Master Drainage Study

Prepared for

City of Tyler, Texas

in conjunction with the

Texas Water Development Board

Submitted By:

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CITY OF TYLER
MASTER DRAINAGE STUDY

EXECUTIVE SUMMARY

The Master Drainage Study was performed for the City of Tyler, in conjunction with the Texas Water Development Board, to update and develop the floodplain information for twelve watersheds identified by the City of Tyler and to prioritize future flood control improvement projects. The purpose of this plan is to provide the City with updated floodplain, floodway and flooding information and to establish a program for future floodplain management improvements.

The scope of the Master Drainage Study included twelve main creeks and several tributaries within the City limits and extra-territorial jurisdiction (ETJ) as identified by the City. The studied streams included:

Black Fork Creek	Henshaw Creek	Shackelford Creek
Butler Creek	Indian Creek	West Mud Creek
Gilley Creek	Little Saline Creek	Wiggins Creek
Harris Creek	Ray Creek	Willow Creek

The Master Drainage Study provides the City of Tyler with the following information:

- Updated hydrologic and hydraulic models for the detailed study areas.
- Hydrologic and hydraulic data for the approximate study areas.
- Updated floodplain and floodway mapping and comparisons to FEMA mapping.
- Identification of flooding and erosion problem areas.
- Evaluation of improvement alternatives for identified problem areas and associated flood reduction.
- Cost opinions for identified improvements.
- Project prioritization for 31 specific areas.
- Development of a Master Drainage Plan.
- Completion of a Master Drainage Study Report that summarizes the overall project.

The completion of the Master Drainage Study included the development of detailed hydrologic models for approximately 263 square miles of basins to define flood discharges ranging from the 2-year to the 500-year event. Flood discharges were determined for both existing conditions and the ultimate development conditions within the watershed. The resulting flood discharges were used in developing the hydraulic models.

Detailed hydraulic models were developed for approximately 114 miles of streams. These hydraulic models represent existing conditions based on City topographic data and field survey information for identified stream cross-sections, bridges and culverts. The hydraulic models generated water surface elevations that were used to delineate the limits of the 100-year floodplain. The hydraulic modeling also included the generation of floodways that are in accordance with FEMA guidelines. The floodplain and floodway limits are mapped on the available topographic maps.

The hydraulic results and mapping provided information on the extent of flooding throughout the City. This information showed that existing drainage structures were overtopped by up to 11 feet. Significant areas of road overtopping and structure flooding were identified and evaluated to determine the extent of future improvements required to reduce the impact of flooding and the corresponding risk. Thirty-one locations were identified for this evaluation. Hydraulic models were generated at these locations to determine the scope of improvements necessary to reduce overtopping and structure flooding. Cost opinions were developed for each of the improvement areas to identify the magnitude of the future costs as compared to the benefits produced by the proposed improvements.

The flooding information and cost opinions provide the basic information necessary for the project prioritization. Each of the 31 project areas were evaluated and ranked based on selected parameters. The resulting rankings were the basis for the overall Master Drainage Plan. The following table summarizes the results of the Master Drainage Plan development.

Master Drainage Plan Summary

1	WMC Trib C at FM 2493	$ 549,300	Installation of 4 - 8' by 8' RCB's.	9 feet	Eliminates 4.5' of road overflow and removes 20 residences from the floodplain	City funding, TxDOT, USACE, TWDB Loan and FMA.
2	WMC at Loop 323	$ 99,800	Installation of 4 - 6' by 6' RCB's.	3 feet	Eliminates 0.9' of road overflow and removes 2 structures from the floodplain	City funding and TxDOT.
3	Blackfork D2 at Beckham	$ 453,200	Installation of 2 - 10' by 7' RCB's.	6.4 feet	Eliminates 1.7' of road overflow and removes multiple structures from the floodplain	City funding, USACE, TWDB Loan and FMA.
4	Blackfork D4 at Fifth	$ 99,200	Installation of 1 - 8' by 8' RCB.	3.3 feet	Eliminates 1.8' of road overflow	City Funding
5	Willow at Erwin	$1,100,500	Installation of channel and box culvert improvements.	5.5 feet	Eliminates 1.5' of road overflow and removes multiple structues from the floodplain	City funding, USACE, TWDB Loan and FMA.
6	Blackfork at E. Fifth	$ 412,100	Installation of 6 - 10' by 10' RCB's.	4.6 feet	Eliminates 2' of road overflow and removes multiple structues from the floodplain	City funding, USACE, TWDB Loan and FMA.
7	Blackfork D3 at E Front	$ 252,900	Installation of 2 additional 10' by 10' RCB's.	5.46 feet	Eliminates road overflow and removes structues from the floodplain	City funding, USACE, TWDB Loan and FMA.
8	Henshaw at HWY 69 South	$1,462,100	Installation of a bridge with a 120' top width.	2.2 feet	Eliminates 1.2' of road overflow and removes multiple structues from the floodplain	City funding and TxDOT.
9	WMC Trib C at Broadway	$ 83,900	Installation of 3 additional 8' by 8' RCB's.	4 feet	Eliminates 4' of road overflow and removes multiple structues from the floodplain	City funding, USACE, TWDB Loan and FMA.
10	WMC at HWY 69 South	$2,596,800	Installation of a bridge with a 90' top width.	3.3 feet	Eliminates 2.7' of road overflow	City funding and TxDOT.

* All costs are based on 2007 $'s and include 30% contingencies. Costs do not include potential land acquisition costs, engineering, and permitting costs.

The final component of the Master Drainage Plan is the development of an implementation plan. The preceding table provides information on potential funding sources for the selected projects. Coordination with the identified agencies may provide funding that will reduce the overall funding required from the City and will impact the final implementation plan. The City should use the Master Drainage Plan as a tool to implement future projects and should adjust the plan as necessary to address these funding opportunities and additional local non-engineering considerations.

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PROJECT OBJECTIVES

The purpose of the Master Drainage Study is to update and develop the floodplain information for twelve watersheds identified by the City of Tyler and to prioritize future flood control improvement projects. To achieve these objectives, Nathan D. Maier Consulting Engineers, Inc. (NDMCE) has completed engineering tasks which include the site reconnaissance and data collection, review of existing floodplain information from both the City and Smith County, development of new hydrology models, development of new hydraulic models, evaluation of existing and future 100-year floodplain, determination of flood hazard areas, development of updated floodways, consideration of funding sources and recommendations for future flood control improvement projects. The purpose of this plan is to provide the city with updated floodplain and flooding information and to establish a program for future floodplain management improvements.

BACKGROUND

The City of Tyler Master Drainage Study addresses the flooding conditions for twelve main creeks and several of their tributaries within the city limits and extra-territorial jurisdiction (ETJ) as identified by the City. The following is a list of the studied reaches:

Black Fork Creek	Henshaw Creek	Shackelford Creek
Butler Creek	Indian Creek	West Mud Creek
Gilley Creek	Little Saline Creek	Wiggins Creek
Harris Creek	Ray Creek	Willow Creek

The MDS reaches are divided into detailed study and simple study areas. Approximately 114 miles of creek were analyzed in a detailed modeling approach and 77 miles were identified as minor creeks and tributaries and were analyzed using approximate methods. The modeling approaches will be discussed in further detail in the following sections.

Previous Studies

Some previous studies have been preformed for the City of Tyler and Smith County. In 1989 The C.T. Brannon Corporation prepared a study titled Storm Water Analysis and Management Plan (SWAMP) for the City of Tyler. This study provided hydrology and hydraulic analysis for Willow, Black Fork and its tributaries and West Mud and its tributaries. This study included an inventory of existing storm drainage facilities throughout the City, ground survey, hydrologic study of the areas mentioned above with computed water surface profiles for current and future development, possible areas for improvement projects in flood prone areas and also provided assistance to the city in obtaining amendments from FEMA to reflect the results from the SWAMP study.

Also previous studies include the FEMA FIS (Flood Insurance Study) study for Smith County and also for the City of Tyler. The FIS study by FEMA has detailed mapping for Black Fork Creek and some of its tributaries, Willow Creek, West Mud Creek and some of its tributaries, Henshaw Creek and Shackelford Creek. All other basins in the MDS study are considered Zone A non-detailed areas on the FEMA maps. The other basins are Indian Creek, Butler Creek, Gilly Creek, Harris Creek, Ray Creek and Wiggins Creek. The FEMA FIS study is currently in the process of being updated and has been submitted to the City for a 90 day review and appeal period. The final version of updated FIS study should finished in late 2008.

Data Collection

As part of this study, NDMCE collected various data from the City of Tyler. This information included the SWAMP and FIS studies outlined above, information from the City's GIS system, hydraulic studies from various projects through out the City and also as-built plans for roadways and hydraulic structures in the City. The information from the GIS system included two-foot contour maps inside the City limits and four-foot contour maps for the ETJ. These maps were developed from the City's 2003 aerial survey. Also included in the GIS information was an inventory of the City's infrastructure including streets and buildings. The City also provided soil and land use information based on current development and also future development. Dunkin Sefko & Associates provided the future land use outside the corporate limits from a previous City of Tyler planning project.

Field Reconnaissance

NDMCE performed field reconnaissance to become familiar with the City of Tyler's infrastructure as well as the characteristics of the study area. This included reviewing hydraulic structures, hydraulic conditions, existing improvements in the area, density of vegetation and any other visual conditions that might impact hydraulic modeling. This reconnaissance provided information to establish locations of for surveyed cross-section and also to establish mannings roughness n-values for specific reaches. Appendix A is a selection of photos from the field reconnaissance.

Survey

Survey data was acquired at every bridge structure in the study area to allow for detailed bridge modeling. The survey of the bridge structures included flow line of channel, location of piers, top of bank, toe of bank, low chord of bridge, centerline of roadway and edge of pavement. For culvert structures the size of barrels was also noted. In addition to surveyed bridges, additional cross-sections were surveyed at bridges to add better detail to the hydraulic models and to give better characteristics of the actual channel than the aerial topographical information provided. In the approximate study areas detailed survey was not taken. The size of culvert structure and distance from flow line of culvert to the top low point of the roadway was acquired in the approximate study areas.

HYDROLOGY

General Methodology

New hydrologic models were developed using the U.S. Army Corps of Engineers (USACE) HEC-1 Watershed Modeling Computer Program. The models were developed based on existing development and ultimate development within the watershed based on the City of Tyler GIS data for existing and future landuse information. The difference between the existing development and a fully urbanized condition in the study area is fairly significant; the impact of the future development on the flood discharges will be discussed in a following section.

Flood discharges were computed for the existing development and ultimate development conditions for the 2-, 5-, 10-, 25-, 50-, 100- and 500-year flood events using the revised hydrology model. The resulting discharges were compared to previous FEMA discharges and the SWAMP study and reviewed with the City for final acceptance. The updated flood discharges, based on revised hydrologic models, were used for the hydraulic analysis portion of the study.

A series of 24-hour design storms were developed with a computational interval of 5 minutes. The Snyder's unit hydrograph procedure was utilized in this study. The HEC-1 program analyzes the incremental rainfall amounts in a critical pattern and generates runoff which, when applied to the unit hydrograph, produces a runoff hydrograph associated with each sub-area based on its drainage area, design rainfall conditions and sub-area rainfall losses. The resulting hydrographs from the individual sub-areas are routed downstream based on derived storage-discharge relationships and combined with hydrographs from other sub-areas to provide a total runoff hydrograph.

The hydrologic parameters that are required to generate runoff hydrograph include drainage area, land use conditions, unit hydrograph parameters and routing information.

Drainage Areas

Drainage areas were delineated for the hydrologic modeling using the available 2-foot contour data within the ETJ and 4-foot contour interval data outside the ETJ as based on the City GIS data. Each of the twelve watersheds were analyzed separately and then combined on an overall map. *Exhibit 1* shows the watershed delineation for the study area. The total drainage area analyzed for the MDS is approximately 263 square miles. The watersheds and their respective drainage areas are presented in Table 1.

Table 1 – Watershed Drainage Areas

Watershed	Drainage Area (mi^2)
Black Fork Creek	50.31
Harris Creek	93.87
Indian Creek	23.77
Ray Creek	18.04
West Mud Creek	59.37
Willow Creek	6.34

Watershed	Drainage Area (mi^2)
Gilley Creek	12.50
Henshaw Creek	7.57
Little Saline Creek	10.47
Shackelford Creek	10.40
Wiggins Creek	17.24
Butler Creek	12.22

Land Use and Soil Conditions

The objective of this study is to determine the impact of the existing and ultimate developed conditions for the twelve watersheds in the MDS study area. The existing and future urbanization of the study area have an affect on the hydrologic response of the watersheds. In general, the affect of development on the watershed is to increase the peak discharge for some flood events and reduce the response time of the watershed. The type of land use for the existing development conditions was based on the City of Tyler existing land use information as available from the GIS department. The GIS data covered only the area with the corporate City Limits. The existing land use outside of the corporate limits was digitized from the 2003 aerial photography and combined with the City information. The ultimate development landuse determination was established from the future landuse information provided with the GIS data for the area within the corporate limits. Dunkin Sefko & Associates provided the future landuse outside the corporate limits from a previous City of Tyler planning project.

Soil mapping was provided with the City GIS data. This included the Natural Resources Conservation Service (NRCS) Soil Survey Geographic (SSURGO) Database mapping for Smith County. The SSURGO soils information includes the hydrologic soils group identification, which is used to calculate the initial and constant loss rates for the Snyder's Unit Hydrograph method.

Unit Hydrographs

Unit hydrographs were derived for the sub-areas based on Snyder's unit hydrograph procedure. Development of Snyder's unit hydrographs is highly dependent on the C_t and C_p (Snyder's coefficients) values that can be used in the procedure. The C_t value can be used to determine the watershed lag time and the C_p value is used in the determination of the peak discharge for the unit hydrograph.

The soil types in the MDS study area vary greatly, with sandy clay and sandy soils being the predominant types. Previous studies of Snyder's coefficients in the region resulted in a range of typical values for the C_p coefficient. The C_p for all watersheds was assigned as approximately 0.72, based upon previous studies by the Corps of Engineers.

The response of a sub-area to a rainfall event is called the time-of-concentration (T_c). The Snyder's C_t value was not used in this study for determining the lag time since there is not appropriate information for determining this value. For the purpose of this study the lag time was based on the time-of-concentration which was computed based on drainage system characteristics included type of drainage system, system slope, resistance to flow, travel lengths and flow velocities. This information was used to compute incremental travel times and overall sub-area time of concentration (T_c). The basin lag time (T_p) was computed as to be approximately 0.6 T_c.

Rainfall Distributions

The design storm point rainfall values used in the models were based on historical rainfall depths, frequencies and storm characteristics. Point rainfall depths were taken from National Oceanic and Atmospheric Administration Technical Memorandum (NOAA) Hydro-35 and the National Weather Service Technical Paper 40 (TP-40). The accumulated point rainfall values used to model recurrence intervals from 2 to 500 years are shown on Table 2. A computational interval of 5 minutes was used for the generation of all hydrographs.

Table 2 – Rainfall Distribution

Duration	Rainfall Depth (in.) per Return Period						
Time	2-yr	5-yr	10-yr	25-yr	50-yr	100-yr	500-yr
5-min	0.52	0.59	0.65	0.73	0.80	0.87	1.03
15-min	1.10	1.27	1.39	1.59	1.74	1.89	2.23
60-min	1.98	2.44	2.77	3.24	3.61	3.98	4.83
2-hrs	2.49	3.25	3.77	4.38	4.88	5.42	6.63
3-hrs	2.75	3.50	4.15	4.81	5.37	5.96	7.25
6-hrs	3.25	4.79	5.00	5.91	6.67	7.45	9.25
12-hrs	3.79	5.08	6.00	6.96	7.90	8.90	11.25
24-hrs	4.45	5.90	6.95	8.18	9.25	10.3	12.87

Rainfall Losses

Rainfall losses due to surface detention, soil wetting and infiltration are characterized in the hydrology model by initial and constant rainfall losses. The initial loss represents rainfall lost to surface detention, plant interception, soil wetting etc., while constant losses reflect the soil ability to allow infiltration as the rainfall continues. Rainfall lost to depression storage is not available as direct runoff and eventually infiltrates the soil and becomes evaporation. Once initial and constant losses are fulfilled, then surface runoff begins. Both the initial and constant losses depend on land use conditions, hydrologic soil types and the frequency of rainfall event. More extreme rainfall events usually have lower initial and constant loss rates due to prior rainfall conditions and higher antecedent moisture conditions.

Initial and constant losses were computed for each watershed based on the SSURGO soils mapping. The hydrologic soil type is an indication of the soil's ability to store and infiltrate rainfall amounts and is categorized in four groups from type A to type D. In general, hydrologic soils type A are generally sandy soils with higher infiltration and percolation rates, while type D soils are typically heavy clay soils with low percolation rates. The initial and constant losses for these types of soils are shown on Table 3.

Table 3 – Soil Loss Rates

Hydrologic Soil Group	Initial Rainfall Loss (in.)	Constant Rainfall Loss (in./hr.)
A	1.50	0.50
C	0.75	0.15
B	1.00	0.30
D	0.50	0.05

The percent impervious area was also computed for the sub-areas of each watershed by determining the percent of land use type within the sub-area. The percent of each land use type was determined from a combination of the City GIS existing land use data as well as aerial photography. Each individual land use type was assigned a general percent of impervious area as shown in Table 4.

Table 4 – Land Use Percent Impervious

Land Use Type	% Impervious
Commercial	85%
Office	95%
Retail	95%
Light Industrial	70%
Medium Industrial	72%
Heavy Industrial	81%
Multi-Family	70%
Town Home	65%
Duplex	53%
Single Family	47%
Public/Semi-Public	20%
Golf Course	5.0%
Parks	2.0%
Vacant	5.0%
Zoo	5.0%
Manufactured Home	65%
Unidentified	5.0%
0.5 Ac Residential	25%
1 Ac Residential	20%
2 Ac Residential	12%
Water	100%
Right-of-Way	77%
Bare Soil	10%
Row Crops	10%

The values in Table 4 were then applied to the percentage of land use type within each sub-area to determine the weighted percent of imperviousness within the sub-area. The remaining portion of the sub-area that is pervious will have initial and constant losses. The amount of losses that will result depends on the hydrologic soil types within the sub-area. The weighted initial and constant loss rates, as well as the percent impervious, as calculated for each sub-area can be found in the HEC-1 input model on the LU card for the Snyder's unit hydrograph method.

Runoff Hydrograph and Flow Attenuation

The HEC-1 hydrology-modeling program developed runoff hydrographs by applying the rainfall excess amounts to the Snyder's unit hydrograph. This involved multiplying the rainfall excess amounts by the unit hydrograph ordinates to develop runoff hydrographs for each time step. The resulting runoff hydrographs were then summed to develop total runoff hydrographs for each sub-basin. Local runoff hydrographs are combined with overall stream hydrographs and routed downstream through a stream network. The routing process accounts for a lag time between sub-area runoffs and stream attenuation due to floodplain storage.

HEC-1 models the impact of the storage volume relationship using the Modified-Puls routing method. This method performs routing calculations by defining the difference of inflow and outflow of a stream reach as a change in the storage of the reach for the computational time period. Routing information consists of a storage-discharge data based on the relationship of the stream reaches storage volume and outflow information. This data can be obtained by either the hydraulic model's storage volumes at a specific discharge for a stream reaches or in the hydrology model by developing a typical cross section in the stream reach. The Modified-Puls storage-volume relationship method was used for all routing reaches where the data was available from the initial HEC-RAS model. The storage-volume relationships were checked after the development of the HEC-RAS model and were adjusted as necessary. The normal depth channel storage routing method was used in reaches where storage-volume data was not available from the HEC-RAS models. The number of routing steps was computed based on the overall reach length; approximate flow velocity and computational time increment.

HEC-1 Results

The HEC-1 model was used to generate runoff hydrographs for 2-, 5-, 10-, 25-, 50-, 100- and 500-year flood events. Appendix B is a summary of values used in each HEC-1 model, including area of drainage basin, lag time for existing and ultimate conditions, and percent impervious for existing and ultimate conditions. The 100-year flood hydrographs were developed for both existing land use conditions as well as ultimate developed conditions. The peak discharges from the HEC-1 model were incorporated into the hydraulic model for the flood study. Appendix B presents the complete list of results from the HEC-1 models for the twelve watersheds studied as a part of the Master Drainage Study.

Early in the Master Drainage Study, an evaluation was made of available stream flow information that could be used for model calibration. A statistical analysis was performed for Mud Creek (376 sq. mi.) and Big Sandy (231 sq. mi). Both of these gages are noted by the USGS to be impacted by diversions or flood storage. In particular, the discharges on Mud Creek are impacted by the storage available at Lake Tyler. Comparison of the statistical analyses from these gauges with previous studies and the modeling for the Tyler watersheds found that the gauges did not provide any data useful for model calibration. No other historical data were available for use with this study. Appendix I is a hydrology summary to the approach mentioned above. It should be noted that the numbers in Appendix I are from early in this study and do not reflect the final hydrology numbers presented in this study.

Existing Conditions vs. Ultimate Development

One of the objectives of this Master Drainage Study is to determine the impact of increased urbanization on flood conditions for the City of Tyler. To evaluate the impact of additional development on flood discharge values, hydrology models were developed for future ultimate development land use conditions based on the City of Tyler Comprehensive Future Landuse Plan within the corporate limits and the ETJ. The MDS assumes that all areas contributing runoff to the watersheds will be developed to their maximum potential.

Ultimate development has several effects on the hydrologic response of watersheds. In general, the urbanization of a watershed will increase the peak discharge experienced within the basin. The increases in discharge and runoff are usually influenced by the following components:

- Increased impervious area
- Decreased time of concentration for individual sub-areas and a reduction of channel flow times
- Changes in timing of individual sub-area runoffs with respect to one another
- Loss of floodplain storage

The increase in impervious area was determined from ultimate landuse conditions presented on the Comprehensive Future Landuse plan. One of the hydrologic effects of urban development is a reduction of the basin lag time (T_p) for individual sub-areas. As the lag time values are decreased, the sub-areas respond more rapidly to rainfall events. This generally increases the peak values for unit hydrographs and alters the timing between individual sub-areas. To account for the effect of increased urbanization, the T_p values were analyzed for several individual sub-areas on selected watersheds. An average decrease of 20% in the fully urbanized lag time was determined. This decrease was then applied to sub-areas that were not fully developed in existing conditions.

As shown in Appendix B, the increases in peak discharge due to ultimate urbanization vary from watershed to watershed. Generally, increases ranged from 4% to 16%. These increases depend greatly on the type of ultimate development in the watershed as well as the existing hydrologic soil characteristics. Typically, higher impervious area has a greater affect on sandier soils. Sandy soils have a higher infiltration rate and as the impervious percentage increases, more runoff occurs. Because clay soils have lower infiltration rates, it has higher rainfall excess in existing conditions and does not have as great of an increase when urbanized as sandy soils.

HYDRAULICS

General Methodology

The hydraulic analysis uses the discharges developed by the hydrology models to compute the impacts of the calculated flows on the river or stream system. The hydraulic analysis uses these flows to determine the water surface elevations, flow velocities and other associated hydraulic variables.

In addition to the hydrologic analysis program, HEC-1, the U.S. Army Corps of Engineers (USACE) Hydrologic Engineering Center developed HEC-RAS, a general river analysis program. HEC-RAS is one-dimensional modeling software typically used to perform hydraulic computations on stream networks. Given the broad acceptance of HEC-RAS as a hydraulic analysis tool, it was chosen for the required hydraulic analysis portion of this study. The most recently available version of HEC-RAS v3.1.3 released in May 2005 was utilized.

