Final Report: Little River Hydrologic and Hydraulic Analyses

Texas Water Development Board Contract 40013

Prepared For:
Texas Water Development Board
City of Cameron

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Executive summary

The City of Cameron accepted a Flood Infrastructure Fund (FIF) Category I grant from the Texas Water Development Board (TWDB) in 2021. The scope of the grant's work was to complete a hydrologic study and hydraulic analysis to map the 100-yr and 500-yr floodplain elevations along the Little River. A hydraulic impact study and a sediment transport analysis of a preliminary design for a proposed raw water pump station for the City of Cameron was completed. The design of the proposed raw water pump station is not part of the Little River Watershed Study.

The project area is within the Lower Little River watershed (HUC #1207020403) and is mostly contained within Milam County, Texas starting near the county's western border and ending near the Brazos River. Floodplain maps were developed for approximately 19 miles along the Little River as shown in **Figure ES-1**. The City of Cameron was the key stakeholder for this project and 26 other stakeholders were invited to two public meetings. The invited stakeholders were

	Stakeholder		Stakeholder
1	Bell County	14	City of Jarrell
2	Milam County	15	Central Texas Council of Governments
3	McLennan County	16	Capitol Area Council of Governments
4	Williamson County	17	Heart of Texas Council of Governments
5	Falls County	18	Region G (Brazos) Water Planning Group
6	City of Temple	19	Brazos River Authority
7	City of Rogers	20	Sonterra Municipal Utility District
8	City of Buckholts	21	Bell County WCID #2
9	City of Holland	22	Bell-Milam-Falls Water Supply Corporation
10	City of Bartlett	23	Armstrong Water Supply Corporation
11	City of Little River-Academy	24	Salem Elm Ridge Water Supply Corporation
12	City of Troy	25	North Milam Water Supply Corporation
13	City of Moody	26	Minerva Water Supply Corporation

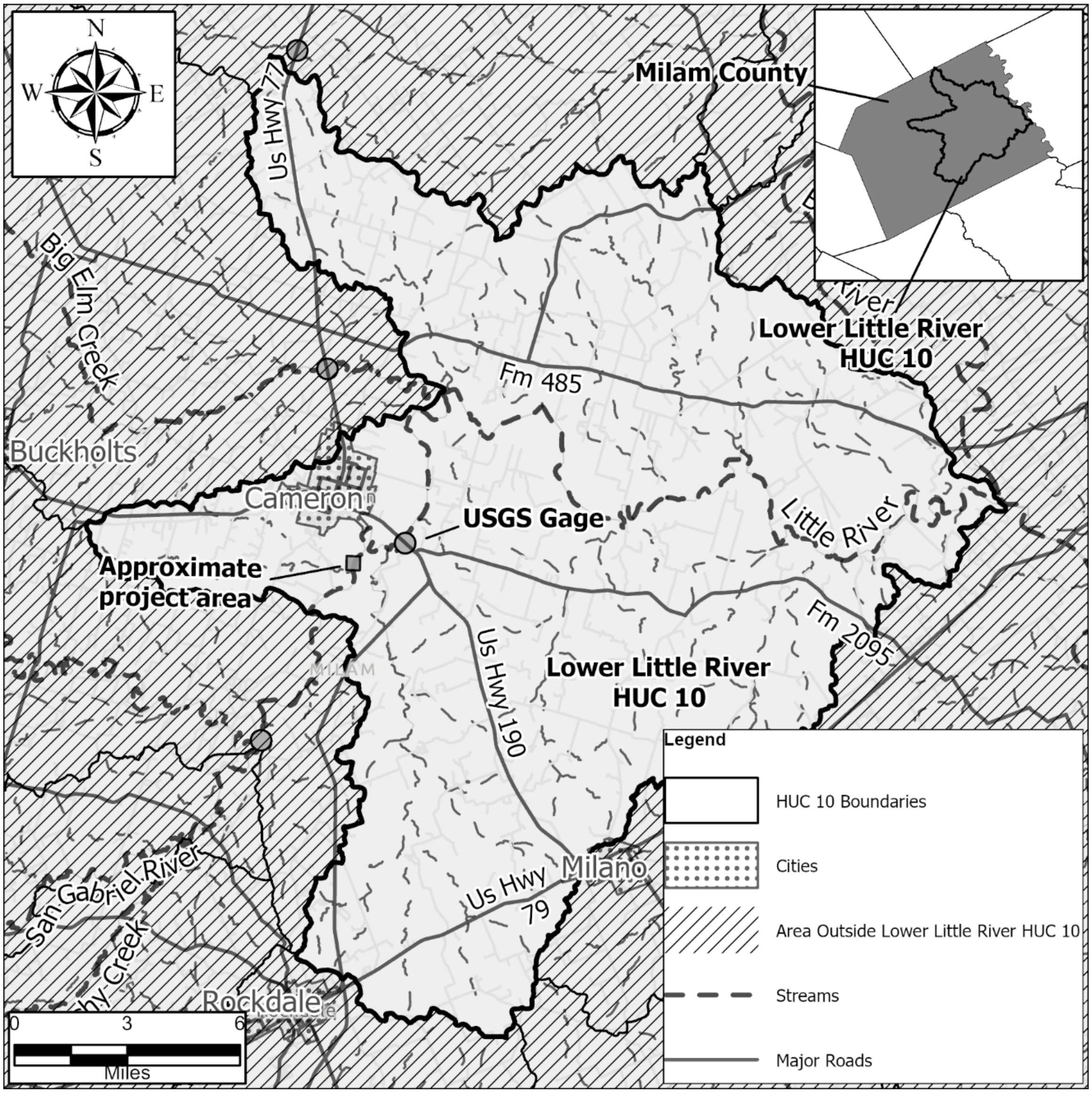

Figure ES-1. Lower Little River study area overview

Peak discharge for 2-, 5-, 10-, 25-,50-, 100- and 500-year return interval floods were calculated using a hydrologic software model developed for the Lower Brazos River Flood Planning Study. The 100-year and 500-year floodwater elevations were calculated using the US Army Corps of Engineer's HEC-RAS software modeling program (Version 6.2). The HEC-RAS model was calibrated to measured water surface elevations at a USGS stream gage station. The calculated water surface elevations from the calibrated HEC-RAS model were used to create a series of floodplain maps were developed for the entire 19-mile study area.

The hydraulic impact study was completed using the calibrated HEC-RAS model's calculated water surface elevations as a baseline (reference) condition. The calibrated HEC-RAS model was then modified using the geometry of a preliminary design of the proposed raw water pump station. The hydraulic impact study showed the preliminary design of the proposed raw water pump intake resulted in slight increases (0.05' or less) of the maximum water surface elevation during the 100-year flood event. These increases were localized around the pump intake structure. There are no habitable structures in the floodplain and the 100-year flood water elevations appear to satisfy local and county floodplain development ordinances. The proposed water intake structure and the low head dam do not cause any negative hydraulic impacts.

The sediment modeling analysis was completed using sediment size data collected near the proposed raw water intake and the embedded two dimensions (2D) sediment transport modeling in the US Army Corps of Engineer's HEC-RAS software modeling program (Version 6.2). The sediment modeling results showed no change in bed elevation around the tower of the proposed raw water intake pump station but suggest that sediment deposits will happen at least 1,8000 feet upstream of the low head dam. The no change in bed elevation was consistent with sediment competency results.

It is recommended the water intake structure be built in the proposed location. It is recommended that the hydraulic impact study and sediment transport analysis be updated if the preliminary design for the raw water pump intake changes during final design.

1 Study background and purpose

The City of Cameron accepted a Flood Infrastructure Fund (FIF) grant from the Texas Water Development Board (TWDB) in 2021, which will support the hydrologic and hydraulic analyses presented in this report. This study seeks to establish the 100-yr and 500-yr floodplain elevations along the Little River and support the design of a proposed pump station for the City of Cameron. These floodplains will be established for a portion of the Little River within the Lower Little River Hydrological Unit Code (HUC10) watershed and will include inflows from the following drainage basins: Upper Little River, Big Elm Creek, and Lower Little River. This report documents the modeling effort for the Lower Little River watershed which consists of an evaluation of existing conditions, calibration, alternatives analysis, and sediment management analysis.

This study, referred to as the Little River Hydrologic and Hydraulic Analysis, was funded by the TWDB as part of their FIF Category I grant program. A key stakeholder for this project was the City of Cameron who submitted an application and was awarded a TWDB FIF Category I grant. The grant agreement included a scope of work. The scope of work included seven tasks whose proposed work is summarized below,

Task 1: Project Management – This task included general project maintenance for schedule, budget and coordination of the project. This task included facilitating two public meetings and client, contractor meetings.

Task 2: Data Collection - Work in this task involved obtaining existing hydrologic models and terrain data.

Task 3: Hydrologic Analysis – The existing Lower Brazos Flood Protection Planning Study hydrologic model was reviewed and adapted to calculate discharges for the little River Hydrologic and Hydraulic Analysis. Seven storm events were modeled.

Task 4: Hydraulic Analysis – A two dimensional (2D) hydraulic model (HEC-RAS version 6.0 or newer) was developed. The model was calibrated to known USGS stream gage data. The model was ran for the seven storm events. Water surface elevations for the 100- and 500-year storms were extracted and used to create floodplain maps.

Task 5: Alternatives Analysis and Downstream Impact Assessment –The existing conditions hydraulic model was amended to include proposed conditions of the City of Cameron’s proposed water intake infrastructure. Changes in floodwater elevations and water velocity were evaluated and documented.

Task 6: Sediment Behavior and Sediment Deposition Mitigation – Adapt the 2D hydraulic model and collect sediment data. Use sediment data and model to estimate changes in riverbed elevation due to sediment accumulation due to the proposed water intake structure.

Task 7: Little River Study Report – A draft technical report was prepared and summarized the findings of the hydrologic analysis and hydraulic analysis. The draft technical report was submitted to the Texas Water Development Board (TWDB) and the City of Cameron. Comments were incorporated into the report and the report finalized and delivered to the City and TWDB along with the required electronic files.

The City administered the grant and held public meetings at their town hall. The City of Cameron was the key stakeholder in the project. Twenty-six other stakeholders were identified and were invited to the two public meetings. The 26 stakeholders were:

	Stakeholder		Stakeholder
1	Bell County	14	City of Jarrell
2	Milam County	15	Central Texas Council of Governments
3	McLennan County	16	Capitol Area Council of Governments
4	Williamson County	17	Heart of Texas Council of Governments
5	Falls County	18	Region G (Brazos) Water Planning Group
6	City of Temple	19	Brazos River Authority
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11	City of Little River-Academy	24	Salem Elm Ridge Water Supply Corporation
12	City of Troy	25	North Milam Water Supply Corporation
13	City of Moody	26	Minerva Water Supply Corporation

1.1 Level of detail and communities

The hydrologic model is sufficiently detailed to support detailed hydraulic modeling along the Little River mainstem. In total, there are 70 subbasins with an average area of 60 square miles within the hydrologic model. There are nine flow input locations within the hydraulic model. The 2D hydraulic model is comprised primarily of 200 ft computational cells with finer resolution cells (100 ft – 150 ft) along the main channel and relevant hydraulic structures. The City of Cameron is located within the watershed near the area of interest for this study and can be seen on **Figure 1-1.**

1.2 Watershed details

The study area falls within the Little River HUC-8 (12070204) and the Lower Little River HUC-10 (1207020403). The only community within the watershed is the City of Cameron. The watershed is centered around Little River, with inflows from several tributaries along the main stem. **Table 1-1** summarizes the modeled stream and its drainage area.

Table 1-1. Stream summary – Lower Little River.

Stream Name	Reach Length (mi)	Watershed Area (mi^2)
Little River	42.7	281

There are three United States Army Corps of Engineers (USACE) flood control reservoirs directly upstream of the watershed: Lake Granger, Lake Belton, and Lake Stillhouse. These reservoirs are monitored by the USACE and output a controlled flow rate. When there is a significant storm event, the USACE will close the gates of the reservoirs, cutting off flow into the Little River watershed. For more information on the reservoirs, see **Section 3.2.1**. An overview of the study area is shown in **Figure 1-1.**

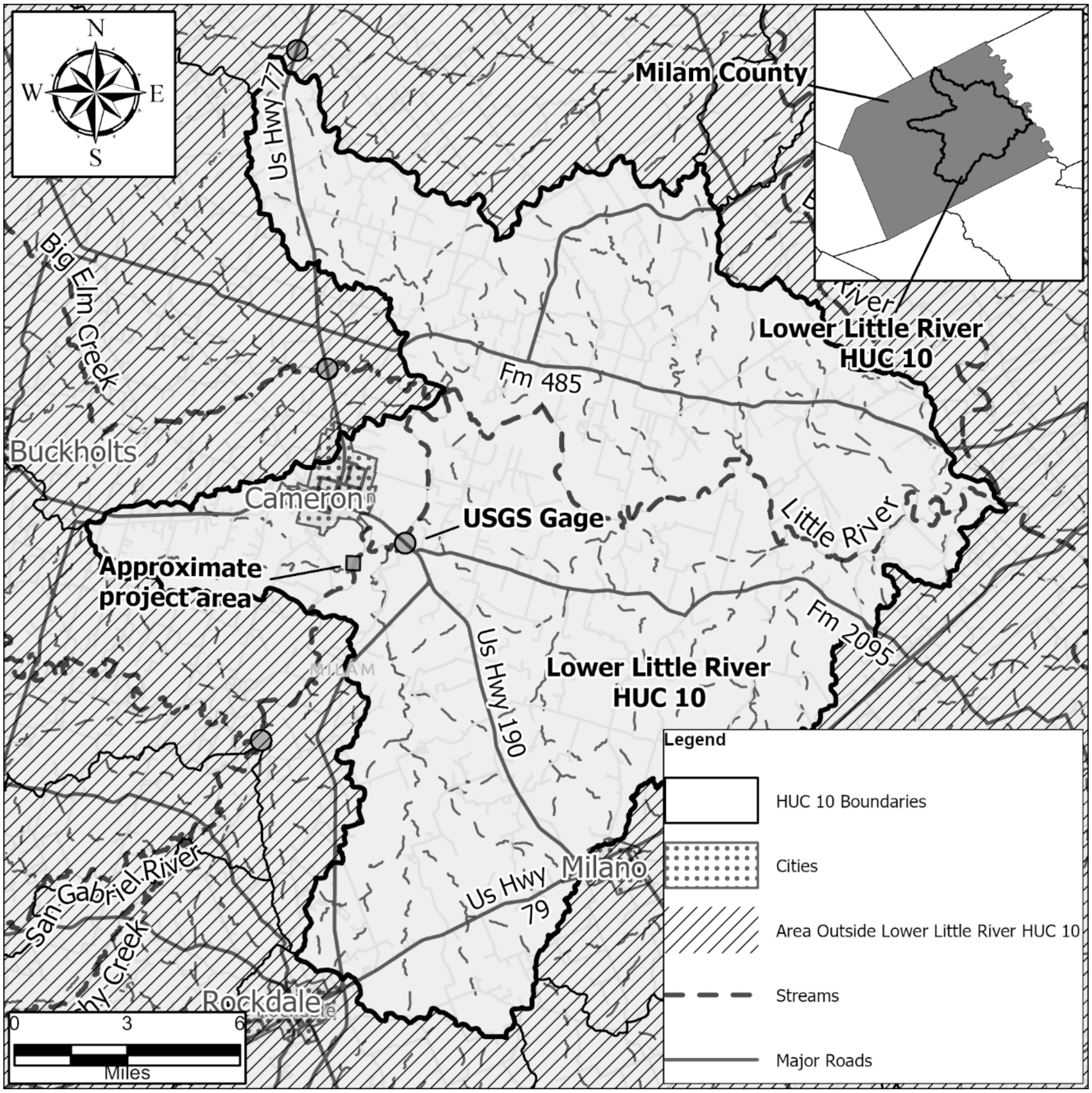

Figure 1-1. **Lower Little River study area overview**

1.3 Previous studies

There are two studies readily available which are relevant to this analysis: the Lower Brazos Flood Protection Planning Study (Lower Brazos Study) completed by Halff Associates and the Brazos River Authority in 2019 and a one-dimensional Base Level Engineering (BLE) study completed in 2020. This study primarily leverages the large-scale hydrologic model produced by the Lower Brazos Flood Protection Planning Study. The BLE study was used only as a reference for results comparison. The Lower Brazos study includes a large-scale hydrologic model, that covers the Brazos River watershed from several USACE reservoirs in Central Texas to the Gulf Coast. This hydrologic model was simulated for several frequency events and was calibrated to multiple historical events. Additional information on this hydrologic model is found in **Section 3.2**. **Figure 1-2** shows the extents of the two existing studies described above in relation to the Little River HUC10 watershed being modeled as part of this analysis.

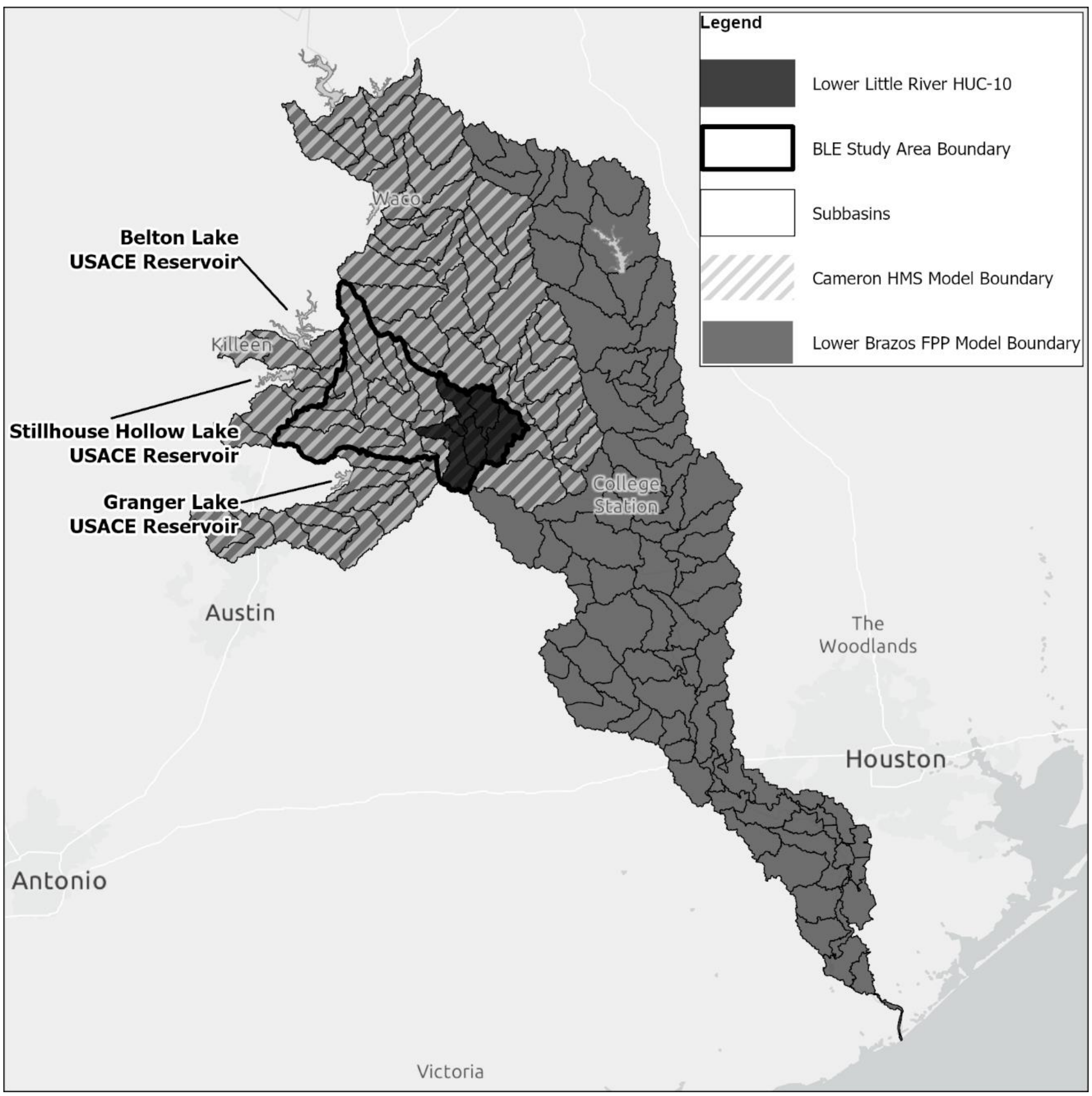

Figure 1-2. Previous study extents

1.4 Gage data and flood history

There is one USGS stream gage within the Lower Little River watershed. As depicted in **Figure 1-1**, the USGS gage is located near the City of Cameron on Highway 36. This gage location is characterized by dual bridge openings with an approximate total opening width of approximately 3000 ft. The period of record for Little Rv nr Cameron is 1917 to present. The Lower Little River watershed has a history of recorded flooding dating back to the early 1900s, where the stream gage recorded peak elevations greater than the current National Weather Service flood stage on multiple occasions. This stream gage records both stage and flow, with annual peak data summarized in **Figure 1-3** and **Figure 1-4**. Observed data from this stream gage is utilized in statistical analysis, hydrologic calibration, and hydraulic calibration presented in **Section 3.1**, **Section 3.2.4**, and **Section 4.6** respectively. As mentioned in **Section 1.2**, there are three USACE controlled reservoirs upstream of the study area which completed construction in 1980. The construction of these reservoirs generally decreased recorded peak stream flows through the study, and the sharp decrease in the magnitude of peak events post-1980 can be clearly seen on **Figure 1-3**.

Table 1-2. USGS stream gage summary

Site Number	Name	Period of Record
08106500	Little Rv nr Cameron, TX	1917-present

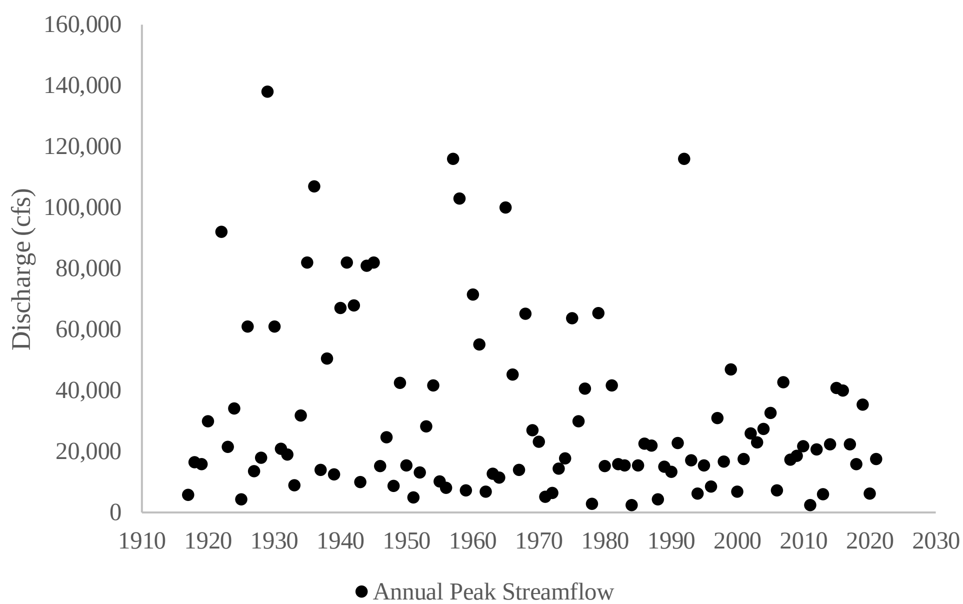

Figure 1-3. Annual peak streamflow – USGS 08106500 Little Rv nr Cameron, TX

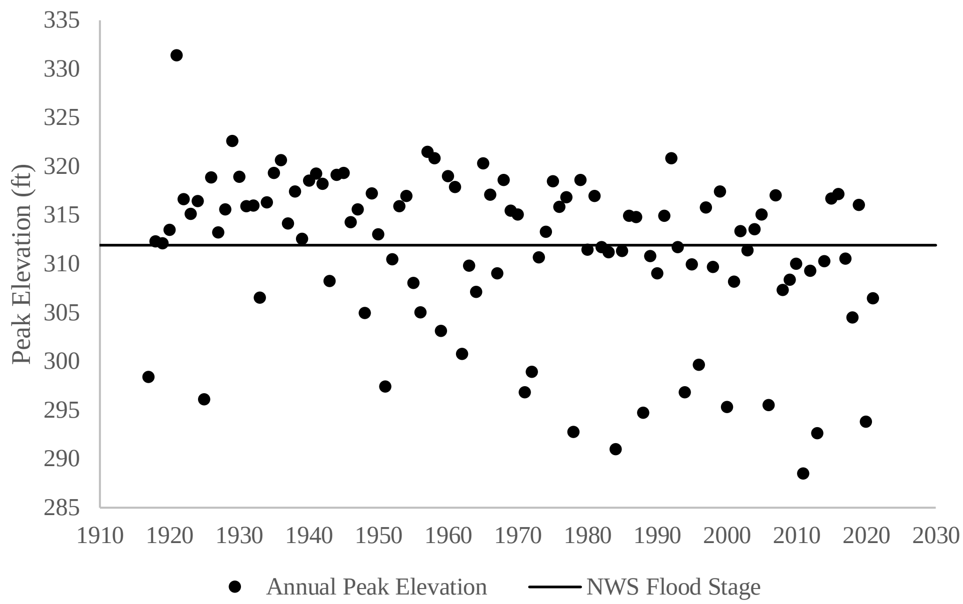

Figure 1-4. Annual peak elevation – USGS 08106500 Little Rv nr Cameron, TX

2 Data collection

This section describes the data that was collected and/or reviewed for this analysis, including geospatial, physical, and observed data.

2.1 Terrain

High resolution 3DEP LiDAR data was available from The National Map, hosted by the United States Geological Survey (USGS) for the entire watershed. These datasets were mosaicked into a 3 ft DEM which was used to generate the terrain in HEC-RAS v6.1. Bathymetry is not readily available within the study area and was not incorporated into the terrain. Bathymetry adjustments made in the Sediment Management model are discussed in **Section 6.1.3.**

2.2 Land cover and impervious cover

The modeling team collected the National Land Cover Dataset 2019 (MRLC, 2019) and incorporated it into the hydraulic model. The land cover dataset was used to determine the initial Manning’s roughness values in the study area. The team also collected the latest NLCD impervious cover layer, but it was only used as a comparison point for the existing hydrologic model to assess if significant impervious cover changes had occurred since the model’s development. The team found minimal differences through the study area. The parameterization of these base land cover datasets is described in **Section 4.3**.

2.3 Soil data

The modeling team collected and reviewed SSURGO Soil Data encompassing the watershed from the USDA Web Soil Survey website (WSS, 2021). Soil characteristics determine the infiltration capacity of the watershed. This analysis leveraged the existing Brazos River study described in **Section 1.3**, so the most recent soil data was only reviewed for agreement with the dataset used in the Brazos River study. Additional information on how soil data was utilized to determine infiltration parameters in the Brazos River study can be found in **Section 3.2.3**.

2.4 Hydraulic structures

Physical data for the hydraulic structures included in the 2D model was developed using a combination of as-built data received from the Bryan Texas Department of Transportation (TxDOT) district, field measurements collected by FNI, and TxDOT online resources. The provided as-builts included data for the SH 36 bridge crossing the Little River. Data for the BNSF railroad crossing just downstream of SH 36 and the existing low-head dam for the pump station was developed from field visit and field survey information, respectively. Survey field data points for the BNSF Railway bridge were collected by FNI using a Trimble R8s Integrated GNSS system, and survey data for the existing low-head dam was provided by the pump station design team. Two additional county road bridge crossings, far removed from the area of interest, were incorporated into the hydraulic model based on the TxDOT crossing online resource. This resource consists of a GIS feature class that contains horizontal information about crossings such as deck width, pier size, and number of piers. This dataset does not contain vertical information, so all elevation data was estimated from LiDAR.

2.5 Sediment samples

Sediment data was collected by FNI at one location upstream and downstream of the proposed intake location. A modified Wolman pebble count and bedload collection following Abt and Bunte "Sampling Surface and Subsurface Particle-Size Distributions in Wadable Gravel and Cobble-Bed Streams for Analyses in Sediment Transport, Hydraulics, and Streambed Monitoring" (2001) at each location. Both bedload samples were then subjected to a Sieve Analysis test following ASTM D6913 guidelines, a Sediment Load Specific Gravity test following ASTM D854 guidelines, and a Hydrometer test following ASTM D7928 guidelines. This data was then used in the sediment transport model discussed in **Section 5.3.3**.

2.6 Precipitation

Frequency event precipitation data was collected and incorporated into the hydrologic model by FNI, which is based on NOAA Atlas 14 precipitation frequency estimates. Gridded ASCII rasters for the following partial duration series (PDS) annual chance events were obtained from NOAA’s Precipitation Frequency Data Server online resource: 50%, 20%, 10%, 4%, 2%, 1%, and 0.2%. Changes made to these storm events in the model are described in **Section 3.4.**

3 Hydrology

This section describes the methods behind the hydrologic analysis including statistical hydrology, existing hydrologic model evaluation and flood hydrograph calculations.

3.1 Statistical hydrology

3.1.1 Gage locations and period of record

There is one USGS stream gage within the Little River watershed. As depicted in **Figure 3-1**, the gage is located on Little River at State Highway 36 near Cameron. This gage location is characterized by bridge openings with an approximate total opening width of 3000 ft. The period of record for Little Rv nr Cameron is 1916 to 2020; however, the observed flow data during this period of record are non-homogenous, including both unregulated flows from before the construction of the three upstream USACE reservoirs and regulated flows post-construction. A Bulletin 17C analysis (England et. al., 2018) was performed on a truncated period of record to estimate flood flow frequencies at the gage. The truncated period of record represents homogenous, regulated flows after the three USACE upstream reservoirs began operation in 1980. This results in an effective period of record from 1980 to present, approximately 42 years. A low flow outlier threshold of 0 cfs was determined according to the Multiple Grubbs-Beck Test. The Bulletin 17C station skew was utilized with no adjustment for regional skew. A summary of the Little River gage near Cameron can be seen in
Table 3-1.

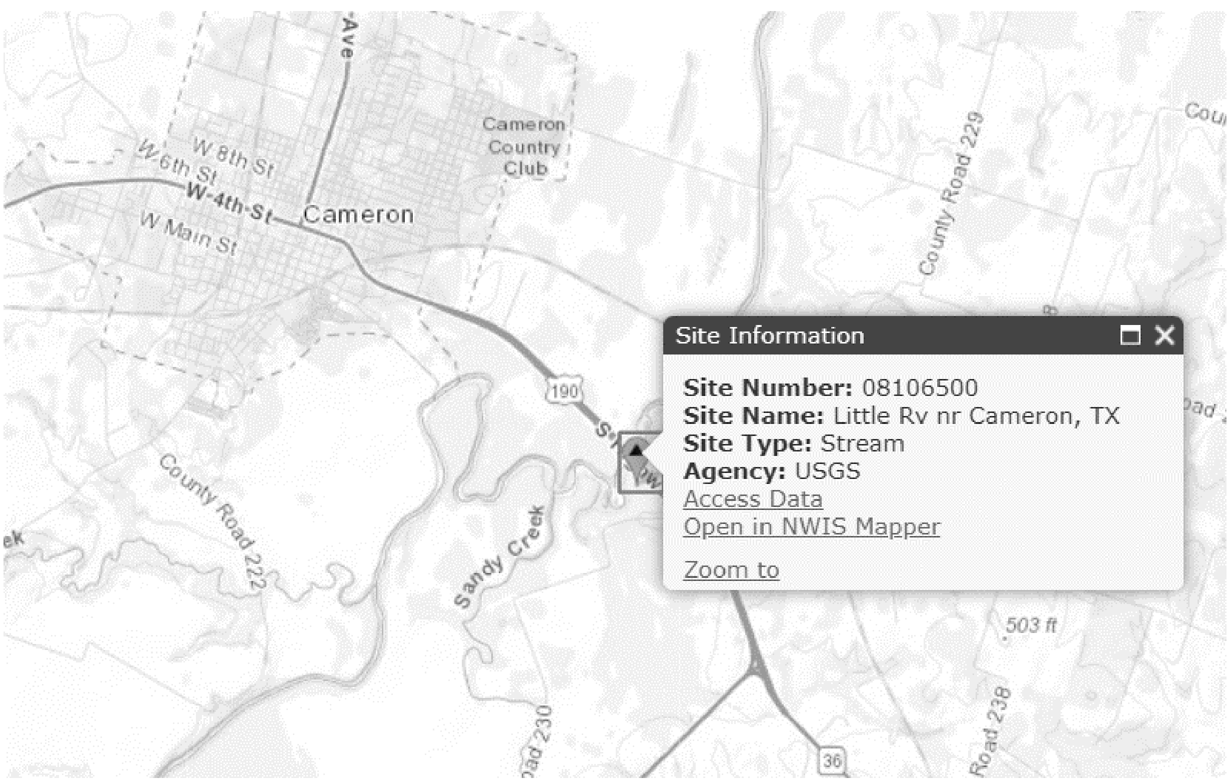

Figure 3-1. USGS Little Rv nr Cameron, TX location map

Table 3-1. Summary of USGS gages

Site Number	Name	Drainage Area (sq. mi.)	Period of Analysis	Low Flow Outlier Threshold (cfs)	Station Skew
08106500	Little Rv nr Cameron, TX	7,065	11/1/916 - Present	0	-0.2444

3.1.2 Bulletin 17C analysis

The annual peak streamflow data is plotted in **Figure 3-2**. **Table 3-2** summarizes the estimated peak streamflow values for various annual exceedance frequency return periods from the Bulletin 17C analysis. The flow frequency curve is plotted in **Figure 3-3**. All figures, tables, and analyses for the Bulletin 17C analysis were made using HEC-SSP Version 2.2.

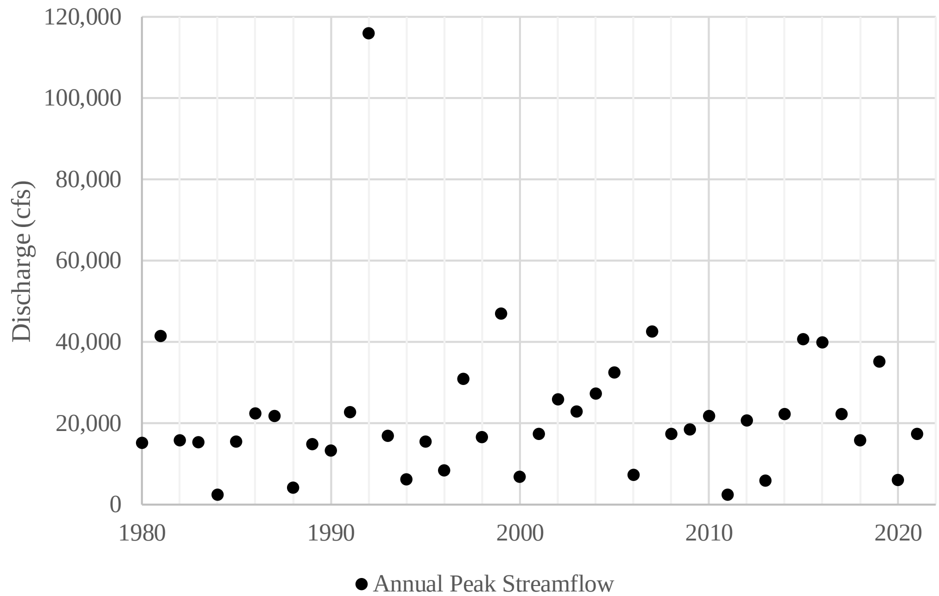

Figure 3-2. USGS gage 08106500 Little Rv nr Cameron, TX – annual peak streamflow

Table 3-2. USGS gage 08106500 Little Rv nr Cameron, TX – Bulletin 17C analysis results

	Peak Streamflow (cfs) by Return Period (years)						
	2-yr	**5-yr**	**10-yr**	**25-yr**	**50-yr**	**100-yr**	**500-yr**
Lower 95% CI	14,553	26,954	35,725	45,136	49,993	53,459	58,423
Estimate	18,195	33,076	43,575	56,940	66,741	76,296	97,541
Upper 95% CI	22,731	40,544	55,989	79,421	99,319	121,964	188,767

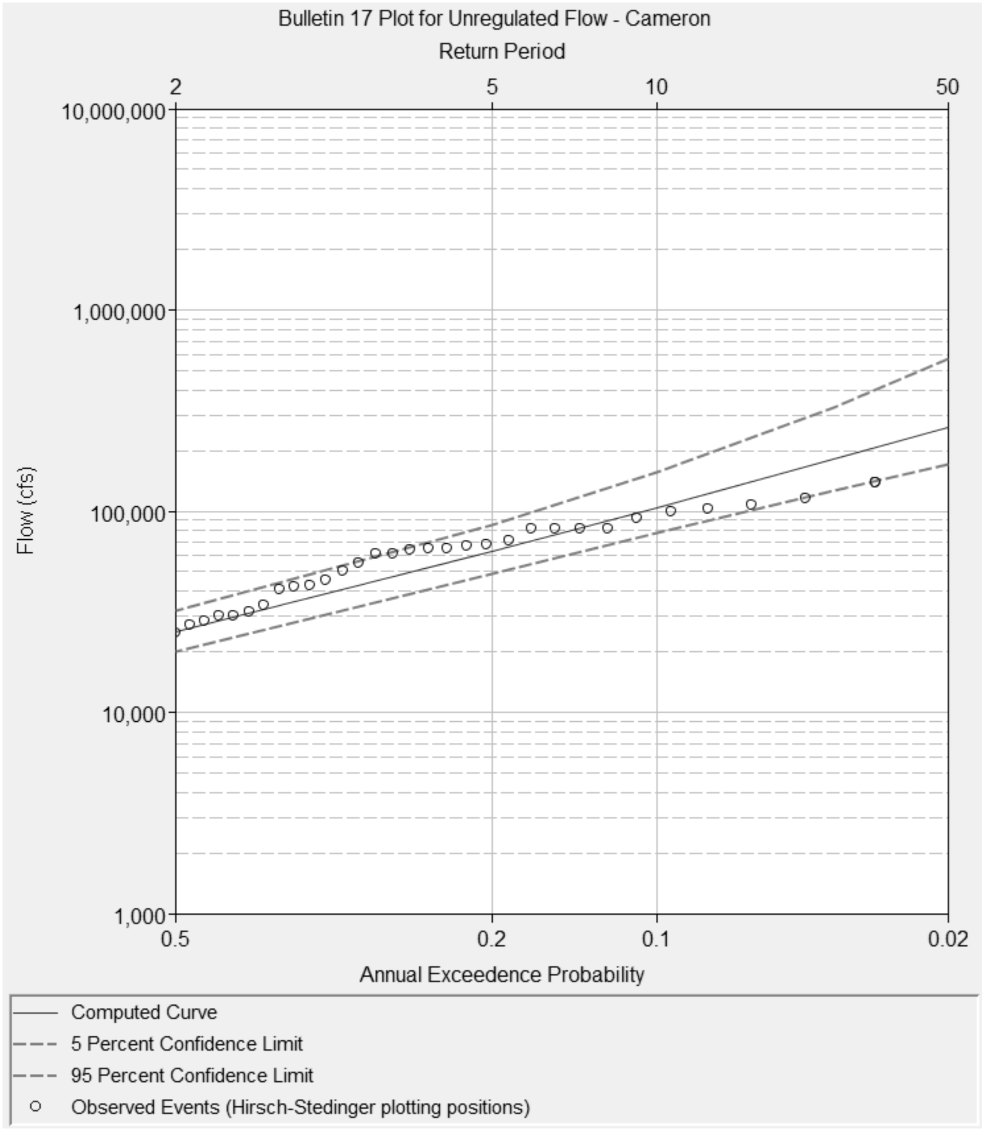

Figure 3-3. **USGS gage 08106500 Little Rv nr Cameron, TX – bulletin 17C AEP plot**

3.2 Existing model evaluation

As mentioned in **Section 1.3**, an existing hydrologic model of the Brazos River was leveraged for this study. This section documents the study team's evaluation of the existing model for applicability to this analysis. Generally, subbasin parameters from the existing Lower Brazos Study were not modified while updates were made to model version, extents, and rainfall. The complete Lower Brazos Flood Protection Planning Study report is within the attached digital data.

