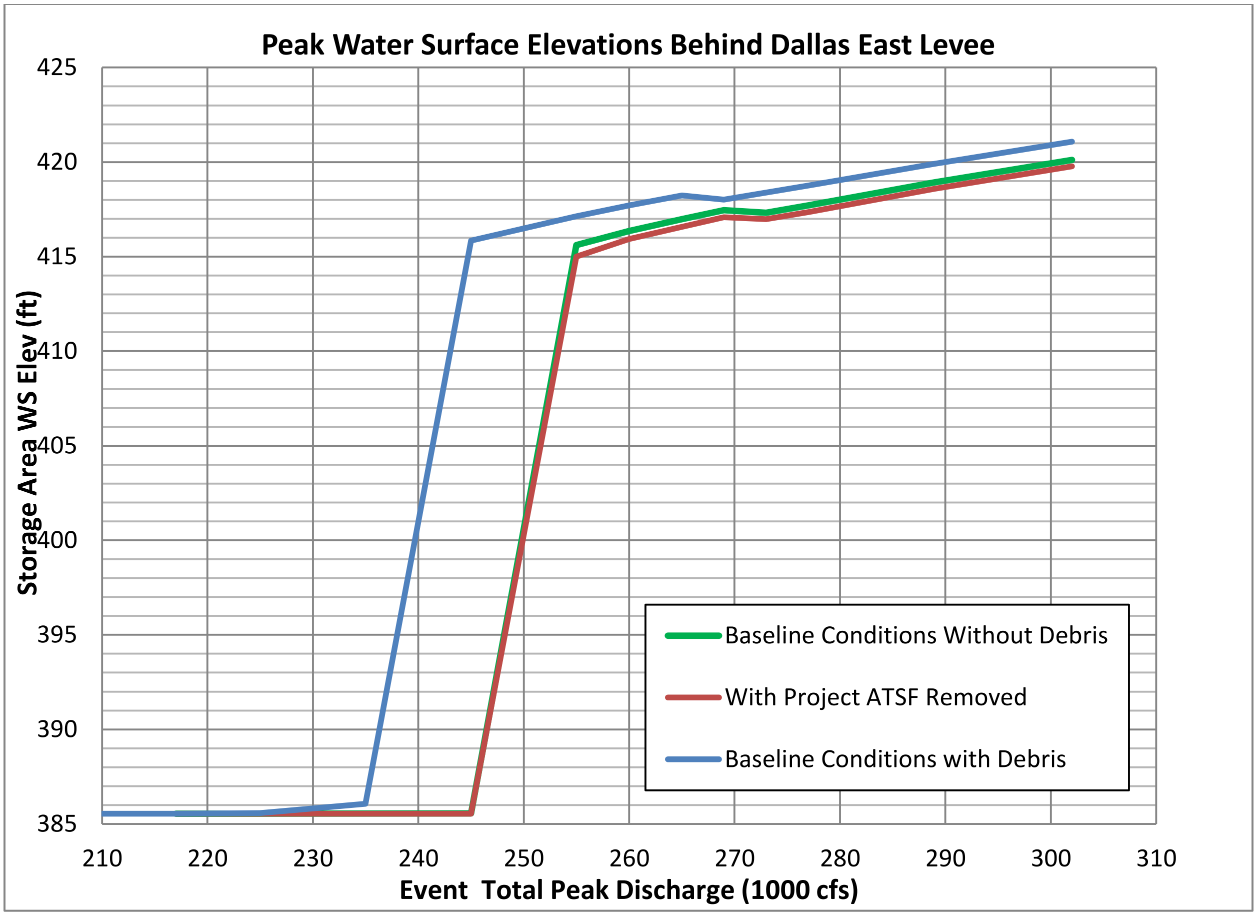


Figure 5-4 Effects of AT&SF Bridge Debris Accumulation on the East Levee

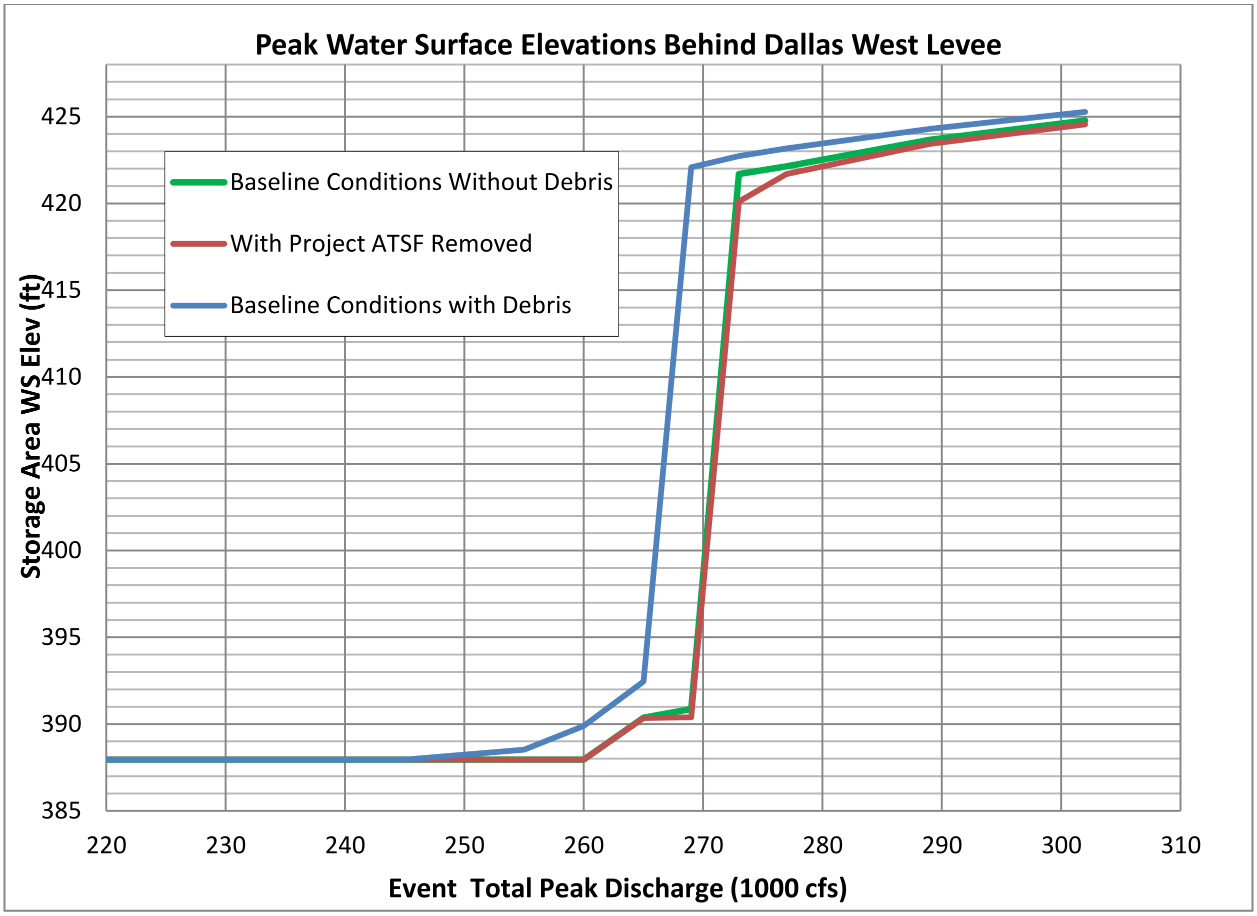


Figure 5-5 Effects of AT&SF Bridge Debris Accumulation on the West Levee

Table 5-2 With and Without Project for the AT&SF Bridge Modification Plan With Debris Accumulation

	Baseline Conditions with Debris Accumulation		**With AT&SF Bridge Removal Project**		**Difference With Project**	
Event (1000 cfs)	*East Levee WS Elev (feet)*	*West Levee WS Elev (feet)*	*East Levee WS Elev (feet)*	*West Levee WS Elev (feet)*	*East Levee WS Elev (feet)*	*West Levee WS Elev (feet)*
217	385.54	387.95	385.54	387.95	0.00	0.00
245	415.85	387.95	385.54	387.95	-30.31	0.00
255	417.13	388.52	415	387.95	-2.13	-0.57
260	417.71	389.89	415.93	387.95	-1.78	-1.94
265	418.24	392.45	416.58	390.36	-1.66	-2.08
269	418.01	422.08	417.08	390.38	-0.93	-31.70
273	418.39	422.73	416.98	420.11	-1.41	-2.62
277	418.76	423.17	417.35	421.7	-1.41	-1.47
289	419.91	424.30	418.59	423.43	-1.32	-0.87
302	421.08	425.28	419.77	424.55	-1.31	-0.73

5.3 LEVEE HEIGHT MODIFICATIONS

Levee height modification was one of the measures analyzed for flood risk management potential. This measure involved using earthen fill to raise the portions of the levees that are below the target water surface profile elevations up to the target water surface profile elevations. A total of six different heights were analyzed, which are associated with peak flow rates on the Trinity River of 260,000 (260k), 265k, 269k, 273k, 277k, and 289k cfs. These flow rates translated to roughly half a foot increments in levee height. Portions of the levees that were already at or above the target elevation would be left unmodified since these areas would be superior in height to the modified areas.

5.3.1 Modeling Methodology

This measure was modeled by setting the elevations for each HEC-RAS lateral structures that define the levee crest height to match the HEC-RAS steady flow water surface profile where the lateral structure elevation (existing levee crest elevation) was below the desired capacity. Existing levee elevations that exceeded the desired capacity were not modified. For example, the 277k raise increased the levee crest elevations to a levee height that prevents levee overtopping for the 277k event and thus resulted in no flooding in the protected area for flood events up to 277k cfs. Flood events that exceed 277k cfs would begin overtopping of the levee system for this levee raise alternative. The same array of unsteady flow events were analyzed with the modified levee crest HEC-RAS geometry for each levee raise alternative. The breach settings were left the same as in baseline conditions, except the trigger water surface elevations for breach initiation were raised to 0.5 foot above the modified levee crest at the breach location. To provide more complete economic information, the levee height modification alternatives were analyzed both with and without the AT&SF Bridge modification in place.

5.3.2 Modeling Results

The resulting protected area inundation elevations for the East and West Levees for the levee height modification alternatives with the AT&SF Bridge removed are shown in Figure 5-6 and Figure 5-7 and in tabular form in Table 5-3 and Table 5-4. Similarly, the results for the levee height modification alternatives with the AT&SF Bridge in place are shown in Figure 5-8 and Figure 5-9 and in Table 5-5 and Table 5-6. In general, the levee height modifications reduced the frequency of overtopping, delayed initiation of each overtopping levee breach, and lowered the maximum water surfaces in the protected areas. For a detailed discussion of how breach parameters were developed, refer to Section 4.4.

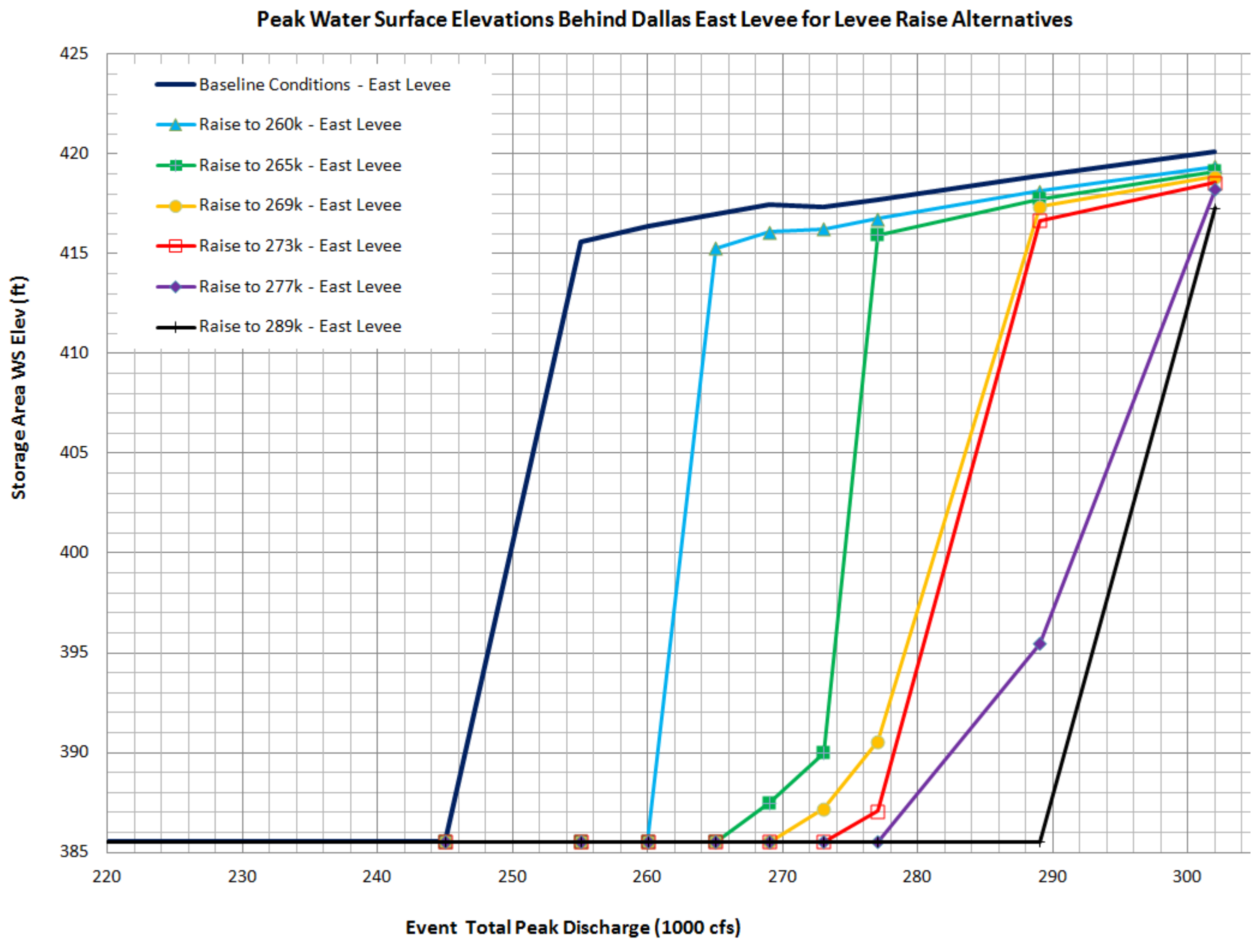


Figure 5-6 East Levee Results for the Levee Raise Alternatives with the AT&SF Bridge Modification

Table 5-3 Max Water Surface Elevations behind the East Levee for the Levee Raise Alternatives with the AT&SF Bridge Modification

Event Name	*Baseline*	*260k Raise*	*265k Raise*	*269k Raise*	*273k Raise*	*277k Raise*	*289k Raise*
217k	385.54	385.54	385.54	385.54	385.54	385.54	385.54
245k	385.56	385.54	385.54	385.54	385.54	385.54	385.54
255k	415.61	385.54	385.54	385.54	385.54	385.54	385.54
260k	416.36	385.54	385.54	385.54	385.54	385.54	385.54
265k	416.98	415.29	385.54	385.54	385.54	385.54	385.54
269k	417.46	416.08	387.50	385.54	385.54	385.54	385.54
273k	417.32	416.26	390.00	387.19	385.54	385.54	385.54
277k	417.71	416.76	415.94	390.54	387.06	385.54	385.54
289k	418.93	418.13	417.75	417.36	416.68	395.48	385.54
302k	420.12	419.39	419.13	418.88	418.57	418.23	417.29

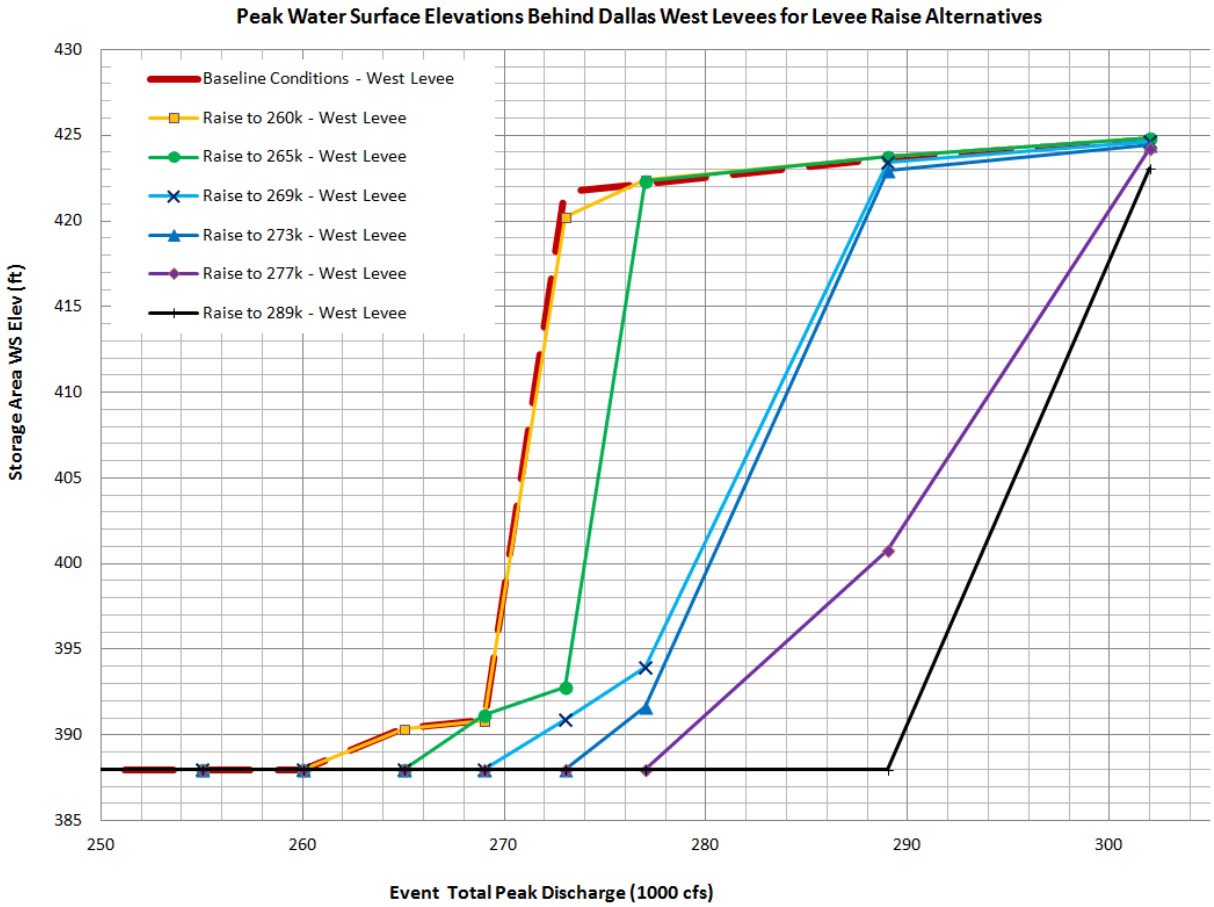


Figure 5-7 West Levee Results for the Levee Raise Alternatives with the AT&SF Bridge Modification

Table 5-4 Maximum Water Surface Elevations behind the West Levee for the Levee Raise Alternatives with the AT&SF Bridge Modification

Event Name	*Baseline*	*260k Raise*	*265k Raise*	*269k Raise*	*273k Raise*	*277k Raise*	*289k Raise*
217k	387.95	387.95	387.95	387.95	387.95	387.95	387.95
245k	387.95	387.95	387.95	387.95	387.95	387.95	387.95
255k	387.95	387.95	387.95	387.95	387.95	387.95	387.95
260k	387.95	387.95	387.95	387.95	387.95	387.95	387.95
265k	390.37	390.38	387.95	387.95	387.95	387.95	387.95
269k	390.87	390.84	391.15	387.95	387.95	387.95	387.95
273k	421.69	420.25	392.83	390.91	387.95	387.95	387.95
277k	422.14	422.42	422.32	393.94	391.61	387.95	387.95
289k	423.67	423.74	423.73	423.41	422.95	400.79	387.95
302k	424.77	424.84	424.84	424.61	424.44	424.24	423.11

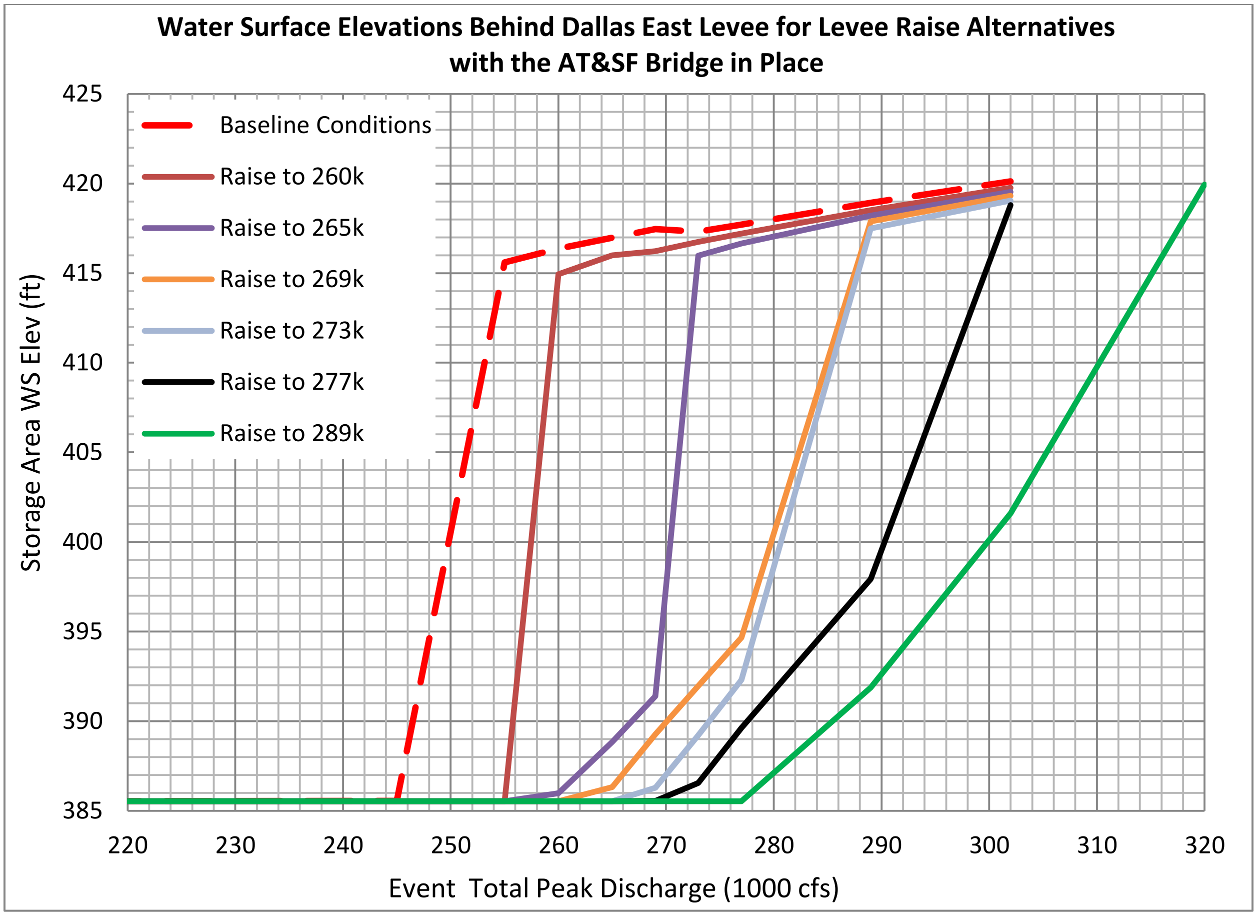


Figure 5-8 East Levee Results for the Levee Raise Alternatives with the AT&SF Bridge in Place

Table 5-5 Max Water Surface Elevations behind the East Levee for the Levee Raise Alternatives with the AT&SF Bridge in Place

Event	*Baseline*	*260k Raise*	*265k Raise*	*269k Raise*	*273k Raise*	*277k Raise*	*289k Raise*
217k	385.54	385.54	385.54	385.54	385.54	385.54	385.54
245k	385.56	385.54	385.54	385.54	385.54	385.54	385.54
255k	415.61	385.54	385.54	385.54	385.54	385.54	385.54
260k	416.36	414.94	385.99	385.54	385.54	385.54	385.54
265k	416.98	416.00	388.84	386.33	385.54	385.54	385.54
269k	417.46	416.22	391.39	389.26	386.29	385.56	385.54
273k	417.32	416.74	415.98	391.97	389.22	386.56	385.54
277k	417.71	417.20	416.65	394.66	392.32	389.62	385.54
289k	418.93	418.52	418.20	417.90	417.49	397.94	391.89
302k	420.12	419.77	419.53	419.31	419.05	418.79	401.60

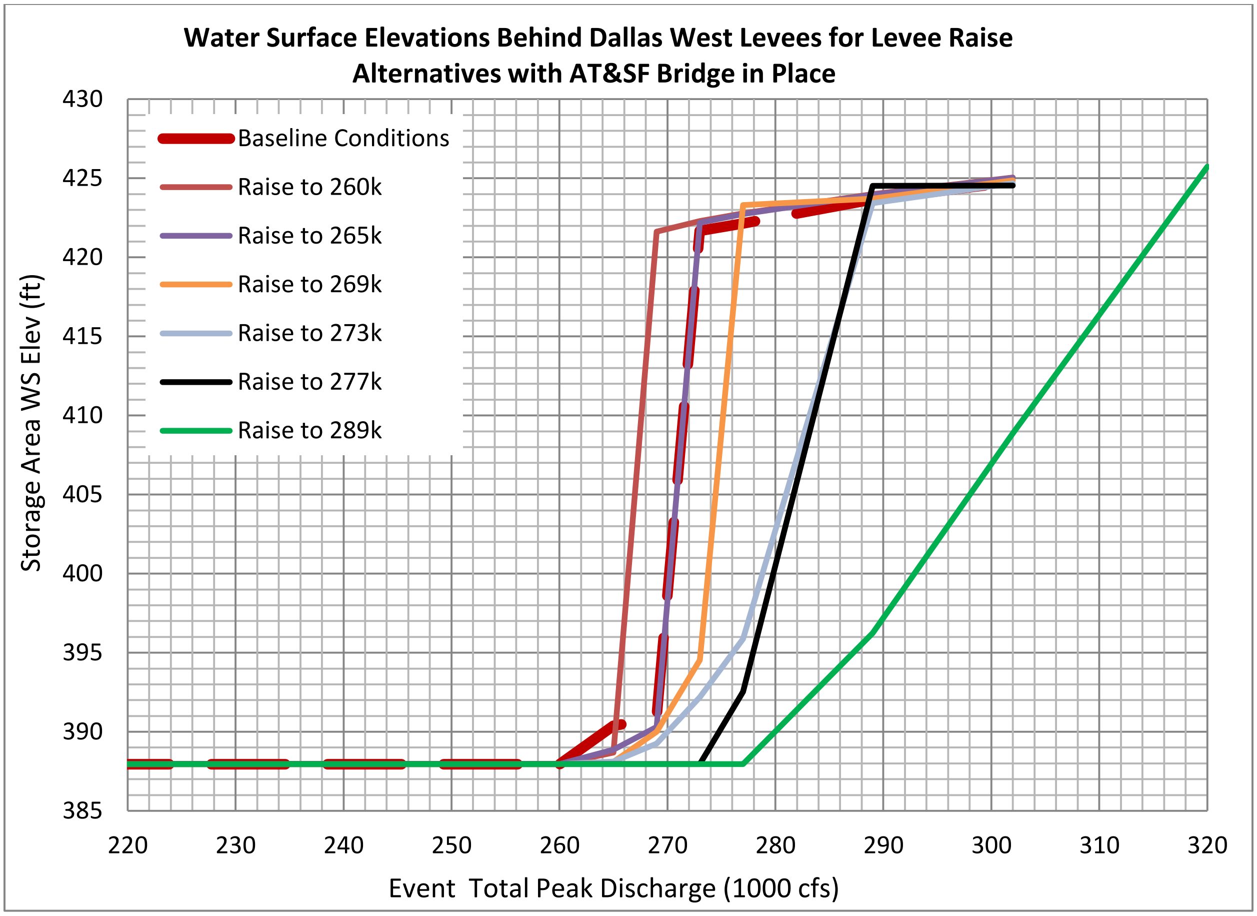


Figure 5-9 West Levee Results for the Levee Raise Alternatives with the AT&SF Bridge in Place

Table 5-6 Max Water Surface Elevations behind the West Levee for the Levee Raise Alternatives with the AT&SF Bridge in Place

Event	*Baseline*	*260k Raise*	*265k Raise*	*269k Raise*	*273k Raise*	*277k Raise*	*289k Raise*
217k	387.95	387.95	387.95	387.95	387.95	387.95	387.95
245k	387.95	387.95	387.95	387.95	387.95	387.95	387.95
255k	387.95	387.95	387.95	387.95	387.95	387.95	387.95
260k	387.95	387.95	387.95	387.95	387.95	387.95	387.95
265k	390.37	388.69	388.87	388.10	388.12	387.95	387.95
269k	390.87	421.63	390.30	390.00	389.25	387.95	387.95
273k	421.69	422.30	422.20	394.55	392.19	387.95	387.95
277k	422.14	422.79	422.75	423.31	395.87	392.54	387.95
289k	423.67	423.99	423.96	423.72	423.44	424.52	396.25
302k	424.77	425.04	425.04	424.87	424.69	424.55	408.88

In Figure 5-9, one may notice that the 260k levee raise actually increased flooding elevations behind the West Levee during the 265k flood event. This phenomenon is due to the existing levee crest currently providing a higher level of protection on the West levee than it does on the East levee, and the fact that the levees are allowed to overtop and breach simultaneously within the same HEC-RAS run. The 260k levee raise plan is only raising low spots on the East levee, as the West levee crest already exceeds the 260k cfs water surface elevations. Raising the East levee alone raised the water surface elevations on the river enough to trigger an overtopping breach on the West Levee during the 265k flood event. These induced damages on the West Levee are believed to be a real phenomenon and were accounted for in the economic analysis.

5.4 Armoring

Levee armoring was another one of the measures analyzed for flood risk management potential. This measure potentially provides benefits by reducing the risk of levee breaching using armoring at the crest and interior slopes for the locations on the levee crest that are lower than the target water surface profile. This measure cannot prevent levee breaching for some flood events that exceed the overflow capacity of the armored locations and rise to an overtopping level where breaching may occur at a location on the levee that is beyond the armored portions of the levee. Therefore, this measure only provides benefits for a relatively narrow range of overtopping flood events.

This measure involves armoring the East and West Levee (including the Elm Fork and West Fork levee segments) in all areas where the existing levee height is below the target water surface profile elevations. A total of eight different armoring alternatives were analyzed, which are associated with peak flow rates on the Trinity River of 255,000 (255k), 260k, 265k, 269k, 273k, 277k, 289k, and 302k cfs.

The armoring will be placed using articulated concrete block (ACB). The ACBs will begin 10 feet down from the crest of the riverside slope and continue across the crest of the levee and down the entire landside slope of the levee, extending 50 feet out from the toe of the levee. The existing side slopes would not be modified. The armoring would be placed on the existing surface of the levees in all identified areas. Two additional materials for armoring were considered, including turf reinforcement mats and scour protection mats. These two methods provided significant cost savings; however, all materials would need to be tested to determine technical viability for their application.

5.4.1 Modeling Methodology

Modeling for this measure used the same HEC-RAS geometry as the AT&SF Bridge Modification Plan. It was assumed that the armoring would prevent the levee from breaching at the armored sections of the levee. However, when the unarmored sections of the levee that are adjacent to the armoring are overtopped, those unarmored sections would still be vulnerable to breach. Armored sections of the levee are only those areas of the levee where the crest is lower than the target flood event water surface profile. Therefore, the armoring alternatives were modeled by setting the breach trigger elevation at which breach initiation would occur to be 0.5 foot above the target water surface elevation at the breach location. This assumption represents a breach initiation at an overtopping depth of 0.5 foot on the unarmored sections of the levee near the upstream or downstream limits of the armored section of the levee. This is consistent with the assumption used for overtopping scenarios, that the breach trigger would be 0.5 foot above the lowest elevation of the unarmored section of the levee crest. This means that if the flood event overtopped the levee by a minimum of 0.5 foot at the unarmored section of the levee, a breach would initiate. For a detailed discussion of how breach parameters were developed, refer to section 4.4.

5.4.2 Modeling Results

The resulting protected area elevations for the East and West Levee are shown in Figure 5-10 and Figure 5-11 and Table 5-7 and Table 5-8 below. The armoring measures did not reduce the frequency of overtopping but did delay the initiation of the breach for the overtopping events.

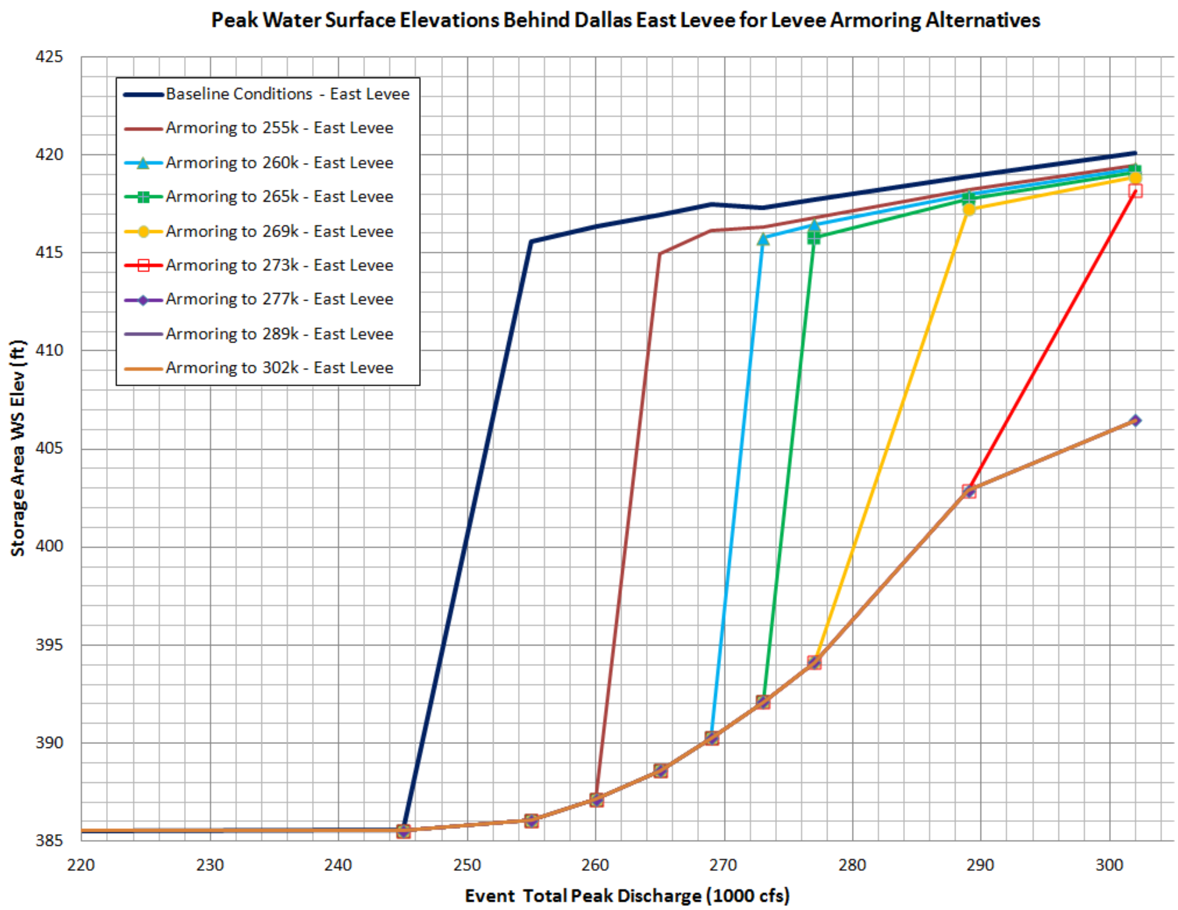


Figure 5-10 Inundation Elevations behind the East Levee for the Armoring Alternatives

Table 5-7 Max Water Surface Elevations behind the East Levee for the Armoring Alternatives

Event Name	*Baseline*	**Armoring Measure**							
		255k	*260k*	*265k*	*269k*	*273k*	*277k*	*289k*	*302k*
217k	385.54	385.54	385.54	385.54	385.54	385.54	385.54	385.54	385.54
245k	385.56	385.54	385.54	385.54	385.54	385.54	385.54	385.54	385.54
255k	415.61	386.07	386.07	386.07	386.07	386.07	386.07	386.07	386.07
260k	416.36	387.15	387.15	387.15	387.15	387.15	387.15	387.15	387.15
265k	416.98	414.98	388.59	388.59	388.59	388.59	388.59	388.59	388.59
269k	417.46	416.14	390.26	390.26	390.26	390.26	390.26	390.26	390.26
273k	417.32	416.32	415.78	392.09	392.09	392.09	392.09	392.09	392.09
277k	417.71	416.83	416.45	415.77	394.11	394.11	394.11	394.11	394.11
289k	418.93	418.20	417.99	417.75	417.22	402.88	402.88	402.88	402.88
302k	420.12	419.46	419.30	419.14	418.87	418.19	406.48	406.48	406.48

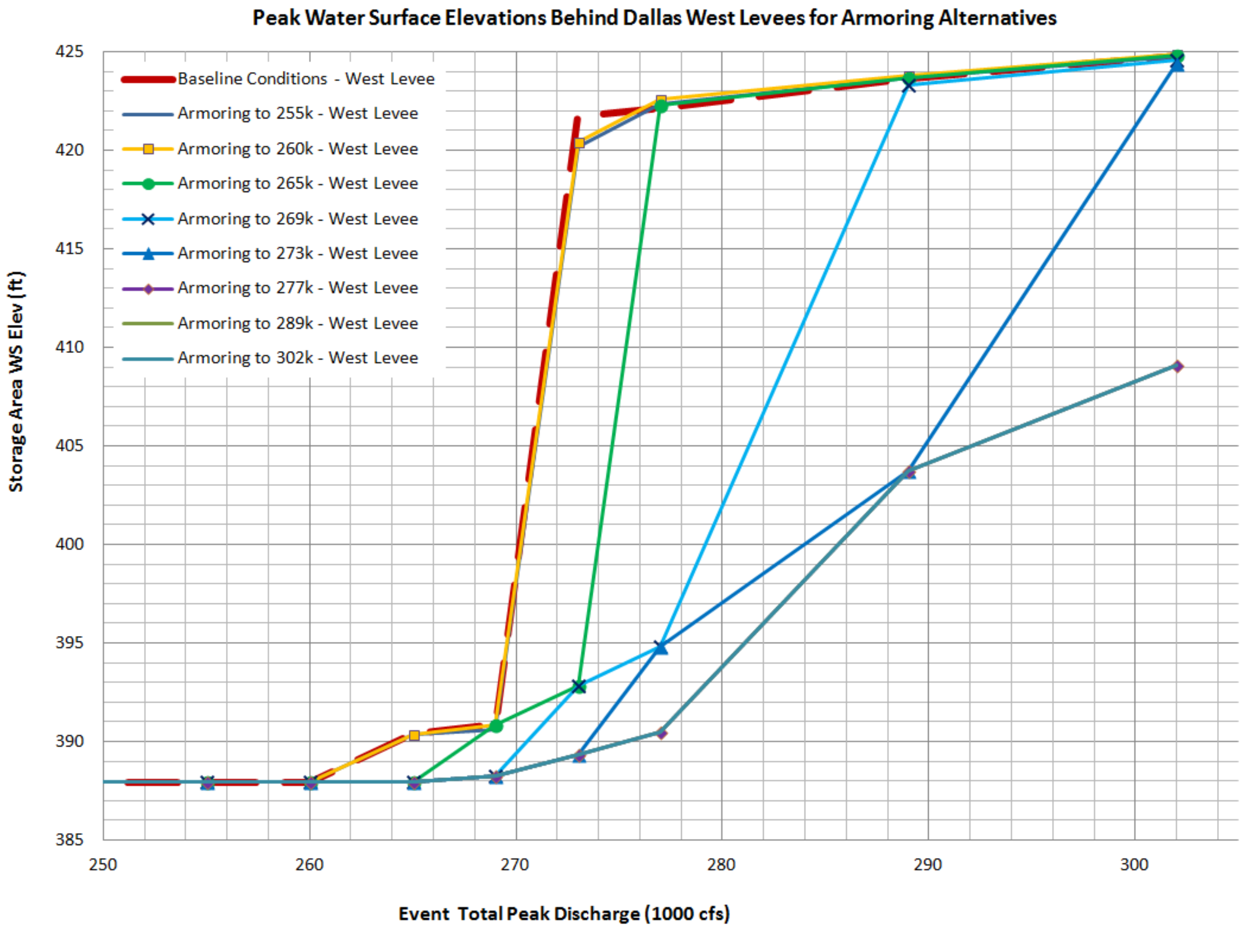


Figure 5-11 Inundation Elevations behind the West Levee for the Armoring Alternatives

Table 5-8 Max Water Surface Elevations behind the West Levee for the Armoring Alternatives

Event Name	*Baseline*	**Armoring Measure**							
		255k	*260k*	*265k*	*269k*	*273k*	*277k*	*289k*	*302k*
217k	387.95	387.95	387.95	387.95	387.95	387.95	387.95	387.95	387.95
245k	387.95	387.95	387.95	387.95	387.95	387.95	387.95	387.95	387.95
255k	387.95	387.95	387.95	387.95	387.95	387.95	387.95	387.95	387.95
260k	387.95	387.95	387.95	387.95	387.95	387.95	387.95	387.95	387.95
265k	390.37	390.37	390.37	387.96	387.96	387.96	387.96	387.96	387.96
269k	390.87	390.62	390.83	390.83	388.23	388.23	388.23	388.23	388.23
273k	421.69	420.16	420.41	392.83	392.83	389.37	389.37	389.37	389.37
277k	422.14	422.33	422.56	422.28	394.80	394.80	390.50	390.50	390.50
289k	423.67	423.68	423.81	423.69	423.31	403.72	403.72	403.72	403.72
302k	424.77	424.77	424.89	424.82	424.56	424.41	409.08	409.08	409.08

5.5 CONTROLLED OVERTOPPING

Proposed levee modifications may include what is commonly called “levee resiliency measures.” Resiliency measures are expected to reasonably provide cost effective flood risk reduction either alone or in combination with other types of flood risk reduction measures or alternatives. Since the highest risk of flooding from levee failure for the Dallas Floodway Levees has been identified as overtopping with subsequent levee breach, resiliency measures are expected to focus on reducing the risk of flooding or depths of flooding associated with overtopping failure of the levees. One of the resiliency measures considered is referred to as “Controlled Overtopping.”

The Controlled Overtopping measure is one of the design considerations outlined in ETL 1110-2-299 discussed at the beginning of this section. This measure potentially provides benefits by localizing the initial overtopping in an area that has the least negative impact and reducing the risk of levee breaching using armoring at the crest and interior slopes.

Controlled Overtopping analysis presented herein primarily focuses on the potential for reduction of flood damage by means of altering the timing of the overtopping inundation and potentially delaying or preventing the breaching of the levee once it has been overtopped. The HEC-RAS unsteady flow model for the Dallas Floodway system was used to analyze the effects of controlled overtopping measures and at various lengths, levee heights, and locations when combined with the 277k levee raise alternative. The modeling was used to compute the incremental benefits of reduction of flood inundation depths for overtopping flood events. HEC-FDA was used to identify the benefits and residual damages for levee controlled overtopping measures. This controlled overtopping measure could be described as a notch in the levee having armoring on the levee crest and landside slope to reduce the chance of a levee breach while flow is within the notch. However, similar to the Armoring alternatives, the controlled overtopping measure cannot prevent levee breach for some flood events that exceed the notch’s capacity and rise to an overtopping level where a breach may occur at a location on the levee that is outside the notch. Therefore, this measure only provides benefits for a relatively narrow range of overtopping flood events.

5.5.1 Modeling Methodology

The Fort Worth District has evaluated overtopping scenarios by adding an armored notch for controlled overtopping at two different locations on the levees. The first location for the notch was located as far downstream as possible on each levee with the expectation that the most downstream location would provide the highest benefits. For the East levee, this notch was located in between the Corinth Street and DART Rail Line bridges, and on the West levee, the notch was placed in between the Corinth Street and I-35E bridges. Then, a second notching location near the Hampton Bridge on both levees was also considered to determine if the location for the structure was a critical factor.

The notching alternatives assumed the levee would be raised to the 277k water surface profile. Notches were evaluated with depths of 0.5 feet and 1.0 foot below the 277k water surface profile at varying widths of 100 feet, 1,000 feet and 1,500 feet at the downstream end of the East and West Levee. A notch of 2-feet deep by 1,500-feet wide was also evaluated as the maximum feasible. A notch having a maximum depth of 2 feet was used because a notch deeper than 2 feet would increase the frequency at which the existing levee system would experience overtopping.

The HEC-RAS geometry for the 277k levee raise plan was used as the base geometry for these alternatives. The lateral structure elevations representing the levees were then modified in the location of the proposed notch by lowering the levee crest elevations by the proposed amount. The same array of unsteady flow events were run with the modified HEC-RAS geometry. Since the notch would be armored to prevent breach at that location, the breach settings were left at the same location and elevation as in the 277k levee raise plan.

5.5.2 Modeling Results

The notching alternatives at the downstream end of the levee system concluded that the controlled overtopping design for any of the modeled notch sizes does not reduce depths of flooding significantly on the landward side of the East and West Levees. Therefore, a complete economic analysis was not warranted for this alternative at the downstream location. In addition, none of the alternative notch sizes at this controlled overtopping location indicated a change in the occurrence of the levee breach. For each notch alternative tested at the downstream location, Table 5-9 and Table 5-10 give the resulting water surface elevations on the respective interiors of the East and West Levees. Figure 5-12 and Figure 5-13 then compare the interior flooding depths of the largest notches evaluated with the levee raise with no notch.