Detailed Hydraulic Analysis

The City staff identified approximately 114 miles of major and minor creeks to be analyzed by detailed methods. Cross-sections were coded with existing ground geometry based on the GIS 2-foot contour mapping, coded on average every 400 feet, and at any obstructions or major changes in the creek. In addition to using the GIS 2-foot contours, NDMCE also utilized surveyed cross-sections in various locations throughout each reach in order to determine more detailed channel characteristics. Cross-sections taken from the GIS information were modified inside the channel banks based on information taken from the various surveyed cross-sections.

The 1989 SWAMP study analyzed 29 miles of stream in detail. The cross section locations from the SWAMP study were located on the current mapping used in this study. Cross-sections in the

MDS study were also taken in approximately the same locations in order to have a better comparison in the two studies.

Within the 114 miles of detailed study reaches, NDM identified 161 culverts and bridge structures to be analyzed for this study. These 161 structures were verified with survey data, and that data was used to model the structures into HEC-RAS. Information used to model structures in HEC-RAS are size and shape of hydraulic structure, flow line, center line of roadway, edge of pavement and data for sloping abutments and wing walls.

Mannings n-values throughout the City of Tyler vary greatly from 0.013 for concrete lined channel to 0.10 for thick vegetation growth in the floodplain overbanks. Typical Mannings n-values taken from HEC-RAS used for the MDS are summarized in Table 5.

Table 5 – Typical Mannings n-Values

Type of Channel and Description	Medium	Normal	Maximum
Natural Streams			
Main Channels			
Clean, straight, full, no rifts or deep pools	0.025	0.03	0.033
Same as above, but more stones and weeds	0.03	0.035	0.04
Clean, winding, some pools and shoals	0.033	0.04	0.045
Sluggish reaches, weedy, deep pools	0.05	0.07	0.08
Very weedy reaches, deep pools, or floodways with heavy stands of timber and brush	0.07	0.1	0.15
Flood Plains			
Pasture no brush			
Short grass	0.025	0.03	0.035
High grass	0.03	0.035	0.05
Brush			
Scattered brush, heavy weeds	0.035	0.05	0.07
Light brush and trees, in winter	0.035	0.05	0.06
Light brush and trees, in summer	0.04	0.06	0.08
Medium to dense brush, in winter	0.045	0.07	0.11
Medium to dense brush, in summer	0.07	0.1	0.16
Trees			
Cleared land with tree stumps, no sprouts	0.03	0.04	0.05
Heavy stand of timber, few down trees, little undergrowth, flow below branches	0.08	0.1	0.12
Dense willows, summer, straight	0.11	0.15	0.2
Lined or Built-Up Channels			
Concrete			
Trowel finish	0.011	0.013	0.015
Float Finish	0.013	0.015	0.016
Finished, with gravel bottom	0.015	0.017	0.02
Unfinished	0.014	0.017	0.02
Concrete bottom float finished with sides of:			
Dressed stone in mortar	0.015	0.017	0.02
Random stone in mortar	0.017	0.02	0.024
Cement rubble masonry, plastered	0.016	0.02	0.024
Cement rubble masonry	0.02	0.025	0.03
Dry rubble on riprap	0.02	0.03	0.035

Starting water surface elevations were discussed with the City of Tyler and an agreement was made as to what values would be used for this study. Starting water surface elevations for the individual streams were started at critical depth. Critical depth is defined as the minimum energy in the cross-section. For Tributaries of West Mud, Black Fork, Gilley and Harris Creeks the starting water surfaces were based on the ratio of drainage areas. If the ratio of the drainage area for the main channel to tributary channel was 15:1 or less, then the water surface elevation from the 25-year storm was used to start the tributary for the 100-year event. If the ratio of drainage area was over 15:1 then coincident water surfaces were used. For mapping purposes backwater elevations from the main creeks 100-year profile was used.

Modeling Results

West Mud Creek and Tributaries

West Mud and all of its tributaries have an overall drainage area of 59.37 square miles, including Shackelford and Henshaw Creeks. The detailed hydraulic modeling for West Mud Creek involved approximately 16.1 miles of creek for the main channel and 24.85 miles of tributaries. Table 6 shows the length of each stream in the detailed analysis of the West Mud Basin.

Table 6 – West Mud Basin Stream Lengths

Creek Label	Length (mi)	Creek Label	Length (mi)
West Mud	16.10	M-11	2.27
Shackelford	7.20	M-11	1.30
Henshaw	4.90	M-B	1.40
M-A	2.34	M-C	2.56
M-A1	0.60	M-C1	0.57
M-A2	1.14	M-C2	0.57

West Mud Creek and its tributaries can be classified as a partially urban stream and partially rural stream. The lower portion of West Mud creek is almost all considered rural stream with a very wide floodplain area, in some locations measuring almost a half-mile wide. Dense vegetation in the overbanks can be seen in most of the lower portion of West Mud with Mannings n-values approaching 0.10 in some locations. Shackelford and Henshaw Creeks combine with West Mud in the lower section of the creek on the south side of the City. Both of these creeks can also be considered rural streams with wide floodplain areas and dense vegetation in the overbanks.

The upper portion of West Mud and its tributaries would be considered urban stream with combinations of natural channel, canalized and concrete lined sections. Mannings n-values in the overbanks for these location vary greatly from as high as 0.10 in very dense vegetated areas to as low as 0.035 in manicured park areas.

West Mud Creek located on the southern side of the City of Tyler creates significant flooding in the city. The lower portion of the creek located in a less developed portion of the city currently does not create major flooding problems. The push for new development in the City of Tyler is located in the southern sector of the city so future flooding could be a problem. The upper portion of West Mud Creek runs through a heavily developed area of the city. There are currently 11 bridges that cross the main channel of West Mud, nine of which are overtopped by the 100-year ultimate water surface elevation. Table 7 shows the roadways that cross the main channel of West Mud and the amount of water that overtops the roadway. Along the main channel of West Mud there are approximately 73 building structures located within the 100-year ultimate floodplain. The major flooding along West Mud is located just upstream of Shiloh Road at station 880+00 flooding 11 structures. Starting at station 910+00 upstream of New Copeland Road there are 12 structures within the floodplain. Just upstream of Easy Street there are 13 structures within the floodplain, six along Sybil Drive and seven along Broadway.

Table 7 – West Mud Creek Roadway Overtopping

Roadway	Cross Section	Depth of Overtopping (ft)
CR 129	25526	5.61
FM 346	37193.5	3.17
Hwy 69 S.	40446	1.62
FM 2813	55765	2.9
Grande	77852	Not Overtopped
Broadway	78977	2.7
Rieck	83554	2.13
Shiloh	86424	Not Overtopped
New Copeland	89488	0.37
Easy	92191	2.4
Loop 323	93762	1.08

The following Table 8 shows a summary of the modeling results for West Mud Creek. These tables show a summary of the existing 100-year, 2-year and the 100-year ultimate water surface elevations. The tables have been condensed for this report; and the full summary table can be found in Appendix C.1. For detailed mapping of the existing100-year and 100-year ultimate water surface elevations and cross section locations see Appendix D.1. Profile of the creek can be seen in Appendix E.1

Table 8 – West Mud Creek Results Summary

West Mud Creek									
	100 Year Existing			2 Year Ultimate			100 Year Ultimate		
Station	Q (cfs)	V (ft/s)	WSE (ft)	Q (cfs)	V (ft/s)	WSE (ft)	Q (cfs)	V (ft/s)	WSE (ft)
10000	44313	14.79	354.24	12851	11.35	352.37	47207	15	354.38
14618	44313	4.67	364.48	12851	3.08	360.3	47207	4.79	364.76
20732	44313	2.51	371.85	12851	2.06	367.4	47207	2.56	372.15
25496	44313	3.97	380.62	12851	1.7	378.17	47207	4.06	380.93
25511	C.R. 129								
25526	44313	3.97	380.94	12851	1.71	378.38	47207	4.07	381.23
29238	42730	4.35	382.35	13408	2.33	378.84	44491	4.38	382.63
36588	24724	4.23	385.69	9134	2.79	381.99	25549	4.26	385.89
37169	24724	14.92	388.33	9134	12.6	383.02	25549	15.08	388.39
37193.5	F.M. 346								
37218	24724	6.14	390.09	9134	8.09	384.48	25549	6.23	390.19
40290	24724	14.08	391.65	9134	13.47	387.05	25549	14.2	391.73
40315	U.S. 69 South								
40340	24724	6.58	393.35	9134	7.03	389.17	25549	6.67	393.43
40423	24724	11.53	393.04	9134	8.08	389.22	25549	11.71	393.12
40446	U.S. 69 South								
40469	24609	5.24	394.01	9198	6.53	389.66	25468	5.3	394.11
46495	24359	5.03	396.14	9298	4.32	392.84	25257	5.09	396.27
52884	23879	4.22	405.29	9275	3.28	402.32	24878	4.28	405.45
55743	23727	11.54	414.04	9295	11.63	409.2	24750	15.39	413.14
55765	F.M. 2813								
55787	23727	8.49	415.7	9295	8.5	410.05	24750	9.37	415.47
59107	23727	4.12	416.94	9295	4.53	412.2	24750	4.32	416.9
63395	23593	4.95	420.52	9446	4.41	417.7	24530	5	420.65
68254	22230	12.97	429.77	8968	8.95	427.07	23014	13.15	429.89
72303	22226	7.35	436.24	8967	5.48	432.9	22907	7.36	436.39
76714	15325	7.03	447.5	6219	4.3	444.2	15481	6.96	447.63
77742	15325	7.83	448.95	6219	4.73	445.78	15481	7.85	449.03

West Mud Creek (Cont.)									
	100 Year Existing			2 Year Ultimate			100 Year Ultimate		
Station	Q (cfs)	V (ft/s)	WSE (ft)	Q (cfs)	V (ft/s)	WSE (ft)	Q (cfs)	V (ft/s)	WSE (ft)
77852	Grande St.								
77962	15325	7.25	449.7	6219	4.48	446.15	15481	7.26	449.78
78848	15055	9.15	458.46	6146	8.12	452.83	15203	9.22	458.49
78977	S. Broadway								
79106	15055	8.78	462.63	6146	7.37	456.63	15203	8.82	462.68
82802	14456	9.33	466.26	6415	8.14	461.56	14580	9.33	466.32
83528	11027	11.13	469.96	4978	14.36	464.37	11019	11.13	469.96
83554	Ashburn Dr.								
83580	11027	5.21	470.81	4978	4.67	467.74	11019	5.2	470.81
86313	11027	9.86	472.48	4978	6.8	469.6	11019	9.85	472.48
86388	11027	12.67	472.04	4978	7.65	469.61	11019	12.66	472.04
86424	Shiloh Rd.								
86460	11027	9.09	475.02	4978	7.24	470.07	11019	9.09	475.01
88833	3323	4.95	477.98	1588	6.24	474.53	3338	4.96	477.99
89441	3323	5.8	479.69	1588	3.57	477.26	3338	5.83	479.7
89488	Copeland Rd.								
89535	3323	4.84	480.28	1588	3.58	477.4	3338	4.85	480.29
89927	3323	4.73	483.07	1588	6.17	480.2	3338	4.73	483.08
89942.5	Culvert								
89958	3323	4.31	483.33	1588	5.74	480.68	3338	4.31	483.34
90510	4266	8.99	483.55	2060	9.25	481.34	4278	8.99	483.56
90525.5	Culvert								
90541	4266	6.99	484.85	2060	5.51	483.02	4278	7	484.86
91474	4266	7.07	489.25	2060	4.69	487.68	4278	7.08	489.26
92167	3488	8.68	491.64	1646	6.26	489.29	3499	8.68	491.65
92191	Kidd Dr.								
92215	3488	6.15	492.37	1646	6.02	489.8	3499	6.15	492.38
93627	1885	3.34	500.18	869	13.09	496.3	1891	3.34	500.2
93694.5	Loop 323								
93762	1885	2.12	506.1	869	13.21	503.38	1891	2.18	506.06
93968	1885	3.34	506.1	869	1.36	506.38	1891	3.41	506.07

Tributary M-2 of West Mud creek is located just east of the intersection of Broadway and Grande Blvd. Barbee Drive is the only street that crosses Tributary M-2 and it is overtopped by the 100-year ultimate water surface. No other structures along Tributary M-2 are located within the 100-year ultimate floodplain.

Tributary M-1 is located on the west side of the Main channel of West Mud Creek just west of the intersection of Grande Blvd. and Hollytree Drive. There are three roadways crossing Tributary M-1 of West Mud Creek, two of which are overtopped by the 100-year ultimate water surface. Table 9 shows the roadways that cross Tributary M-1 and the amount of water going over the roadway. There are approximately 12 building structures located within the 100-year floodplain, seven of which are located just upstream of Grande Blvd at station 40+00 and five at the intersection of North Star Drive.

Table 9 – Tributary M-1 of West Mud Creek Roadway Overtopping

Roadway	Cross Section	Depth of Overtopping (ft)
Hollytree	2022.5	1.87
Grande	2552	Not Overtopped
North Star	5070	2.43

The following Table 10 shows a summary of the modeling results for Tributary M-1 of West Mud Creek. This table shows a summary of the existing 100-year, 2-year and the 100-year ultimate water surface elevations. The tables have been condensed for this report, the full summary table can be found in Appendix C.1. For detailed mapping of the existing100-year and 100-year ultimate water surface elevations and cross section locations see Appendix D.1. Profile of the creek can be seen in Appendix E.1

Table 10 – West Mud Tributary M-1 Creek Results Summary

West Mud Creek Tributary M1									
	100 Year Existing			2 Year Ultimate			100 Year Ultimate		
Station	Q (cfs)	V (ft/s)	WSE (ft)	Q (cfs)	V (ft/s)	WSE (ft)	Q (cfs)	V (ft/s)	WSE (ft)
1000	5581	1.78	443.16	2028	1.63	440.38	5419	1.69	443.24
1965	5581	6.81	446.31	2028	14.95	441.08	5419	6.61	446.31
2022.5	Hollytree Dr.								
2080	5581	1.7	447.87	2028	0.88	446.45	5419	1.66	447.84
3454	5581	9.71	447.67	2028	4.22	446.48	5419	9.44	447.66
3506	5581	16.75	447.59	2028	7.11	446.34	5419	16.6	447.42
3591	Grand Blvd.								
3676	5581	8.37	458.5	2028	8.14	446.99	5419	8.52	457.69
5040	3678	2.67	459.75	1655	7.75	453.84	3844	3.42	458.99
5070	Apartment Bridge								
5082	3678	3.31	459.76	1655	4.3	455.74	3844	4.07	459.01
6600	3168	7.23	463.05	1424	6.16	461.66	3311	6.33	463.56
6742	3168	6.43	464.94	1424	13.23	462.26	3311	8.75	464.06
6774.5	North Star Blvd.								
6807	3168	4.82	465.72	1424	2.74	465.05	3311	5.17	465.64
6873	3168	6.49	465.76	1424	3.82	465.06	3311	6.95	465.7

Tributary MA of West Mud Creek is located on the west side of the main Channel of West Mud just east of the intersection of Grande Blvd. and Hollytree Drive. There are 4 roadways that cross Tributary MA and two of those are overtopped by the 100-year ultimate water surface. Table 11 shows the roadways along Tributary MA and the amount of water overtopping the roadway. Thirty-three structures are located within the floodplain for Tributary MA. Ten of those structures are located just upstream of Rieck Road along Spring Creek Drive. The other major flooding are along Tributary MA is located between Loop 323 and Woodland Drive. This portion of Tributary MA currently has 19 structures within the 100-year ultimate floodplain.

MA has two tributaries, MA-1 and MA-2. Tributary MA-1 has two roadways that cross the creek, and tributary MA-2 has three roadways that cross the channel. Table 12 shows the roadways for Tributary MA-1 and Table 13 shows the roadways for Tributary MA-2. There are currently two structures located within the 100-year ultimate floodplain along Tributary MA-1; one located upstream of Rice Road approximately at station 12+00, and one located just upstream of Charleston Road.

Table 11 – Tributary MA of West Mud Creek Roadway Overtopping

Roadway	Cross Section	Depth of Overtopping (ft)
Grande	1423.5	Not Overtopped
Rieck	4817	1.13
Rice	7277	1.16
Loop 323	12082	Not Overtopped

Table 12– Tributary MA-1 of West Mud Creek Roadway Overtopping

Roadway	Cross Section	Depth of Overtopping (ft)
Rice	568.5	0.37
Charleston	3002.5	2.11

Table 13 – Tributary MA-2 of West Mud Creek Roadway Overtopping

Roadway	Cross Section	Depth of Overtopping (ft)
Private Drive	986	Not Overtopped
FM 2493	1525	1.37
Loop 323	3085	1.14

The following Table 14 through Table 16 show a summary of the modeling results for Tributary MA of West Mud Creek and its tributaries . These tables show a summary of the existing 100-year, 2-year and the 100-year ultimate water surface elevations. The tables have been condensed for this report; the full summary table can be found in Appendix C.1. For detailed mapping of the existing100-year and 100-year ultimate water surface elevations and cross section locations see Appendix D.1. Profile of the creek can be seen in Appendix E.1

Table 14 – West Mud Tributary A Creek Results Summary

West Mud Creek Tributary M-A									
	100 Year Existing			2 Year Ultimate			100 Year Ultimate		
Station	Q (cfs)	V (ft/s)	WSE (ft)	Q (cfs)	V (ft/s)	WSE (ft)	Q (cfs)	V (ft/s)	WSE (ft)
671	5343	9.76	443.48	2278	7.22	442.75	5615	10.09	443.51
1360	5343	13.86	446.15	2278	8.14	444.49	5615	14.07	446.35
1382	Grand Blvd.								
1404	5343	9.31	449.21	2278	7.01	445.63	5615	9.01	449.81
1406	5343	9.22	449.23	2278	7	445.64	5615	8.92	449.83
1423.5	Grand Blvd.								
1441	5343	8.5	450.3	2278	11.77	443.68	5615	7.93	451.23
2488	5343	6.1	451.77	2278	6.57	447.98	5615	5.58	452.48
4197	5005	10.97	459.4	2158	8.85	457.79	5172	11.13	459.45
4783	5005	4.48	467.32	2158	6.26	462.09	5172	4.5	467.54
4817	Reick Rd.								
4851	5005	6	468.52	2158	6.4	462.95	5172	5.96	468.66
5162	5005	4.42	468.91	2158	4.74	463.71	5172	4.48	469.03
5568	5005	5.32	469.19	2158	5.32	464.55	5172	5.39	469.31
6188	5005	6.8	470.05	2158	6.3	466.33	5172	6.78	470.19
7023	4110	5.87	472.28	1796	4.86	469.17	4239	5.91	472.42
7230	4110	8.43	475	1796	11.21	473.04	4239	8.68	475.01
7277	Fox Cove Lane								
7324	4110	4.19	480.03	1796	6.97	475.53	4239	4.22	480.09
8395	4110	3.15	481.1	1796	4.26	479.11	4239	3.16	481.17
10002	4110	8.14	487.05	1796	8.06	485.35	4239	8.15	487.12
11210	1877	5.69	493.04	865	4.39	491.88	1901	5.73	493.06
12007	1877	8.53	496.35	865	5.73	494.83	1901	8.59	496.38
12082	Loop 323								
12157	1877	5.46	499.28	865	4.5	495.49	1901	5.46	499.39
13292	919	4.5	506.73	434	3.99	504.9	941	4.56	506.75
13633	919	3.81	508.54	434	3.57	506.81	941	3.84	508.58
13654	Woodland Hills Dr.								
13675	919	4.45	508.87	434	4.27	506.98	941	4.49	508.91
13725	919	4.75	508.99	434	3.63	507.18	941	4.81	509.02

Table 15 – West Mud Tributary A-1 Creek Results Summary

West Mud Creek Tributary M-A.1									
	100 Year Existing			2 Year Ultimate			100 Year Ultimate		
Station	Q (cfs)	V (ft/s)	WSE (ft)	Q (cfs)	V (ft/s)	WSE (ft)	Q (cfs)	V (ft/s)	WSE (ft)
300	1332	4.42	471.14	568	4.83	467.97	1350	4.35	471.27
537	1332	9.98	475.16	568	8.19	473.65	1350	10.03	475.18
568.5	Rice Rd								
600	1332	5.31	478.33	568	5.82	475.46	1350	5.34	478.35
2200	1332	8.46	487.9	568	6.47	487.08	1350	8.47	487.92
2980	737	6.09	491.67	343	8.8	488.68	748	6.14	491.67
3002.5	Charleston								
3025	737	6.02	492.27	343	7.61	489.41	748	6.08	492.28
3150	737	3.83	492.81	343	4.81	490.87	748	3.86	492.82

Table 16 – West Mud Tributary A-2 Creek Results Summary

West Mud Creek Tributary M-A.2									
	100 Year Existing			2 Year Ultimate			100 Year Ultimate		
Station	Q (cfs)	V (ft/s)	WSE (ft)	Q (cfs)	V (ft/s)	WSE (ft)	Q (cfs)	V (ft/s)	WSE (ft)
400	1747	10.91	490	725	9.55	487.64	1784	10.94	490.06
833	1747	11.96	493.46	725	7.27	491.21	1784	12.11	493.53
986	Culvert								
1139	1747	8.09	497.13	725	6.63	491.92	1784	8.07	497.38
1477	1747	4	498.23	725	13.26	494.88	1784	3.84	498.47
1525	Jacksonville Highway								
1573	1747	1.87	500.79	725	8.23	498.66	1784	1.88	500.85
2770	564	5.35	506.48	248	10.21	503.09	582	5.41	506.49
3085	Loop 323								
3400	564	1.3	513.88	248	5.87	510.46	582	1.33	513.91
4500	564	4.85	524.74	248	4.28	524.27	582	4.87	524.77
6032	564	6.27	531.74	248	4.51	530.89	582	6.35	531.78

Tributary MC of West Mud Creek is located on the west side of the main channel just south of the intersection of Donnybrook Ave. and Loop 323. Tributary MC has 12 roadways that cross the creek, and all 12 of the roadways are overtopped by the 100-year ultimate water surface elevation. Table 17 shows the roadways that cross Tributary MC and the amount of water that is over the roadway. Significant flooding occurs along Tributary MC. There are approximately 82 structures that are located within the 100-year ultimate water surface. The major flooding areas along tributary MC are located just upstream of Broadway with nine structures flooding; between FM 2493 and Robertson with 19 structures flooding; and upstream of Camellia with 14 structures within the floodplain.

Table 17 – Tributary MC of West Mud Creek Roadway Overtopping

Roadway	Cross Section	Depth of Overtopping (ft)
Loop 323	2383.5	0.29
Donnybrook	3620	2.9
Broadway	5340	2.44
Old Bullard	6762.5	4.45
Amherst	8390	2.41
Whittle	9032.5	1.78
Fair	9285	1.74
Green	10260	2.66
FM 2493	11385	4.52
Robertson	12350	2.65
Camellia	13032.5	4.46
Azalea	13417.5	2.72

Tributary MC-1 is located on the east side of Tributary C just north of the intersection of Donneybrook Ave. and Loop 323. There are three roadway that cross the creek all of which are overtopped by the 100-year ultimate water surface. Table 18 shows the roadway that cross Tributary MC-1 and the amount of water over the roadway. Eleven structures along Tributary MC-1 are located inside the 100-year ultimate floodplain just upstream of New Copeland Road.