3.2.1 Model summary

The existing hydrologic model was developed with HEC-HMS v4.3 and encompasses a total drainage area of 9,766 square miles, from Central Texas to the Gulf Coast. The modeled area is divided into 154 subbasins with an average area of 63.4 square miles. The model includes multiple historical event simulations using Gage Adjusted Radar Rainfall (GARR) applied to each subbasin. Three historical events were simulated in the hydrologic model (August 2017, Memorial Day 2016, Tax Day 2016) which were calibrated to multiple USGS stream gages distributed throughout the Brazos River watershed. This includes the Little River stream gage near Cameron, Texas which is most relevant to this analysis. The calibrated hydrologic parameters were then applied to the 50%, 20%, 10%, 4%, 2%, 1%, and 0.2% AEP frequency events. The frequency events were simulated with NOAA Atlas 14 precipitation depths and an elliptical storm. The model uses an elliptical storm, rather than a balanced frequency storm, due to the size of the watershed. In general, USACE recommends using an elliptical storm for watersheds with contributing areas greater than 1,000 square miles. The elliptical storm parameters were varied through an iterative approach until a critical peak flow was produced at a desired evaluation point. For the Lower Brazos Study, the evaluation point was the Brazos Rv at Richmond, TX USGS stream gage within Fort Bend County, Texas.

3.2.2 Simulated reservoir releases

A critical element to the Cameron Little River analysis is the assumption regarding reservoir releases for frequency events. Since frequency events are hypothetical, reservoir releases must be assumed, rather than based on observed gage data for historical events. As discussed in **Section 1.1**, there are three USACE reservoirs upstream of the Cameron Little River study area, and results through the Little River are sensitive to the simulated releases.

USACE monitors the Brazos Rv nr Hempstead, TX and Brazos River at Richmond, TX gages as control points when determining reservoir releases from the reservoirs at the upstream end of the Lower Brazos watershed. The threshold flow at these two stream gages is 60,000 cfs, so if stream flows are anticipated to exceed this threshold, USACE will hold water within the flood control pools of the reservoirs in accordance with their respective Water Control Plans. As a conservative assumption, the hydrologic model assumes this initial release of 60,000 cfs from the USACE reservoirs, which is then "shut off" when the response hydrograph begins to increase at the Brazos Rv nr Hempstead, TX gage. Constant releases from each reservoir were ratioed according to their contributing area based on the total value of 60,000 cfs. An example of the simulated reservoir release is shown in **Figure 3-4**.

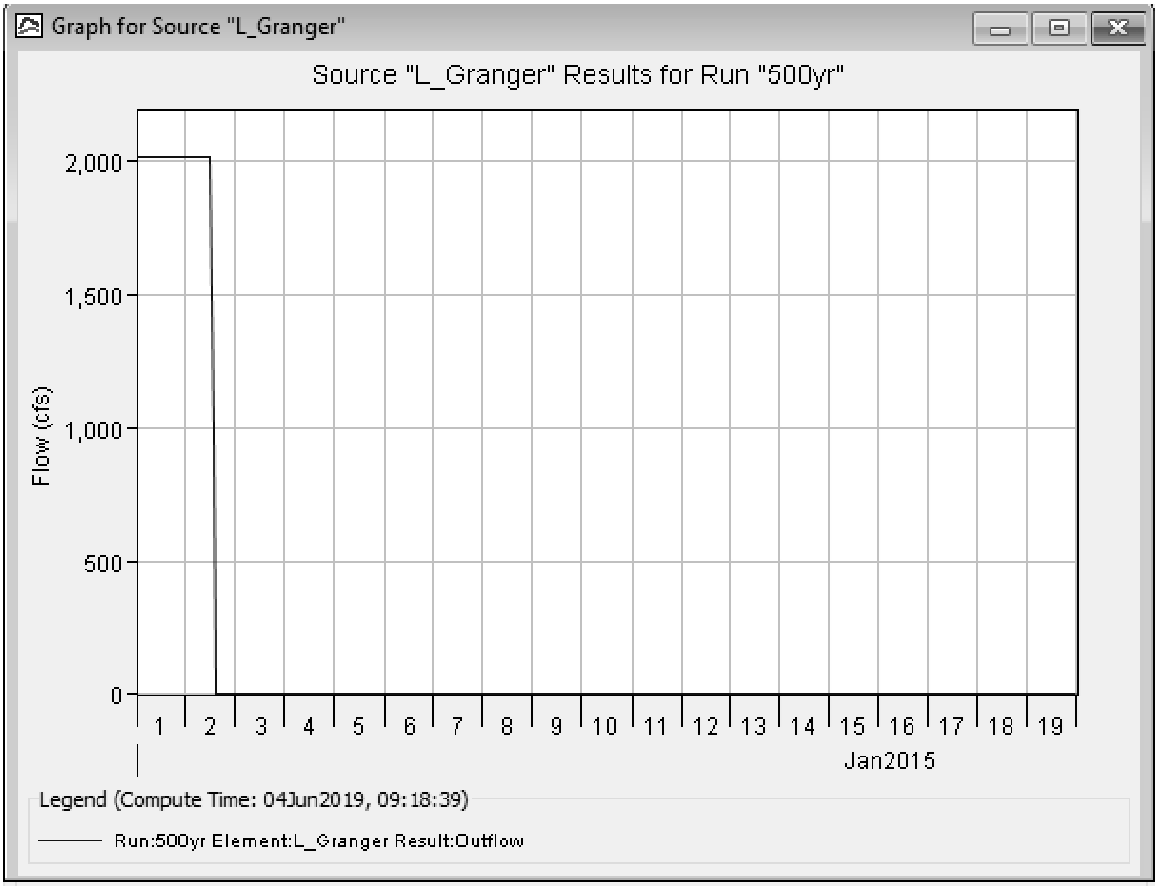

Figure 3-4. Simulated reservoir release example

3.2.3 Hydrologic features and parameters

After evaluating the existing model, the study team determined that the following existing hydrologic features and parameters from the Lower Brazos study did not require modification for the Cameron Little River analysis.

- Subbasins
- Channel Routing
- Transform
- Infiltration

A summary of the Lower Brazos methodology for the area near the Cameron Little River study area is presented in the sections below.

Subbasins

The contributing area upstream of USGS 08106500 Little Rv nr Cameron, TX consists of 28 subbasins with an average area of 53 square miles. Within the Little River study area there are 6 subbasins with an average area of 47 square miles. The Little River HUC10 also receives inflows from the Big Elm Creek watershed, which is divided into 7 subbasins with an average area of 46 square miles.

Channel routing

Modified Puls routing was utilized for the portion of the Lower Brazos hydrologic model relevant to the Cameron Little River analysis. Modified Puls storage-outflow relationships were computed for the Lower Brazos Study with a low detail HEC-RAS v4.1 model.

Transform

The Lower Brazos study applied Snyder's unit hydrograph through the Little River watershed. Snyder's unit hydrograph considers the time distribution of rainfall, the initial rainfall losses to interception and depression storage, and an infiltration rate that decreases during the storm (Source: USACE Engineering Manual (EM 1110-2-1405) Flood-Hydrograph Analysis and Computations). The two parameters associated with the Snyder's unit hydrograph are lag time and Snyder's peaking coefficient. The Snyder's unit hydrograph utilizes the time to peak of the unit hydrograph in hours. The time to peak represents the lag time from the midpoint of the unit rainfall duration.

$$t_p = C_p * (L - L_{ca})^{0.3}$$

t_p = Time to peak of unit hydrograph, hours
C_p = Snyder's peaking coefficients depending upon units and drainage basin characteristics derived by the USACE and others.
L = River mileage from the given station to the upstream limits of the drainage area
L_{ca} = River mileage from the design point (basin discharge location) to the centroid of gravity of the drainage area

The average calibrated Snyder unit hydrograph parameters for the contributing area of USGS 08106500 Little Rv nr Cameron, TX are presented in **Table 3-3**. These averages include subbasins from the Little River (LIT), Leon River (LEO), San Gabriel River (SAN), and Brushy Creek (BRU) basins.

Table 3-3. Cameron Little River hydrology transform parameter summary

Average Snyder Cp	Average Snyder Tp (hr)
0.32	7.64

Infiltration

The Initial and Constant Loss Rate method was used to determine infiltration through the Little River watershed. Initial weighted percent impervious values were determined by GIS methods for each sub-basin based on USGS Land Use Land Cover (LULC) maps. The base initial loss was assumed to be twenty percent of the maximum storage for each sub-basin. The constant loss rate represents the ultimate infiltration capacity of the soils. Constant loss rates were determined for each subbasin using the NRCS Soil Survey Geographic (SSURGO) Database hydrologic soil group maps. The range of constant loss rates for a given soil type are presented in **Table 3-4**. The average calibrated initial and constant loss parameters for the contributing area of USGS

08106500 Little Rv nr Cameron, TX are presented in **Table 3-5**. A summary of the soil types through the Lower Little River HUC10 are also provided in **Figure 3-5**. These averages include subbasins from the Little River (LIT), Leon River (LEO), San Gabriel River (SAN), and Brushy Creek (BRU) basins.

Table 3-4. Range of constant loss rates by soil group

Hydrologic Soil Group	Description	Minimum Constant Loss Rate (in/hr)	Maximum Constant Loss Rate (in/hr)
A	Deep sand, deep loess, aggregated silts	0.30	0.45
B	Shallow loess, sandy loam	0.15	0.30
C	Clay loams, shallow sandy loam, soils low in organic content, and soils usually high in clay	0.05	0.15
D	Soils that swell significantly when wet, heavy plastic clays, and certain saline soils	0.00	0.05

Table 3-5. Cameron Little River infiltration parameter summary

Annual Exceedance Probability	Average Initial Loss (in)	Average Constant Loss (in/hr)
50%	0.80	0.30
20%	0.80	0.05
10%	0.80	0.02
4%	0.80	0.02
2%	0.80	0.04
1%	0.80	0.05
0.2%	0.80	0.14

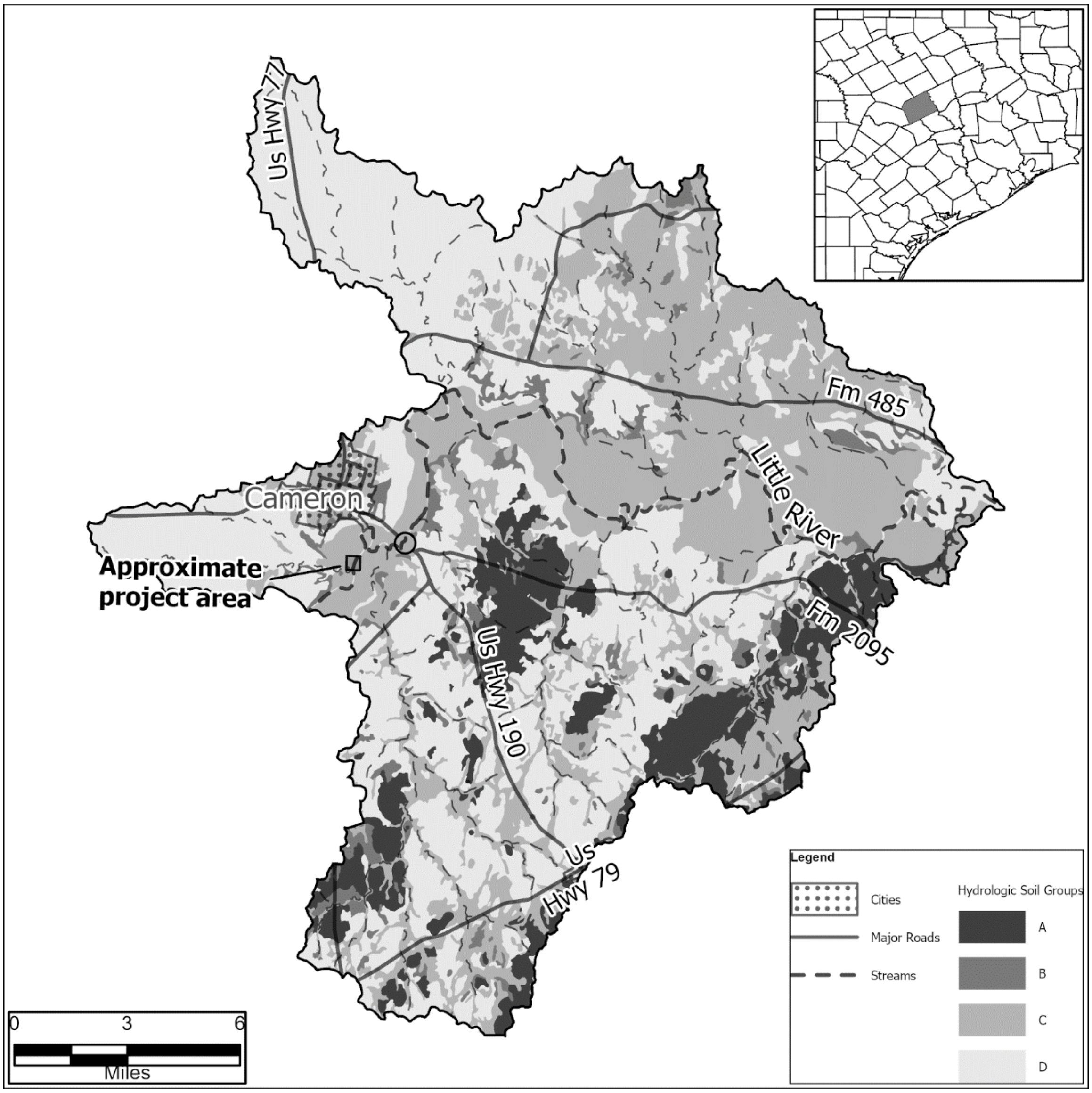

Figure 3-5. Lower Little River hydrologic soil groups

3.2.4 Historical events and calibration

Three historical events were simulated for hydrologic calibration for the Lower Brazos River Flood Protection Planning Study's HEC-HMS model. Rainfall data was based on Gage-Adjusted Radar Rainfall (GARR) sourced from the National Weather Service multisensory precipitation estimator rainfall product for each subbasin. Hydrologic parameters were modified in the Lower Brazos River Flood Protection Planning Study's HEC-HMS model to match observed hydrograph shapes, peak flow, and runoff volume for USGS stream gages in the study area. **Table 3-6** summarizes the three historical events. **Figure 3-6** through **Figure 3-8** and **Table 3-7** through **Table 3-9** present comparisons of modeled versus observed data for each event at USGS 08106500 Little Rv nr Cameron, TX.

The purpose of these comparisons is to determine whether Lower Brazos River Flood Protection Planning Study's HEC-HMS model calculates discharges that are acceptable for the Little River Hydrologic and Hydraulic Analysis.

Table 3-6. Historical event summary

Event	Date
Hurricane Harvey	August 23, 2017 – September 8, 2017
Memorial Day 2016	May 25, 2016 – June 15, 2016
Tax Day 2016	April 25, 2016 – May 25, 2016

Hurricane Harvey 2017

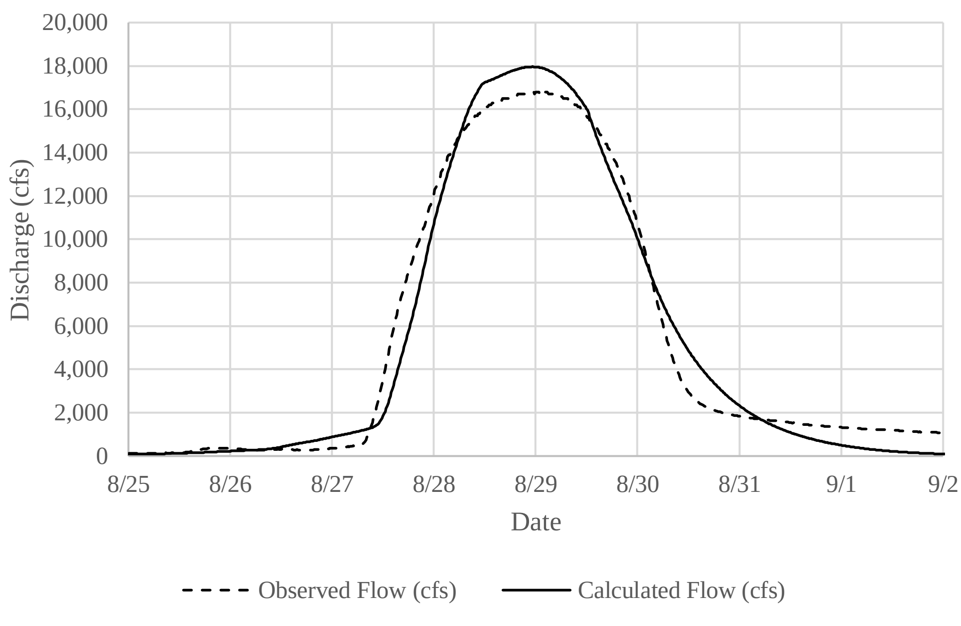

Figure 3-6. Hurricane Harvey 2017 observed vs. calculated flow hydrograph at USGS Little Rv nr Cameron, TX

Table 3-7. Hurricane Harvey 2017 calibration results summary

Event	Peak Discharge (cfs)			Volume (inches)			Time of Peak		
	Observed	Simulated	% Diff	Observed	Simulated	% Diff	Observed	Simulated	Diff (hr)
HARVEY2017	16,800	17,961	6.9	4.73	4.30	-9.1	28Aug2017 2330	28Aug2017 2315	- 0.25

Memorial Day 2016

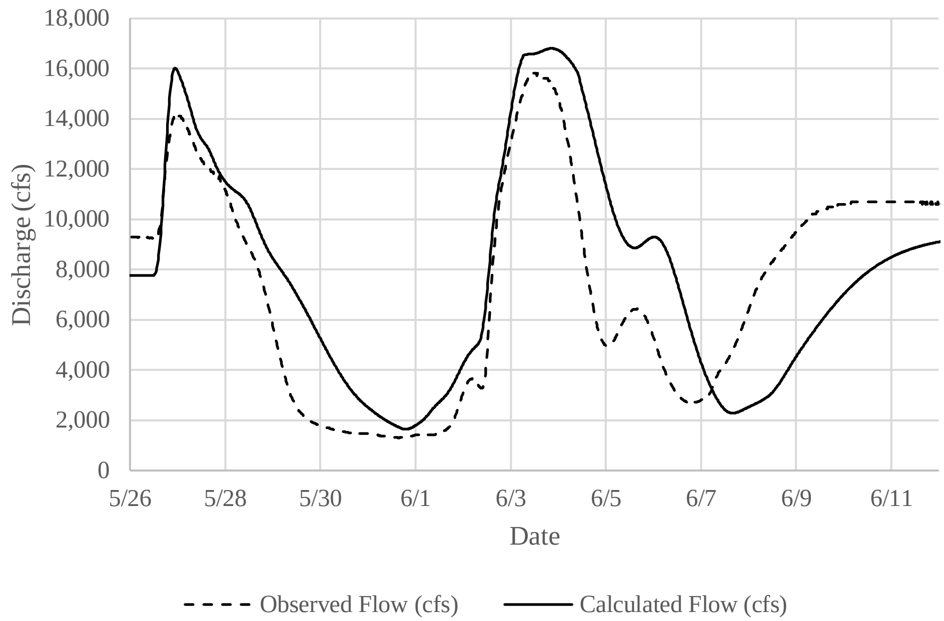

Figure 3-7. Memorial Day 2016 observed vs. calculated flow hydrograph at USGS Little Rv nr Cameron, TX

Table 3-8. Memorial Day 2016 calibration results summary

Event	Peak Discharge (cfs)			Volume (inches)			Time of Peak		
	Observed	Simulated	% Diff	Observed	Simulated	% Diff	Observed	Simulated	Diff (hr)
MEMDAY2016	15,800	16,798	6.3%	16.28	16.69	2.5%	03Jun2016 10:30	03Jun2016 20:45	10.25 hr

Tax Day 2016

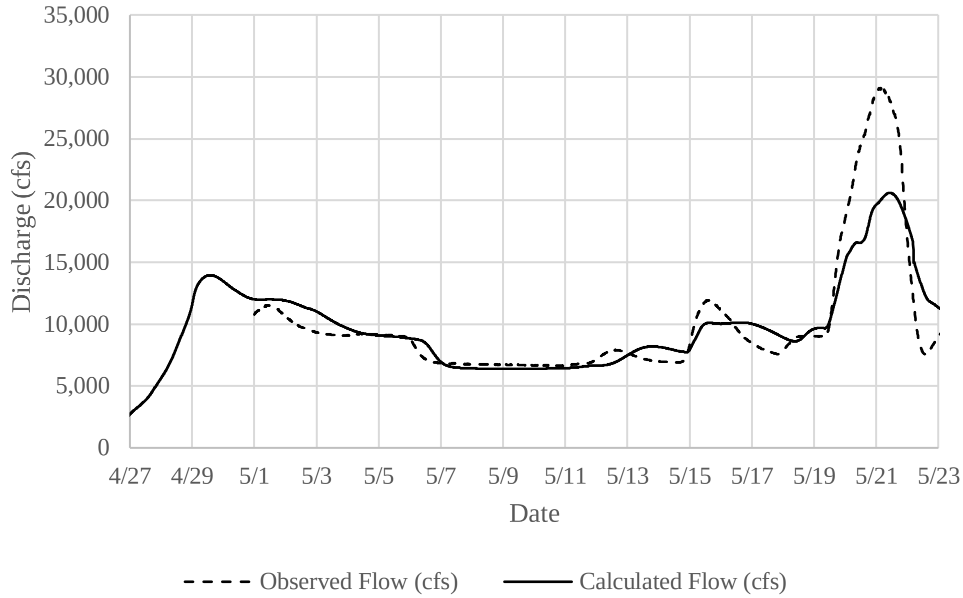

Figure 3-8. Tax Day 2016 observed vs. calculated flow hydrograph at USGS Little Rv nr Cameron, TX

Table 3-9. Tax Day 2016 calibration results summary

Event	Peak Discharge (cfs)			Volume (inches)			Time of Peak		
	Observed	Simulated	% Diff	Observed	Simulated	% Diff	Observed	Simulated	Diff (hr)
TAXDAY2016	29,100	20,626	-29.1%	21.93	-	-	21May2016 02:00	21May2016 10:30	8.5 hr

Conclusions

The calibration efforts in the Lower Brazos River Flood Protection Planning Study included some model calibration to improve the consistency of the simulated discharges to the observed discharges at the USGS stream gage near Cameron, Texas. As demonstrated by the figures and tables presented in the preceding sections, the simulated hydrographs were calibrated to the observed data with varying degrees of success. The Hurricane Harvey and Memorial Day 2016 simulated discharges were similar to observed discharges. The peak intensities for the Tax Day 2016 event were not similar.

Due to the large scale of the Lower Brazos River Flood Protection Planning Study, whose primary goal was to calculate peak discharges in Fort Bend County, far removed from Cameron, the Memorial Day 2016 and Tax Day 2016 events required a wide simulation window (~2 weeks). This accounted for travel time from the upstream end of the model through Fort Bend County. The wide simulation window calculated the multiple peaks for both the Memorial Day 2016 and Tax Day 2016 events which was similar to observed hydrologic trends. However the simulated maximum peak of the Tax Day 2016 flood was less than the observed maximum . peak. Based on these considerations, the Lower Brazos River Flood Protection Planning Study's HEC-HMS model was not calibrated for the Little River Hydrologic and Hydraulic Analysis. The simulated Tax Day 2016 event was not a good match for the observed values therefore it was not used in calibrating the hydraulic model which is discussed in **Section 4.6.**

3.3 Updates to the existing model

This section documents the identification and implementation of updates to the Lower Brazos Study's hydrologic model.

3.3.1 Model extent and version updates

The Lower Brazos River Flood Protection Planning Study's HEC-HMS model was truncated at the hydrologic junction (J_Bryan) associated with the USGS Brazos Rv at SH 21 nr Bryan, TX stream gage to reduce the size of the model and remove portions of the model not relevant to the Cameron Little River analysis. The model extents include all existing information upstream of this point, which include the Cameron study area. The truncated hydrologic model layout was shown on **Figure 1-2**. The study team also updated the existing HEC-HMS model to version 4.9 from version 4.3.

3.3.2 Rainfall updates

The methodology of applying rainfall for frequency events in the Little River watershed was changed from using an elliptical storm to using a hypothetical storm over the entire study area with a depth area reduction curve applied. This change was made because the original elliptical storm in the Lower Brazos model was centered over an area far downstream of the Little River study area. For a complete discussion over the updates made to applying rainfall, refer to **3.4.1**

3.4 Frequency events

3.4.1 Rainfall

NOAA Atlas 14 rainfall rasters were applied by the FNI study team to HEC-HMS by using the Hypothetical Storm type paired with a nested hyetograph rainfall distribution curve. For each frequency event, the 24-hr partial duration series (PDS) precipitation depth raster was added to the model as a precipitation-frequency grid and then tied to a meteorologic model. As a point of reference, **Table 3-10.NOAA atlas 14 study area averaged** precipitation depths presents the average precipitation depths for the contributing area of USGS gage 08106500 Little Rv nr Cameron, Texas. To calculate each curve, the depth for each duration was averaged across the study area. Using the average precipitation depths in **Table 3-10**, the NRCS recommended approach based on WinTR-20 was applied to develop distributions for each frequency event. This approach to frequency storm development mimics the balanced frequency storm typically utilized through the Frequency Storm meteorologic model type. These rainfall distribution curves can be seen tabularly in **Table 3-11** and graphically in **Figure 3-9**. **Figure 3-10** presents the 100-yr, 24-hr duration precipitation depths across the modeled area.

Table 3-10. NOAA atlas 14 study area averaged precipitation depths

Duration	Precipitation Depth (in)						
	50% AEP	20% AEP	10% AEP	4% AEP	2% AEP	1% AEP	0.20% AEP
5	0.52	0.63	0.73	0.86	0.95	1.05	1.29
10	0.83	1.01	1.17	1.37	1.53	1.69	2.05
15	1.04	1.27	1.46	1.71	1.90	2.09	2.55
30	1.46	1.78	2.03	2.38	2.63	2.89	3.55
60	1.91	2.33	2.68	3.15	3.50	3.86	4.81
120	2.35	2.92	3.40	4.08	4.60	5.15	6.60
180	2.61	3.28	3.86	4.68	5.34	6.03	7.88
360	3.06	3.90	4.64	5.73	6.62	7.60	10.20
720	3.49	4.47	5.39	6.76	7.93	9.23	12.90
1440	3.95	5.10	6.19	7.84	9.25	10.90	15.40

Table 3-11. Frequency event rainfall distribution patterns

Percentage of Total Duration	Percentage of Total Rainfall						
	50% AEP	20% AEP	10% AEP	4% AEP	2% AEP	1% AEP	0.20% AEP
0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
25.0	5.8	6.2	6.5	6.9	7.1	7.7	8.1
37.5	11.3	11.8	12.5	13.5	14.2	15.1	16.9
43.8	17.0	17.8	18.8	20.2	21.1	22.3	24.4
45.8	20.3	21.4	22.5	24.0	25.1	26.4	28.6
47.9	25.8	27.2	28.4	29.9	31.1	32.3	34.4
49.0	31.5	32.5	33.6	34.8	35.8	36.7	38.5
49.5	36.8	37.5	38.2	39.1	39.7	40.4	41.7
49.7	43.4	43.8	44.1	44.5	44.8	45.2	45.8
50.0	50.0	50.0	50.0	50.0	50.0	50.0	50.0
50.4	56.6	56.2	55.9	55.5	55.2	54.8	54.2
50.5	63.2	62.5	61.8	60.9	60.3	59.6	58.3
51.0	68.5	67.5	66.4	65.2	64.2	63.3	61.5
52.1	74.2	72.8	71.6	70.1	68.9	67.7	65.6
54.2	79.7	78.6	77.5	76.0	74.9	73.6	71.4
56.3	83.0	82.2	81.2	79.8	78.9	77.7	75.6
62.5	88.7	88.2	87.5	86.5	85.8	84.9	83.1
75.0	94.2	93.8	93.5	93.1	92.9	92.3	91.9
100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0

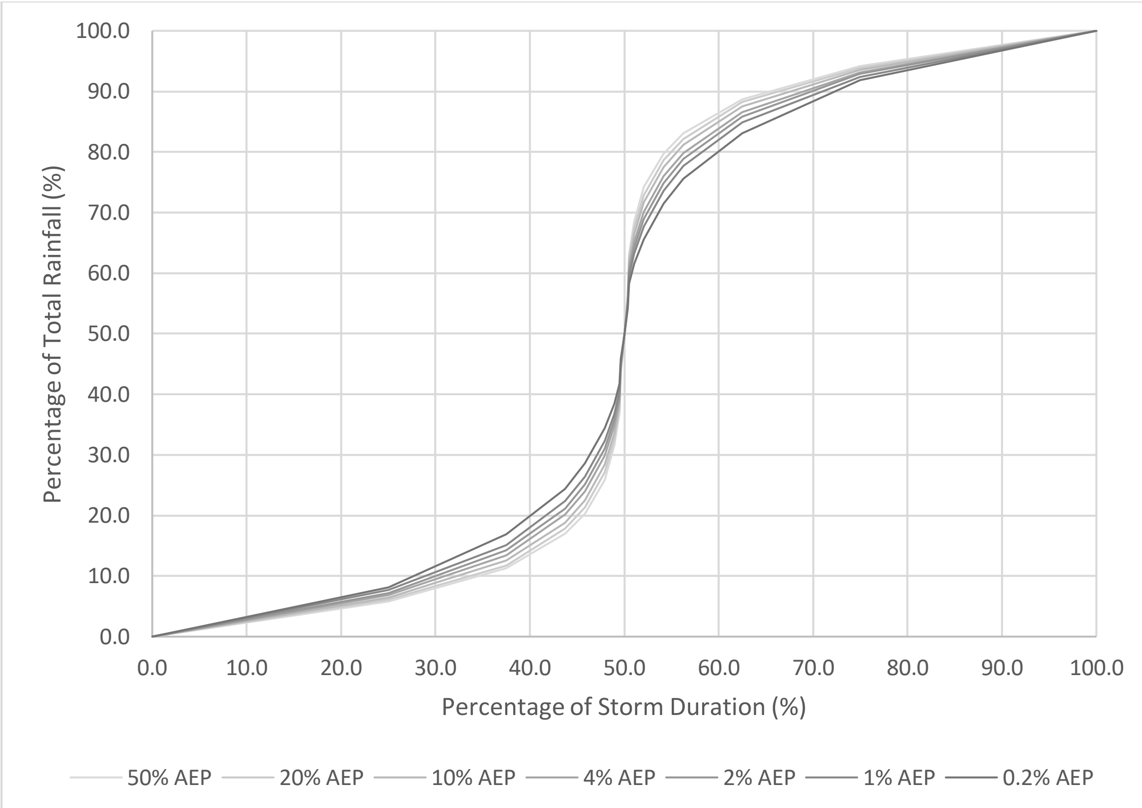

Figure 3-9. Frequency event rainfall distribution pattern

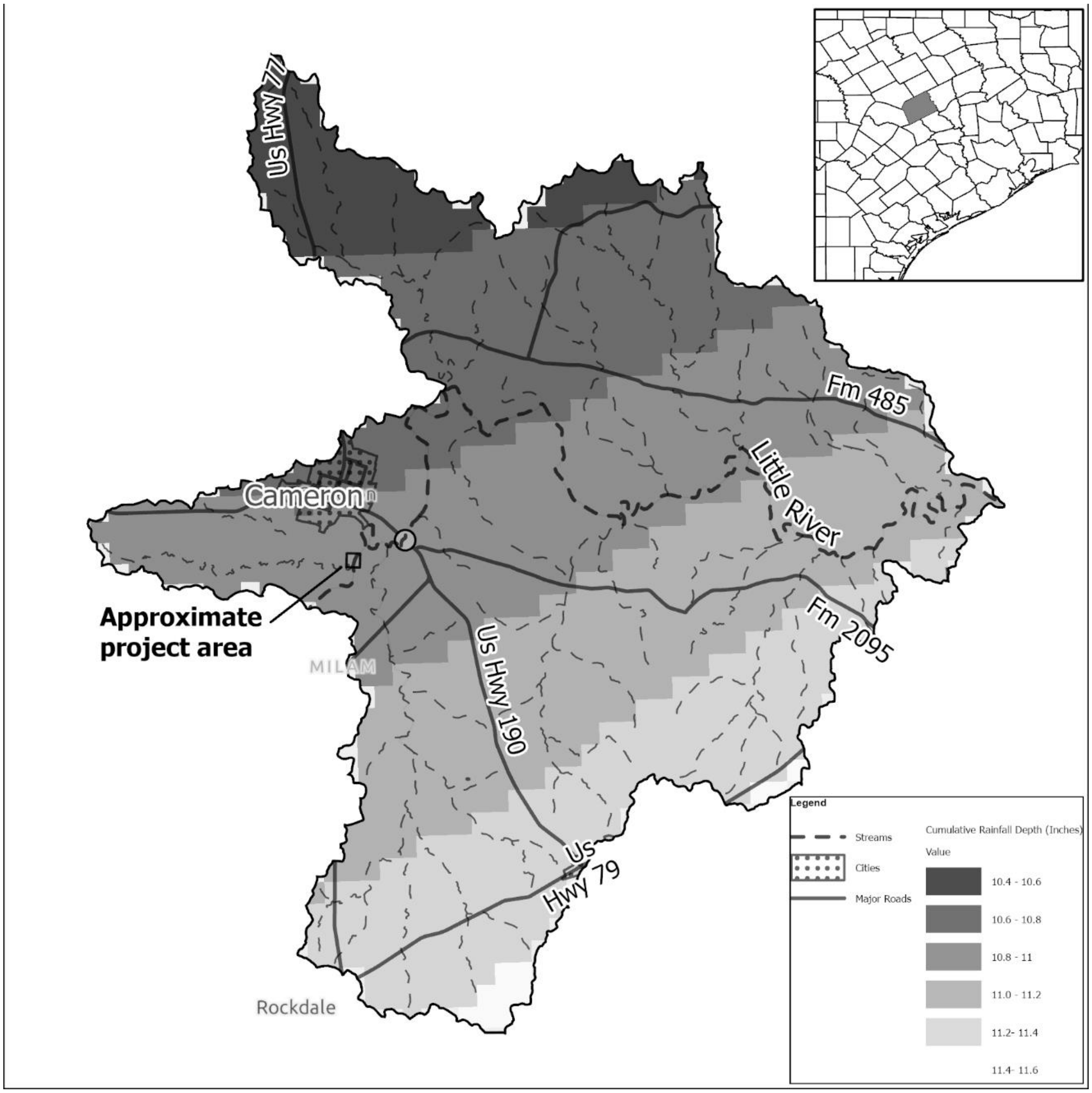

Figure 3-10. 100-yr 24-hr NOAA atlas 14 rainfall depths

3.4.2 Depth area analyses

The National Oceanic and Atmospheric Administration (NOAA) Atlas 14 precipitation frequency estimates are applicable to discrete spatial locations. When applying the precipitation frequency estimates across broader areas, reductions must be applied to account for the limited areal extents of actual storm events. Generally, elliptical storms are recommended for contributing areas greater than 1,000 square miles, but a simplified approach has been applied to the Cameron Little River analysis. To account for areal reduction, the study team applied user-specified depth-area reduction (DAR) curve to each hypothetical storm and simulated the events using Depth-Area Analyses in HEC-HMS. The depth-area reduction curve was developed from Interagency Flood Risk Management Watershed Hydrology Assessments (InFRM WHA) for nearby river basins. A DAR curve for the Cameron Little River study area was developed by applying the most conservative reduction factor to each storm area. The DAR curves are shown in **Table 3-12** and **Figure 3-11**.