Table 5-9 Water Surface Elevations behind the East Levee for the Downstream Notch Location

Event (1000 cfs)	*Raise to 277k, No Notch*	*With 100' x 0.5' Notch*	*With 1000' x 0.5' Notch*	*With 1500' x 0.5' Notch*	*With 100' x 1' Notch*	*With 1000' x 1' Notch*	*With 1500' x 1' Notch*	*With 1500' x 2' Notch*
217	385.54	385.54	385.54	385.54	385.54	385.54	385.54	385.54
245	385.54	385.54	385.54	385.54	385.54	385.54	385.54	385.54
255	385.54	385.54	385.54	385.54	385.54	385.54	385.54	385.82
260	385.54	385.54	385.54	385.54	385.54	385.54	385.54	386.59
265	385.54	385.54	385.54	385.54	385.54	385.54	385.54	387.45
269	385.54	385.54	385.54	385.54	385.54	385.54	385.54	388.25
273	385.54	385.54	385.54	385.54	385.56	385.77	385.97	388.80
277	385.54	385.54	385.54	385.54	385.63	386.31	386.47	389.19
289	395.48	395.55	395.80	395.99	395.69	396.93	397.32	398.69
302	418.23	418.23	418.22	418.24	418.23	418.27	418.28	418.35

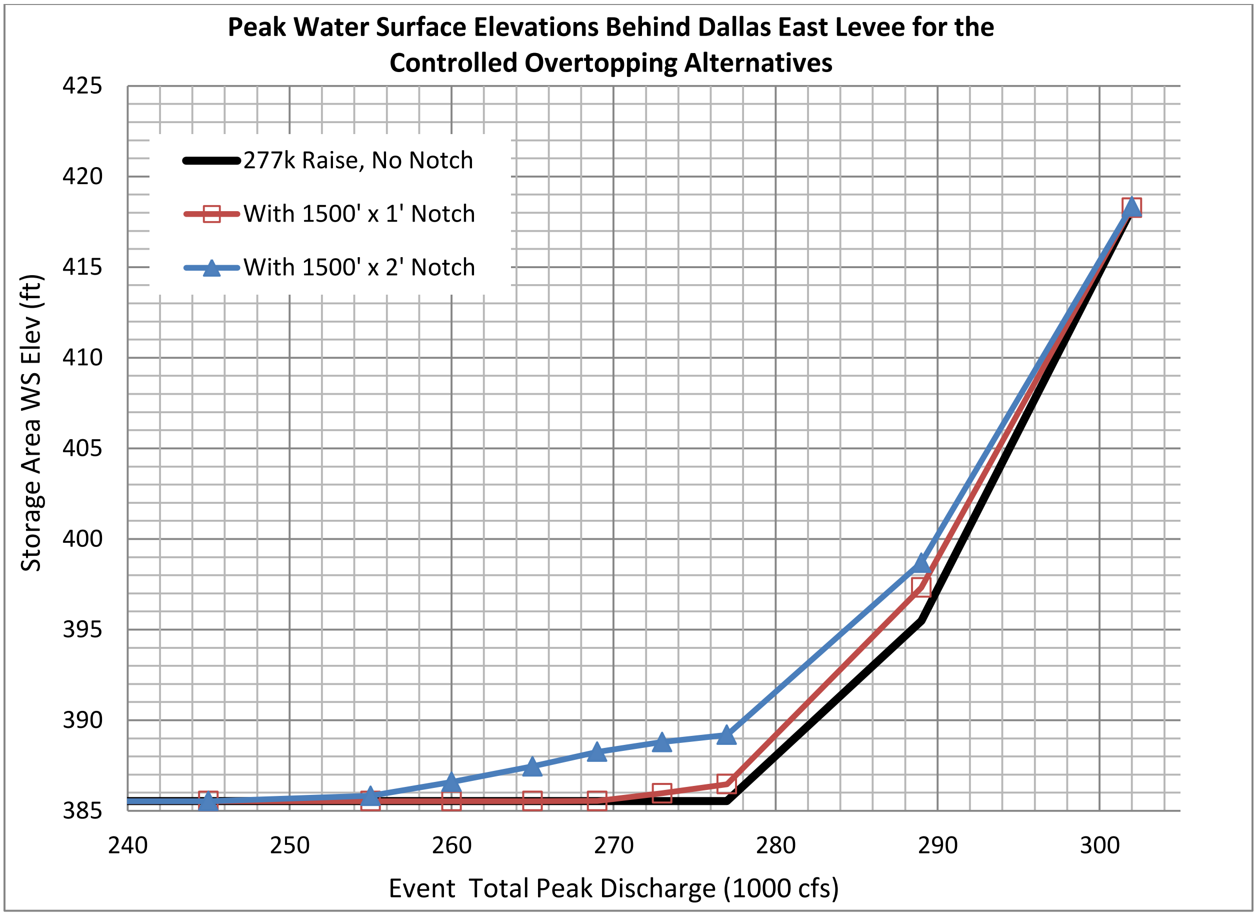


Figure 5-12 Flooding Depths behind the East Levee for the Downstream Controlled Overtopping Alternatives

Table 5-10 With Project Max Water Surface Elevations behind the West Levee for the Downstream Notch Location

Event (1000 cfs)	*277k Raise, No Notch*	*With 100' x 0.5' Notch*	*With 1000' x 0.5' Notch*	*With 1500' x 0.5' Notch*	*With 100' x 1' Notch*	*With 1000' x 1' Notch*	*With 1500' x 1' Notch*	*With 1500' x 2' Notch*
217	387.95	387.95	387.95	387.95	387.95	387.95	387.95	387.95
245	387.95	387.95	387.95	387.95	387.95	387.95	387.95	387.95
255	387.95	387.95	387.95	387.95	387.95	387.95	387.95	394.54
260	387.95	387.95	387.95	387.95	387.95	387.95	387.95	397.40
265	387.95	387.95	387.95	387.95	387.95	388.04	388.07	400.78
269	387.95	387.95	387.95	387.95	389.34	391.60	392.12	402.66
273	387.95	387.95	387.95	387.95	390.28	394.18	394.92	404.56
277	387.95	389.16	390.82	391.31	391.58	395.81	396.38	406.45
289	400.79	400.77	400.59	400.64	400.74	401.87	402.69	410.80
302	424.24	424.24	423.98	424.18	424.22	424.23	424.21	423.54

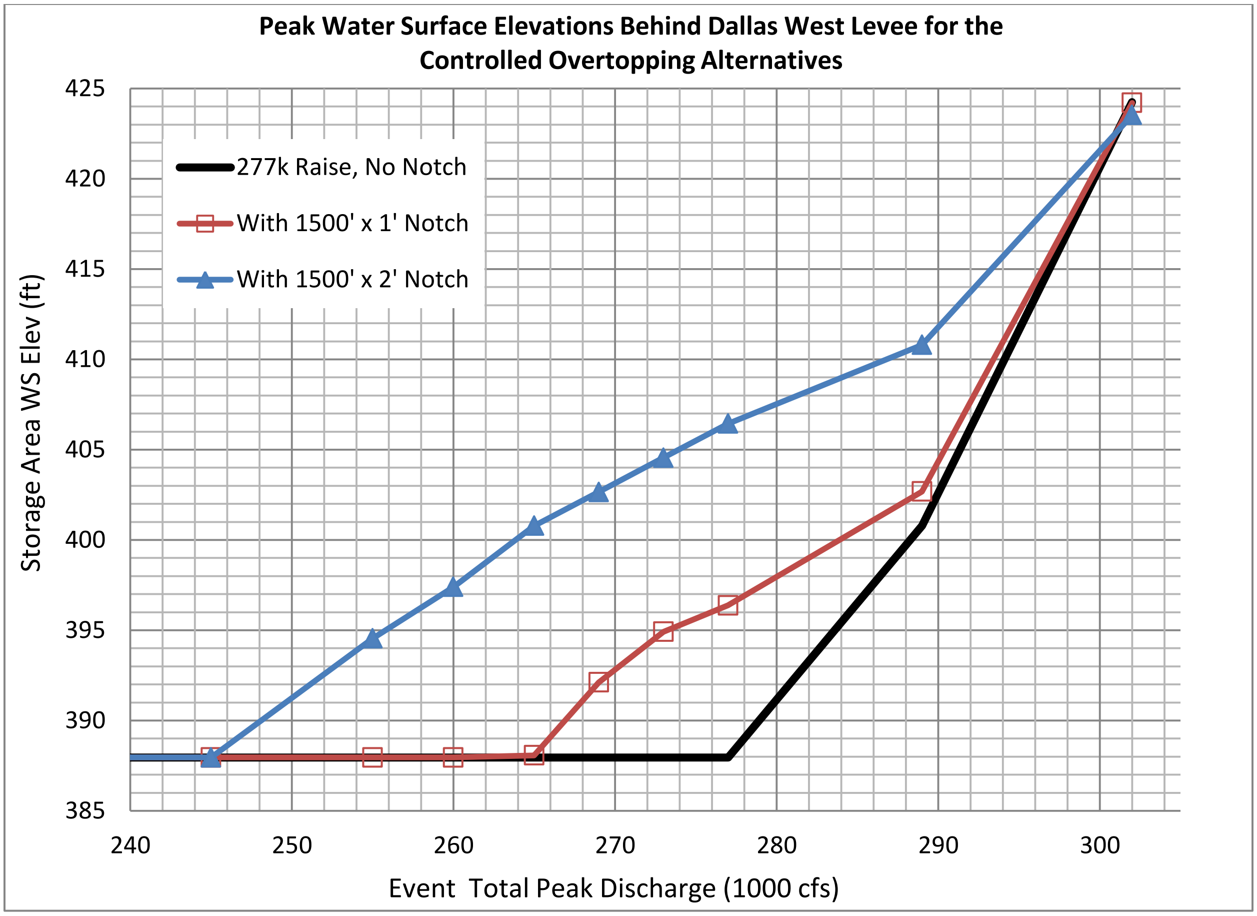


Figure 5-13 Flooding Depths behind the West Levee for the Downstream Controlled Overtopping Alternatives

These results show that the addition of a notch at the downstream end of the levee raise plan does not significantly reduce flooding depths on the interior of the levees, and for some events, it actually increases flooding depths when compared to the same levee raise plan with no notch. This is because the notch at the downstream end was not able to lower the water surface elevations on the river enough to prevent breach for the entire 9 miles of levee upstream, as shown in the results for the 302k event.

The second notch location at the Hampton Bridge resulted in more significant change to the inundation depths when compared to the levee raise alternative without notching. The hydraulic results indicated that the interior inundation depths were higher for the more frequent flood events and lower for the rarer events, as shown in Figure 5-14 and Figure 5-15. There was also an alteration in the occurrence of the levee breach for the highest flood event analyzed. The results for this location indicate that an economic analysis may or may not result in flood damage reduction benefits compared to the without notching alternative.

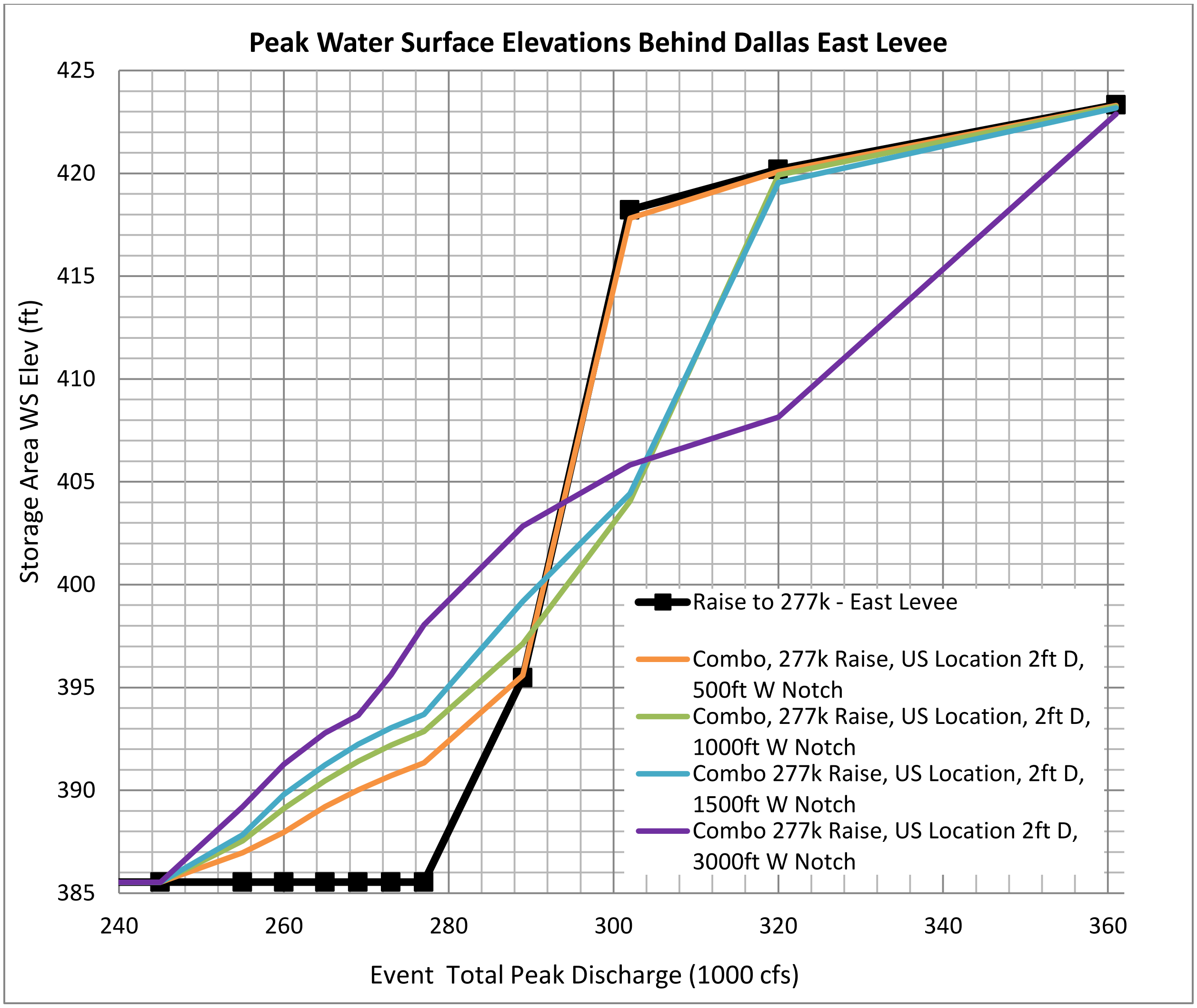


Figure 5-14 Flooding Depths behind the East Levee for the Controlled Overtopping Alternatives at Hampton

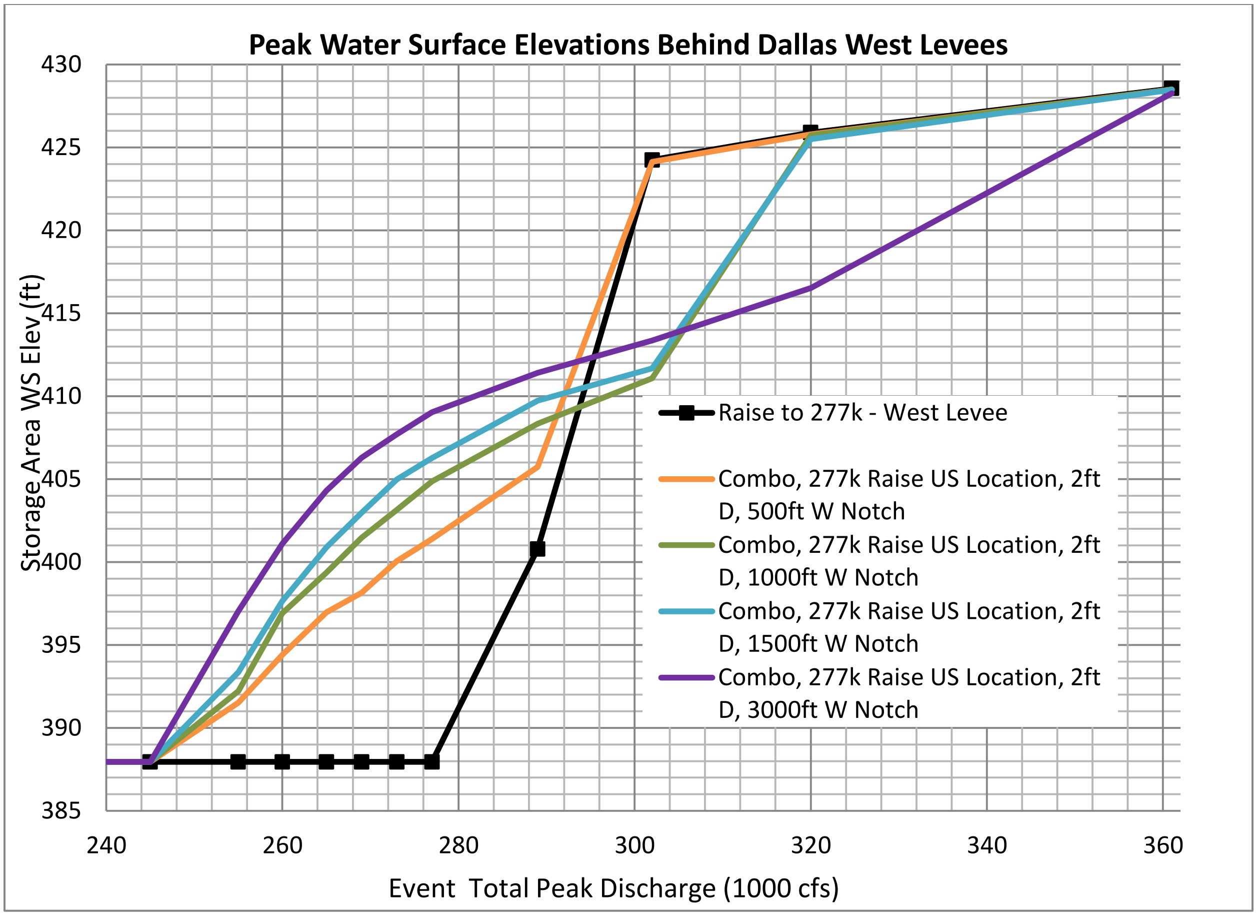


Figure 5-15 Flooding Depths behind the West Levee for the Controlled Overtopping Alternatives at Hampton

5.6 Seepage Walls

Problems associated with internal erosion due to seepage at the levee foundation were identified in the Risk Assessment as risks near to exceeding the tolerable risk guideline. Seepage cut-off walls and clay lining on the river side of the East and West Levees were evaluated to show a reduction in risk for that failure mode. The cut-off wall would be constructed down to the bedrock elevation and keyed-into the bedrock to a depth of 5 feet on the river side of the levees.

5.6.1 Modeling Methodology

Seepage walls were proposed to reduce risk by preventing a breach by the internal erosion (also known as under seepage) failure mode. This is an entirely different failure mode with different breach characteristics than were analyzed for overtopping, and it required a separate analysis for baseline conditions. Therefore, a new hydraulic analysis was performed for baseline and with project conditions that considered only the internal erosion (or under seepage) failure mode in order to evaluate the economic benefits of the proposed seepage walls.

5.6.2 Baseline Conditions Modeling for Under Seepage Failure

Inundation depths were computed through unsteady flow model runs in HEC-RAS version 4.2 for under seepage levee failure with breach initiating during a range of flow events. This modeling effort used the same HEC-RAS geometry file as was used for the Dallas Floodway Feasibility Study base conditions overtopping analysis. However, a different set of inflow events and different breach settings were used to analyze the under seepage failure mode.

A total of eight loading conditions were modeled with peak flows ranging from 120,000 cfs (which is approximately 50% of levee height) to 302,000 cfs (highest event in overtopping analysis). The specific eight inflow events that were modeled are listed in Table 5-11. Most of the modeled flow events have peak stages that fall between 50% and 100% of levee height, which corresponds to the range of concern on the geotechnical fragility curves. The shape of the inflow hydrographs was taken from the future SPF hydrograph for the Trinity River at Dallas, which peaks at 277,000 cfs. A series of multiplication factors have been applied to the hourly flows of this hydrograph to scale the hydrograph up or down to the desired peak discharge at the Commerce Street gage. This methodology is consistent with what was used for the overtopping analysis of the Dallas Floodway Feasibility Study.

Table 5-11 Modeled Inflow Events for the Internal Erosion Failure Mode

Event Name	*SPF Multiplier*	*Peak Discharge at the Commerce St Gage (cfs)*	*Significance of Peak Discharge*
120k	0.47	120,000	100-year Future Peak Discharge
145k	0.57	145,000	
160k	0.65	160,300	
190k	0.75	190,400	
217k	0.85	217,400	2 feet below threshold for overtopping
245k	0.96	245,000	Threshold for overtopping the East Levee
260k	1.018	259,900	1 foot over threshold for overtopping
302k	1.184	302,100	Highest event in overtopping analysis

Levee breaches due to under seepage were assumed to occur at only one location on each levee. The selected failure locations for the East and West Levees are near the Hampton Pump Station, which is where the most critical geotechnical sections are located. The modeled failure location on the East Levee was at Center Station 4170 on Lateral Structure 140590, which corresponds to levee station 311+00. The modeled failure location on the West Levee was at Center Station 550 on Lateral Structure 135100, which corresponds to levee station 250+00. Breach analysis for the under seepage failure mode was based on the assumption that only one levee will breach at a time during each flood event model run. The geotechnical fragility curves, as shown in the figures below, indicate that the internal erosion failure mode has a very low probability of failure; therefore, the chance that both levees would fail due to under seepage during the same flood event is extremely remote. Separate HEC-RAS plans were created for the failures on the East and West Levees, respectively. Coupled with the eight inflow events, this resulted in a total of 16 separate HEC-RAS plans for baseline conditions.

The elevations, timing, initiation and progression of the modeled levee failures were estimated by geotechnical and hydraulic collaboration. Geotechnical recommendations were summarized in a memorandum for record (McCleskey 2012). They recommended breach invert elevations of 405 on the East Levee, and 400 on the West Levee. The river stage must be above 50% of levee height before breach may occur. These recommendations were taken directly from the original Risk Assessment analysis.

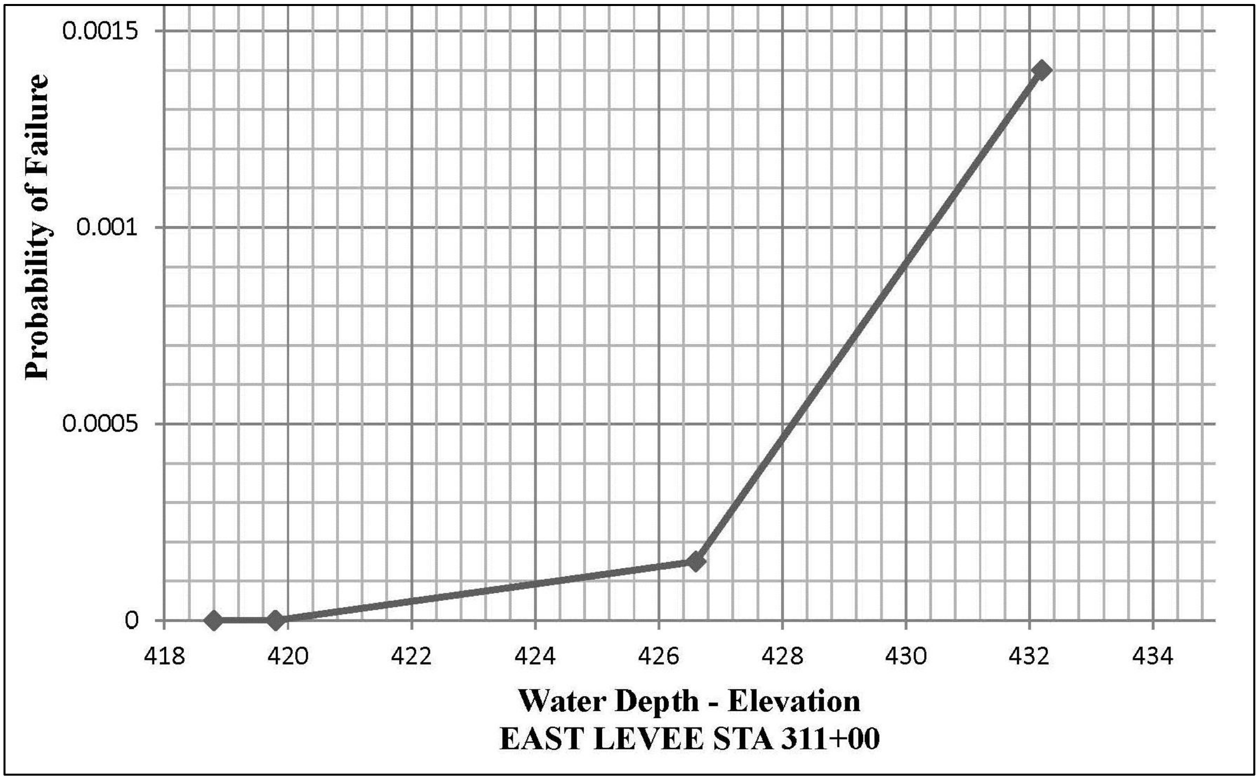


Figure 5-16 East Levee Fragility Curve due to Internal Erosion

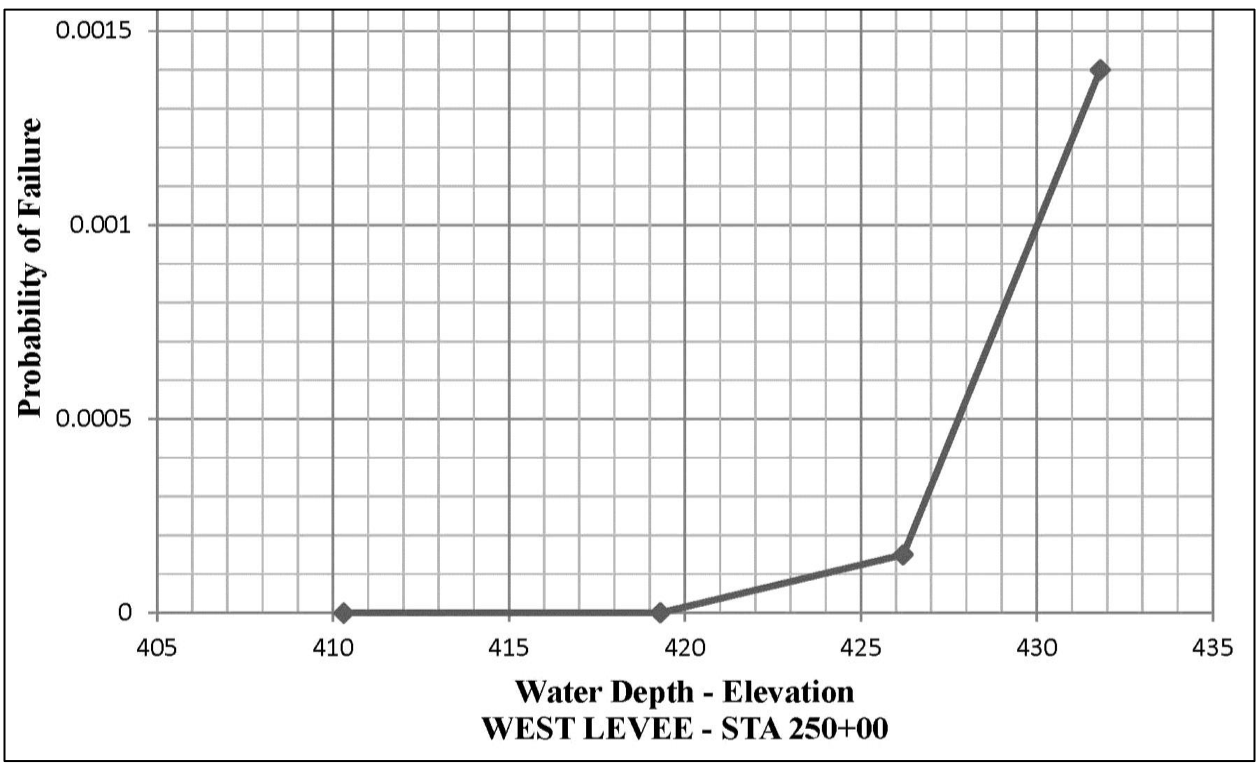


Figure 5-17 West Levee Fragility Curve due to Internal Erosion

The breach settings for under seepage were kept as consistent as possible with the final breach settings for overtopping. The simplified physical method within HEC-RAS 4.2b was used for both, and similar erosion rates were used (i.e. 14 feet/hour at velocities of 11 feet/sec) in both analyses. However, it was acknowledged that these erosion rates might be more appropriate for the later part of the event, once the roof of soil above the initial piped opening has collapsed and open weir flow is occurring.

The levee breaches caused by under seepage were modeled as piping failures in HEC-RAS. The same breach trigger was used for all modeled flow events. The breach was set to initiate when the river stage at the breach location reached the minimum failure elevation at 50% of levee height. Other breach settings were estimated by hydraulic engineers and adjusted based on initial results. For example, the mass wasting feature was used to slow down the initial breach progression during the early part of the piping failure. The final breach settings that were used in the HEC-RAS model are shown in Table 5-12.

Table 5-12 Final Breach Settings for Under Seepage Failures under Baseline Conditions

	East Levee	*West Levee*
Lat Str Breached	140590	135100
Breach Method	Simplified Breach	Simplified Breach
Center Station	4170	550
Max Bottom Width	500	500
Min Bottom Elev	405	400
Left Side Slope	0	0
Right Side Slope	0	0
Breach Weir Coeff	2.6	2.6
Failure Mode	Piping	Piping
Piping Coefficient	0.5	0.5
Initial Piping Elev	405	400
Initial Piping Diameter	0.1	0.1
Mass Wasting	Yes	Yes
Width (feet)	30	30
Duration (hour)	12	12
Final Bottom Elev	405	400
Trigger Failure at	WS Elev	WS Elev
Threshold WS	419.8	419.3

Erosion Rates (Downcutting & Widening)	
Velocity (feet/sec)	*Erosion (feet/hour)*
0	0
8	0
11	14
20	14

5.6.3 With-Project Conditions for Seepage Walls

For with-project conditions, it was assumed that the seepage cut-off walls would prevent a breach from forming due to the internal erosion (under seepage) failure mode. Therefore, with-project conditions for the seepage walls were modeled by turning off the levee breach settings and allowing no breach failure to occur for events with peak stages up to the top of levee. For events that overtopped levee, it was assumed that the water surface elevations behind the levees for with project conditions would be the same as the elevations for Future Without-Project Conditions for the overtopping with breach condition.

5.6.4 Modeling Results

The final modeling results were summarized in terms of final breach widths and maximum water surface elevations in the storage areas behind the levees. The water surface elevations behind the levees were then input into HEC-FDA as part of the levees' interior-exterior relationships. Figure 5-18 below shows the breach progression on the East Levee for each under seepage event modeled for baseline conditions, and Figure 5-19 shows the final breach progression for the West Levee. Figure 5-20 then summarizes the resulting maximum water surface elevations behind the levees for baseline and With-Project Conditions.

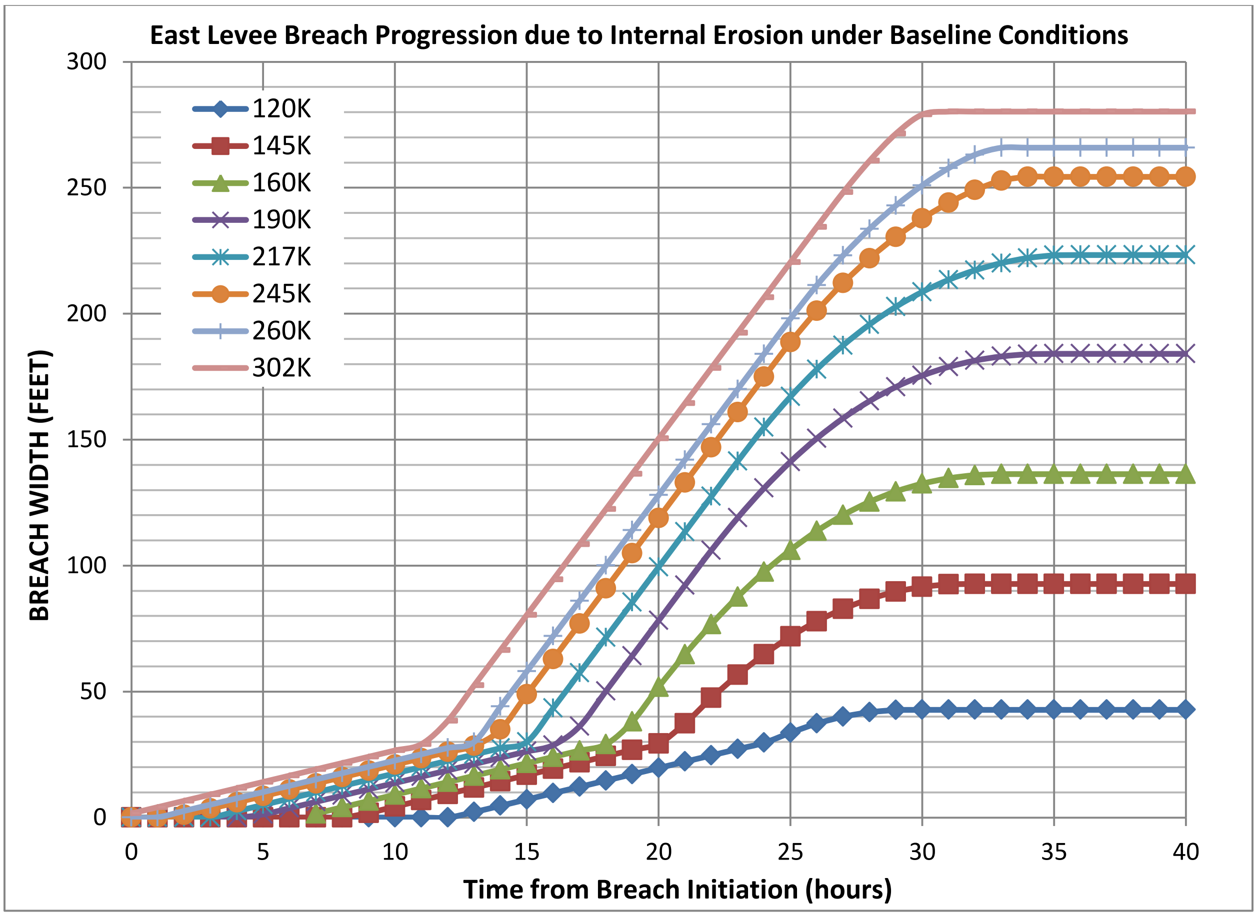


Figure 5-18 East Levee Breach Progression due to Internal Erosion under Baseline Conditions

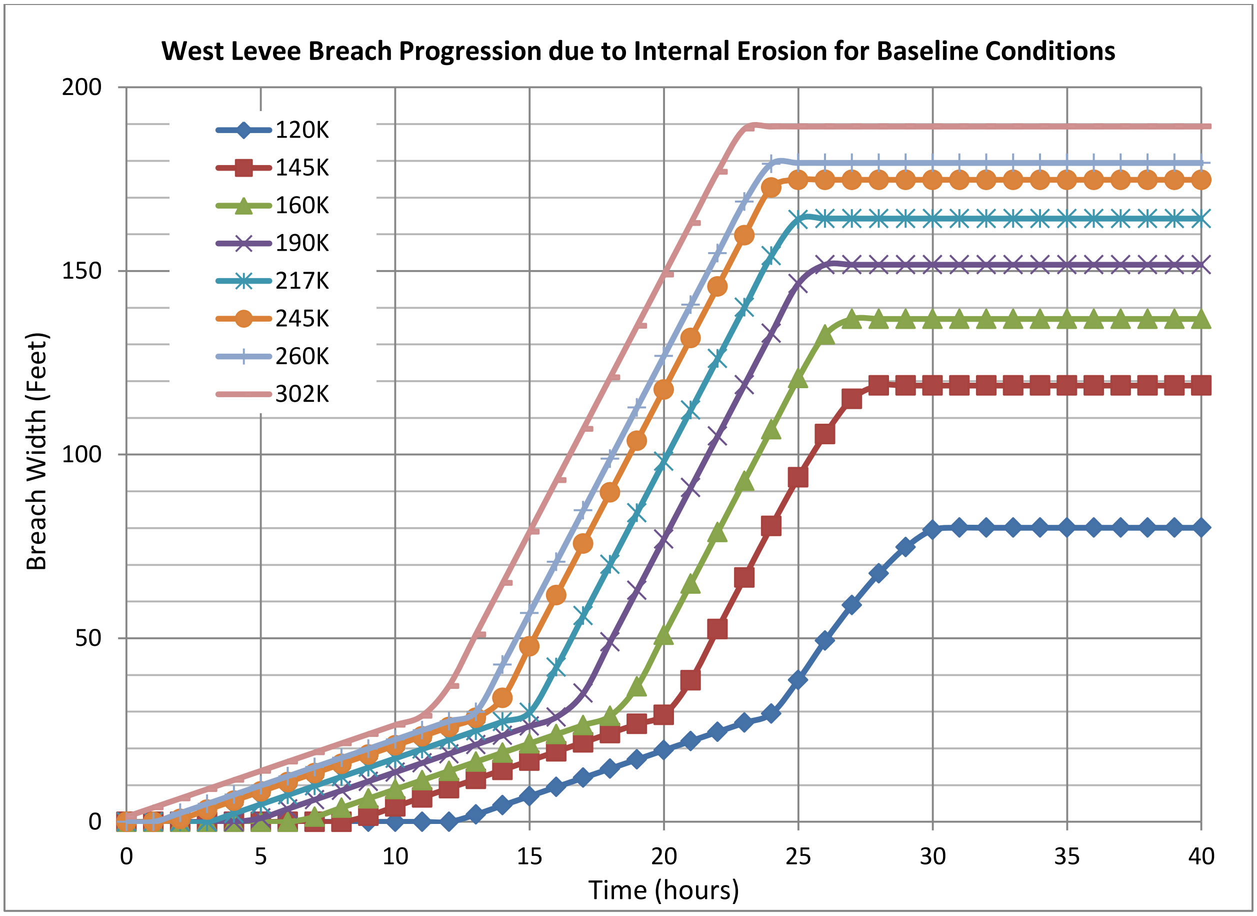


Figure 5-19 West Levee Breach Progression due to Internal Erosion for Baseline Conditions

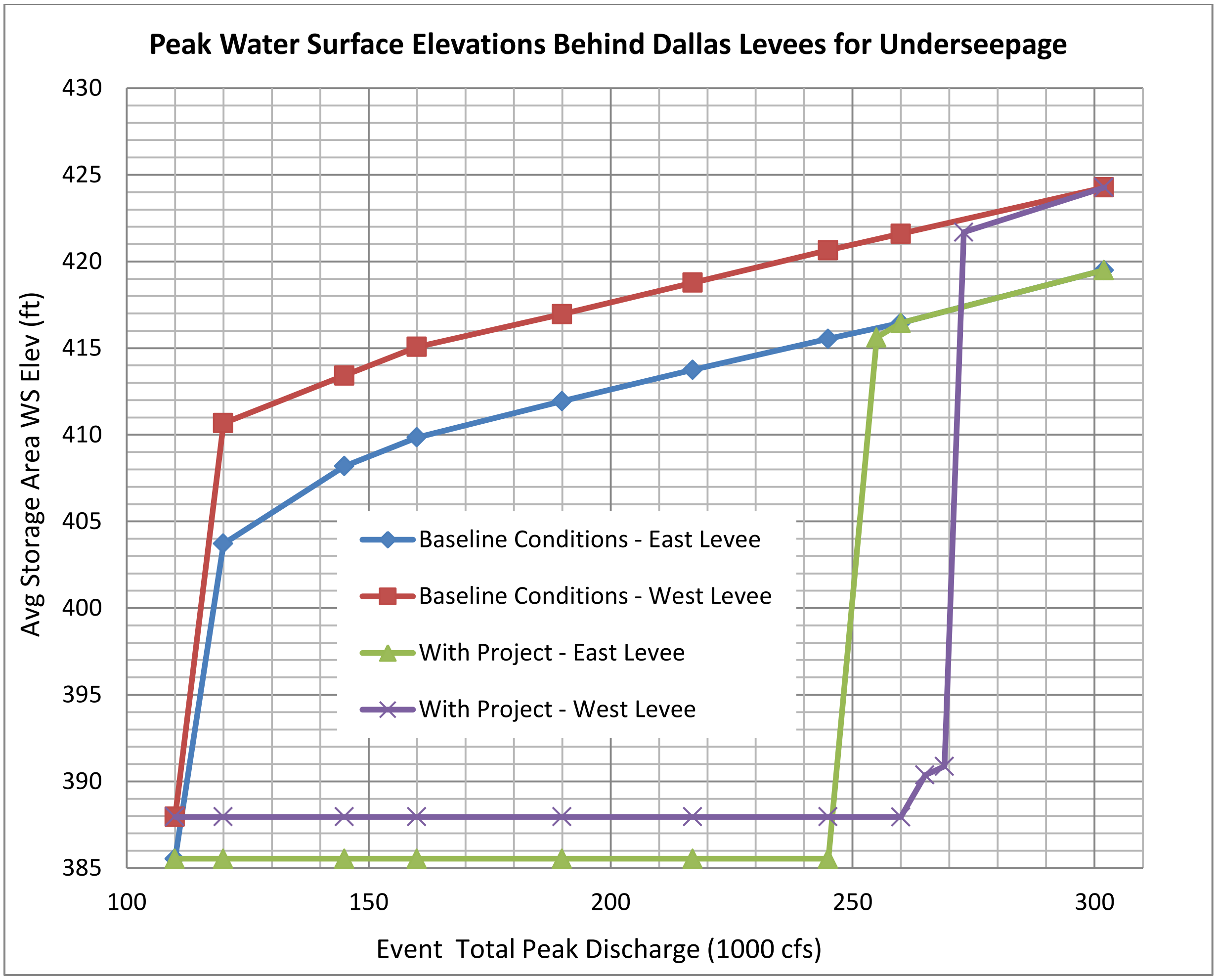


Figure 5-20 Maximum Water Surface Elevations Behind the Levees for Internal Erosion Baseline and With-Project Conditions

5.7 TENTATIVELY SELECTED PLAN FOR FLOOD RISK MANAGEMENT

The FRM NED Plan based on economics was found to be the 277k levee raise with the 3:1 levee side slopes along with the AT&SF Bridge modification. That plan had the highest net benefits out of all of the analyzed alternatives. Therefore, the combination of the 277k levee raise with the AT&SF Bridge Modification Plan is the selected plan for FRM and includes excavation within the Dallas Floodway for borrow material to construct the levee raise.

This plan is regarded as meeting the requirements for "safe and predictable performance" per ER 1101-2-101 with consideration for meeting USACE geotechnical construction standards and includes levee superiority at all locations where the existing levee crest is currently higher than the levee raise segments. The District determined not to recommend additional superiority beyond what already exists in the system with the Recommended Plan in place. However, further superiority for the levee raise locations that may be in the vicinity of critical infrastructure, such as at pump stations, will be considered in the final design phase. These potential changes are expected to be minor design refinements to the Recommended Plan.

The District evaluated the performance of Controlled Overtopping alternatives as well as Armoring alternatives combined with superiority. Both alternatives have similar functions and neither were found to be cost effective at reducing risk for flood damage or life safety.

6.0 THE COMPREHENSIVE ANALYSIS PHASE

6.1 PURPOSE OF THE COMPREHENSIVE ANALYSIS PHASE

Following identification of the Tentatively Selected Plan for FRM, the subsequent phase, known as Comprehensive Analysis, was developed to analyze the combination of all project features and to develop a plan to recommend as the MDFP. See Section 6.10 for additional detail of the MDFP. The H&H analysis for the Comprehensive Analysis considered various combinations of projects using the best available information for each project to determine if some combinations of project features potentially have a negative impact on flood risk.

Overall, the goal in the Comprehensive Analysis was to ensure the projects would function on a system-wide basis and the combined features would not impact the functioning of the MDFP. During this process, the Corps reviewed each feature to ensure that it could meet the Corps engineering and safety standards. The Comprehensive Analysis modeled features collectively and not on an individual basis as described later. In the determination of “hydraulic neutrality” a process of plan comparison in the H&H analysis was used to evaluate if the 1988 Upper Trinity River ROD H&H criteria is met.

The 1988 ROD H&H criteria was originally developed for the purpose of limiting potential increases in flood risk in the Trinity River corridor due to floodplain developments and has been applied to the USACE Section 404 regulatory process in the Upper Trinity River corridor since 1988. While the USACE is not constrained by this regulatory process for development of projects that are consistent with USACE mission objectives, it was expected that the Dallas Floodway Feasibility Study would identify a plan that would be a combination of USACE mission objectives (FRM, Ecosystem Restoration, and Recreation, etc.), projects by local interests, and other agencies such as the Federal Highway Administration (FHWA). These local interest type projects on the Trinity River and tributaries have historically been subject to the ROD criteria and all the local Section 408 projects described herein are evaluated independently using the 1988 ROD H&H criteria. Therefore, it was deemed appropriate for the USACE development of an overall Tentatively Selected Plan for the Dallas Floodway to use the ROD criteria to evaluate these combinations of project components that have varying and sometimes competing hydrologic and hydraulic impacts. This evaluation process is consistent with the original intent of the ROD criteria and ensures that projects that may have significant FRM, ER, and Recreation benefits for the City of Dallas are designed in such a way that minimizes any potential negative flood risk impacts beyond the limits of the Dallas Floodway.

Due to the scope and complexity of the project components proposed within the Dallas Floodway, it was expected that not all combinations of projects proposed would meet the ROD criteria at every location in the Trinity River corridor. However applying the criteria consistently between proposed plans through the preliminary design process provides a means of comparing the flood risk impacts of one plan to another in the interest of making the best risk informed decisions. In addition, the ROD was written with the intent that “the Regional EIS, its public review, and this ROD serve only to establish the best overall public interest as it applies to the Trinity River and its tributaries”. This statement from the ROD has been interpreted to mean that it represents overall benefits to Upper Trinity River floodplain developments only with regard to limiting flood risk increases and environmental impacts. The ROD further states, "Variance from the criteria would be made only if public interest factors not accounted for in the Regional EIS overwhelmingly indicate that the "best overall public interest" is served by allowing such variance. Therefore, it is also presumed that there may be circumstances or proposed developments that may have greater or wider ranging public benefits than those accounted for in the Regional EIS, which would justify a variance to the criteria. Thus, the ROD criteria is used to ensure that overall FRM project goals of the Dallas Floodway Feasibility Study are met and represent the "best overall public interest" even if the preliminary design does not meet every point of the ROD criteria.

The following is a brief description of the history and development of the 1988 ROD as well as its local permitting companion that followed, the Trinity River CDC Process.

6.1.1 The Upper Trinity River 1988 ROD

During the Dallas-Fort Worth Metroplex development boom in the mid-1980s, the USACE began to receive numerous requests for federal Section 404 permits within the Trinity River floodplain for commercial and residential development. Individually or cumulatively, these projects were considered to have the potential to compromise existing flood control protection afforded to floodplain residents, and to impact wetlands and other natural resources. The USACE Fort Worth District Engineer determined that it was necessary to develop a regional perspective to evaluate the impacts of individual permit decisions in accordance with the spirit and intent of the National Environmental Policy Act (NEPA) and other applicable laws.

Therefore, during 1984 through 1988, the USACE prepared a regional environmental impact statement "for the sole purpose of establishing a permitting strategy for the Trinity River and its tributaries." The *Regional Environmental Impact Statement Trinity River and Tributaries – 1988* (TREIS) determined that the cumulative impact of allowing individual development projects in the Trinity River floodplain could be both measurable and significant. The TREIS also indicated that the permitting approach adopted by the USACE had the potential to significantly reduce flood hazards.

Based on the TREIS findings, the USACE issued a ROD in April 1988 specifying criteria the USACE would use to evaluate Section 404 permit applications in the Trinity River Corridor. This discussion deals primarily with the evaluation of Hydraulic Impacts for projects located within the Standard Project Flood floodplain of the Elm Fork, the West Fork and the main stem Trinity River and is presented in the ROD as:

"Hydraulic Impacts - No rise in the 100-year or SPF elevation for the proposed condition will be allowed."

"The maximum allowable loss in storage capacity for the 100-year and SPF discharges will be 0% and 5% respectively."

"Alterations in the floodplain may not create or increase an erosive water velocity on or off-site."

In addition, the ROD further states that the cumulative impacts of other projects in the vicinity will be considered and is presented in the ROD as:

"Cumulative Impacts - The upstream, adjacent, and downstream effects of the applicant's proposal will be considered. The proposal will be reviewed on the assumption that adjacent projects will be allowed to have an equitable chance to be built, such that the cumulative impacts of both will not exceed the common criteria."

The Upper Trinity River ROD hydrologic and hydraulic criteria have been used since the signing of the ROD in 1988 as a measure to evaluate the impacts of proposed developments in the TREIS study area for Section 404 permit actions. In addition to the development of the ROD criteria for evaluation of Section 404 permit actions by the USACE in the Trinity River corridor, a local development permitting strategy for use by local governments (cities and counties) within their own local jurisdictions in the Upper Trinity River corridor was being proposed and became known as the Trinity River CDC process.

The Regional Trinity River CDC Process

In response to the TREIS and ROD in 1988, the cities and counties in the Trinity River Corridor formed the Trinity River Steering Committee (Steering Committee), facilitated by the North Central Texas Council of Governments. The Steering Committee adopted a Draft Statement of Principles for Common Permit Criteria (January 1988), a Resolution for a Joint Trinity River CDC Process (December 1988), and a Regional Policy Position on the Trinity River Corridor (January 1989).

In addition to the policy-oriented Steering Committee, a technically-oriented Flood Management Task Force was formed, comprised of city and county staff. The Steering Committee directed the Flood Management Task Force to develop a process and manual based on the criteria outlined in the USACE ROD. The result was the publication of the 1st Edition of the CDC Manual, drafted by the Flood Management Task Force following a two and one-half year period of intense discussion and negotiation. The Trinity River Corridor Steering Committee approved the first edition of the CDC Manual on May 23, 1991. Nearly two years later, all participating cities and counties had officially amended their floodplain ordinances to adopt the CDC Common Regional Criteria and process. This Common Regional Criteria is nearly identical to the H&H criteria established in the 1988 TREIS ROD with the only difference being the statement: "No increase in the 100-year flood water surface elevation and no significant increase in the Standard Project Flood water surface elevation." The Common Regional Criteria have been used for CDC permit actions to evaluate proposed projects within the Regulatory Zone of the Upper Trinity watershed. Second, third, and fourth editions of the CDC Manual have been approved since 1991. Although the CDC Manual serves as a guide for the H&H analysis required for CDC development activity permit applications, the H&H technical portions of the CDC Manual are also used to describe the H&H evaluation and analysis procedures for Section 404 permit actions since the H&H aspects of the two programs are similar.

6.1.2 Methodology for Applying the ROD Criteria

The ROD criteria are used to ensure that projects are designed in such a way that there are no flood rises in the water surface profile and that there are no valley storage losses for the 100-year flood and less than 5% valley storage loss for the SPF event. The evaluation process requires that a permit applicant prepare a HEC-RAS hydraulic model generally using the current Upper Trinity River CDC HEC-RAS model as a base condition. The CDC HEC-RAS model is maintained and usually distributed by the USACE to be used for evaluation of any and all projects that require a Section 404 Permit or a CDC Permit. In the case of the Dallas Floodway Feasibility Study, the base model was developed from the current CDC HEC-RAS model and updated slightly for existing conditions. The Dallas Floodway Feasibility Study model was also geo-referenced. Further discussion of the Dallas Floodway Feasibility Study Existing Conditions model is provided below in Section 6.2.

Often the development of a With-Project HEC-RAS model requires that additional cross sections be added to the CDC Model to adequately represent the proposed floodplain geometry changes due to the proposed project. In the event that additional cross sections are needed, a new base model is developed with the addition of cross sections to the original CDC HEC-RAS model. This newly developed model is referred to in the CDC Manual as the "Revised CDC Model" and is then used for comparison to the With-Project Model to evaluate the project's hydraulic impacts. Both the Revised CDC Model and the With-Project Model will generally have the same number of model cross sections in the same locations so that the analysis of the impacts of the proposed project reveals only the changes in floodplain geometry and land use. In addition to this comparison, a comparison between the original CDC model and the Revised CDC Model is required as a check to insure that the addition of cross sections to the base model does not skew the comparison results. The Revised CDC Model and the With-Project Model are then used to compute water surface profiles through the river reach impacted by the proposed development and a comparison of the water surface elevations on a cross section by cross section basis is made.

