Upper Sabine River Flood Protection Planning Study

FINAL REPORT

TWDB Project No. 40058

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1.0 Executive summary

The Texas Water Development Board (TWDB) approved a Flood Infrastructure Fund (FIF) Category 1 grant application with the Sabine River Authority of Texas (SRA) to provide Flood Protection Planning for the Upper Sabine River Basin (TWDB FIF ID #40058, Commitment No. GN1001352). SRA contracted with Freese and Nichols, Inc. (FNI) to assist in fulfilling the requirements of the planning activity. The scope of this planning study included the identification and evaluation of existing flood hazards, the necessary data collection and stakeholder engagement to facilitate the identification of proposed projects, and the development and evaluation of proposed projects to reduce flood risk within the study area.

The planning study area for the Upper Sabine River Basin Flood Protection Planning Study includes approximately 269 square miles of drainage area in the upper Sabine River basin in Upshur, Smith, Gregg, Harrison, and Rusk Counties. The planning study includes all or portions of HUC-10 watersheds 1201000205 Rabbit Creek – Sabine River and 1201000206 Cherokee Bayou-Sabine River. Generally, this area includes a section of the Sabine River and the entire contributing area through the cities of Kilgore and Longview. Because of this, the Cities of Longview and Kilgore were the major stakeholders within the study.

The scope of services for the project included elements necessary to engage stakeholders within the study area, collect an adequate amount of data for review to develop accurate models, evaluate existing conditions flood hazards, and develop a plan to address the primary flood hazards through major infrastructure projects. The evaluation of existing conditions included the development of an existing conditions model, calibration of the model to historical storm events, and assessment of the existing conditions flood hazards. Developing a plan to address the primary flood hazards included the identification and evaluation of different mitigation alternatives, analysis of project benefits, development of conceptual project layouts, and development of a plan to implement the recommended projects.

The development of the models for the Upper Sabine River Basin followed rain-on-mesh (ROM) methodology. The use of rain-on-mesh methodology allowed the hydrologic and hydraulic models to be integrated into a single model and facilitated the use of spatially varying data. This methodology resulted in models that evaluate hydrology and hydraulics simultaneously and provide hydraulic results at the cell level. ROM also provided the flexibility to identify flood risk at any location within the model domain, add details to the model within areas of interest, and identify both fluvial and pluvial flood risks.

Models were calibrated using 3 storm events from 2016 and 2019, frequency events ranging from the 2-year to the 500-year events were used to evaluate localized flood prone areas, roadway flooding, and structural flooding. Criteria was developed using information from stakeholders to score and prioritize the potential mitigation areas.

Based on the prioritization evaluation the following areas were identified for flood mitigation project development:

- Bighead Creek
- Harris Creek Tributary Flooding (Drain No. 4)

- Guthrie Creek
- Longview Underpasses
- Iron Bridge Creek
- Upper Wade Creek
- Lower Wade Creek
- Grace Creek
- Turkey Creek Tributary
- Upper Turkey Creek
- Upper Guthrie Creek
- SH 135 Underpass
- Elm Branch

To facilitate the efficient evaluation of the potential flood mitigation projects (FMPs), truncated models were developed for each of the areas based upon the model of the full study area. The truncated models incorporated refinements to capture additional detail. The refined models were used as the baseline for flood mitigation project development within their respective project areas. Of the analyzed areas, 7 resulted in proposed mitigation alternatives, with 8 total Flood Mitigation Projects (FMPs). The recommended projects are summarized below in **Table 1**.

Table 1: Summary of recommended Flood Mitigation Projects (FMPs).

Stream	Description	Benefits
Bighead Creek (Kilgore)	Channel improvements along Bighead Creek and Turkey Creek including bridge improvements to Stone Road (FM 2204) and Dudley Road (FM 1249).	100-year level of service (LOS) at Dudley Road 500-year LOS at Stone Road
State Highway 135 (Kilgore)	Addition of inlets and a bypass storm sewer to a pump station. Pump station connects to channel improvements and detention pond.	Reduced flood depths and drain times during the 5-, 25-, and 100-year events. 5-year: 3.5-ft depth reduction 5-year: Less than 1-hour ponding time
Drain No. 4 (Longview)	Increasing channel capacity through geometry improvements and concrete lining, property buyout, and bridge improvements to Golfcrest Drive, N Lane Wells Drive, Pine Tree Road, and W Loop 281.	WSE reductions along Pine Tree Road: 5-year: 0.25- to 1.5-ft 25-year: up to 2-ft 100-year: up to 3.25-ft
High Street Underpass (Longview)	Improvements to storm sewer draining underpass and storm sewer trunkline along Cotton Street with small detention pond at existing outfall.	Reduced flood depths and drain times during the 5-, 25-, and 100-year events. 5-year: 5-ft depth reduction 5-year: Less than 30-min drain time
Green Street and Mobberly Avenue Underpasses (Longview)	Improvements to storm sewer draining underpasses and storm sewer trunkline, with channel improvements at existing Green Street outfall and detention pond at existing Mobberly Avenue outfall with 65 cfs pump station.	Reduced flood depths and drain times during the 5-, 25-, and 100-year events. 5-year: 9-ft reduction in depth, 10-hour reduction in drain time at Mobberly Avenue underpass 5-year: 2-ft reduction in depth, 30-min drain time at Green Street

Stream	Description	Benefits
Iron Bridge Creek (Longview)	Increasing channel capacity through channel geometry improvements, concrete lining, two detention ponds, and bridge improvements to Raney Drive, Lemmons Drive, Wells Street, and Millie Street.	WSE reductions along Wells Street: 5-year: up to 0.75-ft 25-year: up to 1.75-ft 100-year: up to 2.5-ft
Lower Wade Creek and Grace Creek (Longview)	Proposed stormwater network in surrounding neighborhood, widening nearby drainage ditch, and adding an additional 150-foot opening to Loop 281	100-year LOS for Loop 281. Reduced ponding and structural flooding in surrounding neighborhood
Elm Branch (Longview)	Increasing channel capacity through in-line detention storage via channel benching, property acquisition, downstream channel improvements, and bridge improvements to Irving Street, Ralph Street, and Miles Street.	5-year: 2.5- to 5-ft WSE reduction 25-year: 2- to 5.5-ft WSE reduction 100-year: 2- to 5.5-ft WSE reduction

Of those 8 FMPs, 3 were ready with all required items to be incorporated as FMPs at time of the Amended Sabine Regional Flood Plan in early 2023. The 3 which were placed were the following:

- Bighead Creek (FMP ID: 043000031)
- Drain No. 4 (FMP ID: 043000032)
- Iron Bridge Creek (FMP ID: 043000033)

The next steps for the projects summarized above is the development of preliminary engineering reports to advance the projects. This could be done through leveraging the feasibility level cost estimates developed for this project to budget for future design phases and/or pursuing funding for the projects. Additionally, the refinement and adoption of an updated drainage criteria manual will be critical to minimize/eliminate the possibility of future development adversely impacting downstream communities.

It is noted that this is a planning level study. Proposed projects within this study are for planning purposes to identify projects that could lead to flood risk reduction and are not fully ready to be constructed. More detailed engineering, especially subsurface utility investigation and planning – which was not part of this study – is a vital component that would need to be performed. Additionally, other disciplines such as transportation engineering, construction engineering, geotechnical engineering, site civil engineering, and others are needed to fully expand on a project with plan production and other engineering services to get these planning level projects ready for construction. Construction costs in this report are also planning level costs and should not be used for bidding.

The Areas of Interest (AOI) identified in this study as well as the proposed mitigation projects have been coordinated with the Sabine Regional Flood Plan for their inclusion in the regional plan, and ultimately into the state flood plan. Three (3) of the projects were ready at the time for the amended plan in early 2023; thus, 3 FMPs were included from this study into the amended Sabine RFP and it is anticipated the remaining projects will be amended into the regional and state flood plans in late 2024 and early 2025.

2.0 Introduction

The Texas Water Development Board (TWDB) approved a Flood Infrastructure Fund (FIF) Category 1 grant application with the Sabine River Authority of Texas (SRA) to provide Flood Protection Planning for the Upper Sabine River Basin (TWDB FIF ID #40058, Commitment No. GN1001352). SRA contracted with Freese and Nichols, Inc. (FNI) as the technical consultant to assist in fulfilling the requirements of the planning activity.

The major stakeholders of the study were the City of Kilgore, the City of Longview, and the Sabine River Authority. The scope of services for the project included elements necessary to engage these key stakeholders, collect data for review to develop representative models, evaluate existing conditions flood hazards, and develop a plan to address the primary flood hazards through major infrastructure projects. The evaluation of existing conditions included the development of an existing conditions model, calibration of the model to historical storm events, and assessment of the existing conditions flood hazards. Developing a plan to address the primary flood hazards included the identification and evaluation of different mitigation alternatives to advance a proposed alternative, analyze the benefits of the project, develop conceptual project layout, and provide a plan to implement recommended projects.

The study area is focused on several Hydrologic Unit Code (HUC) boundaries, shown by the black polygon outline in **Figure 1.** It includes approximately 269 square miles of drainage area in the Upper Sabine River basin in Upshur, Smith, Gregg, Harrison, and Rusk Counties. The planning study includes all or portions of HUC-10 watersheds 1201000205 Rabbit Creek – Sabine River and 1201000206 Cherokee Bayou Sabine River. This area includes a section of the Sabine River and the entire drainage area through the cities of Kilgore and Longview. Other municipal corporate limits are also included based on the extents of HUC-10 watershed boundaries.

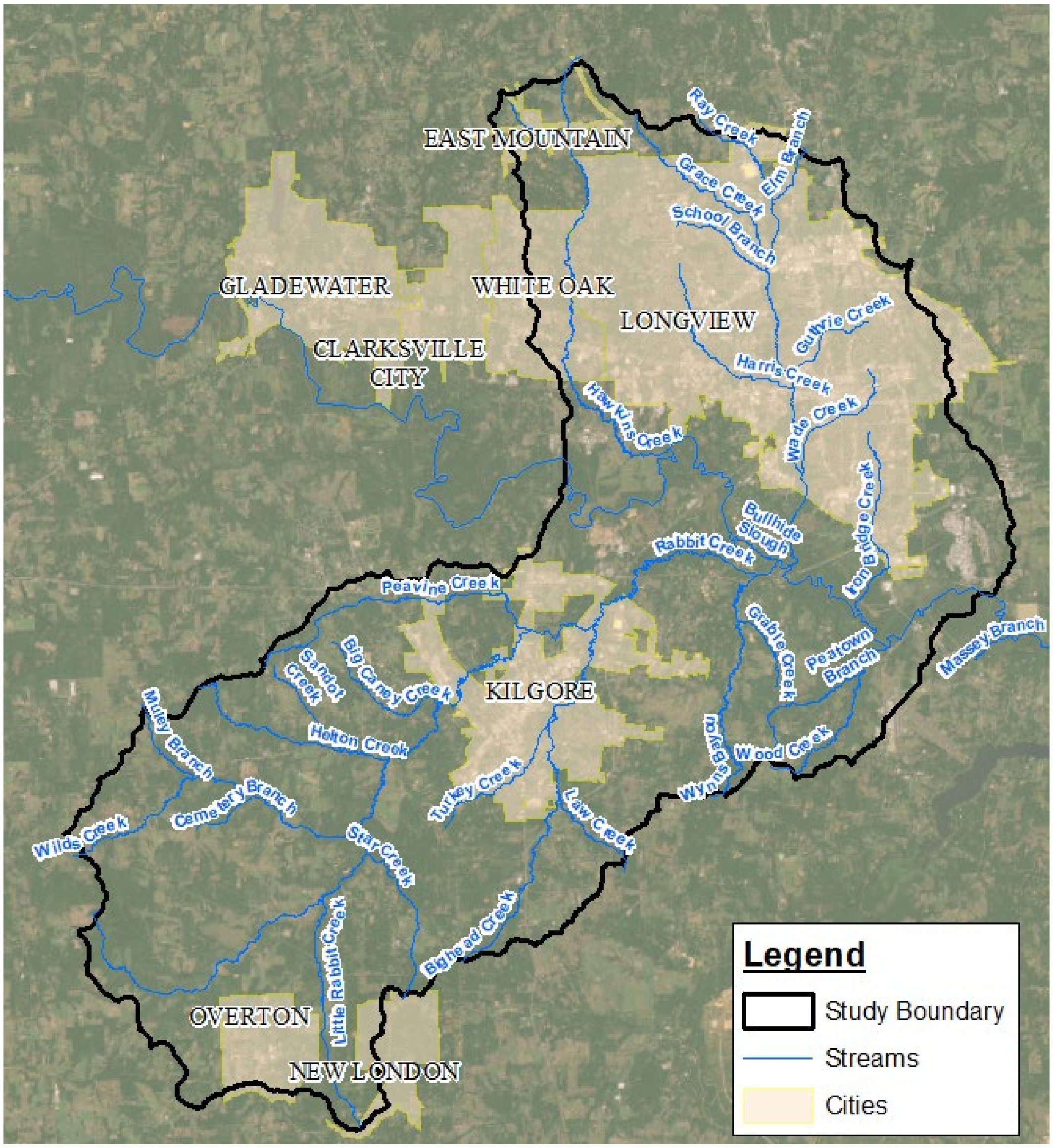


Figure 1: Study Area.

The scope was developed to address the community's need to identify and evaluate existing flood hazards, followed by proposing effective mitigation projects to reduce these flood hazards. Previous effort was done to analyze flooding within the Upper Sabine River Basin, the models of which were not adequate to fully assess the flood risk in the study area. These models, discussed further in **Section 4.1**, were utilized in the development of hydrologic and hydraulic models that were created as a part of this study to address the community's flood risk needs.

3.0 Stakeholder coordination and public engagement

From the outset of the project, the project team coordinated with the major stakeholders of the study: the City of Kilgore, the City of Longview, and the Sabine River Authority. The two cities are the largest populous areas in the region and coordination was done at several points during the project. The Sabine River runs between both cities with the City of Longview residing to the north of the river and the City of Kilgore to the south.

These meetings were held during the development of the existing conditions models, calibration storms, and mitigation options. Correspondence during the project was done to validate the model and confirm the inundation and flood prone areas within the model were representative of the storm events and flooding the cities experienced.

3.1 Stakeholder coordination meetings

The FNI study team and SRA met with the City of Kilgore and the City of Longview in separate meetings on March 22nd, 2022 and August 17th, 2022 to discuss the project and align the study's goals with goals the cities have for flood mitigation. Further coordination occurred with the cities and the public in 2023 when preliminary mitigation strategies were modeled, benefits evaluated benefits, and preliminary design performed.

Initial discussions with the City of Kilgore largely centered around the Turkey Creek and Bighead Creek corridors that bisect the city in a south-to-north direction. During heavy rainfall events, the city historically has had to place barricades along several east-west roads traversing the above-mentioned streams, as the roadways are susceptible to flooding. The City communicated that flood events cause transportation issues on their major thoroughfares for residents and emergency services. When flooded, emergency personnel must take a significantly longer route than normal which puts people and property at greater risk. These discussions helped provide the FNI team with insight necessary to accurately represent the City of Kilgore's flooding issues in the modeling.

Initial discussions with the City of Longview centered around transportation corridors while also emphasizing local channels. The City desired several mitigation projects north and south of US-80. During model calibration, the City of Longview provided a database of local storm sewer trunklines which were incorporated into the FNI modeling. During the meeting in August, the city confirmed many of the locations identified as flooding in the models. Informal discussions were also held with the cities during the public meetings.

Summaries of these stakeholder meetings are included in **Appendix B**.

3.2 Public engagement

As required by the contract, three (3) public meetings were held in the study area. The public meetings are outlined below. Coordination was done with local CBS news station to advertise for the public meetings, and they were all posted on SRA's website for visibility. Generally, there were no comments or input from the public at the public meetings.

Public meeting 1 was held during the evening of September 26, 2022 in conjunction with the Regional Flood Planning (RFP) public meeting at the Longview Expo Center. This public meeting was held to show the existing conditions model results and gather input from the public regarding flood prone areas to be taken into consideration during the screening process in identifying the most hazardous areas for flood mitigation consideration. Several staff members from the City of Longview were present at the meeting and flood prone areas were discussed with the FNI team to confirm many locations within the calibration results.

Public meeting 2 was held during the evening of August 22, 2023 at the Longview Convention Center to highlight the identification of areas of interest for flood mitigation, cover the selected areas for flood mitigation, and outline the identified flooding issues at each location. Staff members from the City of Longview were present at this meeting and further discussions were had between FNI and City staff regarding flood risk comparison between the FIF study's models and the current FEMA effective maps.

Public meeting 3 was held during the evening of November 7, 2023 at the Longview Convention center. The purpose of this meeting was to cover the proposed mitigation solutions for the overall project area.

3.3 Coordination effort with non-NFIP participants

During the application process, two non-participating communities were identified within the Upper Basin Study area. Sabine River Authority coordinated with elected officials, city staff, and TWDB staff to draft and recommend adoption of floodplain ordinances that meet the minimum NFIP standards.

On November 30, 2020, the City of East Mountain entered into an Interlocal Agreement with Upshur County titled "Interlocal Agreement Between the City of East Mountain, Texas and Upshur County, Texas for Flood Plain Management" that allows the County Floodplain Administrator to serve as the City's Floodplain Administrator.

Following discussions with the Sabine River Authority and TWDB staff, the City of Lakeport adopted B level ordinances (as provided by TWDB staff) at their Council Meeting held on December 8, 2020.

4.0 Existing conditions flood hazard

4.1 Modeling methodology

Background

The following data was evaluated and used in support of the planning study: previous studies, plans, GIS data, and gage information. This data was supplemented by field reconnaissance done by FNI in August 2022. The previous studies available for review included steady flow HEC-RAS models of Eastman Lake Creek, Oakland Creek, Grace Creek, and Harris Creek. These models were last modified for the 2012 Bridge and Culvert Improvements project for the City of Longview. These models were considered outdated due to the methodology, older model version, and did not include hydrologic information. The Gregg County Flood Insurance Study (FIS) models were obtained later in the project to supplement bridge and culvert data for inclusion in the newly developed 2D FIF model (FEMA, 2014). The FIS models were steady-state HEC-RAS version 3.1.3 models that were not georeferenced and were low to moderate in their level of detail. Additionally, the hydrologic model was a HEC-1 model which was deemed to be outdated. Furthermore, the latest FIS report for Gregg County, effective in 2014, notes several of the major streams such as Bighead Creek, Turkey Creek in Kilgore were last modeled in 1996 while other major streams in the Longview area were last modeled for the 1990 FIS.

Modeling strategy refinement

A refinement in modeling strategy to more effectively achieve the planning study's objective was proposed to and accepted by SRA and TWDB. The original scope anticipated traditional separate hydrologic and hydraulic 1D HEC-RAS models with variations in detail as summarized below.

- Low Detail – Typically undeveloped, rural areas. Simplified hydrology and 1D unsteady flow hydraulic modeling of large riverine drainage features.
- Medium Detail – Low to moderate development, fringe areas of municipalities. Standard hydrology, 1D unsteady flow hydraulic modeling of large riverine and major local channel drainage features.
- High Detail – Highly developed, urbanized areas. Detailed hydrology and high-resolution open channel hydraulic modeling of local drainage features. Limited 1D/2D and closed conduit (major storm sewer trunkline) modeling to identify sources of flood risk.

In the time between the project scope development and project initiation, there were advances in modeling methodology, computational speed, and data availability. This advancement allowed FNI to utilize a Rain-on-Mesh (ROM) methodology to improve the level of detail for the entire study area to either a "Medium" or "High" level of detail classification. The hydrologic and hydraulic calculations of ROM are integrated into a single model with spatially distributed rainfall, losses, and routing. For each cell within the model, HEC-RAS computes the applied precipitation, evaluates the infiltration to determine the excess rainfall (runoff depth), and routes the runoff volume between neighboring cells based on the surface topography and surface roughness characteristics. The ROM approach provides detailed hydrologic and hydraulic evaluations of the entire watershed within a single integrated model run and produces flood risk information for the entire model domain instead of a limited stream length. A summary of the level of detail improvements recognized across the study area are summarized in **Table 2.** The modeling software used for the ROM analysis was HEC-RAS 6.3.1.

Table 2: Level of detail scoped versus modeled.

Detail Level	Type	Scoped	Modeled
Low Detail	Hydrology	Traditional simplified methods, lumped parameter hydrology.	Spatially varied soils, land use, and rainfall allowing for a more detailed evaluation compared to lumped parameter hydrology.
Low Detail	Hydraulics	1D unsteady, large/major riverine streams only.	2D hydraulics of entire drainage system (large riverine, major local channels, and minor tributaries), lower level of detail around bridges/culverts using breaklines and terrain modifications to evaluate bridges.
Medium Detail	Hydrology	Traditional hydrology methods, lumped parameter hydrology to be done in fringes of urban areas.	Spatially varied soils, land use, and rainfall allowing for a more detailed evaluation compared to lumped parameter hydrology throughout the entire study area rather than on urban fringes alone.

Detail Level	Type	Scoped	Modeled
Medium Detail	Hydraulics	Traditional 1D modeling of large riverine and major local channels.	2D hydraulics of entire drainage system (large riverine, major local channels, and minor tributaries), modeling of bridges and culverts in detail.
High Detail	Hydrology	Traditional hydrology methods, lumped parameter hydrology to be done in urban areas.	Spatially varied soils, land use, and rainfall allowing for a more detailed evaluation compared to lumped parameter hydrology throughout the entire study area rather than on urban fringes alone.
High Detail	Hydraulics	High resolution of open channels. Limited 1D/2D modeling. Limited storm sewer modeling.	Incorporation of storm drains into existing conditions models. Utilization of ICM for further detailed evaluation of local storm sewers. Very detailed modeling of urban streams with bridges and culverts.

As shown above, the hydrology and hydraulic methodologies used provided greater detail than the original scope with no change to the ultimate outcomes for the project and no change to the project's budget. Additionally, the methods generated far greater flood risk detail than the original scope would have created. This is primarily due to the hydraulic modeling of the entire project area using a 2D grid rather than modeling in 1D only on urban or large riverine systems. This adds to the level of detail for the local communities to better understand flood risk in the region. Additional details on the development of the existing conditions model are provided in the subsequent sections.

4.2 Hydrologic inputs

Precipitation data

Precipitation data had two primary sources depending on type. Historical storms used for calibration used National Stage IV Qualitative Precipitation Estimates (QPE) downloaded from the National Weather Service. The historic storm events evaluated were: March 2016, April 2016, and May 2019. Additional detail regarding these storm events is provided in **Section 4.4**. Hypothetical design storms were based on NOAA Atlas 14 data, measured in Lakeport, Texas. Because the calibration is based on rainfall intensities smoothed to the hour, only a 15-minute minimum duration was used to generate the hypothetical design storm hyetographs.

Hypothetical storms were evaluated for the 2-, 5-, 10-, 25-, 50-, 100-, and 500-year frequency events utilizing Atlas 14, Volume 11 (Texas) rainfall data. Rainfall data for the hypothetical storms is based on a Balanced Frequency, 24-hour storm applied uniformly throughout the model domain. Areal Reduction Factor was not considered for this study area for conservatism. The point rainfall depths were inserted into a HEC-HMS model, to generate the applied hyetographs on a 5-minute interval, with a 15-minute minimum duration to retain similitude with the calibration rainfall intensities. The storm depth was centered at 50% of the total duration. The depths used to generate hyetographs are shown in **Table 3**.

Table 3: Balanced frequency storm definition, inches.

Duration	2-year	5-year	10-year	25-year	50-year	100-year	500-year
15-min	0.943	1.20	1.39	1.65	1.85	2.04	2.51
60-min	1.71	2.19	2.55	3.03	3.37	3.73	4.67
2-hr	2.11	2.75	3.23	3.90	4.40	4.93	6.36
3-hr	2.35	3.09	3.67	4.46	5.08	5.75	7.55
6-hr	2.77	3.70	4.44	5.48	6.32	7.24	9.71
12-hr	3.23	4.36	5.28	6.58	7.64	8.82	12.10
24-hr	3.75	5.10	6.19	7.75	9.04	10.50	14.40

Infiltration data

Due to the modified modeling approach, there were no lumped hydrology parameters developed for this model. Within the ROM model that was developed, Initial and Constant loss method was utilized. This method uses a hypothetical single soil layer to account for changes in moisture content and assumes that the maximum potential rate of precipitation loss is constant throughout the simulation. The initial abstraction is accounted for in this method with an initial loss parameter.

To develop the loss parameters, soils data was utilized. Soils data was based on gridded SSURGO (gSSURGO) data from the Natural Resources Conservation Service. The soils data is dated October 2021. The loss parameters summarized below are based on calibration with historic storm events. The calibration process and historic event calibration values are further discussed in **Section 4.4 and Appendix H, Tech Memo 1**.

Initial deficit

Initial Deficit was based on processing each soil type for storage within the topsoil horizon, then averaging individual soil properties to each Hydrologic Soil Group. The Initial Deficit is a proportion of the Maximum Deficit and the initial deficit values were calibrated to the design event values modeled. Data for the hypothetical events is shown in **Table 4**. The calibration process and historic event calibration values are further discussed in **Section 4.4** and **Appendix H, Tech Memo 1**.

Table 4: Initial deficit by hydrologic soil group.