Table 18 – Tributary MC-1 of West Mud Creek Roadway Overtopping

Roadway	Cross Section	Depth of Overtopping (ft)
New Copeland	395	0.49
Shephard	1717.5	1.73
Shannon	2752.5	2.11

The following Table 19 thorough Table 21 show a summary of the modeling results for Tributary MC of West Mud Creek and its tributaries. These tables show a summary of the existing 100-year, 2-year, and the 100-year ultimate water surface elevations. The tables have been condensed for this report; the full summary table can be found in Appendix C.1. For detailed mapping of the existing 100-year and 100-year ultimate water surface elevations and cross section locations see Appendix D.1. Profile of the creek can be seen in Appendix E.1

Table 19 – West Mud Tributary M-C Creek Results Summary

West Mud Creek Tributary M-C									
	100 Year Existing			2 Year Ultimate			100 Year Ultimate		
Station	Q (cfs)	V (ft/s)	WSE (ft)	Q (cfs)	V (ft/s)	WSE (ft)	Q (cfs)	V (ft/s)	WSE (ft)
300	7604	5.46	477.23	3524	6.8	472.84	7647	5.49	477.23
2312	7483	7.91	485.09	3492	5.18	482.23	7519	7.93	485.11
2383.5	Loop 323								
2455	7483	6.09	487.76	3492	5.39	483.06	7519	6.09	487.78
3585	5956	4.46	488.84	2796	3.76	484.82	5989	4.47	488.86
3620	Donnybrook								
3655	5956	4.48	489.03	2796	3.58	485.36	5989	4.49	489.05
5280	5956	6.79	496.73	2796	7.2	491.48	5989	6.77	496.77
5340	South Broadway								
5400	5956	6.6	498.19	2796	5.94	493.29	5989	6.61	498.23
6710	5777	3.33	499.88	2551	4.85	495.74	5817	3.33	499.91
6762.5	Old Bullard								
6815	5777	3.64	499.91	2551	4.16	496.56	5817	3.63	499.95
7785	5777	8.26	501.75	2551	6.85	498.5	5817	8.26	501.79
8055	3777	7.89	502.49	1630	5.38	499.34	3811	7.91	502.53
8087.5	Buckingham Pl.								
8120	3777	5.98	503.28	1630	4.54	500.22	3811	6	503.31
8355	3777	6.16	504.05	1630	4.83	500.77	3811	6.18	504.08
8390	Beechwood Dr.								
8425	3777	6.57	505.56	1630	5.34	501.86	3811	6.6	505.58
9000	3376	13.62	505.88	1485	11.21	502.76	3396	13.32	506.04
9032.5	Whittle St.								
9065	3376	6.71	509.04	1485	6.01	506.19	3396	6.7	509.07
9250	3376	6.52	509.82	1485	6.46	506.75	3396	6.51	509.85
9285	Fair Lane								
9320	3376	8.23	510.18	1485	5.89	507.78	3396	8.23	510.2
10210	3548	4.8	513.55	1556	5	509.93	3564	4.81	513.56
10235	Green								
10260	3548	5.51	513.72	1556	4.92	511.27	3564	5.52	513.73
10470	3548	7.89	513.91	1556	4.91	511.83	3564	7.91	513.92
10542.5	Southbound Sunnybrook								
10615	3548	5.57	515.49	1556	3.45	513.78	3564	5.56	515.51
11330	2564	12.51	516.19	1158	9.75	514.09	2571	12.53	516.19
11385	Jacksonville Highway								
11440	2564	3.67	529.86	1158	3.76	520.89	2571	3.68	529.85
12315	2796	4.23	529.98	1332	8.02	521.75	2814	4.27	529.97
12350	Robertson								
12385	2796	3.9	530.28	1332	3.01	528.36	2814	3.92	530.28
13000	2796	3.49	530.4	1332	3.02	528.45	2814	3.51	530.41
13032.5	Camellia St.								
13065	2796	2.79	530.5	1332	2.46	528.56	2814	2.8	530.51
13390	2796	4.46	530.48	1332	4.46	528.51	2814	4.47	530.49
13417.5	Azalea								
13445	2796	4.27	530.57	1332	3.83	528.83	2814	4.28	530.58
13500	2796	4.36	530.58	1332	3.81	528.84	2814	4.38	530.59

Table 20 – West Mud Tributary M-C.1 Creek Results Summary

West Mud Creek Tributary M-C.1									
	100 Year Existing			2 Year Ultimate			100 Year Ultimate		
Station	Q (cfs)	V (ft/s)	WSE (ft)	Q (cfs)	V (ft/s)	WSE (ft)	Q (cfs)	V (ft/s)	WSE (ft)
300	2111	4.22	488.32	1006	3.65	483.67	2111	4.2	488.34
360	2111	6.45	488.16	1006	5.61	483.57	2111	6.43	488.18
395	New Copeland Rd.								
430	2111	5.33	490.69	1006	5.7	483.93	2111	5.32	490.71
1100	1086	3.84	491.23	527	9.3	487.2	1086	3.82	491.25
1685	1086	6.33	495.55	527	5.15	493.79	1086	6.33	495.55
1717.5	Shepherd Ln.								
1750	1086	5.74	499.46	527	3.52	498.66	1086	5.74	499.46
2715	868	4.68	508.88	432	5.38	507.3	868	4.68	508.88
2752.5	Shannon Dr.								
2790	868	5.55	509.95	432	3.78	509.34	868	5.55	509.95

Table 21 – West Mud Tributary M-C.2 Creek Results Summary

West Mud Creek Tributary M-C.2									
	100 Year Existing			2 Year Ultimate			100 Year Ultimate		
Station	Q (cfs)	V (ft/s)	WSE (ft)	Q (cfs)	V (ft/s)	WSE (ft)	Q (cfs)	V (ft/s)	WSE (ft)
70	1879	3.55	502	856	2.62	498.97	1880	3.54	502.03
85	1879	5.49	501.88	856	4.76	498.83	1880	5.46	501.91
125	Sunnybrook								
165	1879	6.31	503.2	856	6.26	500.38	1880	6.3	503.21
415	1879	8.37	503.79	856	6.52	501.34	1880	8.31	503.82
450	Old Bullard Rd.								
485	1879	4.29	506.02	856	2.81	504.51	1880	4.29	506.02
1760	1879	5.67	513.56	856	4.26	510.66	1880	5.68	513.57
1800	Fair Lane								
1840	1879	4.53	516.88	856	3.99	513.14	1880	4.55	516.87
2995	1704	10.47	524.23	797	8.83	522.56	1704	10.47	524.23

Tributary B of West Mud Creek is located on the east side of the main channel just south of the intersection of Rieck Road and New Copeland Road. Tributary B has three roadways that cross the creek channel, all of which are overtopped by the 100-year ultimate water surface. Table 22 shows the roadway that cross the creek and the amount of water that overtops. Significant flooding occurs along Tributary B with 12 structures located inside the floodplain. Ten of those 12 structures are just upstream of Rieck Road on the south side of the creek along Quail Creek. The culvert structure at New Copeland causes significant flooding on the upstream side of the roadway putting two structures inside the floodplain.

Table 22 – Tributary B of West Mud Creek Roadway Overtopping

Roadway	Cross Section	Depth of Overtopping (ft)
Rieck	437.5	3.54
New Copeland	3120	1.08
Paluxy	6940	1.46

The following Table 23 shows a summary of the modeling results for Tributary B of West Mud Creek. This table shows a summary of the existing 100-year, 2-year and the 100-year ultimate water surface elevations. The tables have been condensed for this report; the full summary table can be found in Appendix C.1. For detailed mapping of the existing100-year and 100-year ultimate water surface elevations and cross section locations see Appendix D.1. Profile of the creek can be seen in Appendix E.1

Table 23 – West Mud Tributary B Creek Results Summary

West Mud Creek Tributary B									
	100 Year Existing			2 Year Ultimate			100 Year Ultimate		
Station	Q (cfs)	V (ft/s)	WSE (ft)	Q (cfs)	V (ft/s)	WSE (ft)	Q (cfs)	V (ft/s)	WSE (ft)
200	3620	3.97	466.27	1466	4.13	461.56	3760	4.09	466.32
390	3620	7.97	466.26	1466	12.7	461.1	3760	8.18	466.31
437.5	Reick Rd.								
485	3620	6.01	471.63	1466	7.12	464.47	3760	6.13	471.8
1915	3615	5.17	472.32	1549	9.14	466.04	3723	5.17	472.51
3025	3122	12.06	474.89	1342	9.91	472.49	3226	12.15	475
3075	3122	12.3	478.08	1342	9.96	475.66	3226	12.38	478.21
3120	New Copeland Dr.								
3165	3122	4.03	481.7	1342	3.56	479.52	3226	4.11	481.74
4400	3112	8.95	484.52	1399	7.04	483.05	3198	9.05	484.56
6500	3112	5.99	498.33	1399	5.62	496.29	3198	6.02	498.4
6880	3439	12.92	499.72	1564	5.97	499.61	3517	13.21	499.72
6940	Paluxy Dr.								
7000	2967	3.63	504.42	1329	2.22	503.43	3021	3.66	504.46
7410	2967	3.26	504.58	1329	1.99	503.49	3021	3.29	504.62

Tributary M-11 of West Mud Creek is located on the southern side of the City of Tyler on the west side of the main channel. There are three roadways that cross the channel, all of which are overtopped by the 100-year ultimate water surface. Table 24 shows the roadway that cross the channel and the amount of water overtopping. Significant flooding occurs upstream of Woodlands drive. There are 24 structures within the study area that are located inside the floodplain starting at station 108+00 to the limit of this study station 120+00.

Table 24 – Tributary M-11 of West Mud Creek Roadway Overtopping

Roadway	Cross Section	Depth of Overtopping (ft)
Holly Creek	8435	2.76
Pinehurst	8891.5	2.57
Woodlands	10072.5	1.8

The following Table 25 shows a summary of the modeling results for Tributary M-11 of West Mud Creek. This table shows a summary of the existing 100-year, 2-year and the 100-year ultimate water surface elevations. The tables have been condensed for this report; the full summary table can be found in Appendix C.1. For detailed mapping of the existing100-year and 100-year ultimate water surface elevations and cross section locations see Appendix D.1. Profile of the creek can be seen in Appendix E.1

Table 25 – West Mud Tributary M-11 Creek Results Summary

West Mud Creek Tributary M-11									
	100 Year Existing			2 Year Ultimate			100 Year Ultimate		
Station	Q (cfs)	V (ft/s)	WSE (ft)	Q (cfs)	V (ft/s)	WSE (ft)	Q (cfs)	V (ft/s)	WSE (ft)
989	4186	2.39	418.62	1533	8.13	415.51	4348	2.41	418.7
3200	4186	5.54	430.55	1533	3.72	428.84	4348	5.61	430.63
4302	2995	3.94	435.33	1211	5.35	432.97	3095	3.92	435.44
6310	2995	4.47	444.34	1211	3.8	442.36	3095	4.5	444.43
7953	2995	11.13	455.44	1211	7.19	454.32	3095	11.29	455.49
8405	2995	12.34	458.34	1211	7.53	456.44	3095	12.64	458.39
8435	Holly Creek Dr.								
8465	2995	4.31	461.67	1211	2.25	460.79	3095	4.36	461.76
8842	1691	3.67	461.85	726	2.9	460.78	1726	3.65	461.94
8891.5	Pinehearst St.								
8941	1691	3.01	463.37	726	1.64	462.28	1726	3.05	463.41
10025	1691	10.78	463.84	726	8.42	462.13	1726	10.87	463.89
10072.5	Woodlands Dr.								
10120	1691	4.76	470.7	726	3.81	469.77	1726	4.86	470.7
12020	1691	3.23	478.99	726	2.93	477.72	1726	3.24	479.02

Henshaw Creek

Henshaw Creek's location on the southwestern side of the City just outside of most urban areas does pose a moderate risk of flooding problems for building structures. There are currently 10 structures located inside the 100-year ultimate floodplain. Five of those are located at approximately station 52+00, two at approximately 214+00, two at 222+00 and one at 256+00. There are six roadways that cross the creek, and all six are overtopped by the 100-year ultimate water surface elevation. One of the roadways, State Highway 69 South, would be considered a major artery in and out of the City of Tyler. Table 26 shows the roadway crossing and the depth of water that overtops the roadway.

Table 26 – Henshaw Creek Roadway Overtopping

Roadway	Cross Section	Depth of Overtopping (ft)
St Hwy 69 s.	4884.5	1.49
FM 346	5241.5	2.35
CR 132	16822.5	4.18
CR 2818	21316	2.76
CR 165	22002	1.69

The following Table 27 shows a summary of the modeling results for Henshaw Creek. This table shows a summary of the existing 100-year, 2-year and the 100-year ultimate water surface elevations. The tables have been condensed for this report; the full summary table can be found in Appendix C.1. For detailed mapping of the existing100-year and 100-year ultimate water surface elevations and cross section locations see Appendix D.1. Profile of the creek can be seen in Appendix E.1

Table 27 – Henshaw Creek Results Summary

	Henshaw Creek								
	100 Year Existing			2 Year Ultimate			100 Year Ultimate		
Station	Q (cfs)	V (ft/s)	WSE (ft)	Q (cfs)	V (ft/s)	WSE (ft)	Q (cfs)	V (ft/s)	WSE (ft)
2031	8981	7.36	381.52	2235	8.28	377.47	9574	7.54	381.7
4800	8981	5.31	388.01	2235	10.69	383.79	9574	5.66	388.01
4884.5	South Broadway								
4969	8981	1.76	392.56	2235	5.75	387.03	9574	1.86	392.64
5206	8981	1.91	392.63	2235	4.62	388.08	9574	2	392.72
5241.5	F.M. 346								
5277	8257	1.94	392.7	2088	4.24	388.39	8799	2.03	392.78
7500	8257	4.65	393.64	2088	5.22	391.61	8799	4.68	393.77
8939	7785	11.03	401.29	2113	12.76	398.19	8306	11.31	401.38
8964	C.R. 137								
8989	7785	5.1	404.23	2113	2.25	402.54	8306	5.29	404.34
11148	7785	3.41	408.97	2113	2.94	406.22	8306	3.45	409.16
13581	7684	9.73	419.56	2450	6.78	417.2	8179	9.91	419.72
15509	7028	5.37	426.99	1910	4.71	423.06	7532	5.46	427.25
16810	6841	8.36	431.75	2021	13.18	428.05	7334	8.48	431.98
16822.5	Cox Rd., C.R. 132								
16835	6841	7.16	431.98	2021	2.98	430.7	7334	6.66	432.61
18504	6676	8.16	439.57	1848	5.49	435.94	7157	8.49	439.7
20197	6335	4.77	444.27	1606	3.19	440.26	6786	4.87	444.53
21279	5641	6	447.61	1411	3.79	444.35	6118	6.14	447.88
21316	C.R. 2813								
21353	5641	4.99	449.26	1411	1.86	447.49	6118	5.22	449.43
21973	5641	6.58	452.82	1411	4.22	450.41	6118	6.87	452.97
22002	F.M. 2493								
22031	5641	4.81	454.09	1411	3.88	451.25	6118	5.11	454.19
23983	5059	7.62	456.76	1413	7.38	454.95	5499	7.49	456.99
25966	4494	8.39	466.95	1525	6.35	464.64	4952	8.63	467.18
26415	4494	5.18	469.47	1525	8.3	466.62	4952	5.32	469.74
26438	C.R. 165								
26461	4494	5.73	470.01	1525	2.78	468.57	4952	6.07	470.17
26471	4494	5.92	470.04	1525	2.96	468.57	4952	6.25	470.2

<u>Shackelford Creek</u>

Shackelford Creek's location on the southeastern side of the city just outside of most urban areas does pose a moderate risk of flooding problem for building structures. There is currently no major structure located inside the 100-year ultimate floodplain. There are four roadways that cross the creek, and all four are overtopped by the 100-year ultimate water surface elevation. Table 28 shows the roadway crossing and the depth of water that overtops the roadway.

Table 28 – Shackelford Creek Roadway Overtopping

Roadway	Cross Section	Depth of Overtopping (ft)
FM 346	6849	2.56
Skidmore	23245	4.81
CR 110	28120	4.02
Paluxy	34181	2.79

The following Table 29 shows a summary of the modeling results for Shackelford Creek. This table shows a summary of the existing 100-year, 2-year and the 100-year ultimate water surface elevations. The tables have been condensed for this report; the full summary table can be found in Appendix C.1. For detailed mapping of the existing100-year and 100-year ultimate water surface elevations and cross section locations see Appendix D.1. Profile of the creek can be seen in Appendix E.1

Table 29 – Shackelford Creek Results Summary

Shackelford Creek									
	100 Year Existing			2 Year Ultimate			100 Year Ultimate		
Station	Q (cfs)	V (ft/s)	WSE (ft)	Q (cfs)	V (ft/s)	WSE (ft)	Q (cfs)	V (ft/s)	WSE (ft)
1986	13955	15.56	384.1	3318	10.49	381.42	14669	15.8	384.22
3698	13955	5.85	390.48	3318	3.56	386.04	14669	5.96	390.68
6406	13955	6.29	394.51	3318	5.92	390.46	14669	6.34	394.65
6810	13955	8.7	395.36	3318	8.87	392.31	14669	8.86	395.5
6849	F.M. 346								
6888	13955	6.55	397.41	3318	12.91	391.49	14669	6.64	397.56
8297	13955	2.88	398.9	3318	1.6	394.92	14669	2.94	399.1
10195	13725	9.42	402.16	3505	6.27	398.8	14319	9.55	402.31
12154	13725	6.14	409.56	3505	4.47	405.38	14319	6.2	409.74
14009	13725	7.1	413.72	3505	4.55	410.1	14319	7.19	413.89
15495	13837	5.25	417.04	3753	4.6	413.62	14341	5.27	417.18
16992	13837	8.06	420.54	3753	5.14	417.65	14341	8.16	420.65
19493	13837	8.03	425.53	3753	5.74	422.13	14341	8.11	425.66
21200	13837	5.78	430.35	3753	3.94	426.3	14341	5.86	430.5
23005	11952	10.82	434.98	3494	7.52	431.66	12973	11.18	435.26
23219	11952	10.9	438.48	3494	12.82	433.18	12973	11.25	438.81
23245	Skidmore Lane								
23271	11952	5.83	442.33	3494	2.79	439.38	12973	6.04	442.6
25317	11952	8.39	444.89	3494	5.41	440.51	12973	8.61	445.27
27609	10469	4.5	449.76	3141	6.08	446.43	11009	4.51	450.01
28086	10469	10.96	451.25	3141	2.55	452.55	11009	11.2	451.39
28120	Cumberland Rd. (C.R. 110)								
28154	10469	5.3	453.94	3141	2.06	452.58	11009	5.5	454.03
29000	10469	10.72	454.78	3141	4.5	452.74	11009	11.07	454.91
30981	7036	8.23	459.2	2891	7.59	455.58	7405	8.29	459.43
32992	7036	9.63	469.16	2891	9.99	465.82	7405	9.6	469.41
34151	4714	7.33	477.01	1962	17.15	474.95	4972	7.73	477.01
34181	Paluxy Dr. (F.M. 756)								
34211	4714	4.89	479.67	1962	2.12	479.5	4972	5	479.79
35815	4714	13.11	488.77	1962	9.89	486.85	4972	13.25	488.92
37395	4714	8.47	496.46	1962	6.86	494.1	4972	8.58	496.64
38114	4714	6.57	501.29	1962	6.11	498.94	4972	6.63	501.45

Black Fork Creek and Tributaries

Black Fork and all of its tributaries have an overall drainage area of 50.31 square miles including the Willow Creeks basin. The detailed hydraulic modeling for Black Fork Creek has approximately 15.15 miles of creek for the main channel and 13.57 miles of tributaries. Table 30 shows the length of each stream in the detailed analysis of the Black Fork Creek Basin.

Table 30 – Black Fork Creek Basin Stream Lengths

Creek Label	Length (mi)	Creek Label	Length (mi)
Black Fork	15.15	Willow	5.00
BF-1	1.42	BF-M1	0.82
BF-D	2.71	BF-D1	1.04
BF-D2	0.62	BF-D3	0.71
BF-D4	0.85	BF-D5	0.40

Most of Black Fork Creek and the lower portion of Willow Creek would be classified as rural stream. The upper portion of Black Fork Creek and the tributaries in this study would be considered Urbanized channel. Lower Black Fork Creek much like West Mud creek has a very wide floodplain area, with some areas measuring over a half-mile wide. The vegetation in the lower portion of the creek would also be considered very dense with mannings n-values approaching 0.10 in the overbanks.

The upper portion of Black Fork Creek, Upper Willow and the tributaries studied for Black Fork Creek would be considered urban stream with combinations of natural channel, channeled sections and concrete lined sections. Mannings n-values in the overbanks for upper Black Fork and its tributaries vary greatly from as high as 0.10 in very dense vegetated areas to as low as 0.035 in manicured park areas.