The second part of the evaluation is the valley storage computation. Valley storage is a measure of the floodplain volume capacity. Changes in floodplain volume due to developments can result in changes in the timing of flood peaks and potentially increase the flood event peak flow. A significant loss of valley storage may in turn increase the risk of flood damage downstream of the proposed development. Valley storage change is required to be computed to determine if a loss of valley storage would occur due to the project implementation and to quantify the magnitude of the change. Since the hydraulic impact evaluation is a peak flow analysis for the 100-year and the SPF events, the valley storage evaluation is also a peak flow determination. Valley storage is defined as the water volume that occupies the floodplain during the passing of the flood event and in this evaluation only the volume at the flood event peak is computed. Valley storage change resulting from the proposed project is based on a comparison of the valley storage or water volume that originally exists on a project site for either flood event to the valley storage with the project in place for both on-site and off-site areas if a valley storage change occurs off-site. On-site valley storage is the peak flow water volume below the 100-year and the SPF water surface that exists <u>only on the project site</u>. All other areas upstream, downstream, and on the opposite side of the river from the project site must be excluded in the on-site valley storage computation in order to afford adjacent property owners the same opportunity for development. While the ROD criteria limit the impacts of proposed projects to no rise in the water surface profile, it does not preclude a lowering of the water surface profile. However, if a proposed project results in a lowering of the water surface profile off-site, this would be regarded as a loss in valley storage and must be computed in the total valley storage change.

Valley storage change resulting from a project is expressed both in terms of the actual change in water volume but also in terms of a percent of the original valley storage that existed on the project site and can be either a valley storage loss (less than original on-site volume) or a valley storage gain (greater than original on-site volume). In order to compute the percent change in valley storage, the on-site valley storage for both flood events must be determined for Pre-Project conditions. This computation can be accomplished in one of several ways. The most common method may be the use of the "average end area" method using closely spaced cross sections, which trace the on-site terrain and reflect the area between the ground surface of the floodplain and the water surface on the project site. Each cross sectional area is then averaged with the next cross section and multiplied times the cross section spacing distance to compute the volume of water above the project site. Other methods employ the use of specialized software that create digital terrain models that may already be employed during the design of the project to compute the actual cut/fill volumes for the project. In some cases, the HEC-RAS model may be used to compute valley storage since the model uses valley cross sections and computes water volume in the course of the hydraulic computations using the average end area method. This method normally would not be used for on-site valley storage since in most cases the computation cannot be limited to only the project site and normally is not as accurate as other methods for on-site valley storage. However, the HEC-RAS model would in most cases be used to compute valley storage off-site since it is usually the only method available for off-site areas.

In some cases, valley storage losses occurring on one project site may be compensated at another site if it is within a reasonable distance with consideration for hydrologic impacts. Both sites would then be regarded as one project and permit conditions would be contingent on both sites for as long as the permit is valid. In this case, both sites would be used to compute the on-site valley storage. This is the normal process for smaller projects or projects that involve one or two disconnected sites. However, for a more complex project that involves multiple sites or may be spread out over a long river reach, defining the on-site conditions becomes more complex. This is the case with some projects proposed for the Dallas Floodway Feasibility Study and Comprehensive Analysis. At the outset of the study, a special process was developed for the valley storage evaluation.

The ROD criteria also contain the stated requirement that *"Alterations in the floodplain may not create or increase an erosive water velocity on or off-site".* This statement is somewhat dependent on engineering judgment, but it is generally interpreted as requiring projects to be designed such that no adverse flow patterns and velocities are created that result in erosive conditions that did not exist in the without-project condition. This would apply to on-site locations, but applies especially to the river channel and also to off-site areas. The criteria is intended to require the designer/planner of the project to evaluate the potential for significant changes to flow velocity in the floodplain during the occurrence of the two evaluation flood events, the 1% AEP flood and the SPF flood, even if these potential changes occur offsite. Since there are no specific limits of flow velocity change that accompany the criteria and no specific evaluation process described, the criteria is generally evaluated in terms of changes in computed average flow velocities for overbank and river channel areas. The criteria does not apply to localized erosive flow velocities that may be created near proposed obstructions, such as bridge piers, since erosion protection measures in almost all cases must accompany these types of structural elements that are subject to flood flows. The criteria would also not prohibit slight increases in computed river channel flow velocities for with-project conditions where flow velocities for such high flow events are almost always at levels that are considered erosive in the without-project condition. Therefore, the flow velocity evaluation is intended to identify significant increases in average flow velocities that may create erosive conditions that did not exist prior to the project implementation and may require design changes to mitigate these risks.

6.1.3 The Dallas Floodway Feasibility Study H&H Evaluation Process

Following the development of Without-Project or Existing Conditions HEC-RAS hydraulic models for the Dallas Floodway alternatives, it was determined that the valley storage computational procedure could not be accomplished in the usual manner and an alternate computational procedure for evaluating potential valley storage losses was adopted.

The alternate procedure was adopted because the determination of on-site valley storage in the usual manner by considering only the individual project sites that are actually modified would be very complex because of the large scale of some projects and also the large number of disconnected project sites. Also, further complicating the process was the fact that other separable smaller scale proposed projects within the same river reach with potential further modifications of some of the same land areas modified by the larger scale projects would also require H&H impact analysis and a strict onsite valley storage determination for these projects would be just as problematic. The adopted procedure differs from the usual computational approach in that it uses a river reach definition of "on-site" valley storage.

The selected Trinity River reach where the Dallas Floodway project modifications are proposed is considered the on-site pre-project valley storage area and the Dallas Floodway Existing Conditions Model is used to compute the “on-site” floodplain volume for this reach. The “on-site” reach is established by using selected HEC-RAS model Trinity River cross sections for the upstream and downstream limits of the reach. The HEC-RAS cross sections selected to define the “on-site” river reach limits are from cross section 107551 to cross section 148136. Cross section 107551 is a short distance downstream from the AT&SF railroad bridge and cross section 148136 is at the confluence of the West Fork and Elm Fork Trinity River. Valley storage changes within this reach for the With-Project alternatives are compared to the established Pre-Project floodplain volume for both the 100-year and SPF flood events to determine total volume change (acre-feet) as well as percent change. This adopted procedure is consistent with the overall goals of the ROD criteria to preserve valley storage by river reach. Any proposed projects within this reach identified as a local Section 408 project will use the Dallas Floodway Existing conditions HEC-RAS model as a baseline to prepare a HEC-RAS model for the proposed project and determine whether or not the project meets the ROD criteria as a stand-alone project. Each individual With–Project HEC-RAS model will then be used in the Comprehensive Analysis process to develop a cumulative impact analysis for this reach of the Trinity River. This approach will be used to determine if all proposed projects within the selected reach will have impacts that are within the limits of the ROD H&H Criteria. In this way, each local Section 408 project within this selected reach of the Trinity River will be analyzed independently and also cumulatively as part of the Comprehensive Analysis to determine if the ROD H&H criteria can be reasonably met.

The evaluation process requires that a With-Project and a base conditions or without-project HEC-RAS hydraulic model be prepared. For the purposes of this Dallas Floodway Feasibility Study Comprehensive Analysis, the Future Without-Project (Future Without Project) Model will serve as the base conditions model. Each model is then used to compute steady flow water surface profiles through the river reach impacted by the proposed development. A comparison of the water surface elevations is then made on a cross section by cross section basis to verify that there is no rise in the water surface profiles due to With-Project condition.

The Future Without Project HEC-RAS model is used to compute the total 100-year and SPF floodplain volumes between the upstream and downstream cross sections that define the project boundaries. The upstream and downstream HEC-RAS model limits used for comparison are from cross section 107551 to 148136. The With-Project volume will then be compared to the Future Without Project volume to determine the total change in valley storage due to the proposed project. These comparisons will be made for the two different comprehensive plans, the BVP with Trinity Parkway in the future condition and the BVP without Trinity Parkway in the future condition. The plans are two designs evaluated in “Alternative 2” presented in the Environmental Impact Statement (EIS) for the BVP and IDP. See additional detail in the main report (Section 3.2) regarding preparation of the EIS and other related Comprehensive Analysis criteria. In addition, Section 3.11.2 provides a description of the Dallas Floodway Project EIS.

The first step in this evaluation process is to develop the HEC-RAS models for the Future Without-Project and the With-Project conditions. The process for developing these models included the following steps:

1. Update the base model geometry to represent Existing Conditions.
2. Create a Future Without-Project model.
3. Create the With-Project model for the FRM plan.
4. Create a With-Project model for the BVP without Trinity Parkway plan.
5. Create a With-Project model for the BVP with Trinity Parkway plan.

The following sections provide more detail on the project features included in each of those models and how they were developed from the base HEC-RAS model.

6.2 The Baseline Conditions HEC-RAS Model

The current Dallas Floodway Feasibility Study HEC-RAS steady flow model for Without-Project Conditions was used as the base conditions hydraulic model for the Comprehensive Analysis. This is the same base model that was used to create the Risk Assessment unsteady flow model described in section 4.3. The Dallas Floodway Feasibility Study steady flow model was developed specifically for this study with cross section and bridge geometry information used in the current version of the Upper Trinity River CDC model. The Dallas Floodway Feasibility Study model is a geo-referenced model that extends from near Hutchins, TX at the downstream end up to near the interstate 35E crossing on the Elm Fork and Grand Prairie, TX on the West Fork. The downstream boundary condition for the Dallas Floodway Feasibility Study steady flow model is about 15 river miles downstream of the Dallas Floodway and was defined with a rating curve. The Dallas Floodway Feasibility Study steady flow model cross sections terminate at the crest of the levees such that all flow is confined to the floodway between the levee crests. The Dallas Floodway Feasibility Study model was geo-referenced using the Texas North Central State Plane (feet) coordinate system. HEC-RAS version 4.1.0 was used for the Comprehensive Analysis.

The hydraulic models for the Dallas Floodway Feasibility Study have a long history. The development of the initial hydraulic models on the Trinity River began in 1991 for the entire Upper Trinity River region. After the USACE ROD was issued, a consistent permitting process was developed in the form of the Trinity River CDC, which required a common hydraulic model for all permitting actions. The first hydraulic models of the Trinity River, Elm Fork and West Forks were developed using HEC-2 to support those efforts. Since then, the hydraulic models for the Dallas Floodway have been repeatedly updated with better survey data, newer software versions, and structural changes in the floodway. The following sections summarize the data sources that are being applied in the current Dallas Floodway Feasibility Study hydraulic model.

6.2.1 Topographic Data

The original topographic mapping of the Trinity River floodplain was compiled from aerial photography flown in February and March of 1991. The 1991 mapping complies with National Map Accuracy Standards and has a vertical accuracy of plus or minus 0.5 feet. This data was based on the NGVD 29 vertical datum. After the 1991 elevation data was collected, the City of Dallas completed a Channel Widening and Levee Fill project in the mid-1990s. Therefore, additional topographic survey data for the Dallas Floodway levees and the Trinity River channel within the floodway was obtained in order to update the HEC-RAS model within the Dallas Floodway for Existing Conditions.

In 2003, a new levee survey generated one-foot contour interval topographic data for both the East and West Dallas Floodway Levees extending from the DART Rail Line Bridge (River Station 1083+80) to the Union Pacific Railroad Bridge (River Station 1216+23) to capture changes to the levees due to the City of Dallas' river channel widening and levee fill project completed in the 1990s. This survey also included a ground survey of the entire levee crest for both the East and West Levees with a ground elevation determined every 100 feet along the levees. This levee crest survey was used for the lateral weir input to the HEC-RAS unsteady flow analysis for levee overtopping.

Also in 2003, a river channel one-foot contour interval topographic survey for the existing river channel with bathymetry was developed. This survey extends from the DART Rail Line Bridge (River Station 1083+80) to the confluence of the Elm Fork and the West Fork (River Station 1481+36). This survey was intended to capture changes due to the river channel widening and levee fill project and any geomorphic and sedimentation changes to the river channel. Both the 2003 levee slope and crest survey and the river channel survey use the NAVD 88.

From these data sources, a composite digital terrain model for the Dallas Floodway between the downstream end of the floodway at the AT&SF Railroad bridge and the Elm Fork and West Fork confluence was created by combining the 1991 topographic data for the floodplain between the river channel and the levees with the 2003 1-foot contour bathymetry survey of the river channel and the 2003 1-foot contour survey of the levees. During the creation of this composite DTM, the 1991 data components were converted to NAVD 88. The final Dallas Floodway Feasibility Study HEC-RAS model cross sections were cut from this composite digital terrain model to update the HEC-RAS existing conditions model on the main stem of the Trinity River within the Dallas Floodway. In addition, later 1-foot contour interval topographic survey data within the Dallas Floodway Extension project reach immediately downstream of the Dallas Floodway was collected in 2006 using NAVD 88. The HEC-RAS model cross sections were updated using this data downstream of the Dallas Floodway. Therefore, the vertical datum of the model data is essentially a combination of NGVD 29 and NAVD 88. The most upstream and downstream reaches of the HEC-RAS model data were not converted to the later NAVD 88 because within the study area, the difference between NGVD 29 and NAVD 88 is less than one inch. Therefore, for the purposes of this feasibility study, the difference is regarded as insignificant.

6.2.2 Bridge Data and Modeling Approach

The Dallas Floodway Feasibility Study HEC-RAS model contains approximately 32 bridges, which span the Dallas Floodway, the downstream Trinity River, the Elm Fork and the West Fork. The Dallas Floodway Feasibility Study HEC-RAS model data for all existing bridges were taken from original bridge plans if available or best available survey data.

Due to the wide variety of pier shapes and types in these bridges, the “bridge modeling approach” was adjusted in some cases from the standard energy equation to pressure and/or weir flow at the bridge. This determination of bridge modeling approach was made based on the high flow (SPF Flood) water surface profile.

6.2.3 Model Calibration

Following the development of the initial HEC-2 hydraulic models for the Dallas Floodway and the Dallas Floodway Extension project reach of the Trinity River and Tributaries, calibration models were developed to determine if the models would reasonably reproduce water surface profiles from an actual flood event. A number of high watermarks from the June 1989 and May 1990 flood events within the Dallas Floodway and the Dallas Floodway Extension project reaches became available and represented the highest and most recent flood events that had occurred since the major reservoir projects in the watershed had been constructed. These high watermarks were supplemented with USGS gage data and used in the model calibration process. The 1991 topographic data represented floodplain hydraulic conditions at the time of the June 1989 and May 1990 floods sufficiently to be used without revision for the calibration model development.

The HEC-2 calibration model was computed with the USGS 1990 flood estimate for peak flow to compute a 1990 flood water surface profile and this profile was compared to the high watermarks that were available for the 1990 flood. The peak flow for the 1990 flood was estimated by the USGS to be 82,300 cfs at the USGS “At Dallas Gage” located at the Commerce Street Bridge. Manning’s roughness coefficients and other model parameters were adjusted in the HEC-2 calibration model until a reasonable water surface profile match to the high watermarks was achieved. The resulting water surface profile generally ranged from 0.2 to 0.5 foot above the high watermarks. Since 1991, no major flood events have occurred on the Trinity River at Dallas that would allow for significant updates to the model calibration.

6.2.4 Roughness Values

Initial Manning's roughness coefficients were estimated by field surveys, aerial photographs, and using the USGS “Guide for Selecting Manning’s Roughness Coefficients for Natural Channels and Flood Plains” by Arcement and Schneider. Manning’s roughness coefficients were then adjusted during calibration to achieve a reasonable water surface profile match to the high watermarks.

The final Manning’s roughness N-values used within the main stem Dallas Floodway are 0.055 for the overbanks and 0.035 for the river channel from Houston Street to the confluence of the West Fork and the Elm Fork. The roughness values used in the reach from the DART Rail Line Bridge to Houston Street are 0.055 for the overbanks and 0.030 for the river channel. The use of a lower roughness value for the river channel in the reach from the DART Rail Line Bridge to Houston Street is due to the river channel-widening project.

Downstream of the DART Rail Line bridge, channel roughness values transition from 0.030 to 0.050 in the reach between the DART and MLK Jr Blvd bridges, and from the MLK Jr Blvd Bridge to the downstream end of the model, the channel roughness values vary between 0.05 and 0.07, depending on density of the natural vegetation. Downstream of the DART Bridge and the floodway, the overbank roughness values generally vary between 0.05 and 0.20, as estimated from the land cover and vegetation in that reach.

Channel roughness values on the West Fork vary from 0.055 at the upstream end to 0.039 in the reach adjacent to the West Levee, while channel roughness values on the Elm Fork vary from 0.05 at the upstream end to 0.035 in the reach adjacent to the East Levee. Overbank N-values on the Elm Fork and West Fork generally range between 0.05 and 0.16, depending on the density of the natural vegetation.

6.3 The Dallas Floodway Feasibility Study Existing Conditions HEC-RAS Model

In recent years, several local projects have been approved for construction through the Section 408 permitting process that were not included in the Dallas Floodway Feasibility Study existing conditions HEC-RAS model developed following the 2003 survey data. For the Dallas Floodway Feasibility Study Comprehensive Analysis, any projects that were completed or had started construction as of March 31, 2012 are included as an update to the Dallas Floodway Feasibility Study Comprehensive Analysis existing conditions. Therefore, the base HEC-RAS steady flow model was updated to include those projects in the Existing Conditions geometry. Local projects were included that have both a potential hydraulic floodplain impact and sufficient design development to enable the estimation of any hydraulic impacts through hydraulic modeling. The Existing Conditions HEC-RAS model was computed both with the final existing conditions frequency flows for Dallas Floodway and the future flows for comparison to the Future Without-Project Conditions model.

6.3.1 Projects Included for the Dallas Floodway Feasibility Study Comprehensive Analysis Existing Conditions

Projects that have been included under Existing Conditions are described below; however, not all of the projects listed warranted actual changes to the HEC-RAS model. Research into the projects’ design plans was conducted by the Fort Worth District to determine whether the projects warranted any changes to the existing cross section geometry. More detailed descriptions of these projects and location information is provided in the main report.

1. Dallas Floodway Extension Project - The Dallas Floodway Extension project consists of the following major components: the Upper and Lower Chain of Wetlands, the Cadillac Heights Levee, the Lamar Street Levee, the Rochester Park Levee, and recreation features immediately downstream of the existing Dallas Floodway Levee System. The project area covers approximately 9,500 acres. Construction of the Dallas Floodway Extension project is on-going and scheduled to be completed in 2016; however, construction of all components of the project are assumed in place as described in the authorizing documents and associated NEPA documentation.

2. DART Orange Line - The DART Orange Line light rail project is 14-miles long and will connect existing DART lines to the Irving/Las Colinas area, ultimately providing rail service to Dallas-Fort Worth Airport. The project includes a new bridge over the Elm Fork Trinity River upstream from the Dallas Floodway East Levee. Construction began in 2009 and was completed in 2012.

3. Great Trinity Forest Land Acquisition and Trails - The Great Trinity Forest Master Plan Concept, approved by the Dallas City Council on March 26, 1997, proposes the development of multipurpose trails for recreation, education, and transportation. It also outlines the acquisition and preservation of bottomland hardwood forest within the Trinity River Corridor. The project area covers 6,000 acres; land acquisition and development continues and was scheduled to finish in 2012.

4. Hampton Road Bridge - A new six-lane bridge was constructed to replace the four-lane bridge at the Hampton/Inwood crossing of the Dallas Floodway. The project area is approximately 28 acres and construction of this project was completed in 2010.

5. Margaret Hunt Hill Bridge – The Margaret Hunt Hill Bridge is one of three proposed signature bridges that will span the Dallas Floodway. The structure is located between the Continental Avenue and Union Pacific Railroad bridges, connecting Singleton Boulevard in West Dallas across the Trinity River to Woodall Rodgers in downtown Dallas. This bridge is part of the proposed Woodall Rodgers extension designed to relieve traffic congestion. The project area is approximately ten acres and construction was completed in 2011.

6. Pavaho Pumping Plant - The City of Dallas and the USACE recently improved the Pavaho Pumping Plant in order to reduce the potential stormwater flooding impacts to people and property in the City of Dallas and extend the service life of the existing facility for at least another 50 years. Improvements include constructing a new pump station, improving the existing Pavaho Pump Station, utilizing the two existing gravity sluices, and installing a new junction box that would connect flow from the existing and proposed Pavaho Pump Stations. The project area is approximately 3.75 acres. Construction began September 2010 and was completed in 2012.

7. Santa Fe Trestle Trail Project - The extended Santa Fe Trestle Trail is a hike and bike trail that provides access to Moore Park, located off East 8th Street south of downtown Dallas, and is approximately 10.4 acres. The trail extends from Moore Park on the east side of the Trinity River and crosses the Trinity River via the abandoned AT&SF railroad truss bridge and portions of the old railroad trestle and continues to a parking lot constructed south of the planned Trinity Parkway. The trail currently ends at an access road near the downstream end of the East Levee. Construction was completed in 2011.

8. Sylvan Bridge - The Sylvan Bridge replaces the Sylvan Avenue approaches and low water crossing over the Trinity River with a single bridge structure that spans the Floodway. The proposed project involves the upgrade of the existing two-lane conveyance to a six-lane bridge, a left turn lane,

sidewalks, and pedestrian railing along both sides of the bridge. The bridge also includes two shared-use travel lanes (one in each direction) to accommodate a bike route along the bridge, a ramp to provide access to Crow Lake Park, and the relocation of the existing boat ramp at Crow Lake Park. The project area covers approximately 15.4 acres. Although construction is ongoing, the USACE has included this project in existing conditions as the regulatory review process was completed prior to 2010.

9. Texas Buckeye Trail - The City of Dallas added an additional 1.6 miles of hard surface trails to the Texas Buckeye Trail. The trail is located at the end of Bexar Street in Rochester Park. A three-quarter-mile spur from the trail takes visitors to a large grove of Texas Buckeye trees (*Aesculus arguta*) located adjacent to the Trinity River. Construction ended in 2009.

10. Trinity Overlook Park - The City of Dallas completed the Trinity Overlook Park in October 2008, which is located just south of the western approach to the Commerce Street Bridge and is less than half an acre. The Trinity Overlook Park includes shade tents and interpretive displays providing information on the Dallas Floodway, the Trinity Lakes, and new signature bridges.

11. Trinity River Audubon Center - The Trinity River Audubon Center is a 120-acre facility located south of South Loop 12 and east of IH-45. The Center provides a place for presenting educational and environmental interests in the TRC; eco-tourism activities; aquatic, archaeological, and historical exhibits; and theme gardens at the center of the Great Trinity Forest's trail system. The Trinity River Audubon Center opened in 2008.

12. Trinity River Standing Wave - This project includes the construction of in-stream standing wave structures for recreational use, and covers approximately 9 acres. In addition to the in-stream components, the project includes a shore component consisting of a canoe launch, small trails, a parking area, and ingress/egress points (launch and take-out) supported by retaining walls. Construction was completed in 2011.

13. Trinity Strand Trail - The Trinity Strand Trail is a 7.8-mile, hike/bike, commuter, and recreational trail that will run along the course of the original Trinity River, also known as the Old Meanders, through the heart of the Dallas Design District (located on the west side of Stemmons Freeway at Oak Lawn Avenue, consists of over 370 designer shops and showrooms). Construction of the trail began in 2009, and Phase I construction began in 2011.

14. Trinity Trails - The Trinity Trails includes an extensive network of trails within the Trinity River Corridor with 3.5 miles of trails that are designed for compatibility with environmentally sensitive areas, 7 miles of soft surface trails, and 26 miles of hard surface trails with pedestrian bridges across the river. Phase I includes an EcoPark trailhead and an entry to the Joppa Nature Preserve, Phase II will end at the Trinity River Audubon Center. Construction of Phase II was estimated to conclude in the fall of 2011.

15. Oncor Towers (West Levee Norwood 345 kV Transmission Line) - This project includes installing new power lines and consolidating existing lines. Oncor Electric Delivery has installed a new 345-kilovolt (kV) power transmission line from West Levee Switching Station located in Dallas to the Norwood Switching Station located in Irving. The City of Dallas and Oncor worked cooperatively to avoid routing a new line along the levees of the Trinity River and to relocate existing power transmission lines along the Trinity River. The transmission line covers almost 7 miles, 1 mile of which is underground. The project includes five new transmission towers within the Dallas Floodway on the main stem Trinity River and four new towers on the Elm Fork Trinity River. This project is complete and began service in the summer of 2010.

6.3.2 Results from the Existing Conditions HEC-RAS Model

The model update of the Dallas Floodway Feasibility Study Comprehensive Analysis Existing Conditions model has resulted in effectively no change to the water surface profiles for both the 100-year and SPF flood events from the previous existing conditions model. This is as expected since each of the permitted projects in this list that have resulted in floodplain modifications has undergone an independent hydraulic analysis to ensure that each project meets the requirements of the 1988 ROD criteria. Not all of the projects listed have resulted in changes to the existing conditions model due to their design or location. For example: the Pavaho Pumping Plant Project did not result in any physical changes to the floodplain within the Dallas Floodway and the Trinity Overlook Park is located on top of the West Levee.

The water surface profiles for the Dallas Floodway Existing Conditions are shown on Figures 6-6 through 6-9 for the Trinity River main stem and the Elm and West Forks. Eight profiles are shown from the 2-year (50% AEP) flood event to the SPF (0.04% AEP) flood event. These profiles are presented with computations performed using the current watershed conditions peak flow hydrologic analysis. The profiles show the relationship between the SPF water surface and the existing levee crest for both the East and West Levees. As shown, the current SPF flood event would be expected to overtop both the East and West Levees. The estimated incipient overtopping flow for the East Levee is approximately 245,000 cfs under existing conditions. This flood event is estimated at a 1500-year return period with a 0.067% AEP. The incipient overtopping flow for the West Levee is slightly higher at an estimated 255,000 cfs.

The water surface elevations computed within the Dallas Floodway for the 100-year and the SPF flood events for current watershed conditions are provided in Table 6-1. In this table, a comparison is provided between the Existing Conditions model with the current flows and the Future Without-Project model with the future flows. This comparison reveals primarily the water surface elevation change due to the estimated future increased peak flows for each flood event. Consequently, the water surface elevations for the Future Without Project Conditions computed with the future watershed conditions flows are higher than the Existing Conditions and are shown as positive values and shown in red indicating a rise in the water surface profile.

Table 6-1 Existing Conditions versus Future Without-Project Water Surface Elevations

		1% AEP Flood Event W.S. Elev.			**SPF Flood Event W.S. Elev.**		
Location	*River Station*	*Existing - Current Flows*	*Future Without Project -Future Flows*	*Difference*	*Existing - Current Flows*	*Future Without Project - Future Flows*	*Difference*
Trinity River Main Stem							
DS of AT&SF RR	107551	412.66	413.09	0.43	423.21	423.69	0.48
US of DART Rail	109035	413.72	414.17	0.45	425.06	425.42	0.36
Corinth St.	110009	413.77	414.22	0.45	425.08	425.46	0.38
IH 35E	114773	414.60	415.04	0.44	426.21	426.53	0.32
Houston St.	116243	414.95	415.43	0.48	426.85	427.24	0.39
IH 30	118966	415.89	416.39	0.50	428.00	428.43	0.43
Commerce St.	120765	416.36	416.86	0.50	428.55	428.99	0.44
UPRR	121639	416.82	417.32	0.50	429.18	429.56	0.38
MHH Bridge	122562	417.06	417.56	0.50	429.45	429.83	0.38
Sylvan Ave.	128158	418.43	418.94	0.51	431.06	431.48	0.42

		1% AEP Flood Event W.S. Elev.			SPF Flood Event W.S. Elev.		
Location	*River Station*	*Existing - Current Flows*	*Future Without Project -Future Flows*	*Difference*	*Existing - Current Flows*	*Future Without Project - Future Flows*	*Difference*
Hampton Rd.	134883	419.84	420.31	0.47	432.45	432.87	0.42
Westmoreland	140734	421.10	421.55	0.45	433.54	433.98	0.44
Confluence	148136	422.84	423.27	0.43	434.94	435.4	0.46
Elm Fork Trinity River							
Shady Grove Rd.	3190	423.45	423.87	0.42	435.52	435.98	0.46
SH 356	4826	423.52	423.93	0.41	435.57	436.03	0.46
BNSF	6689	423.59	424.01	0.42	435.66	436.12	0.46
SH 183	14544	424.05	424.78	0.73	436.59	437.09	0.50
US East Levee	18521	424.22	425.11	0.89	436.95	437.47	0.52
SH 482	22546	424.58	425.54	0.96	437.43	437.94	0.51
Loop 12	29438	425.82	426.79	0.97	438.45	438.56	0.11
West Fork Trinity River							
Loop 12	9763	426.18	426.63	0.45	437.68	438.16	0.48
US West Levee	12811	426.93	427.37	0.44	438.21	438.68	0.47
MacArthur Blvd	28841	435.78	436.1	0.32	443.67	444.13	0.46

6.4 The Future Without-Project HEC-RAS Model

Following the updating of the Existing Conditions model, a second HEC-RAS model was developed to represent Future Without-Project (Future Without Project) Conditions. This model includes all of the projects included in the existing conditions model as well as additional local projects that are reasonably foreseeable as part of future conditions. The Future Without-Project HEC-RAS model geometry was created by including the model geometry for these projects as described below and results are computed with the future conditions frequency flows for Dallas Floodway. The results of the Future Without Project model serve as the base line for comparison to the "With-Project" models for determination of "hydraulic neutrality" by evaluation of the overall project with regard to the 1988 H&H ROD criteria.

6.4.1 Projects Included Under Future Without-Project Conditions

The projects identified for Future Without-Project Conditions started construction after March 31, 2012, or they are regarded as future Section 408 projects because they were not included in the 2007 WRDA authority. Projects that have been included under Future Without-Project Conditions are described below; however, not all of the projects listed warranted changes to the HEC-RAS model due to either the project located outside the floodplain or the project does not have proposed physical modifications to the floodplain. Research into the projects' design plans was conducted by the Fort Worth District to determine whether the projects warranted any changes to the existing cross section geometry. For example, some of the individual IDP pump station projects have proposed riverside levee modifications resulting in encroachments of the floodway due to large discharge pipes running up and over the levees and some proposed pump station modifications do not result in floodplain encroachments. These pump station project plans are at the 35% level and do not currently include any mitigation features for potential floodplain impacts.

1) Beckley Avenue Improvements - The City of Dallas plans to improve Beckley Avenue at Commerce Street by adding four new vehicle lanes, reinforced concrete sidewalks, a new major drainage system, and upgraded water and wastewater mains. The project area is approximately three acres. Construction was estimated to conclude fall 2014. This project is not located within the floodway.

2) Belleview Trail Connector -The City of Dallas proposes to construct a trail connecting development, entertainment, and art districts via mass transit in the Cedars District. The trail would be slightly less than an acre and would connect the proposed Trinity Park to the DART Cedars Station. This project does not currently have an estimated start date. This project is not located within the floodway.

3) Bernal Trail - The City of Dallas would extend the existing Bernal Trail to link the Westmoreland Heights area to the Trinity Levee Trail along the west levee. The trail would go from Emma Carter Park to Tipton Park, and would be approximately 4.6 acres. Construction has not yet begun. This project is not located within the floodway.

4) Charlie Pump Station - This project includes proposed improvements to Dallas' Pump Station Charlie, located on the West Levee of the Trinity River. This pump station is part of the City's IDP, but the west levee IDP features were not included in the 2007 WRDA authorization. Therefore, the project is included under future without-project conditions as a local Section 408 project. This project's proposed design results in a floodplain encroachment and required a modification for the Future Without Project model geometry at Cross-section no. 115434.

5) Continental Pedestrian Bridge - As a result of the construction of the Margaret Hunt Hill Bridge, the Continental Avenue Bridge would be converted from vehicular to pedestrian and bicycle use. The project area would be approximately 2.6 acres, and construction is estimated to begin after the completion of the Margaret Hunt Hill Bridge in 2011. This project results in modifications to the existing Continental Avenue Bridge crossing of the Dallas Floodway but does not result in any modifications to the floodplain. Therefore, no modifications to the Future Without Project model were required.

6) Dallas Horseshoe Project - This project focuses on the IH-30 and IH-35 interchange on the western edge of downtown Dallas, locally known as the "Mixmaster;" and the portion of IH-30 south of downtown, locally known as the "Canyon." The project is a complete replacement of existing bridges for traffic lanes in both directions at both floodway-crossing locations. Preliminary modeling data for both bridge crossings was input to the Future-Without Project model by interpolating new cross sections at cross sections 118500, 114663, 114371, 114219, and 113902. Existing Conditions cross sections located at 114641, 114572, 114510, 114457, 114243, 114183, 114116, and 114054 were deleted. Preliminary bridge geometry data was input to the Future Without Project model for three individual bridges, one at the IH-30 crossing and two at the IH-35 crossing. Construction has not yet begun on either bridge crossing location at the time this analysis was performed.

7) Dallas Watersports Complex - The Dallas Watersports Complex (DWC) would include a waterskiing cableway, a pro-shop, snack bar, full-service restaurant, and viewing deck. The DWC would be located on Fishtrap Lake at the intersection of Hampton Road and Singleton Boulevard in West Dallas, and cover approximately 42 acres. This project does not currently have an estimated start date.

8) Delta Pump Station - This project includes proposed improvements to Dallas' Pump Station Delta, located on the West Levee of the Trinity River. This pump station is part of the City's IDP, but the west levee IDP features were not included in the 2007 WRDA authorization. Therefore, the project is included under Future Without-Project Conditions as a local Section 408 project. This project does not

result in any modifications to the floodplain. Therefore, no modifications to the Future Without-Project model were required.

9) DWU Waterlines - There are four Dallas Water Utilities (DWU) water lines that cross the Dallas Floodway on the main stem of the Trinity River. The Westmoreland/Mockingbird 48-inch water line was built in 1954 and will be replaced with a same size replacement line that is planned to go up and over both levees. The Hampton/Inwood 36-inch water line was built in 1943 and will be replaced with a same size replacement line that is planned to go up and over both levees. The Houston Street 24-inch water line was rebuilt in 1953 and will be replaced with a same size replacement line that is planned to go up and over both levees. The Corinth Street 24-inch water line was rebuilt in 1957 and will be replaced with a 48-inch replacement line that is planned to go up and over the West Levee, but underneath the East Levee. Implementation of these four lines would be performed in two phases; first phase would include upgrade and relocation between the levees within the Dallas Floodway, while the second phase would extend these four lines across the levees. This project's proposed design results in floodplain encroachments at four locations within the Dallas Floodway and required modifications for the Future Without-Project model geometry at cross section nos. 110086, 116111, 131788, and 140272.

10) EF2 Wastewater Interceptor Line and Laterals - This project consists of a new 108-inch diameter wastewater interceptor that would be installed parallel to and riverward, of an existing 90-inch wastewater line located within the Dallas Floodway and immediately adjacent to the Northwest Levee in Irving. Also included in this project are four lateral wastewater lines (points of entry) that are proposed to cross beneath the levee and connect to either the existing 90-inch line or the new 108-inch line. The project area would be approximately 3.7 acres, and construction was anticipated to begin in 2012 and last 18 months. This project does not result in any modifications to the floodway. Therefore, no modifications to the Future Without-Project model were required.

11) Elm Fork Flood Improvements and Parks - The Elm Fork Flood Improvements and Parks project includes enacting flood protection improvements, recreation facilities, and environmental restoration in the Elm Fork area. In addition, the project includes constructing an athletic complex south of Walnut Hill, a dog park, trails, and associated amenities. The project would cover approximately 4,150 acres. Construction of the flood protection improvements has not yet begun, and the Elm Fork Athletic Complex was estimated to begin in 2012. This project is located upstream from the Dallas Floodway and no modeling information was available prior to the development of the Dallas Floodway Feasibility Study Future Without-Project model. Therefore, no modifications to the Future Without-Project model were made.

12) Hampton Wetlands - Located near the proposed Trinity Parkway between the Sylvan and Westmoreland bridges, the Hampton Wetlands would mitigate the loss of wetland caused by the construction of the parkway. Additional proposed biofiltration Wetlands beyond the mitigation requirements would serve as biofiltration cells related to the proposed parkway storm runoff. Owing to their adjacency, the totality of cells contained in these combined wetlands would result in major increases in habitat benefits for the park and the greater Dallas urban region. No modeling information was available prior to the development of the Dallas Floodway Feasibility Study Future Without-Project model. Therefore, no modifications to the Future Without-Project model were made. During the study, it was determined they will no longer be managed as wetlands, but as hydraulic mitigation. Mitigation for environmental features impacted by the Trinity Parkway is expected to be conducted at a mitigation bank outside the Floodway.

13) IH-20 Gateway Park - The City of Dallas proposes to construct the IH-20 Gateway Park north of the intersection of IH-20 and Dowdy Ferry Road. The park would include picnic and fishing stations around the existing pond and canoe access to the Trinity River. The park would cover approximately 75 acres. Construction was estimated to begin in 2012. This project was included as a cumulative project because it affects recreational resources. No modeling information was available prior to the development of the Dallas Floodway Feasibility Study Future Without-Project model. Therefore, no modifications to the Future Without-Project model were made.

14) Irving Northwest Levee Repair - This 23-acre project would complete the rehabilitation of the Irving Northwest Levee for re-certification and re-accreditation for protection from up to and including the 100-year riverine flood event. This project consists of installing a slurry wall on the riverside toe of the existing levee (approximately 13,000 feet long and 25 feet deep) to minimize potential seepage issues associated with the levee during major flood events. It would also include the rehabilitation of a portion of the levee, by either overlaying with clay material or grouting the sand to reduce the potential for through seepage of the levee during flood events. This project was put on hold due to issues with soil borings. No modeling information was available prior to the development of the Dallas Floodway Feasibility Study Future Without-Project model. Therefore, no modifications to the Future Without-Project model were made.

15) Jefferson Memorial Bridge - The TxDOT proposes constructing a new Jefferson-Memorial Bridge to replace the existing Jefferson Street Bridge. The new bridge would provide a direct connection to and from IH-35E and could potentially include an interchange with the proposed Trinity Parkway. As the existing Jefferson Street bridge would be removed, and the proposed Trinity Parkway ramps would be relocated from existing Houston/Jefferson to the new Jefferson-Memorial, the impact on existing Trinity River H&H models would be considered neutral. No modeling information was available prior to the development of the Dallas Floodway Feasibility Study Future Without-Project model. Therefore, no modifications to the Future Without-Project model were made.

16) Joppa Gateway Park - The City of Dallas plans to construct the Joppa Gateway Park as an expansion and improvement of the existing South Central Park. The park would feature a spray ground, expanded trails, an open play field area, an additional small pavilion with picnic/barbeque stations, site furnishings, and would repair and upgrade the existing basketball court. The project was estimated to be under design in 2011. This project is not located within the floodplain.

17) Loop 12 Bridge - Under this project, the Loop 12 corridor near the western SH-183 crossing would be reconstructed to accommodate eight general-purpose lanes (plus auxiliary lanes), four continuous frontage road lanes (plus auxiliary lanes near ramp locations and cross-streets), and a reversible High-Occupancy Vehicle (HOV)/Managed facility. The project area would cover approximately 34 acres; the estimated construction date is April 2015. No modeling information was available prior to the development of the Dallas Floodway Feasibility Study Future Without-Project model. Therefore, no modifications to the Future Without-Project model were made.

18) Loop 12 Gateway Park - The City of Dallas proposes to construct the Loop 12 Gateway Park in a 2.15-mile long greenbelt running from the intersection of Loop 12 and IH-45, east to the Trinity River. The park would be approximately 153 acres and would feature streetscape and solar-powered lighting enhancements, signage, picnic pavilions, a parking area, and internal concrete trails. Construction was estimated to begin by 2012. No modeling information was available prior to the development of the Dallas Floodway Feasibility Study Future Without-Project model. Therefore, no modifications to the Future Without-Project model were made.

19) Martin Luther King Jr. Gateway and Cedar Crest Bridge Improvements - The City of Dallas proposes to construct the MLK Jr. Gateway and Cedar Crest Bridge Improvements, which would feature ball fields, trails, picnic areas, a lake, parking, and a connection from Cedar Crest Bridge down to the Gateway for access. Construction was estimated to begin summer 2012. No modeling information was available prior to the development of the Dallas Floodway Feasibility Study Future Without-Project model. Therefore, no modifications to the Future Without-Project model were made.

20) Moore Gateway Park - Moore Gateway Park would be a regional gateway providing access to the Dallas Floodway. Moore Gateway Park will be approximately 28.5 acres and will include athletic fields, a large pavilion, and access to the proposed Trinity River Standing Wave (refer to Section 4.2.1.11). Construction had an estimated completion date of summer 2013. No modeling information was available prior to the development of the Dallas Floodway Feasibility Study Future Without-Project model. Therefore, no modifications to the Future Without-Project model were made.

21) Pavaho Wetlands - The City of Dallas proposed to construct approximately 70 acres of stormwater wetlands adjacent to the Pavaho Pumping Plant outfall. The wetland design would include a high flow channel for runoff from larger storm events. Cross section numbers 124626 through 129999 were modified to reflect the HEC-RAS model project data provided by A/E for input to the Future Without-Project model.

22) Riverfront Boulevard - This 27-acre project involves converting Riverfront Boulevard (formerly Industrial Boulevard) to a 1.5-mile eight-lane thoroughfare with a 150-foot wide right of way. Riverfront Boulevard would become a "complete street" and include landscape zones, bicycle lanes, and pedestrian sidewalks. The project would also include an upgrade of the drainage system and replacement/upgrade of existing water and wastewater transmission and distribution lines. Construction was estimated to be complete in November 2013. This project is not located within the floodplain.

23) Rochester Gateway Park Improvements - The City of Dallas proposes to construct the Rochester Gateway Park in the Rochester Park neighborhood. The park would be approximately 983 acres and would feature kiosks, a parking area, lighting, a trailhead link between the Great Trinity Forest and the residential areas around Rochester Park, and a soft surface link up and over the levee. No modeling information was available prior to the development of the Dallas Floodway Feasibility Study Future Without-Project model. Therefore, no modifications to the Future Without-Project model were made.

24) S.M. Wright Project - The TxDOT is undertaking design plans and environmental studies for improvements to US-175/S.M. Wright Freeway. The 48.5-acre study area includes improvements to IH-45 from S.M. Wright Freeway (US-175) to south of Lamar Street (1.7 miles), S.M. Wright Freeway from IH-45 to SH-310 near Budd Street (2.5 miles), and providing direct connecting ramps between US-175 and IH-45 (1.5 miles). This project would reduce traffic flow and convert the elevated, 10-lane high-speed S.M. Wright Freeway to a 6-lane low-speed, signalized, at-grade arterial without bridges. Subject to funding availability, construction near Budd Street was estimated to begin January 2015. No modeling information was available prior to the development of the Dallas Floodway Feasibility Study Future Without-Project model. Therefore, no modifications to the Future Without-Project model were made.

25) SH-183 Bridge - The TxDOT is planning a new bridge crossing at the Elm Fork of the Trinity River as part of an overall development plan for SH-183. The TxDOT is studying several alternatives in order to develop a plan for improvements; currently the bridge design would cover approximately 76 acres. In addition to the bridge, alternatives include revising the HOV lanes to provide three lanes in each direction. Subject to funding availability, construction was estimated to begin January 2015. No modeling information was available prior to the development of the Dallas Floodway Feasibility Study

Future Without-Project model. Therefore, no modifications to the Future Without-Project model were made.

26) Texas Horse Park - The 500-acre Texas Horse Park (initially proposed as the Trinity Equestrian Center), would be located northeast of the intersection of Loop 12 and Pemberton Hill Road. The Center would host world-class equestrian competitions of all types, provide riding trails, stabling/boarding, and offer a variety of riding programs. Construction was estimated to begin in 2012. Modeling information was not available prior to the development of the Dallas Floodway Feasibility Study Future Without-Project model. Therefore, no modifications to the Future Without-Project model were made.

27) Trinity Lakes Streetcar Loop - The proposed Trinity Lake Streetcar Loop would better connect Oak Cliff and West Dallas to downtown. The 4.75-mile route would zigzag from the convention center hotel, down the east-west commercial district, and up to the Arts District. It is intended to create economic development opportunities for downtown along with West Dallas, the Design District, and Oak Cliff. This project is not located within the floodplain.

28) Trinity Portland Pump Station - This project by the City of Dallas includes a new pump station located near the existing Trinity Portland station on the West Levee of the West Fork Trinity River. This pump station is part of the City's IDP, but the west levee IDP features were not included in the 2007 WRDA authorization. Therefore, the project is included under Future Without-Project Conditions as a local Section 408 project. This project's proposed design results in a floodplain encroachment and required a modification for the Future Without-Project model geometry at cross section number 3831 on the West Fork Trinity River.

Subsequent to developing the Future Without-Project Conditions HEC-RAS model, Section 4013 of the WRRDA of 2014 (Public Law 113-121), provided a technical correction to Section 5141(a)(2) of the WRDA 2007. Section 5141 was amended by inserting "and the Interior Levee Drainage Study Phase-II report, Dallas, Texas, dated January 2009," after "September 2006." Thus, the WRRDA authorization adds the West Levee IDP to the Section 5141 of WRDA 2007 authorization. The hydrologic and hydraulic analysis for the Comprehensive Analysis is not affected by the authorization change since the Comprehensive Analysis modeled features collectively and not on an individual basis.

6.4.2 Results from the Future Without-Project HEC-RAS Model

The following hydraulic modeling results of the Dallas Floodway Feasibility Study Future Without-Project HEC-RAS hydraulic model includes a comparison of the Dallas Floodway Feasibility Study Comprehensive Analysis Existing Conditions model results to the Future Without-Project model results. This comparison has indicated very small changes to the water surface profiles for both the 100-year and SPF flood events from the existing conditions model. This is as expected since most of the permitted projects in this list either are not located in the floodplain or have been designed to ensure that the project meets the requirements of the 1988 ROD criteria. However, not all of the projects included in the Future Without-Project model have advanced to a level of development to include design to mitigate for any potential negative floodplain impacts as noted in some project descriptions above.

The water surface profiles for the Dallas Floodway Future Without-Project only are shown on Figures 6-10 through 6-13 for the Trinity River main stem and the Elm and West Forks. The profiles are shown for the 2-year (50 % AEP) flood event up to the SPF (0.04% AEP) flood event. The water surface profiles for the Dallas Floodway Future Without-Project compared to the Existing Conditions are shown on Figures 6-14 through 6-17 for the Trinity River main stem and the Elm and West Forks. The profiles are shown for the 100-year (1 % ACE) flood event and the SPF (0.04% ACE) flood event. All of these profiles are presented for computations performed using the future watershed conditions peak flow hydrologic analysis. The existing conditions model has also been computed with the future flows in this case rather than the current watershed conditions flows to provide a modeling results comparison based solely on project geometry differences. The profiles also show the relationship between the SPF water surface and the existing levee crest for both the East and West Levees.

The water surface elevations computed within the Dallas Floodway for the 100-year and the SPF flood events for future watershed conditions for both Future Without-Project Conditions and Existing Conditions (with future watershed flows) are provided in Table 6-2. In this comparison table, the positive values shown in red indicate a water surface rise in the Future Without-Project Conditions compared to the Existing Conditions.

Table 6-2 Existing Conditions versus Future Without-Project Water Surface Elevations

		1% AEP Flood Event W.S. Elev.			**SPF Flood Event W.S. Elev.**		
Location	*River Station*	*Existing*	*Future Without -Project*	*Difference*	*Existing*	*Future Without -Project*	*Difference*
Trinity River Main Stem							
DS of AT&SF RR	107551	413.09	413.09	0.00	423.69	423.69	0.00
US of DART Rail	109035	414.17	414.17	0.00	425.42	425.42	0.00
Corinth St.	110009	414.22	414.22	0.00	425.46	425.46	0.00
IH 35E	114773	415.05	415.04	-0.01	426.65	425.53	-0.09
Houston St.	116243	415.42	415.43	0.01	427.32	426.24	-0.08
IH 30	118966	416.36	416.39	0.03	428.49	428.43	-0.06
Commerce St.	120765	416.83	416.86	0.03	429.04	428.99	-0.05
UPRR	121639	417.29	417.32	0.03	429.62	429.56	-0.06
MHH Bridge	122562	417.54	417.56	0.02	429.89	429.83	-0.06
Sylvan Ave.	128158	418.91	418.94	0.03	431.53	431.48	-0.05
Hampton Rd.	134883	420.32	420.31	-0.01	432.93	432.87	-0.06
Westmoreland	140734	421.55	421.55	0.00	434.02	433.98	-0.04
Confluence	148136	423.27	423.27	0.00	435.43	435.40	-0.03
Elm Fork Trinity River							
Shady Grove Rd.	3190	423.87	423.87	0.00	436.02	435.98	-0.04
SH 356	4826	423.93	423.93	0.00	436.07	436.03	-0.04
BNSF	6689	424.01	424.01	0.00	436.15	436.12	-0.03
SH 183	14544	424.78	424.78	0.00	436.12	436.09	-0.03
US East Levee	18521	425.11	425.11	0.00	437.49	437.47	-0.02
SH 482	22546	425.54	425.54	0.00	437.97	437.94	-0.03
Loop 12	29438	426.79	426.79	0.00	438.59	438.56	-0.03
West Fork Trinity River							
Loop 12	9763	426.61	426.63	0.02	438.17	438.16	-0.01
US West Levee	12811	427.36	427.37	0.01	438.69	438.68	-0.01
MacArthur Blvd	28841	436.10	436.10	0.00	444.13	444.13	0.00

A valley storage comparison has been computed for the Future Without-Project Conditions compared to the Existing Conditions. The valley storage change for the Future Without-Project is -0.11% for the 1 % ACE flood event and -0.45% for the SPF compared to the Existing Conditions. This means that the Future Without-Project Conditions results in a small valley storage loss for both flood events. In this comparison, the Existing Conditions is regarded as the base condition and the percent change is computed as a percentage of the onsite volume for the Existing Conditions for the same river reach as the onsite volume for the Future Without-Project Conditions.