Hydrologic Soil Group	Initial Deficit (in)
A	1.3
B	1.3
C	1.3
D	1.3
CD	1.3
BD	1.3

Maximum deficit

Maximum Deficit was based on processing each soil type for storage within the topsoil horizon, then averaging individual soil properties to each Hydrologic Soil Group. The final values for the frequency events are shown in **Table 5**. The calibration process and historic event calibration values are further discussed in **Section 4.4** and **Appendix H, Tech Memo 1.**

Table 5: Maximum Deficit by hydrologic soil group.

Hydrologic Soil Group	Max Deficit (in)
A	2.08
B	2.65
C	3.1
D	2.87
CD	3.57
BD	3.43

Constant loss

Constant Loss was based on the Saturated Hydraulic Conductivity (Ksat), determined by Hydrologic Soil Group. The range of values specified by HEC-RAS User's Manual are shown in **Table 6**, which were used as a starting point for calibration, as well as the final hypothetical storm Ksat values. The calibration process and historic event calibration values are further discussed in **Section 4.4** and **Appendix H, Tech Memo 1.**

Table 6: Constant loss ranges by hydrologic soil group.

Hydrologic Soil Group	Ksat Range (in/hr)*	Final Ksat (in/hr)
A	0.30 - 0.45	0.3
B	0.15 - 0.30	0.15
C	0.05 – 0.15	0.05
D	0.00 – 0.05	0.001
CD	-	0.001
BD	-	0.001

*SCS, 1986; Skaggs and Khaleel 1982

4.3 Hydraulic inputs

Terrain development

The topography for the project area was based on a USGS dataset classified as "Red River LIDAR" from the Texas Natural Resources Information Service (TNRIS) dated March 2017 and has a 70-centimeter resolution. FNI used the preprocessed Digital Elevation Model (DEM) provided by the TNRIS server. There was a small area of NoData in the raster that was filled with USGS National Elevation Dataset (NED) dated 2013, which has a 30-meter resolution. Neither data source provides an accurate estimate of the channel bottom for the Sabine River and other streams that had a depth of flow at the time of LiDAR collection due to LiDAR's inability to effectively collect data beneath the water surface. Since the larger channels have some additional flow depth blocked by the data collection methodology, channel modifications were added to account for this additional depth.

For the Sabine River, this modification was based on a constant slope between the two stage-discharge USGS gaging stations, which monitor flow based on the current surveyed channel cross-section and flow at the gage location within the study area, above and below Longview. The flowline of the modification was based on the minimum elevation of the rating curve at each station, and the hydraulic properties of the channel were compared against the rating curve to ensure an appropriate representation of the bathymetry. Modifications following this approach are stored in the “Channel” Modification Layer.

Leveraging storm sewer information provided by the City of Longview, polylines were used to perform terrain modifications to implement the storm sewers into the terrain in the HEC-RAS model such that the model allowed water to utilize these pathways to simulate stormwater conveyance. This path was chosen as HEC-RAS currently does not have the capabilities to model storm sewer networks and simulating rain-on-mesh with no underground conveyance would have overrepresented overland flow. Where major storm drains were identified based on deep ponding results in urbanized areas (indicating a localized depression from an inlet), or any other instance where detailed review of the results indicated the presence of a storm drain system, additional modifications to represent the storm sewer were implemented. The elevations for each storm drain were derived from the LIDAR data, with an assumed constant slope between the two points. Flowlines for the upstream end were assumed to have a minimum depth of 4 feet based on typical inlet box configurations. Outlet elevations were based on the daylight elevation or a connecting elevation to a larger pipe size. The maximum width of the modification was rounded to the nearest foot. The features representing the storm sewer are stored in the “Storm Drain” Modification Layer within the HEC-RAS model.

Where identifiable streams cross major roads and railroads, there was evidence for a culvert based on aerial imagery, and the area draining to the culvert was of sufficient size to affect the model results, approximations of culverts were added as channel modifications if structure data was not available. The inlet and outlet elevations for each culvert were derived from the LIDAR data, with an assumed constant slope between the two points. The maximum width of the channel modification was rounded to the nearest foot of approximated structure size.

Geometry development

The modeled area includes approximately 269 square miles of the upper Sabine River basin as shown in **Figure 1.** The area was modeled as a single 2D flow area. In order to appropriately capture key topographic features, breaklines were implemented. Hydraulic structures such as major bridges and culverts were based on a statewide dataset made available online by TxDOT. The “Bridges” dataset is a statewide GIS point feature class that includes horizontal information for all bridges maintained by TxDOT. This includes all bridge-class culverts as well. For each bridge or bridge-class culvert in the study area, FNI added a storage area and 2D connection (SA/2D) based on the TxDOT Bridges dataset in order to define hydraulic connection upstream and downstream of the hydraulic structure within the 2D flow area.

Due to the limited data available within the TxDOT dataset, several assumptions were made to represent each bridge structure. The horizontal data available for bridges includes the following:

- Number of spans within the structure
- Maximum span width

- Total span width
- Deck distance

For most bridges, the spans are evenly distributed and the number of spans times the maximum span is equal to the total span width. For bridges with unevenly distributed spans, the largest span was identified based on the aerial imagery, with the remaining spans distributed evenly. Deck distance is used directly in the model to represent the structural width of the bridge in the direction of flow. Piers were generally assumed to be 4 feet in diameter, and deck thickness was also assumed to be 4 feet. Existing HEC-RAS model geometry in Longview showed similar bridge components and validated this approach.

Much like for bridges, the horizontal data available from TxDOT for bridge-class culverts is limited and requires interpretation of the data and surrounding context to determine parameters. The data available from TxDOT includes the following:

- Number of culvert barrels
- Maximum barrel width

For all structures derived from this dataset, all culvert barrels are assumed to be identical. All structures in the dataset have a rectangular cross-section. The culvert rise is based on a minimum of 2 feet of cover to the crown of the road and a maximum of the span width. Backcheck against some existing HEC-RAS model geometry in Longview validated this approach.

Land Cover data is based on the National Land Cover Dataset (NLCD) 2019, version 2.0. Both the land cover and impervious area datasets, dated June 2021, were used. The 2D mesh was generated with horizontally varied n-values. The percent impervious dataset is used and modified as appropriate for truncated existing models to reflect true conditions. Surface roughness, or Manning's n value, was based on the NLCD classification, with some overrides in certain circumstances at significant channels and roadways. The Sabine River and other major tributaries within the study area, generally named streams, were digitized to better follow the terrain data, and were then buffered to assign a channel-specific roughness since this level of detail is not available in the NLCD dataset. FNI used the "TxDOT_Roadway_Inventory" dataset for road centerlines to identify areas of reduced surface roughness too detailed to be identified within the NLCD. The TxDOT dataset included estimates of road surface width, which were buffered to create a polygon of coverage. Manning's n values were calibrated using the historic event data. The calibration process and historic event calibration values are further discussed in **Appendix H, Tech Memo 1**. **Table 7** summarizes the design event Manning's n value by land use.

Table 7: Manning's n values by land use.

Land Use	Design Events
Road / NoData	0.020
Pasture-Hay	0.062
Developed, Open Space	0.038
Mixed Forest	0.250
Evergreen Forest	0.200

Land Use	Design Events
Shrub-Scrub	0.190
Emergent Herbaceous Wetlands	0.129
Deciduous Forest	0.250
Developed, Low Intensity	0.077
Open Water	0.030
Grassland-Herbaceous	0.062
Developed, Medium Intensity	0.100
Woody Wetlands	0.194
Barren Land Rock-Sand-Clay	0.039
Developed, High Intensity	0.090
Cultivated Crops	0.062
Channel	0.053

Stability

In some cases, the configuration of other model elements, topography, etc. combine to produce areas of localized model instability. In these areas, the instability was addressed by adding terrain modifications to the model to smooth hydraulic results while still capturing the necessary topographic details. Modifications following this approach are stored in the most appropriate type of Modification Layer based on the type of modification applied.

Gage data

Gage data used in the project was limited to USGS streamflow gages listed in **Appendix H, Tech Memo 1**. FNI used HEC-SSP to evaluate the available gage data on the Sabine River to determine the prevailing flow rates through the model, based on Bulletin 17C methodology. Because the gage 08020900: Sabine River below Longview only has 26 years of record, FNI also included gages at Hawkins (08019200), Gladewater (08020000), and Beckville (08022040) as additional reference points.

FNI performed gage analysis of all 4 gages since 1986, the year when both Lake Fork and Lake Tawakoni reservoirs were fully filled and operational. The limited record of the Hawkins (24 years) and Longview (26 years) gages allowed for comparisons along the Sabine River in multiple locations. The results of the Bulletin 17C Analysis are shown in **Appendix H, Tech Memo 1**.

The stream gages were also used to evaluate the validity of the existing conditions models and adjust the models to provide a more consistent result between the modeled and observed results. The three gages used were Sabine River below Longview, which was used in the gage analysis, as well as Sabine River above Longview (08020450) and Kilgore (08020700). The Gladewater gage was used as an inflow boundary condition to the model. Further analysis of the calibration results of the are included in **Appendix H, Tech Memo 1**.

Table 8: Summary of USGS gaging stations used.

Gage Number	Description	Period of Record	Purpose
08020700	Rabbit Ck nr Kilgore	2015-Present	Calibration
08020450	Sabine Rv abv Longview	1995-Present	Inflow, Calibration, Gage Analysis
08020900	Sabine Rv bl Longview	1995-Present	Calibration
08019200	Sabine Rv nr Hawkins	1997-Present	Gage Analysis
08020000	Sabine Rv nr Gladewater	1932-Present	Inflow, Gage Analysis
08022040	Sabine Rv nr Beckville	1938-Present	Gage Analysis

Boundary conditions

An inflow boundary condition is applied to the Sabine River at the upstream end of the model, representing the flow in the Sabine River between the USGS gages at Gladewater (08020000) and Above Longview (08020450). Depending on the types of storm events for calibration, different inflow conditions were used to approximate the inflow to the model domain. There are several limitations with using unmodified gages data to establish the observed inflows into the model domain.

The upstream distance to the USGS gage at Gladewater (08020000) includes approximately 152 square miles of drainage area which is not represented by the flow measured at that gage. Due to the location of the scoped model extents, it was infeasible to adjust the studied 2D boundary to align with the drainage area of the gage without accounting for additional drainage area. To remain within scope, the gage was determined to best be used as a calibration parameter. The gage just upstream of Longview (08020450) does not have a complete rating curve and is only suited for analysis of low flows.

Limitations also existed with using unmodified gages data located downstream of the study area to establish the observed outflows from the model domain. Notably, the gage downstream of Longview (08020900) already accounts for the rainfall to be simulated within the model. Instead, a normal depth boundary condition was applied at the downstream end of the model, representing the flow going downstream toward the Beckville gage (08022040).

All other boundary conditions were applied as precipitation to the model and utilized the spatial capabilities of HEC-RAS to generate runoff and route it downstream.

4.4 Calibration

FNI conducted an analysis that includes the evaluation of several historical storms. Gage Adjusted Radar Rainfall (GARR) precipitation data was entered in the existing conditions hydrologic models to compare modeled results to observed data for the study area. Stream gage data, High Water Marks (HWM), and modeled historical storms were used to evaluate the validity of the existing conditions models and adjust the models to provide a more consistent result between the modeled and observed results. **Table 9** shows a summary of calibration statistics used to evaluate the quality of results in each event.

Table 9: Calibration performance rating system.

Performance Rating	NSE	RSR	PBIAS
Very Good	0.65 < NSE < 1.00	0.00 < RSR < 0.60	PBIAS < + 15
Good	0.55 < NSE < 0.65	0.60 < RSR < 0.70	+ 15 < PBIAS < + 20
Satisfactory	0.40 < NSE < 0.55	0.70 < RSR < 0.80	+ 20 < PBIAS < + 30
Unsatisfactory	NSE < 0.40	RSR > 0.80	PBIAS > + 30

NSE = Nash-Sutcliffe efficiency; RSR = RMSE-observations standard derivation ratio; PBIAS = Percentage bias.

March 2016 storm event

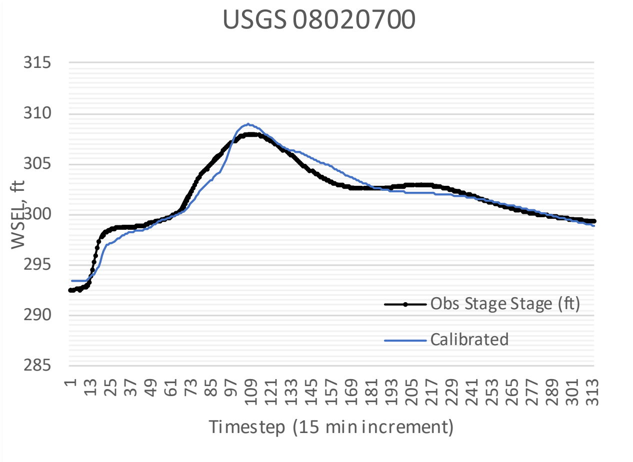
The March 2016 Storm Event was selected because it produced the second highest stage at USGS 08020700, Rabbit Creek near Kilgore. The event was simulated from March 8, 2016 18:00 UTC to March 11, 2016 24:00 UTC. During this time, nearly 11.5 inches of rainfall were measured, which represents approximately a maximum return frequency of the 2-day, 50-year storm. For shorter durations of 12 hours or less, the storm was approximately a 10-year to 25-year return frequency.

Prior to the storm event, the Antecedent Precipitation Tool (APT), produced by USACE rated the conditions as Drier than Normal within a Wet Season, with a Palmer Drought Severity Index of "Severe Wetness".

Prevailing flows in the Sabine River were approximately 1,400 cfs, as measured at USGS gage 08020900, with a measured peak flow of 19,800 cfs.

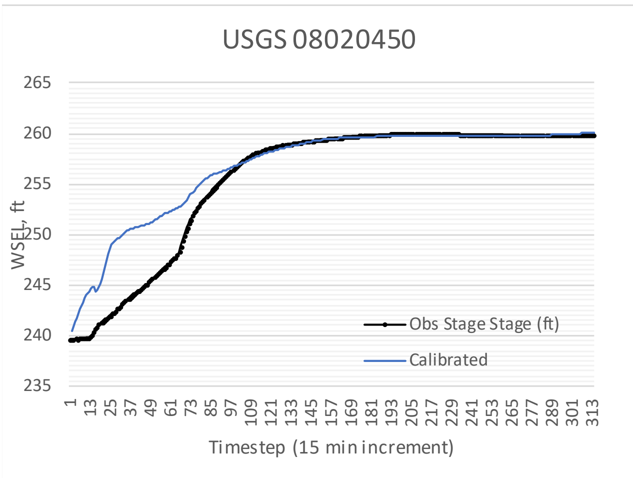
To achieve the amount of flow at both gages on the Sabine River required the Gladewater flows to be shifted by 5 hours and multiplied by 150%. Other than the initial conditions, the simulated water surface elevations match the observed water surface elevations at both Sabine River gages very closely. Despite the initial conditions differences, model metrics were classified as good.

The simulated water surface elevation at the Rabbit Creek Gage matched the general shape of the observed water surface elevation. The simulated peak water surface elevation was 0.91 feet above the observed peak water surface elevation, though the peaks occurred at the exact same time. The good match to the observed hydrograph shape is reflected in the Statistical Performance metrics. A summary of model performance is shown in **Figure 2**.



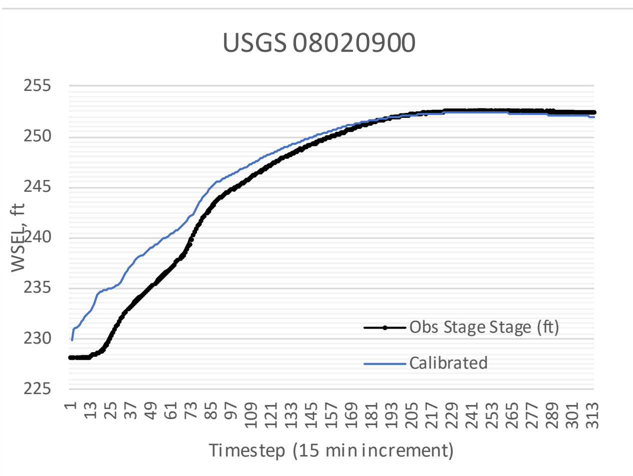
	Observed	Simulated	Difference
Peak WSE (ft)	308.01	308.92	0.91
Time of Peak (min)	1605	1605	0

Statistical Performance (Depth)		
PBIAS	-1.1%	V. Good
Mean Error	-0.15	V. Good
RMSE	0.84	Good



	Observed	Simulated	Difference
Peak WSE (ft)	259.99	260.08	0.09
Time of Peak (min)	3045	4710	1665

Statistical Performance (Depth)		
PBIAS	4.7%	Good
Mean Error	1.17	Good
RMSE	2.54	Good



	Observed	Simulated	Difference
Peak WSE (ft)	252.68	252.41	-0.27
Time of Peak (min)	3780	3690	-90

Statistical Performance (Depth)		
PBIAS	4.1%	Good
Mean Error	1.09	Good
RMSE	1.97	Good

Figure 2: March 2016 Storm Event Model Performance.

April 2016 storm event

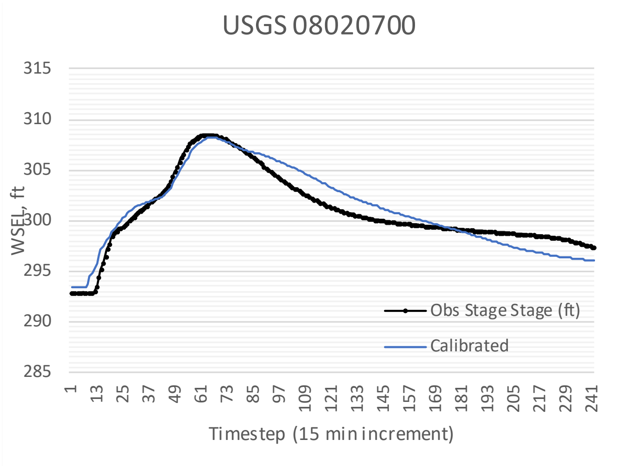
The April 2016 Storm Event was selected because it produced the highest stage at USGS 08020700, Rabbit Creek near Kilgore. The event was simulated from April 30, 2016 00:00 UTC to May 2, 2016 12:00 UTC. During this time, between 5 to 7 inches of rainfall were measured, which represents approximately a maximum return frequency of the 6-hour, 25-year storm.

Prior to the storm event, APT rated the conditions as Wetter than Normal within a Wet Season, with a Palmer Drought Severity Index of "Severe Wetness".

Prevailing flows in the Sabine River were approximately 6,700 cfs, as measured at USGS gage 08020900, with a measured peak flow of 15,300 cfs.

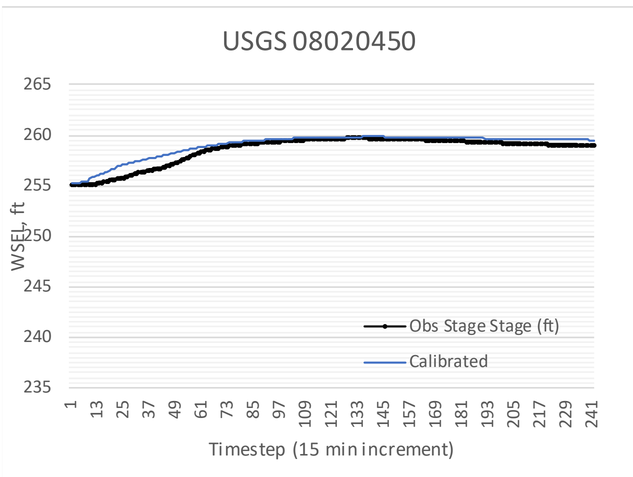
To achieve the amount of flow at both gages on the Sabine River required the Gladewater flows to be shifted by 5 hours and multiplied by 155%. The simulated initial water surface elevations on the Sabine River tracked slightly ahead of the observed water surface elevations, however the difference in peak water surface elevation was within a tenth of a foot, and within 2 hours of the peak timing. The simulated water surface elevations match the observed water surface elevations at both Sabine River gages very closely. Model metrics, such as Percent Bias, Mean Error, and Root Mean Squared Error were classified as good or very good.

The simulated water surface elevation at the Rabbit Creek Gage matched the general shape of the observed water surface elevation. The simulated peak water surface elevation was 0.30 feet below the observed peak water surface elevation, with the peaks occurring within 15 minutes of each other. The good match to the observed hydrograph shape is reflected in the Statistical Performance metrics. A summary of model performance is shown in **Figure 3**.



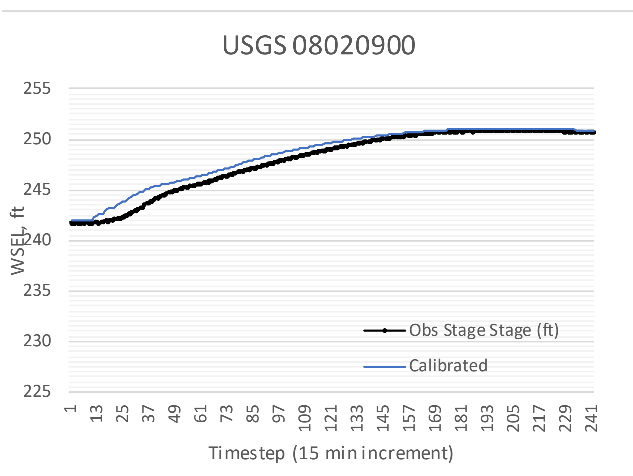
	Observed	Simulated	Difference
Peak WSE (ft)	308.52	308.22	-0.30
Time of Peak (min)	975	990	15

Statistical Performance (Depth)		
PBIAS	1.7%	**Good**
Mean Error	0.22	**V. Good**
RMSE	1.22	**Good**



	Observed	Simulated	Difference
Peak WSE (ft)	259.78	259.86	0.08
Time of Peak (min)	1920	2040	120

Statistical Performance (Depth)		
PBIAS	1.4%	**Good**
Mean Error	0.39	**V. Good**
RMSE	0.50	**V. Good**



	Observed	Simulated	Difference
Peak WSE (ft)	250.97	251.05	0.08
Time of Peak (min)	2970	2925	-45

Statistical Performance (Depth)		
PBIAS	2.8%	**Good**
Mean Error	0.48	**V. Good**
RMSE	0.61	**Good**

Figure 3: April 2016 storm event model performance.

May 2019 storm event

The May 2019 Storm Event was selected because it produced the third highest stage at USGS 08020700, Rabbit Creek near Kilgore. The event was simulated from May 2, 2019 00:00 UTC to May 4, 2019 12:00 UTC. During this time, approximately 5.5 inches of rainfall were measured, which represents approximately a maximum return frequency of the 6-hour, 10-year storm. In the general Kilgore area, was approximately a 10-year return frequency for 3- to 12-hour durations.

Prior to the storm event, APT rated the conditions as Normal within a Wet Season, with a Palmer Drought Severity Index of “Extreme Wetness”.

Prevailing flows in the Sabine River were approximately 9,900 cfs, as measured at USGS gage 08020900, with a measured peak flow of 15,300 cfs.

To achieve the amount of flow at both gages on the Sabine River required the Gladewater Flows be shifted by 10 hours and multiplied by 190%. The simulated initial water surface elevations on the Sabine River track slightly ahead of the observed water surface elevations, however the simulated water surface elevation is generally parallel to the observed values. Model metrics were classified as good or very good.

The simulated water surface elevation at the Rabbit Creek Gage matched the general shape of the observed water surface elevation. The simulated peak water surface elevation was 0.23 feet below the observed peak water surface elevation, with the simulated peak occurring within 2 hours and 15 minutes after the observed peak. The good match to the observed hydrograph shape is reflected in the Statistical Performance metrics. A summary of model performance is shown in **Figure 4**.