Black Fork Creek is located on the northern side of the City of Tyler and is partially located in a rural setting and the upper portion is located in a urban portion of the city. There are 15 roadways that cross over the main channel of Black Fork Creek, 12 of which are over topped by the 100-year ultimate water surface. Table 31 shows all the roadways that cross Black Fork Creek and the amount of water that overtops the roadway. The lower portion of the creek does not pose a major flooding problem for structures. Currently there are two structures located in the floodplain at station 302+00, three structures just upstream of loop 323 and eight structures located at Hwy 271. Major flooding problems start at the Missouri Pacific Railroad with 35 structures located in the floodplain from the Railroad to Commerce Street. Upstream of Erwin Street there are14 structures located within the floodplain. At Front Street there are 11 structures located inside the floodplain. Upstream of Golden Street there are currently 10 structures in the floodplain along Pinkerton starting at station 822+00. Upstream of Fifth Street there are four more structures located inside the floodplain. Table 32 shows a summary of the modeling results for Black Fork Creek. The table shows a summary of the existing 100-year, 2-year and the 100-year ultimate water surface elevations. The tables have been condensed for this report; the full summary table can be found in Appendix C.2. For detailed mapping of the existing100-year and 100-year ultimate water surface elevations and cross section locations see Appendix D.2. Profile of the creek can be seen in Appendix E.2

Table 31 – Black Fork Creek Roadway Overtopping

Roadway	Cross Section	Depth of Overtopping (ft)
Baron Verner	14274	5.42
CR 46	28024	6.45
HWY 110	39085	6.78
CR 427	44281	6.89
Mineola Hwy	48119	4.98
Loop 323	55632	1.93
Broadway	61613	Not Overtopped
FM 14	65109	Not Overtopped
Gentry	67702	4.01
Railroad	70607	Not Overtopped
Commerce	76262.5	4.02
Erwin	77933.5	3.51
Front	79811.5	2.97
Golden	81569.5	2.17
Fifth	86223.5	3.41

Table 32 – Black Fork Creek Results Summary

Black Fork Creek									
	100 Year Existing			2 Year Ultimate			100 Year Ultimate		
Station	Q (cfs)	V (ft/s)	WSE (ft)	Q (cfs)	V (ft/s)	WSE (ft)	Q (cfs)	V (ft/s)	WSE (ft)
10149	30763	1.92	380.00	7516	0.47	380.00	34119	2.13	380.00
13786	30763	8.27	385.65	7516	3.94	381.58	34119	8.72	386.06
14261	30763	13.73	389.85	7516	12.30	382.52	34119	14.25	390.14
14274	Baron Verner Rd.								
14287	30763	10.29	390.97	7514	6.86	386.04	34119	10.76	391.27
17454	30894	3.38	393.61	7516	2.71	387.81	34174	3.52	394.10
19255	30894	3.41	395.33	7516	2.64	390.02	34174	3.44	395.84
22563	31557	8.70	400.09	7645	5.52	394.91	34281	8.94	400.54
25149	31498	8.14	403.90	7553	4.44	398.47	34204	8.42	404.36
28008	30233	17.41	407.93	7387	9.27	402.58	32388	17.41	408.30
28024	C.R. 46								
28040	30233	12.66	409.33	7387	8.93	403.30	32388	13.00	409.60
32119	29902	2.37	412.41	7370	1.40	405.22	31992	2.45	412.83
34114	29902	3.30	412.87	7370	2.78	405.72	31992	3.38	413.30
38191	29868	6.03	417.11	7677	4.84	411.91	31878	6.13	417.49
39061	29843	15.44	418.86	7684	7.60	414.78	31842	15.55	419.20
39085.5	Highway 110								
39110	29843	15.22	419.13	7684	6.59	415.32	31842	9.92	423.14
42604	23880	2.52	424.57	6861	2.22	417.47	25011	2.33	425.85
44266	23880	11.41	425.09	6861	12.22	420.07	25011	8.95	426.35
44281	C.R. 427								
44296	23880	7.71	425.82	6861	2.58	425.06	25011	6.67	426.89
48047	24027	6.24	429.39	7018	4.21	426.18	25114	6.02	429.86
48097	24085	10.16	429.06	7164	5.91	426.05	25203	9.54	429.59
48119	Mineola Highway								
48141	24085	10.51	429.97	7164	5.50	426.68	25203	10.40	430.27
48169	24085	11.84	429.80	7164	5.45	426.73	25203	11.70	430.11
48193	Mineola Highway								
48217	24085	9.68	431.32	7164	5.61	426.97	25203	9.79	431.50
51561	22247	2.68	433.21	7206	2.38	428.27	23393	2.74	433.41
53595	18981	4.69	434.45	6349	3.34	430.81	19505	4.68	434.63
55612	18567	16.98	436.00	6341	6.16	435.45	19036	16.19	436.67
55632	W NW Loop 323								
55652	18567	10.25	438.95	6341	4.86	436.14	19036	10.33	439.17
55701	18567	10.72	439.02	6341	4.76	436.22	19036	10.73	439.26
55734	W NW Loop 323								
55767	18567	9.29	439.67	6341	4.53	436.67	19036	9.18	439.93
59124	18485	6.14	445.59	6505	6.27	441.87	18884	6.11	445.70
60632	18350	4.18	448.31	6487	3.21	444.81	18713	4.20	448.40
61587	18350	6.29	448.70	6487	5.43	445.14	18713	6.31	448.79
61613	N. Broadway Ave.								
61639	18350	6.38	448.81	6487	5.21	445.28	18713	6.40	448.89
65093	16179	10.11	458.54	6457	5.53	454.76	16904	10.41	458.73
65109	F.M. 14								
65125	16154	9.27	458.84	6461	5.08	454.86	16879	9.54	459.05

Black Fork Creek (cont.)									
	100 Year Existing			2 Year Ultimate			100 Year Ultimate		
Station	Q (cfs)	V (ft/s)	WSE (ft)	Q (cfs)	V (ft/s)	WSE (ft)	Q (cfs)	V (ft/s)	WSE (ft)
67635	16154	5.38	467.59	6461	4.33	464.64	16879	5.47	467.74
67702	E. Gentry Pkwy.								
67769	16154	5.26	467.76	6461	3.60	465.53	16879	5.35	467.89
70530	10122	18.89	467.04	4552	9.37	465.91	10247	18.89	467.20
70568.5	Railroad								
70607	10122	11.28	478.88	4552	9.10	469.63	10247	11.19	479.30
73721	7545	1.94	481.11	3232	4.06	471.21	7576	1.86	481.50
75612	7545	2.94	481.21	3232	4.59	474.69	7576	2.80	481.59
76234	10919	10.84	482.33	3701	14.63	474.15	10890	10.84	482.32
76262.5	Commerce St.								
76291	10919	11.91	483.25	3701	15.66	477.45	10890	11.90	483.25
77890	10330	12.18	489.28	3570	17.41	482.98	10293	12.16	489.27
77933.5	E. Erwin St.								
77977	10330	6.64	490.19	3570	4.12	488.18	10293	6.65	490.17
79433	10330	9.09	492.73	3570	7.33	489.44	10293	9.08	492.71
79738	10330	12.11	493.76	3570	7.53	490.31	10293	12.11	493.74
79811.5	E. Front St.								
79885	10330	7.19	495.57	3570	8.53	492.01	10293	7.17	495.56
81219	6922	5.59	497.40	3060	4.54	493.96	6948	5.62	497.39
81534	6922	9.94	497.62	3060	7.83	494.28	6948	9.99	497.61
81569.5	Golden Rd.								
81605	6922	10.51	497.63	3060	8.48	494.82	6948	10.58	497.61
82911	5614	4.71	503.86	2506	4.15	499.79	5627	4.71	503.88
86122	5614	14.17	514.44	2506	9.69	511.19	5627	14.18	514.45
86172	5614	10.58	517.42	2506	12.09	511.68	5627	10.55	517.45
86223.5	E. Fifth St.								
86275	5614	11.67	518.87	2506	11.59	515.16	5627	11.62	518.91
87423	3534	3.22	521.67	1521	2.47	518.53	3540	3.22	521.68
88613	3023	6.85	525.79	1381	6.68	522.83	3028	6.85	525.79
89972	1589	6.32	531.26	737	7.00	529.08	1591	6.32	531.26

Tributary BF-M1 is located on the north side of the City of Tyler on the south side of the main channel. There are two roadways that cross the channel and both are over topped the 100-year ultimate water surface. Table 33 shows the roadways and the amount of water that overtops the roadway. BF-M1 does not pose any major flooding problem. There are currently only four structures located inside the floodplain, starting at station 33+00. Table 34 shows a summary of the modeling results for Black Fork tributary BF-M1. The table shows a summary of the existing 100-year, 2-year and the 100-year ultimate water surface elevations. The tables have been condensed for this report; the full summary table can be found in Appendix C.2. For detailed mapping of the existing100-year and 100-year ultimate water surface elevations and cross section locations see Appendix D.2. Profile of the creek can be seen in Appendix E.2

Table 33 – Tributary BF-M1 of Black Fork Creek Roadway Overtopping

Roadway	Cross Section	Depth of Overtopping (ft)
Porter	2074	2.12
Devine	2855	2.59

Table 34 – Black Fork Tributary BF-M.1 Creek Results Summary

Black Fork Creek Tributary BF-M-1									
	100 Year Existing			2 Year Ultimate			100 Year Ultimate		
Station	Q (cfs)	V (ft/s)	WSE (ft)	Q (cfs)	V (ft/s)	WSE (ft)	Q (cfs)	V (ft/s)	WSE (ft)
500	2471	3.85	496.4	1172	4.34	493.47	2477	3.87	496.39
2030	2471	14	506.66	1172	10.93	504.27	2477	14	506.67
2074	Porter Dr. / Don St.								
2118	2471	7.07	509.36	1172	5.52	507.7	2477	7.07	509.37
2830	2471	5.6	515.6	1172	9.53	512.45	2477	5.59	515.61
2855	Devine St.								
2880	2471	4.14	516.25	1172	2.78	514.39	2477	4.14	516.26
3365	2471	5.39	518.57	1172	16.76	516.52	2477	5.4	518.57
3387.5	Culvert								
3410	2471	6.95	519.87	1172	2.51	520.85	2477	6.96	519.88
4330	2471	4.71	523.05	1172	2.81	521.62	2477	4.72	523.06

Tributary D of Black Fork Creek currently has 15 roadways that cross the channel, 12 of which are overtopped by the 100-year ultimate water surface. Table 35 shows the roadways that cross Tributary D and the amount of water that overtops the roadway. Significant flooding occurs along Tributary D with 11 structures upstream of Commerce Street, five structures upstream of Houston

Street and 10 structures downstream of Donnybrook inside the 100-year ultimate floodplain. Table 36 shows a summary of the modeling results for Black Fork Creek Tributary D. The table shows a summary of the existing 100-year, 2-year and the 100-year ultimate water surface elevations. The tables have been condensed for this report; the full summary table can be found in Appendix C.2. For detailed mapping of the existing100-year and 100-year ultimate water surface elevations and cross section locations see Appendix D.2. Profile of the creek can be seen in Appendix E.2

Table 35 – Tributary D of Black Fork Creek Roadway Overtopping

Roadway	Cross Section	Depth of Overtopping (ft)
Laurel St.	1457	2.17
Railroad	3013	Not Overtopped
Commerce St.	5434.5	3.57
Fleischel Ave.	5790	4.9
W. Locust St	6982	1.69
Saunders Ave.	7605	2.41
High Ave.	7850	2.85
E. Earle St.	9261.5	1.01
E. Front St.	9710	Not Overtopped
crossover	10773	2.54
E. Houston St.	11121	2.19
Railroad	11496	Not Overtopped
Victory Dr.	12067	9.92
Victory Dr.	12327.5	8.59
Donnybrook	13143.5	0.12

Table 36 – Black Fork Tributary D Creek Results Summary

Black Fork Creek Tributary D									
	100 Year Existing			2 Year Ultimate			100 Year Ultimate		
Station	Q (cfs)	V (ft/s)	WSE (ft)	Q (cfs)	V (ft/s)	WSE (ft)	Q (cfs)	V (ft/s)	WSE (ft)
500	10433	4.7	468.59	4666	3.1	465.94	10485	4.63	468.74
1438	10433	14.03	469.36	4666	10.67	465.62	10485	14.1	469.36
1457	Laurel St.								
1476	10433	7.85	471.62	4666	6.76	467.67	10485	7.85	471.65
2899	6255	12.69	472.75	2996	7.79	469.83	6271	12.7	472.78
3013	Railroad								
3127	6255	11.06	483.44	2996	9.01	474.48	6271	11.04	483.53
5407	6955	8.83	485.6	3126	14.46	478.57	6976	8.63	485.7
5434.5	Commerce St.								
5462	6955	5.98	486.11	3126	4.76	483.35	6976	5.9	486.19
5690	6955	3.63	486.83	3126	2.36	483.81	6976	3.61	486.89
5790	Fleischel Ave / E. Oakwood St.								
5890	6955	4.13	486.86	3126	2.74	484.07	6976	4.1	486.94
6726	2175	2.54	488.07	1223	4.08	485.06	2179	2.51	488.12
6982	W. Locust St / E. Erwin St								
7238	2175	4.72	488.34	1223	4.05	487.07	2179	4.69	488.36
7569	2175	8.31	490.19	1223	16.37	489.57	2179	8.29	490.2
7605	Saunders Ave.								
7641	2175	4.61	491.96	1223	1.63	493.71	2179	4.57	492
7833	2175	3.1	492.22	1223	1.3	493.73	2179	3.09	492.25
7850	High Ave.								
7867	2175	3.48	493.17	1223	1.76	493.75	2179	3.49	493.17
9243	2175	9.69	496.85	1223	6.92	495.31	2179	9.7	496.86
9261.5	E. Earle St.								
9280	2175	5.02	498.67	1223	6.01	495.79	2179	5.01	498.68
9636	2175	6.6	501.25	1223	8.9	498.22	2179	6.61	501.26
9710	E. Front St.								
9784	2175	6.17	503.06	1223	7.97	499.65	2179	6.17	503.08
10741	3016	10.72	506.09	1356	14.28	502.39	3020	10.72	506.09
10773	crossover								
10805	3016	6.21	508	1356	4.42	506.36	3020	6.23	507.99
10896	3016	8.03	510.18	1356	11.33	505.79	3020	8.01	510.19
11121	E. Houston St.								
11346	3016	4.89	517.91	1356	6.86	514.37	3020	4.9	517.91
11461	3016	8.64	517.61	1356	4.95	514.97	3020	8.66	517.6
11496	Railroad								
11531	3016	5.26	526.33	1356	4.56	516.59	3020	5.26	526.34
12029	1818	1.02	526.82	798	6.43	516.74	1819	1.01	526.84
12067	Victory Dr.								
12105	1818	0.82	526.83	798	11.62	517.2	1819	0.82	526.85
12313	1818	0.89	526.84	798	2.48	519.61	1819	0.89	526.85
12327.5	Victory Dr.								
12342	1818	1.01	526.87	798	2.78	519.8	1819	1.01	526.85

Black Fork Creek Tributary D (cont)									
	100 Year Existing			2 Year Ultimate			100 Year Ultimate		
Station	Q (cfs)	V (ft/s)	WSE (ft)	Q (cfs)	V (ft/s)	WSE (ft)	Q (cfs)	V (ft/s)	WSE (ft)
13046	1818	5.18	527.15	798	13.94	522.64	1819	5.2	527.14
13143.5	Donnybrook Ave.								
13241	1818	8.8	529.47	798	4.47	529.23	1819	8.78	529.48
14005	2043	13.7	533.05	986	9.81	530.93	2043	13.7	533.05
14105	Culvert								
14205	2043	5.84	540.6	986	5.24	534.67	2043	5.84	540.6
14295	2043	2.82	541.13	986	8.38	535.35	2043	2.82	541.13

Tributary D.1 is located on the lower end of Tributary D of Black Fork Creek. There are eight structures that cross over the creek, all of which are overtopped by the 100-year ultimate water surface. Table 37 shows the roadways that are located along Tributary D.1 and the amount of water that is over the roadway. Currently there is some major flooding along tributary D.1 with 16 structures located inside the 100-year ultimate floodplain. Table 38 shows a summary of the modeling results for Black Fork Creek Tributary D.1. The table shows a summary of the existing 100-year, 2-year and the 100-year ultimate water surface elevations. The tables have been condensed for this report; the full summary table can be found in Appendix C.2. For detailed mapping of the existing100-year and 100-year ultimate water surface elevations and cross section locations see Appendix D.2. Profile of the creek can be seen in Appendix E.2

Table 37– Tributary D.1 of Black Fork Creek Roadway Overtopping

Roadway	Cross Section	Depth of Overtopping (ft)
Gentry	1863.5	2.34
Carlyle	3049	1.91
Chruch	3751.5	3.64
Fanin	4247.5	2.13
Railroad	4371.5	0.91
Spring	4646.5	11.25
Queen	4976	3
Broadway	5289	3

Table 38 – Black Fork Tributary D.1 Creek Results Summary

Black Fork Creek Tributary D.1									
	100 Year Existing			2 Year Ultimate			100 Year Ultimate		
Station	Q (cfs)	V (ft/s)	WSE (ft)	Q (cfs)	V (ft/s)	WSE (ft)	Q (cfs)	V (ft/s)	WSE (ft)
295	4329	2.81	474.41	1747	3.08	469.64	4367	2.82	473.44
1798	2786	7.73	478.96	1036	5.27	474.99	2815	7.76	479.01
1863.5	Gentry Parkway								
1929	2786	11.86	487.21	1036	8.12	484.9	2815	11.88	487.24
3015	2786	4.56	489.33	1036	10.01	485.56	2815	4.6	489.37
3049	Carlyle Ave.								
3083	2786	7.2	489.24	1036	3.94	487.57	2815	7.25	489.25
3725	2648	12.61	492.25	1268	13	487.69	2663	12.53	492.31
3751.5	Church Ave.								
3778	2648	4.26	494.24	1268	5.67	491.51	2663	4.23	494.27
4214	2203	8.57	494.62	1056	12.07	491.43	2215	8.53	494.65
4247.5	Fanin Ave.								
4281	2203	5.63	496.51	1056	3.87	495.55	2215	5.62	496.53
4343	2203	9.16	501.22	1056	15.43	494.09	2215	9.18	501.22
4371.5	Railroad								
4400	2203	1.01	510.97	1056	1.88	504.22	2215	1.02	510.96
4614	2203	0.77	510.98	1056	1.17	504.25	2215	0.77	510.97
4646.5	Spring Ave.								
4679	2203	1.02	510.98	1056	1.3	504.26	2215	1.02	511
4839	2203	1.59	510.97	1056	1.68	504.27	2215	1.59	510.99
4976	Queen St.								
5113	2203	3.05	510.94	1056	2.82	508.18	2215	3.01	511.01
5114	2203	3.05	510.94	1056	2.82	508.18	2215	3.01	511.01
5289	Broadway Ave								
5464	2203	4.13	510.97	1056	3.93	509.11	2215	4.08	511.02
5509	2203	2.62	511.09	1056	1.81	509.3	2215	2.61	511.14

Tributary D.2 is located along Tributary D of Black Fork Creek. There are seven structures that cross over the creek, all of which are overtopped by the 100-year ultimate water surface. Table 39 shows the roadways that are located along Tributary D.2 and the amount of water that is over the roadway. Currently there is some minor flooding along tributary D.2 with 9 structures located inside the 100-year ultimate floodplain. Table 40 shows a summary of the modeling results for Black Fork Creek Tributary D.2. The table shows a summary of the existing 100-year, 2-year and the 100-year ultimate water surface elevations. The tables have been condensed for this report; the full summary table can be found in Appendix C.2. For detailed mapping of the existing100-year and 100-year ultimate water surface elevations and cross section locations see Appendix D.2. Profile of the creek can be seen in Appendix E.2

Table 39 – Tributary D.2 of Black Fork Creek Roadway Overtopping

Roadway	Cross Section	Depth of Overtopping (ft)
Townsend	191	2.85
Beverly	1269	0.95
Locust	1426	0.05
Beckham	2059	1.78
Earwin	2513.5	3.59
Adams	2798.5	1.28
Center	2996.5	1.21

Table 40 – Black Fork Tributary D.2 Creek Results Summary

Black Fork Creek Tributary D.2									
	100 Year Existing			2 Year Ultimate			100 Year Ultimate		
Station	Q (cfs)	V (ft/s)	WSE (ft)	Q (cfs)	V (ft/s)	WSE (ft)	Q (cfs)	V (ft/s)	WSE (ft)
130	2010	1.25	487.09	992	3.24	484.12	2020	1.23	487.16
161	2010	1.51	487.09	992	6.74	483.91	2020	1.48	487.16
191	Townsend Ave.								
221	2010	2.44	487.1	992	2.1	484.97	2020	2.42	487.17
1247	2010	5.5	489.89	992	3.3	488.52	2020	5.52	489.9
1269	Beverly Ave.								
1291	2010	4.13	491.33	992	3.54	488.92	2020	4.14	491.34
1391	2010	8.61	490.87	992	6.22	488.74	2020	8.64	490.88
1426	Locust St.								
1461	2010	6.16	493.9	992	4.45	491	2020	6.19	493.89
1909	1298	15.36	494.08	656	12.25	491.39	1298	15.36	494.08
2059	Beckham Ave.								
2209	1298	2.31	503.42	656	8.05	496.01	1298	2.31	503.42
2450	1298	2.08	503.45	656	8.73	496.77	1298	2.08	503.45
2513.5	Erwin St. / Thompson Ave.								
2577	1298	2.08	503.46	656	6.73	497.81	1298	2.08	503.46
2769	1298	8.78	502.95	656	10.21	497.88	1298	8.78	502.95
2798.5	Adams Ave.								
2828	1298	3.08	504.39	656	7.07	499.51	1298	3.08	504.39
2933	1298	5.34	504.3	656	8.71	499.54	1298	5.34	504.3
2996.5	Center St								
3060	1298	10.65	504.31	656	6.95	502.65	1298	10.65	504.31
3284	1298	15.78	507.93	656	12.54	505.12	1298	15.78	507.93

Tributary D.3 is located along Tributary D of Black Fork Creek. There are four structures that cross over the creek, all of which are overtopped by the 100-year ultimate water surface. Table 41 shows the roadways that are located along Tributary D.3 and the amount of water that is over the roadway. Currently there is some minor flooding along tributary D.3 with 14 structures located inside the 100-year ultimate floodplain. Table 42 shows a summary of the modeling results for Black Fork Creek Tributary D.3. The table shows a summary of the existing 100-year, 2-year, and the 100-year ultimate water surface elevations. The tables have been condensed for this report, the full summary table can be found in Appendix C.2. For detailed mapping of the existing100-year and 100-year ultimate water surface elevations and cross section locations see Appendix D.2. Profile of the creek can be seen in Appendix E.2

Table 41 – Tributary D.3 of Black Fork Creek Roadway Overtopping

Roadway	Cross Section	Depth of Overtopping (ft)
Elm	722.5	1.84
Early	1353.5	3.82
Front	1950.5	1.01
Houston	2912	6.45

Table 42 – Black Fork Tributary D.3 Creek Results Summary

Black Fork Creek Tributary D.3									
	100 Year Existing			2 Year Ultimate			100 Year Ultimate		
Station	Q (cfs)	V (ft/s)	WSE (ft)	Q (cfs)	V (ft/s)	WSE (ft)	Q (cfs)	V (ft/s)	WSE (ft)
275	2593	10.05	488.9	1020	8.63	487.96	2599	10.05	488.9
697	2593	9.79	491.27	1020	7.52	489.09	2599	9.77	491.28
722.5	Elm St.								
748	2593	8.7	491.83	1020	7.07	489.86	2599	8.71	491.84
1332	2593	9.91	497.1	1020	5.51	493.84	2599	9.93	497.1
1340	2593	5.96	498.4	1020	5.44	493.89	2599	5.95	498.41
1353.5	Earle St.								
1367	2593	4.76	498.55	1020	5.09	494.37	2599	4.76	498.56
1851	2593	4.56	503.5	1020	3	498.36	2599	4.56	503.51
1950.5	E. Front St.								
2050	2593	1.39	511.6	1020	9.57	503.58	2599	1.4	511.58
2764	1895	2.03	511.66	850	15.99	505.91	1899	2.05	511.64
2802	E. Front St. / E. Houston St.								
2840	1895	1.96	511.67	850	1.4	509.88	1899	1.97	511.66
2884	1895	1.36	511.69	850	1.02	509.89	1899	1.37	511.68
2912	E. Houston St.								
2940	1895	1.78	511.7	850	1.33	509.9	1899	1.79	511.69
3773	1895	1.72	511.87	850	2.98	509.94	1899	1.73	511.86

Tributary D.4 is located on the upper end of Tributary D of Black Fork Creek. There are 11 structures that cross over the creek, all of which are overtopped by the 100-year ultimate water surface. Table 43 shows the roadways that are located along Tributary D.4 and the amount of water that is over the roadway. Currently there is major structure flooding along tributary D.4 with 27 structures located inside the 100-year ultimate floodplain. There are currently 7 structures upstream of the railroad along Turtle Creek, and 20 structures at Lake Drive. Table 44 shows a summary of the modeling results for Black Fork Creek Tributary D.4. The table shows a summary of the existing 100-year, 2-year and the 100-year ultimate water surface elevations. The tables have been condensed for this report; the full summary table can be found in Appendix C.2. For detailed mapping of the existing100-year and 100-year ultimate water surface elevations and cross section locations see Appendix D.2. Profile of the creek can be seen in Appendix E.2

Table 43 – Tributary D.4 of Black Fork Creek Roadway Overtopping

Roadway	Cross Section	Depth of Overtopping (ft)
Drive #1	570	5.5
Drive #2	697.5	4.8
Drive #3	805	2.89
Drive #4	917.5	1.73
Dodge St.	1165	1.76
Drive #5	1312.5	2.27
Lake St.	1947.5	2.5
First St.	2570	1.67
Second St.	3172.5	2.79
Third St.	3582.5	2.25
Fifth St.	4337.5	1.71