All of the following water surface elevations and valley storage comparisons for alternatives are compared to the Future Without-Project Conditions. Therefore, the percent change in valley storage is computed regarding the Future Without-Project onsite volume as the base condition. The computed onsite volume for the Existing condition is 37,582 acre-feet for the 1% ACE flood event and 65,100 acre-feet for the SPF event. The computed onsite volume for the Future Without-Project Conditions is 37,555 acre-feet for the 1% ACE flood event and 64,937 acre-feet for the SPF event. The onsite valley storage volume has been computed as the volume of water occupied by the floodplain at the peak flow for each flood event from river station 107551 to 148136. This river reach extends from downstream of any proposed floodplain modifications for any alternatives upstream to the confluence of the Elm Fork and West Fork.

6.5 THE WITH-PROJECT FRM PLAN MODEL

The first "With Project" HEC-RAS geometry created for the Comprehensive Analysis was for the Dallas Floodway TSP for FRM (component of the MDFP), as described in section 5.7 of this appendix. The FRM plan model has been computed with the future watershed conditions peak flows for direct comparison to the Future Without-Project Conditions. However, the FRM Plan has not been developed to achieve hydraulic neutrality. With the effects of the partial removal of the AT&SF Bridge, no rise in water surface profiles are expected compared to the Future Without-Project Conditions. However, due to the lowering of the water surface profiles upstream of the modified bridge, valley storage loss is expected with no reasonable means of achieving hydraulic neutrality with respect to the 1988 ROD H&H criteria, especially for the 100-year flood event. Therefore, the FRM Plan model results compared to the Future Without-Project Conditions is used in this analysis solely to document the effects of the FRM project alone as an interim condition.

6.5.1 Projects Included in the Flood Risk Management Plan Model

The With-Project FRM Plan HEC-RAS model was developed by creating a FRM Plan model that encompasses all of the project features of the Future Without-Project model with the added FRM Plan features. The added FRM components include (1) the proposed AT&SF Bridge modification, (2) the levee raise to 277k with 3:1 riverside levee slopes on both levees (except where 4:1 riverside levee slopes already exist), and (3) the excavated borrow areas needed for the levee raise construction. The levee raise does not require levee slope modifications since the existing riverside levee slopes are currently at 3:1 or flatter. Nor does the model require modification for the levee raise height since the HEC-RAS steady flow model cross sections do not extend past the levee crest for either levee. This means that only flow within the floodway is computed and there is no computation of flow for overtopping of the levee crest. By definition of the FRM Plan, no overtopping of the levees will occur up to and including the 277,000 cfs (SPF) flood event. In effect, the only model revisions required for this effort are for the model input for the AT&SF Bridge modification plan and the excavation for the borrow areas. Floodplain cross sections were modified at cross section numbers 139920, 140096, 140272, 141789, and 142004 based on supplied maps from the Fort Worth District's Civil Design Section for borrow area locations and depths.

6.5.2 Results from the FRM HEC-RAS Model

The following hydraulic modeling results of the Dallas Floodway Feasibility Study FRM Plan HEC-RAS hydraulic model include a comparison of the Dallas Floodway Feasibility Study FRM Plan model results to the Future Without-Project model results. This comparison indicates the impact of the AT&SF bridge modification plan as part of the FRM Plan. The water surface profiles for both the 100-year and SPF flood events are shown with reference to the proposed levee crest height modification as part of the FRM Plan. For clarity, the East levee and the West Levee crest profiles are shown on separate figures on the Trinity River main stem. The proposed levee crest is shown as effectively matching the 277,000 cfs (SPF) water surface profile where the existing levee is lower than the 277k water surface profile. As described above, the FRM Plan does not include floodplain modifications to ensure that the project meets the requirements of the 1988 ROD H&H criteria. However there are no water surface rises associated with the FRM Plan compared to the Future Without-Project Conditions but a valley storage loss would be expected for both flood events.

The water surface profiles for the Dallas Floodway FRM Plan compared to the Future Without-Project Conditions are shown on Figures 6-18 through 6-23 for the Trinity River main stem and the Elm and West Forks. The profiles are shown for the 100-year (1% AEP) flood event and the SPF (0.04% AEP) flood event. All of these profiles are presented for computations performed using the future watershed conditions peak flow hydrologic analysis. The profiles show the relationship between the SPF water surface and the proposed levee crest height for both the East and West Levees.

The water surface elevations computed within the Dallas Floodway for the 100-year and the SPF flood events for future watershed conditions for both the FRM Plan and the Future Without-Project Conditions are provided in Table 6-3. In this comparison table, there are no water surface rises for the FRM Plan compared to the Future Without-Project Conditions. Again, this is primarily due to the significant lowering of the water surface elevations with implementation of the AT&SF Bridge Modification included in the FRM Plan.

Table 6-3 Flood Risk Management Plan versus Future Without-Project Water Surface Elevations

		1% AEP Flood Event W.S. Elev.			**SPF Flood Event W.S. Elev.**		
Location	*River Station*	*Future Without -Project*	*FRM Plan*	*Difference*	*Future Without -Project*	*FRM Plan*	*Difference*
Trinity River Main Stem							
DS of AT&SF RR	107551	413.09	413.09	0.00	423.69	423.69	0.00
US of DART Rail	109035	414.17	413.65	-0.52	425.42	424.54	-0.88
Corinth St.	110009	414.22	413.71	-0.51	425.46	424.47	-0.99
IH 35E	114663	415.04	414.59	-0.45	426.53	425.65	-0.88
Houston St.	116243	415.43	415.01	-0.42	427.24	426.40	-0.84
IH 30	118966	416.39	416.04	-0.35	428.43	427.70	-0.73
Commerce St.	120765	416.86	416.53	-0.33	428.99	428.30	-0.69
UPRR	121639	417.32	417.01	-0.31	429.56	428.98	-0.58
MHH Bridge	122562	417.56	417.27	-0.29	429.83	429.27	-0.56
Sylvan Ave.	128158	418.94	418.70	-0.24	431.48	431.00	-0.48
Hampton Rd.	134883	420.31	420.13	-0.18	432.87	432.46	-0.41
Westmoreland	140734	421.55	421.40	-0.15	433.98	433.61	-0.37
Confluence	148136	423.27	423.16	-0.11	435.40	435.09	-0.31

		1% AEP Flood Event W.S. Elev.			SPF Flood Event W.S. Elev.		
Location	*River Station*	*Future Without -Project*	*FRM Plan*	*Difference*	*Future Without -Project*	*FRM Plan*	*Difference*
Elm Fork Trinity River							
Shady Grove Rd.	3190	423.87	423.78	-0.09	435.98	435.69	-0.29
SH 356	4826	423.93	423.84	-0.09	436.03	435.74	-0.29
BNSF	6689	424.01	423.91	-0.10	436.12	435.83	-0.29
SH 183	14544	424.78	424.70	-0.08	437.09	436.83	-0.26
US East Levee	18521	425.11	425.03	-0.08	437.47	437.22	-0.25
SH 482	22546	425.54	425.47	-0.07	437.94	437.71	-0.23
Loop 12	29438	426.79	426.74	-0.05	438.56	438.35	-0.21
West Fork Trinity River							
Loop 12	9763	426.63	426.59	-0.04	438.16	437.95	-0.21
US West Levee	12811	427.37	427.34	-0.03	438.68	438.49	-0.19
MacArthur Blvd	28841	436.10	436.10	0.00	444.13	443.94	-0.19

6.6 The With-Project "BVP without Trinity Parkway" Model

The With-Project "BVP without Trinity Parkway" model includes all of the projects from the FRM model with the addition of those projects that were included in the 2007 WRDA authorization. This includes, most notably, the BVP from the City of Dallas and the Interior Drainage IDP components that are located on the East Levee. The modeling effort for the BVP plus IDP does not include additional modeling for project phasing options. Project phasing will be examined in future efforts when specific construction phases for the BVP are proposed. The With-Project model for the BVP without Trinity Parkway was computed with the future watershed conditions hydrology for direct comparison to the Future Without-Project Conditions.

6.6.1 Description of the Project Features

The specific project features evaluated under the Comprehensive Analysis include all reasonably foreseeable projects in the study area. These proposed projects include proposed plans by the City of Dallas and others that are located within the existing Dallas Floodway study area. These plans include the BVP, the IDP, various Local Features and Section 408 projects, including the Trinity Parkway Riverside Alternative 3C.

The BVP includes the complete City of Dallas plan for lakes, river relocation, recreation features, and riverside levee slope modification to 4:1 consistent with the 277k levee raise, and it encompasses most of the Dallas Floodway corridor. The IDP includes pumping plant modifications and some of these have riverside levee modifications to the floodway. Local Features include a number of local projects proposed by various entities that were not part of the federal project authorization for WRDA 2007. These projects are subject to the Section 408 approval process.

The Trinity Parkway is a project proposed by the North Texas Tollway Authority and FHWA. This study only includes analysis for the proposed Trinity Parkway Alternative 3C included in the FHWA EIS for the Trinity Parkway. It encompasses the construction of 9-miles of new tollway stretching from the IH-35E/SH-183 interchange in the northwest to the US-75/SH-310 interchange in the southeast. A major portion of the proposed six-lane tollway is located within the floodway, directly adjacent to the riverside of the Dallas Floodway East Levee. The Trinity Parkway is considered a local feature for this analysis but these project features are not included in the With-Project BVP without Trinity Parkway model.

Because of the potential floodplain impacts of the Trinity Parkway, two different overall plans were analyzed for the Comprehensive Analysis. The first plan is referred to as the "BVP without Trinity Parkway", and it includes the FRM, BVP, IDP, and all the local features and Section 408 projects, except for the Trinity Parkway. The second plan is called BVP with Trinity Parkway, and it includes all of the same project features as in the first plan, but includes slight modifications to the BVP to accommodate the space required for the Trinity Parkway project features. This plan is discussed in Section 6.7.

6.6.2 Projects Included in the With-Project "BVP without Trinity Parkway" Model

The following projects are included in the With-Project model for BVP without Trinity Parkway. This list is in addition to all of the projects that were carried over from the FRM Plan model.

Able Pump Station – This project includes proposed improvements to Dallas' Sump A and Pump Station Able, located on the East Levee of the Trinity River, south of the Dallas Central Business District. This pump station is part of the City's IDP, and the east levee IDP features were included in the 2007 WRDA project authorization. This project's proposed design results in a floodplain encroachment and required a modification for the BVP without Trinity Parkway model geometry at cross section numbers 115764, 115937, and 116111.

Balanced Vision Plan – The BVP by the City of Dallas is a comprehensive plan for the development of the Dallas Floodway corridor for the purposes of recreation and environmental quality. The BVP includes the creation of new lakes, the relocation of the main river channel, and the addition of various recreational features within the floodplain. The project is linear in nature and affects approximately 7.5 miles of the Upper Trinity River Main Stem in the City of Dallas. Figure 6-1 shows the location of the proposed BVP project.

Baker Pumping Plant **-** The City of Dallas and USACE are planning to improve the Baker Pumping Plant in order to reduce the potential storm water flooding impacts to people and property in the City of Dallas and extend the service life of existing facilities for at least another 50 years. Currently, the Baker Pumping Plant includes two pump stations that drain the associated sump. Improvements would include constructing a new pump station (which would work along with the 1975 Baker Pump Station), rehabilitating the 1975 Baker Pump Station to modernize the electrical system of the building, extending the Baker Pumping Plant outfall 300 feet into the Dallas Floodway, and taking off line or decommissioning the Old Baker Pumping Plant. The project area would be approximately 4.54 acres, and construction was estimated to begin fall 2011. This pump station is part of the City's IDP, and it is also included in the 2007 WRDA project authorization. This project does not result in any modifications to the floodway. Therefore, no modifications to the BVP without Trinity Parkway model were required.

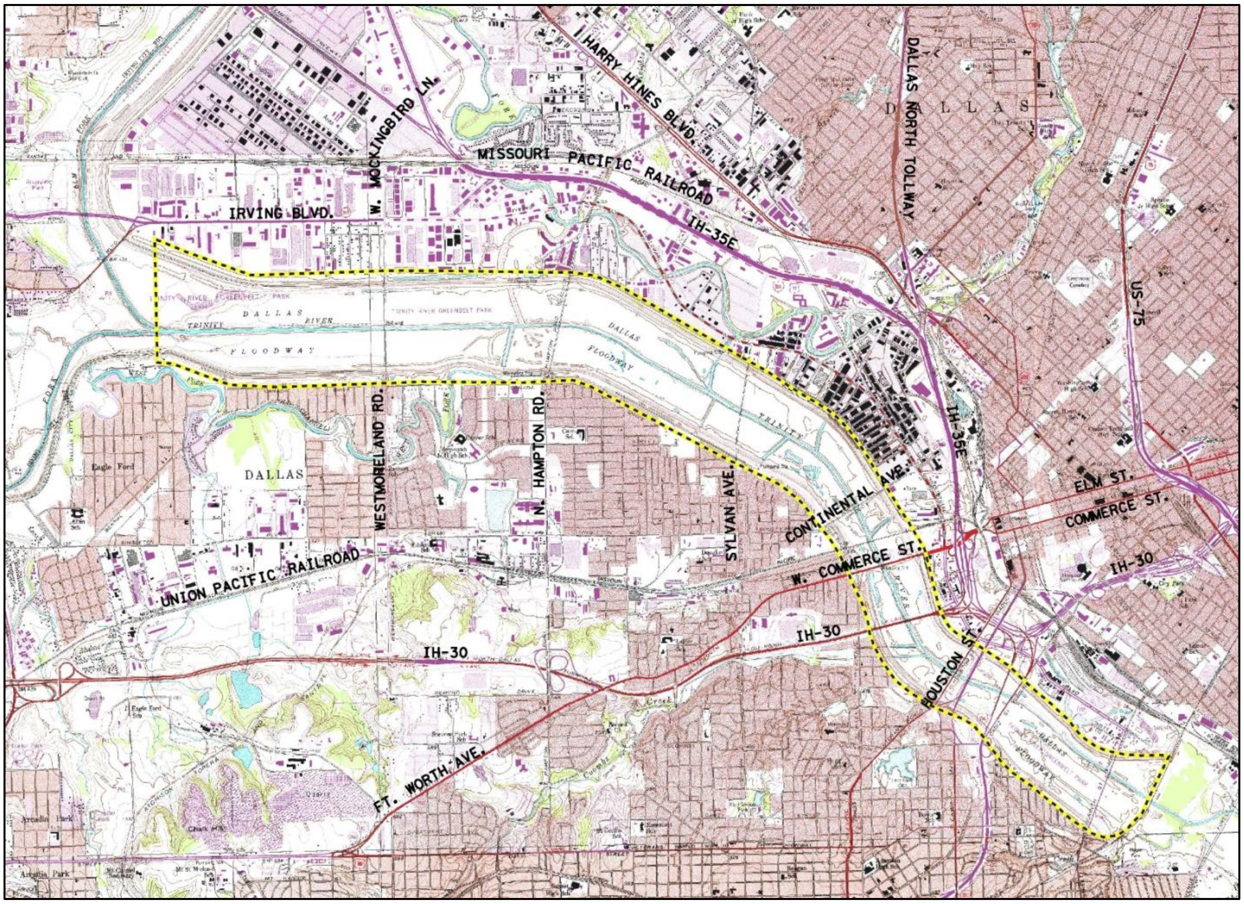


Figure 6-1 Location Map of the Balanced Vision Plan

Hampton 3 Pump Station - This project includes proposed improvements to Dallas' Pump Station Hampton 3, located on the East Levee of the Trinity River. This pump station is part of the City's IDP, and the east levee IDP features were included in the 2007 WRDA project authorization. This project's proposed design results in a floodplain encroachment and required a modification for the BVP without Trinity Parkway model geometry at cross section numbers 136515, 136721, and 136927.

Riverside Levee Side Slope Modifications – The City of Dallas intends to flatten the riverside levee side slopes on the approximately 3:1 existing side slopes to 4:1 for several intended purposes. The purposes include: (1) Improve the efficiency and safety for levee mowing operations, (2) Reduce the long term maintenance cost associated with repairing skin slides by reducing the frequency and severity of these slides that have occurred in the past, and (3) Provide for easier and safer pedestrian access on the levee slopes when the floodway is used for recreation purposes. It should be noted that a major portion of the Dallas Floodway levees currently have riverside levee slopes that are at or near a 4H:1V slope. The reason is that the City of Dallas completed a river channel and levee modification in the 1990s for a portion of the levee system to include a change to a 4H:1V riverside slope. This levee side slope modification project included both the East and West Levees extending from near the AT&SF Railroad Bridge at the downstream end of the floodway to near the Continental Street Bridge. Therefore, it is expected that this portion of the levees would require only minimal side slope revisions in some areas of this reach. The proposed side slope modifications will be done in a manner that is consistent with the 277k levee height modification that is proposed in the FRM plan. This modification required revisions to all of the cross sections in the HEC-RAS BVP without Trinity Parkway model that did not already have a 4:1 riverside levee slope.

6.6.3 HEC-RAS Modeling Methodology for the BVP without Trinity Parkway

Due to the extensive scope of the BVP, a preliminary HEC-RAS model was developed for the BVP without Trinity Parkway by the City of Dallas' A/E contractor (URS Engineering). The development of the Dallas Floodway Feasibility Study Comprehensive Analysis model used the A/E provided model as the base geometry, and then added the other project features to create the BVP without Trinity Parkway model. All of the previous projects listed above for the Existing Conditions Model, the Future Without-Project Model, and the FRM Plan Model were added to create the BVP without Trinity Parkway model. Additionally, the BVP without Trinity Parkway cross sections were modified to include the riverside 4:1 levee side slope modifications, the Hampton 3 Pump Station, and the Able Pump Station as described above. For the East Levee IDP features, HEC-RAS model data was not available, so the cross section modifications for the IDP were derived from the design drawings for each pump station having riverside levee encroachments.

HEC-RAS roughness coefficients (Manning's n-values)for the BVP without Trinity Parkway model were selected by the City's A/E contractor based on engineering judgment, standard references (Chow's 1959 Open Channel Hydraulics), USACE recommendations and a July 21, 2003 Carter & Burgess "SARIP – Assigning Manning's "n" Values for Vegetation Associations" memo. Various zones were defined within the proposed project and each was assigned a range of N-values, as shown in Table 6-4. A proposed lake roughness coefficient of 0.025 was selected at the recommendation of the USACE in order to be consistent with coefficients used downstream of the Trinity Lakes project area for other USACE projects such as the Chain of Wetlands. A channel n-value of 0.035 was used for the Trinity River Main Stem revised reach (Dallas, Aug 2010).

Table 6-4 Manning's N-values used in the BVP without Trinity Parkway Hydraulic Model

Zone	*Land Use*	*N-values*
A	Maintained lawn, short grass, turf	0.035
B	Existing levee sides, grasses, and unrevised overbank areas	0.055
C	Tree masses with woody understory	0.15
D	Tree masses with tall grass understory, and Pecan Grove	0.075
E	River Terrace planting	0.15
F	Lake Terrace planting, tall grasses, marshes	0.055
G1	Wetlands and meadows with low tree density	0.055
G2	Wetlands and meadows with medium tree density	0.065
H	Cypress wetlands, density varies	0.075 - 0.15
I	Concrete parking lots with trees 40' apart on average	0.045
J	Overflow parking lots, short grass and trees 60' apart on average	0.055
K	Concrete	0.02
L	Meadow areas, tall grasses	0.05
	Oxbow Lake	0.035
	Proposed Natural Lake	0.025
	Proposed Urban Lake	0.025
	Proposed West Dallas Lake	0.025
	Existing Trammel Crow Lake	0.035

6.6.4 Modeling Variables related to the Risk Register

There are a number of variables related to the preliminary design of the BVP and the other smaller projects included in the hydraulic analysis for the Dallas Floodway Feasibility Study Comprehensive Analysis that represent a potential risk to the overall impacts to the floodway. These risks are discussed qualitatively with emphasis on their relative impacts to this evaluation of hydraulic neutrality although there may be other risks related to constructability, costs, etc.

First, as discussed above, the hydraulic modeling using HEC-RAS requires estimates for roughness coefficients for the various land uses proposed in the floodway. It can be seen that lower values are associated with smoother surfaces such as concrete paving and higher values are generally associated with rougher surfaces such as vegetated surfaces. The values chosen for the modeling are generally regarded as best estimates based on engineering judgment, the afore-mentioned industry accepted roughness coefficient guides, and potential values obtained through model calibration with known water surface measurements and flow values. Roughness values used for the hydraulic analysis have a potential range of values that may apply to a particular land use type, but this potential range of values is rarely well known. Generally the values chosen for design are estimated conservatively with use of values that are higher in the estimated potential range of values. Therefore, there is a risk that the appropriate roughness values may be higher or lower than those selected for use, which may affect the final results.

Another variable in the design of the BVP is the design for the relocated river channel. The preliminary design for the relocated river channel has a wider range of channel dimensions with variable side slopes and bottom widths compared to the existing river channel. This may represent a risk to the hydraulic analysis due to the fact that the river channel flow generally has a significant impact on the overall flow capacity in the floodplain. There may also be other risks associated with regard to erosion and deposition processes which may require additional costs to alter these processes after construction.

Because of the preliminary nature of the BVP design, some assumptions for the relocated river channel with regard to existing bridges have been made. The proposed alignment of the relocated river channel results in the river channel crossing the existing bridge at a location where, in most cases, the bridge pier bents are more closely spaced and/or have not been designed for excavation around the lower portions of the piers. This results in a potential for either relocation of pier bents or reinforcement of existing piers. In most cases, the modeling of these bridge pier impacts has been assumed that the existing pier bents are to remain in the current locations but are strengthened by a concrete collar constructed around the rows of piers. In most cases, the assumption for this strengthening has substantially widened the lower portions of these piers in the proposed river channel in the hydraulic modeling process. This results in a risk that when the final design for the modifications of these bridge piers is complete, the hydraulic impacts for these bridge piers within the channel may be different.

6.6.5 Fluvial Geomorphology Assessment

The channel realignment portion of the BVP will result in arguably the most significant change to the Trinity River channel in many decades. The existing channel appears to have remained relatively stable since the USACE reconstruction of the channel in the 1950s. The BVP for the Trinity River Corridor proposes physical changes to the channel and floodway including restoration of channel meanders, creation of a mid-channel island, alterations to channel geometry, and construction of three lakes in the floodway adjacent to the channel. To support the design of the channel realignment, the City of Dallas hired a contractor to perform a geomorphic assessment of the proposed project. The purpose of that assessment was to characterize and quantify historical channel and floodway change and to document design decisions and supporting analyses that led to the final design of the channel realignment (CH2MHill 2008).

USACE reviewed the findings of the City's geomorphology report in order to assure that the realigned channel will remain stable through the floodway and especially that potential channel geomorphology will not pose a threat to levee stability. The review was performed by a nationally recognized expert in the field of geomorphology, Mr. Michael Spoor of the USACE, Huntington District. In his review, Mr. Spoor concluded that the proposed project should function as designed with no adverse impacts to flood conveyance or levee stability. Mr. Spoor also provided a list of comments regarding additional items to consider in the final design of BVP features. The comments from his geomorphology review will be provided to the designers for consideration during future design phases.

6.6.6 Results from the "BVP Without Trinity Parkway" HEC-RAS Model

The following hydraulic modeling results of the Dallas Floodway Feasibility Study BVP without Trinity Parkway HEC-RAS hydraulic model includes a comparison of that model's results to the Future Without-Project model results. The water surface profiles for both the 100-year and SPF flood events are shown with reference to the improved levee crest height that is consistent with the FRM Plan. For clarity, the East levee and the West Levee crest profiles are shown on separate figures for the Trinity River main stem. The BVP without Trinity Parkway project has been modeled with the proposed features and projects as described above and does not include further design of floodplain modifications to ensure that the project meets the requirements of the 1988 ROD H&H criteria.

The water surface profiles for the Dallas Floodway BVP without Trinity Parkway compared to the Future Without-Project Conditions are shown on Figures 6-24 through 6-29 for the Trinity River main stem and the Elm and West Forks. The profiles are shown for the 100-year (1% ACE) flood event and the SPF (0.04% ACE) flood event. All of these profiles are presented for computations performed using the future watershed conditions peak flow hydrologic analysis. The profiles show the relationship between the SPF water surface and the proposed levee crest height for both the East and West Levees.

The water surface elevations computed within the Dallas Floodway for the 100-year and the SPF flood events for future watershed conditions for both the BVP without Trinity Parkway and the Future Without-Project Conditions are provided in Table 6-5. In this comparison table, the positive values shown in red indicate a water surface rise for the BVP without Trinity Parkway compared to the Future Without-Project Conditions.

Table 6-5 BVP without Trinity Parkway versus Future Without-Project Water Surface Elevations

		1% AEP Flood Event W.S. Elev.			SPF Flood Event W.S. Elev.		
Location	*River Station*	*Future Without -Project*	*BVP w/o Trinity Parkway*	*Difference*	*Future Without -Project*	*BVP w/o Trinity Parkway*	*Difference*
Trinity River Main Stem							
DS of AT&SF RR	107551	413.09	413.09	0.00	423.69	423.69	-0.00
US of DART Rail	109035	414.17	413.63	-0.54	425.42	424.51	-0.91
Corinth St.	110009	414.22	413.72	-0.50	425.46	424.62	-0.84
IH 35E	114663	415.04	414.79	-0.25	426.53	425.94	-0.59
Houston St.	116243	415.43	415.15	-0.28	427.24	426.54	-0.70
1% ACE max rise	117672	415.91	416.18	0.27	427.85	427.44	-0.41
IH 30	118966	416.39	416.35	-0.04	428.43	427.70	-0.73
Commerce St.	120765	416.86	416.60	-0.26	428.99	428.06	-0.93
UPRR	121639	417.32	416.76	-0.56	429.56	428.35	-1.21
MHH Bridge	122562	417.56	416.89	-0.67	429.83	428.54	-1.29
Sylvan Ave.	128158	418.94	418.13	-0.81	431.48	430.11	-1.37
Hampton Rd.	134883	420.31	419.83	-0.48	432.87	431.88	-0.99
Westmoreland	140734	421.55	420.81	-0.74	433.98	432.89	-1.09
Confluence	148136	423.27	423.05	-0.22	435.40	434.70	-0.70
Elm Fork Trinity River							
Shady Grove Rd.	3190	423.87	423.70	-0.17	435.98	435.32	-0.66
SH 356	4826	423.93	423.77	-0.16	436.03	435.38	-0.65
BNSF	6689	424.01	423.85	-0.16	436.12	435.48	-0.64
SH 183	14544	424.78	424.66	-0.12	437.09	436.54	-0.55
US East Levee	18521	425.11	424.99	-0.12	437.47	436.94	-0.53
SH 482	22546	425.54	425.44	-0.10	437.94	437.44	-0.50
Loop 12	29438	426.79	426.72	-0.07	438.56	438.10	-0.46
West Fork Trinity River							
Loop 12	9763	426.63	426.64	0.01	438.16	437.77	-0.39
US West Levee	12811	427.37	427.38	0.01	438.68	438.33	-0.35
MacArthur Blvd	28841	436.10	436.10	0.00	444.13	444.09	-0.04

The BVP without Trinity Parkway plan has been modeled to evaluate the floodplain impacts and determine the hydraulic neutrality with reference to the 1988 ROD H&H criteria. The criteria are primarily evaluated on four points. These four points are: water surface rise due to the project for the 1% AEP and SPF flood events and valley storage loss for the 1% AEP and SPF flood events. As shown on the water surface elevation tables there are no water surface rises due to the project for the SPF flood event but there is a short reach where a water surface rise occurs for the 1% ACE flood event. This rise extends from river station 117403 to river station 118500, just downstream of the I-30 Bridge. The maximum rise for the 1% AEP event is at river station 117672 and is 0.27 feet. This rise occurs within the floodway on the Trinity River Main stem where both levees provide protection from flooding risk for the 1% AEP flood event to the City of Dallas. No rises are indicated upstream of the Elm Fork and West Fork confluence. The 0.01 rise noted in Table 6-5 on the West Fork is considered negligible from a computational standpoint. This analysis indicates that since no water surface rise occurs for the SPF flood event and the 1% ACE flood event for areas upstream of the project, there would be no increase in flood risk for these areas for either flood event. However, since water surface rises occur for the 1% ACE flood event, this plan as currently designed fails to meet the requirements of the ROD criteria.

The valley storage change for the BVP without Trinity Parkway is estimated at -0.83% for the 1% ACE and -5.1 % for the SPF compared to the Future Without-Project Conditions. This means that the project results in a valley storage loss for both flood events. The project as currently designed does not meet the ROD criteria for the 1% ACE flood event or the SPF event because no valley storage loss is allowed for the 1% ACE and no loss greater than 5% is allowed for the SPF.

As shown in Table 6-5, the computed water surface elevations for the BVP without Trinity Parkway are significantly lower than the Future Without-Project Conditions at most locations within the floodway. The extensive reach where water surface elevations have been lowered results in the estimated valley storage loss associated with this plan. It should be noted that while the 4:1 riverside levee side slope modifications were not evaluated as a stand-alone alternative, previous hydraulic analyses showed that the change from 3:1 to 4:1 alone has a very minor impact on computed water surface profiles within the floodway. This is due partly to the fact that a major portion of the Dallas Floodway already has 4:1 riverside levee slopes as described in item 5 of Section 6.6.1 above, but also is due to the change occurring at the outer limits of the floodplain where flow velocity is generally lowest. It would be expected that riverside levee flattening would create a loss of conveyance and result in a potential water surface rise. However, all of the borrow material for the levee side slope modification is designed to be obtained from the floodway and when the levee flattening is combined with the BVP in this alternative, any small hydraulic impacts of the levee side slopes are overcome by the impacts of the proposed lakes and river realignments. Even if considering that overall flow area within the floodway for with-project conditions were the same, given that borrow material for the levee side slope modifications and other changes are obtained from the floodway, the hydraulic efficiency of the overall floodway is greater due to the larger modified channel and extensive areas of open water created by the proposed lakes. The primary reason for the observed lowering of the water surface profiles with the BVP versus the Future Without-Project is the lower hydraulic roughness of the lakes resulting in the overall lowering of the hydraulic roughness of the floodway.

6.7 THE WITH-PROJECT "BVP WITH TRINITY PARKWAY" MODEL

The "BVP with Trinity Parkway" model includes all of the projects from the "BVP without Trinity Parkway" model with the addition of the Trinity Parkway project. This alternative allows evaluation comprehensively both with and without the Trinity Parkway project.

6.7.1 Projects Included in the "BVP with Trinity Parkway" Model

The following is a description of the Trinity Parkway project. All of the other projects that were carried over from the BVP without Trinity Parkway model are described in the preceding sections.

Trinity Parkway - The Trinity Parkway is a local project proposed by the North Texas Tollway Authority (NTTA). It encompasses the construction of 9-miles of new tollway stretching from the IH-35E/SH-183 interchange in the northwest to the US-75/SH-310 interchange in the southeast. The NTTA has evaluated several alternatives for this project. This BVP with Trinity Parkway analysis is for only one of these alternatives and is referred to as Alternative 3C by the NTTA. A major portion of the proposed six-lane tollway is located within the Dallas Floodway, adjacent to the East Levee.

6.7.2 HEC-RAS Modeling Methodology for the "BVP with Trinity Parkway" Model

Similarly to the development of the BVP without Trinity Parkway plan, a preliminary HEC-RAS model was developed for the BVP with Trinity Parkway Alternative by the City of Dallas' A/E contractor (URS Engineering). The development of the Comprehensive Analysis model used the A/E provided model as the base geometry, and then added the other project features to the model. Additionally, project modeling revisions were added to the BVP with Trinity Parkway model to update for recent changes to the Trinity Parkway design. These model updates were provided by the NTTA A/E contractor (Halff Associates, Inc.). First, all of the previous projects included in the updated FRM model were added to the model. Then, the cross sections were modified to include the riverside 4:1 levee side slope modifications. Finally, the IDP features were added. For the East Levee IDP features, HEC-RAS model data was not available, so the cross section modifications for the IDP were derived from the design drawings for each pump station having riverside levee encroachments.

The Manning's roughness values used to represent with-project conditions for the BVP with Trinity Parkway hydraulic model are essentially the same as those shown in Section 6.6.3 for the BVP without Trinity Parkway model with the exception of roughness values used to model the specific Parkway areas. The roughness values used for the Parkway are dominated by paved surfaces which have used a Manning's roughness coefficient of 0.02. Other areas of the Trinity Parkway use values ranging up to 0.07 depending on the type of vegetation selected.

6.7.3 Results from the "BVP with Trinity Parkway" HEC-RAS Model

The following hydraulic modeling results of the Dallas Floodway Feasibility Study BVP with Trinity Parkway HEC-RAS hydraulic model includes a comparison of that model's results to the Future Without-Project model results. The water surface profiles for both the 100-year and SPF flood events are shown with reference to the improved levee crest height consistent with the FRM Plan. For clarity, the East levee and the West Levee crest profiles are shown on separate figures on the Trinity River main stem. As described above, the BVP with Trinity Parkway does not include floodplain modifications by the USACE to ensure that the project meets the requirements of the 1988 ROD H&H criteria.

The water surface profiles for the Dallas Floodway BVP with Trinity Parkway compared to the Future Without-Project Conditions are shown on Figures 6-30 through 6-35 for the Trinity River main stem and the Elm and West Forks. The profiles are shown for the 100-year (1% ACE) flood event and the SPF (0.04% ACE) flood event. All of these profiles are presented for computations performed using the future watershed conditions peak flow hydrologic analysis. The profiles show the relationship between the SPF water surface and the proposed levee crest height for both the East and West Levees.

The water surface elevations computed within the Dallas Floodway for the 100-year and the SPF flood events for future watershed conditions for both the BVP with Trinity Parkway and the Future Without-Project Conditions are provided in Table 6-6. In this comparison table, the positive values shown in red indicate a water surface rise for the BVP with Trinity Parkway compared to the Future Without-Project Conditions.

Table 6-6 BVP with Trinity Parkway versus Future Without-Project Water Surface Elevations

		1% AEP Flood Event W.S. Elev.			**SPF Flood Event W.S. Elev.**		
Location	*River Station*	*Future Without -Project*	*BVP with Trinity Parkway*	*Difference*	*Future Without -Project*	*BVP with Trinity Parkway*	*Difference*
Trinity River Main Stem							
DS of AT&SF RR	107551	413.09	413.09	0.00	423.69	423.69	0.00
US of DART Rail	109035	414.17	413.81	-0.36	425.42	424.57	-0.85
Corinth St.	110009	414.22	413.91	-0.31	425.46	424.77	-0.69
	113902	414.80	414.85	0.05	426.20	425.87	-0.33
IH 35E	114663	415.04	414.99	-0.05	426.53	426.08	-0.44
Houston St.	116243	415.43	415.40	-0.03	427.24	426.67	-0.57
1% ACE max rise	117672	415.91	416.47	0.56	427.85	427.30	-0.55
IH 30	118966	416.39	416.63	0.24	428.43	427.52	-0.91
Commerce St.	120765	416.86	416.88	0.02	428.99	427.87	-1.12
UPRR	121639	417.32	417.02	-0.30	429.56	428.14	-1.42
MHH Bridge	122562	417.56	417.17	-0.39	429.83	428.31	-1.52
Sylvan Ave.	128158	418.94	418.49	-0.45	431.48	430.21	-1.27
Hampton Rd.	134883	420.31	420.25	-0.06	432.87	432.19	-0.68
	136721	420.67	420.78	0.11	433.17	432.73	-0.44
Westmoreland	140734	421.55	421.12	-0.43	433.98	433.16	-0.82
Confluence	148136	423.27	423.06	-0.21	435.40	434.79	-0.61
Elm Fork Trinity River							
Shady Grove Rd.	3190	423.87	423.71	-0.16	435.98	435.40	-0.58
SH 356	4826	423.93	423.78	-0.15	436.03	435.46	-0.57
BNSF	6689	424.01	423.86	-0.15	436.12	435.56	-0.56
SH 183	14544	424.78	424.66	-0.12	437.09	436.61	-0.48
US East Levee	18521	425.11	425.00	-0.11	437.47	437.01	-0.46
SH 482	22546	425.54	425.44	-0.10	437.94	437.51	-0.43
Loop 12	29438	426.79	426.72	-0.07	438.56	438.16	-0.40
West Fork Trinity River							
Loop 12	9763	426.63	426.64	0.01	438.16	437.82	-0.34
US West Levee	12811	427.37	427.38	0.01	438.68	438.37	-0.31
MacArthur Blvd	28841	436.10	436.10	0.00	444.13	444.10	-0.03

The BVP with Trinity Parkway plan has been modeled to evaluate the floodplain impacts and determine the hydraulic neutrality with reference to the 1988 ROD H&H criteria. The criteria are primarily evaluated on four points. These four points are: water surface rise due to the project for the 1% ACE and SPF flood events and valley storage loss for the 1% ACE and SPF flood events. As shown on the water surface elevation tables there are no water surface rises due to the project for the SPF flood event but there are some localized areas where a water surface rise occurs for the 1% ACE flood event. First, at river station 113563 and 113902, a rise of 0.04 foot and 0.05 foot, respectively, are indicated. Secondly, a rise extends from river station 117294 to River station 120765. The maximum rise for the 1% ACE event is at river station 117672 and is 0.56 foot. Thirdly, a rise occurs from river station 136310 to river station 137598. In this reach, the maximum rise for the 1% ACE event is at river station 136721 and is 0.11 foot. These rises occur within the floodway on the Trinity River main stem where both levees provide protection from flooding risk for the 1% ACE flood event to the City of Dallas. The 0.01-foot rise noted in Table 6-7 on the West Fork is considered negligible from a computational standpoint. This analysis indicates that since no water surface rise occurs for the SPF flood event and the 1% ACE flood event for areas upstream of the project, there would be no increase in flood risk for these areas for either flood event. This analysis indicates that since water surface rises occur for the 1% ACE flood event, this plan fails to meet the requirements of the ROD criteria.

The valley storage change for the BVP with Trinity Parkway has been computed at -5.1% for the SPF and approximately -2.7% for the 1% ACE compared to the Future Without Project Conditions. This means that the project results in a valley storage loss for both flood events. The project as currently designed does not meet the ROD criteria for the 1% ACE flood event or the SPF event. Please see the discussion on the unsteady flow hydraulic model results for the BVP with Trinity Parkway in Section 6.8 for an estimate of the approximate downstream impacts resulting from a valley storage loss of this magnitude.

6.8 Comparison of Plans in Unsteady HEC-RAS

Most of the hydraulic analysis for the Comprehensive Analysis phase was accomplished through steady flow HEC-RAS for riverine flow that is confined to the floodway without overtopping, which is the type of analysis that was originally intended in the ROD criteria evaluation process. The ROD criteria is written in terms of change in water surface or valley storage at the peak of the flood event; therefore, steady flow analysis is normally all that is needed to evaluate performance of a project against the ROD criteria. However, both Steady Flow and Unsteady Flow methods were employed within the Comprehensive Analysis phase for differing purposes. The Steady Flow methods provide a more consistent computation method for comparison of plans to determine if the ROD criteria are met and is considered a better method for computing the change in the water surface profiles. This is especially true when numerous bridges or other abrupt changes in the water surface are expected if floodplain and modeling conditions exist that conform to the modeling assumptions for one dimensional flow. The Steady Flow method is a robust and consistent method for determining water surface elevation change and valley storage change if these flow conditions exist. Since most of the plans evaluated did not meet the ROD criteria completely, primarily for valley storage loss, the Unsteady Flow method was used to provide a means of estimating the magnitude of the downstream impacts for valley storage loss. The valley storage impact analysis was deemed important for better informing decision makers on the potential for changes in flood risk impacts since the ROD criteria was not met for valley storage change.

One capability that unsteady flow analysis has, which steady flow does not, is modeling the attenuation in peak flow as a hydrograph moves through a system. This allows the modeler to quantify what the effects of a loss in valley storage would be in terms of peak discharge downstream. This unsteady flow analysis was only performed for Future Without-Project and the BVP with Trinity Parkway conditions. However, since the BVP with and without Trinity Parkway models had similar water surface profiles in steady flow, the results were expected to be similar in unsteady flow as well.

6.8.1 Hydrographs Used

Unsteady flow analysis was performed for the two events which are the basis of the ROD criteria, the 100-year and the SPF. The inflow hydrographs for the unsteady analysis were taken from the Upper Trinity River rainfall-runoff model for the future 100-year and future SPF events. Small multiplication factors were applied to the hourly flows of these hydrograph to scale the hydrograph up or down to the desired peak discharge at the Commerce Street gage. Figure 6-2 and Figure 6-3 illustrate the shape of the hydrographs as well as the proportionate contributions of the Elm Fork and the West Fork. These proportions were maintained in the scaled hydrographs. Table 6-7 contains the final two inflow hydrographs and their associated multiplication factors. In the HEC-RAS flow file, these inflow hydrographs were entered at the upstream ends of the Elm Fork and the West Fork reaches.

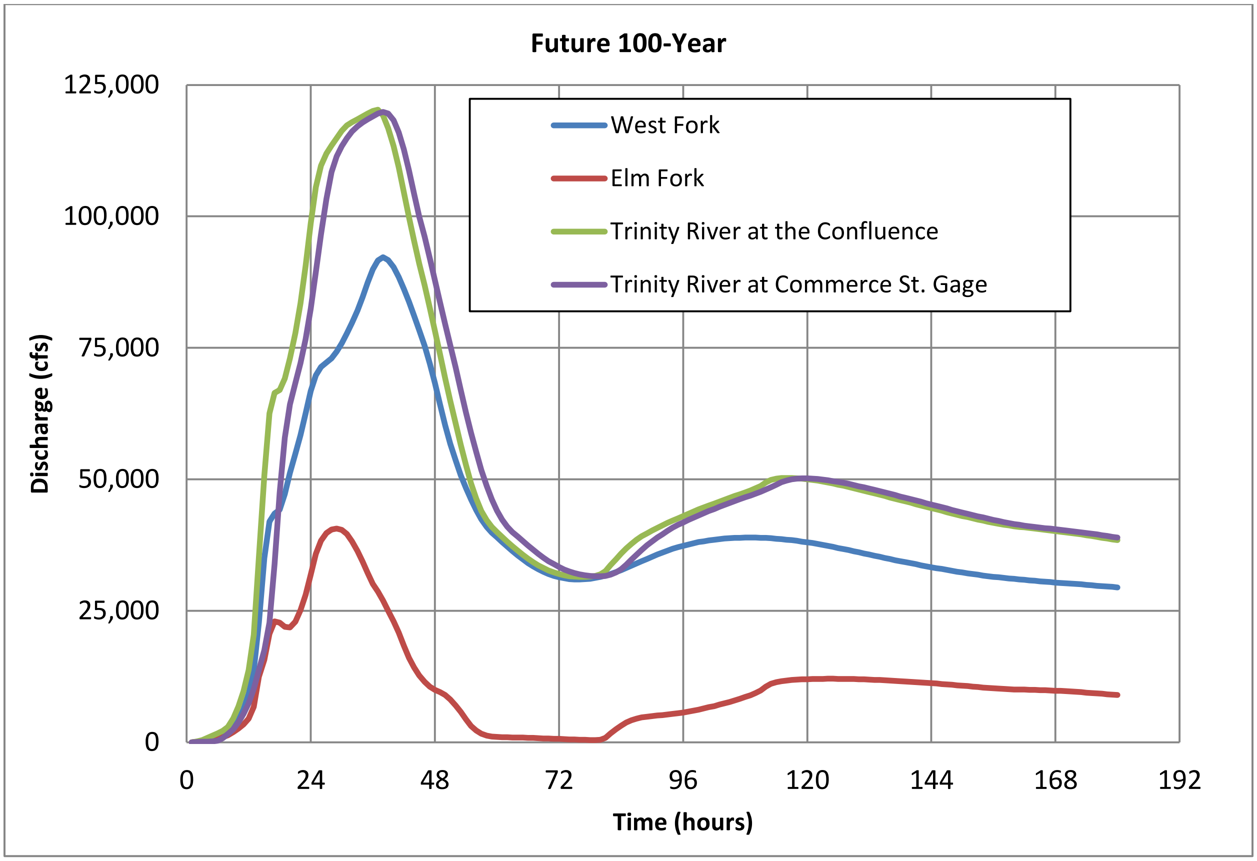


Figure 6-2 Future 100-year Hydrographs Used in the Unsteady Analysis

Table 6-7 Unsteady Hydrograph Peaks at Commerce

Event #	*Hydrograph Multiplier*	*Peak Q at Commerce (cfs)*	*Peak WS Elev at Commerce (feet)*	*Significance*
1	1.03998	120,100	416.6	100-year Future Peak Q
2	1.08072	277,000	428.6	SPF Future

Notes: Commerce location is the upstream side of Commerce Street Bridge (RS 120765).

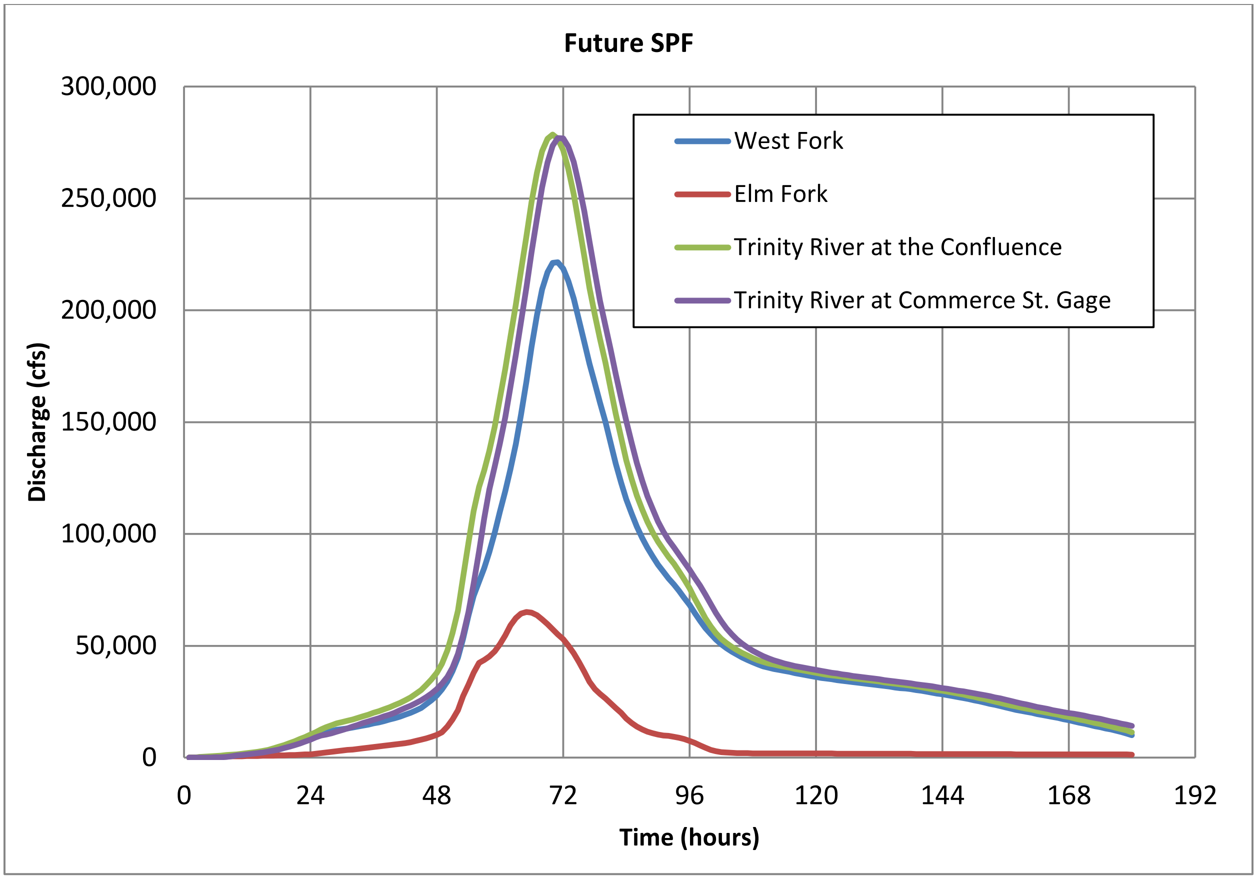


Figure 6-3 Future SPF Hydrographs Used in the Unsteady Analysis

6.8.2 Adjustments to the Model Geometries for Unsteady Flow

The Future Without-Project Conditions and BVP with Trinity Parkway steady flow models were used as the basis for the development of the unsteady flow models. There were several, relatively minor changes to the steady flow models that were required to convert the models to unsteady flow and to calibrate the models. Below is a summary of the changes that were made to the model geometries for both Future Without-Project and BVP with Trinity Parkway.