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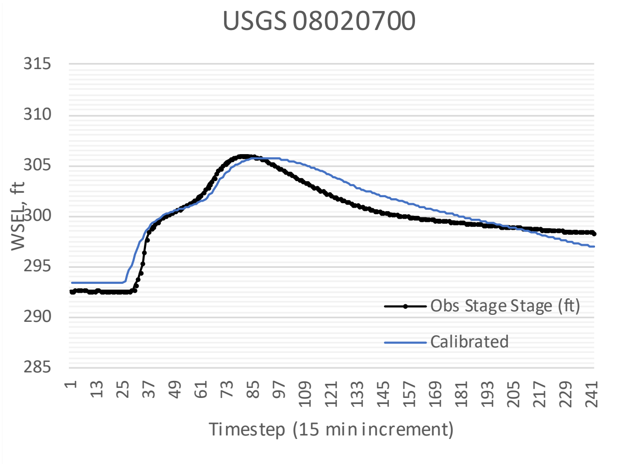
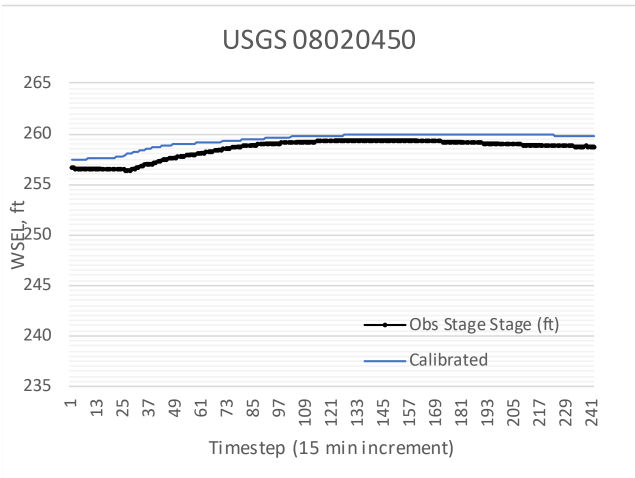


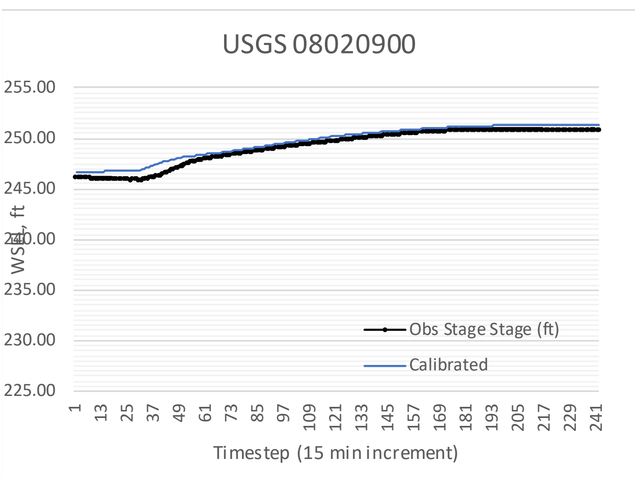
	Observed	Simulated	Difference
Peak WSE (ft)	306.00	305.77	-0.23
Time of Peak (min)	1215	1350	135

Statistical Performance (Depth)		
PBIAS	4.2%	Good
Mean Error	0.52	Good
RMSE	1.15	Good



	Observed	Simulated	Difference
Peak WSE (ft)	259.44	259.95	0.51
Time of Peak (min)	2040	2520	480

Statistical Performance (Depth)		
PBIAS	2.8%	Good
Mean Error	0.79	Good
RMSE	0.85	Good



	Observed	Simulated	Difference
Peak WSE (ft)	251.00	251.33	0.33
Time of Peak (min)	3030	3210	180

Statistical Performance (Depth)		
PBIAS	1.3%	Good
Mean Error	0.38	V. Good
RMSE	0.43	V. Good

Figure 4: May 2019 storm event model performance.

4.5 Hypothetical storm evaluation

Following the calibration of the model, hypothetical storms were evaluated for the 2-, 5-, 10-, 25-, 50-, 100-, and 500-year frequency events utilizing Atlas 14, Volume 11 (Texas) rainfall data. Calibrated parameters of the Manning's n, initial deficit, and saturated hydraulic conductivity values were utilized for the hypothetical storm evaluation. A normal slope boundary condition was used at the downstream end of the model of 0.0001. Tabular summaries of input data for the calibrated parameters and the precipitation are provided in **Section 4.2**.

4.6 Analysis results

The results of the evaluated storm events (hypothetical and historic) are provided in **Appendix H**. Water surface profiles for the hypothetical storms are provided in **Appendix I**.

4.7 Flood hazard identification

Prioritization of flood mitigation areas

Using the input from the stakeholders and the hydraulic model's results, Areas of Interest (AOI) for flood mitigation were initially identified as areas that experience flood risks which could be aided by the implementation of a flood mitigation project(s). These areas were evaluated further to determine feasibility of implementing a project, severity of the flooding, and community impact potential. Boundaries of potential flooding areas were created (shown in **Figure 5**) and were identified where the following was observed:

- Multiple structures (residential and commercial) potentially inundated by a 5-year storm event
- Structures with potentially deep flood depths (see **Table 18**)
- Roadways at stream crossings overtopped by a 5-year storm event
- Significant length or depth of roadway overtopping during a 100-year storm event
- Potentially flooded underpasses
- Reported flooding from local news sources, or City Engineer

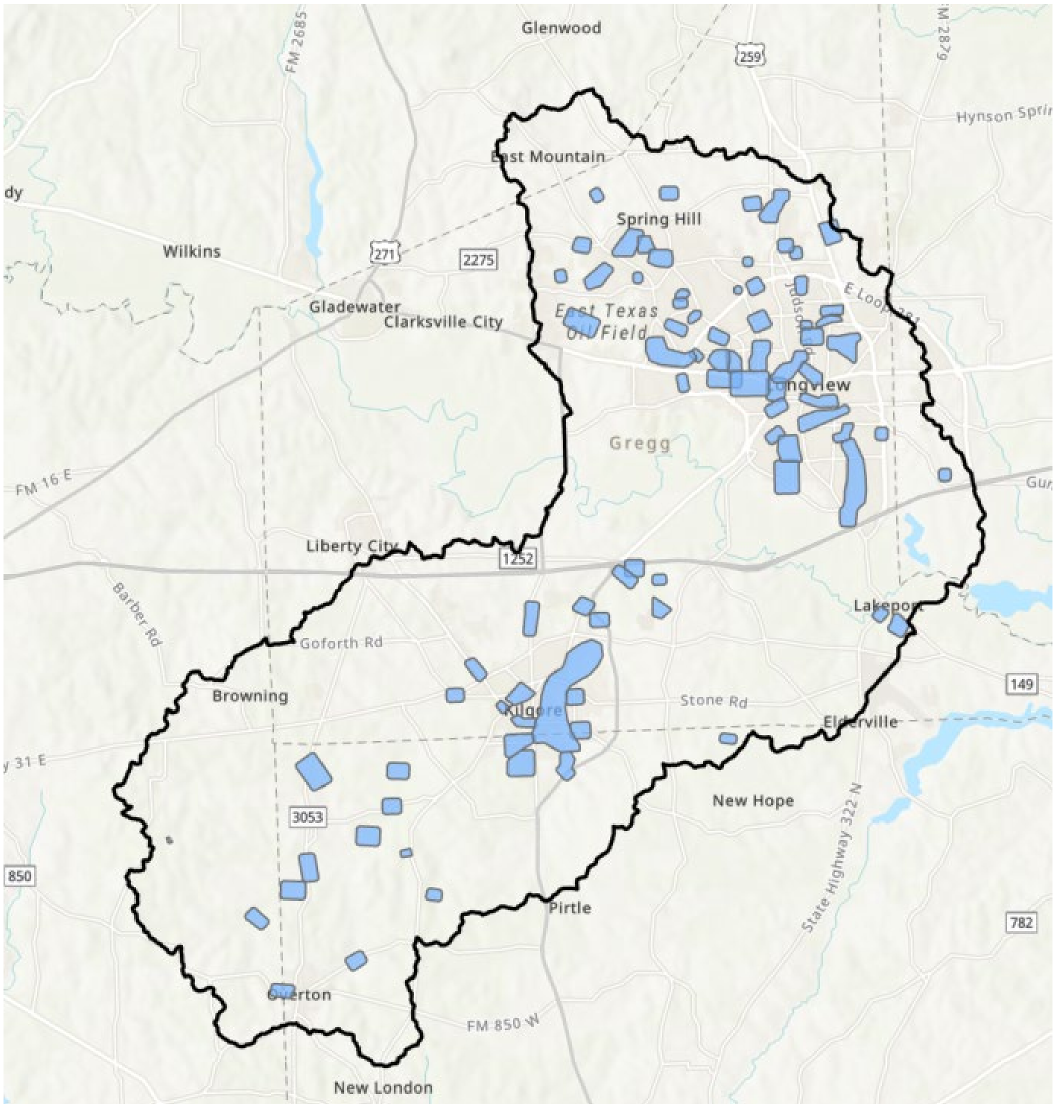


Figure 5: Project boundary (black) and identified AOIs (blue).

Once the AOIs were identified, GIS processes were performed and metrics within each AOI boundary were determined for the scoring system described in the sections below. The scoring system helped to provide an objective analysis ultimately differentiating between projects ranging between those that have a great mitigation need versus those with a lower flood mitigation need.

Prioritization scoring criteria

A total of 3 categories were developed for the prioritization criteria with 12 total metrics evaluated. A numerical analysis was chosen to provide an objective and impartial assessment for prioritizing the AOIs to be further evaluated for potential mitigation alternatives. A scoring range from 0 to 10 was used for each metric based on the percentile ranges calculated per criteria as well as the technical consultant team's engineering judgement. Additionally, each criterion is weighted between 5% to 15% according to its evaluated importance. The 12 metrics are weighted so that the total possible accumulative score per project is 10.

Listed below in **Table 10** are the details for the different categories, their scoring metrics, and the weighted percentage assigned.

Table 10: Prioritization categories and considerations.

Flooding Category	Metrics	Individual Weighting
General	Ability to Improve/Implement Project	5%
General	Type of Roadway Flooding	5%
General	Reported Flooding	10%
General	Critical Facilities Affected	5%
General	Population Impacted	10%
General	Social Vulnerability Index (SVI)	5%
Roadway	First Instance Overtopping Roadway	10%
Roadway	Depth of Roadway Flooding	10%
Roadway	Length of Roadway Overtopping	5%
Roadway	Accessibility/Detour Routes	5%
Structural	Severity of Structural Flooding	15%
Structural	Number of Structures Flooded	15%

Ability to improve/implement project

The ability to improve/implement each project was analyzed visually to determine the amount of area available for construction and mitigation or if acquiring land is infeasible due to existing development. Areas with minimal space for improvement or mitigation were given a lower score, while those with considerable space were given a higher score, and a weighted value of 5% was applied to this criterion. An area identified as "Low" indicates minimal space for construction or mitigation signifying a project may not be as feasible or practical for implementation and construction. Conversely, areas with very high amount of undeveloped land indicate there is significant space for potential flood mitigation indicating it may be more likely that a project can be implemented and potentially have less challenges compared and improvements in the area. **Table 11** shows the scoring categories.

Table 11: Scoring for ability to improve/implement project.

Weighted Percentage: 5%	Low	Medium Low	Medium	High	Very High
Points	2	4	6	8	10

Type of roadway flooding

Roadway designations as defined by the Texas Department of Transportation (TxDOT) were used to categorize the severity of roadway flooding for this metric. Areas were scored based upon the type of roadway inundated during the 100-year event. Higher scores were given to major highways and roadways that are likely to: have higher traffic volumes, be utilized as evacuation routes, and be utilized by critical facility vehicles during a flood event. A weighted value of 5% was applied to this criterion. **Table 12** shows the scoring categories.

Table 12: Scoring for type of roadway flooding.

Weighted Percentage: 5%	None	Local Road Flooding	Arterial Road Flooding	County/FM Road Flooding	Major Highway Flooding
Points	0	4	6	8	10

Reported flooding

The number of reported flooding locations from the City Engineer or staff, news articles, and other available sources per area of interest was calculated for this metric. These were locations identified during the meetings with the stakeholders in 2022, news reports found online when researching flooding in Longview, Kilgore, and the surrounding counties, as well as a list of flooded structures provided by the City Engineer for the City of Longview. A record or instance of any of the items listed above represents a single report as counted in the scoring process. Scoring was based upon the number or reports provided within the AOI. A weighted value of 10% was applied to this criterion. **Table 13** shows the scoring categories.

Table 13: Scoring for reported flooding.

Weighted Percentage: 10%	None	Single Report	Multiple Reports
Points	0	6	10

Critical facilities

This metric is a measurement of the number of critical facilities within the identified boundary for each area of interest that may be affected by flooding. Critical facilities were defined as schools, hospitals, fire stations, and national shelter facilities. Scoring was based on the number of critical facilities within the AOI boundary. Higher scores were given to areas with multiple critical facilities. A weighted value of 5% was applied to this criterion. **Table 14** shows the scoring categories.

Table 14: Scoring for critical facilities.

Weighted Percentage: 5%	None	Single Critical Facility	Multiple Critical Facilities
Points	0	6	10

Population impacted

The population impacted was calculated by adding the total number of people per structure affected by the 100-year flood event in each area of interest. Scoring for this metric was based upon number of people within the AOI that may be affected during a 100-year flood event. Higher scores were given to areas that had a larger number of people impacted. An area with greater than 1,000 people impacted by the 100-year flooding was assigned the highest possible score as it is expected that areas with larger populations within an area may have a higher risk to people and property compared to areas of lower population. A weighted value of 10% was applied to this criterion. **Table 15** shows the scoring categories.

Table 15: Scoring for Population Impacted.

Weighted Percentage: 10%	Less Than 10 People	10 to 100 People	100 to 1,000 People	Greater Than 1,000 People
Points	0	2	6	10

Social vulnerability index (SVI)

SVI is a rating system from the Centers for Disease Control and Prevention (CDC) and Agency for Toxic Substances and Disease Registry (ATSDR) that measures the external stresses on human health and is used to identify communities that most likely need support as a result of a disaster. A lower SVI value correlates to a lower vulnerability, while a higher SVI relates to a higher vulnerability. The average SVI per area of interest was calculated for each area and a weighted value of 5% was applied to this criterion. **Table 16** shows the scoring categories.

Table 16: Scoring for SVI.

Weighted Percentage: 5%	Least Vulnerable: SVI = 0 to 0.25	Low: SVI = 0.25 to 0.5	Moderate: SVI = 0.5 to 0.75	Most Vulnerable: SVI = 0.75 to 1.0
Points	2	3	6	10

Severity of roadway overtopping

This metric is an analysis of the 2-, 5-, 25-, and 100-year storm events interaction with the roadways at stream crossings within the areas of interest in the existing conditions model. The first instance in which a roadway overtops by one of the listed events within an AOI designates the frequency of the roadway overtopping. Roadways that begin to become overtopped for higher frequency storm events, such as a 2-year or a 5-year event, may be susceptible to more frequent flooding. A higher score was applied to more frequently flooded roadways and a weighted value of 10% was applied to this criterion. **Table 17** shows the scoring categories.

Table 17: Scoring for severity of roadway overtopping.

Weighted Percentage: 10%	No Overtopping	100-Year Overtopping	25-Year Overtopping	5-Year Overtopping	2-Year Overtopping
Points	0	3	5	8	10

Depth of roadway flooding in 100-year event

The 100-year depth of flooding was analyzed for the overtopped roads within the study area. Many of these occurred at stream crossings, but some others were flooded from potential overwhelmed storm sewers backing up runoff onto the roadways and causing overland flooding. Deeper inundation over a roadway indicates that the road may be impassible during a flood event or is more susceptible to flooding. A weighted value of 10% was applied to this criterion and roadway flooding greater than 6 feet deep was considered the highest possible score. **Table 18** shows the scoring categories.

Table 18: Scoring for depth of roadway flooding.

Weighted Percentage: 10%	None	Minimal: Depth = Less Than 1 Foot Deep	Moderate: Depth = 1 To 3 Feet Deep	Major: Depth = 3 to 6 Feet Deep	Extreme: Depth = Greater Than 6 Feet Deep
Points	0	3	5	8	10

Length of roadway overtopping in 100-year event

The length of roadway overtopping was determined by measuring the inundation extents of the 100-year event along the length of the roadway. Higher scores were given to areas with greater lengths of inundated roadways. A weighted value of 5% was applied to this criterion and areas with flooding that inundated over one mile of roadway were considered the highest possible score. **Table 19** shows the scoring categories.

Table 19: Scoring for length of roadway overtopping.

Weighted Percentage: 5%	None	Minimal: Less Than 0.1 miles	Moderate: 0.1 to 0.25 miles	High: 0.25 to 0.5 miles	Major: Greater Than 0.5 miles
Points	0	2	5	8	10

Accessibility/detour routes

This is an indirect measure of the ability to detour the overtopped roadways within a flood mitigation area. Each flood mitigation area was individually assessed into three classifications: major detour, inconvenient detour, or detour possible. Areas were classified as a major detour if inundation may cause severe traffic delays or if it left structures inaccessible by roadway. Areas that had potential ways to bypass the inundated streets, but the route was considered significantly inconvenient, were classified as an inconvenient detour. If a detour was apparent in the immediate vicinity, the area was classified as "detour possible". A weighted value of 5% was applied to this criterion. **Table 20** shows the scoring categories.

Table 20: Scoring for accessibility/detour routes.

Weighted Percentage: 5%	Detour Possible	Inconvenient Detour	Major Detour
Points	2	6	10

Severity of structural flooding

This metric is an analysis of possible structural flooding from the 5-, 25-, and 100-year storm events within the areas of interest based on the existing conditions model results. The first instance in which a structure is inundated by one of the events designates the severity of the structural flooding. A structure which floods during a 5-year event is more severe compared to a structure that begins to flood during a 100-year event; thus, it the areas which experience structural flooding at a 5-year level is a more severe flooding issue and is assigned a higher. A weighted value of 15% was applied to this criterion. **Table 21** shows the scoring categories.

Table 21: Scoring for severity of structural flooding.

Weighted Percentage: 15%	None	100-Year Flooding	25-Year Flooding	5-Year Flooding
Points	0	3	6	10

Number of structures flooded

This is a measure of the number of structures inundated from the 100-year event. The greater the number of inundated structures within an AOI boundary, the higher the potential for flood damages in an area. AOIs with a greater number of potentially flooded structures are given a higher score compared to an area with fewer structures. A weighted value of 15% was applied to this criterion and flooding affecting over 100 structures for an area was considered the highest possible score. **Table 22** shows the scoring categories.

Table 22: Scoring for number of structures flooded.

Weighted Percentage: 15%	None	Minimal: Less than 5 structures	Low: 6 to 15 structures	Moderate: 16 to 50 structures	High: 51 to 100 Structures	Extreme: Greater than 100 structures
Points	0	2	3	5	7	10

Prioritization results

Once the scores were generated, a ranking of the scores was performed to identify those which scored high and would be prioritized in the next stages of the project for more detailed evaluation. **Table 23** presents the top 30 scoring AOI based on the ranking criterion outlined above.

Table 23: Highest 30 scoring AOIs.

Score (out of 10)	Location	Description
9.45	Kilgore	Bighead Creek Flooding
8.55	Longview	Harris Creek Tributary Flooding
8.45	Longview	Longview Underpass Flooding
8.35	Longview	Guthrie Creek Corridor Flooding
7.90	Longview	Upper Wade Creek Flooding
7.60	Longview	Lower Wade Creek Flooding
7.55	Kilgore	Upper Turkey Creek Neighborhood Flooding
7.50	Longview	Upper Guthrie Creek Neighborhood Flooding
7.40	Longview	Iron Bridge Creek Neighborhood Flooding
7.30	Longview	Elm Branch Flooding
7.30	Kilgore	Turkey Creek Tributary Neighborhood Flooding
7.20	Longview	Tammy Lynn Drive Neighborhood Flooding
7.20	Longview	Harris Creek at US 80 Flooding
7.15	Longview	Grace Creek Flooding at US 80
7.05	Longview	Judson Road Neighborhood Flooding
7.05	Kilgore	Rabbit Creek at SH 42 Overtopping
6.95	Longview	US 281 Underpass
6.85	Kilgore	SH 135 Underpass Flooding
6.45	Longview	Drake Blvd Neighborhood Flooding
6.40	Longview	Coushatta Hills Creek Neighborhood Flooding
6.25	Longview	HG Mosely Roadway Flooding
6.20	Longview	Eastman Road Flooding
6.15	Longview	Eden Drive Flooding
6.10	Kilgore	Rabbit Creek at SH 31 Overtopping
6.05	Longview	Whispering Pines Roadway Flooding
6.05	Kilgore	Florence Street Neighborhood Flooding
6.00	Longview	Secretariat Trail Neighborhood Flooding
6.00	Longview	Lorraine Court Neighborhood Flooding
6.00	Longview	Grace Creek at Loop 281 Overtopping (South)
5.95	Longview	Harley Ridge Road Flooding (South)

Generally, the highest ranking AOIs were located in populated areas of the study area. This is due to the criteria placing an emphasis on locating structural flooding and traffic mobility as these were main concerns of stakeholders during the in-person meetings in 2022. The City of Longview expressed specific concern in flooding at the High Street underpass, which is located just north of the Longview Fire Department and utilized frequently as a critical route during emergencies.

Figure 6 below outlines the distribution of the project rankings for all 84 AOIs. The ranking criteria created a stratified list with a clear distinction between high scoring projects versus ones that scored lower. This graphical representation of the data identifies the areas of the greatest need for flood mitigation which will be targeted for flood mitigation solutions in the remainder of the project.

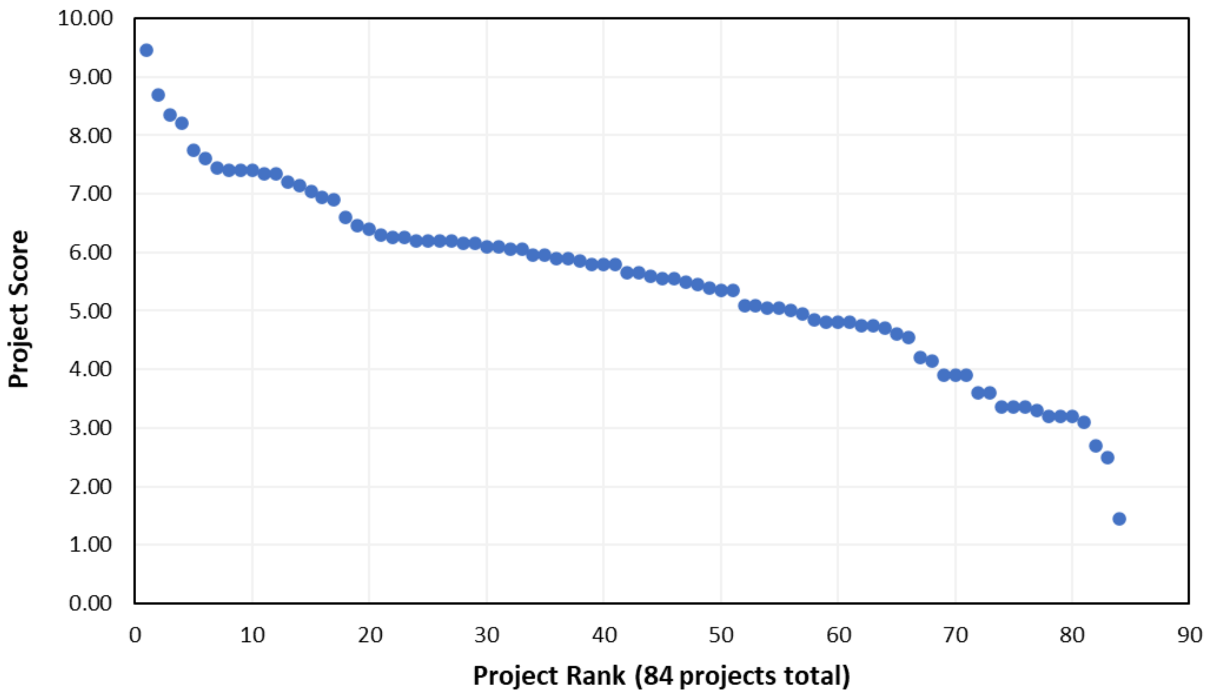


Figure 6: Project ranking plot.

The projects in **Table 23** scored the highest in the ranking process and a flood mitigation alternative to each one may have a potentially significant benefit to the region. **Figure 7** shows the locations of the 10 highest ranked projects in red, projects ranked between 11 and 20 in orange, with the other AOIs shown in blue. A full graphical view of the AOIs with the 100-year inundation boundary from the existing conditions model can also be viewed in an online web map for this project at the following link: SRA21707 Upper Sabine Stakeholder Web App (arcgis.com).

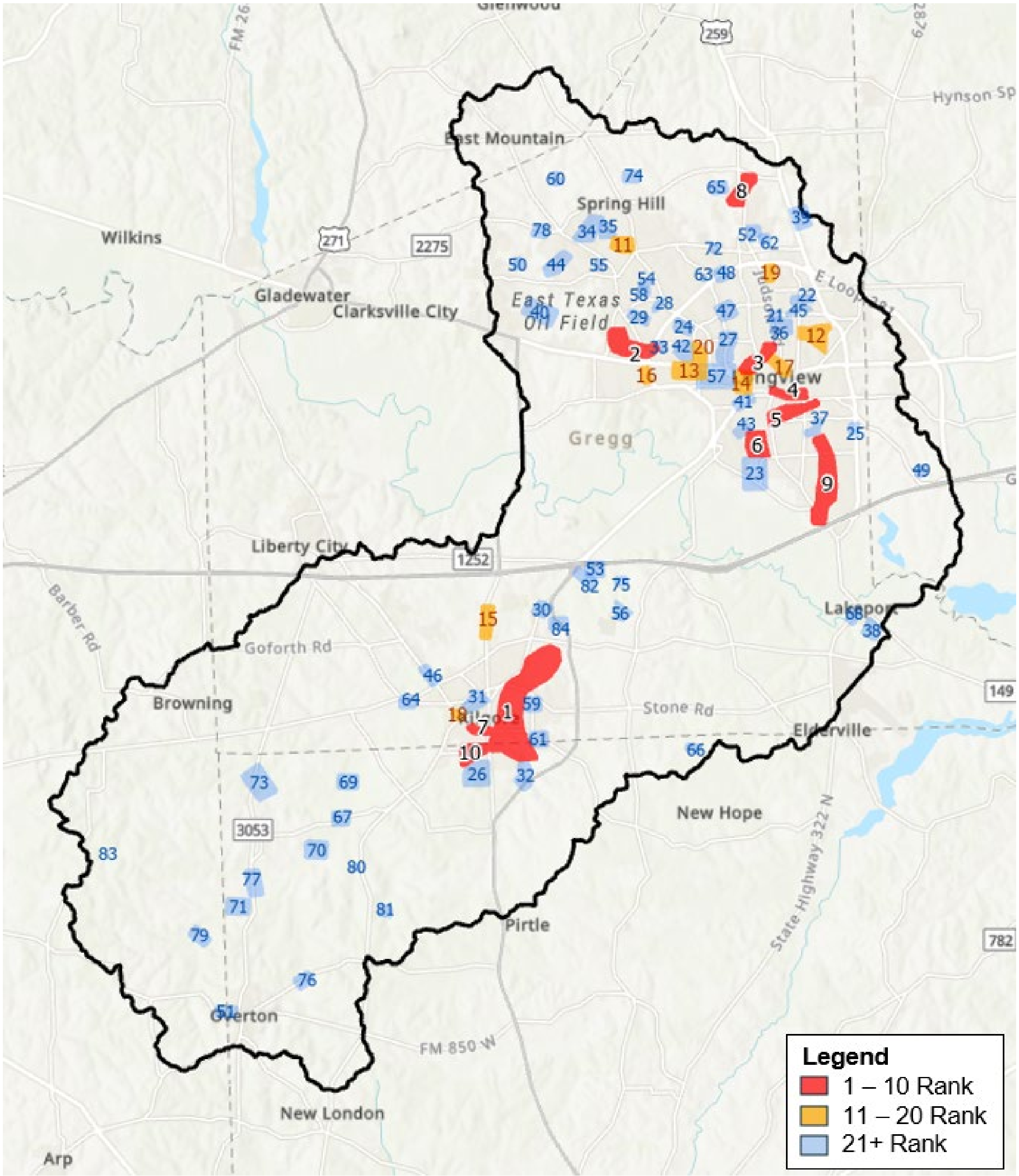


Figure 7: AOI projects with ranking numbers.