Table 44 – Black Fork Tributary D.4 Creek Results Summary

Black Fork Creek Tributary D.4									
	100 Year Existing			2 Year Ultimate			100 Year Ultimate		
Station	Q (cfs)	V (ft/s)	WSE (ft)	Q (cfs)	V (ft/s)	WSE (ft)	Q (cfs)	V (ft/s)	WSE (ft)
300	1224	0.72	526.76	572	4.57	516.83	1226	0.72	526.78
550	1224	1.49	526.76	572	9.84	518.69	1226	1.49	526.78
570	Drive #1								
590	1224	1.5	526.78	572	4.08	521.43	1226	1.5	526.8
665	1224	1.69	526.79	572	8.87	520.94	1226	1.68	526.8
697.5	Drive #2								
730	1224	1.94	526.8	572	9.64	521.22	1226	1.94	526.82
785	1224	3.49	526.78	572	11.39	521.62	1226	3.48	526.79
805	Drive #3								
825	1224	3.65	526.82	572	7.62	523.49	1226	3.62	526.84
900	1224	8.5	526.66	572	11.29	524.04	1226	8.47	526.67
917.5	Drive #4								
935	1224	4.9	527.71	572	6.45	525.87	1226	4.88	527.73
1130	1224	11.07	528.03	572	7.22	526.84	1226	11.07	528.04
1165	Dodge St.								
1200	1224	3.04	532.92	572	2.06	531.74	1226	3.03	532.93
1295	1224	3.9	532.96	572	2.91	531.75	1226	3.89	532.97
1312.5	Drive #5								
1330	1224	3.15	533.09	572	2.1	532.05	1226	3.15	533.1
1920	1224	3.67	540.7	572	4.88	539.08	1226	3.67	540.7
1947.5	Lake St.								
1975	1224	2.41	540.84	572	1.91	539.66	1226	2.42	540.84
2540	1224	8.88	543.32	572	6.97	542.72	1226	8.89	543.33
2570	First St.								
2600	723	3.34	544.15	338	2.24	543.53	724	3.37	544.14
3145	723	7.91	550.07	338	11.67	547.91	724	7.91	550.07
3172.5	Second St.								
3200	723	3.57	552.93	338	3.04	551.88	724	3.46	552.99
3535	723	7.9	557.19	338	6.31	556.68	724	7.91	557.2
3582.5	Third St.								
3630	723	5.41	558.2	338	4.34	557.6	724	5.51	558.18
4250	723	12	565.76	338	9.93	563.64	724	12	565.77
4337.5	Fifth St.								
4425	723	2.73	576.35	338	1.61	575.38	724	2.73	576.35
4465	723	2.25	576.37	338	1.25	575.4	724	2.25	576.38

Tributary D.5 is located on the upper end of Tributary D of Black Fork Creek. There are 4 structures that cross over the creek, all of which are overtopped by the 100-year ultimate water surface. Table 45 shows the roadways that are located along Tributary D.5 and the amount of water that is over the roadway. Currently there is major flooding along tributary D.5 with 12 structures located inside the 100-year ultimate floodplain. Table 46 shows a summary of the modeling results for Black Fork Creek Tributary D.5. The table shows a summary of the existing 100-year, 2-year and the 100-year ultimate water surface elevations. The tables have been condensed for this report; the full summary table can be found in Appendix C.2. For detailed mapping of the existing100-year and 100-year ultimate water surface elevations and cross section locations see Appendix D.2. Profile of the creek can be seen in Appendix E.2

Table 45 – Tributary D.5 of Black Fork Creek Roadway Overtopping

Roadway	Cross Section	Depth of Overtopping (ft)
Lindsay	345	3.17
Shaw	720	2.49
First	1472.5	2.6
Second	1832.5	3.08

Table 46 – Black Fork Tributary D.5 Creek Results Summary

Black Fork Creek Tributary D.5									
	100 Year Existing			2 Year Ultimate			100 Year Ultimate		
Station	Q (cfs)	V (ft/s)	WSE (ft)	Q (cfs)	V (ft/s)	WSE (ft)	Q (cfs)	V (ft/s)	WSE (ft)
200	930	2.15	541.13	449	7.57	536.51	930	2.15	541.13
315	930	9.13	542.85	449	11.44	540.13	930	9.13	542.85
345	Lindsey Lane								
375	930	2.97	547.71	449	1.74	546.72	930	2.97	547.71
695	930	6.99	547.64	449	4.59	546.67	930	6.99	547.64
720	Shaw St.								
745	930	2.64	550.84	449	1.66	549.96	930	2.64	550.84
1225	930	9.16	559.74	449	7.66	559.22	930	9.16	559.74
1425	930	9.34	563.81	449	14.52	560.4	930	9.34	563.81
1472.5	First St.								
1520	930	2.95	564.74	449	1.92	563.95	930	2.95	564.74
1800	930	7.71	565.64	449	7.83	563.96	930	7.71	565.64
1832.5	Second St.								
1865	930	3.3	570.39	449	2.17	569.56	930	3.3	570.39
2120	930	10.17	571.12	449	8.8	569.65	930	10.17	571.12

Willow Creek

Willow Creek is located on the northern side of the City is a combination of urban and rural stream. The lower portion of the creek is in a rural area posing little flooding risk to structures. The upper portion of the creek runs through a very urban area of the City of Tyler with multiple structures located inside the 100-year ultimate floodplain. Three structures are located at station

192+00 upstream of Loop 323. Seven of those structures are located downstream of Parkdale Street, and three structures flooding upstream of Parkdale. One structure is located upstream of Lyons Street. The roadway culvert at Erwin Street causes major flooding upstream of that structure putting over 22 structures inside the 100-year ultimate floodplain. Table 47 shows all the roadway structures that cross Willow creek inside the study area and the amount of water that overtops each structure. Table 48 shows a summary of the modeling results for Willow Creek for the existing 100-year, 2-year, and the 100-year ultimate water surface elevations. The table has been condensed for this report, the full summary table can be found in Appendix C.2. For detailed mapping of the existing 100-year and 100-year ultimate water surface elevations and cross section locations see Appendix D.2. Profile of the creek can be seen in Appendix E.2

Table 47 – Willow Creek Roadway Overtopping

Roadway	Cross Section	Depth of Overtopping (ft)
SH 110	9653	2.16
Loop 323	12762	Not Overtopped
Parkdale	20603	3.3
Forest	21213.5	1.11
Erwin	23657	0.88
SH 64	23970	2.72
SH 64 Cross	24440	1.94
SH 64 Cross	25119	0.8
SH 64 Cross	25367	0.44
Front	25999	1.69

Table 48 – Willow Creek Results Summary

Willow Creek									
	100 Year Existing			2 Year Ultimate			100 Year Ultimate		
Station	Q (cfs)	V (ft/s)	WSE (ft)	Q (cfs)	V (ft/s)	WSE (ft)	Q (cfs)	V (ft/s)	WSE (ft)
1600	7833	1.15	423.22	2500	0.85	416.5	8054	1.04	424.72
5594	7596	7.48	428.89	2576	4.72	426.27	7934	7.86	428.87
6985	6900	14.55	443.03	2448	9.12	440.01	7135	14.67	443.2
9608	6911	19.09	446.09	2452	6.56	446.46	7151	19.3	446.35
9653	Van Highway, S.H. 110								
9698	6911	1.66	451.87	2452	6.26	447.4	7151	1.64	452.14
10991	5656	3.34	452.02	2034	2.35	448.22	5791	3.28	452.28
12650	5656	7.4	456.83	2034	10.08	451.95	5791	7.46	456.94
12706	Loop 323								
12762	5656	3.07	461.13	2034	8.15	454.01	5791	3.1	461.22
15908	4222	2.76	463.44	1682	2.37	460.63	4280	2.75	463.52
17640	4202	6	465.61	1893	5.09	464.42	4241	6.04	465.61
20533	4202	4.21	479.92	1893	4.51	477.03	4241	4.21	479.95
20603	Parkdale Dr.								
20673	4202	4.22	479.96	1893	3.92	477.55	4241	4.21	480
21071	2623	5.98	480.73	1223	10.82	477.72	2649	6	480.76
21213.5	Forest Ave.								
21356	2623	5.73	488.34	1223	5.51	485.69	2649	5.77	488.36
23585	2383	8.09	495.04	1168	6.52	492.72	2404	8.13	495.07
23657	W. Erwin St.								
23729	2383	4.79	499.93	1168	4.95	495.9	2404	4.83	499.94
23730	1945	5.4	499.86	1013	6.69	495.68	1960	5.43	499.88
23790	W. Erwin St.								
23850	1945	2.84	500.59	1013	5.65	497.15	1960	2.84	500.61
23939	1945	2.05	500.65	1013	6.71	497.15	1960	2.05	500.67
23970	S.H. 64 exit lane								
24001	1945	2.22	500.67	1013	3.72	498.19	1960	2.22	500.68
24407	1945	2.46	500.7	1013	6.03	498.17	1960	2.47	500.72
24440	S.H. 64 crossover								
24473	1945	4.31	500.74	1013	3.23	499.61	1960	4.32	500.75
25097	931	5.01	503.67	442	7.57	502.13	935	5.01	503.68
25119	S.H. 64 crossover								
25141	931	4.63	505.33	442	6.92	503.22	935	4.62	505.35
25343	931	10.32	505.48	442	8.01	504	935	10.32	505.5
25367	S.H. 64 crossover								
25391	931	8.15	507.39	442	11.01	505.5	935	8.19	507.39
25674	638	10.1	510.01	303	11.16	508.47	641	10.15	510.01
25999	Front St								
26324	638	1.46	521.68	303	5.33	516.08	641	1.48	521.66
26374	638	3	521.61	303	8.45	516.64	641	3.03	521.59

Harris Creek and Tributaries

Harris Creek has two major tributaries that were part of this study; Wiggins Creek and Ray Creek. Harris Creek has an overall drainage area of 93.87 square miles. The detailed hydraulic modeling for Harris Creek has approximately 10.97 miles of creek for the main channel and 6.88 miles of Ray Creek and 4.29 miles of Wiggins Creek.

Harris Creek and its tributaries are located on the northeast side of the City of Tyler. All of Harris Creek, Wiggins Creek and Ray Creek would be considered rural channel. The floodplain for each creek is very wide in some locations. The overbanks for these creeks are covered with dense trees and vegetation with overall high Mannings n-values. The following Table 49 shows a summary of the modeling results for Harris Creek. This table shows a summary of the existing 100-year, 2-year, and the 100-year ultimate water surface elevations. The tables have been condensed for this report, the full summary table can be found in Appendix C.3. For detailed mapping of the existing100-year and 100-year ultimate water surface elevations and cross section locations see Appendix D.3. Profile of the creek can be seen in Appendix E.3

Harris Creek is located in the eastern part of the city, outside of most urban areas, and does not pose an immediate flooding problem for building structures. There is currently one structure located at approximately station 152+00 and another structure located at approximately 402+00 that are inside the 100-year ultimate floodplain. Out of the 4 roadways that cross the creek 3 are overtopped by the 100-year ultimate water surface elevation. Table 50 shows the roadway crossing and the depth of water that overtops the roadway.

Table 49 – Harris Creek Results Summary

Harris Creek									
	100 Year Existing			2 Year Ultimate			100 Year Ultimate		
Station	Q (cfs)	V (ft/s)	WSE (ft)	Q (cfs)	V (ft/s)	WSE (ft)	Q (cfs)	V (ft/s)	WSE (ft)
300	33690	18.83	329.52	9432	12.85	328.17	37211	19.28	329.67
2211	33690	4.85	334.29	9432	2.91	331.38	37211	5.06	334.59
4167	33690	7.93	336.5	9432	5.28	333.2	37211	8.23	336.86
6164	33706	5.75	342.17	9482	3.2	337.34	37263	6.03	342.67
8500	33706	5.22	343.4	9482	4.15	338.24	37263	5.35	343.93
10080	33673	4.25	344.18	9522	4.46	339.37	37352	4.33	344.71
11511	33972	9.41	345.76	9626	7.14	341.92	37866	9.84	346.24
11606	33972	14.17	346.7	9626	12.54	341.2	37866	14.68	347.06
11624.5	Old Longview Rd.								
11643	33972	7.38	348.52	9626	7.49	343.41	37866	7.86	348.84
13321	33972	5.31	350.23	9626	4.27	345.57	37866	5.52	350.69
15033	33972	8.16	351.39	9626	6.08	346.69	37866	8.44	351.89
16384	33972	6.17	353.42	9626	5.68	348.76	37866	6.28	353.96
18021	30687	4.92	354.62	9660	3.86	350.53	34484	5.12	355.13
19986	30687	7.11	359.73	9660	5.52	355.37	34484	7.25	360.31
22314	34963	5.33	362.01	9807	3.29	357.53	39731	5.66	362.61
24212	34963	5.16	364.75	9807	2.97	359.99	39731	5.44	365.42
26213	34963	5.13	366.73	9807	6.34	361.63	39731	5.34	367.45
26572	34963	9.27	371.78	9807	9.28	366.61	39731	10.02	372.05
28294	14630	5.09	375.36	4509	4.2	369.59	15418	5.1	375.75
29625	14630	12.88	375.77	4509	11.7	369.43	15418	13.21	375.91
29673	Culvert								
29721	14630	6.11	378.87	4509	3.4	376.54	15418	6.29	378.98
31316	14630	4.22	380.02	4509	1.97	376.88	15418	4.36	380.2
33622	14463	7.52	381.55	4723	5.7	377.67	15352	7.68	381.78
35511	14463	4.08	385.4	4723	3.29	381.23	15352	4.15	385.68
37470	14391	6.54	388.07	4860	4.97	385.39	15823	6.56	388.37
39685	14391	4.86	392.72	4860	4.25	390.24	15823	4.92	393
40100	14391	11.21	397.67	4860	9.04	394.28	15823	11.47	397.92
40129.5	S.H. 31								
40159	14391	3.33	401.03	4860	8.64	395.3	15823	3.52	401.21
41894	14391	5.76	403.34	4860	5.26	399.73	15823	5.87	403.7
43751	11465	5.5	406.74	4180	4.45	404.21	12213	5.55	406.98
45538	11429	6.61	414.16	4197	6.7	411.03	12172	6.68	414.41
45564	11429	11.48	414.6	4197	8.34	411.02	12172	11.62	414.82
45589	Bridge								
45614	11429	6.86	416.94	4197	6.61	412.23	12172	7.13	417.07
46985	11134	5.6	418.98	4077	6	415.15	11967	5.73	419.23
49005	7682	6.18	423.21	2848	4.87	421.38	8363	6.31	423.41
51015	7250	7.09	429.58	2856	6.08	427.8	7959	7.27	429.79
52986	6110	7.84	439.9	2433	6.45	438.42	6716	7.97	440.1
54998	3613	5.37	451.25	1520	3.86	449.95	4002	5.6	451.43
56993	3613	8.34	462.98	1520	5.8	461.19	4002	8.73	463.22
57930	3613	7.83	467.89	1520	5.88	465.86	4002	8.05	468.2

Table 50 – Harris Creek Roadway Overtopping

Roadway	Cross Section	Depth of Overtopping (ft)
CR 248	11624.5	4.45
FM 2767	29673	2.98
ST HWY 31	40129.5	Not Overtopped
CR	45313	4.1

Ray Creek is located on the eastern side of the City just outside of most of the urban areas, and does pose a minor risk of flooding problem for building structures. There is currently no major structure located inside the 100-year ultimate floodplain. There are three roadways that cross the creek, and all three are overtopped by the 100-year ultimate water surface elevation. Table 51 shows the roadway crossing and the depth of water that overtops the roadway. Table 52 shows a summary of the modeling results for Ray Creek for he existing 100-year, 2-year, and the 100-year ultimate water surface elevations. The table has been condensed for this report, the full summary table can be found in Appendix C.3. For detailed mapping of the existing100-year and 100-year ultimate water surface elevations and cross section locations see Appendix D.3. Profile of the creek can be seen in Appendix E.3

Table 51 – Ray Creek Roadway Overtopping

Roadway	Cross Section	Depth of Overtopping (ft)
County Road	3943	3.2
Railroad	20624.5	Not Overtopped
County Road	26586.5	0.14
Railroad	34215.5	Not Overtopped
Gladewater Hwy	35768	2.37

Table 52 – Ray Creek Results Summary

Ray Creek									
	100 Year Existing			2 Year Ultimate			100 Year Ultimate		
Station	Q (cfs)	V (ft/s)	WSE (ft)	Q (cfs)	V (ft/s)	WSE (ft)	Q (cfs)	V (ft/s)	WSE (ft)
1962	22083	14.1	332.09	6093	9.76	330.94	23735	14.49	332.17
3936	21421	7.77	339.16	6041	2.33	339.01	23099	7.86	339.32
3943	Bridge								
3950	21421	3.62	341.88	6041	1.51	340.34	23099	3.79	342.02
5566	21421	9.55	345.9	6041	6.33	342.81	23099	9.79	346.15
7487	20823	8.32	353.29	5947	4.19	348.35	22808	8.68	353.68
9500	21117	9.79	358.33	5957	7.45	351.58	22855	10.16	358.59
11695	21117	3.94	362.52	5957	3.05	357.64	22855	4.05	362.88
13693	17563	14.03	368.49	5252	13.16	363.37	18775	14.51	368.62
15976	18105	7.2	372.34	5370	6.09	367.46	19313	7.31	372.64
18230	18405	7.7	375.41	6271	7.14	370.42	19614	7.85	375.71
20245	15848	5.38	378.16	5263	4.19	374.43	16806	5.42	378.45
20611	15848	16.46	377.82	5263	10.73	374.62	16806	16.76	378.07
20624.5	Railroad								
20638	15848	11.75	380	5263	8.78	375.25	16806	14.22	380
23418	10819	3.4	382.71	3646	5.25	378.85	11492	3.23	383.24
25820	10819	2.54	388.99	3646	1.43	387.09	11492	2.64	389.09
26574	10819	11.71	391.69	3646	7.26	390	11492	11.84	391.82
26586.5	Bridge								
26599	10819	7.36	392.93	3646	6.42	390.39	11492	10.48	392.13
28270	10819	9.15	399.75	3646	6.81	397.42	11492	9.29	399.92
29698	11253	10.9	404.46	4021	8.29	402.17	12143	10.98	404.69
30908	10410	12.05	408.7	4491	11.27	405.97	11333	12.14	408.99
32802	10410	12.27	418.77	4491	8.41	416.31	11333	12.53	419.07
34186	9386	8.5	424.74	4200	5.42	422.12	10261	9.07	424.92
34215.5	Railroad								
34245	9386	8.02	426.06	4200	5.46	422.5	10261	8.29	426.63
35753	6008	17.03	430.47	2509	8.19	429.28	6535	10.55	434.13
35768	Old Gladwater Highway								
35783	6008	7.63	435.57	2509	6.42	430.44	6535	6.85	436.67
36305	6008	14.09	435.32	2509	10.49	431.42	6535	12.51	436.55

Wiggins Creek is located on the eastern side of the city just outside of most urban areas, and does pose a minor risk of flooding problem for building structures. There is currently no major structure located inside the 100-year ultimate floodplain. There are four roadways that cross the creek, and all of which are overtopped by the 100-year ultimate water surface elevation. Table 53 shows the roadway crossing and the depth of water that overtops the roadway. Table 54 shows a summary of the modeling results for Wiggins Creek for the existing 100-year, 2-year, and the 100-year ultimate water surface elevations. The table has been condensed for this report, the full summary table can be found in Appendix C.3. For detailed mapping of the existing100-year and

100-year ultimate water surface elevations and cross section locations see Appendix D.3. Profile of the creek can be seen in Appendix E.3

Table 53 – Wiggins Creek Roadway Overtopping

Roadway	Cross Section	Depth of Overtopping (ft)
Railroad	3480	2.4
Lawhon	5347.5	5.31
Hwy 155	6062	2.21
Church Road	17593.5	4.02

Table 54 – Wiggins Creek Results Summary

Wiggins Creek									
	100 Year Existing			2 Year Ultimate			100 Year Ultimate		
Station	Q (cfs)	V (ft/s)	WSE (ft)	Q (cfs)	V (ft/s)	WSE (ft)	Q (cfs)	V (ft/s)	WSE (ft)
3447	13372	13.78	327.01	3133	11.15	321.44	15953	14.93	327.57
3469	13372	12.51	329.72	3133	10.24	322.5	15953	13.01	330.38
3480	Railroad								
3491	13372	11	331.36	3133	7.95	324	15953	12.87	331.47
5334	13372	3.67	333.2	3133	8.08	326.35	15953	3.84	333.92
5347.5	Lawhon								
5361	13372	2.83	333.34	3133	6.43	327.25	15953	2.98	334.04
6036	13372	9.16	333.06	3133	4.44	328.89	15953	9.23	333.81
6062	Old Highway 155								
6088	13372	5.99	334.18	3133	3.85	329.26	15953	6.45	334.63
8286	16112	11.38	335.72	3403	6.93	330.6	17729	11.41	336.25
10202	15840	5.16	339.64	3369	4.69	334.97	17409	5.18	340.08
12588	17059	11.93	347.01	3481	5.96	342.56	18328	12.35	347.26
14831	16976	8.9	355.19	3479	6	349.21	18163	9.06	355.55
17021	16880	9.76	361.62	3758	6.01	356.11	18100	9.94	361.97
17583	16880	12.95	366.04	3758	14.53	359.47	18100	13.3	366.18
17593.5	Harris Creek Church Rd.								
17604	16880	5.4	367.11	3758	2.46	363.95	18100	5.63	367.27
19522	15443	4.35	368.03	3697	2.14	364.21	16686	4.53	368.27
21488	15220	3.3	372.6	4428	2.06	369.33	16375	3.39	372.87
22658	15220	13.53	380.01	4428	15.85	376.41	16375	14.56	380.01

<u>Gilley Creek and Tributaries</u>

Gilley Creek and one of its major tributaries are part of this detailed study. The overall drainage area of Gilley Creek is 12.5 square miles. The detailed hydraulic modeling for the main channel of Gilley Creek is approximately 5.41 miles and the tributary G-1 has approximately 2.11 miles of stream. Gilley Creek is located on the southwest side of the City of Tyler and drains into Lake

Tyler. The channel for Gilley Creek and its tributaries would be considered a rural watershed. The lower portion of the floodplain is very wide in some locations with widths of over 1000-feet. Being in the rural area the floodplain is mostly undeveloped with heavy amounts of vegetation and undergrowth. Mannings n-values in the overbanks ranged from 0.04 for grass pasture land to 0.10 for dense vegetation. The following Table 55 and Table 56 show a summary of the modeling results for Gilley and Gilley tributary G-1. These tables show a summary of the existing 100-year, 2-year, and the 100-year ultimate water surface elevations. The tables have been condensed for this report, the full summary table can be found in Appendix C.4. For detailed mapping of the existing100-year and 100-year ultimate water surface elevations and cross section locations see Appendix D.4. Profile of the creek can be seen in Appendix E.4

Gilley Creek and Tributary G-1's location on the southeastern part of the city outside of most urban areas does not pose an immediate flooding problem for building structures as there are no building located inside the 100-year ultimate floodplain. Inside the study area there are 5 roadways that cross the two creeks. Out of the five roadways that cross the two creeks four of those are overtopped by the 100-year ultimate water surface elevation. Table 57 and Table 58 show the roadway crossing and the depth of water that overtops the roadway.

