RESOLUTION NO. 4137

A RESOLUTION OF THE CITY COUNCIL OF THE CITY OF DENISON, TEXAS APPROVING THE CONCEPTUAL STORMWATER MASTER PLAN; INCORPORATING RECITALS; PROVIDING AN EFFECTIVE DATE; AND FINDING AND DETERMINING THAT THE MEETING AT WHICH THIS RESOLUTION WAS PASSED WAS OPEN TO THE PUBLIC AS REQUIRED BY LAW.

WHEREAS, the City Council of the City of Denison, Texas ("City") previously retained Huitt-Zollars Inc. to perform an evaluation of key areas of stormwater concern and to identify improvements that will allow the City to address and prevent current and potential stormwater hazards; and

WHEREAS, the City Council has determined that it is in the best interest of the citizens of the City to approve the Conceptual Stormwater Master Plan, a copy of which is attached hereto as **Exhibit "A"**.

NOW THEREFORE BE IT RESOLVED BY THE CITY COUNCIL OF THE CITY OF DENISON, TEXAS, THAT:

SECTION 1: Recitals Incorporated. The findings recited above are incorporated as if fully set forth in the body of this Resolution.

SECTION 2: Approval of Master Plan. That the City Council of the City hereby approves the Conceptual Stormwater Master Plan, which is attached hereto as **Exhibit "A"** and incorporated herein.

SECTION 3: Effective Date. This Resolution shall take effect immediately upon its passage.

SECTION 4: Open Meeting. It is officially found, determined and declared that the meeting at which this Resolution is adopted was open to the public and public notice of the time, place and subject matter of the public business to be considered at such meeting, including this Resolution was given, all as required by Chapter 551, as amended, Texas Government Code.

On motion by Mayor Pro Tem Crawley, seconded by Council Member Courtright, the above and foregoing Resolution was passed and approved at a Regular Meeting of the City Council of the City of Denison, Texas, on this the 7th day of August 2023.

Ayes: Hander, Courtright, Thorne, Gott, Massey, Crawley and Thomas
Nays:
Abstentions:

Janet Gott
JANET GOTT, MAYOR

CITY OF DENISON
Denison
TEXAS
TEXAS 1872

ATTEST:

Christine Wallentine
Christine Wallentine, City Clerk

EXHIBIT "A"

CONCEPTUAL STORMWATER MASTER PLAN

City of Denison Storm Water Master Plan (SWMP)

Denison
TEXAS

Conceptual Stormwater Master Plan Report

Submitted:
July 17, 2023

Prepared for:
City of Denison
300 West Main Street
Denison, TX 75020

Prepared by:
Huitt-Zollars, Inc.
5430 LBJ Freeway, Suite 1500
Dallas, Texas 75240

STATE OF TEXAS
JOHN HO
58873
LICENSED
PROFESSIONAL ENGINEER

HUITT-ZOLLARS

ADVANCEDESIGN

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EXECUTIVE SUMMARY

Introduction

Huitt-Zollars, Inc. (HZ) has been under contract with the City of Denison (the City) to provide a Conceptual Stormwater Master Plan (SWMP) to identify current drainage-related issues and aid in the planning of future drainage projects.

Project Location

The City of Denison encompasses an area of approximately 29-square miles within Grayson County, Texas. Denison is part of the two principal cities that make up the Sherman-Denison metropolitan statistical area. Located near the northern border of Texas with the State of Oklahoma, the City of Denison and its extra-territorial jurisdiction area drainages ultimately drain to the Red River. The City lies in a drainage area of approximately 64 square miles (Figure 1). Within these limits, there are several water storage reservoirs including Lake Randell, Loy Lake, Waterloo Lake, and Country Club Lake.

Purpose

The City of Denison, Texas wishes to prepare a 10-year Capital Improvement Program (CIP) to help solve flooding and stormwater related problems. To accomplish this objective, a Conceptual SWMP is to be prepared to develop planning-level hydrology, identify areas of potential flood hazards and determine where flood mitigation projects are needed. The extent of the current analysis is for the city limits, not including a newly planned area that borders Lake Texoma to the northwest. Extra-territorial areas outside of the city limits Once the areas of most concern are identified, conceptual solutions will be developed for each selected hazard area and budgets prepared for possible inclusion within a future planned Capital Improvement Program (CIP).

Model Approach

The study consists of developing two drainage models and interpreting the results of the models to develop conceptual projects for the City. The results are compared with FEMA Flood Insurance Rate Maps and known flooding issues to assist in the identification of problem areas and potential mitigation projects.

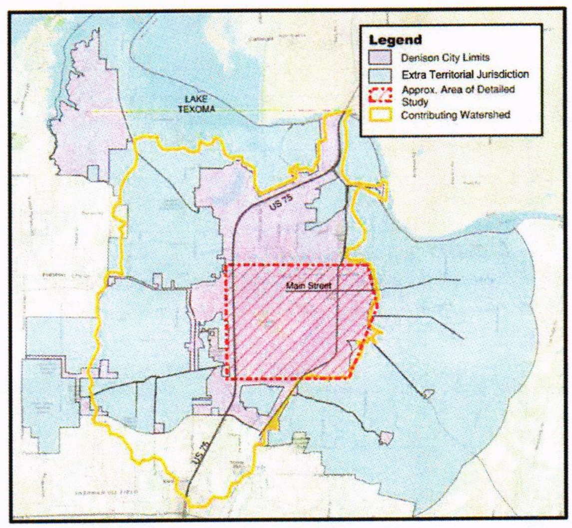

Figure E 1: Study Area Map

Proposed Improvement Projects

Thirteen potential improvement projects have been identified (Figure E 2), and cost estimates have been included in this study (Table E1). Potential improvement projects were identified based on the severity of the flood hazard. Project priority was categorized between high, medium, and low priority. High priority is based on a flood hazard that potentially impacts multiple structures; medium priority is assigned to projects where the flooding may pose a hazard to members of the public but do not impact multiple structures, and low priority is assigned to projects where flooding is anticipated but the hazards are limited to local flooding that does not significantly impact public safety or any structures. The improvements include stormwater infrastructure such as culverts, storm drains, and engineered channels and ditches.

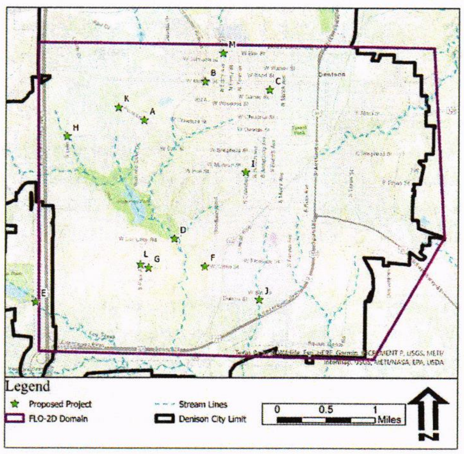

Figure E 2: Proposed Project Location Map

Table E 1: Summary of Cost and Priority for Each Project

PROJECT	LOCATION	COST	PRIORITY
A	Flora Lane and Rivercrest Circle	$ 2,690,000	High
B	Morton and Maurice	$ 3,548,000	High
C	Sears Street	$ 5,229,000	High
D	Loy Lake Road and Waterloo Creek Crossing	$ 1,272,000	High
E	Loy Lake Road Culvert Crossing	$ 1,279,000	Medium
F	Coffin and Woodlawn	$ 3,433,000	Medium
G	Stream Downstream of Coffin Street and Park Avenue	$ 768,000	Medium
H	Lillis Lane and Crawford Street	$ 1,689,000	Medium
I	Chandler Avenue near Munson Street	$ 415,000	Medium
J	Dubois Between Scullin and Barrett	$ 676,000	Medium
K	Flora Lane Near Lum Lane	$ 307,000	Low
L	Coffin Street and Park Avenue	$ 771,000	Low
M	Eddy Avenue Between Elm Street and Martin Luther King Street	$ 462,000	Low

A Flora Lane and Rivercrest Circle

Flooding along Flora Lane and at Rivercrest Circle results from street drainage along Flora and a stream that passes under Flora Lane and Rivercrest Circle. Flooding impacts have been shown to impact the homes south of Flora Lane, which are built with lower finished floor elevations than the road deck for Flora Lane. High priority is assigned to this project. The proposed improvements include a new storm drain inlet on Flora Lane, a shallow detention basin north of Flora within Katy Practice Fields, and upgrades to the culverts under Flora Lane.

B Morton Street and Maurice Avenue

Flooding along Morton, and at its intersection with Maurice, is the result of inadequate storm drain infrastructure along Morton Street and Maurice Avenue. Observed flooding along this project location has historically impacted buildings along the roadway. High priority is assigned for this project. The improvements include the addition of a storm drain system within both Morton and Maurice and conveyance to an existing stream.

C Sears Street

Drainage concentrates along Sears Street between Tone Avenue and Fannin Avenue, causing flooding. Analysis has shown that there is potential of flooding above the top-of-curb on Sears, which would flood the homes in the surrounding neighborhood. High priority is assigned to this project. The improvements include a storm drain system within Sears with a discharge to an existing drainageway.

D Waterloo Creek and Loy Lake Road

Waterloo Creek, downstream of Waterloo Lake is known to flood Loy Lake Road during intense storm events. Loy Lake Road is a major collector street which connects the communities east and west of Waterloo Creek, and high flooding would potentially be a safety risk for traffic as well as a hazard for emergency response between the two connected areas. High priority is assigned to this project. The proposed improvements include additional culverts to pass the 100-year discharge and downstream channel improvements.

E Loy Creek and Loy Lake Road

A stream crossing of Loy Creek under Loy Lake Road, downstream of the Loy Lake spillway, is undersized and prone to clogging with debris. Loy Lake Road is a major collector street for traffic heading east-to-west, and significant flooding of the roadway would potentially impact access, including access for emergency response to the connected communities. High priority is assigned at this location. The proposed improvements include the addition of culverts with capacity to convey the 100-year flow from Loy Creek and the Loy Lake spillway and a catch basin to capture debris on the upstream end of the proposed pipes.

F Coffin Street and Woodlawn Boulevard

Flooding at the intersection of Coffin and Woodlawn occurs when the capacities of an existing channel and culvert are exceeded. Medium priority level is assigned. A development is planned west of the intersection and would be affected by flooding at the intersection. The proposed improvements include improving the capacity of the channel and adding a storm drain system that receives the flow from the existing channel and passes that and street drainage along Coffin through the proposed community before releasing it to Waterloo Creek.

G Stream Downstream of Coffin Street and Park Avenue

Downstream of the intersection of Coffin Street and Park Avenue, storm water is collected in a stream where it transects residential properties located on Renaissance Drive. A number of these properties have individual stream crossings built on the stream bed which allow access to the other side of the properties. After a series of constructed stream crossings, which include concrete culvert crossings and a wood bridge crossing, water is released to a natural stream via a grade control structure. Downstream of this grade control structure, significant scour and lateral migration of the stream towards the residences on Renaissance Drive has been reported. Stream bank stabilization with a gabion retaining wall and installation of a new grade control structure is proposed to prevent a potential hazard to the foundational stability of the nearby homes. Due to the potential to undermine the stability of homes and the uncertainty of the hazard poses by the scour and erosion, a medium priority is assigned. Analysis of the stability of the soils near this location is advised so that the priority level may be re-evaluated.

H Intersection of Lillis Lane and Crawford

At the intersection of Lillis Lane and Crawford the existing culvert crossing under the road is undersized and prone to clogging. Flooding would overtop the drainage across intersection roadway and potentially cause flooding at nearby properties. Medium priority level is assigned. The improvements include adding box culverts with capacity to convey the 100-year flood.

I Chandler Avenue near Munson Street

Reported flooding incidents have occurred at the two homes on Munson Street northeast of the intersection of Chandler Avenue and Munson Street. This area lacks a curb and gutter system, using a roadside drainage ditch to pass flow downstream. The flooding occurs because the road and roadside ditch on the east side of Chandler Avenue is at a higher elevation than the ground and finished floors of the homes in question. It would be expected that for any significant rainfall event, some form of ponding of stormwater near the residential structures would occur. Medium priority is assigned to this project. The proposed improvements include adding a storm drain inlet into the alley where ponding initially occurs. The inlet would pass runoff via reinforced concrete pipe through a vacant residential parcel near the problem area and tie-in with the existing storm drainage system.

J Dubois Street

A stream crossing under Dubois Street between Scullin Avenue and Barrett Avenue is prone to flooding due to undersized culverts and heavy debris. Medium priority is assigned for this project. The proposed improvements include new culverts with capacity for the 100-year flood.

K Flora Lane and Lum Lane

A recently constructed home downstream of Flora Lane near Lum Lane could experience flooding if overbank flow rises above the right-side channel banks. Low priority is assigned to this project. The proposed improvements include straightening the channel and armoring of the right channel embankment.

L Coffin Street and Park Avenue

Flooding occurs at the intersection of Coffin and Park due to inadequate roadside ditches and culverts along Coffin Street. Flooding impacts are largely limited to the roadway along Coffin and the intersection at Coffin and Park. Low priority is assigned to this project. The improvements include upgrading the drainage infrastructure along Coffin, including culverts and roadside drainage.

M Eddy Avenue

Flooding hazards are presented at Eddy Avenue between Elm Street and Martin Luther King Street. Low Priority Level is assigned to this project because flooding impacts are largely limited to the street and sedimentation. The improvements include a storm drain system down Eddy Avenue that outlets to existing drainage. The system includes two inlets.

1.0 Introduction

Huitt-Zollars, Inc. (HZ) has been under contract with the City of Denison (the City) to provide a Conceptual Stormwater Master Plan (SWMP) to identify current drainage-related issues and aid in the planning of future drainage projects.

1.1 Project Location

The City of Denison encompasses as area of approximately 29-square miles within Grayson County, Texas. Denison is part of the two principal cities that make up the Sherman-Denison metropolitan statistical area. Located near the northern border of Texas with the State of Oklahoma, the City of Denison and its extra-territorial jurisdiction area drainages ultimately drain to the Red River. The City lies in the larger Bois D'Arc Watershed defined by FEMA in a Baseline Engineering Study (Ref. 1), and the City lies within a combined drainage area of approximately 64 square miles (Figure 1 and Figure 2). Within these limits, there are several water storage reservoirs including Lake Randell, Loy Lake, Waterloo Lake, and Country Club Lake.

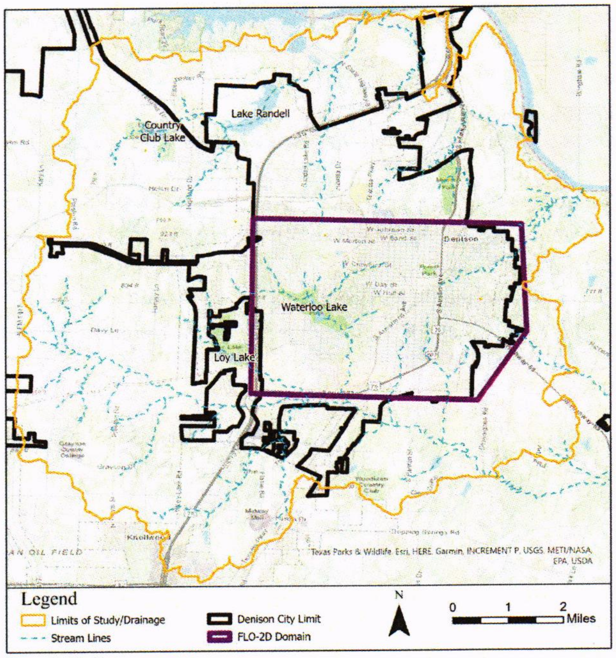

Figure 1: Site Map

1.2 Purpose

The City of Denison, Texas wishes to prepare a 10-year Capital Improvement Program (CIP) to help solve flooding and stormwater related problems. To accomplish this objective, a Conceptual SWMP is to be prepared to develop planning-level hydrology, identify areas of potential flood hazards and determine where flood mitigation projects are needed. Once the areas of most concern are identified, conceptual solutions will be developed for each selected hazard area and budgets prepared for possible inclusion within a future planned Capital Improvement Program (CIP).

The Conceptual SWMP will develop planning level hydrology, identify areas of potential flood hazards and, in conjunction with data collection efforts and stakeholder involvement, will be used to identify where flood mitigation measures are needed. Once the areas of most concern are determined, they will be narrowed down to a list of the top 10. Conceptual solutions will be identified for each selected hazard area and used to prepare budget estimates. If the combined budget estimates exceed the anticipated budget for the 10-year period, the number of projects will be pared down.

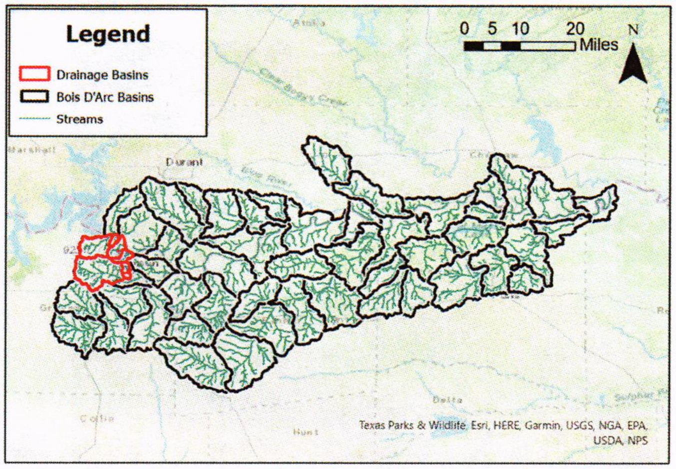

Figure 2: Bois D'Arc Watershed Drainage Map

2.0 Data Collection

As part of data collection, relevant data was gathered pertinent to the development of the hydrology and flood models and for identifying potential improvement projects.

2.1 Topographic Data

Topographic data was used for the development of both the HEC-HMS model (Ref. 11) (Appendix D) and the FLO-2D Model (Appendix E). The dataset is a mosaicked set from two sources - the United States Geological Survey (USGS) and the Texas Strategic Mapping Program (StratMap):

- USGS LiDAR Point Cloud TX West Central from 2018 (Ref. 2)
- StratMap LiDAR for North and Central Texas (Ref. 3)
- USGS 1/3 Arc Second resolution elevation data (Ref. 4)

The mosaicked elevation dataset extents are shown in Figure 3. All areas outside the extents of the higher resolution data in Figure 3 are sampled from the USGS coarse DEM data.

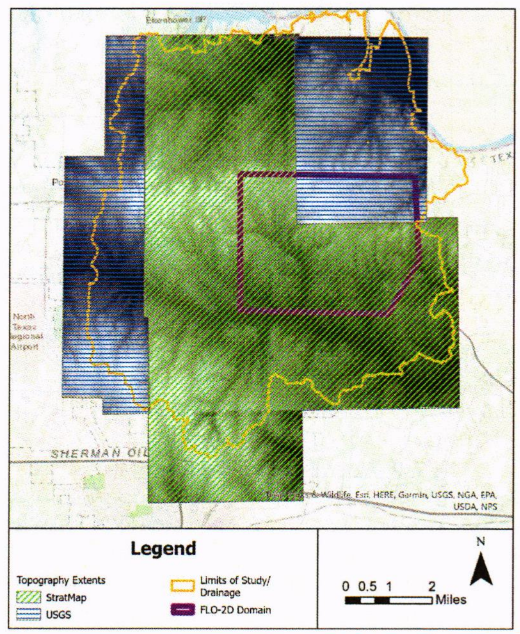

Figure 3: Topography Map

2.2 Known Areas of Historic Flooding

The City provided HZ with a list of historic flooding complaints from residents. These areas were identified prior to modeling efforts and were evaluated against the FLO-2D model results to determine the severity of flood hazard at each location. Additional information at some historic flooding areas was provided as part of the site visit.

2.3 Field Visit

HZ conducted a field visit on November 2, 2022. As part of the site visit the project team (the Team) visited various locations of interest to the City as well as noted areas of historic flooding. Field photographs were taken at areas of interest, and the Team noted characteristics and observations of the various locations visited. A detailed description of the field visit is provided in Appendix C.

2.4 FEMA Flood Insurance Rate Maps and Reports

Previous hydrologic studies within the study limits include studies by the Federal Emergency Management Agency (FEMA) to establish Flood Insurance Rate Maps (FIRMs) that are used for insurance purposes.

2.4.1 Flood Insurance Studies

A Flood Insurance Study (FIS) conducted by the Federal Emergency Management Agency (FEMA) for Grayson County encompasses different parts of the City of Denison within each of its three volumes (Ref. 5). In the FIS for Grayson County, a hydrologic analysis was performed at the watershed level to determine the peak discharge associated with a range of annual recurrence intervals for various streams and tributaries. Based on these peak discharges, mapping of both floodplains and floodways are depicted on Flood Insurance Rate Maps (FIRMs). The results presented on the FIRMs are used to set flood insurance rates for Grayson County and the resulting data is available for states or communities to implement sound floodplain management. Areas that are mapped on FIRM panels, which are publicly available from FEMA, with the reference FIRM Panel ID number are shown in Figure 4.

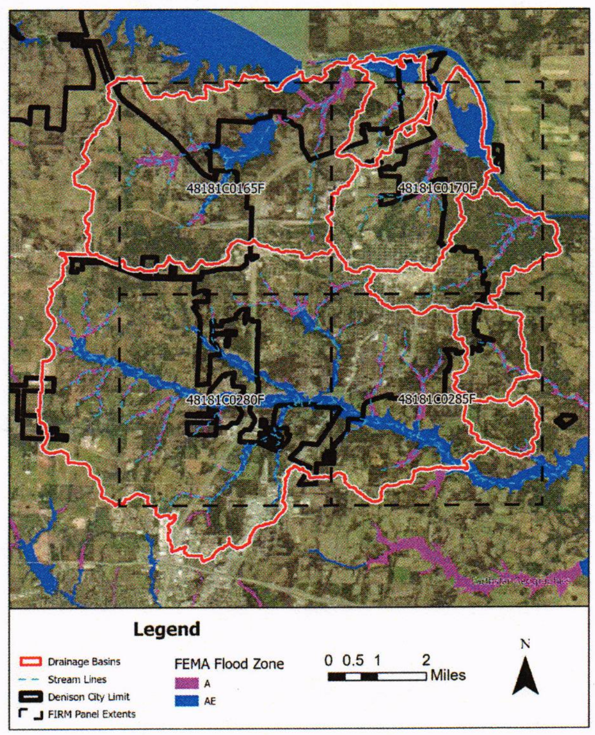

Figure 4: FEMA Flood Insurance Zones

2.4.2 Base Level Engineering Studies

A Base Level Engineering (BLE) study conducted for the Bois D' Arc Watershed for FEMA in January 2021 (Figure 2, Ref. 1) includes drainage courses in the vicinity of the City of Denison and beyond. The BLE study serves as a validation of the areas with defined flood zone hazards from the existing FIS. The study includes the determination of peak flows for various sub-basins within the watershed. The analysis is less detailed than that required for a typical FIS, however, the BLE study approach provides a quick method for mapping approximate flooding extents.

The base sub-basins from the BLE Study were also used for this SWMP study, thus a comparison can be made between the BLE Study and the approximate flood hazards estimated by the FLO-2D model.

2.5 As-Built Stormwater Infrastructure

As-Built GIS data has been provided by the City showing known stormwater infrastructure, including culverts, catch basins, and manholes. For most of these data, accompanying pictures have been provided. The stormwater infrastructure is provided as point data, although much of the information for the associated structures is incomplete. For most structures, including culverts and storm drain inlets/catch basins, it was necessary to make assumptions regarding dimensions, lengths, and invert elevations using available topography data and photographs. Additional data includes sanitary sewer and water utility data, both provided as polyline datasets.

2.6 Reservoir Data

Three major reservoirs (Figure 1) are modeled as part of this study; Loy Lake, Waterloo Lake, and Lake Randell. Waterloo Lake and Lake Randell are owned and operated by the City, while Grayson County operates Loy Lake. The fourth largest reservoir, Country Club Lake, was omitted from the study, as it is privately owned and has the least available stormwater storage. Data for the reservoirs are provided by the National Inventory of Dams (NID) (Ref. 6) and the Critical Infrastructure Division of the Texas Commission on Environmental Quality (TCEQ). Reservoir data is used in both the HEC-HMS (Appendix D) and FLO-2D models (Appendix E).

2.7 Existing and Proposed Land Cover and Land Use Data

Existing land cover data used for the HEC-HMS (Appendix D) and FLO-2D (Appendix E) models in this report are from the National Land Cover Database (NLCD) dataset for 2019 (Ref. 7). These data provide spatially explicit land cover data based on Landsat imagery and geospatial ancillary datasets. More detailed description of the land cover and its use in modeling efforts are included in Appendix D and Appendix E.

Modeling efforts rely heavily on the land cover dataset, but land use data is valuable to contextualize both existing conditions and future conditions. Zoning data from the City available through the City of Denison interactive zoning map is used for reference for existing land use data, and future land use data is available from the City of Denison Comprehensive Plan (Ref. 8).

2.8 Soils Data

Soils data for the project comes from the gridded soil survey geographic database (gSSURGO) from the United States Department of Agriculture (Ref. 9). The data provide a 10-meter gridded dataset across the United States to give detailed soil survey data. The soil data was used to determine the losses in the HEC-HMS and FLO-2D models. An in-depth description of the soils data and how it is used for each model are provided In Appendix D and Appendix E.

2.9 Building Data

Building data is for two purposes in the study. The first purpose is to identify structures that are at risk of flooding, and the second is so that buildings are modeled in the two-dimensional FLO-2D Model. Buildings used in this model comes from the State of Texas (Ref. 10), and as shown in Figure 5.

2.10 Planned Major Street Infrastructure

Planned street infrastructure is available through the City of Denison Comprehensive Plan (Ref. 8)

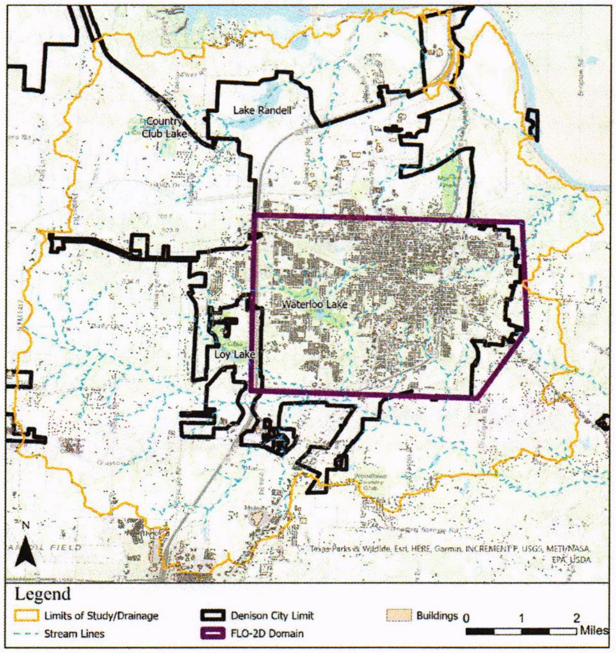

Figure 5: Building Data Map

3.0 Hydrologic and Hydraulic Analysis

This study evaluates planning-level hydrologic and hydraulic modeling that is used to identify areas of potential flood hazards and to aid in their mitigation. Hydrologic modeling includes a lumped-parameter, large sub-basin model that uses the HEC-HMS hydrologic software (Ref. 11) to evaluate peak flows and flood volumes, and a smaller two-dimensional flood model using FLO-2D software (Ref. 12) to evaluate flow patterns and approximate floodplain limits. The domain of the FLO-2D model used in this study is shown in Figure 1.

The HEC-HMS model is developed using sub-basin boundaries originating from a Base Level Engineering (BLE) study prepared for the Department of Homeland Security (DHS) and the Federal Emergency Management Agency (FEMA) to analyze the Bois D'Arc Watershed (Ref. 1). The sub-basins are modeled using a synthetic storm developed from gridded rainfall depth-duration-frequency data from the National Oceanic and Atmospheric Administration (NOAA) Atlas 14 Precipitation Frequency Data Server (PFDS) (Ref. 13). The HEC-HMS model is developed for existing conditions based on existing land use and soils data. Details on the methods, inputs, and results of the HEC-HMS hydrologic model are provided in Appendix D.

The FLO-2D two-dimensional flood model is developed to reflect existing topography, land uses, and soils data for the urban core of Denison (Figure 5). The FLO-2D model's primary purpose is to contextualize flow patterns and floodplain extents, which are used to identify areas of potential flood hazard. Details on the methods, inputs, and results of the FLO-2D flood model are provided in Appendix E.

Hydraulics for the various projects are evaluated at a planning level, and then used to establish a base estimate of cost for each proposed project. The City of Denison's stormwater standards (Ref. 14) serve as a guideline for all proposed solutions.

3.1 HEC-HMS Model

The HEC-HMS model was developed to evaluate the 100-year flood peak discharge and flood volumes. The sub-basins analyzed within the model are shown in Figure 6. The output discharge from the 100-year model was used in tandem with the results of the FLO-2D model to evaluate flood hazards and develop a conceptual design for each proposed project.

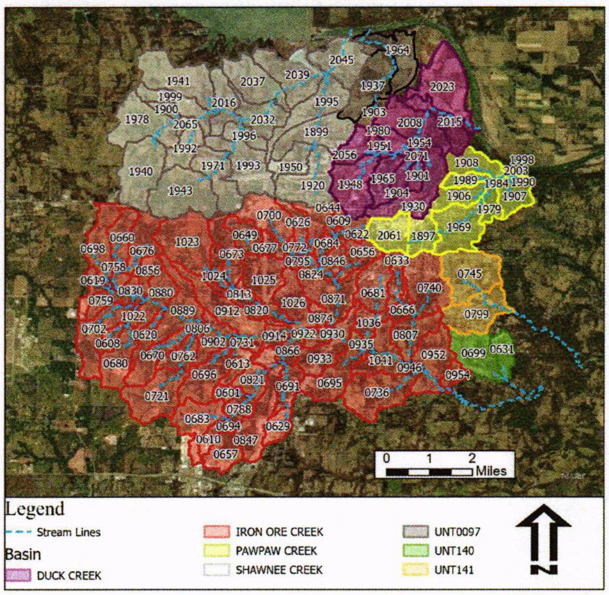

Figure 6: HEC-HMS Modeled Sub-Basins

3.2 FLO-2D Model

The FLO-2D model is used to estimate the flood flow patterns associated with the 100-year flood. A 20' x 20' grid was built with topographic mapping and hydraulic structure elements to model the flow within this urban environment. Because this model was developed for the purpose of evaluating the flood patterns, it should not be used to evaluate design flows or for detailed design. Both flood depth and velocity are identified with the model results.

Flood depths are shown in Figure 7 and flood velocities are shown in Figure 8. The results show that many areas with flood depths exceeding 1-foot are located at stream crossings with undersized culverts, or within streets without storm drain systems. Further detail about the FLO-2D model is provided in Appendix E.

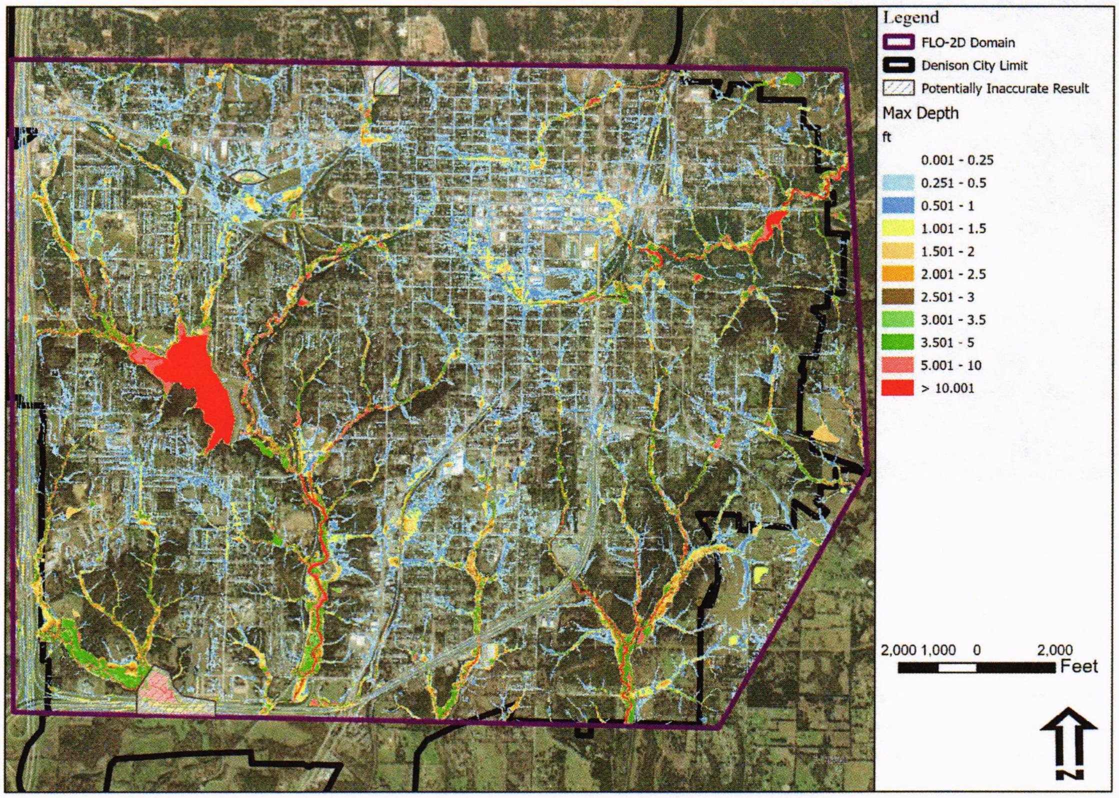

Figure 7: Maximum Depth and Areas of Noted High Flood Hazard

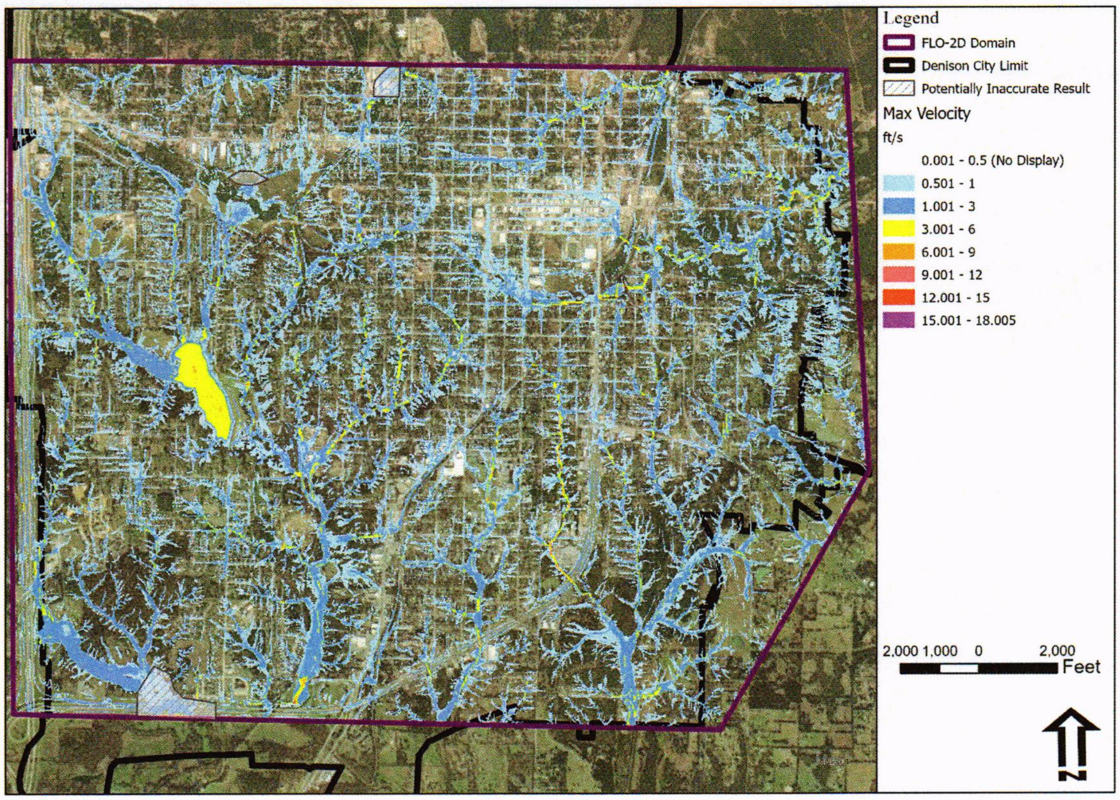

Figure 8: Maximum Velocity and Areas of Noted High Flood Hazard

3.3 Hydraulic Analysis

Evaluation of existing conditions and proposed improvement projects were conducted on a planning level at areas of noted concern derived from model results and historic flooding complaints. Detailed calculations for each area of proposed improvements are provided in Appendix F.

3.3.1 Storm Drain Systems

Storm drain systems were evaluated using design nomographs from Denison's Storm Drainage System Design Manual (Ref.14) for street gutter capacity and inlet capacity. For street gutters, triangular gutters with a ¼-inch per foot cross slope and 4-inch curb heights were assumed. For inlet capacity, standard curb opening inlets on grade were assumed. Storm drain conduits were sized using the Manning's equation for a full-flowing circular pipe laid at an assumed slope of 1.0%.

3.3.2 Culverts

Culverts were evaluated at the 100-year design flow using the HY-8 software (Ref. 15) developed by the Federal Highways Administration (FHWA). HY-8 is a public domain software intended for the design of culverts based on approaches and protocols from the FHWA publication, "Hydraulic Design Series 5: Hydraulic Design of Highway Culverts (Ref. 16). For a given design flow, existing and proposed conditions were evaluated to determine the adequacy of culverts to convey the design flow.

3.3.3 Engineered Channels

Engineered channels were evaluated and designed using the FHWA Hydraulic Toolbox (Ref. 17); a program that performs routine hydrologic and hydraulic analysis and design computations. Channel hydraulic analysis was performed for existing or proposed channels assuming steady, uniform flow for the design discharge and channel geometry.

4.0 Proposed Improvement Projects and Cost Estimates

For this initial study, 13 improvement projects are presented for the City to consider for inclusion in a future CIP. Of the proposed improvement projects, priority rankings have been provided based on hazard severity, including potential damage to property or life. High priority is determined for a flood hazard that potentially impacts multiple structures or would be a potential hazard to human life or safety; medium priority is assigned to projects where the flooding may cause minor flooding of any structures or cause a disturbance to highly trafficked areas by vehicles, and low priority is assigned to projects where flooding is anticipated but the hazards are limited to local flooding that does not significantly impact public safety or any structures. The locations of each of these projects are provided in Figure 9, and summaries of each conceptual project and cost are provided in Appendix A. A summary of the estimated costs for each project is provided in Table 1 and detailed cost estimates are provided in Appendix B.

Table 1: Summary of Cost for Each Proposed Project

PROJECT	LOCATION	COST		PRIORITY
A	Flora Lane and Rivercrest Circle	$	2,690,000	High
B	Morton and Maurice	$	3,548,000	High
C	Sears Street	$	5,229,000	High
D	Loy Lake Road and Waterloo Creek Crossing	$	1,272,000	High
E	Loy Lake Road Culvert Crossing	$	1,279,000	Medium
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K	Flora Lane Near Lum Lane	$	307,000	Low
L	Coffin Street and Park Avenue	$	771,000	Low
M	Eddy Avenue Between Elm Street and Martin Luther King Street	$	462,000	Low

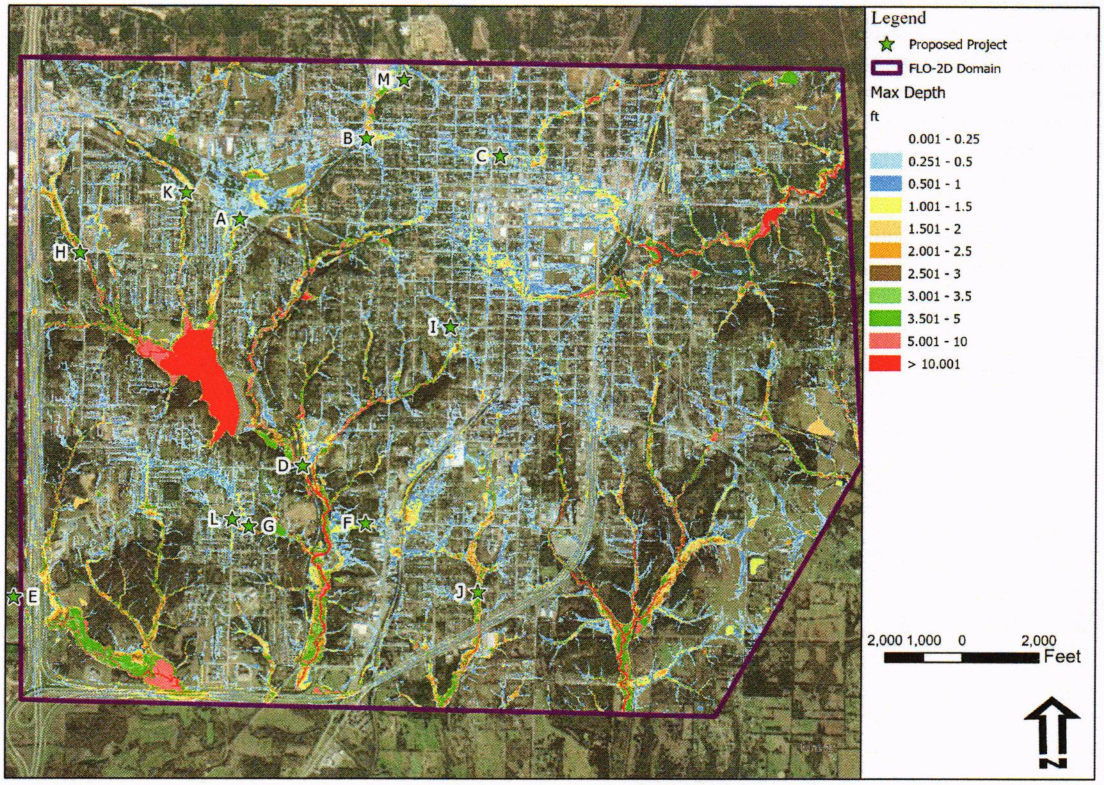

Figure 9: Proposed Projects Location Map

4.1 Project A – Drainage Improvements at Flora Lane and Rivercrest Circle

Priority - High

The Flora-Rivercrest improvement project is located where an existing stream bound for Waterloo Lake passes under Flora Lane and then under Rivercrest Circle just south of Katy Practice Fields. A residential community is situated south of Flora Lane, where the homes are built at elevations lower than the top-of-curb for Flora. As a result, any runoff that ponds above the top-of-curb would potentially flood the downstream homes (Figure 10). Flooding complaints from homes adjacent to the culvert crossing at Flora indicate that the culvert and drainage channel lack adequate capacity. Field visit observations similarly noted signs of flooding extending out-of-banks on the channel between Flora and Rivercrest.

Due to the potential to flood residences and historic reports of flooding in this area, high priority has been assigned. The proposed project includes building a storm drain system with inlets along Flora, culvert improvements at Flora, installation of a detention pond upstream in Katy Practice Fields, and channel improvements to the drainage channel downstream of the culvert crossings between Flora and Rivercrest.

Estimated Project Cost $2,690,000

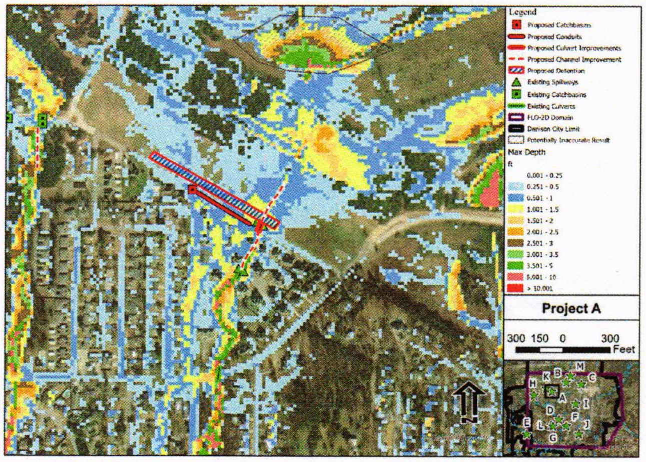

Figure 10: Proposed Project A

4.2 Project B – Morton Street and Maurice Avenue Storm Drain Expansion

Priority - High

The Morton-Maurice improvement project is located on Morton Street approximately 1,400-feet from the intersection with Perry Avenue until the intersection with Maurice Avenue, and on Maurice Avenue from the intersection with Gandy Street to the intersection with Bond Street. Morton Street uniquely falls under state jurisdiction as a state route, so this improvement project requires coordination between the State and the City. Morton and Maurice act as primary conveyance corridors for urban runoff from the surrounding blocks (Figure 11), and there are limited existing storm drain infrastructure in the area. Fairly regularly, ponding along Morton occurs during and after rainfall events as the existing drainage infrastructure is overwhelmed. Historic flooding has occurred at buildings along Morton and Maurice.

High priority is assigned to the Morton-Maurice improvement project due to the frequency of flood occurrence and the potential flood hazard to multiple nearby buildings. The proposed improvements include building a storm drain system centered along both Morton Street and Maurice Avenue, with catch basins designed to intercept urban runoff and convey runoff to existing downstream drainage located on Bond Street between Maurice and Brown.

Estimated Project Cost $3,548,000

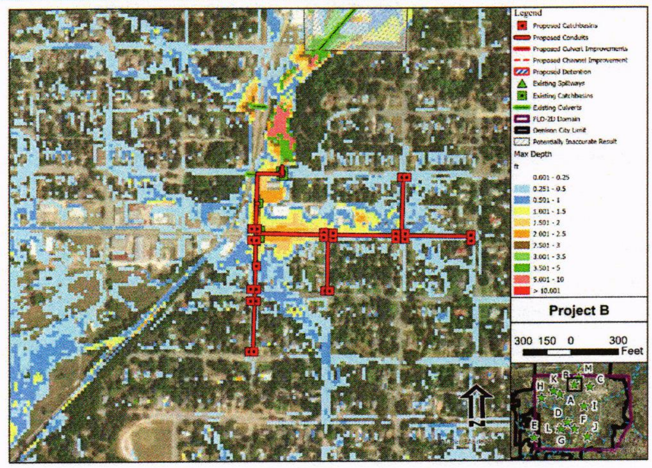

Figure 11: Proposed Project B

4.3 Project C – Sears Street Storm Drain Expansion

Priority - High

The Sears Street improvement project is located on Sears Street within Downtown Denison between Tone Avenue and Fannin Avenue. Sears Street is a principal conveyance corridor for the urban runoff from the surrounding neighborhoods, which outlets to a curb inlet at the intersection of Sears and Fannin (Figure 12). Sears Street lacks stormwater infrastructure upstream of the intersection of Sears and Fannin. Flooding complaints indicate that Sears Street experiences flooding during storm events, and modeling indicates that flooding at Sears Street poses a hazard to the homes along Sears as well as the nearby neighborhoods.

The Sears Street improvement project has high priority due to the hazard posed to homes within the area. The project is a storm drain centered along Sears Street with laterals and catch basins with capacity to intercept street runoff and convey it to the existing outfall on Fannin Avenue between Bond Street and Walker Street.

Estimated Project Cost $5,229,000

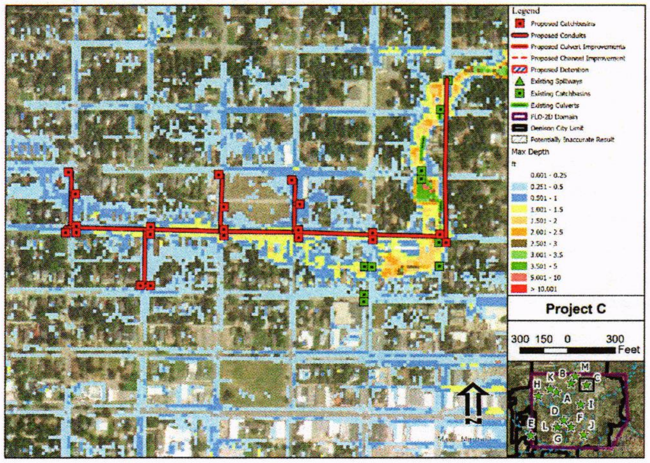

Figure 12: Proposed Project C

4.4 Project D – Waterloo Creek Stream Crossing Improvement at Loy Lake Road

Priority - High

The Waterloo Creek-Loy Lake Road improvement project is located at the stream crossing for Waterloo Creek at Loy Lake Road (Figure 13). The stream crossing is located approximately 4,600-feet downstream of Waterloo Lake's emergency spillway. A four-barrel, 60-inch diameter circular culvert passes stream flow. Above this stream crossing, a 200-foot span pedestrian bridge was constructed in 2021 to allow Katy Trail to cross Loy Lake Road with a clearance of about 17-feet. During high flows, the culvert crossing is overwhelmed and overtops Loy Lake Road, which serves as a major collector street for east-west travel, including emergency response. The roadway crossing is within a FEMA "Zone AE" floodplain and flood zone.

Due to the importance of Loy Lake Road to emergency response and the high hazard high flood flows could present to vehicle and pedestrian traffic, this project is given high priority. The proposed project includes adding culverts with capacity for the 100-year flood. Clearance between the pedestrian bridge and road deck is of particular concern, where a minimum clearance of at least 16-feet is provided by the City's Drainage System Design Manual (Ref. 14). The potential design is additionally constrained by nearby utilities, which may require relocation for project construction. Downstream earthwork is required to fit the additional culverts.

Estimated Project Cost $1,272,000

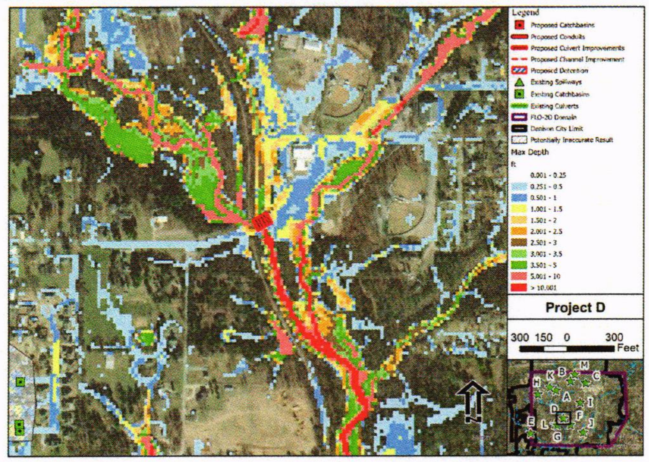

Figure 13: Proposed Project D

4.5 Project E – Loy Creek Stream Crossing at Loy Lake Road

Priority - High

The Loy Lake Road improvement project is located at the culvert crossing for Loy Creek at Loy Lake Road; a major collector street running southwest to northeast (Figure 14). Located just downstream of the spillway for Loy Lake, this culvert crossing lacks capacity for the 100-year flood flow, which is significantly larger than the existing culvert crossing capacity. The culvert crossing lies within the delineated 100-year floodway from FEMA's FIRM, and the surrounding floodplain is classified as "Zone AE", denoting that base flood elevations are determined. Base flood elevations indicate that the existing roadway is below the flood elevation to a depth of approximately 12-feet.

Flood depths pose a significant hazard for pedestrians and vehicles, resulting in a high priority designation. The proposed project includes culvert improvements to increase capacity and inlet improvements with a stilling basin to reduce the clogging potential for culverts.

Estimated Project Cost $1,279,000

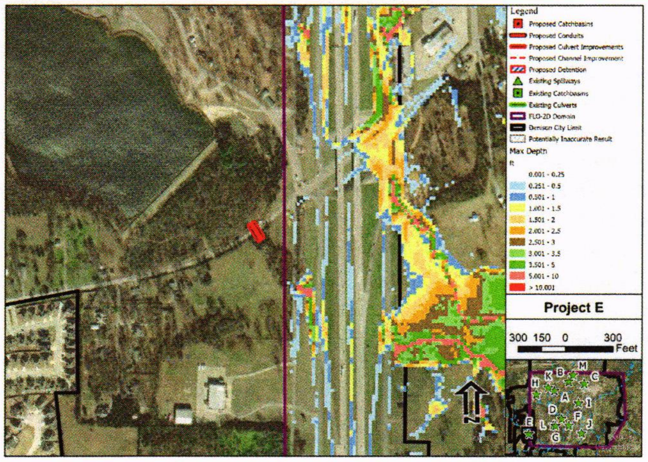

Figure 14: Proposed Project E

4.6 Project F – Coffin Street and Woodlawn Avenue Intersection

Priority - Medium

The Coffin-Woodlawn improvement project is located at the intersection of Coffin Street and Woodlawn Boulevard. During intense storm events ponding occurs at the intersection, likely from flow originating from upstream on Coffin Street where an existing drainage channel overflows (Figure 15). To the west of the intersection, a proposed mixed-use development is planned which would also be impacted by flooding at the intersection.

Due to the potential impact to the proposed development west of the intersection, a medium priority is assigned. The proposed project includes improvements to the upstream drainage channel that passes under Coffin, culvert improvements for the crossing under Coffin Street, and adding an additional storm drain inlet on the northeast corner of the Coffin/Woodlawn intersection.

Estimated Project Cost $3,433,000

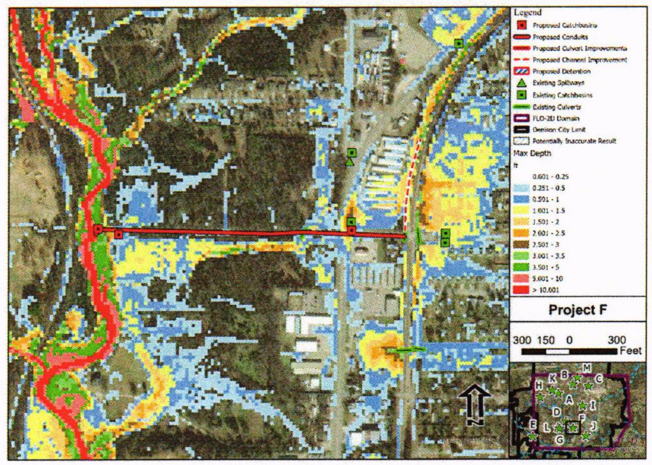

Figure 15: Proposed Project F

4.7 Project G – Stream Downstream of Coffin Street and Park Avenue

Priority - Medium

Downstream of the culvert crossing at the intersection of Coffin Street and Park Avenue, identified in Project L, an existing stream transects properties located on Renaissance Drive. The streambanks have experienced significant scour and lateral migration towards the residences to the south. The stream receives flow from the culvert at Coffin and Park and two storm drain systems located both north and south of the stream.

Downstream of the intersection of Coffin Street and Park Avenue, storm water runoff passes through a 72-inch diameter reinforced concrete pipe, where it is combined with flow from the storm drain systems along Park Avenue, Renaissance Drive, Turtlecreek, and Park Village. Downstream of the 72-inch pipe, the combined flow discharges to a stream channel which passes through the backyards of residential properties on Renaissance Drive. Several property owners have constructed stream crossings over the streambed to provide access to the property across the stream and added various forms of bank protection. Downstream of the stream crossings, flow is discharged over a grade control structure as it is released to a natural unconfined stream where flow meanders easternly towards a pond at Deer Lake Drive.

While no flood incidents have been reported in this area, a significant scour hole and lateral migration of the stream towards the residential structures on Renaissance Drive has been reported in the unconfined portion of the stream downstream of the grade control structure. If bank scour continues and allows lateral migration toward the homes on Renaissance Drive, it could pose a hazard to the structural stability of the foundations for the homes.

A new grade control structure at the location of the existing grade control structure, which ties into a gabion retaining wall along the southwest bank of the stream, is proposed to mitigate the risk for further lateral migration of the stream. The gabions would serve as an erosion control and soil stability measure while the new grade control structure would prevent head-cutting erosion from progressing further upstream. Potential complications include construction access, since the project location is within private property and would require a construction entrance that passes through various residential properties.

Although no flooding hazards are present at this project location, the potential for foundation instability for residential structures results in a medium priority rating. An evaluation of the hazard to the residences along Renaissance Drive is recommended, where the priority rating may be re-evaluated based on the results.

Estimated Project Cost $768,000

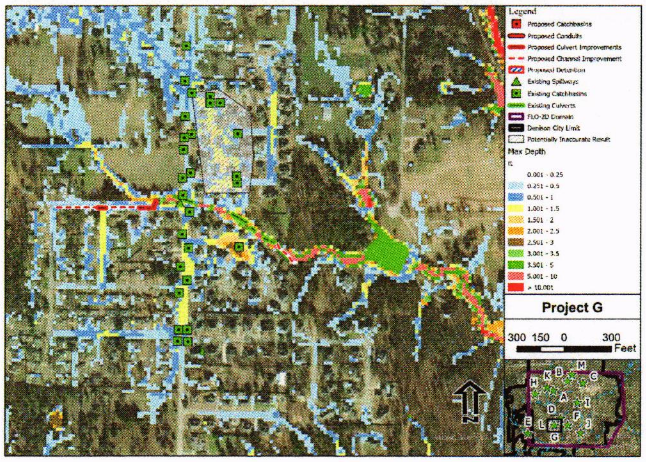

Figure 16: Proposed Project G

4.8 Project H – Lillis Lane and Crawford Street Intersection

Priority - Medium

The Lillis-Crawford improvement project is located at the intersection of Lillis Lane and Crawford Street. A stream draining to Waterloo Lake crosses under the intersection via an existing circular concrete pipe culvert. This improvement project lies within a "Zone A" flood zone according to FEMA's FIRM, and the culvert crossing has been historically prone to clogging. The flooding impacts at this location include intersection flooding and flooding of homes located within the 100-year floodplain just downstream of the intersection (Figure 17).

The proposed solution improves the capacity of the culvert crossing to the 100-year flood discharge and includes inlet improvements to reduce the potential for clogging and to facilitate maintenance. Due to the flooding hazards in this location, which include flooding crossing the street intersection and potentially spilling into adjacent properties downstream, medium priority level is assigned.

Estimated Project Cost $1,689,000

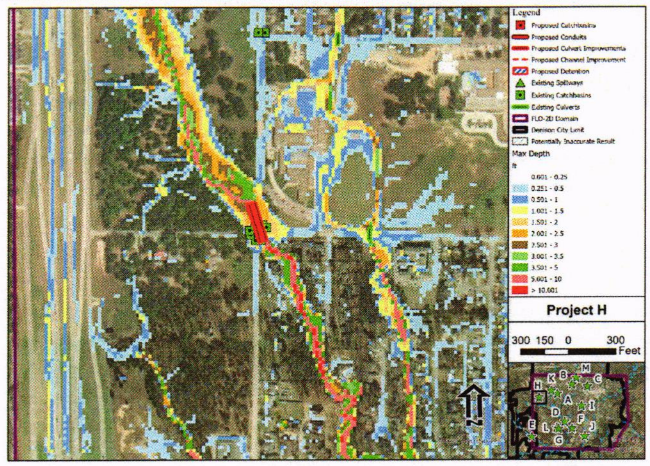

Figure 17: Proposed Project H

4.9 Project I – Chandler Avenue near Munson Street

Priority - Medium

This project is located on Chandler Avenue near Munson Street, and the system extends southwardly to near the intersection of Chandler Avenue and Monterey Street. The two residences located on Munson Street on the northeast corner of the intersection of Munson Street and Chandler Avenue have reported flooding incidents. In the location of noted flood complaints, storm water is handled with a roadside ditch that runs north-to-south along Chandler Avenue. In the ditches on both sides of Chandler Avenue, flow passes under driveways and alleys through corrugated metal culverts and is intercepted by two storm drain inlets near the intersection of Chandler and Munson. The layout of the receiving storm drain system is uncertain because of limited data but appears to pass flow to an existing stormwater channel that runs north-to-south between Chandler Avenue and Scullin Avenue. The existing storm drainage system does not use any curb and gutter system and passes all flows through surface drainage to the existing stormwater channel between Chandler Avenue and Scullin Avenue. The capacity of this stormwater system is unknown without detailed as-built data and/or survey.

Flooding initiates in the alley north of the two homes, near the intersection of the alley with Chandler Avenue. In this location, there is a localized low point near the property line between the two homes, where Chandler Avenue and the roadside ditch are at a higher elevation than the ground for the properties to the east. Flooding analysis with FLO-2D has confirmed that the flow paths in this area push drainage, which originates from the north, towards the low-point near the property line between the two residences. Here, flow passes toward the south, flowing through both properties rather than around.

The proposed project consists of a storm drain pipe, which begins with an inlet at the low point in the alley east of Chandler Avenue between Munson Street and Day Street. From here, flow travels to the south along the property line between the second and third house from the intersection within an existing vacant property. The storm drain will then pass under Munson Street and tie in with the existing catch basin and storm drain on Munson Street. This storm drain outlets to an existing storm water drainage channel located south of Munson Street. Grading should be performed in the alley to ensure that drainage is directed toward either existing storm water infrastructure along the roadside ditch or towards the proposed storm drain inlet in the alley.

This solution assumes that the existing storm water channel running south from Munson Street to Texas Street has capacity to accept this storm water discharge. The current layout suggests that the existing channel is the intended concentration point for localized runoff from these blocks of residences. Capacity in this existing channel should be checked with detailed survey data and hydraulic analysis to ensure the stream does not overflow during extreme storm events. Right-of-Way acquisition will be required along the proposed pipe alignment through an existing vacant parcel.

Since the existing flooding reported at this location is limited to two homes and the magnitude of the flooding is fairly limited, this project is given a medium priority classification. The additional capacity offered by this project would be beneficial to reduce existing flooding at residences while also adding to the resiliency of the existing system.

Estimated Project Cost $415,000

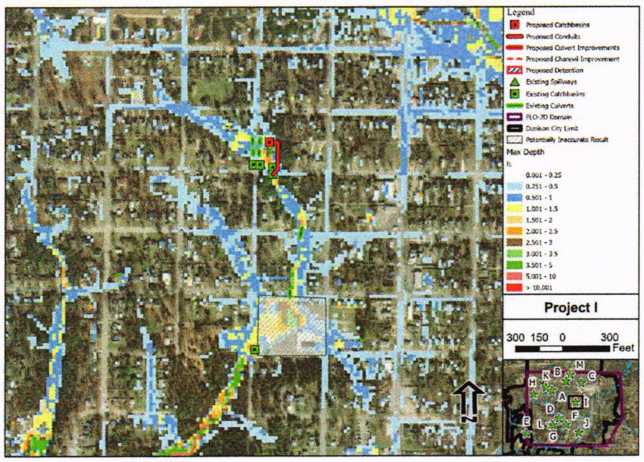

Figure 18: Proposed Project I

4.10 Project J – Dubois Street Stream Crossing between Scullin and Barrett

Priority - Medium

The Dubois Street improvement project is located at a stream crossing located between Scullin Avenue and Barrett Avenue. The existing stream crossing consists of a multiple-barrel circular concrete culvert crossing. The roadway is an east-west crossing for traffic across the existing stream, and the closest additional crossing is about 1,700 feet north at Coffin Street. The upstream and downstream areas are heavily vegetated, and the culvert is prone to clogging by debris. Flooding at the roadway occurs in this location during high stream flow (Figure 19). The roadway and nearby homes are located within a "Zone A" hazard classification by FEMA's FIRM, but a base flood elevation is not determined.

Analysis of the existing culvert crossing indicates that the culvert is undersized for the 100-year flood. Since the roadway is a major traffic crossing for the stream and the roadway is crossed by the 100-year flood, medium priority is assigned to this project. The proposed improvements include adding culverts with sufficient capacity to convey the 100-year flood flow.