5.0 Flood mitigation planning alternatives analysis

5.1 Selected flood mitigation modeling areas

Based on the results from the flood hazard identification, the list below of identified areas in **Table 24** was created, concurrently with the stakeholders, as the proposed areas to be evaluated in further detail. The projects proposed in this plan are intended to be in alignment with the stakeholder's priorities such that the problematic flooding areas are addressed, and a plan is outlined to improve those areas. The evaluated projects where a project was not proposed are discussed further in **Section 5.2** and the evaluated projects with a proposed project are detail in **Section 5.3**. Images shown for each location show the depth raster for existing conditions from the HEC-RAS models. In those depth rasters, deeper blue indicates deeper water while lighter blue colors indicate shallower flooding.

Table 24: Selected areas for flood mitigation alternatives.

Location	Stream	Description
Kilgore	Bighead Creek	Bighead Creek Flooding
Longview	Harris Creek	Harris Creek Tributary Flooding
Longview	Iron Bridge Creek	Iron Bridge Creek Neighborhood Flooding
Longview	N/A	Longview Underpass Flooding
Longview	Wade Creek	Upper Wade Creek Flooding
Longview	Wade Creek	Lower Wade Creek Flooding
Kilgore	Turkey Creek	Turkey Creek Tributary Neighborhood Flooding
Longview	Guthrie Creek	Upper Guthrie Creek Neighborhood Flooding
Longview	Guthrie Creek	Guthrie Creek Corridor Flooding
Longview	Elm Branch	Elm Branch Flooding
Longview	Grace Creek	Grace Creek Flooding at US 80
Kilgore	N/A	SH 135 Underpass Flooding

5.2 Evaluated Areas of Interest (AOIs) without proposed projects

The areas presented in Section 5.2 were evaluated with modelling efforts, but do not currently have proposed projects within this study. These areas presented issues that were not able to be resolved under the current project and specific reasonings are outlined in their respective sections. Generally, solutions in these areas require watershed scale improvements, significant land acquisition, and improvements to state or interstate highways. Furthermore, preliminary screening anticipated there would need to be a significant amount of offsetting mitigation needed to achieve no adverse impact, as required for all projects proposed in this FIF study per Regional Flood Planning requirements. During evaluation of other projects in this study, it was determined that flood mitigation to offset improvements was needed in the immediate vicinity of the project. In many of these instances, land for offsetting mitigation was not available adjacent to the site or mitigation was anticipated to be significant in scale which may have made the project's cost so large that benefit-cost ratios were expected to be quite small.

Guthrie Creek corridor flooding

Overview
Guthrie Creek runs through the City of Longview and is constrained on both banks by development. The lack of conveyance capacity and lack of right-of-way within the channel and backwater from Grace Creek further limits opportunity for improvement in this area.

Identified root cause of flooding
The Guthrie Creek corridor is within a large drainage area of Longview that is fully developed. The channel is confined by development along its banks and has significant structural flooding in the 100-year event. Based on a profile of the 25-year frequency event (similar to the City's largest storm it has experienced), there do not appear to be significant head losses at structures indicating the bridges along Guthrie Creek may not be major restrictions to flow. However, the downstream water surface elevations are quite flat nearing the confluence with Grace Creek indicating tailwater influences from Grace Creek may be the driving force behind high water surface elevations along Guthrie Creek. **Figure 8** shows the existing 100-year inundation extents modeled along Guthrie Creek.

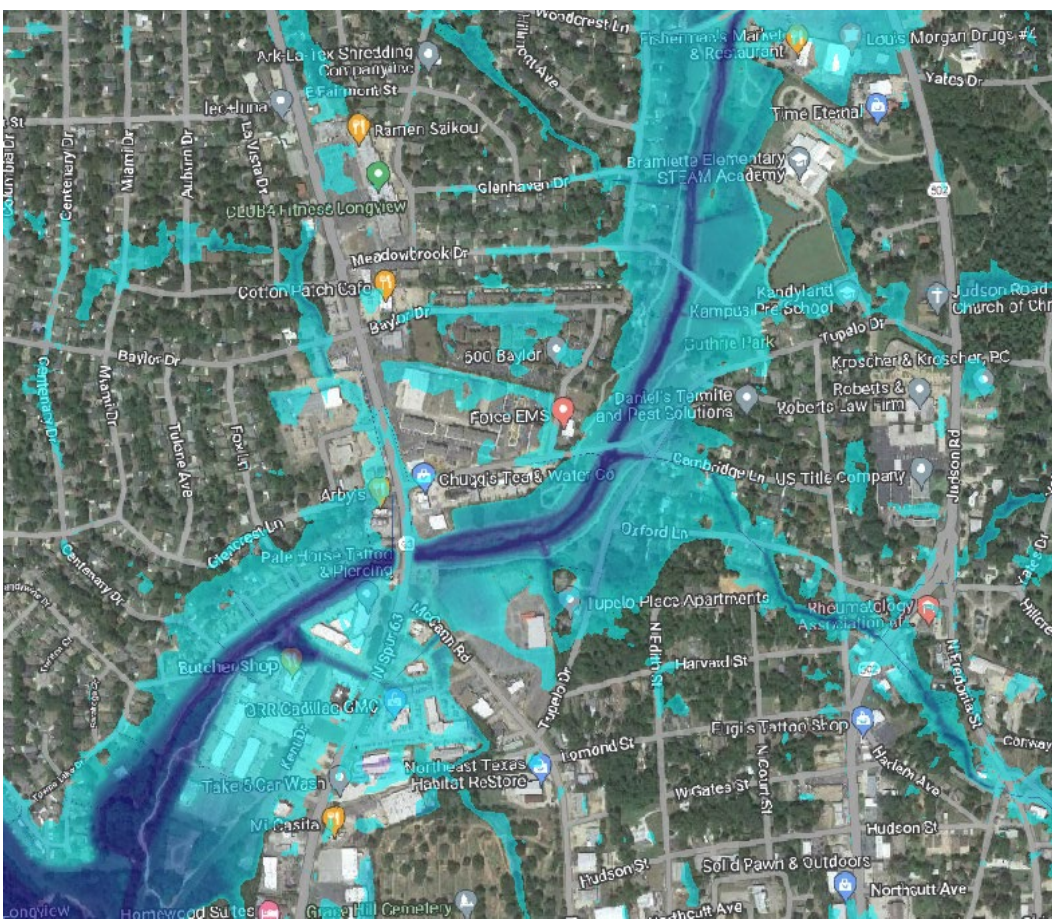


Figure 8: Guthrie Creek Corridor 100-year inundation.

Mitigation measures evaluated
Guthrie Creek Corridor improvements were modeled in HEC-RAS 6.3.1. Channel improvements were attempted, but severely limited by development on each side of the channel. A u-shaped concrete-lined channel alternative within the corridor was implemented. The channel improvement did not yield significant changes in WSE. Additionally, while a large u-shaped channel could be constructed, it would be quite deep and would likely have a significant safety hazard for residents in the area. Area for the implementation of detention is unavailable throughout the channel corridor.

A large regional detention basin approx. 23 acres in size was also evaluated near the confluence of Guthrie Creek, Oakland Creek, and Johnston Creek. It assumed a buyout of the shopping center at Triple Creek Drive and construction of a detention pond in the shopping center's current location. Preliminary results showed only about 0.5 foot of benefit on Guthrie Creek and the change in the floodplain extent in the critical area of Towne Lake was quite minimal. Thus, even a large regional pond showed minimal results and it did not appear to be economically reasonable to continue forward with mitigation in the area.

Recommendation
There are no projects recommended through the Guthrie Creek corridor as part of this study. The fully developed nature of the corridor adjacent to the top of bank limits the benefits possible within the project area without large-scale buyouts. A tunnel could be considered; however, the magnitude of the cost compared to benefits was estimated to be very low, so the possibility was not pursued as part of this study. Additionally, this area is influenced by tailwater conditions of Grace Creek and potential restrictions at downstream bridge and railroad crossings; thus, mitigation measures to aid Guthrie Creek would be reduced due to the downstream influences.

Upper Wade Creek flooding

Overview
Located in the southern portion of Longview, Wade Creek is a small tributary of Grace Creek. The Upper Wade Creek AOI extends through the southern developed section of Longview from East Cotton Street to South High Street. This portion of Wade Creek, which is narrow in size and closely surrounded by development, contains ten culvert crossings and existing conditions modeling shows extensive structural inundation during the 100-year event.

The upper portion of the creek often runs parallel to local roads south of Cotton Street. In many areas, the channel utilizes vertical or nearly vertical walls. Some examples of this can be seen at the following intersections:

- Oden Street at Clover Lane
- S. Mobberly Ave. at Sylvan Drive
- Timpson Street at Green Street

The constrained nature of the channel's corridor, especially by roadways, may potentially limit the number of potential improvements.

Identified root cause of flooding

Upper Wade Creek is situated in a large drainage area of Longview that is heavily developed. The channel is confined to a narrow area with many road crossings and encroachments along its banks from structures. Wade Creek can quickly overtop during a large rainfall event, inundating roadways and flooding nearby structures. Structure flooding and roadway overtopping begins in events as small as the 2-year event, with widespread inundation shown in the 100-year event. **Figure 9** shows the 100-year inundation extents modeled along Upper Wade Creek.

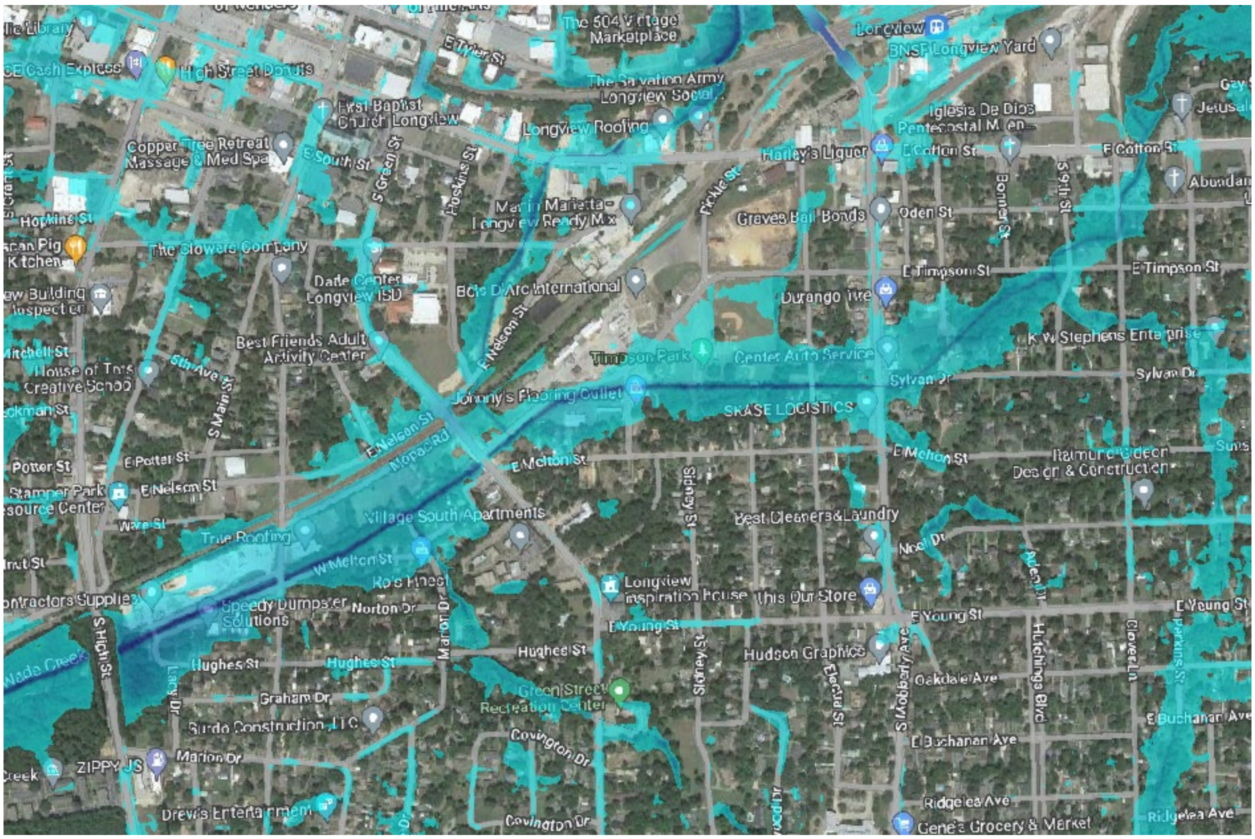


Figure 9: Upper Wade Creek 100-year inundation.

Mitigation measures evaluated

Improvements were modeled in HEC-RAS 6.3.1 to better convey stormwater through the narrow channel down to Lower Wade Creek and Grace Creek. Potential improvements modeled included combinations of converting the narrow channel to a u-shaped concrete-lined channel, improving all ten culvert crossings to larger bridge openings, buying out three structures located at the edge of Wade Creek and using the space for detention, as well as benching at the downstream end. The improvements alleviated some, but not a substantial amount of the residential flooding along Upper Wade Creek. Thus, it was determined that the benefits were not large enough to outweigh the potential cost to construct the improvements plus structure and land acquisition. Additional improvements were tested to include the maximum amount of detention available above East Cotton Street, as well as detention below South High Street, both in the undeveloped area available. However, these improvements did not provide significant benefits to the structural flooding occurring.

Detention was also simulated on a feasibility level in the upstream area near Whaley St. This was simulated by blocking off flow using a high berm simulating the effects of reduced flow due to volume storage. Results of the feasibility test showed minimal water surface elevation change in the area flowing through neighborhoods indicating that upstream detention would not be effective for flood mitigation.

Recommendation

The structural mitigation measures tested on Upper Wade Creek noted above alleviated some of the structural and roadway flooding but were not substantial enough to be recommended as part of this study. The large drainage area and developed nature of the land immediately adjacent to upper Wade Creek constrained the benefits of the possible alternatives. Significant buyouts and land acquisition would be needed to construct channel improvements or create detention. Land is not readily available due to the highly developed nature of this watershed and the creek running through the neighborhoods. While no structural project is proposed in the upper Wade Creek area as part of this study, it is recommended to perform regular maintenance of the channel to remove debris and vegetation growth for more efficient drainage.

It was noted in Google Street View and aerial images that there are sections of the creek with heavy vegetation growth. Thus, it is recommended that maintenance be done on the channel to achieve better efficiency of drainage.

Turkey Creek Tributary neighborhood flooding

Overview

Turkey Creek Tributary is located in Kilgore, Texas. It consists of mixed natural channel, storm sewer, and concrete lined channel starting south of Houston Street and West of Florence Street and located primarily within residential area following south of Crim Avenue and Dudley Road until the confluence with Turkey Creek.

Identified root cause of flooding

The low elevation in the neighborhood between Florence Street and S Martin Street is causing flooded structures during a 5-year event. Existing storm sewer data was not available, resulting in an assumed storm sewer system being added to the model to simulate anticipated flow patterns. The model results showed that the assumed storm sewer system added in the neighborhood did not achieve a 5-year LOS, indicating a potential capacity issue. Additionally, the crossing at S Henderson Boulevard appears undersized and acts as a restrictor. **Figure 10** and **Figure 11** shows the existing 5-year and 100-year inundation extents modeled along Turkey Creek, respectively.

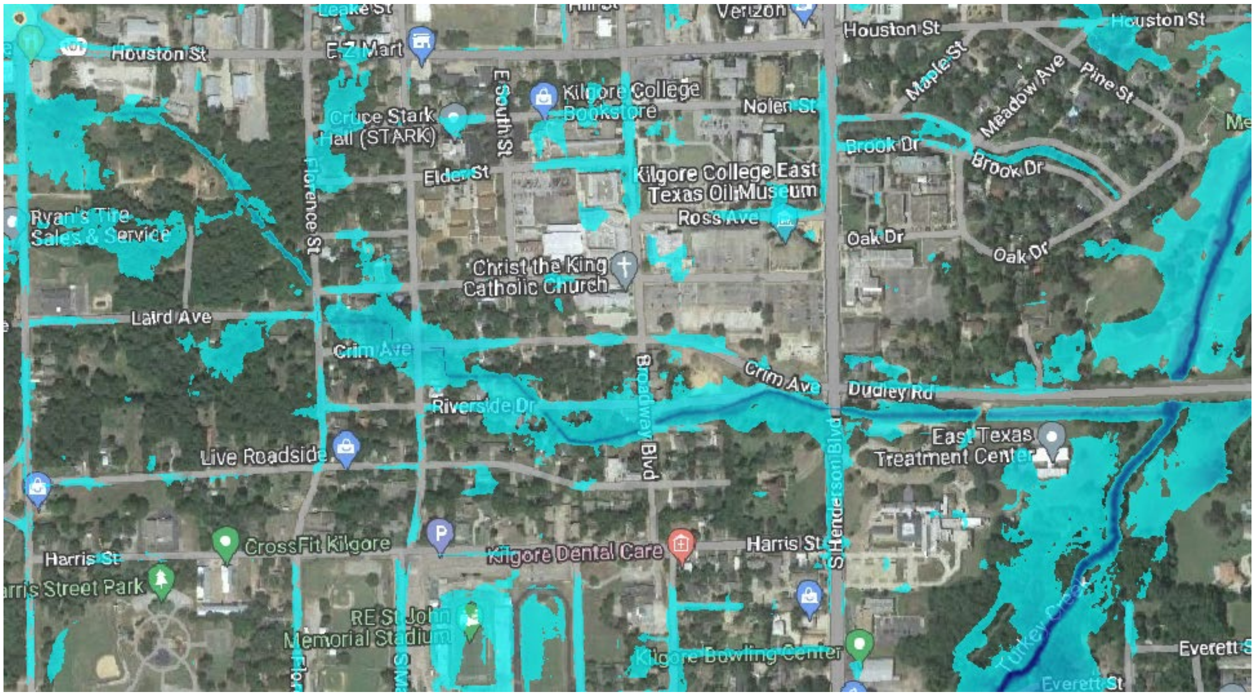


Figure 10: Turkey Creek Tributary 5-year inundation.

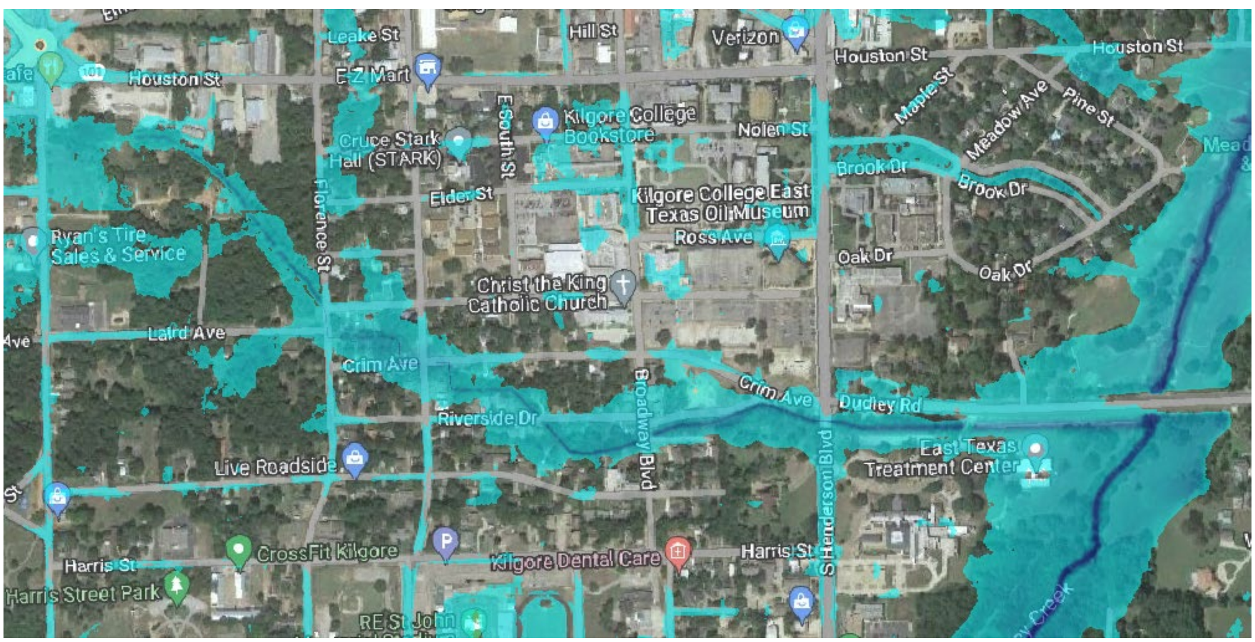


Figure 11: Turkey Creek Tributary 100-year inundation.

Mitigation measures evaluated

The proposed storm sewer system was upsized in the neighborhood along with inline detention and upstream channel clearing. The proposed inline detention upstream of S Henderson Boulevard helped during a 5-year and 100-year rainfall event, but the S Henderson Boulevard crossing acted as a restrictor. This led to adding upsizing of culvert crossings as mitigation for this area. This caused minor changes, with S. Henderson Boulevard still acting as a restrictor and the roadways being overtopped. Previous mitigation efforts indicated that Dudley Road (FM

1249) acted as an obstruction for Turkey Creek. The removal of FM 1249 was proposed to see if inundation along Turkey Creek Tributary would decrease from the lack of blockage along Turkey Creek. It was observed that removing FM 1249 did not relieve inundation along the Turkey Creek Tributary.

Recommendation

Further correspondence with the City of Kilgore led to de-prioritization of this area as the City indicated that flooding was not a large issue through this neighborhood. No proposed mitigation will occur for this AOI at this time.

Upper Guthrie Creek neighborhood flooding

Overview

Upper Guthrie Creek runs through the west side of Longview and is heavily restricted by development on both sides of the channel. Lack of conveyance capacity within the neighborhood storm sewer system and the channel results in structural flooding adjacent to Guthrie Creek.

Identified root cause of flooding

The structural flooding adjacent to Guthrie Creek is the result of undersized storm sewer and limited conveyance capacity within the channel. This results in ponding along the streets and potential structural flooding. It should be noted that storm sewer capacity was approximated in existing conditions due to lack of data. Additionally, the conveyance capacity of Guthrie Creek is further limited by backwater from Grace Creek. **Figure 12,** shows the existing 100-year inundation extents modeled along Upper Guthrie Creek.

Figure 12: Upper Guthrie Creek 100-year inundation.

Mitigation measures evaluated
Upper Guthrie Creek improvements were modeled in HEC-RAS 6.3.1. Due to the channel being fully developed on both sized of the channel, a concrete u-channel was evaluated. The implementation of a concrete u-channel did not result in significant changes to the inundation extents and resulted in adverse impacts (ie. increased water surface elevations) along Tulip Lane To mitigate the impacts, storm sewer improvements were implemented along Tulip Lane to provide additional conveyance capacity. The improvements along Tulip Lane removed the structures adjacent to Tulip Lane from the inundation extents but resulted in impacts downstream of the improvements. A small detention pond was added near Pegues Place but was unable to resolve the adverse impacts. The areas downstream moving into the Guthrie Creek corridor were evaluated for detention but did not have adequate undeveloped space for detention to be implemented.

Recommendation
While the improvements to the neighborhood drainage infrastructure was able to provide some benefit, the area lacked sufficient space to mitigate the flow increases from the conveyance improvements. The fully developed nature of the project area limited the opportunity for improvements.

Judson Road neighborhood flooding

Overview
The Judson Road Neighborhood AOI is located in central Longview along N Fredonia Street near its intersections with Spur 502 (Judson Road) and N Center streets. Existing conditions model results showed there was potential for roadway flooding in what appears to be a major transportation route from central Longview. The area is drained by storm sewers and a small channel which ultimately outfalls into Guthrie Creek near Tupelo Drive and Cambridge Lane.

Identified root cause of flooding
The flooding shown in the existing conditions model may be related to the modeled storm sewer system becoming overwhelmed or the local channel not having capacity to contain the runoff. The drainage area is fully developed and does not have significant areas for potential mitigation.