Table 55 – Gilley Creek Results Summary

Gilley Creek									
	100 Year Existing			2 Year Ultimate			100 Year Ultimate		
Station	Q (cfs)	V (ft/s)	WSE (ft)	Q (cfs)	V (ft/s)	WSE (ft)	Q (cfs)	V (ft/s)	WSE (ft)
767	18561	12.59	379.35	4710	8.3	378.06	19214	12.33	379.46
1015	18561	6.58	383.22	4710	3.89	381.12	19214	6.71	383.27
1062.5	F.M. 848								
1110	18561	5.87	383.99	4710	3.07	381.85	19214	5.97	384.06
2704	17748	8.33	388.33	4678	4.51	384.29	18416	8.48	388.47
4676	17748	4.51	393.23	4678	3.26	387.3	18416	4.54	393.45
7022	18298	6.85	397.58	4909	9.05	392.76	18896	6.83	397.77
9014	18050	7.04	403.14	4895	4.17	398.94	18626	7.11	403.28
11315	17124	4.34	408	4993	3.33	404.17	17714	4.37	408.14
13627	17124	9.15	413.35	4993	6.52	410.21	17714	9.25	413.46
15638	17517	6.24	422.01	5246	4.77	417.61	18064	6.3	422.17
18000	17363	13.05	425.06	5321	10.82	422.1	17908	12.93	425.23
19606	8636	7.39	434.38	2781	6.59	431.44	9132	7.54	434.57
19670	8636	8.02	435.18	2781	10.39	431.21	9132	8.48	435.18
19702.5	C.R. 262								
19735	8636	4.04	438.97	2781	1.94	436.76	9132	4.2	439.07
21249	7897	5.94	440.73	2760	4.16	437.6	8383	6.1	440.93
23323	7458	6.42	448.36	2817	5.7	445.75	7967	6.54	448.55
24440	7458	5.42	454.83	2817	4.16	451.92	7967	5.54	455.06
25943	5853	4.91	458.92	2310	3.83	456.26	6238	5	459.16
27301	4641	6.54	466.9	1397	3.94	464.86	4964	6.71	467.05
28395	3612	10.98	470.55	1571	6.25	468.86	3819	11.4	470.68
28433	University Blvd.								
28471	3612	4.37	473.48	1571	3.25	469.78	3819	4.41	473.84
28580	3612	5.11	473.87	1571	5.26	470.22	3819	5.11	474.23

Table 56 – Gilley Tributary G.1 Creek Results Summary

Gilley Trib G-1									
	100 Year Existing			2 Year Ultimate			100 Year Ultimate		
Station	Q (cfs)	V (ft/s)	WSE (ft)	Q (cfs)	V (ft/s)	WSE (ft)	Q (cfs)	V (ft/s)	WSE (ft)
492	8487	9.99	425.23	2652	8.34	422.1	8583	14.45	423.99
1132	8487	10.59	429.44	2652	7.43	427.73	8583	10.97	429.38
1149	Mackey Rd. (C.R. 2122)								
1166	8487	8.13	430.45	2652	4.81	428.69	8583	8.19	430.46
2500	7371	6.26	437.06	2505	5.28	433.78	7491	6.31	437.1
4500	7371	11.07	444.32	2505	9.47	441.36	7491	11.09	444.37
5106	7371	11.76	449.03	2505	16.82	444.12	7491	11.79	449.08
5130.5	C.R. 2120								
5155	7371	12.72	449.45	2505	5.35	448.66	7491	12.82	449.48
7070	6447	8.45	458.53	2505	7.29	456.2	6523	8.53	458.54
9000	6390	7.87	470.03	2616	10.62	467.7	6385	7.8	470.05
11148	4518	7.66	478.03	1828	5.41	475.91	4548	7.68	478.05

Table 57 – Gilley Creek Roadway Overtopping

Roadway	Cross Section	Depth of Overtopping (ft)
FM 848	1062.5	3.06
CR 2120	19735	3.89
University	28433	Not Overtopped

Table 58 – Tributary G-1 Creek Roadway Overtopping

Roadway	Cross Section	Depth of Overtopping (ft)
CR 2122	1149	4.27
CR 2120	5130.5	2.48

Indian Creek

The main channel of Indian Creek is the only detailed portion of this study. The overall drainage area of Indian Creek is 23.77 square miles. The main channel of Indian Creek that is part of the detailed portion of this study is approximately 8.93 miles.

Indian Creek is located just west of the City of Tyler. The entire creek channel for Indian Creek is located in a rural area with very little development around the floodplain. The lower portion of the creek has a wide floodplain that in some areas measures over 1000-feet wide. The overbanks are covered with dense vegetation with Mannings n-values ranging from 0.04 for grass pasture to 0.10

for dense trees and undergrowth. The following Table 59 show a summary of the modeling results for Indian Creek. These tables show a summary of the existing 100-year, 2-year, and the 100-year ultimate water surface elevations. The tables have been condensed for this report, the full summary table can be found in Appendix C.5. For detailed mapping of the existing100-year and 100-year ultimate water surface elevations and cross section locations see Appendix D.5. Profile of the creek can be seen in Appendix E.5

Indian Creek is located on the western part of the city outside of most urban areas does not pose an imitate flooding problem for building structures as there are no major buildings located inside the 100-year ultimate floodplain. Inside the study area there are 6 roadways that cross the creek. Out of the 6 roadways that cross the creek five are overtopped by the 100-year ultimate water surface elevation. Table 60 shows the roadway crossing and the depth of water that overtops the roadway.

Table 59 – Indian Creek Results Summary

Indian Creek									
	100 Year Existing			2 Year Ultimate			100 Year Ultimate		
Station	Q (cfs)	V (ft/s)	WSE (ft)	Q (cfs)	V (ft/s)	WSE (ft)	Q (cfs)	V (ft/s)	WSE (ft)
10489	13256	10.47	349.05	4392	7.72	348.56	15609	11.08	349.13
12149	13256	10.24	355.25	4392	11.75	351.29	15609	10.52	355.59
12167	F.M. 2661								
12185	13256	2.78	356.92	4392	1.48	355.4	15609	3.06	357.18
14227	13256	15.71	359.96	4392	6.67	357.94	15609	18.62	359.89
14242	Railroad								
14257	13256	7.2	363.54	4392	6.46	358.61	15609	7.24	364.18
15673	13256	1.76	364.54	4392	1.37	360.02	15609	1.9	365.21
17639	13256	3.12	365.98	4392	2.01	362.67	15609	3.31	366.66
19328	13256	4.26	368.29	4392	3.55	364.72	15609	4.42	369.02
21266	12829	9.48	370.53	4502	7.13	367.64	14516	9.64	371.07
22556	12829	9.14	376.18	4502	5.66	371.9	14516	9.59	376.8
24441	10592	4.69	379.7	3287	3.24	374.84	12524	5.57	380.42
25271	10592	8.41	380.11	3287	10.32	374.84	12524	8.05	380.97
25286	Dean Rd.								
25301	10592	7.44	380.25	3287	8.96	376.84	12524	7.54	381.03
27131	10592	2.47	382.04	3287	1.25	378.71	12524	2.66	382.86
27831	10592	12.05	384.53	3287	11.02	378.16	12524	12.78	384.98
27843.5	C.R. 1139								
27856	10592	10.83	384.93	3287	9.29	379.32	12524	11.56	385.38
29713	10539	2.48	386.68	3381	2.98	381.65	12448	2.65	387.28
31771	9078	5.11	390.07	2928	3.86	387.4	11304	5.44	390.76
33576	9078	11.97	396.31	2928	7.91	393.25	11304	12.93	397.11
33641	9078	11	398.14	2928	7.23	394.41	11304	12.15	399.1
33664.5	Spur 364								
33688	9078	8.73	399.54	2928	4.93	395.27	11304	9.57	400.76
35536	9078	2.32	403.12	2928	3.32	397.63	11304	2.48	404.39
36431	7499	3.5	403.49	2201	3.91	398.42	9162	3.45	404.81
36935	7499	12.39	406.96	2201	9.98	398.63	9162	12.87	407.78
36952.5	Greenbriar Road								
36970	7499	5.41	408.84	2201	9.63	399.4	9162	5.78	409.55
37853	4408	3.35	409.17	1059	2.19	403.26	5418	3.71	409.91
39737	4408	3.59	410.01	1059	5.1	404.11	5418	3.71	411.08
41546	4408	6.67	412.8	1059	2.96	409.08	5418	7.44	413.44
43341	4331	0.24	433.42	1040	0.08	429.59	5319	0.28	434.28
48225	10951	8.95	433.69	5013	6.64	431.8	12315	8.69	434.45
50332	3107	2.98	441.55	1518	3.37	440.27	3490	2.92	441.84
51717	3107	9.54	457.87	1518	9.88	455.69	3490	9.6	458.18
51741	3107	12.43	458.22	1518	9.16	456.49	3490	12.79	458.65
51825	Loop 323								
51909	3107	3.66	465.16	1518	4.73	460.31	3490	3.91	465.4
53454	2868	9.81	470.6	1421	6.28	470.23	3155	10.04	470.72
55059	1699	3.94	483.69	850	5.05	481.28	1822	3.75	484.09
57141	1699	8.8	502.14	850	7.57	500.97	1822	8.93	502.3

Table 60 – Indian Creek Roadway Overtopping

Roadway	Cross Section	Depth of Overtopping (ft)
FM 2661	12167	2.33
Railroad	14242	1.68
Dean	25286	3.69
CR 1139	27843.5	3.59
Spur 364	33664.5	Not Overtopped
Greenbriar	36952.5	4.46
Loop 323	51825	1.3

Butler Creek

The main channel of Butler Creek is the only detailed portion of this study. The overall drainage area of Butler Creek is 12.22 square miles. The main channel of Butler Creek that is part of the detailed portion of this study is approximately 6.20 miles.

Butler Creek is located just west of the City of Tyler and drains directly into Lake Palestine. The entire creek channel for Butler Creek is located in a rural area with very little development around the floodplain. The lower portion of the creek has a wide floodplain that in some areas measures over 1000-feet wide. The overbanks are covered with dense vegetation with Mannings n-values ranging from 0.04 for grass pasture to 0.10 for dense trees and undergrowth. Table 61 shows a summary of the existing 100-year, 2-year, and the 100-year ultimate water surface elevations. The table has been condensed for this report, the full summary table can be found in Appendix C.6. For detailed mapping of the existing100-year and 100-year ultimate water surface elevations and cross section locations see Appendix D.6. Profile of the creek can be seen in Appendix E.6

Butler Creek is located on the western side of the City just outside of most urban areas and does not pose a flooding problem for building structures. There is currently one structure located at approximately station 420+00 that is inside the 100-year ultimate floodplain. Out of the five roadways that cross the creek all five are overtopped by the 100-year ultimate water surface elevation. Table 62 shows the roadway crossing and the depth of water that overtops the roadway.

Table 61 – Butler Creek Results Summary

Butler Creek									
	100 Year Existing			2 Year Ultimate			100 Year Ultimate		
Station	Q (cfs)	V (ft/s)	WSE (ft)	Q (cfs)	V (ft/s)	WSE (ft)	Q (cfs)	V (ft/s)	WSE (ft)
10053	18099	15.62	360.95	5564	10.66	358.19	18653	15.66	361.07
11637	18099	4.58	366.75	5564	3.88	362.11	18653	4.61	366.89
12864	18099	9.55	369.70	5564	9.69	366.12	18653	9.48	369.85
12880.5	Galilee Rd., C.R. 1131								
12897	18099	6.46	370.48	5564	5.35	366.96	18653	7.01	370.24
14780	16887	4.65	374.81	5491	2.62	371.28	17353	4.73	374.89
16659	16887	5.97	377.09	5491	3.03	372.16	17353	6.07	377.23
18447	16887	6.31	381.86	5491	5.49	378.54	17353	6.32	381.97
19742	13716	3.67	384.07	3907	2.49	380.02	14020	3.69	384.18
21046	13716	7.21	389.55	3907	6.01	385.73	14020	7.22	389.64
21094	13716	9.06	389.79	3907	8.13	386.92	14020	9.00	389.89
21109	Dean Rd., C.R. 1141								
21124	13716	7.81	390.19	3907	4.48	388.22	14020	7.88	390.23
22499	13716	3.86	393.94	3907	2.64	389.70	14020	3.88	394.04
24373	13716	4.49	395.51	3907	4.95	391.61	14020	4.49	395.61
26066	13716	6.54	398.96	3907	3.97	396.45	14020	6.56	399.02
26974	10278	1.38	402.31	3343	0.92	398.21	10452	1.40	402.38
28712	10278	2.84	402.69	3343	1.82	398.65	10452	2.86	402.77
30544	9531	4.22	404.46	3394	3.09	401.95	9676	4.23	404.52
30748	9531	11.56	406.00	3394	8.07	404.76	9676	11.56	406.03
30762	Greenbriar Rd., C.R. 1125								
30776	9531	9.89	406.45	3394	6.40	405.10	9676	9.90	406.49
31692	9531	5.39	408.59	3394	6.26	406.22	9676	5.40	408.63
31742	9531	10.28	409.50	3394	13.35	406.69	9676	10.36	409.52
31755.5	Lake Placid Rd, C.R. 1113								
31769	9531	4.18	410.39	3394	1.76	409.82	9676	4.21	410.42
33085	8323	5.50	412.20	3143	3.54	410.44	8436	5.52	412.24
34540	8323	5.34	419.01	3143	3.92	416.32	8436	5.36	419.06
36302	7184	3.23	423.16	2972	2.33	420.83	7270	3.24	423.20
37979	5834	7.63	427.23	2504	6.39	425.48	5905	7.64	427.26
39463	5834	7.72	435.14	2504	6.53	433.91	5905	7.74	435.16
41132	3530	9.45	441.72	1559	6.31	441.18	3570	9.46	441.73
42071	2548	2.79	454.51	1164	15.45	450.32	2574	2.82	454.51
42124.5	Highway 155								
42178	2548	1.89	456.76	1164	1.26	455.54	2574	1.91	456.77
42752	2548	1.89	456.86	1164	1.20	455.58	2574	1.90	456.87

Table 62 – Butler Creek Roadway Overtopping

Roadway	Cross Section	Depth of Overtopping (ft)
CR 1131	12880.5	5.19
CR 1141	21109	5.9
CR 1125	30762	3.13
CR 1113	31755.5	2.42
Hwy 155	42124.5	2.35

Non-Detailed Study Areas

The City staff identified approximately 85 miles of minor creeks to be analyzed by non-detailed methods, this included all of Little Saline Creek. No detailed study was conducted for Little Saline Creek. As with the detailed study areas cross-sections were coded with existing ground geometry based on the GIS 2-foot contour mapping, coded on average every 1000 feet and at any roadway crossing. Cross-sections taken from the GIS information were modified inside the channel banks based on information provided by the City of Tyler. No detailed field survey was conducted for the non-detailed study areas, but general information was obtained for roadway crossing. This information included photos of the structure, distance from top of roadway to top of culvert or bridge opening and size of the opening.

Mannings n-values throughout the City of Tyler vary greatly from 0.013 for concrete lined channel to 0.10 for thick vegetation growth in the floodplain overbanks. Typical Mannings n-values taken from HEC-RAS were used for the non-detailed areas.

This information above was used to create HEC-RAS models for each of the non-detailed reaches. Once the HEC-RAS model was created water surface elevation were then calculated at each roadway. Appendix F has a summary of the HEC-RAS models for each reach and calculated water-surface elevations at roadway locations.

PROPOSED IMPROVEMENT LOCATIONS

The following section discusses improvements for individual areas to provide a concept for future improvements. The proposed improvements are conceptual in nature and are only intended to assist the City of Tyler in prioritizing the needed improvements and developing a program to implement the improvements. The description of each reach includes an opinion of probable construction cost. The full engineering design of individual projects could result in an increase or decrease in the amount of actual improvement construction needed. The improvements outlined in this report and also the associated opinion of probable construction cost could vary significantly from those finally required. The need for improvements and the associated cost of those improvements will continue to increase over time.

West Mud Creek

State Hwy 69 (Broadway)

Existing structure consists of 6-9'x9' box culverts with a 100-year ultimate water surface of 462.68'. The existing roadway is currently over topped by approximately 2.68 feet of water. To eliminate roadway from being overtopped the box culverts would need to be removed and replaced with a bridge structure that has a top width of 90-feet and a concrete lined channel to be tied into existing concrete channel. The low chord set at elevation 458 and will remain in floodplain but weir flow over roadway will be eliminated. With the new structure in place the upstream water surface elevation is lowered from 462.68 to 459.32. The roadway is the only structure removed from the floodplain; additional channel improvement would be necessary to remove other structures from the floodplain.

Shiloh Road

The existing structure consists of a bridge with a top width of approximately 100-feet. The current 100-year ultimate water surface elevation is 475.01-feet and overtops the roadway surface by approximately 0.12-feet. Channel improvements under bridge and 75-feet downstream will reduce the water surface to elevation of 473.98 almost a foot under the top of the roadway. The proposed channel would be concrete lined with a 30-foot bottom with 2:1 sides to extend through the bridge to a distance of 75-feet downstream. Roadway is the only structure removed from the floodplain but the low chord of bridge still remains within the floodplain. Existing structure has a pedestrian crossing under bridge that will need to be preserved in the channel improvements.

New Copeland

The existing structure in the location is fairly new and consists of 5-10'x10' box culverts. The 100-year water surface upstream of structure is 480.29, overtopping the roadway by approximately 0.34-feet. In order to stop weir flow over roadway in this location and save existing structure channel improvements down stream of structure will be needed. A concrete lined channel with a 30' bottom and 2:1 sides would be extended from New Copeland structure to approximately 600' downstream. The improvements will drop the upstream water surface at New Copeland to an elevation of 478.23, over 1.7-feet below the roadway surface. The roadway and three houses along Cambridge Drive will be removed from the floodplain with these improvements.

Loop 323

Existing structure consists of 2-5'x 6' box culverts with the 100-year water surface elevation overtopping the roadway by 0.87-feet. Replacing the existing structure with 4-6'x6' box culverts reduces the upstream water surface from 506.06 to 503.06 and stops the roadway from flooding. In addition to the roadway being removed from the floodplain two additional structures are also removed from the floodplain.

West Mud Creek Tributary B

New Copeland Road

The existing structure under New Copeland Road is 3-8'x 9' box culverts. The 100-year ultimate water surface elevation overtops the roadway by 0.79-feet. Adding capacity to the existing structure will reduce the water surface almost one foot removing the roadway from the floodplain. The water surface is reduced from 481.74 to 479.99, almost two feet. Improvements needed to add capacity to structure would be three more8'x 9' box culverts.

West Mud Creek Tributary C

Loop 323

Existing structure is 6-10'x10' box culverts. The 100-year ultimate water surface elevation overtops the roadway by 0.15'. By adding an additional 10'x 10' box culvert to the existing structure the water surface elevation is lowered from 487.76 to 486.72. The roadway and one structure on the west side of channel are removed from the floodplain. In order to remove the four structures on the east side of the channel the water surface elevation would need to be reduced by approximately 4'. Additional structure capacity and channel improvements would be needed to achieve such a drastic reduction in water surface elevation.

Broadway

Existing structure is 5-8'x8' box culverts. The roadway is overtopped by 100-year ultimate water surface elevation by almost 4' causing multiple structures to flood upstream of the roadway. Adding three more 8'x8' boxes and concrete lining 500' upstream channel lowers the water surface from 498.23 to 494.3 removing approximately 10 structures from the floodplain. Concrete channel needed is a 30' bottom with 2:1 sides.

FM 2493

Existing structure under FM 2493 is one 16'x8' box culvert. Existing structure is undersized causing the roadway to be overtopped by the 100-year ultimate water surface elevation by 4.5' Replacing the structure with 4-8'x8' lowers the water surface elevation from 529.85 to 520.65, removing approximately 20 structures from the floodplain.

West Mud Creek Tributary A

Loop 323

Existing structure consists of 3-5'x5' and 2-10'x10' box culverts. The roadway is not overtopped by the 100-year water surface elevation but upstream flooding occurs. Replacing the existing culverts does not alleviate the flooding problem upstream of this area. The upstream channel is a concrete lined channel with vertical walls. Fences are built up to the edge of the existing channel. In order to stop upstream flooding improvements will be needed to the existing channel. Since private property comes to the edge of the existing channel, widening of the channel might not be possible, so lowering the channel is the other option. If the existing channel is lowered the culvert structure will also need to be replaced due to grade. The selected alternative includes lowering the existing channel approximately 1' and replacing the existing culvert with 5-10'x 5' boxes. Proposed channel would have vertical walls and a bottom width of approximately 20'. In order to remove all structures flooding in this area the channel will need to be concrete lined up to the next structure at Woodland Drive 1300 feet. A drop structure will also be needed downstream of Woodland Drive to control the grade of the channel.

West Mud Creek Tributary A-2

Private Culvert/Old Jacksonville/Loop 323

Improvements at Old Jacksonville and Loop 323 depend on improvements made at a private culvert downstream of Old Jacksonville. The private structure consists of 2-10'x 7' box culverts. These culverts were not surveyed for this study; flow lines and exact size of the culverts will need to be verified for a detailed study and design in this area. Adding one more 10'x 7' box to the private structure drops the 100-year water surface elevation upstream of the structure from 497.38 to 494.77.

Old Jacksonville

Existing structure consists of one 10'x 7' box culvert. By adding capacity to the downstream channel the downstream water surface elevation is lowered in this area but weir flow over the roadway is still occurs due to insufficient capacity of the existing box culvert. Adding two 10'x7' boxes will lower the upstream water surface elevation and eliminate the weir flow. After improving the private structure and also improving the Jacksonville culvert, the upstream water surface elevation is lowered from 500.35 to 489.12.

Loop 323

Existing structure consists of one 7'x6' box culvert. The existing culvert does not have the capacity to carry the 100-year design flow. Due to the configuration of the existing culvert, adding capacity may not be possible. Replacing the existing structure with 2-7'x5' boxes and also lining the downstream channel will eliminate the weir flow over the roadway. Downstream channel consists of a 15' bottom with 2:1 sides slopes. All improvements are dependent on a private structure downstream of Old Jacksonville.

Henshaw Creek

US HWY 69

Existing structure is 6-9'x6' box culverts. This structure is under capacity and causes weir flow over the highway of approximately 1.22'. Replacing this culvert with a bridge will eliminate the weir flow over the roadway and drop the water surface from 392.41 to 390.24. The proposed bridge structure is trapezoidal with a top width of 120-foot with 3:1 sides and the proposed channel is a five-foot deep concrete lined low flow channel with a 20' bottom and 2:1 sides.

Shackelford Creek

CR 110

Existing structure is 2-108" reinforced concrete pipe (RCP) and currently has 3.5' of weir flow over CR 110 (Cumberland Road) during the 100-year ultimate event. In order to eliminate the road overtopping the road must be raised to a minimum of 453.5, over 3' higher than the existing roadway. Also the existing culverts must be replaced with a bridge structure. The proposed bridge structure consists of a 30' bottom pilot channel, with 3:1 sides and a bridge top width of 150'. This lowers the 100-year ultimate water surface elevation from 454.08 to 453.23. The low chord of the bridge will still be within the ultimate water surface elevation but will eliminate the weir flow over the road.

Indian Creek

Loop 323

The existing structure is 4-7'x 7' box culverts. The roadway is currently overtopped by the 100-year ultimate water surface elevation by almost 1.3'. By adding two more box culverts the water surface elevation is lowered from 465.4 to 463.75, eliminating the weir flow over the roadway.

Gilley Creek

CR 2120

The existing structure is one 96" RCP and one 72" RCP. The current roadway is overtopped by 3.89'. Replacing the existing RCP's with a bridge structure and raising the roadway approximately one foot will eliminate the weir flow over the roadway. The minimum elevation the roadway would need to be raised to 436 from 435.18. The proposed bridge would need a top width of 170' and a channel with a 20' bottom and 2:1 sides. With the improvements the water surface elevation can be reduced from 439.07 to 435.76.

Harris Creek

FM 2767

The existing structure over Harris Creek is a bridge with a top width of 110', three sets of piers and a natural channel. The current structure is overtopped by 3.33' of weir flow. In order to remove the road surface from the floodplain, the structure must be replaced and the roadway needs to be raised 3.3' to 472. The bridge structure needs to be expanded to a top width of 460' with a natural channel and piers spaced at 40' centers. The improvements lower the water surface from 472.05 to 471.4.

HWY 31

The existing structure at Highway 31 is 6-10'x10' box culverts. The roadway is overtopped by over 1.5'. In order to remove weir flow at this location the existing box culverts need to be replaced with a bridge structure. The new structure needs to have a 210' top width, a concrete channel with 2:1 sides and a bottom width of 30'. The new structure will eliminate weir flow and lower the 100-year ultimate water surface from 401.21 to 397.6.

Willow Creek

Loop 323

The existing structure consists of 3-10'x10' box culverts. The roadway is currently overtopped by the 100-year ultimate water surface elevation by 1.22'. Adding capacity to the existing structure will lower the water surface from 461.22 to 458.35. Improvements to this area would need to be 2 more 10'x10' boxes.

Erwin Street and Upstream Channel.

The Existing structure under Erwin Street consists of varying size box and circular culverts. The circular culvert is in a lower pilot channel then the box culverts. Under Erwin Street an additional tributary to Willow connects to the box structure. Also under Erwin Street at the point where the tributary connects there is a drop structure that drops the channel bottom approximately four-feet. The roadway is overtopped by over 1.5' causing multiple structures upstream to flood.