Estimated Project Cost $676,000

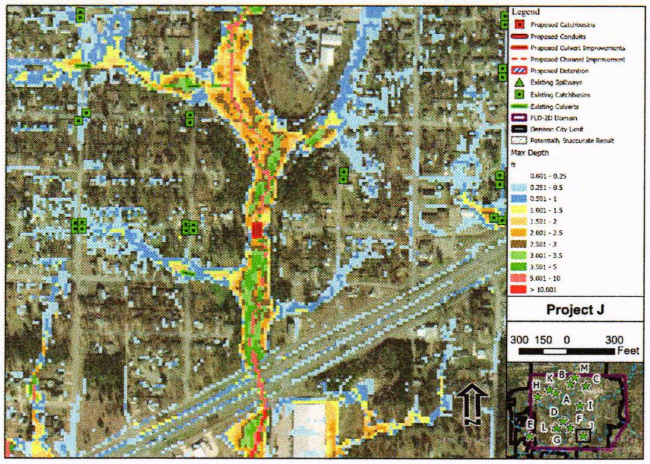

Figure 19: Proposed Project J

4.11 Project K – Drainage Improvement at Flora Lane near Lum Lane

Priority - Low

The Flora-Lum improvement project is located at the stream crossing located just west of the intersection of Flora Lane and Lum Lane (Figure 20). This location was recently improved in 2021 with a dual-barrel concrete box culvert and riprap outlet protection. After the culvert crossing improvement, a residence was built on the property downstream of the crossing and has reported intermittent flooding from the nearby stream. According to the Grayson County Appraisal District, this residence was constructed some time in 2022, after the culvert crossing improvement was completed.

The downstream house is the only impacted structure, which is outside the main stream channel but could be impacted by overbank flow, so this project is given low priority. The proposed improvement modifies the downstream stream to move the alignment away from the existing home in an engineered channel and armoring it to prevent lateral migration.

Estimated Project Cost $307,000

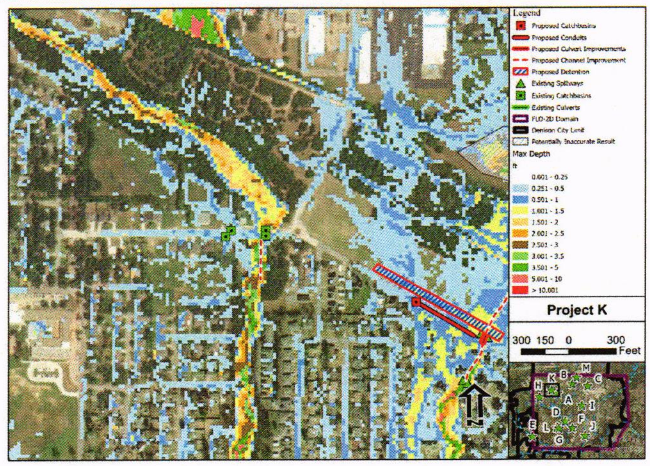

Figure 20: Proposed Project K

4.12 Project L – Coffin Street and Park Avenue Drainage and Culvert Improvements

Priority - Low

The Coffin-Park improvement project lies between the intersection of Coffin Street and Park Avenue, and the intersection of Caprice Avenue and Coffin Street. An existing stream, ultimately draining to Iron Ore Creek, passes under the three-way intersection of Coffin Street and Park Avenue through an existing culvert. Park Avenue is outfitted with a storm drain system that outlets just downstream of the Coffin and Park intersection. While Park Avenue contains a storm drain system, Coffin Street, which lacks curbs and gutters, conveys storm water through a roadside ditch that runs adjacent to Coffin on the south side of the road prior to discharging via culvert northward just upstream of the intersection of Coffin and Park.

Flooding at the intersection of Coffin and Park occurs after intense rainfall events. Flooding is suspected to occur when the roadside drainage ditch along Coffin overflows and discharges onto the roadway, which ultimately drains to the intersection of Coffin and Park. Flooding impacts are limited to the roadway since the homes adjacent to Coffin Street are elevated multiple feet above the roadway deck (Figure 21), so the priority level is low.

The proposed improvements include improving or adding culverts under the roadways and driveways along Coffin Street and upgrading the roadside channel to ensure adequate capacity for storm runoff.

Estimated Project Cost $771,000

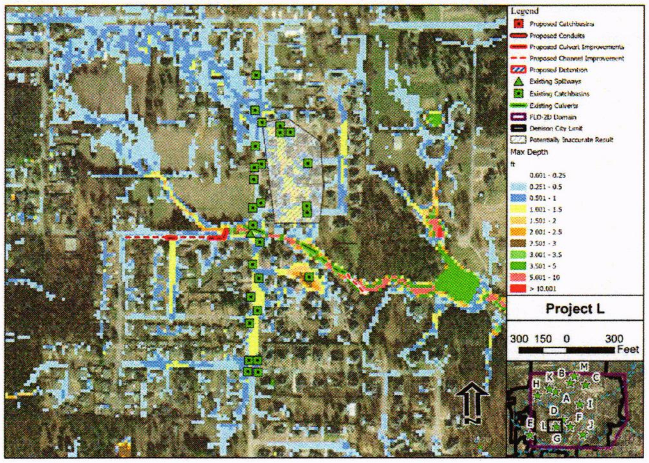

Figure 21: Proposed Project L

4.13 Project M – Eddy Avenue between Elm Street and Martin Luther King Street

Priority - Low

The Eddy Avenue improvement project lies between Elm Street and Martin Luther King Street. The roadway is fairly steep, and excessive roadway ponding and sediment deposition have occurred in this area. The limits of flooding impacts include roadway ponding along Eddy Avenue and at its intersections with Elm Street and Martin Luther King Street (Figure 22).

Low priority is assigned to this project due to the limited hazards presented by existing drainage problems. The proposed solution would add a storm drain with inlets along Eddy to reduce gutter flow depths along Eddy and within both intersections. Flow from the storm drain will discharge to the existing drainage passing under Martin Luther King Street.

Estimated Project Cost $462,000

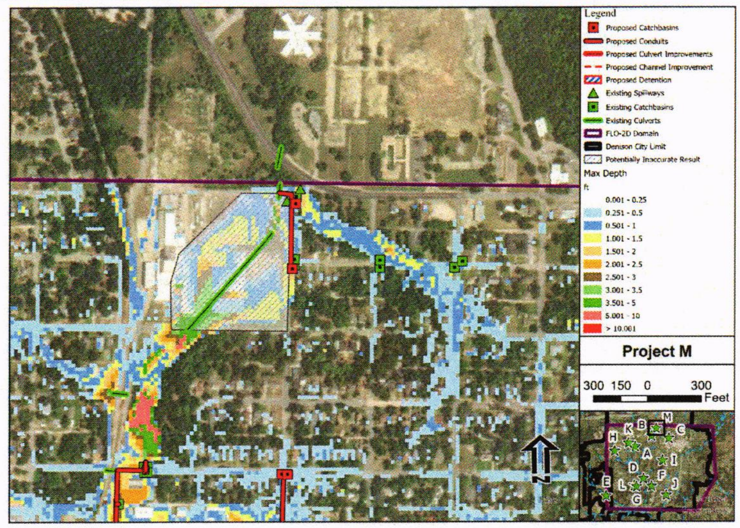

Figure 22: Proposed Project M

5.0 Recommendations

Each of the thirteen proposed projects presented herein have been developed to a limited level of detail due to the availability of more detailed documentation of existing infrastructure. Each project needs to be developed to a 15% level of design to better determine the extent of existing and required infrastructure improvements. Once this data is available, hydrologic and hydraulic models should be updated. New site-specific topographic mapping with existing storm drainage infrastructure should be denoted along with any known or identified utilities.

Cost estimates to date have been inflated due to unknowns. Cost estimates were deemed appropriate at the time of study completion, but are subject to change based upon local and national wage and material price changes.

5.1 Other Areas of Interest

This report focuses on local flooding problems identified by the City during a field investigation as reported in Appendix C. Figure 23 illustrates other areas that either potentially lie within the floodplain, receive flood complaints, or are shown to have high flood hazard from the FLO-2D model results. These areas were not investigated in this study, but they have the potential to put existing and/or proposed structures at risk. In some cases, this risk may already be shown on the FEMA FIRMs. Additional field investigation and review of flooding complaints is recommended to determine if the identified flood hazards are representative of actual conditions, and whether infrastructure improvements have already been completed to address such concerns.

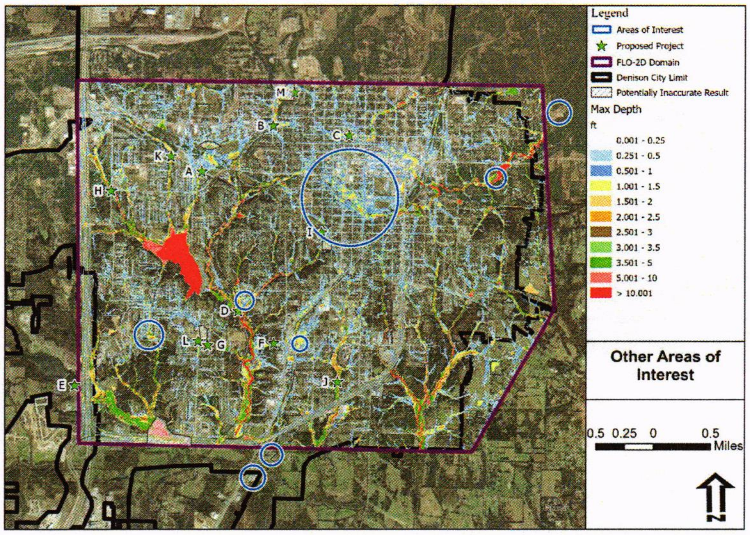

Figure 23: Other Areas of Interest

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Appendix A: Summary of Proposed Improvement Projects

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Exhibits

Project A – Flora Lane & Rivercrest Circle

A Project A – Flora Lane & Rivercrest Circle

Priority - High

A.1 Location

Flooding hazards have been identified on Flora Lane near the intersection with Rivercrest Circle. Katy Practice Fields, a City of Denison-owned park, is located to the north of Flora Lane. Existing drainage paths intersect within Katy Practice Fields and cross over Flora Lane, either as sheet drainage, or via a culvert east of River Crest. The culvert and street scuppers release drainage into an armored channel that travels southward between existing homes. The channel continues further south and crosses Rivercrest Circle through three large culverts, which release flow to a fairly vegetated drainageway.

A.2 Drainage Problem

Homes located along the south side of Flora Lane are generally built lower than the road grades, resulting in flooding of the front yards, through conveyance, and noted flooding of homes in more extreme rainfall events. The driveways for the homes on Flora are elevated lower than the top-of-curb elevation for Flora, and both sides of Flora Lane are curbed without any stormwater infrastructure. The result is that all runoff from Flora is collected in the gutters for the length of Flora Lane between Lum Lane and Crawford Street. Street runoff in the southern gutters overtops at low-points along the curbs and is directed into the front yards.

The culvert crossing and armored channel for the existing drainageway at Flora are severely undersized for the 100-year event. The crossing at Rivercrest, although larger, is also undersized. Historic flooding of the homes adjacent to the stream has been noted. Visual evidence from an HZ field visit shows water marks that reach above the bottom fence line for the home adjacent to the drainage channel between Flora Lane and Rivercrest Circle. The culvert crossing at Flora Lane is a sole circular concrete culvert, while the downstream culvert crossing at Rivercrest Circle is three circular concrete pipes. With the existing configuration, during extreme storm events, runoff would pond and spill over Flora Lane, which would flood the adjacent homes that are situated at a lower elevation than the road deck at Flora Lane. Due to the hazard presented to multiple homes south of Flora Lane, high priority is assigned.

Figure A-1:Culvert Crossing Under Flora Lane on Upstream End

Figure A-2: Photograph Facing Downstream Channel from Flora Culvert Crossing

Figure A-3: Photograph from Upstream End of Culvert Crossing at Rivercrest Circle

Figure A-4: Photograph at Outlet of Culvert Crossing Under Rivercrest Circle

A.3 Assumptions

The following assumptions were made to develop the conceptual drainage solution:

- The project lies within the "644" sub-basin in the HEC-HMS model, to estimate the design flow, discharge is based on a unit discharge (discharge per square mile) basis using the 100-year peak flows estimated from the HEC-HMS model for sub-basin 644. Unit discharge for sub-basin 644, in cfs per square mile, is multiplied by the contributing drainage area for each structure to determine peak flow. Detailed estimation of peak flow is provided in Appendix F.
- Conduits for the storm drain were sized assuming full-flowing, uniform and steady flow conditions (Manning's equation) for the total contributing area to the storm drain and final design should refine the storm drains.
- Storm drain inlets are sized for the street capacity using design nomographs from the *Storm Drainage System Design Manual* for Denison, Texas (Ref. 1), assuming the following
  - Longitudinal Street Slope of 0.01 foot/foot
  - Street Cross Slope = ¼" per foot with center crown
  - 4" Curb Height
  - Standard curb opening inlet on grade at each catch basin
- Storm drain inlets are placed at areas upstream of driveways elevated lower than the top of curb along the south side of Flora Lane.
- Drainage downstream of the culvert at Rivercrest Circle is assumed to have capacity to convey flood flows.
- Culvert crossings were analyzed with estimated sizes and pipe inverts in lieu of detailed as-built information using HY-8 software.
- Grouted riprap similar to existing channel is assumed for the armored channel between Flora and Rivercrest.

A.4 Conceptual Design

The design consists of a storm drain system, culvert improvement at the Flora Lane stream crossing, a detention basin to reduce runoff peak flow in Katy Practice Fields, and channel improvements both upstream of Flora at Katy Practice Fields and in the channel between Flora and Rivercrest. The detention basin is intended to reduce the 100-year discharge to 318 cfs, which would meet the capacity of the existing stream crossing at Rivercrest Circle. The Flora Lane culvert is proposed to be upgraded to a capacity of 318 cfs, similar to the existing culvert crossing at Rivercrest. A summary of the design is provided in Table A-1. The proposed layout overlaid on the FLO-2D model results are provided in Exhibit A.

Priority High

Table A-1: Design Summary at Flora Lane & Rivercrest Circle

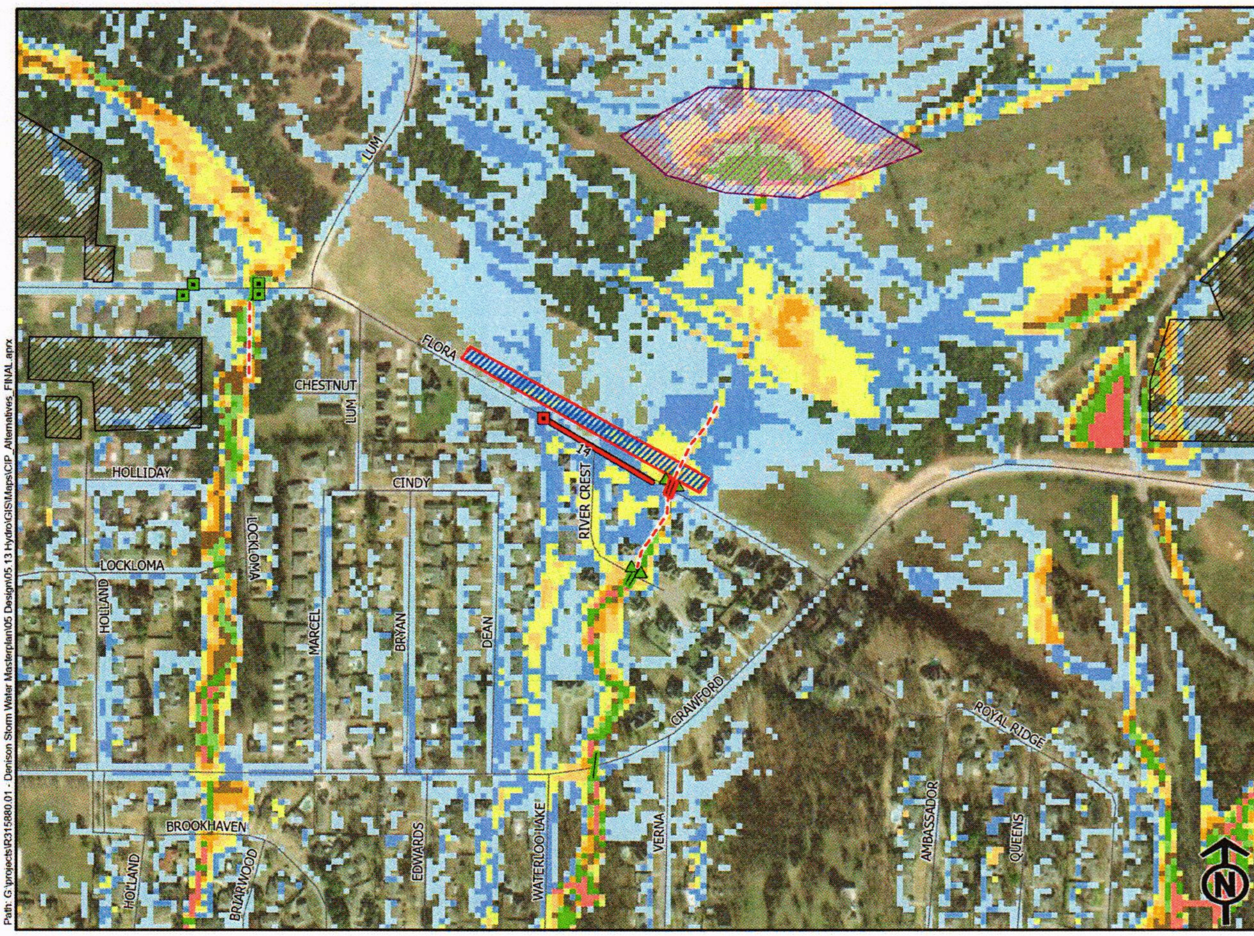
Total Stream Crossing Contributing Area (mi²)	0.28
Discharge, 100-year (cfs)	579
Discharge, 10-year (cfs)	353
Storm Drain Contributing Area (mi²)	0.002
Storm Drain Discharge (cfs)	4
Main Conduit Size (inch)	24
Inlet Length (ft)	4
Proposed Culvert at Flora Lane	4 x 4' W x 3' H Concrete Box
Existing Rivercrest Circle Culvert	3 x 48" Dia. Circular Concrete
Proposed Culvert Capacity (cfs)	318
Proposed Detention Volume (acre-ft)	7.7
Proposed Channel	10' Wide x 2.5' Deep Trapezoid – Grouted Riprap

A.5 Cost Estimate

The following table (Table A-2) summarizes the estimated cost for the proposed improvements at Flora Lane and Rivercrest Circle. Detailed breakdown of cost is provided in Appendix B.

Table A-2: Estimation of Cost – Flora Lane & Rivercrest Circle

ITEM DESCRIPTION	UNIT	QUANTITY	UNIT COST		TOTAL COST	
REMOVE ASPHALT PAVEMENT	SY	112	$	50	$	5,600
REMOVE CURB AND GUTTER	LF	40	$	15	$	600
REMOVE & DISPOSE REINFORCED CONCRETE PIPE	LF	50	$	45	$	2,250
REMOVE HEADWALL	EA	2	$	350	$	700
UNCLASSIFIED STREET EXCAVATION	CY	260.00	$	50	$	13,000
4' STD. CURB INLET	EA	1	$	13,000	$	13,000
CONC BOX CULV (4 FT X 4 FT)	LF	200	$	500	$	100,000
CITY OF DENISON TYPE B MANHOLE	EA	1	$	13,500	$	13,500
24" CLASS III REINF. CONC. PIPE	LF	500	$	200	$	100,000
SLOPED CONCRETE HEADWALL	EA	2	$	8,000	$	16,000
CONNECT TO EXISTING DRAINAGE STRUCTURE	EA	1	$	5,000	$	5,000
HMAC ASPHALT PAVEMENT (2" TYPE D)	SY	112	$	40	$	4,480
6" SEPARATE CONCRETE CURB & GUTTER	LF	40	$	55	$	2,200
GROUTED RIPRAP TYPE A	SY	1,871	$	300	$	561,300
EARTHWORK (EROSN & SEDMT CONT, IN VEH)	CY	12,800	$	50	$	640,000
				SUB-TOTAL =	**$**	**1,477,630**
				(APPROX. 40%) =	**$**	**591,070**
				CONTINGENCY SUB-TOTAL =	**$**	**2,068,700**
				ADMIN, ENGINEERING, CONSTRUCTION ADMIN (APPROX. 30%) =	**$**	**621,300**
				TOTAL =	**$**	**2,690,000**



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CITY OF DENISON
300 W Main St
Denison, TX 75020

Denison TEXAS

City of Denison
Conceptual SWMP
R315880.01

- Proposed Catchbasins
- Proposed Conduits
- Proposed Culvert Improvements
- Proposed Channel Improvement
- Existing Spillways
- Existing Catchbasins
- Existing Culverts
- Streets
- Proposed Detention
- Potentially Inaccurate Result
- Further Detail Required

Proposed Development
- Commercial
- Mixed
- Residential

Max Depth
ft
- 0.001 - 0.25
- 0.251 - 0.5
- 0.501 - 1
- 1.001 - 1.5
- 1.501 - 2
- 2.001 - 2.5
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- 5.001 - 10
- > 10.001

Appendix A
Exhibit A

Proposed Storm Water Layout
Project A

0 125 250 500
Feet

HUITT-ZOLLARS
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Dallas, Texas 75240

Project B – Morton Street and Maurice Avenue

B Project B – Morton Street & Maurice Avenue

Priority - High

B.1 Location

The intersection at Morton Street and Maurice Avenue experiences flooding after intense rainfall events and Morton Street experiences ponding fairly regularly. While Maurice Avenue falls under the jurisdiction of the City of Denison, Morton Street is a State Highway under the jurisdiction of the Texas Department of Transportation (TXDOT). Existing storm drain infrastructure includes a storm drain system with five grated inlets near the intersection of Morton and Maurice. The main storm drain outfalls to an existing drainageway east of the intersection of Maurice and Bond. The receiving drainageway, which flows from south to north, is an unnamed tributary to Duck Creek. A railroad crosses Morton west of Maurice and then parallels Maurice heading north.

B.2 Drainage Problem

The intersection at Morton Street and Maurice Avenue is a concentration point for urban runoff from the west, south, and east prior to flowing northward along Maurice. An area of approximately 60.7 acres contributes to urban runoff at this intersection. Presently, five storm drain inlets at this intersection are intermittently overwhelmed during storm events, resulting in ponding at the intersection. Urban runoff concentrates along Morton Street, which serves as a local low-point. Stormwater infrastructure data provided by the City of Denison indicates that although runoff concentrates along Morton, it lacks a storm drain system at regular intervals to capture stormwater runoff.

Flooding has occurred that impacts buildings along Morton Street, and flooding complications are expected to occur as long as the current storm water infrastructure along both Morton Street and Maurice Avenue are in place. High priority is assigned.

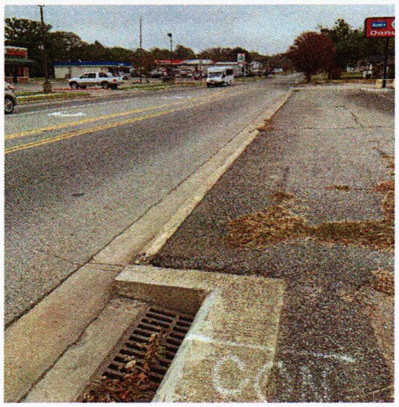


Figure B-1: Photograph Facing East at Storm Drain Inlet on Maurice

Figure B-2: Existing Storm Drain Inlet at Intersection of Morton and Maurice

B.3 Assumptions

The following assumptions were made to develop the conceptual drainage solution:

- The project lies within the "1948" sub-basin in the HEC-HMS model, to estimate the design flow, discharge is based on a unit discharge (discharge per square mile) basis using the 100-year peak flows estimated from the HEC-HMS model for sub-basin 1948. Unit discharge for sub-basin 1948, in cfs per square mile, is multiplied by the contributing drainage area at the storm drain outlet of 0.09 square miles to determine maximum system peak flow. Detailed estimation of peak flow is provided in Appendix F.
- Conduits for the storm drain are sized assuming full-flowing, uniform and steady flow conditions (Manning's equation) for the total contributing area. The storm drain should be refined during preliminary and final design.
- Storm drain inlets are sized for the street capacity using design nomographs from the *Storm Drainage System Design Manual* for Denison, Texas (Ref. 1), assuming the following
  - Longitudinal Street Slope of 0.01 foot/foot
  - Street Cross Slope = ¼" per foot with center crown
  - 4" Curb Height
  - Standard curb opening inlet on grade at each catch basin
- Storm drain inlets are placed at areas of flow concentration noted from the two-dimensional FLO-2D model for the 100-year storm. A more detailed hydrologic analysis is required for sizing and placement of storm drain inlets.
- Although Morton Street falls under State jurisdiction, a conceptual storm drain system is laid out with a main storm drain along Morton Street, which is where urban runoff concentrates. Coordinated efforts between the City and the State will be required.
- Without detailed as-built data for the existing storm drain system near Maurice and Morton, it is assumed that larger conduits will be required to either supplement or replace the existing drainage system.
- Stormwater in the proposed storm drain system outlets to an existing drainageway on Bond Street between Brown and Maurice. It is assumed that the existing drainageway downstream of the storm drain has capacity.

B.4 Conceptual Design

The proposed main storm drain is situated along Morton Street before turning north along Maurice Avenue. The ultimate outfall is to an existing drainageway north of Bond Street between Brown Avenue and Maurice Avenue. Storm drain inlets will be placed along Maurice and at intersections with Brown, Eddy and Perry. An additional storm drain lateral is proposed south along to capture urban runoff from the south. A summary of the design is provided in Table B-1. The proposed layout overlaid on the FLO-2D model results are provided in Exhibit B.

Priority High

Table B-1: Design Summary at Morton and Maurice

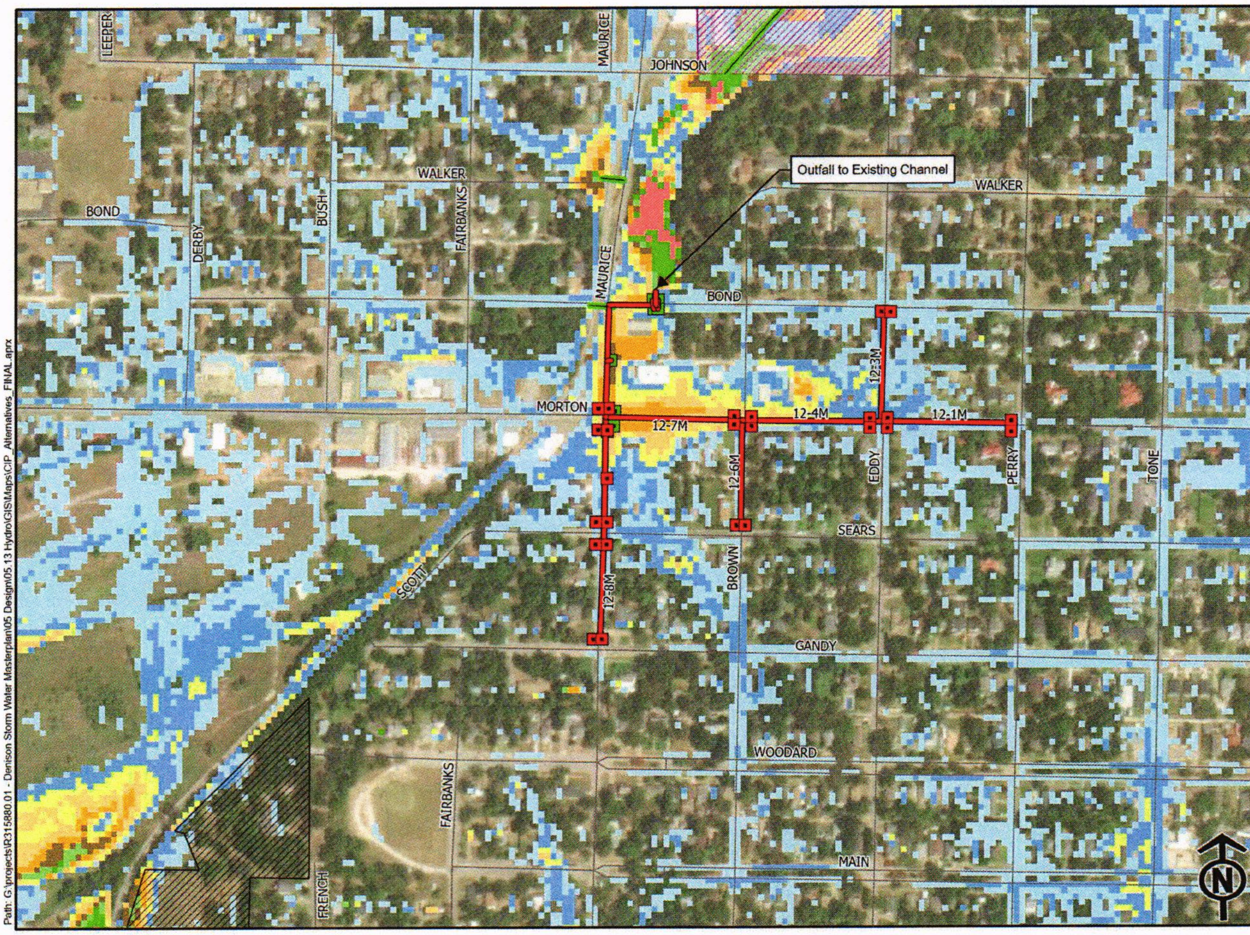
Total Contributing Area (mi²)	0.09
Discharge, 100-year (cfs)	174
Discharge, 10-year (cfs)	104
Max Main Conduit Size (inch)	66

B.5 Cost Estimate

The following table (Table B-2) summarizes the estimated cost for the proposed improvements along Morton Street and Maurice Avenue.

Table B-2: Estimation of Cost – Morton Street and Maurice Avenue

ITEM DESCRIPTION	UNIT	QUANTITY	UNIT COST	TOTAL COST
REMOVE ASPHALT PAVEMENT	SY	195	$ 50	$ 9,750
REMOVE CURB AND GUTTER	LF	350	$ 15	$ 5,250
UNCLASSIFIED STREET EXCAVATION	CY	292	$ 50	$ 14,600
14' STD. CURB INLET	EA	25	$ 13,000	$ 325,000
18" CLASS III REINF. CONC. PIPE	LF	474	$ 190	$ 90,060
24" CLASS III REINF. CONC. PIPE	LF	1,467	$ 200	$ 293,400
30" CLASS III REINF. CONC. PIPE	LF	97	$ 300	$ 29,100
36" CLASS III REINF. CONC. PIPE	LF	791	$ 350	$ 276,850
42" CLASS III REINF. CONC. PIPE	LF	491	$ 420	$ 206,220
54" CLASS III REINF. CONC. PIPE	LF	528	$ 600	$ 316,800
66" CLASS III REINF. CONC. PIPE	LF	55	$ 850	$ 46,750
CITY OF DENISON TYPE B MANHOLE	EA	21	$ 13,500	$ 283,500
CONNECT TO EXISTING DRAINAGE STRUCTURE	EA	5	$ 5,000	$ 25,000
HMAC ASPHALT PAVEMENT (2" TYPE D)	SY	195	$ 40	$ 7,800
6" SEPARATE CONCRETE CURB & GUTTER	LF	350	$ 55	$ 19,250
			SUB-TOTAL =	$ 1,949,330
			CONTINGENCY (APPROX. 40%) =	$ 779,770
			CONTINGENCY SUB-TOTAL =	$ 2,729,100
			ADMIN, ENGINEERING, CONSTRUCTION ADMIN (APPROX. 30%) =	$ 818,900
			TOTAL =	$ 3,548,000



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- Potentially Inaccurate Result
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Proposed Development
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- Mixed
- Residential

Max Depth
ft
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- 3.001 - 3.5
- 3.501 - 5
- 5.001 - 10
- > 10.001

Appendix A
Exhibit B

Proposed Storm Water Layout
Project B

0 125 250 500
Feet

HUITT-ZOLLARS
5430 LBJ Freeway, Suite 1500
Dallas, Texas 75240

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Project C: Sears Street

C Project C – Sears Street

Priority - High

C.1 Location

Intermittent historical flooding has occurred along Sears Street in the downtown area between Tone Avenue and Bennett Avenue. Drainage from Sears Street ultimately outlets to an existing drainage channel that runs parallel to Fannin Avenue and drains from south to north.

C.2 Drainage Problem

Runoff from the catchment contributing to the existing drainage channel at the intersection of Sears and Fannin concentrates along Sears between Tone and Burnett. Flooding on Sears, which is where flow concentrates until the intersection at Fannin, appears to exceed the street capacity of Sears and flood southward from Sears into local neighborhoods. At Fannin Avenue drainage is intercepted by the street and is directed northward to the existing drainage channel. From this existing channel, flows pass through a series of stormwater structures to pass downstream. The hydrologic analysis indicates that the existing conditions along Sears pose a flooding risk for the properties along Sears Street and the surrounding neighborhoods.

The existing storm drain system in this area does not have capacity to adequately carry storm runoff along Sears Street to the existing storm water conveyance channel along Fannin Avenue. The proposed solution would add a network of catch basins and conduits sized to carry runoff to the existing storm water channel at the intersection of Sears and Fannin.

The flooding along Sears has been shown, based on the flooding model, to potentially impact the housing in the surrounding blocks of Sears Street. As flow exceeds the street capacity of Sears, which is at a low point, flow would pass through the surrounding properties to the south and the nearest alley. High priority is assigned because of the potential hazard to many homes.

Figure C-1: Sears Street Looking East at Barrett Avenue toward Fannin Avenue

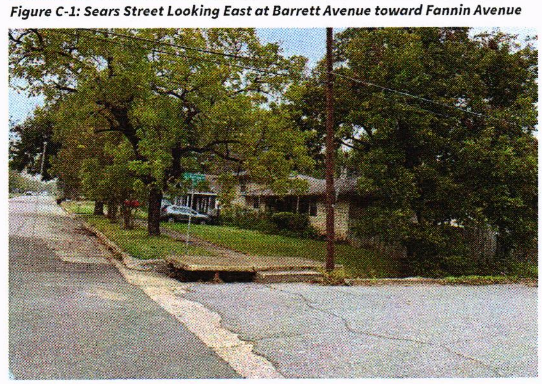


Figure C-2: Photograph of Curb Inlet at Sears Street and Fannin Avenue

Figure C-3: Outfall channel at Bond Street west of Fannin Avenue

C.3 Assumptions

The following assumptions were made to develop the conceptual drainage solution:

- The project lies within the "1930" sub-basin in the HEC-HMS model, to estimate the design flow, discharge is based on a unit discharge (discharge per square mile) basis using the 100-year peak flows estimated from the HEC-HMS model for sub-basin 1930. Unit discharge for sub-basin 1930, in cfs per square mile, is multiplied by the contributing drainage area for the contributing drainage area of 0.12 square miles for the storm drain outlet to determine the maximum system peak flow. Detailed estimation of peak flow is provided in Appendix F.
- Conduits for the storm drain were sized assuming full-flowing, uniform and steady flow conditions (Manning's equation) for the total contributing area to the storm drain and final design should refine the storm drains.
- Storm drain inlets were sized for the street capacity along Sears using design nomographs from the *Storm Drainage System Design Manual* for Denison, Texas (Ref. 1), assuming the following
    - Longitudinal Street Slope of 0.01 foot/foot
    - Street Cross Slope = ¼" per foot with center crown
    - 4" Curb Height
    - Standard curb opening inlet on grade at each catch basin
- Storm drain inlets are placed at areas of flow concentration noted from the two-dimensional FLO-2D model for the 100-year storm. A more detailed hydrologic analysis is required for sizing and placement of storm drain inlets.
- It is assumed that the storm drain system will discharge to the existing stormwater channel on Fannin between Bond Street and Walker Street, where the data on the system is not enough to confirm capacity for the runoff from the storm drain. The drainage is assumed to have adequate capacity, but must be confirmed.

C.4 Conceptual Design

Along Sears Street, a storm drain system with a main under Sears and catch basins located along Sears and the associated cross streets between Tone and Fannin. The main pipe will continue down Fannin Avenue, where it will outlet to an existing drainage between Bond Street and Walker Street. A summary of the design is provided in Table C-1. The proposed layout overlaid on the FLO-2D model results are provided in Exhibit C.

Priority High

Table C-1: Design Summary at Sears Street

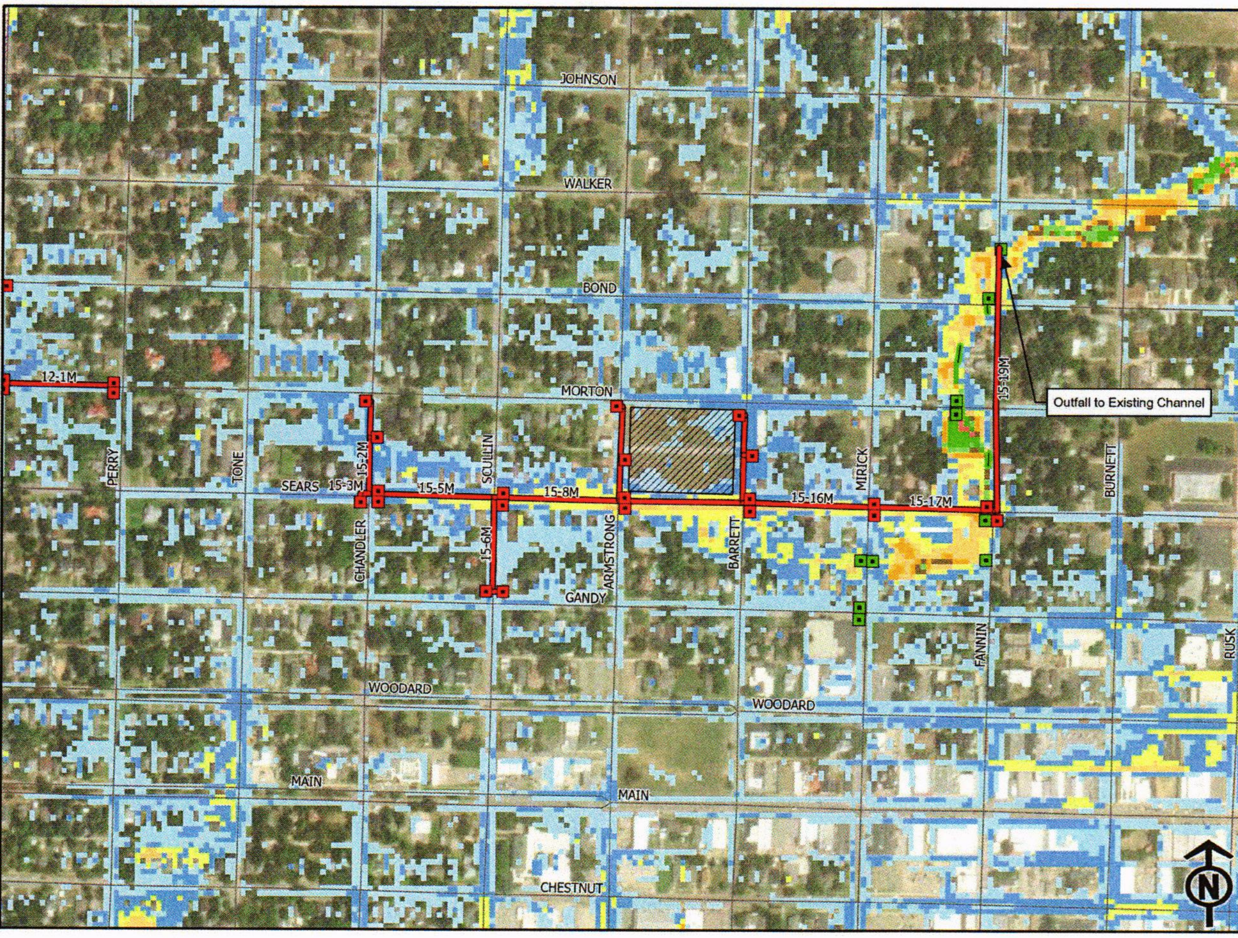
Total Contributing Area (mi^2)	0.12
Discharge, 100-year (cfs)	203
Discharge, 10-year (cfs)	117
Max Main Conduit Size (inch)	66
Inlet Length (ft)	14

C.5 Cost Estimate

Table (Table C-2) summarizes the estimated cost for the proposed improvements at Sears Street.

Table C-2: Estimation of Cost – Sears Street

ITEM DESCRIPTION	UNIT	QUANTITY	UNIT COST	TOTAL COST
REMOVE ASPHALT PAVEMENT	SY	164	$ 50	$ 8,200
REMOVE CURB AND GUTTER	LF	294	$ 15	$ 4,410
UNCLASSIFIED STREET EXCAVATION	CY	245	$ 50	$ 12,250
14' STD. CURB INLET	EA	21	$ 13,000	$ 273,000
18" CLASS III REINF. CONC. PIPE	LF	1,017	$ 190	$ 193,230
24" CLASS III REINF. CONC. PIPE	LF	916	$ 200	$ 183,200
30" CLASS III REINF. CONC. PIPE	LF	473	$ 300	$ 141,900
36" CLASS III REINF. CONC. PIPE	LF	33	$ 350	$ 11,550
42" CLASS III REINF. CONC. PIPE	LF	460	$ 420	$ 193,200
48" CLASS III REINF. CONC. PIPE	LF	940	$ 500	$ 470,000
54" CLASS III REINF. CONC. PIPE	LF	465	$ 600	$ 279,000
66" CLASS III REINF. CONC. PIPE	LF	995	$ 850	$ 845,750
CITY OF DENISON TYPE B MANHOLE	EA	17	$ 13,500	$ 229,500
CONNECT TO EXISTING DRAINAGE STRUCTURE	EA	1	$ 5,000	$ 5,000
HMAC ASPHALT PAVEMENT (2" TYPE D)	SY	164	$ 40	$ 6,560
6" SEPARATE CONCRETE CURB & GUTTER	LF	294	$ 55	$ 16,170
			SUB-TOTAL =	**$ 2,872,920**
			CONTINGENCY (APPROX. 40%) =	**$ 1,149,180**
			CONTINGENCY SUB-TOTAL =	**$ 4,022,100**
			ADMIN, ENGINEERING, CONSTRUCTION ADMIN (APPROX. 30%) =	**$ 1,206,900**
			TOTAL =	**$ 5,229,000**



CITY OF DENISON
300 W Main St
Denison, TX 75020

Denison TEXAS

City of Denison
Conceptual SWMP
R315880.01

- Proposed Catchbasins
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- Existing Catchbasins
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- Potentially Inaccurate Result
- Further Detail Required

Proposed Development
- Commercial
- Mixed
- Residential

Max Depth
ft
- 0.001 - 0.25
- 0.251 - 0.5
- 0.501 - 1
- 1.001 - 1.5
- 1.501 - 2
- 2.001 - 2.5
- 2.501 - 3
- 3.001 - 3.5
- 3.501 - 5
- 5.001 - 10
- > 10.001

Appendix A
Exhibit C

Proposed Storm Water Layout
Project C

0 125 250 500
Feet

HUITT-ZOLLARS
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Dallas, Texas 75240

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Project D: Loy Lake Road at Waterloo Creek Crossing

D Proposed Project D – Loy Lake Road at Waterloo Creek Crossing

Priority - High

D.1 Location

Loy Lake Road at Waterloo Creek Crossing is located approximately 4,600-feet downstream of Waterloo Lake's emergency spillway. Waterloo Lake is manmade and is situated within Waterloo Lake Regional Park, which is owned and operated by City of Denison. The lake has approximately 52-acres of surface area with a maximum death of approximately 20-feet. The lake was constructed in the early 1950s and is a popular spot for recreational activities.

Waterloo Creek runs from north to south through part of the City of Denison, generally from Crawford Street to Spur 503. It is approximately 3.8 miles long, and is a tributary of Iron Ore Creek. Loy Lake Road is a busy commercial corridor with one 18-foot lane in each direction, a 5-foot median, curbs and gutters on both sides, and a 5-foot sidewalk on the north side. The road is classified as a major collector street in the *Master Thoroughfare Plan for the City of Denison Extra-Territorial Jurisdiction area* (Ref. 3). There is an existing 4-barrel, approximately 36-inch diameter culvert under Loy Lake Road to convey the Waterloo Creek flow downstream. The roadway crossing is currently within a FEMA "Zone AE" Floodplain and Floodway.

Figure D-1: Photograph of the Pedestrian Bridge Over Loy Lake Road Facing West

At the crossing of Loy Lake Road, a 200-foot span pedestrian bridge was constructed in 2021 to cross Loy Lake Road as a part of the City's Capital Projects; Katy Trail Phase 1. The pedestrian bridge has approximately 20-feet of clearance from the current road grade.

D.2 Drainage Problem

Historically, the roadway crossing of Waterloo Creek at Loy Lake Road experiences flooding during heavy storm events. The existing culvert crossing is at the road low point, with steep slopes on both sides. Flood water overtopping makes vehicle travel difficult or impassable, and represents a safety risk for vehicle and pedestrian traffic. As a major east-west collector street, flood-induced closures negatively impact emergency response times, as emergency responders must use alternative routes to cross Waterloo Creek. Addressing this issue could be a critical need.

According to the Flood Insurance Study (FIS) (Ref. 4) completed in 1993, the 100-year flow rate at the culvert crossing is 2,250-cfs, and the 50-year flow rate is 2,040-cfs. The regulatory water surface elevation upstream of the roadway crossing is at 645.7-feet, and the roadway elevation is at approximately 641.8-feet. Thus, the water depth over the road is approximately 3.9-feet, which could result in infrastructure damage and pose a considerable safety hazard to pedestrians and motorists. Because of the potential safety hazard and the disruption to emergency response services as a result of flooding across the roadway, high priority is assigned.

D.3 Assumptions

The following assumptions were made to develop the conceptual drainage solution:

- Construction As-Built Plans from the Theresa Drive Water Line Phase 2 (Ref. 6) show the existing structure at Loy Lake Road is four 60" Reinforced Concrete Pipe culverts. Additionally, the plans note a "minimum anticipated vertical clearance" of 17.0-feet across the road. These dimensions should be confirmed for detailed analysis.
- Downstream utilities are a constraint, but the locations must be determined.
- HY-8 software is used for the culvert hydraulic analysis.
- The FEMA Flood Insurance Study (FIS) flow rate is used for the design concept instead of the HEC-HMS model results.
- Without the available as-built plans, the available LiDAR data will be used to estimate the roadway elevations and culvert crossing invert elevations.
- The City of Denison Storm Drainage System Design Manual will be used as the design concept development criteria. The City requires all drainage structures to be designed to carry the 100-year frequency design flow without overtopping.

D.4 Conceptual Design

In order to mitigate the flood risk over the roadway, the capacity of the existing culvert should be increased to convey the 100-year flow without overtopping. Additionally, in order to keep proper vehicle clearance beneath the pedestrian bridge, a minimum clearance of at least 16-feet shall be provided.

The existing culvert has a capacity of approximately 620-cfs, whereas the 100-year flow rate from FIS study is 2,250-cfs. A 7-barrel, 10-foot-wide by 5-foot high box culvert would have the capacity to convey the 100-year flow without overtopping the road. With the proposed box culvert, the culvert crossing roadway not need to be raised, but sufficient cover should be maintained above the box while keeping a minimum of 16-feet of clearance between the roadway and the pedestrian bridge overhead. The clearance for the pedestrian bridge should be evaluated to ensure adequate clearance is provided. From LiDAR data, the slope towards the culvert crossing from the west is approximately 20-percent, and from east is approximately 2-percent. The geometry of the channel downstream is approximately 40-feet, wide and would need to be widened to fit the seven 10-foot wide boxes.

A summary of the design is provided in Table D-1. The proposed layout overlaid on the FLO-2D model results and FEMA flood zones are provided in Exhibit D.

Priority High

Table D-1: Design Summary at Loy Lake Road and Waterloo Creek Crossing

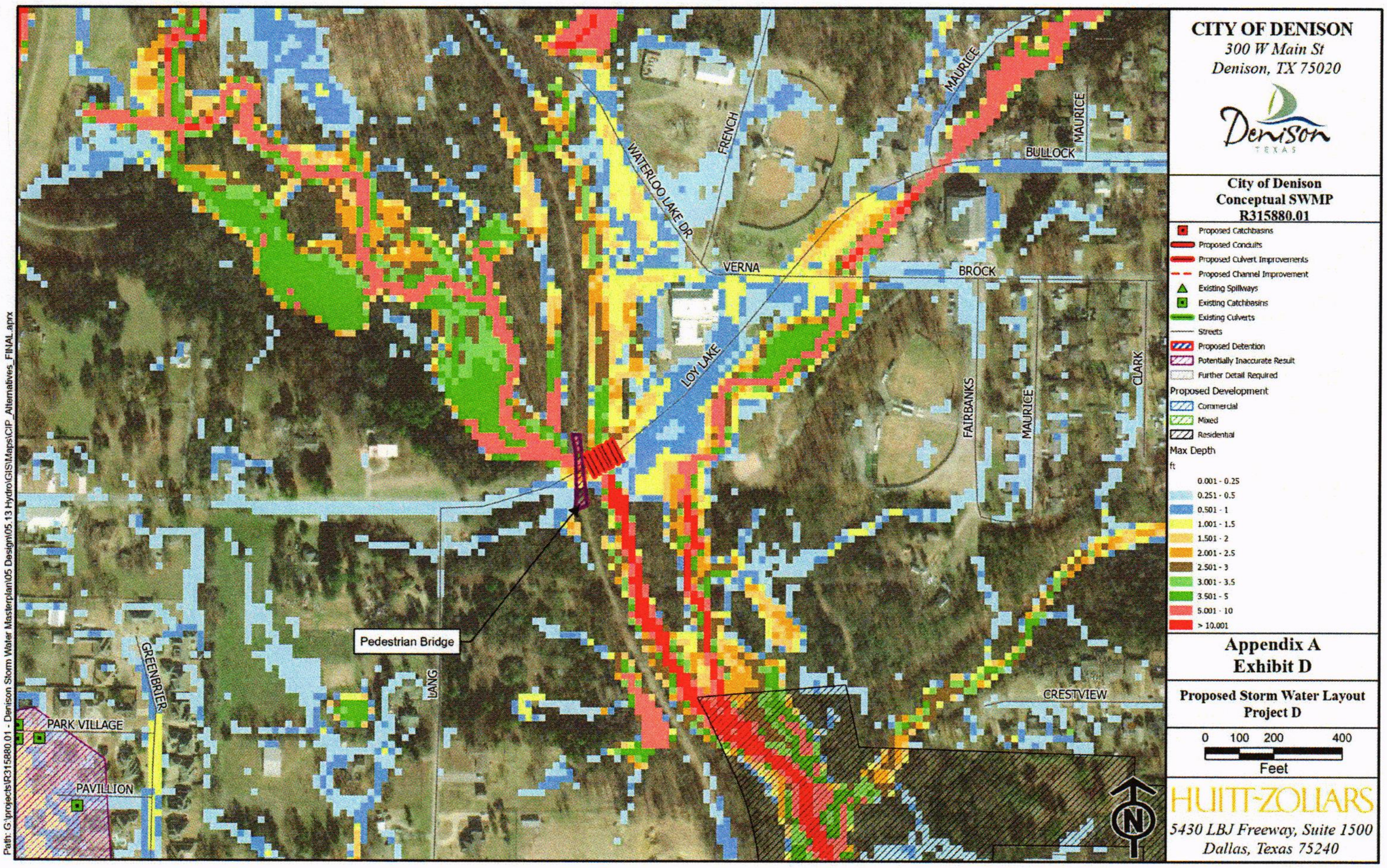
Discharge, 100-year (cfs)	2,250
Proposed Culvert	7 x 10' W x 5' H Box Culvert
Proposed Culvert Capacity (cfs)	2,485

D.5 Cost Estimate

The following table (Table D-2) summarizes the estimated cost for the proposed improvements at Loy Lake Road and Waterloo Creek Crossing. Detailed breakdown of cost is provided in Appendix B.

Table D-2: Estimation of Cost – Loy Lake Road and Waterloo Creek Crossing

ITEM DESCRIPTION	UNIT	QUANTITY	UNIT COST	TOTAL COST
REMOVE ASPHALT PAVEMENT	SY	412	$ 50	$ 20,600
REMOVE CURB AND GUTTER	LF	148	$ 15	$ 2,220
REMOVE & DISPOSE REINFORCED CONCRETE PIPE	LF	200	$ 45	$ 9,000
REMOVE INLET	EA	1	$ 1,400	$ 1,400
REMOVING CONC (MEDIANS)	SY	41	$ 15	$ 617
UNCLASSIFIED STREET EXCAVATION	CY	1,097	$ 50	$ 54,850
TXDOT INLET	EA	1	$ 13,000	$ 13,000
CONC BOX CULV (10 FT X 5 FT)	LF	350	$ 1,200	$ 420,000
SLOPED CONCRETE HEADWALL	EA	14	$ 8,000	$ 112,000
HMAC ASPHALT PAVEMENT (2" TYPE D)	SY	412	$ 40	$ 16,480
EARTHWORK (EROSN & SEDMT CONT, IN VEH)	CY	741	$ 50	$ 37,037
6" SEPARATE CONCRETE CURB & GUTTER	LF	148	$ 55	$ 8,140
CONC MEDIAN	SY	41	$ 75	$ 3,083
			SUB-TOTAL =	**$ 698,427**
			CONTINGENCY (APPROX. 40%) =	**$ 279,373**
			CONTINGENCY SUB-TOTAL =	**$ 977,800**
			ADMIN, ENGINEERING, CONSTRUCTION ADMIN (APPROX. 30%) =	**$ 294,200**
			TOTAL =	**$ 1,272,000**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Appendix A
Exhibit D
Proposed Storm Water Layout
Project D
0 100 200 400
Feet
HUITT-ZOLLARS
5430 LBJ Freeway, Suite 1500
Dallas, Texas 75240
N
WATERLOO LAKE DR
FRENCH
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CLARK
Pedestrian Bridge
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Project E: Loy Lake Road at Loy Creek

E Project E – Loy Lake Road at Loy Creek

Priority - High

E.1 Location

Loy Lake Road is classified as a major collector street, that runs southwest to northeast, crossing US Highway 75 just east of Loy Lake. Loy Lake is a reservoir operated by Grayson County situated on Loy Creek. The primary purpose of Loy Lake is recreation, and according to the National Inventory of Dams (NID), the reservoir has a normal storage of 370 acre-feet with a maximum capacity of 642 acre-feet, and captures a drainage area of approximately 2.4 square miles. The lake, which was constructed in 1933, is held by an earthen dam with an uncontrolled spillway which outlets south and east of the dam towards Loy Lake Road. At Loy Lake Road a double pipe culvert crossing under Loy Lake Road outfalls to Loy Creek.

E.2 Drainage Problem

The culvert crossing for Loy Creek under Loy Lake Road rarely receives major runoff flow since it is mostly limited upstream by Loy Lake. Since the Loy Lake spillway is unregulated, the culvert crossing will only receive runoff from the larger sub-basin contributing to Loy Lake when discharge of the Loy Lake spillway occurs. For all other rainfall events, only the catchment immediately downstream of the Loy Lake spillway produces runoff at the culvert crossing.

Since the catchment immediately downstream of the Loy Lake spillway is relatively flat and heavily vegetated, the culvert crossing is prone to clogging with debris. The hydraulic performance of the culvert crossing could be hindered by the heavy vegetation and the potential for debris produced downstream of the Loy Lake spillway.

Loy Lake Road is located within a designated regulatory floodway and "Zone AE" floodplain (Figure E-1). A regulatory floodway means the channel of a river/watercourse and the adjacent lands must be reserved in order to discharge the base flood without cumulatively increasing the water surface elevation more than a designated height. "Zone AE" denotes a 100-year floodplain where base flood elevations are provided. The water surface of the regulatory floodway at the roadway is elevation 668.3-feet. While detailed survey of the road deck at the Loy Lake Road culvert crossing is not available, available LiDAR survey at the location indicates the road deck is approximately at the elevation 655.4-feet, suggesting that for the 100-year flood, the roadway could be inundated to a depth of greater than 12-feet. Because the potential flood depth above the roadway is so high for the 100-year flood and because the roadway serves as a major emergency response purpose between east and west, a high priority is assigned.

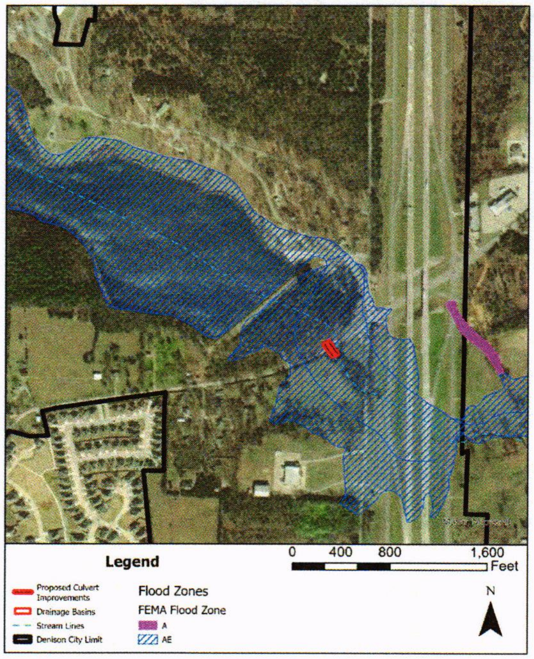


Figure E-1: Loy Lake Road FEMA Flood Zone Map

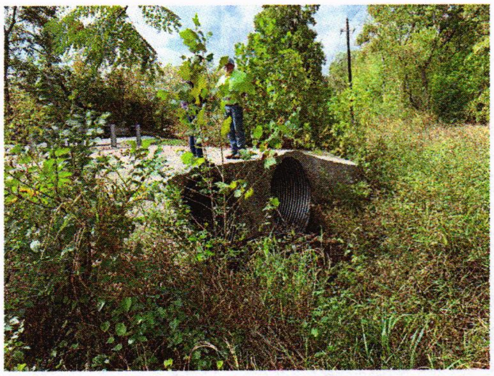


Figure E-2: Photograph of Inlet for Culvert Crossing for Loy Creek at Loy Lake Road

Figure E-3: Photograph of Outlet for Culvert Crossing for Loy Creek at Loy Lake Road

E.3 Assumptions

The following assumptions were made to develop the conceptual drainage solution:

- With an unregulated spillway, flood mitigation is provided by the freeboard storage of Loy Lake, however, the level of protection provided by Loy Lake is uncertain because the water level is not managed. As an approximation of the flood peak attenuation provided by Loy Lake, the reservoir was assumed to be at the normal storage of 370 acre-feet at the beginning of the 100-year flood event.
- The stream crossing lies part-way within sub-basin 820, while also receiving the upstream contributing drainage for Loy Lake, the modelled 100-year discharge for reach R-813-820 from HEC-HMS is added to the area-based partial discharge from sub-basin 820 to determine the total design flow.
- The receiving channel downstream of Loy Lake Road contains storm water infrastructure to pass flow through U.S. Highway 75, but as-built information on this infrastructure is unavailable. It is assumed that the downstream infrastructure has capacity for extreme flood events and does not influence the tailwater at Loy Lake Road.

- Without detailed survey and as-built information at the culvert crossing of Loy Lake Road, the culvert size (2 x 48" CMP pipes) based on field visit photographs and with assumed invert elevations and channel geometry from available LiDAR data.

E.4 Conceptual Design

The conceptual design at the culvert crossing for Loy Creek at Loy Lake Road includes a stilling basin upstream of the culvert crossing to collect debris and upgraded culverts with capacity for the 100-year flow. The proposed culvert is tabulated in Table E-1 and the layout is shown in Exhibit E.

Priority High

Table E-1: Design Summary for Loy Creek Crossing at Loy Lake Road

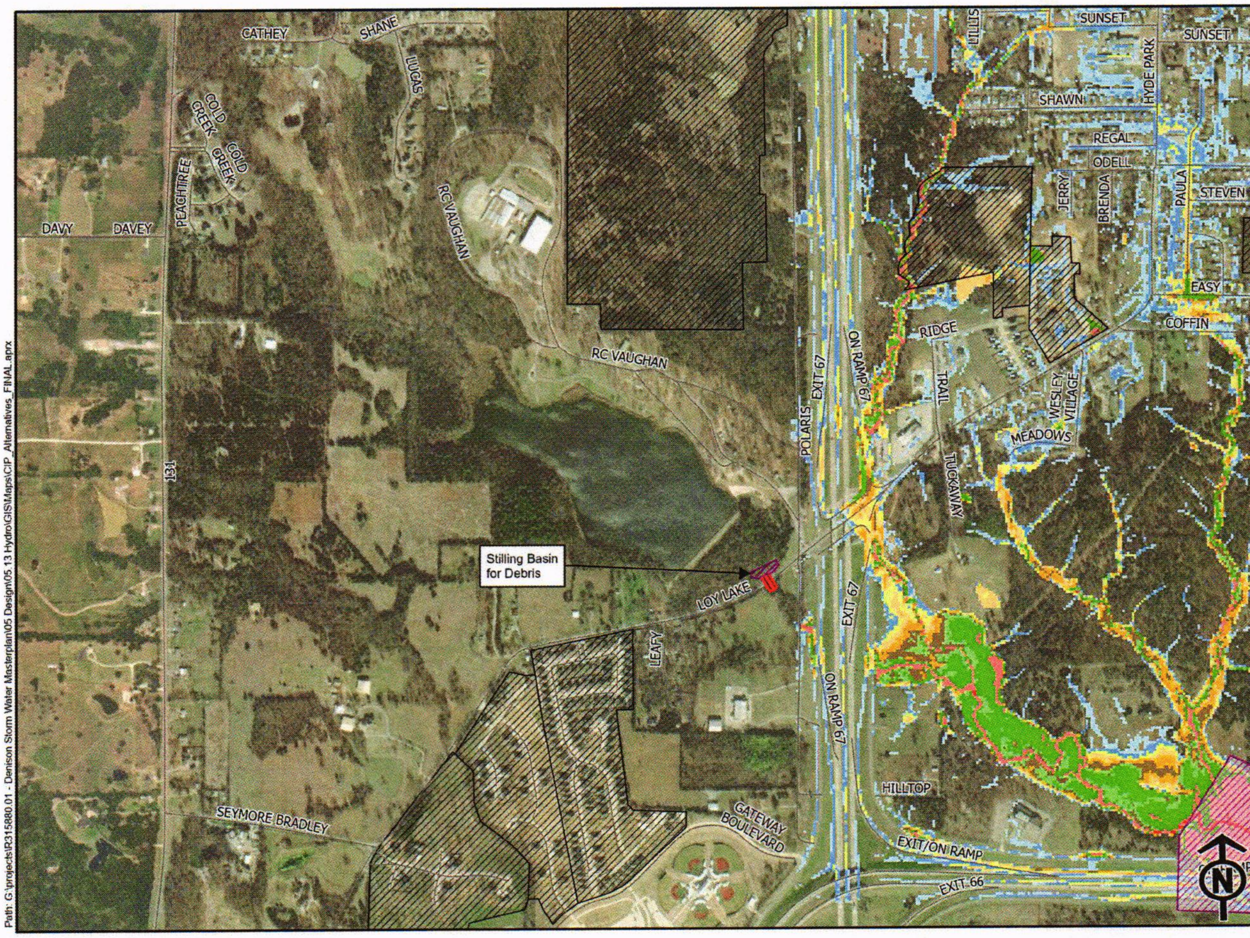
Total Contributing Area (mi^2)	2.4
Discharge, 100-year (cfs)	3290
Discharge, 10-year (cfs)	1780
100-year Flood Elevation (ft)	668.3
Proposed Culvert	6 x 8' W x 6' H Concrete Box

E.5 Cost Estimate

The following table (Table E-2) summarizes the estimated cost for the proposed improvements at Loy Creek. Detailed breakdown of cost is provided in Appendix B.