Mitigation measures evaluated
An initial detention pond was evaluated adjacent to N Fredonia Street across from Conway Street. Initial results showed some reductions, but none were significant to reduce the simulated flooding on the roadways where flooding was detected in existing conditions models. Subsequent coordination regarding this area indicated flooding was not a concern; thus, mitigation modeling ceased for this AOI.

Recommendation
Coordination with the City Engineer for the City of Longview indicated that the area does not experience issues with flooding. Thus, efforts were directed to other locations for potential flood risk reduction solutions.

5.3 Evaluated mitigation alternatives with proposed projects

Using the existing conditions HEC-RAS model, truncated models of the flood mitigation alternatives in **Table 24** were created and the modeled alternatives are detailed below. A summary of the evaluated alternatives, key benefits, cost, and constraints is shown in **Table 25**.

Table 25: Summary of flood mitigation alternatives.

Location	Stream	Mitigation Description	Benefit	Planning Level Cost
Kilgore	Bighead Creek – Dudley Rd	Channel improvements along Bighead Creek and Turkey Creek including bridge improvements to Dudley Road (FM 1249).	100-year LOS at Dudley Road	$18,756,000
Kilgore	Bighead Creek – Stone Rd	Increasing channel capacity through in-line detention storage via channel benching, property acquisition, bridge improvements to Stone Road and conveyance improvements downstream.	500-year LOS at Stone Road	$161,594,000
Longview	Drain No. 4 (Harris Creek Tributary)	Increasing channel capacity through geometry improvements and concrete lining, property buyout, and crossing improvements to Golfcrest Drive, N Lane Wells Drive, Pine Tree Road, and W Loop 281.	WSE reductions along Pine Tree Road: 5-year: 0.25- to 1.5-ft 25-year: up to 2-ft 100-year: up to 3.25-ft	$27,395,000
Longview	Longview Underpasses - High Street Underpass (Longview)	Improvements to storm sewer draining underpass and storm sewer trunkline along Cotton Street with small detention pond at existing outfall.	Reduced flood depths and drain times during the 5-, 25-, and 100-year events. 5-year: 5-ft depth reduction 5-year: Less than 30-min drain time	$6,604,000
Longview	Longview Underpasses - Green Street and Mobberly Avenue	Improvements to storm sewer draining underpasses and storm sewer trunkline, with channel improvements at existing Green Street outfall and detention pond at existing Mobberly Avenue outfall with 65 cfs pump station.	Reduced flood depths and drain times during the 5-, 25-, and 100-year events. 5-year: 9-ft reduction in depth, 10-hour reduction in drain time at Mobberly Avenue underpass 5-year: 2-ft reduction in depth, 30-min drain time at Green Street	$32,020,000
Longview	Iron Bridge Creek	Increasing channel capacity through channel geometry improvements, concrete lining, two detention ponds, and bridge improvements to Raney Drive, Lemmons Drive, Wells Street, and Millie Street.	WSE reductions along Wells Street: 5-year: up to 0.75-ft 25-year: up to 1.75-ft 100-year: up to 2.5-ft	$33,834,000
Longview	Grace/Lower Wade Creek	Proposed stormwater network in surrounding neighborhood, widening nearby drainage ditch, and adding an additional 150-foot opening to Loop 281.	100-year LOS for Loop 281. Reduced ponding and structural flooding in surrounding neighborhood	$29,515,000
Kilgore	SH 135 Underpass	Addition of inlets and a bypass storm sewer to a pump station. Pump station connects to channel improvements and detention pond.	Reduced flood depths and drain times during the 5-, 25-, and 100-year events. 5-year: 3.5-ft depth reduction 5-year: Less than 1-hour ponding time	$10,414,000

Location	Stream	Mitigation Description	Benefit	Planning Level Cost
Longview	Elm Branch	Increasing channel capacity through in-line detention storage via channel benching, property acquisition, downstream channel improvements, and bridge improvements to Irving Street, Ralph Street, and Miles Street.	5-year: 2.5- to 5-ft WSE reduction 25-year: 2- to 5.5-ft WSE reduction 100-year: 2- to 5.5-ft WSE reduction	$6,520,000

Bighead Creek flooding

Overview

Bighead Creek, along with Turkey Creek and other minor tributaries, in the City of Kilgore were modeled due to the overtopping of thoroughfares, connecting the east and west sides of the City, during major storm events. The main roads of focus for the study were Pentecost Road, Stone Road (FM 2204), Houston Street, and Dudley Road (FM 1249), listed from downstream (north) to upstream (south) along Bighead Creek. Both Pentecost Road and Stone Road are crossed by Bighead Creek while Houston Street and Dudley Road are intersected by Turkey Creek and Bighead Creek.

Identified root cause of flooding

These four roadways are overtopped by Bighead Creek, as well as Turkey Creek for Houston Street and Dudley Road, which causes mobility issues when trying to cross the City of Kilgore from east-west during large rainfall events. Dudley Road crosses Bighead Creek via a 110-foot bridge crossing, and Turkey Creek via five parallel 10-foot by 10-foot reinforced concrete box culverts. Those crossings act as constrictions, causing Dudley Road to be overtopped at both locations during the 25-year event. The existing conditions for the two crossings under Houston Street at Bighead Creek and Turkey Creek are five 9-foot by 7-foot RCBs and two 10-foot by 6-foot RCBs, respectively. The segment of Houston Street crossing Bighead Creek is overtopped by the 2-year event and has a flood depth of over 8 feet during the 100-year event. The Stone Road corridor is overtopped by the 10-year event and has a flood depth of over 6 feet in the 100-year event.

In existing conditions, the Stone Road bridge span is 115 feet. There is a constriction downstream where Bighead Creek flows under Stone Road, which coupled with the Bighead Creek floodplain corridor extending wide and deep, are the main constraints on floodplain improvements.

Mitigation measures evaluated

The modeling software used to model Bighead Creek and its tributaries was HEC-RAS 6.3.1. The two crossings under Dudley Road were brought to a 100-year design service level through several mitigation measures, shown on **Exhibit 1**. At the crossing of Bighead Creek, the existing section of road is proposed to be elevated by a maximum height of 3 feet, including widening the channel section 125 feet and adding benching to improve conveyance capacity in Bighead Creek at Dudley Road. This improvement does affect the bike trail located along Bighead Creek.

At the crossing of Turkey Creek, the existing parallel storm boxes are proposed to be replaced with a 128-foot open-span bridge crossing. In addition, the grass-lined channel was widened and included benching to improve conveyance capacity in Turkey Creek at Dudley Road. This improvement does affect one golf cart bridge. Also noted was that the modeled improvements to Stone Road discussed later in this section did not negatively impact the improvements to Dudley Road and vice versa.

At Houston Street, the depth of flooding over the roadway at Bighead Creek and constriction of Stone Road limited the mitigation efforts. Detention both upstream and downstream of Houston Street was modeled but considered ineffective in this area due to the size and depth of the floodplain. There are also transportation tie-ins with parking lots and adjacent streets that would require reconfiguration and reconstruction. Creating a design substantial enough to provide service over Bighead Creek while also mitigating adverse impacts created downstream during a 25-year event included improving the crossing to a bridge opening, raising the roadway 8 feet, and then adding an additional ten 10-foot by 10-foot RCBs to the crossing.

The improvements to Stone Road consisted of raising the roadway embankment and increasing the span of the bridge to 275 feet to allow conveyance of the 100-year flood event & 500-year flood event. Downstream of Pentecost Road, channel improvements were implemented to allow the channel to better manage the increased conveyance. Channel benching to increase storage within the channel was implemented upstream and downstream of the crossing. To maximize the additional in-channel storage, weirs were added upstream and downstream of Stone Road to create in-line detention. Each weir is 5-foot tall to avoid classification as a dam and has two 36-inch pipes for low flow. Holistically the proposed improvements include roadway and bridge improvements at Stone Road, 2,500 linear feet of channel improvements downstream of Pentecost Road, 100 linear feet of reinforced concrete pipe, and channel benching. The proposed improvements layout is shown in **Exhibit 9**.

Recommendation
The Dudley Road improvements, including updating both crossing to wider bridge openings, benching along the grass-lined channel, and raising the section of road near Bighead Creek by a maximum height of 3 feet, are recommended to be implemented based on the provided benefits. Improvements to the Houston Street crossings are not proposed, as they were deemed cost prohibitive and east-west access in Kilgore would be provided by Stone Road and Dudley Road. The Stone Road proposed improvements include roadway and bridge improvements, 2,500 linear feet of channel improvements downstream of Pentecost Road, 100 linear feet of reinforced concrete pipe, and channel benching. Additionally, both Stone Road (FM 2204) and Dudley Road (FM 1249) are TxDOT roads, and additional coordination would be needed for the mitigation improvements. A summary of the crossing improvements can be found below in **Table 26**.

Table 26: Road crossing improvements.

Roadway	Channel	Existing Conditions	Proposed Improvements
Dudley Road (FM 1249)	Bighead Creek	110-ft open-span bridge	• Elevate low chord a maximum of 3-ft • Widen bridge span to 125-ft • Add benching to US/DS of roadway crossing
Dudley Road (FM 1249)	Turkey Creek	5 – 10-ft by 10-ft RCBs	• Replace RCBs with RCBs • Update crossing to bridge span of 128-ft • Add benching to channel section at roadway crossing

Roadway	Channel	Existing Conditions	Proposed Improvements
Stone Road	Bighead Creek	115-ft open-span bridge	• Raise roadway embankment • Widen bridge span to 275-ft • Channel benching and in-line detention with weirs along Bighead Creek

Analysis of Flood Mitigation Project Benefits

The resulting benefits to the Bighead Creek study area for the Dudley Road improvements include increasing the LOS to 100-year and reducing the 100-year flood risk of an estimated 4 structures. The improvements will provide greater emergency transportation access to the City of Kilgore during major storm events. For more details on the risk reduction provided from this proposed channel improvement refer to **Appendix D**.

Exhibit 2 through **Exhibit 8** display the floodplain reduction for Dudley Road (FM 1249) improvements for the 2-year, 5-year, 10-year, 25-year, 50-year, 100-year, and 500-year storm events, respectively. **Exhibit 10** through **Exhibit 16** display the floodplain reduction for Stone Road improvements for the 2-year, 5-year, 10-year, 25-year, 50-year, 100-year, and 500-year storm events, respectively.

The alternative analysis and the ultimate recommended project was not designed to meet a specific LOS, but rather the intent was for the project to maximize benefit while working within the fixed constraints of the project site. For the Dudley Road and Stone Road improvement projects, the goal was to reduce roadway flooding along Dudley Road, therefore the LOS was determined by the lowest, most-frequent, storm event that did not overtop the roadway beyond 0.5-ft of depth. The existing LOS for Dudley Road was determined to be the 25-year event and the proposed LOS was estimated to be the 100-year event. The existing LOS for Stone Road was determined to be the 2-year event and the proposed LOS was estimated to be the 500-year event.

Table 27 and **Table 29** show the flow at key locations for the Dudley Road improvement project and Stone Road improvement project, respectively, while **Table 28** and **Table 30** show water surface elevation at key locations for the Dudley Road and Stone Road projects, respectively.

Table 27: Flows at key locations for Dudley Road.

Crossing	100yr Existing (cfs)	100yr Proposed (cfs)	500yr Existing (cfs)	500yr Proposed (cfs)
DS Dudley St at Turkey Creek	6663	6671	10286	10290
DS Dudley St at Bighead Creek	15115	14507	25088	25087

Table 28: WSE at key locations for Dudley Road.

Crossing	5yr Existing WSE (FT) NAVD88	5yr Proposed WSE (FT) NAVD88	25yr Existing WSE (FT) NAVD88	25yr Proposed WSE (FT) NAVD88	100yr Existing WSE (FT) NAVD88	100yr Proposed WSE (FT) NAVD88
US Dudley Rd – Bighead Creek Crossing	304.87	302.62	308.37	305.77	309.52	309.79
DS Dudley Rd – Bighead Creek Crossing	303.06	302.18	305.45	304.95	307.32	307.17
US Dudley Rd – Turkey Creek Crossing	310.49	307.94	311.91	309.61	312.65	310.76
DS Dudley Rd – Turkey Creek Crossing	309.01	307.65	310.00	309.16	310.52	310.10
US Pentecost Rd	297.93	298.01	301.48	301.57	303.61	303.54
DS Pentecost Rd	296.39	296.44	299.67	299.78	301.95	301.88
US Stone Rd	301.25	301.28	303.79	303.89	305.99	305.92
DS Stone Rd	299.20	299.27	302.68	302.79	305.03	304.96
US Bighead Ck Boundary Condition	311.35	311.33	313.77	313.63	315.56	315.39
Bighead Ck Trib - Boundary Condition	290.91	290.91	291.94	291.94	292.68	292.68
US Bighead Ck Boundary Condition	282.16	282.19	285.42	285.53	288.30	288.27
DS Bighead Ck Boundary Condition	279.68	279.70	282.04	282.15	284.27	284.25

For more details on the risk reduction provided from this proposed channel improvement refer to **Appendix D**. **Exhibit 10** through **Exhibit 16** displays the floodplain reduction for the 2-year, 5-year, 10-year, 25-year, 50-year, 100-year, and 500-year storm events, respectively.

Table 29: Flows at key locations for Stone Road.

Crossing	100yr Existing (cfs)	100yr Proposed (cfs)	500yr Existing (cfs)	500yr Proposed (cfs)
DS Dudley St at Turkey Creek	14115	13388	26216	25053
DS Dudley St at Bighead Creek	14214	13492	26488	25311

Table 30: WSE at key locations for Stone Road.

Crossing	5yr Existing WSE (FT) NAVD88	5yr Proposed WSE (FT) NAVD88	25yr Existing WSE (FT) NAVD88	25yr Proposed WSE (FT) NAVD88	100yr Existing WSE (FT) NAVD88	100yr Proposed WSE (FT) NAVD88
US Dudley Rd – Bighead Creek Crossing	304.39	304.19	307.29	306.56	309.46	309.07
DS Dudley Rd – Bighead Creek Crossing	303.16	302.91	305.70	304.80	307.76	306.95
US Dudley Rd – Turkey Creek Crossing	309.26	309.25	311.10	311.10	312.23	312.25
DS Dudley Rd – Turkey Creek Crossing	308.35	308.34	309.40	309.40	310.09	310.10
US Pentecost Rd	297.23	296.21	302.12	301.13	304.26	303.49
DS Pentecost Rd	295.09	293.25	298.64	296.53	301.14	299.25
US Stone Rd	301.62	297.99	304.47	302.62	306.64	305.40
DS Stone Rd	298.11	297.62	302.73	302.19	305.01	304.75
Boundary Condition - US FM 349	280.65	280.69	284.39	284.16	287.34	286.99
Boundary Condition - DS FM 349	277.58	277.60	280.27	280.10	282.74	282.46
Boundary Condition - Bighead Ck Tributary	289.76	286.93	290.60	288.55	291.21	289.90
Boundary Condition - DS Turkey Bighead	275.03	275.07	277.96	277.79	280.36	280.07
Boundary Condition - US Bighead Ck	311.16	309.52	313.70	312.02	315.56	314.03

Project Implementation & Constraints

A high-level evaluation of constraints and environmental data was analyzed for the feasibility analysis of these projects. Existing wetlands (US Fish & Wildlife Service, 2023), Railroad Commission (RRC) underground pipelines (RRC, 2023), overhead powerlines, critical habitats (NOAA Fisheries, 2023), Texas Historic Commission (THC) historic sites (THC, 2023) and other location-specific constraints were considered in the analysis.

Project constraints for Dudley Road include wetlands along both Turkey Creek and Bighead Creek. An existing bike trail along Bighead Creek and one golf cart bridge crossing Turkey Creek would also need to be adjusted to implement the new bridge crossing and channel improvements. No underground pipelines, overhead powerlines, critical habitats, or historic sites were found in the area. **Exhibit 1** shows the conceptual project layout with the potential constraints identified.

Project constraints for Stone Road also include wetlands along Bighead Creek and the existing bike trail. Subsurface data will be required to better understand the pipeline constraints. No overhead powerlines, critical habitats, or historic sites were found in the area. **Exhibit 9** shows the conceptual project layout with the potential constraints identified.

No Negative Impact
These two projects were found to meet the no adverse impact requirement, as described by TWDB as a maximum increase of 2D water surface elevation rounded to 0.3 feet (<0.35 ft) at each computation cell. It should be noted that within the Stone Road model there are flow increases at the downstream boundary of the model in the 2-year, 5-year, and 10-year hypothetical events. These increases are all less than 5% of the total flow and can likely be removed with further refinement of the bridge crossing. As shown on **Table 28** and **Table 30**, there are small impacts less than 0.35 ft to the frequency events, however, these impacts could be mitigated with additional mitigation measures such as detention.

Drain No. 4 (Harris Creek Tributary)

Overview
Located in the City of Longview, Harris Creek Tributary (also known as Drain No. 4 within the Gregg County Flood Insurance Study) begins at Creekside Drive and extends to Harris Creek around Avenue B. This area was identified as an area of interest due to the potential structural flooding present along this segment of the tributary from Scenic Drive to Avenue B based on existing conditions modeling in this study and the potential overtopping of several local roads in smaller events. This reach of the tributary also crosses Golfcrest Drive, N Lane Wells Drive, Pine Tree Road, and W Loop 281. Risk of flood damage appears to be present in rainfall events as frequently as the 5-year event.

Identified root cause of flooding
Based on an existing conditions 100-year event using a 24-hour duration storm, structural flooding appears to be present along the stream corridor from Scenic Drive to Avenue B. The project area was modified to include the contributing watershed and extended further downstream of the confluence to be able to evaluate for any potential downstream impacts. This truncated model was modeled in HEC-RAS version 6.3.1. The flooding in the area appears to be a result of Drain No. 4 being undersized for the conveyance passing through this area.

South of Harris Creek Tributary, between Golfcrest Drive and N Lane Wells Drive, the 5-year structural flood depths range from 0.5 feet to 2.0 feet. Mitigation measures in the local area aim to reduce the risk of structural flooding from Scenic Drive to W Loop 281. Channel and roadway crossing improvements of varying magnitude to reduce water surface elevation along the tributary were evaluated. In addition, the implementation of detention volume was also considered.

Mitigation measures evaluated
Improvements were modeled in HEC-RAS version 6.3.1. Starting at Scenic Drive the existing channel has concrete lining for approximately 1,000 linear feet, so expanding the concrete lining along the tributary to N Lane Wells was explored. While expanding the concrete lining did present some water surface elevation reductions, they were not significant enough to alleviate the risk of structural flooding in the area. Inline detention was coupled with the concrete lining to re-

evaluate the benefit in the area, however that did not result in significant added improvements compared to concrete lining alone. Upsizing the roadway crossing south of Golfcrest Drive to Pine Tree Road resulted in additional water surface elevation reductions relative to concrete lining alone. Inline detention was re-evaluated with the roadway crossing improvements and concrete lining; however, the addition of inline detention did not improve the water surface elevation reductions. Inline detention was removed from consideration as it did not seem to provide significant benefits. Additional conveyance capacity was still needed to alleviate flooding, and with limited right-of-way restricting potential locations, channel deepening was explored.

Upgrading the concrete-lined channel to also include channel deepening, from Golfcrest Dr. to Pine Tree Road, resulted in a significant reduction in water surface elevation. The water surface elevation reductions were significant relative to concrete lining alone and concrete lining coupled with crossing improvements. Although the reductions were significant, the roadway crossings through this stretch were improved to fully maximize the benefits. The addition of crossing improvements further reduced water surface elevations, compared to without. Although the results showed water surface elevation reductions the potential for further reductions were explored, since significant head losses were apparent at the crossings.

A pilot channel coupled with the concrete-lined channel deepening was evaluated. The pilot channel continued to reduce the water surface elevation, but significant head losses at the crossings were still apparent, so the crossings from Golfcrest Drive to Pine Tree Road were further improved to account for the pilot channel. Expanding the channel deepening and crossing improvements to slightly downstream of Loop 281, including some buyouts of properties to be able to improve the crossing at Loop 281 were explored to further evaluate the benefits.

Some increases in water surface elevation were apparent downstream of Loop 281, so channel clearing from Loop 281 and Avenue B were analyzed to reduce these impacts. Channel clearing was deemed useful in reducing impacts and was then carried over into the final proposed project. Full details of the conceptual project layout are shown on **Exhibit 17**.

Recommendation
Ultimately, the concrete lined channel deepening, a pilot channel, and crossing improvements from Scenic Drive through Loop 281 provided the most significant amount of water surface elevation reductions. To improve crossings at Pine Tree Road and Loop 281 and maximize benefits, some property acquisitions will be necessary between these crossings. All crossing improvements can be found in **Table 31**, below.

Table 31: Road crossing improvements.

Crossing	Existing Conditions	Proposed Improvements
Golfcrest Drive	Single 4.5-ft diameter culvert	35-ft span bridge
North Lane Wells Drive	Single 5-ft diameter culvert	30-ft span bridge
Pine Tree Road	Dual 5.5-ft diameter culvert	25ft x 10ft bridge opening
Loop 281	Single 10-ft x 7-ft box culvert	52-ft span bridge

Analysis of flood mitigation project benefits

The proposed channel improvements provide about 58 structures with reduced flood risk in the 100-year event. These improvements also removed the floodplain from 34 structures in the 100-year event. For more details on the risk reduction provided from this proposed channel improvement refer to **Appendix D**. **Exhibit 18** through **Exhibit 24** displays the floodplain reduction for the 2-year, 5-year, 10-year, 25-year, 50-year, 100-year, and 500-year storm events, respectively. The benefits to the 100-year event along Drain No. 4 from Scenic Drive to Avenue B are shown below in the profile plot on **Figure 13**.

Figure 13: Existing and proposed WSE profile comparison along Drain No. 4.

The alternative analysis and the ultimate recommended project was not designed to meet a specific LOS, but rather the intent was for the project to maximize benefit. The focus of this area was to minimize the risk of structural flooding resulting from the undersized channel. The LOS was estimated to be the lowest, most frequent, storm event analyzed that fully contained conveyance in the channel along the stretch of improvements and did not overflow beyond 0.5 feet above natural ground elevation, which was the assumed finished floor elevations for buildings. The existing LOS was determined to be 2-year, and the proposed is 100-year. **Table 32** and **Table 33** show flows and water surface elevation at several key locations for Drain No. 4, respectively.

Table 32: Flows at key locations – Drain No. 4.

Crossing	100yr Existing (cfs)	100yr Proposed (cfs)	500yr Existing (cfs)	500yr Proposed (cfs)
Golfcrest Dr	1054	1093	1545	1611
Hwy 281	1722	2271	2593	3184
US Avenue B & confluence with Harris Ck	1782	2375	2693	3307
Marshall Ave	6098	6011	9829	9863
Pine Tree Rd	1162	2054	2024	2950
DS Model Harris Creek	6278	6180	10146	10148

Table 33: WSE at key locations – Drain No. 4.

Crossing	5yr Existing WSE (FT) NAVD88	5yr Proposed WSE (FT) NAVD88	25yr Existing WSE (FT) NAVD88	25yr Proposed WSE (FT) NAVD88	100yr Existing WSE (FT) NAVD88	100yr Proposed WSE (FT) NAVD88
US Avenue B	325.78	325.68	326.67	326.58	327.64	327.54
DS Avenue B	324.12	323.84	326.09	325.94	327.22	327.08
US Hwy 281	336.48	333.75	338.37	335.63	338.82	336.73
DS Hwy 281	333.48	330.31	335.20	331.30	335.90	332.02
US Pine Tree	342.36	337.70	343.30	339.21	343.79	340.63
DS Pine Tree Rd	337.37	334.72	339.12	336.23	339.60	337.13
US Golfcrest Dr	356.05	351.64	356.47	353.10	356.85	353.53
DS Golfcrest	354.99	350.32	356.04	351.18	356.50	351.71
US Marshall Ave	307.87	307.77	309.35	309.25	310.76	310.72
DS Marshall Ave	307.69	307.58	309.24	309.14	310.67	310.64
Boundary Condition - US Harris CK	339.94	339.94	341.61	341.61	342.69	342.69
Boundary Condition - DS Harris CK	304.47	304.48	305.88	305.91	306.95	306.96

Project Implementation & Constraints

A high-level evaluation of constraints and environmental data was analyzed for the feasibility analysis of this project. Existing wetlands (US Fish & Wildlife Service, 2023), underground pipelines (RRC, 2023), overhead powerlines, critical habitats (NOAA Fisheries, 2023), historic sites (THC, 2023) and other location-specific constraints were considered in the analysis. There are multiple conflicts with storm lines and wetlands along the length of the channel improvements. The recommended channel improvements and required maintenance access will create a need for additional right-of-way and removal of encroachments from adjacent parcels. Three parcels are recommended for acquisition as part of this project. The exact channel right-of-way is not known, so there is potential for additional right-of-way needs along the channel improvements, that will be determined during detailed engineering and final design. Channel improvements downstream of Scenic Drive and downstream of Pine Tree Road conflict with the existing storm network system that will be removed. Several existing culvert crossings will be upgraded to pier and bent bridge crossing, with consideration for traffic control and access management to minimize neighborhood impacts during construction. Overhead powerlines, critical habitats, and historic sites were analyzed, but were not identified for this project. **Exhibit 17** shows the final proposed project to reduce structural flooding with the potential constraints identified.