Multiple improvements will be needed in this area in order to alleviate the existing flooding problem. Existing structures at Erwin Street, Glenwood Crossover and Elm Crossover need to be replaced. Also reconstruction of the existing concrete/stone lined channel will be necessary.

The proposed structure at Erwin Street would need to be 4-8'x 8' boxes. The upstream flow line of the new structure will need to be lowered to the elevation of the existing pilot channel of 488.

The existing structures at the Glenwood Crossover and Elm Street Crossover will need to be replaced with 3-8'x 8' boxes, and the flow lines at each will need to be lowered approximately 1' in order to add capacity to the concrete channel.

The proposed concrete channel would need to be lowered approximately 1' from its current elevation. The proposed channel will need to be concrete lined with vertical concrete walls due to space constraints of the street that is located on both sides of the channel.

The improvements listed above will lower the 100-year ultimate water surface elevation through the project area approximately 5.5' at Erwin Street, and approximately 1' at each of the crossovers. This project will remove multiple structures from the existing 100-year ultimate floodplain.

Black Fork Creek Tributary D4

5^Th^ Street

The existing culvert is a 4'x 4' box culvert. The roadway is overtopped by the 100-year ultimate water surface elevation by 1.81'. Replacing the existing structure with an 8'x8' box culvert will eliminate the current weir flow and reduce the upstream water surface elevation from 576.35 to 573.03.

Black Fork Creek Tributary D3

E. Front Street

The existing structure under Front Street is one 10'x10'. The roadway is overtopped by the 100-year ultimate water surface elevation. Also several structures upstream of the roadway are within the floodplain. Adding one 10'x10' box will reduce the water surface elevation and bring the roadway and some structures out of the floodplain. Adding an additional 10'x10' box will reduce the water surface elevation enough to bring all the structures immediately upstream of Front out of the floodplain.

Black Fork Creek Tributary D2

Beckham Street

The existing structure under Beckham Street is an 11.5'x7' box culvert. The roadway is overtopped by 1.69'. To reduce the water surface elevation in this location the existing structure needs to be replaced with two 10'x7' box culverts tied into the existing concrete lined channel. This improvement will reduce the water surface elevation from 503.42 to 496.81, removing the roadway and multiple structures upstream of the roadway from the floodplain.

Black Fork Creek Tributary D1

Gentry Parkway

Gentry Parkway is currently overtopped by the 100-year ultimate water surface elevation by 2.34. The existing structure consists of one 10'x7' box culvert. This structure is under capacity and needs to be replaced with 3-10'x10' box culverts. This improvement will reduce the water surface elevation from 487.24 to 483.24 removing the roadway and multiple structures upstream of the structure from the floodplain.

Black Fork Creek Tributary D

Front Street/Upstream Channel to Douglas Crossover.

The existing structure at Front Street is not overtopped by the 100-year ultimate water surface elevation but does cause some structure flooding upstream. Adding capacity to the existing 2-10' x 7' box culverts and also channel improvements will reduce the upstream flooding. Adding one more 10'x 7' box culvert will reduce the upstream water surface elevation from 504.92 to 502.53. The proposed channel from Front Street to the Douglas Crossover is a concrete lined channel with a 20' bottom and 2:1 side slopes.

Beckham/Houston

The existing structure under Beckham/Houston Street is 2-7'x 7' box culverts. The roadway is overtopped by 1.91' of weir flow. This area also has several structures that are built over the top of the structure. These structures are also in the floodplain. Adding two 96" RCP's to this system will lower the 100-year ultimate water surface elevation from 517.91 to 515.41, eliminating the roadway overflow and also removing the structures from the floodplain. Construction in this area will be difficult due to the existing building structures. More detailed survey will be needed in order to do a detailed design in the area.

Missouri Pacific Railroad

Existing structure under the rail consists of one 11' RCP and one 6' RCP. The capacity of these two pipes is far less then the design flow of the 100-year ultimate storm. This causes significant flooding upstream. Replacing the two pipes with larger boxes will significantly reduce the water surface elevation upstream and alleviate flooding. Adding two 12'x12' box culverts reduces the upstream water surface elevation by over 10'. Open cutting of railroads is usually not an option for construction so other construction methods will most likely be needed in this area. One alternative for construction could be jacking new boxes through the existing railroad berm. Another alternative would be to build a temporary rail until construction is complete. Adding capacity to structures like this one needs more detailed hydrology and hydrologic modeling. Adding capacity to the structure will greatly affect valley storage and may increase flows downstream.

Black Fork Creek

State Hwy 69

The existing structures are two bridges, one for northbound traffic and one for southbound traffic. The 100-year ultimate water surface elevation overtops each structure by several feet. In order to alleviate the roadway flooding in the area the bridge structures need to be replaced and the roadway needs to be raised to a minimum of 433.5. This will provide a 1' freeboard from the water surface elevation to the top of the roadway, although the low chord of the bridges remains in the floodplain. The proposed bridge structures would maintain a natural channel under the bridge and the bridge opening would be 400' wide. The ultimate water surface elevation upstream of the bridge structures will rise from 431.5 to 432.53, over one 1'. There are no structures that will flood immediately upstream of the structure due to this rise. If a rise is not acceptable a larger opening would be necessary.

Loop 323

The existing structures are two bridges, one for each direction of traffic. The 100-year ultimate water surface elevation overtops each structure by almost two feet. In order to alleviate the roadway flooding in the area the bridge structures need to be replaced, the roadway raised and the channel downstream of structure improved. The channel improvements consist of 3000-foot of trapezoidal channel that is grass lined with 3:1 sides and a 40' bottom. The two bridge structures need to be replaced with bridges that have a 200' top width, 40' bottom and concrete lined with 2:1 sides. The roadway needs to be raised from 438 to a minimum of 440 to provide 1' of freeboard. The low chord of these bridges would still be within the 100-year ultimate water surface elevation.

Gentry Parkway

The existing structure is 6-8'x 7' box culverts. The roadway is overtopped by the 100-year ultimate water surface elevation by over 4'. Replacing the culverts with a bridge and downstream channel improvements will be necessary to remove this structure from the floodplain. Channel improvements consist of a concrete trapezoidal channel with 2:1 sides and a 40' bottom to extend from Gentry to a location 500' downstream. From that point the channel will change to a grass lined trapezoidal channel with 3:1 sides, 30' bottom to a location downstream to the next structure. Also in order to reduce the water surface elevation the channel overbanks needs to be cleaned and thick brush removed. The proposed bridge structure will need to raise the road from 463.88 to 468. The structure would need a top width of 300', a concrete lined pilot channel with 2:1 sides and a 40' bottom. These improvements will lower the upstream water surface elevation from 467.89 to 466.2, removing the structure from the floodplain.

Railroad/Commerce/Erwin/Front

In order to reduce flooding problems in this area these four structures would need to be improved together.

The existing structure at the railroad consists of 4-10'x 10' box culverts. The railroad is not overtopped by the 100-year ultimate water surface elevation but it does cause significant flooding upstream of the structure. By adding 4 more boxes to this system the water surface elevation can be reduced from 479.3 to 471.31, almost 8-foot. Open cutting of railroads is usually not an option for construction so other construction methods will most likely be needed in this area. One alternative for construction could be jacking new boxes through the existing railroad berm. Another alternative would be to build a temporary rail until construction is complete. Adding capacity to structures like this one needs more detailed hydrology and hydrologic modeling. Adding capacity to the structure will greatly effect valley storage and may increase flows downstream.

The existing structure at Commerce Street consists of 3-10'x 9' box culverts. The improvements at the railroad drop the downstream water surface elevation at Commerce Street from 481.10 to 478.40. In order to reduce the downstream water surface elevation even more additional channel improvements will be needed. The proposed channel improvements consist of a trapezoidal grass lined channel with a 30' bottom and 3:1 sides. The grass-lined channel will extend 200' downstream of the Commerce Street structure. The existing culverts under Commerce Street do not have the capacity to carry the 100-year ultimate flow. Replacing the structure and lowering the structure will also be necessary to remove the roadway from the floodplain. The proposed structure is 10-10'x12' box culverts with the flow line lowered 1' from the existing culverts. The upstream water surface elevation with these improvements and the Railroad improvements is lowered from 483.25 to 478.91.

The structure at Erwin Street is 2-10'x 10' box culverts. The roadway is overtopped by the 100-year ultimate water surface elevation by almost 4'. In order to lower the water surface to remove the roadway and upstream structures from the floodplain, improvements need to be made to the structure and also the downstream channel. The downstream channel improvements include a trapezoidal grass lined channel with a 30' bottom and 3:1 sides. The channel will extend from the Erwin Street structure to the Commerce Street structure. Also the overbanks need to be cleaned to reduce the n values to approximately 0.045. The existing structure at Erwin Street needs to be replaced with a bridge. The bridge needs to have a top width of 210' and the top of road needs to be raised almost 4' to 490'. The channel needs to have a 4' pilot channel and be concrete lined with 2:1 sides. These improvements drop the upstream water surface elevation from 490.17 to 488.61.

Existing structure at Front Street is 3-10'x 10' box culverts. The 100-year ultimate water surface elevation overtops the roadway by over 2.5'. To remove this structure from the floodplain channel and structure improvements will be needed. Proposed improvements to the channel include cleaning overbanks to reduce the n values to approximately 0.045, creating a grass lined trapezoidal channel from Front Street to Erwin Street. Channel needs a 30' bottom with 3:1 side slopes. The channel reduces the downstream water surface elevation at Front Street from 493.90 to 490.5. The existing box culverts at Front Street are under sized to carry the 100-year ultimate design flow. Replacing the structure with 10-12'x 10' box culverts reduces the upstream water surface elevation from 495.56 to 492.75, almost three feet.

E. Fifth Street

The existing structure at E. Fifth Street is 4-7'x 5' box culverts. The existing roadway is overtopped by over 2' of weir flow. Replacing the box culverts and also improving the channel downstream is needed to remove the roadway from the floodplain. The channel improvements include a trapezoidal grass lined channel with a 20' bottom and 3:1 side slopes that extend 1000' downstream. The existing structure under the roadway is under capacity to carry the 100-ultimate design flow. Replacing the structure with 6-10'x 10' box culverts will reduce the upstream water surface elevation from 418.15 to 413.5. This will remove the roadway and also several structures from the floodplain.

EROSION AND EROSION CONTROL

There are a number of factors related to the erosion process within a natural stream system and improved channels. These are variation in rainfall intensity, duration and frequency; overall channel slope; type of soil; degradation of protective vegetation; development and increased impervious area; and structures constructed in and along the channel. Possible options for controlling erosion are divided into general categories of structural improvements, biological controls and biotechnical improvements. All of the improvement options require continual monitoring and maintenance to ensure the long-term success of the improvement.

Structural improvements could include many types of improvements and various materials and could include the following:

1. Full Channel Lining
2. Partial Channel Lining
3. Drop Structures and Grade Control Structures
4. In-Channel Dams
5. Slope Stabilization and Channel Excavation
6. Retaining Walls

The use of biological measures to control erosion is the use of vegetation that will provide protection and add strength to the channel side slopes. Under some circumstances, vegetation can provide a buffer area between high velocities and the channel soils. Vegetation can strengthen the slopes by the effect of the root system and the uptake of soil moisture by the planting. The additional benefit of biological control includes the aesthetic value of the vegetation and the habitat provided for the wildlife along the stream. Biotechnical improvements combine the benefits of both structural and biological controls. Even though the use of biological measures is not recommended as the primary means of erosion protection, it can be used as a successful erosion control plan. In general, biotechnical control should attempt to incorporate the strength of both structural and biological elements. The inclusion of biological elements will increase the maintenance required of improvements, but will provide stabilizing benefits to warrant its use.

Several areas through out the City of Tyler have erosive problems. NDMCE looked at a few of these areas in the proposed improvement areas mentioned above. These areas include West Mud Creek at New Copeland and Black Fork Tributary D at Front Street. These project not only deal address flooding they also looked at erosive velocities and erosive problems noted in the area.

FLOODWAYS

Floodways are defined as the areas that can be filled or encroached upon for the discharge of the base flood so that the cumulative increase in water surface elevation is not more than a designated amount. That designated amount is not to exceed one foot for FEMA floodways.

As part of this study, NDMCE modeled the 100-year ultimate floodways. In areas where there was currently a FEMA floodway NDMCE tried to match them the best as possible. The areas with existing floodways are West Mud Creek main channel, West Mud Creek Tributary A, West Mud Tributary C, West Mud Tributary B, Black Fork Creek main channel, Willow Creek, and Black Fork Creek Tributary D. All other FEMA areas were Zone A and did not have current floodway delineation to match. After an analysis was complete for the 100-year ultimate floodways, they were mapped along with the floodplain delineation. Floodways can be seen in the exhibits in Appendix D.

PROBABLE CONSTRUCTION COSTS

Initial opinions of probable construction costs were calculated from conceptual designs and a projection of construction quantities. The construction cost was adjusted with a 30 percent contingency to determine the total construction costs. The total cost for each project does not include an estimate of engineering fees and other special services not included in the basic engineering services and does not include construction inspection fees to the City.

A summary of the opinion of probable construction costs for all of the improvement projects are provided in Table 63 and full calculations for each project are located in Appendix G. These costs vary from as little as $100,000 to over $6,000,000, are based on conditions at the time of this report for current construction costs and should be used as a guide to measure the magnitude of the construction project. These costs are expected to increase over time as construction costs increase.

Table 63 – Summary of Probable Construction Costs

Location	Cost	Location	Cost
WMC at HWY 69 South	$ 2,596,800	Harris at FM 2767	$ 1,485,300
WMC at Shiloh	$ 665,000	Harris at HWY 31	$ 962,800
WMC at New Copeland	$ 208,000	Willow at Loop 323	$ 215,800
WMC at Loop 323	$ 99,800	Willow at Erwin	$ 1,100,500
WMC Trib B at New Copeland	$ 261,800	Blackfork D4 at Fifth	$ 199,200
WMC Trib C at Loop 323	$ 165,600	Black fork D3 at E Front	$ 252,900
WMC Trib C at Broadway	$ 483,900	Black Fork D2 at Beckham	$ 453,200
WMC Trib C at FM 2493	$ 549,300	Black Fork D1 at Gentry	$ 468,700
WMC Trib A at Loop 323	$ 1,657,300	Black Fork D at Front	$ 1,352,500
WMC Trib A-2 Private Culvert	$ 285,700	Black Fork D at Railroad	$ 206,700
WMC Trib A-2 at Jacksonville	$ 183,300	Black Fork at HWY 69	$ 6,242,000
WMC Trib A-2 at Loop 323	$ 747,300	Black Fork at Loop 323	$ 2,212,600
Henshaw at HWY 69 South	$ 1,462,100	Black Fork at Gentry	$ 2,298,500
Shackelford at CR 110	$ 783,100	Black Fork at Railroad*	$ 5,459,000
Indian at Loop 323	$ 387,300	Black Fork at Fifth	$ 412,100
Gilley at CR 2120	$ 598,000		

PROJECT PRIORITIZATION

A point system was developed for individual projects so that they could be prioritized based on engineering, safety and environmental considerations. The following provides a summary of the categories used in the prioritization system.

1. Relative Cost – 1 to 5 points was provided based on the opinion of probable construction cost. For projects with a relative cost of less then $500,000, a maximum of 5 points were given. Projects with anticipated costs greater then $2,000,000 only received 1 point.
2. Construction Negatively Impacts Others – In areas where construction will effect adjacent land owners, a low point value of 1 was assigned. In areas with little to no effect on adjacent landowners, the highest value of 5 was given. Negative impacts would consist of access to property due to raising roadway, expansion of channels that take additional right-of-way or having to move access to a property due to construction equipment.

3. Dependence on Other Projects – Projects that are independent of other improvements were given a maximum of 5 points. Other projects that could only be completed in conjunction with other improvements were given fewer points.
4. Minimize Future Improvements and Maintenance – Projects that would offset the cost of future improvements in the area or would cut down on maintenance costs were given the most points. Other projects were given lower point values.
5. Right-of-Way Acquisition – Projects that would stay inside the existing easements were given the highest points. Projects that would require an additional ROW were given fewer points.
6. Involvement with Railroad – Projects that would involve coordination and cooperation of the railroad were given the least amount of points.
7. Permitting – Projects that would require an extensive permitting process were given the least points. Projects that require very little permitting were giving the most points.
8. Depth of Roadway Overtopping – 0' to 1' of flooding over the roadway was given a value of 1, 1' to 2' of overtopping was given a point value of 3, and everything over 2' of flooding was given a 5.
9. Potential Loss of Life – The greater the potential of flooding and the heavier the use of a roadway the greater the potential for loss of life. A value of 1 was given if that potential was very low and up to a value of 10 was given if the potential loss of life was significant.
10. Potential Loss of Private Property – The greater the extent of flooding in an area the greater risk of loss of private property. A value of 1 was given in areas that have a low risk of loss of property, up to a value of 10 for high-risk areas.
11. Potential Loss of Public Property – The greater the extent of flooding in an area the greater risk of loss of public property. A value of 1 was given in areas that have a low risk of loss of property up to a value of 10 for high-risk areas.
12. Preservation of Environmental Features and Water Quality – Areas where environmental features were not preserved were given a value of 1, the better the project preserved the natural channel or added to water quality the higher the value to a maximum value of 10.

A matrix system was developed which weighs the overall benefits of each project area on the basis of a maximum of 72 points. A project ranking was then developed from the points that were accumulated in the priority analysis. Table 64 displays the matrix system and rankings from this procedure. Table 65 summarizes the ranking in numerical order based on this approach. This procedure provides a method by which to measure relative benefits of protecting the individual project area. This ranking should not be considered as final analysis of benefits, but should be used as a tool in developing an improvement strategy. Other consideration such as the construction order of projects and the ability to complete overall reaches of protection must also be considered.

TABLE 63 PRIORITIZATION FACTORS

Location	Cost ($)	CONSTRUCTION FEASIBILITY: Relative Cost	Roadway Use	Construction Negative Impact On Others	Construction Dependant on Other Projects	Minimize Cost Of Future Improvements & Maintenance	ROW Acquisition	Involvement with Railroad or others	Permitting	PUBLIC SAFETY: Depth of Roadway Overtopping	Potential Loss of Life	Potential For Loss Of Private Property	Potential For Loss Of Public Property	QUALITY OF LIFE: Preservation Of Environmental Features and Water Quality	PRIORITY EVALUATION: Rating	Priority Ranking
WMC at HWY 69 South	$ 2,596,800	1	1	1	3	3	3	3	3	5	8	8	8	2	49	10
WMC at Shiloh	$ 665,000	3	3	5	5	1	3	3	3	3	5	2	4	4	44	18
WMC at New Copeland	$ 208,000	5	5	3	3	3	1	3	1	3	5	3	4	3	42	20
WMC at Loop 323	$ 99,800	5	5	3	5	3	3	3	3	1	5	5	4	8	53	2
WMC Trib B at New Copeland	$ 261,800	5	5	3	5	1	3	3	3	1	2	5	3	8	47	13
WMC Trib C at Loop 323	$ 165,600	5	5	3	5	1	1	3	3	1	3	5	3	9	47	12
WMC Trib C at Broadway	$ 483,900	3	3	3	3	1	3	3	1	5	6	7	7	4	49	9
WMC Trib C at FM 2493	$ 549,300	3	3	3	5	3	3	3	3	5	6	7	7	8	59	1
WMC Trib A at Loop 323	$ 1,657,300	1	3	3	1	3	1	3	1	1	2	7	3	3	32	31
WMC Trib A-2 Private Culvert	$ 285,700	5	5	1	1	1	1	3	3	1	1	3	1	8	34	29
WMC Trib A-2 at Jacksonville	$ 183,300	5	5	3	1	3	1	3	1	3	5	2	4	5	41	22
WMC Trib A-2 at Loop 323	$ 747,300	3	3	3	1	3	1	3	1	3	5	2	4	3	35	28
Henshaw at HWY 69 South	$ 1,462,100	1	3	5	5	3	3	3	3	3	6	4	4	6	49	8
Shackelford at CR 110	$ 783,100	1	3	5	5	3	1	3	3	5	3	1	1	6	40	23
Indian at Loop 323	$ 387,300	5	5	3	5	3	3	3	3	3	3	1	1	5	43	19
Gilley at CR 2120	$ 598,000	3	3	5	5	3	1	3	3	5	2	1	1	5	40	25
Harris at FM 2767	$ 1,485,300	1	3	5	5	3	1	3	3	5	4	2	2	8	45	17
Harris at HWY 31	$ 962,800	1	3	3	5	3	1	3	3	3	5	2	2	6	40	24
Willow at Loop 323	$ 215,800	5	5	3	5	1	3	3	3	3	4	2	2	8	47	11
Willow at Erwin*	$ 1,100,500	1	3	1	1	3	3	3	1	5	9	9	9	3	51	5
Blackfork D4 at Fifth	$ 199,200	5	5	3	5	3	3	3	3	3	3	4	4	7	51	4
Blackfork D3 at E Front	$ 252,900	5	5	3	5	1	3	3	3	3	3	4	4	8	50	7
Blackfork D2 at Beckham	$ 453,200	5	5	3	5	3	3	3	3	5	4	3	3	7	52	3
Blackfork D1 at Gentry	$ 468,700	3	3	3	5	1	3	3	3	5	4	2	3	8	46	15
Blackfork D at Front	$ 1,352,500	1	3	3	1	1	1	3	1	1	3	6	5	4	33	30
Blackfork D at Railroad	$ 206,700	5	5	1	5	3	1	1	1	1	3	5	3	8	42	21
Blackfork at HWY 69	$ 6,242,000	1	1	3	5	3	1	3	3	5	7	3	3	8	46	16
Blackfork at Loop 323	$ 2,212,600	1	3	3	3	3	1	3	1	5	8	6	5	5	47	14
Blackfork at Gentry	$ 2,298,500	1	1	3	3	3	1	3	1	1	5	6	5	4	37	27
Blackfork at Railroad*	$ 5,459,000	1	1	1	1	3	1	1	1	5	6	7	5	5	38	26
Blackfork at E. Fifth	$ 412,100	5	5	3	3	3	1	3	1	5	6	6	5	4	50	6
		1 – 2mill+ 3- 500k to 5- 0 to 100k	1 – Low 3- Medium 5- High	5 – Low 3- Medium 1- High	1 – Major Project 3 – Moderate Level 5 – No	1 – No 3 – Yes	3 - No 1 – Yes	3 - No 1 – Yes	3 - Minor 1 – Major	1 – 0' to 1' 3- 1 to 2' 5- Over 2'	1 – Low to 10 – High	1 – Low to 10 – High	1 – Low to 10 – High	1 – Low to 10 – High		

Table 65 – Project Ranking

Project Location	Ranking
WMC Trib C at FM 2493	1
WMC at Loop 323	2
Blackfork D2 at Beckham	3
Blackfork D4 at Fifth	4
Willow at Erwin	5
Blackfork at Fifth	6
Blackfork D3 at E Front	7
Henshaw at HWY 69 South	8
WMC Trib C at Broadway	9
WMC at HWY 69 South	10
Willow at Loop 323	11
WMC Trib C at Loop 323	12
WMC Trib B at New Copeland	13
Blackfork at Loop 323	14
Blackfork D1 at Gentry	15
Blackfork at HWY 69	16
Harris at FM 2767	17
WMC at Shiloh	18
Indian at Loop 323	19
WMC at New Copeland	20
Blackfork D at Railroad	21
WMC Trib A-2 at Jacksonville	22
Shackelford at CR 110	23
Harris at HWY 31	24
Gilley at CR 2120	25
Blackfork at Railroad	26
Blackfork at Gentry	27
WMC Trib A-2 at Loop 323	28
WMC Trib A-2 Private Culvert	29
Blackfork D at Front	30
WMC Trib A at Loop 323	31

FUNDING SOURCES

An evaluation of potential federal and state agency programs was made to identify potential financial assistance sources. The following outlines the information obtained concerning these potential funding sources.