- Htab Parameters - Hydraulic table parameters were modified for all bridges and cross sections by increasing number of curves and number of points on those curves. A maximum discharge was also specified in the bridge tables.
- Unsteady flow expansion and contraction coefficients - Coefficients at various locations. Values of 0.1, 0.3 (Contraction, Expansion) were used for standard cross sections and values of 0.2, 0.3 were used at most bridges. Maximum values of 0.5, 0.8 were limited to the areas where severe contraction and expansion would take place, primarily around the abandoned AT&SF Bridge.
- Cross Section Additions - Addition of two cross sections to meet minimum requirement of two cross sections between bridges.
- Junction Method - The junction method was modified from "Force Equal WS Elevations" to an "Energy Balance Method," and reach lengths were added to the downstream cross sections of the Elm Fork and West Fork to allow HEC-RAS to calculate head loss through the junction.

- Bridge Method **-** Bridge low flow computation methods at two bridges were modified to improve model stability. The two methods were compared in steady flow and the change in water surface was not significant (0.02 foot Maximum).
- Low Water Crossing Removed **-** One low water crossing was removed to improve model stability. The effects of removing the crossing were tested in steady flow and did not change the water surface. This was likely due to the small size of the crossing and the depth of flooding over the crossing.
- Manning's Roughness Parameters **-** Modified parameters to better match the steady flow model results. Roughness values were increased up to a maximum of 0.005 on the Main Stem and 0.003 on the Elm Fork and West Fork.

Prior to calibration, the unsteady flow model produced a water surface that was lower than the steady flow model by more than 1 foot in a number of locations. After calibration, the unsteady water surface profiles generally differed from the steady flow profiles by less than 0.2 foot at every cross section from the confluence of the Elm Fork and the West Fork (River Station 148136) to the Commerce Street Gage (River Station 120729). There is about a half-mile stretch surrounding Sylvan Avenue (128092.5) where the difference approaches 0.3 foot. From the Commerce Street Gage to the downstream end of the Dallas Levees, the Trinity River profiles generally differed by less than 0.1 foot. There is about a 700 feet reach upstream of the Abandoned AT&SF Bridge (108287) where the difference approaches 0.3 foot. On the Elm Fork and West Forks, the unsteady and steady flow profiles generally differed by about 0.2 and 0.3 foot, respectively. Therefore, when comparing the unsteady water surface results against the ROD criteria, one should keep in mind that the margin of error in the unsteady computations is on the order of 0.2 foot.

6.8.3 Comparison of Results for Future Without-Project and the BVP with Trinity Parkway

Table 6-8 and Table 6-9 provide a summary of the results that were obtained from the unsteady flow analysis. The BVP with Trinity Parkway project changes generally lowered the water surface along the Main Stem while increasing the water surface at certain locations. There was an overall decrease in valley storage of less than 1%, and that change resulted in a small (less than 1%) increase in peak flow downstream of the Trinity Parkway. This increase in peak discharge resulted in an increase in water surface of about 0.1 foot in the reach downstream of the project. The increase in water surface was not fully attenuated at the downstream limit of the model. However, it should be noted that this unsteady model does not fully account for the available storage volumes downstream of the floodway. For example, the additional storage volume available in the area of White Rock Creek (82361) but not included in the model would be expected to increase the attenuation of the peak flows and decrease the computed change in peak flow. The unsteady peak flow increases were then entered into the steady flow model, and downstream water surface impacts were compared to the values shown in Table 6-8 and Table 6-9. The steady flow results were similar (0.05 foot higher for the 100YR and 0.15 foot higher for the SPF).

The Trinity Parkway Floodwall could potentially be overtopped for more than a day (26 hours) if an SPF event passed through the floodway while a 100-year event would not overtop the Trinity Parkway floodwall. This is only an indication of how long the river may be above the Trinity Parkway Floodwall and not necessarily the amount of time the Trinity Parkway could be inundated. Actual amount of time that the Trinity Parkway would be inundated during an SPF event would depend on the method used to drain the water from the Parkway.

Table 6-8 Summary Table from the 100-Year Unsteady Flow Analysis

Information	*Value*	*Station*
Max Change in Peak Discharge below Trinity Parkway (cfs)	600	103960
Max WS Elev Increase below Trinity Parkway (feet)	0.07	103533
Change in Valley Storage on Main Stem (acre-feet)	-239	NA
Change in Valley Storage on Main Stem (%)	-0.2	NA
Duration River is above Parkway Floodwall (hours)	0	117801

Table 6-9 Summary Table from the SPF Unsteady Flow Analysis

Information	*Value*	*Station*
Max Change in Peak Discharge below Trinity Parkway (cfs)	2,200	103960
Max WS Elev Increase below Trinity Parkway (feet)	0.12	103453
Change in Valley Storage on Main Stem Only (acre-feet)	-1,300	NA
Change in Valley Storage on Main Stem Only (%)	-0.5	NA
Duration River is above Parkway Floodwall (hours)	26	117801

6.8.4 Limitations of the Unsteady Flow Analysis

There are certain limitations that should be considered when interpreting the unsteady results. As was mentioned previously, calibration was performed in order to create similar peak water surface profiles between the unsteady Future Without-Project Conditions and the steady Future Without-Project Conditions. The same calibration was performed for the unsteady BVP with Trinity Parkway condition and the steady BVP with Trinity Parkway condition. The level of calibration generally had an error band up to 0.2 foot. For example, when calibrating the Future Without-Project model a difference of 0.0 foot may have been obtained at a given cross section while the BVP with Trinity Parkway model had a difference of -0.2 foot. Where the difference between the Future Without-Project model and the BVP model may be -1.0 foot in steady flow, the difference between the two conditions in unsteady flow may be -1.2 feet, due to differences in the calibration levels. This suggests that care should be given when using the unsteady results to compare water surface differences less than 0.2 foot against the ROD criteria of no rise, as well as evaluating against the "No Change" valley storage criteria for the 100-year. The calibration differences for the main stem from the confluence to the downstream end of the Trinity Parkway are illustrated in Figure 6-4 below. The calibration differences might be reduced with further calibration, but it is not likely to completely remove the differences to the hundredth of a foot (0.00 foot), as would be necessary to demonstrate zero rise in water surface. There are some locations where a cross section is computing a value too high but is adjacent to a cross section that is computing too low. Improving the situation at one cross section could worsen the situation at the adjacent cross section.

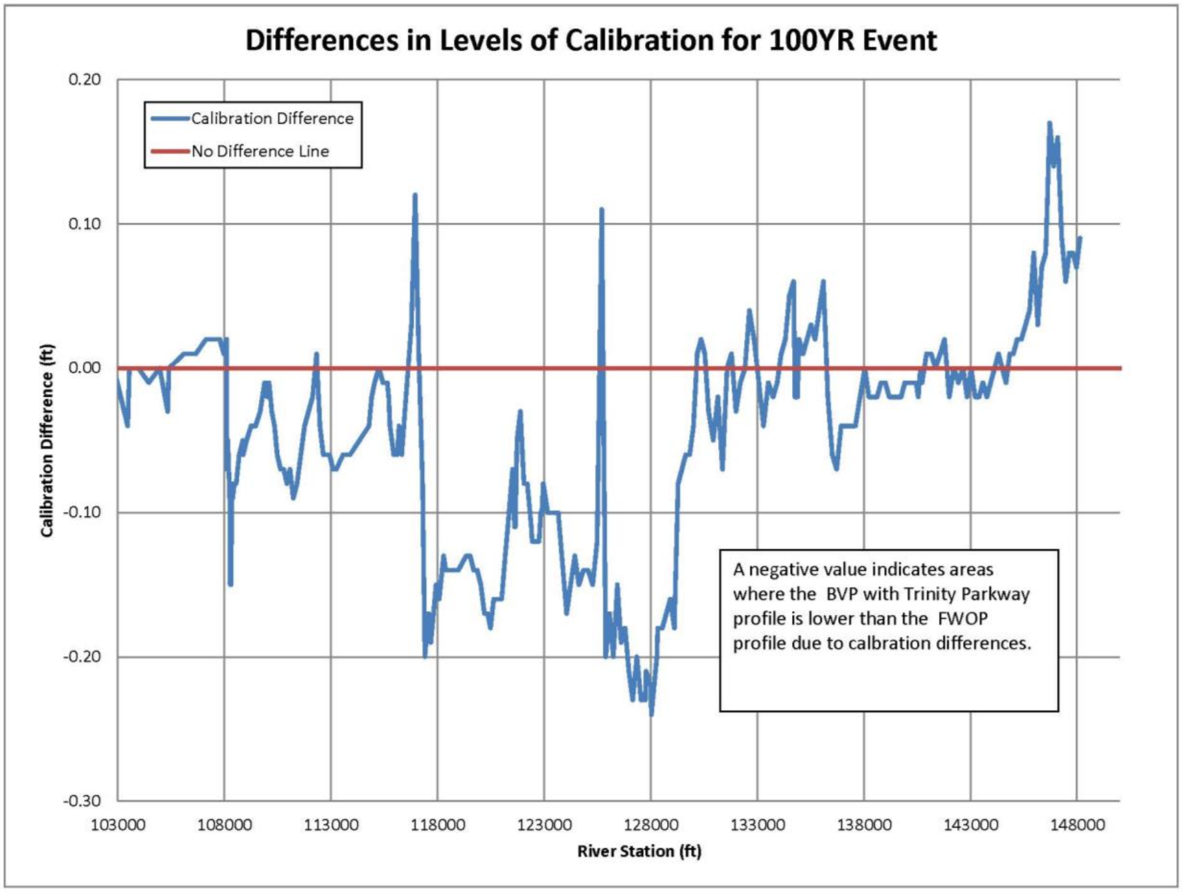


Figure 6-4 Differences in the Levels of Calibration

Therefore, it is not recommended that the unsteady results be used to judge whether a plan meets the ROD criteria, down to the hundredth of the foot in water surface, but the unsteady analysis is helpful in quantifying the effects of not meeting the criteria. For example, the steady flow results for the BVP with Trinity Parkway showed that the project did not meet the ROD criteria for the 100-year event (no rise and the 0% valley storage loss). However, the unsteady flow model is used to estimate the effects downstream of the project of the valley storage change. Again, as shown in Tables 6-9 and 6-10, the estimated increase in the peak water surface is 0.07 foot for the 1% ACE flood event and is 0.12 foot for the SPF (277,000 cfs) flow compared to the Future Without-Project Conditions.

6.9 CONCLUSIONS AND RECOMMENDATIONS FROM THE COMPREHENSIVE ANALYSIS

The 1988 Upper Trinity River ROD H&H Criteria has been used since inception to limit potential flood risk increases in the Upper Trinity River floodplain by limiting the hydraulic effects of floodplain developments to no rise in the peak flow water surface on neighboring or adjacent developments. Additionally, the valley storage loss criteria are designed to have the same limiting effects on valley storage losses, which may have the impact of increasing flood risk on downstream developments. It should be evident from the analysis presented in this report that for proposed projects on the scale of the BVP and the Dallas Floodway levees, meeting the criteria at every location along the entire reach of such a large project can be very challenging. The computed impacts for the preliminary design for the comprehensive plan as presented are not to be construed as absolute values but are intended to be indicators of potential impacts for plan comparison purposes. There remain many variables in both the final design of the project features and the assumptions used in the analysis, which may have some impact on the end results.

To summarize the H&H results for the Comprehensive Analysis, each of the four main H&H criteria points are presented for both the SPF flood event and the 1% ACE flood event. First, the ROD criteria for water surface rise for the SPF flood event is met at every location within the Dallas Floodway and upstream. However, the significant lowering of the water surface at some locations results in the computation of a total valley storage loss that exceeds the allowable 5% of onsite valley storage. The computed valley storage loss for the comprehensive plan only within the main stem Trinity River portion (downstream of the confluence) of the Dallas Floodway is approximately 3.7%. However, the change in the peak water surface elevation upstream of the Dallas Floodway extends far upstream before attenuating to a near zero change. This lowering of the water surface upstream of the project results in an additional valley storage loss that exceeds the total 5% valley storage loss limit for the SPF. Potentially, it would seem that if the SPF water surface could be raised upstream of the confluence, then the valley storage loss upstream could be reduced to meet the ROD criteria by making project design changes or to proposed vegetation density downstream of the confluence. However, raising the SPF water surface upstream of the confluence will result in increasing the 1% ACE flood event water surface upstream of the confluence as well. Currently the 1% ACE water surface upstream of the confluence is nearly ideal with a near zero change from Future Without-Project Conditions on the West Fork and up to about 0.15 foot lower than the Future Without-Project on the Elm Fork (see Table 6-7). It can be seen that attempting to reduce valley storage loss for one flood event may cause an undesirable water surface rise on the other. Attempting to raise the water surface profile for the SPF upstream of the confluence by means of project design changes would result in a tradeoff of achieving a benefit of reducing flood risk increases downstream while increasing the flood risk for upstream areas, which could be considered more significant in terms of water surface rise.

The Comprehensive Analysis indicates that the water surface elevation rise indicated for the 1% ACE flood event are limited to the areas of the Dallas Floodway on the main stem Trinity River. There is no water surface rise indicated for the SPF flood event. Therefore, there would be no increased risk of flooding due to levee overtopping impacts for this reach of the Trinity River since the East and West Levees are protecting both sides of the floodplain on the main stem of the Trinity River. There could be a very small increased risk to the levee for the 1% ACE flood event with consideration for levee through-seepage or under-seepage resulting in a levee piping failure from an increased hydraulic loading for this flood event. However, the levee system Risk Assessment did not indicate a significant risk from these failure modes. As shown, no water surface rises occur for the 1% ACE flood event or the SPF flood event upstream of the confluence, therefore, no increased risk of flooding would occur to areas upstream of the Dallas Floodway that are not protected by the Dallas Floodway levees.

With regard to the valley storage loss impacts estimation using the Unsteady flow modeling discussed in Section 6.8, the computed maximum water surface rise downstream of the project for the 1% ACE flood event is 0.07 foot and is 0.12 foot for the SPF event. While this would be regarded as a potential increase in flood risk, it should be considered insignificant with consideration for actual damages that potentially could be realized. First, the immediate areas downstream of the Dallas Floodway are impacted by the Dallas Floodway Extension Project reach. The Dallas Floodway Extension project as described above is designed to ultimately provide flood risk benefits up to the SPF flood event with completion of the proposed levees. Since the levees have not been constructed, the very small rise estimated for the SPF flood event may be compensated for in the final design for the Dallas Floodway Extension levees at an insignificant additional cost to provide the same flood risk as designed. If the levee construction components for the Dallas Floodway Extension are extensively delayed or eliminated, the hydraulic benefits currently realized by completion of the chain of wetlands components of the Dallas Floodway Extension project more than compensate for any expected rise due to the estimated valley storage loss for the Dallas Floodway proposed projects. Secondly, downstream of the Dallas Floodway Extension project area, there are few structures subject to flooding by the SPF or 1% ACE flood event. Therefore, the very small potential for additional flood risk in the downstream areas would have very insignificant economic impacts.

In conclusion, while additional design refinement efforts may be able to reduce the valley storage losses noted and/or reduce the water surface rises for the 1% ACE flood event within the Dallas Floodway on the main stem Trinity River, meeting the ROD criteria on every point is likely not achievable for such a large and complex combination of projects. Further reducing the negative impacts for valley storage loss to some extent may be achievable, but since these estimated impacts are relatively insignificant, efforts to further reduce them are not likely to be cost effective at this level of design. At the current level of design for the various project components considered, the level of compliance with regard to meeting the goals of the 1988 ROD criteria is estimated to be very nearly optimal.

It is expected that further design efforts at this feasibility level for the purposes of meeting the ROD criteria would not result in significant gains in reducing potential increases in flood risk downstream of the Dallas Floodway project compared to the Future Without-Project conditions. However, further hydraulic analysis will be required for selected construction phases continuing through the final design stages to ensure that any potential flood risk impacts are analyzed for determination of hydraulic neutrality and remain technically sound (or hydraulically neutral) using the same criteria as used for this Dallas Floodway Feasibility Study. It is also expected that this ongoing analysis will be utilized to further reduce or minimize potential flood risk increases as design opportunities arise during the final design stages of the various project components.

6.10 Recommended Modified Dallas Floodway Project

During the Comprehensive Analysis and an in-depth review of the City of Dallas' BVP and IDP, the decision was made to select a subset of the proposed plan to become the Recommended Modified Dallas Floodway Project. All BVP and IDP features were determined to have the potential to be technically sound following proper design and construction. However, the determination of technical soundness does not necessarily dictate the inclusion of all of these features. The BVP and IDP features that are recommended were determined based on their contributions to meeting the overall project objectives of the FRM and ecosystem restoration..

The Recommended Modified Dallas Floodway Project includes:

- NED Plan (the 277,000 cfs levee raise with the AT&SF Railroad Bridge modification and EAP improvements)
- Levee Side Slope Flattening to 4H:1V
- The IDP Phase I (Hampton and Baker pump stations, and the Nobles Branch sump improvements)
- The IDP Phase II (Charlie, Delta, and New Trinity Portland pump stations)
- River Relocation
- Corinth Wetlands

While the above-mentioned features will be cost shared, it is assumed that the sponsor, City of Dallas, will continue with plans to construct the remainder of the BVP and the IDP as local features. Please see the main report for further details on the Recommended Modified Dallas Floodway Project and the approval process for the local features.

7.0 REFERENCES

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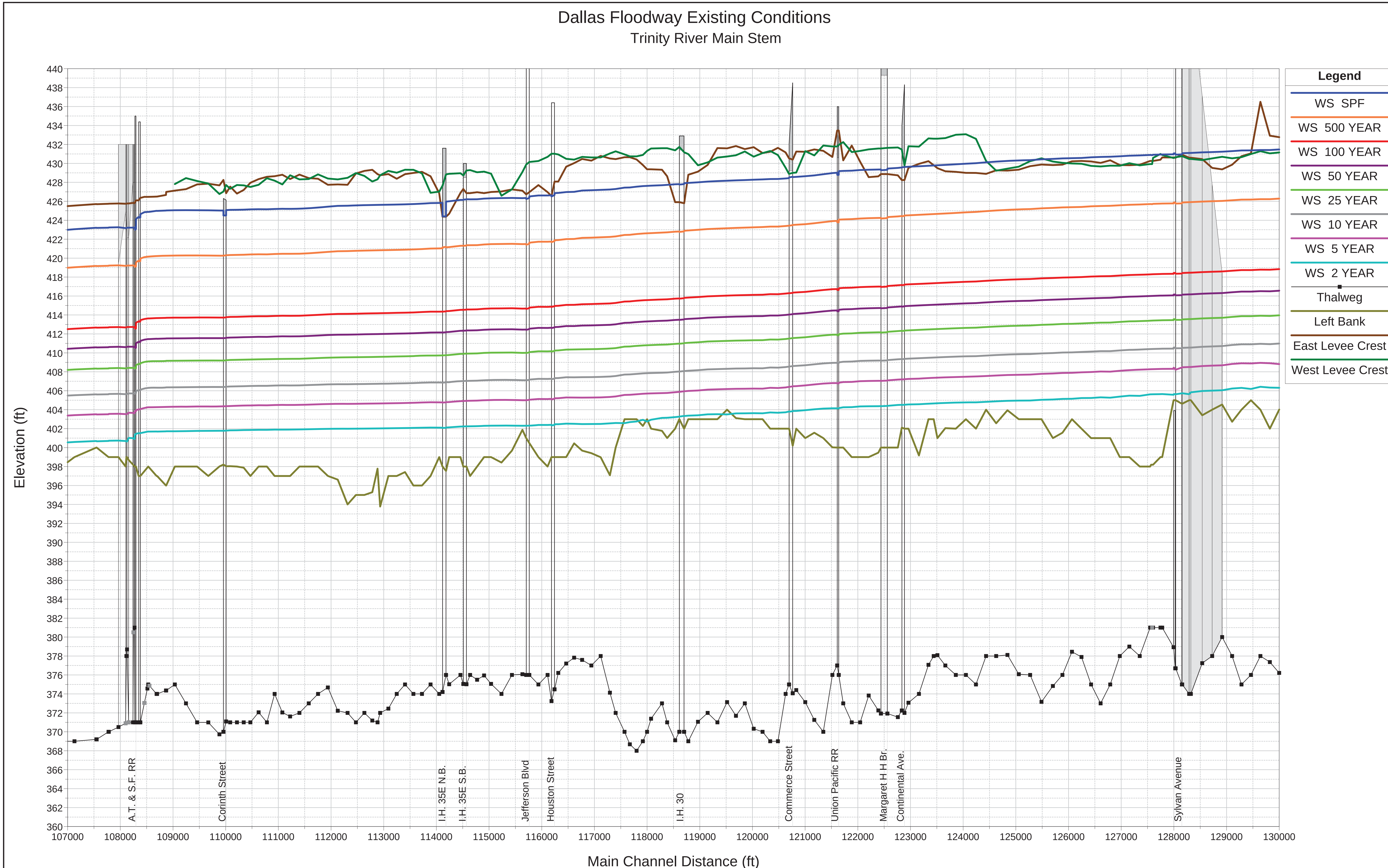
Dallas Floodway Existing Conditions
Trinity River Main Stem
Elevation (ft)
Main Channel Distance (ft)
Legend
WS SPF
WS 500 YEAR
WS 100 YEAR
WS 50 YEAR
WS 25 YEAR
WS 10 YEAR
WS 5 YEAR
WS 2 YEAR
Thalweg
Left Bank
East Levee Crest
West Levee Crest
A.T. & S.F. RR
Corinth Street
I.H. 35E N.B.
I.H. 35E S.B.
Jefferson Blvd
Houston Street
I.H. 30
Commerce Street
Union Pacific RR
Margaret H H Br.
Continental Ave.
Sylvan Avenue

Figure 6-6

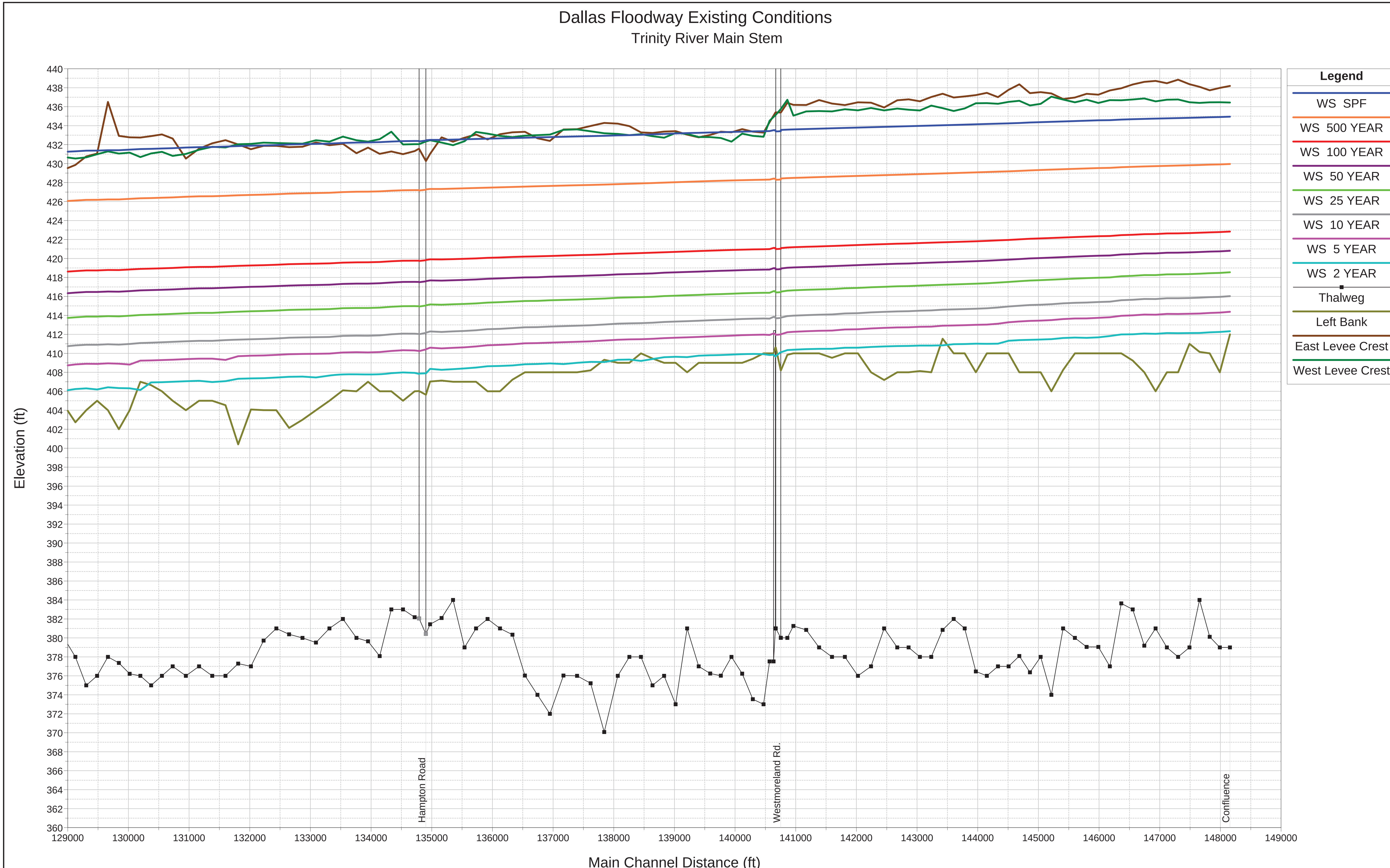
Dallas Floodway Existing Conditions
Trinity River Main Stem
Elevation (ft)
Main Channel Distance (ft)
Legend
WS SPF
WS 500 YEAR
WS 100 YEAR
WS 50 YEAR
WS 25 YEAR
WS 10 YEAR
WS 5 YEAR
WS 2 YEAR
Thalweg
Left Bank
East Levee Crest
West Levee Crest
Hampton Road
Westmoreland Rd.
Confluence

Figure 6-7

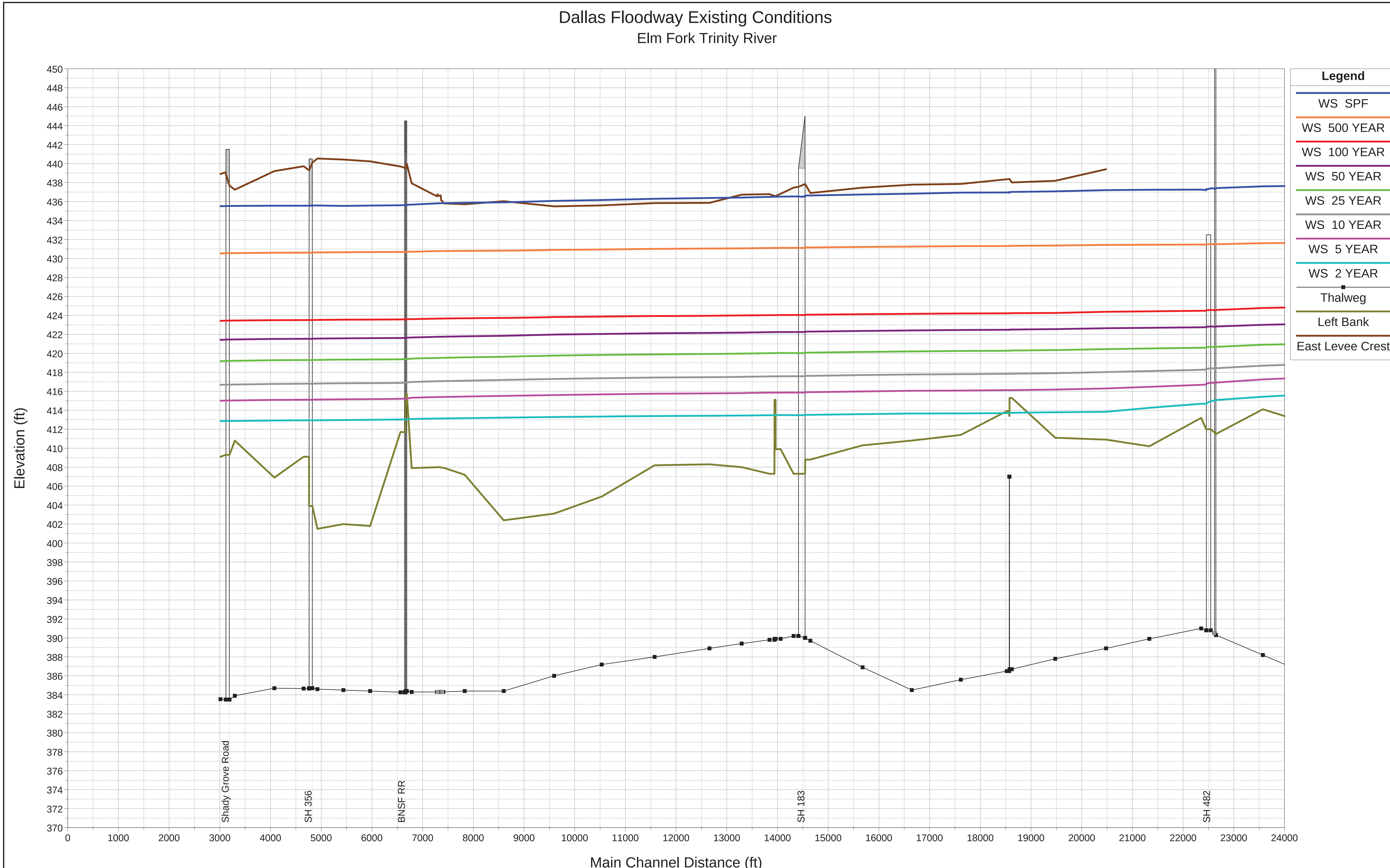
Dallas Floodway Existing Conditions
Elm Fork Trinity River
Legend
WS SPF
WS 500 YEAR
WS 100 YEAR
WS 50 YEAR
WS 25 YEAR
WS 10 YEAR
WS 5 YEAR
WS 2 YEAR
Thalweg
Left Bank
East Levee Crest
Shady Grove Road
SH 356
BNSF RR
SH 183
SH 482
Elevation (ft)
Main Channel Distance (ft)
450
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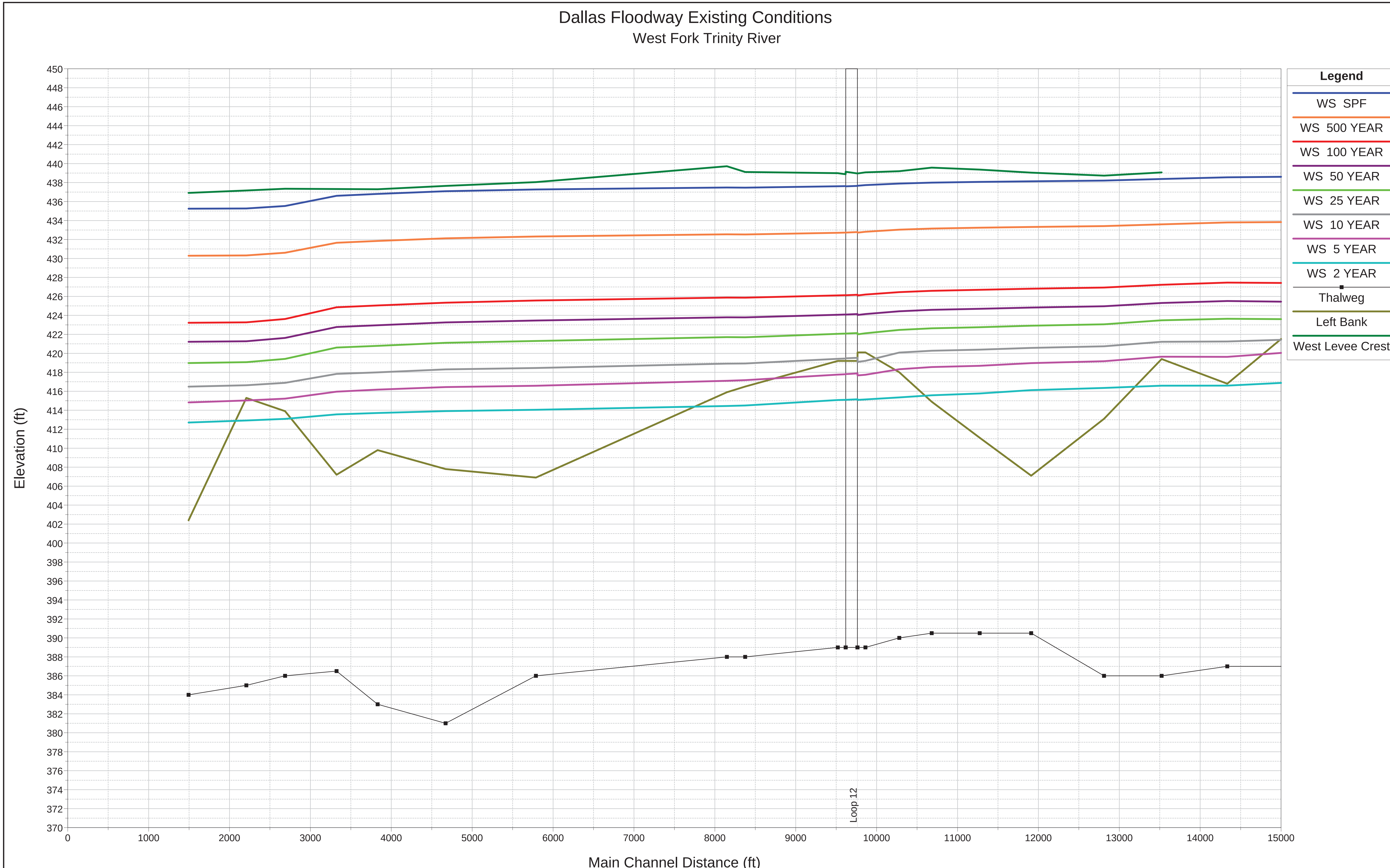
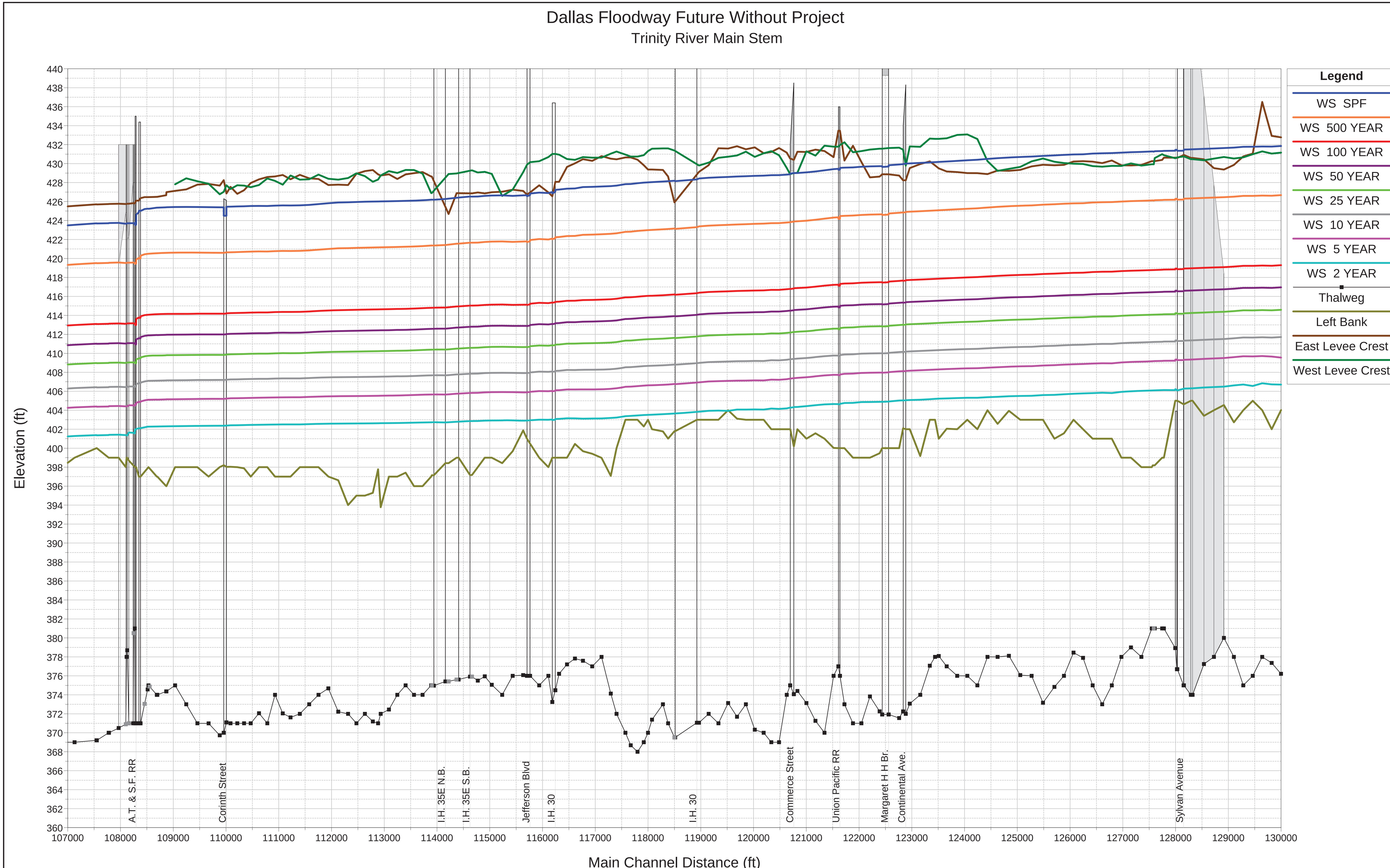
Figure 6-8



Dallas Floodway Existing Conditions
West Fork Trinity River
Elevation (ft)
Main Channel Distance (ft)
Loop 12
Legend
WS SPF
WS 500 YEAR
WS 100 YEAR
WS 50 YEAR
WS 25 YEAR
WS 10 YEAR
WS 5 YEAR
WS 2 YEAR
Thalweg
Left Bank
West Levee Crest

Figure 6-9



Dallas Floodway Future Without Project
Trinity River Main Stem
Legend
WS SPF
WS 500 YEAR
WS 100 YEAR
WS 50 YEAR
WS 25 YEAR
WS 10 YEAR
WS 5 YEAR
WS 2 YEAR
Thalweg
Left Bank
East Levee Crest
West Levee Crest
Elevation (ft)
Main Channel Distance (ft)
A.T. & S.F. RR
Corinth Street
I.H. 35E N.B.
I.H. 35E S.B.
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Union Pacific RR
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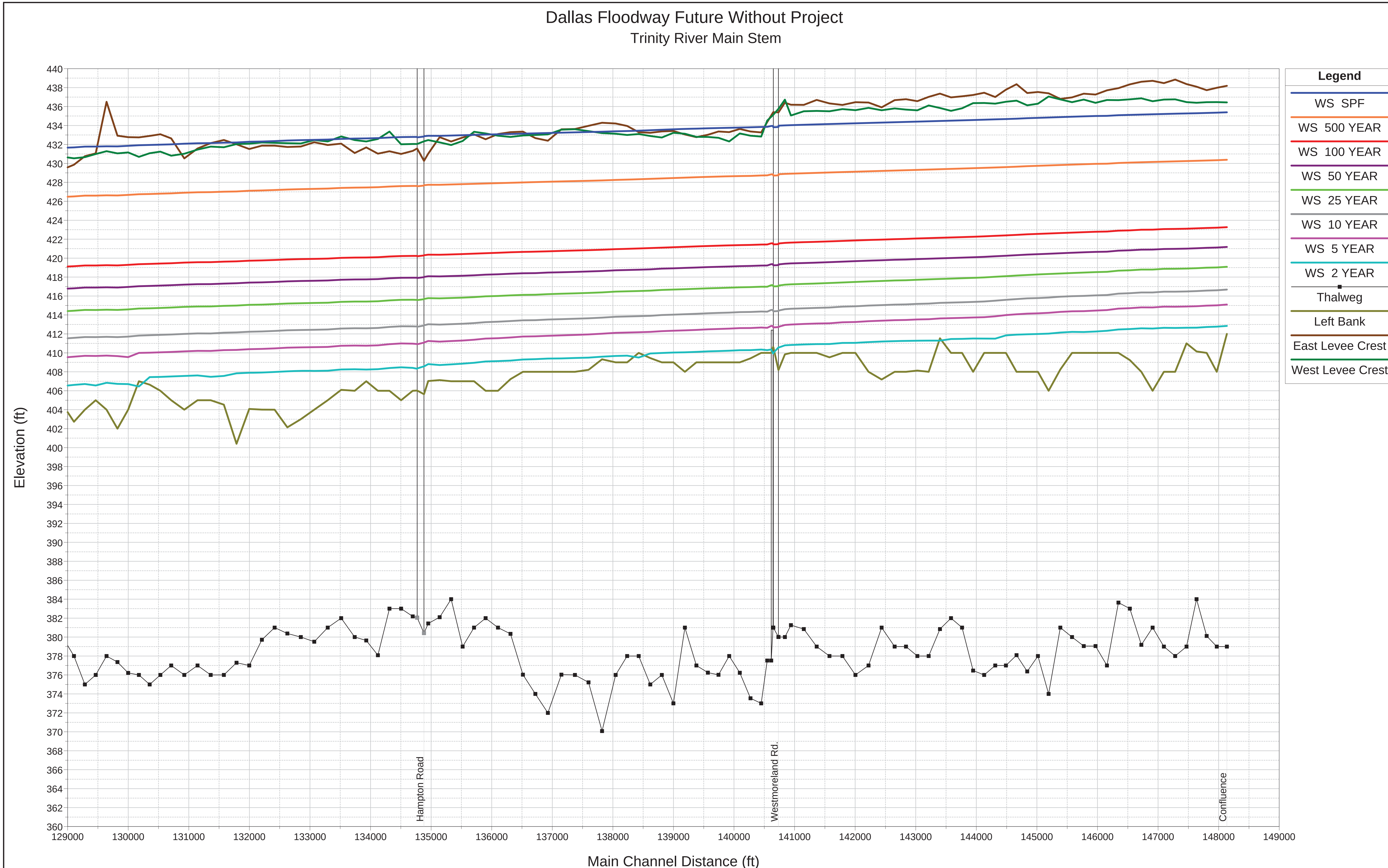
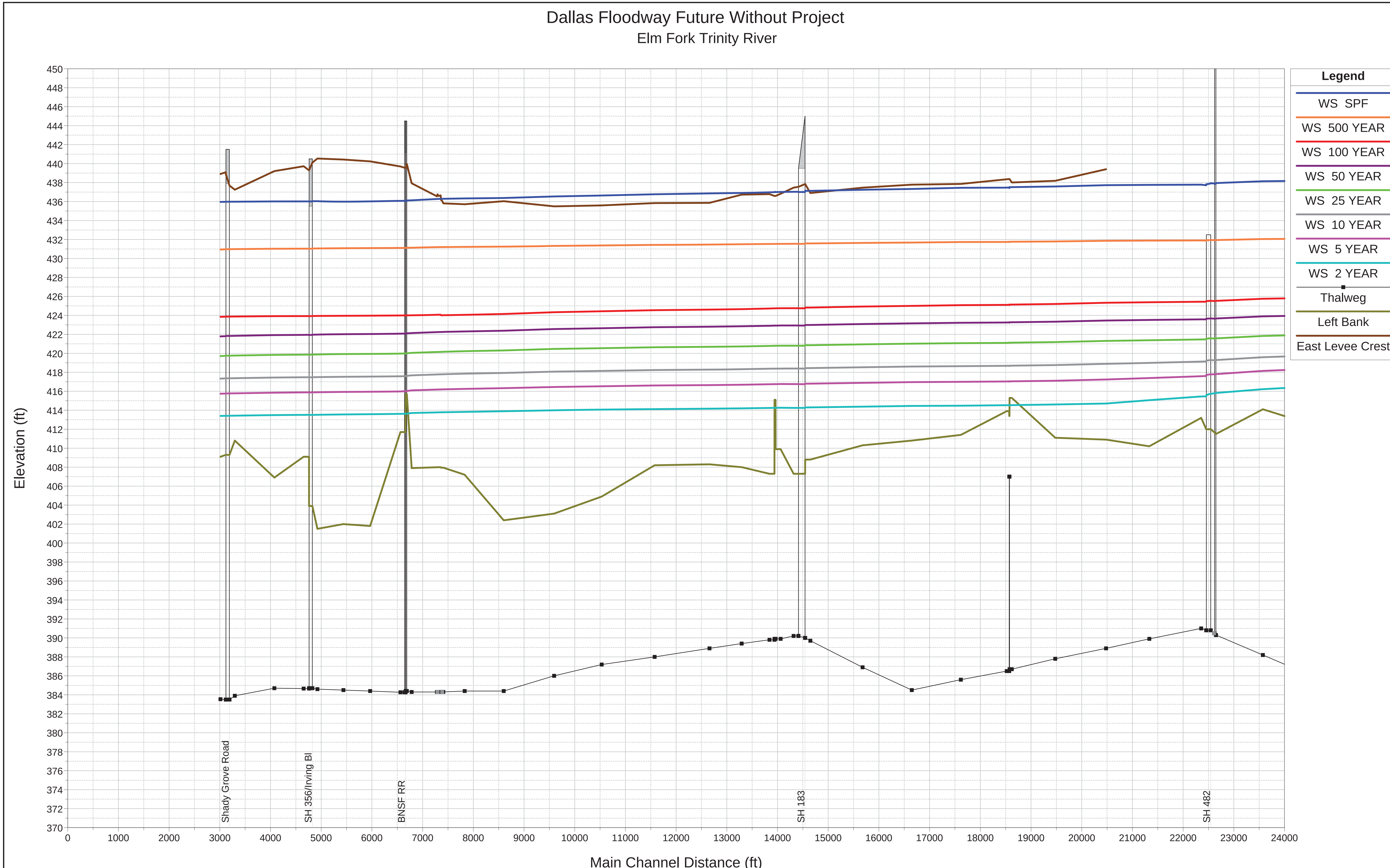
Continental Ave.
Sylvan Avenue

Figure 6-10



Dallas Floodway Future Without Project
Trinity River Main Stem
Elevation (ft)
Main Channel Distance (ft)
Legend
WS SPF
WS 500 YEAR
WS 100 YEAR
WS 50 YEAR
WS 25 YEAR
WS 10 YEAR
WS 5 YEAR
WS 2 YEAR
Thalweg
Left Bank
East Levee Crest
West Levee Crest
Hampton Road
Westmoreland Rd.
Confluence

Figure 6-11



Dallas Floodway Future Without Project
Elm Fork Trinity River
Elevation (ft)
Main Channel Distance (ft)
Legend
WS SPF
WS 500 YEAR
WS 100 YEAR
WS 50 YEAR
WS 25 YEAR
WS 10 YEAR
WS 5 YEAR
WS 2 YEAR
Thalweg
Left Bank
East Levee Crest
Shady Grove Road
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BNSF RR
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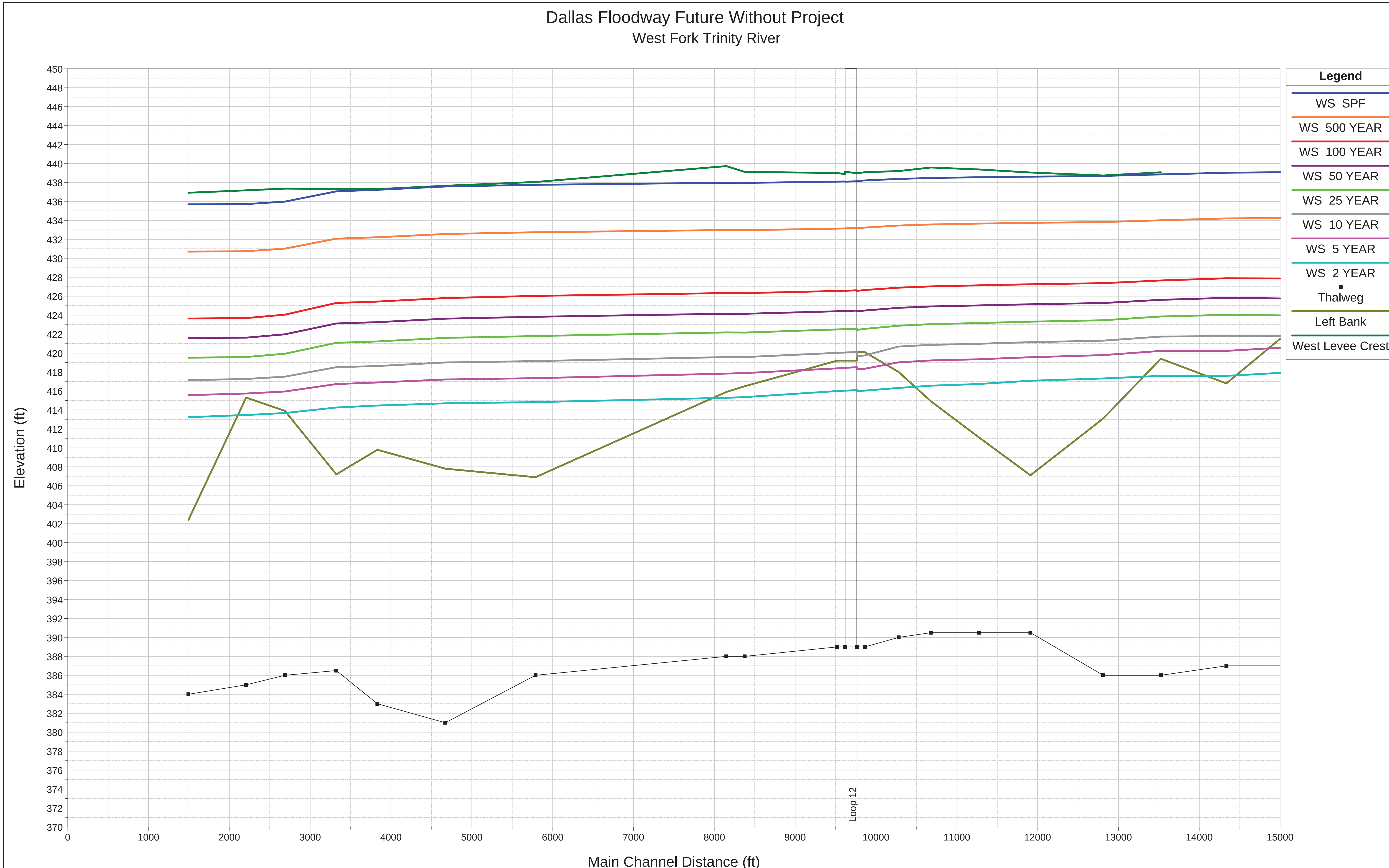
Figure 6-12