Table E-2: Estimation of Cost – Loy Creek Crossing at Loy Lake Road

ITEM DESCRIPTION	UNIT	QUANTITY	UNIT COST		TOTAL COST	
REMOVE ASPHALT PAVEMENT	SY	185	$	50	$	9,250
REMOVE & DISPOSE REINFORCED CONCRETE PIPE	LF	200	$	45	$	9,000
REMOVE HEADWALL	EA	2	$	350	$	700
UNCLASSIFIED STREET EXCAVATION	CY	555.00	$	50	$	27,750
CONC BOX CULV (8 FT X 6 FT)	LF	450	$	1,200	$	540,000
SLOPED CONCRETE HEADWALL	EA	2	$	8,000	$	16,000
REMOVE EXISTING TREES	EA	7	$	1,700	$	11,900
EARTHWORK (EROSN & SEDMT CONT, IN VEH)	CY	1,614	$	50	$	80,700
HMAC ASPHALT PAVEMENT (2" TYPE D)	SY	185	$	40	$	7,400
				SUB-TOTAL =	**$**	**702,700**
				CONTINGENCY (APPROX. 40%) =	**$**	**281,100**
				CONTINGENCY SUB-TOTAL =	**$**	**983,800**
				ADMIN, ENGINEERING, CONSTRUCTION ADMIN (APPROX. 30%) =	**$**	**295,200**
				TOTAL =	**$**	**1,279,000**



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- Proposed Catchbasins
- Proposed Conduits
- Proposed Culvert Improvements
- Proposed Channel Improvement
- Existing Spillways
- Existing Catchbasins
- Existing Culverts
- Streets
- Proposed Detention
- Potentially Inaccurate Result
- Further Detail Required

Proposed Development
- Commercial
- Mixed
- Residential

Max Depth
ft
- 0.001 - 0.25
- 0.251 - 0.5
- 0.501 - 1
- 1.001 - 1.5
- 1.501 - 2
- 2.001 - 2.5
- 2.501 - 3
- 3.001 - 3.5
- 3.501 - 5
- 5.001 - 10
- > 10.001

Appendix A
Exhibit E

Proposed Storm Water Layout
Project E

0 250 500 1,000
Feet

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Project F – Coffin Street & Woodlawn Boulevard

F Proposed Project F – Coffin Street & Woodlawn Boulevard

Priority - Medium

F.1 Location

At the intersection of Coffin Street and Woodlawn Boulevard flooding occurs during intense storm events. The existing stormwater infrastructure consists of a curb inlet at the northeast corner of the intersection. Drainage from the surrounding catchment area flows toward this intersection from Coffin Street. East of the railroad, neighborhood flooding and ponding occurs behind the railroad tracks. Additional flow from a drainage ditch west of, and parallel to, the railroad tracks contributes to Coffin and the intersection (Exhibit F).

Katy Trails Village, a mixed-use proposed development is planned to the west of intersection on Coffin Street. The planned development includes improvements to Coffin Street west of the intersection of Coffin and Woodlawn and the associated drainage.

F.2 Drainage Problem

The intersection is prone to flooding which occurs when runoff travels down Coffin Street from the railroad crossing. The flooding associated with runoff from Coffin Street is likely due to a lack of stormwater capacity at both the existing drainage channel and culvert crossing at Coffin Street near the railroad crossing. Existing topographic LiDAR survey indicates that the drainage channel lacks capacity, and field investigation indicates that the channel is heavily vegetated, further reducing the capacity of the channel.

Other potential drainage problems may result from not having sufficient inlet capacity at the intersection of Coffin and Woodlawn. Additional inlets that tie-in with the existing curb inlet would add additional capacity at the intersection.

The flooding at the intersection has the potential to continue down Coffin Street to impact the proposed development at Katy Trails Village. This project has been assigned a medium priority.

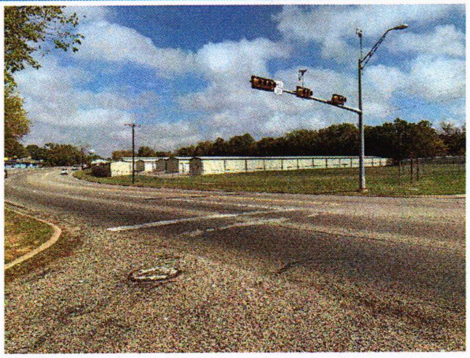


Figure F-1: Photograph Facing Northeast at Intersection of Coffin and Woodlawn

Figure F-2: Photograph facing Northward toward Drainage Channel on Coffin near Railroad

F.3 Assumptions

The following assumptions were made to develop the conceptual drainage solution:

- The project lies within the "871" sub-basin in the HEC-HMS model, to estimate the design flow, discharge is based on a unit discharge (discharge per square mile) basis using the 100-year peak flows estimated from the HEC-HMS model for sub-basin 871. Unit discharge for sub-basin 871, in cfs per square mile, is multiplied by the contributing drainage area at the storm drain location of 0.09 square miles to determine peak flow. Detailed estimation of peak flow is provided in Appendix F.
- Storm drain inlets are placed at areas of flow concentration noted from the two-dimensional FLO-2D model for the 100-year storm. A more detailed hydrologic analysis is required for sizing and placement of storm drain inlets.
- Existing culvert crossing sizes were assumed based on site photographs due to a lack of existing as-built information, including invert elevations, culvert dimensions, and detailed surveys upstream and downstream of culverts. Culverts were modeled based on existing photographs and LiDAR survey.
- Steady, uniform flow was assumed for the drainage channel analysis

F.4 Conceptual Design

Along Coffin Street, a storm drain that captures flow from the existing channel is proposed that travels westward down Coffin Street and through the Katy Trails Village proposed development, where it will discharge to Waterloo Creek. The proposed storm drain includes additional inlets as needed for flow along Coffin Street. Additionally, channel improvements are proposed to add capacity for the 100-year runoff at the drainage channel along the railroad until Coffin Street, where flow will be picked up by the proposed storm drain. A summary of the design is provided in Table F-1. The proposed layout overlaid on the FLO-2D model results are provided in Exhibit F.

Priority Medium

Table F-1: Design Summary at Coffin Street and Woodlawn Boulevard

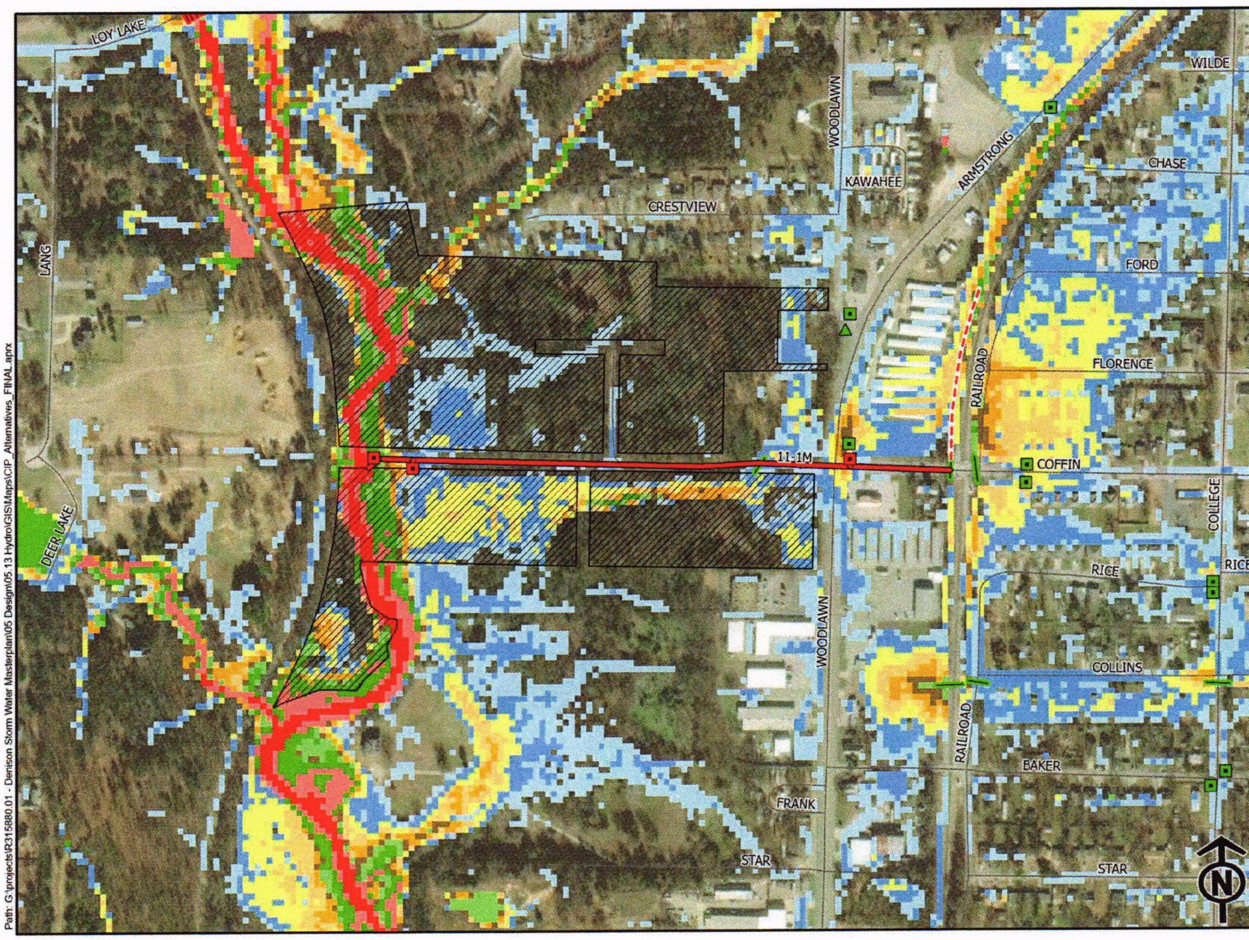
Railroad Crossing Channel		Storm Drain	
Total Contributing Area (mi^2)	0.09	**Total Contributing Area (mi^2)**	0.10
Discharge, 100-year (cfs)	178	**Discharge, 100-year (cfs)**	206
Discharge, 10-year (cfs)	104	**Discharge, 10-year (cfs)**	120
Culvert Requirement	3 x 36" Dia. Concrete Circular	**Main Pipe Diameter**	60" Dia. Reinforced Concrete Pipe
Channel Requirement	5' Wide x 4' Deep Trapezoidal	**Inlet Type**	10' Standard Curb Inlet

F.5 Cost Estimate

The following table (Table F-2) summarizes the estimated cost for the proposed improvements at Coffin Street and Woodlawn Boulevard.

Table F-2: Estimation of Cost – Coffin Street and Woodlawn Boulevard

ITEM DESCRIPTION	UNIT	QUANTITY	UNIT COST	TOTAL COST
REMOVE ASPHALT PAVEMENT	SY	17	$ 50	$ 850
REMOVE & DISPOSE CMP PIPE	LF	50	$ 40	$ 2,000
REMOVE & REPLACE PAVEMENT, CURB, & GUTTER	LS	1	$ 8,000	$ 8,000
UNCLASSIFIED STREET EXCAVATION	CY	25	$ 50	$ 1,250
CITY OF DENISON TYPE B MANHOLE	EA	3	$ 13,500	$ 40,500
10' STD. CURB INLET	EA	3	$ 11,000	$ 33,000
18" CLASS III REINF. CONC. PIPE	LF	100	$ 190	$ 19,000
66" CLASS III REINF. CONC. PIPE	LF	2,000	$ 850	$ 1,700,000
SLOPED CONCRETE HEADWALL	EA	2	$ 8,000	$ 16,000
HMAC ASPHALT PAVEMENT (2" TYPE D)	SY	17	$ 40	$ 680
EARTHWORK (EROSN & SEDMT CONT, IN VEH)	CY	1,289	$ 50	$ 64,450
			SUB-TOTAL =	$ 1,885,730
			CONTINGENCY (APPROX. 40%) =	$ 754,370
			CONTINGENCY SUB-TOTAL =	$ 2,640,100
			ADMIN, ENGINEERING, CONSTRUCTION ADMIN (APPROX. 30%) =	$ 792,900
			TOTAL =	$ 3,433,000



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- Proposed Catchbasins
- Proposed Conduits
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Proposed Development
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Max Depth
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- 2.501 - 3
- 3.001 - 3.5
- 3.501 - 5
- 5.001 - 10
- > 10.001

Appendix A
Exhibit F

Proposed Storm Water Layout
Project F

0 125 250 500
Feet

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Project G: Stream Downstream of Coffin Street & Park Avenue

G Proposed Project G – Stream Downstream of Coffin Street and Park Avenue

Priority - Medium

G.1 Location

Located just east of the intersection of Coffin Street and Park Avenue, the existing stormwater drainage from the storm drain systems along Park Avenue, Renaissance Drive, Turtlecreek, and Park Village combine in an existing stream before emptying into an existing pond near Deer Lake Drive.

The existing 48" diameter reinforced concrete culvert at the intersection of Coffin Street and Park Avenue empties into an existing stream for about 250-feet before being captured in a 72" diameter reinforced concrete culvert which combines this flow with storm drain flow originating from the north. The 72" diameter culvert releases flow into a channel that transects the properties along Renaissance Drive, separating each home from the rest of the property on the other side of the existing stream.

At least 4 of the properties on Renaissance Drive have built stream crossings for access to the other side of the stream, where at least three are 24" concrete pipe culverts and one is a wooden bridge crossing. After the final stream crossing, there is a grade control structure before the water is released to natural drainage (Figure G-2). Once in a natural, unconfined channel, the stream meanders easternly before releasing into the existing pond at Deer Lake Drive.

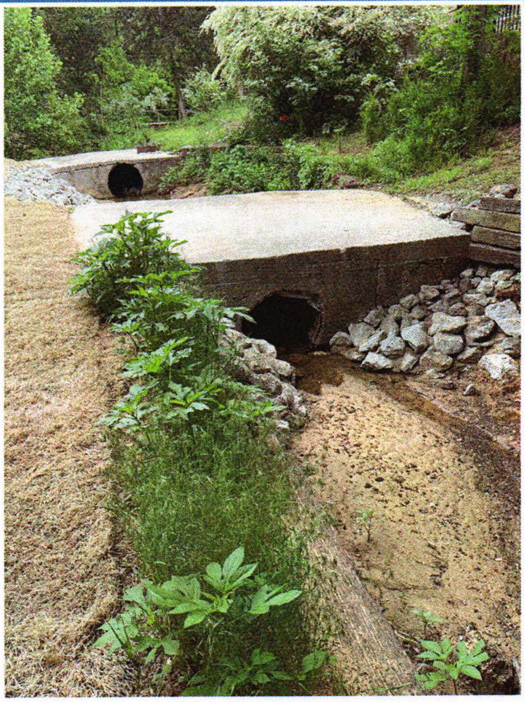


Figure G-1: Photograph of stream crossings with 12" culverts

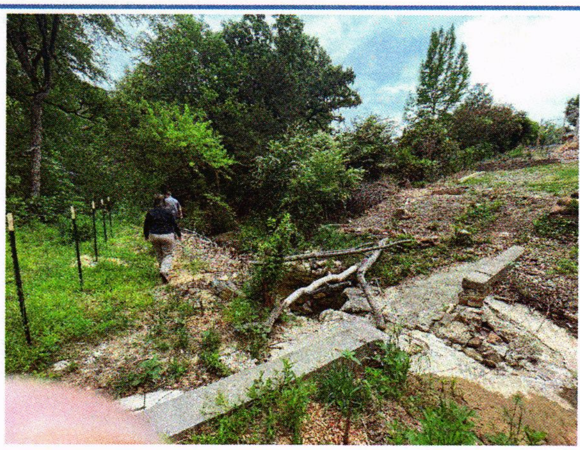


Figure G-2: Photograph of Downstream Grade Control Structure, Looking Downstream

G.2 Drainage Problem

Erosion of the right bank (facing downstream) of the existing stream is causing scour holes to form near the homes on Renaissance Drive. Additional analysis is required to determine whether these scour holes pose a potential structural stability threat to the structures on these properties, but if the erosion is allowed to continue, further scour and erosion may continue towards the homes along Renaissance Drive.

Scour and meandering downstream of a series of hydraulic structures is a symptom of erosive flows causing channel instability downstream of a grade control structure. The stream crossings constructed by property owners do not appear to be engineered structures sized for the appropriate storm event, since the 100-year flow in this stream is estimated to be about 350 cfs, which is well in excess of the capacity of a single 24-inch pipe. The existing 72" culvert appears to have adequate capacity, but the 12" culvert crossings are severely undersized. Evidence of sedimentation and erosion along the series of stream crossings were noted upon field visit by HZ, and sediment accumulation can be noted in Figure G-1. Sedimentation at undersized stream crossings increases flow erosivity further downstream.

Without proper armoring and erosion control, further degradation at the banks of the existing stream is likely. Engineered structures are required to stabilize the stream channel and prevent further erosion from degrading the stream bank in the direction of the homes on Renaissance Drive.

G.3 Assumptions

The following assumptions were made to develop the conceptual drainage solution:

- Design flows were estimated based on a hydrology model at a flow per area ratio for the contributing area to specific flow points for this project.
- The size of structures in the stream were estimated based on observations from an HZ field visit.
- No detailed analysis of scour, erosion, or sedimentation is performed. Detailed analyses of these stream morphology properties should be conducted in a detailed engineering analysis for design.
- No structural analysis of the soils near Renaissance Drive is performed and should be performed to determine the potential structural hazard.

G.4 Conceptual Design

The engineered solution in this area is to provide sufficient armoring with gabions along the west bank. Gabions, which are engineered cages filled with rocks, would serve as both retaining walls and erosion protection for the bank, and would be placed from the estimated potential scour depth to the top bank of the stream channel. The gabions would extend from the bottom of the existing stream meander up to the existing grade control structure shown in Figure G-2. Based on the condition of the existing grade control structure, it will be replaced with a gabion grade control structure.

The grade control structure and gabion wall along the western bank of the stream would prevent further lateral migration of the existing stream and prevent further head cut erosion. The proposed design would not extend further upstream to the existing stream crossing structures. Since one bank of the stream would be armored for scour, the other bank would remain vulnerable to potential erosion and scour. Because there are no existing structures in the vicinity of the west bank, it has been assumed that no erosion protection is necessary along the east bank of the stream.

While it may be advantageous to extend bank protection further upstream to additional properties above the grade control structure, the upstream drainage appears to be fairly constrained which would limit additional bank erosion without additional intervention. Additionally, the existing built structures along the stream upstream of the grade control structure would complicate construction of bank protection with existing property owners. The constrained channel upstream of the grade control structure does not show signs of significant scour or lateral migration like is seen downstream. With an upgraded grade control structure, no excessive stream migration or scour hole development is anticipated in the upstream portion of the stream. For this reason, no additional erosion protection is proposed upstream of the grade control structure.

Priority Medium

Table G-1: Design Summary at Stream near Coffin Street and Park Avenue

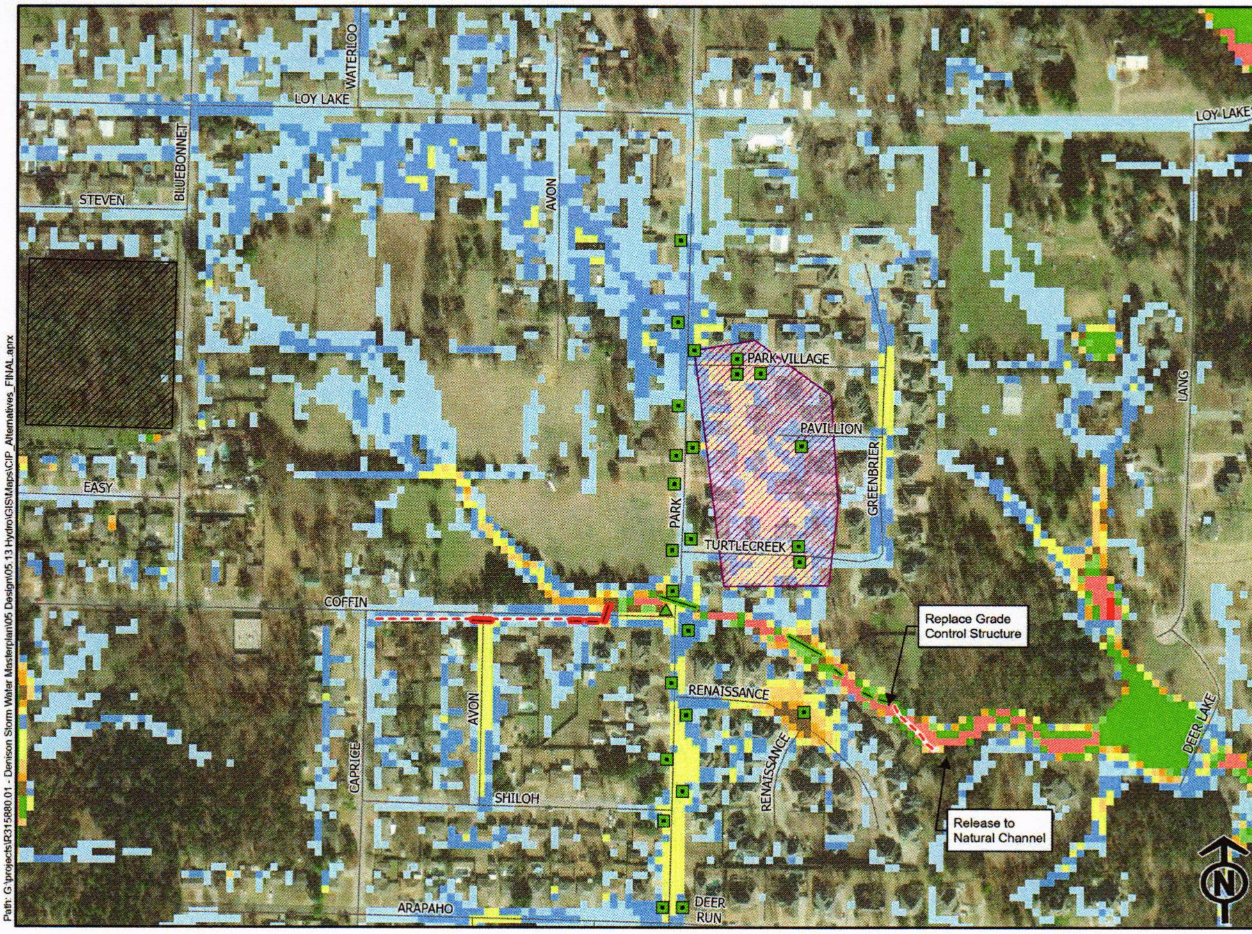
Contributing Area (mi²)	0.196
Discharge, 100-year (cfs)	350

G.5 Cost Estimate

The following table (Table G-2) summarizes the estimated cost for the proposed improvements. Detailed breakdown of cost is provided in Appendix B.

Table G-2: Estimation of Cost –Stream near Coffin Street and Park Avenue

ITEM DESCRIPTION	UNIT	QUANTITY	UNIT COST	TOTAL COST
REMOVE EXISTING TREES	EA	20	$ 1,700	$ 34,000
MOBILIZATION & SITE PREPARATION	LS	1	$ 100,000	$ 100,000
STREAM CONSTRUCTION ACCESS	LS	1	$ 25,000	$ 25,000
UNCLASSIFIED STREAM EXCAVATION	CY	570	$ 80	$ 45,600
BACKFILL	CY	400	$ 30	$ 12,000
REMOVE GRADE CONTROL STRUCTURE	EA	1	$ 5,000	$ 5,000
GRADE CONTROL STRUCTURE	CY	30	$ 800	$ 24,000
GABIONS (3' X 3') (GALV)	CY	420	$ 400	$ 168,000
FILTER FABRIC	SY	400	$ 20	$ 8,000
			SUB-TOTAL =	**$ 421,600**
			CONTINGENCY (APPROX. 40%) =	**$ 168,700**
			CONTINGENCY SUB-TOTAL =	**$ 590,300**
			ADMIN, ENGINEERING, CONSTRUCTION ADMIN (APPROX. 30%) =	**$ 177,700**
			TOTAL =	**$ 768,000**



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R315880.01

Proposed Catchbasins
Proposed Conduits
Proposed Culvert Improvements
Proposed Channel Improvement
Existing Spillways
Existing Catchbasins
Existing Culverts
Streets
Proposed Detention
Potentially Inaccurate Result
Further Detail Required

Proposed Development
Commercial
Mixed
Residential

Max Depth
ft
0.001 - 0.25
0.251 - 0.5
0.501 - 1
1.001 - 1.5
1.501 - 2
2.001 - 2.5
2.501 - 3
3.001 - 3.5
3.501 - 5
5.001 - 10
> 10.001

Appendix A
Exhibit G

Proposed Storm Water Layout
Project G

0 100 200 400
Feet

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Project H – Lillis Lane and Crawford Street

H Project H – Lillis Lane and Crawford Street

Priority - Medium

H.1 Location

At the intersection of Lillis Lane and Crawford Street, an existing storm drain system and culvert crossing pass storm water under the roadway towards Waterloo Lake. The Rosemary, a planned mixed-use development, is permitted north and west of the intersection along Lillis Lane near the intersections with Elk Street and Viking Street (see Exhibit H).

H.2 Drainage Problem

At this intersection, the upstream drainage is heavily vegetated and has the potential to generate debris, which could clog the existing culvert. This drainage course lies within the FEMA 100-year floodplain, which is classified as "Zone A". Zone A is an Approximate Zone meaning a detailed analysis has not performed in the area and that no base flood elevations are available. The existing culvert crossing under the intersection is likely prone to clogging and is undersized for the 100-year flood.

Detailed as-built information for the existing culvert crossing is currently unavailable, including information on the culvert size, invert elevations, and any survey of upstream and downstream drainageways. Photographs provided by the City of Denison were used to evaluate the existing conditions are shown on Figure H-1 and Figure H-2. The photographs appear to show that the upstream end consists of concrete circular pipes, while the downstream end consists of a concrete box, likely changing shape when storm drain inlets connect under the intersection of Lillis and Crawford.

Flooding hazards in this area include flooding across the intersection when the capacity of the existing structure is exceeded. Homes located southwest of this intersection may be impacted by flood flows cascading over the intersection and potentially directing flood flows into the homes. Depending on flood flows across the intersection, traffic along the intersection may be disrupted. Due to these potential hazards, medium priority is assigned.

Figure H-1: Photograph at Inlet of Culvert Crossing at Lillis and Crawford

Figure H-2: Photograph of Outlet at Lillis and Crawford

H.3 Assumptions

The following assumptions were made to develop the conceptual drainage solution:

- The project lies within the "700" sub-basin in the HEC-HMS model, to estimate the design flow, discharge is based on a unit discharge (discharge per square mile) basis using the 100-year peak flows estimated from the HEC-HMS model for sub-basin 700. Unit discharge for sub-basin 700, in cfs per square mile, is multiplied by the contributing drainage area at the structure location of 0.57 square miles to determine peak flow. Detailed estimation of peak flow is provided in Appendix F.
- Existing inlets and storm drain conduits are assumed to have capacity for the design storm since the flooding problems appear to be associated with the culvert crossing under the intersection.
- Without detailed as-built information on the culvert crossing, the following existing configurations were tested, and the invert elevations were assumed based on available LiDAR survey:
  - Two 48" concrete circular culverts
  - Single 5' x 5' box culvert

H.4 Conceptual Design

Analysis of the existing culvert crossing found that neither the two 48" circular culverts nor the 5' x 5' concrete box has enough capacity to convey the 100-year flow. The conceptual design incorporates a larger culvert crossing under the intersection at Lillis and Crawford, sized for the 100-year flow. Inlet improvements on the upstream end are recommended to reduce the clogging potential. A summary of the design is provided in Table H-1. The proposed layout overlaid on the FLO-2D model results are provided in Exhibit H.

Priority Medium

Table H-1: Design Summary at Lillis & Crawford

Total Contributing Area (mi²)	0.57
Discharge, 100-year (cfs)	1203
Discharge, 10-year (cfs)	734
Required Culvert for 100-year	Six 5' W x 5' H Concrete Box

H.5 Cost Estimate

The following table (Table H-2) summarizes the estimated cost for the proposed improvements at Lillis and Crawford. Detailed breakdown of cost is provided in Appendix B.

Table H-2: Estimation of Cost – Lillis & Crawford

ITEM DESCRIPTION	UNIT	QUANTITY	UNIT COST	TOTAL COST
REMOVE ASPHALT PAVEMENT	SY	567	$ 50	$ 28,350
REMOVE CURB AND GUTTER	LF	300	$ 15	$ 4,500
REMOVE & DISPOSE REINFORCED CONCRETE PIPE	LF	300	$ 45	$ 13,500
REMOVE HEADWALL	EA	2	$ 350	$ 700
UNCLASSIFIED STREET EXCAVATION	CY	1,512.00	$ 50	$ 75,600
CONC BOX CULV (5 FT X 5 FT)	LF	1,500	$ 500	$ 750,000
SLOPED CONCRETE HEADWALL	EA	2	$ 8,000	$ 16,000
HMAC ASPHALT PAVEMENT (2" TYPE D)	SY	567	$ 40	$ 22,680
6" SEPARATE CONCRETE CURB & GUTTER	LF	300	$ 55	$ 16,500
			SUB-TOTAL =	**$ 927,830**
			CONTINGENCY (APPROX. 40%) =	**$ 371,170**
			CONTINGENCY SUB-TOTAL =	**$ 1,299,000**
			ADMIN, ENGINEERING, CONSTRUCTION ADMIN (APPROX. 30%) =	**$ 390,000**
			TOTAL =	**$ 1,689,000**

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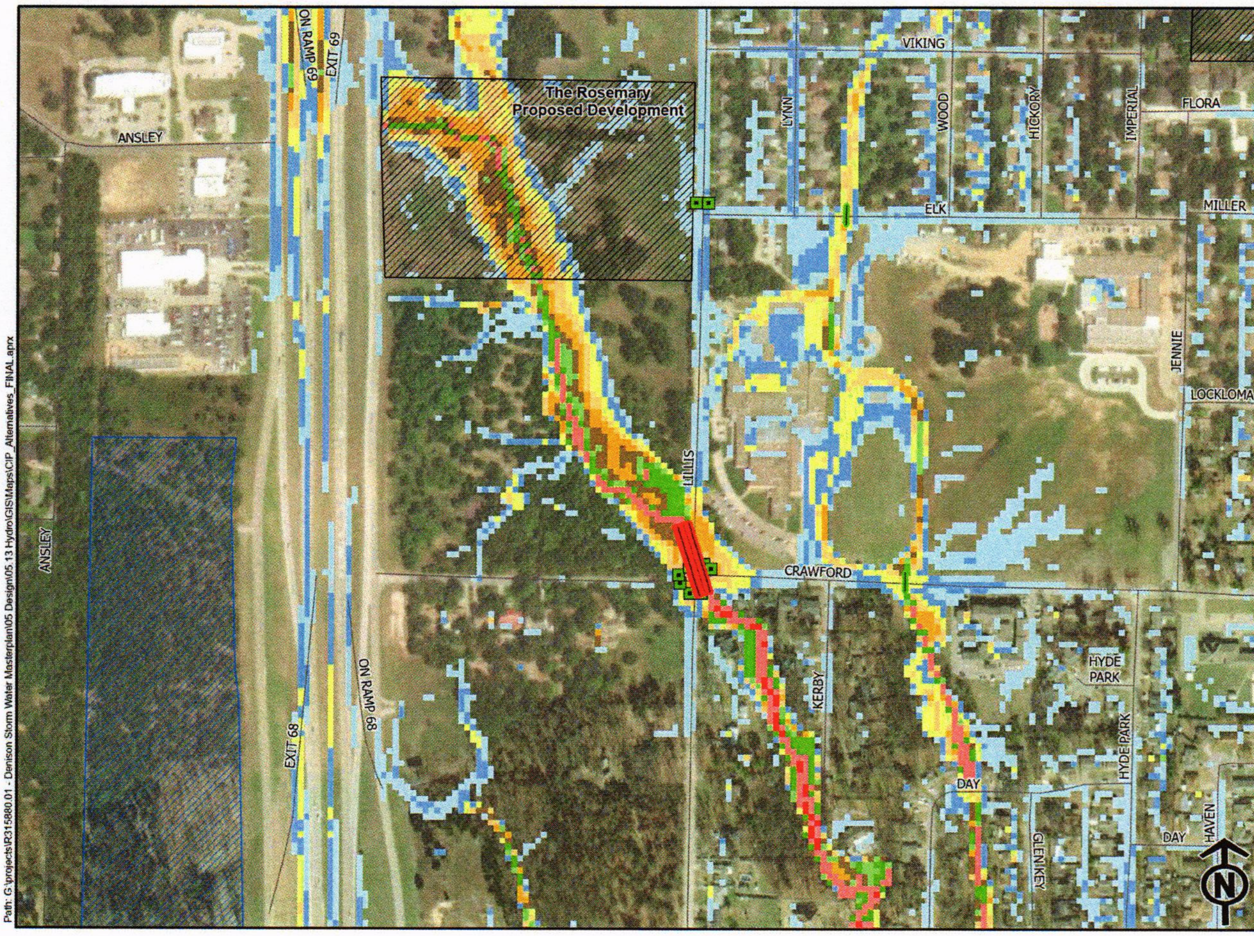

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- Proposed Catchbasins
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- Streets
- Proposed Detention
- Potentially Inaccurate Result
- Further Detail Required

Proposed Development

- Commercial
- Mixed
- Residential

Max Depth
ft

- 0.001 - 0.25
- 0.251 - 0.5
- 0.501 - 1
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- 2.001 - 2.5
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- > 10.001

Appendix A
Exhibit H

Proposed Storm Water Layout
Project H

0 125 250 500
Feet

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Project I: Chandler Avenue near Munson Street

I Proposed Project I – Chandler Avenue near Munson Street

Priority - Medium

I.1 Location

Chandler Avenue between Day Street and Munson Street is a residential street that lacks a curb and gutter system. While there is not a curb and gutter system, roadside drainage ditches run along the east and west sides of the street with corrugated metal circular culverts allowing the drainage to pass under driveways and alleys. At the intersection of Chandler Avenue and Munson Street, both roadside ditches connect to catch basins with an approximately 12-foot curb inlet opening on the street and a 24" reinforced concrete pipe inlet from the drainage ditch. Additional storm drain inlets are placed on both the north and south sides of Munson Street approximately 130 feet east of the intersection with Chandler Avenue.

After the intersection, the layout of the receiving storm drainage system is unclear, and there is no available as-built information on the system for review. From review of aerial imagery and available topography, it appears that the storm drainage outfalls to surface drainage on the south side of Munson Street, approximately 130 feet east of the intersection of Munson Street and Chandler Avenue. In the drainage south of Munson Street, drainage travels via open channel between Chandler Avenue and Scullin Avenue, passing through a culvert under Hull Street, where it enters a storm drain under Texas Street which outlets to natural drainage near the intersection of Chandler Avenue and Monterey Street.

I.2 Drainage Problem

Two homes on the northeast corner of Chandler Avenue and Munson Street have experienced flooding issues starting in the alley north of the homes. Water accumulates near the shared property line and runs north-to-south between the two properties. Analysis of the drainage near this area shows that there is an existing low point in the northern alley near the property line between the two properties, and after running along the property line, storm water passes to an existing storm drain inlet on Munson Street.

The grading in the area is fairly steep, draining north-to-south, and all areas to the north and east of Chandler Avenue drain down towards the low point near these two properties. Grading near the alley is such that rather than directing storm water towards the existing drainage ditch and culverts along Chandler Avenue, flow is pushed eastward towards the two properties that experience flooding problems. Figure I-1 is a photograph from a field visit, where the grading can be seen to slope towards the home nearest to the alley as well as toward the fence for the second home to the east.

Figure I-1: Photograph looking east from Chandler Avenue into gravel alley between Munson Street and Day Street.

Analysis with the hydrology model and FLO-2D model suggests that flow from the north funnels along Chandler Avenue near this location. The east side of Chandler Avenue has a 100-year discharge of 12 cfs, which is largely contributed to by the residential block to the north. While the east side of Chandler

Avenue has a 100-year discharge of 12 cfs, most of the contributing drainage area at the intersection of Chandler and Munson comes from the north and west of Chandler Avenue, with the total peak discharge at the intersection estimated at 64 cfs.

While it is difficult to estimate the capacity of the receiving storm drain system without detailed as-built information, the existing topographic survey suggests that the drainage channel that runs under Texas Street could be about 1 to 2 feet wide on the bottom and around 3 feet deep. The total contributing drainage area for this system is 0.092 square miles, which corresponds to a peak 100-year discharge of 173 cfs. Aerial imagery appears to show the channel is highly vegetated, reducing the hydraulic capacity of the system. Based on a 2-foot bottom width channel, the channel would not have the capacity to convey the 100-year discharge, which could pose a hazard to the properties adjacent to the existing channel.

I.3 Assumptions

The following assumptions were made to develop the conceptual drainage solution:

- Assumed the sizes of existing stormwater infrastructure which must be confirmed prior to design.
- Assumed the design 100-year flow is determined on a unit discharge basis, where the peak discharge is estimated by multiplying the contributing drainage area at various locations of interest by the ratio of 100-year discharge to total drainage area for the encompassing sub-basin
- The City of Denison Storm Drainage System Design Manual will be used as the design concept development criteria. The City requires all drainage structures to be designed to carry the 100-year frequency design flow without overtopping.
- Assumed the existing stormwater channel running from Munson Street to Texas Street has adequate capacity.

I.4 Conceptual Design

In order to mitigate flooding at the problematic area near the intersection of Chandler Avenue and Munson Street, the proposed solution is a storm drain system running from the low point where ponding initially occurs in the alley and passing runoff southward through a vacant parcel east of the two properties where flooding has been reported. Grading efforts will be made to ensure that drainage is directed either towards the existing infrastructure on Chandler Avenue or towards the proposed storm drain inlet. Right of Way acquisition will be required from the vacant parcel where the alignment for the pipe will pass. The pipe will pass under Munson Street and tie-in with the existing stormwater infrastructure that discharges to an existing drainage ditch which runs north-to-south from Munson Street to Texas Street. There is not detailed survey data available for the drainage ditch, which at points along its alignment runs parallel to residences. As part of this assessment, it has been assumed that the drainage ditch has capacity for the design storm runoff, and prior to detailed design, this assumption should be validated to ensure that no flood hazard is presented to the nearby residences.

The design includes a 4' x 4' square drop inlet at the low-point in the alley between Chandler Avenue and Scullin Avenue. A 30" reinforced concrete pipe drains this inlet and connects to the existing drainage infrastructure.

A summary of the design is provided in Table I-1. The proposed layout overlaid on the FLO-2D model results and are provided in Exhibit I

Priority Medium

Table I-1: Design Summary at Chandler Avenue near Munson Street

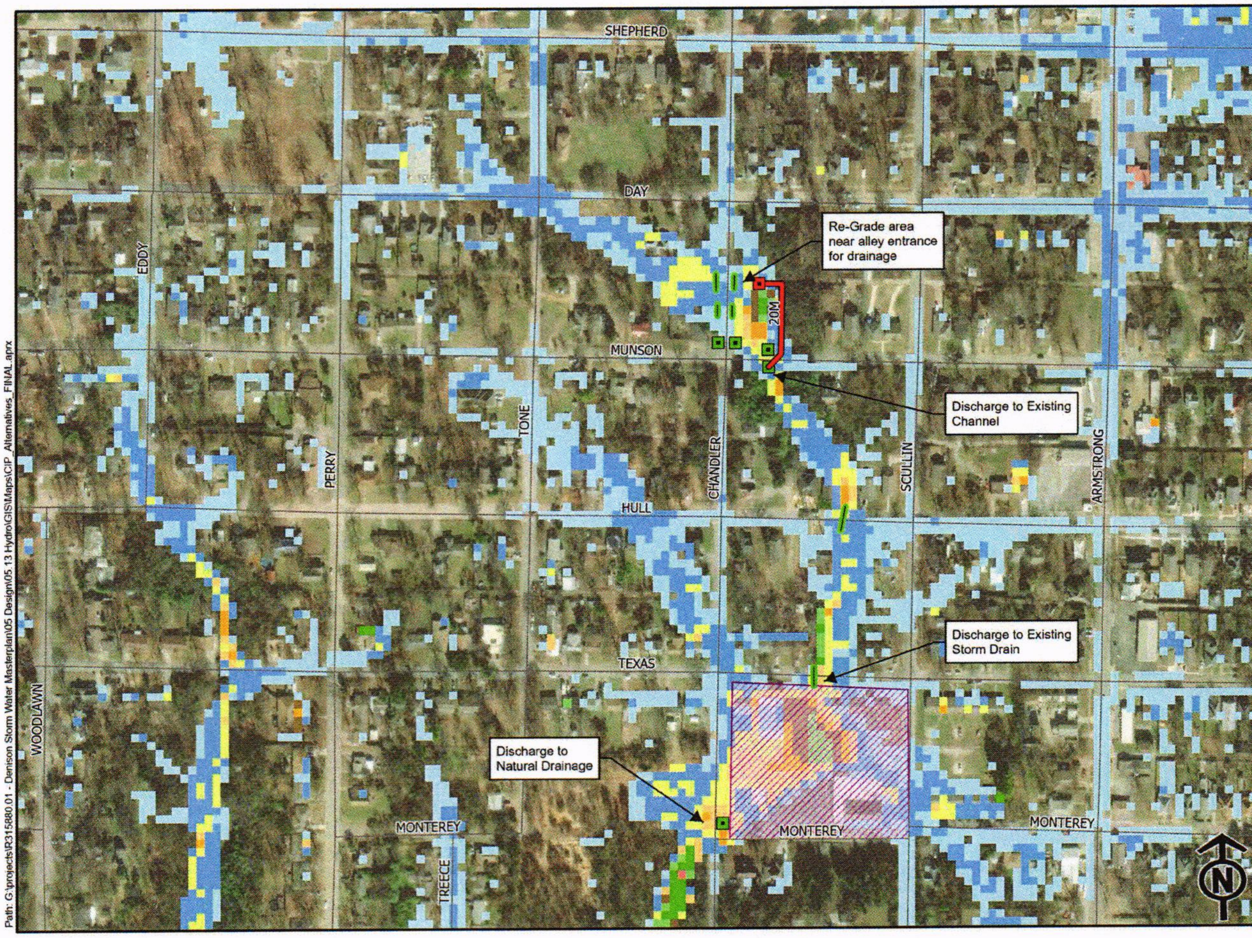
Contributing Area Alley Low-Point (square miles)	0.0066
Design Discharge for Inlet and Storm Drain Pipe (cfs)	12
Proposed Inlets in Alley Off Chandler Ave	4' x 4' Square Drop Inlet
Storm Drain Pipe	30" Reinforced Concrete Pipe

I.5 Cost Estimate

The following table (Table I-2) summarizes the estimated cost for the proposed improvements at Chandler Avenue near Munson Street. Detailed breakdown of cost is provided in Appendix B.

Table I-2: Estimation of Cost –Chandler Avenue near Munson Street

ITEM DESCRIPTION	UNIT	QUANTITY	UNIT COST	TOTAL COST
REMOVE EXISTING TREES	EA	20	$ 1,700	$ 34,000
30" CLASS III REINF. CONC. PIPE	LF	275	$ 300	$ 82,500
CONNECT TO EXISTING DRAINAGE STRUCTURE	EA	1	$ 5,000	$ 5,000
4' x 4' SQUARE DROP INLET	EA	1	$ 10,000	$ 10,000
CITY OF DENISON TYPE B MANHOLE	EA	2	$ 13,500	$ 27,000
REMOVE & REPLACE PAVEMENT, CURB, & GUTTER	LS	1	$ 8,000	$ 8,000
UNCLASSIFIED STREET EXCAVATION	CY	104	$ 50	$ 5,200
ROW ACQUISITION	SF	1,225	$ 35	$ 42,875
EARTHWORK (EROSN & SEDMT CONT, IN VEH)	CY	264	$ 50	$ 13,200
			SUB-TOTAL =	**$ 227,775**
			CONTINGENCY (APPROX. 40%) =	**$ 91,125**
			CONTINGENCY SUB-TOTAL =	**$ 318,900**
			ADMIN, ENGINEERING, CONSTRUCTION ADMIN (APPROX. 30%) =	**$ 96,100**
			TOTAL =	**$ 415,000**



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Appendix A
Exhibit I

Proposed Storm Water Layout
Project I

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Path: G:\projects\R315880.01 - Denison Storm Water Masterplan\05 Design\05.13 Hydro\GIS\Maps\CIP_Alternatives_FINAL.aprx

Project J – Dubois Between Scullin & Barrett

J Project J – Dubois between Scullin & Barrett

Priority - Medium

J.1 Location

Located on the 800 block of Dubois Street, between Scullin Avenue and Barrett Avenue, the road crosses a stream tributary to Iron Ore Creek. A culvert crossing consisting of five circular culverts at the main drainage path allows runoff to pass under the road at this location. Upstream and downstream of the culvert crossing, the stream banks are densely forested, which can produce heavy debris, including sediment, vegetative debris, and rubbish. There is utility pipe about three feet downstream of the culvert crossing that runs parallel to Dubois Street (Figure I-1 J-3).

J.2 Drainage Problem

The culvert crossing under Dubois Street is undersized and prone to debris blockage. During the site visit, logs, sediment, and other debris had partially blocked many of the culvert pipes (Figure J-1). The culverts are undersized such that sustained high-intensity flows tend to overtop the roadway. This culvert crossing lies within a FEMA FIS 100-year floodplain with "Zone A" classification. The 100-year discharge is estimated at 1,220 cfs. Over the road deck, the approximate flood elevation for the 100-year flood event is 656.8-feet. The existing stream crossing is estimated to have a capacity of about 416 cfs. Detailed survey for road deck elevations at this crossing is not available, but based on available LiDAR survey available from TNRIS, the road deck could be as low as 656-feet, meaning the 100-year base flood would inundate the roadway. Dubois is an important east-west connector street since several streets to the north and south do not connect, therefore emergency access is inhibited with larger storm flows.

Because the crossing is one of the road crossings allowing east-west access and that flood flows would disrupt traffic and that flood flows could pose a potential hazard for vehicular traffic, a medium priority is assigned.

Figure J-1: Photograph of Inlet for Culvert Crossing at Dubois Street

Figure J-2: Photograph of Stream Channel Upstream of Culvert Crossing at Dubois

Figure I-1 J-3: Photograph of Downstream of Culvert Crossing at Dubois with Utility Pipe

J.3 Assumptions

The following assumptions were made to develop the conceptual drainage solution:

- The project lies within the "681" sub-basin in the HEC-HMS model, to estimate the design flow, discharge is based on a unit discharge (discharge per square mile) basis using the 100-year peak flows estimated from the HEC-HMS model for sub-basin 681. Unit discharge for sub-basin 681, in cfs per square mile, is multiplied by the contributing drainage area of 0.61 square miles to determine peak flow of 1,220 cfs. Detailed estimation of peak flow is provided in Appendix F.
- 100-year base flood elevations are assumed to be at the elevation from the BLE study at this culvert crossing, with an elevation of 656.8-feet.
- Without detailed survey of the road deck at the culvert crossing for Dubois Street the roadway deck is assumed to be inundated during the 100-year flood event.

J.4 Conceptual Design

Conceptual engineering improvements at Dubois Street between Scullin Avenue and Barrett Avenue include culvert improvements to increase capacity such that the culverts are able to pass a 100-year flood in addition to maintenance plans to keep the improved culverts debris-free. Detailed hydraulic analysis and flood analysis is required to determine the flood depth of the 100-year flood in this location. A summary of the improvements is provided in Table J-1.

Priority Medium

Table J-1: Design Summary at Dubois Street

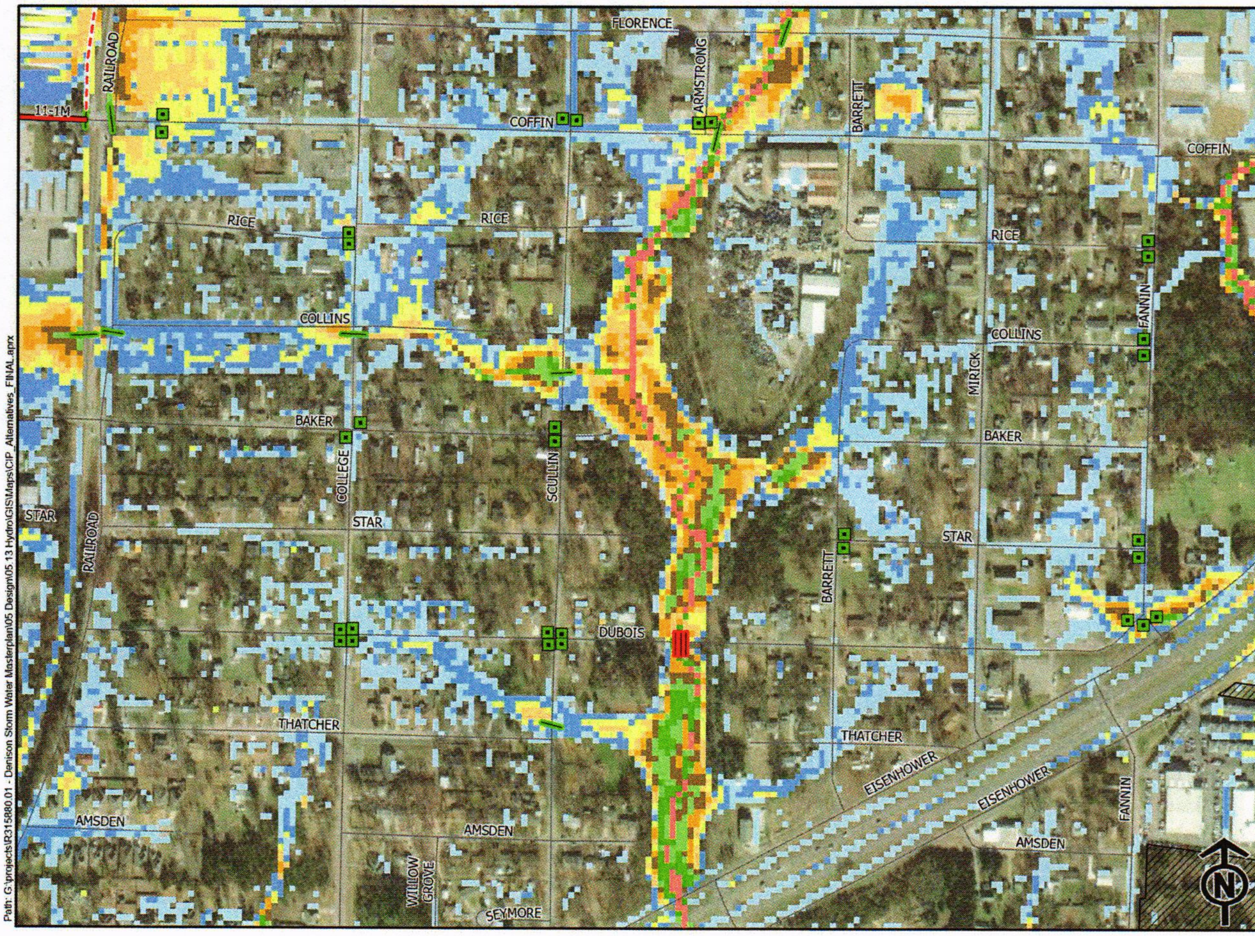
Total Contributing Area (mi^2)	0.61
Discharge, 100-year (cfs)	1220
Discharge, 10-year (cfs)	713
Proposed Culvert	5 x 10' W x 4' H Concrete Box

J.5 Cost Estimate

The following table (Table J-2) summarizes the estimated cost for the proposed improvements at Dubois Street. Detailed breakdown of cost is provided in Appendix B.

Table J-2: Estimation of Cost – Dubois Street

ITEM DESCRIPTION	UNIT	QUANTITY	UNIT COST	TOTAL COST
REMOVE ASPHALT PAVEMENT	SY	210	$ 50	$ 10,500
REMOVE & DISPOSE REINFORCED CONCRETE PIPE	LF	250	$ 45	$ 11,250
REMOVE HEADWALL	EA	2	$ 350	$ 700
UNCLASSIFIED STREET EXCAVATION	CY	490	$ 50	$ 24,500
CONC BOX CULV (10 FT X 4 FT)	LF	250	$ 1,200	$ 300,000
SLOPED CONCRETE HEADWALL	EA	2	$ 8,000	$ 16,000
HMAC ASPHALT PAVEMENT (2" TYPE D)	SY	210	$ 40	$ 8,400
			SUB-TOTAL =	$ 371,350
			CONTINGENCY (APPROX. 40%) =	$ 148,550
			CONTINGENCY SUB-TOTAL =	$ 519,900
			ADMIN, ENGINEERING, CONSTRUCTION ADMIN (APPROX. 30%) =	$ 156,100
			TOTAL =	$ 676,000



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- Proposed Catchbasins
- Proposed Conduits
- Proposed Culvert Improvements
- Proposed Channel Improvement
- Existing Spillways
- Existing Catchbasins
- Existing Culverts
- Streets
- Proposed Detention
- Potentially Inaccurate Result
- Further Detail Required

Proposed Development
- Commercial
- Mixed
- Residential

Max Depth
ft
- 0.001 - 0.25
- 0.251 - 0.5
- 0.501 - 1
- 1.001 - 1.5
- 1.501 - 2
- 2.001 - 2.5
- 2.501 - 3
- 3.001 - 3.5
- 3.501 - 5
- 5.001 - 10
- > 10.001

Appendix A
Exhibit J

Proposed Storm Water Layout
Project J

0 125 250 500
Feet

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Project K – Flora Lane near Lum Lane

K Project K – Flora Lane near Lum Lane

Priority - Low

K.1 Location

West of the traffic circle at the intersection of Lora Lane and Lum Lane, a stream passes through a culvert crossing under Flora Lane. The crossing consists of two 7' x 3' concrete box culverts with riprap outlet protection downstream and two curb inlet catch basins for street runoff. The catchment upstream of the culvert crossing is highly forested, and the downstream channel is moderately forested with a moderately meandering stream and defined banks. The receiving stream runs through residential properties.

K.2 Drainage Problem

Flooding problems have been reported from the single-family home directly downstream of the culvert crossing. The home is a fairly new structure, with aerial imagery suggesting it was built some time in 2021. The culvert crossing is also a recently-built structure, built some time prior to the construction of this single-family home in 2021, but after 2019. The single-family home lies within the original flow path of the drainageway, and is protected by modest elevation and bank protection. Although detailed survey at this location is not available, based on observations from a field visit at this location, the home may be at a lower elevation than the culvert outlet and the roadway at Flora Lane. With the current configuration of the culvert crossing and downstream meandering channel, flow exceeding the banks of the channel could cause flooding of the home. Because the home is a single structure that was built in this location after the construction of the existing storm water infrastructure, and because flooding hazards would occur for overbank flooding with typically low flood depths, this is assigned low priority.

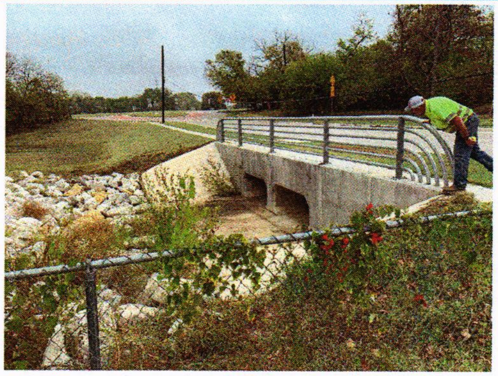

Figure K-1: Photograph of Outlet for Culvert Crossing on Flora Lane near Lum Lane

Figure K-2: Downstream from Culvert Crossing at Flora Lane near Lum Lane

K.3 Assumptions

The following assumptions were made to develop the conceptual drainage solution:

- The stream crossing lies within sub-basin 626, and the design 100-year flow is determined on a unit discharge basis, where the peak discharge is estimated by multiplying the contributing drainage area at this location by the ratio of 100-year discharge to total drainage area for sub-basin 626.
- The storm drain system along Flora and culverts are recently updated and designed for the 100-year flow, so the designs were assumed to be adequately designed.
- The receiving stream channel downstream of the impacted single-family home is assumed to have adequate capacity for the 100-year flood without impact to other existing structures downstream.

K.4 Conceptual Design

The conceptual design is to improve the drainage channel downstream of Flora Lane with an engineered riprap-armored channel. The downstream channel would be improved to attempt to provide capacity for the 100-year flood event. A summary of the design is provided in Table K-1. The proposed layout overlaid on the FLO-2D model results are provided in Exhibit K.

Priority Low

Table K-1: Design Summary at Flora Lane near Lum Lane

Total Contributing Area (mi²)	0.14
Discharge, 100-year (cfs)	266
Discharge, 10-year (cfs)	156
Existing Culvert Crossing	2 x 7' W x 3' H Concrete Box
Downstream Channel	15' Wide x 3' Deep Trapezoid
Median Riprap Size (inch)	12

K.5 Cost Estimate

The following table (Table K-2) summarizes the estimated cost for the proposed improvements at Flora Lane near Lum Lane. Detailed breakdown of cost is provided in Appendix B.

Table K-2: Estimation of Cost – Flora Lane near Lum Lane

ITEM DESCRIPTION	UNIT	QUANTITY	UNIT COST	TOTAL COST
EARTHWORK (EROSN & SEDMT CONT. IN VEH)	CY	472	$ 50	$ 23,611
GROUTED RIPRAP TYPE A	SY	483	$ 300	$ 144,900
			SUB-TOTAL =	**$ 168,511**
			CONTINGENCY (APPROX. 40%) =	**$ 67,489**
			CONTINGENCY SUB-TOTAL =	**$ 236,000**
			ADMIN, ENGINEERING, CONSTRUCTION ADMIN (APPROX. 30%) =	**$ 71,000**
			TOTAL =	**$ 307,000**

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- Proposed Catchbasins
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- 2.501 - 3
- 3.001 - 3.5
- 3.501 - 5
- 5.001 - 10
- > 10.001

Appendix A
Exhibit K

Proposed Storm Water Layout
Project K

0 125 250 500
Feet

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Project L – Coffin Street & Park Avenue

L Project L – Coffin Street & Park Avenue

Priority - Low

L.1 Location

Flooding can occur at the intersection of Coffin Street and Park Avenue and includes ponding and overtopping of Park Avenue to the east. This is a three-way intersection where Coffin Street approaches from the west and terminates at Park Avenue. An existing culvert crossing of Park Avenue passes drainage under the intersection from west to east. Park Avenue contains a fairly built-out curb, gutter, and storm drain system while Coffin Street does not have any curbs or gutters. A roadside drainage ditch runs along the south side of Coffin Street until it crosses through a culvert just west of its intersection with Park Avenue. The drainage ditch passes under existing residence driveways via various culverts and is fairly heavily vegetated.

L.2 Drainage Problem

At the intersection of Coffin Street and Park Avenue, intermittent ponding occurs when water exceeds the capacity of the drainage ditch along Coffin Street. Coffin lacks a curb and gutter, relying on a roadside drainage ditch to convey runoff. During high intensity storm events, such as the 10-year and 100-year storm events, this existing roadside ditch and culverts that convey flow under driveways and ultimately under Coffin are undersized, and runoff would spill onto the roadway and flood toward the intersection of Coffin and Park.

Flooding impacts are largely limited to the intersection, which has spillways to route flow downstream when ponding is high enough. Because of this limited flooding hazard, low priority is assigned.

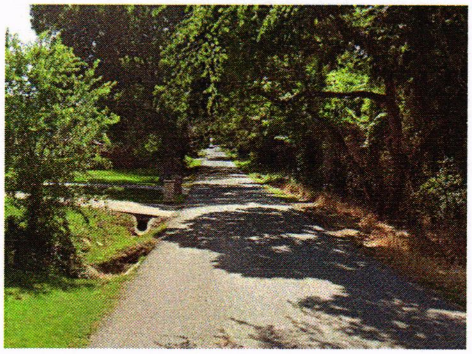


Figure L-1: Coffin Street Showing Ditch and Cross-Street Culvert

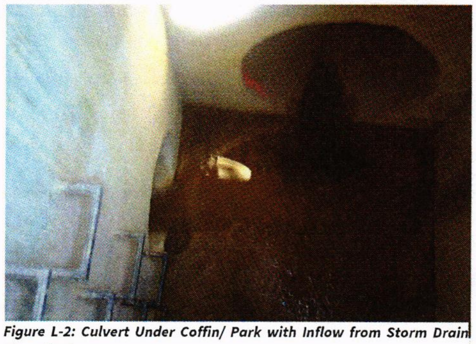


Figure L-2: Culvert Under Coffin/ Park with Inflow from Storm Drain along Park

L.3 Assumptions

The following assumptions were made to develop the conceptual drainage solution:

- The project lies within the "871" sub-basin in the HEC-HMS model, to estimate the design flow, discharge is based on a unit discharge (discharge per square mile) basis using the 100-year peak flows estimated from the HEC-HMS model for sub-basin 871. Unit discharge for sub-basin 871, in cfs per square mile, is multiplied by the contributing drainage area at each structure location to determine peak flow. Detailed estimation of peak flow is provided in Appendix F.
- Without proper as-built data for the existing storm water infrastructure, the existing conditions at the culvert crossing under Coffin Street are assumed. Design plans for the street improvements at Park Avenue show the culvert under Park Avenue is a Class III 48" reinforced concrete pipe (Ref. 2); inverts were assumed based on topography. Dimensions are based on photographs provided by the City of Denison, and pipe invert and roadway elevations using existing LiDAR data.
- The existing trapezoidal geometry of the roadside drainage ditch along Coffin Road is based on available LiDAR survey and photographs.
- The existing storm drain system along Park Avenue is assumed to have capacity for high-intensity storm events. Additional analysis to confirm this assumption is required for detailed design.
- To prevent vegetation growth that could negatively impact the hydraulic capacity of the roadside ditch, concrete lining is proposed.

L.4 Conceptual Design

The conceptual design improves the drainage along Coffin Street, upstream of the intersection with Park Avenue. Driveways along Coffin near the intersection of Avon and Coffin, and under Coffin at the terminus of the roadside drainage should be improved. Additionally, improvements are proposed to the roadside drainage ditch along Coffin to allow it to carry additional stormwater runoff. A summary of the design is provided in Table L-1. The proposed layout overlaid on the FLO-2D model results are provided in Exhibit L.

Priority Low

Table L-1: Design Summary at Coffin Street and Park Avenue

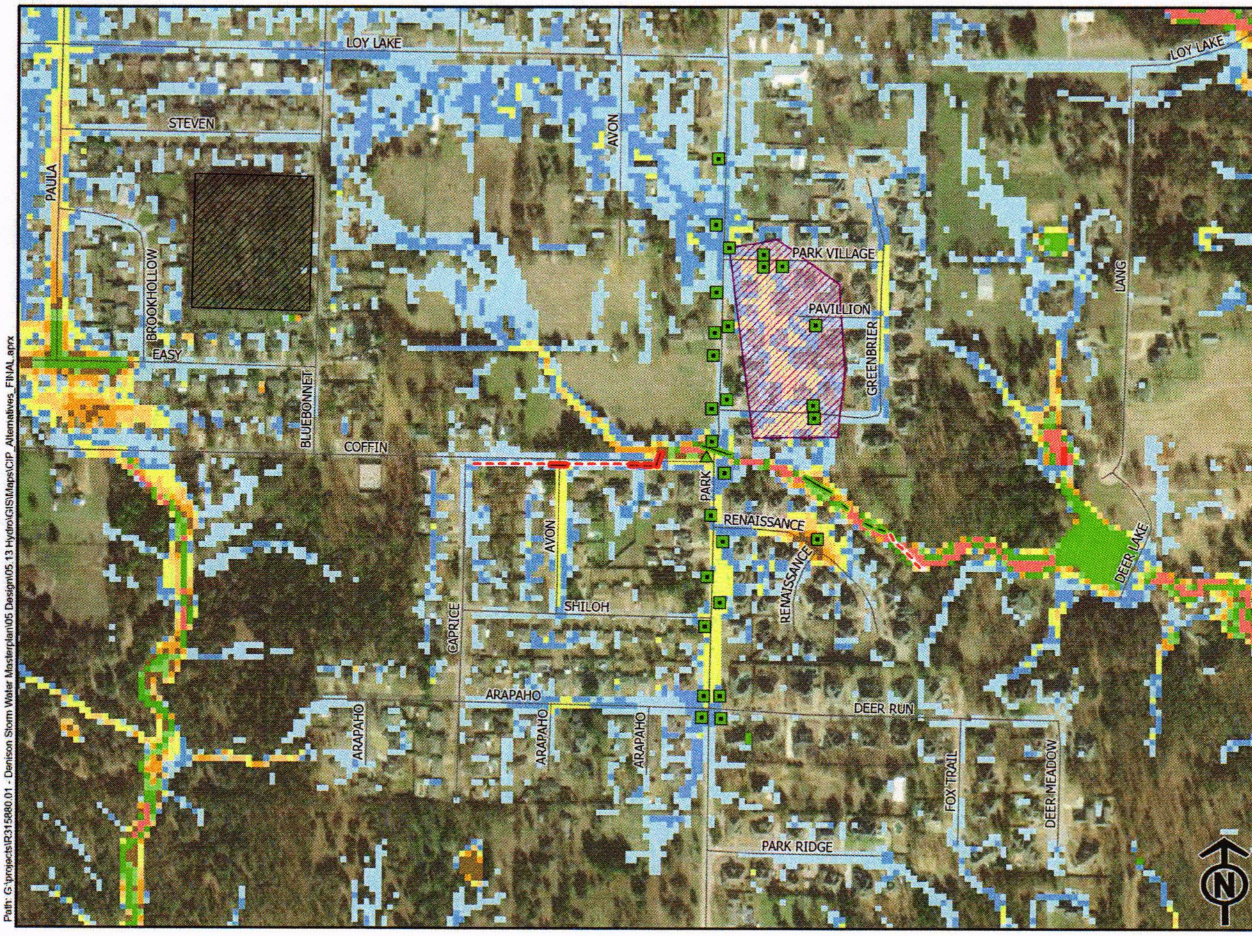
Roadside Ditch / Culvert - Coffin	
Total Contributing Area (mi^2)	0.02
Discharge, 100-year (cfs)	40
Discharge, 10-year (cfs)	23
Culvert Requirement	48" Dia. Concrete Circular (Existing)
Channel Requirement	5' Wide x 3' Deep Concrete Trapezoid

L.5 Cost Estimate

The following table (Table L-2) summarizes the estimated cost for the proposed improvements at Coffin and Park. Detailed breakdown of cost is provided in Appendix B.