No negative impact

This planning level project was found to meet the no adverse impact requirement, as described by TWDB as a maximum increase of 2D water surface elevation rounded to 0.3 feet (<0.35 ft) at each computation cell. Although **Table 33** shows slight water surface elevation increases, below 0.3ft, some water-surface elevation increases above 0.35 ft are also noted along the upstream portion of the channel but are determined to be a result of modeling computation instabilities at sudden geometry changes or breakover points rather than true water surface elevation increases. These increases are most notable in smaller storm events, like the 2-year and 5-year events but appear to diminish in gravity with larger storm events. This also led to the conclusion that the water surface elevation increase was a result of modeling instabilities as a result of the drastic difference in elevation from the banks of the channel to the channel bottom with the proposed channel improvements. These modeling instabilities were deemed improbable for the purposes of this feasibility study and are expected to be removed in detailed design.

Longview underpass flooding

Overview

Located in the City of Longview, the three roadways of North High Street, South Green Street, and South Mobberly Avenue pass underneath railroad tracks running east-west across the City. Based on record data provided by the City of Longview, the existing stormwater network appears to be undersized and does not adequately drain stormwater runoff during major rain events based on existing conditions modeling results. Historical flooding has been noted in these three underpasses, which quickly become impassable during substantial rainfall events. Several news articles were also found during the data collection phase regarding the flooding in the underpasses as follows:

- Heavy rains bring flooding to Longview area | | news-journal.com
- PHOTOS: Motorists challenge flooded underpasses | News | news-journal.com
- Officials responding to numerous areas around Longview due to high water | cbs19.tv
- Traffic Alert: Underpasses at S High/Cotton and S Green/Nelson in Longview reopened after flooding (kltv.com)
- Rain causing flooding issues in parts of Longview (kltv.com)

Shown below in **Figure 14**, which was taken during a field visit in August 2023, is the severe drop into the underpass. Notably, the roadway surface at the lowest level of the underpass cannot be seen from eye level in vehicles; thus, drivers cannot see whether the underpass is flooded until they have already entered the underpass where it is difficult to brake and turn around. This is consistent with information presented to the project team from City of Longview regarding drivers entering flooded waters in the underpass and needing rescue from the local fire department.

Adjacent streets near the underpasses – S. Court Street, S. Center Street, and S. Fredonia Street – are all on-grade intersections with the railroad. The High Street underpass, along with the Green Street and Mobberly Underpasses are critical emergency vehicle access routes, especially for the Longview Fire Station located 1 block away from the High Street underpass. This underpass allows emergency vehicle access from south of the railroad to the north side even when a train is passing.

Figure 14: Longview Underpass at High Street.

Identified root cause of flooding

The three underpasses identified receive stormwater runoff from urbanized, highly impervious drainage areas within the City of Longview. The curb inlets located at the lowest elevation of the underpass are connected to 18-inch storm sewers are inadequate to convey flows during the 5-year event. In addition to the small storm sewers draining the low point of the underpass, the GIS data of the provided by the City of Longview also showed a 24-inch RCP behind the western wall of the High Street underpass which can be seen in **Figure 15**. This line was modeled in addition to the 18-inch lines draining the low point of the underpass. The image below shows the storm sewer data provided by the City.

Figure 15: Existing storm sewers near the High Street underpass.

During extreme storms such as the 100-year event, or high intensity and short duration rainfall events, the underpass drainage system is rapidly overwhelmed, resulting in deep areas of ponding that are impassible and hazardous to the safety of the traveling public. Thus, the infrastructure draining the underpass does not have adequate capacity to drain the area which backs up water and the underpass becomes flooded.

Mitigation measures evaluated

A 2D ICM 2023.2.0 model was created of the drainage area for this portion of Longview to model the existing and proposed storm sewer system conditions during the 5-, 25-, and 100-year events. The existing City of Longview stormwater GIS shapefiles and Google Street View were used to create the models, which were the main sources for locating the inlets, stormwater lines, and sizes. However, there were gaps in the shapefiles, requiring some reasonable assumptions to be made about sizes, locations, drainage directions, and invert elevations of the network.

Due to the high depths of inundation shown starting in the 5-year event, the proposed condition stormwater design goal was to limit the amount of time where flood depths exceeded 0.5 feet in each underpass while also keeping any water surface elevation impacts to below the standard 0.35 feet required by TWDB. Each underpass was looked at independently to improve the flood conditions of these areas while optimizing the drainage design. The conceptual project layout is shown on **Exhibit 25**.

For High Street, the existing stormwater network (shown by the white lines in the figures below) included two inlets connected to 18-inch RCPs that then increased in size to the downstream outfall pipe size of 36-inches. The ICM model also included a 24” RCP line running behind the western wall in addition to the two 18-inch drains in the underpass. The storm sewer flowed west of the underpass outfalling in an existing concrete lined channel, shown on **Figure 16**. For the proposed conditions, pipe sizes were increased to 48-inch RCPs (represented by the dashed black line) along the same existing trunkline. The concrete channel outfall area at West Cotton Street and Grigsby Street was widened within a 1-acre parcel owned by the City of Longview to create in-line storage volume to accommodate the improved storm sewer system flow. The stormwater network was upsized to the maximum capacity possible within the available right-of-way for drainage infrastructure.

Figure 16: High Street existing drainage layout.

Green Street's existing stormwater network in the underpass included two inlets with 18-inch RCPs that reduced down to a 12-inch outfall. The outfall pipe discharges to a small drainage ditch east of the underpass that flows into a 36-inch RCP, that increases to a 48-in outfall south of the underpass. The layout of the existing network is shown below on **Figure 17**. The existing Green Street main trunkline was upsized to 6-foot by 3-foot RCBs to convey the existing flows east to the small open ditch. 0.1 acres of an adjacent City-owned parcel was used to widen the ditch to increase conveyance capacity. This next upsized section of the trunkline included 48-inch RCPs and 60-inch RCPs. The outfall of the line was directed into another City-owned parcel, north of the intersection of East College Street and East Nelson Street, of which 1.1 acres were used for detention.

Figure 17: Green Street existing drainage layout.

South Mobberly Avenue, the underpass with the greatest depth of inundation, was found to be drained by four inlets connected to 18-inch pipes that then flowed south, increasing in size downstream to the 36-inch outfall, shown on **Figure 18.** The main trunkline of this stormwater network was updated to 4-foot by 2-foot RCB and 6-foot by 4-foot RCB to convey the existing flow. The outfall of this trunkline was in a City-owned parcel in the northwest corner of East Timpson Street and Electra Street that was modeled as a 4.5-acre over-excavated detention area with a 65 cubic foot per second (cfs) pump outfall to contain more volume, maintain a dry bottom, and reduce any downstream impacts. This parcel does seem to currently be utilized as a laydown yard but will need to be repurposed to relieve flooding in the South Mobberly Avenue underpass. Additional alternatives of smaller detention ponds with no pump were modeled, however, too much conveyance capacity in the proposed storm sewer network caused the detention pond to overtop and too little capacity provided minimal relief to the underpass.

Figure 18: South Mobberly Avenue existing drainage layout.

Recommendation

Projects are recommended for each of these three underpasses to improve the safety and welfare of the traveling public and emergency vehicles during larger or heavier rainfall events in Longview. It is also important to note that adjacent intersections are on-grade with the railroad tracks; however, these underpasses serve as critical emergency access routes from north and south of the railroad tracks, especially for the fire station located close by. Thus, these 3 underpasses cannot be filled and brought up to on-grade intersections like the adjacent ones because they must continue to serve as emergency access routes.

These projects do not fully alleviate flooding to make the underpass completely dry during storm events but are effective at reducing the drain times during the 5-, 25-, and 100-year events as well as reducing the flood depths. The recommended project for the North High Street is upsizing the existing line to 48-inch RCPs and widening the existing concrete channel outfall to a 12 acre-feet detention pond at the corner of West Cotton Street and Grigsby Street. The recommendation for the Green Street underpass was to upsize the stormwater section to 6-foot by 3-foot RCBs, widen the City owned outfall ditch, upsize the next section of stormwater line to 48- and 60-inch RCPs, which would then outfall into a 10 acre-feet proposed detention pond. South Mobberly Avenue underpass is recommended to increase the existing stormwater line to 4-foot by 2-foot RCB and 6-foot by 4-foot RCB to convey the existing inundation to a proposed 105 acre-feet over-excavated detention pond at East Timpson Street and Electra Street with a 65 cfs pump outfall.

Based on the coordination with stakeholders, the High Street underpass was prioritized as the most critical underpass to improve. Due to this prioritization, the High Street underpass was selected to be further developed with a scope of work for a PER, discussed in **Section 5.5**. A summary of the underpass improvements can be found below in **Table 34**.

Table 34: Longview Underpass improvements.

Crossing	Existing Conditions	Proposed Improvements
N High St	• 18-inch storm sewers drain	• Upsizing line to 48-inch RCPs • Widening the existing concrete channel outfall to a 12 acre-feet detention area at West Cotton Street and Grigsby Street
N Green St	• 18-inch storm sewers drain	• Upsize the stormwater section to 6-ft x 3-ft RCBs • Widen the City owned outfall ditch • Upsize the next section of stormwater line to 48- and 60-inch RCPs, to outfall into a 10 acre-feet proposed detention pond
S Mobberly Ave	• 18-inch storm sewers drain	• Increase the existing stormwater line to 4-ft x 2-ft and 6-ft x 4-ft RCB to convey the existing inundation to proposed 105 acre-feet over-excavated detention area at East Timpson Street and Electra Street with a 65 cfs pump outfall

Analysis of flood mitigation project benefits

The mitigation alternatives for each underpass provide beneficial reductions in drain times and flood depths for improved mobility in the City of Longview during a storm event. The High Street underpass showed reduced drain times of less than 30 minutes during the 5-year event and a reduction in flood depths by over 5 feet, providing greater safety and mobility during a major event. South Green Street improvements provided a reduction in flood depths of 2 feet and a drain time of less than 30 minutes for the 5-year event. South Mobberly Avenue saw the greatest benefits during a 5-year event, with over 8 feet of depth reduction and over 10 hours of reduction in the drain time. For more details on the risk reduction provided from these proposed underpass improvements, refer to **Appendix D**. **Exhibit 26** through **Exhibit 28** display the floodplain reduction for the 5-year, 25-year, and 100-year storm events, respectively. Note that the 2-, 10-, 50-, and 500-year storm events were not modeled for the underpasses. **Table 35** shows the depths above 0.5 feet at each underpass. **Table 36** shows the duration of flooding above 0.5 feet at each underpass. **Table 37** and **Table 38** show the flow and water surface elevation at the outfall of each underpass, respectively.

Table 35: Existing and proposed depth above 0.5 feet at each underpass.

Underpass Name	5yr Existing (FT)	5yr Proposed (FT)	25yr Existing (FT)	25yr Proposed (FT)	100yr Existing (FT)	100yr Proposed (FT)
N High St	6.85	1.46	9.16	4.72	9.76	6.49
S Green St	3.08	1.07	3.71	1.25	4.13	2.60
S Mobberly Ave	13.78	5.00	13.92	10.80	14.02	13.34

Table 36: Existing and proposed time of depth inundation above 0.5 feet at each underpass.

Underpass	5yr Existing	5yr Proposed	25yr Existing	25yr Proposed	100yr Existing	100yr Proposed
N High St	1.5 Hours	Less Than 30 Minutes	2 Hours	Less Than 1 Hour	2.5 Hours	Less Than 1 Hour
S Green St	1 Hour	Less Than 30 Minutes	2 Hours	Less Than 1 Hour	2.5 Hours	Less Than 1 Hour
S Mobberly Ave	15.5 Hours	4 Hours	19.5 Hours	5.5 Hours	22.5 Hours	9 Hours

Table 37: Flow at outfall locations.

Underpass Outfalls	5yr Existing (cfs)	5yr Proposed (cfs)	25yr Existing (cfs)	25yr Proposed (cfs)	100yr Existing (cfs)	100yr Proposed (cfs)
N High St	163	165	171	188	175	192
S Green St	568	526	699	713	807.72	800
S Mobberly Ave	56	67	83	71	122	74

Table 38: WSE at outfall locations.

Underpass Outfalls	5yr Existing WSE (FT) NAVD88	5yr Proposed WSE (FT) NAVD88	25yr Existing WSE (FT) NAVD88	25yr Proposed WSE (FT) NAVD88	100yr Existing WSE (FT) NAVD88	100yr Proposed WSE (FT) NAVD88
N High St	276.91	276.94	277.04	277.35	277.11	277.41
S Green St	299.12	299.06	299.90	300.17	300.53	300.66
S Mobberly Ave	306.35	306.53	306.80	306.55	307.05	306.64

The recommended project was not designed to meet a specific LOS, but rather the intent was for the project to maximize benefit. For the Longview underpasses, the goals of the projects were to reduce the depths and durations of flooding at each underpass. The LOS of the project was based on the lowest, most-frequent, storm event in which the underpass had had ponding above 0.5-foot of depth for less than an hour. Since the analysis only involved the 5-year, 25-year, and 100-year events, the LOS for some scenarios with ponding deeper than 0.5 for over an hour was assumed to be smaller than the analyzed events. For High Street, the existing LOS was estimated to be a 2-year event and the proposed LOS was estimated to be a 100-year event. For the Green Street underpass, the existing LOS was estimated to be a 5-year event and the proposed LOS was estimated to be a 100-year event. For the Mobberly Avenue underpass, the existing LOS was estimated to be a 2-year event and the proposed LOS was estimated to be a 2-year event, however, the drain time was reduced by over 10 hours for the 5-year event.

Project implementation & constraints
A high-level evaluation of constraints and environmental data was analyzed for the feasibility analysis of these projects. Existing wetlands (US Fish & Wildlife Service, 2023), Railroad Commission (RRC) underground pipelines (RRC, 2023), overhead powerlines, critical habitats (NOAA Fisheries, 2023), Texas Historic Commission (THC) historic sites (THC, 2023) and other location-specific constraints were considered in the analysis. Project constraints included underground pipeline crossings with proposed underground storm lines and one proposed detention pond. Land acquisition will also be required for the South Green Street channel improvements, and the city owned parcel's current function as a laydown yard will need to be changed where the proposed detention pond is located for South Mobberly Avenue. Work within the ROW of many of Longview's streets will be required, which will necessitate a detailed traffic control plan. **Exhibit 25** shows the conceptual project layout with the potential constraints identified.

No negative impact
This project was found to meet the no adverse impact requirement, as described by TWDB as a maximum increase of 2D water surface elevation rounded to 0.30 feet (<0.35 ft) at each computation cell. As shown on **Table 38**, there are small impacts less than 0.35 ft to the frequency events, however, these impacts could be mitigated with additional mitigation measures in the final design.

Iron Bridge Creek neighborhood flooding

Overview
Iron Bridge Creek is a tributary to the Sabine River and is located adjacent to LeTourneau University, in southeast Longview. The channel is constricted between neighborhoods, particularly from Raney Drive through Millie Street – with an existing concrete-lined trapezoidal channel through this stretch. In existing conditions, structural flooding is present in various neighborhoods along the stream corridor. Due to the adjacent neighborhoods, channel improvements were limited from Raney Drive to Millie Street. Opportunity to add detention volume was also limited by development near Iron Bridge Creek and was evaluated exclusively where adequate undeveloped space was available.

Identified root cause of flooding
The channel appears to have limited capacity for the amount of stormwater runoff that flows through this area from the upstream drainage area. The undersized channel combined with the limited channel right-of-way between neighborhoods leaves minimal opportunities to alleviate the structural flooding present in this area. From Raney Drive to Millie Street structural flooding is present in as frequent events as the 25-year event where structural flood depth ranges from 0.5 ft to 2.5 ft. Mitigation measures will aim to reduce structural flooding in the local area.

Mitigation measures evaluated
Iron Bridge Creek Corridor improvements were modeled in HEC-RAS 6.2. The initial evaluation implemented several rounds of detention ponds upstream of Raney Drive to determine the best results to reduce the volume downstream. Preliminary modeling results showed detention alone was not effective in reducing the structural flooding in the project area.

In addition to a detention pond upstream of Raney Drive, channel improvements downstream were evaluated. To increase the conveyance and create in-channel storage, a concrete u-shaped channel was implemented from Raney Drive to Millie Street. Roadway crossings along the improved channel reach were all modified to maximize the benefit. Immediately downstream of the improvements, Millie Street and Margo Street appeared to be severely overtopped. Additional mitigation alternatives were explored to attempt to reduce overtopping at these crossings as well as further improve the in-channel storage capacity needed to contain the increased volume coming from upstream. Channel benching was added from Millie Street, through Margo Street, to slightly downstream of Margo Street.

After the channel benching was implemented, an additional detention pond was explored to mitigate the increased conveyance as a result of the benching. Detention was investigated north of Millie Street to attempt to offload some volume coming from a before the channel benching began, however that did not provide significant reductions. This second pond was then coupled with a third detention pond north of Johnson Street to catch the increased conveyance downstream of all channel benching and channel improvements. With the addition of the third pond, north of Johnson Street, the impacts were offset. The second pond, north of Millie Street was then re-evaluated to determine if the combined storage area from both downstream ponds was vital to mitigate impacts, or if the pond downstream of all improvements was producing the benefit. No significant changes were apparent when the pond north of Millie Street was removed, so detention in that area was no longer evaluated, only the pond north of Johnson Street.

During internal QC, the detention pond north of Johnson Street was deemed infeasible due to recent residential construction apparent in the same area. New Google Street View images, taken in February 2023, showed a new home had been constructed along Johnson Street; however, this house is not shown in the latest aerials which were taken in September 2021. After the detention pond downstream of improvements was deemed infeasible, new mitigation measures were necessary to offset the increased flows and water surface elevation increases due to the increased conveyance as a result of the channel benching. With overhead powerlines creating a concern on the west banks of Iron Bridge Creek from Margo Street to Old Elderville Road and apparent construction occurring in the open space east of Iron Bridge Creek within this stretch, there is no other appropriate area to provide detention to mitigate these increased flows and water surface elevations. The channel benching was then reduced iteratively to determine the necessary reduction to not cause downstream impacts. It was determined that all channel benching needed to be removed to fully remove these impacts, which in turn also removed the benefit of reduced overtopping at Millie Street and Margo Street.

In order to try and re-gain some of the reduction in overtopping at Millie Street and Margo Street, other mitigation measures were evaluated within this stretch, since backwater was severely apparent in this stretch. This stretch was also very heavily forested, so an initial test was run to see how channel clearing and clearing adjacent to the channel would provide benefit. The forest clearing test provided a positive sign of benefits. A grass-lined channel widening combined with clearing adjacent to the channel from Millie Street was evaluated. This alternative did seem to significantly reduce overtopping at Millie Street, however, did result in water surface elevation increases downstream of Margo Street. The previous detention pond evaluated north of Millie Street was then re-evaluated under the new mitigation options to create storage area to offload some conveyance coming downstream before a smaller tributary enters Iron Bridge Creek upstream of Millie Street, which was ultimately successful. The overall conceptual project layout is shown on **Exhibit 29**.

Recommendation
The proposed project consists of two detention ponds, channel improvements from Raney Drive to Millie Street in the form of a U-channel, and a grass-lined channel from Millie Street to Margo Street coupled with forest clearing in this stretch. The combined storage area from detention ponds located South of E Birdsong Street and North of Millie Street is approximately 54 ac-ft. There are approximately 4,500 linear feet (LF) of channel improvements from Raney Drive to Margo Street. A summary of the crossing improvements can be found in **Table 39**, below.

Table 39: Road crossing improvements.

Crossing	Existing Conditions	Proposed Improvements
Raney Dr	Dual 8-ft x 5-ft box culverts	34-ft open span bridge
Lemmons Dr	Four – 5-ft diameter circular culverts	54-ft open span bridge
Wells St	Three – 10-ft x 6-ft box culverts	53-ft open span bridge
Millie St	Three – 10-ft x 10-ft box culverts	52-ft open span bridge
Margo St	110-ft open span bridge	110-ft open span bridge

Analysis of flood mitigation project benefits
The proposed channel improvements provide about 55 structures with reduced flood risk in the 100-year event. These improvements also removed 20 structures from the floodplain in the 100-year event. For more details on the risk reduction provided from this proposed channel improvement refer to **Appendix D**. **Exhibits 30-36** displays the floodplain reduction for the 2-year, 5-year, 10-year, 25-year, 50-year, 100-year, and 500-year storm events, respectively. The benefits to the 100-year event along Iron Bridge Creek from E Birdsong Street to Margo Street are shown below on **Figure 19**.

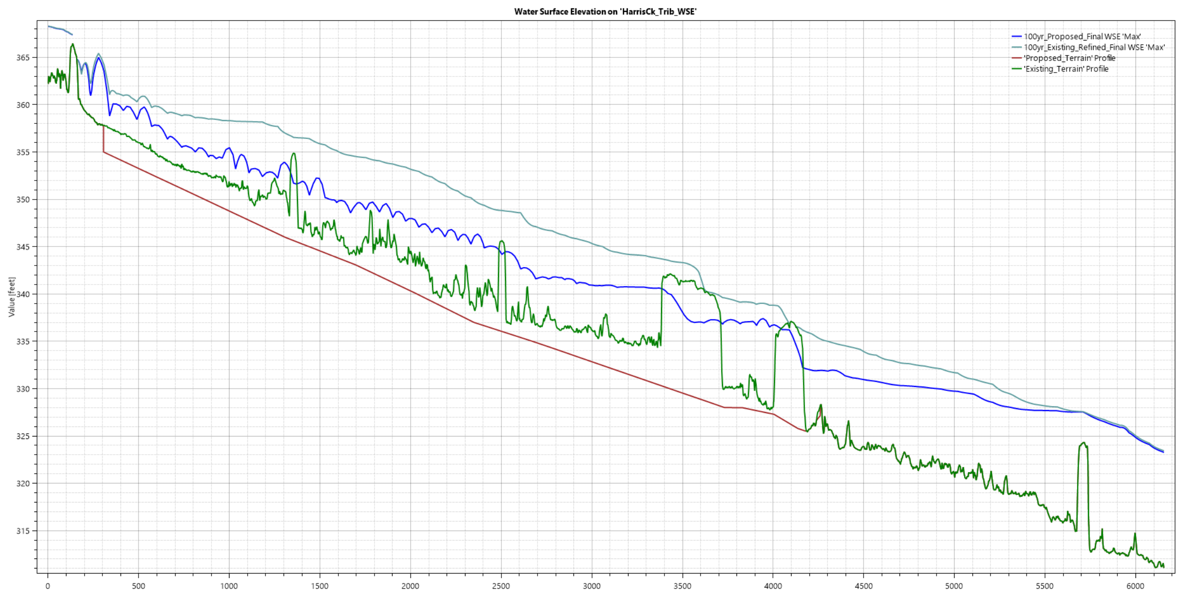


Figure 19: Existing and proposed WSE profile comparison along Iron Bridge Creek.

The alternative analysis and the ultimate recommended project was not designed to meet a specific LOS, but rather the intent was for the analysis to maximize benefit. The focus of this area was to reduce the risk of structural flooding resulting from the undersized channel. The LOS was estimated to be the lowest, most frequent, storm event analyzed that fully contained conveyance in the channel along the stretch of improvements and did not overflow beyond 0.5 feet above natural ground elevation, which was the assumed finished floor elevations for buildings. The existing LOS was determined to be 2-year, and the proposed is 10-year. **Table 40** and **Table 41** show the flow and water surface elevation at key locations for Iron Bridge Creek, respectively.

Table 40: Flows at key locations.

Crossing	100yr Existing (cfs)	100yr Proposed (cfs)	500yr Existing (cfs)	500yr Proposed (cfs)
DS Raney Dr	2254	1672	2896	3287
DS Lemmons Dr	2478	1990	3198	3627
DS Wells Dr	2513	2316	3313	3687
DS Millie St	2890	2850	3955	4113
DS Margo St	3112	3040	4506	4494
DS Estes Pkwy	3249	3241	4864	4762
DS I-20	2969	2764	4971	4868
DS Iron Bridge Ck 1	2973	2780	4992	4885
DS Iron Bridge Ck 2	1650	1613	2029	1992

Table 41: WSE at key locations.

Crossing	5yr Existing WSE (FT) NAVD88	5yr Proposed WSE (FT) NAVD88	25yr Existing WSE (FT) NAVD88	25yr Proposed WSE (FT) NAVD88	100yr Existing WSE (FT) NAVD88	100yr Proposed WSE (FT) NAVD88
US Raney Dr	311.17	306.70	311.97	308.59	312.52	310.21
DS Raney Dr	310.92	306.39	311.77	308.10	312.32	309.39
US Lemmons Dr	310.00	305.47	310.81	307.19	311.31	308.34
DS Lemmons Dr	308.40	305.13	309.61	306.84	310.48	308.07
US Wells St	303.92	299.84	305.06	301.53	305.78	303.34
DS Wells St	303.21	300.45	304.68	301.72	305.48	303.12
US Millie St	295.48	291.88	297.39	293.80	298.50	295.90
DS Millie St	294.66	290.98	296.66	292.90	298.16	294.50
US Margo St	289.29	288.78	291.14	290.78	292.58	292.40
DS Margo St	288.91	288.43	290.36	290.00	291.28	291.14
US Estes Pkwy	283.21	282.99	284.07	283.84	285.03	284.80
DS Estes Pkwy	281.43	280.90	283.21	282.75	284.64	284.32
US I-20	277.86	277.07	281.31	280.27	283.50	283.16
DS I-20	275.72	275.18	277.37	276.96	278.38	278.10
Boundary Condition - DS Iron Bridge Ck	268.15	267.80	269.15	268.90	269.92	269.67

Project implementation & constraints

A high-level evaluation of constraints and environmental data was analyzed for the feasibility analysis of these projects. Existing wetlands (US Fish & Wildlife Service, 2023), Railroad Commission (RRC) underground pipelines (RRC, 2023), overhead powerlines, critical habitats (NOAA Fisheries, 2023), Texas Historic Commission (THC) historic sites (THC, 2023) and other location-specific constraints were considered in the analysis. There are potential wetlands conflicts along the length of the proposed channel improvements. Along the stretch of proposed channel improvements, there are also several connections the existing storm sewer network links back into the channel that could be a potential conflict. The proposed forest clearing and channel improvements south of Millie Street conflict with an underground pipeline crossing through the channel. Several existing culvert crossings will be upgraded to bridge openings, which will require a traffic control plan to help navigate neighborhood traffic. Overhead powerlines, critical habitats, and historic sites were analyzed, but were not identified for this project. **Exhibit 29** shows the final proposed project to reduce structural flooding with the potential constraints identified.