Dallas Floodway Future Without Project
West Fork Trinity River
Legend
WS SPF
WS 500 YEAR
WS 100 YEAR
WS 50 YEAR
WS 25 YEAR
WS 10 YEAR
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WS 2 YEAR
Thalweg
Left Bank
West Levee Crest
Loop 12
Elevation (ft)
Main Channel Distance (ft)
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Figure 6-13

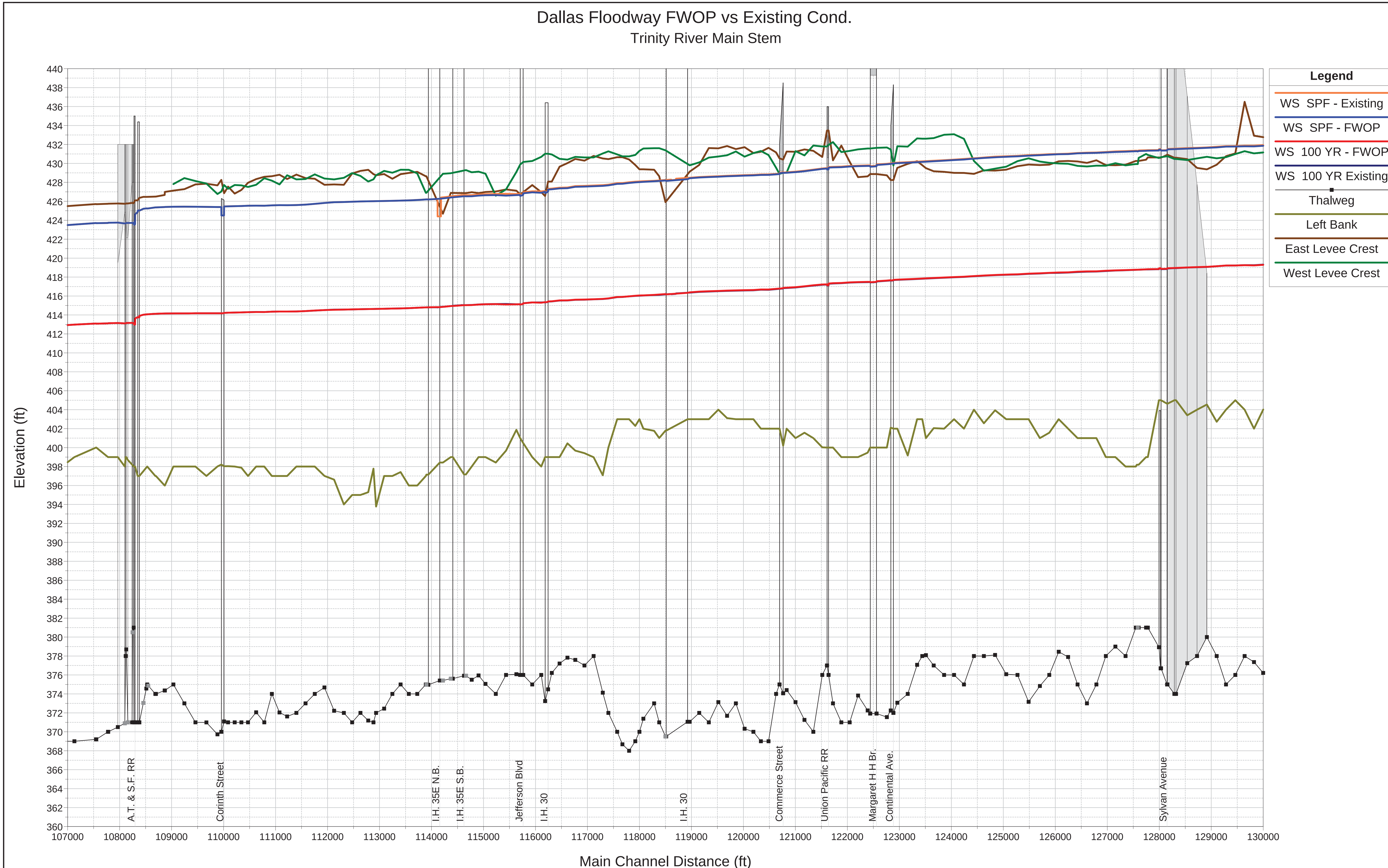
Dallas Floodway FWOP vs Existing Cond.
Trinity River Main Stem
Legend
WS SPF - Existing
WS SPF - FWOP
WS 100 YR - FWOP
WS 100 YR Existing
Thalweg
Left Bank
East Levee Crest
West Levee Crest
Elevation (ft)
Main Channel Distance (ft)
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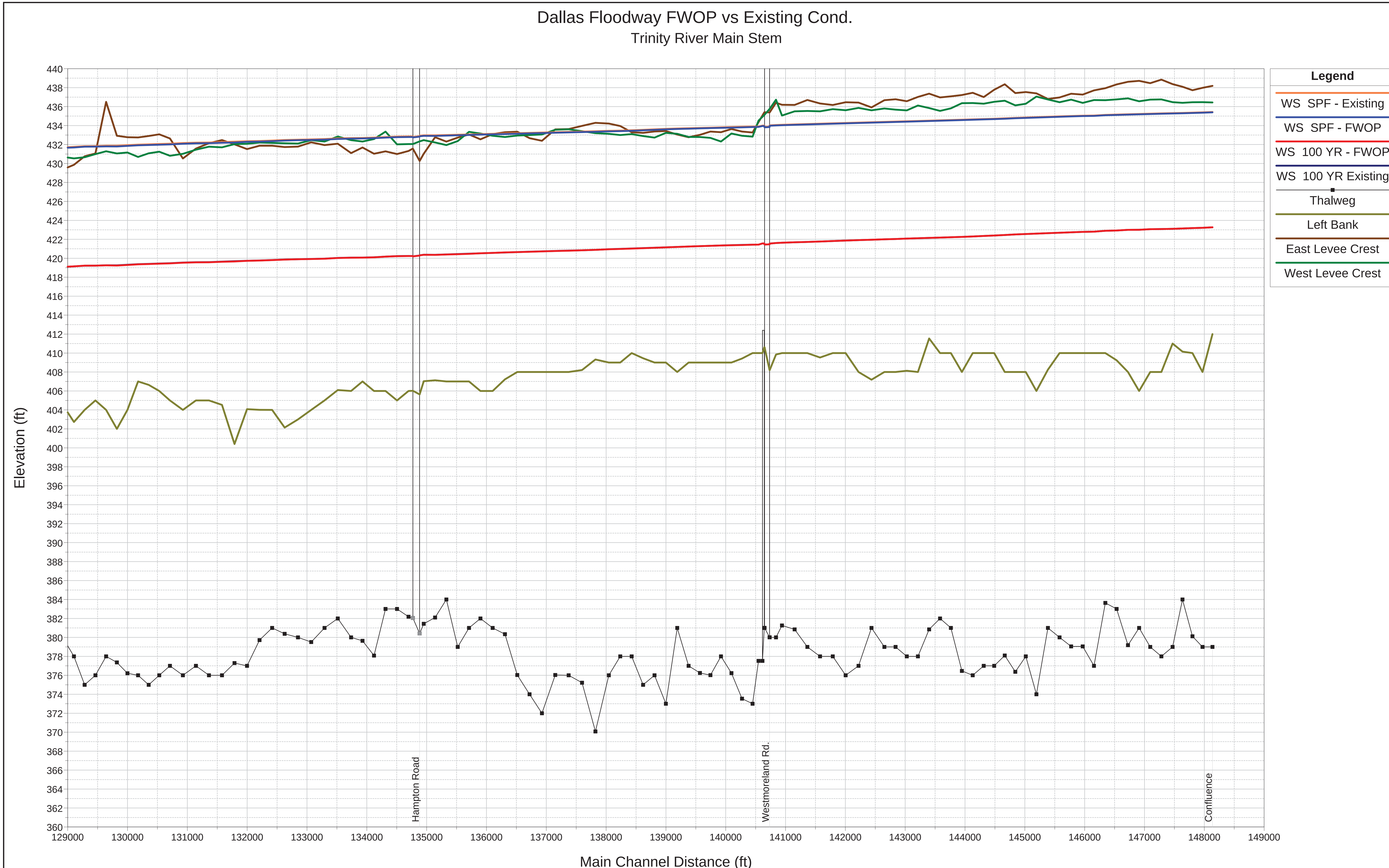
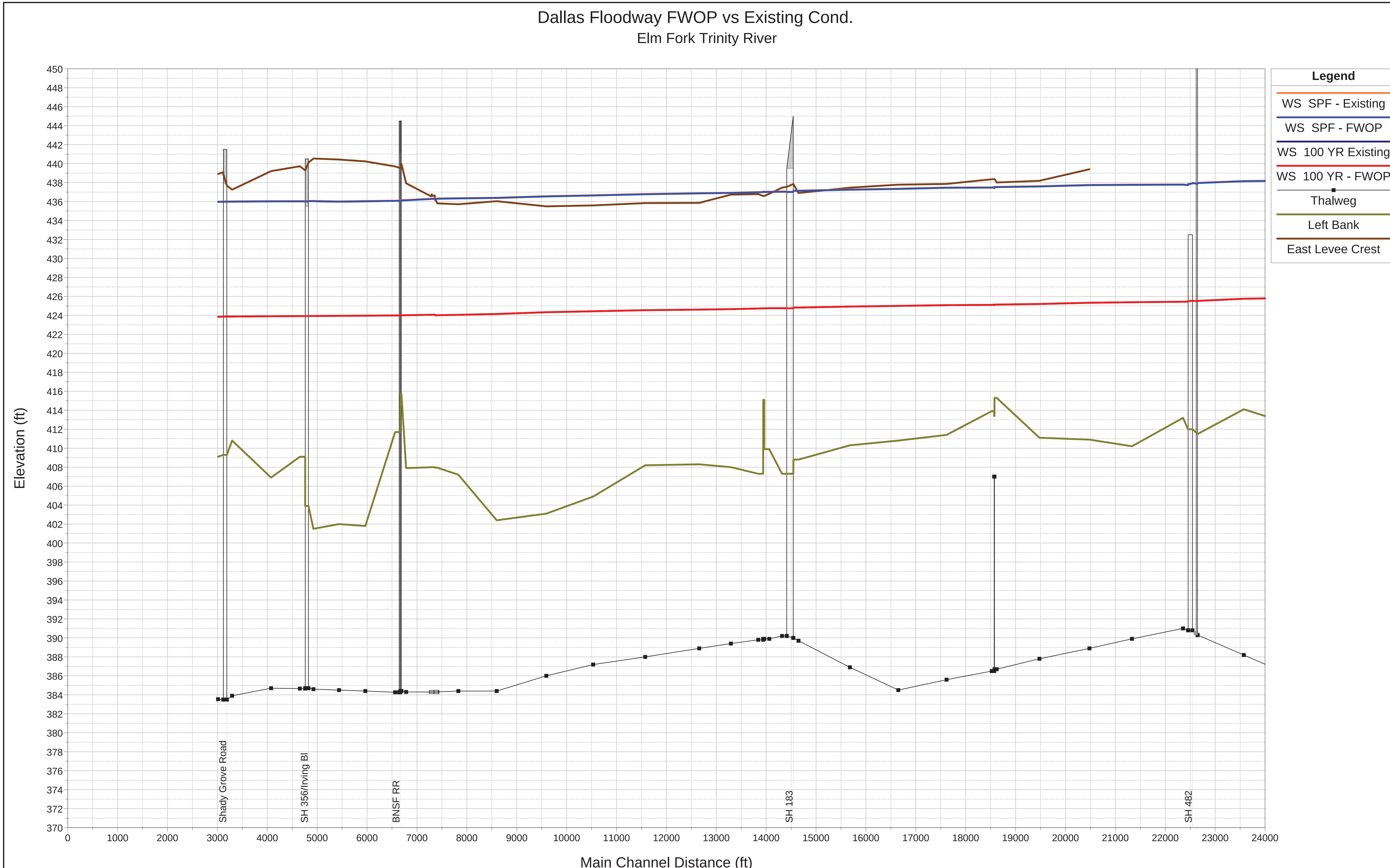
Figure 6-14



Dallas Floodway FWOP vs Existing Cond.
Trinity River Main Stem
Legend
WS SPF - Existing
WS SPF - FWOP
WS 100 YR - FWOP
WS 100 YR Existing
Thalweg
Left Bank
East Levee Crest
West Levee Crest
Elevation (ft)
Main Channel Distance (ft)
Hampton Road
Westmoreland Rd.
Confluence

Figure 6-15



Dallas Floodway FWOP vs Existing Cond.
Elm Fork Trinity River
Legend
WS SPF - Existing
WS SPF - FWOP
WS 100 YR Existing
WS 100 YR - FWOP
Thalweg
Left Bank
East Levee Crest
Elevation (ft)
Main Channel Distance (ft)
Shady Grove Road
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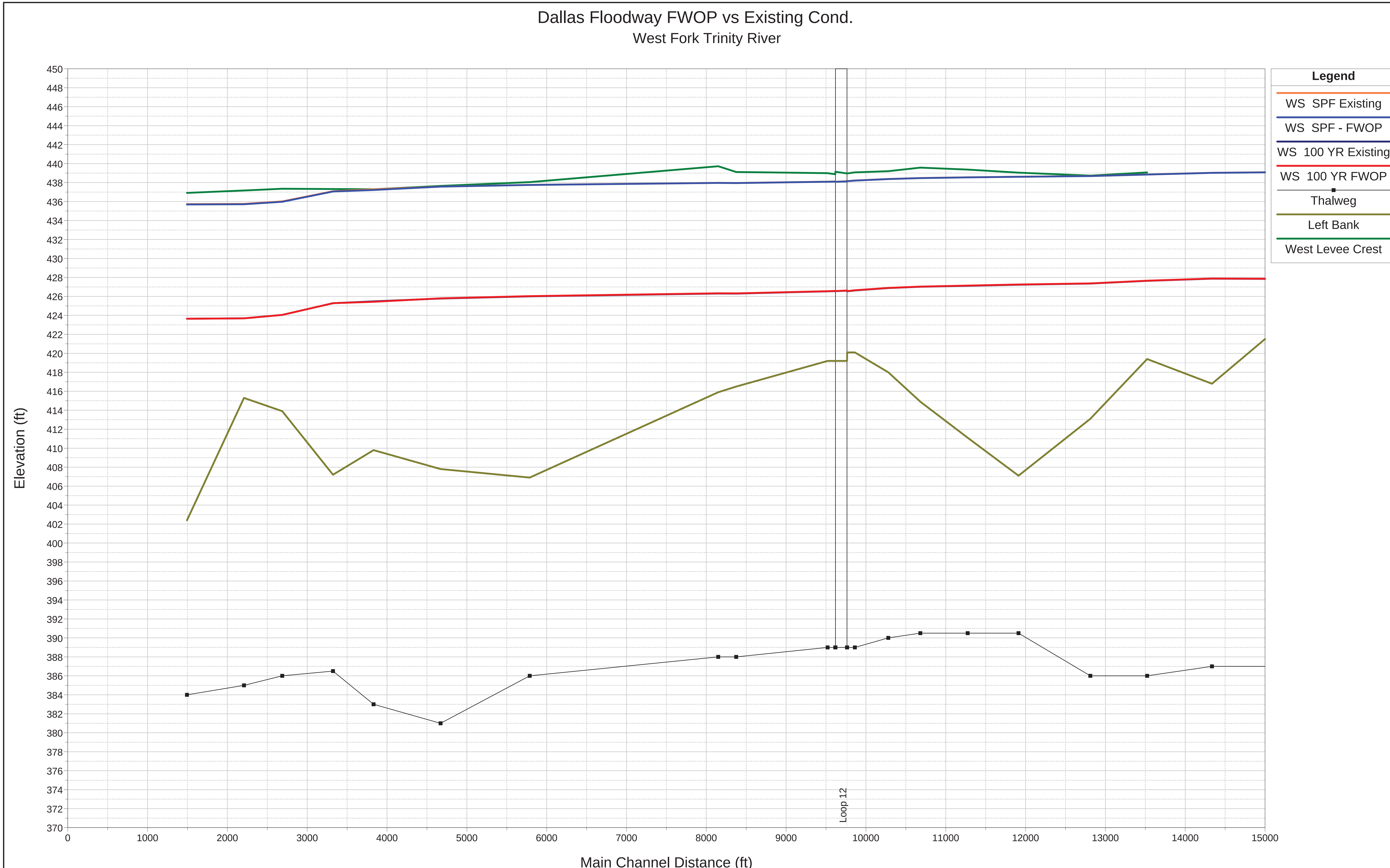
Figure 6-16


Dallas Floodway FWOP vs Existing Cond.
West Fork Trinity River
Elevation (ft)
Main Channel Distance (ft)
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Thalweg
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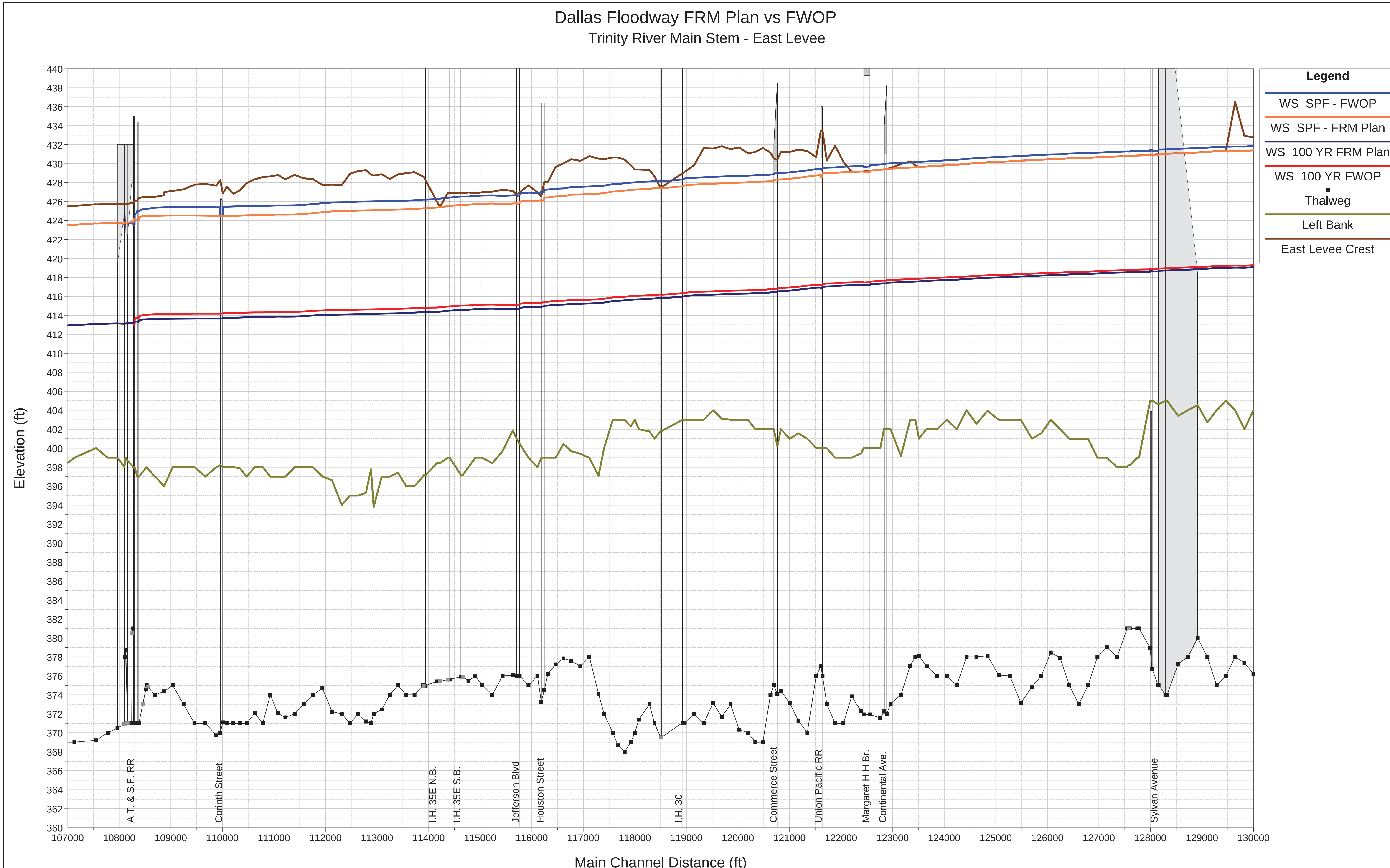
Figure 6-17



Dallas Floodway FRM Plan vs FWOP
Trinity River Main Stem - East Levee
Legend
WS SPF - FWOP
WS SPF - FRM Plan
WS 100 YR FRM Plan
WS 100 YR FWOP
Thalweg
Left Bank
East Levee Crest
Elevation (ft)
Main Channel Distance (ft)
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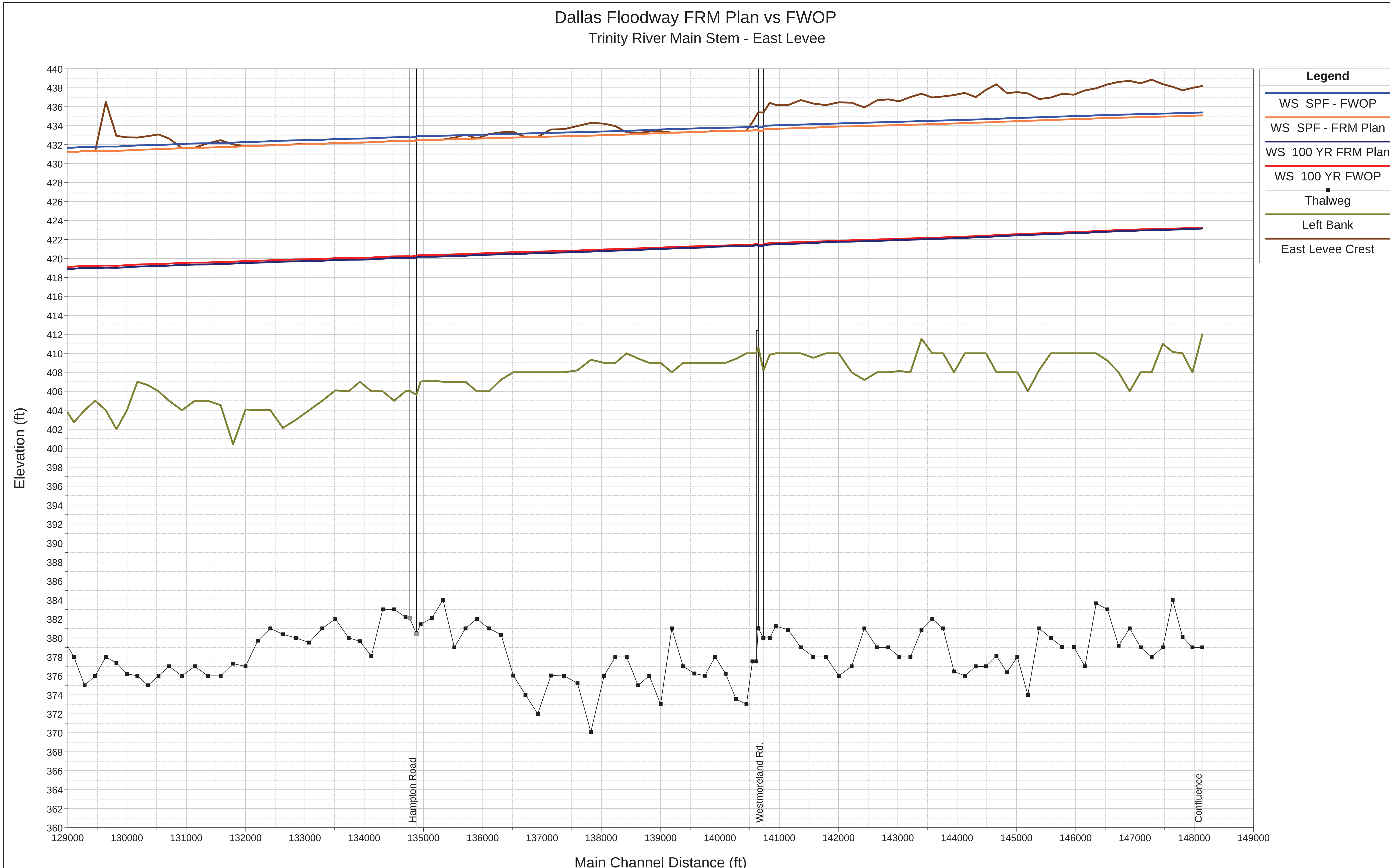
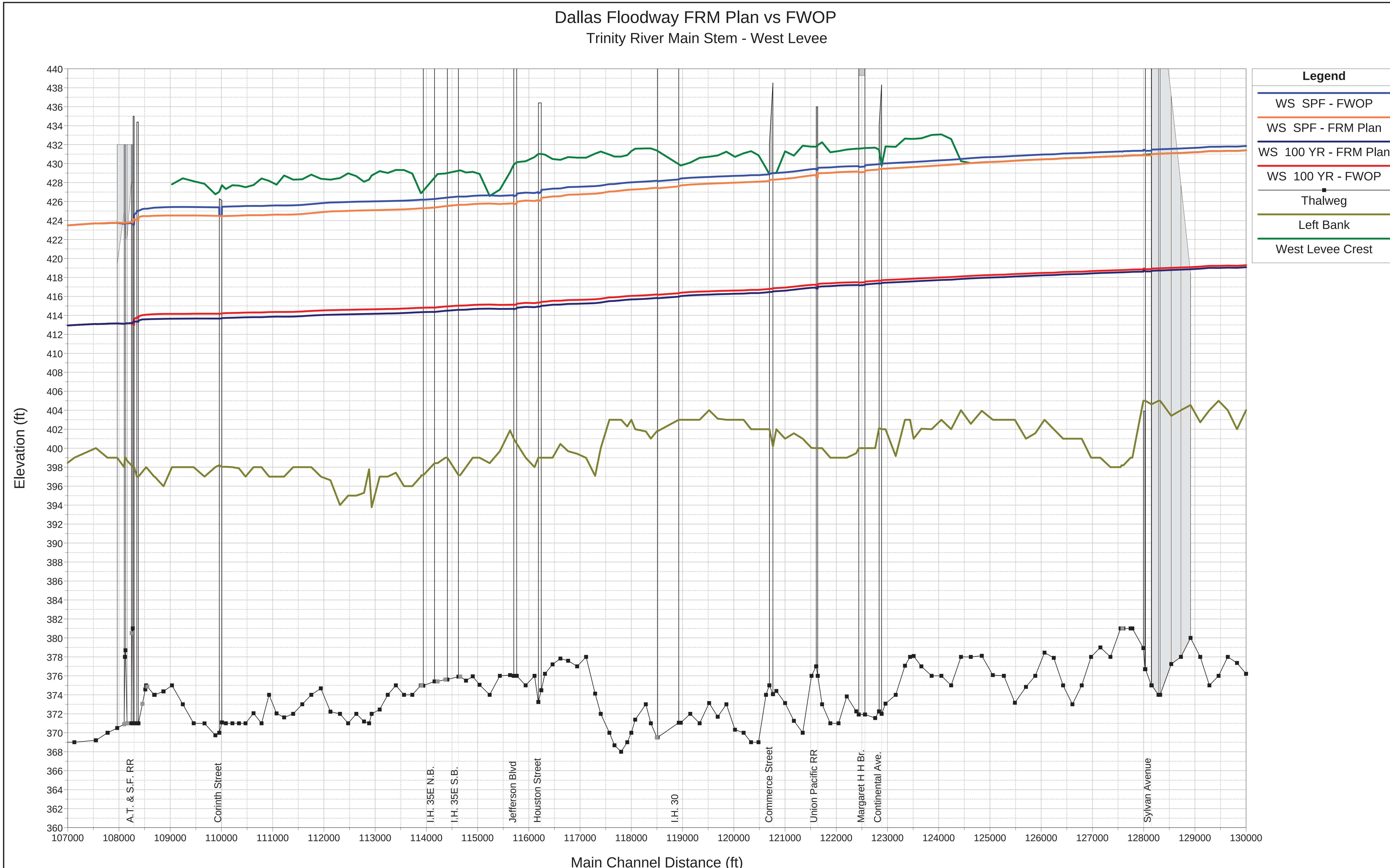
Figure 6-18



Dallas Floodway FRM Plan vs FWOP
Trinity River Main Stem - East Levee
Legend
WS SPF - FWOP
WS SPF - FRM Plan
WS 100 YR FRM Plan
WS 100 YR FWOP
Thalweg
Left Bank
East Levee Crest
Elevation (ft)
Main Channel Distance (ft)
Hampton Road
Westmoreland Rd.
Confluence

Figure 6-19


Dallas Floodway FRM Plan vs FWOP
Trinity River Main Stem - West Levee
Legend
WS SPF - FWOP
WS SPF - FRM Plan
WS 100 YR - FRM Plan
WS 100 YR - FWOP
Thalweg
Left Bank
West Levee Crest
Elevation (ft)
Main Channel Distance (ft)
A.T. & S.F. RR
Corinth Street
I.H. 35E N.B.
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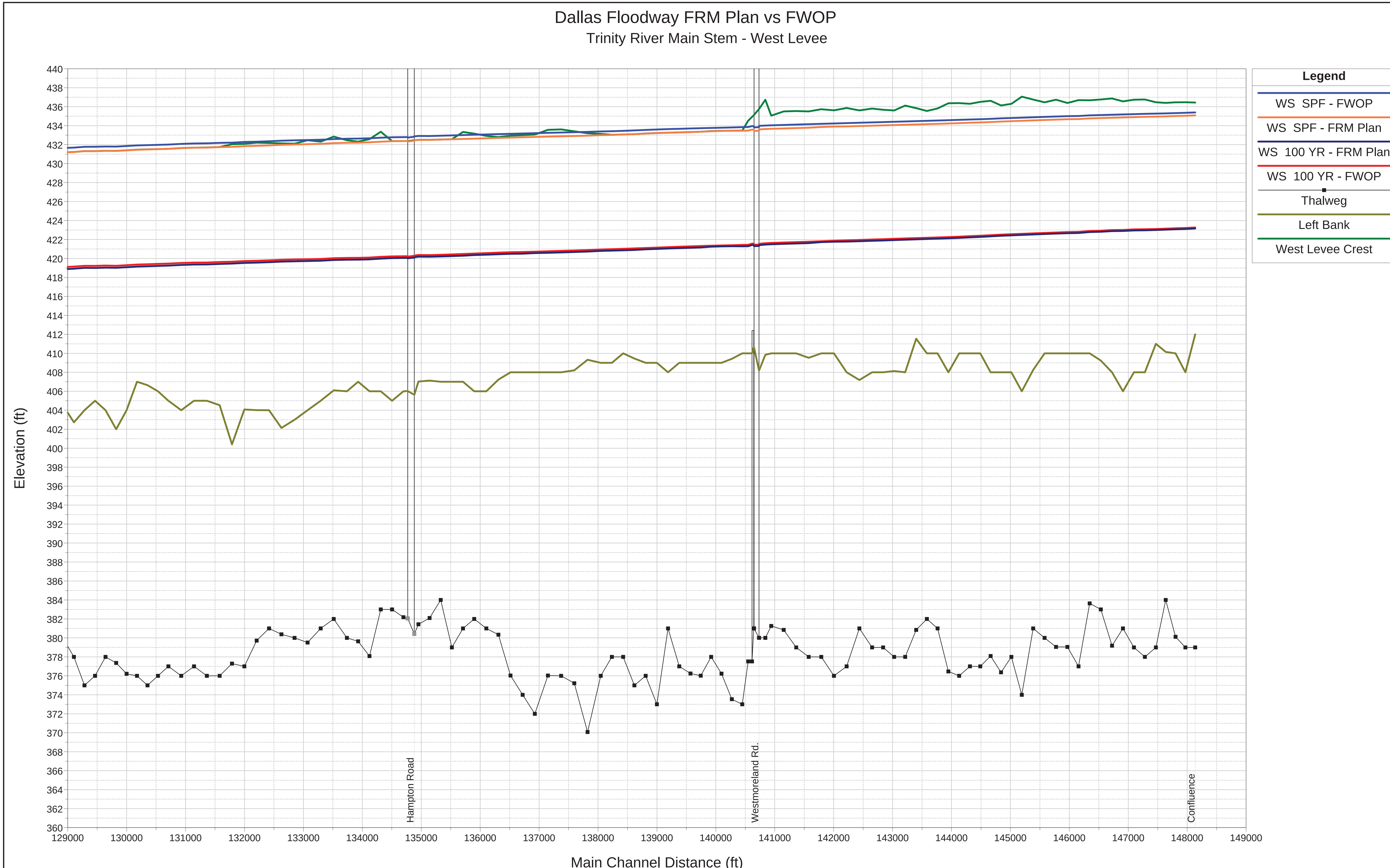
Sylvan Avenue

Figure 6-20



Dallas Floodway FRM Plan vs FWOP
Trinity River Main Stem - West Levee
Legend
WS SPF - FWOP
WS SPF - FRM Plan
WS 100 YR - FRM Plan
WS 100 YR - FWOP
Thalweg
Left Bank
West Levee Crest
Elevation (ft)
Main Channel Distance (ft)
Hampton Road
Westmoreland Rd.
Confluence
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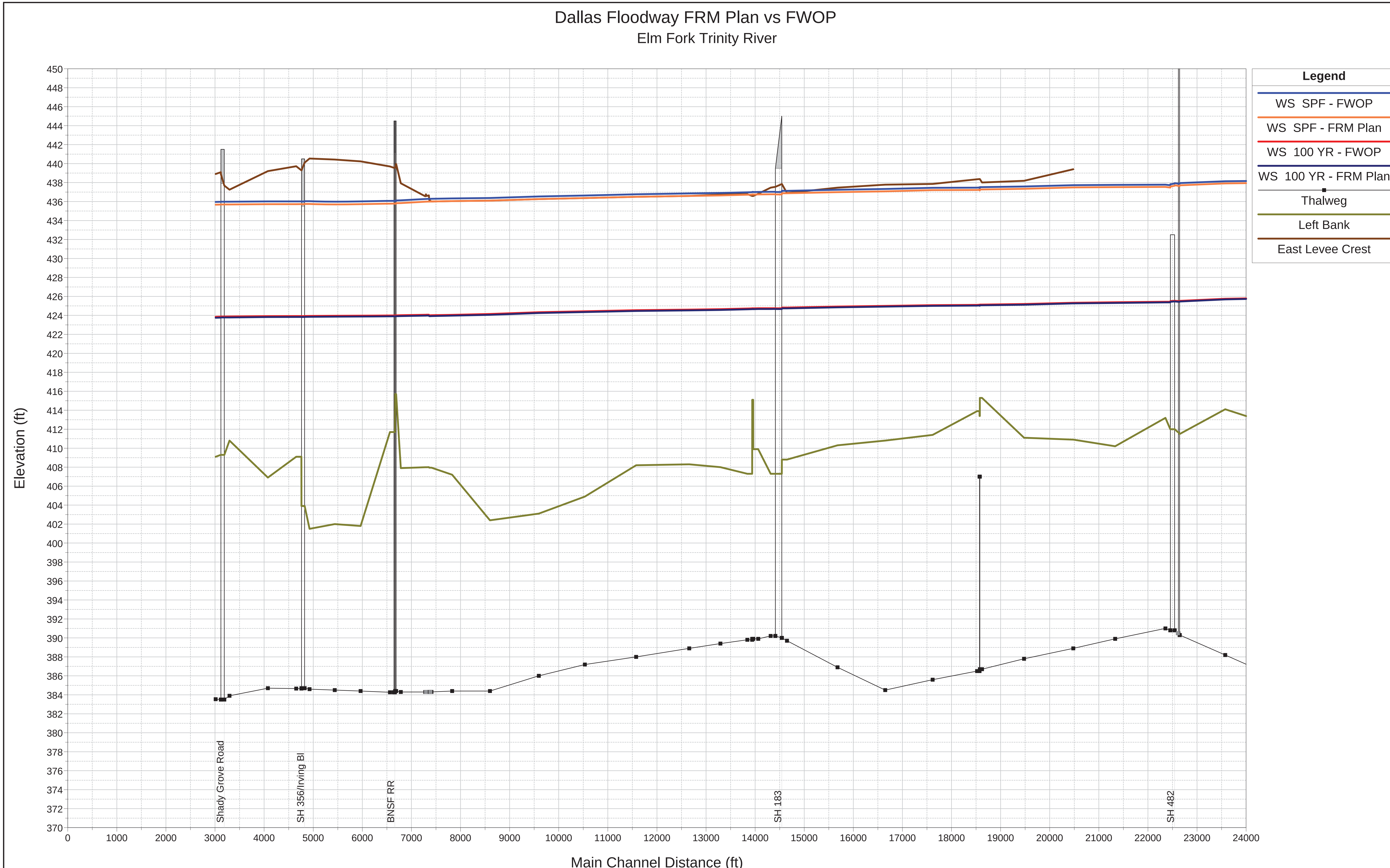
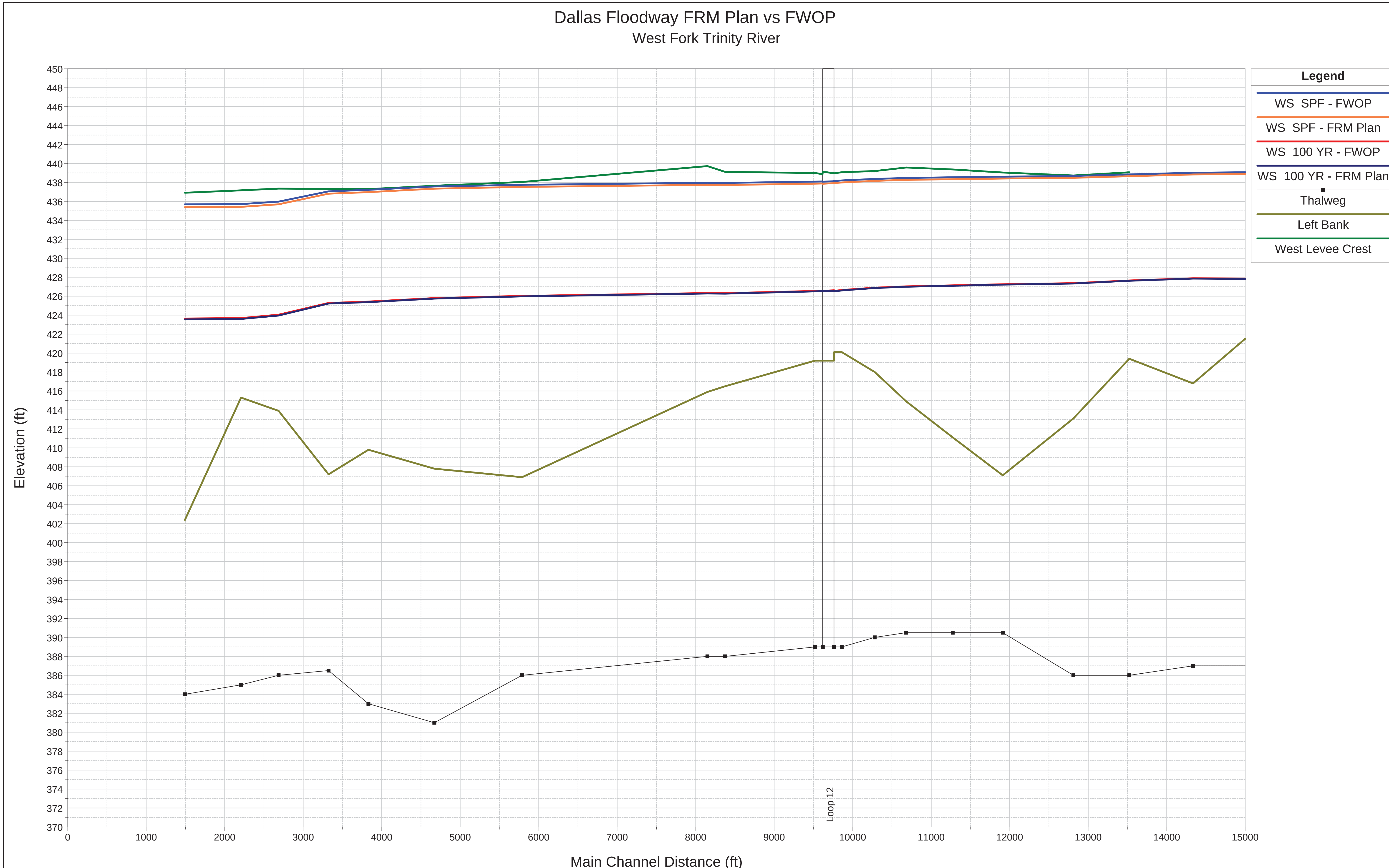
Figure 6-21



Dallas Floodway FRM Plan vs FWOP
Elm Fork Trinity River
Legend
WS SPF - FWOP
WS SPF - FRM Plan
WS 100 YR - FWOP
WS 100 YR - FRM Plan
Thalweg
Left Bank
East Levee Crest
Elevation (ft)
Main Channel Distance (ft)
Shady Grove Road
SH 356/Irving Bl
BNSF RR
SH 183
SH 482

Figure 6-22



Dallas Floodway FRM Plan vs FWOP
West Fork Trinity River
Legend
WS SPF - FWOP
WS SPF - FRM Plan
WS 100 YR - FWOP
WS 100 YR - FRM Plan
Thalweg
Left Bank
West Levee Crest
Loop 12
Elevation (ft)
Main Channel Distance (ft)
450
448
446
444
442
440
438
436
434
432
430
428
426
424
422
420
418
416
414
412
410
408
406
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384
382
380
378
376
374
372
370
0
1000
2000
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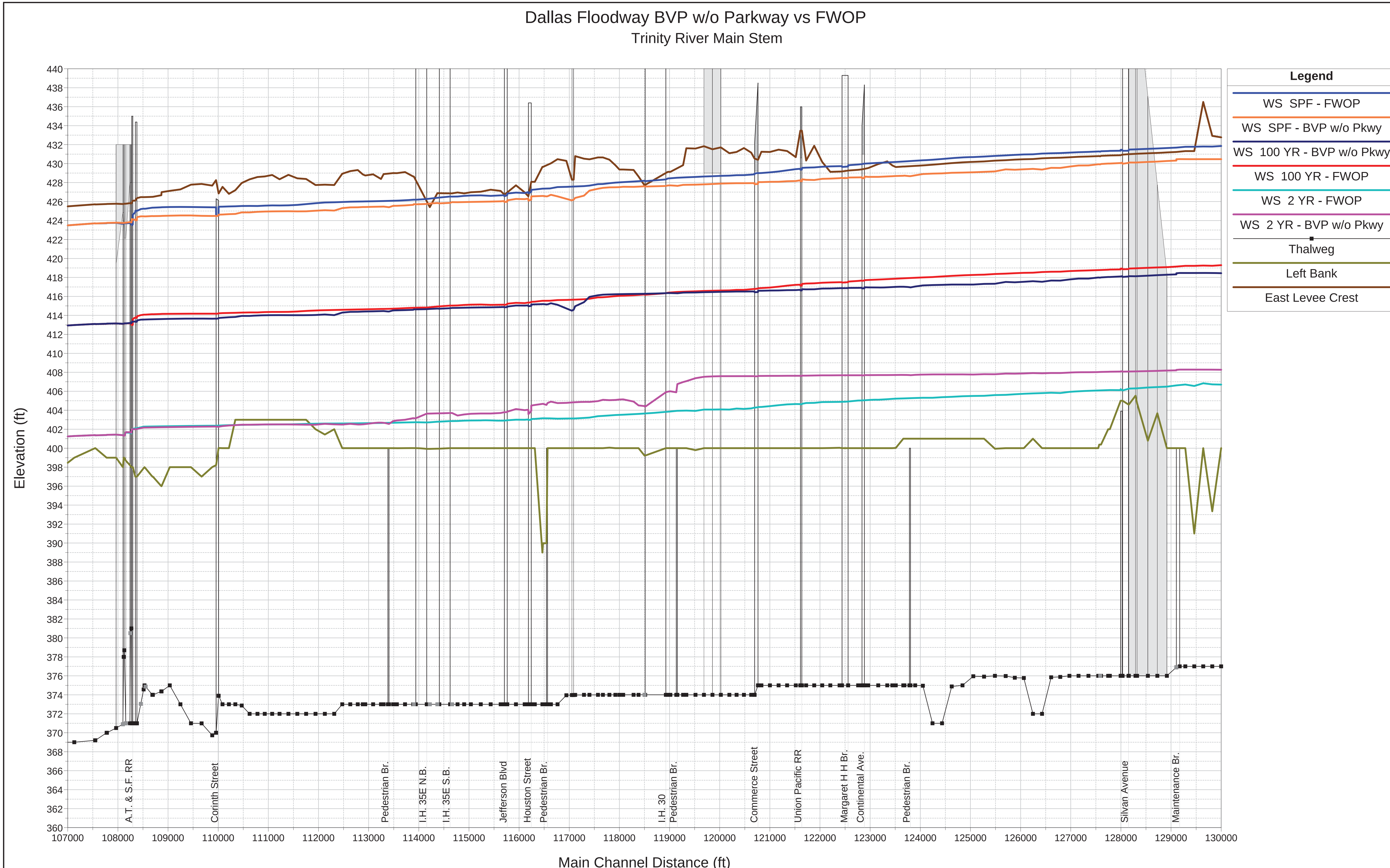
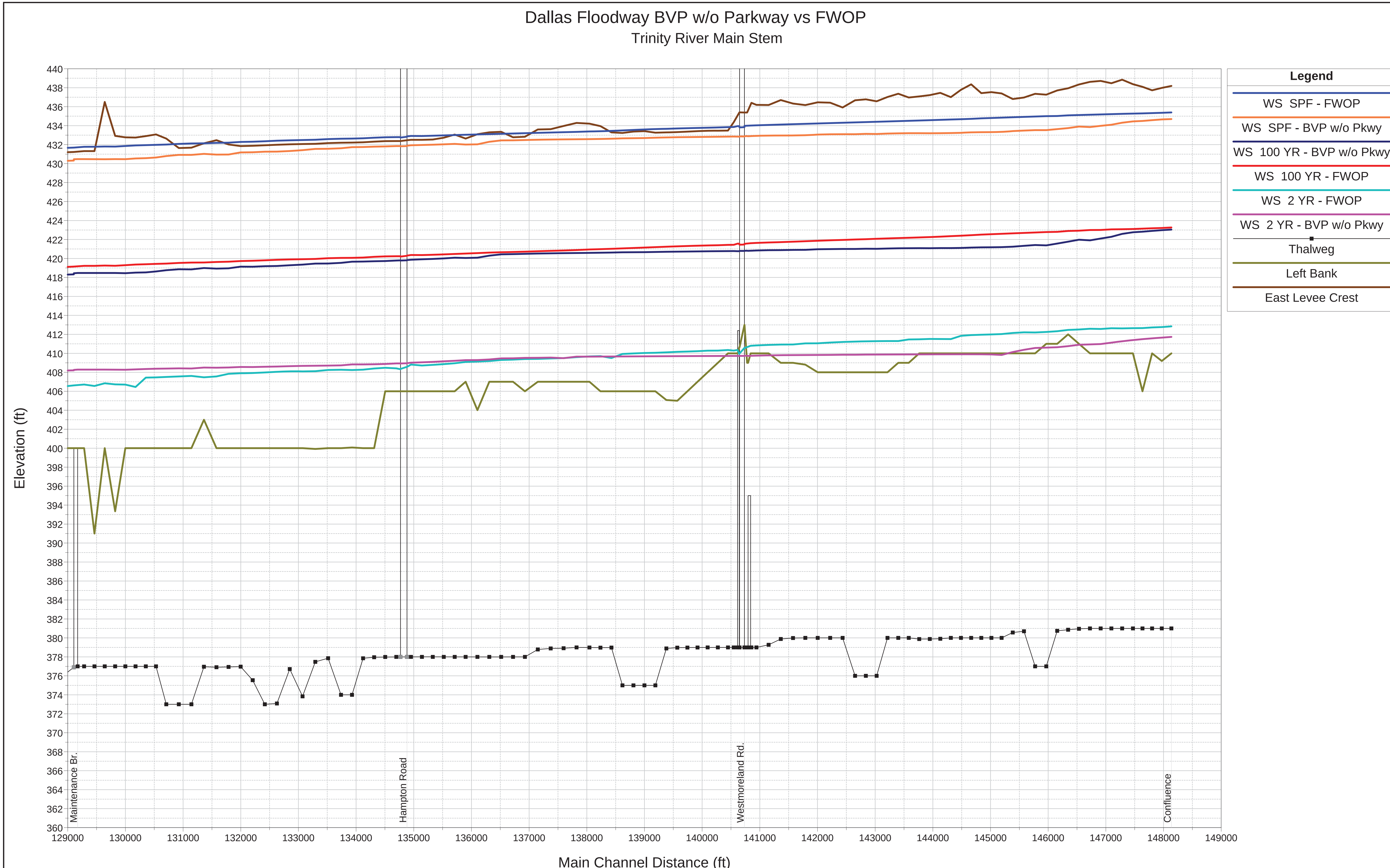
4000
5000
6000
7000
8000
9000
10000
11000
12000
13000
14000
15000

Figure 6-23

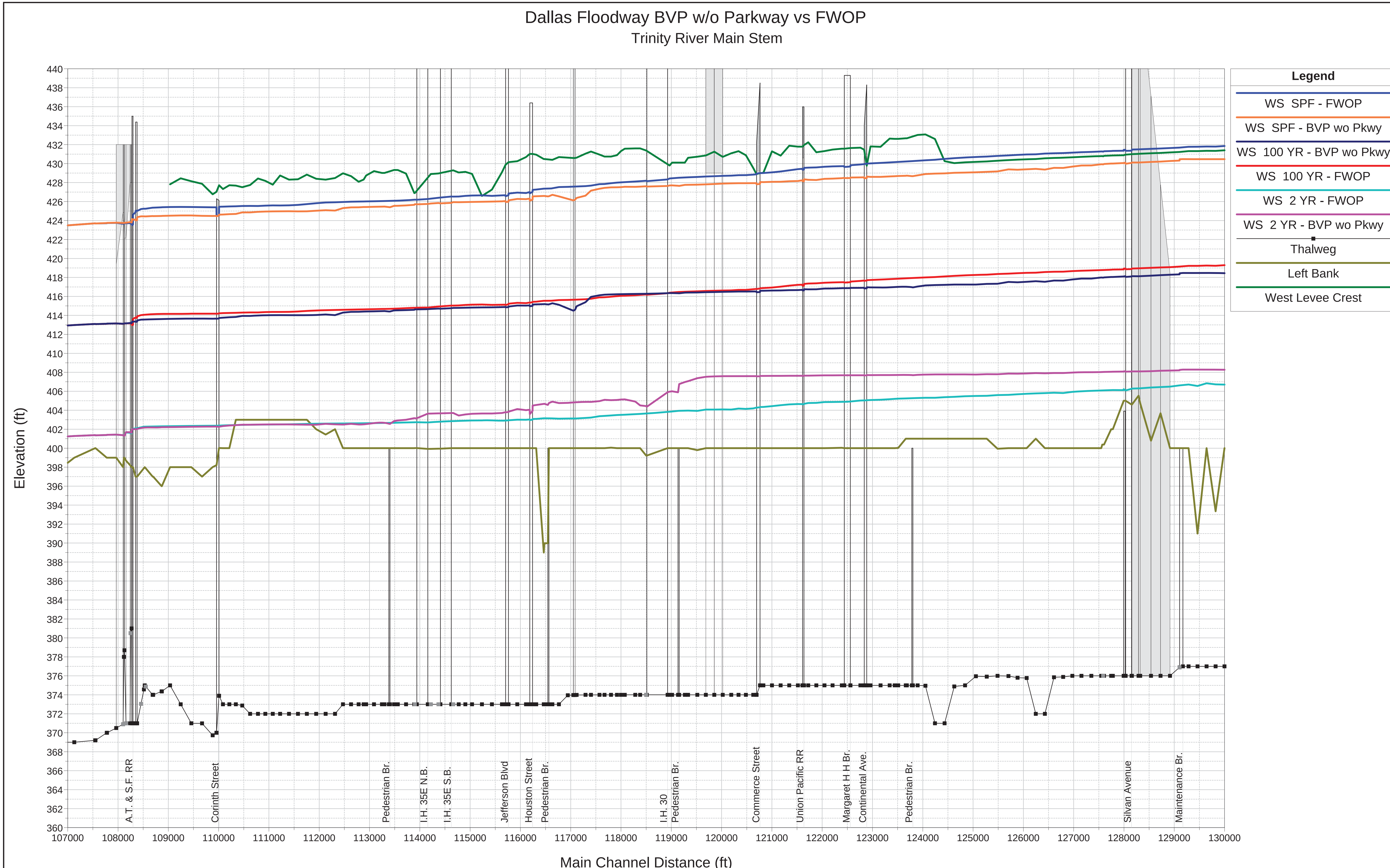


Dallas Floodway BVP w/o Parkway vs FWOP
Trinity River Main Stem
Elevation (ft)
Main Channel Distance (ft)
Legend
WS SPF - FWOP
WS SPF - BVP w/o Pkwy
WS 100 YR - BVP w/o Pkwy
WS 100 YR - FWOP
WS 2 YR - FWOP
WS 2 YR - BVP w/o Pkwy
Thalweg
Left Bank
East Levee Crest
A.T. & S.F. RR
Corinth Street
Pedestrian Br.
I.H. 35E N.B.
I.H. 35E S.B.
Jefferson Blvd
Houston Street
Pedestrian Br.
I.H. 30
Pedestrian Br.
Commerce Street
Union Pacific RR
Margaret H H Br.
Continental Ave.
Pedestrian Br.
Silvan Avenue
Maintenance Br.