Table 3-12. Adopted depth area reduction curve from InFRM WHA

Storm Area (sq. mi.)	Guadalupe River	Trinity River	Neches River	Selected Curve
10	0.987	1.000	1.000	1.000
30	0.967	0.977	0.970	0.977
50	0.953	0.960	0.957	0.960
100	0.930	0.940	0.936	0.940
200	0.905	0.902	0.903	0.905
300	0.880	0.875	0.883	0.883
400	0.865	0.855	0.867	0.867
600	0.835	0.834	0.835	0.835
800	0.812	0.818	0.814	0.818
1,000	0.795	0.804	0.800	0.804
1,500	0.760	0.775	0.772	0.775
2,000	0.730	0.752	0.744	0.752
2,667	0.705	0.726	0.718	0.726
3,500	0.680	0.699	0.689	0.699
4,000	0.665	0.685	0.672	0.685
4,500	0.650	0.672	0.665	0.672
5,000	0.640	0.658	0.657	0.658
6,000	0.620	0.637	0.642	0.642
6,500	0.612	0.626	0.635	0.635
7,000	0.600	0.617	0.628	0.628
8,000	0.585	0.599	0.617	0.617
9,000	0.570	0.581	0.606	0.606
10,000	0.555	0.564	0.594	0.594

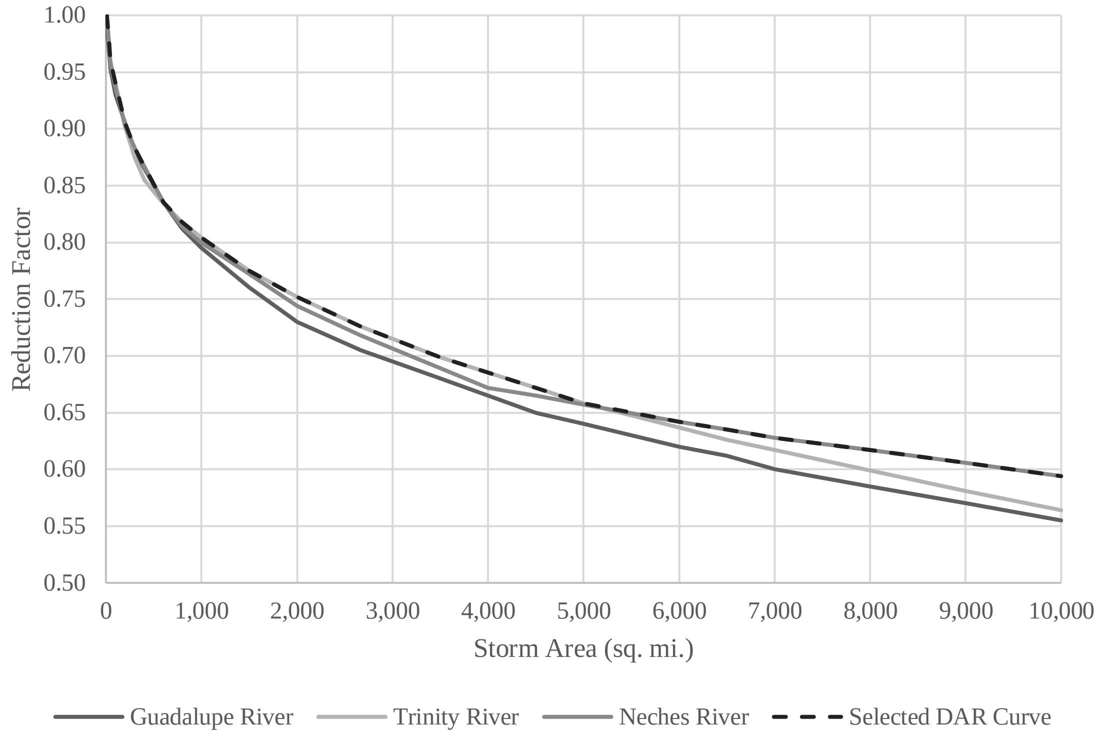

Figure 3-11. Adopted depth area reduction curve

For each frequency event, the evaluation point of the depth-area analyses was set at the HEC-HMS element (J_Cameron) associated with USGS 08106500 Little Rv nr Cameron, TX. Additional evaluation points were added downstream of the gage to properly develop input flow hydrographs for HEC-RAS, but all hydrologic results presented in this document will reference the J_Cameron evaluation point.

3.4.3 Sensitivity analysis and storm area

The DAR factor determined according to storm area, as discussed in **Section 3.4.2** and presented in **Table 3-12**, is critical in determining peak flows through Little River near Cameron, Texas. The three USACE reservoirs upstream of the study area present complicating factors in determining the appropriate storm area, so the study team conducted various sensitivity tests with combinations of reservoir releases and storm area to select the scenario most appropriate to this study. The two potential scenarios are as follows:

1. Assumed reservoir releases discussed in **Section 3.2.2** and storm area that includes contributing area to the reservoirs.
2. No reservoir releases and storm area that includes only the contributing area downstream of the reservoirs. A "local" event.

The study team simulated these two scenarios in the HEC-HMS model, and peak flows results measured at the HEC-HMS element (J_Cameron) associated with USGS gage 08106500 Little

Rv nr Cameron, TX are presented in **Table 3-13**. A comparison of the 100-yr hydrographs is shown in **Figure 3-12**.

Table 3-13. Storm area sensitivity analysis results

Scenario	Simulated Reservoir Release (cfs)	Storm Area (mi2)	50% AEP	20% AEP	10% AEP	4% AEP	2% AEP	1% AEP	0.2% AEP
1	9,267	7,104	24,760	42,480	59,130	78,418	93,610	112,058	153,148
2	0	1,491	21,377	45,454	66,259	91,301	112,370	135,188	198,958

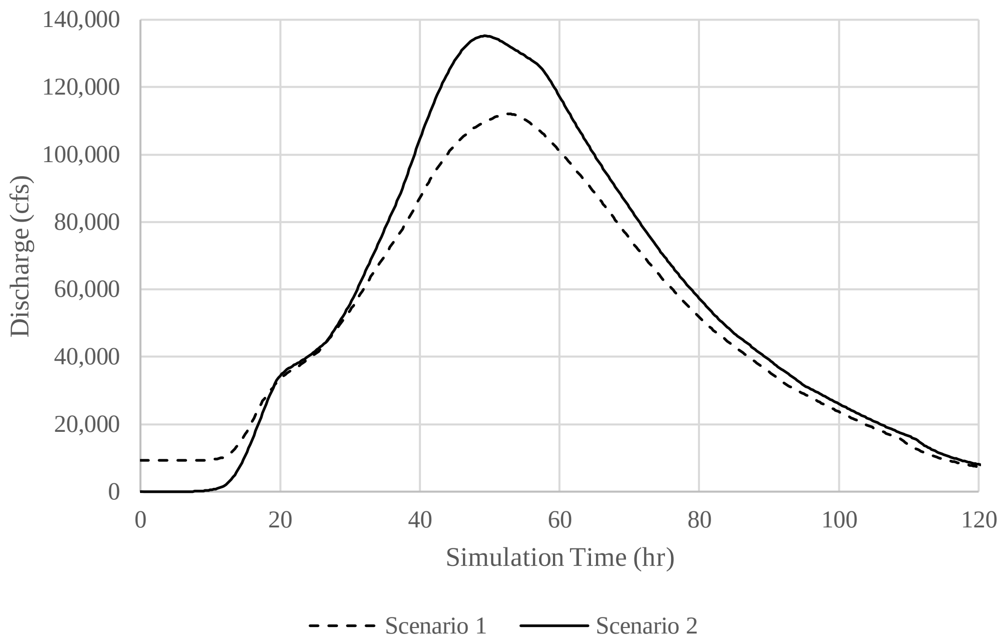

Figure 3-12. Storm area sensitivity analysis results, 100-year (1% AEP)

Based on the results of this analysis, the study team determined that Scenario #2, the "local" event, is more applicable to the Cameron Little River analysis. In addition, it produces more conservative peak flows for less frequent events, which will result in more conservative floodplain extents and lead to a more resilient design of the proposed Little River pump station. The hydrologic results presented later in this document reflect this determination.

3.4.4 Hydrologic parameters

Hydrologic parameters were not changed from those included the Lower Brazos Flood Protection Planning model used as a base for this analysis.

3.4.5 Results summary

Table 3-14 summarizes the peak flows for each frequency event. The results for each location presented in the table correspond to the depth area analysis simulation which used that evaluation point. **Figure 3-13** shows the hydrograph for each frequency event at the Cameron gage.

Table 3-14. Frequency event peak flow results at key locations (cfs)

Location Description	HEC-HMS Element	2-yr	5-yr	10-yr	25-yr	50-yr	100-yr	500-yr
USGS Gage 08106500 Little Rv nr Cameron, TX	J_Cameron	21,377	45,454	66,259	91,301	112,370	135,188	198,958
Little River Confluence with Brazos River	J_LIT_130	23,872	50,424	73,594	101,191	123,304	149,542	221,813

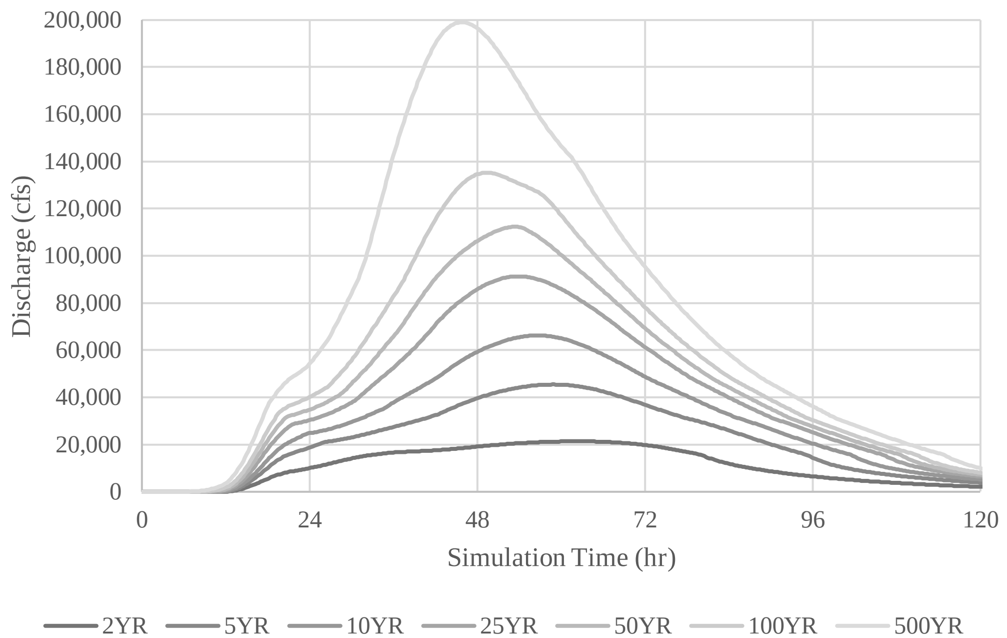

Figure 3-13. Frequency event hydrographs at USGS Gage 08106500 Little Rv nr Cameron, TX

3.5 Comparison of frequency flow estimates

Table 3-15 and **Figure 3-14** demonstrate that the HEC-HMS model produced by the Little River analysis is producing peak flows that are higher than the statistical analysis described in **Section 3.1** and those produced Lower Brazos Flood Protection Planning Study, but peak flows are lower than what was simulated in the Little River Base Level Engineering model. Comparisons to these two studies are informative for the Little River analysis, but the various studies do not use methodologies that are directly comparable. The Lower Brazos study utilized an elliptical storm, with an evaluation point that had a contributing area much larger than the area of interest for this study. If the Lower Brazos study focused on the USGS gage near Cameron, Texas with its elliptical storm analysis, it would have resulted in larger peak flows. One-dimensional Base Level Engineering analyses utilize regional regression equations to develop peak flows for hydraulic models which are much less precise than rainfall-runoff models.

Table 3-15. Frequency event peak flow comparison (cfs)

Study Name	2-yr	5-yr	10-yr	25-yr	50-yr	100-yr	500-yr
Little River HEC-HMS	21,377	45,454	66,259	91,301	112,370	135,188	198,958
Bulletin 17C Lower 95% CI	14,553	26,954	35,725	45,136	49,993	53,459	58,423
Bulletin 17C Estimate	18,195	33,076	43,575	56,940	66,741	76,296	97,541
Bulletin 17C Upper 95% CI	22,731	40,544	55,989	79,421	99,319	121,964	188,767
Little River BLE	-	-	61,040	89,551	114,048	141,054	214,241
Lower Brazos Flood Protection Planning Study	17,265	31,578	46,352	58,867	66,640	74,897	89,186

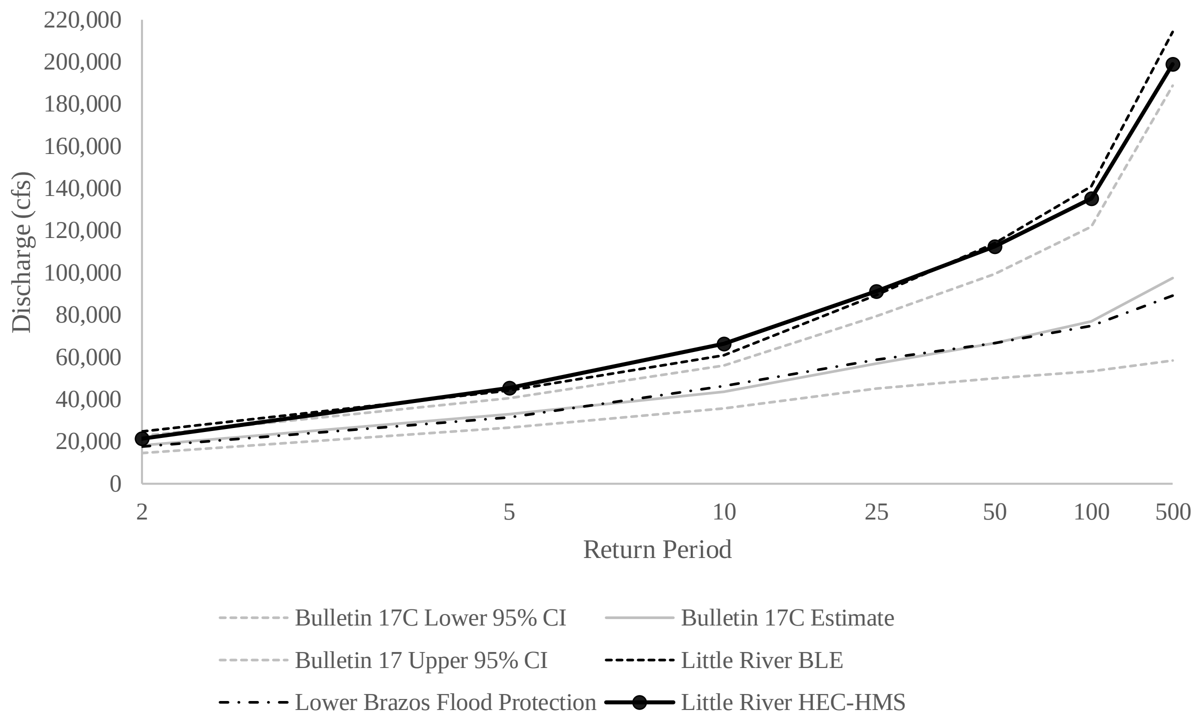

Figure 3-14. Frequency event peak flow comparison at USGS 08106500 Little Rv nr Cameron, TX

The hydrologic model from Little River study produces peak flows that are slightly outside of the 95% confidence interval of the statistical analysis, but the study team believes that the results remain reasonable and can be used to determine floodplain extents through the study area and support the design of the Little River pump station. The conservative peak flows produced by this study will also lead to conservative water surface elevations in the hydraulic model, resulting in a more resilient design of the Little River pump station. Due to the uncertain nature of the hydrologic results, the study team also conducted a sensitivity analysis that provides insight into how sensitive floodplain extents and water surface elevations are to applied frequency event flows. This information is found in **Section 4.7**.

4 Hydraulics

This section describes the methods used to develop the hydraulic model and floodplain boundary results.

4.1 Model overview and extents

FNI created a hydraulic model that includes all streams listed in Table 1-1 entirely within a single two-dimensional (2D) HEC-RAS model. HEC-RAS is an industry standard modeling software that can handle large scale two-dimensional unsteady flow simulations and resolve floodplain characteristics in a wide variety of hydraulic contexts. This, along with its sediment transfer modeling capabilities, made it the software of choice for this study. The primary stream in the watershed is the Little River, within the Lower Little River HUC10 watershed. However, a small section of the Brazos River was also included in the hydraulic model to allow for backwater effects from the Brazos River at the most downstream portion of the Little River. The Brazos River portion of the hydraulic model includes limited detail, and no hydraulic structures. The intent of its inclusion is solely to provide a reasonable tailwater condition for the Little River. The study area was presented in **Figure 1-1**, and a full depiction of the hydraulic model layout can be found in **Figure 4-3.**

4.2 2D mesh development

The terrain datasets described in **Section 2.1** were used to create a 3 ft x 3 ft composite terrain of the study area in HEC-RAS v6.1. The base 2D mesh uses a 200 ft grid cell size and a boundary equivalent to the Lower Little River HUC10 boundary with minor modifications. Near the confluence of Big Elm Creek and Little River, the 2D mesh was extended into the Big Elm Creek HUC10 to allow for backwater into Big Elm Creek from Little River. This backwater can be seen in **Figure 4-15**.

Breaklines were added so that channel banks along the Little River mainstem and other relevant topographic features such as roadway embankments and berms were captured in the 2D mesh. The Little River has an average channel width of approximately 300 ft. Breaklines with cell sizes ranging from 125 ft to 150 ft were added along the Little River centerline. This resulted in well-defined channels throughout the study area with 1-2 cells spanning the channels. A stream centerline breakline was also included for the Brazos River, and breaklines were also added to capture major roadway embankments. The mesh regenerates without the need for manual edits. An example of the level of detail built into the 2D mesh is shown below in **Figure 4-1**.

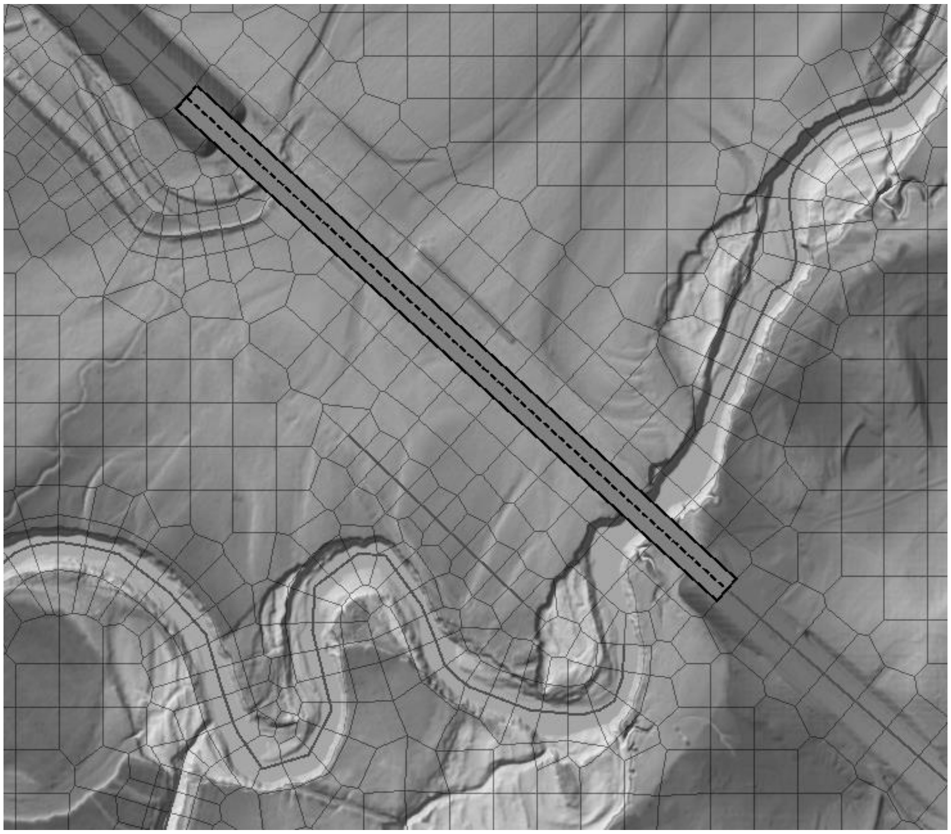

Figure 4-1. Example mesh detail

4.3 Hydraulic parameter development

The 2019 National Land Cover Dataset (NLCD) (MRLC, 2019) was used to determine primary land uses within the watershed. Manning's roughness values were assigned in accordance with the values listed in **Table 4-1**, from the HEC-RAS Reference Manual. Roughness values were manually refined near the channels where the NLCD delineations do not adequately capture the land cover using calibration regions. The initial estimates of Manning's roughness values are a primary point of calibration, which is covered in **Section 4.6**. **Figure 4-2** shows the spatial variation in NLCD land cover in the study area.

Table 4-1. HEC-RAS land cover and manning's roughness

NLCD Gridcode	Land Cover Classification	Manning's Roughness
0	NoData	0.035
11	Open Water	0.035
21	Developed, Open Space	0.04
22	Developed, Low Intensity	0.08
23	Developed, Medium Intensity	0.12
24	Developed, High Intensity	0.15
31	Barren Land (Rock/Sand/Clay)	0.03
41	Deciduous Forest	0.10
42	Evergreen Forest	0.15

43	Mixed Forest	0.12
52	Shrub/Scrub	0.08
71	Grassland/Herbaceous	0.04
81	Pasture/Hay	0.045
82	Cultivated Crops	0.05
90	Woody Wetlands	0.07
95	Emergent Herbaceous Wetlands	0.05

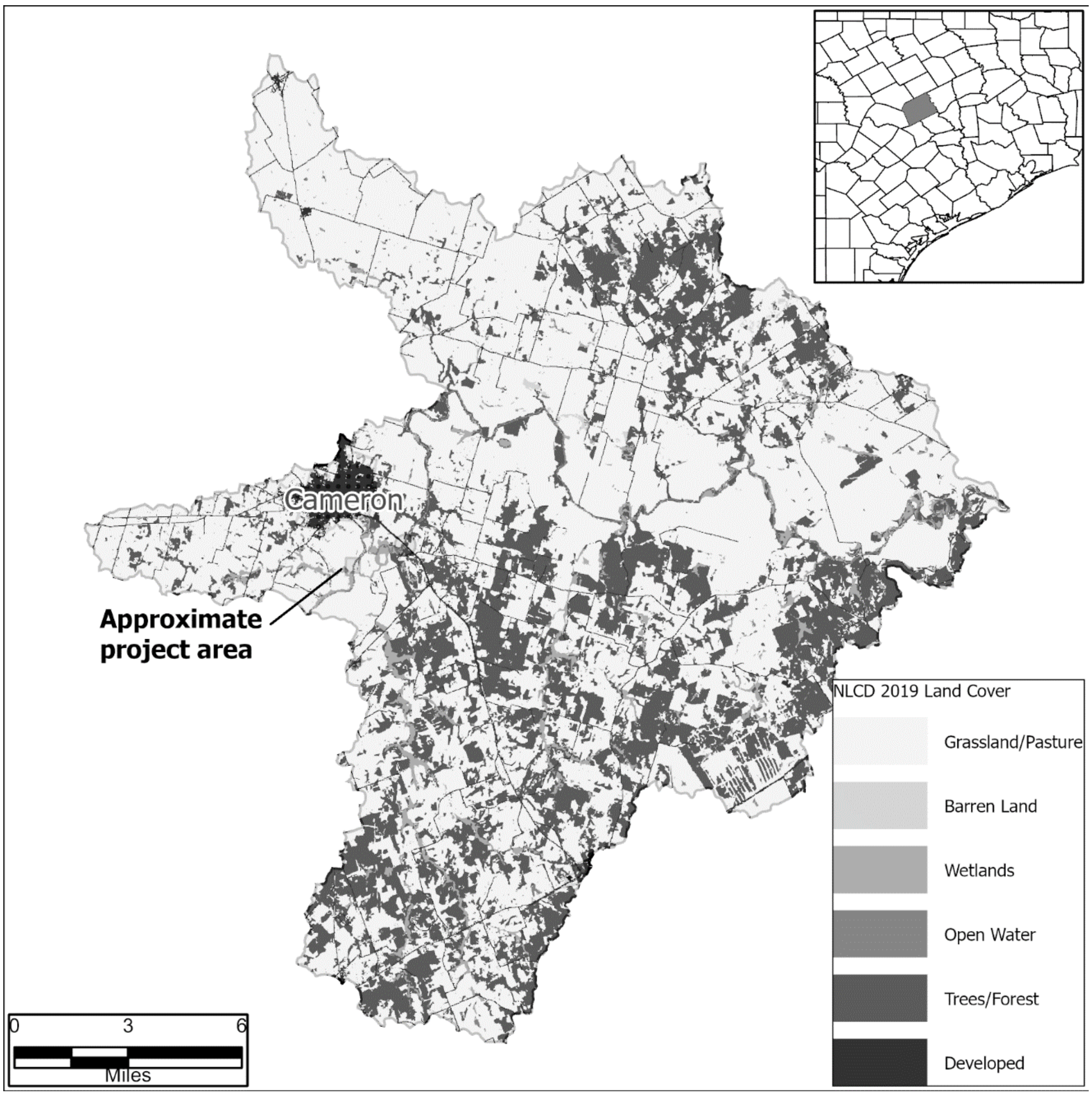

Figure 4-2. Lower Little River NLCD 2019 land cover

4.4 Boundary conditions

Inflow hydrographs for the hydraulic model were obtained from the calibrated rainfall-runoff hydrologic model for the 2-, 5-, 10-, 25-, 50-, 100-, and 500-yr events and the historical events mentioned in **Section 3.2.4**. Hydrographs taken from the rainfall-runoff model were applied to the 2D mesh as internal boundary condition lines at the outlet of each HEC-HMS subbasin. In locations where subbasin boundaries intersected with roadways, the internal boundary condition line was placed slightly upstream of the subbasin boundary to mitigate instabilities at the hydraulic structure. A normal depth external boundary condition was applied to the outlet of the study area for each frequency event along the Brazos River. The location of each boundary condition line is shown on **Figure 4-3** through **Figure 4-4.**

4.5 Hydraulic features

The 2D hydraulic model includes three hydraulic structures: the BNSF Railroad Bridge, the SH 36 Bridge, and the Existing Low Head Dam. The modeling team reviewed the study area for other hydraulic structures in the area such as reservoirs, levees, and culverts but found none of any relevance. Berms, ridges, and embankments were captured in the model using breaklines. TxDOT as-builts and FNI-gathered survey data were relied upon to accurately represent the structures in the model. The following items for bridges and low-head dams were obtained from the gathered data. When this data was missing from the provided data, the items were estimated.

- Bridges
  - Bridge deck high and low chord
  - Deck width
  - Abutments
  - Number and location of piers
  - Pier size
- Low-head dam
  - Weir crest elevation
  - Weir crest width

For structures where no as-built data is available, a desktop review was performed to estimate structure configuration based on aerial imagery, LiDAR, nearby structures, and online TxDOT resources. Items such as deck width and number of piers can be reasonably estimated from these data sources. Other items, such as pier width, must be assumed.

Hydraulic property curves (i.e., free- and submerged flow rating curves) were carefully reviewed for both bridges to ensure smooth transitions between flow regimes. The number of curves, number of points on the curves, and maximum elevations/flows were adjusted when necessary. The hydraulic property curves and bridge modeling approach for each structure was evaluated and adjusted as needed for low and high flow scenarios. **Table 4-2** lists the structures included in the hydraulic model and their respective data sources. The location of each structure can be seen on **Figure 4-3** through **Figure 4-4.**

Table 4-2. Modeled hydraulic structures

Stream	Connection Name	Roadway/Railroad Name	Data Source
Little River	LR_IC1	Existing Low Head Dam	Field Survey
Little River	LR_IC2	SH 36	TxDOT Plans
Little River	LR_IC3	BNSF Railroad	Field Measurement
Little River Relief	LR_IC4	BNSF Railroad	Field Measurement
Little River	LR_IC5	CR 277	Estimate
Little River	LR_IC6	CR 264	Estimate

Near the existing low head dam, the existing pump station intake structure was incorporated into the model through the terrain modification feature within RASMapper. The extents, location, and elevation of intake structure were based on plan data. A single breakline was utilized to capture the structure in the 2D mesh.

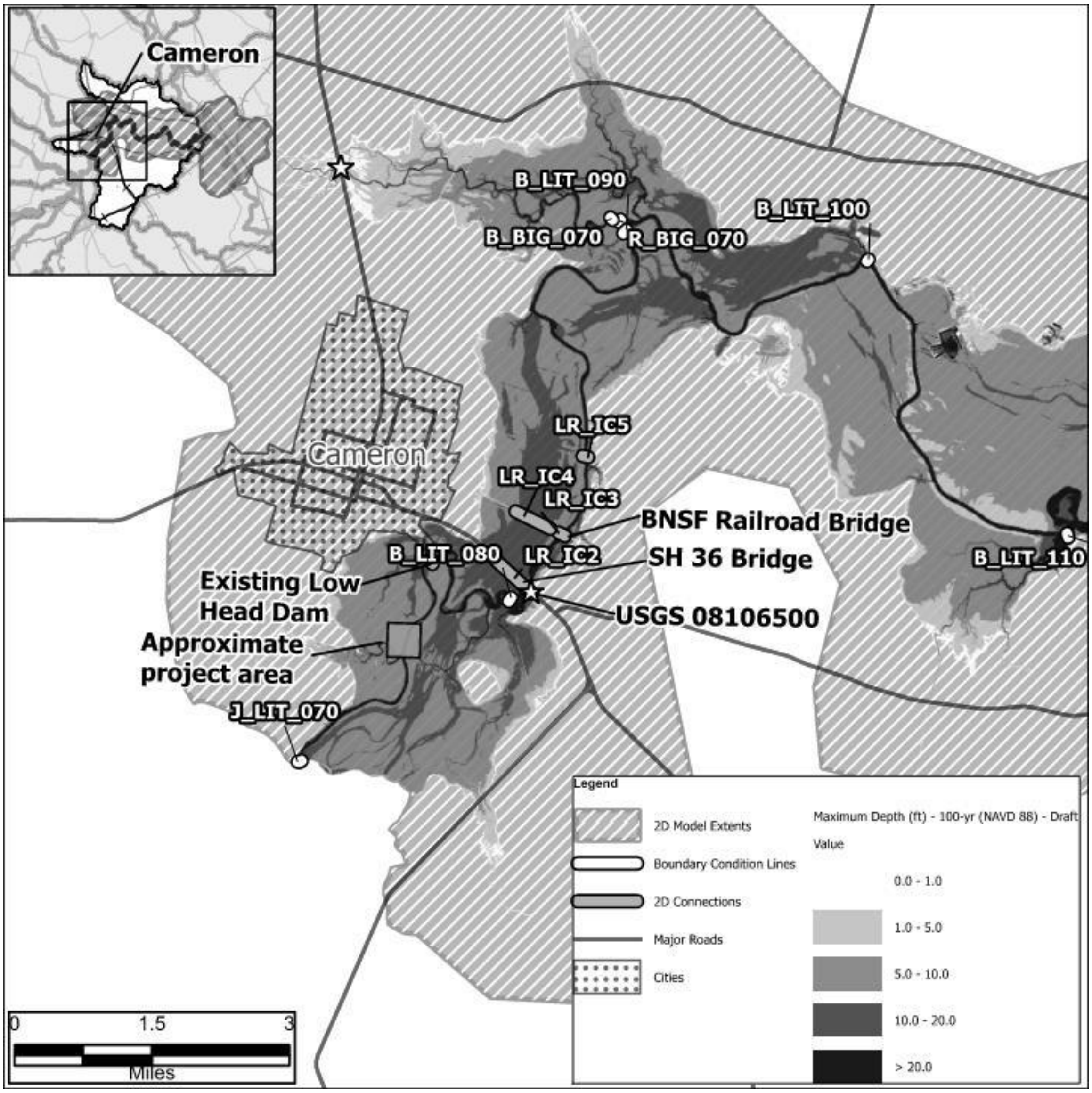

Figure 4-3. Lower Little River hydraulic model layout

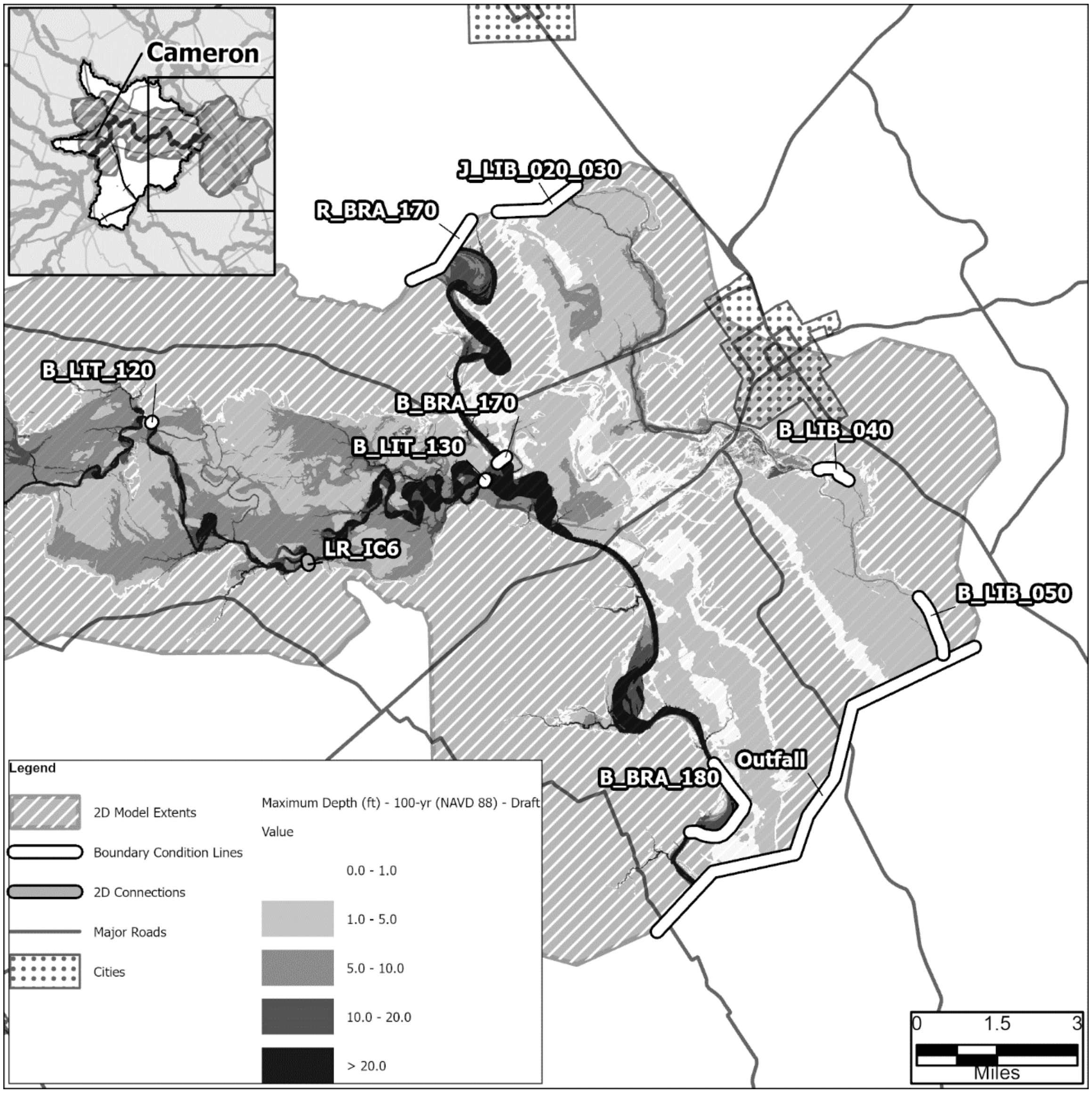

Figure 4-4. Lower Little River hydraulic model layout

4.6 Calibration and validation

After developing the initial HEC-RAS model described in the preceding sections, the study team calibrated the hydraulic model to verify it was accurately simulating the response of the watershed over a range of events. The model was calibrated to two observed events: Hurricane Harvey 2017 and Memorial Day 2016, which were present in the existing hydrologic model used as a base for this study. These two observed events are small in magnitude through the Little River and do not produce peak stages comparable to the 100-yr and 500-yr floodplain which are critical to a resilient design of the pump station. In absence of observed events that are comparable to the 100-yr and 500-yr frequency events, the study team also used the published USGS rating curve of USGS gage 08106500 Little Rv nr Cameron, TX as a validation point to verify the results of the hydraulic model.