Table L-2: Estimation of Cost – Coffin Street & Park Avenue

ITEM DESCRIPTION	UNIT	QUANTITY	UNIT COST	TOTAL COST
REMOVE & REPLACE PAVEMENT, CURB, & GUTTER	LS	1	$ 8,000	$ 8,000
REMOVE & DISPOSE REINFORCED CONCRETE PIPE	LF	150	$ 45	$ 6,750
REMOVE HEADWALL	EA	6	$ 350	$ 2,100
UNCLASSIFIED STREET EXCAVATION	CY	2	$ 50	$ 100
30" CLASS III REINF. CONC. PIPE	LF	290	$ 300	$ 87,000
SLOPED CONCRETE HEADWALL	EA	8	$ 8,000	$ 64,000
CONCRETE CHANNEL	SY	1,240	$ 185	$ 229,400
EARTHWORK (EROSN & SEDMT CONT, IN VEH)	CY	517	$ 50	$ 25,850
			SUB-TOTAL =	$ 423,200
			CONTINGENCY (APPROX. 40%) =	$ 169,300
			CONTINGENCY SUB-TOTAL =	$ 592,500
			ADMIN, ENGINEERING, CONSTRUCTION ADMIN (APPROX. 30%) =	$ 178,500
			TOTAL =	$ 771,000



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Proposed Conduits
Proposed Culvert Improvements
Proposed Channel Improvement
Existing Spillways
Existing Catchbasins
Existing Culverts
Streets
Proposed Detention
Potentially Inaccurate Result
Further Detail Required

Proposed Development
Commercial
Mixed
Residential

Max Depth
ft
0.001 - 0.25
0.251 - 0.5
0.501 - 1
1.001 - 1.5
1.501 - 2
2.001 - 2.5
2.501 - 3
3.001 - 3.5
3.501 - 5
5.001 - 10
> 10.001

Appendix A
Exhibit L

Proposed Storm Water Layout
Project L

0 125 250 500
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Project M – Eddy Avenue between Elm Street & Martin Luther King Street

M Project M – Eddy Avenue between Elm Street & Martin Luther King Street

Priority - Low

M.1 Location

Eddy Avenue on the block from Elm Street to Martin Luther King Street experiences roadway flooding after intense rainfall events. The existing roadway is fairly steep, with an average slope of about 5% and with slopes up to 8%. Existing storm water infrastructure includes an existing culvert under Martin Luther King Street to the west of Eddy Avenue and a storm drain system on Elm Street and on Martin Luther King Street. The existing culvert under Martin Luther King flows south to north and is an unnamed tributary to Duck Creek. Locally, storm runoff concentrates along Eddy. Existing storm drain inlets are placed at the intersections at both Elm and Martin Luther King streets.

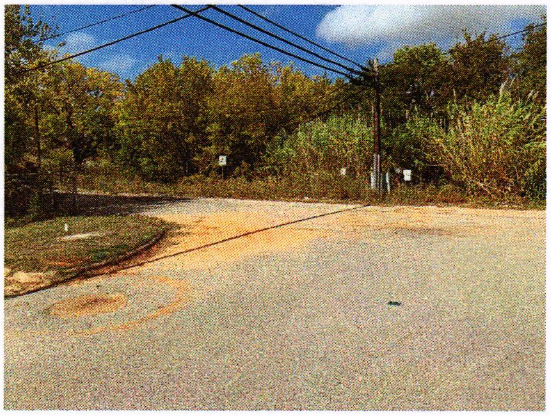


Figure M-1: Photograph at Intersection of Eddy and Martin Luther King

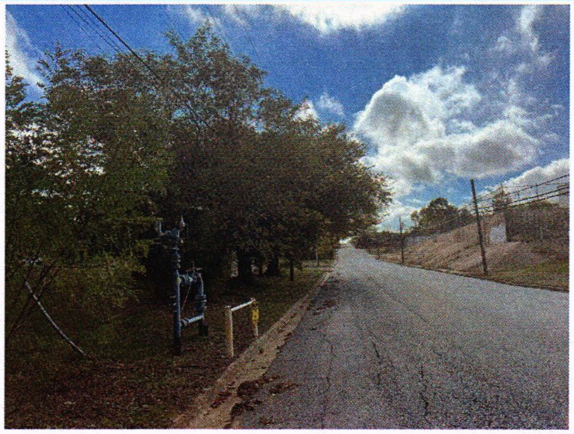


Figure M-2: Photograph Facing South along Eddy from Intersection with Martin Luther King

M.2 Drainage Problem

Drainage concentrates locally on Eddy Avenue, capturing flows originating from Elm Street and Johnson Street. Visual evidence at the intersections of both Eddy and Elm and Eddy and Martin Luther King indicates ponding and deposited sediment. The existing storm drainage infrastructure along Eddy is inadequate, with storm drain inlets on Elm and Martin Luther King streets but no inlets along Eddy Avenue where drainage concentrates. An existing spillway on the west side of Eddy just south of the intersection with Martin Luther King Street is the lone storm water drainage structure along Eddy. Visual evidence of ponding and sedimentation is mostly on the east side of Eddy Avenue where there are no storm drainage structures. The flood hazards presented at this location largely include partial road ponding and sediment deposition. Accordingly, this low priority is assigned.

M.3 Assumptions

The following assumptions were made to develop the conceptual drainage solution:

- No detailed inlet sizing has been made. Additional standard curb inlet at 10-foot length are planned at the intersections of Elm-Eddy and Martin Luther King-Eddy where intermittent ponding has been noted. The inlets are assumed to mirror those that are existing, and tie into existing infrastructure
- Without detailed as-built storm drain information for the existing storm drain system along Elm and Martin Luther King Street, estimates for the hydraulic capacity of the existing storm drainage system are not possible. In lieu of additional information, the storm drain system is assumed adequate to add new inlet structures on Eddy Avenue.

M.4 Conceptual Design

The conceptual design includes a storm drain with two additional standard storm drain inlets along Eddy Avenue. The results from the FLO-2D model in the area of this project may not reflect the 100-year flooding in the area because many of the storm drainage structures in this area are not accounted for in the model. As a result, the two additional inlets along Eddy are provided to capture street runoff along Eddy from Johnson Street until Martin Luther King Street. The inlets are sized for the 100-year flow, and are summarized in Table M-1. The proposed layout overlaid on the FLO-2D model results are provided in Exhibit M.

Priority Low

Table M-1: Design Summary at Eddy Avenue

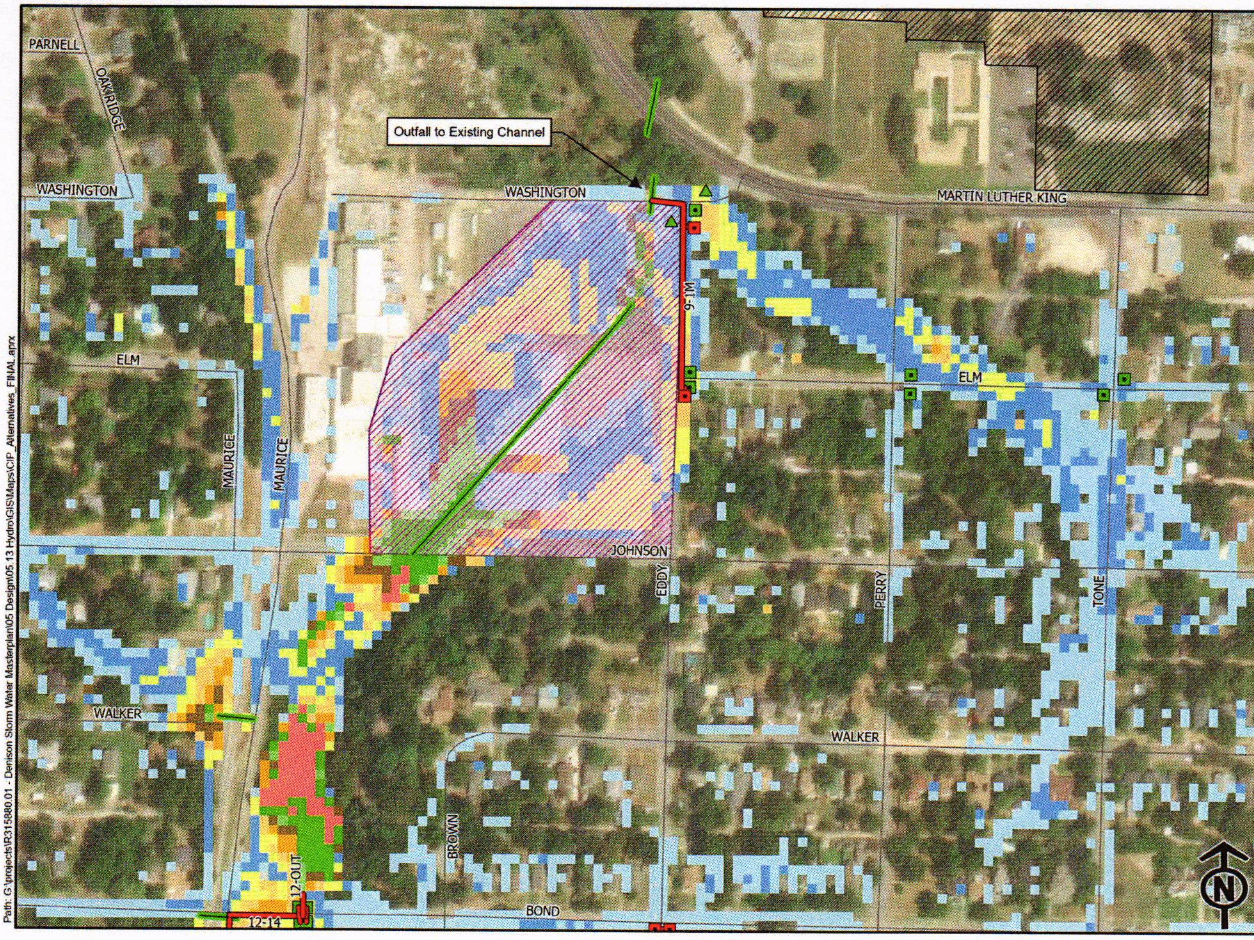
Total Contributing Area (mi^2)	0.0658
Inlet Design Discharge, 100-year (cfs)	12
Inlet Discharge, 10-year (cfs)	7
Inlet Design	Two 12' Standard Curb Inlets on Eddy Ave.
Main Conduit Size	30" Reinforced Concrete Pipe

M.5 Cost Estimate

The following table (Table M-2) summarizes the estimated cost for the proposed improvements at Eddy Avenue.

Table M-2: Estimation of Cost – Eddy Avenue

ITEM DESCRIPTION	UNIT	QUANTITY	UNIT COST	TOTAL COST
REMOVE & REPLACE PAVEMENT, CURB, & GUTTER	LS	1	$ 8,000	$ 8,000
12' STD. CURB INLET	EA	2	$ 13,000	$ 26,000
CITY OF DENISON TYPE B MANHOLE	EA	3	$ 13,500	$ 40,500
30" CLASS III REINF. CONC. PIPE	LF	500	$ 300	$ 150,000
18" CLASS III REINF. CONC. PIPE	LF	100	$ 190	$ 19,000
CONNECT TO EXISTING DRAINAGE STRUCTURE	EA	2	$ 5,000	$ 10,000
			SUB-TOTAL =	**$ 253,500**
			CONTINGENCY (APPROX. 40%) =	**$ 101,400**
			CONTINGENCY SUB-TOTAL =	**$ 354,900**
			ADMIN, ENGINEERING, CONSTRUCTION ADMIN (APPROX. 30%) =	**$ 107,100**
			TOTAL =	**$ 462,000**



CITY OF DENISON
300 W Main St
Denison, TX 75020

Denison TEXAS

City of Denison
Conceptual SWMP
R315880.01

Appendix A
Exhibit M

Proposed Storm Water Layout
Project M

HUITT-ZOLLARS
5430 LBJ Freeway, Suite 1500
Dallas, Texas 75240

N REFERENCES

1. Denison, *Texas Storm Drainage System Design Manual*. April 2017.

2. TCB, Inc. Construction Plans for South Park Avenue & Crawford Street Road Roadway Improvements. Prepared for City of Denison, Texas. 2006.

3. Compas PTS JV, Bois D'Arc Watershed, TX Base Level Engineering (BLE) Results. Prepared for DHS/FEMA Region VI. August 2021.

4. City of Denison. *Comprehensive Plan*. Prepared by Huitt-Zollars and Marshdarcy Partners. 2018.

5. Federal Emergency Management Agency, Flood Insurance Study Volume 1 of 3: Grayson County, Texas and Incorporated Areas. Revised September 1, 2022. Flood Insurance Study Number 48181CV001B.

6. Teague Nall and Perkins, Inc. Construction Plans for Theresa Drive Water Line Phase 2. Prepared for the City of Denison. June 2020 – Stamped with "As-built".

Appendix B: Estimation of Probable Cost

Engineer's Opinion of Cost: Unit Cost Reference List

REMOVAL		
ITEM DESCRIPTION	UNIT	UNIT COST
REMOVE ASPHALT PAVEMENT	SY	$50.00
ROTO MILL 2" ASPHALT PAVEMENT	SY	$14.00
REMOVE CONCRETE DRIVEWAY	SY	$25.00
REMOVE GRAVEL DRIVEWAY	SY	$25.00
REMOVE CONCRETE PAVEMENT	SY	$25.00
REMOVE CONCRETE FLUME	SY	$35.00
REMOVE CURB AND GUTTER	LF	$15.00
REMOVE & DISPOSE CMP PIPE	LF	$40.00
REMOVE & DISPOSE REINFORCED CONCRETE PIPE	LF	$45.00
REMOVE INLET	EA	$1,400.00
REMOVE & RESET MAIL BOX	EA	$400.00
REMOVE & RESET EX. GATE	EA	$1,900.00
REMOVE EXISTING TREES	EA	$1,700.00
REMOVE & REPLACE PAVEMENT, CURB, & GUTTER	LS	$8,000.00
REMOVE & REPLACE CHAIN LINK FENCE	LF	$120.00
REMOVE & REPLACE WIRE FENCE	LF	$85.00
REMOVE & DELIVER EXISTING WATER METER TO CITY	EA	$240.00
REMOVE & DELIVER EXISTING FIRE HYDRANT TO CITY	EA	$725.00
REMOVE EX. WASTEWATER MANHOLE	EA	$1,300.00
REMOVE EX. WASTEWATER CLEANOUT	EA	$600.00
REMOVE EX. CONCRETE CELLAR	EA	$6,200.00
REMOVE HEADWALL	EA	$350.00
REMOVE GRADE CONTROL STRUCTURE	EA	$5,000.00
ROW ACQUISITION	SF	$35.00

Engineer's Opinion of Cost: Unit Cost Reference List

DRAINAGE		
ITEM DESCRIPTION	UNIT	UNIT COST
18" CLASS III REINF. CONC. PIPE	LF	$190.00
21" CLASS III REINF. CONC. PIPE	LF	$210.00
21" CLASS III REINF. CONC. PIPE (CEMENT STABILIZED BACKFILL)	LF	$350.00
24" CLASS III REINF. CONC. PIPE	LF	$200.00
24" CLASS III REINF. CONC. PIPE (CEMENT STABILIZED BACKFILL)	LF	$300.00
30" CLASS III REINF. CONC. PIPE	LF	$300.00
36" CLASS III REINF. CONC. PIPE	LF	$350.00
42" CLASS III REINF. CONC. PIPE	LF	$420.00
48" CLASS III REINF. CONC. PIPE	LF	$500.00
54" CLASS III REINF. CONC. PIPE	LF	$600.00
66" CLASS III REINF. CONC. PIPE	LF	$850.00
72" CLASS III REINF. CONC. PIPE	LF	$900.00
COMBINATION FOUR GRATE INLET	EA	$15,000.00
TXDOT INLET	EA	$13,000.00
4' STD. CURB INLET	EA	$13,000.00
10' STD. CURB INLET	EA	$11,000.00
10' RECESSED CURB INLET	EA	$9,500.00
12' STD. CURB INLET	EA	$13,000.00
14' STD. CURB INLET	EA	$13,000.00
14' RECESSED CURB INLET	EA	$14,000.00
4' x 4' SQUARE DROP INLET	EA	$10,000.00
5' x 5' SQUARE DROP INLET	EA	$10,000.00
CITY OF DENISON TYPE B MANHOLE	EA	$13,500.00
CONNECT TO EXISTING PIPE	EA	$5,000.00
CONNECT TO EXISTING DRAINAGE STRUCTURE	EA	$5,000.00
GROUTED RIPRAP TYPE A	SY	$300.00
FILTER FABRIC	SY	$20.00
GABIONS (3' X 3') (GALV)	CY	$400.00
SLOPED CONCRETE HEADWALL	EA	$8,000.00
TXDOT CH-FW-45 FOR 2-72" RCP	EA	$20,500.00
SWPPP	LS	$99,500.00
TRENCH SAFETY	LF	$20.00
TELEVISION INSPECTION	LF	$11.00
30" STORM PLUG	EA	$7,500.00
48" STORM PLUG	EA	$3,000.00
EROSION CONTROL MEASURES	LS	$136,000.00
CONCRETE CHANNEL	SY	$185.00
CONCRETE FLUME	SF	$100.00
TXDOT METAL BEAM GUARD FENCE	LF	$100.00
CONC BOX CULV (8 FT X 6 FT)	LF	$1,200.00
CONC BOX CULV (10 FT X 4 FT)	LF	$1,200.00
CONC BOX CULV (10 FT X 5 FT)	LF	$1,200.00
CONC BOX CULV (10 FT X 6 FT)	LF	$1,200.00
CONC BOX CULV (10 FT X 7 FT)	LF	$1,300.00
CONC BOX CULV (4 FT X 4 FT)	LF	$500.00
CONC BOX CULV (5 FT X 5 FT)	LF	$500.00
CONC BOX CULV (6 FT X 5 FT)	LF	$700.00
STREAM CONSTRUCTION ACCESS	LS	$25,000.00
UNCLASSIFIED STREAM EXCAVATION	CY	$80.00
GRADE CONTROL STRUCTURE	CY	$800.00
RIPRAP CONCRETE BAG RETAINING WALL	SF	$100.00
ACCESS BRIDGE	EA	$4,000.00
BACKFILL	CY	$30.00

Engineer's Opinion of Cost:
Unit Cost Reference List

PAVING		
ITEM DESCRIPTION	**UNIT**	**UNIT COST**
MOBILIZATION & SITE PREPARATION	LS	$100,000.00
TRAFFIC CONTROL	LS	$440,000.00
UNCLASSIFIED STREET EXCAVATION	CY	$50.00
7" CONCRETE PAVEMENT	SY	$120.00
HMAC ASPHALT PAVEMENT (2" TYPE D)	SY	$40.00
HMAC ASPHALT PAVEMENT (2" TYPE D, 4" TYPE B)	SY	$90.00
HMAC ASPHALT PAVEMENT (3" TYPE C, 5" TYPE B)	SY	$100.00
6" CEMENT STABILIZED SUBGRADE	SY	$6.00
CEMENT FOR SUBGRADE STABILIZATION (23LB/SY)	TON	$500.00
4" REINFORCED CONCRETE SIDEWALK	SF	$10.00
6" REINFORCED CONCRETE SIDEWALK	SF	$10.00
3" MOUNTABLE CURB	LF	$12.00
3" VERTICAL CURB	LF	$35.00
6" CONCRETE CURB	LF	$5.00
6" SEPARATE CONCRETE CURB & GUTTER	LF	$55.00
CONCRETE DRIVEWAY PAVEMENT	SY	$110.00
GRAVEL DRIVEWAY	SY	$90.00
INTERLOCKING PAVING STONES	SY	$160.00
GRASS BLOCK SODDING WITH 4" TOP SOIL	SY	$14.00
BARRIER FREE RAMPS	EA	$3,500.00
BARRIER FREE RAMPS AT ROUNDABOUT	EA	$3,000.00
TEMPORARY ASPHALT PAVING FOR MAINTENANCE OF TRAFFIC	SY	$35.00
2" SCHEDULE 40 STREETLIGHT CONDUIT	LF	$20.00
HIGH EARLY STRENGTH CONCRETE (CONTINGENCY ITEM)	SY	$130.00
4" DOUBLE YELLOW THERMOPLASTIC STRIPE	LF	$4.00
4" SOLID WHITE THERMOPLASTIC STRIPE	LF	$2.00
4" SOLID YELLOW THERMOPLASTIC STRIPE	LF	$2.00
24" WHITE CROSSWALK MARKER	LF	$16.00
ADVISORY MARKER	EA	$600.00
TYPE II A-A BUTTON	EA	$6.00
18" WHITE YIELD TRIANGLE	LF	$40.00
24" WHITE STOP BAR	LF	$15.00
REMOVE SMALL ROADWAY SIGN	EA	$250.00
REMOVE & RELOCATE SMALL ROADWAY SIGN	EA	$1,000.00
SMALL ROADWAY SIGN	EA	$850.00
TRAFFIC ARROW	EA	$300.00
TRAFFIC WORD	EA	$400.00
EARTHWORK (EROSN & SEDMT CONT, IN VEH)	CY	$50.00
REMOVING CONC (MEDIANS)	SY	$15.00
CONC MEDIAN	SY	$75.00

City of Denison Conceptual SWMP
Appendix B
Estimation of Probable Cost

Engineer's Opinion of Cost
Project Reference List

Project A
Flora Lane and Rivercrest Circle

ITEM DESCRIPTION	UNIT	QUANTITY	UNIT COST	TOTAL COST
REMOVE ASPHALT PAVEMENT	SY	112	$ 50	$ 5,600
REMOVE CURB AND GUTTER	LF	40	$ 15	$ 600
REMOVE & DISPOSE REINFORCED CONCRETE PIPE	LF	50	$ 45	$ 2,250
REMOVE HEADWALL	EA	2	$ 350	$ 700
UNCLASSIFIED STREET EXCAVATION	CY	260	$ 50	$ 13,000
4' STD. CURB INLET	EA	1	$ 13,000	$ 13,000
CONC BOX CULV (4 FT X 4 FT)	LF	200	$ 500	$ 100,000
CITY OF DENISON TYPE B MANHOLE	EA	1	$ 13,500	$ 13,500
24" CLASS III REINF. CONC. PIPE	LF	500	$ 200	$ 100,000
SLOPED CONCRETE HEADWALL	EA	2	$ 8,000	$ 16,000
CONNECT TO EXISTING DRAINAGE STRUCTURE	EA	1	$ 5,000	$ 5,000
HMAC ASPHALT PAVEMENT (2" TYPE D)	SY	112	$ 40	$ 4,480
6" SEPARATE CONCRETE CURB & GUTTER	LF	40	$ 55	$ 2,200
GROUTED RIPRAP TYPE A	SY	1,871	$ 300	$ 561,300
EARTHWORK (EROSN & SEDMT CONT, IN VEH)	CY	12,800	$ 50	$ 640,000
			SUB-TOTAL =	**$ 1,477,630**
			CONTINGENCY (APPROX. 40%) =	**$ 591,070**
			CONTINGENCY SUB-TOTAL =	**$ 2,068,700**
			ADMIN, ENGINEERING, CONSTRUCTION ADMIN (APPROX. 30%) =	**$ 621,300**
			TOTAL =	**$ 2,690,000**

Project B
Morton and Maurice

ITEM DESCRIPTION	UNIT	QUANTITY	UNIT COST	TOTAL COST
REMOVE ASPHALT PAVEMENT	SY	195	$ 50	$ 9,750
REMOVE CURB AND GUTTER	LF	350	$ 15	$ 5,250
UNCLASSIFIED STREET EXCAVATION	CY	292	$ 50	$ 14,600
14' STD. CURB INLET	EA	25	$ 13,000	$ 325,000
18" CLASS III REINF. CONC. PIPE	LF	474.00	$ 190	$ 90,060
24" CLASS III REINF. CONC. PIPE	LF	1,467	$ 200	$ 293,400
30" CLASS III REINF. CONC. PIPE	LF	97	$ 300	$ 29,100
36" CLASS III REINF. CONC. PIPE	LF	791	$ 350	$ 276,850
42" CLASS III REINF. CONC. PIPE	LF	491	$ 420	$ 206,220
54" CLASS III REINF. CONC. PIPE	LF	528	$ 600	$ 316,800
66" CLASS III REINF. CONC. PIPE	LF	55	$ 850	$ 46,750
CITY OF DENISON TYPE B MANHOLE	EA	21	$ 13,500	$ 283,500
CONNECT TO EXISTING DRAINAGE STRUCTURE	EA	5	$ 5,000	$ 25,000
HMAC ASPHALT PAVEMENT (2" TYPE D)	SY	195	$ 40	$ 7,800
6" SEPARATE CONCRETE CURB & GUTTER	LF	350	$ 55	$ 19,250
			SUB-TOTAL =	**$ 1,949,330**
			CONTINGENCY (APPROX. 40%) =	**$ 779,770**
			CONTINGENCY SUB-TOTAL =	**$ 2,729,100**
			ADMIN, ENGINEERING, CONSTRUCTION ADMIN (APPROX. 30%) =	**$ 818,900**
			TOTAL =	**$ 3,548,000**

Project C
Sears Street

ITEM DESCRIPTION	UNIT	QUANTITY	UNIT COST	TOTAL COST
REMOVE ASPHALT PAVEMENT	SY	164	$ 50	$ 8,200
REMOVE CURB AND GUTTER	LF	294	$ 15	$ 4,410
UNCLASSIFIED STREET EXCAVATION	CY	245	$ 50	$ 12,250
14' STD. CURB INLET	EA	21	$ 13,000	$ 273,000
18" CLASS III REINF. CONC. PIPE	LF	1,017	$ 190	$ 193,230
24" CLASS III REINF. CONC. PIPE	LF	916	$ 200	$ 183,200
30" CLASS III REINF. CONC. PIPE	LF	473	$ 300	$ 141,900
36" CLASS III REINF. CONC. PIPE	LF	33	$ 350	$ 11,550
42" CLASS III REINF. CONC. PIPE	LF	460	$ 420	$ 193,200
48" CLASS III REINF. CONC. PIPE	LF	940	$ 500	$ 470,000
54" CLASS III REINF. CONC. PIPE	LF	465	$ 600	$ 279,000
66" CLASS III REINF. CONC. PIPE	LF	995	$ 850	$ 845,750
CITY OF DENISON TYPE B MANHOLE	EA	17	$ 13,500	$ 229,500
CONNECT TO EXISTING DRAINAGE STRUCTURE	EA	1	$ 5,000	$ 5,000
HMAC ASPHALT PAVEMENT (2" TYPE D)	SY	164	$ 40	$ 6,560
6" SEPARATE CONCRETE CURB & GUTTER	LF	294	$ 55	$ 16,170
			SUB-TOTAL =	**$ 2,872,920**
			CONTINGENCY (APPROX. 40%) =	**$ 1,149,180**
			CONTINGENCY SUB-TOTAL =	**$ 4,022,100**
			ADMIN, ENGINEERING, CONSTRUCTION ADMIN (APPROX. 30%) =	**$ 1,206,900**
			TOTAL =	**$ 5,229,000**

Project D
Loy Lake Road and Waterloo Creek Crossing

ITEM DESCRIPTION	UNIT	QUANTITY	UNIT COST	TOTAL COST
REMOVE ASPHALT PAVEMENT	SY	412	$ 50	$ 20,600
REMOVE CURB AND GUTTER	LF	148	$ 15	$ 2,220
REMOVE & DISPOSE REINFORCED CONCRETE PIPE	LF	200	$ 45	$ 9,000
REMOVE INLET	EA	1	$ 1,400	$ 1,400
REMOVING CONC (MEDIANS)	SY	41	$ 15	$ 617
UNCLASSIFIED STREET EXCAVATION	CY	1,097	$ 50	$ 54,850
TXDOT INLET	EA	1	$ 13,000	$ 13,000
CONC BOX CULV (10 FT X 5 FT)	LF	350	$ 1,200	$ 420,000
SLOPED CONCRETE HEADWALL	EA	14	$ 8,000	$ 112,000
HMAC ASPHALT PAVEMENT (2" TYPE D)	SY	412	$ 40	$ 16,480
EARTHWORK (EROSN & SEDMT CONT, IN VEH)	CY	741	$ 50	$ 37,037
6" SEPARATE CONCRETE CURB & GUTTER	LF	148	$ 55	$ 8,140
CONC MEDIAN	SY	41	$ 75	$ 3,083
			SUB-TOTAL =	**$ 698,427**
			CONTINGENCY (APPROX. 40%) =	**$ 279,373**
			CONTINGENCY SUB-TOTAL =	**$ 977,800**
			ADMIN, ENGINEERING, CONSTRUCTION ADMIN (APPROX. 30%) =	**$ 294,200**
			TOTAL =	**$ 1,272,000**

City of Denison Conceptual SWMP
Appendix B
Estimation of Probable Cost

Engineer's Opinion of Cost
Project Reference List

Project E
Loy Lake Rod Culvert Crossing

ITEM DESCRIPTION	UNIT	QUANTITY	UNIT COST	TOTAL COST
REMOVE ASPHALT PAVEMENT	SY	185	$ 50	$ 9,250
REMOVE & DISPOSE REINFORCED CONCRETE PIPE	LF	200	$ 45	$ 9,000
REMOVE HEADWALL	EA	2	$ 350	$ 700
UNCLASSIFIED STREET EXCAVATION	CY	555	$ 50	$ 27,750
CONC BOX CULV (8 FT X 6 FT)	LF	450	$ 1,200	$ 540,000
SLOPED CONCRETE HEADWALL	EA	2	$ 8,000	$ 16,000
REMOVE EXISTING TREES	EA	7	$ 1,700	$ 11,900
EARTHWORK (EROSN & SEDMT CONT, IN VEH)	CY	1,614	$ 50	$ 80,700
HMAC ASPHALT PAVEMENT (2" TYPE D)	SY	185	$ 40	$ 7,400
			SUB-TOTAL =	**$ 702,700**
			CONTINGENCY (APPROX. 40%) =	**$ 281,100**
			CONTINGENCY SUB-TOTAL =	**$ 983,800**
			ADMIN, ENGINEERING, CONSTRUCTION ADMIN (APPROX. 30%) =	**$ 295,200**
			TOTAL =	**$ 1,279,000**

Project F
Coffin and Woodlawn

ITEM DESCRIPTION	UNIT	QUANTITY	UNIT COST	TOTAL COST
REMOVE ASPHALT PAVEMENT	SY	17	$ 50	$ 850
REMOVE & DISPOSE CMP PIPE	LF	50	$ 40	$ 2,000
REMOVE & REPLACE PAVEMENT, CURB, & GUTTER	LS	1	$ 8,000	$ 8,000
UNCLASSIFIED STREET EXCAVATION	CY	25	$ 50	$ 1,250
CITY OF DENISON TYPE B MANHOLE	EA	3.00	$ 13,500	$ 40,500
10' STD. CURB INLET	EA	3	$ 11,000	$ 33,000
18" CLASS III REINF. CONC. PIPE	LF	100	$ 190	$ 19,000
66" CLASS III REINF. CONC. PIPE	LF	2,000	$ 850	$ 1,700,000
SLOPED CONCRETE HEADWALL	EA	2	$ 8,000	$ 16,000
HMAC ASPHALT PAVEMENT (2" TYPE D)	SY	17	$ 40	$ 680
EARTHWORK (EROSN & SEDMT CONT, IN VEH)	CY	1,289	$ 50	$ 64,450
			SUB-TOTAL =	$ 1,885,730
			(APPROX. 40%) =	$ 754,370
			NCY SUB-TOTAL =	$ 2,640,100
			(APPROX. 30%) =	$ 792,900
			TOTAL =	$ 3,433,000

Project G
Stream Downstream of Coffin Street and Park Avenue

ITEM DESCRIPTION	UNIT	QUANTITY	UNIT COST	TOTAL COST
MOBILIZATION & SITE PREPARATION	LS	1	$ 100,000	$ 100,000
STREAM CONSTRUCTION ACCESS	LS	1	$ 25,000	$ 25,000
UNCLASSIFIED STREAM EXCAVATION	CY	570	$ 80	$ 45,600
BACKFILL	CY	400	$ 30	$ 12,000
REMOVE GRADE CONTROL STRUCTURE	EA	1	$ 5,000	$ 5,000
GRADE CONTROL STRUCTURE	CY	30	$ 800	$ 24,000
GABIONS (3' X 3') (GALV)	CY	420	$ 400	$ 168,000
FILTER FABRIC	SY	400	$ 20	$ 8,000
			SUB-TOTAL =	$ 421,600
			CONTINGENCY (APPROX. 40%) =	$ 168,700
			CONTINGENCY SUB-TOTAL =	$ 590,300
			ADMIN, ENGINEERING, CONSTRUCTION ADMIN (APPROX. 30%) =	$ 177,700
			TOTAL =	$ 768,000

Project H
Lillis Lane and Crawford Street

ITEM DESCRIPTION	UNIT	QUANTITY	UNIT COST	TOTAL COST
REMOVE ASPHALT PAVEMENT	SY	567	$ 50	$ 28,350
REMOVE CURB AND GUTTER	LF	300	$ 15	$ 4,500
REMOVE & DISPOSE REINFORCED CONCRETE PIPE	LF	300	$ 45	$ 13,500
REMOVE HEADWALL	EA	2	$ 350	$ 700
UNCLASSIFIED STREET EXCAVATION	CY	1,512	$ 50	$ 75,600
CONC BOX CULV (5 FT X 5 FT)	LF	1,500	$ 500	$ 750,000
SLOPED CONCRETE HEADWALL	EA	2	$ 8,000	$ 16,000
HMAC ASPHALT PAVEMENT (2" TYPE D)	SY	567	$ 40	$ 22,680
6" SEPARATE CONCRETE CURB & GUTTER	LF	300	$ 55	$ 16,500
			SUB-TOTAL =	**$ 927,830**
			CONTINGENCY (APPROX. 40%) =	**$ 371,170**
			CONTINGENCY SUB-TOTAL =	**$ 1,299,000**
			ADMIN, ENGINEERING, CONSTRUCTION ADMIN (APPROX. 30%) =	**$ 390,000**
			TOTAL =	**$ 1,689,000**

Project I

Chandler Avenue near Munson Street

ITEM DESCRIPTION	UNIT	QUANTITY	UNIT COST	TOTAL COST
30" CLASS III REINF. CONC. PIPE	LF	275	$ 300	$ 82,500
CONNECT TO EXISTING DRAINAGE STRUCTURE	EA	1	$ 5,000	$ 5,000
4' x 4' SQUARE DROP INLET	EA	1	$ 10,000	$ 10,000
CITY OF DENISON TYPE B MANHOLE	EA	2.00	$ 13,500	$ 27,000
REMOVE & REPLACE PAVEMENT, CURB, & GUTTER	LS	1	$ 8,000	$ 8,000
UNCLASSIFIED STREET EXCAVATION	CY	104	$ 50	$ 5,200
ROW ACQUISITION	SF	1,225	$ 35	$ 42,875
EARTHWORK (EROSN & SEDMT CONT, IN VEH)	CY	264	$ 50	$ 13,200
			SUB-TOTAL =	$ 227,775
			CONTINGENCY (APPROX. 40%) =	$ 91,125
			CONTINGENCY SUB-TOTAL =	$ 318,900
			ADMIN, ENGINEERING, CONSTRUCTION ADMIN (APPROX. 30%) =	$ 96,100
			TOTAL =	$ 415,000

Project J

Dubois Between Scullin and Barrett

ITEM DESCRIPTION	UNIT	QUANTITY	UNIT COST		TOTAL COST	
REMOVE ASPHALT PAVEMENT	SY	210	$	50	$	10,500
REMOVE & DISPOSE REINFORCED CONCRETE PIPE	LF	250	$	45	$	11,250
REMOVE HEADWALL	EA	2	$	350	$	700
UNCLASSIFIED STREET EXCAVATION	CY	490	$	50	$	24,500
CONC BOX CULV (10 FT X 4 FT)	LF	250	$	1,200	$	300,000
SLOPED CONCRETE HEADWALL	EA	2	$	8,000	$	16,000
HMAC ASPHALT PAVEMENT (2" TYPE D)	SY	210	$	40	$	8,400
				SUB-TOTAL =	$	371,350
				CONTINGENCY (APPROX. 40%) =	$	148,550
				CONTINGENCY SUB-TOTAL =	$	519,900
				ADMIN, ENGINEERING, CONSTRUCTION ADMIN (APPROX. 30%) =	$	156,100
				TOTAL =	$	676,000

Project K

Flora Lane near Lum Lane

ITEM DESCRIPTION	UNIT	QUANTITY	UNIT COST	TOTAL COST
EARTHWORK (EROSN & SEDMT CONT, IN VEH)	CY	472	$ 50	$ 23,611
GROUTED RIPRAP TYPE A	SY	483	$ 300	$ 144,900

SUB-TOTAL =	**$**	**168,511**
CONTINGENCY (APPROX. 40%) =	**$**	**67,489**
CONTINGENCY SUB-TOTAL =	**$**	**236,000**
ADMIN, ENGINEERING, CONSTRUCTION ADMIN (APPROX. 30%) =	**$**	**71,000**
TOTAL =	**$**	**307,000**

Project L

Coffin Street and Park Avenue

ITEM DESCRIPTION	UNIT	QUANTITY	UNIT COST	TOTAL COST
ITEM DESCRIPTION	UNIT	QUANTITY	UNIT COST	TOTAL COST
REMOVE & REPLACE PAVEMENT, CURB, & GUTTER	LS	1	$ 8,000	$ 8,000
REMOVE & DISPOSE REINFORCED CONCRETE PIPE	LF	150	$ 45	$ 6,750
REMOVE HEADWALL	EA	6	$ 350	$ 2,100
UNCLASSIFIED STREET EXCAVATION	CY	2	$ 50	$ 100
30" CLASS III REINF. CONC. PIPE	LF	290	$ 300	$ 87,000
SLOPED CONCRETE HEADWALL	EA	8	$ 8,000	$ 64,000
CONCRETE CHANNEL	SY	1,240	$ 185	$ 229,400
EARTHWORK (EROSN & SEDMT CONT, IN VEH)	CY	517	$ 50	$ 25,850
			SUB-TOTAL =	**$ 423,200**
			CONTINGENCY (APPROX. 40%) =	**$ 169,300**
			CONTINGENCY SUB-TOTAL =	**$ 592,500**
			ADMIN, ENGINEERING, CONSTRUCTION ADMIN (APPROX. 30%) =	**$ 178,500**
			TOTAL =	**$ 771,000**

Project M
Eddy Avenue Between Elm Street and Martin Luther King Street

ITEM DESCRIPTION	UNIT	QUANTITY	UNIT COST	TOTAL COST
ITEM DESCRIPTION	UNIT	QUANTITY	UNIT COST	TOTAL COST
REMOVE & REPLACE PAVEMENT, CURB, & GUTTER	LS	1	$ 8,000	$ 8,000
12' STD. CURB INLET	EA	2	$ 13,000	$ 26,000
CITY OF DENISON TYPE B MANHOLE	EA	3	$ 13,500	$ 40,500
30" CLASS III REINF. CONC. PIPE	LF	500	$ 300	$ 150,000
18" CLASS III REINF. CONC. PIPE	LF	100	$ 190	$ 19,000
CONNECT TO EXISTING DRAINAGE STRUCTURE	EA	2	$ 5,000	$ 10,000
			SUB-TOTAL =	**$ 253,500**
			CONTINGENCY (APPROX. 40%) =	**$ 101,400**
			CONTINGENCY SUB-TOTAL =	**$ 354,900**
			ADMIN, ENGINEERING, CONSTRUCTION ADMIN (APPROX. 30%) =	**$ 107,100**
			TOTAL =	**$ 462,000**

Appendix C: Field Visit Report

City of Denison
Conceptual Stormwater Master Plan
Field Reconnaissance Report

PROJECT: City of Denison Conceptual Stormwater Master Plan

LOCATION: Denison and Surrounds

PROJECT NO.: R315880.01 DATE: November 2, 2022

PURPOSE: Field review of known flooding problem areas

ATTENDEES: Paul Hoskin – HZ
Christian Aguirre - HZ
Doug Olds - Denison

PURPOSE:

A meeting was held to introduce the team to the City staff after which a city tour was conducted by Doug Olds of the noted areas where there have been historical problems with flooding and resulting maintenance. At the time of the field visit a different numbering system was in place. The old and new numbering systems are provided below:

DISCUSSION:

Area C (15) Sears Street - Scullin to Fannin

Drainage concentrates within curbs of Sears Street, commencing west of Scullin Street until Fannin. The flow extends south of Sears street and flows between homes on its way to Fannin Street. Many homes in this area appear to be low-lying. At Fannin, drainage enters a curb inlet and flows north through neighborhood drainage channels.

Area L (13) Coffin Street and Park Avenue

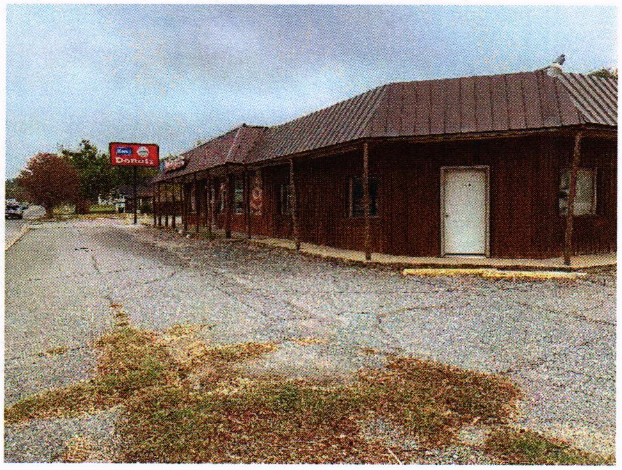
Flow occurs down Coffin Street and floods the intersection with Park Avenue Drainage also

Coffin Street Showing Ditch and Cross-Street Culvert

reaches the north side of Coffin Street and flows within a drainageway before reaching a culvert crossing of Park Avenue.

Area B (12) Morton and Maurice

The intersection at Morton Street and Maurice Avenue experiences flooding after intense rainfall events. While Maurice Avenue falls under the jurisdiction of the City of Denison, Morton Street is a State Highway under the jurisdiction of the Texas Department of Transportation (TXDOT). Existing storm drain infrastructure includes a storm drain system with five grated inlets near the intersection of Morton and Maurice. The main storm drain outfalls to an existing drainageway east of the intersection of Maurice and Bond. The receiving drainageway, which flows from south to north, is an unnamed tributary to Duck Creek. Flooding of buildings occurs at the southeast corner and at the Donut Shop.



Area F (11) Coffin and Woodlawn

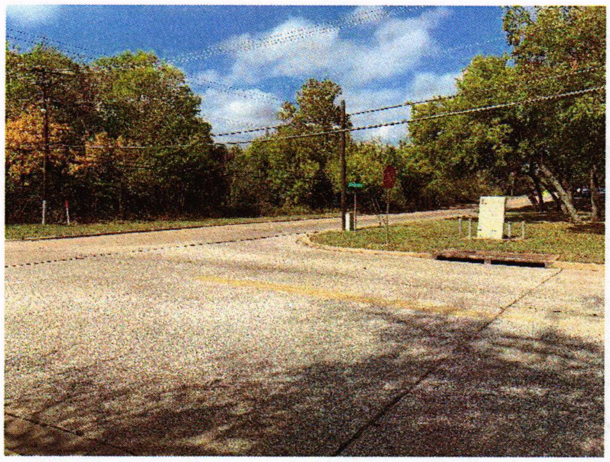
Drainage flows south along the west side of the railroad tracks and enters Coffin Street where it combines with flow from Woodlawn to flood the intersection. The flow continues west along

Coffin Street, which steepens before ending at Gerrard Street. The drainage continues west and outfalls into Waterloo Creek.

There is a planned development for this area. Any improvements proposed should be coordinated with that project.

City of Denison
Conceptual Stormwater Master Plan
Field Reconnaissance Report

Area H (10) Lillis and Crawford



Drainage originates from Morton Street at a Walmart store. Surface drainage is conveyed under Lillis through a 5'x 5' concrete box culvert. The road overtops if there are obstruction that block the culvert flow. There is an area at the northwest corner that is proposed as a multi-family site.

November 2, 2022

City of Denison
Conceptual Stormwater Master Plan
Field Reconnaissance Report

Area M (9) Eddy Street between Elm & Martin Luther King Street

This area may be a lower priority, but backyards are being flooded, and new homes are stating to build up.

City of Denison
Conceptual Stormwater Master Plan
Field Reconnaissance Report

(5) Bus Facility

Flows cross under Highway 91 and have caused flooding of the bus facility. This drainage way is called Iron Ore Creek, and it is in a FEMA designated floodplain.

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City of Denison
Conceptual Stormwater Master Plan
Field Reconnaissance Report

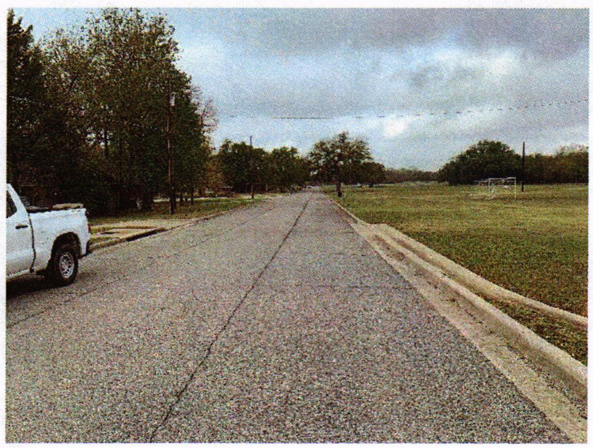
Area A (14) Flora and Rivercrest






City of Denison
Conceptual Stormwater Master Plan
Field Reconnaissance Report

Houses south of Flora Lane are subject to flooding because they are mostly built below the road grades. When the road overtops it can flow down driveways, into front yards, and then through side yards. Finished floors are relatively low and homes are at risk from flooding. A single 36" diameter culvert takes flow from Katy Practice Fields to the north of Flora Lane and directs into a grouted riprap channel. The channel may be undersized and the top of bank is higher than the adjacent home's floor elevation. At Rivercrest Circle, the channel flow is conveyed through 3 – 42" diameter concrete pipes.



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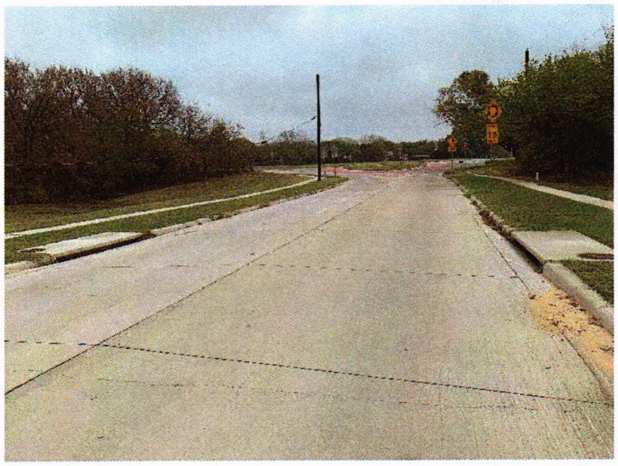
City of Denison
Conceptual Stormwater Master Plan
Field Reconnaissance Report

Area K (18) Flora and Lum



Concrete box culverts were constructed recently and convey flow from north to south. A home constructed after the culverts were completed appears to lie within the path of the wash and could be at flood risk since the finished floor appears low relative to Flora Lane. A small amount of bank protection is provided to help turn the flow eastward.

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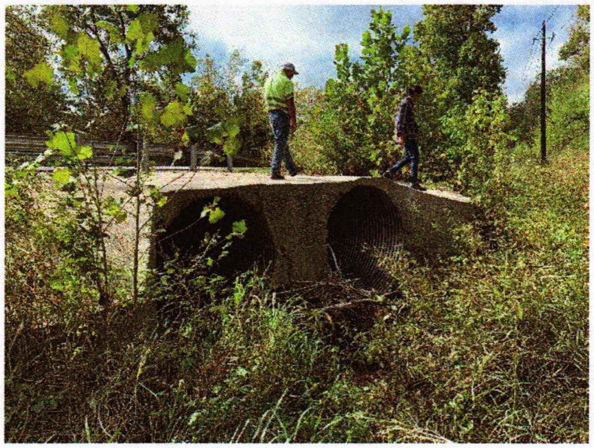
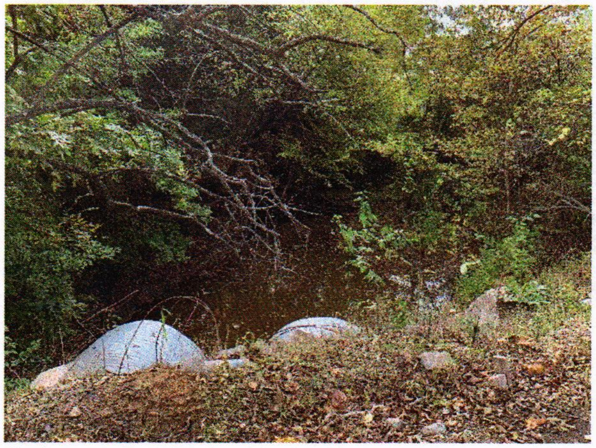
City of Denison
Conceptual Stormwater Master Plan
Field Reconnaissance Report

Area E (17) Loy Lake Road south of Loy Lake



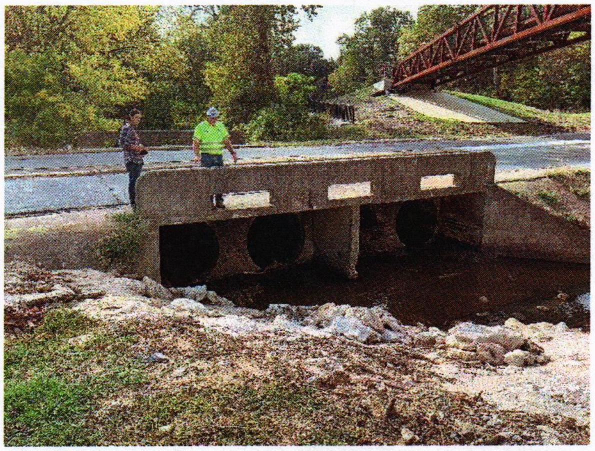

Culverts across Loy Lake Road receive debris from lower watershed. Possible inlet concerns.

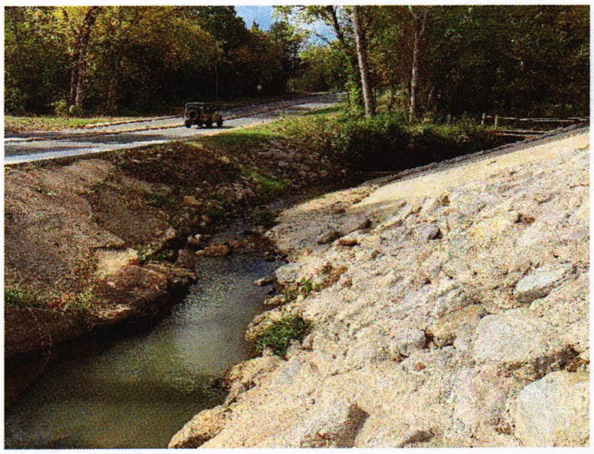
November 2, 2022

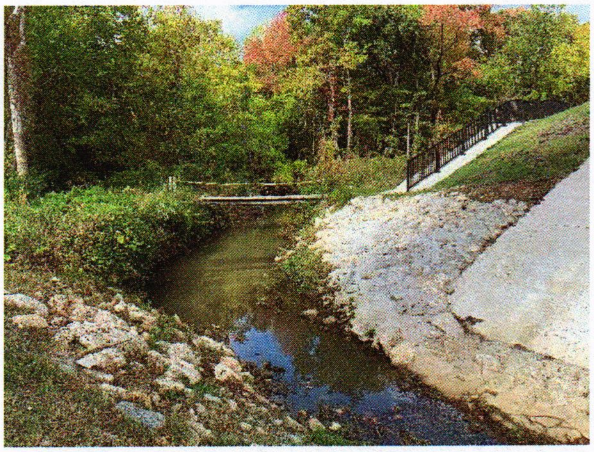
City of Denison
Conceptual Stormwater Master Plan
Field Reconnaissance Report

Area D (12) Loy Lake Road and Waterloo Creek

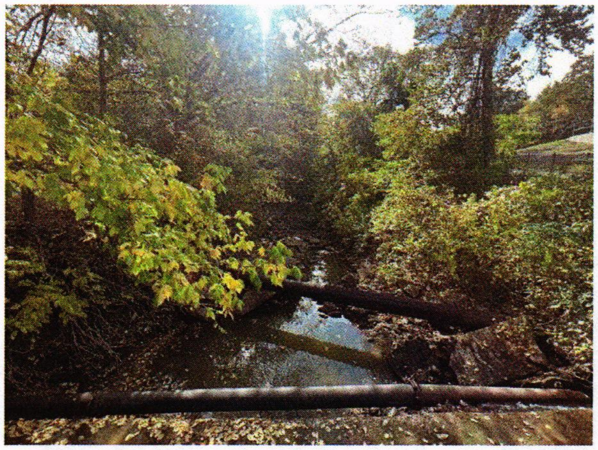
A four barrel culvert, situated with Loy Lake Park, crosses Loy Lake Road and conveys the flow from within Waterloo Creek. Recent construction of a pedestrian bridge across Loy Lake Road has encroached upon the wash causing a constriction in the channel. This, in combination with limited culvert size causes the road to overtop.







At the downstream end of the culvert, water and/or sewer lines are exposed and may require realignment with any culvert replacement.



November 2, 2022

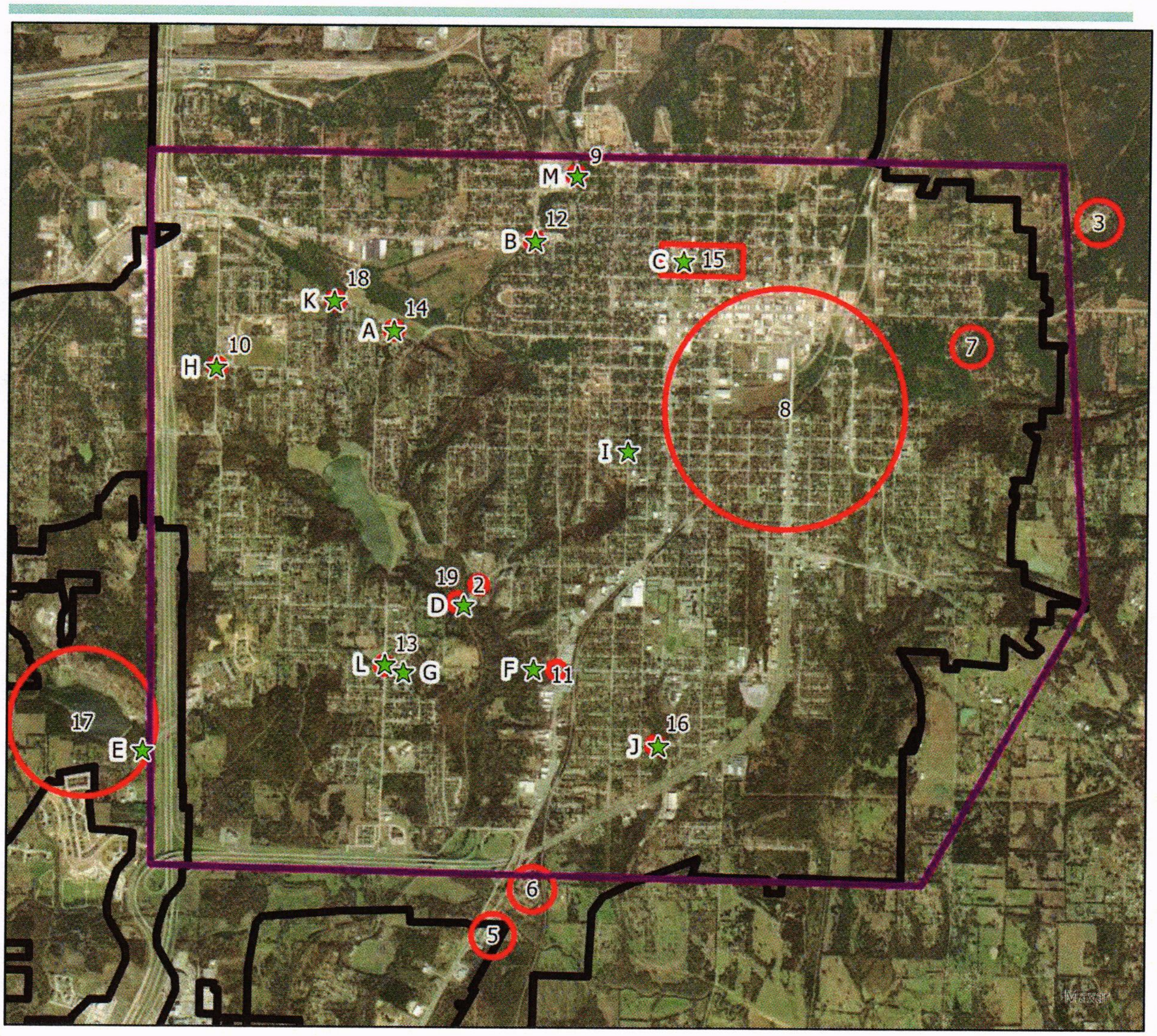
City of Denison
Conceptual Stormwater Master Plan
Field Reconnaissance Report

Area J (16) 800 Block of Dubois Street

Dubois Street provides east-west access to neighborhoods on either side of the creek. The next east-west street is Coffin Street located approximately 1,700 feet to the north. During heavy rainfall events the road can overtop, either due to a lack of capacity or because of debris flows. A log is shown at the culvert entrance. This watershed has plenty of potential debris.

November 2, 2022

City of Denison
Conceptual Stormwater Master Plan
Field Reconnaissance Report



HUITT-ZOLLARS

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Appendix D: HEC-HMS Model Methods, Inputs, and Results

Table of Contents

List of Tables

List of Figures

Attachments

1.0 Introduction

Huitt-Zollars, Inc. (HZ) has been under contract with the City of Denison (the City) to provide a Conceptual Stormwater Master Plan (SWMP) to identify current drainage-related issues and aid in the planning of future drainage projects.

1.1 Project Location

The City of Denison encompasses as area of approximately 29-square miles within Grayson County. Denison is part of the two principal cities that make up the Sherman-Denison metropolitan statistical area. Located near the northern border of Texas with the State of Oklahoma, the City of Denison and its extra-territorial jurisdiction area drainages ultimately drain to the Red River. The City lies in that larger Bois D'Arc Watershed, and the total drainage area contributing to the City is approximately 64 square miles. A handful of reservoirs are located within the City, including Lake Randell, Loy Lake, Waterloo Lake, and Country Club Lake. A map presented the project location is provided in Figure 1.

1.2 Modeling Software

The Hydrologic Modeling System (HEC-HMS) software, version 4.3, was used to model hydrology within the study area. The model is developed as drainage basins that are represented by various connected nodes, including sub-basins, junctions, and reservoirs. The modeling is performed on a basin-by-basin basis, such that each drainage basin is modeled separately.

2.0 Design Standards

The study is developed in accordance with the current City of Denison standards. Storm drainage standards are addressed in the *Denison, Texas Storm Drainage System Design Manual* (Ref. 1). Hydrology guidance from the integrated stormwater management (iSWM) Technical Manual for Hydrology (Ref. 2) was used for hydrology model development.

3.0 Modeling Methods

The HEC-HMS model is developed to integrate sub-basins, precipitation, infiltration losses, hydrograph transformation, and hydrograph routing into a modeling network representing each of seven drainage basins within the study area. The schematics used for each drainage basin are included in Attachment 1.

3.1 Drainage Basin & Sub-Basin Delineation

Drainage basins and sub-basins were consistent with the drainage basins and sub-basins from the Federal Emergency Management Agency (FEMA) Bois D'Arc Watershed, TX Base Level Engineering (BLE). Report (Ref. 3). Within the BLE study, the Denison study area is divided into seven drainage basins: Duck Creek, Iron Ore Creek, Pawpaw Creek, Shawnee Creek, UNT0097, UNT140, and UNT141 (Figure 2). Input drainage area for each sub-basin is tabulated in Attachment 2.

3.2 Design Storm

Synthetic frequency storms were developed in HEC-HMS using gridded rainfall depth-duration-frequency partial duration data from the National Oceanic and Atmospheric Administration (NOAA) Atlas 14 Precipitation Frequency Data Server (PFDS) for the 10-year and 100-year return periods (Ref. 4). For most design criteria, the 100-year storm results are the design discharge, but the 10-year storm is being evaluated as an additional scenario. In the HEC-HMS software, the frequency storm method produces a synthetic storm from statistical precipitation data. The frequency storm method is applied using the maximum depth for a given duration and frequency within each drainage basin.

The frequency storm method is applied over a 1-day period with a 5-minute intensity duration and an intensity position at 50 percent for each sub-basin within a drainage basin. The precipitation data for each basin for the 10-year and 100-year return interval storms are tabulated in Attachment 3.

3.3 Losses

Infiltration losses were estimated using the Soil Conservation Service (SCS) hydrologic method (curve number method) for existing land use and soil conditions within the study area. Existing land use data is developed from the National Land Cover Database (NLCD) from 2019 (Ref. 5). Soil data comes from the Soil Survey Geographic Database (SSURGO) for the study area (Ref. 6). The relationships between land cover, hydrologic soil group, and curve number (CN), developed from Texas Department of

Transportation (TX-DOT) guidance (Ref. 7), are provided in Table 1. Inputs for each sub-basin are provided in Attachment 4.

3.3.1 Curve Number

Curve number is an empirical loss parameter used in the SCS hydrologic method which represents the runoff potential for a given sub-basin (Ref. 8). Curve numbers are a function of the soil type and the cover conditions for a given sub-basin. Under the curve number method, the runoff for a watershed is a function of rainfall, CN, and initial abstraction. Initial abstraction encompasses all losses before runoff begins and includes water retained in surface depressions, water intercepted by vegetation, evaporation, and infiltration.

$$Q = \frac{(P - I_a)^2}{(P - I_a) + \left(\frac{1000}{CN} - 10\right)}$$

Where:

Q = Runoff (inches)

P = Rainfall (inches)

I_a = Initial Abstraction (inches)

CN = Curve Number

The initial abstraction is approximated by the following equation:

$$I_a = 0.2\left(\frac{1000}{CN} - 10\right)$$

3.3.2 Land Cover and Land Use

Existing land cover data for the hydrologic and hydraulic analyses in this study are from the National Land Cover Database (NLCD) dataset for 2019 (Ref. 5). Land use information from the City's interactive zoning map and future land use from Denison's comprehensive plan (Ref. 9) inform the land cover and use for a given area. Land cover from NLCD is presented in Figure 4.