Figure 6-24

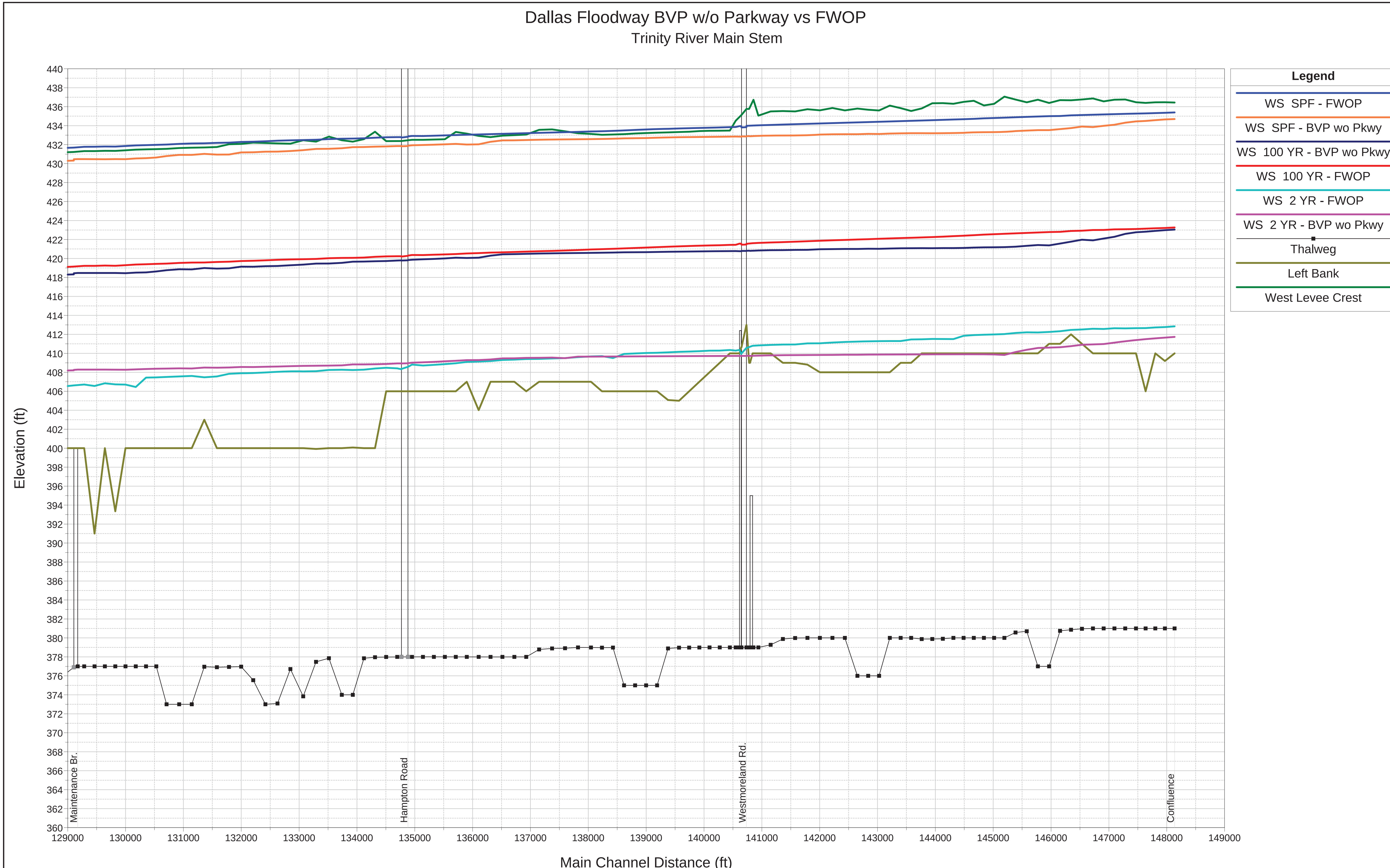


Dallas Floodway BVP w/o Parkway vs FWOP
Trinity River Main Stem
Legend
WS SPF - FWOP
WS SPF - BVP w/o Pkwy
WS 100 YR - BVP w/o Pkwy
WS 100 YR - FWOP
WS 2 YR - FWOP
WS 2 YR - BVP w/o Pkwy
Thalweg
Left Bank
East Levee Crest
Elevation (ft)
Main Channel Distance (ft)
Maintenance Br.
Hampton Road
Westmoreland Rd.
Confluence



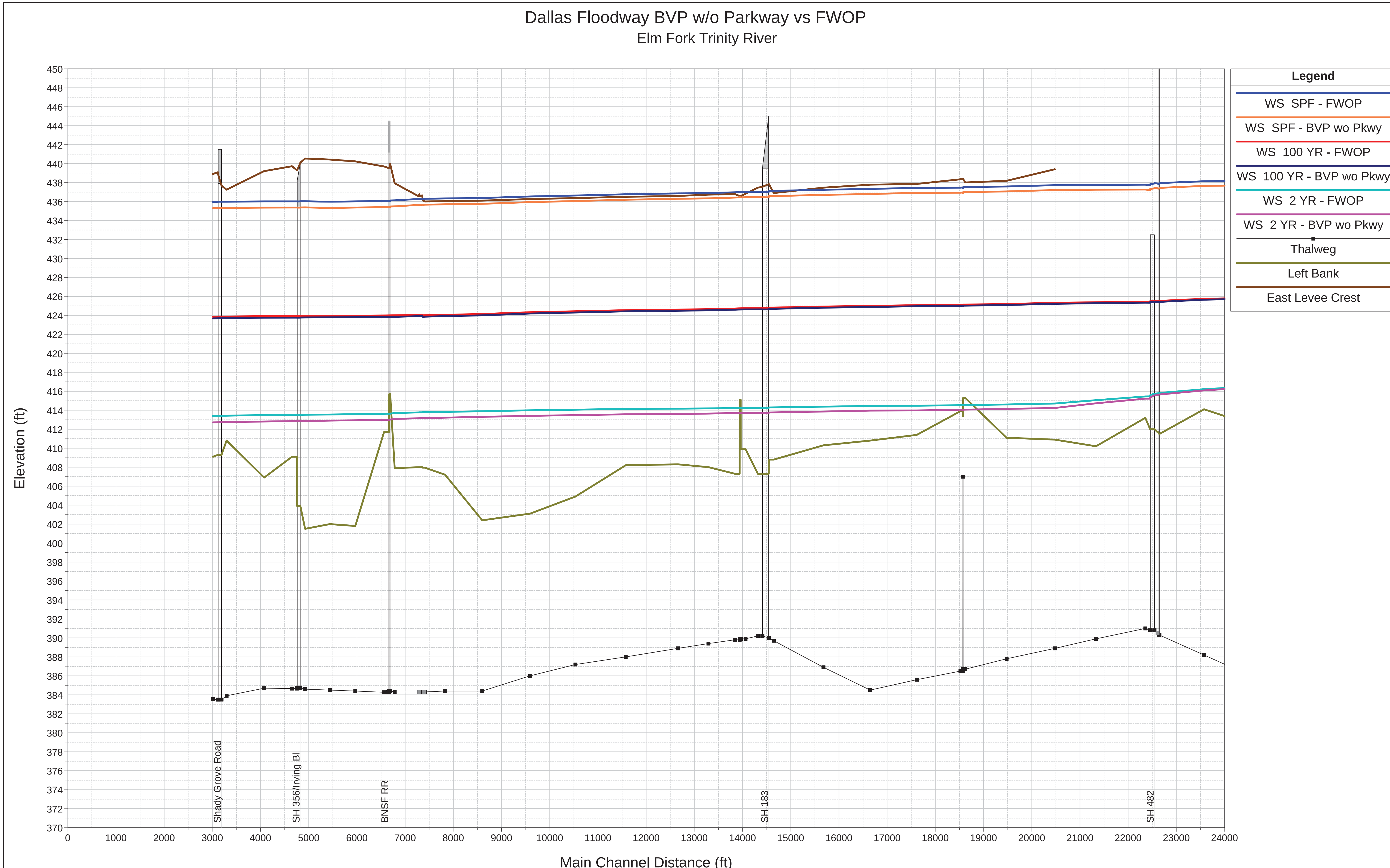
Dallas Floodway BVP w/o Parkway vs FWOP
Trinity River Main Stem
Legend
WS SPF - FWOP
WS SPF - BVP wo Pkwy
WS 100 YR - BVP wo Pkwy
WS 100 YR - FWOP
WS 2 YR - FWOP
WS 2 YR - BVP wo Pkwy
Thalweg
Left Bank
West Levee Crest
Elevation (ft)
Main Channel Distance (ft)
A.T. & S.F. RR
Corinth Street
Pedestrian Br.
I.H. 35E N.B.
I.H. 35E S.B.
Jefferson Blvd
Houston Street
Pedestrian Br.
I.H. 30
Pedestrian Br.
Commerce Street
Union Pacific RR
Margaret H H Br.
Continental Ave.
Pedestrian Br.
Silvan Avenue
Maintenance Br.

Figure 6-26



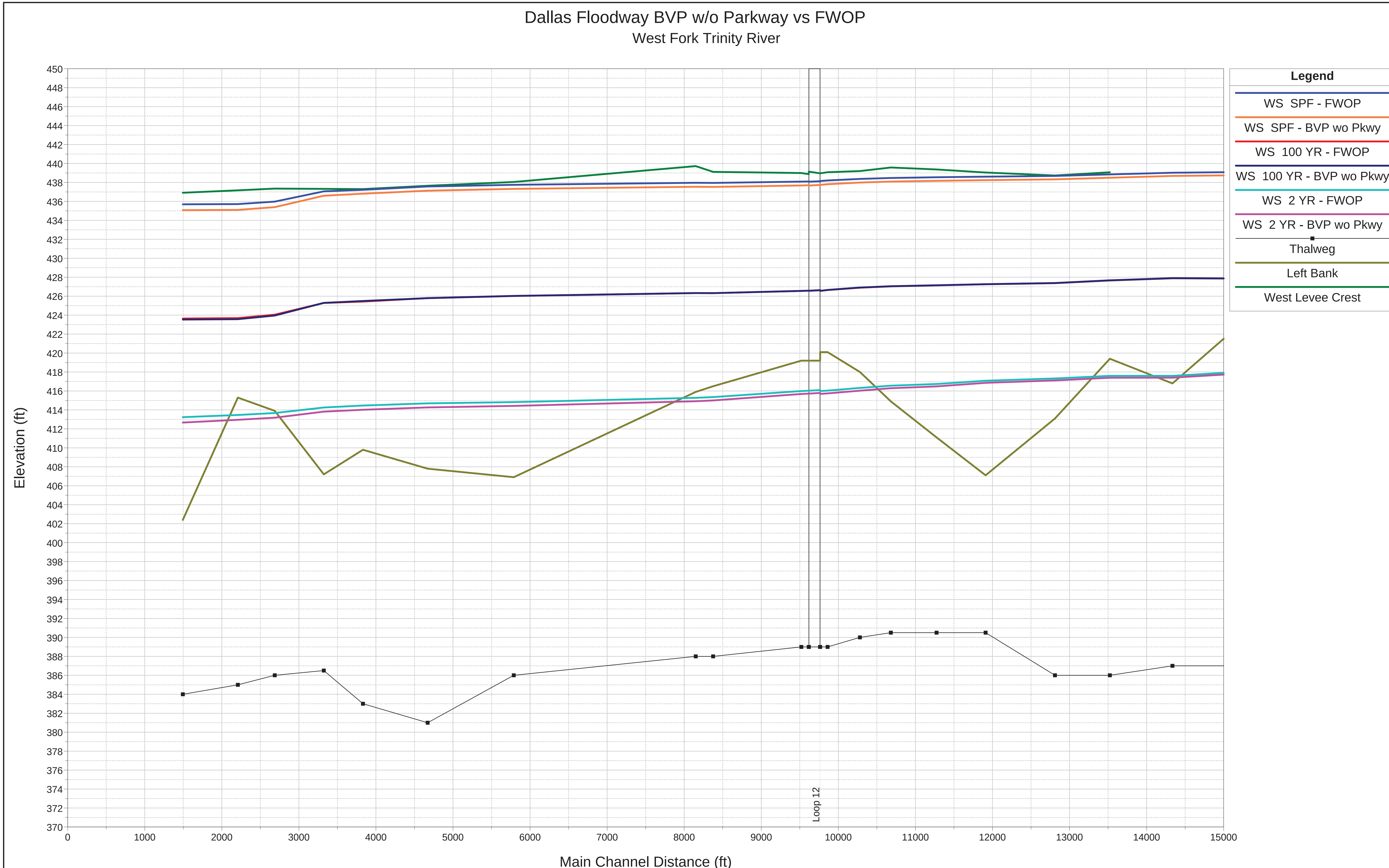
Dallas Floodway BVP w/o Parkway vs FWOP
Trinity River Main Stem
Elevation (ft)
Main Channel Distance (ft)
Legend
WS SPF - FWOP
WS SPF - BVP wo Pkwy
WS 100 YR - BVP wo Pkwy
WS 100 YR - FWOP
WS 2 YR - FWOP
WS 2 YR - BVP wo Pkwy
Thalweg
Left Bank
West Levee Crest
Maintenance Br.
Hampton Road
Westmoreland Rd.
Confluence

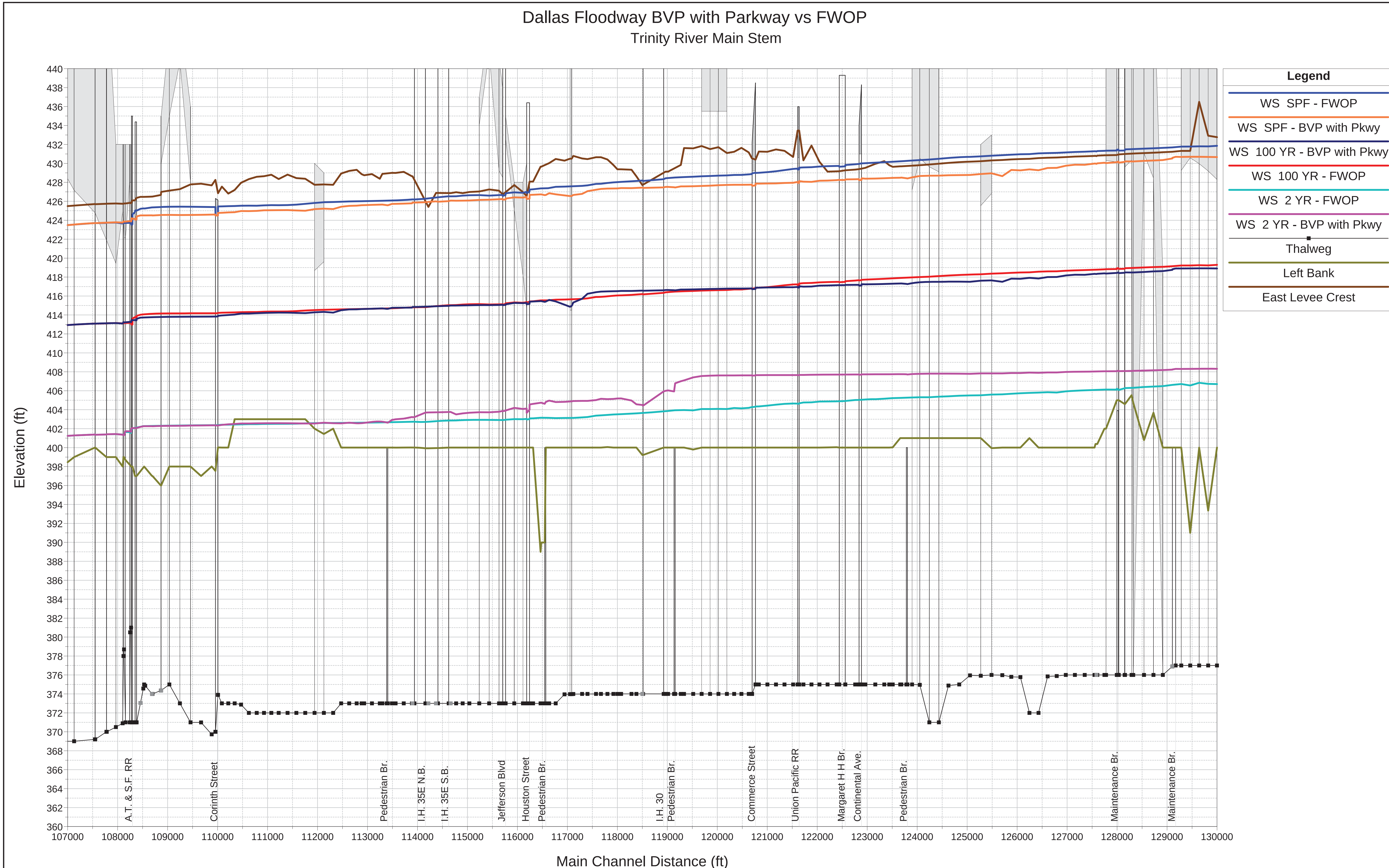
Figure 6-27



Dallas Floodway BVP w/o Parkway vs FWOP
Elm Fork Trinity River
Elevation (ft)
Main Channel Distance (ft)
Shady Grove Road
SH 356/Irving Bl
BNSF RR
SH 183
SH 482
Legend
WS SPF - FWOP
WS SPF - BVP wo Pkwy
WS 100 YR - FWOP
WS 100 YR - BVP wo Pkwy
WS 2 YR - FWOP
WS 2 YR - BVP wo Pkwy
Thalweg
Left Bank
East Levee Crest

Figure 6-28


Dallas Floodway BVP w/o Parkway vs FWOP
West Fork Trinity River
Legend
WS SPF - FWOP
WS SPF - BVP wo Pkwy
WS 100 YR - FWOP
WS 100 YR - BVP wo Pkwy
WS 2 YR - FWOP
WS 2 YR - BVP wo Pkwy
Thalweg
Left Bank
West Levee Crest
Loop 12
Elevation (ft)
Main Channel Distance (ft)
Figure 6-29



Dallas Floodway BVP with Parkway vs FWOP
Trinity River Main Stem
Legend
WS SPF - FWOP
WS SPF - BVP with Pkwy
WS 100 YR - BVP with Pkwy
WS 100 YR - FWOP
WS 2 YR - FWOP
WS 2 YR - BVP with Pkwy
Thalweg
Left Bank
East Levee Crest
Elevation (ft)
Main Channel Distance (ft)
A.T. & S.F. RR
Corinth Street
Pedestrian Br.
I.H. 35E N.B.
I.H. 35E S.B.
Jefferson Blvd
Houston Street
Pedestrian Br.
I.H. 30
Pedestrian Br.
Commerce Street
Union Pacific RR
Margaret H H Br.
Continental Ave.
Pedestrian Br.
Maintenance Br.
Maintenance Br.

Figure 6-30

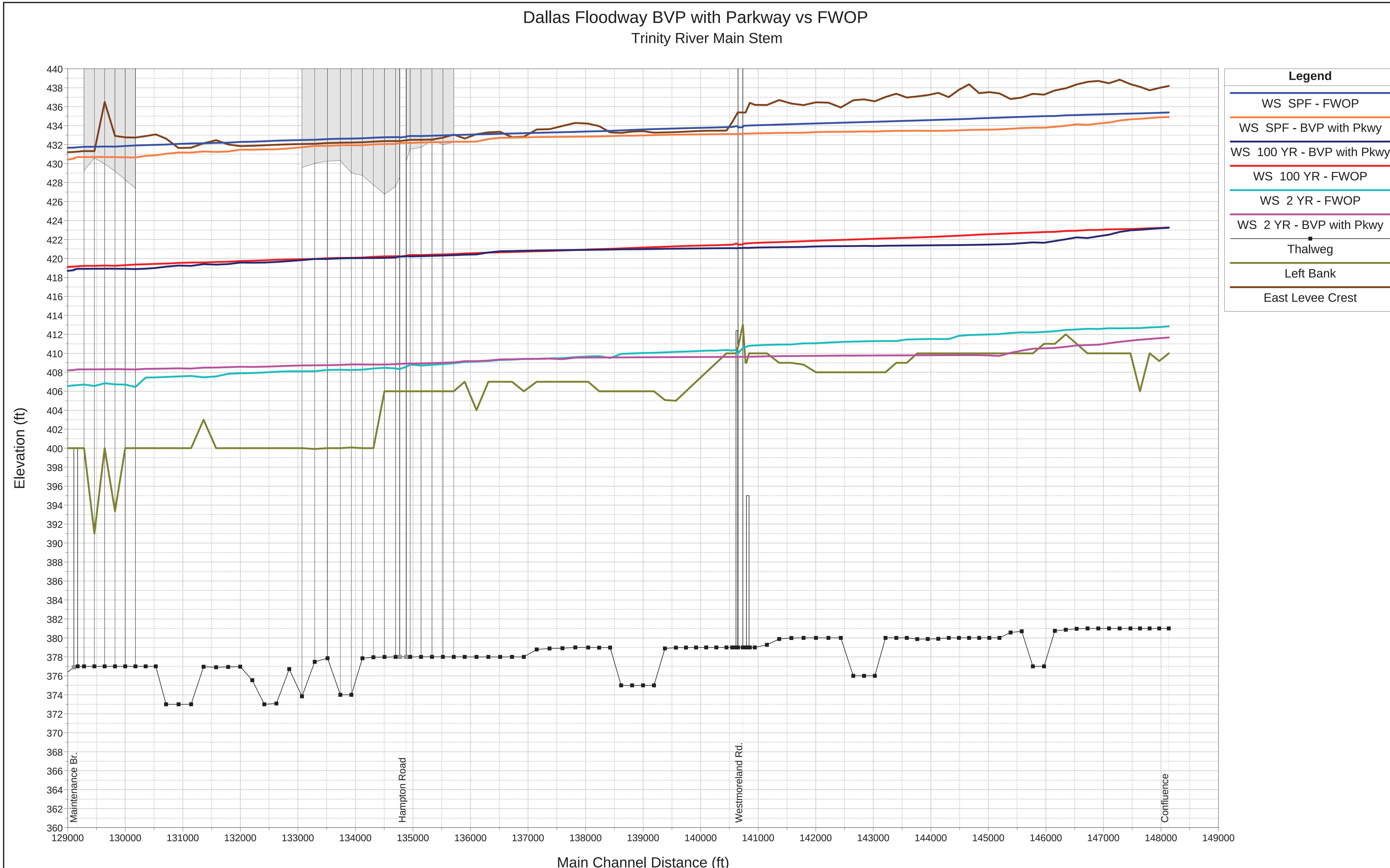
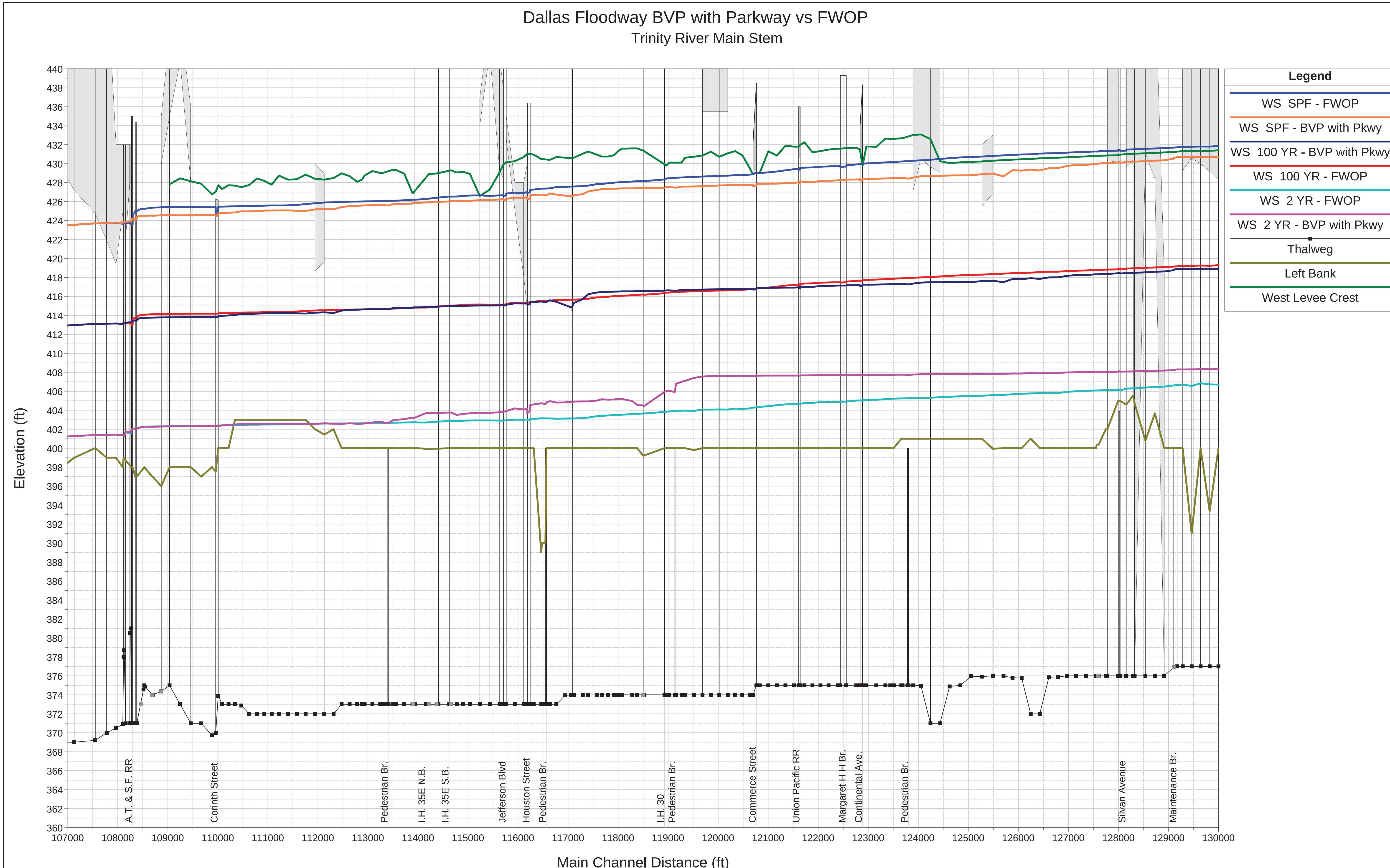
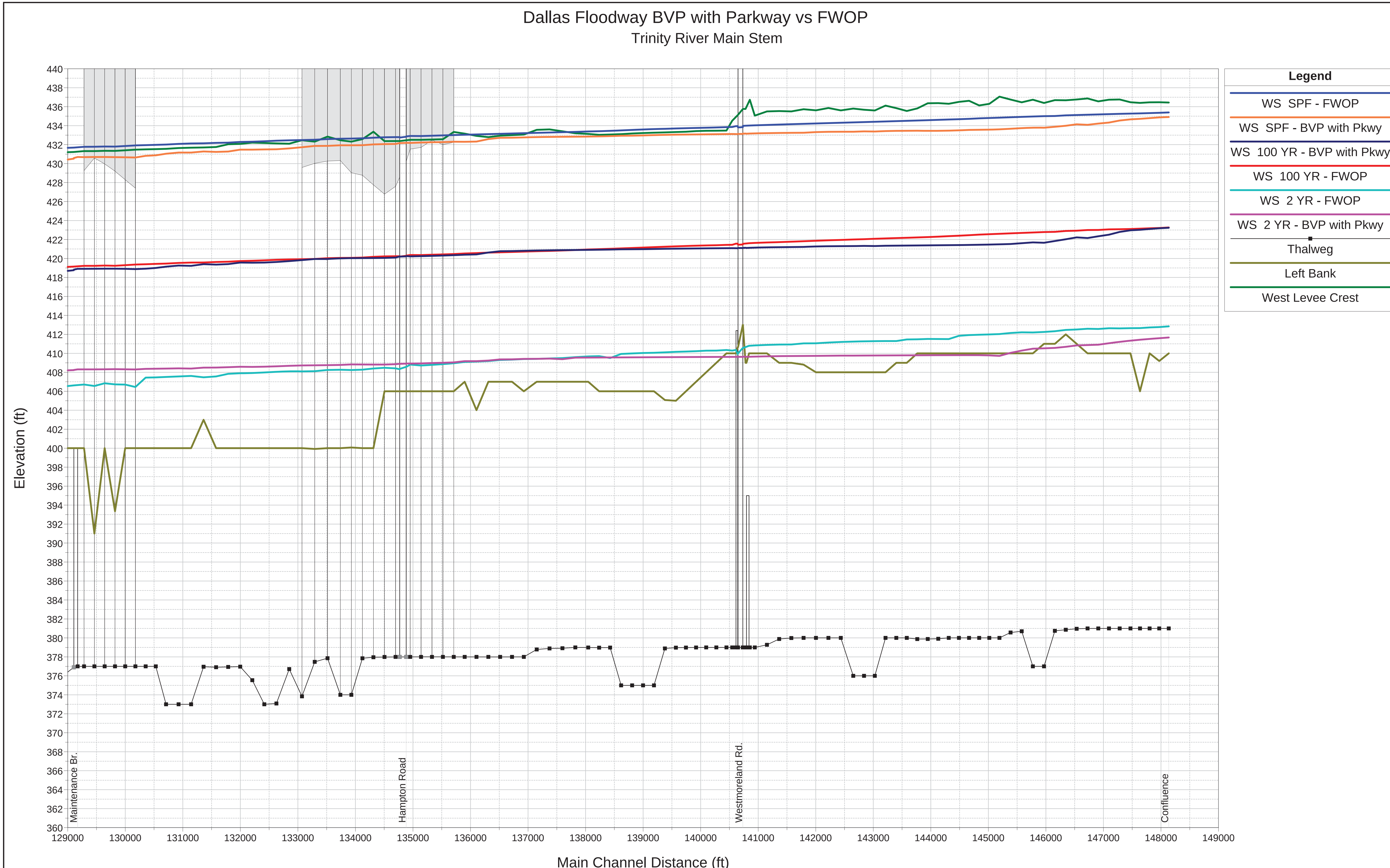

Dallas Floodway BVP with Parkway vs FWOP
Trinity River Main Stem
Elevation (ft)
Main Channel Distance (ft)
Maintenance Br.
Hampton Road
Westmoreland Rd.
Confluence
Legend
WS SPF - FWOP
WS SPF - BVP with Pkwy
WS 100 YR - BVP with Pkwy
WS 100 YR - FWOP
WS 2 YR - FWOP
WS 2 YR - BVP with Pkwy
Thalweg
Left Bank
East Levee Crest

Figure 6-31



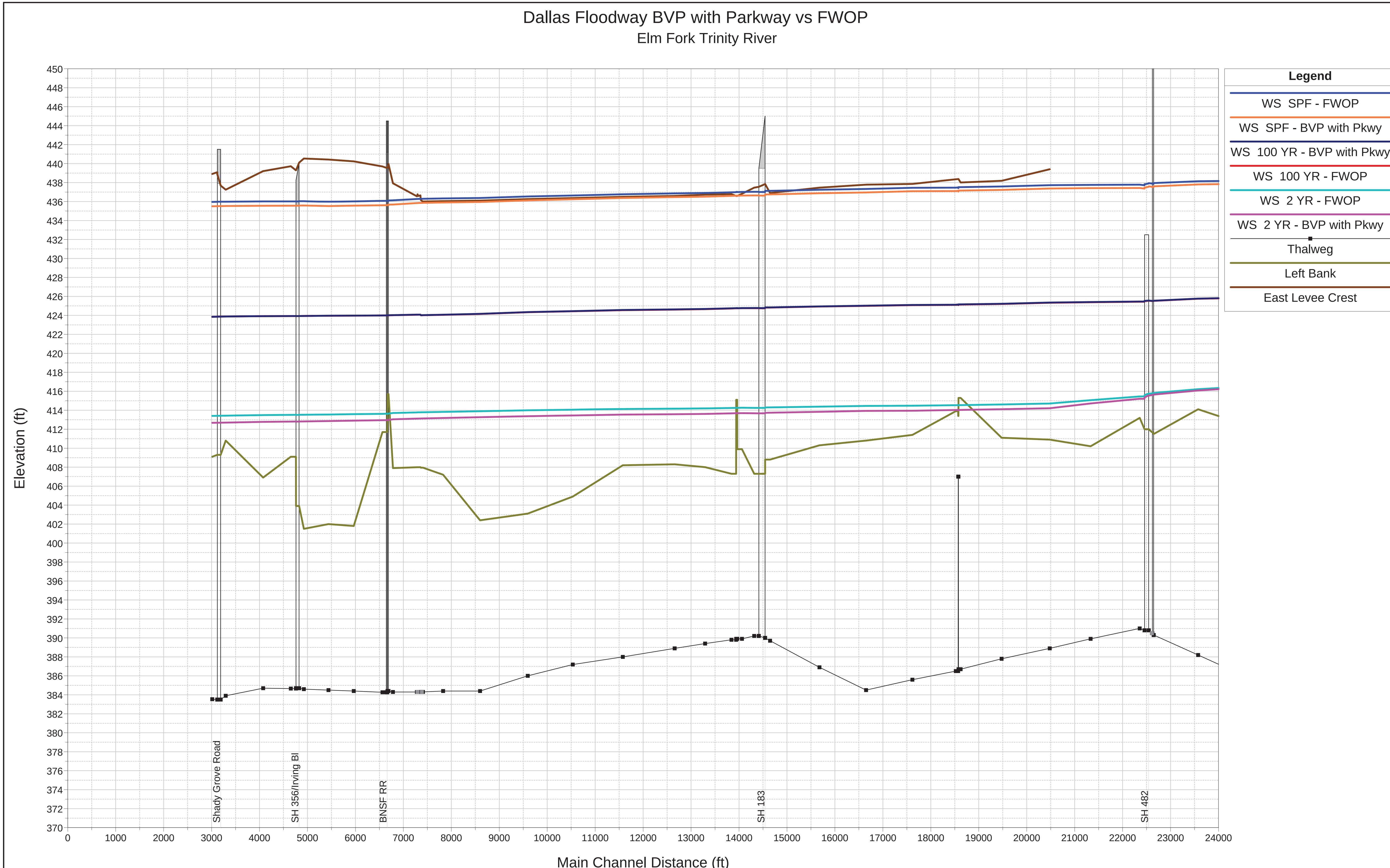
Dallas Floodway BVP with Parkway vs FWOP
Trinity River Main Stem
Elevation (ft)
Main Channel Distance (ft)
Legend
WS SPF - FWOP
WS SPF - BVP with Pkwy
WS 100 YR - BVP with Pkwy
WS 100 YR - FWOP
WS 2 YR - FWOP
WS 2 YR - BVP with Pkwy
Thalweg
Left Bank
West Levee Crest
A.T. & S.F. RR
Corinth Street
Pedestrian Br.
I.H. 35E N.B.
I.H. 35E S.B.
Jefferson Blvd
Houston Street
Pedestrian Br.
I.H. 30
Pedestrian Br.
Commerce Street
Union Pacific RR
Margaret H H Br.
Continental Ave.
Pedestrian Br.
Silvan Avenue
Maintenance Br.

Figure 6-32



Dallas Floodway BVP with Parkway vs FWOP
Trinity River Main Stem
Elevation (ft)
Main Channel Distance (ft)
Legend
WS SPF - FWOP
WS SPF - BVP with Pkwy
WS 100 YR - BVP with Pkwy
WS 100 YR - FWOP
WS 2 YR - FWOP
WS 2 YR - BVP with Pkwy
Thalweg
Left Bank
West Levee Crest
Maintenance Br.
Hampton Road
Westmoreland Rd.
Confluence

Figure 6-33


Dallas Floodway BVP with Parkway vs FWOP
Elm Fork Trinity River
Legend
WS SPF - FWOP
WS SPF - BVP with Pkwy
WS 100 YR - BVP with Pkwy
WS 100 YR - FWOP
WS 2 YR - FWOP
WS 2 YR - BVP with Pkwy
Thalweg
Left Bank
East Levee Crest
Elevation (ft)
Main Channel Distance (ft)
Shady Grove Road
SH 356/Irving Bl
BNSF RR
SH 183
SH 482
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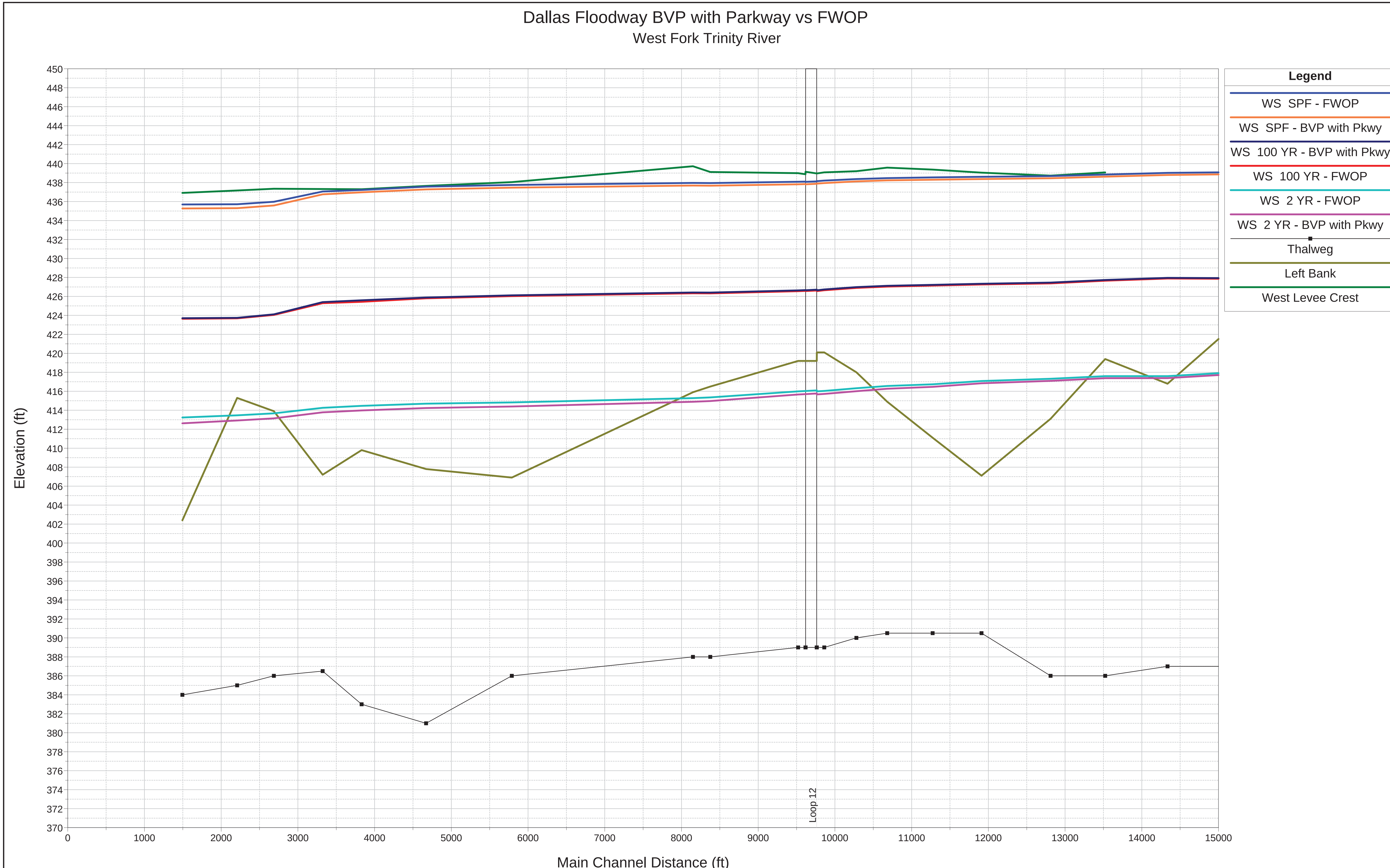
24000

Figure 6-34



Dallas Floodway BVP with Parkway vs FWOP
West Fork Trinity River
Elevation (ft)
Main Channel Distance (ft)
Loop 12
Legend
WS SPF - FWOP
WS SPF - BVP with Pkwy
WS 100 YR - BVP with Pkwy
WS 100 YR - FWOP
WS 2 YR - FWOP
WS 2 YR - BVP with Pkwy
Thalweg
Left Bank
West Levee Crest

Figure 6-35

Findings and Recommendations Regarding Hydrology for the Dallas Floodway Project

US Army Corps of Engineers
BUILDING STRONG®

Final Report

29 May 2012

Contents

Executive Summary

The Corps of Engineers Committee on Hydrology met with members of the Fort Worth District and Southwestern Division via webinar on 1 March 2012, 8 March 2012, and 16 March 2012 to discuss the hydrology for the Dallas Floodway Project. Additional informal communications have also taken place via e-mail and telephone subsequent to the formal webinar meetings. The purpose of this report is to summarize the findings and recommendations of the Committee on Hydrology regarding the issues discussed during these meetings. Recommendations are focused on the following issues: 1) determination of the standard project flood hydrograph and peak discharge estimate, 2) application of depth area duration relationships to frequency rainfall events, 3) frequency analysis for the period of record data, 4) selection of the peak discharge frequency relationship, 5) estimation of the return period for the standard project flood peak discharge, and 6) determination of interior versus exterior inundation relationships for estimation of consequences.

Committee on Hydrology

Engineer Regulation 15-2-14 prescribes the objectives, composition, and responsibilities of the Committee on Hydrology. The purpose of the Committee on Hydrology is to render consulting services on specific problems as requested by various elements of the Corps of Engineers. The Committee on Hydrology assists field elements in defining problems, developing plans for solutions to problems, and identifying appropriate expertise to perform necessary investigations and studies. The Committee on Hydrology makes recommendations on specific issues having important bearing on project design or investigation programs. The Committee on Hydrology recommendations are advisory rather than directive in nature. The Committee on Hydrology members are recognized national experts in the field of hydrologic engineering. The members of the Committee on Hydrology are listed below.

- Jeff Harris (HEC) – Committee Chair
- Doug Clemetson (NWO)
- Patrick Foley (MVP)
- David Margo (RMC)
- Tamara Masong (SPA)
- Sean Smith (SAJ)
- Glendon Stevens (NAP)
- Cary Talbot (ERDC)
- Jerry Webb (HQ)
- David Williams (SWT)

- Michael Wong (POH)

Determination of the Standard Project Flood

USACE policy for determination of standard project flood estimates is provided by Engineer Regulation 1110-2-1464. For projects located east of the 105th meridian, such as the Dallas Floodway, the policy requires that the standard project flood estimate be developed using the procedures described in Engineer Manual 1110-2-1411. For projects located west of the 105th meridian, the standard project flood may be estimated as 50% of the probable maximum flood. Southwestern Division policy documented in CESWD-ED-WH memorandum dated 14 December 1992 conforms to these USACE policy requirements.