The primary calibration parameter in the hydraulic model is the Manning's roughness coefficient. Manning's roughness values are applied to the HEC-RAS model in two locations, bridge internal cross sections and a land cover table which defines roughness values within the 2D mesh based on the NLCD 2019 dataset. Adjustments to Manning's roughness in the hydraulic model are known to influence the timing, peak flow, and peak stage of a hydraulic models' simulated response hydrograph. The study team adjusted Manning's roughness values in a consistent manner throughout the watershed by scaling the initial roughness estimates as needed to produce quality results in the observed historical events and USGS Rating Curve calibration described later in this section. All land classifications in the NLCD layer were adjusted uniformly, except the "Open Water" classification which was kept at the standard value. The final Manning's roughness values following calibration are presented in **Table 4-3**.

Table 4-3. Calibrated HEC-RAS land cover and manning's roughness

NLCD Gridcode	Land Cover Classification	Initial Manning's Roughness	Calibrated Manning's Roughness
-	Channel	0.04	0.035
0	NoData	0.035	0.035
11	Open Water	0.035	0.035
21	Developed, Open Space	0.04	0.07
22	Developed, Low Intensity	0.08	0.14
23	Developed, Medium Intensity	0.12	0.21
24	Developed, High Intensity	0.15	0.26
31	Barren Land (Rock/Sand/Clay)	0.03	0.05
41	Deciduous Forest	0.10	0.18
42	Evergreen Forest	0.15	0.26
43	Mixed Forest	0.12	0.21
52	Shrub/Scrub	0.08	0.14
71	Grassland/Herbaceous	0.04	0.07
81	Pasture/Hay	0.045	0.08
82	Cultivated Crops	0.05	0.09
90	Woody Wetlands	0.07	0.12
95	Emergent Herbaceous Wetlands	0.05	0.09

4.6.1 Historical events

This section describes the historical storm events used to calibrate the existing conditions hydraulic model.

Hurricane Harvey 2017

The Hurricane Harvey 2017 event was primarily contained within the channel banks of the Little River, so the study team used this event solely to calibrate channel Manning's roughness values. Initial simulations showed that the model was slightly overpredicting water surface elevations compared to the observed data, so the initial channel Manning's roughness estimate of 0.04 was reduced to 0.035. Comparisons of hydrologic, hydraulic, and USGS observed data are presented in **Figure 4-5** through **Figure 4-6**, and **Table 4-4** through **Table 4-5**. These tables and figures demonstrate that hydrologic model, hydraulic model, and observed data reasonably agree for the Hurricane Harvey 2017 event.

The calibrated hydraulic model produces peak elevations and flows that agree with the hydrologic model and are within 1.04 ft and 11% of the observed data, respectively. The hydraulic model shows slight increases in simulated flows compared to HEC-HMS, which were also higher than the observed data. So, the hydraulic model accurately simulated a higher water surface elevation than was observed. As mentioned in **Section 2.1,** bathymetric data was not readily available for use in this model, which may contribute to the discrepancy in the rising limb of the response hydrograph shown in **Figure 4-6**. A review of **Figure 4-6** also shows that the timing and shape of the response hydrograph agrees with the observed data better than the timing metric in **Table 4-5** suggests.

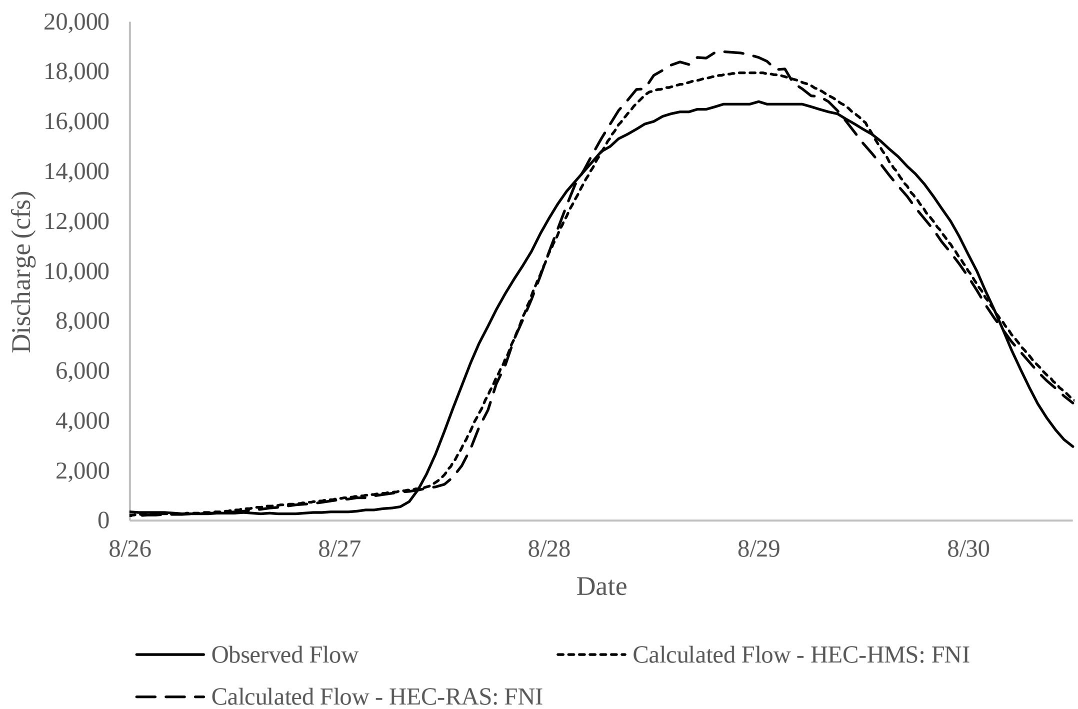

Figure 4-5. Harvey 2017 observed vs. calculated flow hydrograph at USGS Little Rv nr Cameron, TX

Table 4-4. Harvey 2017 observed vs. calculated flow at USGS Little Rv nr Cameron, TX

Data Source	Peak Flow (cfs)	Time of Peak
USGS 08106500	16,800	29Aug2017 0000
HEC-HMS: FNI	17,961	28Aug2017 2315
HEC-RAS: FNI	18,791	28Aug2017 2000

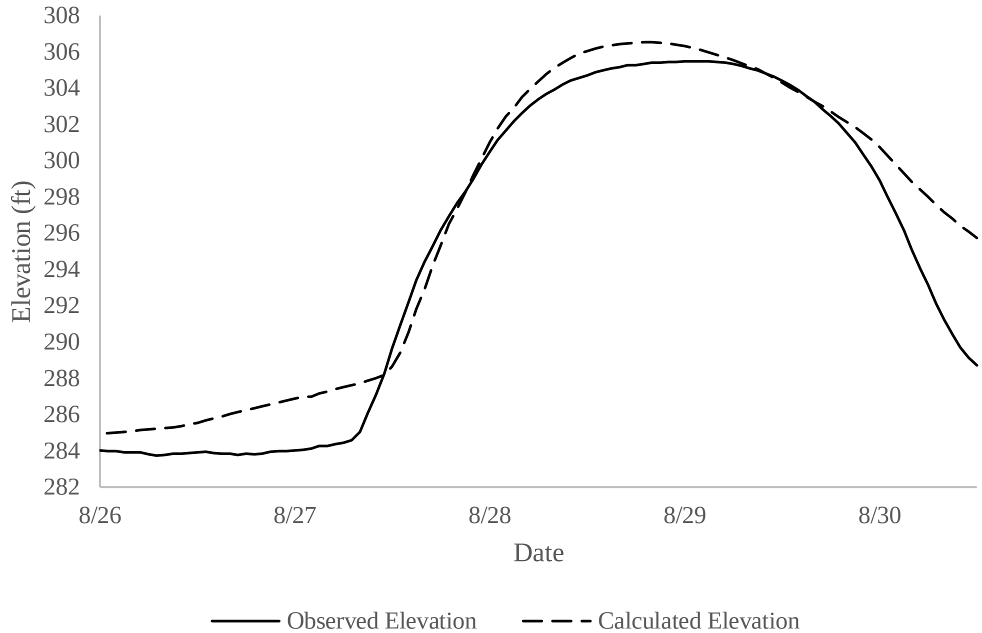

Figure 4-6. Harvey 2017 observed vs. calculated elevation at USGS Little Rv nr Cameron, TX

Table 4-5. Harvey 2017 observed vs. calculated elevation at USGS Little Rv nr Cameron, TX

Data Source	Peak Elevation (ft)	Time of Peak
USGS 08106500	305.49	29Aug2017 0000
HEC-RAS	306.53	28Aug2017 1900
Difference	1.04	-5 hr

Memorial Day 2016

Like the Harvey 2017 event, the Memorial Day 2016 event was primarily contained within the channel banks of the Little River. So, the channel Manning's roughness adjustment from 0.04 to 0.035 was also applied to the Memorial Day event. Comparisons of hydrologic, hydraulic, and USGS observed data are presented in through **Figure 4-8**, and **Table 4-6** through **Table 4-7**. These tables and figures demonstrate that hydrologic model, hydraulic model, and observed data reasonably agree for the Hurricane Harvey 2017 event.

The calibrated hydraulic model produces peak elevations and flows that agree with the hydrologic model and are within 0.57 ft and 8% of the observed data, respectively. Like the Harvey simulation, the hydraulic model produces slightly higher water surface elevations than observed by USGS gage 08106500 because the calibrated hydrologic flows are higher than observed. The shape of the stage hydrograph shown in **Figure 4-8** doesn't match the observed data well, but it does provide peak results that agree with the observed data. Although the overall shape of the stage hydrograph does not follow the observed data, it does reproduce the shape and timing provided by inflows from the calibrated hydrologic model.

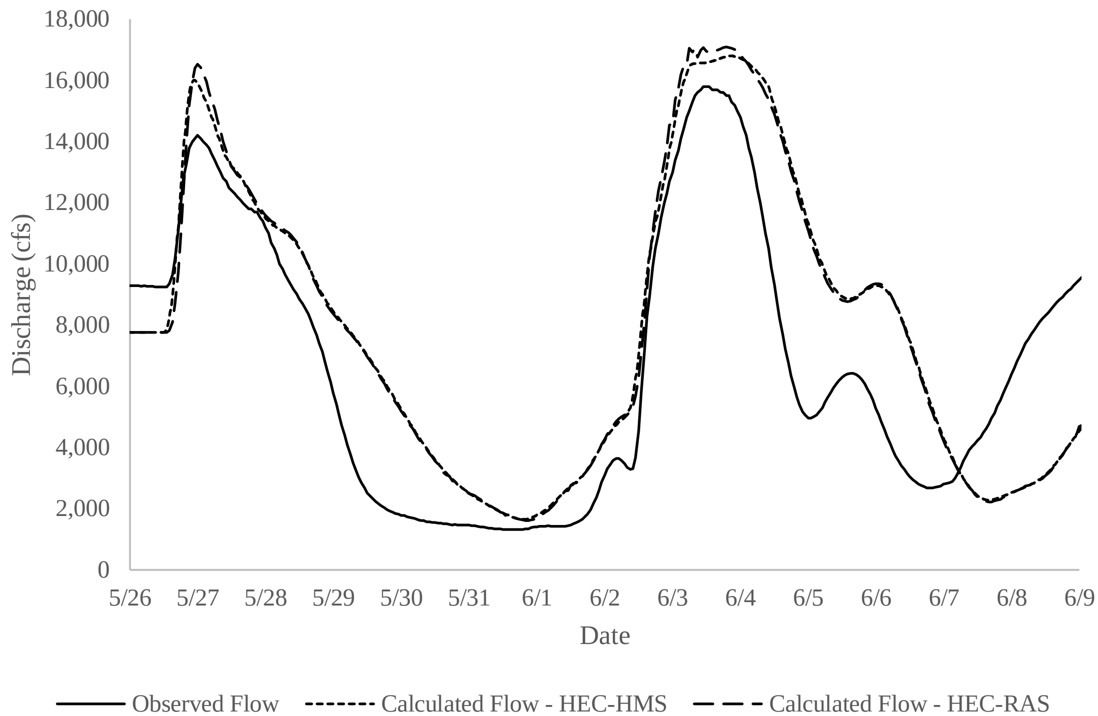

Figure 4-7. Memorial Day 2016 observed vs. calculated flow hydrograph at USGS Little Rv nr Cameron, TX

Table 4-6. Memorial Day 2016 observed vs. calculated flow hydrograph at USGS Little Rv nr Cameron, TX

Data Source	Peak Flow (cfs)	Time of Peak
USGS 08106500	15,800	03Jun2016 1100
HEC-HMS	16,798	03Jun2016 2045
HEC-RAS	17,086	03Jun2016 1900

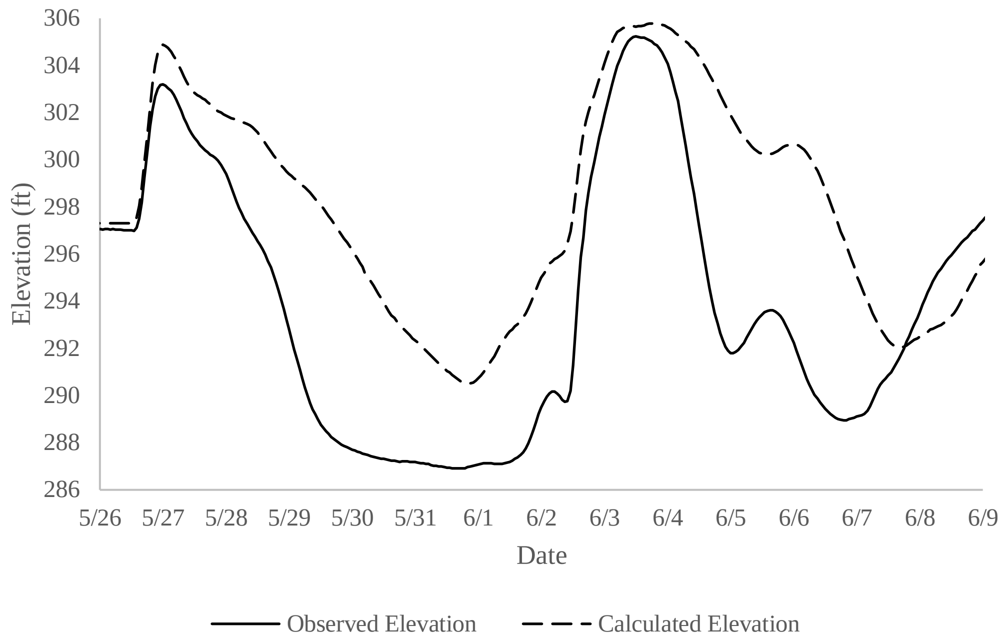

Figure 4-8. Memorial Day 2016 observed vs. calculated elevation at USGS Little Rv nr Cameron, TX

Table 4-7. Memorial Day 2016 observed vs. calculated elevation at USGS Little Rv nr Cameron, TX

Data Source	Peak Elevation (ft)	Time of Peak
USGS 08106500	305.23	03Jun2016 1200
HEC-RAS	305.80	03Jun2016 1900
Difference	0.57	7 hr

4.6.2 USGS rating curve

Figure 4-9 presents data related to the published USGS gage rating curve, including the rating curve, USGS field measurements, and results from the calibrated hydraulic model. The study team used the USGS rating curve as a point of comparison for hydraulic calibration and adjusted Manning's roughness values in the HEC-RAS model to obtain a closer match between the model's calculated rating curve versus the USGS rating curve. The results of the hydraulic model, represented by "Historical Event Peak (HEC-RAS)" and "Frequency Event Peak (HEC-RAS)" series in **Figure 4-9** align reasonably well with the USGS rating curve.

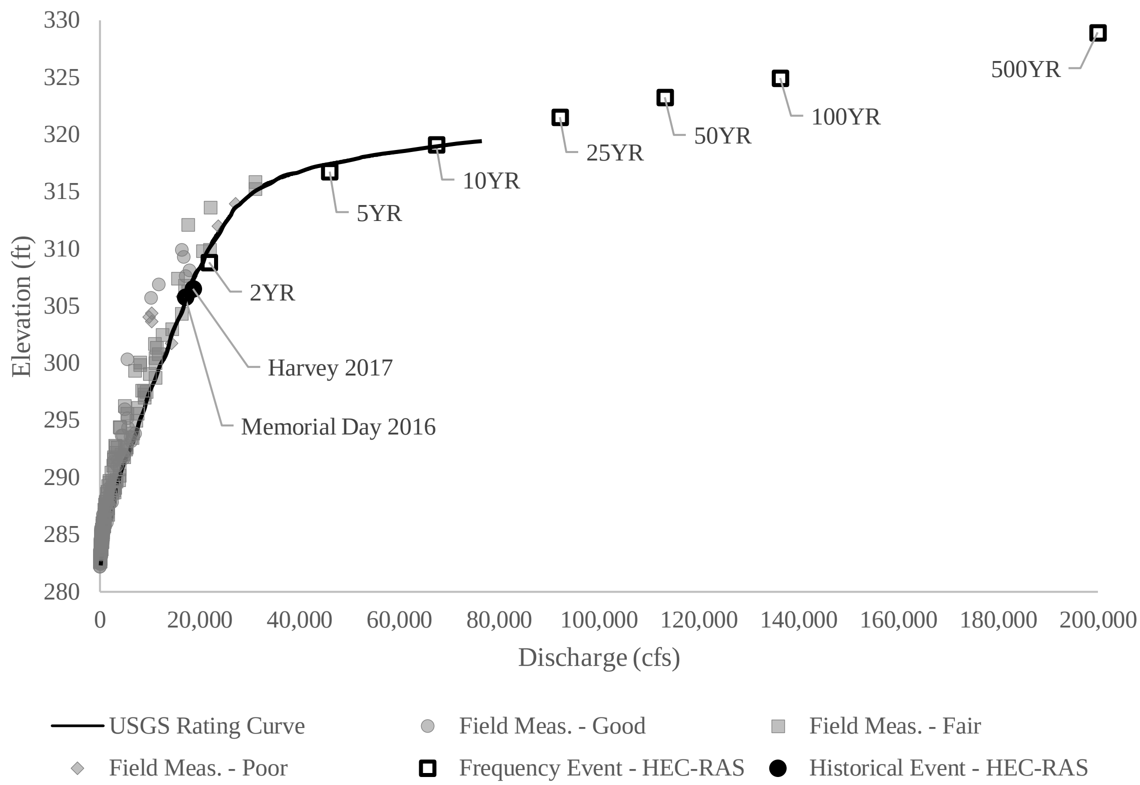

Figure 4-9. USGS 08106500 Little Rv nr Cameron, TX – rating curve comparison

4.7 Floodplain extents sensitivity analysis

Frequency event flows produced by the HEC-HMS model developed for this study are conservative and higher than what is calculated by the statistical analysis presented in **Section 3.1**. To demonstrate that peak flows are not overly conservative, the study team conducted a sensitivity analysis on floodplain extents for multiple flow conditions.

Two flow scenarios were analyzed as part of the sensitivity analysis: unaltered HEC-HMS results and HEC-HMS results scaled down to match peak flows calculated by the statistical analysis. In the 100-yr event the HEC-HMS model produced a peak flow of 135,188 cfs at USGS 08106500 (J_Cameron) while the statistical analysis calculated a peak flow of 76,296 cfs. The modeling team added a flow ratio factor of 0.56 to match the statistical peak flow to the original HEC-HMS inflow hydrographs and simulated this plan in the calibrated hydraulic model. The same methodology was applied to the 500-YR event as well. The differences in floodplain extents near the City of Cameron are shown in the figures below.

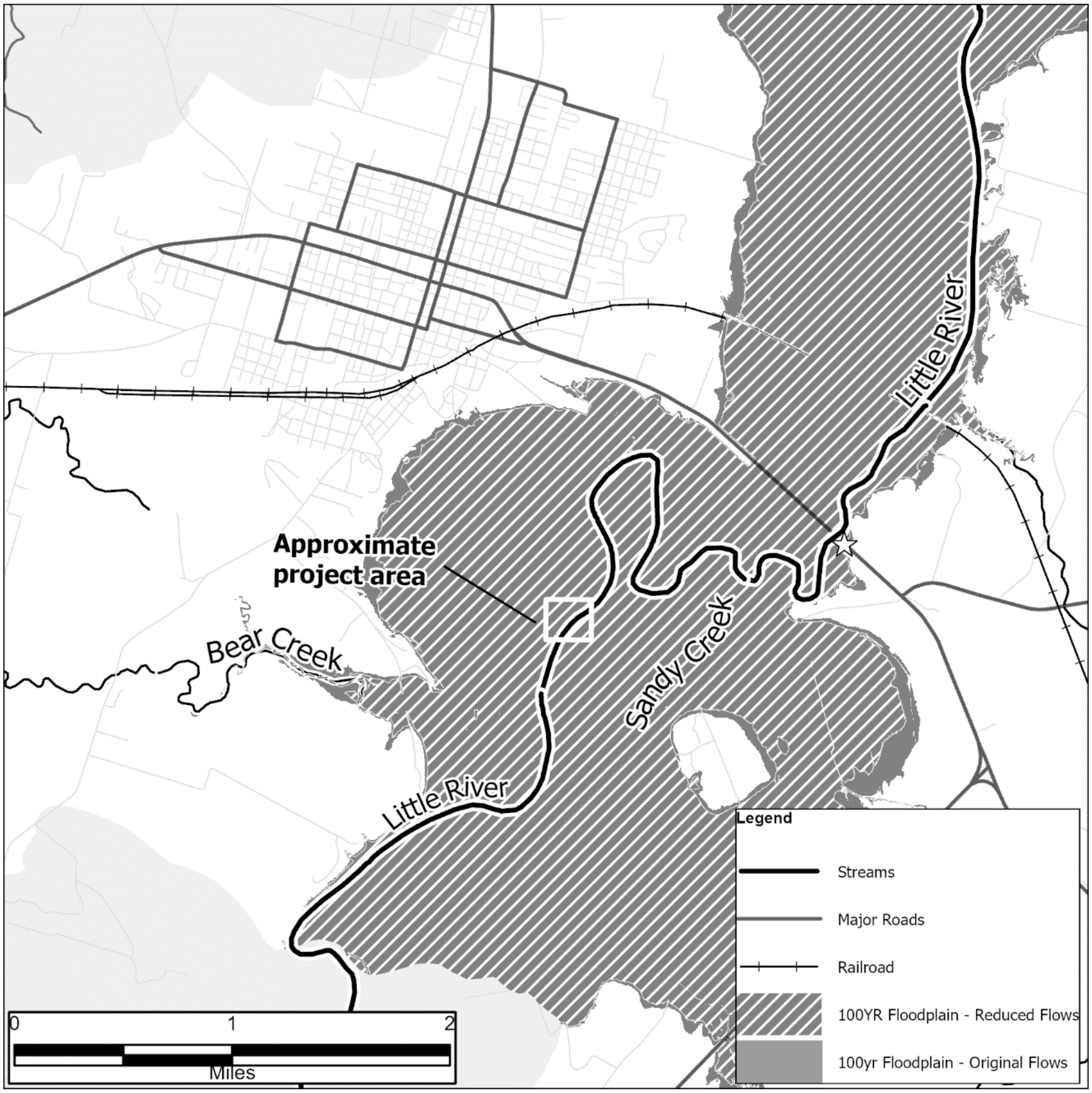

Figure 4-10. floodplain extents comparison – 100-yr

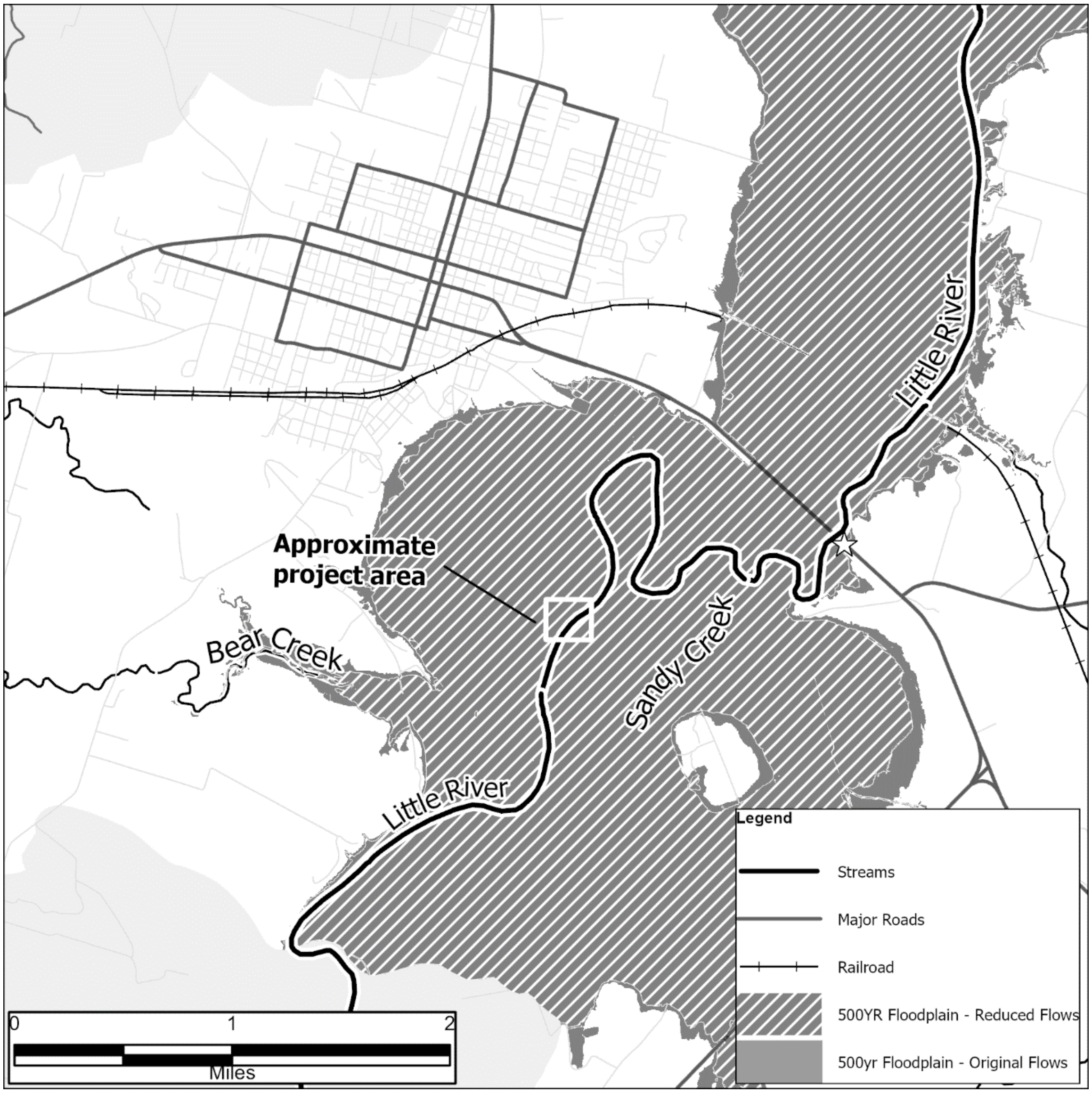

Figure 4-11. Floodplain extents comparison – 500-yr

This sensitivity analysis revealed that reducing peak flows from the hydrologic model does not substantially change floodplain extents in the 100-yr event. Therefore, the study team recommends maintaining the conservative peak flows resulting from the HEC-HMS model which will lead to a more resilient design of the proposed Little River pump station.

4.8 Frequency events

Following the calibration of the hydraulic model, the frequency events were then simulated in HEC-RAS by applying the frequency event results from the hydrologic model. Inflow hydrographs were applied for each event using the methodology described in **Section 4.4**, and the model's results were recorded at the USGS stream gage on Little River near Cameron, Texas. All hydraulic results presented in the following sections are in reference to the hydrologic depth-area analysis set at USGS gage 08106500 Little Rv nr Cameron, TX (J_Cameron).

4.8.1 Results summary

Table 4-8 and **Table 4-9** summarize the peak flow and peak elevation for each frequency event at SH 36, where USGS gage 08106500 is located. **Figure 4-12** and **Figure 4-13** present the flow and stage hydrographs for each frequency event at the same location. **Figure 4-14** through **Figure 4-19** present the 100-yr and 500-yr floodplains through the Lower Little River HUC10.

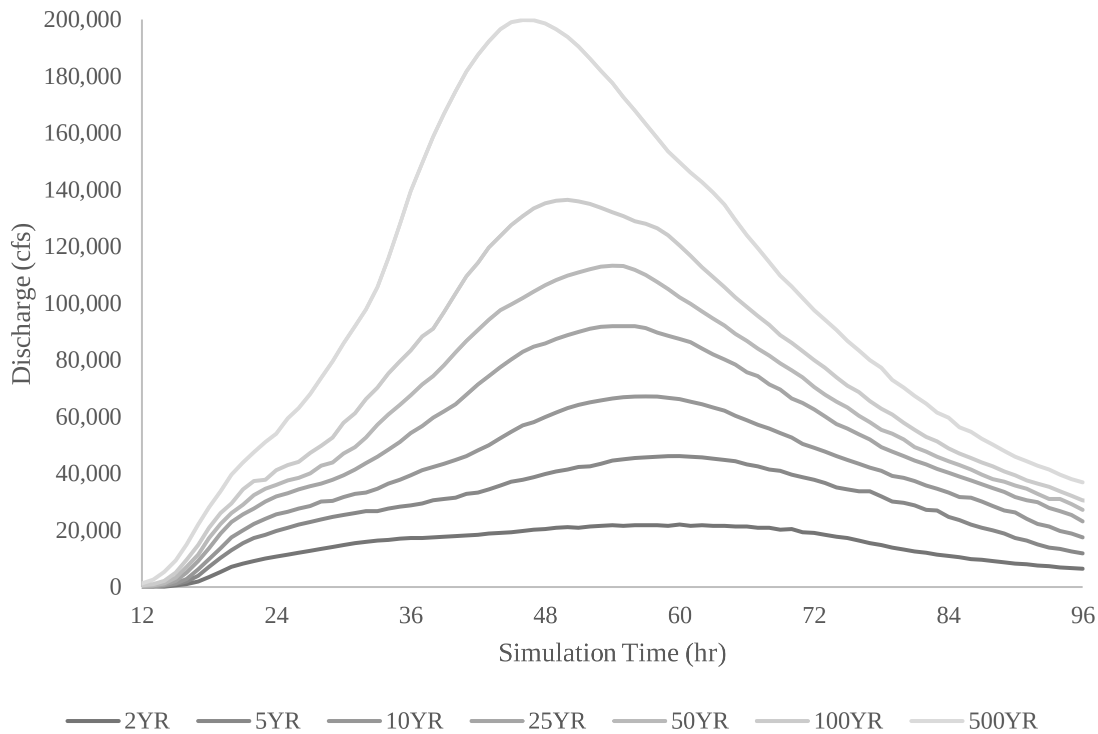

Figure 4-12. Frequency event flow hydrographs at SH 36 (USGS 08106500)

Table 4-8. Frequency event peak flow at SH 36 (USGS 08106500)

Return Interval	Peak Discharge (cfs)
2YR	22,029
5YR	46,123
10YR	67,164
25YR	91,925
50YR	113,236
100YR	136,441
500YR	199,800

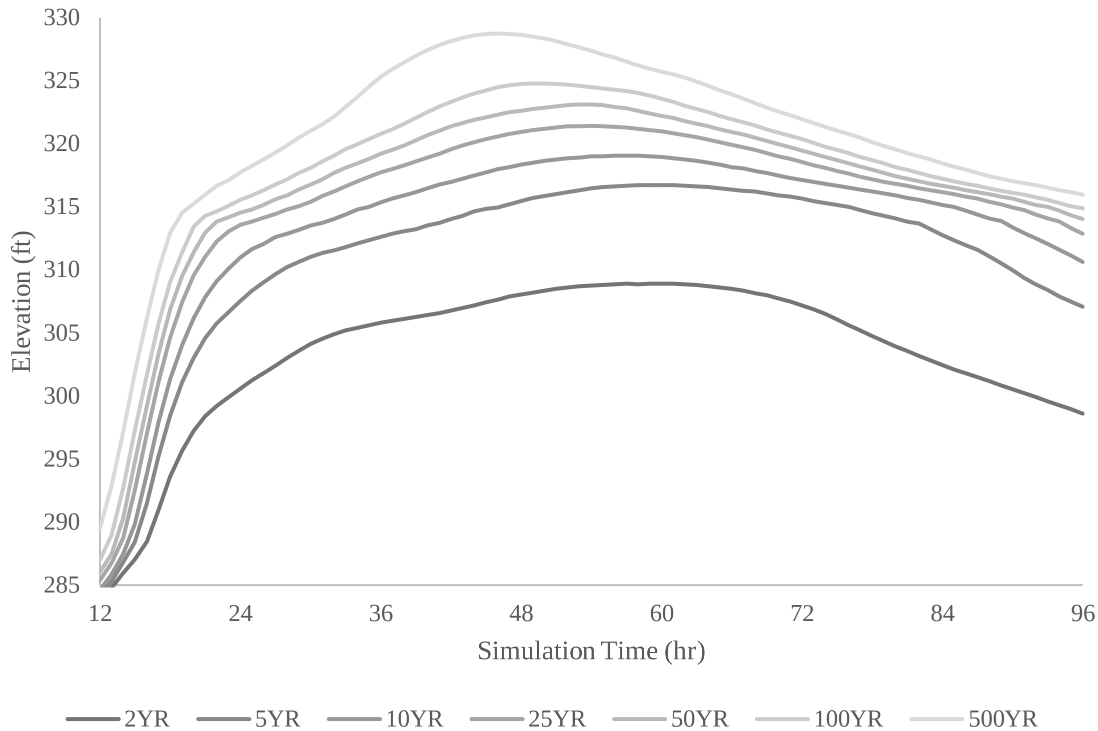

Figure 4-13. Frequency event stage hydrographs at SH 36 (USGS 08106500)

Table 4-9. Frequency Event Peak Stage at SH 36 (USGS 08106500)

Return Interval	Peak Elevation (ft)
2YR	308.90
5YR	316.71
10YR	319.05
25YR	321.41
50YR	323.11
100YR	324.77
500YR	328.74