3.3.3 Soils

The SCS hydrologic method bases sub-basin losses on four hydrologic soil groups – A, B, C, and D. The soil groups represent increasing runoff potential, where Group A soils have the lowest runoff potential (highest loss) and Group D soils have the highest runoff potential (lowest loss). The hydrologic soil groups are presented in Figure 5 for the study area.

Table 1: Curve Number for Given NLCD Cover Type and Hydrologic Soil Group

NLCD Value (2011)	NLCD Description	TX-DOT Description	Curve Number			
			A	B	C	D
11	Open Water	Water	100	100	100	100
21	Developed, Open Space	Open Space - Good Condition	39	61	74	80
22	Developed, Low Intensity	Residential - 1/4 acre	61	75	83	87
23	Developed, Medium Intensity	Residential - 1/8 acre	77	85	90	92
24	Developed, High Intensity	Commercial and Business	89	92	94	95
31	Barren Land	Saltbrush - Good	49	68	79	84
41	Deciduous Forest	Woods grass combination- Fair	43	65	76	82
42	Evergreen Forest	Woods - Fair	36	60	73	79
43	Mixed Forest	Woods grass combination- Fair	43	65	79	84
52	Shrub/Scrub	Brush - Fair	35	56	70	77
71	Herbaceous	Herbaceous-Fair	60	71	81	89
81	Hay/Pasture	Pasture - Good	39	61	74	80
82	Cultivated Crops	Row crops (SR) - Good	67	78	85	89
90	Woody Wetlands	Water	100	100	100	100
95	Emergent Herbaceous Wetlands	Water	100	100	100	100

3.4 Hydrograph Transformation

In accordance with the City of Denison design standards, the Snyder synthetic unit hydrograph method was used for hydrology modeling. In the HEC-HMS software, the inputs for the standard Snyder method are lag time, t_p, as defined by the following equation, and a peaking coefficient, C_p, which is estimated as a function of characteristic basin slope and the level of development with storm drains (Table 2). Basin slope is divided between flat, moderate, and steep, while storm sewer density is divided into sparsely sewered, moderately sewered, and highly sewered areas.

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

Where:

t_p = Lag Time (hours)

C_t = Empirical Coefficient (Table 2)

L = Stream Distance from Point of Design to Upper Limit of Drainage (miles)

L_{ca} = Stream Distance from Point of Design to Drainage Centroid (miles)

Table 2: Snyder Unit Hydrograph Coefficient Estimates (Ref. 1)

Drainage Area Characteristics	Ct	Cp
Sparsely Sewered Area		
Flat Basin Slope (less than 0.50%)	0.65	0.55
Moderate Basin Slope (0.5% to 0.8%)	0.60	0.58
Steep Basin Slope (greater than 0.8%)	0.55	0.61
Moderately Sewered Area		
Flat Basin Slope (less than 0.50%)	0.55	0.63
Moderate Basin Slope (0.5% to 0.8%)	0.50	0.66
Steep Basin Slope (greater than 0.8%)	0.45	0.69
Highly Sewered Area		
Flat Basin Slope (less than 0.50%)	0.45	0.70
Moderate Basin Slope (0.5% to 0.8%)	0.40	0.73
Steep Basin Slope (greater than 0.8%)	0.35	0.77

To account for variation within each sub-basin, area-weighting based on the sewer density adjusted values of C_t and C_p for each sub-basin. The level of sewer density assigned to each NLCD land use type is detailed in Table 3.

Table 3: Assigned Sewer Density for NLCD Cover Types

NLCD Value (2019)	Description	Assigned Sewer Density
11	Open Water	Excluded
21	Developed, Open Space	Sparsely Sewered Area
22	Developed, Low Intensity	Sparsely Sewered Area
23	Developed, Medium Intensity	Moderately Sewered Area
24	Developed, High Intensity	Highly Sewered Area
31	Barren Land	Sparsely Sewered Area
41	Deciduous Forest	Sparsely Sewered Area
42	Evergreen Forest	Sparsely Sewered Area
43	Mixed Forest	Sparsely Sewered Area
52	Shrub/Scrub	Sparsely Sewered Area
71	Herbaceous	Sparsely Sewered Area
81	Hay/Pasture	Sparsely Sewered Area
82	Cultivated Crops	Sparsely Sewered Area
90	Woody Wetlands	Sparsely Sewered Area
95	Emergent Herbaceous Wetlands	Sparsely Sewered Area

Inputs for hydrograph transformation by Snyder Unit Hydrograph and the associated calculations are provided in Attachment 5.

3.5 Flow Routing

Flow routing is implemented through the Muskingum-Cunge routing method. Inputs to the HEC-HMS model for Muskingum-Cunge routing include data representing channel geometry and characteristics. A general eight-point cross-section represents the channel geometry for each reach (Figure 6), and the roughness for each reach is based on the NLCD land cover classification within a 100-foot buffer of the reach (Table 4). A reference flow at half of the 100-year peak flow within a reach from the BLE study is specified to discretize each reach into sub-reaches and time sub-intervals. Inputs are provided in Attachment 6.

Table 4: Roughness Values Based on Land Use for Flow Routing

NLCD Code (2019)	Description	Assigned Value, n
11	Open Water	0.035
21	Developed, Open Space	0.040
22	Developed, Low Intensity	0.085
23	Developed, Medium Intensity	0.120
24	Developed, High Intensity	0.150
31	Barren Land	0.030
41	Deciduous Forest	0.160
42	Evergreen Forest	0.160
43	Mixed Forest	0.160
52	Shrub/Scrub	0.130
71	Herbaceous	0.050
81	Hay/Pasture	0.045
82	Cultivated Crops	0.045
90	Woody Wetlands	0.125
95	Emergent Herbaceous Wetlands	0.085

3.6 Baseflow

Baseflow is not modelled. Baseflow is assumed to not have a significant contribution to peak flows, which are the primary output of this study.

3.7 Reservoirs

There are three reservoirs modelled in the HEC-HMS Model, Loy Lake, Waterloo Lake, and Lake Randell. While Waterloo Lake and Lake Randell fall under the jurisdiction of the City, Loy Lake is operated by Grayson County. Although other, smaller ponds and lakes are found within the model domain, the model incorporated these three reservoirs to incorporate the attenuation provided by reservoir storage that reduces the peak discharge downstream. Summary data for the three modeled reservoirs are included in Table 5. The model assumes that the reservoir is at normal water level at the start of the 100-year, 24-hour storm using data from the National Inventory of Dams (NID) (Ref. 10). Without stage-storage data, stage-storage was estimated based on interpolation and extrapolation from existing topography data. Reservoir data includes stage-storage and stage-discharge relationships. These relationships are included in Attachment 7.

Table 5: Reservoir Data Summary

Reservoir	Owner	Primary Purpose	Hydraulic Height (Ft)	Max Storage (Acre-Ft)	Normal Storage (Acre-Ft)	Drainage Area (Square Mile)
Waterloo Lake	City of Denison	Flood Risk Reduction	53	2545	99	2.3
Lake Randell	City of Denison	Water Supply	55	6824	5690	10.1
Loy Lake	Grayson County	Recreation	25	642	370	2.4

3.7.1 Loy Lake

Loy Lake is a reservoir located west of the US-75 highway, just outside of the city limits for Denison. The lake's primary purpose is recreation. The earthen dam was completed in 1933, with a rise of about 25-feet and a regulatory storage of 642 acre-feet (ac-ft). Loy Lake is drained by an uncontrolled concrete spillway, and has a normal storage capacity of 370 ac-ft. The resulting additional capacity for stormwater during the design storm is 272 ac-ft. Loy Lake drains an area of approximately 2.4 square miles, and its spillway has capacity for 14,053 cubic feet per second.

3.7.2 Waterloo Lake

Waterloo Lake is a reservoir located north of Loy Lake Road in Denison. Its primary purpose is flood risk reduction, and it is operated by the City. The earthen dam was completed in 1991, with a rise of about 53-feet and a regulatory storage of 2,545 ac-ft. Waterloo Lake is drained by an uncontrolled concrete spillway and has a normal storage capacity of 99 ac-ft. The resulting additional capacity for stormwater during the design storm is 2,446 ac-ft. Waterloo Lake drains an area of approximately 2.3 square miles.

3.7.3 Lake Randell

Lake Randell is a reservoir located at the northwest limits of the City of Denison. Its primary purpose is potable water supply, and it is operated by the City. The earthen dam was completed in 1909, with a rise of about 55-feet and regulatory storage of 7,500 ac-ft. Randell lake is drained by a controlled spillway and has a normal storage volume of 4,525 ac-ft. The resulting additional capacity for stormwater during the design storm is 2,975 ac-ft. Lake Randell drains an area of approximately 10.3 square miles, and its spillway has capacity for 33,675 cubic feet per second.

4.0 Results

The modeling results for the 100-year and 10-year, 24-hour storms are presented in Attachment 8 and Attachment 9. The results show that the reservoirs play an important role in flood mitigation, as the results are highly sensitive to the water level at the beginning of the simulation. Additionally, the results provide discharges which were used to develop the proposed improvement projects as part of this study.

5.0 REFERENCES

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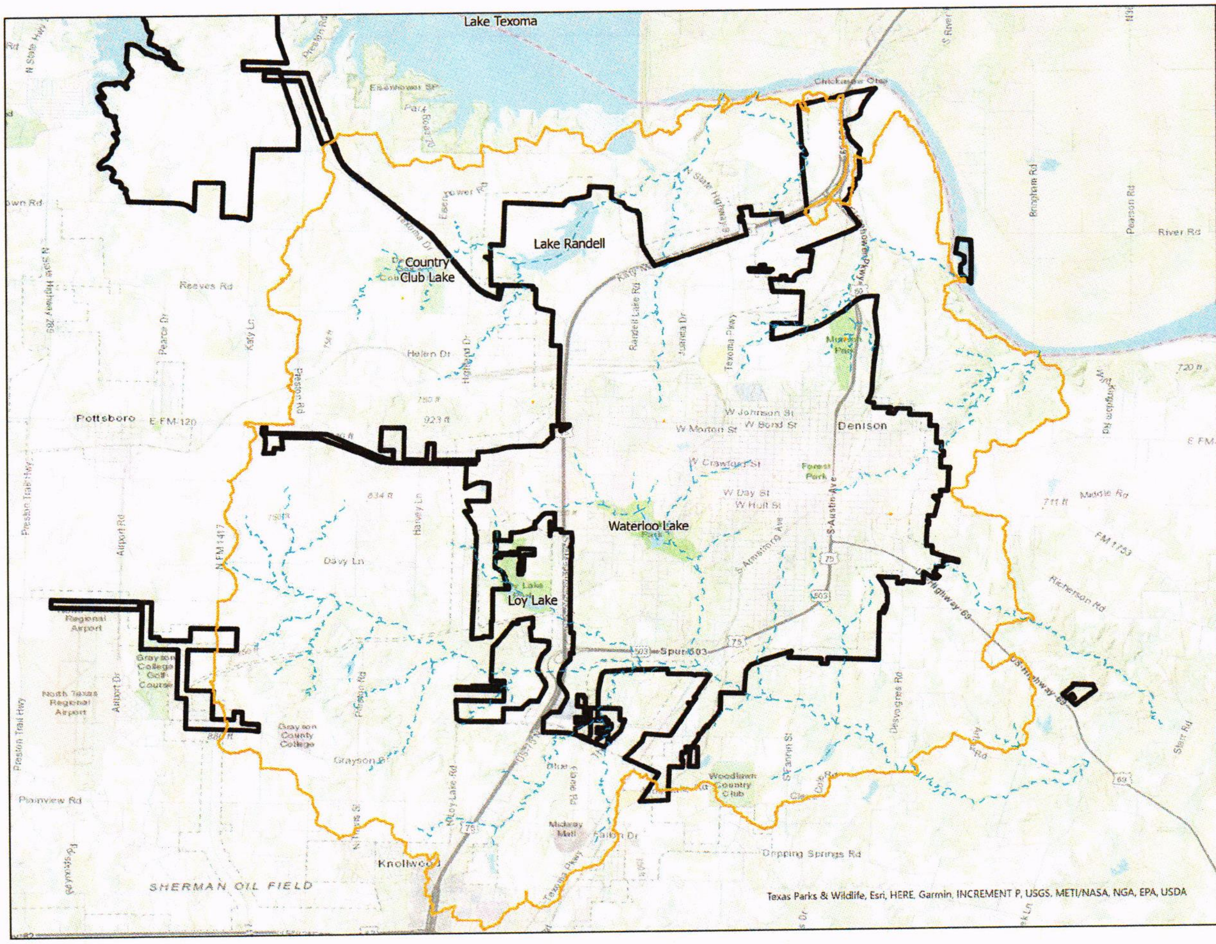
FIGURES

Figure 1: Model Location Map



CITY OF DENISON
300 W Main St
Denison, TX 75020

City of Denison
Conceptual Stormwater Master Plan
R315880.01

Legend
- Limits of Study/Drainage
- Stream Lines
- Denison City Limit

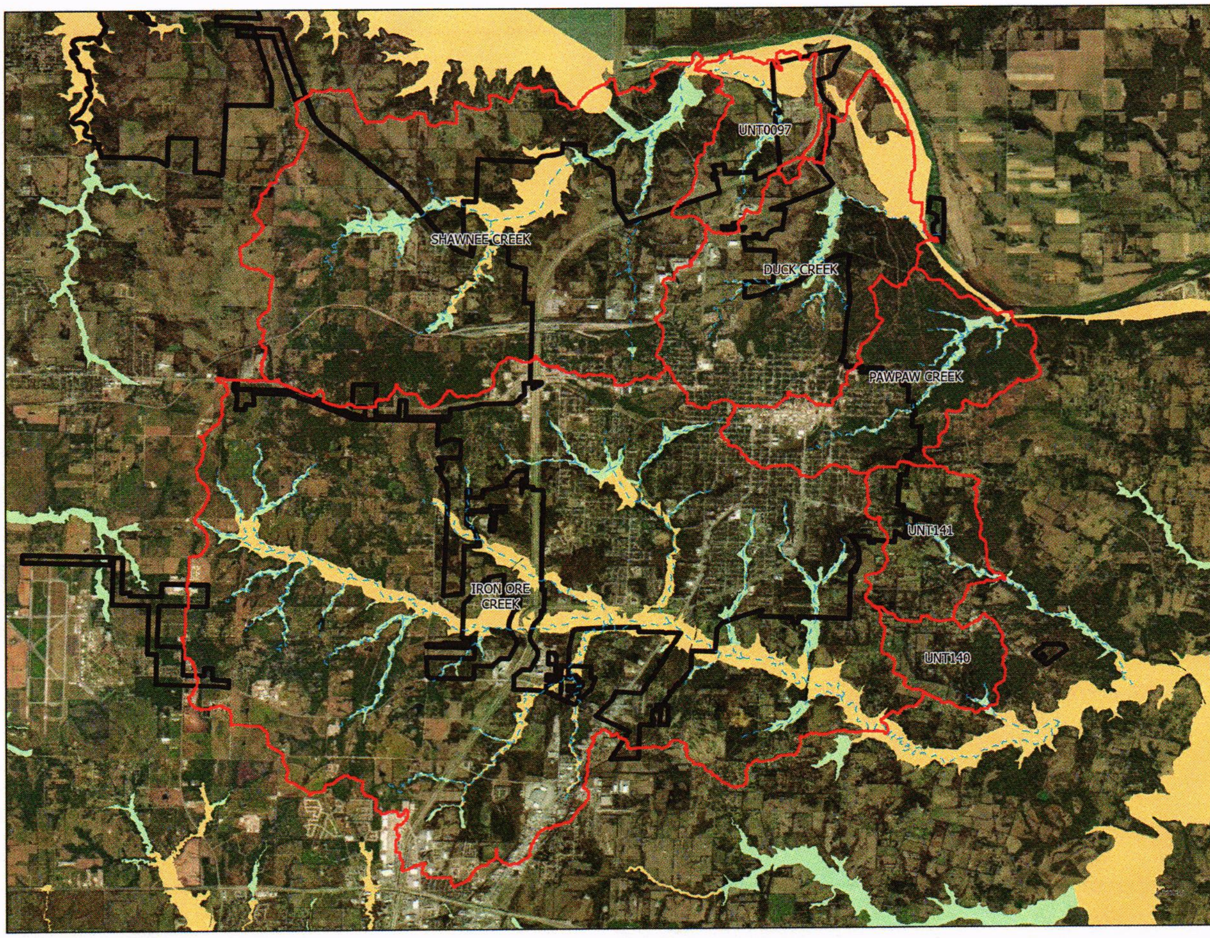
Appendix D
Figure 1

Model Location Map

0 1 2 Miles

HUITT-ZOLLARS
5430 LBJ Freeway, Suite 1500
Dallas, Texas 75240

Figure 2: Drainage Map



CITY OF DENISON
300 W Main St
Denison, TX 75020

Denison TEXAS

City of Denison
Conceptual Stormwater Master Plan
R315880.01

Legend

- Drainage Basins
- Stream Lines
- Denison City Limit

FEMA Flood Zone

- A
- AE

N

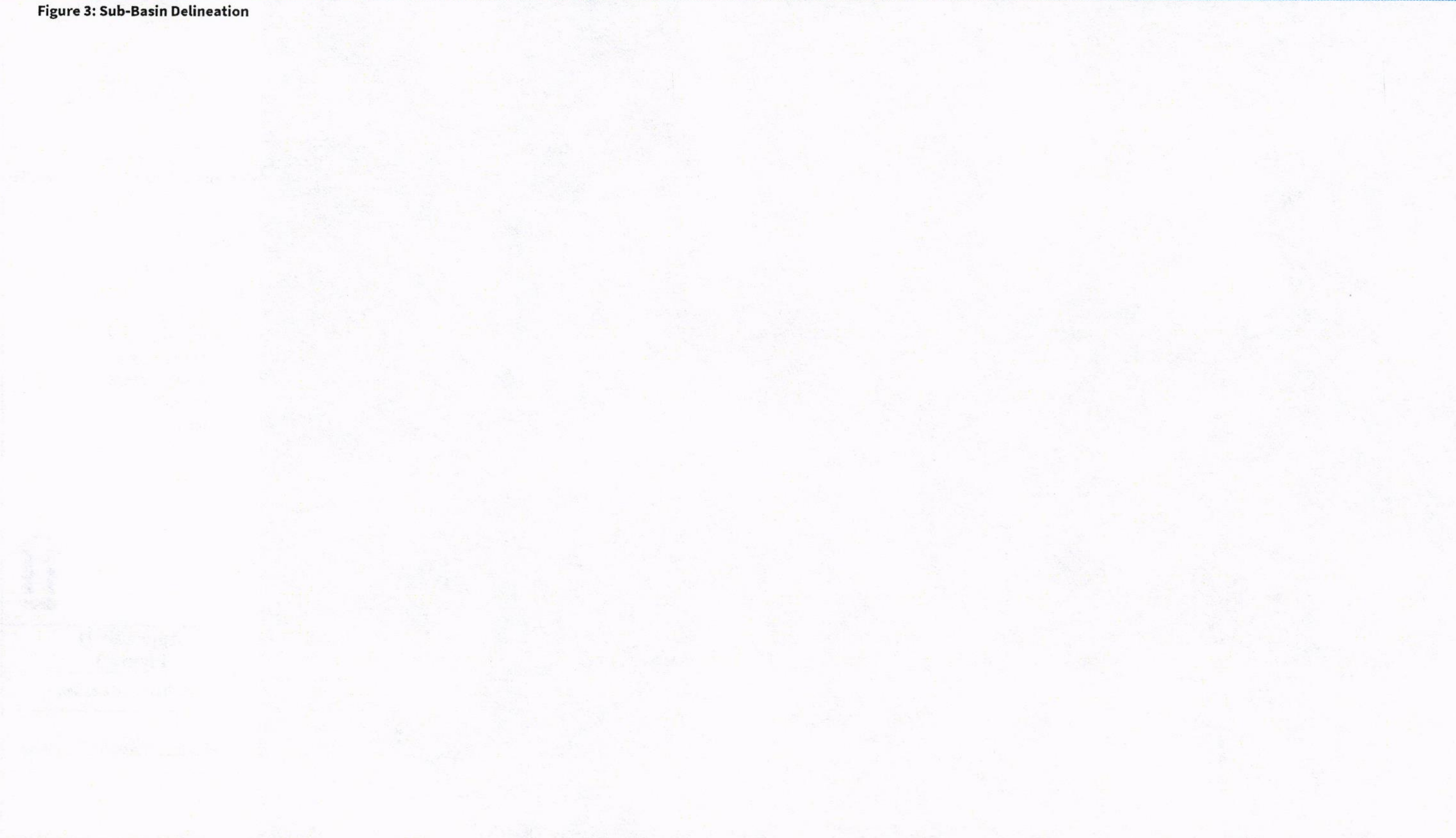
Appendix D
Figure 2

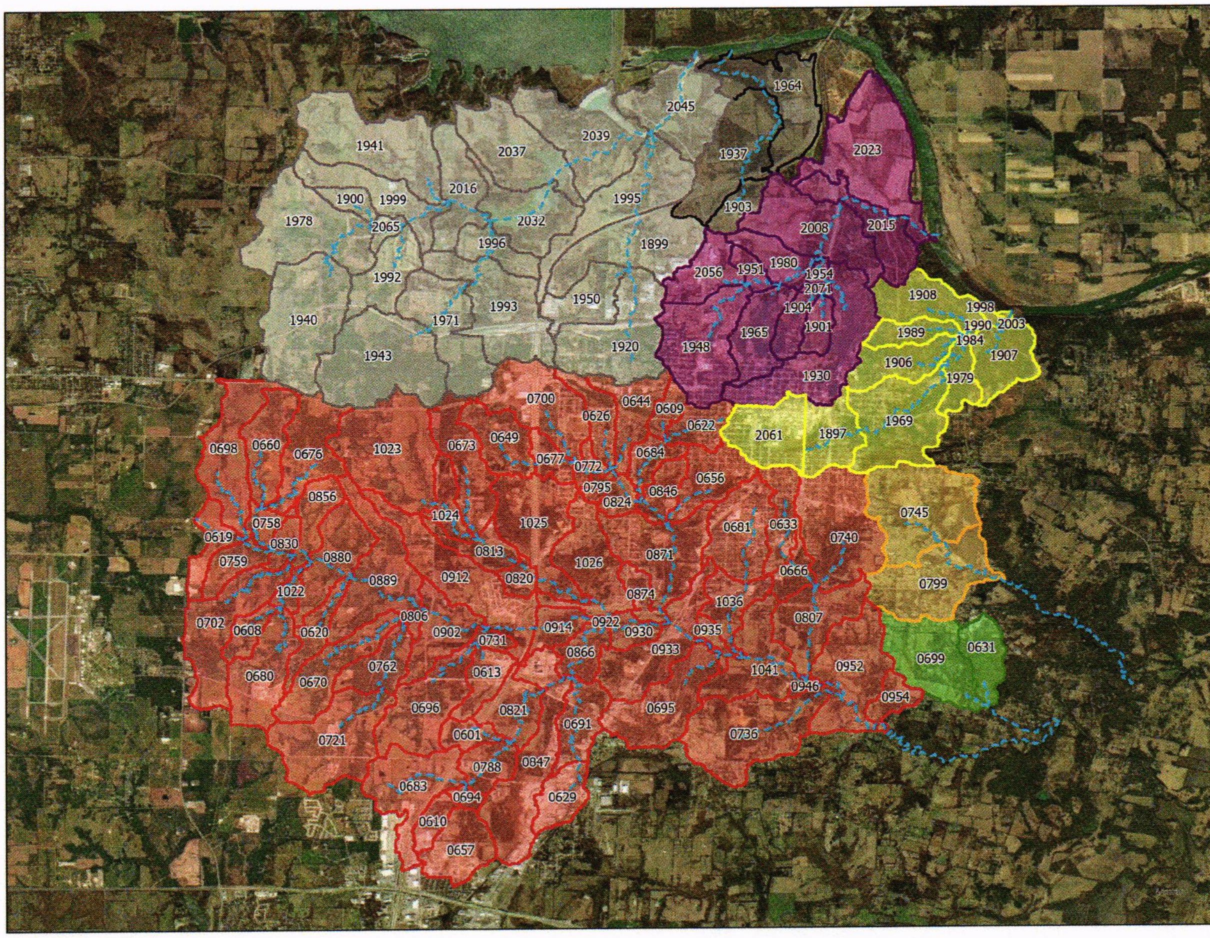
Drainage Map

0 1 2 Miles

HUITT-ZOLLARS
5430 LBJ Freeway, Suite 1500
Dallas, Texas 75240

Figure 3: Sub-Basin Delineation





CITY OF DENISON

300 W Main St
Denison, TX 75020

Denison
TEXAS

City of Denison
Conceptual Stormwater Master Plan
R315880.01

Legend

Stream Lines

Basin

- DUCK CREEK
- IRON ORE CREEK
- PAWPAW CREEK
- SHAWNEE CREEK
- UNT0097
- UNT140
- UNT141

N

Appendix D
Figure 3

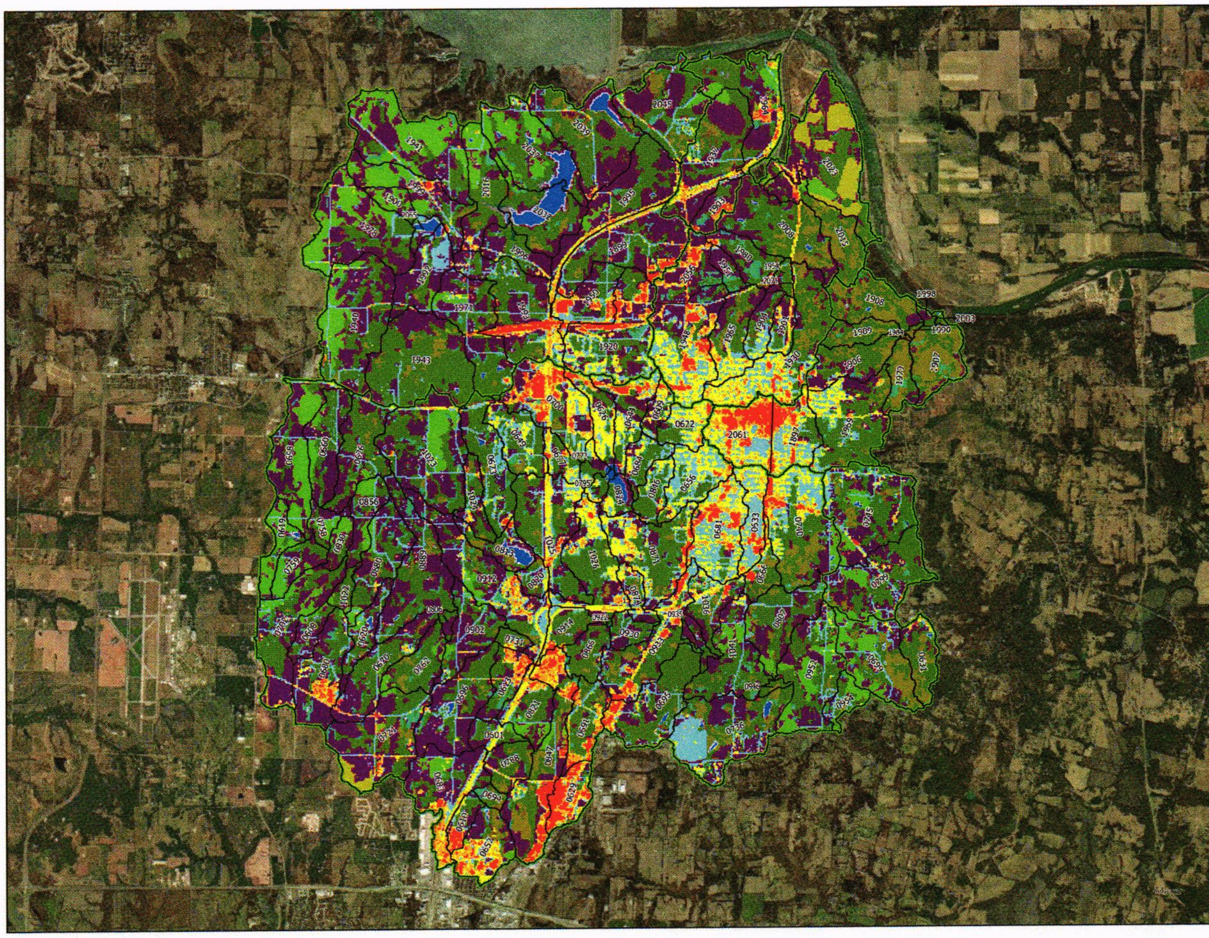
Sub-Basin Delineation

0 1 2 Miles

HUITT-ZOLLARS

5430 LBJ Freeway, Suite 1500
Dallas, Texas 75240

Figure 4: Land Cover Map



CITY OF DENISON
300 W Main St
Denison, TX 75020

City of Denison
Conceptual Stormwater Master Plan
R315880.01

Legend
- Sub-Basins
- Basins

Land Cover
- Open Water
- Developed Open Space
- Developed Low Intensity
- Developed Medium Intensity
- Developed High Intensity
- Barren Land
- Deciduous Forest
- Evergreen Forest
- Mixed Forest
- Shrub/Scrub
- Herbaceous
- Hay/Pasture
- Cultivated Crops
- Woody Wetlands
- Emergent Herbaceous Wetlands

N

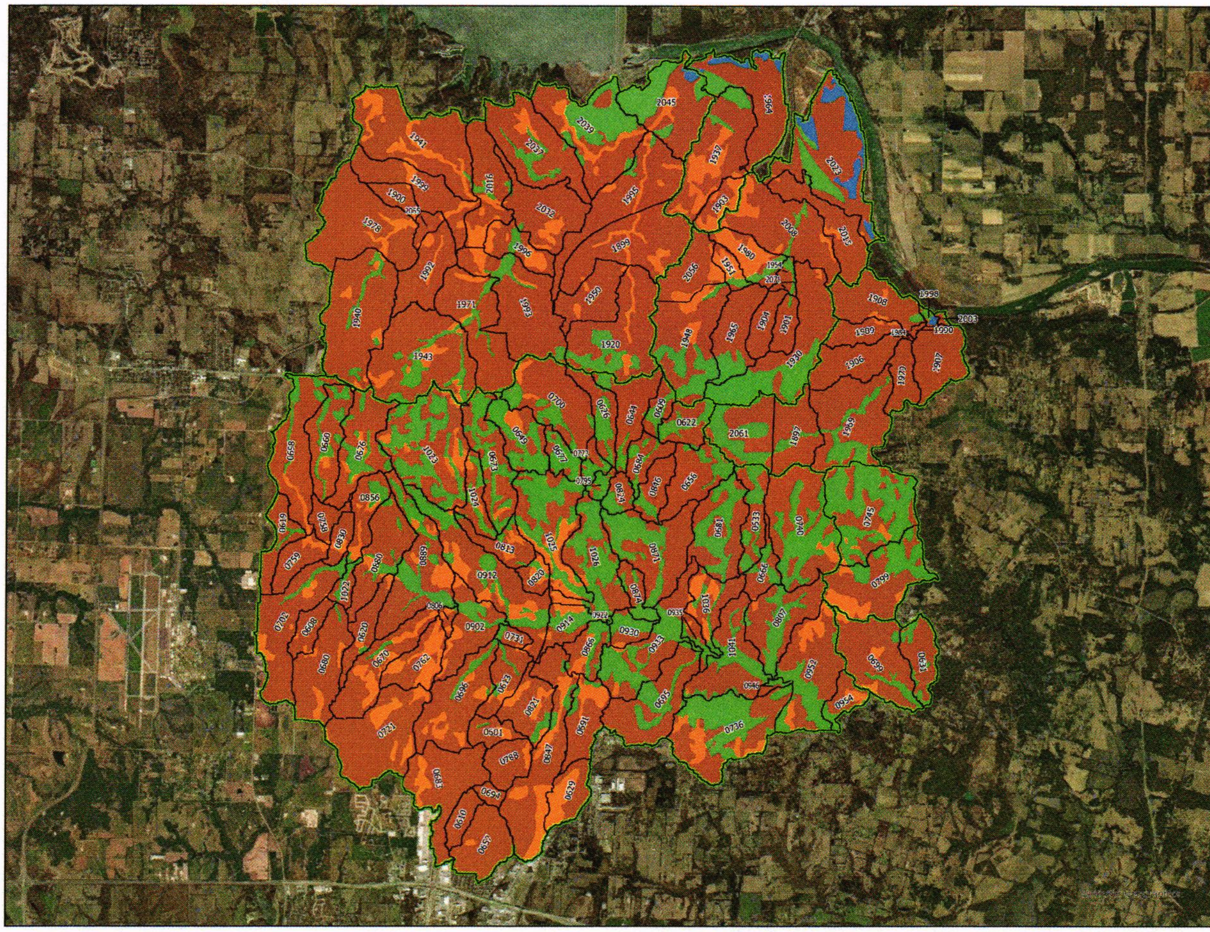
Appendix D
Figure 4

Land Cover Map

0 0.5 1 2 Miles

HUITT-ZOLLARS
5430 LBJ Freeway, Suite 1500
Dallas, Texas 75240

Figure 5: Soils Map



CITY OF DENISON
300 W Main St
Denison, TX 75020

Denison TEXAS

City of Denison
Conceptual Stormwater Master Plan
R315880.01

Legend
- Sub-Basins
- Basins

Hydrologic Soil Group
- A
- B
- B/D
- C
- C/D
- D

N

Appendix D
Figure 5

Soils Map

0 0.5 1 2 Miles

HUITT-ZOLLARS
5430 LBJ Freeway, Suite 1500
Dallas, Texas 75240

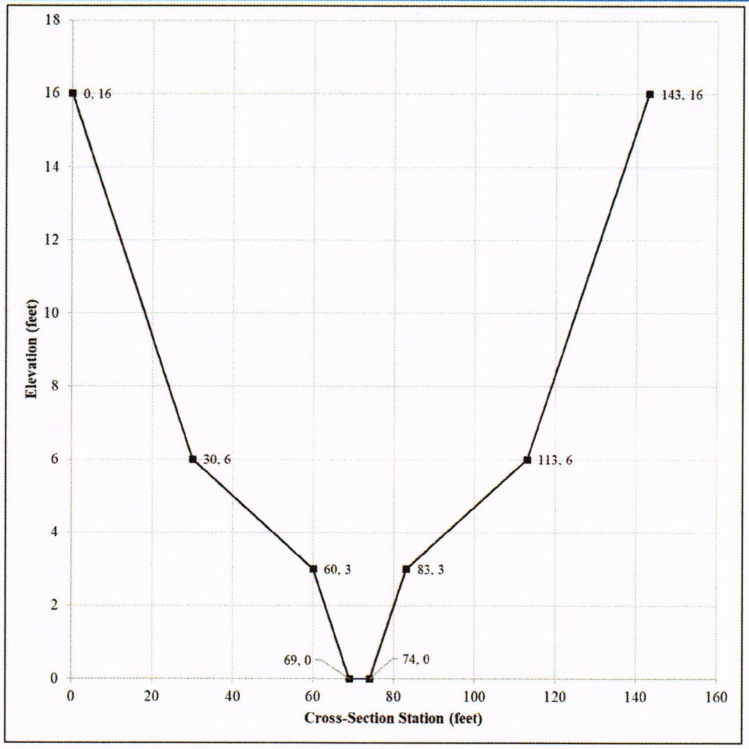
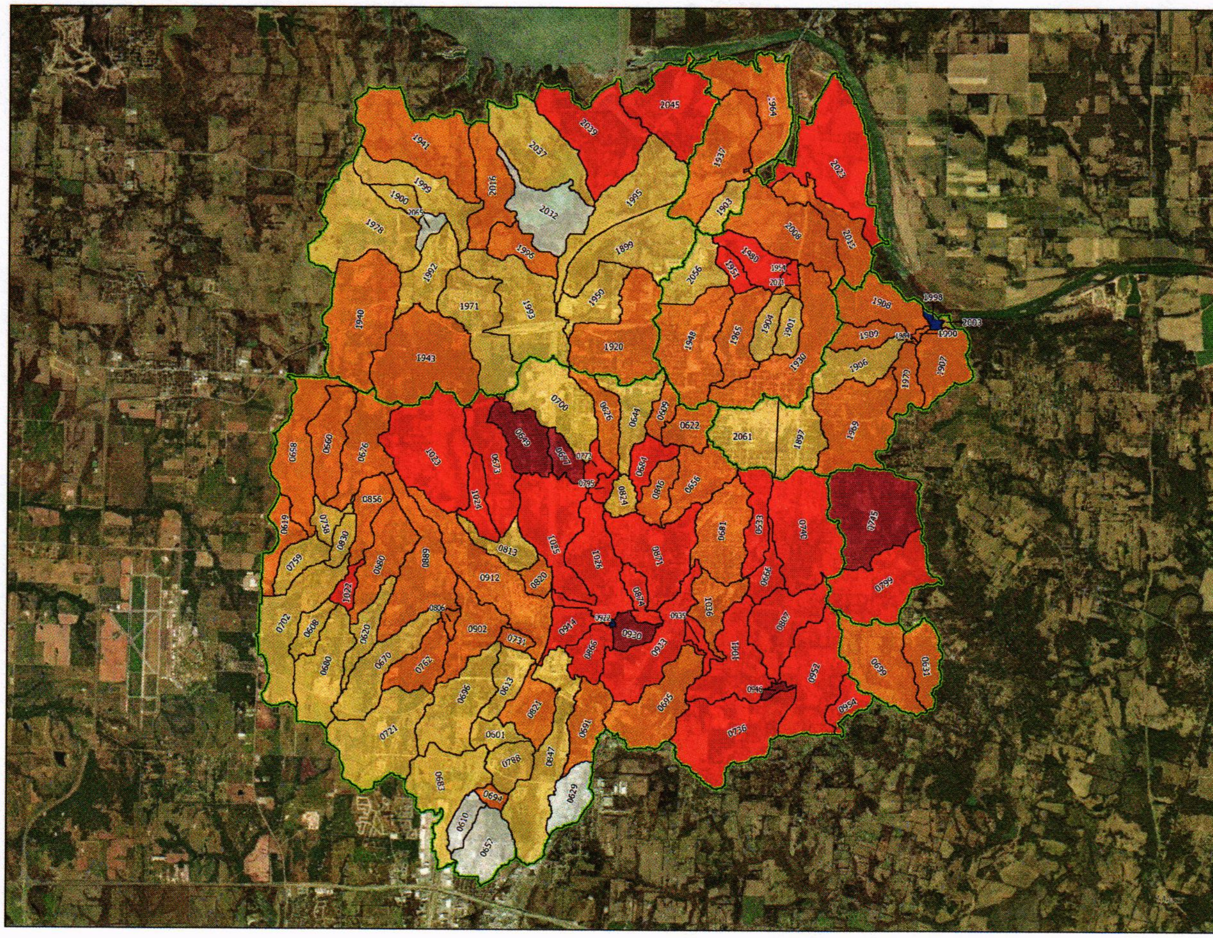


Figure 6: Eight-Point Muskingum-Cunge Cross-Section

Figure 7: Assigned Curve Number Map



CITY OF DENISON
300 W Main St
Denison, TX 75020

Denison TEXAS

City of Denison
Conceptual Stormwater Master Plan
R315880.01

Legend
- Sub-Basins
- Basins

CN
- 63 - 66
- 67 - 70
- 71 - 74
- 75 - 78
- 79 - 82
- 83 - 86
- 87 - 91

N

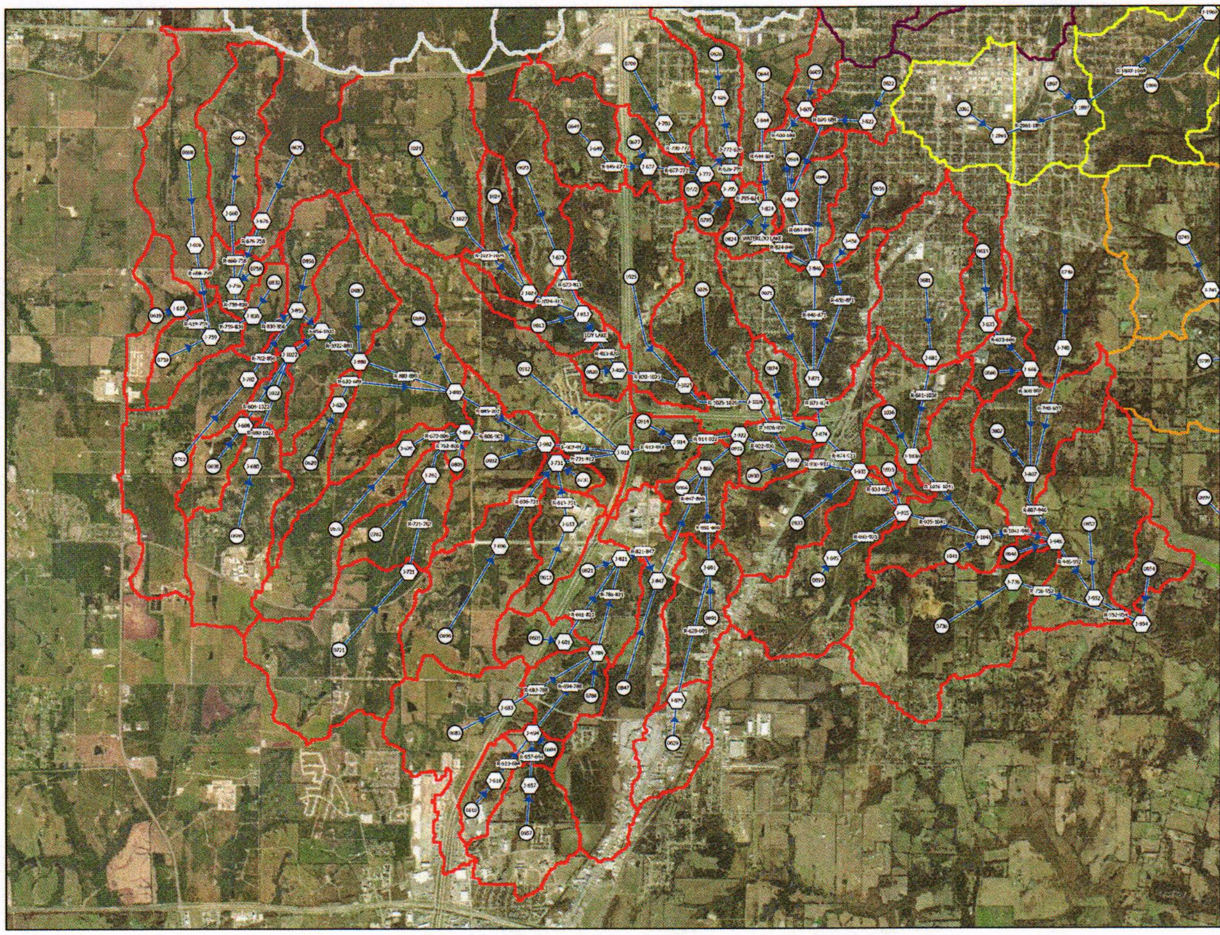
Appendix D
Figure 7

Assigned Curve Number Map

0 0.5 1 2 Miles

HUITT-ZOLLARS
5430 LBJ Freeway, Suite 1500
Dallas, Texas 75240

Attachment 1: Drainage Basin Input Schematics



CITY OF DENISON

300 W Main St
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Denison
TEXAS

City of Denison
Conceptual Stormwater Master Plan
R315880.01

Legend

- Junction
- Subbasin
- Reach
- Reservoir
- Network

Subbasin Boundaries

Basin

- DUCK CREEK
- IRON ORE CREEK
- PAWPAW CREEK
- SHAWNEE CREEK
- UNT0097
- UNT140
- UNT141

N

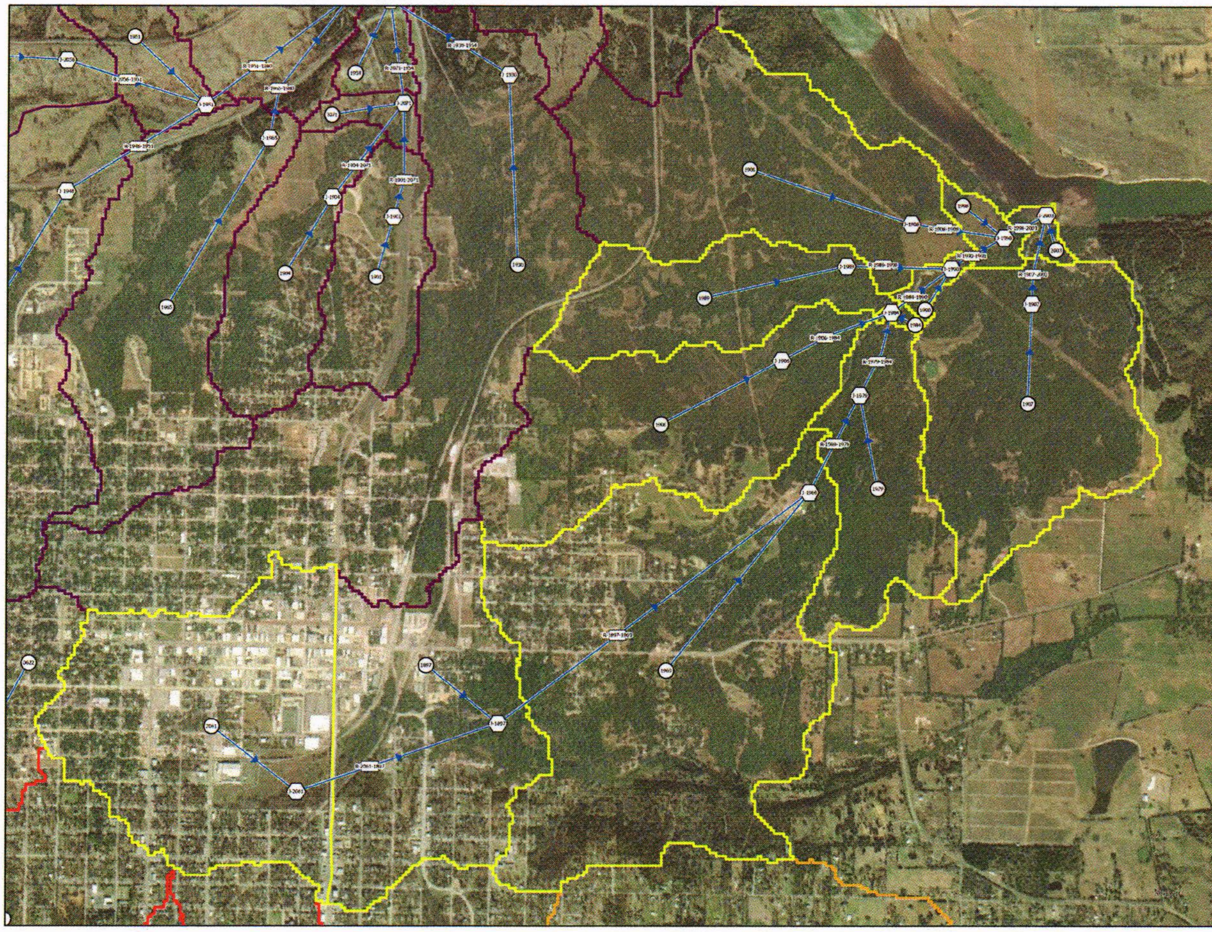
Attachment 1-A

Iron Ore Creek HEC-HMS Schematic

0 0.5 1 Miles

HUITT-ZOLLARS

5430 LBJ Freeway, Suite 1500
Dallas, Texas 75240



CITY OF DENISON
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Denison
TEXAS

City of Denison
Conceptual Stormwater Master Plan
R315880.01

Legend
- Junction
- Subbasin
- Reach
- Network
- Reservoir

Subbasin Boundaries

Basin
- DUCK CREEK
- IRON ORE CREEK
- PAWPAW CREEK
- SHAWNEE CREEK
- UNT0097
- UNT140
- UNT141

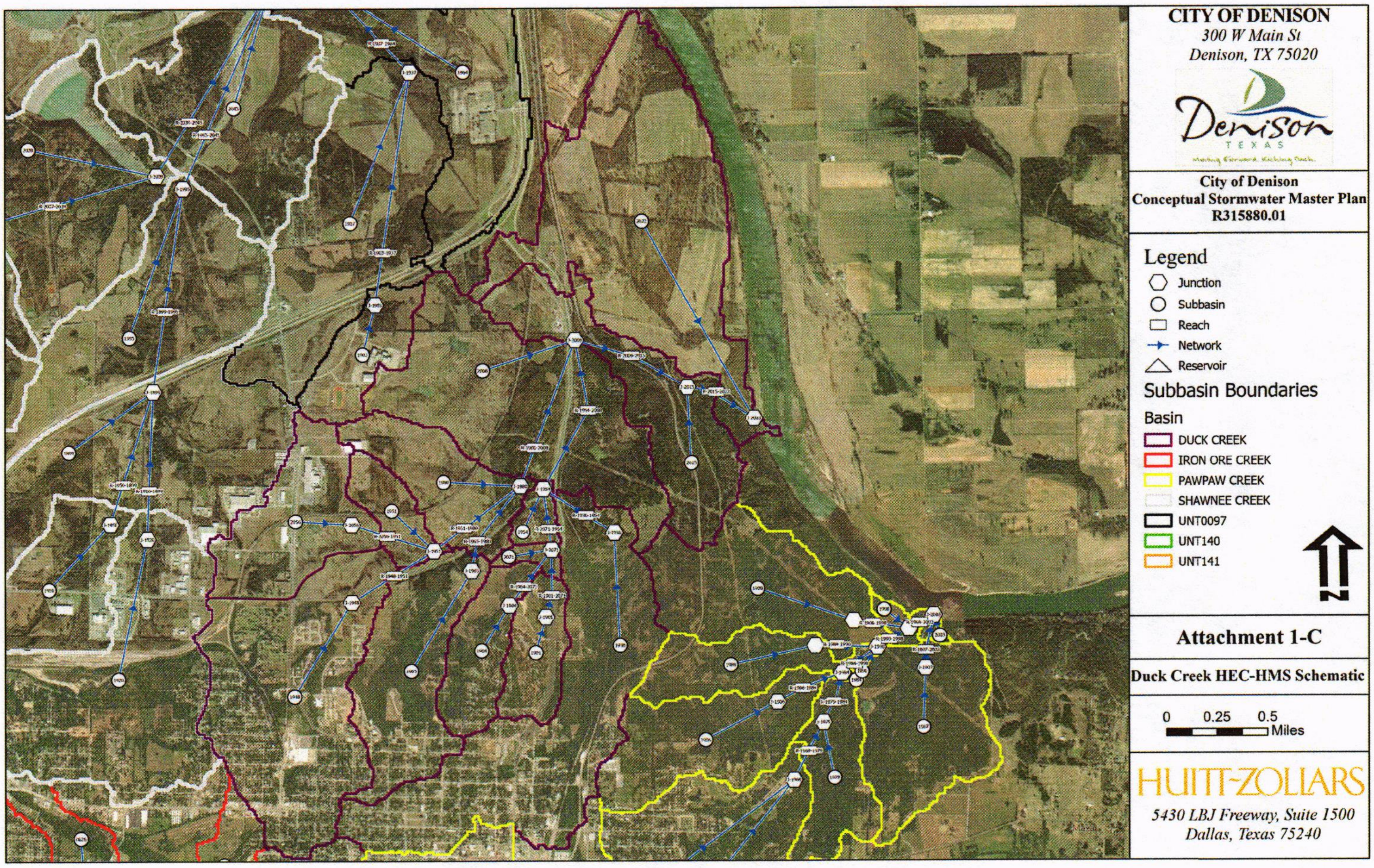
N

0 0.25 0.5 Miles

Attachment 1-B

Pawpaw Creek HEC-HMS Schematic

HUITT-ZOLLARS
5430 LBJ Freeway, Suite 1500
Dallas, Texas 75240


CITY OF DENISON
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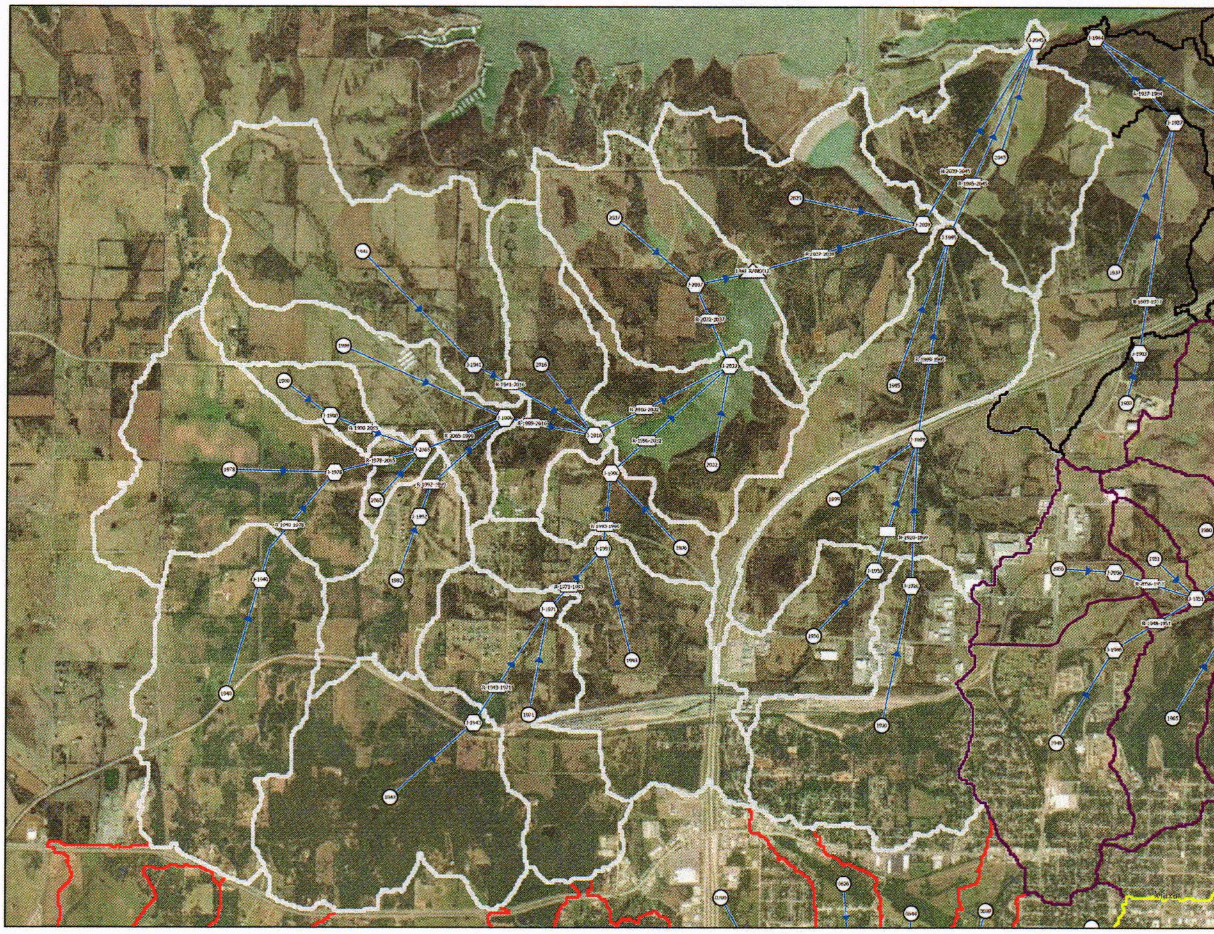
Denison
TEXAS
City of Denison
Conceptual Stormwater Master Plan
R315880.01
Legend
Junction
Subbasin
Reach
Network
Reservoir
Subbasin Boundaries
Basin
DUCK CREEK
IRON ORE CREEK
PAWPAW CREEK
SHAWNEE CREEK
UNT0097
UNT140
UNT141
N
Attachment 1-C
Duck Creek HEC-HMS Schematic
0 0.25 0.5
Miles
HUITT-ZOLLARS
5430 LBJ Freeway, Suite 1500
Dallas, Texas 75240



CITY OF DENISON
300 W Main St
Denison, TX 75020

City of Denison
Conceptual Stormwater Master Plan
R315880.01

Legend
- Junction
- Subbasin
- Reach
- Network
- Reservoir

Subbasin Boundaries

Basin
- DUCK CREEK
- IRON ORE CREEK
- PAWPAW CREEK
- SHAWNEE CREEK
- UNT0097
- UNT140
- UNT141

N

Exhibit 1-D

Shawnee Creek HEC-HMS Schematic

0 0.25 0.5 Miles

HUITT-ZOLLARS
5430 LBJ Freeway, Suite 1500
Dallas, Texas 75240

CITY OF DENISON

300 W Main St

Denison, TX 75020

City of Denison

Conceptual Stormwater Master Plan

R315880.01

Legend

Junction

Subbasin

Reach

Network

Reservoir

Subbasin Boundaries

Basin

DUCK CREEK

IRON ORE CREEK

PAWPAW CREEK

SHAWNEE CREEK

UNT0097

UNT140

UNT141

N

Attachment 1-E

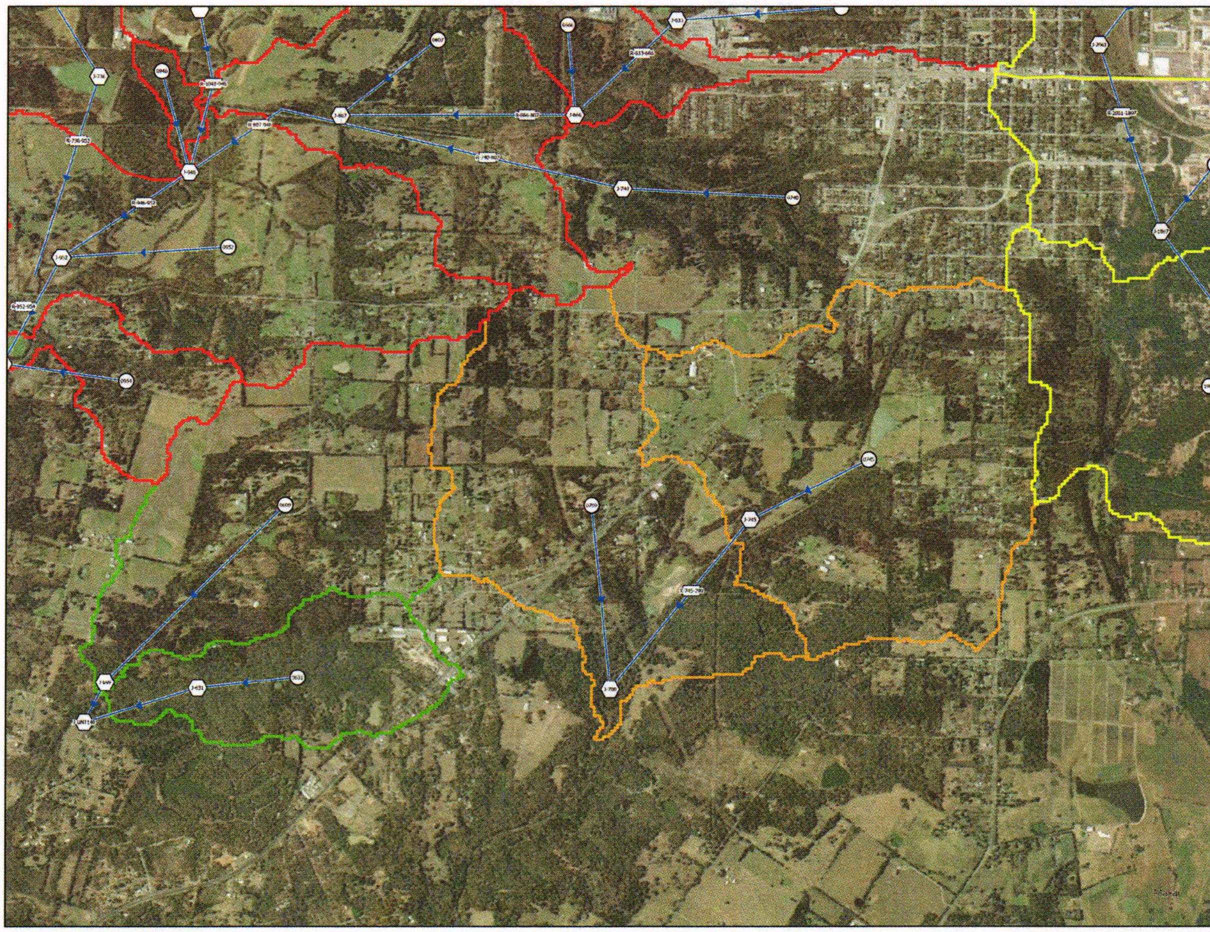
UNT0097 HEC-HMS Schematic

0 0.25 0.5 Miles

HUITT-ZOLLARS

5430 LBJ Freeway, Suite 1500

Dallas, Texas 75240



CITY OF DENISON
300 W Main St
Denison, TX 75020

City of Denison
Conceptual Stormwater Master Plan
R315880.01

Legend

- Junction
- Subbasin
- Reach
- Network
- Reservoir

Subbasin Boundaries

Basin

- DUCK CREEK
- IRON ORE CREEK
- PAWPAW CREEK
- SHAWNEE CREEK
- UNT0097
- UNT140
- UNT141

N

Attachment 1-F

UNT140 & UNT141
HEC-HMS Schematic

0 0.25 0.5 Miles

HUITT-ZOLLARS
5430 LBJ Freeway, Suite 1500
Dallas, Texas 75240

Attachment 2: Sub-Basin Input – Areas

City of Denison Conceptual Stormwater Master Plan (SWMP)
Attachment 2
Sub-Basin Input - Areas

Sub-Basin	Area (Square Mile)
Duck Creek	
1901	0.188
1904	0.226
1930	1.023
1948	0.932
1951	0.135
1954	0.056
1965	0.489
1980	0.326
2008	0.909
2015	0.528
2023	1.250
2056	0.415
2071	0.035
Pawpaw Creek	
1897	0.471
1906	0.441
1907	0.488
1908	0.520
1969	0.945
1979	0.255
1984	0.007
1989	0.241
1990	0.020
1998	0.035
2003	0.019
2061	0.586
Shawnee Creek	
1899	0.927
1900	0.159
1920	1.011
1940	1.160
1941	1.160
1943	1.225
1950	0.447
1971	0.820
1978	1.082
1992	0.484
1993	1.000
1995	0.872
1996	0.311
1999	0.580
2016	0.570
2032	0.655
2037	0.872
2039	1.046
2045	0.844
2065	0.094
UNT0097	
1903	0.220
1937	0.913
1964	0.809
UNT140	
631	0.347
699	0.790
UNT141	
745	1.079
799	0.879

Sub-Basin	Area (Square Mile)
Iron Ore Creek	
1022	0.113
1023	1.178
1024	0.254
1025	0.879
1026	0.723
1036	0.362
1041	0.698
601	0.187
608	0.198
609	0.162
610	0.175
613	0.297
619	0.271
620	0.278
622	0.288
626	0.314
629	0.324
633	0.360
644	0.438
649	0.451
656	0.488
657	0.477
660	0.515
666	0.209
670	0.601
673	0.614
676	0.635
677	0.186
680	0.668
681	0.687
683	0.755
684	0.233
691	0.408
694	0.079
695	0.754
696	0.757
698	0.765
700	0.793
702	0.838
721	1.001
731	0.089
736	1.097
740	1.060
758	0.156
759	0.262
762	0.393
772	0.112
788	0.286
795	0.083
806	0.054
807	0.637
813	0.282
820	0.139
821	0.399
824	0.152
830	0.149
846	0.252
847	1.002
856	0.310
866	0.276
871	0.842
874	0.182
880	0.578
889	0.883
902	0.497
912	0.983
914	0.252
922	0.006
930	0.181
933	0.760
935	0.111
946	0.044
952	0.894
954	0.201