The Dallas Floodway project was originally designed based on a standard project flood estimate (as documented in the 1952 Definite Project Report) for a drainage area of about 6100 mi^2 with a 24 hour basin average rainfall depth of about 8.6 inches for a storm that was centered over the 1300 mi^2 of uncontrolled drainage area. The resulting peak discharge was about 226,000 cfs on the Trinity River below the confluence of the West and Elm Forks. The original standard project flood for the project was developed prior to publication of Engineer Manual 1110-2-1411; however, it was judged that the methods used were reasonably comparable and any differences would have a negligible impact on peak discharge estimates.

The current Dallas Floodway Project is based on a revised standard project flood estimate (as documented in the Dallas Floodway Extension General Reevaluation Report) with a peak discharge of about 269,000 cfs at the Dallas gage located near Commerce Street (USGS 08057000). The current standard project flood estimate is based on a total rainfall amount equal to 50% of the probable maximum precipitation determined in accordance with Hydrometeorological Reports 51 and 52. The storm, which was centered over the uncontrolled drainage area, had a 72 hour basin average rainfall depth of about 11.2 inches.

The Hydrologic Engineering Center and Risk Management Center estimated the standard project flood under current hydrologic conditions using the procedures described in Engineer Manual 1110-2-1411. The peak discharge for this estimate was approximately 245,000 cfs which is an increase of about 19,000 cfs (8% increase) compared to the 1952 estimate and a decrease of about 24,000 cfs (9% decrease) compared to the current estimate.

The Committee on Hydrology recommends that the current standard project flood estimate based on 50% of the probable maximum precipitation with peak discharge of about 269,000 cfs be used for the Dallas Floodway Project. Since this approach is not in conformance with the requirements of Engineer Regulation 1110-2-1464, the Committee on Hydrology recommends that the Fort Worth District initiate a policy waiver request to formally obtain and document approval of the approach. This recommendation is based on the fact that the Engineer Manual for standard project flood determinations is outdated and that probable maximum precipitation estimates are more current. The approach results in a standard project flood estimate that is more conservative than the normal procedure, but the results appear to be reasonable and supportable. The recommended standard project flood hydrograph is shown in Figure 1.

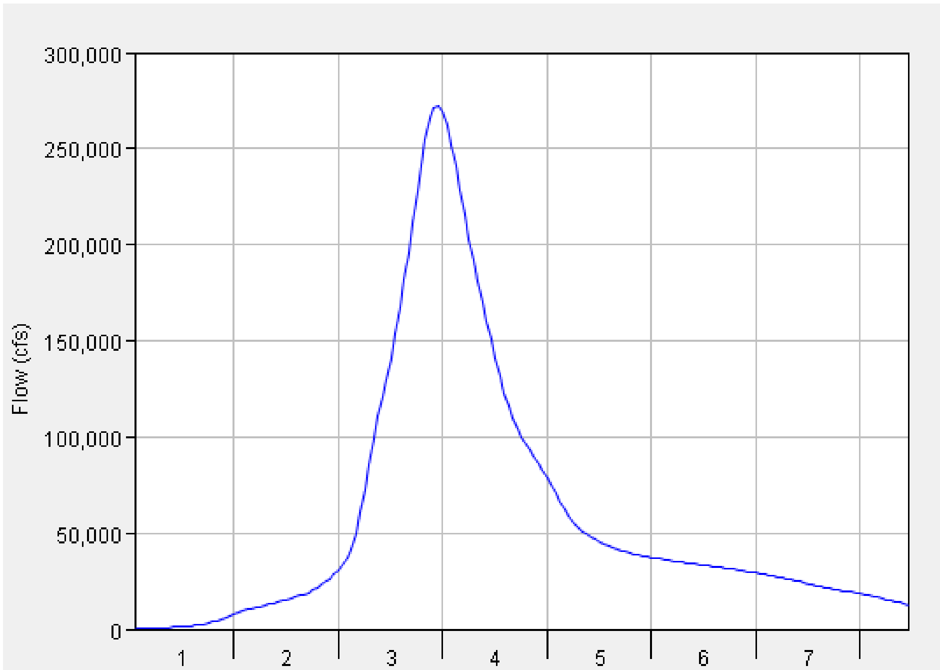


Figure 1 – Standard Project Flood Hydrograph at Dallas gage (USGS 08057000)

Depth Area Duration Relationships

It is common practice to simulate frequency based precipitation events using a hydrologic model to obtain a peak discharge estimate for a given frequency flood. This information helps to inform selection of the peak discharge frequency relationship particularly for events beyond the period of record. The approach is described in Chapter 13 of Engineer Manual 1110-2-1417.

Point rainfall estimates can be obtained from publications such as NOAA Technical Paper 40, NOAA Technical Paper 49, and NOAA Atlas 14. A balanced hyetograph approach can be used to obtain a temporal distribution of precipitation for the simulated precipitation event. The spatial distribution can be obtained by applying appropriate depth area duration relationships. Figure 13-1 in EM 1110-2-1417, which is a reproduction of Figure 15 in Technical Paper 40, is one such relationship. The computer software HEC-1 and HEC-HMS both implement this relationship. The software also implements an index hydrograph methodology to simplify the process needed to obtain consistent frequency estimates at multiple sites within a basin.

Frequency based precipitation events were simulated for the Dallas Floodway Extension General Reevaluation Report using HEC-1 in order to inform development and selection of the peak discharge frequency function. The hydrologic model utilized the TP40 based depth area duration functions in HEC-1 and the index hydrograph methodology. This approach overestimated the rainfall and resulting peak discharges by about 15-25% due to several contributing factors. First, it is not appropriate to use the index hydrograph method, which assumes a uniform spatial rainfall distribution, to model this watershed due to the size and the effects of regulation. The watershed has a drainage area of about 6100 mi^2 of which about 80% is significantly influenced by regulation. The index hydrograph method cannot spatially distribute the rainfall appropriately under these conditions. Second, the depth area duration relationships that were used are only valid for relatively small watersheds that are no more than a few hundred square miles in size. For larger watersheds, extrapolation of the TP40 depth area duration relationships results in an implicit assumption that rainfall depth remains constant with increasing area size. This assumption is not supported by known observations, physical phenomenon, or scientific theory. Figure 2 provides examples of several published depth area duration relationships. Note that all of the relationships, except for TP40, show a similar trend of decreasing rainfall depth with increasing area. The extrapolation of the TP40 relationship is shown as a dashed line because the National Oceanographic and Atmospheric Administration does not support extrapolation of the TP40 depth area duration relationship beyond 400 square miles. It is important to note that the HMR52 depth area duration relationships cannot be directly estimated from the all season PMP values in Figures 18 through 47 of HMR 51. Use of these values is not correct and will underestimate the rainfall and resulting peak discharge. Proper HMR52 depth area duration relationships can be obtained by post processing output from the HMR52 program.

The Committee on Hydrology applied the HMR52 depth area duration relationships to the hydrologic model to obtain a more appropriate spatial and temporal distribution of rainfall for the frequency based precipitation events. This was accomplished by estimating a ratio of the point precipitation values for each frequency event to the point PMP precipitation values. These ratios were computed for the 1, 2, 3, 6, 12, 24, 48, and 72 hour durations. A sample of the computations for the 6 hour duration is provided in Table 1.

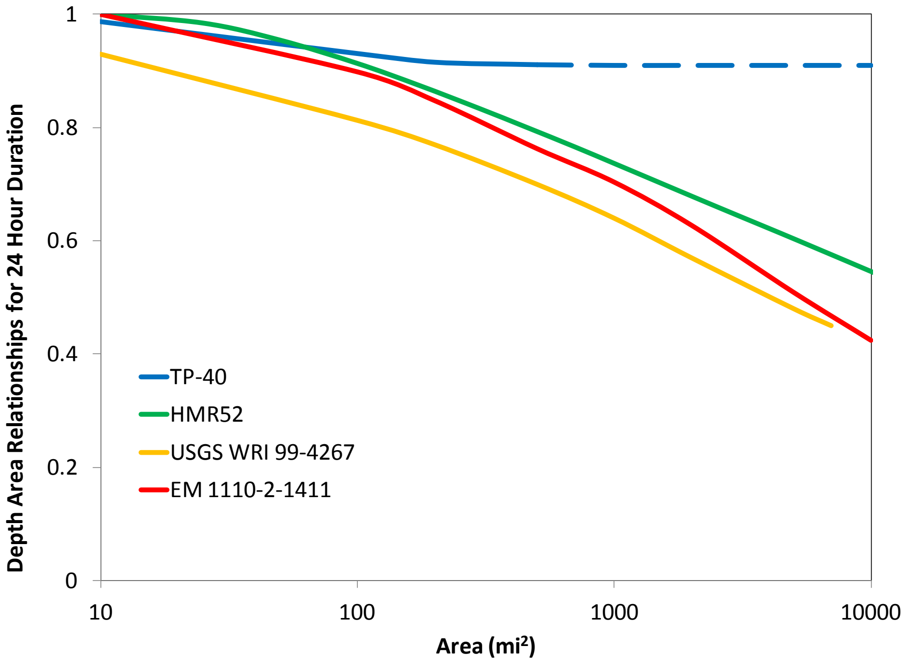


Figure 2 – Example Depth Area Duration Relationships

Table 1 – Example PMP Ratio Computation for 6 Hour Duration

Return Period (Year)	Point Precipitation for 6 Hour Duration	Ratio of Frequency Precipitation to PMP
2	3.0	3.0 / 24.4 = 0.12
5	3.9	3.9 / 24.4 = 0.16
10	4.7	4.7 / 24.4 = 0.19
25	5.3	5.3 / 24.4 = 0.22
50	6.2	6.2 / 24.4 = 0.25
100	6.9	6.9 / 24.4 = 0.28
500	9.8	9.8 / 24.4 = 0.40
PMP	24.4	n/a

Ratios for the 1, 2, and 3 hour durations had to be further adjusted to maintain consistency between the 1 to 6 hour precipitation ratio for the frequency precipitation events and the 1 to 6 hour precipitation ratio for the PMP. This is necessary because the 1 to 6 hour ratio from TP 40 is about 0.63 and the 1 to 6 hour ratio from HMR 52 is about 0.33. Failure to make this adjustment will bias the computed ratios and the results will not be correct or consistent. After making the proper adjustments, a consistent ratio was obtained across all durations and for all frequency events. A sample of the adjustments for the 1 hour duration is provided in Table 2.

Table 2 – Example PMP Ratio Adjustment for 1 Hour Duration

Return Period (Year)	Point Precipitation for 1 Hour Duration	Point Precipitation for 6 Hour Duration	Ratio of 1 Hour Point Precipitation to PMP	Ratio of 1 Hour to 6 Hour Precipitation	PMP 1 to 6 Hour Ratio Divided by Frequency 1 to 6 Hour Ratio	Adjusted 1 Hour Ratio
2	1.9	3.0	1.9 / 8.2 = 0.23	1.9 / 3.0 = 0.63	0.33/0.63 = 0.53	0.53*0.23 = 0.12
5	2.5	3.9	2.5 / 8.2 = 0.31	2.5 / 3.9 = 0.64	0.33 / 0.64 = 0.53	0.53 * 0.35 = 0.16
10	2.9	4.7	2.9 / 8.2 = 0.35	2.9 / 4.7 = 0.62	0.33 / 0.62 = 0.54	0.54 * 0.35 = 0.19
25	3.4	5.3	3.4 / 8.2 = 0.41	3.4 / 5.3 = 0.64	0.33 / 0.64 = 0.53	0.53 * 0.41 = 0.22
50	3.8	6.2	3.8 / 8.2 = 0.47	3.8 / 6.2 = 0.62	0.33 / 0.62 = 0.54	0.54 * 0.47 = 0.25
100	4.3	6.9	4.3 / 8.2 = 0.53	4.3 / 6.9 = 0.62	0.33 / 0.62 = 0.54	0.54 * 0.53 = 0.28
500	6.1	9.8	6.1 / 8.2 = 0.75	6.1 / 9.8 = 0.62	0.33 / 0.62 = 0.54	0.54* 0.75 = 0.40
PMP	8.2	24.4	n/a	8.2 / 24.4 = 0.33	n/a	n/a

The ratios from Table 1 were then applied to the PMP using the HMR 52 program. The resulting temporal distribution of precipitation over the maximum 6 hour period had to be manually adjusted using HEC-DSSVue to maintain consistency with the maximum 6 hour distribution from TP40 because the HMR 52 program uses a different distribution for the maximum 6 hour period. The adjustment was made for completeness even though it did not affect the peak discharge estimates at the Dallas gage (USGS 08057000). A sample of the adjustments made for one of the sub basins is provided in Figure 3.

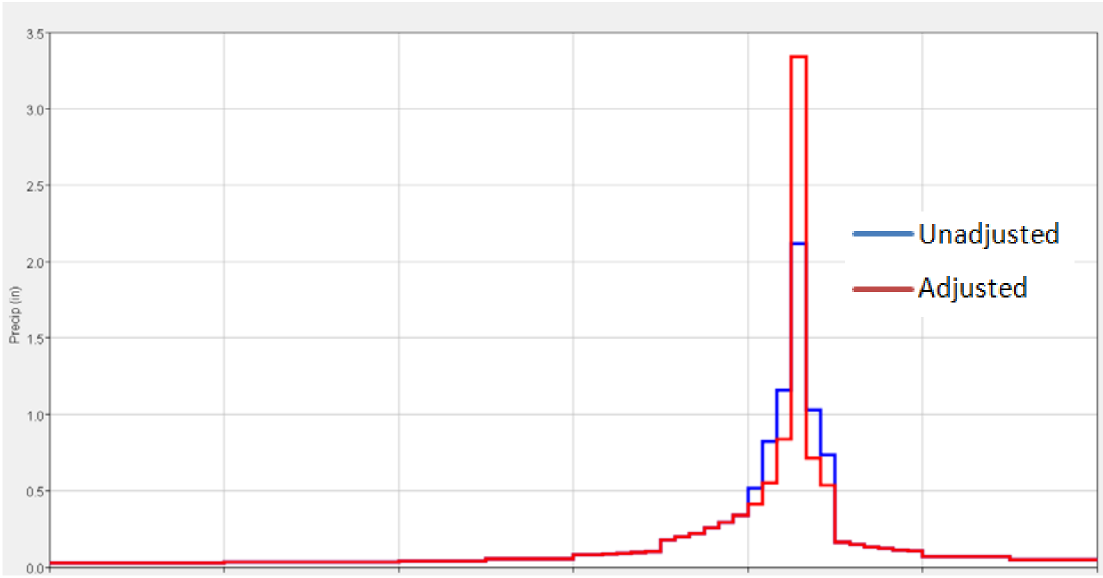


Figure 3 – Example Manual Adjustment for Maximum 6 Hour Rainfall Distribution

A summary of the recommended PMP ratios for each frequency event and the resulting peak discharge estimate is provided in Table 3. A comparison of the resulting depth area duration relationships showed reasonable agreement with TP40 for small area sizes and good agreement with HMR52 for all drainage area sizes across all frequencies and durations. An example of this comparison is provided in Figure 4 for the 6 hour duration and in Figure 5 for the 1 hour duration.

Table 3 – PMP Ratios and Peak Discharge Estimates

Return Period (Years)	PMP Ratio	Peak Discharge Estimate (cfs)
2	0.12	24,500
5	0.16	33,700
10	0.19	44,100
25	0.22	56,800
50	0.25	73,600
100	0.28	94,500
500	0.40	165,200

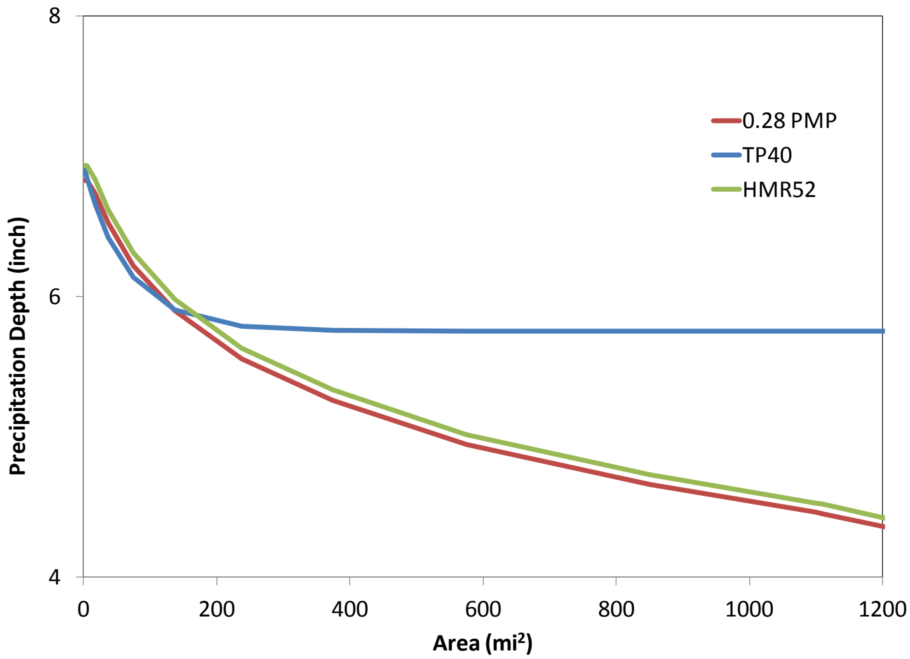


Figure 4 – Sample Depth Area Duration Relationship for 6 Hour Duration

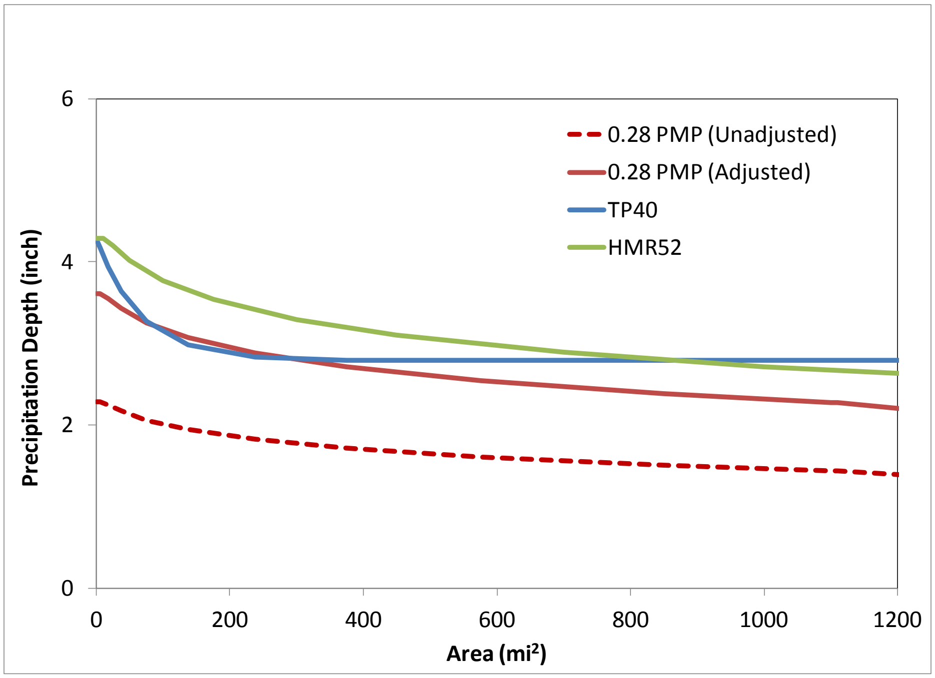


Figure 5 – Sample Depth Area Duration Relationship for 1 Hour Duration

Fort Worth District has proposed an alternate set of PMP ratios to represent the frequency based precipitation events. These ratios are based on a similar methodology; however, no adjustments were made to account for differences between TP40 and HMR52 with respect to the temporal distribution of the maximum 6 hour period and the spatial distribution of precipitation depth with increasing area. As a result, the computed ratios and resulting peak discharges are overestimated and inconsistent across the various frequencies, durations, and area sizes.

This overestimation is demonstrated in Figure 6 which provides a comparison between the depth area duration relationships obtained using the Fort Worth District ratios, the TP40 depth area duration relationship, and the HMR52 depth area duration relationship. Note that the Fort Worth Districts PMP ratio of 0.33 for the 100 year return period shows poor agreement with both TP40 and HMR52 across all drainage area sizes. Also note that precipitation depths obtained from the 0.33 PMP ratio for areas less than about 150 mi^2 are actually greater than the TP40 point precipitation values which is not physically possible. Similar inconsistencies in the results were observed across all of the frequency precipitation events for which Fort Worth District estimated PMP ratios.

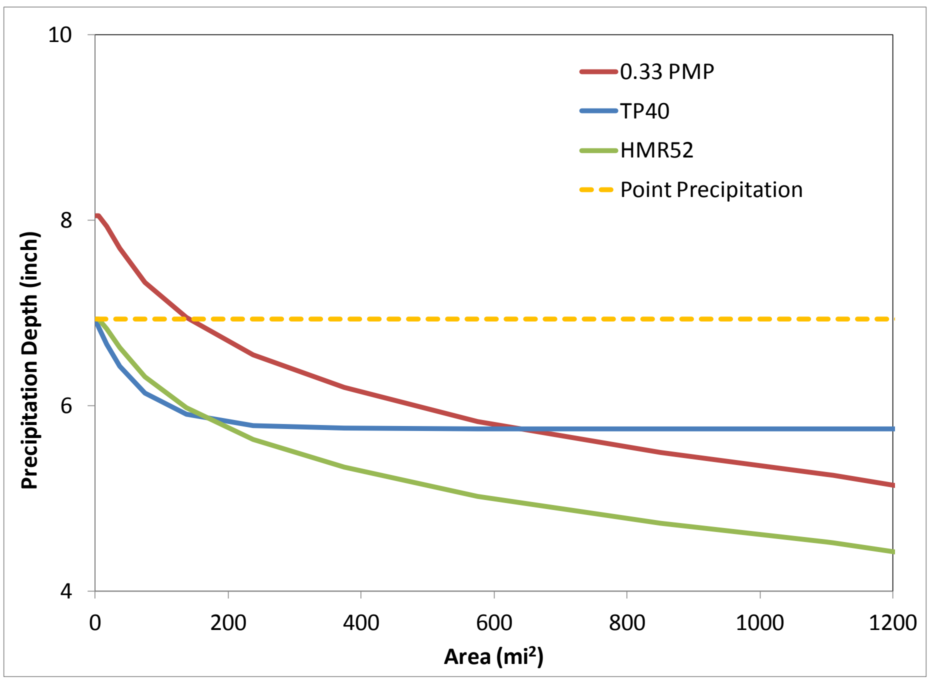


Figure 6 – Sample Depth Area Duration Relationship for SWF PMP Ratio Method

The Committee on Hydrology recommends that HMR52 depth area duration relationships be applied to obtain the spatial and temporal distribution of rainfall for the frequency based precipitation events. Additional adjustments for the temporal distribution of the maximum 6 hour period should also be made. The Committee on Hydrology recommends that the PMP ratios presented in Table 3 be adopted for use in HMR52 for computation of the spatial and temporal rainfall distribution. The resulting rainfall should be applied to the HEC-1 hydrologic model to obtain the peak discharge estimates in Table 3. The resulting peak discharges should be used to inform selection of the peak discharge frequency relationship. The Committee on Hydrology also supports initiating a new study to develop regional depth area duration relationships. Results of this study will be important to ensure that appropriate and consistent depth area duration relationships are available for the Dallas Floodway study and numerous other studies throughout the region. A sufficient number of storms of various sizes, durations, and magnitudes need to be evaluated to support adoption of regional depth area duration relationships. The Committee on Hydrology recommends that the study be subjected to an Agency Technical Review. Upon completion of the study, the Committee on Hydrology should be consulted to obtain a recommendation as to whether the proposed regional depth area duration relationships should be adopted for use.

Period of Record Frequency Analysis

It is common practice to use historic observations to inform selection of a peak discharge frequency relationship. Appropriate consideration must be given to issues such as record length, regulation effects, and homogeneity of the available data. Procedures for hydrologic frequency analysis are provided by Engineer Manual 1110-2-1415.

An analytical peak discharge frequency relationship at the Dallas gage (USGS 08057000) was developed for the 1995 General Reevaluation Report using a 38 year period of record and Bulletin 17B methods. The period of record includes water years 1955 through 1992 based on the substantial completion of upstream regulating projects which occurred in water year 1955 and the most current year for which data was available at the time of the study which was water year 1992. The analytical peak discharge frequency relationship has not been updated for the current study. Parameters for this relationship are a mean of 4.278, a standard deviation of 0.334, and a weighted skew of 0.1.

An analytical peak discharge frequency relationship at the Dallas gage (USGS 08057000) has been developed for a 57 year period of record using Bulletin 17B methods. The period of record includes water years 1955 through 2011 based on the substantial completion of upstream regulating projects which occurred in water year 1955 and the most recently available data from water year 2011. Parameters for this relationship are a mean of 4.310, a standard deviation of 0.302, and a weighted skew of -0.1.

A comparison of the two analytical frequency relationships is presented in Figure 7.

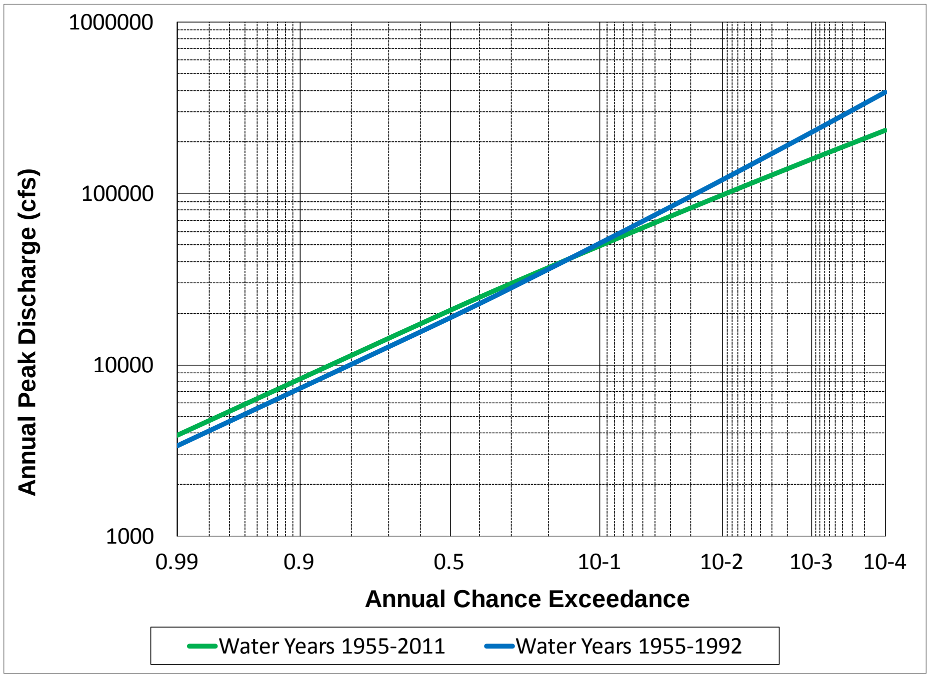


Figure 7 – Comparison of Analytical Peak Discharge Frequency Relationships

The Committee on Hydrology recommends that the full period of record of readily available data for water years 1955 through 2011 be used for the period of record frequency analysis. The Committee on Hydrology questions the use of regulated discharges with Bulletin 17B analytical methods for development of the peak discharge frequency relationship. Bulletin 17B procedures are only applicable to discharges that are not appreciably altered by reservoir regulation. Fort Worth District believes that reservoir regulation does not significantly affect the shape of the peak discharge frequency relationship at the Dallas gage (USGS 08057000). The Committee on Hydrology recommends that further analyses be undertaken to provide evidence in support of this claim. The Committee on Hydrology recommends that the period of record data be adjusted to reflect unregulated conditions and that an unregulated peak discharge frequency relationship be developed. A regulated versus unregulated relationship should then be applied to obtain a regulated peak discharge frequency relationship. This relationship can then be used to inform selection of the peak discharge frequency relationship. Results of this analysis will be important for assessing the shape of the frequency relationship for the portion that extends beyond the period of record data.

Average annual discharges have roughly doubled in magnitude over time due to changes within the watershed. The urbanization impacts on peak discharges for less frequent floods is likely to be less significant. In typical cases, urbanization results in an increase in the mean and a decrease in the standard deviation of the annual peak discharges. This results in an increase in peak discharges for more frequent events. Changes in the peak discharges for less frequent events tend to be less significant. Results obtained from the hydrologic model, which has been calibrated to reflect calendar year 2000 urbanization conditions, show reasonable agreement with historic observations for water years 1955 through 2011. Comparison of the hydrologic modeling results with the period of record data supports the claim that urbanization effects may not be significant. There are, however, some uncertainties and unknowns regarding the potential impacts of urbanization on peak discharge frequency estimates. The Committee on Hydrology supports initiating a new study to investigate urbanization impacts. Results of this study will be important to ensure that urbanization impacts on the peak discharge frequency relationships are appropriately addressed for the Dallas Floodway study and numerous other studies throughout the region. The study should be conducted using methods that are consistent with the guidelines in Engineer Manual 1110-2-1415. The Committee on Hydrology recommends that the study be subjected to an Agency Technical Review. Upon completion of the study, the Committee on Hydrology should be consulted to obtain a recommendation as to how urbanization impacts should be considered when selecting a peak discharge frequency relationship.

Peak Discharge Frequency Relationship

USACE guidance for selecting a peak discharge frequency relationship is provided by Engineer Manual 1110-2-1417. The recommended approach is to develop discharge frequency estimates using multiple methods. This approach has been applied to the Dallas Floodway Project using period of record analysis and frequency rainfall hydrologic modeling methods.

Additional information to support selection of the peak discharge frequency relationship can be obtained from an estimate of the precipitation frequency for the standard project flood event. Figure 8 summarizes the frequency associated with the point rainfall for various precipitation events. Based on this figure, the standard project flood precipitation frequency is estimated to have an annual chance exceedance of about 0.00033 (3000 year return period).

The Committee on Hydrology recommends that the analytical peak discharge frequency relationship for water years 1955 through 2011 with an adopted skew of 0.1 be used for purposes of the feasibility scoping meeting. The adopted skew value of 0.1 is recommended to provide better agreement with the hydrologic modeling results for flood events that are beyond the period of record. This peak discharge frequency relationship reflects the results obtained by resolving the issues related to the limited record length and application of proper depth area duration relationships. This relationship, which is summarized in Figure 9, has a mean of 4.310, a standard deviation of 0.302, and a skew of 0.1. This relationship represents the best estimate based on the current understanding of the hydrology.

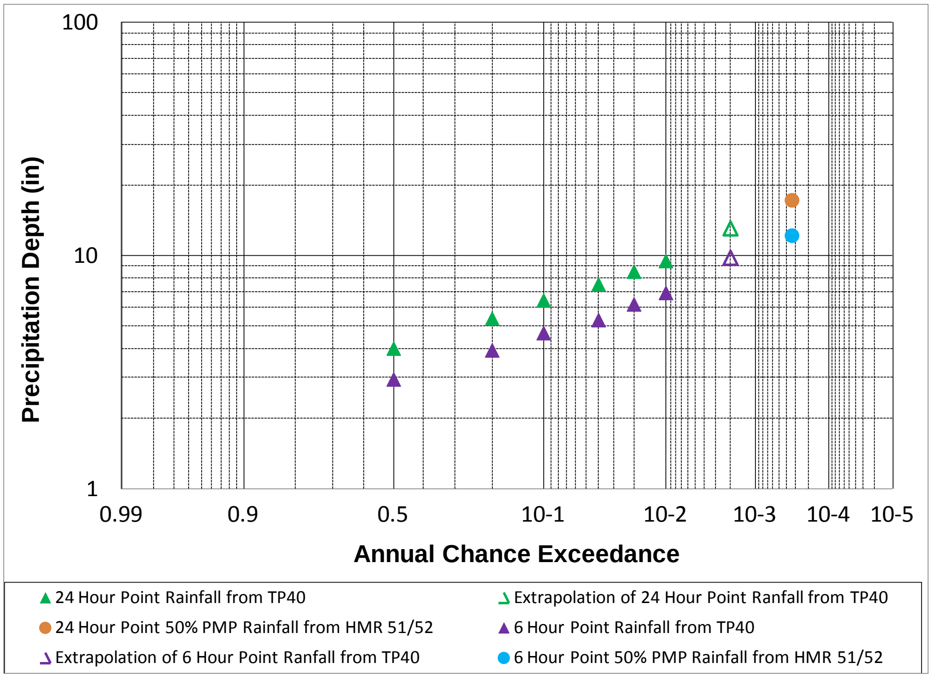


Figure 8 – Precipitation Frequency Estimates

The Committee on Hydrology recommends that the analytical peak discharge frequency relationship for water years 1955 through 1992 be evaluated as a sensitivity case for purposes of the feasibility scoping meeting recognizing that this relationship may be somewhat conservative due to the limited record length and overestimation of the frequency rainfall peak discharge estimates. This relationship, which is summarized in Figure 10, has a mean of 4.278, a standard deviation of 0.334, and a skew of 0.1.

The Committee on Hydrology recommends that selection of a final peak discharge freqeuncy relationship for the Dallas Floodway Project be deferred until completion of the unregulated frequency analysis, the study on regional depth area duration relationships, and the study on urbanization impacts. The Committee on Hydrology should be consulted to obtain a recommendation regarding adoption of a final peak discharge frequency relationship.

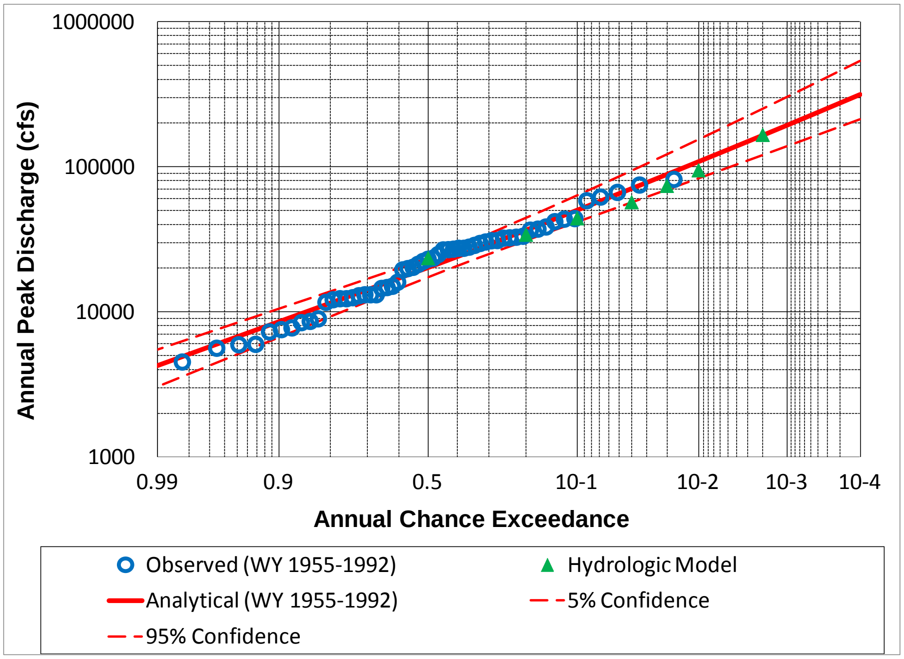


Figure 9 – Analytical Frequency Relationship for Water Years 1955 Through 2011 Based on Committee on Hydrology Analyses

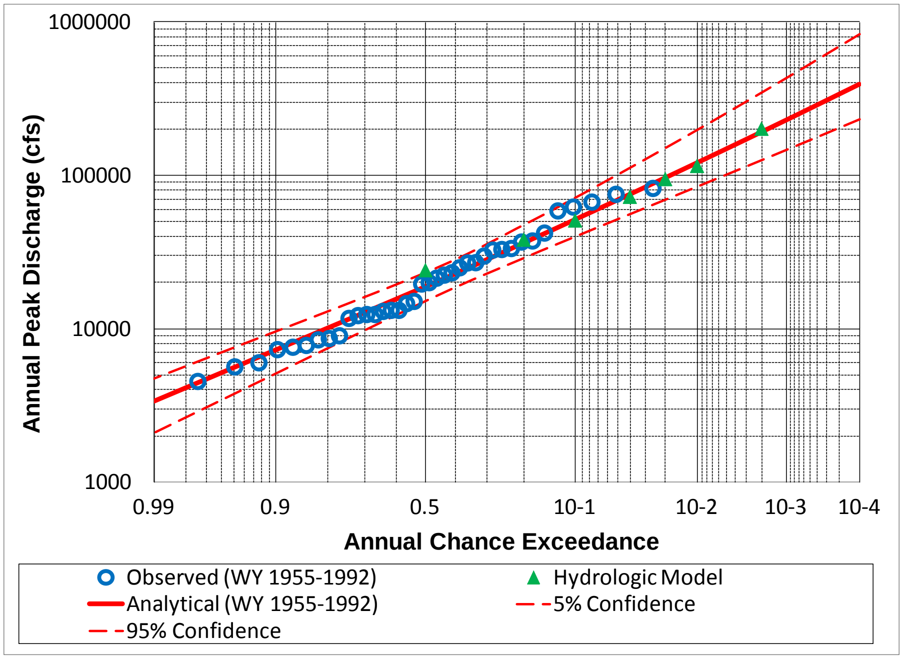


Figure 10 – Analytical Frequency Relationship for Water Years 1955 Through 1992 Based on General Reevaluation Report

Standard Project Flood Frequency Estimate

The impact of uncertainty due to small sample size can be quantified with the expected probability adjustment. This adjustment is based on the argument that an annual chance exceedance discharge estimate made with a given sample represents the median estimate that would be obtained with successive samples of the same size. However, the probability distribution of the estimate is skewed, so the average (expected value) of the samples exceeds the median estimate. The consequence of this is that if a very large number of estimates of flood magnitude are made over a region, more annual chance floods will occur than expected. For example, more 100 year floods would occur in the United States on the average than would be expected if the median estimate is used.

Prior to USACE adopting explicit probability and uncertainty analysis methods in the mid 1990s, it was standard practice to communicate flood frequencies in terms of their expected probability to ensure that the average number of flood occurrences would be appropriately characterized and considered in the formulation and design of projects. When the expected probability adjustmend is applied to the frequency relationship in Figure 10, the expected annual chance exceedance probability for the standard project flood peak discharge of 269,000 cfs would be about 0.00125 (800 year return period). Current USACE practice, which has been in place since the mid 1990s, is to use the median probability estimate to support analyses such as those performed using HEC-FDA. The expected probability adjustment is not used for the inputs to these analyses because the bias in the probability distribution is already explicitly accounted for in the uncertainty sampling and computations. This approach is described in various Engineer Manuals including 1110-2-1619. The analytical frequency relationship in Figure 10 would yield a median annual chance exceedance probability for the standard project flood peak discharge of 269,000 cfs of about 0.0005 (2000 year return period). The estimates are summarized in Figure 11 for both the expected and median estimates. Both estimates (800 year for expected probability and 2000 year for median probability) are technically correct as long as they are characterized, communicated, and applied in the proper context. For the current study and consistent with current USACE policy, which requires explicit probability and uncertainty analysis methods, the median return period estimate of 2000 years is more appropriate based on the data in Figure 9.

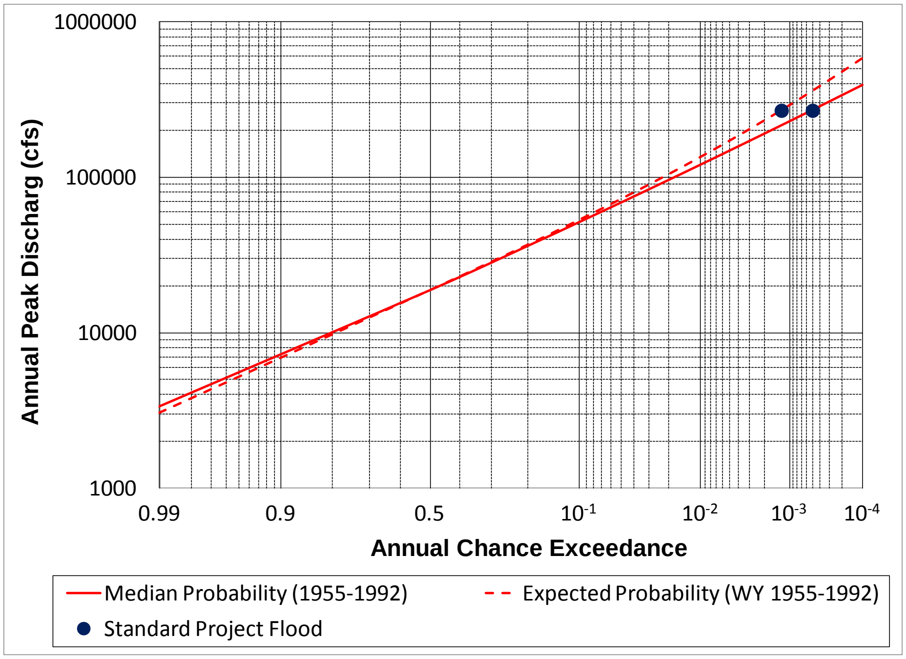


Figure 11 – Standard Project Flood Frequency Based on General Reevaluation Report Frequency Relationship (Water Years 1955 – 1992)

The analytical frequency relationship in Figure 9 would yield a median annual chance exceedance probability for the standard project flood peak discharge of 269,000 cfs of about 0.00022 (4500 year return period). This estimate is summarized in Figure 12.

The Hydrology Committee recommends that the median estimate be used to characterize the return period for the standard project flood peak discharge estimate and that this estimate be used for purposes of HEC-FDA and similar probabilistic analyses. For purposes of the feasibility scoping meeting, the Committee on Hydrology recommends a 4500 year return period estimate for the base condition consistent with Figures 10 and 12. A 2000 year return period estimate consistent with Figures 9 and 11 should be evaluated as a sensitivity case. The Committee on Hydrology recommends that selection of a final estimate be deferred until selection of a final frequency relationship is made based on the results of the unregulated frequency analysis, the regional depth area duration relationship study, and the urbanization impact study.

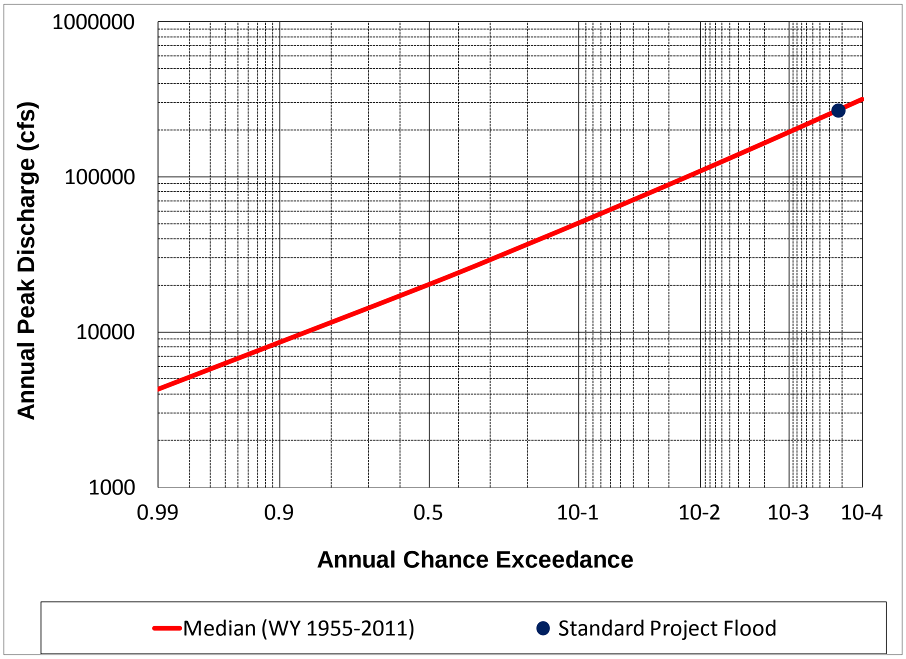


Figure 12 – Standard Project Flood Frequency Based on Committee on Hydrology Relationship (Water Years 1955 – 2011)

Inundation Depth for Estimating Consequences

Analyses conducting using HEC-FDA assume that breach and/or capacity exceedance results in inundation of the leveed area. HEC-FDA does not explicitly distinguish between breach prior to overtopping, overtopping with breach, and overtopping without breach inundation scenarios. The peak water surface elevation for the resulting inundation of the leveed area is determined by an interior versus exterior function provided by the user or by a default assumption that the resulting inundation elevation of the leveed area is equal to the water surface elevation in the river. Consequences are then estimated using a peak water surface elevation versus consequence function.

The HEC-FDA model for the Dallas Floodway Project does not conform to standard practices regarding the use of interior versus exterior inundation relationships. As a result, the consequence estimates may be overestimated and somewhat conservative. It is also difficult to review and judge the credibility of the analysis due to the non standard approach. The Committee on Hydrology recommends using the existing HEC-FDA analyses for the feasibility scoping meeting recognizing that the consequence estimates may be somewhat conservative. The Committee on Hydrology recommends that the Fort Worth District consult with the Mapping, Modeling, and Consequence Production Center (POC Ron Goldman) to develop unsteady flow hydraulic models for use in developing interior versus exterior relationships. These relationships should be used in the HEC-FDA model for the final feasibility study analyses. The Committee on Hydrology recommends adapting existing HEC-RAS models which have previously been developed to support the feasibility study and the base condition risk assessment if the MMC agrees that it is practical to do so.

Summary of Findings and Recommendations

The Committee on Hydrology provides the following summary of findings and recommendations to address concerns related to the hydrology for the Dallas Floodway Project.

1) The standard project flood estimate should be based on 50% of the probable maximum precipitation developed in accordance with Hydrometeorological Reports 51 and 52. Fort Worth District should initiate a policy waiver request to obtain and document formal approval for this approach.

2) The HEC-1 hydrologic model appears to be reasonably calibrated with respect to the transformation of rainfall to runoff based on Fort Worth District's evaluation of several historic events. It is recommended that the model be converted to HEC-HMS which is now the preferred USACE model format. Continued use of the current model parameters is recommended. Changes should be made in terms of how rainfall is applied as an input to the model. Committee on Hydrology recommendations regarding the spatial and temporal distribution and magnitude of rainfall should be adopted for use in the application of the model to obtain discharge hydrographs and peak discharge estimates at locations throughout the watershed. Specifically, the index hydrograph method should not be used and the depth area duration relationships should be based on Hydrometeorological Report 52. Regional depth area duration relationships could also be used pending the results of additional study.

3) The depth area duration relationships from Hydrometeorological Report 52 should be used for simulation of frequency rainfall events to obtain frequency peak discharge estimates. The PMP ratios developed by the Committee on Hydrology should be used. The PMP ratios developed by the Fort Worth District should not be used.

4) Additional study to investigate regional depth area duration relationships is appropriate and supported by the Committee on Hydrology. Upon completion of the study, the Committee on Hydrology should be consulted to obtain a recommendation as to whether the regional depth area relationship should be adopted for use.

5) A regulated versus unregulated peak discharge relationship should be developed at key locations within the watershed. This relationship should be used to obtain unregulated annual peak discharge estimates for water years 1955 through 2011. An unregulated peak discharge frequency relationship should be developed using Bulletin 17B procedures for this period of record. The regulated versus unregulated relationship should then be applied to obtain a regulated peak discharge frequency relationship.

6) Additional study to investigate urbanization impacts is appropriate and supported by the Committee on Hydrology. Upon completion of the study, the Committee on Hydrology should be consulted to obtain a recommendation as to how urbanization impacts should be considered when selecting a peak discharge frequency relationship.

7) The median analytical frequency relationship based on water years 1955 through 2011 (Figure 9) with an adopted skew of 0.1 should be used as the base case for purposes of the feasibility scoping meeting. The median analytical frequency relationship based on water years 1955 through 1992 (Figure 10) should be used as a sensitivity case for purposes of the feasibility scoping meeting. Selection of a final frequency relationship should be deferred until completion of the additional recommended unregulated, regional depth area duration and urbanization impact studies. Upon completion of these studies, the Committee on Hydrology should be consulted to obtain a recommendation on selection of the final peak discharge frequency relationship.

8) A 4500 year return period should be associated with the standard project flood as a base case for purposes of the feasibility scoping meeting. This estimate is based on the 1955 through 2011 median analytical peak discharge frequency relationship (Figure 9). A 2000 year return period should be associated with the standard project flood as a sensitivity case for purposes of the feasibility scoping meeting. This estimate is based on the 1955 through 1992 median analytical peak discharge frequency relationship (Figure 10). Selection of a final return period estimate for the standard project flood should be deferred until the final frequency relationship is identified.

9) Existing interior versus exterior assumptions in the HEC-FDA model should be used for purposes of the feasibility scoping meeting with recognition that the consequence estimates may be somewhat conservative. The Modeling, Mapping, and Consequence Production Center (POC Ron Goldman) should be consulted to develop unsteady flow hydraulic models for use in developing interior versus exterior relationships. These

relationships should be used in the HEC-FDA model for the final feasibility study analyses. Existing HEC-RAS models which have already been developed for the feasibility study and the base condition risk assessment should be adapted for this purpose to the extent that it is practical to do so.