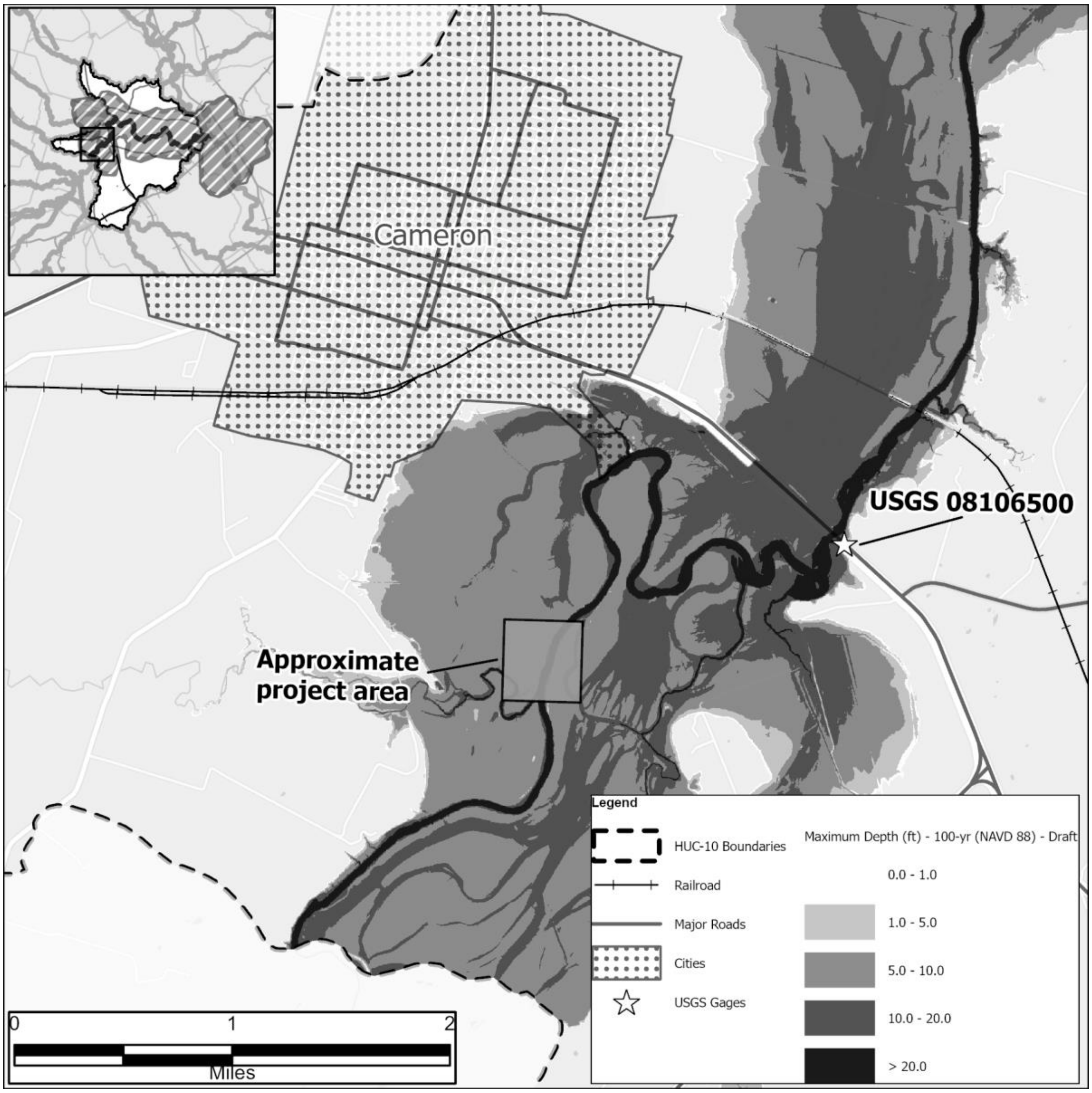

Figure 4-14. **100-yr floodplain mapping**

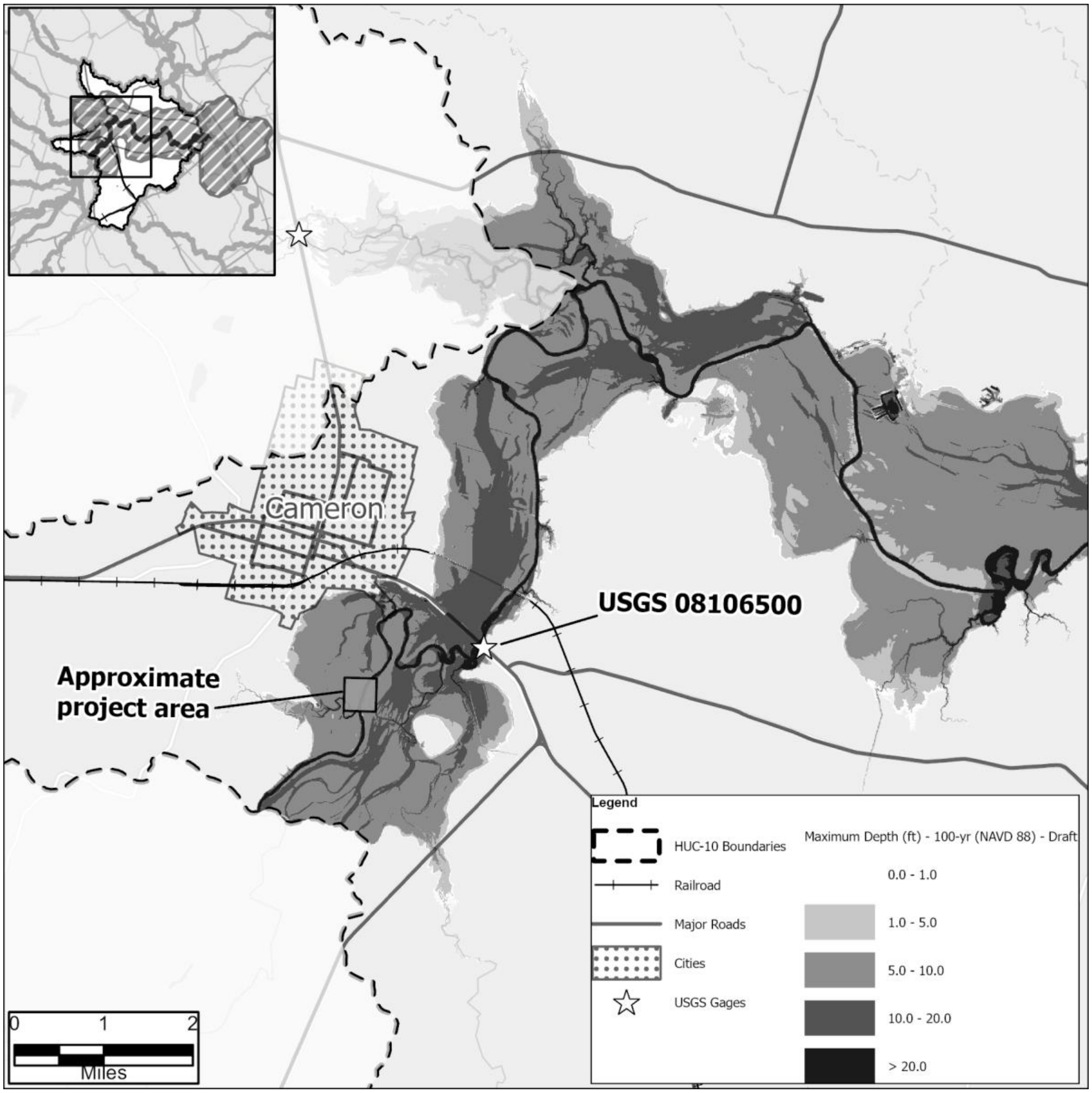

Figure 4-15. 100-yr floodplain mapping

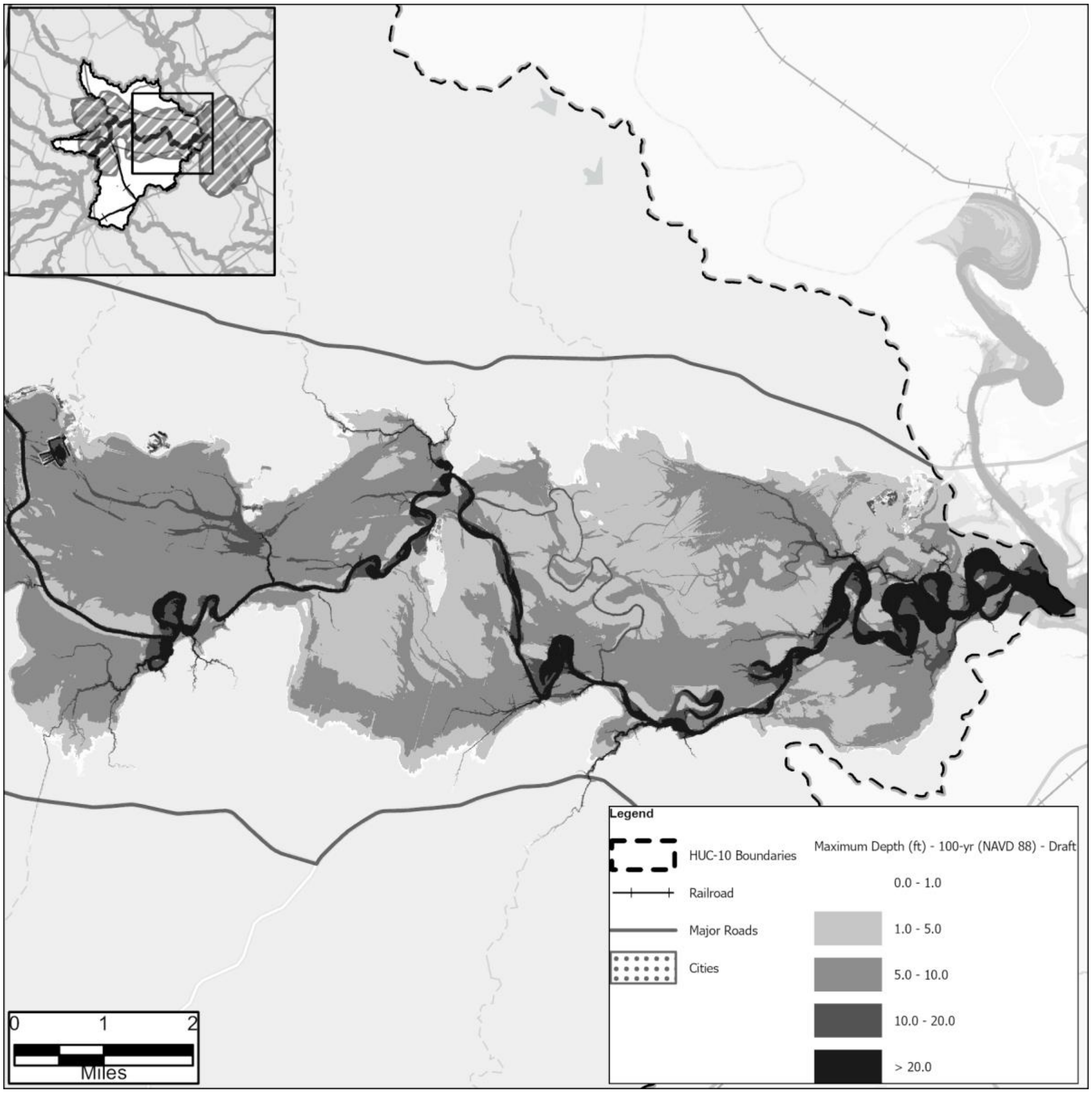

Figure 4-16. 100-yr floodplain mapping

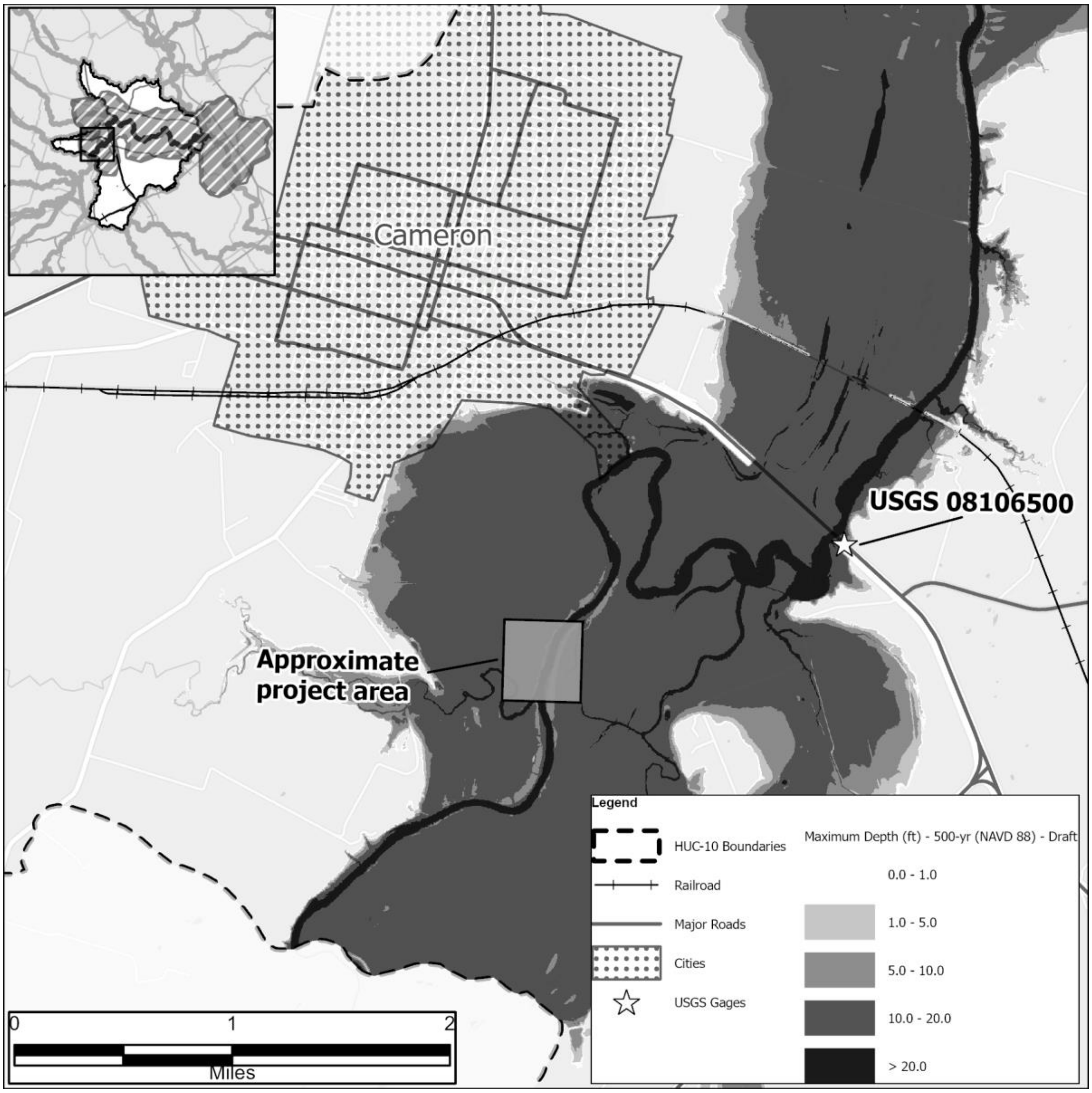

Figure 4-17. 500-yr floodplain mapping

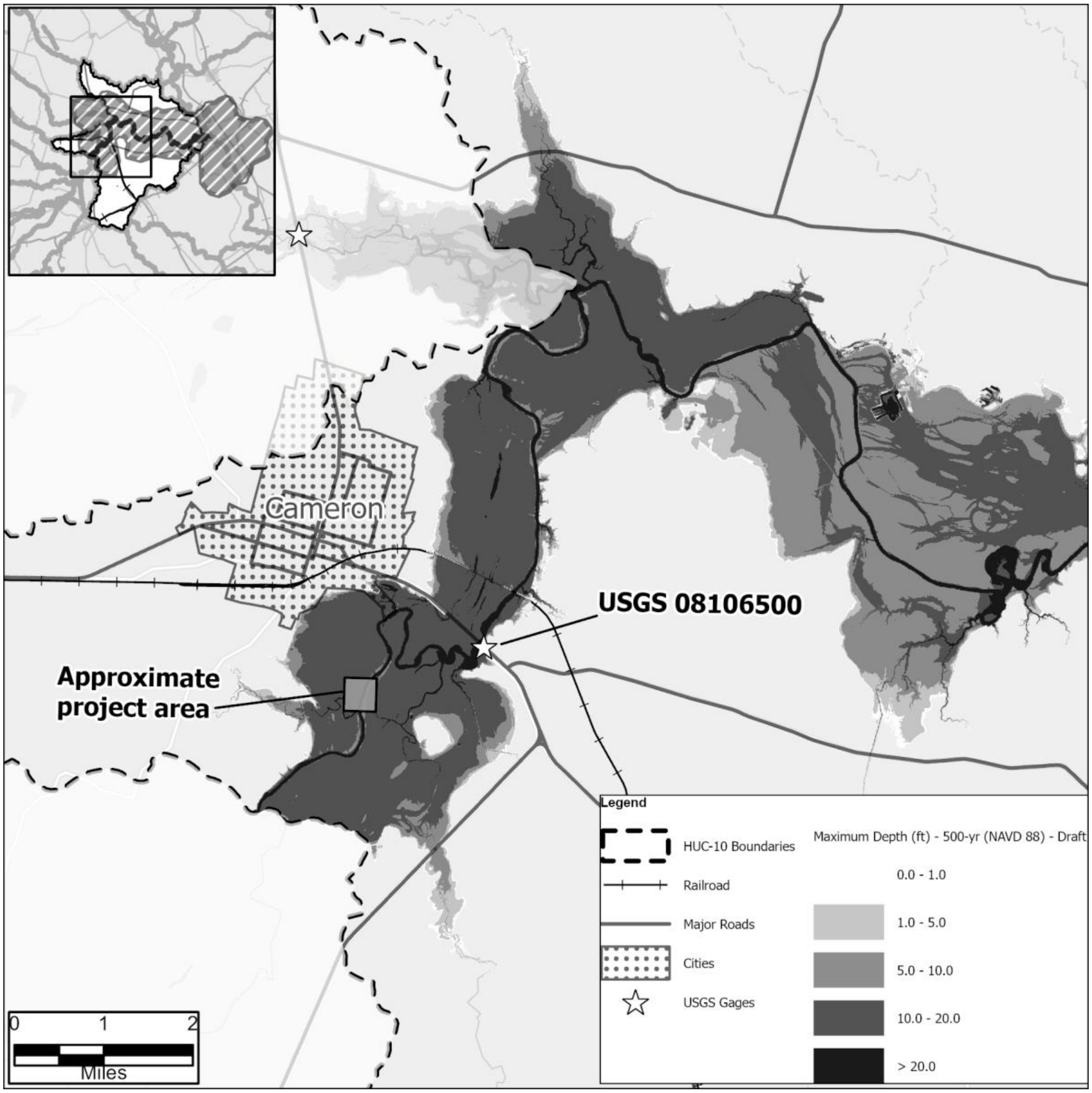

Figure 4-18. 500-yr floodplain mapping

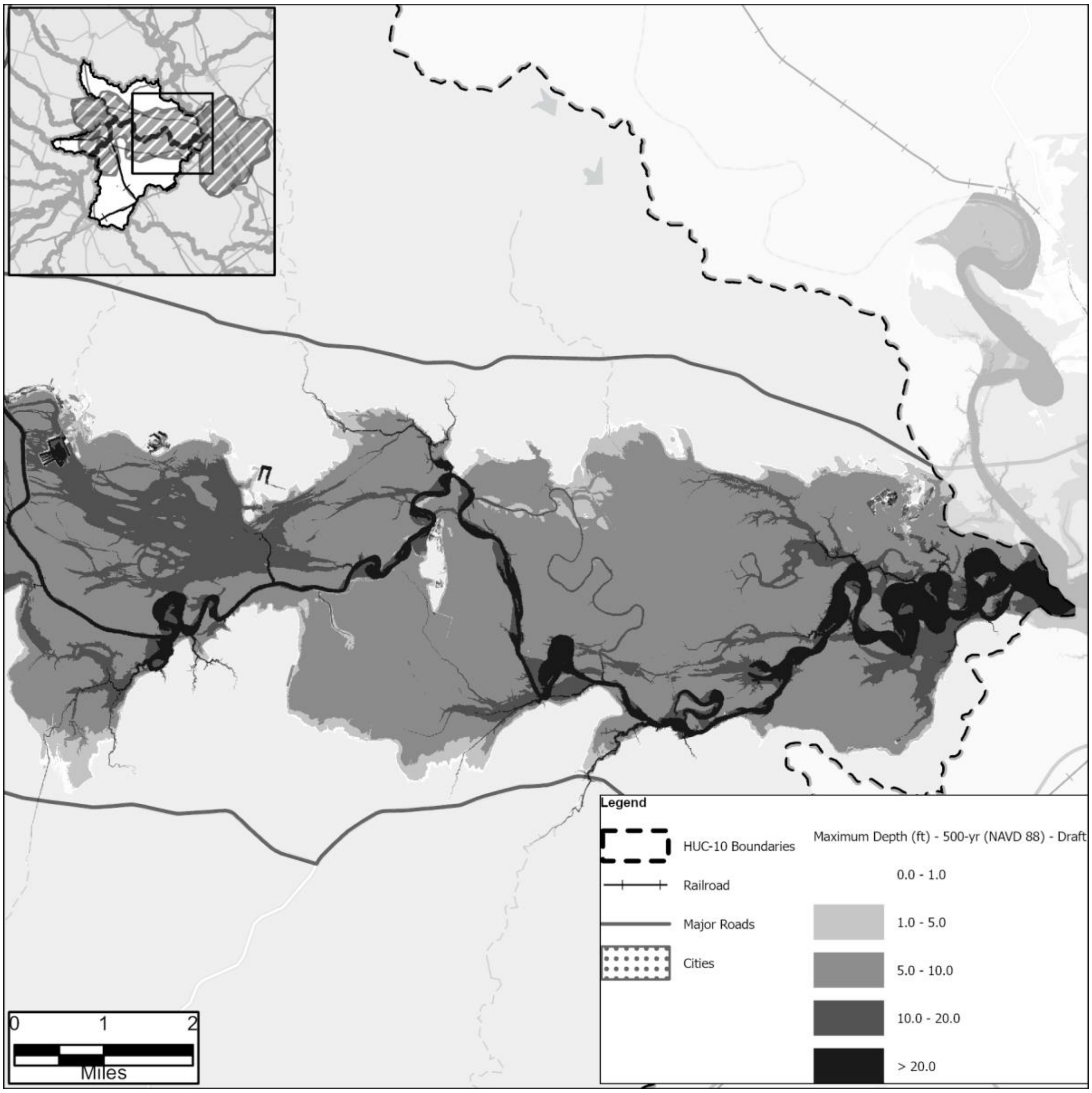

Figure 4-19. 500-yr floodplain mapping

5 Alternatives analysis

This section describes the analysis of the City of Cameron's proposed low head dam and pump station intake structure on flood water elevations.

5.1 Hydraulic model development

This section gives an overview of how the existing conditions hydraulic model was modified to reflect the preliminary design of the proposed low head dam and pump station intake structure.

5.1.1 Proposed condition overview

As mentioned in the **Executive Summary**, this hydraulic and hydrologic analysis is intended to support the design of a new raw water pump station for the City of Cameron. In relation to the hydraulic analysis described in this report, the primary features that describe the proposed condition are:

- Construction of new low-head dam
- Construction of new pump station intake structure
- Notch existing low-head dam that passes normal streamflow without impounding water

The proposed dam will be a concrete structure constructed across the main channel of the Little River, with a total weir length of approximately 100 ft at an elevation of approximately 291.0 ft (NAVD 88). The channel invert elevation at the proposed location is approximately 285.0 ft (NAVD 88), resulting in a maximum total dam height of 6.00 ft. The weir width will be approximately 1.5 ft. **Figure 5-1** shows a plan view of the proposed low-head dam. The new low-head dam and intake structure will be constructed approximately 0.9 miles upstream of the existing intake structure.

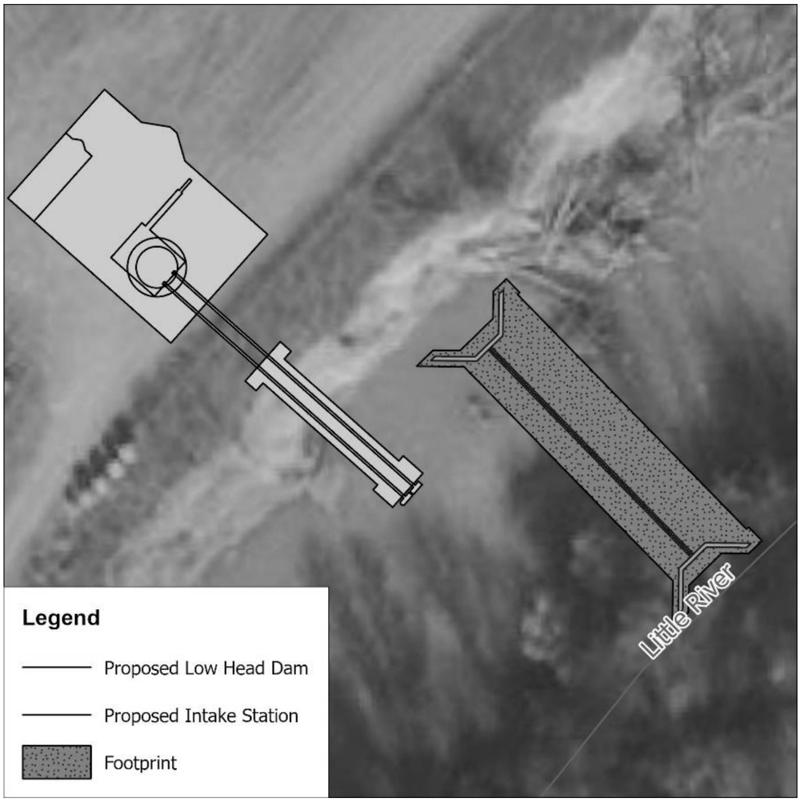

Figure 5-1. Proposed low head dam plan-view

5.1.2 Terrain development

Due to a lack of bathymetric data for the section of Little River included in this analysis, the proposed condition necessitates modifications to the terrain. The modifications to the proposed condition terrain are described in this section.

Upstream of proposed low-head dam

The existing low-head dam has a weir crest elevation of 293.5 ft (NAVD 88), which results in a normal channel water surface elevation of 293.5 ft (NAVD 88). This normal water surface elevation is reflected in the terrain data described in **Section 2.1.** The new low-head dam will have a weir crest elevation of 291.0 ft (NAVD 88), which is lower than the existing low head-dam along with the normal water surface elevation in the channel. To develop a proposed condition terrain that agrees with the proposed low-head dam, the study team developed a proposed condition channel terrain through an interim HEC-RAS 2D geometry modification that lowered the channel elevation upstream of the new low-head dam to an elevation of 291.0 ft (NAVD 88).

Between existing low-head dam and proposed low-head dam

In the proposed condition, the existing low-head dam will be notched in a manner that will pass normal stream flows without impounding water. This notch will remove the normal water surface elevations currently caused by the existing weir, so the terrain between the existing dam and proposed dam needs to reflect channel invert elevations rather than a normal water surface elevation.

In the existing condition, normal stream flow conditions result in an elevation of approximately 289 ft (NAVD 88) downstream of the existing dam, which drops at an average slope of 0.0001 ft/ft as the Little River continues to its confluence with the Brazos River. To simulate a proposed condition that agrees with the existing terrain downstream of the existing dam, the study team extended the channel terrain upstream to the proposed dam location at slope of 0.0001 ft/ft beginning at elevation 289 ft (NAVD 88). This results in a channel elevation of approximately 289.5 ft (NAVD 88) just downstream of the proposed dam. **Figure 5-2** summarizes the resulting channel terrain used in the proposed condition simulations.

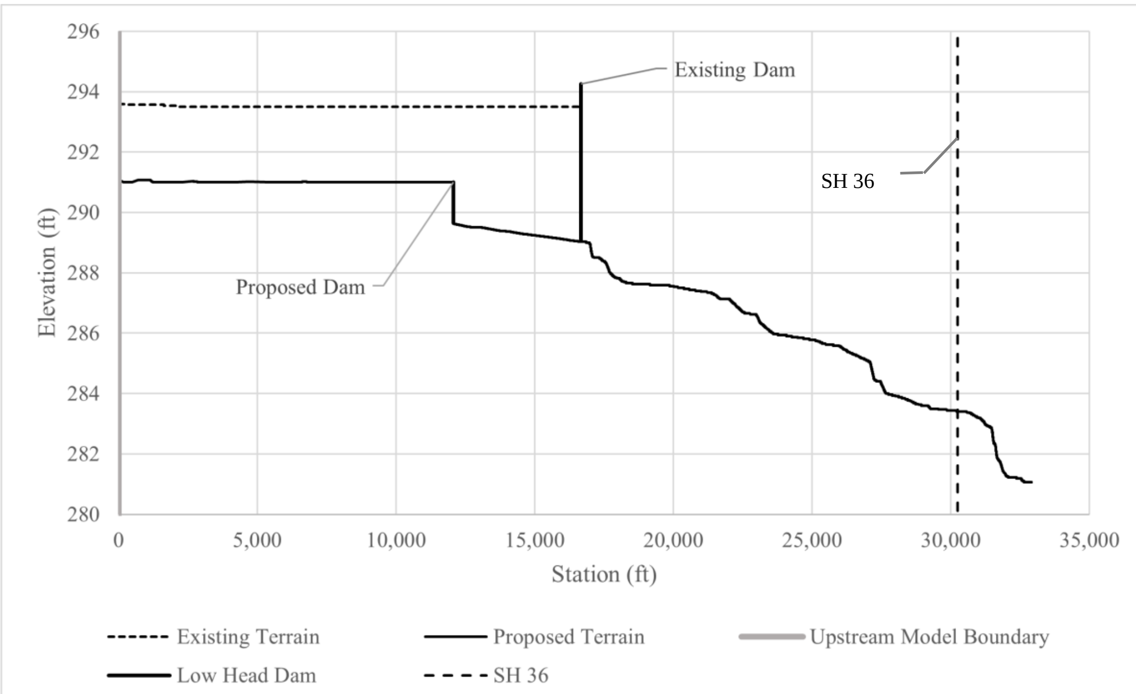

Figure 5-2. Proposed condition channel terrain profile

5.1.3 Proposed Condition HEC-RAS geometry

This section describes the changes to the hydraulic model geometry for the proposed low-head dam and existing low-head dam.

Proposed low-head dam

The proposed low-head dam was incorporated into the HEC-RAS geometry through the addition of a SA/2D connection. The SA/2D connection follows the terrain data along channel banks, then represents the proposed concrete structure with a flat weir crest elevation of 291 ft. The weir portion of the connection is approximately 100 ft long, and the weir is 1.5 ft wide. Like the existing condition, the proposed intake structure was incorporated into the model through a terrain modification, and a single breakline was applied so that the structure is captured by the 2D mesh.

Existing low-head dam

In the proposed condition, the existing low-head dam will be notched in a manner that allows normal stream flow conditions to pass without being impounded by the structure. The study team evaluated average daily stream flow at USGS gage 08106500 Little River near Cameron, Texas and found an average daily peak flow of 352 cfs. The study team found that a 40 ft wide rectangular notch is sufficient to pass the 352 cfs under a normal depth flow condition, where the energy grade slope within the channel matches the slope of the terrain. Figure 5-3 demonstrates this condition.

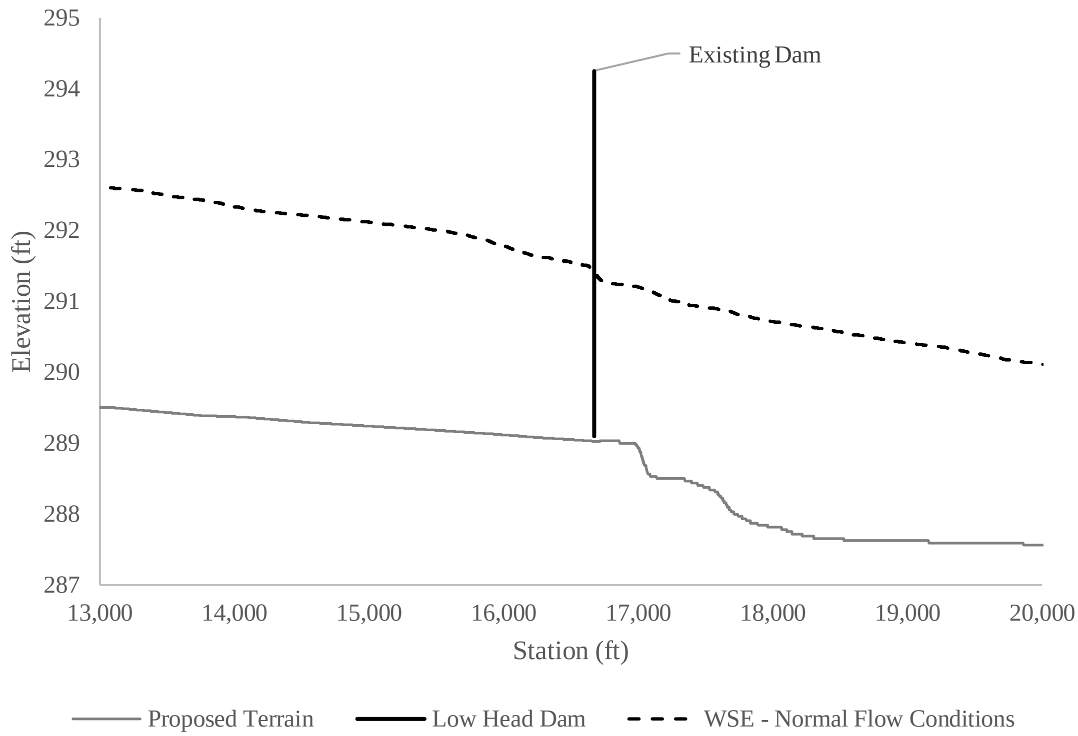

Figure 5-3. Notched existing dam – normal flow conditions

5.2 Frequency events

Following the development of the proposed condition hydraulic model, the same set of frequency events applied to the existing condition model were then simulated in the proposed condition. The model's results were recorded at the USGS stream gage on Little River near Cameron, Texas. All hydraulic results presented in the following sections are in reference to the hydrologic depth-area analysis set at USGS gage 08106500 Little Rv nr Cameron, TX (J_Cameron).

5.2.1 Results summary and comparison

Table 5-1 and **Table 5-2** summarizes the peak flow and peak elevation for each frequency event at SH 36, where USGS gage 08106500 is located. **Figure 5-4** and **Figure 5-5** compare the existing and proposed hydrographs for the 100-yr and 500-yr events at the same location. **Figure 5-6** and **Figure 5-7** show the difference in maximum water surface elevations near the project location for the 100-yr and 500-yr events, respectively. All water surface elevation comparisons downstream of what is shown in these two figures indicate a decrease in proposed conditions.