No negative impact

This project was found to meet the no adverse impact requirement, as described by TWDB as a maximum increase of 2D water surface elevation rounded to 0.3 feet (<0.35 ft) at each computation cell. As shown in **Table 41**, there are no increases to water surface elevations at the key locations. Some water-surface elevation increases above 0.35 ft are noted along the upstream portion of the channel but are determined to be a result of modeling computation instabilities at sudden geometry changes or breakover points rather than true water surface elevation increases. These increases are most notable in smaller storm events, like the 2-year and 5-year events but appear to diminish in scale with larger storm events. This also led to the conclusion that the water surface elevation increase were a result of modeling instabilities as a result of the drastic difference in elevation from the banks of the channel to the channel bottom with the proposed channel improvements. These modeling instabilities were deemed improbable for the purposes of this feasibility study and are expected to be removed in detailed design.

Lower Wade and Grace Creek flooding

Overview

The lower portion of Wade Creek appears to experience structural flooding during the 100-year event from both direct rainfall and backwater from Grace Creek at Loop 281.

Identified root cause of flooding

The neighborhood surrounding both sides of Lower Wade Creek at Fuller Drive and Webster Street has short 24- to 42-inch stormwater outfalls but no significant stormwater network according to the City of Longview's storm sewer data. During a 100-year rainfall event, runoff quickly ponds along neighborhood streets and may inundate structures. In addition, backwater flooding from Grace Creek at Loop 281 causes some structural flooding in the neighborhood as well. These two sources of flooding were the main focus of the mitigation measures evaluated.

Mitigation measures evaluated

Upper Wade Creek mitigation measures were modeled in HEC-RAS 6.3.1. The existing section of Wade Creek through the surrounding neighborhood is a 30-foot wide concrete channel of maximum width. Alternatives to deepen this concrete section were ineffective at providing relief. In addition, the floodplain of Grace Creek in this area is wide and deep, rendering detention both upstream and downstream of the nearby neighborhood ineffective to alleviate flooding. Based on these existing constraints, a proposed design to alleviate neighborhood flooding included adding an extensive underground stormwater management network throughout the neighborhood. An additional 150-foot wide opening at Loop 281, west of the main bridge section, was proposed where a 36-inch culvert currently is located to relieve the backwater flooding on Grace Creek. The small ditch to the northeast of the neighborhood near Benton Street was also widened to 140 feet assist with drainage. These mitigation measures, shown on **Exhibit 37**, caused no impacts downstream to Grace Creek.

Recommendation

The mitigation measures employed on lower Wade Creek reduced structural flooding during the 100-year event and are recommended. These measures included designing a stormwater management network in the surrounding neighborhood, widening a drainage ditch adjacent to the neighborhood, and adding an additional 150-foot opening to Loop 281 downstream. A summary of proposed improvements are listed below, in **Table 42**.

Table 42: Lower Wade and Grace Creek improvements.

Crossing	Existing Conditions	Proposed Improvements
Loop 281	36-inch culvert	150-ft wide opening
Neighborhood flooding	Minimal neighborhood drainage	Underground stormwater network & 140-foot ditch widening

Analysis of flood mitigation project benefits

The proposed improvements to both the neighborhood and Loop 281 remove 9 structures and reduce the flood risk of 65 structures during the 100-year event. These improvements also provide a greater LOS to Loop 281 and the local neighborhood streets. For more details on the risk reduction provided from this proposed channel improvement refer to **Appendix D**. **Exhibits 38** through **44** displays the floodplain reduction for the 2-year, 5-year, 10-year, 25-year, 50-year, 100-year, and 500-year storm events, respectively.

The recommended project was not designed to meet a specific LOS but rather the intent was for the project to maximize benefit. For the Lower Wade and Grace Creek improvement project, the goal was to reduce roadway flooding along Loop 281 and reduce structural flooding as a result of an undersized storm sewer system in the nearby neighborhood. The LOS was initially determined separately for the improvement at Loop 281 and the neighborhood storm network improvements. The lowest, most-frequent, storm event that did not overtop Loop 281 or cause local roadway flooding were the storm network system was improved was used to determine the overall project LOS. For the roadway, the lowest storm event that did not overtop the roadway beyond 0.5-ft of depth in existing conditions was evaluated to assess LOS. The LOS at Loop 281 in existing conditions was identified to be the 25-year event, and the proposed was determined to be the 50-year event. For the local storm network improvements, the lowest storm event that did not cause local roadways to overtop above 0.5 ft as a result of the storm sewer system was used to determine the LOS. The existing LOS for the local neighborhood improvements was determined to be the 2-year event and the proposed LOS was estimated to be the 10-year event. Therefore, the overall existing LOS of the project was estimated to be the 2-year, and the proposed was determined to be the 10-year. **Table 43** and **Table 44** show the flow and water surface elevation at key locations for Iron Bridge Creek, respectively.

Table 43: Flows at key locations.

Crossing	100yr Existing (cfs)	100yr Proposed (cfs)	500yr Existing (cfs)	500yr Proposed (cfs)
DS HWY 81	23685	23195	42022	41816
Grace Creek/Wade Creek Confluence	23820	23355	42750	42547
Lower Wade Creek	3009	3026	7654	7746

Table 44: WSE at key locations in Lower Wade/Grace Creek.

Crossing	5yr Existing WSE (FT) NAVD88	5yr Proposed WSE (FT) NAVD88	25yr Existing WSE (FT) NAVD88	25yr Proposed WSE (FT) NAVD88	100yr Existing WSE (FT) NAVD88	100yr Proposed WSE (FT) NAVD88
US Garfield Dr	262.87	262.48	265.39	264.90	269.48	269.31
DS Garfield Dr	262.53	262.34	265.36	264.90	268.55	268.35
US Hwy 281	258.73	258.11	262.54	261.53	266.09	265.88
DS Hwy 281	257.79	257.49	260.25	260.28	264.53	264.39
Confluence	261.65	260.93	264.62	263.81	267.58	267.31
Boundary Condition - Grace Ck	265.91	265.54	268.42	268.30	271.60	271.48
Boundary Condition - Grace Ck Trib 1	265.94	265.59	268.38	268.27	271.55	271.44
Boundary Condition - Grace Ck Trib 2	266.10	265.76	268.45	268.34	271.60	271.48
Boundary Condition - Wade Ck Trib	265.53	265.58	266.09	266.09	267.70	267.46
Boundary Condition - US Grace Ck, DS confluence	265.23	264.79	267.91	267.76	271.19	271.05
Boundary Condition - DS Grace Ck	257.03	256.77	259.38	259.48	263.99	263.86

Project implementation & constraints

A high-level evaluation of constraints and environmental data was analyzed for the feasibility analysis of these projects. Existing wetlands (US Fish & Wildlife Service, 2023), Railroad Commission (RRC) underground pipelines (RRC, 2023), overhead powerlines, critical habitats (NOAA Fisheries, 2023), Texas Historic Commission (THC) historic sites (THC, 2023) and other location-specific constraints were considered in the analysis. Project constraints included wetlands and underground pipeline crossings along Loop 281 and in the neighborhood that is in conflict with the proposed crossing improvement and underground storm, respectively. Work within the ROW of Loop 281 and local neighborhood roads will be required, which will necessitate a detailed traffic control plan. **Exhibit 37** shows the conceptual project layout with the potential constraints identified.

No negative impact

This project was found to meet the no adverse impact requirement, as described by TWDB as a maximum increase of 2D water surface elevation rounded to 0.3 feet (<0.35 ft) at each computation cell.

Kilgore SH 135 underpass flooding

Overview

The State Highway 135 underpass is located underneath a Union Pacific Railroad (UPRR) on the west side of Kilgore just northwest of a traffic circle that is currently being reconstructed at SH135's intersection with Houston Street and S Commerce Street. The underpass' low point is approximately 15 feet below the nearby roundabout's elevation and is drained by 4 inlets. Two of these inlets are on-grade inlets between the roundabout and the low point of the underpass and drain via a 36" RCP north and outfall just north of the UPRR. The other two are located at the sag point of the underpass and drain into a single 24" RCP which drains north for 2,420 feet to ultimately outfall west of Bates Street. **Figure 20**, shown below, was taken during a field visit to the underpass in August 2023 where the photo was taken of the underpass viewing southeast.

Figure 20: SH 135 underpass in Kilgore, viewing southeast.

Identified root cause of flooding

The primary root cause of flooding is the small 24" storm sewers that drain the underpass. During a 100-year event, over 10 feet of flood depth occurs in the State Highway 135 underpass that slowly drain through the undersized existing storm sewer. Deep ponding also occurs during smaller storms, such as the 5-year event, which appears that it may be impassible by traffic during intense rainfall events.

Mitigation measures evaluated

The existing and proposed storm sewer system serving the underpass was modeled in 2D in ICM 2023.2.0 for the 5-, 25-, and 100-year events. The State Highway 135 plan set, provided by the City of Kilgore, and Google Street View were used to create the model. These were the main sources for locating the inlets, stormwater lines, and sizes. There were some unavailable culvert crossing sizes; thus, reasonable assumptions were made about sizes, locations, drainage directions, and invert elevations.

Model results indicated the underpass has impassible levels of inundation starting at the 5-year event. The proposed condition's goal was to limit the amount of time flood depths above 0.5 feet in the underpass while also keeping any water surface elevation impacts downstream to below the standard 0.35 feet required.

The proposed design included two new inlets in the sag, adjacent to the existing drainage line and inlets that were left alone. A 4-foot by 2-foot RCB connected the two inlets and drained to a proposed 90 CFS pump. Larger pump sizes were tested to see if the flood depth of the underpass could be reduced, however, these larger sized pumps caused downstream impacts above the standard 0.35 feet. The pump then outfalls through a 60-inch RCP to the undeveloped area northeast of the underpass. 20 acre-feet of excavation is proposed in the floodplain of this undeveloped area to help mitigate downstream impacts. The conceptual project layout is shown on **Exhibit 45**.

Recommendation
The proposed pump and outfall line are recommended for the State Highway 135 underpass to improve traffic and mobility during major flood events in Kilgore. This project does not fully alleviate the flooding of the underpass but is effective at reducing the drain times during the 5-, 25-, and 100-year events. The recommended project includes two additional inlets at the sag of the underpass, with a 4-foot by 2-foot RCB connecting the inlets, draining to a 90 CFS pump that then flows into a 60-inch RCP outfall northeast of the underpass in the undeveloped floodplain, 20 acre-feet of which included mitigation cut to reduce impacts. A summary of the underpass improvements can be found in **Table 45**, below.

Table 45: SH 135 Underpass improvements.

Crossing	Existing Conditions	Proposed Improvements
State Highway 135 underpass	• 4 inlets with low point approximately 15 ft below roundabout elevation • Two inlets are on-grade inlets and drain via a 36" RCP north. • Two other inlets located and drain into a single 24" RCP north.	• Two additional inlets at the sag of the underpass, with a 4-ft x 2-ft RCB connecting the inlets • A 90 CFS pump that flows into a 60-in RCP outfall northeast of the underpass in undeveloped floodplain • 20 acre-feet of the undeveloped floodplain to be cut to reduce impacts

Analysis of flood mitigation project benefits
The mitigation alternative for the underpass provides beneficial reductions in drain times and flood depths for improved mobility in the City of Kilgore during a storm event. The SH 135 underpass showed reduced drain times of less than 1 hour during the 5-year event and a reduction in flood depths by over 3 feet. For more details on the risk reduction provided from this proposed underpass improvement, refer to **Appendix D**. **Exhibits 46-48** displays the floodplain reduction for the 5-year, 25-year, and 100-year storm events, respectively. Note that the 2-, 10-, 50-, and 500-year storm events were not modeled for the underpasses.

Table 46 shows the depths above 0.5 feet and **Table 47** shows the duration of flooding above 0.5 feet in the underpass. **Table 48** and **Table 49** show the flow and water surface elevations at the outfall of the underpass, respectively.

Table 46: Existing and proposed depth above 0.5 feet at underpass.

Underpass	5yr Existing (FT)	5yr Proposed (FT)	25yr Existing (FT)	25yr Proposed (FT)	100yr Existing (FT)	100yr Proposed (FT)
SH 135	5.55	1.98	8.51	3.21	10.43	5.49

Table 47: Existing and proposed time of depth inundation above 0.5 feet at underpass.

Underpass	5yr Existing	5yr Proposed	25yr Existing	25yr Proposed	100yr Existing	100yr Proposed
SH 135	2.5 Hours	Less Than 1 Hour	4 Hours	1.5 Hours	5.5 Hours	2 Hours

Table 48: Flow at outfall location.

Underpass Outfall	5yr Existing (cfs)	5yr Proposed (cfs)	25yr Existing (cfs)	25yr Proposed (cfs)	100yr Existing (cfs)	100yr Proposed (cfs)
SH 135	124	91	189	151	241	193

Table 49: WSE at outfall locations.

Underpass Outfall	5yr Existing WSE (FT) NAVD88	5yr Proposed WSE (FT) NAVD88	25yr Existing WSE (FT) NAVD88	25yr Proposed WSE (FT) NAVD88	100yr Existing WSE (FT) NAVD88	100yr Proposed WSE (FT) NAVD88
SH 135	345.06	344.89	345.35	345.17	345.56	345.39

The alternative analysis and the ultimate recommended project was not evaluated to meet a specific LOS, but rather the intent was for the project to maximize benefit. For the State Highway 135 Underpass, the goal of the project was to reduce the depth and duration of flooding. The LOS of the project was based on the lowest, most-frequent, storm event in which the underpass had ponding above 0.5-foot of depth for less than an hour. The analysis only involved the 5-year, 25-year, and 100-year events and the lowest of these events for existing conditions showed over 0.5-foot of depth for longer than an hour. Therefore, the LOS was assumed to be smaller than the analyzed events and is considered a 2-year LOS. The proposed LOS was found to be 5-year.

Project implementation & constraints

A high-level evaluation of constraints and environmental data was analyzed for the feasibility analysis of these projects. Existing wetlands (US Fish & Wildlife Service, 2023), Railroad Commission (RRC) underground pipelines (RRC, 2023), overhead powerlines, critical habitats (NOAA Fisheries, 2023), Texas Historic Commission (THC) historic sites (THC, 2023) and other location-specific constraints were considered in the analysis. Project constraints included an underground pipeline crossing with proposed underground storm line. In the area of channel improvements, potential wetland conflicts are delineated. Partial parcel acquisition will also be required for this channel improvement. Work within the ROW of SH 135 will be required, which will necessitate a detailed traffic control plan. **Exhibit 45** shows the conceptual project layout with the potential constraints identified.

No negative impact
This project was found to meet the no adverse impact requirement, as described by TWDB as a maximum increase of 2D water surface elevation rounded to 0.3 feet (<0.35 ft) at each computation cell.

Elm Branch flooding

Overview
Elm Branch is a small tributary to Ray Creek and is located in northeast Longview, TX. The channel is relatively small and meanders through a neighborhood west of Spur 502. The City Engineer noted structural flooding in the area during historic storms along Irving Street and several roadways overtop in the local area during smaller storm events. Additionally, the neighborhood drains via roadside ditch which can become overwhelmed relatively quickly in large storm event.

Identified root cause of flooding
The flooding in the area appears to be the small channel relative to the size of the drainage area. At Spur 502, the contributing drainage area is approximately 520 acres draining through a mix of large-lot developed property and undeveloped areas.

Along Irving Street where historically flooded structures were identified, the 25-year flood depth (approximate level of the historic storm for the area) is approximately 1 to 1.5 feet deep. Mitigation measures in the local area will aim to reduce structural flooding in the lower Elm Branch area as there are few structures upstream of Spur 502 which appear to be at risk of flooding.

Mitigation measures evaluated
Mitigation modeling was performed with HEC-RAS version 6.3.1 and improvements were focused primarily on storm events up to the 25-year storm. Flooding in the residential area in excess of the 25-year event appears to be more influenced by backwater from Ray Creek; thus, improvements in the lower end of Elm Branch are not as significant. The 100-year event still has several feet of benefit, but this is localized closer to Spur 502 and Irving Street.

The possibility of detention was evaluated upstream of Spur 502 to reduce the flows in Lower Elm Branch to relieve the channel of excess flows. An artificial embankment at Spur 502 was simulated to completely block the upstream drainage area simulating effects of stored water (potential detention) and reduced flows in the lower portion of the creek. Results from this quick run showed a reduction in water surface but did not fully alleviate the flooding of lower Elm Branch downstream of Spur 502 as water still exceeded the channel’s banks. Thus, the reason the channel floods is due to limited capacity which is exacerbated by the large upstream drainage area. Detention upstream may have some improvement, but it did not appear that it would make a significant improvement and mitigation is likely better in the immediate local area. Because it did not appear to have significant benefits, upstream detention was not pursued for this area as it may not be very cost effective.

Channel benching was evaluated next between Spur 502 and Miles Street to create storage volume in the local area which was coupled with channel improvements from Miles Street to the confluence with Ray Creek. Benching was constrained to areas of open space to limit the number of structural buyouts needed for implementation. In addition to the creation of storage volume via benching, improvements to Irving Street, Ralph Street, and Miles Street were also evaluated to allow the water to drain more efficiently. These improvements are shown in **Exhibit 49**.

The potential implementation of this project could yield water surface reductions by several feet in the local neighborhood with no downstream impacts to Ray Creek. The southern roads of Miles Street and Ralph Street may not be able to be fully dry during a 100-year flood as Ray Creek produces a high stage which backs up Elm Branch flooding the intersections with Farmer Street. However, during smaller storms like the 5-year event, the water is contained within the channel banks in proposed conditions while it was outside of banks in existing conditions.

The bridges are not proposed to be elevated above more extreme event water surface elevations due to potential issues with tie ins with driveways for single family residential structures.

Recommendation

The recommended project for this area includes channel benching from Spur 502 (Judson Road) to Miles Street, and channel improvements downstream of Miles Street to the confluence with Ray Creek. It also incorporates conveyance improvements to Irving Street, Ralph Street, and Miles Street culverts. There is 1 structure along Irving Street that would require a buy-out in order to construct the benching on the north side of the creek. A summary of crossing improvements can be found below, in **Table 50**.

Table 50: Road crossing improvements.

Crossing	Existing Conditions	Proposed Improvements
Irving St.	3 – 6’ X 4’ box culverts	Channel benching w/ Bridge at crossing
Ralph St.	3 – 6’ X 4’ box culverts	Channel benching w/ Bridge at crossing
Miles St.	3 – 6’ X 4’ box culverts	Channel benching w/ Bridge at crossing

Analysis of flood mitigation project benefits

The proposed channel improvements provide 12 structures with reduced flood risk in the 100-year event. These improvements also remove 9 structures from the floodplain in the 100-year event. For more details on the risk reduction provided from this proposed channel improvement refer to **Appendix D**. **Exhibits 50** through **56** displays the floodplain reduction for the 2-year, 5-year, 10-year, 25-year, 50-year, 100-year, and 500-year storm events, respectively. The benefits to the 100-year event along Elm Branch from Judson Road to Ray Creek are shown below on **Figure 21**.

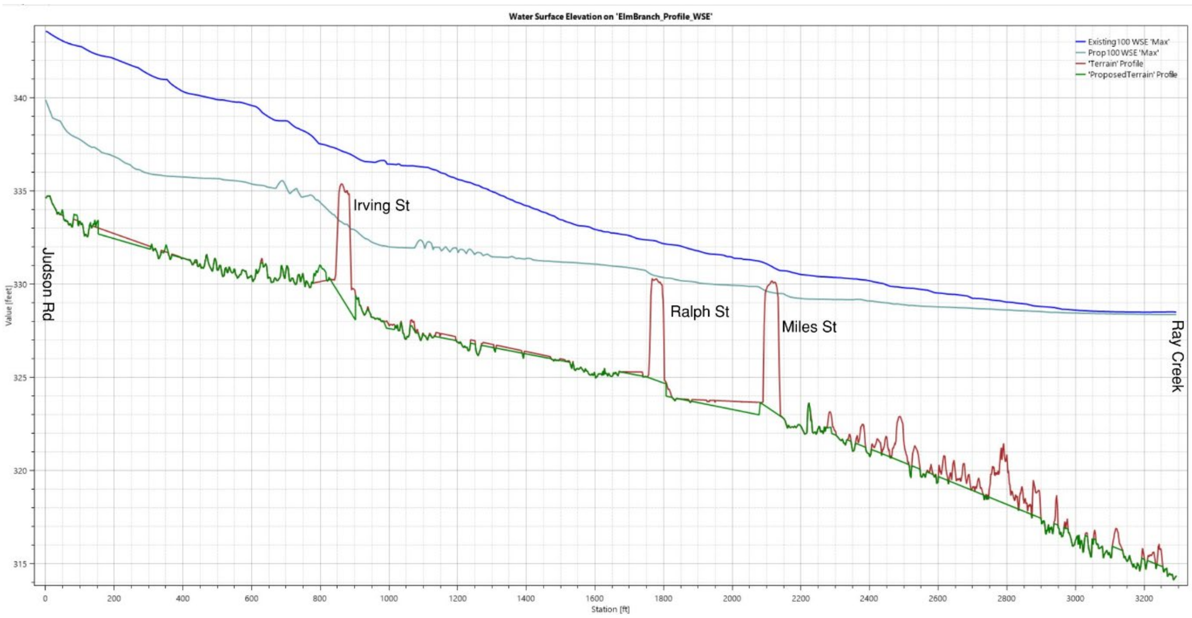


Figure 21: Existing and proposed WSE profile comparison along Elm Branch.

The alternative analysis and the ultimate recommended project was not designed to meet a specific LOS, but rather the intent was for the project to maximize benefit. For the Elm Branch improvement project, the goal was to reduce structural flooding. The LOS was estimated to be the lowest, most frequent, storm event analyzed that fully contained conveyance in the channel along the stretch of improvements and did not overflow beyond 0.5 feet above natural ground elevation, which was the assumed finished floor elevations for buildings. The existing LOS was determined to be 2-year, and the proposed is 25-year. **Table 51** and **Table 52** show the flow and water surface elevation at key locations for Elm Branch, respectively.

Table 51: Flows at key locations.

Crossing	100yr Existing (cfs)	100yr Proposed (cfs)	500yr Existing (cfs)	500yr Proposed (cfs)
Elm Branch / Wade Creek Confluence	5318	5219	8291	8289
DS Miles St	1900	1935	2513	2510
DS Ralph St	1877	1942	2438	2547
DS Irving St	1880	1909	2514	2514
DS Judson Rd	1782	1826	2383	2397

Table 52: WSE at key locations.

Crossing	5yr Existing WSE (FT) NAVD88	5yr Proposed WSE (FT) NAVD88	25yr Existing WSE (FT) NAVD88	25yr Proposed WSE (FT) NAVD88	100yr Existing WSE (FT) NAVD88	100yr Proposed WSE (FT) NAVD88
US Judson Rd	343.40	342.83	346.11	345.52	348.52	347.85
DS Judson Rd	341.78	338.16	342.96	339.08	343.53	339.57
US Ralph St	331.53	329.23	332.11	330.24	332.41	330.78
DS Ralph St	331.24	328.62	331.82	329.74	332.13	330.33
US Irving St	336.44	332.79	337.12	333.60	337.46	334.06
DS Irving St	335.65	331.43	336.36	332.31	336.71	332.76
US Miles St	330.33	328.03	330.95	329.27	331.28	329.89
DS Miles St	329.72	327.44	330.24	328.54	330.68	329.33
DS Elm Branch & Ray Ck Confluence	325.89	325.81	327.40	327.25	328.49	328.37
Boundary Condition - US Elm Branch	334.44	334.44	335.95	335.94	336.96	336.95
Boundary Condition - DS Elm Branch	315.51	315.40	317.69	317.58	319.33	319.24

Project implementation & constraints

A high-level evaluation of constraints and environmental data was analyzed for the feasibility analysis of these projects. Existing wetlands (US Fish & Wildlife Service, 2023), Railroad Commission (RRC) underground pipelines (RRC, 2023), overhead powerlines, critical habitats (NOAA Fisheries, 2023), Texas Historic Commission (THC) historic sites (THC, 2023) and other location-specific constraints were considered in the analysis. Existing wetlands, pipelines, and powerlines were considered. Some project-specific constraints include issues with the channel benching improvements crossing over wetlands area, and establishing a traffic control plan for when crossing improvements occur. Near the confluence of Elm Branch with Ray Creek, an underground pipeline crosses Elm Branch and the proposed channel improvements could conflict with the existing pipeline. Overhead powerlines, critical habitats, and historic sites were analyzed, but were not identified for this project. **Exhibit 49** shows the final proposed project to reduce structural flooding with the potential constraints identified.