City Funding

The City of Tyler uses two general sources for funding of drainage improvement projects. The Capital Improvement Projects are generally funded through the existing ½-cent sales tax. The City also has a Stormwater Utility Fund that receives funds through a fee that is included on the utility bills. These are the two basic sources of funds for City improvement projects.

Cost Sharing

In cases where both public and private property is involved in improvement projects, some communities have pursued cost-sharing between public and private sources. This includes cities that have implemented a cost-sharing program for some erosion control type projects. Another approach to the general concept of cost sharing would be the development of impact type fees within the identified improvement areas for the drainage improvements.

Community "Development" Block Grant Program

Some communities have pursued financial assistance for drainage project through the Community Block Grant Program. The program is generally administered through the Department of Housing and Urban Development. The primary statutory objective of the CDBG program is to develop viable communities by providing decent housing and a suitable living environment and by expanding economic opportunities, principally for persons of low- and moderate-income. The available programs include Entitlement Communities and State Administered CDBG's.

The State Administered CDBG must ensure that at least 70 percent of its CDBG grant funds are used for activities that benefit low- and moderate-income persons over a one-, two-, or three-year time period selected by the State. This general objective is achieved by granting "maximum feasible priority" to activities which benefit low- and moderate-income families or aid in the prevention or elimination of slums or blight. Under unique circumstances, States may also use their funds to meet urgent community development needs. A need is considered urgent if it poses a serious and immediate threat to the health or welfare of the community and has arisen in the past 18 months.

These funds are difficult to obtain and the request for funds would need to be carefully worded and targeted for specific areas. Funds are primarily restricted to economically depressed areas and the availability of funds would be in competition with other municipalities within depressed areas.

U. S. Army Corps of Engineers (USACE)

Section 205 funds are administered by the USACE for flood control improvements. These funds are targeted towards flood control. The Federal participation is limited to $5 million. The time frame from initial contacts with the USACE to project construction is usually about 5 years. Usually, historical flooding and repetitive damage would be required before a project could be considered.

The project evaluation proceeds in three phases. Phase I is the initial appraisal. It is generally undertaken by the USACE and fully funded by the Federal program. This phase usually takes 6 months to a year to complete. Phase 2 is the feasibility level study. Costs for the study that are in excess of $100,000 are shared 50-50 between the USACE and the local community. The local community cost sharing includes in-kind services to facilitate public meetings, contracted services, and community services. Once an agreement is reached between the local community and USACE regarding the type of project and extent, the plans and specifications are developed by the USACE in the project cost. Projects must be justified by a cost benefit ratio in the initial appraisal and feasibility phases. Construction costs are generally shared at 65% federal and 35% local sponsor.

Texas Water Development Board (TWDB)

The TWDB is currently funding a portion of the Master Drainage Study through a grant for planning projects. There are other opportunities for low interest loans through the TWDB.

The TWDB administers the State Loan Program which provides financial assistance funds dedicated to financing flood control projects. The types of financial assistance provided are low interest loans. Low-interest loans are available for the following pertinent items:

1. development of flood management plans;
2. construction of storm water retention basins;

3. enlargement of stream channels;
4. modification or reconstruction of bridges;
5. acquisition of floodplain land for use in public open space;
6. acquisition and removal of buildings located in a floodplain; and
7. relocation of residents of buildings removed from a floodplain.

The low interest loans are required to have a security instrument that typically requires a public bond. The rates are set based on the Board's borrowing costs and the risk exposure. The maximum financing life of a loan is 50 years.

The Board's staff reviews and approves the design plans and specifications, bid documents and bid award. During the construction process, the Board's staff monitors the progress.

The TWDB also administers the Flood Mitigation Assistance (FMA) Program which is a federal grant program. The TWDB administers the program under an agreement with the Federal Emergency Management Agency (FEMA). This program provides federal funding to assist States and communities in implementing measures to reduce or eliminate the long-term risk of flood damage to buildings, manufactured homes, and other structures insurable under the National Flood Insurance Program (NFIP). This program is a pre-disaster grant program.

This program can fund Project Grants for projects that reduce the risk of flood damage to structures that are insurable under the NFIP. Such activities include:

- Acquisition of insured structures and real property;
- Relocation or demolition of insured structures;
- Dry flood proofing of insured structures;
- Elevation of insured structures; and
- Minor, localized structural projects that are not fundable by other State or Federal programs.

Texas Department of Transportation (TxDOT)

Several of the bridges and culverts that were analyzed for potential improvements included multiple highway and Farm-to-Market roads. Any improvements in these areas should be coordinated with TxDOT to determine if there are any plans to improve these roadways. Any future plans to improve these roadways by TxDOT may help to reduce the City's overall cost for resolving these problem areas. These potential areas include the following:

1. Structures at multiple crossing of US 69.
2. Structures at multiple crossings of Loop 323.
3. Structures on Highway 31.
4. Structures on FM 2493 and FM 2767

Normal TxDOT drainage improvements may not meet the requirements of these areas from the Master Drainage Plan. The City of Tyler may be responsible for the betterment cost to upgrade these drainage systems.

REGULATORY

There are several entities with regulatory requirements that impact the ability to perform improvements within the waterways in the City of Tyler. These regulations are generally related to impacts on the floodplain elevation at improvement sites and environmental issues. These include the Federal Emergency Management Agency (FEMA) regulations, Section 404 (Clean Water Act) permitting related to activities within jurisdictional waters of the United States, and construction activity requirements. The following paragraphs provide a brief discussion on the requirements related to the possible flood control improvements that are included in the Master Drainage Plan.

FEMA Requirements

There are two general issues related to FEMA in regards to the Master Drainage Plan improvements. These issues are the FEMA requirements for flood plain management and the difference between the FEMA models and the models developed for this study.

The National Flood Insurance Program (NFIP) defines the requirements for construction activities within the floodway of a studied stream. Several of the creeks that are included in this study are also detailed studied streams in the Flood Insurance Study for the City of Tyler. Any improvements related to the Master Drainage Plan would be located within the floodways of these studied streams. It is anticipated that a submittal would be required to FEMA for most of the improvement plans. This could also require public notification for the adjustment of any flood elevations or floodway limits within a designated project area.

The Master Drainage Study was being completed at the time of the recent FEMA update for the City of Tyler and Smith County. Therefore, the results of the Master Drainage Study have not been incorporated into the FEMA models or report. The report exhibits displays the graphical difference between the 100-year floodplain and floodway limits for the Master Drainage Study as compared to the updated FEMA information. Any submittals to FEMA for drainage improvements would need to be based on the current FEMA models or the submittal would need to include the Master Drainage Study information as updated baseline information within the project area. An alternative would be for the City of Tyler to submit all of the Master Drainage Study data as a LOMR submittal to update all of the data for the City to FEMA.

404 Permitting

A regulatory permit would be required for most drainage improvement projects to satisfy the requirements under Section 404 of the Clean Water Act (CWA). This permit is required when jurisdictional waters of the United States (or wetlands) are impacted by the placement of fill or some excavation activities. Jurisdictional waters are typically the bottom of the channel and the sideslopes up to the ordinary highwater mark. The study streams would all include some jurisdictional waters. Any culvert, bridge or channel improvement project would require either a nationwide or individual 404 permit.

A nationwide permit can usually be obtained for culvert and bridge projects. There are limitations as to the extent of the improvements that are allowed under Nationwide Permit 14 for Linear

Transportation Projects. These include notification requirements, stream distance limitations and the permit does not allow for stream channelization.

If a project does not fall under a nationwide permit then an individual permit is required. Obtaining an individual 404 permit is a longer and more involved process. The submittal for the permit will require the completion of an Alternative Analysis. The permit process requires that that the impacts of the project be evaluated in three steps. These steps are as follows:

1. Methods by which impacts to jurisdictional areas could be avoided.
2. Methods by which unavoidable impacts can be minimized.
3. A mitigation plan to offset the impacts that cannot be avoided or minimized.

Any Alternatives Analysis will need to document the extent of and permanence of the beneficial and/or detrimental effects of the potential project activities, the need for the proposed activity, and the practicability of using reasonable alternative methods to accomplish the desired objectives. Of great importance to the project evaluation is the USACE public interest review. The public and private benefits and detriments of all factors relevant to the project are evaluated and balanced. Relevant factors may include conservation, economics, wetlands, cultural resources, navigation, fish and wildlife values, water supply, and any other factors judged important to the needs and welfare of the people. Natural functions of the creek channel and associated riparian corridor will be considered which may include water quality improvement (through sediment retention and nutrient removal and transformation), habitat cover and food sources, aesthetics, greenbelts and natural amenities, screening and noise abatement, and carbon dioxide uptake.

Compensatory mitigation will be required for unavoidable impacts and may include restoration or reestablishment of functions and characteristics that have either ceased to exist or exist in a substantially degraded state. Restoration can include the reestablishment of natural hydrology to an area by the removal or disabling of manmade structures. Mitigation can also be provided by enhancement of an aquatic resource or adjacent area such as improvement of the diversity of the existing plant community, active management for wildlife enhancement and/or ecosystem reestablishment, and minor activities to restore hydrology to its natural condition. Mitigation may also involve creation of an appropriate aquatic resource where one did not formerly exist or preservation of an ecologically important resource in perpetuity to protect a high quality resource that might otherwise be lost. Restoration and enhancement are preferred to creation because they are normally less expensive and more successful, and less likely to adversely affect existing upland and open water habitats. The USACE will require assurances that mitigation areas are established as proposed and maintained as little as possible. Vegetation that does not survive the 2-year establishment period will have to be replaced.

Mitigation ratios (i.e., the amount of aquatic resource to be created, enhanced, restored, or preserved to compensate for the impacts to an aquatic resource resulting in lost functions) can be variable depending on project specific circumstances. Since replacement of lost functions from impacted aquatic resource areas is the purpose of mitigation, the proposed mitigation should address the replacement of those functions. Therefore, development of mitigation areas is generally directed toward replacement of the type of aquatic resource area impacted or "in-kind" mitigation. Current USACE policy regarding mitigation for impacts to stream channels with mature riparian corridors is approximately 2:1 ratio, depending on the quality of the area being disturbed.

As a part of the 404 permit process, it is necessary to satisfy the requirements of Section 401 of the Clean Water Act (CWA) which constitutes a Water Quality Certification. This certification is issued by the Texas Commission on Environmental Quality (TCEQ). This certification is required from the State to secure a 404 permit from the USACE and is usually handled as a part of the 404 permit process.

Securing the necessary permits for a project can be a long and involved process. Typical permitting can require anywhere from 6 months to well over 1 year. The time period can vary for which a 404 permit is valid. Some permits only have a 2-year duration while others are for longer periods. Extensions of the permits may be granted, but should not be counted on in the planning of a project. Adequate time must be included in the permit period to allow for the determination of appropriate project funding strategies, securing the necessary funding (including bond sales), and preparing the construction plans.

MASTER DRAINAGE PLAN

The preceding sections of this report provide the base information needed to develop an overall Master Drainage Plan (MDP) and to provide an implementation approach for the MDP. The MDP is based on data from the reconnaissance, determination of flooding conditions, evaluation of improvement alternatives, opinion of probable construction costs, prioritization of project areas, evaluation of funding sources, and the consideration of potential regulatory requirements.

The project rankings developed from the priority analysis provide a basis for determining future improvements. Some of the project areas have the potential towards funding from other sources, while a majority of projects may need to be funded by the City of Tyler.

The ranking of the top ten projects and pertinent data related to costs and potential funding sources are shown in Table 66. The projects shown in Table 66 are ranked high due to their relative cost, potential loss of life, potential loss of property, and reduction of flood levels. All improvements that have the potential of receiving funding from other sources should be given high priority in the planning and design phases. This may allow those improvements to be incorporated into the overall planning with other agencies and therefore reduce the overall cost to the City.

Table 66 – Master Drainage Plan Projects

Location	Cost ($)	Permitting	Priority Ranking	Improvement Description	Funding Options
WMC Trib C at FM 2493	$ 549,300	Minor	1	Installation of 4 - 8' by 8' RCB's lowers flood levels by over 9 feet, eliminates 4.5' of road overflow and removes approximately 20 residences from the floodplain.	City funding, TxDOT, USACE, TWDB Loan and FMA.
WMC at Loop 323	$ 99,800	Minor	2	Installation of 4 - 6' by 6' RCB's lowers flood levels by approximately 3 feet, eliminates approximately 0.9' of road overflow and removes approximately 2 structures from the floodplain.	City funding and TxDOT.
Blackfork D2 at Beckham	$ 453,200	Minor	3	Installation of 2 - 10' by 7' RCB's lowers flood levels by approximately 6.4 feet, eliminates approximately 1.7' of road overflow and removes multiple structures from the floodplain.	City funding, USACE, TWDB Loan and FMA.
Blackfork D4 at Fifth	$ 199,200	Minor	4	Installation of 1 - 8' by 8' RCB lowers flood levels by approximately 3.3 feet and eliminates approximately 1.8' of road overflow.	City Funding
Willow at Erwin	$ 1,100,500	Major	5	Installation of channel and box culvert improvements lowers flood levels by approximately 5.5 feet, eliminates approximately 1.5' of road overflow and removes multiple structures from the floodplain..	City funding, USACE, TWDB Loan and FMA.
Blackfork at E. Fifth	$ 412,100	Major	6	Installation of 6 - 10' by 10' RCB's lowers flood levels by approximately 4.6 feet, eliminates approximately 2' of road overflow and removes multiple structures from the floodplain.	City funding, USACE, TWDB Loan and FMA.
Blackfork D3 at E Front	$ 252,900	Minor	7	Installation of 2 additional 10' by 10' RCB's lowers flood levels, eliminates road overflow and removes structures from the floodplain.	City funding, USACE, TWDB Loan and FMA.
Henshaw at HWY 69 South	$ 1,462,100	Minor	8	Installation of a bridge with a 120' top width lowers flood levels by approximately 2.2 feet, eliminates approximately 1.2' of road overflow and removes multiple structures from the floodplain.	City funding and TxDOT.
WMC Trib C at Broadway	$ 483,900	Major	9	Installation of 3 additional 8' by 8' RCB's lowers flood levels by approximately 4', eliminates approximately 4' of road overflow and removes structures from the floodplain.	City funding, USACE, TWDB Loan and FMA.
WMC at HWY 69 South	$ 2,596,800	Minor	10	Installation of a bridge with a 90' top width lowers flood levels by approximately 3.3 feet and eliminates approximately 2.7' of road overflow.	City funding and TxDOT.

An important element in forming an overall program is the consideration of the interaction of the individual improvement reaches. Some improvements are independent of other projects while the success of others will be based on the interaction of individual improvement elements. A general principle to follow is first reduce potential flood discharges and to begin construction at the downstream end of inter-related projects when possible.

Funding Summary

Based on the previous information it appears that there are not many funding sources that would be available to assist in the construction of proposed drainage improvements. The only likely funding opportunities appear to be through the Stormwater Utility Fund, coordination with TxDOT to incorporate necessary improvements into the plans for future roadway projects, and the Flood Mitigation Assistance Program.

Funding opportunities through the community block grants seem less likely because of the competition with areas that may be available to demonstrate "hardship" conditions for completing projects. Funding through USACE Section 205 projects is not likely since there is not a history of repetitive flooding and damages in this area.

Plan Implementation

This study has provided the engineering information necessary to develop an overall Master Drainage Plan for the City. The actual implementation of the Master Drainage Plan, and its individual component projects, will be influenced by factors other than pure engineering parameters. Some of these additional parameters have been considered in the project prioritization. There are other significant factors in an implementation approach that are not incorporated into the prioritization scheme. Additional factors that are key in any implementation are establishing the project funding, addressing areas experiencing development pressures, and other pertinent local issues. This report has addressed possible funding partners but has not included other development and local issues into the prioritization approach. The City should review the Master Drainage Plan and adjust the plan as necessary to address these additional non-engineering considerations. The initial step in the plan implementation needs to be to define the funding for each project. The City should coordinate with TxDOT and the TWDB to determine if any of the specific projects can be incorporated in future TxDOT funded improvements or are candidates for the Flood Mitigation Assistance Program through the TWDB. Projects that are not eligible for these programs will probably need to be funded by the City.

PUBLIC MEETINGS

As part of the contract with the Texas Water Development Board the City of Tyler was required to hold three individual public meetings. The first public meeting was the general project kickoff meeting that was held on November 17th of 2004. The purpose of this meetings was to provide an overview presentation to outline the scope of the project and to provide a forum for interested citizens to express their concerns regarding flooding problems within the City. Comments and questions were accepted from the public after the presentation. There were several discussions related to local flooding issues, but there were no pertinent public comments or questions concerning the project.

A second meeting was held on August 2nd of 2006 at approximately the 50% completion point of the project. A short presentation was made to provide a status update on the project. Exhibits were provided, showing 100-year ultimate floodplain limits and elevations, for review and comment by the public. After the presentation members of the audience were given the opportunity to review the posted exhibits Questions were then directed to City Officials attending the meeting, and representatives from NDMCE. Explanations of the exhibits were provided, some areas of revisions were noted, but no significant comments were provided related to the project.

A third and final meeting was conducted on February 6th of 2008 to finalize the project. A short PowerPoint presentation was conducted that provided an overview of the project scope and discussed details of the final product that would be given to the City of Tyler. After the presentation the audience was given the opportunity to ask questions and give input pertaining to the Master Drainage Study. Final floodplain delineation maps were posted for public review. Written questioner forms were also given out for any questions about the study. There were several comments related to local flooding and erosion problems, but no significant questions were taken with respect to the study. The written forms, received after the public meeting, also did not have any pertinent comments.

SUMMARY

The Master Drainage Study was performed for the City of Tyler, in conjunction with the Texas Water Development Board, to update and develop the floodplain information for twelve watersheds identified by the City of Tyler and to prioritize future flood control improvement projects. The completion of these engineering tasks included site reconnaissance, data collection, review of existing floodplain information from both the City and Smith County, development of new hydrology models, development of new hydraulic models, evaluation of existing and future 100-year floodplain, determination of flood hazard areas, development of updated floodways, consideration of funding sources and recommendations for future flood control improvement projects. The purpose of this plan is to provide the City with updated floodplain, floodway and flooding information and to establish a program for future floodplain management improvements.

The scope of the Master Drainage Study included twelve main creeks and several tributaries within the City limits and extra-territorial jurisdiction (ETJ) as identified by the City. The studied streams included:

Black Fork Creek
Butler Creek
Gilley Creek
Harris Creek
Henshaw Creek
Indian Creek
Little Saline Creek
Ray Creek
Shackelford Creek
West Mud Creek
Wiggins Creek
Willow Creek

The Master Drainage Study provides the City of Tyler with the following information:

- Updated hydrologic and hydraulic models for the detailed study areas.
- Hydrologic and hydraulic data for the approximate study areas.
- Updated floodplain and floodway mapping and comparisons to FEMA mapping.
- Identification of flooding and erosion problem areas.
- Evaluation of improvement alternatives for identified problem areas and associated flood reduction.
- Cost opinions for identified improvements.
- Project prioritization for 31 specific areas.
- Development of a Master Drainage Plan.
- Completion of a Master Drainage Study Report that summarizes the overall project.

The completion of the Master Drainage Study included the development of detailed hydrologic models for approximately 263 square miles of basins to define flood discharges ranging from the 2-year to the 500-year event. Flood discharges were determined for both existing conditions and the ultimate development conditions within the watershed. The resulting flood discharges were used in developing the hydraulic models and are summarized in Appendix B.

Detailed hydraulic models were developed for approximately 114 miles of streams. These hydraulic models represent existing conditions based on City topographic data and field survey information for identified stream cross-sections, bridges and culverts. The hydraulic models generated water surface elevations that were used to delineate the limits of the 100-year floodplain. The resulting hydraulic results are provided in Appendix C. The hydraulic modeling also included the generation of floodways that are in accordance with FEMA guidelines. The floodplain and floodway limits were mapped on the available topographic maps. The corresponding floodplain mapping and flood profiles are provided in Appendices D and E (bound separately), respectively.

The hydraulic results and mapping provided information on the extent of flooding throughout the City. This information showed that existing drainage structures were overtopped by up to 11 feet. Significant areas of road overtopping and structure flooding were identified and evaluated to determine the extent of future improvements required to reduce the impact of flooding and the corresponding risk. Thirty-one locations were identified for evaluation. Hydraulic models were generated at these locations to determine the scope of improvements necessary to reduce overtopping and structure flooding. Cost opinions (provided in Appendix G) were developed for each of the improvement areas to identify the magnitude of the future costs as compared to the benefits produced by the proposed improvements.

The flooding information and cost opinions provide the basic information necessary for the project prioritization. Each of the 31 project areas were evaluated and ranked based on selected parameters. The resulting rankings were the basis for the overall Master Drainage Plan. Table 67 summarizes the results of the Master Drainage Plan development.

Table 67 – Master Drainage Plan Summary

Priority Ranking	Location	Cost ($) *	Improvement Description	Project Benefits		Funding Options
				Flood Reduction	Benefits	
1	WMC Trib C at FM 2493	$ 549,300	Installation of 4 - 8' by 8' RCB's.	9 feet	Eliminates 4.5' of road overflow and removes 20 residences from the floodplain	City funding, TxDOT, USACE, TWDB Loan and FMA.
2	WMC at Loop 323	$ 99,800	Installation of 4 - 6' by 6' RCB's.	3 feet	Eliminates 0.9' of road overflow and removes 2 structures from the floodplain	City funding and TxDOT.
3	Blackfork D2 at Beckham	$ 453,200	Installation of 2 - 10' by 7' RCB's.	6.4 feet	Eliminates 1.7' of road overflow and removes multiple structues from the floodplain	City funding, USACE, TWDB Loan and FMA.
4	Blackfork D4 at Fifth	$ 199,200	Installation of 1 - 8' by 8' RCB.	3.3 feet	Eliminates 1.8' of road overflow	City Funding
5	Willow at Erwin	$ 1,100,500	Installation of channel and box culvert improvements.	5.5 feet	Eliminates 1.5' of road overflow and removes multiple structues from the floodplain	City funding, USACE, TWDB Loan and FMA.
6	Blackfork at E. Fifth	$ 412,100	Installation of 6 - 10' by 10' RCB's.	4.6 feet	Eliminates 2' of road overflow and removes multiple structues from the floodplain	City funding, USACE, TWDB Loan and FMA.
7	Blackfork D3 at E Front	$ 252,900	Installation of 2 additional 10' by 10' RCB's.	1.0 feet	Eliminates road overflow and removes structues from the floodplain	City funding, USACE, TWDB Loan and FMA.
8	Henshaw at HWY 69 South	$ 1,462,100	Installation of a bridge with a 120' top width.	2.2 feet	Eliminates 1.2' of road overflow and removes multiple structues from the floodplain	City funding and TxDOT.
9	WMC Trib C at Broadway	$ 483,900	Installation of 3 additional 8' by 8' RCB's.	4 feet	Eliminates 4' of road overflow and removes multiple structues from the floodplain	City funding, USACE, TWDB Loan and FMA.
10	WMC at HWY 69 South	$ 2,596,800	Installation of a bridge with a 90' top width.	3.3 feet	Eliminates 2.7' of road overflow	City funding and TxDOT.

* All costs are based on 2007 $'s and include 30% contingencies. Costs do not include potential land acquisition costs, engineering, and permitting costs.

The final component of the Master Drainage Plan is the development of an implementation plan. The preceding table provides information on potential funding sources for the selected projects. Possible funding sources include TxDOT, for roadway projects, and the TWDB through the Flood Mitigation Assistance Program. The City should use the Master Drainage Plan as a tool to implement future projects and should adjust the plan as necessary to address additional local non-engineering considerations.