Table 5-1. Frequency event peak flow at SH 36 (USGS 08106500)

Annual Exceedance Probability	Existing Peak Discharge (cfs)	Proposed Peak Discharge (cfs)	Difference (cfs)
50%	22,029	21,921	-108
20%	46,123	46,058	-65
10%	67,164	67,143	-21
4%	91,925	92,221	296
2%	113,236	113,095	-141
1%	136,441	136,355	-86
0.2%	199,800	199,925	125

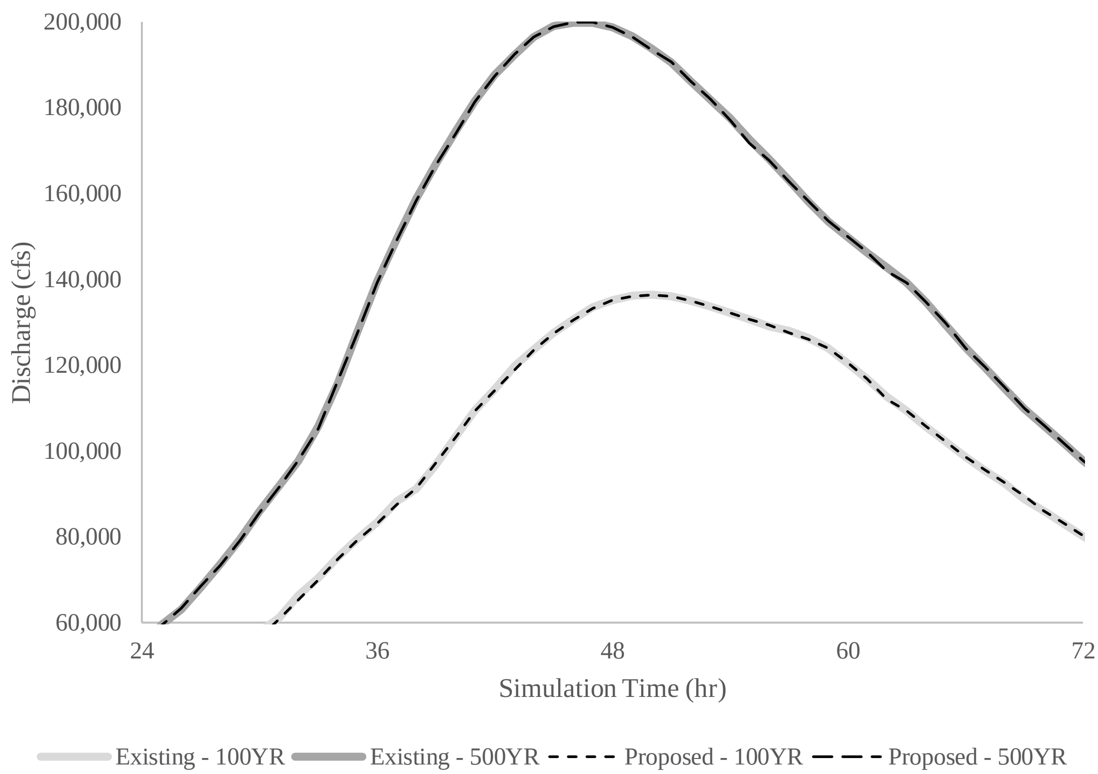

Figure 5-4. Flow hydrograph comparison at SH 36 (USGS 08106500)

Table 5-2. Frequency event peak stage at SH 36 (USGS 08106500)

Return Interval	Existing Peak Elevation (ft)	Proposed Peak Elevation (ft)	Difference (ft)
2YR	308.90	308.94	0.04
5YR	316.71	316.70	-0.01
10YR	319.05	319.05	0.00
25YR	321.41	321.44	0.03
50YR	323.11	323.16	0.05
100YR	324.77	324.82	0.05
500YR	328.74	328.77	0.03

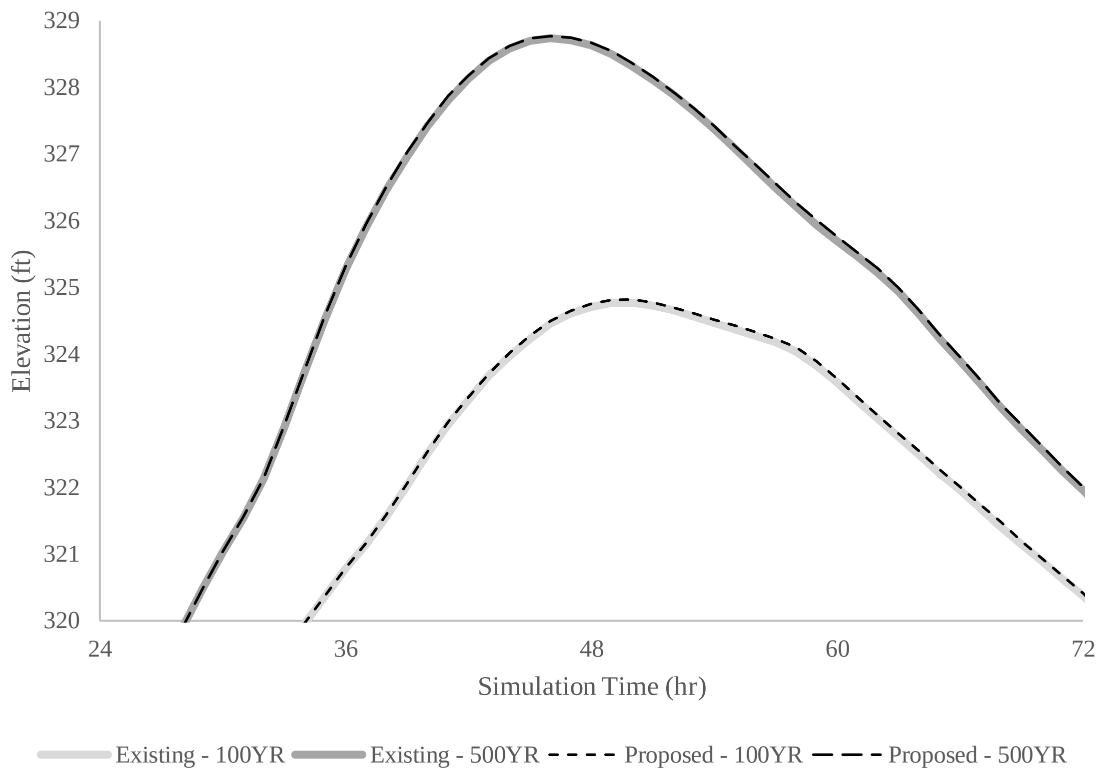

Figure 5-5. Stage hydrograph comparison at SH 36 (USGS 08106500)

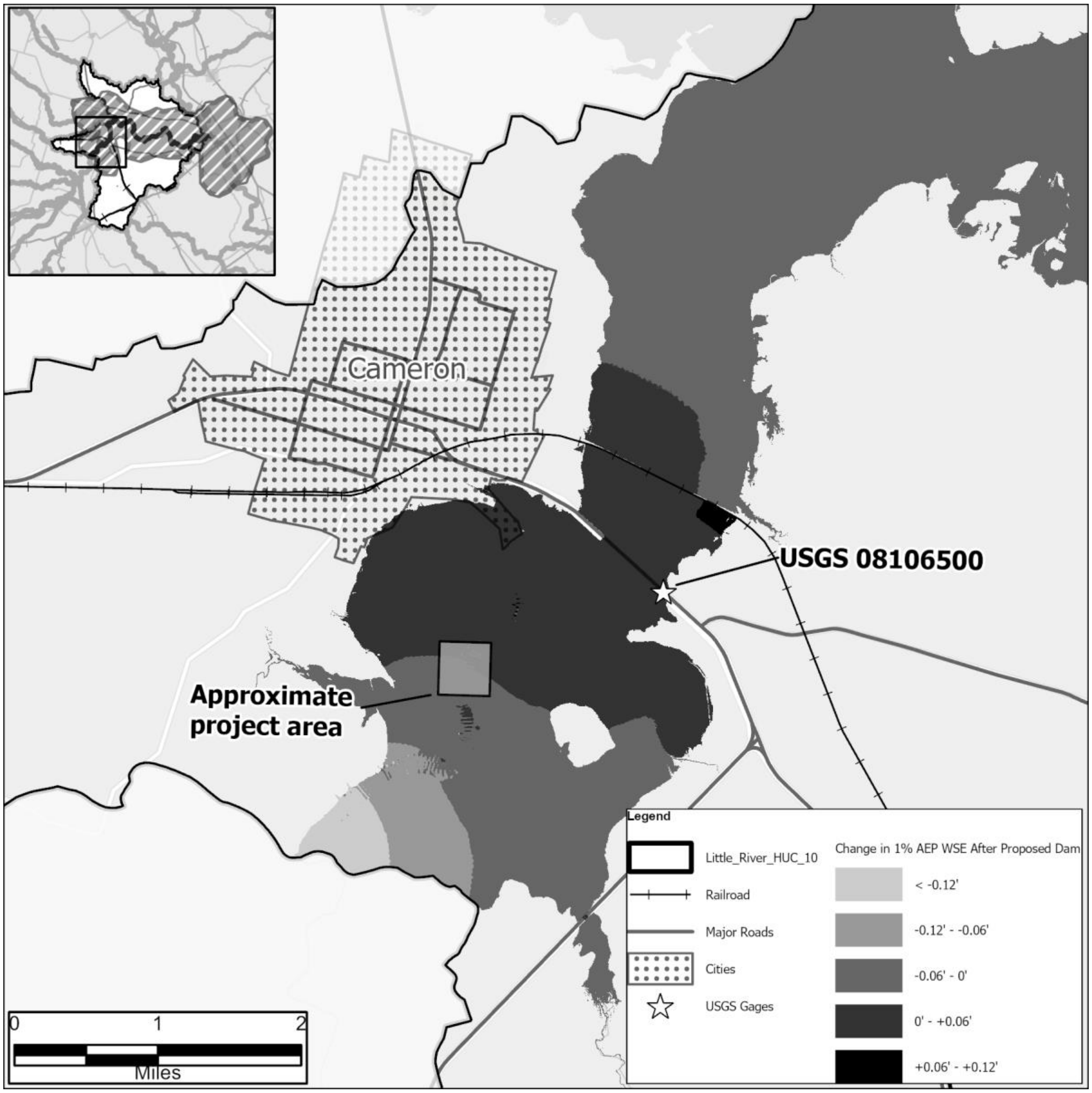

Figure 5-6. Difference in maximum water surface elevation – 1% AEP

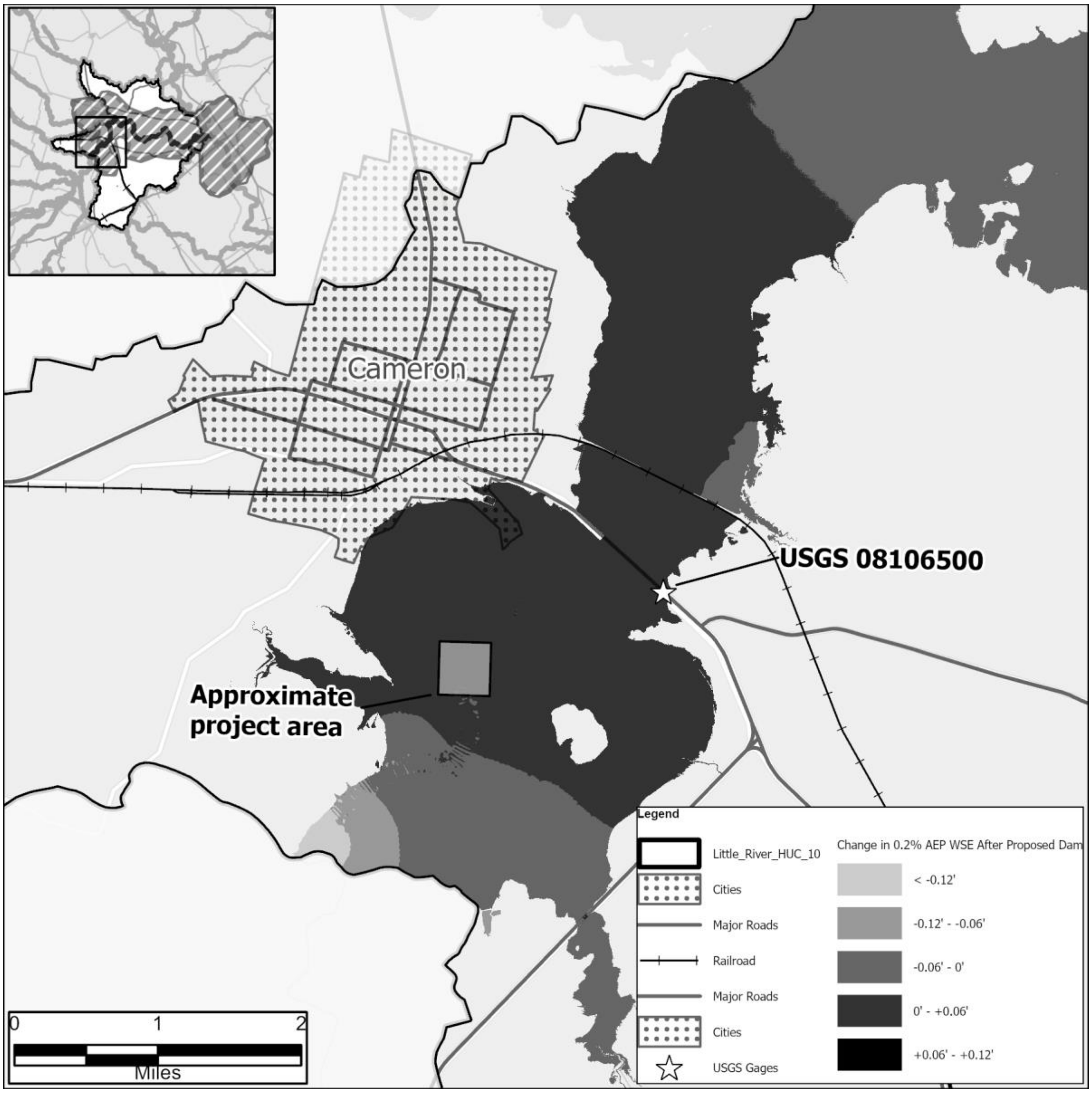

Figure 5-7. Difference in maximum water surface elevation – 0.2% AEP

5.3 Impact analysis

As presented in **Section 5.2.1,** the proposed condition for the Little River Pump Station results in slight increases in maximum water surface elevation near the project site in multiple frequency event simulations. The study team created a structural database and evaluated existing and proposed water surface elevations at each. The methodology and results of this analysis is described in this section.

5.3.1 Structural database

The study team used the publicly available Microsoft Building Footprints shapefile to create a structural database for the Lower Little River HUC10. The Microsoft data consisted of 6,244 features within the Lower Little River HUC10. The study team also conducted a desktop completeness review of the dataset near the floodplain produced by this study and determined that the Microsoft data was sufficient.

An average natural ground elevation was calculated at each structure based on the LiDAR data described in **Section 2.1.** The study team also assumed a 0.50 ft finished floor for each structure in the database. This results in the relevant elevation of Natural Ground + 0.5 ft when considering potential inundation depths.

For structures determined to reside in the floodplains produced by this study, each structure was categorized based on aerial imagery to aid in evaluating results. The applied categories are as follows:

- Residential
- Non-Residential
- Non-Existent
- Shed/Barn

Structures that were deemed ‘Non-Existent’ after preforming a visual check were removed from the dataset. Maximum water surface elevations for all frequency events in both existing and proposed conditions were also extracted in GIS software and incorporated into the structural database.

5.3.2 Results comparison

Of the approximately 6,200 structures within the Lower Little River watershed, 23 structures show greater than 0.0 ft inundation depths in at least one of the evaluated frequency events. **Table 5-3** and **Table** 5-4 summarize the existing and proposed inundation depths for structures within the floodplains produced by this study.

Table 5-3. Existing conditions – inundated structure summary

FID	Category	FFE (NAVD88)	Depth above FFE (ft) 50% AEP	20% AEP	10% AEP	4% AEP	2% AEP	1% AEP	0.2% AEP
58	Shed/Barn	324.01	0.00	0.00	0.00	0.00	0.00	0.02	3.90
154	Shed/Barn	287.04	0.00	0.00	0.00	0.00	0.00	0.00	0.03
427	Shed/Barn	307.33	0.00	0.00	0.00	0.00	0.00	0.00	1.82
450	Shed/Barn	285.10	0.00	0.00	0.00	0.00	0.00	0.00	1.13
451	Shed/Barn	284.77	0.00	0.00	0.00	0.00	0.00	0.00	1.34
763	Shed/Barn	308.76	0.00	0.35	2.31	4.14	5.49	6.81	9.89
764	Shed/Barn	308.83	0.00	0.29	2.26	4.10	5.45	6.77	9.85
899	Shed/Barn	304.40	0.00	0.00	0.00	0.00	0.00	0.00	2.25
900	Residential	303.57	0.00	0.00	0.00	0.00	0.00	0.64	3.06
1369	Shed/Barn	310.87	0.00	2.42	4.33	6.13	7.45	8.74	11.84
1372	Residential	323.58	0.00	0.00	0.00	0.00	0.00	0.00	0.25
2799	Shed/Barn	301.69	0.00	0.00	2.03	3.81	4.99	6.11	8.62
2981	Non-Residential	330.93	0.00	0.00	0.00	0.00	0.00	0.00	0.15
2982	Shed/Barn	323.20	0.00	0.00	0.00	1.17	2.52	3.94	7.66
3016	Shed/Barn	281.33	0.00	0.00	0.00	0.00	0.00	0.00	0.12
3527	Residential	308.03	0.00	0.00	2.01	3.93	5.32	6.67	9.77
4057	Shed/Barn	282.51	0.00	0.00	0.09	1.99	3.41	4.69	7.28
4662	Residential	310.52	0.00	0.00	0.00	0.04	1.43	2.75	5.70
4663	Residential	309.60	0.00	0.00	0.00	0.96	2.36	3.67	6.63
4664	Non-Residential	314.72	0.00	0.00	0.00	0.00	0.00	0.00	2.02
5131	Residential	307.07	0.00	0.00	1.17	3.30	4.73	6.09	9.15
5624	Residential	274.88	0.00	0.00	0.00	0.00	0.00	2.00	5.33
5864	Shed/Barn	310.50	0.00	0.00	1.87	3.65	4.95	6.23	9.27

Table 5-4. Proposed conditions – inundated structure summary

FID	Category	FFE (NAVD88)	Depth above FFE (ft)						
			50% AEP	20% AEP	10% AEP	4% AEP	2% AEP	1% AEP	0.2% AEP
58	Shed/Barn	324.01	0.00	0.00	0.00	0.00	0.00	0.07	3.93
154	Shed/Barn	287.04	0.00	0.00	0.00	0.00	0.00	0.00	0.03
427	Shed/Barn	307.33	0.00	0.00	0.00	0.00	0.00	0.00	1.82
450	Shed/Barn	285.10	0.00	0.00	0.00	0.00	0.00	0.00	1.13
451	Shed/Barn	284.77	0.00	0.00	0.00	0.00	0.00	0.00	1.34
763	Shed/Barn	308.76	0.00	0.34	2.31	4.14	5.49	6.81	9.89
764	Shed/Barn	308.83	0.00	0.28	2.26	4.10	5.45	6.77	9.85
899	Shed/Barn	304.40	0.00	0.00	0.00	0.00	0.00	0.00	2.25
900	Residential	303.57	0.00	0.00	0.00	0.00	0.00	0.64	3.06
1369	Shed/Barn	310.87	0.00	2.40	4.33	6.13	7.44	8.74	11.84
1372	Residential	323.58	0.00	0.00	0.00	0.00	0.00	0.00	0.25
2799	Shed/Barn	301.69	0.00	0.00	2.03	3.81	4.99	6.11	8.62
2981	Non-Residential	330.93	0.00	0.00	0.00	0.00	0.00	0.00	0.15
2982	Shed/Barn	323.20	0.00	0.00	0.00	1.10	2.49	3.93	7.67
3016	Shed/Barn	281.33	0.00	0.00	0.00	0.00	0.00	0.00	0.12
3527	Residential	308.03	0.00	0.00	2.00	3.93	5.32	6.67	9.77
4057	Shed/Barn	282.51	0.00	0.00	0.08	1.99	3.40	4.69	7.28
4662	Residential	310.52	0.00	0.00	0.00	0.03	1.43	2.75	5.70
4663	Residential	309.60	0.00	0.00	0.00	0.96	2.35	3.67	6.63
4664	Non-Residential	314.72	0.00	0.00	0.00	0.00	0.00	0.00	2.02
5131	Residential	307.07	0.00	0.00	1.17	3.30	4.73	6.09	9.15
5624	Residential	274.88	0.00	0.00	0.00	0.00	0.00	2.00	5.33
5864	Shed/Barn	310.50	0.00	0.00	1.87	3.65	4.95	6.23	9.27

Table 5-3 and **Table 5-4** include the same set of 23 structures between existing and proposed conditions. This demonstrates that the proposed condition and associated differences in maximum water surface elevation discussed in **Section 5.1.1** and **5.2.1** respectively, do not add any new structures into the floodplain. Of the inundated structures presented in the above tables, several show increases in maximum water surface elevation when comparing existing and proposed conditions. **Table 5-5** summarizes these differences, where positive values indicate an increase in maximum water surface elevation in proposed conditions. Of the two adversely impacted structures, none are considered habitable structures. Both structures (FID 58 & 2982) are barns or sheds.

Table 5-5. Existing vs. proposed maximum WSEL difference

FID	Category	Δ Depth (ft)						
		50% AEP	20% AEP	10% AEP	4% AEP	2% AEP	1% AEP	0.2% AEP
58	Shed/Barn	0.00	0.00	0.00	0.00	0.00	0.05	0.04
2982	Shed/Barn	0.00	0.00	0.00	-0.06	-0.03	-0.01	0.01

5.3.3 Local ordinance compliance

As part of this study, the FNI team investigated local ordinances and regulations pertaining to floodplain management. As a result of this investigation, the team concluded that the predicted water surface elevation rises seen in **Figure 5-6** and **Figure 5-7** are allowed by FEMA regulations and local floodplain ordinances. Since there are no defined Special Flood Hazard Areas for this portion of the Little River, the acceptability of this study's results in relation to no adverse impact will ultimately be determined by the Floodplain Administrator of the area in question. The governing documents for floodplain development in the City of Cameron are the Milam County Development Permit Application and the Milam County Flood Damage Prevention Ordinance. In both documents, there are no water surface elevation rise regulations for areas outside of regulatory floodways. In the absence of more restrictive local ordinances, FEMA regulations do allow small, localized increases in the 1% AEP WSE. Representatives from the City of Cameron Code Enforcement team were contacted to confirm this assertion and they confirmed that these minor increases are allowed by local ordinances and that this would not preclude a floodplain development permit.

6 Sediment management

This section describes the methods used to calculate existing sediment transport behavior in the study area and how the City of Cameron's proposed water intake infrastructure influence on sediment in the Little River.

6.1 Model development

This section describes the development of the existing and proposed sediment transport software models.

6.1.1 Sediment management summary

FNI created a two-dimensional sediment transport model to compare the following before and after the proposed low head dam is constructed.

- Hydraulic variables that transport sediment (Shear stress and velocity)
- Sediment transport and deposition (Transport capacity and river bed elevation change)

The comparison was completed to provide the City of Cameron's drinking water utility information about the potential of sediment depositing upstream of the low head dam in the vicinity of the new drinking water intake pump house. Sediment modeling results showed an averaged reduction of shear stress, velocity and sediment transport capacity by 23%, 44% and 75% respectively upstream of the low head dam. The findings suggest the low head dam modifies the Little River's hydraulics resulting in sediment deposition upstream of the dam in the region of the intake structure. FNI recommends that the City plan for sediment accumulation around its intake structure and consider modifications of the intake system or sediment removal to avoid interruptions of water intake service.

6.1.2 Model background and purpose

A two-dimensional hydraulic model (2D model) was built to predict water surface elevations for several flooding events; the smallest and most frequent was the 2-year return interval event and the largest and most infrequent was the 500-year return interval flood (as shown in **Table 3-2**). Its hydrology and hydraulics were calibrated as discussed in **Sections 3** and **4.** This model was built to calculate these water surface elevations for 19 miles of the Little River including the region surrounding the proposed water intake facility. Since the distance between the low head dam and the proposed water intake structure is several hundred feet, the 2D model was truncated with its boundaries starting 800 ft upstream and ending 500 ft downstream of the proposed low head dam. The truncated 2D model is referred to the sediment model.

The sediment model's purpose was to compare the impacts of the low head dam on the hydraulic conditions (velocity, shear stress). If the impact is negligible then sediment will continue to be transported similarly to pre-dam conditions and no mitigation for or removal of sediment should be expected. If the impact is notable, then sediment is likely to deposit around the intake structure and the sediment model will calculate the rise in river bed elevation. The City will then decide to either mitigate sediment deposition or plan on removing sediment buildup.

The sediment model was run under existing conditions and then proposed conditions with the structures detailed in **Section 5.1.1** to complete the comparison. The sediment model was run for both physical conditions using discharges that were thought to initiate and sustain sediment transport and occur several times a year. Since the studied discharges occur multiple times during a year, the study's results have a reasonable level of confidence there will be either no change or some change to the river bed elevation year on yearly basis.

6.1.3 Terrain development

This section describes the creation of the existing and proposed terrains for the sediment transport models.

Existing terrain
As described in **Section 2.1,** high resolution 3DEP LiDAR data was used to build the 2D model and the sediment model for the study regions. Finer resolution survey data and bathymetric data was available for the region around the proposed low head dam and drinking water intake structure. The extent of this data was approximately 400 ft in length.

The terrain from the 2D model was longitudinally truncated from approximately 19 miles to a half mile. The downstream limit of the terrain was approximately 500 ft downstream of the low head dam and approximately 1,800 ft upstream of the dam. The proposed drinking water intake is roughly 500 ft upstream of the dam.

The terrain was then modified using the finer resolution survey data and bathymetric data which was collected in June 2021. This data was collected using a rod with either GPS survey grade or total station methods. A total of six cross sections were measured and were evenly spaced apart. available for the model area. Breaklines were created along notable breaks in slope and a terrain constructed with the June 2021 data. The June 2021 terrain replaced the LiDAR data using HEC-RAS's embedded RAS Mapper tool.

Proposed terrain
The existing terrain was modified for proposed conditions by adding in the proposed structures as described in **Section 5.1.1** as modifications to the terrain. The low head dam crest was placed

at an elevation of 291 ft (NAVD 88) and its average height was approximately 4 ft above the adjacent river bed elevation. The water intake tower is approximately 15 ft long and 10 ft wide and the terrain was raised to 325 ft. The terrain modification for the water intake tower spanned two mesh cells.

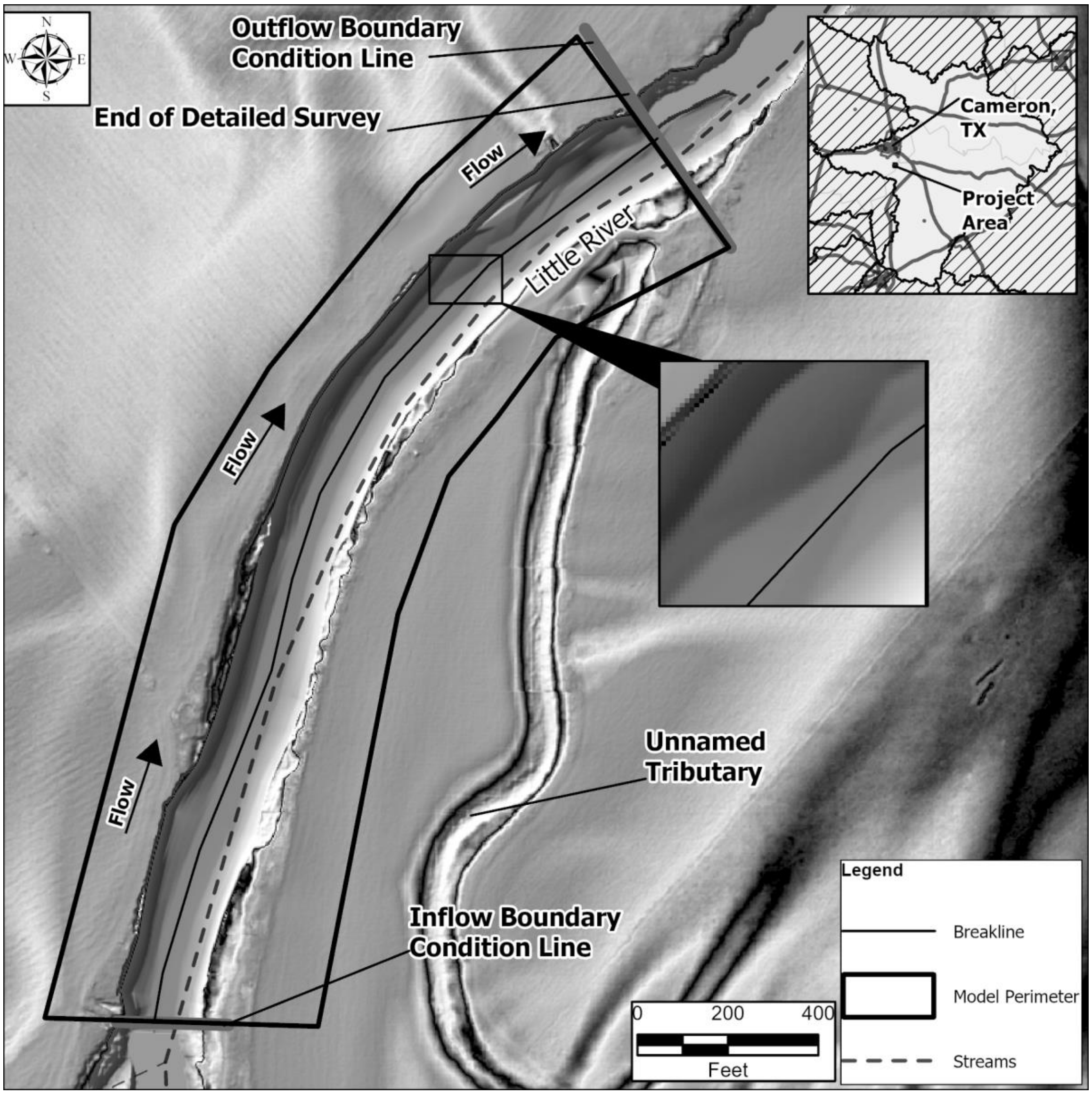

Figure 6-1. **Existing sediment transfer model overview**

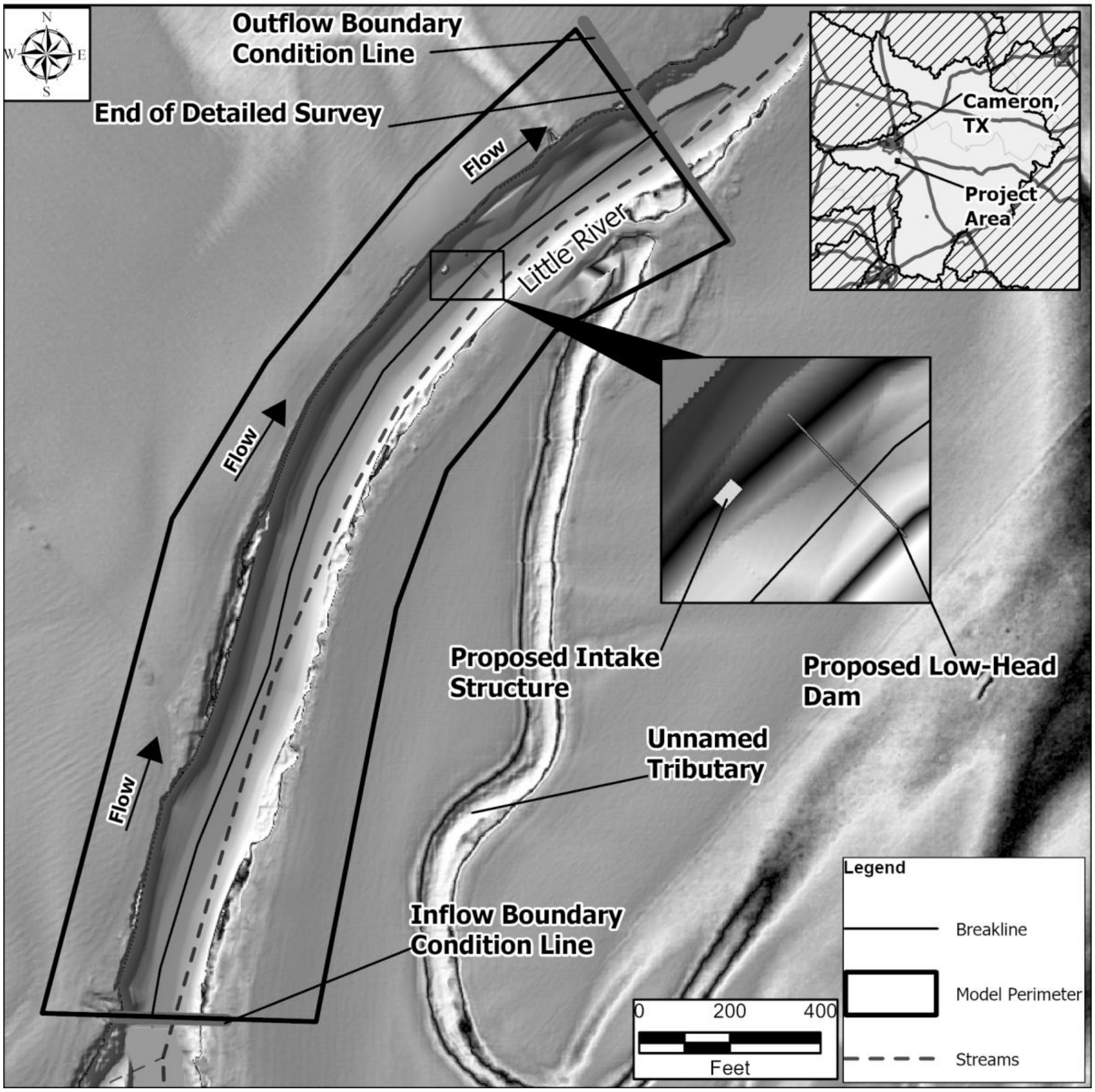

Figure 6-2. Proposed sediment transfer model overview

6.1.4 Model mesh

The model mesh established the spatial resolution for hydraulic and sediment transport calculations. A coarser mesh means a single value will be calculated for a wider area and result in faster processing speeds. A mesh size of 25 ft by 25 ft was chosen for this model as it provided the most detailed data without causing the model to become unstable. The channel bottom width in the sediment model is on average 125 ft wide and the channel top width is on average 250 ft wide. The mesh has 14,250 number of cells. To create a uniform mesh along the channel a singular breakline was drawn in along the channel centerline.

Refinement regions were drawn in to accurately represent Manning's N values in the model using 0.03 for channel bottoms, 0.04 for channel banks, and 0.06 for the floodplains outside the banks. A depiction of the existing conditions model can be found in **Figure 6-1.** To model proposed conditions a copy of this geometry was linked to the proposed terrain described in **Section 6.1.3.** A depiction of the proposed conditions model can be found in **Figure 6-2.**

6.1.5 Model hydrology

The studied flood events in the 2D model were large relative infrequent flooding events with the smallest and most frequently occurring studied flood event being the 2-year flood event. It is widely accepted in the scientific community of geomorphology, the study of the interaction between rivers and landforms that the discharge that statistically transports the most sediment over time occurs more frequently than 2-years. The discharge which transports the most sediment over time is commonly referred to as the channel forming discharge. Wolman and Leopold (1957 (Dodov & Foufoula-Georgiou (2005), Petit & Pauquet (1998) and many other scientists and practitioners agree the discharge has a return interval between 0.5 years to 1.5 years. Therefore, none of the studied discharges in the 2D Model could be used for the Sediment Model.

The National Resource Conservation Service (NRCS) 2007, the US Army Corps of Engineers (Copeland et alt. 2000) and common industry practice use physical indictors in alluvial rivers similar to the Little River to find the water surface elevation that occurs during the channel forming discharge. Once this elevation is found, the discharge can be calculated using software hydraulic models or empirical equations such as the manning's equation.

In March 2022, a field visit was completed to find physical indicators of the channel forming discharge. NRCS 2007 recommends looking for physical indicators at point bars which are features located on the inside of meanders where sediment has recently deposited. A canoe was used during the field visit and two depositional features, approximately 400 ft downstream of the proposed low head dam and 3,900 ft upstream of the proposed dam were found. Each depositional bar had physical indicators of the channel forming discharge.

At the most upstream indicator, the ground elevation, a cross section across the channel and water surface elevation was measured using a survey grade GPS unit. Due to vegetation and steep banks, the same measurements could not be replicated at the downstream indicator. Indicators are seen in **Figure 6-3** and **Figure 6-4** as well as the measured cross section of the most upstream cross section in **Figure 6-5.** A discharge of 1,521 cubic feet per second (cfs) was calculated using a manning's n value of 0.033 (from field observations) and slope of 0.0001 (average daily water surface profile).

The hydrograph of the smallest most frequent studied flood event, the 2-year return interval flood (whose peak discharge is seen in **Table 3-2,** 18,195 cfs) was extracted from the 2D model. The hydrograph was then normalized to match the peak discharge of 1,521 cfs and is shown in **Figure 6-6.** A discharge of 1,521 cfs does not plot in the Schedule 17B analysis of peak discharges as shown in **Figure 3-3.** A flow duration curve was calculated and plotted for 10 years of average daily discharge data in **Figure 6-7.** The discharge of 1,521 cfs is exceeded approximately 27% of the time during the period of record. This means the channel forming flow occurs frequently, likely multiple times a year which will be helpful in predicted sediment transport behavior over the course of the year.

Figure 6-3. Physical indicator at most upstream deposition feature at the point of the rightward most leg of the tripod.

Note, picture taken looking upstream and an inset levee is seen to the left of the tripod, which is a reliable indicator the channel forming elevation is nearby.

Figure 6-4. **Physical indicator at the most downstream deposition feature at the point of the leftward most leg of the tripod.**

Note, picture taken looking downstream and an inset levee is seen behind the tripod, which is a reliable indicator the channel forming elevation is nearby.