No negative impact

This project was found to meet the no adverse impact requirement, as described by TWDB as a maximum increase of 2D water surface elevation rounded to 0.3 feet (<0.35 ft) at each computation cell.

5.4 SVI and LMI for recommended projects

For each of the project areas, the 2020 social vulnerability index (SVI) by census tract was reviewed for potential projects within the medium-high to high categories (CDC, 2020). An SVI of 0.5-0.75 is classified as a medium-high level of vulnerability, while a score of 0.75-1 is considered a high level of vulnerability. The SVI range for each project location is shown below in **Table 53**. This table shows that the Longview Underpasses, Iron Bridge Creek, Harris Creek, Grace / Lower Wade Creek, and Kilgore Underpass projects are all fully within the 0.5 to 1 SVI range. These SVI values are important to consider when implementing projects that benefit vulnerable populations and are typically required when seeking funding from grants and other sources.

Table 53: SVI range within each project area.

Project Name	SVI	Level of Vulnerability
Elm Branch	0.19-0.50	Low-Medium
Longview Underpasses	0.82	High
Iron Bridge Creek	0.74-0.86	Medium-High to High
Bighead Creek – Dudley Road	0.42-0.66	Partially Medium-High
Bighead Creek – Stone Road	0.42	Low-Medium
Harris Creek	0.70-0.85	Medium-High to High
Grace / Lower Wade Creek	0.74	Medium-High
Kilgore SH 135 Underpass	0.95	High

The low- and moderate-income (LMI) area data, based on 2011-2015 American Community Survey (ACS), was also reviewed per project area (HUD. 2019). Proposed projects may qualify for CDBG assistance if the AOI benefits at least 51% of low- and moderate-income persons. **Table 54** below shows the LMI ranges per project area. This table shows that the Longview Underpasses, Iron Bridge Creek, Harris Creek, Grace / Lower Wade Creek, and Kilgore Underpass projects are all fully in areas where the LMI is at least 51%.

Table 54: LMI range within each project area.

Project Name	LMI
Elm Branch	20-28%
Longview Underpasses	75%
Iron Bridge Creek	60-79%
Bighead Creek – Dudley Road	27%
Bighead Creek – Stone Road	33%
Harris Creek	30-72%
Grace / Lower Wade Creek	65-75%
Kilgore SH 135 Underpass	57-66%

5.5 Conceptual project development - PER for High Street underpass

Based on the high ranking of the Longview underpass flooding and coordination with the stakeholders, the High Street underpass was identified as a top priority. A scope of work for a PER was developed for the project and included in **Appendix E**.

5.6 Benefit-cost analysis

TWDB developed the Benefit-Cost Analysis (BCA) Input Tool to facilitate the calculation of flood mitigation benefits from a Flood Mitigation Project (FMP) for regional flood planning. This tool receives input of existing and proposed conditions to determine expected benefits related to the construction of the FMP in question. The benefits considered in the analysis include, but are not limited to, the reduction of damages to residential structures, commercial structures, and flooded street impacts. The BCA Input Tool was modified to handle larger structural datasets included in the analysis. The BCA Input Tool was used in conjunction with the Federal Emergency Management Agency (FEMA) BCA Toolkit v6.0.0.

The opinion of probable cost of the project estimates the total construction cost of the projects, and is included as **Appendix F**. The annual operations & maintenance cost was assumed to be 2.5% of construction costs, not considering costs associated with property acquisition. The project was assumed to have a useful life of 50 years. The project cost used in the BCA includes demolition activities, property acquisition, and construction activities related to channel improvements or crossing improvements. Property acquisition costs were estimated by pulling the appraised value of the parcel from the county appraisal district website and adding a 20% contingency to account for the incentive costs for acquiring the property.

The "Texas Buildings with SVI and Estimated Population (November 2021)" dataset provided by TWDB for Regional Flood Planning was used to determine building sizes and building types. The Finished Floor Elevation (FFE) for all structures was assumed to be 0.5 feet above ground level and the number of stories has been factored into group sizing of structures. Based on the provided building types, structures were reclassified as either residential, commercial, industrial, or agricultural. Public buildings were reclassified as commercial structures. Buildings marked as "Vacant or Unknown" in the TWDB dataset were reclassified as agricultural buildings. Since reducing structural flooding and water surface elevation was the major focus for a majority of the proposed projects, only structural damages were evaluated for those BCAs. For proposed projects that also focused on reducing overtopped roadways, flooded streets/ low water crossing benefit was also included in the BCA, as appropriate. Elm Branch, Harris Creek Tributary, and Iron Bridge Creek projects focused on reducing structural flooding, so only structural damages were considered for their analysis. Bighead Creek Flooding at Dudley Road and the Lower Wade/Grace Creek included the benefit associated with reduced roadway inundation and low water crossings along with structural damage reduction. The Kilgore SH 135 underpass, Longview High Street underpass, and Longview Green St and Mobberly St underpass projects evaluated structural damage reduction along with benefit associated with the reduction of flooding at these underpasses evaluated as a low water crossing for safety and traffic mobility. The depth and duration of flooding at these underpasses was assumed to equate to the similar amount of benefit as this reduction at a low water crossing. The flood depths for each structure within the study areas were determined for the 1%, 4%, and 20% annual chance events. The baseline and project damages for the projects with proposed mitigation alternative are included in the tables below. A breakdown of the Benefit-Cost-Ratios for each project can be found in **Appendix G**. **Table 55** below shows the baseline and with project damages calculated in the BCA as well as the final BCR calculated for the projects.

Table 55: BCR, baseline and with project damages for proposed projects.

Location	BCR	100-yr Baseline	100-yr With Project	100-yr Reduced Damages	25-yr Baseline	25-yr With Project	5-yr Baseline	5-yr With Project
Bighead Creek Flooding – Dudley Road	0.11	$28,400,291	$25,391,904	$3,008,387	$19,887,797	$14,992,747	$8,479,215	$4,352,916
Bighead Creek Flooding – Stone Road	0.09	$26,545,900	$25,619,187	$926,713	$16,450,389	$16,122,143	$6,040,693	$5,251,054
Drain No. 4 (Harris Creek Tributary)	0.22	$14,344,157	$10,827,497	$3,516,660	$8,746,763	$6,372,407	$4,169,197	$2,253,066
Longview Underpass – High St	0.20	$2,572,584	$1,956,637	$615,947	$1,833,666	$1,384,698	$985,147	$513,828
Longview Underpass – Green Mobberly Road	0.54	$15,561,115	$7,307,797	$8,253,318	$12,702,680	$4,445,090	$9,318,884	$2,542,762
Iron Bridge Creek	0.09	$9,419,456	$6,871,455	$2,548,001	$5,168,999	$4,225,522	$2,271,333	$1,914,525
Wade Creek & Grace Creek	0.26	$33,575,270	$28,216,672	$5,358,598	$14,248,519	$13,029,908	$8,091,901	$4,565,948
SH 135 Underpass	0.16	$2,397,224	$990,605	$1,406,619	$1,707,818	$1,293,870	$1,085,518	$517,253
Elm Branch	0.22	$1,504,575	$613,853	$890,722	$1,165,999	$466,260	$641,644	$334,672

5.7 Funding opportunities

There are several funding mechanisms with varying cost-sharing requirements available to fund infrastructure improvement projects. The projects recommended in **Section 5.3** shall be implemented in subsequent cycles of the TWDB Regional Flood Plans and are prime candidates for future TWDB FIF funding. The pursuit of FME or FMP funding for further development of the recommended projects could facilitate the further planning, property acquisition, and construction of the recommended projects. Additionally due to the wide-reaching benefits of the proposed projects, there may be opportunity to pursue Building Resilient Infrastructure and Communities (BRIC) and Flood Mitigation Assistance grants. It should be noted that both participation and standing in the national flood mitigation program may influence eligibility for these grants.

Additionally, implementation of green infrastructure and multiuse (i.e. detention pond that is also park space) in subsequent phases of the proposed projects can allow for improved benefits and increase likelihood of funding. A summary of potential funding opportunities is summarized in the table below.

Table 56: Possible funding summary.

	Flood Infrastructure Fund	**Texas CDBG - Community Development Fund (Grant)**	**Flood Mitigation Assistance Grant**	**BRIC Grant**
Sponsor	TWDB	TDA	FEMA/TWDB	FEMA/TDEM
Grant/Loan	Mixed Loan (0% interest) and Grant	Grant	Grant	Grant
Eligible Funding Activities	Drainage, flood mitigation and flood control such as: -Planning and design activities -Work to obtain regulatory approvals -Construction and implementation of flood projects	Plan, design and construct public works projects in nonentitlement communities	-Project Scoping, Community Flood Projects, Technical Assistance, Flood HMP, and Individual Mitigation Projects, including: -Acquisition for demolition or relocation -Structure elevation or reconstruction -Dry flood-proofing (non-residential and historic) -Minor, localized flood reduction projects -Structural/non-structural retrofitting -Soil Stabilization -HMP (flood hazard only) development or update	-Capability and Capacity Building (Including HMP development, project development, application development) -Mitigation Projects, with a focus on community-wide infrastructure projects and protection of lifelines
Cost Share	Varies	Varies; typically 75% Grant/25% Local	0% Local/100% Federal, for projects involving Severe Repetitive Loss Properties 10% Local/90% Federal, for projects involving Repetitive Loss Properties 25% Local/75% Federal, for projects involving NFIP insured properties	25% Local/ 75% Federal Higher local share receives more points Management costs 100% eligible
Project Funding Cap	Not explicit. Mixed grant and 0% interest rate loan	$350,000	$600,000 for Project Scoping $30 M for community flood projects; $50,000 TA; $25,000 for local HMP update Single family <$250,000 prioritized; Multifamily <$750,000 prioritized	$50 M for projects up to $300,000 per state may be used for planning activities

	Flood Infrastructure Fund	**Texas CDBG - Community Development Fund (Grant)**	**Flood Mitigation Assistance Grant**	**BRIC Grant**
Availability	On-going annual funding program for projects found in developed Texas State regional flood plans unless no funds available in any given year	Biennial	Annual	Annual
Requirements	-Must show what the benefit/cost ratio is (at least 1:1) for construction related projects -Memorandum of understanding with neighboring political subdivisions relating to the management of watershed when same is partially located outside the applicant's jurisdiction -Must have floodplain ordinances or orders in place and enforced to NFIP standards	Eligible applicants are non entitlement general purpose units of local government including cities (<50,000 pop) and counties (<200,000 pop) that are not participating or designated as eligible to participate in the entitlement portion of the federal Community Development Block Grant Program (CDBG)	-Community participation in the NFIP (and in good standing) -Mitigated property insured by NFIP -FEMA-approved and locally adopted HMP -Cost-beneficial project	-States that have had a major disaster declaration in the last 7 years (COVID-19 resets 7 years to start at 2020 for all states) -Community participation in NFIP (for projects located in SFHA) -FEMA-approved HMP -Cost-beneficial project

Acronyms for **Table 56**: BRIC = Building Resilient Infrastructure and Communities; EPA = Environmental Protection Agency; FEMA = Federal Emergency Management Agency; GLO = Texas General Land Office; HMP = Hazard Mitigation Plan; HUD = Department of Housing and Urban Development; NFIP = National Flood Insurance Program; NRCS = Natural Resources Conservation Service; PY = Program Year; SFHA = Special Flood Hazard Area; SFY = State Fiscal Year; TCEQ = Texas Commission on Environmental Quality; TDA = Texas Department of Agriculture; TDEM = Texas Division of Emergency Management; TPWD = Texas Parks and Wildlife; TSWCB = Texas Soil and Water Conservation Board; TWDB = Texas Water Development Board; USACE = United States Army Corps of Engineers; USDA RD = United States Department of Agriculture - Rural Development; WEP = Water and Environment Program

5.8 Project implementation

The projects recommended in **Section 5.3** have been developed to feasibility study level of detail and will require additional planning and design prior to construction. It is recommended that the various stakeholders work these projects into their capital improvement plans if possible and begin developing scopes of work for the preliminary engineering phases of the recommended projects. It is critical during the preliminary engineering phases to further refine the environmental and subsurface utility constraints of the projects to ensure that project implementation is feasible. Concurrently, it is recommended that stakeholders begin planning for pursuit of funding for the recommended projects to minimize direct cost of the various improvements.

6.0 Conclusion

Based on the calibrated existing conditions ROM model results, locations of primary flood hazard areas within the Upper Sabine River Basin were identified. These areas were evaluated with truncated models of proposed mitigation solutions of the existing flood risks. The results of this modeling effort provided 8 total proposed projects, listed below, in 7 areas of the study.

- Bighead Creek – Dudley Road
- Bighead Creek – Stone Road
- Drain No. 4 (Harris Creek Tributary)
- Longview Underpasses
- Iron Bridge Creek
- Lower Wade and Grace Creek
- SH 135 Underpass
- Elm Branch

The 8 total projects showed beneficial reductions in risk of flood hazards within the study area. The 8 projects were discussed with key stakeholders and based on those discussions, as well as the high ranking of the Longview Underpasses, the underpass at High Street was further developed to a scope of work for a PER.

The next steps for the projects summarized above is the development of preliminary engineering reports to advance the projects. This could be done through leveraging the feasibility level cost estimates developed for this project to budget for future design phases or pursuing funding for the projects. It is recommended that the various stakeholders pursue FME or FMP funding through TWDB's FIF program, or utilize an alternative funding source like federal funds, to support the additional planning and design necessary for the above projects to be constructed. Additionally, the refinement and adoption of an updated drainage criteria manual will be critical to minimize/eliminate the possibility of future development adversely impacting downstream communities.

6.1 Coordination with the Sabine Regional Flood Planning Group

The 2023 Sabine Regional Flood Plan (RFP) was also led by Freese and Nichols. The Sabine RFP had a deadline of March 31, 2023, for FMEs, FMSs, and FMPs to be included in the amended Sabine RFP which will ultimately roll into the state's first flood plan later in 2024. In early 2023, all 84 AOIs were included in the amended Sabine RFP. Additionally, 3 of the 8 projects ultimately identified in this study were fully ready with completed modeling, benefit-cost analyses, and data processing to be considered FMPs. From this study, 81 FMEs were created, and 3 FMPs were integrated in the Sabine RFP. The 3 projects from this study which became FMPs in the Sabine RFP were: Bighead Creek at Stone Road (FMP ID 043000031), Harris Creek Tributary (FMP ID 043000032), and Iron Bridge Creek (FMP ID 043000033). These projects were eligible to apply for funding for the 2024/2025 FIF funding cycle and would still be eligible for funding in the next cycle in 2026/2027.

The remaining 5 FMPs identified in this study are anticipated to be amended into the regional flood plan early in the second cycle of regional flood planning commencing in late 2024. It is expected that there will be one amendment for the Texas State Flood Plan ahead of the next FIF funding cycle in 2026/2027. Coordination with the RFPG and amending them into the Sabine RFP and ultimately amendment into the state flood plan will make them eligible for FMP funding for the next funding cycle. **Table 57** below outlines each of the 84 AOIs identified in this Upper Sabine FIF study and their FMX status within this study as well as the Sabine RFP. As noted above, while 8 FMPs were identified within this FIF study, only 3 were ready with all required items at the time of the Amended Sabine Regional Flood Plan in early 2023 to be included into the State Flood Plan. These 3 are shown in red in **Figure 22** long with the other 5 FMPs in orange which were identified within this FIF study, but were not fully ready to be included in the 2023 RFP.

Table 57: FMX Inclusion into Sabine RFP.

RFP FMX ID Number	FMX Name	Upper Sabine FIF Study Classification	2023 Sabine RFP Classification
043000031	Bighead Creek – Roadway Improvements to Stone Road	FMP	FMP
043000032	Harris Creek Tributary Flooding	FMP	FMP
043000033	Iron Bridge Creek Neighborhood Flooding	FMP	FMP
041000091	Upper Sabine FIF Phase 2	FME	FME
041000097	Bighead Creek Flooding – Stone Road	FME	FME
041000098	Guthrie Creek Corridor Flooding	FME	FME
041000099	Longview Underpass Flooding	FMP	FME
041000100	Upper Wade Creek Flooding	FME	FME
041000101	Lower Wade Creek Flooding	FMP	FME
041000102	Turkey Creek Tributary Neighborhood Flooding	FME	FME
041000103	Elm Branch Flooding	FMP	FME
041000104	Upper Turkey Creek Neighborhood Flooding	FME	FME
041000105	Tammy Lynn Drive Neighborhood Flooding	FME	FME
041000106	Upper Guthrie Creek Neighborhood Flooding	FME	FME
041000107	Harris Creek at US 80 Flooding	FME	FME
041000108	Grace Creek Flooding at US 80	FME	FME
041000109	Rabbit Creek at SH 42 Overtopping	FME	FME
041000110	US 281 Underpass	FME	FME

RFP FMX ID Number	FMX Name	Upper Sabine FIF Study Classification	2023 Sabine RFP Classification
041000111	Judson Road Neighborhood Flooding	FME	FME
041000112	SH 135 Underpass Flooding	FMP	FME
041000113	Drake Blvd Neighborhood Flooding	FME	FME
041000114	HG Mosely Roadway Flooding	FME	FME
041000115	Eden Drive Flooding	FME	FME
041000116	Coushatta Hills Creek Neighborhood Flooding	FME	FME
041000117	Grace Creek at Loop 281 Overtopping (South)	FMP	FME
041000118	Whispering Pines Roadway Flooding	FME	FME
041000119	Eastman Road Flooding	FME	FME
041000120	Florence Street Neighborhood Flooding	FME	FME
041000121	Grace Creek at Freemont Street Flooding	FME	FME
041000122	Secretariat Trail Neighborhood Flooding	FME	FME
041000123	Lorraine Court Neighborhood Flooding	FME	FME
041000124	Rabbit Creek at SH 31 Overtopping	FME	FME
041000125	US 259 and Remington Neighborhood Flooding	FME	FME
041000126	Harris Creek Local Road Flooding	FME	FME
041000127	Harley Ridge Road Flooding (South)	FME	FME
041000128	French Drive Neighborhood Flooding	FME	FME
041000129	Middle Guthrie Creek Flooding	FME	FME
041000130	Upper Iron Bridge Creek Neighborhood Flooding	FME	FME
041000131	Lakeport Neighborhood Flooding	FME	FME
041000132	Oak Creek Tributary Neighborhood Flooding	FME	FME
041000133	S Whatley Road Flooding	FME	FME
041000134	Grace Creek at Highway 31 Flooding	FME	FME
041000135	Rockwall Drive Neighborhood Flooding	FME	FME
041000136	Sabine Street and Railroad Flooding	FME	FME
041000137	Hawkins Creek Tributary Flooding	FME	FME
041000138	Coleman Drive Neighborhood Flooding	FME	FME
041000139	Rabbit Creek at SH 135 Overtopping	FME	FME
041000140	Grace Creek at HG Mosley Flooding	FME	FME
041000141	Grace Creek at Loop 281 Flooding (North)	FME	FME
041000142	Circle Road Neighborhood Flooding	FME	FME
041000143	N Whatley Road Flooding	FME	FME
041000144	Meadowbrook Neighborhood Flooding	FME	FME
041000145	Hill Street Flooding	FME	FME
041000146	Rabbit Creek at Fairmont Street Flooding	FME	FME
041000147	Lynnwood Street Neighborhood Flooding	FME	FME
041000148	Harley Ridge Overtopping (North)	FME	FME
041000149	Rolling Meadows Neighborhood Flooding	FME	FME
041000150	Lake Lamond	FME	FME
041000151	Evergreen Street Overtopping	FME	FME
041000152	Higganbotham Road Neighborhood Flooding	FME	FME
041000153	Meadowview Road Flooding	FME	FME
041000154	White Street Neighborhood Flooding	FME	FME
041000155	Airline Road Overtopping	FME	FME
041000156	Patio Street Neighborhood Flooding	FME	FME
041000157	Old Gladewater Highway Overtopping	FME	FME
041000158	Ray Creek at Pliler Precise Rd Overtopping	FME	FME
041000159	Cassidy Lane Overtopping	FME	FME
041000160	County Road 1114 Overtopping	FME	FME

RFP FMX ID Number	FMX Name	Upper Sabine FIF Study Classification	2023 Sabine RFP Classification
041000161	Mitchell Lake Outfall	FME	FME
041000162	County Road 1115 Overtopping	FME	FME
041000163	County Road 138 Overtopping	FME	FME
041000164	County Road 1110 Overtopping	FME	FME
041000165	Grace Creek at Hawkins Pkwy Flooding	FME	FME
041000166	FM 3053/1639 Flooding	FME	FME
041000167	McCann Creek Road Overtopping	FME	FME
041000168	Meadows Lane Flooding	FME	FME
041000169	McKay Street Overtopping	FME	FME
041000170	FM 3053 Overtopping	FME	FME
041000171	Hawkins Creek at George Richey Road Overtopping	FME	FME
041000172	Denman Road Overtopping	FME	FME
041000174	County Road 146 Overtopping	FME	FME
041000175	Rabbit Creek at Spinks Chapman Overtopping	FME	FME
041000176	Jamestown Road Overtopping	FME	FME
041000177	Bighead Creek at US 259 Flooding	FME	FME

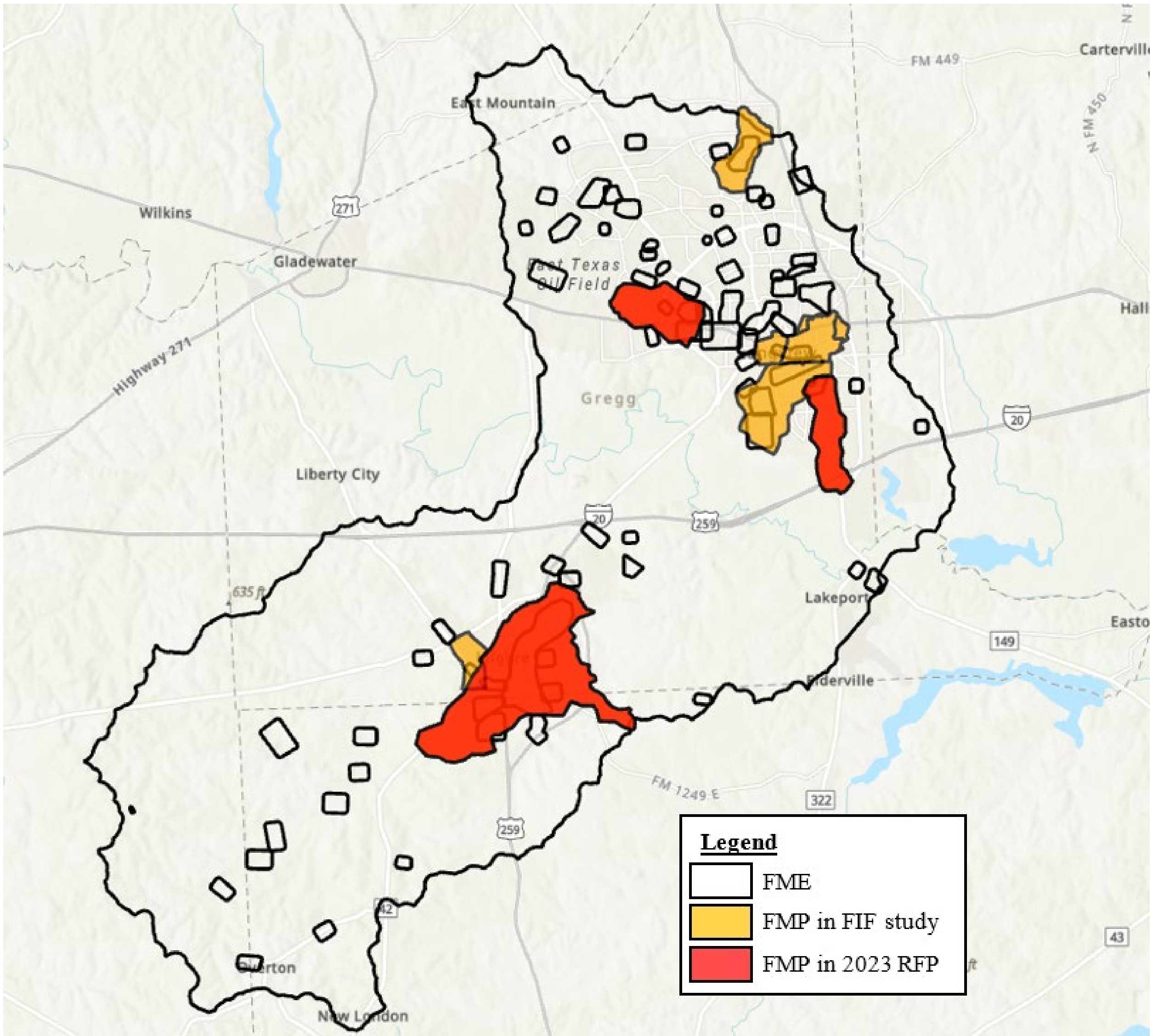


Figure 22: FMXs in FIF Study and 2023 RFP

7.0 References

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Appendix A - Public Meeting Information

Appendix B - Stakeholder Meeting Summaries

Appendix C - Exhibits

Appendix D - RFP Exhibit C Tables

Appendix E - Scope of Work, PER

Appendix F - OPCC

Appendix G - BCAs

Appendix H - Tech Memos

Appendix I - WSE Profiles