Attachment 3: Sub-Basin Input – Design Storm IDF Data

Attachment 3
Sub-Basin Input - Design Storm IDF Data

Drainage Basin Precipitation Inputs
Frequency Storm
From NOAA Atlas 14 Texas Point Frequency Grids

Basin Duration	Iron Ore Creek 100-year Depth (inch/1000)	UNT140 100-year Depth (inch/1000)	UNT141 100-year Depth (inch/1000)	Pawpaw Creek 100-year Depth (inch/1000)	Shawnee Creek 100-year Depth (inch/1000)	Duck Creek 100-year Depth (inch/1000)	UNT0097 100-year Depth (inch/1000)
5min	1015	1003	1006	1008	1018	1011	1014
15min	2009	1987	1992	1997	2016	2004	2009
1h	3658	3626	3634	3642	3677	3653	3663
2h	4850	4789	4802	4815	4871	4833	4845
3h	5663	5578	5597	5613	5683	5637	5648
6h	7067	6962	6984	7004	7086	7031	7040
12h	8421	8341	8356	8369	8438	8390	8395
24h	9806	9764	9769	9772	9819	9783	9784

Basin Duration	Iron Ore Creek 100-year Depth (inch)	UNT140 100-year Depth (inch)	UNT141 100-year Depth (inch)	Pawpaw Creek 100-year Depth (inch)	Shawnee Creek 100-year Depth (inch)	Duck Creek 100-year Depth (inch)	UNT0097 100-year Depth (inch)
5min	1.015	1.003	1.006	1.008	1.018	1.011	1.014
15min	2.009	1.987	1.992	1.997	2.016	2.004	2.009
1h	3.658	3.626	3.634	3.642	3.677	3.653	3.663
2h	4.850	4.789	4.802	4.815	4.871	4.833	4.845
3h	5.663	5.578	5.597	5.613	5.683	5.637	5.648
6h	7.067	6.962	6.984	7.004	7.086	7.031	7.040
12h	8.421	8.341	8.356	8.369	8.438	8.390	8.395
24h	9.806	9.764	9.769	9.772	9.819	9.783	9.784

Attachment 3
Sub-Basin Input - Design Storm IDF Data

Drainage Basin Precipitation Inputs
Frequency Storm
From NOAA Atlas 14 Texas Point Frequency Grids

Basin / Duration	Iron Ore Creek 10-year Depth (inch/1000)	UNT140 10-year Depth (inch/1000)	UNT141 10-year Depth (inch/1000)	Pawpaw Creek 10-year Depth (inch/1000)	Shawnee Creek 10-year Depth (inch/1000)	Duck Creek 10-year Depth (inch/1000)	UNT0097 10-year Depth (inch/1000)
5min	692	691	692	692	692	692	692
15min	1378	1378	1378	1378	1376	1377	1376
1h	2499	2500	2501	2502	2499	2500	2500
2h	3170	3173	3173	3173	3168	3169	3167
3h	3600	3604	3603	3601	3597	3596	3593
6h	4390	4396	4394	4392	4381	4383	4377
12h	5281	5289	5288	5287	5265	5276	5268
24h	6255	6256	6257	6213	6249	6256	6189

Basin / Duration	Iron Ore Creek 10-year Depth (inch)	UNT140 10-year Depth (inch)	UNT141 10-year Depth (inch)	Pawpaw Creek 10-year Depth (inch)	Shawnee Creek 10-year Depth (inch)	Duck Creek 10-year Depth (inch)	UNT0097 10-year Depth (inch)
5min	0.692	0.691	0.692	0.692	0.692	0.692	0.692
15min	1.378	1.378	1.378	1.378	1.376	1.377	1.376
1h	2.499	2.5	2.501	2.502	2.499	2.5	2.5
2h	3.17	3.173	3.173	3.173	3.168	3.169	3.167
3h	3.6	3.604	3.603	3.601	3.597	3.596	3.593
6h	4.39	4.396	4.394	4.392	4.381	4.383	4.377
12h	5.281	5.289	5.288	5.287	5.265	5.276	5.268
24h	6.255	6.256	6.257	6.213	6.249	6.256	6.189

Attachment 4: Sub-Basin Input – Curve Number

City of Denison Stormwater Master Plan (SWMP)
Attachment 4
Sub-Basin Input - Curve Number

Sub-Basin	Composite Curve Number
Duck Creek	
1901	85.16
1904	83.64
1930	79.13
1948	82.45
1951	76.95
1954	76.55
1965	81.60
1980	76.91
2008	82.12
2015	82.19
2023	77.34
2056	86.16
2071	81.79
Pawpaw Creek	
1897	85.23
1906	83.20
1907	80.62
1908	80.81
1969	80.09
1979	80.54
1984	79.44
1989	81.10
1990	80.63
1998	66.49
2003	85.42
2061	84.84
Shawnee Creek	
1899	85.47
1900	82.84
1920	82.07
1940	82.45
1941	82.28
1943	80.01
1950	86.00
1971	82.95
1978	83.82
1992	85.01
1993	83.66
1995	84.95
1996	81.93
1999	83.28
2016	81.75
2032	87.58
2037	83.14
2039	78.10
2045	76.10
2065	89.76
UNT0097	
1903	84.68
1937	82.04
1964	79.94
UNT140	
631	79.06
699	80.09
UNT141	
745	72.57
799	77.85

Sub-Basin	Composite Curve Number
Iron Ore Creek	
1022	76.84
1023	76.03
1024	77.77
1025	74.90
1026	75.81
1036	79.96
1041	77.86
601	83.32
608	84.33
609	79.31
610	86.75
613	83.81
619	81.34
620	83.85
622	79.12
626	80.32
629	90.52
633	78.25
644	84.16
649	72.65
656	80.92
657	87.07
660	79.34
666	78.36
670	82.92
673	75.43
676	79.61
677	73.54
680	85.31
681	79.98
683	86.09
684	78.34
691	82.37
694	80.14
695	79.66
696	83.34
698	81.32
700	84.13
702	83.15
721	84.78
731	79.71
736	75.05
740	76.53
758	83.49
759	82.81
762	79.65
772	77.22
788	83.26
795	75.29
806	82.01
807	77.25
813	84.73
820	79.56
821	81.53
824	82.98
830	82.70
846	79.64
847	84.65
856	78.90
866	75.66
871	78.31
874	77.78
880	81.20
889	78.82
902	80.54
912	79.03
914	75.63
922	63.35
930	73.86
933	77.64
935	78.48
946	73.09
952	77.34
954	77.20

Attachment 5: Sub-Basin Input – Snyder Unit Hydrograph

City of Denison Stormwater Master Plan (SWMP)
Attachment 5
Sub-Basin Input - Snyder Unit Hydrograph

Reference: iSWM Technical Manual – Hydrology
City of Denison – Storm Drainage System Design Manual, April 2017

$$t_p = C_t (L \cdot L_{ca})^{0.3}$$

Sub-basin ID	Area [sq mi]	L [ft]	L [mi]	Top Elevation [ft]	Bottom Elevation [ft]	Lca [ft]	Lca [mi]	Slope [ft/ft]	Basin Slope Characteristic	Ct	Tp [hr]	Cp
Duck Creek												
1901	0.1880	4838.8	0.916	752.7	600.0	2768.6	0.524	0.032	Steep Basin Slope	0.539	0.433	0.619
1904	0.2263	6225.3	1.179	757.4	599.3	3467.5	0.657	0.025	Steep Basin Slope	0.549	0.508	0.611
1930	1.0227	16699.7	3.163	788.8	591.3	5563.2	1.054	0.012	Steep Basin Slope	0.533	0.765	0.624
1948	0.9316	11577.7	2.193	794.3	613.9	6556.2	1.242	0.016	Steep Basin Slope	0.521	0.704	0.633
1951	0.1351	4930.6	0.934	701.6	593.0	2713.5	0.514	0.022	Steep Basin Slope	0.548	0.440	0.611
1954	0.0558	3151.7	0.597	652.8	558.9	1323.8	0.251	0.030	Steep Basin Slope	0.546	0.309	0.613
1965	0.4893	9539.0	1.807	779.9	596.6	4331.6	0.820	0.019	Steep Basin Slope	0.542	0.610	0.616
1980	0.3263	7888.7	1.494	711.9	558.9	4522.1	0.856	0.019	Steep Basin Slope	0.546	0.588	0.613
2008	0.9088	7801.0	1.477	681.6	535.9	2554.5	0.484	0.019	Steep Basin Slope	0.547	0.494	0.613
2015	0.5280	8111.8	1.536	553.0	505.9	2724.3	0.516	0.006	Moderate Basin Slope	0.597	0.557	0.583
2023	1.2499	12619.2	2.390	534.6	498.9	7760.1	1.470	0.003	Flat Basin Slope	0.649	0.946	0.551
2056	0.4149	6573.7	1.245	707.0	616.7	3055.3	0.579	0.014	Steep Basin Slope	0.521	0.472	0.634
2071	0.0354	2071.1	0.392	659.7	588.3	786.2	0.149	0.034	Steep Basin Slope	0.543	0.231	0.616
Pawpaw Creek												
1897	0.4712	5935.3	1.124	752.0	651.6	2232.3	0.423	0.017	Steep Basin Slope	0.499	0.399	0.651
1906	0.4412	9074.1	1.719	740.5	534.8	5533.2	1.048	0.023	Steep Basin Slope	0.547	0.652	0.613
1907	0.4885	7853.3	1.487	708.7	503.1	3558.4	0.674	0.026	Steep Basin Slope	0.550	0.550	0.610
1908	0.5204	8453.2	1.601	677.5	522.8	4682.9	0.887	0.018	Steep Basin Slope	0.550	0.611	0.610
1969	0.9447	11584.6	2.194	763.0	559.9	7086.4	1.342	0.018	Steep Basin Slope	0.547	0.756	0.613
1979	0.2553	8450.1	1.600	718.8	527.8	5315.9	1.007	0.023	Steep Basin Slope	0.550	0.635	0.610
1984	0.0065	819.8	0.155	627.3	522.0	258.5	0.049	0.129	Steep Basin Slope	0.550	0.127	0.610
1989	0.2411	6679.6	1.265	728.0	532.7	3402.5	0.644	0.029	Steep Basin Slope	0.550	0.517	0.610
1990	0.0196	2012.8	0.381	630.2	508.4	963.6	0.183	0.060	Steep Basin Slope	0.550	0.247	0.610
1998	0.0353	2044.2	0.387	587.0	501.6	1083.7	0.205	0.042	Steep Basin Slope	0.550	0.257	0.610
2003	0.0194	1122.7	0.213	621.6	497.7	271.5	0.051	0.110	Steep Basin Slope	0.550	0.142	0.610
2061	0.5859	6407.6	1.214	780.6	719.7	3069.2	0.581	0.010	Steep Basin Slope	0.463	0.417	0.679
Shawnee Creek												
1899	0.9269	9801.9	1.856	737.7	588.4	3203.6	0.607	0.015	Steep Basin Slope	0.534	0.554	0.622
1900	0.1586	5514.7	1.044	751.4	674.3	3078.1	0.583	0.014	Steep Basin Slope	0.550	0.474	0.610
1920	1.0109	9317.2	1.765	791.0	628.5	5457.3	1.034	0.017	Steep Basin Slope	0.527	0.631	0.628
1940	1.1602	13621.2	2.580	890.8	689.2	6807.3	1.289	0.015	Steep Basin Slope	0.549	0.787	0.611
1941	1.1602	15300.6	2.898	799.3	619.8	7765.4	1.471	0.012	Steep Basin Slope	0.550	0.849	0.610
1943	1.2253	9098.7	1.723	900.6	689.6	4646.1	0.880	0.023	Steep Basin Slope	0.550	0.623	0.610
1950	0.4467	15101.9	2.860	796.5	628.5	3958.4	0.750	0.011	Steep Basin Slope	0.513	0.645	0.639
1971	0.8201	10896.4	2.064	910.3	645.6	4720.9	0.894	0.024	Steep Basin Slope	0.547	0.658	0.612
1978	1.0824	10860.6	2.057	767.9	671.1	6206.9	1.176	0.009	Steep Basin Slope	0.550	0.716	0.610
1992	0.4840	8973.6	1.700	825.1	652.9	4509.0	0.854	0.019	Steep Basin Slope	0.549	0.614	0.611
1993	1.0000	12290.5	2.328	818.5	628.7	5727.9	1.085	0.015	Steep Basin Slope	0.529	0.698	0.627
1995	0.8716	17358.7	3.288	740.8	537.8	7554.5	1.431	0.012	Steep Basin Slope	0.543	0.864	0.616
1996	0.3107	5630.7	1.066	725.3	615.7	2100.6	0.398	0.019	Steep Basin Slope	0.549	0.425	0.610
1999	0.5802	13544.0	2.565	784.3	619.8	6107.4	1.157	0.012	Steep Basin Slope	0.545	0.755	0.614
2016	0.5701	7853.6	1.487	730.9	615.7	3098.3	0.587	0.015	Steep Basin Slope	0.548	0.526	0.612
2032	0.6547	6309.8	1.195	707.4	615.7	2761.3	0.523	0.015	Steep Basin Slope	0.550	0.478	0.610
2037	0.8724	11455.9	2.170	731.7	615.7	2831.3	0.536	0.010	Steep Basin Slope	0.550	0.575	0.610
2039	1.0464	12225.7	2.315	698.1	537.7	4821.7	0.913	0.013	Steep Basin Slope	0.549	0.687	0.611
2045	0.8437	9515.8	1.802	601.5	503.9	4000.6	0.758	0.010	Steep Basin Slope	0.548	0.602	0.612
2065	0.0943	3373.3	0.639	723.7	649.2	1434.0	0.272	0.022	Steep Basin Slope	0.550	0.325	0.610
UNT 0097												
1903	0.2199	4323.9	0.819	720.2	623.1	2329.5	0.441	0.022	Steep Basin Slope	0.517	0.381	0.636
1937	0.9129	14665.0	2.777	701.9	526.5	8336.7	1.579	0.012	Steep Basin Slope	0.544	0.848	0.615
1964	0.8090	9433.4	1.787	637.9	503.5	2178.2	0.413	0.014	Steep Basin Slope	0.533	0.486	0.624
UNT140												
631	0.3467	6527.8	1.236	669.4	564.0	3654.3	0.692	0.016	Steep Basin Slope	0.546	0.521	0.614
699	0.7896	10543.1	1.997	693.5	564.0	5349.5	1.013	0.012	Steep Basin Slope	0.550	0.679	0.610
UNT141												
745	1.0793	8269.5	1.566	783.4	629.9	3323.7	0.629	0.019	Steep Basin Slope	0.549	0.546	0.611
799	0.8786	11117.4	2.106	699.4	591.4	5461.9	1.034	0.010	Steep Basin Slope	0.549	0.694	0.611
Iron Ore Creek												
1022	0.1134	4759.9	0.902	748.9	668.3	2741.7	0.519	0.017	Steep Basin Slope	0.550	0.438	0.610
1023	1.1782	10387.2	1.967	883.6	696.7	5585.1	1.058	0.018	Steep Basin Slope	0.549	0.684	0.611
1024	0.2535	7819.3	1.481	807.1	674.7	3522.6	0.667	0.017	Steep Basin Slope	0.546	0.544	0.613
1025	0.8785	12842.5	2.432	790.9	619.8	8884.9	1.683	0.013	Steep Basin Slope	0.544	0.830	0.615
1026	0.7230	10410.7	1.972	741.7	609.8	4776.0	0.905	0.013	Steep Basin Slope	0.542	0.645	0.616
1036	0.3616	8724.6	1.652	696.9	591.9	4805.2	0.910	0.012	Steep Basin Slope	0.540	0.611	0.618
1041	0.6977	10725.9	2.031	701.5	579.8	5775.5	1.094	0.011	Steep Basin Slope	0.550	0.699	0.610
601	0.1874	4902.0	0.928	825.2	680.0	2893.1	0.548	0.030	Steep Basin Slope	0.550	0.449	0.610
608	0.1983	6008.0	1.138	849.7	710.1	3229.6	0.612	0.023	Steep Basin Slope	0.550	0.493	0.610
609	0.1621	5108.9	0.968	794.6	702.8	2303.6	0.436	0.018	Steep Basin Slope	0.535	0.413	0.622
610	0.1755	5020.7	0.951	834.0	708.1	2911.5	0.551	0.025	Steep Basin Slope	0.502	0.413	0.649
613	0.2969	7051.9	1.336	807.7	639.5	3537.8	0.670	0.024	Steep Basin Slope	0.520	0.503	0.634
619	0.2713	6288.0	1.191	769.0	697.9	2175.0	0.412	0.011	Steep Basin Slope	0.549	0.444	0.611
620	0.2780	10092.8	1.912	862.7	670.3	4773.8	0.904	0.019	Steep Basin Slope	0.549	0.648	0.610
622	0.2885	5491.1	1.040	796.7	720.1	2334.1	0.442	0.014	Steep Basin Slope	0.534	0.423	0.623
626	0.3140	8996.2	1.704	812.4	678.1	4252.4	0.805	0.015	Steep Basin Slope	0.531	0.583	0.625
629	0.3238	6120.5	1.159	800.7	712.7	2874.8	0.544	0.014	Steep Basin Slope	0.439	0.382	0.699
633	0.3600	8014.9	1.518	763.6	635.3	3775.6	0.715	0.016	Steep Basin Slope	0.524	0.537	0.631
644	0.4381	8216.2	1.556	791.3	678.1	4756.3	0.901	0.014	Steep Basin Slope	0.527	0.583	0.629
649	0.4508	7603.4	1.440	881.3	712.5	3611.3	0.684	0.022	Steep Basin Slope	0.549	0.547	0.611
656	0.4877	9143.4	1.732	797.7	638.1	4853.9	0.919	0.017	Steep Basin Slope	0.542	0.623	0.617
657	0.4770	6914.3	1.310	842.3	706.2	3663.2	0.694	0.020	Steep Basin Slope	0.511	0.497	0.641
660	0.5155	13327.8	2.524	909.0	710.4	6024.2	1.141	0.015	Steep Basin Slope	0.550	0.755	0.610
666	0.2095	6283.5	1.190	732.3	616.3	1892.8	0.358	0.018	Steep Basin Slope	0.529	0.410	0.627
670	0.6015	14322.8	2.713	886.2	654.0	8397.2	1.590	0.016	Steep Basin Slope	0.548	0.849	0.612
673	0.6145	12021.0	2.277	908.2	673.6	6032.7	1.143	0.020	Steep Basin Slope	0.547	0.729	0.612
676	0.6354	12092.7	2.290	909.8	711.3	6909.0	1.309	0.016	Steep Basin Slope	0.549	0.764	0.611
677	0.1862	4483.1	0.849	824.6	688.6	1528.1	0.289	0.030	Steep Basin Slope	0.538	0.353	0.620
680	0.6676	12594.4	2.385	878.2	706.3	7169.7	1.358	0.014	Steep Basin Slope	0.536	0.762	0.621
681	0.6872	8223.7	1.558	768.0	647.3	4679.7	0.886	0.015	Steep Basin Slope	0.514	0.566	0.639
683	0.7551	8291.9	1.570	852.5	695.0	4588.8	0.869	0.019	Steep Basin Slope	0.529	0.580	0.627
684	0.2326	6295.6	1.192	784.7	650.2	3433.8	0.650	0.021	Steep Basin Slope	0.548	0.508	0.611
691	0.4082	7425.9	1.406	760.3	638.5	3417.7	0.647	0.016	Steep Basin Slope	0.517	0.503	0.636
694	0.0787	2582.7	0.489	811.3	695.5	1028.6	0.195	0.045	Steep Basin Slope	0.550	0.272	0.610
695	0.7539	11534.7	2.185	724.6	594.5	5773.2	1.093	0.011	Steep Basin Slope	0.538	0.699	0.619
696	0.7570	11595.4	2.196	853.8	639.1	7083.2	1.342	0.019	Steep Basin Slope	0.549	0.759	0.611
698	0.7646	15893.9	3.010	853.3	696.5	8382.0	1.587	0.010	Steep Basin Slope	0.550	0.879	0.610
700	0.7927	8776.6	1.662	873.4	688.6	5068.2	0.960	0.021	Steep Basin Slope	0.506	0.582	0.645
702	0.8378	17815.4	3.374	871.9	680.3	9594.7	1.817	0.011	Steep Basin Slope	0.548	0.944	0.612
721	1.0005	10644.8	2.016	874.1	709.4	6156.3	1.166	0.015	Steep Basin Slope	0.546	0.706	0.613
731	0.0886	2067.9	0.392	728.3	628.7	1211.4	0.229	0.048	Steep Basin Slope	0.540	0.262	0.618
736	1.0975	11935.0	2.260	700.6	584.1	6305.5	1.194	0.010	Steep Basin Slope	0.549	0.740	0.611
740	1.0601	10239.7	1.939	758.0	616.4	4944.9	0.937	0.014	Steep Basin Slope	0.536	0.641	0.621
758	0.1563	5378.6	1.019	739.9	682.5	2861.8	0.542	0.011	Steep Basin Slope	0.550	0.460	0.610
759	0.2622	6826.5	1.293	767.4	680.6	3428.5	0.649	0.013	Steep Basin Slope	0.550	0.522	0.610
762	0.3930	7397.7	1.401	813.3	654.1	4502.3	0.853	0.022	Steep Basin Slope	0.550	0.580	0.610
772	0.1121	4264.4	0.808	743.6	678.1	1624.4	0.308	0.015	Steep Basin Slope	0.547	0.360	0.612
788	0.2856	5717.3	1.083	809.4	667.8	3196.0	0.605	0.025	Steep Basin Slope	0.548	0.483	0.612
795	0.0833	2188.9	0.415	747.4	678.1	1099.8	0.208	0.032	Steep Basin Slope	0.545	0.261	0.614
806	0.0536	3024.0	0.573	718.5	648.3	1339.9	0.254	0.023	Steep Basin Slope	0.550	0.308	0.610
807	0.6366	10271.2	1.945	701.2	579.9	4307.5	0.816	0.012	Steep Basin Slope	0.550	0.632	0.610
813	0.2823	7425.0	1.406	741.7	670.8	2522.2	0.478	0.010	Steep Basin Slope	0.545	0.484	0.614
820	0.1385	3303.0	0.626	697.5	641.5	1373.5	0.260	0.017	Steep Basin Slope	0.541	0.314	0.617
821	0.3985	8132.4	1.540	754.3	637.2	3521.7	0.667	0.014	Steep Basin Slope	0.539	0.544	0.618
824	0.1519	4645.2	0.880	729.7	652.2	2542.7	0.482	0.017	Steep Basin Slope	0.548	0.424	0.611
830	0.1491	3282.4	0.622	739.8	678.6	1624.4	0.308	0.019	Steep Basin Slope	0.550	0.335	0.610
846	0.2521	5806.8	1.100	773.9	639.1	2916.8	0.552	0.023	Steep Basin Slope	0.548	0.472	0.611
847	1.0016	16360.2	3.099	801.5	630.0	7807.6	1.479	0.010	Steep Basin Slope	0.505	0.797	0.646
856	0.3101	6955.6	1.317	818.1	668.1	3816.9	0.723	0.022	Steep Basin Slope	0.550	0.542	0.610
866	0.2761	5621.7	1.065	703.9	609.7	3238.0	0.613	0.017	Steep Basin Slope	0.548	0.482	0.611
871	0.8415	10084.1	1.910	721.9	609.4	4924.2	0.933	0.011	Steep Basin Slope	0.538	0.640	0.620
874	0.1819	5456.4	1.033	706.2	603.0	3245.7	0.615	0.019	Steep Basin Slope	0.541	0.472	0.617
880	0.5783	7209.6	1.365	801.6	669.8	2639.1	0.500	0.018	Steep Basin Slope	0.549	0.490	0.611
889	0.8826	14532.9	2.752	828.9	648.4	5426.1	1.028	0.012	Steep Basin Slope	0.550	0.751	0.610
902	0.4970	8220.7	1.557	766.7	628.7	3426.3	0.649	0.017	Steep Basin Slope	0.549	0.551	0.611
912	0.9828	16227.6	3.073	820.2	626.7	6878.4	1.303	0.012	Steep Basin Slope	0.541	0.821	0.617
914	0.2518	9547.9	1.808	713.7	609.8	3577.6	0.678	0.011	Steep Basin Slope	0.538	0.571	0.620
922	0.0060	806.8	0.153	613.3	609.7	548.3	0.104	0.004	Flat Basin Slope	0.650	0.188	0.550
930	0.1807	5376.3	1.018	702.8	603.0	2454.1	0.465	0.019	Steep Basin Slope	0.550	0.439	0.610
933	0.7602	10431.3	1.976	724.5	594.5	5710.5	1.082	0.012	Steep Basin Slope	0.526	0.660	0.629
935	0.1109	5130.8	0.972	652.3	591.9	2734.1	0.518	0.012	Steep Basin Slope	0.550	0.448	0.610
946	0.0441	3728.2	0.706	642.6	579.9	1733.7	0.328	0.017	Steep Basin Slope	0.550	0.355	0.610
952	0.8940	10824.0	2.050	695.6	569.8	4861.6	0.921	0.012	Steep Basin Slope	0.550	0.665	0.610
954	0.2005	5037.7	0.954	665.6	569.8	3118.0	0.591	0.019	Steep Basin Slope	0.550	0.463	0.610

Attachment 6: Routing Input – Muskingum-Cunge

City of Denison Stormwater Master Plan (SWMP)
Attachment 6
Routing Input - Muskingum-Cunge

Reach ID	Initial Type	Length [ft]	Slope [ft/ft]	Mannings n	Space-Time Method	Index Method	Index Flow [cfs]	Shape	Left Manning's n	Right Mannings n	Cross Section
Duck Creek											
R-1901-2071	Discharge = Inflow	1,013	0.012	0.065	Auto DX Auto DT	Flow	381	Eight Point	0.158	0.158	XS1
R-1904-2071	Discharge = Inflow	1,000	0.011	0.065	Auto DX Auto DT	Flow	400	Eight Point	0.158	0.158	XS1
R-1930-1954	Discharge = Inflow	1,788	0.018	0.065	Auto DX Auto DT	Flow	1,156	Eight Point	0.141	0.141	XS1
R-1948-1951	Discharge = Inflow	2,418	0.009	0.065	Auto DX Auto DT	Flow	1,177	Eight Point	0.145	0.145	XS1
R-1951-1980	Discharge = Inflow	4,627	0.007	0.065	Auto DX Auto DT	Flow	1,708	Eight Point	0.151	0.151	XS1
R-1954-2008	Discharge = Inflow	4,746	0.005	0.065	Auto DX Auto DT	Flow	1,797	Eight Point	0.154	0.154	XS1
R-1965-1980	Discharge = Inflow	4,721	0.008	0.065	Auto DX Auto DT	Flow	758	Eight Point	0.151	0.151	XS1
R-1980-2008	Discharge = Inflow	4,714	0.005	0.065	Auto DX Auto DT	Flow	2,411	Eight Point	0.154	0.154	XS1
R-2008-2015	Discharge = Inflow	3,214	0.009	0.065	Auto DX Auto DT	Flow	4,188	Eight Point	0.137	0.137	XS1
R-2015-2023	Discharge = Inflow	1,731	0.004	0.065	Auto DX Auto DT	Flow	4,348	Eight Point	0.117	0.117	XS1
R-2056-1951	Discharge = Inflow	2,525	0.009	0.065	Auto DX Auto DT	Flow	557	Eight Point	0.145	0.145	XS1
R-2071-1954	Discharge = Inflow	2,133	0.014	0.065	Auto DX Auto DT	Flow	831	Eight Point	0.141	0.141	XS1
Iron Ore Creek											
R-1022-880	Discharge = Inflow	3,149	0.001	0.065	Auto DX Auto DT	Flow	1,146	Eight Point	0.131	0.131	XS1
R-1023-1024	Discharge = Inflow	3,666	0.006	0.065	Auto DX Auto DT	Flow	1,488	Eight Point	0.146	0.146	XS1
R-1024-813	Discharge = Inflow	3,402	0.001	0.065	Auto DX Auto DT	Flow	1,639	Eight Point	0.063	0.063	XS1
R-1025-1026	Discharge = Inflow	2,024	0.005	0.065	Auto DX Auto DT	Flow	2,974	Eight Point	0.137	0.137	XS1
R-1026-930	Discharge = Inflow	4,588	0.001	0.065	Auto DX Auto DT	Flow	3,499	Eight Point	0.144	0.144	XS1
R-1036-1041	Discharge = Inflow	6,473	0.002	0.065	Auto DX Auto DT	Flow	1,112	Eight Point	0.147	0.147	XS1
R-1041-946	Discharge = Inflow	1,582	0.001	0.065	Auto DX Auto DT	Flow	12,371	Eight Point	0.160	0.160	XS1
R-601-821	Discharge = Inflow	7,924	0.005	0.065	Auto DX Auto DT	Flow	355	Eight Point	0.153	0.153	XS1
R-608-1022	Discharge = Inflow	4,937	0.008	0.065	Auto DX Auto DT	Flow	320	Eight Point	0.147	0.147	XS1
R-609-684	Discharge = Inflow	5,624	0.009	0.065	Auto DX Auto DT	Flow	254	Eight Point	0.146	0.146	XS1
R-610-694	Discharge = Inflow	1,588	0.008	0.065	Auto DX Auto DT	Flow	304	Eight Point	0.156	0.156	XS1
R-613-731	Discharge = Inflow	1,402	0.008	0.065	Auto DX Auto DT	Flow	495	Eight Point	0.160	0.160	XS1
R-619-759	Discharge = Inflow	2,123	0.008	0.065	Auto DX Auto DT	Flow	320	Eight Point	0.159	0.159	XS1
R-620-889	Discharge = Inflow	6,319	0.003	0.065	Auto DX Auto DT	Flow	432	Eight Point	0.152	0.152	XS1
R-622-684	Discharge = Inflow	5,668	0.012	0.065	Auto DX Auto DT	Flow	349	Eight Point	0.146	0.146	XS1
R-626-795	Discharge = Inflow	668	0.001	0.065	Auto DX Auto DT	Flow	408	Eight Point	0.049	0.049	XS1
R-629-691	Discharge = Inflow	6,413	0.012	0.065	Auto DX Auto DT	Flow	446	Eight Point	0.149	0.149	XS1
R-633-666	Discharge = Inflow	2,480	0.008	0.065	Auto DX Auto DT	Flow	472	Eight Point	0.154	0.154	XS1
R-644-824	Discharge = Inflow	3,148	0.008	0.065	Auto DX Auto DT	Flow	551	Eight Point	0.041	0.041	XS1
R-649-677	Discharge = Inflow	2,479	0.010	0.065	Auto DX Auto DT	Flow	638	Eight Point	0.137	0.137	XS1
R-656-871	Discharge = Inflow	6,724	0.004	0.065	Auto DX Auto DT	Flow	716	Eight Point	0.144	0.144	XS1
R-657-694	Discharge = Inflow	1,663	0.006	0.065	Auto DX Auto DT	Flow	682	Eight Point	0.156	0.156	XS1
R-660-758	Discharge = Inflow	4,222	0.007	0.065	Auto DX Auto DT	Flow	701	Eight Point	0.133	0.133	XS1
R-666-807	Discharge = Inflow	7,013	0.005	0.065	Auto DX Auto DT	Flow	728	Eight Point	0.148	0.148	XS1
R-670-806	Discharge = Inflow	1,536	0.004	0.065	Auto DX Auto DT	Flow	847	Eight Point	0.159	0.159	XS1
R-673-813	Discharge = Inflow	3,067	0.001	0.065	Auto DX Auto DT	Flow	850	Eight Point	0.063	0.063	XS1
R-676-758	Discharge = Inflow	4,311	0.007	0.065	Auto DX Auto DT	Flow	872	Eight Point	0.133	0.133	XS1
R-677-772	Discharge = Inflow	2,191	0.005	0.065	Auto DX Auto DT	Flow	945	Eight Point	0.136	0.136	XS1
R-680-1022	Discharge = Inflow	4,906	0.008	0.065	Auto DX Auto DT	Flow	853	Eight Point	0.147	0.147	XS1
R-681-1036	Discharge = Inflow	6,150	0.009	0.065	Auto DX Auto DT	Flow	815	Eight Point	0.150	0.150	XS1
R-683-788	Discharge = Inflow	5,021	0.005	0.065	Auto DX Auto DT	Flow	1,022	Eight Point	0.158	0.158	XS1
R-684-846	Discharge = Inflow	2,877	0.004	0.065	Auto DX Auto DT	Flow	809	Eight Point	0.151	0.151	XS1
R-691-866	Discharge = Inflow	5,807	0.005	0.065	Auto DX Auto DT	Flow	853	Eight Point	0.150	0.150	XS1
R-694-788	Discharge = Inflow	5,015	0.006	0.065	Auto DX Auto DT	Flow	981	Eight Point	0.158	0.158	XS1
R-695-935	Discharge = Inflow	1,361	0.002	0.065	Auto DX Auto DT	Flow	787	Eight Point	0.125	0.125	XS1
R-696-731	Discharge = Inflow	1,326	0.008	0.065	Auto DX Auto DT	Flow	1,123	Eight Point	0.160	0.160	XS1
R-698-759	Discharge = Inflow	2,047	0.008	0.065	Auto DX Auto DT	Flow	874	Eight Point	0.159	0.159	XS1
R-700-772	Discharge = Inflow	2,097	0.005	0.065	Auto DX Auto DT	Flow	988	Eight Point	0.136	0.136	XS1
R-702-856	Discharge = Inflow	1,653	0.007	0.065	Auto DX Auto DT	Flow	965	Eight Point	0.146	0.146	XS1
R-721-762	Discharge = Inflow	5,630	0.010	0.065	Auto DX Auto DT	Flow	1,227	Eight Point	0.159	0.159	XS1
R-731-912	Discharge = Inflow	2,938	0.001	0.065	Auto DX Auto DT	Flow	1,570	Eight Point	0.130	0.130	XS1
R-736-952	Discharge = Inflow	5,107	0.003	0.065	Auto DX Auto DT	Flow	1,221	Eight Point	0.128	0.128	XS1
R-740-807	Discharge = Inflow	6,994	0.005	0.065	Auto DX Auto DT	Flow	1,320	Eight Point	0.148	0.148	XS1
R-758-830	Discharge = Inflow	1,559	0.002	0.065	Auto DX Auto DT	Flow	1,534	Eight Point	0.148	0.148	XS1
R-759-830	Discharge = Inflow	1,528	0.001	0.065	Auto DX Auto DT	Flow	1,386	Eight Point	0.148	0.148	XS1
R-762-806	Discharge = Inflow	1,536	0.004	0.065	Auto DX Auto DT	Flow	1,557	Eight Point	0.159	0.159	XS1
R-788-821	Discharge = Inflow	7,394	0.004	0.065	Auto DX Auto DT	Flow	1,932	Eight Point	0.153	0.153	XS1
R-795-824	Discharge = Inflow	3,174	0.008	0.065	Auto DX Auto DT	Flow	2,016	Eight Point	0.041	0.041	XS1
R-806-902	Discharge = Inflow	4,374	0.004	0.065	Auto DX Auto DT	Flow	2,162	Eight Point	0.144	0.144	XS1
R-807-946	Discharge = Inflow	1,556	0.001	0.065	Auto DX Auto DT	Flow	2,231	Eight Point	0.160	0.160	XS1
R-813-820	Discharge = Inflow	2,050	0.014	0.065	Auto DX Auto DT	Flow	2,241	Eight Point	0.096	0.096	XS1
R-820-1025	Discharge = Inflow	4,637	0.005	0.065	Auto DX Auto DT	Flow	2,270	Eight Point	0.151	0.151	XS1
R-821-847	Discharge = Inflow	2,140	0.003	0.065	Auto DX Auto DT	Flow	2,206	Eight Point	0.153	0.153	XS1
R-824-846	Discharge = Inflow	2,982	0.004	0.065	Auto DX Auto DT	Flow	2,612	Eight Point	0.151	0.151	XS1
R-830-856	Discharge = Inflow	1,653	0.006	0.065	Auto DX Auto DT	Flow	2,621	Eight Point	0.146	0.146	XS1
R-846-871	Discharge = Inflow	6,800	0.004	0.065	Auto DX Auto DT	Flow	3,258	Eight Point	0.144	0.144	XS1
R-847-866	Discharge = Inflow	5,371	0.004	0.065	Auto DX Auto DT	Flow	2,935	Eight Point	0.150	0.150	XS1
R-871-874	Discharge = Inflow	1,091	0.006	0.065	Auto DX Auto DT	Flow	3,517	Eight Point	0.129	0.129	XS1
R-874-933	Discharge = Inflow	4,260	0.002	0.065	Auto DX Auto DT	Flow	3,950	Eight Point	0.138	0.138	XS1
R-880-889	Discharge = Inflow	5,922	0.004	0.065	Auto DX Auto DT	Flow	4,287	Eight Point	0.152	0.152	XS1
R-889-902	Discharge = Inflow	4,462	0.004	0.065	Auto DX Auto DT	Flow	4,681	Eight Point	0.144	0.144	XS1
R-902-912	Discharge = Inflow	2,907	0.001	0.065	Auto DX Auto DT	Flow	6,098	Eight Point	0.130	0.130	XS1
R-912-914	Discharge = Inflow	5,283	0.003	0.065	Auto DX Auto DT	Flow	7,037	Eight Point	0.153	0.153	XS1
R-914-922	Discharge = Inflow	762	0.001	0.065	Auto DX Auto DT	Flow	6,883	Eight Point	0.152	0.152	XS1
R-922-930	Discharge = Inflow	4,575	0.001	0.065	Auto DX Auto DT	Flow	8,924	Eight Point	0.144	0.144	XS1
R-930-933	Discharge = Inflow	4,216	0.002	0.065	Auto DX Auto DT	Flow	10,124	Eight Point	0.138	0.138	XS1
R-933-935	Discharge = Inflow	1,272	0.002	0.065	Auto DX Auto DT	Flow	12,194	Eight Point	0.125	0.125	XS1
R-935-1041	Discharge = Inflow	6,499	0.002	0.065	Auto DX Auto DT	Flow	12,107	Eight Point	0.147	0.147	XS1
R-946-952	Discharge = Inflow	4,470	0.002	0.065	Auto DX Auto DT	Flow	13,091	Eight Point	0.128	0.128	XS1
R-952-954	Discharge = Inflow	951	0.001	0.065	Auto DX Auto DT	Flow	13,263	Eight Point	0.156	0.156	XS1
Pawpaw Creek											
R-1897-1969	Discharge = Inflow	9,443	0.010	0.065	Auto DX Auto DT	Flow	1,146	Eight Point	0.146	0.146	XS1
R-1906-1984	Discharge = Inflow	655	0.020	0.065	Auto DX Auto DT	Flow	717	Eight Point	0.160	0.160	XS1
R-1907-2003	Discharge = Inflow	788	0.007	0.065	Auto DX Auto DT	Flow	859	Eight Point	0.140	0.140	XS1
R-1908-1998	Discharge = Inflow	1,013	0.021	0.065	Auto DX Auto DT	Flow	753	Eight Point	0.160	0.160	XS1
R-1969-1979	Discharge = Inflow	4,527	0.007	0.065	Auto DX Auto DT	Flow	1,967	Eight Point	0.160	0.160	XS1
R-1979-1984	Discharge = Inflow	447	0.013	0.065	Auto DX Auto DT	Flow	2,136	Eight Point	0.160	0.160	XS1
R-1984-1990	Discharge = Inflow	1,507	0.009	0.065	Auto DX Auto DT	Flow	2,508	Eight Point	0.157	0.157	XS1
R-1989-1990	Discharge = Inflow	1,940	0.012	0.065	Auto DX Auto DT	Flow	455	Eight Point	0.157	0.157	XS1
R-1990-1998	Discharge = Inflow	893	0.008	0.065	Auto DX Auto DT	Flow	2,713	Eight Point	0.160	0.160	XS1
R-1998-2003	Discharge = Inflow	788	0.005	0.065	Auto DX Auto DT	Flow	3,092	Eight Point	0.140	0.140	XS1
R-2061-1897	Discharge = Inflow	4,530	0.015	0.065	Auto DX Auto DT	Flow	592	Eight Point	0.104	0.104	XS1
Shawnee Creek											
R-1899-1995	Discharge = Inflow	7,844	0.006	0.065	Auto DX Auto DT	Flow	2,440	Eight Point	0.151	0.151	XS1
R-1900-2065	Discharge = Inflow	2,140	0.012	0.065	Auto DX Auto DT	Flow	231	Eight Point	0.058	0.058	XS1
R-1920-1899	Discharge = Inflow	4,499	0.009	0.065	Auto DX Auto DT	Flow	1,348	Eight Point	0.109	0.109	XS1
R-1940-1978	Discharge = Inflow	4,974	0.004	0.065	Auto DX Auto DT	Flow	1,391	Eight Point	0.134	0.134	XS1
R-1941-2016	Discharge = Inflow	2,545	0.002	0.065	Auto DX Auto DT	Flow	1,253	Eight Point	0.103	0.103	XS1
R-1943-1971	Discharge = Inflow	5,305	0.008	0.065	Auto DX Auto DT	Flow	1,681	Eight Point	0.138	0.138	XS1
R-1950-1899	Discharge = Inflow	4,499	0.009	0.065	Auto DX Auto DT	Flow	619	Eight Point	0.109	0.109	XS1
R-1971-1993	Discharge = Inflow	3,305	0.005	0.065	Auto DX Auto DT	Flow	2,391	Eight Point	0.151	0.151	XS1
R-1978-2065	Discharge = Inflow	1,980	0.011	0.065	Auto DX Auto DT	Flow	2,288	Eight Point	0.058	0.058	XS1
R-1992-1999	Discharge = Inflow	3,530	0.009	0.065	Auto DX Auto DT	Flow	727	Eight Point	0.140	0.140	XS1
R-1993-1996	Discharge = Inflow	3,048	0.004	0.065	Auto DX Auto DT	Flow	3,242	Eight Point	0.147	0.147	XS1
R-1995-2045	Discharge = Inflow	6,344	0.005	0.065	Auto DX Auto DT	Flow	3,186	Eight Point	0.078	0.078	XS1
R-1996-2032	Discharge = Inflow	4,569	0.001	0.065	Auto DX Auto DT	Flow	3,230	Eight Point	0.038	0.038	XS1
R-1999-2016	Discharge = Inflow	2,443	0.002	0.065	Auto DX Auto DT	Flow	3,275	Eight Point	0.103	0.103	XS1
R-2016-2032	Discharge = Inflow	4,556	0.001	0.065	Auto DX Auto DT	Flow	4,306	Eight Point	0.038	0.038	XS1
R-2032-2037	Discharge = Inflow	2,069	0.001	0.065	Auto DX Auto DT	Flow	6,332	Eight Point	0.036	0.036	XS1
R-2037-2039	Discharge = Inflow	7,627	0.010	0.065	Auto DX Auto DT	Flow	6,456	Eight Point	0.142	0.142	XS1
R-2039-2045	Discharge = Inflow	6,389	0.005	0.065	Auto DX Auto DT	Flow	7,545	Eight Point	0.078	0.078	XS1
R-2065-1999	Discharge = Inflow	3,561	0.008	0.065	Auto DX Auto DT	Flow	2,482	Eight Point	0.140	0.140	XS1
UNT0097											
R-1903-1937	Discharge = Inflow	10,710	0.009	0.065	Auto DX Auto DT	Flow	340	Eight Point	0.144	0.144	XS1
R-1937-1964	Discharge = Inflow	5,945	0.004	0.065	Auto DX Auto DT	Flow	1,342	Eight Point	0.131	0.131	XS1
UNT141											
R-745-799	Discharge = Inflow	5,922	0.007	0.065	Auto DX Auto DT	Flow	1,481	Eight Point	0.146	0.146	XS1

Attachment 7: Reservoir Input Data

Attachment 7
Reservoir Input Data

Loy Lake
HEC-HMS Inputs

Normal Storage (acre-ft)	370
Max Storage (acre-ft)	642
Normal Available Storage(acre-ft)	272
Elevation at Normal Deficit (ft)	664.6
Initial Reservoir Elevation (ft)	664.6

Stage-Area Input

Elevation (ft)	Area (acre)	Note
664	18.6	
674	39	Spillway Crest
680	39	Assumed Areas above Crest
700	39	Assumed Areas above Crest

Stage-Discharge Table

Ogee Spillway Coefficient (ft^0.5/s)	3.3
Weir Length (ft)	185
Crest Elevation (ft)	674

Elevation (ft)	Discharge (cfs)
664	0
674	0
676	1727
678	4884
680	8972

Attachment 7
Reservoir Input Data

Waterloo Lake
HEC-HMS Inputs

Normal Storage (ac-ft)	99
Maximum Storage (ac-ft)	2545
Normal Available Storage(acre-ft)	2446
Elevation at Normal Deficit (ft)	N/A
Initial Reservoir Elevation (ft)	680

Stage-Area Input

Elevation (ft)	Area (acre)	Note
641	10.1	
670	38.3	
680	48.1	
690	80.6	
694	99.9	Spillway Crest
700	128.9	

Stage-Discharge Table

Ogee Spillway Coefficient (ft^0.5/s)	3.3
Weir Length (ft)	130
Crest Elevation (ft)	694

Elevation (ft)	Discharge (cfs)
641	0
670	0
680	0
690	0
694	0
696	1213
698	3432
700	6305

Attachment 7
Reservoir Input Data

Lake Randell
HEC-HMS Inputs

Normal Storage (ac-ft)	5690
Maximum Storage (ac-ft)	6824
Normal Available Storage(acre-ft)	1134
Elevation at Normal Deficit (ft)	619.8
Initial Reservoir Elevation (ft)	619.8

Stage-Area Input

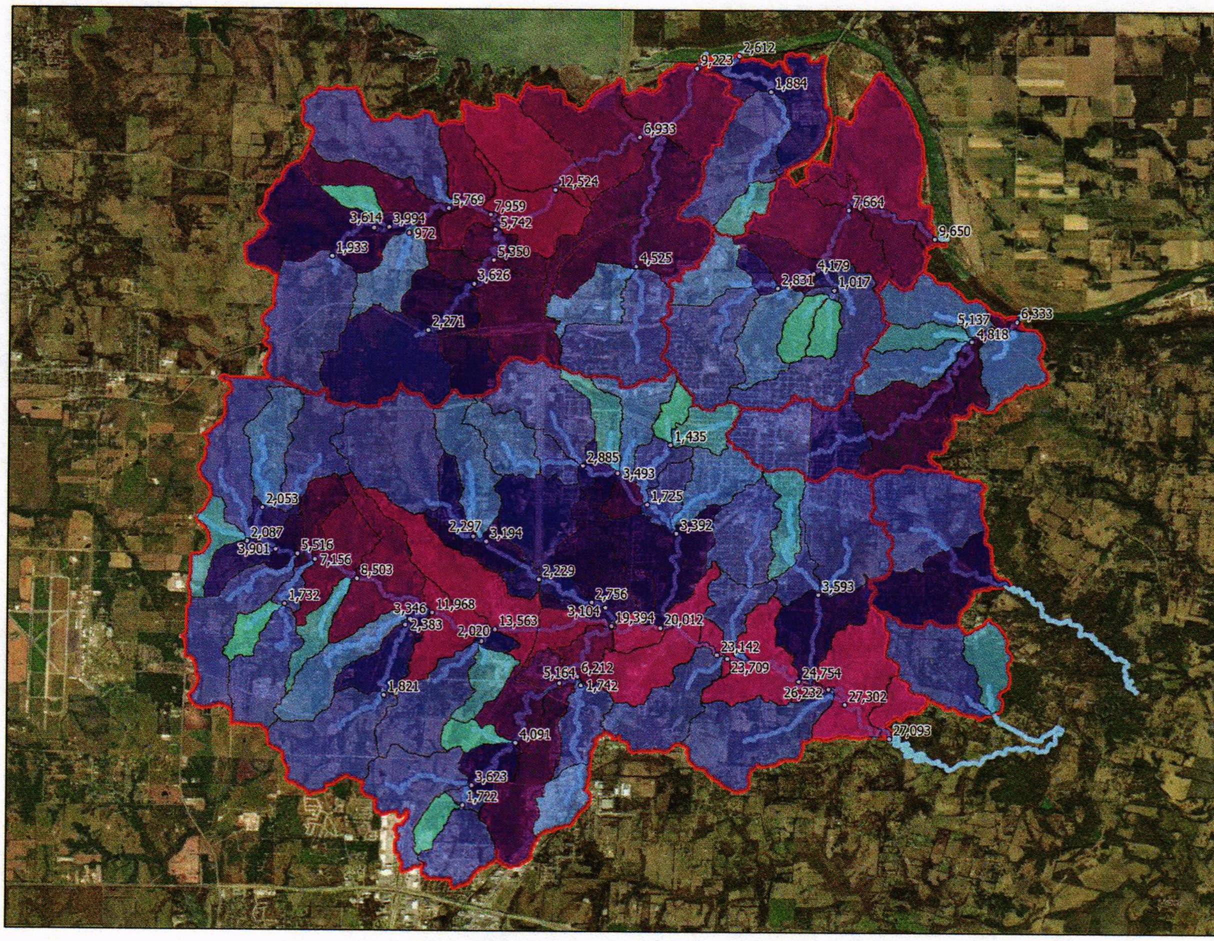
Elevation (ft)	Area (acre)	Note
614.4	238.0	
619.8	291.4	
621.8	311.0	
622.4	312.2	Spillway Crest
628	324.0	

Stage-Discharge Table

Ogee Spillway Coefficient (ft^0.5/s)	3.3
Weir Length (ft)	276
Crest Elevation (ft)	622.4

Elevation (ft)	Discharge (cfs)
614.4	0
619.8	0
621.8	0
622.4	0
624	1843
626	6221
628	12070

Attachment 8: Model Results – Peak Discharge and Runoff Volume



CITY OF DENISON
300 W Main St
Denison, TX 75020

Denison TEXAS

City of Denison
Conceptual Stormwater Master Plan
R315880.01

Legend

- Concentration Point Flow

Discharge (cfs)

- 359 - 500
- 500 - 750
- 750 - 1000
- 1000 - 2000
- 2000 - 3000
- 3000 - 4000
- 4000 - 6000
- 6000 - 10000
- 10000 - 20000
- 20000 - 27302
- Stream Line
- Basins

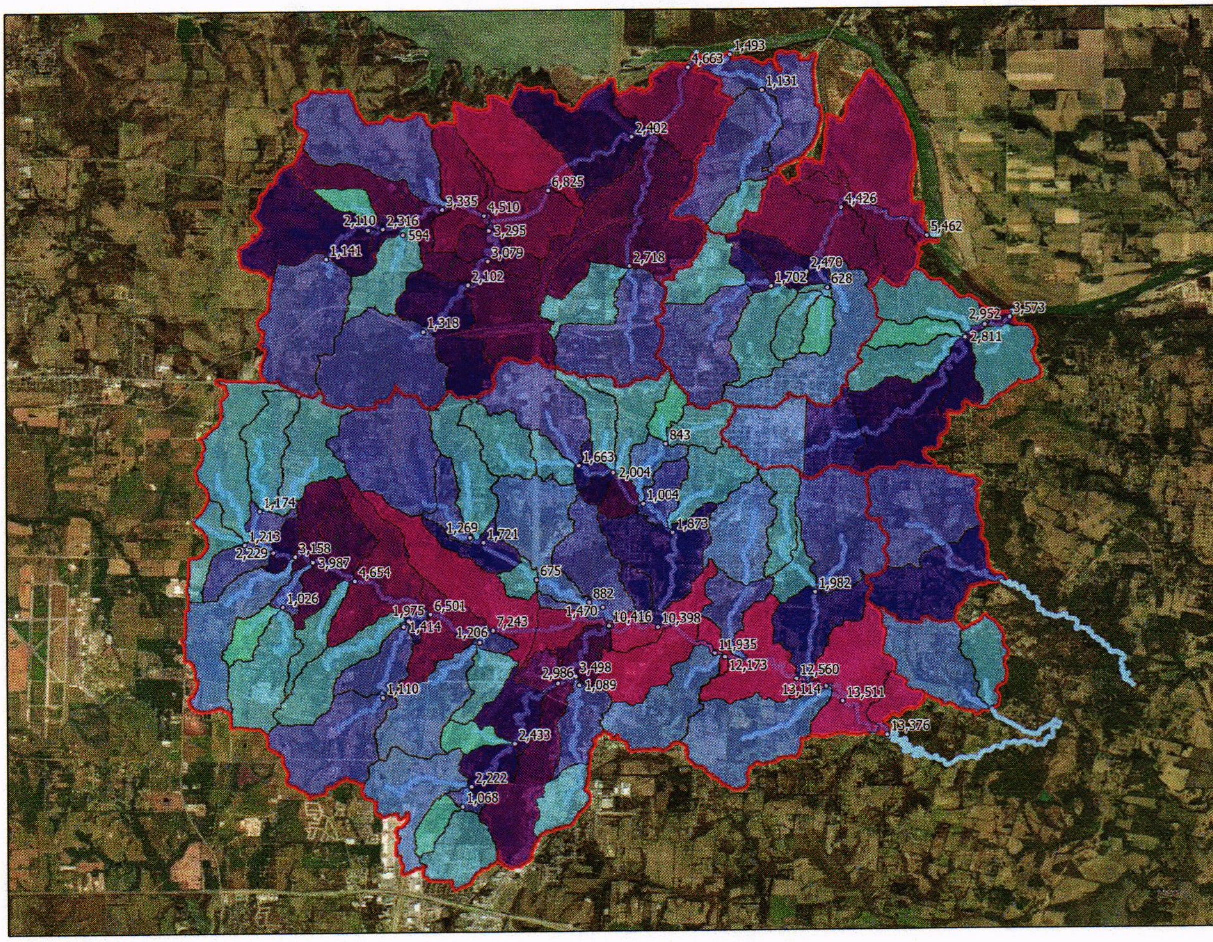
N

Attachment 8

100-year, 24-hour Storm Discharge

0 1 2 Miles

HUITT-ZOLLARS
5050 N 40th Street, Suite 100
Phoenix, Arizona 85018



CITY OF DENISON
300 W Main St
Denison, TX 75020

Denison TEXAS

City of Denison
Conceptual Stormwater Master Plan
R315880.01

Legend

- Concentration Point Flow

10y HMS (cfs)

- 216 - 300
- 300 - 750
- 750 - 1000
- 1000 - 1500
- 1500 - 2000
- 2000 - 2500
- 2500 - 4000
- 4000 - 7000
- 7000 - 10000
- 10000 - 13511
- Stream Line
- Basins

N

Attachment 8

10-year, 24-hour Storm Discharge

0 1 2 Miles

HUITT-ZOLLARS
5050 N 40th Street, Suite 100
Phoenix, Arizona 85018

Attachment 8
Model Results - Peak Discharge and Runoff Volume

100-Year, 24-Hour Design Storm				
Hydrologic Element	Drainage Area (mi^2)	Peak Discharge (cfs)	Time of Peak	Volume (ac-ft)
Duck Creek				
J-1901	0.188	500	01Jan2000, 12:30	79.2
J-1904	0.226	500	01Jan2000, 12:35	93
J-1930	1.023	1700	01Jan2000, 12:50	387.4
J-1948	0.932	1700	01Jan2000, 12:45	374.1
J-1951	1.482	2800	01Jan2000, 12:45	599.5
J-1954	1.528	2700	01Jan2000, 12:45	593
J-1965	0.489	900	01Jan2000, 12:40	194
J-1980	2.297	4200	01Jan2000, 13:00	908.1
J-2008	4.734	7700	01Jan2000, 13:10	1854.7
J-2015	5.262	8300	01Jan2000, 13:15	2061
J-2023	6.512	9700	01Jan2000, 13:15	2511.6
J-2056	0.415	1000	01Jan2000, 12:30	177.6
J-2071	0.450	1000	01Jan2000, 12:35	186.2
R-1901-2071	0.188	500	01Jan2000, 12:35	79.2
R-1904-2071	0.226	500	01Jan2000, 12:35	92.8
R-1930-1954	1.023	1700	01Jan2000, 12:50	386.9
R-1948-1951	0.932	1700	01Jan2000, 12:50	373.1
R-1951-1980	1.482	2800	01Jan2000, 13:00	596.1
R-1954-2008	1.528	2600	01Jan2000, 13:05	588.7
R-1965-1980	0.489	900	01Jan2000, 13:00	192.9
R-1980-2008	2.297	4000	01Jan2000, 13:15	901.8
R-2008-2015	4.734	7600	01Jan2000, 13:15	1849.6
R-2015-2023	5.262	8200	01Jan2000, 13:20	2056.6
R-2056-1951	0.415	1000	01Jan2000, 12:40	177
R-2071-1954	0.450	1000	01Jan2000, 12:40	185.7
1901	0.188	500	01Jan2000, 12:30	79.2
1904	0.226	500	01Jan2000, 12:35	93
1930	1.023	1700	01Jan2000, 12:50	387.4
1948	0.932	1700	01Jan2000, 12:45	374.1
1951	0.135	300	01Jan2000, 12:30	49.5
1954	0.056	100	01Jan2000, 12:20	20.3
1965	0.489	900	01Jan2000, 12:40	194
1980	0.326	600	01Jan2000, 12:40	119.1
2008	0.909	2000	01Jan2000, 12:30	364.2
2015	0.528	1000	01Jan2000, 12:35	211.5
2023	1.250	1600	01Jan2000, 13:00	455.1
2056	0.415	1000	01Jan2000, 12:30	177.6
2071	0.035	100	01Jan2000, 12:15	14.2
Iron Ore Creek				
J-1022	0.979	1700	01Jan2000, 13:00	403.5
J-1023	1.178	1900	01Jan2000, 12:45	423.5
J-1024	1.432	2300	01Jan2000, 12:55	515.8
J-1025	3.345	2800	01Jan2000, 14:15	923.7
J-1026	4.068	3100	01Jan2000, 14:20	1179.9
J-1036	1.049	1900	01Jan2000, 12:55	404.1
J-1041	28.513	24800	01Jan2000, 14:10	9253.4
J-601	0.187	400	01Jan2000, 12:30	76.9
J-608	0.198	400	01Jan2000, 12:30	82.7
J-609	0.162	400	01Jan2000, 12:25	62.2
J-610	0.175	500	01Jan2000, 12:25	76.1
J-613	0.297	700	01Jan2000, 12:30	122.7
J-619	0.271	600	01Jan2000, 12:30	107.7
J-620	0.278	500	01Jan2000, 12:40	114.6
J-622	0.288	700	01Jan2000, 12:30	110.3
J-626	0.314	600	01Jan2000, 12:35	122.2
J-629	0.324	1000	01Jan2000, 12:25	148.6
J-633	0.360	700	01Jan2000, 12:35	135.2
J-644	0.438	900	01Jan2000, 12:35	181.9
J-649	0.451	800	01Jan2000, 12:35	152.0
J-656	0.488	900	01Jan2000, 12:40	191.7
J-657	0.477	1100	01Jan2000, 12:30	207.6
J-660	0.515	800	01Jan2000, 12:50	196.5
J-666	0.570	1100	01Jan2000, 12:40	213.7
J-670	0.601	1000	01Jan2000, 12:55	243.4
J-673	0.614	1000	01Jan2000, 12:45	218.1
J-676	0.635	1000	01Jan2000, 12:50	243.4
J-677	0.637	1100	01Jan2000, 12:40	215.7
J-680	0.668	1200	01Jan2000, 12:45	281.3
J-681	0.687	1400	01Jan2000, 12:35	266.1
J-683	0.755	1600	01Jan2000, 12:35	323.1
J-684	0.683	1400	01Jan2000, 12:50	258.8
J-691	0.732	1700	01Jan2000, 12:45	312.6
J-694	0.731	1700	01Jan2000, 12:35	313.6

Attachment 8

Model Results - Peak Discharge and Runoff Volume

100-Year, 24-Hour Design Storm				
Hydrologic Element	Drainage Area (mi^2)	Peak Discharge (cfs)	Time of Peak	Volume (ac-ft)
J-695	0.754	1300	01Jan2000, 12:45	289.4
J-696	0.757	1300	01Jan2000, 12:50	309.0
J-698	0.765	1200	01Jan2000, 12:55	300.9
J-700	0.793	1700	01Jan2000, 12:35	329.0
J-702	0.838	1300	01Jan2000, 13:00	339.7
J-721	1.001	1800	01Jan2000, 12:45	418.3
J-731	1.143	2000	01Jan2000, 12:45	465.3
J-736	1.097	1700	01Jan2000, 12:50	386.6
J-740	1.060	1800	01Jan2000, 12:40	385.0
J-758	1.307	2100	01Jan2000, 13:05	501.6
J-759	1.298	2100	01Jan2000, 12:45	514.0
J-762	1.394	2400	01Jan2000, 13:00	567.3
J-772	1.542	2900	01Jan2000, 12:45	584.4
J-772-626	1.856	3500	01Jan2000, 12:45	706.7
J-788	1.772	3600	01Jan2000, 12:55	748.9
J-795	1.939	3500	01Jan2000, 12:45	734.6
J-806	2.049	3300	01Jan2000, 13:05	829.8
J-807	2.266	3600	01Jan2000, 13:10	826.3
J-813	2.328	3200	01Jan2000, 13:05	842.0
J-820	2.467	2200	01Jan2000, 14:00	622.7
J-821	2.358	4100	01Jan2000, 13:20	973.0
J-824	2.529	4600	01Jan2000, 12:50	976.9
J-830	2.754	3900	01Jan2000, 13:05	1067.9
J-846	3.464	1700	01Jan2000, 13:00	354.4
J-847	3.359	5200	01Jan2000, 13:25	1386.1
J-856	3.902	5500	01Jan2000, 13:10	1522.9
J-856-1022	4.882	7200	01Jan2000, 13:05	1926.4
J-866	4.368	6200	01Jan2000, 13:35	1783.7
J-871	4.794	3400	01Jan2000, 13:20	855.5
J-874	4.976	3600	01Jan2000, 13:20	922.5
J-880	5.460	7200	01Jan2000, 13:10	2134.0
J-889	6.621	8500	01Jan2000, 13:25	2560.6
J-902	9.166	12000	01Jan2000, 13:30	3564.2
J-912	11.291	13600	01Jan2000, 13:35	4355.5
J-914	11.543	13500	01Jan2000, 13:45	4411.4
J-922	15.917	19400	01Jan2000, 13:45	6183.5
J-930	20.166	20000	01Jan2000, 14:00	7290.7
J-933	25.902	23100	01Jan2000, 14:00	8415.8
J-935	26.766	23700	01Jan2000, 14:00	8724.6
J-946	30.823	26200	01Jan2000, 14:10	10031.3
J-952	32.815	27300	01Jan2000, 14:15	10647.5
J-954	33.015	27100	01Jan2000, 14:20	10683.1
LOY LAKE	2.328	2200	01Jan2000, 14:00	570.3
R-1022-880	4.882	6500	01Jan2000, 13:15	1905.1
R-1023-1024	1.178	1900	01Jan2000, 12:55	421.4
R-1024-813	1.432	2000	01Jan2000, 13:10	508.1
R-1025-1026	3.345	2700	01Jan2000, 14:25	920.9
R-1026-930	4.068	2700	01Jan2000, 14:45	1148.1
R-1036-1041	1.049	1600	01Jan2000, 13:35	394.4
R-1041-946	28.513	24200	01Jan2000, 14:15	9196.2
R-601-821	0.187	400	01Jan2000, 13:15	75.5
R-608-1022	0.198	400	01Jan2000, 12:55	82.1
R-609-684	0.162	400	01Jan2000, 12:50	61.7
R-610-694	0.175	500	01Jan2000, 12:35	75.9
R-613-731	0.297	700	01Jan2000, 12:40	122.5
R-619-759	0.271	600	01Jan2000, 12:40	107.4
R-620-889	0.278	500	01Jan2000, 13:30	112.3
R-622-684	0.288	700	01Jan2000, 12:50	109.5
R-626-795	1.856	3400	01Jan2000, 12:45	704.8
R-629-691	0.324	900	01Jan2000, 12:45	147.7
R-633-666	0.360	700	01Jan2000, 12:45	134.6
R-644-824	0.438	900	01Jan2000, 12:45	181.4
R-649-677	0.451	800	01Jan2000, 12:45	151.5
R-656-871	0.488	800	01Jan2000, 13:15	189.1
R-657-694	0.477	1100	01Jan2000, 12:40	207.0
R-660-758	0.515	800	01Jan2000, 13:05	195.3
R-666-807	0.570	1000	01Jan2000, 13:15	211.2
R-670-806	0.601	900	01Jan2000, 13:00	242.6
R-673-813	0.614	800	01Jan2000, 13:05	215.4
R-676-758	0.635	1000	01Jan2000, 13:05	242.0
R-677-772	0.637	1100	01Jan2000, 12:50	215.0
R-680-1022	0.668	1200	01Jan2000, 13:05	279.9
R-681-1036	0.687	1300	01Jan2000, 13:00	264.3
R-683-788	0.755	1500	01Jan2000, 13:00	320.6