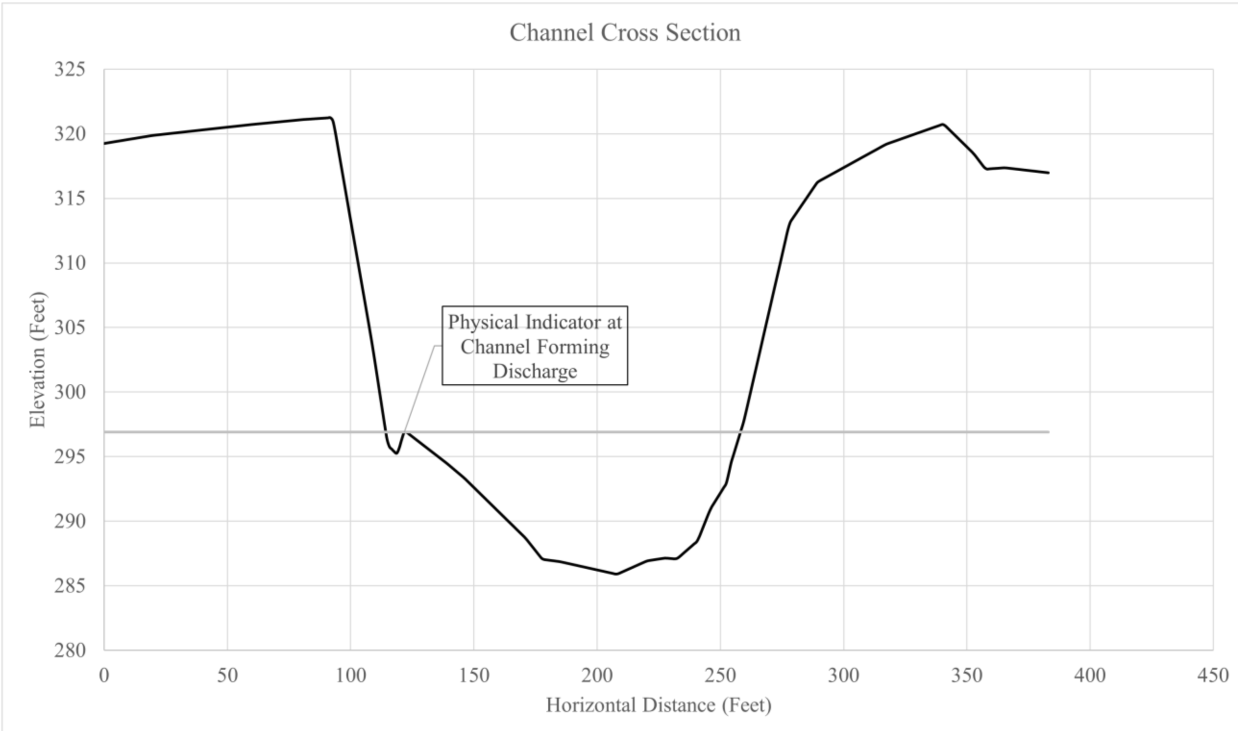

Figure 6-5. Cross section through the indicator at the channel forming discharge for the indicator seen in Figure 6-2

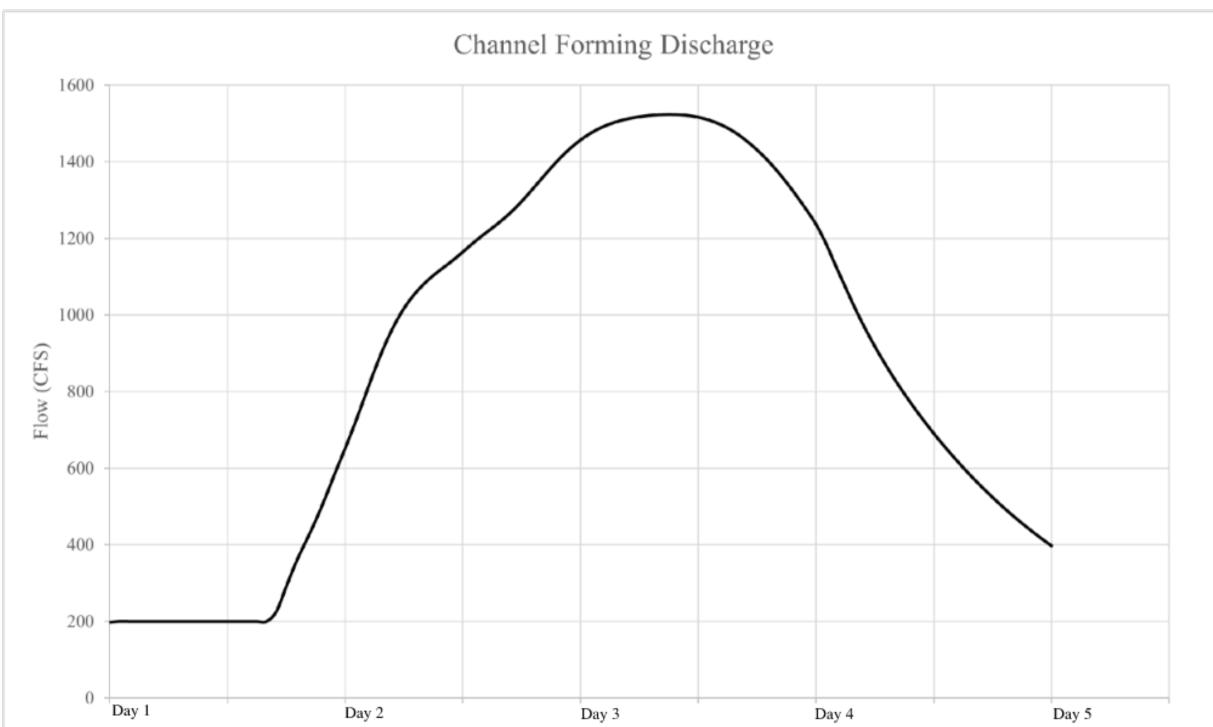

Figure 6-6. Hydrograph of channel forming discharge

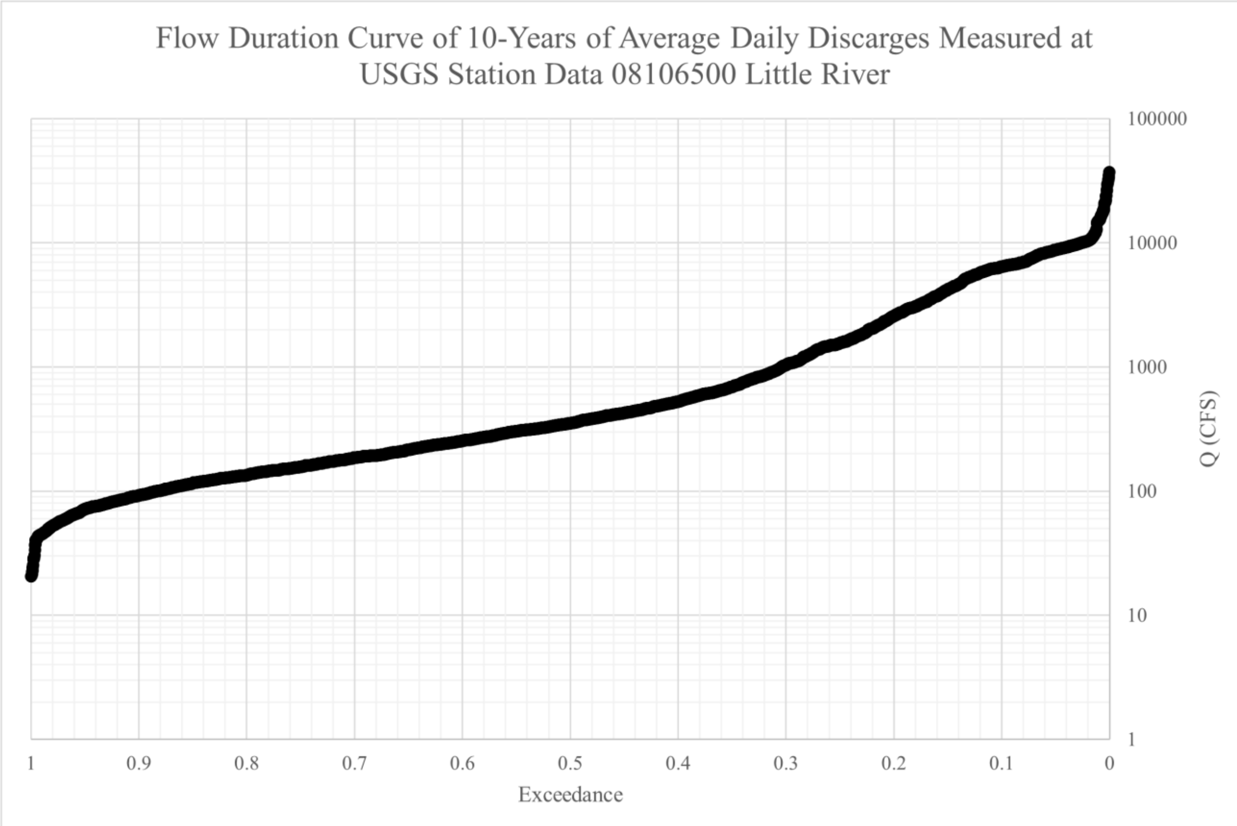

Figure 6-7. Flow duration curve at USGS Gage 08106500

The discharge of 1,521 cfs is exceeded approximately 25% of the time during the period of record

6.1.6 Sediment data

Parameters used in the Sediment Model are listed in **Table 6-1**. As described in **Section 2.3** sediment samples were attained by FNI that were representative of the conditions in the model area. A gradation curve based off of different class fractions found in the sample was used for both the bed gradation and equilibrium boundary conditions (i.e. the size distribution of transported sediment) in the model. This assumption was made because the gradation of the bed could not be measured due to water depth. This gradation curve can be found in **Table 6-2** below.

A static temperature value of 63.3 degrees Fahrenheit, the average of all temperature data available from the nearest USGS gage (08106500), was used. The Laursen-Copeland sediment transfer method was chosen due to its high performance in the very fine sand and very coarse silt range like the conditions found in the Little River. The Rubey fall velocity equation was chosen due to its applicability to sediment size ranges and specific gravities found in the Little River soil samples. A need to specify soil flocculation was identified due to the large percentage of silts and clays found in the soil samples. As the data to produce a flocculation curve was not available the Hwang (1989) floc settling velocity formula was used with coefficients gained from a study of the James River in Virginia. Coefficients from the James River were used due to a similar sediment makeup to the Little River.

Table 6-1. Sediment modeling parameters used

Data Requirement	Source and Selection
Bed Gradations	Particle size distribution of samples obtained from depositional features
Upstream Sediment Boundary Conditions	Particle size distribution of samples obtained from depositional features
Movable Bed Limits	Established between the streambank toes
Subsidence	None assumed
Bed Mixing	None
Temperature	Average temperature from USGS Stream Gage 08106500
Cohesive Properties	Selected Transport Functions were used for All Grain Sizes

Table 6-2. Bed gradation curve

Diam (mm)	Class Fraction (%)
0-0.001	7.38
0.001-0.0014	0.00
0.0014-0.0033	1.48
0.0033-0.0068	1.48
0.0068-0.0094	0.00
0.0094-0.013	1.48
0.013-0.018	1.48
0.018-0.026	1.48
0.026-0.036	1.48
0.036-0.05	1.48
0.05-0.075	5.46
0.075-0.15	5.17
0.15-0.25	7.23
0.25-0.425	5.46
0.425-0.85	2.81
0.85-2	3.84
2-4.75	9.60
4.75-9.525	17.42
9.525-19.05	17.57
19.05-25.4	7.68

6.1.7 Sensitivity analysis

A sensitivity analysis was conducted to determine the effect of changing two factors within the model: adding a consolidation curve to the soil cohesion methods and changing the cell size of the mesh in the model geometry. The existing conditions model was copied and then edited to include a basic consolidation curve. Another copy of the existing conditions model was made and the mesh size changed to 50 ft cell size instead of the 25 ft cell size mesh used in the existing conditions model. Each of these sensitivity models were compared to existing conditions using four factors: max velocity, shear stress, bed elevation, and total load capacity. Figures depicting the results of the shear stress comparisons can be found in **Figure 6-8** and **Figure 6-9**. No differences were found in the bed elevation comparison for both the consolidation curve and the 50 ft cell size models. Additional results comparisons for velocity, shear stress, and total-load concentration can be found in **Section 8.1**. Negligible differences were found between the baseline model and the model including a consolidation curve. The means the modeling results are not influenced by the consolidation curve.

The only variable tested for sensitivity which showed a notable difference between the baseline model and the sensitivity modeling run was using a larger cell size (50 ft cell). The total-load capacity results were notably different due to the coarser cell size. As such in the final model a consolidation curve was left out and the 25 ft cell size mesh of the baseline model was used.

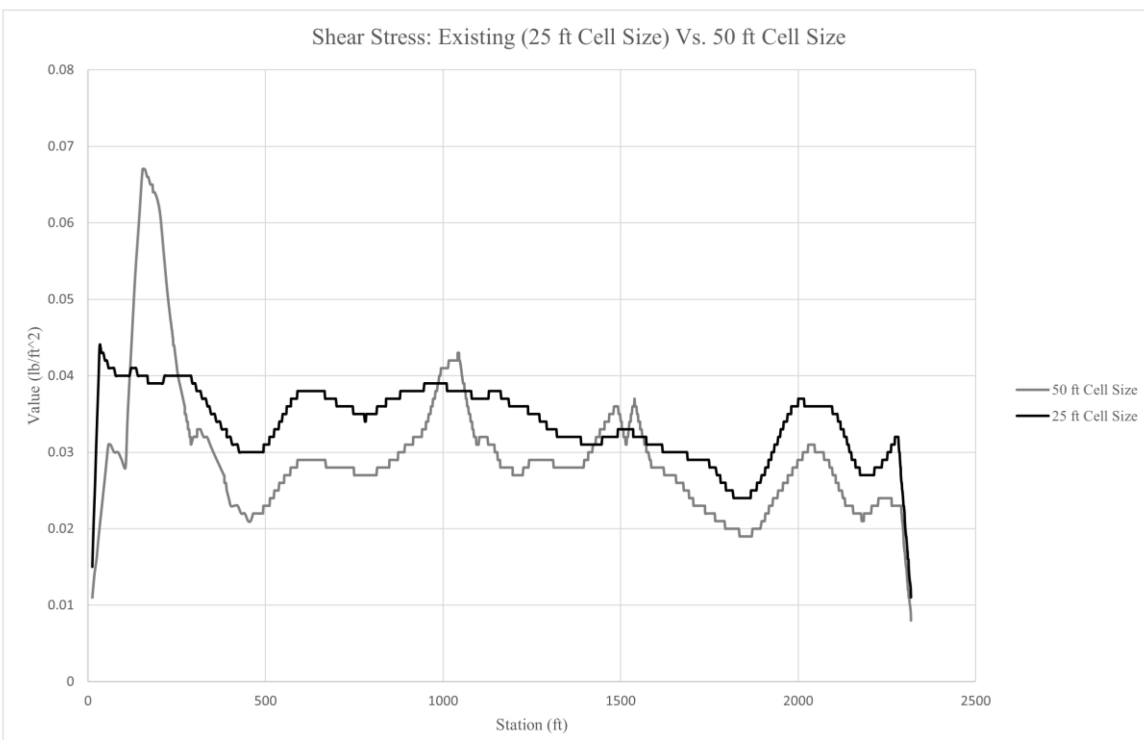

Figure 6-8. Shear stress - channel centerline profile

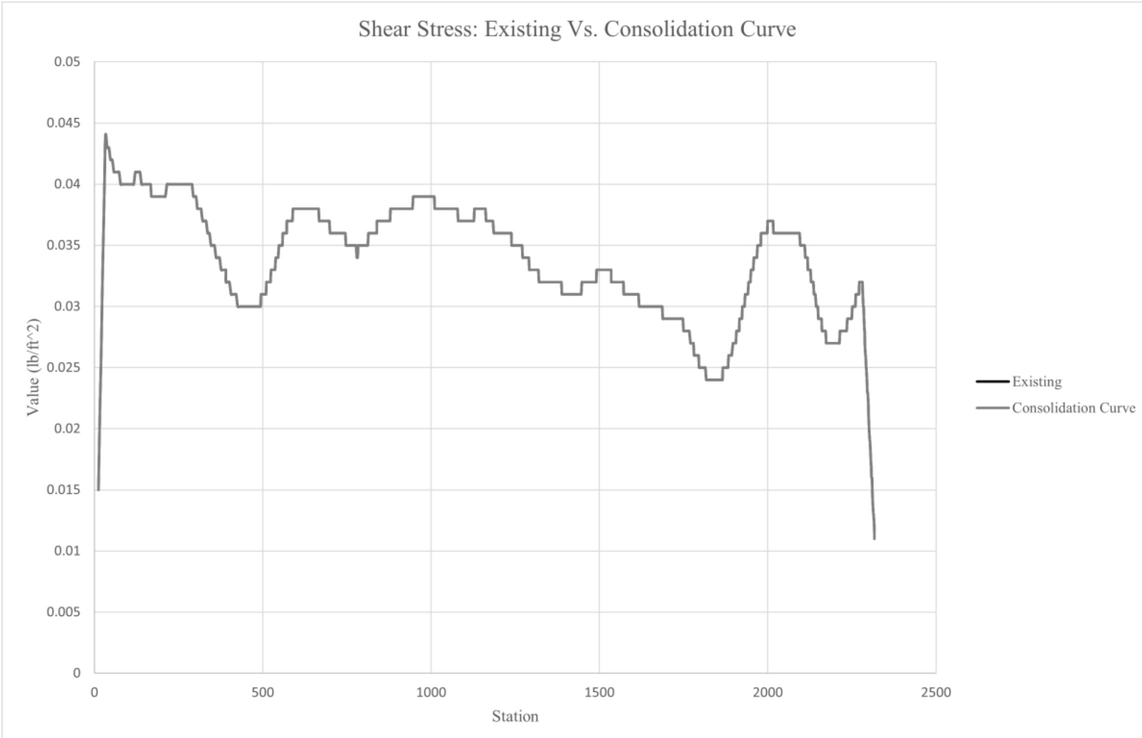

Figure 6-9. Shear stress – channel centerline profile

6.2 Results

This section describes the hydraulic and sediment transport results of the sediment transport models.

6.2.1 Hydraulic results

The low head dam reduces the three main hydraulic variables that transport sediment. Values for shear stress, velocity and sediment transport capacity were extracted from the Sediment Model results export files from RAS were extracted along the Little River's centerline. The same hydraulic variables were also extracted along a cross section through the proposed water intake tower.

Velocity

Velocity was reduced throughout the study area. Velocity is an important hydraulic variable in moving sediment down rivers and creeks. Faster moving water can carry more sediment volume and can carry larger sediment particles.

An average reduction of 0.40 feet per second (ft/sec) (23%) occurred under proposed conditions along the Littler River's centerline. The velocity along the centerline is shown graphically **Figure 6-10** and the general reduction in velocity is assumed to be caused mostly by the low head dam which spans the entire river and extends several feet above the river bed. Also notable in the figure is the preservation of the general pattern where velocity increases and decreases under both conditions. For example, velocity speeds up around station 500 and slows down around station 1,250. The exception is at the low head dam at station 1,800 where the water speeds up to go over the top of the dam. It appears the dam does not result in a pool of still water upstream of it. The average reduction in velocity was 0.26 feet/second (ft/sec) an 18% reduction at the proposed water intake tower. Existing and proposed velocities are seen in **Figure 6-11** at the tower which show slower velocity at the cross section at the tower. At station 70 the velocity goes to zero because this space is occupied by the tower. A plan-view comparison between existing and proposed velocities can be found in **Figure 8-11**.

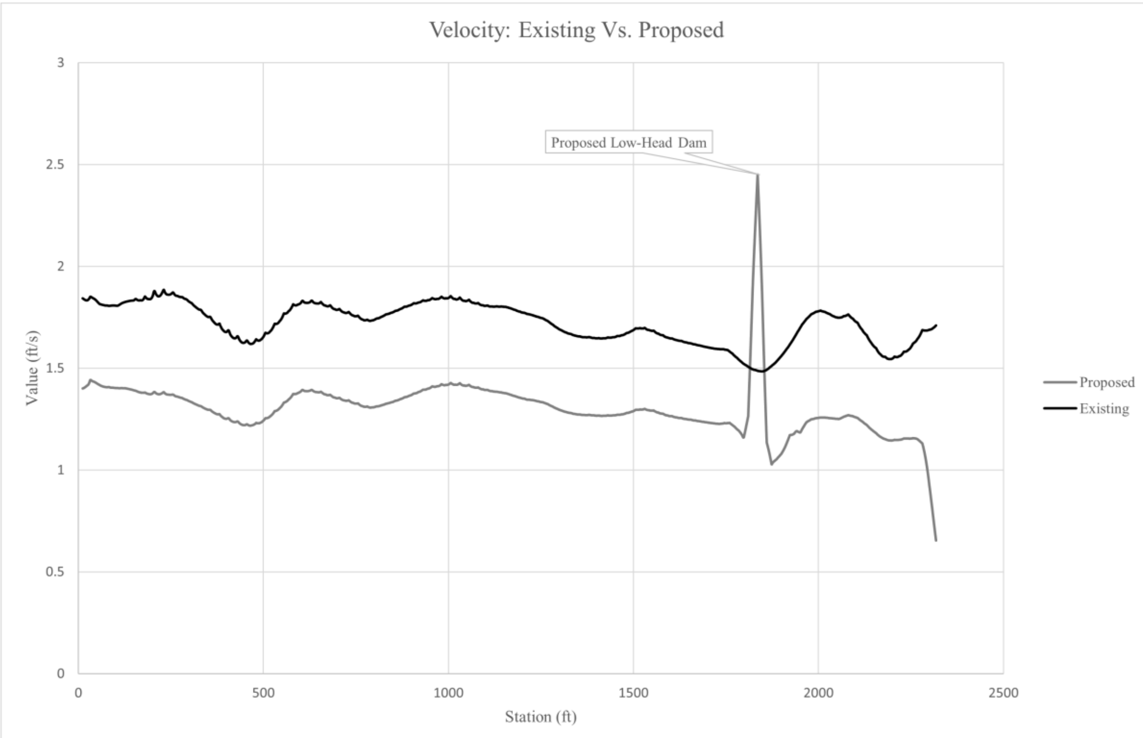

Figure 6-10. Velocity – channel centerline profile

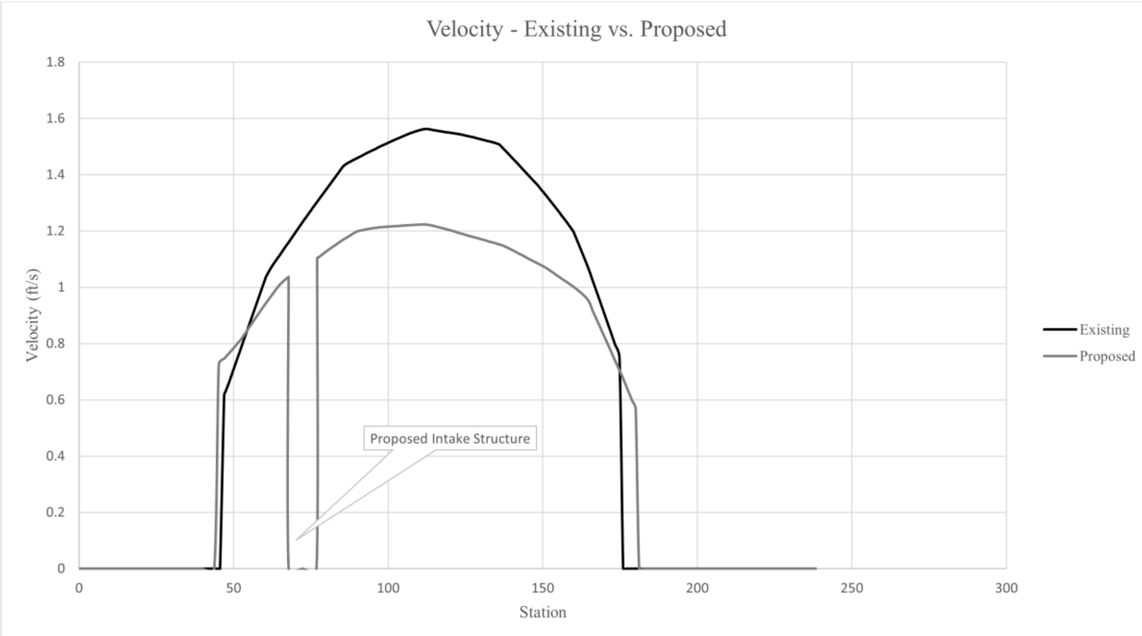

Figure 6-11. Velocity – intake structure profile

Shear stress

Shear stress is a physical force that water applies to the bottom and sides of a river which can move sediment. Higher shear stresses can move larger sediment particles. Shear stress at the tower and centerline were reduced 0.014 pounds per square foot (lbs/ft^2) and 0.015 pounds per square foot (lbs/ft^2) respectively due to the proposed conditions caused by the low head dam and water intake tower. These can be seen graphically in **Figure 6-12** and **Figure 6-13**. The reduction in shear stress follows a similar pattern to velocity in overall reduction and where it speeds up and slows down along the Little River centerline and at the tower. A plan-view comparison between existing and proposed shear-stress can be found in **Figure 8-12.**

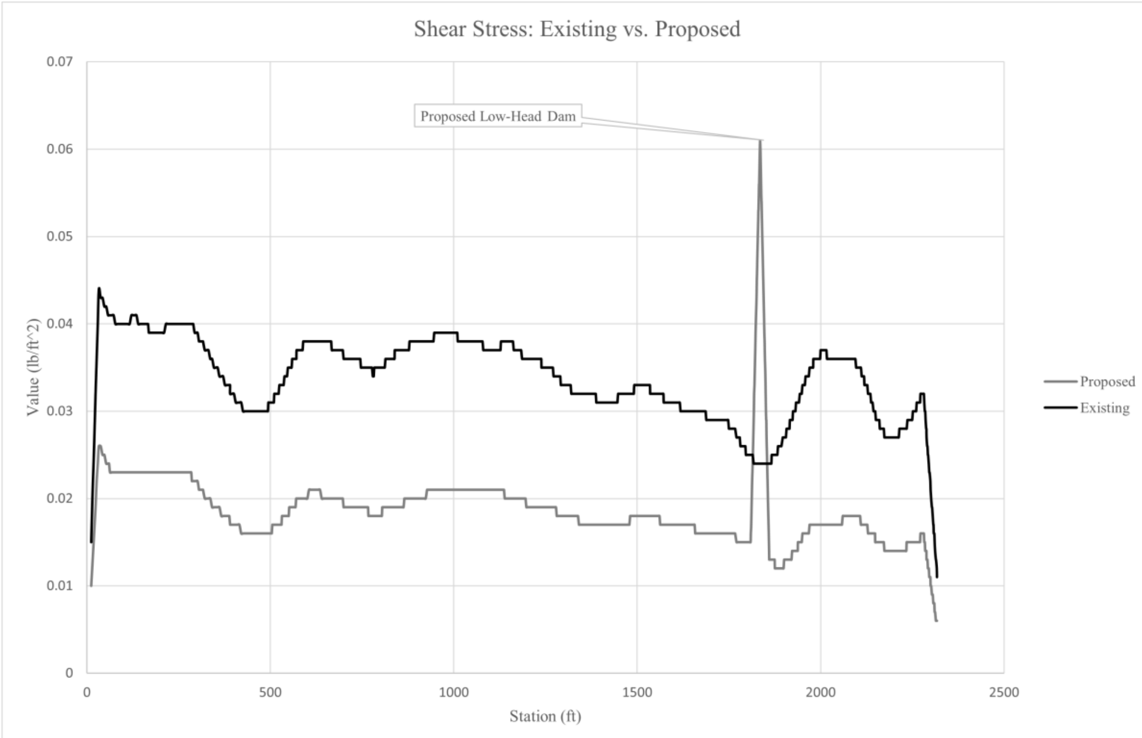

Figure 6-12. Shear stress – channel centerline profile

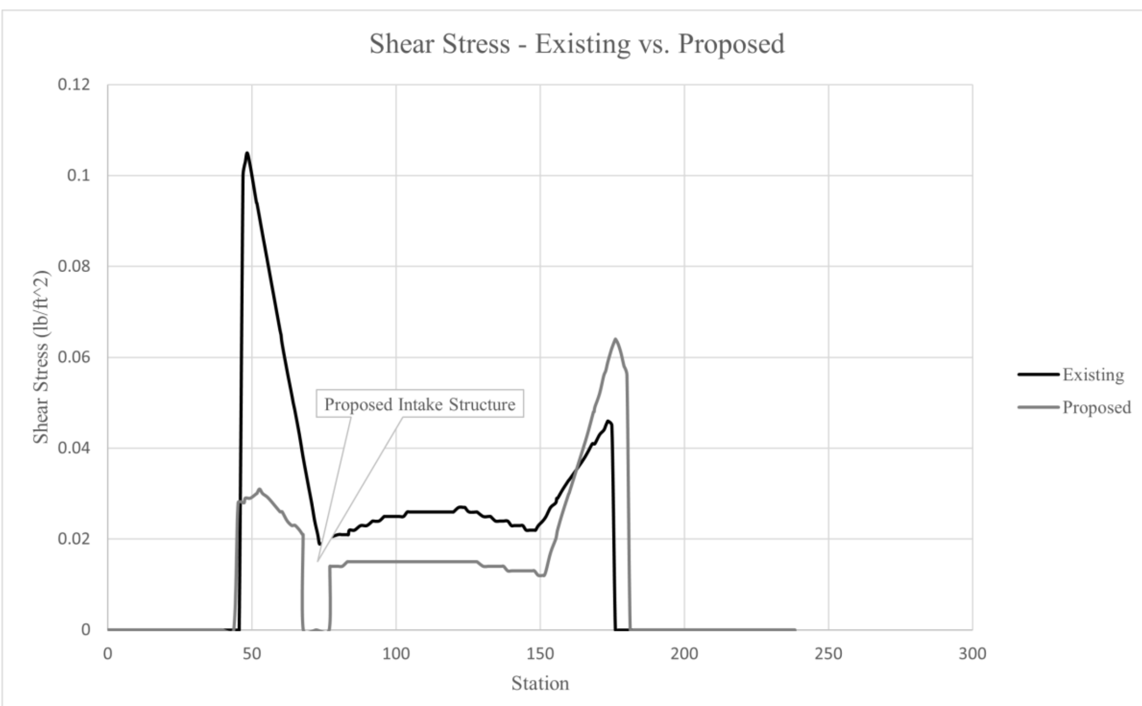

Figure 6-13. Existing vs. proposed – shear stress – intake structure profile

6.2.2 Sediment results

The Sediment Model was run to calculate the river bottom elevation (referred to as the bed elevation) under proposed conditions. If sediment deposited upstream of the dam (due to the muted hydraulic variables discussed in **Section 6.2.1)** the bed elevation would increase and be higher than the bed elevation under existing conditions. Existing and proposed bed elevations are shown in **Figure 6-14** and **Figure 6-15** along the Little River's centerline and across the water intake tower respectively. There was no measurable increase in the channel bed elevation. A second approach was used to evaluate if this finding was reasonable.

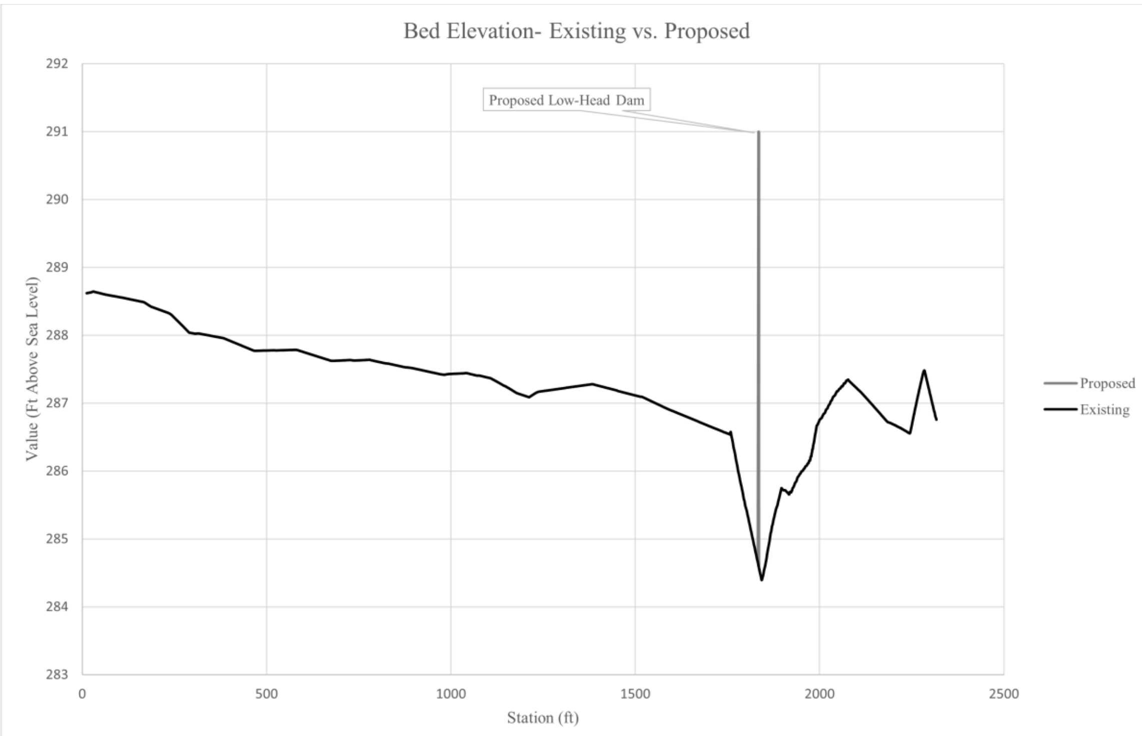

Figure 6-14. Bed elevation –centerline profile

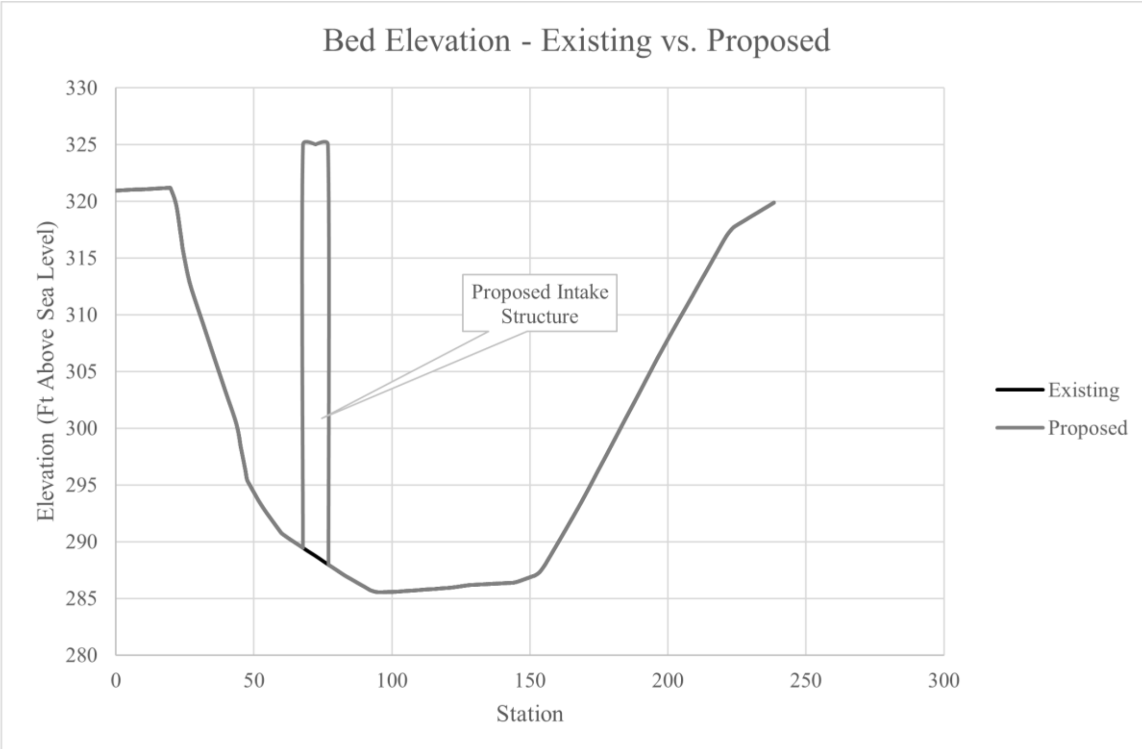

Figure 6-15. Bed elevation – intake structure profile

At discussed in **Section 6.2.1,** the Sediment Model at the water intake tower cross section calculated a drop in shear stress. The drop between the averaged shear stress in the cross section from existing conditions to proposed conditions was 0.014 lbs/ft^2 (0.036 lbs/ft^2 to 0.022 lbs/ft^2). These values were plotted on **Figure 6-16** along with a relationship between shear stress and sediment size. This linear model is referred to as the critical shear stress to initiate movement and carry a particular sediment size. In other words, when the shear stress caused by a river's moving water exceeds the critical shear stress of a particular sediment size, the sediment size is entrained and put into motion.

The diagonal line in **Figure 6-16** is the linear model between critical shear stress and sediment size provided by the US Army Corp of Engineers (Fischenich 2001). Using this relationship, under existing conditions, it is likely the Little River at the water intake tower can move up to a 0.12 inch (3 mm) very fine gravel size piece of sediment. This conclusion was made by finding the intersection of the diagonal line and the horizontal line of the shear stress under existing conditions (0.036 lbs/ft^2). Under proposed conditions, the sediment size the Little River can move at the water intake tower has been reduced to 0.075 inch (1.9 mm), a very coarse piece of sand. Four vertical lines are included in **Figure 6-16** to show the size range of silt and clays, sands, etc**.**

Using the critical shear stress model in **Figure 6-16** it would appear that a sediment particle with a median diameter between 0.075 inches and 0.12 inches should stop moving and deposit on the riverbed at the water intake tower. It is reasonable this would increase the riverbed elevation at this location. This finding conflicts with the Sediment Model finding which calculated no change in bed elevation. A further review was completed to understand the conflict.

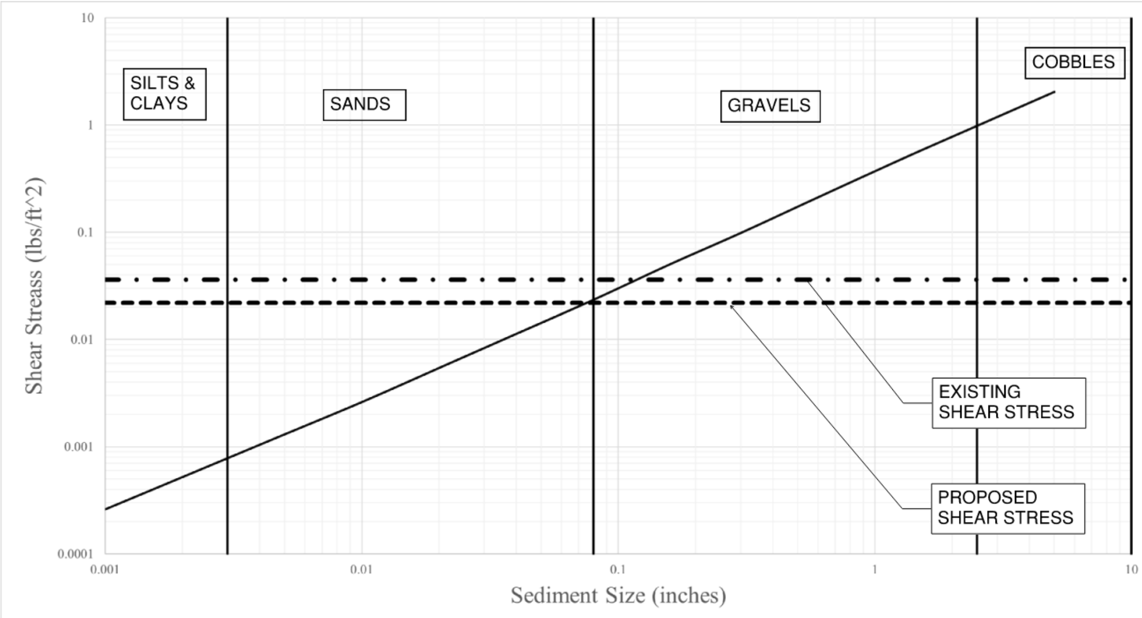

Figure 6-16. **Existing and proposed shear stresses plotted with critical shear stresses**

As discussed in **Section 6.1.5** a representative sample of transported sediment was obtained from a location along the Little River approximately 3,900 ft upstream of the proposed dam. A lab test was completed to determine its sediment size composition (**Figure 6-17)**. The largest sediment size which is moved under proposed condition and existing conditions from **Figure 6-17** were plotted on **Figure 6-16**. **Figure 6-16** shows approximately 4% of the transported sediment would stop moving under proposed conditions. This percentage is low enough to reasonably assume that no measurable change in the bed elevation would occur at the water intake tower, which supports the Sediment Model's findings. A more compelling finding in support of the Sediment Model's prediction of no bed elevation change at the water intake tower is the fact that the reduction of shear stress caused by a combination of the low head dam and water intake tower stops coarse sediments from reaching the intake tower. Without this coarse sediment, the bed elevation shouldn't rise around the intake tower.

In **Figure 6-12,** the maximum calculated shear stress along the Little River's centerline under proposed conditions was 0.025 lbs/ft^2 occurring near station 50 and quickly dropping to 0.022 lbs/ft^2 . The linear model in **Figure 6-16** shows these values will move a piece of sediment between and 0.075 inch and 0.08 inch in size along the centerline, approximately the same size that would stop moving at the intake tower as presented and discussed in **Figure 6-16**. This means the sediment size that would stop moving at the intake tower never reaches the tower during the discharge used in the Sediment Model. This finding confirms the Sediment Model results of no bed change around the intake tower.

It should be expected that bed change will occur upstream of the intake tower. Where the bed change will occur and its distance to the tower is unclear from the Sediment Model findings. It is reasonable that bed change will begin where the dam's hydraulic influence ends which appears to be at least 1,800 ft (the upstream extent of the model).

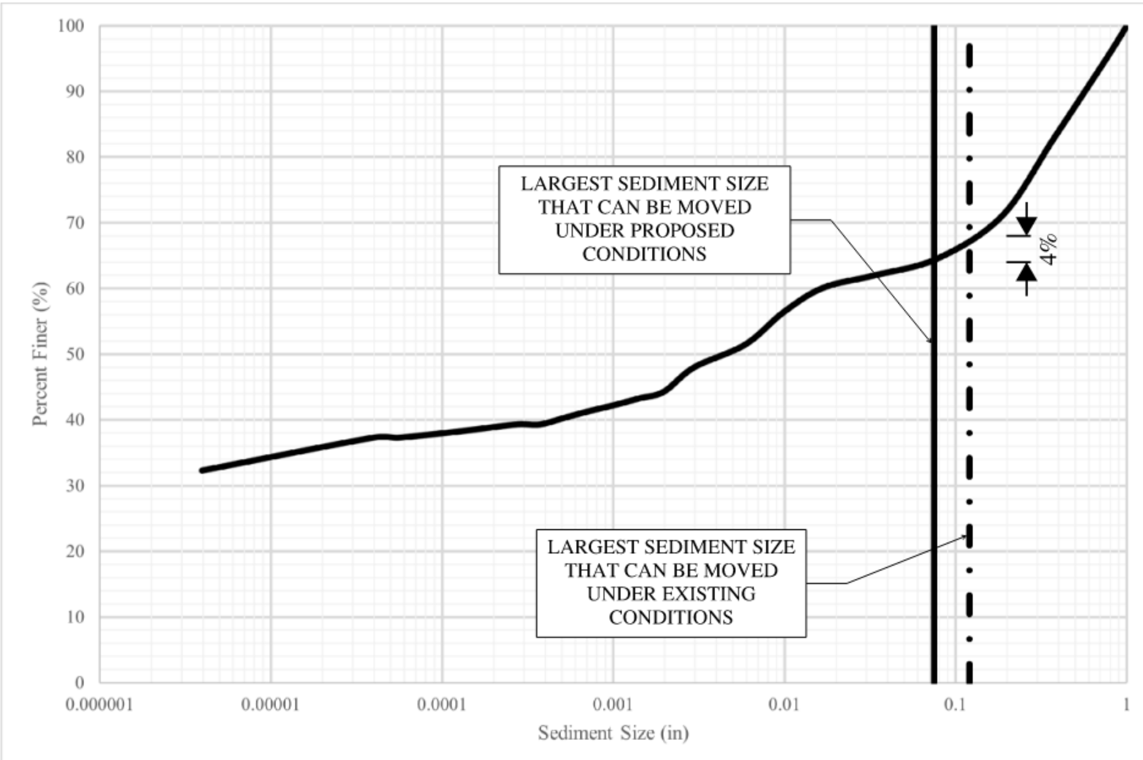

Figure 6-17. Particle size distribution of transported sediment.

7 Conclusions and recommendations

A detailed hydraulic model of existing flood conditions along 19 miles of the Little River was developed and floodplain maps were created along the Little River near Cameron, Texas. A new water intake facility is proposed to be built by the City of Cameron. The detailed hydraulic model was adapted to include this facility. An impact analysis described in **Section 5.2,** showed there were slight increases in maximum water surface elevation near the proposed water intake facility. Elevation increases range from 0.01 ft – 0.05 ft in the 500-year storm event resulting in almost unmeasurable changes in floodplain width around the SH 36 and the BNSF railroad bridges. The change in floodplain width is hard to see in **Figure 7-1** because the change is mostly limited to one pixel in the floodplain rasters. Raster pixel size is 3 feet by 3 feet. **Section 5.3** demonstrated that although there are slight increases in maximum water surface elevation in the proposed condition, the increases do not translate to increase in maximum water surface elevation at any habitable structures in the area.

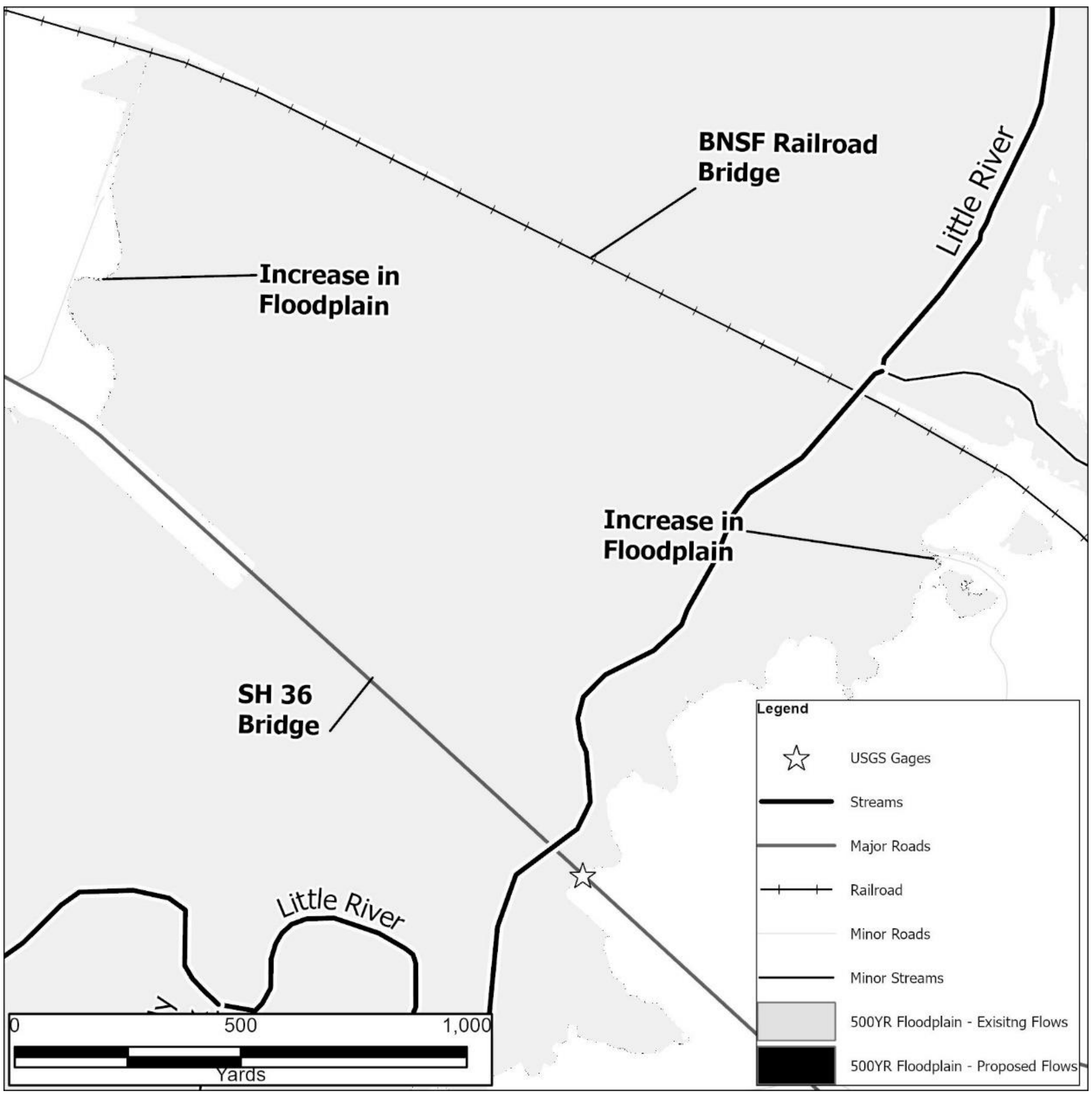

Figure 7-1. **Difference in 500 year floodplain extents**

This condition satisfies all local and federal requirements for floodplain development. Since there are no defined Special Flood Hazard Areas for this portion of the Little River, the acceptability of this study's results in relation to no adverse impact will ultimately be determined by the Floodplain Administrator of the area in question. As noted in **Section 5.3.3**, a Code Enforcement representative from the City of Cameron was contacted to inquire whether the slight water surface elevation rise shown in proposed conditions is acceptable. The Code Enforcement representative confirmed to the FNI team that the proposed conditions modeling was not in violation of any regulations. The proposed water intake structure and the low head dam do not cause any negative hydraulic impacts.

The sediment modeling work calculated a discharge that transported the most amount of sediment over time using physical indicators found during a field visit. This discharge should occur multiple times during the course of a year providing insight into how much if any sediment could build up around the water intake tower. The sediment modeling results showed no change in bed elevation around the tower. These findings were corroborated using an Excel-based linear model which also found that sediment buildup during the studied discharge shouldn't occur. The sediment modeling results did suggest that sediment deposits will happen upstream of the low head dam but should occur at least 1,800 ft upstream.

All seven tasks in the Texas Water Development Board's (TWDB) Category I grant agreement between the TWDB and the City of Cameron were completed successfully.

The hydraulic analysis and sediment modeling were performed during an early phase of the water intake design, a design at approximately thirty percent design maturity. The location of the intake structure and low head dam may change as the design is finalized. It is recommended if the location and dimensions of the proposed infrastructure changes from what has been studied, the hydraulic analysis and sediment modeling should be run again and floodwater elevations, water velocities and sediment behavior checked.

It is also recommended the City and other stakeholders use these floodplain maps to mitigate flood risk. These maps show floodwater inundation width and depth. Development within floodwater inundation should be managed to minimize damage to infrastructure and public safety risks.

8 Appendices

This section includes supplemental results from the sensitivity analysis in the sediment modeling work.

8.1 Additional sensitivity analysis results

The following figures represent results comparisons used in the Sediment Management sensitivity analysis. **Figure 8-1** through **Figure 8-4** show plan view comparisons between the existing (25 ft cell size) and existing (50 ft cell size) models and the existing and consolidation curve models for both velocity and shear stress. Positive values show where the velocity or shear stresses were higher in the existing model while negative values show where the velocity or shear stresses were higher in the existing (50 ft cell size) or consolidation curve models. **Figure 8-5** through **Figure 8-10** show channel centerline profile comparisons for velocity, bed elevation, and total load capacity for both sensitivity analyses.

Figure 8-1. Existing (25 ft cell size) vs. existing (50 ft cell size) comparison – velocity

Figure 8-2. Existing vs. consolidation curve comparison – velocity

Figure 8-3. Existing (25 ft cell size) vs. existing (50 ft cell size) comparison – shear stress

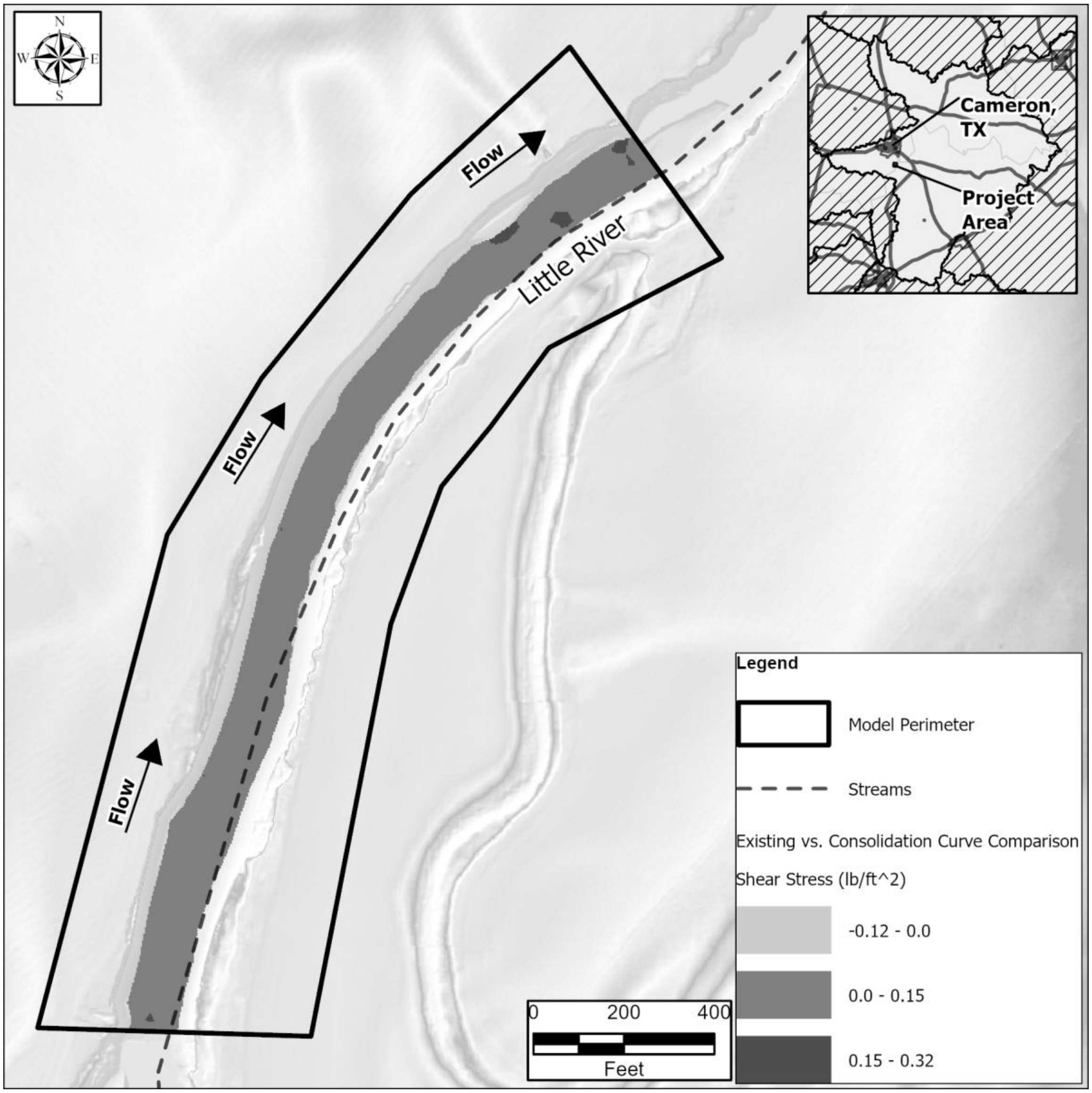

Figure 8-4. Existing vs. consolidation curve comparison – shear stress

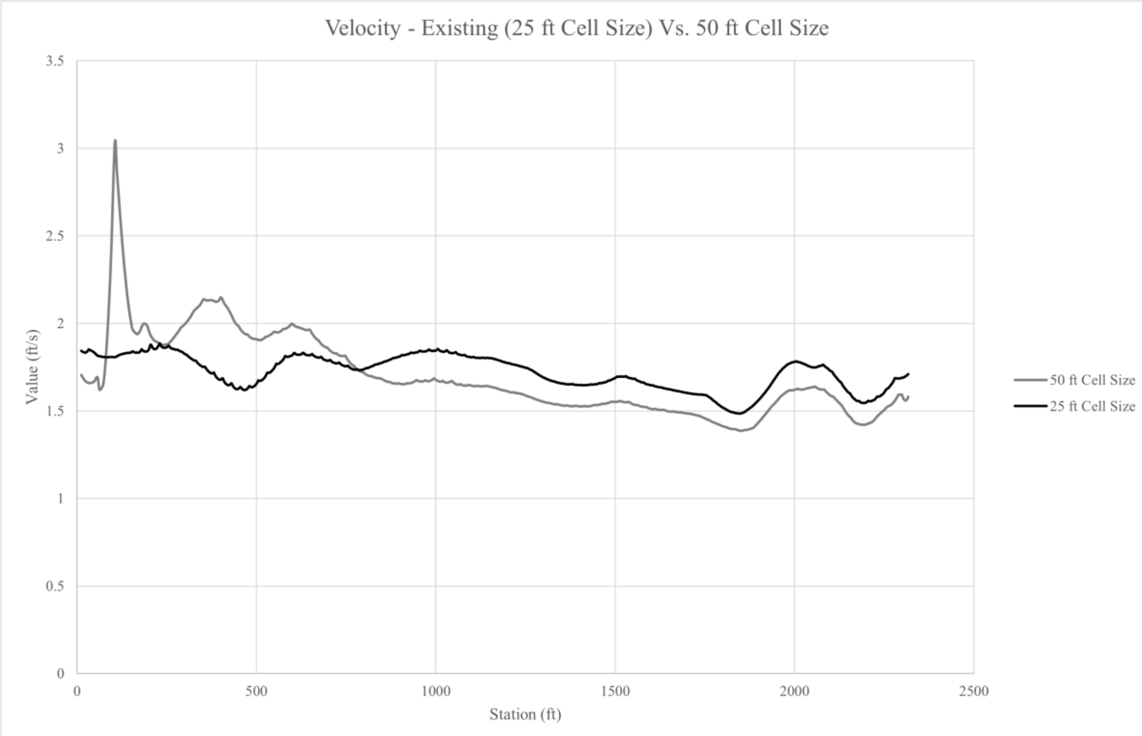

Figure 8-5. **Velocity – channel centerline profile**

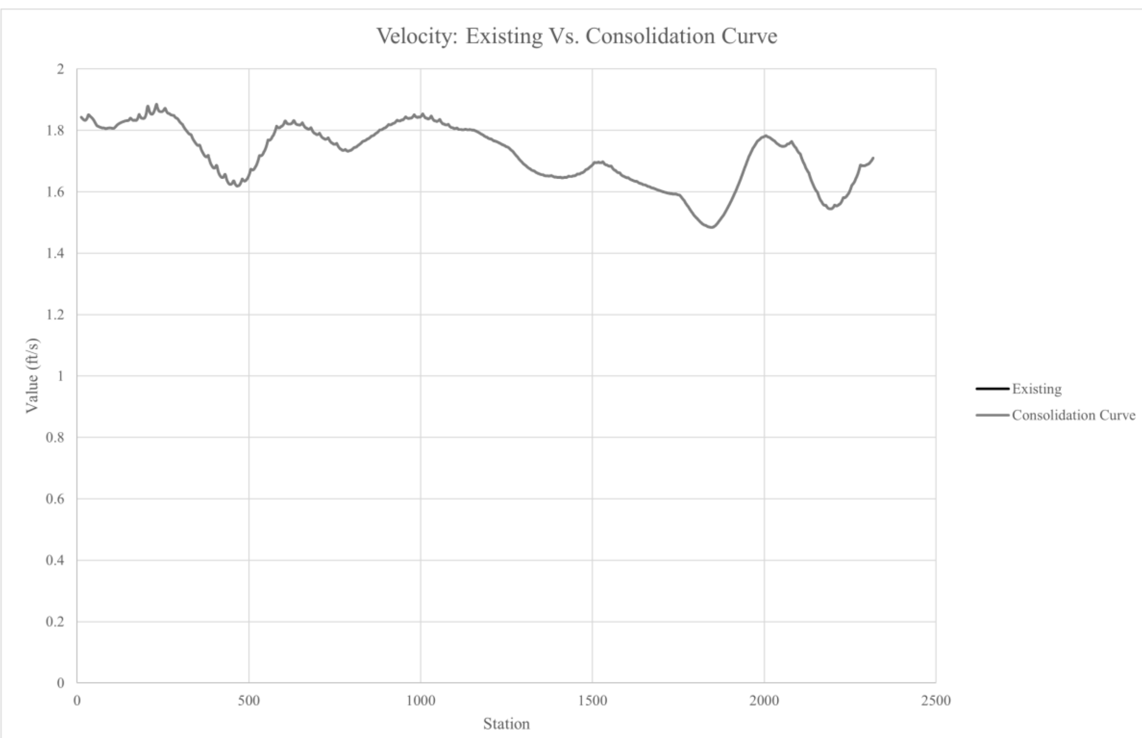

Figure 8-6. **Velocity – channel centerline profile**

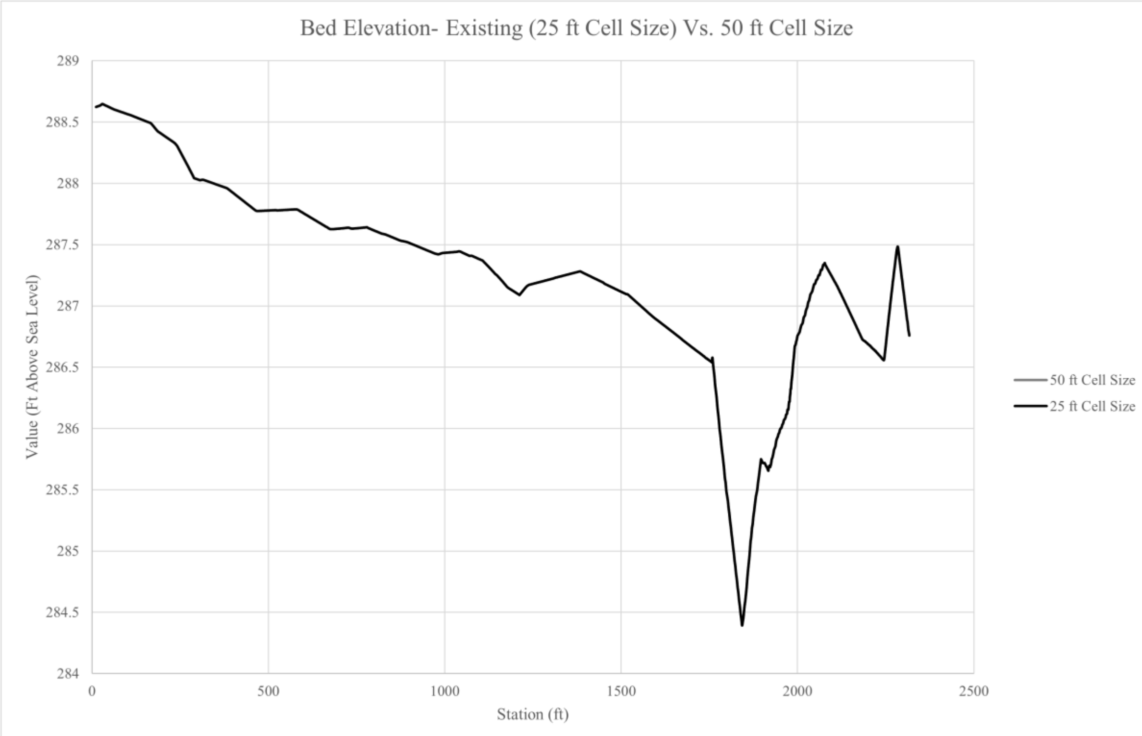

Figure 8-7. Bed elevation – channel centerline profile

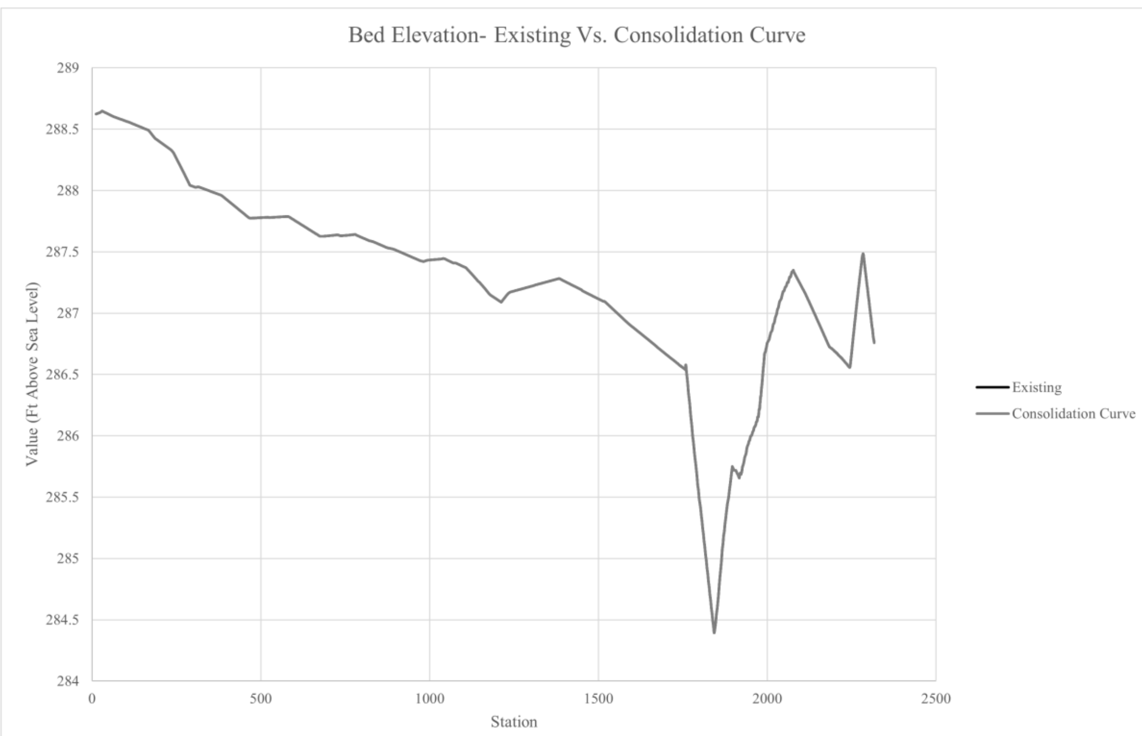

Figure 8-8. Bed Elevation – Channel Centerline Profile

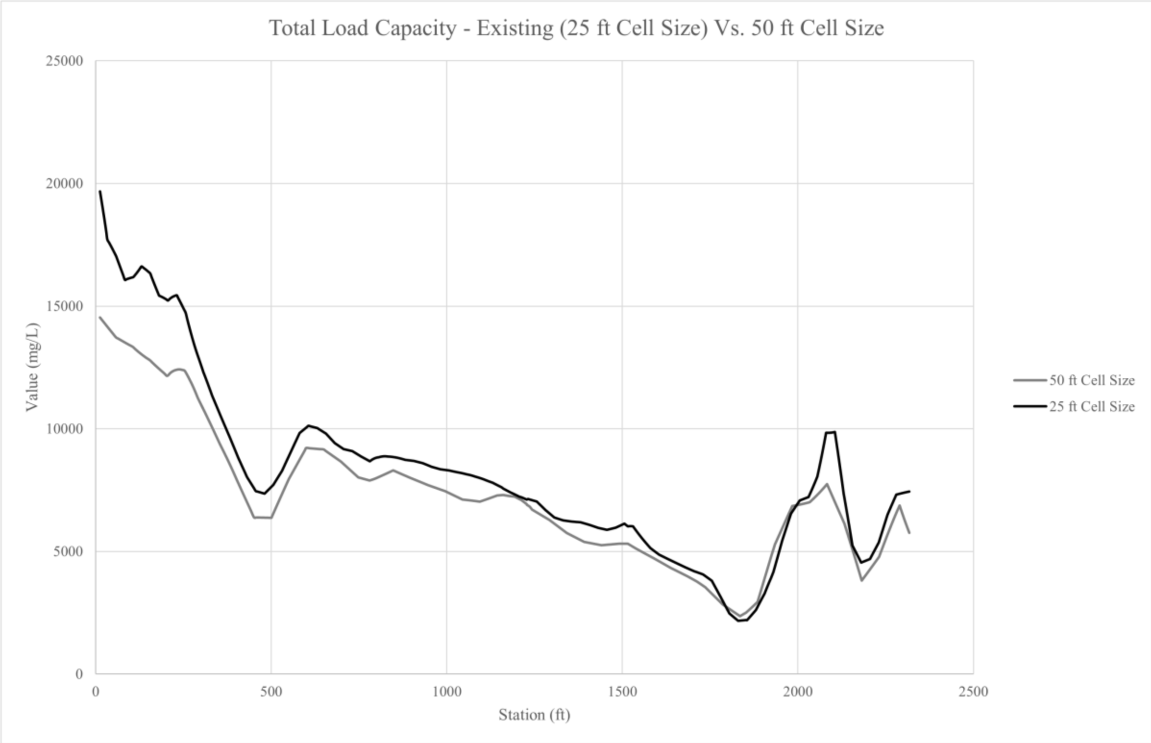

Figure 8-9. **Total load capacity – channel centerline profile**

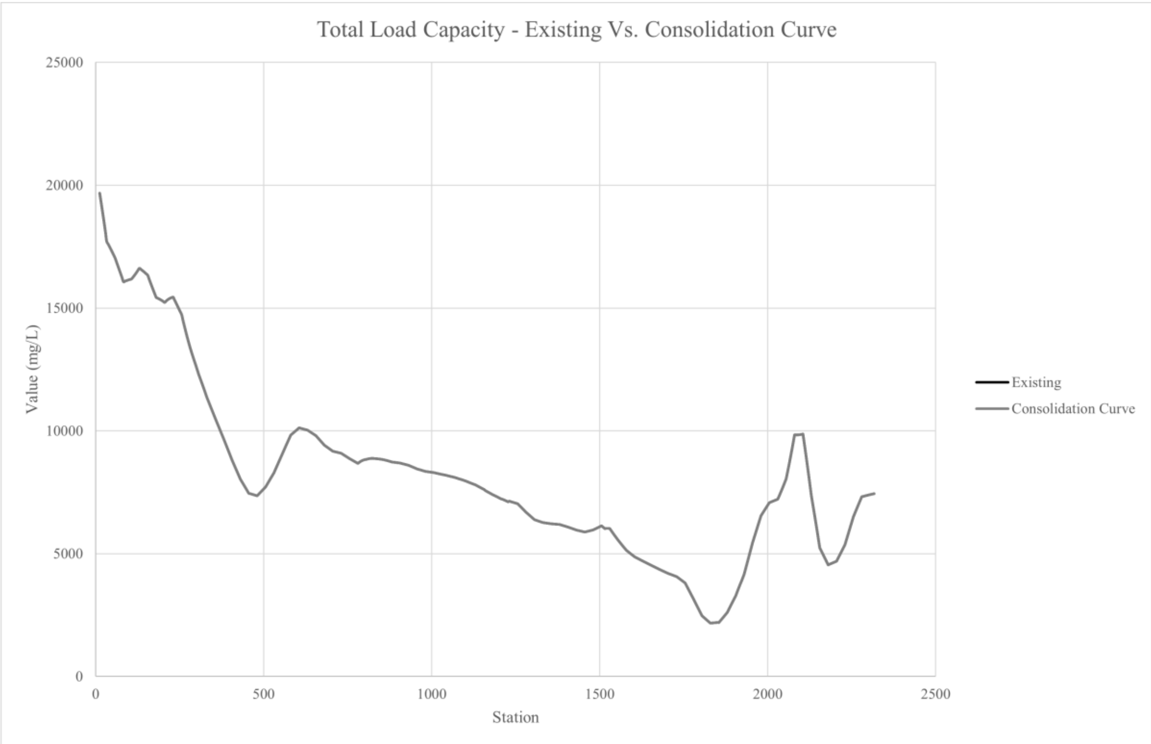

Figure 8-10. **Total load capacity – channel centerline profile**

8.2 Additional existing vs. proposed results

Figure 8-11 and **Figure 8-12** show plan view comparisons between the existing and proposed models for velocity and shear stress, respectively. Positive values show where the velocity or shear stresses were higher in the existing model while negative values show where the velocity or shear stresses were higher in the proposed model.

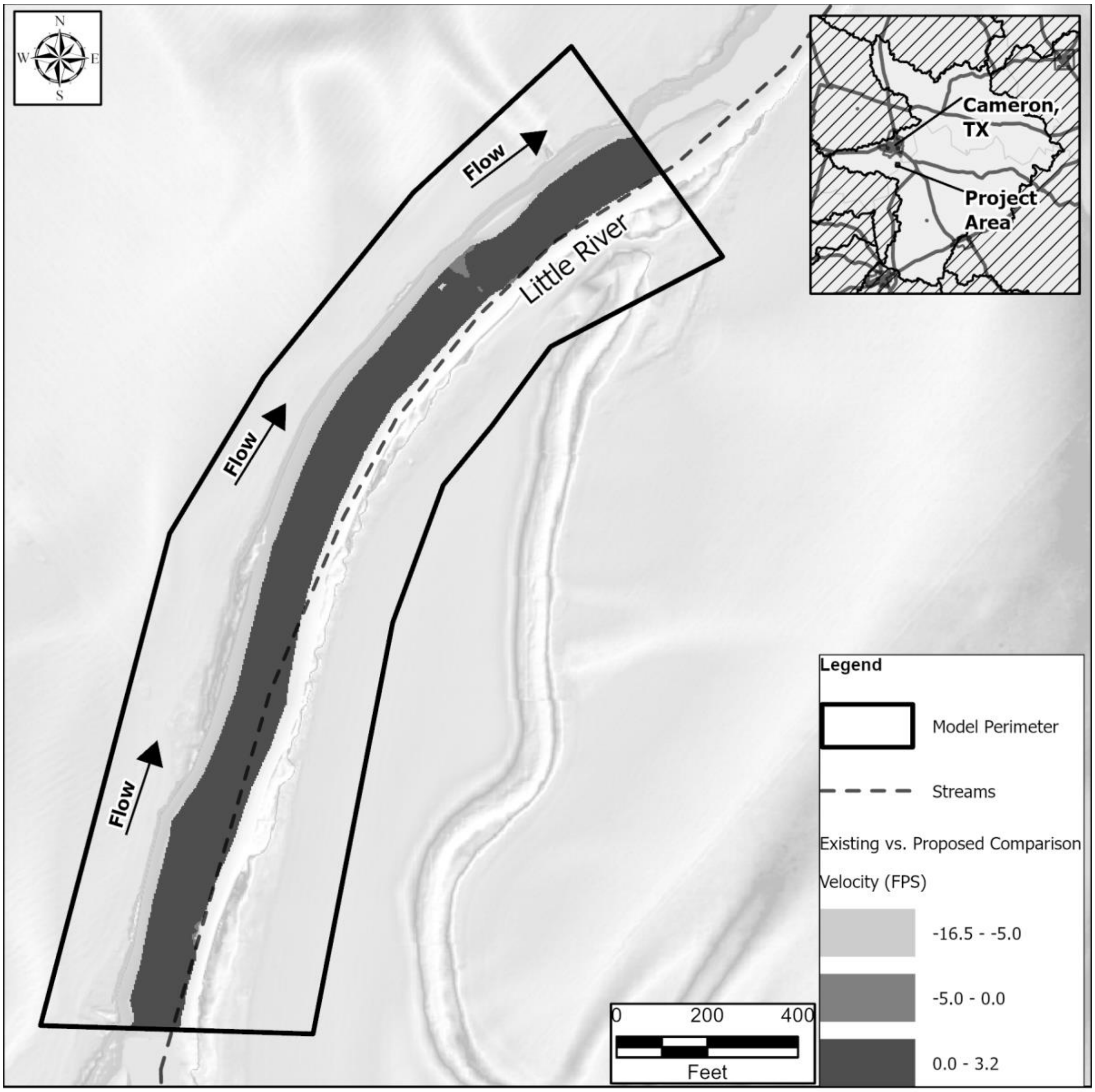

Figure 8-11. Existing vs. proposed comparison – velocity

Figure 8-12. **Existing vs. proposed comparison – shear stress**

9 References

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