Attachment 8
Model Results - Peak Discharge and Runoff Volume

100-Year, 24-Hour Design Storm				
Hydrologic Element	Drainage Area (mi^2)	Peak Discharge (cfs)	Time of Peak	Volume (ac-ft)
R-684-846	0.683	1400	01Jan2000, 13:00	257.2
R-691-866	0.732	1600	01Jan2000, 13:05	309.6
R-694-788	0.731	1700	01Jan2000, 12:55	311.3
R-695-935	0.754	1200	01Jan2000, 12:55	288.1
R-696-731	0.757	1300	01Jan2000, 12:55	308.5
R-698-759	0.765	1200	01Jan2000, 13:05	300.1
R-700-772	0.793	1700	01Jan2000, 12:45	328.0
R-702-856	0.838	1300	01Jan2000, 13:05	339.2
R-721-762	1.001	1800	01Jan2000, 13:05	416.1
R-731-912	1.143	1600	01Jan2000, 13:00	456.3
R-736-952	1.097	1600	01Jan2000, 13:15	382.1
R-740-807	1.060	1700	01Jan2000, 13:10	380.9
R-758-830	1.307	1900	01Jan2000, 13:10	499.0
R-759-830	1.298	1900	01Jan2000, 13:00	508.2
R-762-806	1.394	2400	01Jan2000, 13:05	565.6
R-788-821	1.772	3300	01Jan2000, 13:25	739.1
R-795-824	1.939	3500	01Jan2000, 12:50	733.4
R-806-902	2.049	3200	01Jan2000, 13:20	823.9
R-807-946	2.266	3200	01Jan2000, 13:15	820.0
R-813-820	2.328	2200	01Jan2000, 14:05	569.2
R-820-1025	2.467	2200	01Jan2000, 14:20	615.7
R-821-847	2.358	4000	01Jan2000, 13:30	968.5
R-824-846	2.529	0	01Jan2000, 00:00	0.0
R-830-856	2.754	3900	01Jan2000, 13:10	1066.0
R-846-871	3.464	1600	01Jan2000, 13:25	350.6
R-847-866	3.359	5000	01Jan2000, 13:45	1375.2
R-871-874	4.794	3400	01Jan2000, 13:20	854.7
R-874-933	4.976	3300	01Jan2000, 13:40	910.5
R-880-889	5.460	7000	01Jan2000, 13:30	2114.9
R-889-902	6.621	8400	01Jan2000, 13:35	2546.1
R-902-912	9.166	11100	01Jan2000, 13:40	3527.1
R-912-914	11.291	13300	01Jan2000, 13:45	4321.4
R-914-922	11.543	13300	01Jan2000, 13:50	4398.1
R-922-930	15.917	17900	01Jan2000, 13:55	6080.0
R-930-933	20.166	19700	01Jan2000, 14:10	7223.5
R-933-935	25.902	23000	01Jan2000, 14:05	8394.7
R-935-1041	26.766	23100	01Jan2000, 14:15	8599.7
R-946-952	30.823	25900	01Jan2000, 14:20	9936.1
R-952-954	32.815	27000	01Jan2000, 14:20	10609.1
WATERLOO LAKE	2.529	0	01Jan2000, 00:00	0.0
1022	0.113	200	01Jan2000, 12:30	41.6
1023	1.178	1900	01Jan2000, 12:45	423.5
1024	0.254	500	01Jan2000, 12:35	94.4
1025	0.879	1300	01Jan2000, 12:55	308.0
1026	0.723	1200	01Jan2000, 12:40	259.0
1036	0.362	700	01Jan2000, 12:40	139.8
1041	0.698	1200	01Jan2000, 12:45	259.3
601	0.187	400	01Jan2000, 12:30	76.9
608	0.198	400	01Jan2000, 12:30	82.7
609	0.162	400	01Jan2000, 12:25	62.2
610	0.175	500	01Jan2000, 12:25	76.1
613	0.297	700	01Jan2000, 12:30	122.7
619	0.271	600	01Jan2000, 12:30	107.7
620	0.278	500	01Jan2000, 12:40	114.6
622	0.288	700	01Jan2000, 12:30	110.3
626	0.314	600	01Jan2000, 12:35	122.2
629	0.324	1000	01Jan2000, 12:25	148.6
633	0.360	700	01Jan2000, 12:35	135.2
644	0.438	900	01Jan2000, 12:35	181.9
649	0.451	800	01Jan2000, 12:35	152.0
656	0.488	900	01Jan2000, 12:40	191.7
657	0.477	1100	01Jan2000, 12:30	207.6
660	0.515	800	01Jan2000, 12:50	196.5
666	0.209	500	01Jan2000, 12:25	79.0
670	0.601	1000	01Jan2000, 12:55	243.4
673	0.614	1000	01Jan2000, 12:45	218.1
676	0.635	1000	01Jan2000, 12:50	243.4
677	0.186	400	01Jan2000, 12:25	64.2
680	0.668	1200	01Jan2000, 12:45	281.3
681	0.687	1400	01Jan2000, 12:35	266.1
683	0.755	1600	01Jan2000, 12:35	323.1
684	0.233	500	01Jan2000, 12:35	87.5
691	0.408	900	01Jan2000, 12:30	164.8
694	0.079	200	01Jan2000, 12:20	30.7

Attachment 8
Model Results - Peak Discharge and Runoff Volume

100-Year, 24-Hour Design Storm				
Hydrologic Element	Drainage Area (mi²)	Peak Discharge (cfs)	Time of Peak	Volume (ac-ft)
695	0.754	1300	01Jan2000, 12:45	289.4
696	0.757	1300	01Jan2000, 12:50	309.0
698	0.765	1200	01Jan2000, 12:55	300.9
700	0.793	1700	01Jan2000, 12:35	329.0
702	0.838	1300	01Jan2000, 13:00	339.7
721	1.001	1800	01Jan2000, 12:45	418.3
731	0.089	300	01Jan2000, 12:20	34.3
736	1.097	1700	01Jan2000, 12:50	386.6
740	1.060	1800	01Jan2000, 12:40	385.0
758	0.156	400	01Jan2000, 12:30	64.3
759	0.262	600	01Jan2000, 12:35	106.6
762	0.393	800	01Jan2000, 12:40	151.2
772	0.112	300	01Jan2000, 12:25	41.5
788	0.286	600	01Jan2000, 12:30	117.0
795	0.083	200	01Jan2000, 12:20	29.8
806	0.054	200	01Jan2000, 12:20	21.6
807	0.637	1100	01Jan2000, 12:40	234.3
813	0.282	600	01Jan2000, 12:30	118.4
820	0.139	400	01Jan2000, 12:20	53.5
821	0.399	800	01Jan2000, 12:35	158.5
824	0.152	400	01Jan2000, 12:30	62.0
830	0.149	400	01Jan2000, 12:25	60.7
846	0.252	500	01Jan2000, 12:30	97.1
847	1.002	1800	01Jan2000, 12:50	417.6
856	0.310	600	01Jan2000, 12:35	117.8
866	0.276	600	01Jan2000, 12:30	98.9
871	0.842	1500	01Jan2000, 12:40	315.8
874	0.182	400	01Jan2000, 12:30	67.8
880	0.578	1200	01Jan2000, 12:30	228.9
889	0.883	1400	01Jan2000, 12:50	333.4
902	0.497	1000	01Jan2000, 12:35	194.2
912	0.983	1500	01Jan2000, 12:50	372.2
914	0.252	500	01Jan2000, 12:35	90.0
922	0.006	10	01Jan2000, 12:15	1.7
930	0.181	400	01Jan2000, 12:30	62.6
933	0.760	1300	01Jan2000, 12:40	281.7
935	0.111	200	01Jan2000, 12:30	41.9
946	0.044	100	01Jan2000, 12:25	15.1
952	0.894	1500	01Jan2000, 12:45	329.3
954	0.201	400	01Jan2000, 12:30	74.0
Pawpaw Creek				
J-1897	1.057	2700	01Jan2000, 12:35	443.5
J-1906	0.441	800	01Jan2000, 12:40	179.2
J-1907	0.488	1000	01Jan2000, 12:35	190.3
J-1908	0.520	1000	01Jan2000, 12:40	203.2
J-1969	2.002	4000	01Jan2000, 13:00	802.5
J-1979	2.257	4200	01Jan2000, 13:10	897.1
J-1984	2.705	4800	01Jan2000, 13:10	1078.5
J-1989	0.241	500	01Jan2000, 12:35	94.8
J-1990	2.966	5100	01Jan2000, 13:15	1179.2
J-1998	3.521	5800	01Jan2000, 13:15	1391.8
J-2003	4.029	6300	01Jan2000, 13:15	1588.9
J-2061	0.586	1600	01Jan2000, 12:30	245.6
R-1897-1969	1.057	2600	01Jan2000, 13:00	439.3
R-1906-1984	0.441	800	01Jan2000, 12:45	179.1
R-1907-2003	0.488	1000	01Jan2000, 12:40	190.1
R-1908-1998	0.520	1000	01Jan2000, 12:45	203.0
R-1969-1979	2.002	3900	01Jan2000, 13:15	797.9
R-1979-1984	2.257	4200	01Jan2000, 13:15	896.8
R-1984-1990	2.705	4800	01Jan2000, 13:15	1077.0
R-1989-1990	0.241	500	01Jan2000, 12:40	94.5
R-1990-1998	2.966	5100	01Jan2000, 13:20	1178.4
R-1998-2003	3.521	5700	01Jan2000, 13:20	1390.5
R-2061-1897	0.586	1600	01Jan2000, 12:40	244.7
1897	0.471	1300	01Jan2000, 12:25	198.8
1906	0.441	800	01Jan2000, 12:40	179.2
1907	0.488	1000	01Jan2000, 12:35	190.3
1908	0.520	1000	01Jan2000, 12:40	203.2
1969	0.945	1500	01Jan2000, 12:50	363.3
1979	0.255	500	01Jan2000, 12:40	99.2
1984	0.007	30	01Jan2000, 12:10	2.5
1989	0.241	500	01Jan2000, 12:35	94.8
1990	0.020	100	01Jan2000, 12:20	7.7
1998	0.035	100	01Jan2000, 12:20	10.4

100-Year, 24-Hour Design Storm				
Hydrologic Element	Drainage Area (mi^2)	Peak Discharge (cfs)	Time of Peak	Volume (ac-ft)
2003	0.019	100	01Jan2000, 12:10	8.2
2061	0.586	1600	01Jan2000, 12:30	245.6
Shawnee Creek				
J-1899	2.380	4500	01Jan2000, 12:45	988.0
J-1900	0.160	400	01Jan2000, 12:30	64.7
J-1920	1.010	2000	01Jan2000, 12:40	405.8
J-1940	1.160	1900	01Jan2000, 12:50	467.1
J-1941	1.160	1800	01Jan2000, 12:55	465.3
J-1943	1.230	2300	01Jan2000, 12:40	474.8
J-1950	0.450	900	01Jan2000, 12:40	191.0
J-1971	2.050	3600	01Jan2000, 12:50	805.8
J-1978	2.240	3600	01Jan2000, 12:55	910.3
J-1992	0.480	1000	01Jan2000, 12:40	203.8
J-1993	3.050	5400	01Jan2000, 12:55	1215.9
J-1995	3.260	5800	01Jan2000, 13:00	1347.2
J-1996	3.360	5700	01Jan2000, 13:00	1338.3
J-1999	3.560	5800	01Jan2000, 13:00	1453.8
J-2016	5.290	8000	01Jan2000, 13:10	2136.0
J-2032	9.300	12500	01Jan2000, 13:20	3714.7
J-2037	10.170	13300	01Jan2000, 13:20	4055.8
J-2039	11.220	6900	01Jan2000, 15:05	3328.3
J-2045	15.320	9200	01Jan2000, 14:45	4927.7
J-2065	2.500	4000	01Jan2000, 13:00	1016.6
LAKE RANDELL	10.170	6600	01Jan2000, 15:00	2956.1
R-1899-1995	2.380	4500	01Jan2000, 13:05	982.1
R-1900-2065	0.160	400	01Jan2000, 12:40	64.5
R-1920-1899	1.010	1900	01Jan2000, 12:50	404.3
R-1940-1978	1.160	1900	01Jan2000, 13:05	464.1
R-1941-2016	1.160	1700	01Jan2000, 13:05	461.6
R-1943-1971	1.230	2200	01Jan2000, 12:55	472.1
R-1950-1899	0.450	900	01Jan2000, 13:00	190.0
R-1971-1993	2.050	3600	01Jan2000, 13:00	804.4
R-1978-2065	2.240	3600	01Jan2000, 13:00	909.1
R-1992-1999	0.480	1000	01Jan2000, 12:55	203.0
R-1993-1996	3.050	5300	01Jan2000, 13:00	1213.4
R-1995-2045	3.260	5600	01Jan2000, 13:20	1335.4
R-1996-2032	3.360	4800	01Jan2000, 13:15	1320.8
R-1999-2016	3.560	5500	01Jan2000, 13:10	1446.5
R-2016-2032	5.290	7000	01Jan2000, 13:25	2106.4
R-2032-2037	9.300	12300	01Jan2000, 13:25	3699.1
R-2037-2039	10.170	6600	01Jan2000, 15:05	2936.9
R-2039-2045	11.220	6900	01Jan2000, 15:20	3287.6
R-2065-1999	2.500	4000	01Jan2000, 13:05	1013.8
1899	0.930	2000	01Jan2000, 12:35	393.6
1900	0.160	400	01Jan2000, 12:30	64.7
1920	1.010	2000	01Jan2000, 12:40	405.8
1940	1.160	1900	01Jan2000, 12:50	467.1
1941	1.160	1800	01Jan2000, 12:55	465.3
1943	1.230	2300	01Jan2000, 12:40	474.8
1950	0.450	900	01Jan2000, 12:40	191.0
1971	0.820	1500	01Jan2000, 12:40	333.7
1978	1.080	1900	01Jan2000, 12:45	446.2
1992	0.480	1000	01Jan2000, 12:40	203.8
1993	1.000	1900	01Jan2000, 12:45	411.5
1995	0.870	1400	01Jan2000, 12:55	365.1
1996	0.310	800	01Jan2000, 12:30	124.9
1999	0.580	1000	01Jan2000, 12:45	237.0
2016	0.570	1200	01Jan2000, 12:35	227.9
2032	0.650	1500	01Jan2000, 12:30	287.5
2037	0.870	1800	01Jan2000, 12:35	356.7
2039	1.050	1800	01Jan2000, 12:45	391.4
2045	0.840	1500	01Jan2000, 12:40	304.7
2065	0.090	300	01Jan2000, 12:25	42.9
UNT0097				
J-1903	0.220	600	01Jan2000, 12:25	92.1
J-1937	1.133	1900	01Jan2000, 13:05	453.8
J-1964	1.942	2600	01Jan2000, 13:15	761.6
R-1903-1937	0.220	600	01Jan2000, 13:10	90.8
R-1937-1964	1.133	1800	01Jan2000, 13:25	449.1
1903	0.220	600	01Jan2000, 12:25	92.1
1937	0.913	1400	01Jan2000, 12:55	363.0
1964	0.809	1700	01Jan2000, 12:30	312.5
UNT140				
J-UNT140	1.136	2000	01Jan2000, 12:40	435.1

Attachment 8
Model Results - Peak Discharge and Runoff Volume

100-Year, 24-Hour Design Storm				
Hydrologic Element	Drainage Area (mi^2)	Peak Discharge (cfs)	Time of Peak	Volume (ac-ft)
J-631	0.347	700	01Jan2000, 12:35	131.4
J-699	0.790	1400	01Jan2000, 12:45	303.7
631	0.347	700	01Jan2000, 12:35	131.4
699	0.790	1400	01Jan2000, 12:45	303.7
UNT141				
J-745	1.079	1900	01Jan2000, 12:35	361.3
J-799	1.958	3200	01Jan2000, 12:55	683.1
R-745-799	1.079	1800	01Jan2000, 12:55	358.2
745	1.079	1900	01Jan2000, 12:35	361.3
799	0.879	1500	01Jan2000, 12:45	324.8

Attachment 8

Model Results - Peak Discharge and Runoff Volume

10-Year, 24-Hour Design Storm				
Hydrologic Element	Drainage Area (mi^2)	Peak Discharge (cfs)	Time of Peak	Volume (ac-ft)
Duck Creek				
J-1901	0.188	300	01Jan2000, 12:30	45.3
J-1904	0.226	300	01Jan2000, 12:35	52.5
J-1930	1.023	1000	01Jan2000, 12:50	210.3
J-1948	0.932	1000	01Jan2000, 12:45	209.1
J-1951	1.482	1700	01Jan2000, 12:50	336.6
J-1954	1.528	1600	01Jan2000, 12:45	326
J-1965	0.489	600	01Jan2000, 12:40	107.6
J-1980	2.297	2500	01Jan2000, 13:05	504.3
J-2008	4.734	4400	01Jan2000, 13:15	1026.3
J-2015	5.262	4700	01Jan2000, 13:20	1140.9
J-2023	6.512	5500	01Jan2000, 13:25	1380.2
J-2056	0.415	600	01Jan2000, 12:30	102.4
J-2071	0.450	600	01Jan2000, 12:35	105.6
R-1901-2071	0.188	300	01Jan2000, 12:35	45.3
R-1904-2071	0.226	300	01Jan2000, 12:40	52.4
R-1930-1954	1.023	1000	01Jan2000, 12:55	209.9
R-1948-1951	0.932	1000	01Jan2000, 12:55	208.3
R-1951-1980	1.482	1700	01Jan2000, 13:05	334.1
R-1954-2008	1.528	1500	01Jan2000, 13:05	323.2
R-1965-1980	0.489	500	01Jan2000, 13:00	106.8
R-1980-2008	2.297	2400	01Jan2000, 13:20	500
R-2008-2015	4.734	4400	01Jan2000, 13:20	1022.9
R-2015-2023	5.262	4700	01Jan2000, 13:25	1137.6
R-2056-1951	0.415	600	01Jan2000, 12:40	101.9
R-2071-1954	0.450	600	01Jan2000, 12:40	105.3
1901	0.188	300	01Jan2000, 12:30	45.3
1904	0.226	300	01Jan2000, 12:35	52.5
1930	1.023	1000	01Jan2000, 12:50	210.3
1948	0.932	1000	01Jan2000, 12:45	209.1
1951	0.135	200	01Jan2000, 12:30	26.4
1954	0.056	100	01Jan2000, 12:20	10.8
1965	0.489	600	01Jan2000, 12:40	107.6
1980	0.326	300	01Jan2000, 12:40	63.4
2008	0.909	1200	01Jan2000, 12:35	203.1
2015	0.528	600	01Jan2000, 12:35	117.9
2023	1.250	900	01Jan2000, 13:00	242.6
2056	0.415	600	01Jan2000, 12:30	102.4
2071	0.035	100	01Jan2000, 12:15	7.9
Iron Ore Creek				
J-1022	0.979	1000	01Jan2000, 13:05	227.8
J-1023	1.178	1100	01Jan2000, 12:45	222.7
J-1024	1.432	1300	01Jan2000, 13:00	271.7
J-1025	3.345	900	01Jan2000, 12:50	364.9
J-1026	4.068	1500	01Jan2000, 12:55	499.2
J-1036	1.049	1100	01Jan2000, 13:00	220.0
J-1041	28.513	12600	01Jan2000, 14:35	4883.4
J-601	0.187	300	01Jan2000, 12:30	43.2
J-608	0.198	300	01Jan2000, 12:35	46.8
J-609	0.162	200	01Jan2000, 12:30	33.8
J-610	0.175	300	01Jan2000, 12:25	44.0
J-613	0.297	400	01Jan2000, 12:35	69.2
J-619	0.271	400	01Jan2000, 12:30	59.5
J-620	0.278	300	01Jan2000, 12:40	64.6
J-622	0.288	400	01Jan2000, 12:30	59.8
J-626	0.314	400	01Jan2000, 12:40	66.9
J-629	0.324	600	01Jan2000, 12:25	88.4
J-633	0.360	400	01Jan2000, 12:35	72.7
J-644	0.438	600	01Jan2000, 12:40	102.8
J-649	0.451	400	01Jan2000, 12:35	77.4
J-656	0.488	500	01Jan2000, 12:40	105.4
J-657	0.477	700	01Jan2000, 12:35	120.2
J-660	0.515	500	01Jan2000, 12:50	106.5
J-666	0.570	600	01Jan2000, 12:40	114.8
J-670	0.601	600	01Jan2000, 12:55	136.0
J-673	0.614	500	01Jan2000, 12:50	114.0
J-676	0.635	600	01Jan2000, 12:50	132.2
J-677	0.637	600	01Jan2000, 12:40	110.1
J-680	0.668	700	01Jan2000, 12:50	160.4
J-681	0.687	800	01Jan2000, 12:35	145.2
J-683	0.755	1000	01Jan2000, 12:35	185.6
J-684	0.683	800	01Jan2000, 12:50	139.9
J-691	0.732	1100	01Jan2000, 12:45	179.6
J-694	0.731	1100	01Jan2000, 12:40	180.4

Attachment 8
Model Results - Peak Discharge and Runoff Volume

10-Year, 24-Hour Design Storm				
Hydrologic Element	Drainage Area (mi^2)	Peak Discharge (cfs)	Time of Peak	Volume (ac-ft)
J-695	0.754	800	01Jan2000, 12:45	157.4
J-696	0.757	800	01Jan2000, 12:50	173.3
J-698	0.765	700	01Jan2000, 12:55	165.9
J-700	0.793	1000	01Jan2000, 12:35	186.0
J-702	0.838	800	01Jan2000, 13:00	190.1
J-721	1.001	1100	01Jan2000, 12:45	237.6
J-731	1.143	1200	01Jan2000, 12:45	260.8
J-736	1.097	900	01Jan2000, 12:50	201.4
J-740	1.060	1000	01Jan2000, 12:40	203.5
J-758	1.307	1200	01Jan2000, 13:05	273.1
J-759	1.298	1200	01Jan2000, 12:50	284.1
J-762	1.394	1400	01Jan2000, 13:00	318.2
J-772	1.542	1700	01Jan2000, 12:50	316.9
J-772-626	1.856	2000	01Jan2000, 12:45	383.7
J-788	1.772	2200	01Jan2000, 13:00	428.1
J-795	1.939	2000	01Jan2000, 12:50	398.1
J-806	2.049	2000	01Jan2000, 13:10	464.4
J-807	2.266	2000	01Jan2000, 13:15	438.2
J-813	2.328	1700	01Jan2000, 13:15	447.0
J-820	2.467	700	01Jan2000, 15:05	208.8
J-821	2.358	2400	01Jan2000, 13:30	551.2
J-824	2.529	2700	01Jan2000, 12:50	534.7
J-830	2.754	2200	01Jan2000, 13:10	585.5
J-846	3.464	1000	01Jan2000, 13:00	191.6
J-847	3.359	3000	01Jan2000, 13:35	785.1
J-856	3.902	3200	01Jan2000, 13:10	837.6
J-856-1022	4.882	4200	01Jan2000, 13:10	1065.4
J-866	4.368	3500	01Jan2000, 13:50	1007.4
J-871	4.794	1900	01Jan2000, 13:25	462.1
J-874	4.976	1900	01Jan2000, 13:30	497.9
J-880	5.460	4000	01Jan2000, 13:20	1176.7
J-889	6.621	4700	01Jan2000, 13:35	1405.6
J-902	9.166	6500	01Jan2000, 13:40	1961.4
J-912	11.291	7200	01Jan2000, 13:50	2393.2
J-914	11.543	7200	01Jan2000, 14:05	2417.4
J-922	15.917	10400	01Jan2000, 14:00	3416.9
J-930	20.166	10400	01Jan2000, 14:10	3859.5
J-933	25.902	11900	01Jan2000, 14:15	4453.6
J-935	26.766	12200	01Jan2000, 14:20	4617.7
J-946	30.823	13100	01Jan2000, 14:35	5284.6
J-952	32.815	13500	01Jan2000, 14:45	5592.1
J-954	33.015	13400	01Jan2000, 14:45	5606.0
LOY LAKE	2.328	700	01Jan2000, 14:55	180.5
R-1022-880	4.882	3700	01Jan2000, 13:25	1050.5
R-1023-1024	1.178	1100	01Jan2000, 13:00	221.3
R-1024-813	1.432	1100	01Jan2000, 13:20	267.2
R-1025-1026	3.345	900	01Jan2000, 13:00	363.3
R-1026-930	4.068	1000	01Jan2000, 13:30	477.7
R-1036-1041	1.049	900	01Jan2000, 13:45	213.6
R-1041-946	28.513	12200	01Jan2000, 14:40	4845.4
R-601-821	0.187	200	01Jan2000, 13:15	42.3
R-608-1022	0.198	300	01Jan2000, 12:55	46.5
R-609-684	0.162	200	01Jan2000, 12:50	33.5
R-610-694	0.175	300	01Jan2000, 12:35	43.8
R-613-731	0.297	400	01Jan2000, 12:40	69.1
R-619-759	0.271	400	01Jan2000, 12:40	59.3
R-620-889	0.278	300	01Jan2000, 13:30	62.9
R-622-684	0.288	400	01Jan2000, 12:50	59.3
R-626-795	1.856	2000	01Jan2000, 12:50	382.5
R-629-691	0.324	600	01Jan2000, 12:50	87.8
R-633-666	0.360	400	01Jan2000, 12:50	72.3
R-644-824	0.438	600	01Jan2000, 12:45	102.5
R-649-677	0.451	400	01Jan2000, 12:45	77.1
R-656-871	0.488	500	01Jan2000, 13:25	103.3
R-657-694	0.477	700	01Jan2000, 12:40	119.7
R-660-758	0.515	500	01Jan2000, 13:10	105.7
R-666-807	0.570	600	01Jan2000, 13:20	112.8
R-670-806	0.601	600	01Jan2000, 13:05	135.4
R-673-813	0.614	400	01Jan2000, 13:15	112.5
R-676-758	0.635	600	01Jan2000, 13:10	131.2
R-677-772	0.637	600	01Jan2000, 12:55	109.5
R-680-1022	0.668	700	01Jan2000, 13:10	159.3
R-681-1036	0.687	800	01Jan2000, 13:05	143.8
R-683-788	0.755	900	01Jan2000, 13:00	183.7

Attachment 8

Model Results - Peak Discharge and Runoff Volume

10-Year, 24-Hour Design Storm				
Hydrologic Element	Drainage Area (mi^2)	Peak Discharge (cfs)	Time of Peak	Volume (ac-ft)
R-684-846	0.683	800	01Jan2000, 13:05	138.8
R-691-866	0.732	1000	01Jan2000, 13:10	177.4
R-694-788	0.731	1000	01Jan2000, 13:00	178.7
R-695-935	0.754	700	01Jan2000, 12:55	156.4
R-696-731	0.757	800	01Jan2000, 12:55	173.0
R-698-759	0.765	700	01Jan2000, 13:05	165.3
R-700-772	0.793	1000	01Jan2000, 12:45	185.2
R-702-856	0.838	700	01Jan2000, 13:05	189.7
R-721-762	1.001	1100	01Jan2000, 13:05	235.9
R-731-912	1.143	900	01Jan2000, 13:10	254.7
R-736-952	1.097	800	01Jan2000, 13:20	198.6
R-740-807	1.060	1000	01Jan2000, 13:15	200.7
R-758-830	1.307	1100	01Jan2000, 13:15	271.3
R-759-830	1.298	1100	01Jan2000, 13:00	280.2
R-762-806	1.394	1400	01Jan2000, 13:10	317.0
R-788-821	1.772	2000	01Jan2000, 13:30	421.2
R-795-824	1.939	2000	01Jan2000, 12:55	397.5
R-806-902	2.049	1900	01Jan2000, 13:25	460.4
R-807-946	2.266	1700	01Jan2000, 13:20	431.5
R-813-820	2.328	700	01Jan2000, 15:05	179.7
R-820-1025	2.467	700	01Jan2000, 15:30	204.8
R-821-847	2.358	2300	01Jan2000, 13:40	548.3
R-824-846	2.529	0	01Jan2000, 00:00	0.0
R-830-856	2.754	2200	01Jan2000, 13:15	584.2
R-846-871	3.464	900	01Jan2000, 13:35	189.2
R-847-866	3.359	2900	01Jan2000, 13:55	778.0
R-871-874	4.794	1900	01Jan2000, 13:30	461.6
R-874-933	4.976	1800	01Jan2000, 13:50	490.5
R-880-889	5.460	3900	01Jan2000, 13:40	1162.8
R-889-902	6.621	4600	01Jan2000, 13:50	1394.4
R-902-912	9.166	6000	01Jan2000, 13:55	1937.4
R-912-914	11.291	7100	01Jan2000, 14:05	2370.2
R-914-922	11.543	7100	01Jan2000, 14:05	2408.8
R-922-930	15.917	9500	01Jan2000, 14:15	3349.6
R-930-933	20.166	10200	01Jan2000, 14:25	3812.7
R-933-935	25.902	11900	01Jan2000, 14:20	4438.8
R-935-1041	26.766	11800	01Jan2000, 14:35	4531.1
R-946-952	30.823	12900	01Jan2000, 14:45	5218.2
R-952-954	32.815	13300	01Jan2000, 14:45	5566.7
WATERLOO LAKE	2.529	0	01Jan2000, 00:00	0.0
1022	0.113	100	01Jan2000, 12:30	22.1
1023	1.178	1100	01Jan2000, 12:45	222.7
1024	0.254	300	01Jan2000, 12:35	50.5
1025	0.879	700	01Jan2000, 12:55	160.2
1026	0.723	700	01Jan2000, 12:40	136.0
1036	0.362	400	01Jan2000, 12:40	76.2
1041	0.698	700	01Jan2000, 12:45	138.7
601	0.187	300	01Jan2000, 12:30	43.2
608	0.198	300	01Jan2000, 12:35	46.8
609	0.162	200	01Jan2000, 12:30	33.8
610	0.175	300	01Jan2000, 12:25	44.0
613	0.297	400	01Jan2000, 12:35	69.2
619	0.271	400	01Jan2000, 12:30	59.5
620	0.278	300	01Jan2000, 12:40	64.6
622	0.288	400	01Jan2000, 12:30	59.8
626	0.314	400	01Jan2000, 12:40	66.9
629	0.324	600	01Jan2000, 12:25	88.4
633	0.360	400	01Jan2000, 12:35	72.7
644	0.438	600	01Jan2000, 12:40	102.8
649	0.451	400	01Jan2000, 12:35	77.4
656	0.488	500	01Jan2000, 12:40	105.4
657	0.477	700	01Jan2000, 12:35	120.2
660	0.515	500	01Jan2000, 12:50	106.5
666	0.209	300	01Jan2000, 12:30	42.5
670	0.601	600	01Jan2000, 12:55	136.0
673	0.614	500	01Jan2000, 12:50	114.0
676	0.635	600	01Jan2000, 12:50	132.2
677	0.186	200	01Jan2000, 12:25	33.0
680	0.668	700	01Jan2000, 12:50	160.4
681	0.687	800	01Jan2000, 12:35	145.2
683	0.755	1000	01Jan2000, 12:35	185.6
684	0.233	300	01Jan2000, 12:35	47.1
691	0.408	600	01Jan2000, 12:35	91.8
694	0.079	100	01Jan2000, 12:20	16.8

Attachment 8
Model Results - Peak Discharge and Runoff Volume

10-Year, 24-Hour Design Storm				
Hydrologic Element	Drainage Area (mi^2)	Peak Discharge (cfs)	Time of Peak	Volume (ac-ft)
695	0.754	800	01Jan2000, 12:45	157.4
696	0.757	800	01Jan2000, 12:50	173.3
698	0.765	700	01Jan2000, 12:55	165.9
700	0.793	1000	01Jan2000, 12:35	186.0
702	0.838	800	01Jan2000, 13:00	190.1
721	1.001	1100	01Jan2000, 12:45	237.6
731	0.089	200	01Jan2000, 12:20	18.7
736	1.097	900	01Jan2000, 12:50	201.4
740	1.060	1000	01Jan2000, 12:40	203.5
758	0.156	200	01Jan2000, 12:30	36.2
759	0.262	300	01Jan2000, 12:35	59.6
762	0.393	400	01Jan2000, 12:40	82.2
772	0.112	200	01Jan2000, 12:25	22.1
788	0.286	400	01Jan2000, 12:30	65.7
795	0.083	100	01Jan2000, 12:20	15.6
806	0.054	100	01Jan2000, 12:20	12.0
807	0.637	600	01Jan2000, 12:40	124.7
813	0.282	400	01Jan2000, 12:30	67.3
820	0.139	200	01Jan2000, 12:20	29.1
821	0.399	500	01Jan2000, 12:35	87.6
824	0.152	200	01Jan2000, 12:30	34.7
830	0.149	300	01Jan2000, 12:25	33.9
846	0.252	300	01Jan2000, 12:30	52.9
847	1.002	1100	01Jan2000, 12:50	236.8
856	0.310	400	01Jan2000, 12:35	63.7
866	0.276	300	01Jan2000, 12:35	51.9
871	0.842	900	01Jan2000, 12:40	169.7
874	0.182	200	01Jan2000, 12:30	36.3
880	0.578	700	01Jan2000, 12:35	126.2
889	0.883	800	01Jan2000, 12:50	179.9
902	0.497	600	01Jan2000, 12:35	106.5
912	0.983	900	01Jan2000, 12:55	201.1
914	0.252	300	01Jan2000, 12:40	47.2
922	0.006	10	01Jan2000, 12:15	0.8
930	0.181	200	01Jan2000, 12:30	32.3
933	0.760	800	01Jan2000, 12:45	150.5
935	0.111	100	01Jan2000, 12:30	22.6
946	0.044	100	01Jan2000, 12:25	7.7
952	0.894	900	01Jan2000, 12:45	175.3
954	0.201	200	01Jan2000, 12:30	39.4
Pawpaw Creek				
J-1897	1.057	1600	01Jan2000, 12:35	251.5
J-1906	0.441	500	01Jan2000, 12:40	100.1
J-1907	0.488	600	01Jan2000, 12:35	104
J-1908	0.520	600	01Jan2000, 12:40	111.2
J-1969	2.002	2400	01Jan2000, 13:05	446.2
J-1979	2.257	2500	01Jan2000, 13:20	497.3
J-1984	2.705	2800	01Jan2000, 13:20	598.5
J-1989	0.241	300	01Jan2000, 12:35	52
J-1990	2.966	3000	01Jan2000, 13:25	653.7
J-1998	3.521	3300	01Jan2000, 13:25	769.1
J-2003	4.029	3600	01Jan2000, 13:25	876.8
J-2061	0.586	1000	01Jan2000, 12:30	139.2
R-1897-1969	1.057	1600	01Jan2000, 13:05	248.9
R-1906-1984	0.441	500	01Jan2000, 12:45	100
R-1907-2003	0.488	600	01Jan2000, 12:40	103.8
R-1908-1998	0.520	600	01Jan2000, 12:45	111.1
R-1969-1979	2.002	2300	01Jan2000, 13:20	443.2
R-1979-1984	2.257	2500	01Jan2000, 13:20	497.2
R-1984-1990	2.705	2800	01Jan2000, 13:25	597.6
R-1989-1990	0.241	300	01Jan2000, 12:40	51.9
R-1990-1998	2.966	2900	01Jan2000, 13:25	653.1
R-1998-2003	3.521	3300	01Jan2000, 13:25	768.3
R-2061-1897	0.586	1000	01Jan2000, 12:40	138.6
1897	0.471	800	01Jan2000, 12:25	113
1906	0.441	500	01Jan2000, 12:40	100.1
1907	0.488	600	01Jan2000, 12:35	104
1908	0.520	600	01Jan2000, 12:40	111.2
1969	0.945	900	01Jan2000, 12:50	197.4
1979	0.255	300	01Jan2000, 12:40	54.1
1984	0.007	20	01Jan2000, 12:10	1.4
1989	0.241	300	01Jan2000, 12:35	52
1990	0.020	40	01Jan2000, 12:20	4.2
1998	0.035	40	01Jan2000, 12:20	4.9

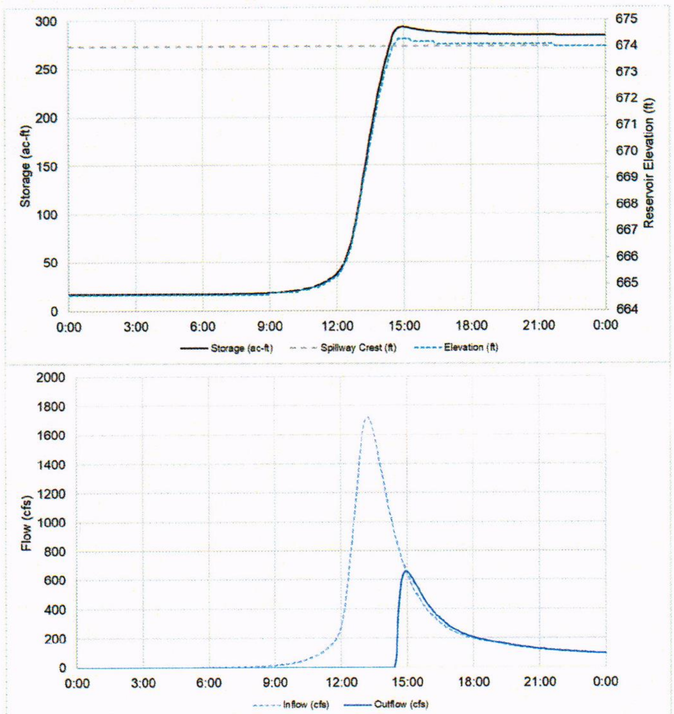
Attachment 8
Model Results - Peak Discharge and Runoff Volume

10-Year, 24-Hour Design Storm				
Hydrologic Element	Drainage Area (mi^2)	Peak Discharge (cfs)	Time of Peak	Volume (ac-ft)
2003	0.019	100	01Jan2000, 12:10	4.7
2061	0.586	1000	01Jan2000, 12:30	139.2
Shawnee Creek				
J-1899	2.380	2700	01Jan2000, 12:50	556.4
J-1900	0.160	200	01Jan2000, 12:30	36.1
J-1920	1.010	1200	01Jan2000, 12:40	224.6
J-1940	1.160	1100	01Jan2000, 12:50	259.2
J-1941	1.160	1100	01Jan2000, 12:55	257.7
J-1943	1.230	1300	01Jan2000, 12:40	258.1
J-1950	0.450	600	01Jan2000, 12:40	109.3
J-1971	2.050	2100	01Jan2000, 12:55	442.4
J-1978	2.240	2100	01Jan2000, 13:00	507.9
J-1992	0.480	600	01Jan2000, 12:40	115.7
J-1993	3.050	3100	01Jan2000, 13:00	672.4
J-1995	3.260	3500	01Jan2000, 13:05	759.4
J-1996	3.360	3300	01Jan2000, 13:00	739.9
J-1999	3.560	3300	01Jan2000, 13:05	814.0
J-2016	5.290	4500	01Jan2000, 13:15	1187.4
J-2032	9.300	6800	01Jan2000, 13:30	2050.8
J-2037	10.170	7200	01Jan2000, 13:30	2238.6
J-2039	11.220	2400	01Jan2000, 16:10	1429.1
J-2045	15.320	4700	01Jan2000, 13:25	2312.8
J-2065	2.500	2300	01Jan2000, 13:05	568.3
LAKE RANDELL	10.170	2300	01Jan2000, 16:05	1233.7
R-1899-1995	2.380	2700	01Jan2000, 13:10	552.6
R-1900-2065	0.160	200	01Jan2000, 12:40	36.0
R-1920-1899	1.010	1200	01Jan2000, 12:50	223.6
R-1940-1978	1.160	1100	01Jan2000, 13:10	257.3
R-1941-2016	1.160	1000	01Jan2000, 13:10	255.2
R-1943-1971	1.230	1300	01Jan2000, 13:00	256.3
R-1950-1899	0.450	500	01Jan2000, 13:00	108.6
R-1971-1993	2.050	2100	01Jan2000, 13:00	441.5
R-1978-2065	2.240	2100	01Jan2000, 13:05	507.1
R-1992-1999	0.480	600	01Jan2000, 12:55	115.1
R-1993-1996	3.050	3100	01Jan2000, 13:05	670.8
R-1995-2045	3.260	3300	01Jan2000, 13:30	751.5
R-1996-2032	3.360	2600	01Jan2000, 13:25	717.6
R-1999-2016	3.560	3100	01Jan2000, 13:20	806.3
R-2016-2032	5.290	3900	01Jan2000, 13:35	1166.5
R-2032-2037	9.300	6700	01Jan2000, 13:35	2039.3
R-2037-2039	10.170	2300	01Jan2000, 16:15	1219.9
R-2039-2045	11.220	2400	01Jan2000, 16:35	1401.4
R-2065-1999	2.500	2300	01Jan2000, 13:15	566.5
1899	0.930	1200	01Jan2000, 12:35	224.3
1900	0.160	200	01Jan2000, 12:30	36.1
1920	1.010	1200	01Jan2000, 12:40	224.6
1940	1.160	1100	01Jan2000, 12:50	259.2
1941	1.160	1100	01Jan2000, 12:55	257.7
1943	1.230	1300	01Jan2000, 12:40	258.1
1950	0.450	600	01Jan2000, 12:40	109.3
1971	0.820	900	01Jan2000, 12:45	186.1
1978	1.080	1200	01Jan2000, 12:45	250.6
1992	0.480	600	01Jan2000, 12:40	115.7
1993	1.000	1100	01Jan2000, 12:45	230.9
1995	0.870	900	01Jan2000, 12:55	206.8
1996	0.310	500	01Jan2000, 12:30	69.1
1999	0.580	600	01Jan2000, 12:50	132.5
2016	0.570	700	01Jan2000, 12:35	125.9
2032	0.650	1000	01Jan2000, 12:30	166.7
2037	0.870	1100	01Jan2000, 12:40	199.3
2039	1.050	1000	01Jan2000, 12:45	209.1
2045	0.840	800	01Jan2000, 12:40	159.9
2065	0.090	200	01Jan2000, 12:25	25.3
UNT0097				
J-1903	0.220	400	01Jan2000, 12:25	51.8
J-1937	1.133	1100	01Jan2000, 13:05	250.2
J-1964	1.942	1500	01Jan2000, 13:20	415.6
R-1903-1937	0.220	300	01Jan2000, 13:10	51
R-1937-1964	1.133	1100	01Jan2000, 13:30	247.1
1903	0.220	400	01Jan2000, 12:25	51.8
1937	0.913	800	01Jan2000, 12:55	199.2
1964	0.809	1000	01Jan2000, 12:35	168.5
UNT140				
J-UNT140	1.136	1200	01Jan2000, 12:40	238.3

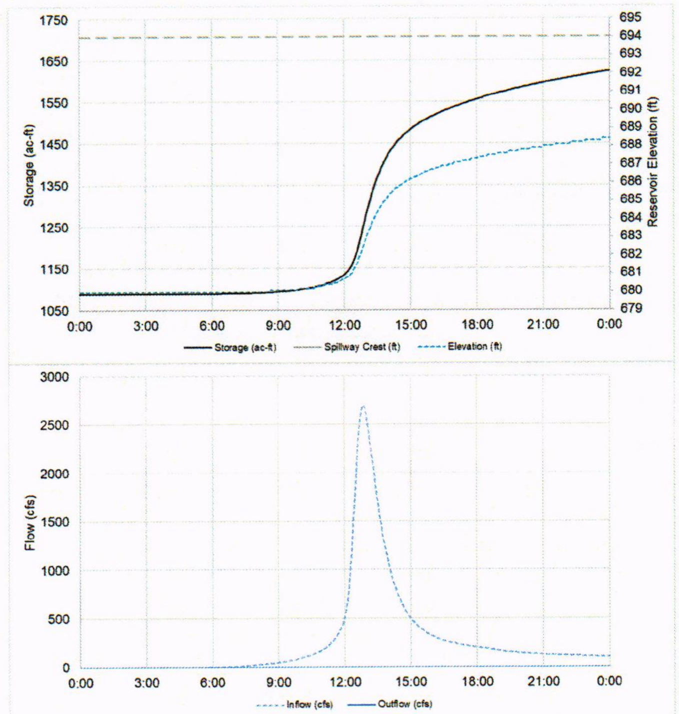
Attachment 8
Model Results - Peak Discharge and Runoff Volume

10-Year, 24-Hour Design Storm				
Hydrologic Element	Drainage Area (mi^2)	Peak Discharge (cfs)	Time of Peak	Volume (ac-ft)
J-631	0.347	400	01Jan2000, 12:35	71.5
J-699	0.790	800	01Jan2000, 12:45	166.8
631	0.347	400	01Jan2000, 12:35	71.5
699	0.790	800	01Jan2000, 12:45	166.8
UNT141				
J-745	1.079	1000	01Jan2000, 12:35	185
J-799	1.958	1700	01Jan2000, 13:00	357.8
R-745-799	1.079	1000	01Jan2000, 13:00	183
745	1.079	1000	01Jan2000, 12:35	185
799	0.879	800	01Jan2000, 12:45	174.8

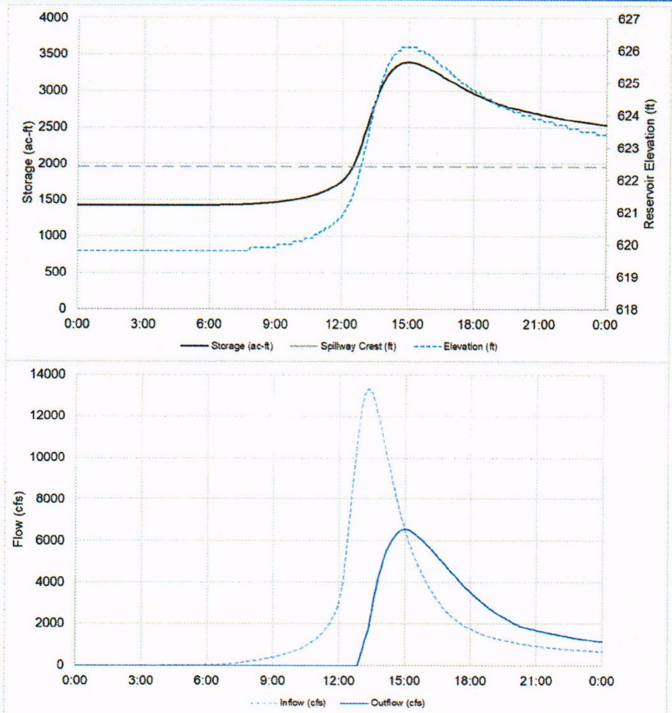
Attachment 9: Model Results – Reservoir Inflow and Outflow



Attachment 9A: Loy Lake Inflow and Outflow



Attachment 9B: Waterloo Lake Inflow and Outflow



Attachment 9C: Lake Randell Inflow and Outflow

Appendix E: FLO-2D Model Methods, Inputs, and Results

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1.0 Introduction

A two-dimensional flood model was developed to evaluate urban flooding near Downtown Denison. The model's purpose is to evaluate the flow patterns and approximate floodplain extents. The results of the model were used to evaluate potential improvement projects to resolve existing or potential future drainage-related issues.

1.1 Model Approach

The two-dimensional flood model uses FLO-2D software Build 19.07.21 (Ref. 1). The model was built using data processing in ArcGIS Pro, and the model was developed using the QGIS FlO-2D Plugin version 0.10.2. Upon completion of model runs, the results were processed into results layers in ArcGIS Pro. The model was built through an iterative process of running the model and updating the model to remove potential errors or make improvements to make the model more representative of the scenario being ran.

The model is developed for the purpose of planning drainage improvement projects and is not intended as a detailed floodplain study. More detailed analysis is required to accurately model urban flooding for the purpose of determining flood elevations resulting from intense storm events.

2.0 Gridded Data

The FLO-2D model is built around a two-dimensional square grid where each grid element is representative of the underlying terrain, including soil properties, elevations, and surface cover.

2.1 Domain & Grid Size

Within the study area for the Denison SWMP, the model domain contains a smaller area of about 12.65 square miles in the urban center of Denison. It is approximately bounded by Highway 503 to the east and south, highway US-75 to the west, and Martin Luther King Street to the North. The domain is shown in reference to the greater study area in Figure 1.

The model is defined as a grid with 20-foot spacing, resulting in a total of 880,663 computational cells. The 20-foot grid was used to strike a balance between model run-time and model accuracy.

2.2 Grid-Assigned Elevation

Each cell within the model grid is assigned an elevation. Topographic LiDAR data from the North and Central Texas LiDAR (Ref. 2) and the United States Geological Survey (USGS) (Ref. 3) were processed to assigned the average elevation within each cell. Where needed, grid elevations are adjusted to more accurately reflect the hydraulic conditions of a grid cell. This is discussed in-depth in Section 5.1. The final elevations assigned to each grid element are displayed in Figure 2.

2.3 Spatially-Varied Manning's n

Spatially-varied Manning's roughness coefficients (n) are assigned based on the land cover dataset from the NLCD (Ref. 4). The spatially varied land cover data is spatially averaged within a cell to determine the average value for a cell. The roughness values corresponding to each land cover designation are tabulated in Table 1. A figure displaying the spatially-distributed roughness assigned in the model is provided in Figure 3.

Table 1: Assigned Manning's n for each Land Cover Type

NLCD Code (2019)	Description	Assigned Value, n
11	Open Water	0.035
21	Developed, Open Space	0.040
22	Developed, Low Intensity	0.085
23	Developed, Medium Intensity	0.120
24	Developed, High Intensity	0.150
31	Barren Land	0.030
41	Deciduous Forest	0.160
42	Evergreen Forest	0.160
43	Mixed Forest	0.160
52	Shrub/Scrub	0.130
71	Herbaceous	0.050
81	Hay/Pasture	0.045
82	Cultivated Crops	0.045
90	Woody Wetlands	0.125
95	Emergent Herbaceous Wetlands	0.085

2.4 Gridded Boundary Conditions

Gridded boundary conditions include specifying cells as points of inflow, outflow, or reservoirs. Gridded Inflow hydrographs and reservoir water surface elevations are included in the INFLOW.DAT file while the model outflow grid points are specified in the OUTFLOW.DAT file. The locations of gridded inflow and outflow to the flood model are shown in Figure 4.

2.4.1 Inflow Grid

At points of inflow, a user-defined hydrograph is defined at each point where inflow from outside the model domain is expected. Four sets of inflow points have been specified at these locations. All four locations are at the western boundary of the model. The flood hydrograph from the HEC-HMS hydrology model are used to determine the discharge hydrograph at each inflow point. In areas where only part of a sub-basin from the HEC-HMS model is used, the hydrograph from the partial sub-basin is multiplied by the fraction of the partial contributing area to the total sub-basin area.

Multiple grid elements are specified at an inflow location to distribute flow across a stream cross-section. For these locations, the total inflow hydrograph is divided evenly between points. At inflow elements, the time series from the HEC-HMS model is modified accordingly to produce the inflow hydrograph on a "per element" basis. The inflow boundary condition time series data is provided in Attachment 1.

$$Q_{in}(t) = \frac{Q_{HMS}(t)}{A_{HMS}} A_{in} * \left(\frac{1}{i_{in}}\right)$$

Where:

$Q_{in}(t)$	=	Inflow at time, t, at each inflow grid element (cfs/grid element)
$Q_{HMS}(t)$	=	Modeled Flow at time, t, from HMS Model
A_{HMS}	=	Area of Contributing Drainage to Junction of Interest (mi^2)
A_{in}	=	Partial Area of Contributing Drainage Upstream of Boundary (mi^2)
i_{in}	=	Number of Inflow Grid Elements for Specified Boundary Condition

Table 2: Summary of Inflow Boundary Condition Locations

Inflow Location ID	HMS Model Reference Element	Modeled 100-year Peak Flow	Total Contributing Drainage Modeled (mi^2)	Area Outside Model Boundary (mi^2)	Fraction of Modelled Flow	Boundary Peak Flow (cfs)	Number of Elements
Junction 649	J-649	785	0.450796	0.450796	1	785	4
Sub-Basin 700 (Partial)	700	1683	0.792728	0.29	0.37	616	5
Sub-Basin 1025 (Partial)	1025	1250	0.878548	0.3	0.34	427	6
Junction 820	J-820	2229	2.466873	2.466873	1	2229	13

2.4.2 Reservoir Initial Water Surface

Reservoir initial water surface conditions are assigned to any cell inside a reservoir within the model domain. Waterloo Lake is the only reservoir modelled within the domain, and was modelled without specifying an initial water surface. Without an initial water surface elevation, the scenario simulates that the water surface level is at the level when the topographic survey was made (since LiDAR surveys only include bare ground, the survey plateaus at the water surface elevation during the survey).

1.1.1.1 Reservoir Water Surface Discussion

The National Inventory of Dams (Ref. 5) lists that the storage capacity of Waterloo Lake is 2,545 acre-feet, while the normal water storage is 99 acre-feet. Based on the StratMap topographic data used in the model, the water surface level is 678-feet above mean sea level, setting the "ground elevation" within the reservoir at that elevation (678-feet). The elevation of 678-feet corresponds to approximately 1,000 acre-feet of storage, which is about 900 acre-feet greater than the normal water storage, but about 1,545 acre-feet less than the total reservoir capacity. Since there is no bathymetric survey below the water surface at 678-feet, and since the available storage above 678-feet is greater than the 100-year, 24-hour runoff volume contributing to Waterloo Lake, any water surface at or below this 678-foot elevation will not produce runoff on the Waterloo Lake Spillway. As such, no initial water surface elevation is specified for Waterloo Lake, which simulates an initial water surface of 678-feet. Under this scenario, the water level in Waterloo Lake does not outflow from the spillway, meaning that none of the 2.3 square miles of drainage upstream of Waterloo Lake contributes to flooding downstream of Waterloo Lake during the simulation.

2.4.3 Outflow Grid

Outflow boundary conditions were specified at every cell at the boundary of the simulation. The outflow grid elements are all specified as floodplain outflow nodes with normal flow conditions. Under this specification, all flow that reaches the outflow grid cells is removed from the model.

2.5 Rainfall

Gridded rainfall data from NOAA Atlas 14 was used to develop a synthetic, center-peaking 24-hour rainfall distribution. The 100-year 24-hour storm was used as the design storm. As most of the model domain falls within the drainage basin for Iron Ore Creek from the HEC-HMS hydrology model, the rainfall distribution for Iron Ore Creek was applied with equal distribution to all model grid elements in the model domain. The design hyetograph for the 100-year, 24-hour frequency storm is shown in Figure 5.

2.6 Losses

Infiltration losses were estimated using the SCS Curve Number (CN) Method (Ref. 6) for existing land use data and soil survey data. CN is an empirical loss parameter which represents the runoff potential for a drainage area. Curve numbers are a function of the soil type and the cover conditions for a given drainage area. The inputs for the CN method include the CN value and an initial abstraction value. An average global loss input was calculated for the model domain based on the land cover (Ref. 4) and soils data (Ref. 7) within the model domain using the relationships tabulated in Table 3. Under the CN method, runoff is calculated as a function of rainfall, CN, and initial abstraction.

$$Q = \frac{(P - I_a)^2}{(P - I_a) + \left(\frac{1000}{CN} - 10\right)}$$

Where:

Q = Runoff (inches)

P = Rainfall (inches)

I_a = Initial Abstraction (inches)

CN = Curve Number

The initial abstraction is approximated by the following equation:

$$I_a = 0.2\left(\frac{1000}{CN} - 10\right) - \ell_d$$

Where:

ℓ_d = Surface Detention Losses (inches)

The initial abstraction typically includes all losses before runoff begins within the model – surface detention losses, water interception by vegetation, evaporation, and infiltration; however, in FLO-2D, since surface detention losses are accounted for in the TOLER.DAT file, the initial abstraction is modified to not include the surface detention loss, which is 0.004 feet. An average uniform CN for the model domain was determined to be 79, with an initial abstraction of 0.48 inches.

Table 3: Curve Number for Given NLCD Cover Type and Hydrologic Soil Group

NLCD Value (2011)	NLCD Description	TX-DOT Description	Curve Number			
			A	B	C	D
11	Open Water	Water	100	100	100	100
21	Developed, Open Space	Open Space - Good Condition	39	61	74	80
22	Developed, Low Intensity	Residential - 1/4 acre	61	75	83	87
23	Developed, Medium Intensity	Residential - 1/8 acre	77	85	90	92
24	Developed, High Intensity	Commercial and Business	89	92	94	95
31	Barren Land	Saltbrush - Good	49	68	79	84
41	Deciduous Forest	Woods grass combination- Fair	43	65	76	82
42	Evergreen Forest	Woods - Fair	36	60	73	79
43	Mixed Forest	Woods grass combination- Fair	43	65	79	84
52	Shrub/Scrub	Brush - Fair	35	56	70	77
71	Herbaceous	Herbaceous-Fair	60	71	81	89

NLCD Value (2011)	NLCD Description	TX-DOT Description	Curve Number			
			A	B	C	D
81	Hay/Pasture	Pasture - Good	39	61	74	80
82	Cultivated Crops	Row crops (SR) - Good	67	78	85	89
90	Woody Wetlands	Water	100	100	100	100
95	Emergent Herbaceous Wetlands	Water	100	100	100	100

2.7 Modeling of Buildings (ARFs)

Polygons representing buildings (Ref. 8) were used to assigned Area Reduction Factors (ARFs) to grid elements with buildings that would restrict flow within each 20-foot square grid. ARFs represent obstructions to floodplain storage, and they range in value from 0 (representing no obstructions) to 1 (fully obstructed grid element). ARFs are important in urban hydrology, where buildings will commonly constrict flow, forcing flow to route around structures.

3.0 Model Control Parameters

The model control parameters are found in the CONT.DAT and TOLER.DAT files. The CONT.DAT file contains the system and global modeling parameters in addition to specifying the output files produced by the model run. The TOLER.DAT file defines the tolerance and numerical stability values and govern model stability. The input control parameters are tabulated in Attachment 2.

4.0 Hydraulic Structures

Hydraulic structures were used to incorporate structures at locations where they are needed to route flow downstream. Because the model's purpose is to be used for planning, only hydraulic structures that meaningfully impact the flooding hazards for the 100-year storm were included in this model. This was accomplished iteratively, starting with no hydraulic structures modeled, where structures were added as need to pass flow downstream and prevent excessive ponding at the upstream side of a roadway. This is discussed more in-depth in Section 5.2. Data on existing storm drain and culvert structures were provided by the City of Denison, but much of the data lacks pipe dimensions or data on inverts. Where detailed information was lacking, inverts were assumed based on the topographic data at each hydraulic structure, and pipe sizes were assumed based on photographs that accompanied the stormwater infrastructure data from the City of Denison.

4.1 Hydraulic Structure Inputs

Hydraulic structures are contained in the HYSTRUC.DAT file, and the hydraulic structure data is summarized in Attachment 3. The input data for each hydraulic structure represents the hydraulic conditions present for each culvert crossing, including pipe size, roughness, and losses. As a simplification, each hydraulic structure was analyzed with the assumption of inlet control at each culvert. This assumption means hydraulic capacity is calculated based solely on the water elevation at the inlet floodplain location. As a result, no tailwater effects or potential upstream flow are considered. Additionally, the area for each culvert was reduced by 50% to account for potential clogging of up to 50% during a flood event.

5.0 Model Adjustments

Model adjustments were iteratively made to improve the accuracy of the results of the FLO-2D model. The adjustments made included adjusting grid elevations and adding hydraulic structures were needed to better represent the hydraulic conditions in certain areas of the model.

5.1 Grid Elevation Adjustments

Grid elevations were adjusted in areas where the average elevation value does not reflect the hydraulic conditions at that grid location. Example areas include the inlets and outlets for hydraulic structures and channels.

5.2 Hydraulic Structures

Hydraulic structures were iteratively added into the model to more accurately model flow traveling through hydraulic structures. The process for adding hydraulic structures involved running the model simulation, plotting the maximum depth, and determine which areas of significant flooding are just upstream of a hydraulic structure. Once hydraulic structures were identified, the approximate length and size were estimated based on topography data and field photographs. For culvert crossings with multiple barrels, a single barrel with an equivalent cross-sectional area is specified. While this process does not account for all hydraulic structures in the model domain, it was sufficient to model the flow patterns within the model domain.

6.0 Results

The results of the flood modelling show the extents of flooding within the model domain. Areas that show high flood impacts were identified as potential locations for improvement projects as part of the SWMP. Flooding results for maximum depth (Attachment 4) and maximum velocity (Attachment 5) during the 100-year, 24-hour flood are included within this appendix, and more in-depth flood maps are included at areas of potential projects in the main report.

7.0 REFERENCES

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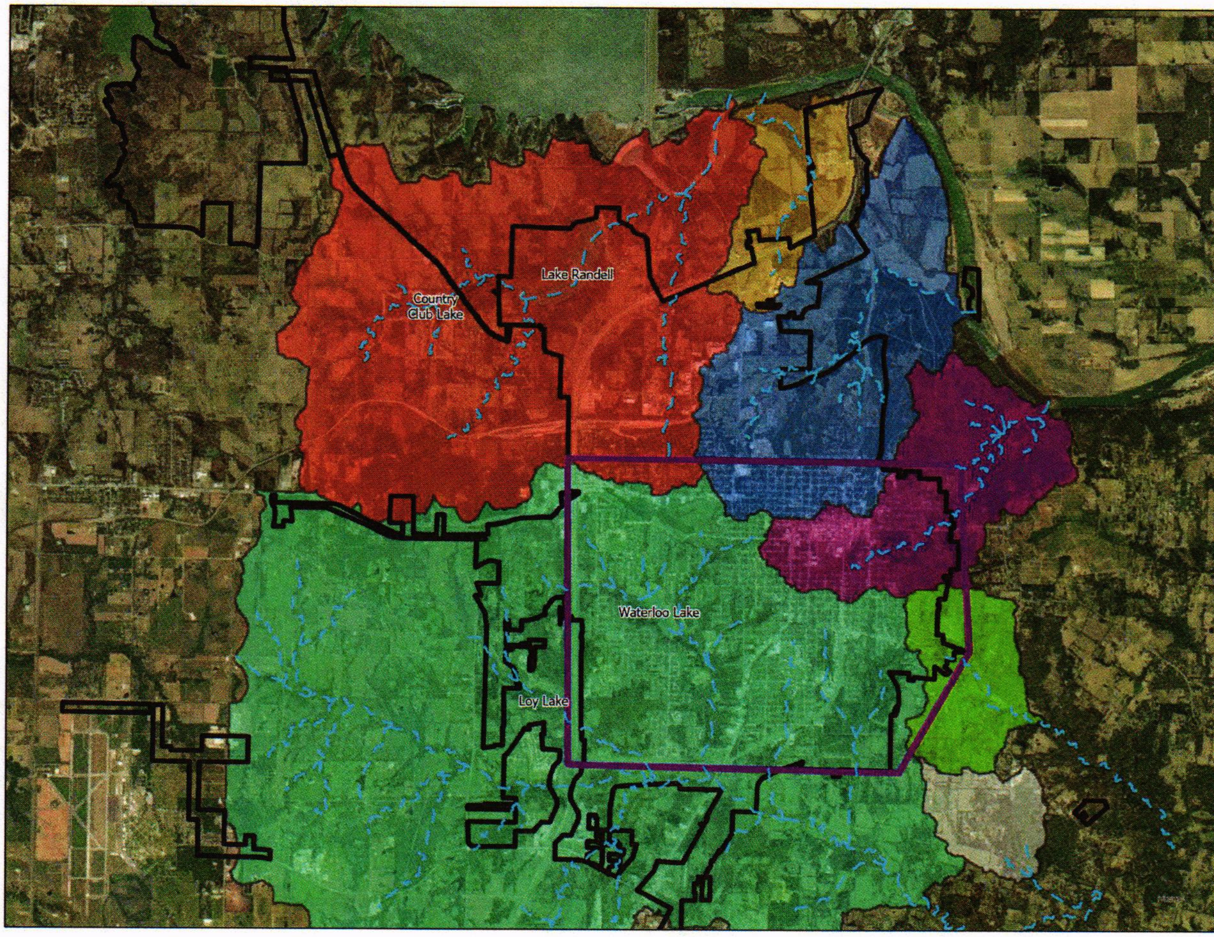
FIGURES

Figure 1: Model Domain



CITY OF DENISON
300 W Main St
Denison, TX 75020

Denison
TEXAS

City of Denison
Conceptual Stormwater Master Plan
R315880.01

Legend
- Stream Lines
- FLO-2D Domain
- Denison City Limit

Basin
- DUCK CREEK
- IRON ORE CREEK
- PAWPAW CREEK
- SHAWNEE CREEK
- UNT0097
- UNT140
- UNT141

N

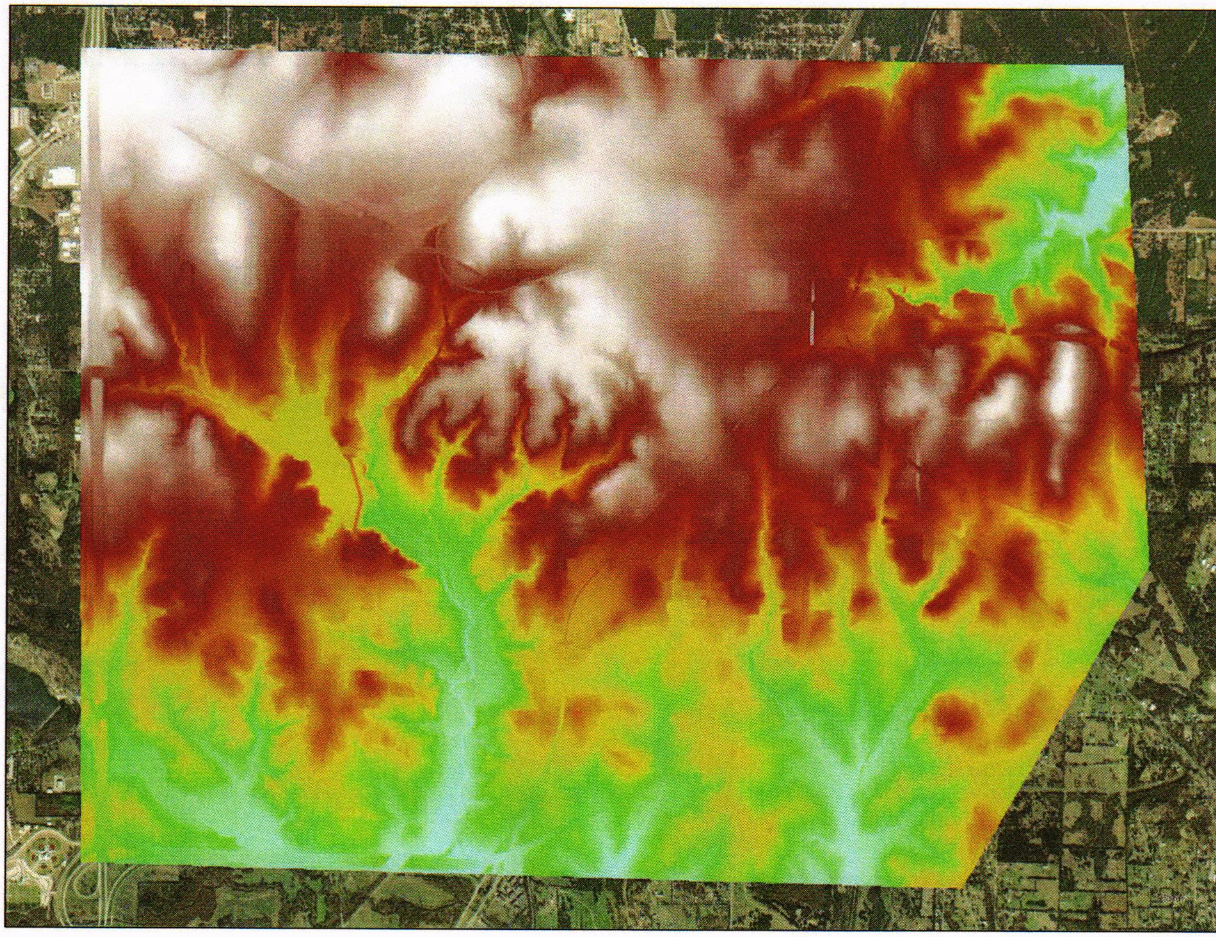
Appendix E
Figure 1

Model Domain

0 1 2 Miles

HUITT-ZOLLARS
5050 N 40th Street, Suite 100
Phoenix, Arizona 85018

Figure 2: Elevation Map



CITY OF DENISON

300 W Main St

Denison, TX 75020

City of Denison
Conceptual Stormwater Master Plan
R315880.01

Legend

Elevation (ft)

822.5
774.5
756.6
738.8
720.9
703.0
685.2
667.3
649.5
631.6
584.5

N

Appendix E
Figure 2

Elevation Map

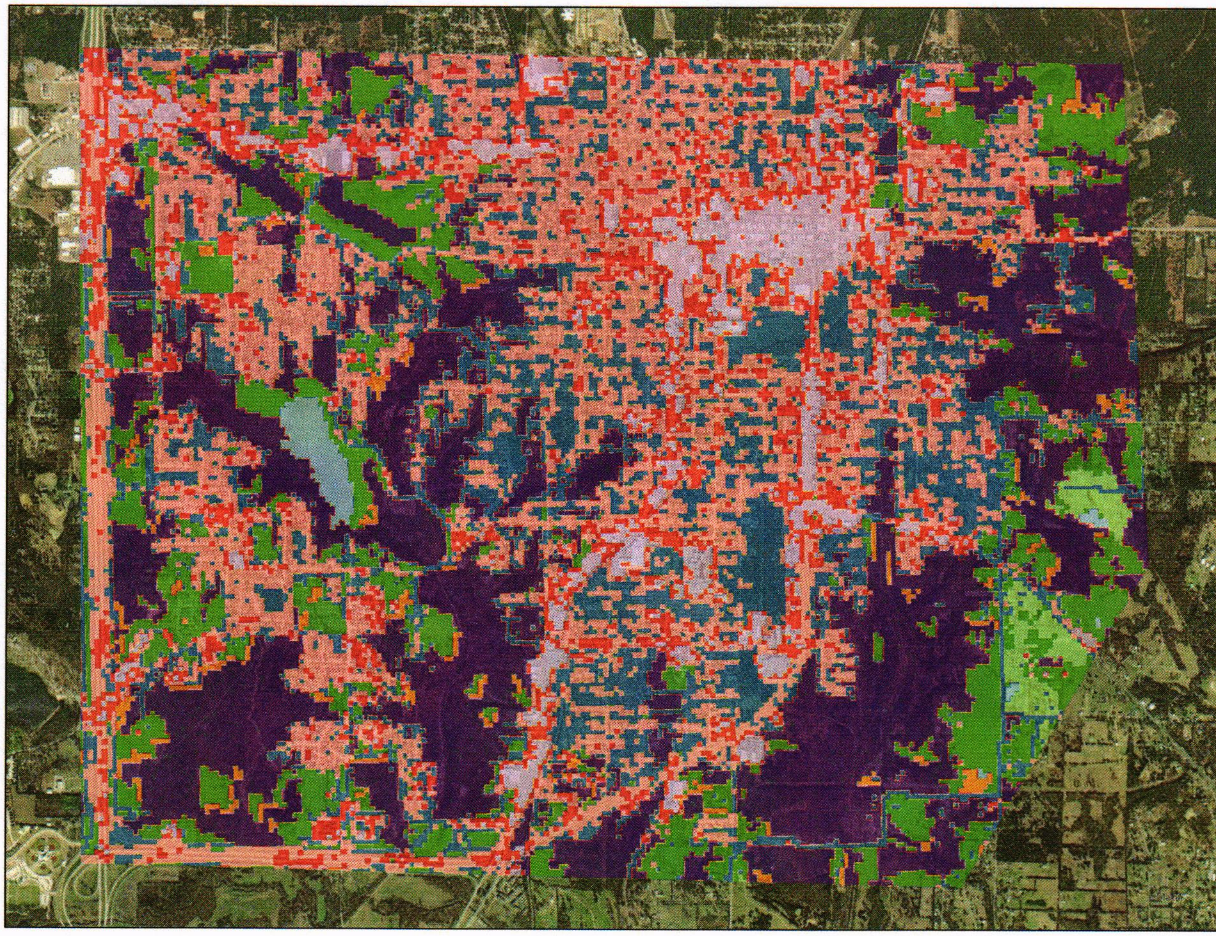
0 0.25 0.5 Miles

HUITT-ZOLLARS

5050 N 40th Street, Suite 100

Phoenix, Arizona 85018

Figure 3: Assigned Roughness



CITY OF DENISON
300 W Main St
Denison, TX 75020

City of Denison
Conceptual Stormwater Master Plan
R315880.01

Legend

- 0.031 - 0.035 (Barren Land & Open Water)
- 0.036 - 0.04 (Developed, Open Space)
- 0.041 - 0.045 (Hay/Pasture/ Cultivated Crops)
- 0.046 - 0.05 (Herbaceous)
- 0.051 - 0.085 (Developed, Low Intensity)
- 0.086 - 0.12 (Developed, Medium Intensity)
- 0.121 - 0.125 (Woody Wetlands)
- 0.126 - 0.13 (Shrub/Scrub)
- 0.131 - 0.15 (Developed, High Intensity)
- 0.151 - 0.16 (Forest)

N

Appendix E
Figure 3

Assigned Roughness

0 0.25 0.5 Miles

HUITT-ZOLLARS
5050 N 40th Street, Suite 100
Phoenix, Arizona 85018

Figure 4: Boundary Conditions

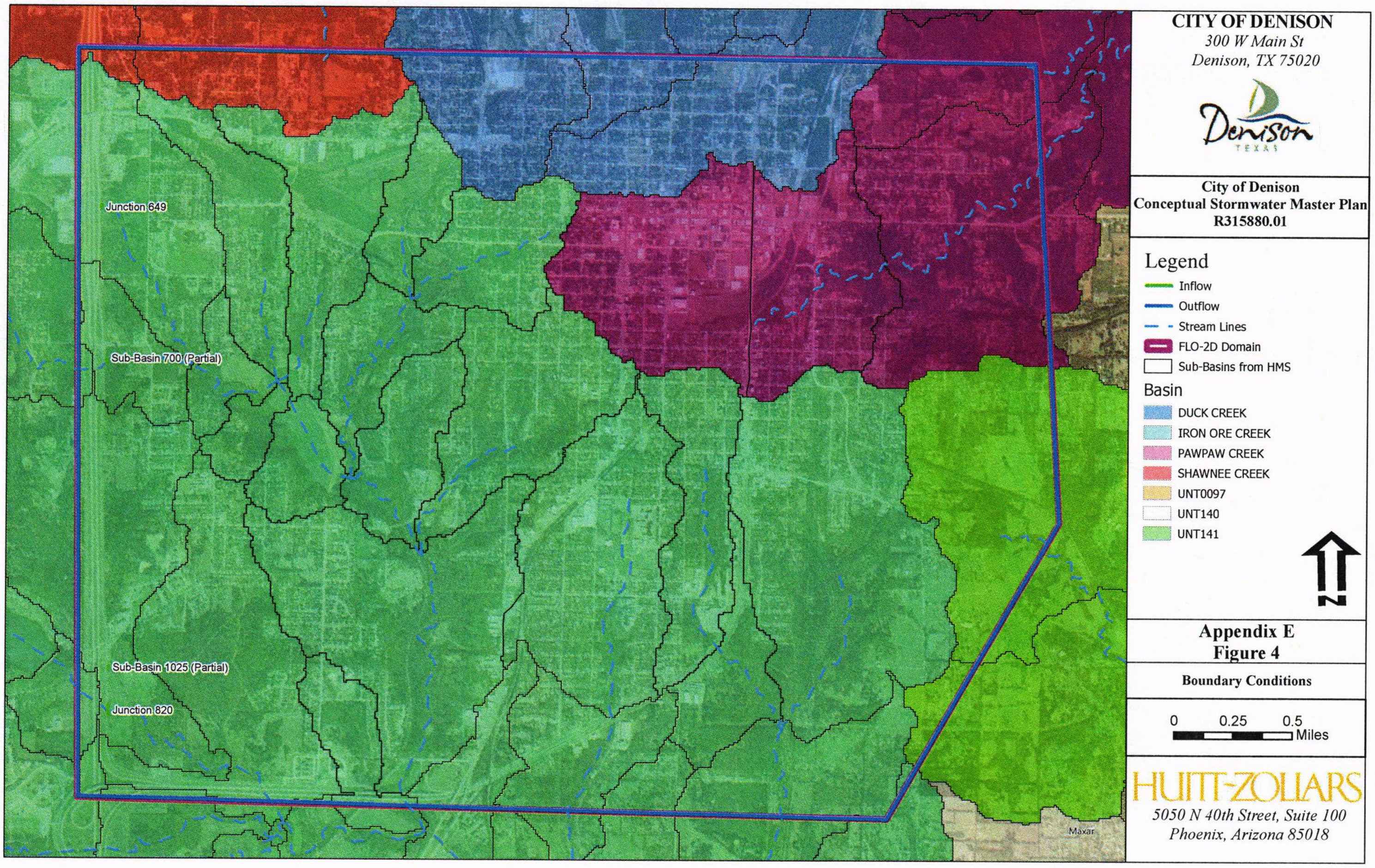

Junction 649
Sub-Basin 700 (Partial)
Sub-Basin 1025 (Partial)
Junction 820
Maxar
CITY OF DENISON
300 W Main St
Denison, TX 75020
Denison TEXAS
City of Denison
Conceptual Stormwater Master Plan
R315880.01
Legend
Inflow
Outflow
Stream Lines
FLO-2D Domain
Sub-Basins from HMS
Basin
DUCK CREEK
IRON ORE CREEK
PAWPAW CREEK
SHAWNEE CREEK
UNT0097
UNT140
UNT141
N
Appendix E
Figure 4
Boundary Conditions
0 0.25 0.5
Miles
HUITT-ZOLLARS
5050 N 40th Street, Suite 100
Phoenix, Arizona 85018

Figure 5: Design Storm Hyetograph

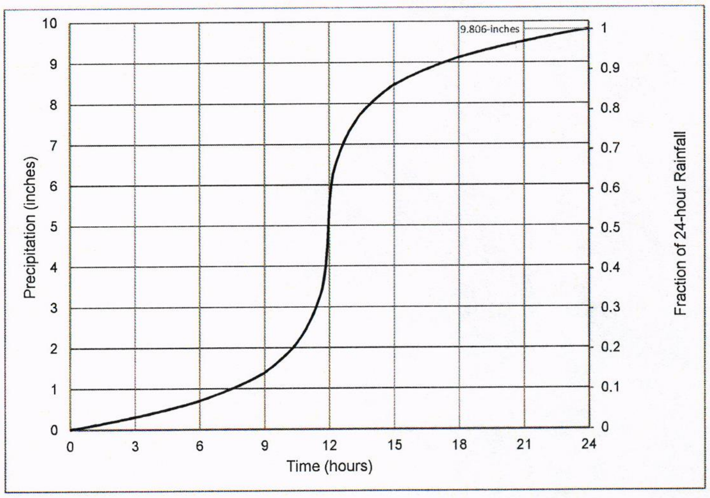
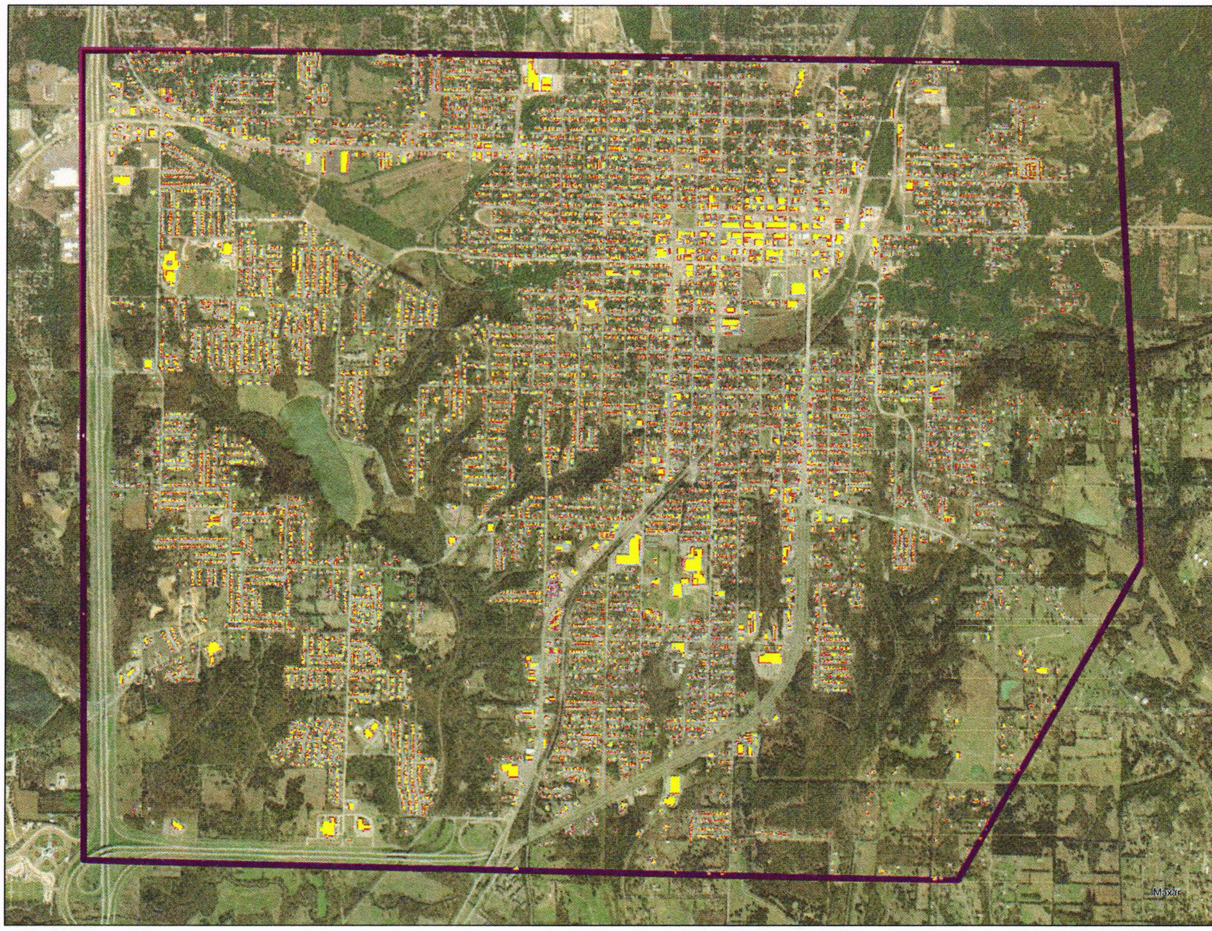


Figure 6: Building Area Reduction Factors



CITY OF DENISON
300 W Main St
Denison, TX 75020

City of Denison
Conceptual Stormwater Master Plan
R315880.01

Legend

ARF

- 0.008 - 0.2
- 0.201 - 0.4
- 0.401 - 0.6
- 0.601 - 0.8
- 0.801 - 0.93
- FLO-2D Domain

N

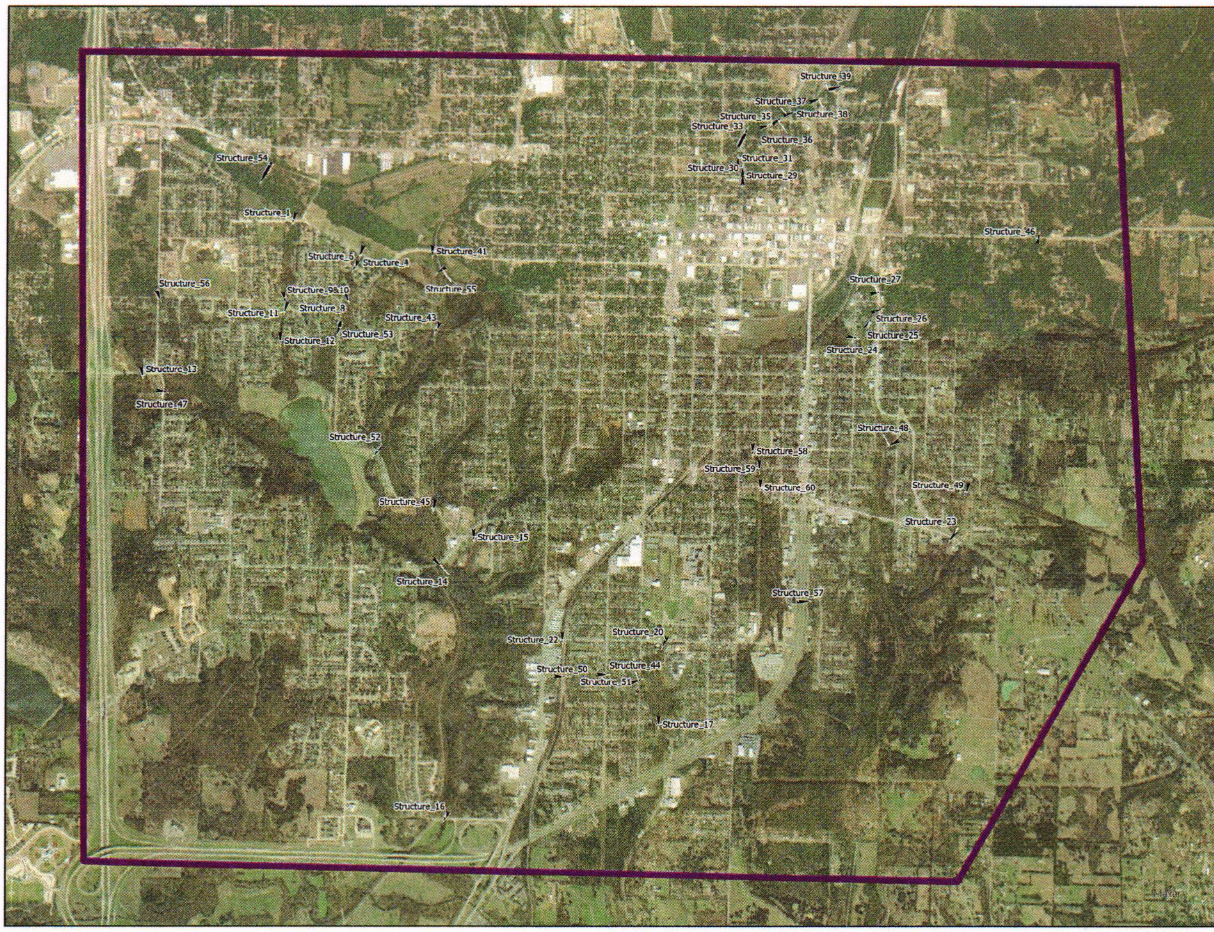
Appendix E
Figure 6

Building Area Reduction Factors

0 0.25 0.5 Miles

HUITT-ZOLLARS
5050 N 40th Street, Suite 100
Phoenix, Arizona 85018

Figure 7: Hydraulic Structure Map



CITY OF DENISON
300 W Main St
Denison, TX 75020

Denison TEXAS

City of Denison
Conceptual Stormwater Master Plan
R315880.01

Legend
- Hydraulic Structure
- FLO-2D Domain

N

Appendix E
Figure 7

Hydraulic Structure Map

0 0.25 0.5 Miles

HUITT-ZOLLARS
5050 N 40th Street, Suite 100
Phoenix, Arizona 85018

ATTACHMENTS

Attachment 1: Inflow Boundary Conditions Data

Time (hour)	Total Inflow Hydrograph (cfs)				Inflow per Element (cfs)			
	Junction 649	Sub-Basin 700 (Partial)	Sub-Basin 1025 (Partial)	Junction 820	Junction 649	Sub-Basin 700 (Partial)	Sub-Basin 1025 (Partial)	Junction 820
0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
0.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
0.2	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
0.3	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
0.4	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
0.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
0.6	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
0.7	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
0.8	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
0.9	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
1	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
1.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
1.2	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
1.3	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
1.4	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
1.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
1.6	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
1.7	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
1.8	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
1.9	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
2	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
2.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
2.2	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
2.3	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
2.4	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
2.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
2.6	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
2.7	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
2.8	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
2.9	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
3	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
3.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
3.2	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
3.3	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
3.4	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
3.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
3.6	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
3.7	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
3.8	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
3.9	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
4	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
4.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
4.2	0.0	0.1	0.0	0.0	0.0	0.0	0.0	0.0
4.3	0.0	0.4	0.0	0.0	0.0	0.1	0.0	0.0
4.4	0.0	0.4	0.0	0.0	0.0	0.1	0.0	0.0
4.5	0.0	0.7	0.0	0.0	0.0	0.1	0.0	0.0
4.6	0.0	0.7	0.0	0.0	0.0	0.1	0.0	0.0
4.7	0.0	0.9	0.0	0.0	0.0	0.2	0.0	0.0
4.8	0.0	1.1	0.0	0.0	0.0	0.2	0.0	0.0
4.9	0.0	1.4	0.0	0.0	0.0	0.3	0.0	0.0
5	0.0	1.8	0.0	0.0	0.0	0.4	0.0	0.0
5.1	0.0	1.9	0.0	0.0	0.0	0.4	0.0	0.0
5.2	0.0	2.2	0.0	0.0	0.0	0.4	0.0	0.0
5.3	0.0	2.4	0.0	0.0	0.0	0.5	0.0	0.0
5.4	0.0	2.9	0.0	0.0	0.0	0.6	0.0	0.0
5.5	0.0	2.9	0.0	0.0	0.0	0.6	0.0	0.0
5.6	0.0	3.4	0.0	0.2	0.0	0.7	0.0	0.0
5.7	0.0	3.8	0.0	1.0	0.0	0.8	0.0	0.1
5.8	0.0	4.0	0.0	1.0	0.0	0.8	0.0	0.1
5.9	0.0	4.3	0.0	1.0	0.0	0.9	0.0	0.1
6	0.0	4.8	0.0	1.0	0.0	1.0	0.0	0.1
6.1	0.0	5.1	0.0	1.0	0.0	1.0	0.0	0.1
6.2	0.0	5.3	0.0	1.0	0.0	1.1	0.0	0.1
6.3	0.0	5.7	0.0	1.6	0.0	1.1	0.0	0.1
6.4	0.0	6.5	0.0	2.0	0.0	1.3	0.0	0.2
6.5	0.0	7.0	0.0	2.0	0.0	1.4	0.0	0.2
6.6	0.0	7.4	0.3	2.0	0.0	1.5	0.1	0.2
6.7	0.0	7.9	0.3	2.4	0.0	1.6	0.1	0.2
6.8	0.0	8.5	0.5	3.0	0.0	1.7	0.1	0.2
6.9	0.0	9.1	0.7	3.0	0.0	1.8	0.1	0.2
7	1.0	9.5	0.7	3.0	0.3	1.9	0.1	0.2
7.1	1.0	10.4	1.1	3.2	0.3	2.1	0.2	0.2
7.2	1.0	10.8	1.4	4.0	0.3	2.2	0.2	0.3

Time (hour)	Total Inflow Hydrograph (cfs)				Inflow per Element (cfs)			
	Junction 649	Sub-Basin 700 (Partial)	Sub-Basin 1025 (Partial)	Junction 820	Junction 649	Sub-Basin 700 (Partial)	Sub-Basin 1025 (Partial)	Junction 820
7.3	1.6	11.2	1.6	4.0	0.4	2.2	0.3	0.3
7.4	2.0	12.0	2.0	4.0	0.5	2.4	0.3	0.3
7.5	2.0	12.5	2.0	4.0	0.5	2.5	0.3	0.3
7.6	3.0	13.0	2.5	5.0	0.8	2.6	0.4	0.4
7.7	3.4	13.7	2.9	5.0	0.9	2.7	0.5	0.4
7.8	4.0	14.4	3.3	5.0	1.0	2.9	0.5	0.4
7.9	4.8	15.0	3.7	5.0	1.2	3.0	0.6	0.4
8	5.0	15.8	4.1	6.0	1.3	3.2	0.7	0.5
8.1	6.0	16.3	4.5	6.0	1.5	3.3	0.8	0.5
8.2	6.4	17.0	4.9	6.4	1.6	3.4	0.8	0.5
8.3	7.6	17.7	5.3	7.0	1.9	3.5	0.9	0.5
8.4	8.0	18.6	5.7	7.0	2.0	3.7	1.0	0.5
8.5	9.0	19.1	6.1	7.0	2.3	3.8	1.0	0.5
8.6	10.0	20.0	6.9	8.0	2.5	4.0	1.1	0.6
8.7	10.4	20.9	7.3	8.0	2.6	4.2	1.2	0.6
8.8	11.6	21.7	7.9	8.6	2.9	4.3	1.3	0.7
8.9	12.0	22.6	8.5	9.0	3.0	4.5	1.4	0.7
9	13.0	23.5	9.2	9.0	3.3	4.7	1.5	0.7
9.1	14.2	24.4	9.7	10.0	3.6	4.9	1.6	0.8
9.2	15.4	25.5	10.5	10.4	3.9	5.1	1.8	0.8
9.3	17.2	27.1	11.3	11.6	4.3	5.4	1.9	0.9
9.4	18.8	28.7	12.1	12.8	4.7	5.7	2.0	1.0
9.5	21.0	30.8	13.0	14.0	5.3	6.2	2.2	1.1
9.6	23.2	33.1	14.2	15.2	5.8	6.6	2.4	1.2
9.7	24.8	35.6	15.4	16.4	6.2	7.1	2.6	1.3
9.8	27.2	37.8	16.8	17.0	6.8	7.6	2.8	1.3
9.9	29.6	40.0	18.2	17.8	7.4	8.0	3.0	1.4
10	32.0	42.2	19.8	19.0	8.0	8.4	3.3	1.5
10.1	34.4	44.8	21.4	20.2	8.6	9.0	3.6	1.6
10.2	37.2	47.0	23.0	21.4	9.3	9.4	3.8	1.6
10.3	40.2	49.4	24.7	22.6	10.1	9.9	4.1	1.7
10.4	43.4	52.1	26.3	23.8	10.9	10.4	4.4	1.8
10.5	46.0	54.7	28.3	25.0	11.5	10.9	4.7	1.9
10.6	49.6	57.5	30.3	27.2	12.4	11.5	5.1	2.1
10.7	53.6	61.0	32.5	29.2	13.4	12.2	5.4	2.2
10.8	58.4	65.2	35.0	32.2	14.6	13.0	5.8	2.5
10.9	64.0	70.0	37.7	35.4	16.0	14.0	6.3	2.7
11	71.0	75.6	40.9	39.0	17.8	15.1	6.8	3.0
11.1	78.2	81.9	44.7	42.6	19.6	16.4	7.4	3.3
11.2	85.8	89.0	48.9	46.6	21.5	17.8	8.1	3.6
11.3	94.8	96.5	53.6	51.4	23.7	19.3	8.9	4.0
11.4	105.2	105.0	58.8	56.2	26.3	21.0	9.8	4.3
11.5	117.0	114.2	64.4	62.0	29.3	22.8	10.7	4.8
11.6	129.4	124.8	71.1	68.2	32.4	25.0	11.9	5.2
11.7	144.2	136.4	78.6	75.4	36.1	27.3	13.1	5.8
11.8	161.6	150.1	87.0	84.4	40.4	30.0	14.5	6.5
11.9	184.8	168.1	97.6	100.0	46.2	33.6	16.3	7.7
12	221.0	196.0	112.2	130.0	55.3	39.2	18.7	10.0
12.1	295.4	251.9	137.7	199.8	73.8	50.4	23.0	15.4
12.2	407.8	335.0	175.4	293.4	102.0	67.0	29.2	22.6
12.3	539.2	431.7	221.4	363.0	134.8	86.3	36.9	27.9
12.4	664.0	524.1	272.0	371.8	166.0	104.8	45.3	28.6
12.5	753.0	592.1	322.8	331.0	188.3	118.4	53.8	25.5
12.6	783.2	617.1	367.2	280.2	195.8	123.4	61.2	21.6
12.7	761.2	602.5	400.4	236.2	190.3	120.5	66.7	18.2
12.8	710.8	560.3	420.5	201.4	177.7	112.1	70.1	15.5
12.9	653.6	510.2	425.9	173.4	163.4	102.0	71.0	13.3
13	599.0	462.2	416.9	151.0	149.8	92.4	69.5	11.6
13.1	550.2	419.2	398.3	133.4	137.6	83.8	66.4	10.3
13.2	504.6	380.6	377.9	178.2	126.2	76.1	63.0	13.7
13.3	463.2	345.8	357.3	489.6	115.8	69.2	59.5	37.7
13.4	426.0	314.8	337.4	923.6	106.5	63.0	56.2	71.0
13.5	392.0	287.1	318.1	1306.0	98.0	57.4	53.0	100.5
13.6	361.2	262.7	299.8	1593.6	90.3	52.5	50.0	122.6
13.7	333.2	240.5	282.3	1826.4	83.3	48.1	47.1	140.5
13.8	307.4	220.5	265.4	2041.2	76.9	44.1	44.2	157.0
13.9	283.8	202.3	249.1	2175.6	71.0	40.5	41.5	167.4
14	261.0	185.7	233.9	2229.0	65.3	37.1	39.0	171.5
14.1	241.8	170.6	219.6	2221.2	60.5	34.1	36.6	170.9
14.2	223.8	157.4	206.0	2178.8	56.0	31.5	34.3	167.6
14.3	208.2	145.7	193.3	2112.8	52.1	29.1	32.2	162.5
14.4	194.2	135.6	181.6	2032.0	48.6	27.1	30.3	156.3
14.5	182.0	126.6	170.8	1943.0	45.5	25.3	28.5	149.5

Time (hour)	Total Inflow Hydrograph (cfs)				Inflow per Element (cfs)			
	Junction 649	Sub-Basin 700 (Partial)	Sub-Basin 1025 (Partial)	Junction 820	Junction 649	Sub-Basin 700 (Partial)	Sub-Basin 1025 (Partial)	Junction 820
14.6	171.4	118.9	160.7	1849.6	42.9	23.8	26.8	142.3
14.7	162.2	112.1	151.6	1765.6	40.6	22.4	25.3	135.8
14.8	153.8	106.2	143.4	1703.2	38.5	21.2	23.9	131.0
14.9	146.2	100.7	135.6	1643.2	36.6	20.1	22.6	126.4
15	139.0	95.4	128.5	1581.0	34.8	19.1	21.4	121.6
15.1	133.0	90.5	122.1	1517.4	33.3	18.1	20.4	116.7
15.2	126.6	85.3	116.3	1453.4	31.7	17.1	19.4	111.8
15.3	118.8	81.1	110.5	1389.8	29.7	16.2	18.4	106.9
15.4	112.0	77.3	104.9	1327.4	28.0	15.5	17.5	102.1
15.5	106.0	73.4	99.9	1267.0	26.5	14.7	16.6	97.5
15.6	100.2	69.5	94.6	1208.6	25.1	13.9	15.8	93.0
15.7	95.4	66.0	89.9	1152.6	23.9	13.2	15.0	88.7
15.8	91.2	62.7	85.4	1098.6	22.8	12.5	14.2	84.5
15.9	86.8	59.9	81.1	1047.4	21.7	12.0	13.5	80.6
16	83.0	57.6	77.0	998.0	20.8	11.5	12.8	76.8
16.1	80.6	55.4	73.4	951.4	20.2	11.1	12.2	73.2
16.2	77.8	53.4	70.2	906.6	19.5	10.7	11.7	69.7
16.3	74.8	51.8	67.1	864.0	18.7	10.4	11.2	66.5
16.4	73.2	50.2	64.2	823.6	18.3	10.0	10.7	63.4
16.5	71.0	48.8	61.7	785.0	17.8	9.8	10.3	60.4
16.6	68.8	47.5	59.3	748.2	17.2	9.5	9.9	57.6
16.7	67.2	46.3	56.9	713.8	16.8	9.3	9.5	54.9
16.8	65.4	45.2	54.7	681.4	16.4	9.0	9.1	52.4
16.9	64.2	44.2	52.0	651.0	16.1	8.8	8.7	50.1
17	63.0	43.3	50.1	623.0	15.8	8.7	8.4	47.9
17.1	61.8	42.4	48.5	595.8	15.5	8.5	8.1	45.8
17.2	60.6	41.6	47.0	571.0	15.2	8.3	7.8	43.9
17.3	59.4	40.9	45.8	548.2	14.9	8.2	7.6	42.2
17.4	58.2	40.2	44.5	526.6	14.6	8.0	7.4	40.5
17.5	57.0	39.3	43.6	506.0	14.3	7.9	7.3	38.9
17.6	56.0	38.8	42.5	488.0	14.0	7.8	7.1	37.5
17.7	55.6	38.0	41.7	470.4	13.9	7.6	6.9	36.2
17.8	54.4	37.6	40.8	454.2	13.6	7.5	6.8	34.9
17.9	53.2	36.9	40.0	439.4	13.3	7.4	6.7	33.8
18	53.0	36.3	39.2	425.0	13.3	7.3	6.5	32.7
18.1	51.8	35.8	38.4	412.8	13.0	7.2	6.4	31.8
18.2	50.6	35.1	37.6	400.0	12.7	7.0	6.3	30.8
18.3	49.4	34.4	36.8	388.6	12.4	6.9	6.1	29.9
18.4	48.2	33.6	36.2	377.8	12.1	6.7	6.0	29.1
18.5	47.0	32.7	35.5	368.0	11.8	6.5	5.9	28.3
18.6	45.8	31.8	34.6	358.6	11.5	6.4	5.8	27.6
18.7	44.6	30.9	33.8	349.8	11.2	6.2	5.6	26.9
18.8	43.4	30.2	33.0	341.4	10.9	6.0	5.5	26.3
18.9	43.0	29.5	32.2	333.4	10.8	5.9	5.4	25.6
19	42.0	29.0	31.4	326.0	10.5	5.8	5.2	25.1
19.1	41.0	28.2	30.6	318.8	10.3	5.6	5.1	24.5
19.2	40.6	27.8	30.1	311.6	10.2	5.6	5.0	24.0
19.3	39.4	27.3	29.5	304.4	9.9	5.5	4.9	23.4
19.4	39.0	26.9	28.8	298.0	9.8	5.4	4.8	22.9
19.5	38.0	26.4	28.3	291.0	9.5	5.3	4.7	22.4
19.6	37.8	26.0	27.8	285.0	9.4	5.2	4.6	21.9
19.7	37.0	25.5	27.1	279.0	9.3	5.1	4.5	21.5
19.8	37.0	25.3	26.7	273.0	9.3	5.1	4.5	21.0
19.9	36.2	25.0	26.3	267.0	9.1	5.0	4.4	20.5
20	36.0	24.6	25.9	262.0	9.0	4.9	4.3	20.2
20.1	35.8	24.5	25.6	256.2	8.9	4.9	4.3	19.7
20.2	35.0	24.1	25.4	251.4	8.8	4.8	4.2	19.3
20.3	35.0	23.9	25.0	246.6	8.8	4.8	4.2	19.0
20.4	34.2	23.6	24.6	241.8	8.6	4.7	4.1	18.6
20.5	34.0	23.5	24.2	238.0	8.5	4.7	4.0	18.3
20.6	33.8	23.1	24.1	233.4	8.4	4.6	4.0	18.0
20.7	33.0	22.8	23.7	229.8	8.3	4.6	4.0	17.7
20.8	33.0	22.5	23.3	225.6	8.3	4.5	3.9	17.4
20.9	32.2	22.4	23.2	221.6	8.1	4.5	3.9	17.0
21	32.0	22.0	22.8	219.0	8.0	4.4	3.8	16.8
21.1	32.0	22.0	22.8	215.4	8.0	4.4	3.8	16.6
21.2	31.6	21.7	22.4	212.2	7.9	4.3	3.7	16.3
21.3	31.0	21.7	22.2	209.2	7.8	4.3	3.7	16.1
21.4	31.0	21.4	21.9	206.4	7.8	4.3	3.6	15.9
21.5	31.0	21.3	21.8	204.0	7.8	4.3	3.6	15.7
21.6	30.8	20.9	21.5	200.6	7.7	4.2	3.6	15.4
21.7	30.0	20.8	21.3	198.2	7.5	4.2	3.6	15.2
21.8	30.0	20.6	21.1	195.8	7.5	4.1	3.5	15.1

Time (hour)	Total Inflow Hydrograph (cfs)				Inflow per Element (cfs)			
	Junction 649	Sub-Basin 700 (Partial)	Sub-Basin 1025 (Partial)	Junction 820	Junction 649	Sub-Basin 700 (Partial)	Sub-Basin 1025 (Partial)	Junction 820
21.9	30.0	20.6	20.9	193.4	7.5	4.1	3.5	14.9
22	29.0	20.2	20.8	191.0	7.3	4.0	3.5	14.7
22.1	29.0	20.1	20.5	189.6	7.3	4.0	3.4	14.6
22.2	29.0	19.8	20.3	187.2	7.3	4.0	3.4	14.4
22.3	29.0	19.8	20.1	185.4	7.3	4.0	3.4	14.3
22.4	28.2	19.5	20.1	183.4	7.1	3.9	3.4	14.1
22.5	28.0	19.5	19.8	181.0	7.0	3.9	3.3	13.9
22.6	28.0	19.4	19.7	179.6	7.0	3.9	3.3	13.8
22.7	28.0	19.1	19.4	177.6	7.0	3.8	3.2	13.7
22.8	28.0	19.1	19.2	176.4	7.0	3.8	3.2	13.6
22.9	27.2	18.8	19.1	174.4	6.8	3.8	3.2	13.4
23	27.0	18.7	19.1	173.0	6.8	3.7	3.2	13.3
23.1	27.0	18.6	18.8	171.6	6.8	3.7	3.1	13.2
23.2	27.0	18.4	18.8	169.6	6.8	3.7	3.1	13.0
23.3	27.0	18.4	18.5	168.4	6.8	3.7	3.1	13.0
23.4	26.2	18.1	18.4	167.2	6.6	3.6	3.1	12.9
23.5	26.0	18.0	18.4	165.0	6.5	3.6	3.1	12.7
23.6	26.0	17.9	18.1	163.8	6.5	3.6	3.0	12.6
23.7	26.0	17.6	18.1	162.6	6.5	3.5	3.0	12.5
23.8	26.0	17.6	17.9	161.4	6.5	3.5	3.0	12.4
23.9	25.2	17.6	17.7	160.2	6.3	3.5	3.0	12.3
24	25.0	17.3	17.7	159.0	6.3	3.5	3.0	12.2
24.1	24.8	17.2	17.4	157.6	6.2	3.4	2.9	12.1
24.2	23.6	16.6	17.3	155.2	5.9	3.3	2.9	11.9
24.3	22.4	15.5	16.6	152.8	5.6	3.1	2.8	11.8
24.4	20.4	14.2	15.8	149.6	5.1	2.8	2.6	11.5
24.5	18.0	12.5	15.0	147.0	4.5	2.5	2.5	11.3
24.6	14.6	10.4	14.1	144.4	3.6	2.1	2.4	11.1
24.7	12.2	8.6	12.9	141.2	3.1	1.7	2.1	10.9
24.8	10.4	7.0	11.5	138.2	2.6	1.4	1.9	10.6
24.9	8.4	5.7	10.1	135.4	2.1	1.1	1.7	10.4
25	7.0	4.8	8.9	132.0	1.8	1.0	1.5	10.2
25.1	5.8	3.9	8.0	128.4	1.4	0.8	1.3	9.9
25.2	4.6	3.2	6.9	124.4	1.2	0.6	1.1	9.6
25.3	4.0	2.5	6.1	120.2	1.0	0.5	1.0	9.2
25.4	3.2	1.9	5.5	115.8	0.8	0.4	0.9	8.9
25.5	3.0	1.8	4.8	110.0	0.8	0.4	0.8	8.5
25.6	2.0	1.4	4.3	105.0	0.5	0.3	0.7	8.1
25.7	2.0	1.1	3.6	99.0	0.5	0.2	0.6	7.6
25.8	1.4	0.9	3.2	93.0	0.3	0.2	0.5	7.2
25.9	1.0	0.7	2.8	87.0	0.3	0.1	0.5	6.7
26	1.0	0.7	2.4	82.0	0.3	0.1	0.4	6.3
26.1	1.0	0.4	2.3	76.0	0.3	0.1	0.4	5.8
26.2	1.0	0.4	1.9	70.4	0.3	0.1	0.3	5.4
26.3	0.4	0.4	1.7	65.6	0.1	0.1	0.3	5.0
26.4	0.0	0.4	1.4	60.8	0.0	0.1	0.2	4.7
26.5	0.0	0.0	1.4	56.0	0.0	0.0	0.2	4.3
26.6	0.0	0.0	1.0	51.2	0.0	0.0	0.2	3.9
26.7	0.0	0.0	1.0	46.8	0.0	0.0	0.2	3.6
26.8	0.0	0.0	0.8	42.6	0.0	0.0	0.1	3.3
26.9	0.0	0.0	0.7	38.6	0.0	0.0	0.1	3.0
27	0.0	0.0	0.7	36.0	0.0	0.0	0.1	2.8
27.1	0.0	0.0	0.7	32.4	0.0	0.0	0.1	2.5
27.2	0.0	0.0	0.5	29.2	0.0	0.0	0.1	2.2
27.3	0.0	0.0	0.3	26.8	0.0	0.0	0.1	2.1
27.4	0.0	0.0	0.3	23.6	0.0	0.0	0.1	1.8
27.5	0.0	0.0	0.3	21.0	0.0	0.0	0.1	1.6
27.6	0.0	0.0	0.3	19.6	0.0	0.0	0.1	1.5
27.7	0.0	0.0	0.3	17.2	0.0	0.0	0.1	1.3
27.8	0.0	0.0	0.3	15.4	0.0	0.0	0.1	1.2
27.9	0.0	0.0	0.1	13.4	0.0	0.0	0.0	1.0
28	0.0	0.0	0.0	12.0	0.0	0.0	0.0	0.9
28.1	0.0	0.0	0.0	10.6	0.0	0.0	0.0	0.8
28.2	0.0	0.0	0.0	8.6	0.0	0.0	0.0	0.7
28.3	0.0	0.0	0.0	7.4	0.0	0.0	0.0	0.6
28.4	0.0	0.0	0.0	6.2	0.0	0.0	0.0	0.5
28.5	0.0	0.0	0.0	6.0	0.0	0.0	0.0	0.5
28.6	0.0	0.0	0.0	4.8	0.0	0.0	0.0	0.4
28.7	0.0	0.0	0.0	4.0	0.0	0.0	0.0	0.3
28.8	0.0	0.0	0.0	3.4	0.0	0.0	0.0	0.3
28.9	0.0	0.0	0.0	3.0	0.0	0.0	0.0	0.2
29	0.0	0.0	0.0	2.0	0.0	0.0	0.0	0.2
29.1	0.0	0.0	0.0	2.0	0.0	0.0	0.0	0.2

Time (hour)	Total Inflow Hydrograph (cfs)				Inflow per Element (cfs)			
	Junction 649	Sub-Basin 700 (Partial)	Sub-Basin 1025 (Partial)	Junction 820	Junction 649	Sub-Basin 700 (Partial)	Sub-Basin 1025 (Partial)	Junction 820
29.2	0.0	0.0	0.0	1.6	0.0	0.0	0.0	0.1
29.3	0.0	0.0	0.0	1.0	0.0	0.0	0.0	0.1
29.4	0.0	0.0	0.0	1.0	0.0	0.0	0.0	0.1
29.5	0.0	0.0	0.0	1.0	0.0	0.0	0.0	0.1
29.6	0.0	0.0	0.0	1.0	0.0	0.0	0.0	0.1
29.7	0.0	0.0	0.0	1.0	0.0	0.0	0.0	0.1
29.8	0.0	0.0	0.0	1.0	0.0	0.0	0.0	0.1
29.9	0.0	0.0	0.0	0.2	0.0	0.0	0.0	0.0
30	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0

Attachment 2: Model Control Parameters

Control Variables (CONT.DAT)

Time Control & Global Data	
Simulation Time (hours)	30
Output Interval (hours)	0.1
Graphics Display	Text Screen
Update Time Interval (hours)	
Units	English
n-value Adjustment	0
Flow Depth for Depth Duration Analysis	0
Bulking Concentration	0
Area Reduction Factor	0
Floodplain Limiting Froude Number	0
Shallow Flow n-value	0.2
Encroachment Depth	0

System Components Switches	
Main Channel	OFF
Levees	OFF
Streets	OFF
Area Reduction Factors (ARF)	ON
Multiple Channels (Rill & Gullies)	OFF

Conveyance Structure Switches	
Hydraulic Structures	ON
Floodway Analysis	OFF
Debris Basin	OFF

Physical Processes Switches	
Rainfall	ON
Evaporation	OFF
Storm Drain	OFF
Infiltration	ON
MODFLO-2D Modelling	OFF
Volume Rating Tables	OFF
Mud/Debris/Sediment	None

Numerical Stability Parameters (TOLER.DAT)

Numerical Stability	
Surface Detention	0.004
Percent Change in Flow Depth	0
Dynamic Wave Stability Coefficient	0

Courant Numbers	
Courant Floodplain	0.6
Channel Courant	0.6
Street Courant	0.6
Change Accelerator Rate	0.1

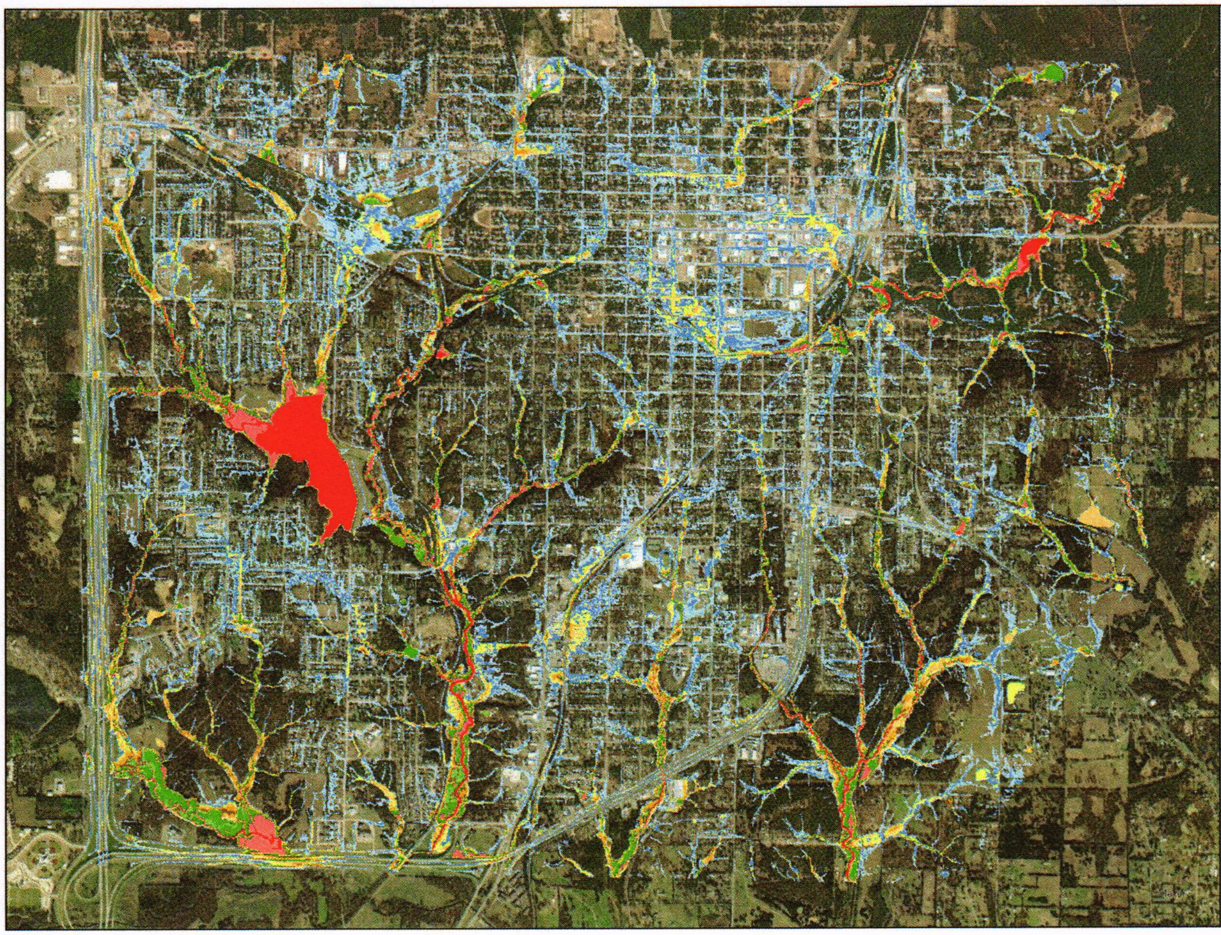
Attachment 3: Hydraulic Structure Input Data

City of Denison Storm Water Master Plan (SWMP)
Appendix E
FLO-2D Model Methods, Inputs, and Results

Attachment 3
Hydraulic Structure Input Data

Hydraulic Structure Name	Structure Type	Inflow Node	Outflow Node	Head Reference Elevation (ft)	Culvert Length (ft)	Culvert Diameter (ft)	Culvert Type	Entrance Type	Culvert n	Entrance Loss Coefficient	Box Width (ft)	Note
Structure_1	Floodplain	180,076	179,243	Floodplain Elevation	65	2.00	Concrete Box	Box - Extended Wingwalls 0 degrees	0.015	0.5	6.0	
Structure_6	Floodplain	233,659	231,989	Floodplain Elevation	60	2.00	Concrete Pipe	Pipe - Square end with Headwall	0.011	0.5	9.4	Multiple Barrel Circle - Converted to Box
Structure_4	Floodplain	238,667	237,833	Floodplain Elevation	70	2.12	Concrete Pipe	Pipe - Square end with Headwall	0.011	0.5	0.0	
Structure_8	Floodplain	226,997	227,836	Floodplain Elevation	65	2.83	Concrete Pipe	Pipe - Square end with Headwall	0.011	0.5	0.0	
Structure_9&10	Floodplain	172,633	173,473	Floodplain Elevation	73	1.50	Concrete Box	Box - Straight headwall	0.015	0.4	4.0	
Structure_11	Floodplain	173,480	172,647	Floodplain Elevation	61	2.00	Concrete Pipe	Pipe - Square end with Headwall	0.011	0.5	9.4	Multiple Barrel Circle - Converted to Box
Structure_12	Floodplain	169,331	169,335	Floodplain Elevation	73	2.00	Concrete Pipe	Pipe - Square end with Headwall	0.011	0.5	9.4	Multiple Barrel Circle - Converted to Box
Structure_13	Floodplain	49,946	50,786	Floodplain Elevation	115	2.50	Concrete Box	Box - Wingwalls at 30 to 75 degrees, square edge	0.015	0.4	3.0	
Structure_14	Floodplain	306,820	309,337	Floodplain Elevation	83	1.50	Concrete Pipe	Pipe - Socket end with Headwall	0.011	0.2	9.4	Multiple Barrel Circle - Converted to Box
Structure_15	Floodplain	337,811	337,815	Floodplain Elevation	60	1.50	Concrete Box	Box - Square edged on three edges, headwall	0.015	0.5	4.0	
Structure_16	Floodplain	314,624	312,952	Floodplain Elevation	87	1.50	Concrete Box	Box - Square edged on three edges, headwall	0.015	0.5	12.0	
Structure_17	Floodplain	499,186	499,189	Floodplain Elevation	52	1.50	Concrete Pipe	Pipe - Socket end with Headwall	0.011	0.2	11.8	Multiple Barrel Circle - Converted to Box
Structure_20	Floodplain	504,987	502,471	Floodplain Elevation	80	2.00	Concrete Box	Box - Wingwalls at 30 to 75 degrees, square edge	0.015	0.4	8.0	
Structure_22	Floodplain	415,092	415,098	Floodplain Elevation	115	1.41	CMP	Corrugated with headwall	0.02	0.5	0.0	
Structure_23	Floodplain	755,957	751,744	Floodplain Elevation	175	2.00	Concrete Box	Box - 45 degree wingwalls	0.015	0.3	4.0	
Structure_24	Floodplain	665,484	669,700	Floodplain Elevation	100	2.00	Concrete Box	Box - 45 degree wingwalls	0.015	0.3	4.0	
Structure_25	Floodplain	680,642	682,325	Floodplain Elevation	70	2.00	Concrete Box	Box - Square edged on three edges, headwall	0.015	0.5	4.0	
Structure_26	Floodplain	685,689	690,746	Floodplain Elevation	125	2.00	Concrete Box	Box - Square edged on three edges, headwall	0.015	0.5	8.0	
Structure_27	Floodplain	686,513	689,884	Floodplain Elevation	90	2.00	Concrete Box	Box - Wingwalls at 30 to 75 degrees, square edge	0.015	0.4	4.0	
Structure_29	Floodplain	569,284	569,282	Floodplain Elevation	30	0.25	Concrete Box	Box - Extended Wingwalls 0 degrees	0.015	0.5	3.0	
Structure_30	Floodplain	569,276	569,274	Floodplain Elevation	35	1.50	Concrete Box	Box - Square edged on three edges, headwall	0.015	0.5	2.0	
Structure_31	Floodplain	565,057	565,053	Floodplain Elevation	88	1.50	Concrete Box	Box - Square edged on three edges, headwall	0.015	0.5	2.0	
Structure_33	Floodplain	564,204	573,445	Floodplain Elevation	600	1.50	Concrete Box	Box - Square edged on three edges, headwall	0.015	0.5	5.0	
Structure_35	Floodplain	589,440	592,807	Floodplain Elevation	86	1.50	Concrete Box	Box - Square edged on three edges, headwall	0.015	0.5	5.0	
Structure_36	Floodplain	596,173	602,062	Floodplain Elevation	168	1.50	Concrete Box	Box - Square edged on three edges, headwall	0.015	0.5	5.0	
Structure_37	Floodplain	607,110	613,002	Floodplain Elevation	133	2.50	Concrete Box	Box - Wingwalls at 30 to 75 degrees, square edge	0.015	0.4	8.0	
Structure_38	Floodplain	628,153	635,737	Floodplain Elevation	200	2.00	Concrete Box	Box - Wingwalls at 30 to 75 degrees, square edge	0.015	0.4	10.0	
Structure_39	Floodplain	641,629	654,268	Floodplain Elevation	320	2.00	Concrete Box	Box - Wingwalls at 30 to 75 degrees, square edge	0.015	0.4	10.0	
Structure_41	Floodplain	300,628	300,637	Floodplain Elevation	55	2.12	Concrete Pipe	Pipe - Socket end with Headwall	0.011	0.2	0.0	
Structure_43	Floodplain	304,902	303,230	Floodplain Elevation	78	2.12	Concrete Pipe	Pipe - Socket end with Headwall	0.011	0.2	0.0	
Structure_44	Floodplain	481,493	484,012	Floodplain Elevation	63	2.83	CMP	Corrugated projecting	0.02	0.9	0.0	
Structure_45	Floodplain	303,408	302,575	Floodplain Elevation	105	2.83	CMP	Corrugated projecting	0.02	0.9	0.0	
Structure_46	Floodplain	821,045	822,443	Floodplain Elevation	180	3.54	CMP	Corrugated projecting	0.02	0.9	0.0	
Structure_47	Floodplain	67,485	71,661	Floodplain Elevation	100	2.00	Concrete Box	Box - Square edged on three edges, headwall	0.015	0.5	4.0	
Structure_48	Floodplain	701,856	698,482	Floodplain Elevation	85	1.50	Concrete Box	Box - Wingwalls at 30 to 75 degrees, square edge	0.015	0.4	3.0	
Structure_49	Floodplain	766,826	766,001	Floodplain Elevation	118	1.00	Concrete Box	Box - Square edged on three edges, headwall	0.015	0.5	2.0	
Structure_50	Floodplain	414,294	417,650	Floodplain Elevation	80	1.00	Concrete Box	Box - Square edged on three edges, headwall	0.015	0.5	2.0	
Structure_51	Floodplain	452,079	454,599	Floodplain Elevation	60	2.83	CMP	Corrugated projecting	0.02	0.9	0.0	
Structure_52	Floodplain	253,937	251,430	Floodplain Elevation	97	3.54	CMP	Corrugated projecting	0.02	0.9	0.0	
Structure_53	Floodplain	222,003	218,674	Floodplain Elevation	312	2.00	Concrete Box	Box - Wingwalls at 30 to 75 degrees, square edge	0.015	0.4	4.0	
Structure_54	Floodplain	161,623	151,624	Floodplain Elevation	500	1.50	Concrete Box	Box - Wingwalls at 30 to 75 degrees, square edge	0.015	0.4	4.0	
Structure_55	Floodplain	309,869	304,845	Floodplain Elevation	140	2.12	Concrete Pipe	Pipe - Socket end with Headwall	0.011	0.2	0.0	
Structure_56	Floodplain	61,537	65,721	Floodplain Elevation	250	2.50	Concrete Box	Box - Wingwalls at 30 to 75 degrees, square edge	0.015	0.4	3.0	
Structure_57	Floodplain	616,875	629,511	Floodplain Elevation	420	2.12	Concrete Pipe	Pipe - Socket end with Headwall	0.011	0.2	0.0	
Structure_58	Floodplain	579,668	579,674	Floodplain Elevation	70	2.12	Concrete Pipe	Pipe - Socket end with Headwall	0.011	0.2	0.0	
Structure_59	Floodplain	585,581	585,585	Floodplain Elevation	70	1.50	Concrete Box	Box - Square edged on three edges, headwall	0.015	0.5	4.0	
Structure_60	Floodplain	586,442	586,447	Floodplain Elevation	70	1.50	Concrete Box	Box - Square edged on three edges, headwall	0.015	0.5	4.0	

Attachment 4: Flood Results Map – Maximum Depth



CITY OF DENISON
300 W Main St
Denison, TX 75020

Denison TEXAS

City of Denison
Conceptual SWMP
R315880.01

Legend
Max Depth
ft
0.001 - 0.25
0.251 - 0.5
0.501 - 1
1.001 - 1.5
1.501 - 2
2.001 - 2.5
2.501 - 3
3.001 - 3.5
3.501 - 5
5.001 - 10
> 10.001

N

Appendix E
Attachment 4

Flood Results Map - Maximum Depth

0 0.25 0.5 Miles

HUITT-ZOLLARS
5430 LBJ Freeway, Suite 1500
Dallas, Texas 75240

Attachment 5: Flood Results Map – Maximum Velocity

CITY OF DENISON
300 W Main St
Denison, TX 75020

Denison TEXAS

City of Denison
Conceptual Stormwater Master Plan
R315880.01

Legend

Max Velocity

ft/s

- 0.001 - 0.5 (No Display)
- 0.501 - 1
- 1.001 - 3
- 3.001 - 6
- 6.001 - 9
- 9.001 - 12
- 12.001 - 15
- 15.001 - 18.005

N

Appendix E
Attachment 5

Flood Results Map - Maximum Velocity

0 0.25 0.5 Miles

HUITT-ZOLLARS
5430 LBJ Freeway, Suite 1500
Dallas, Texas 75240

Appendix F: Hydraulic Evaluation Calculations

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Attachments

1.0 Introduction

As part of the development of each proposed improvement plan for the City of Denison (the City), Huitt-Zollars, Inc. (HZ) evaluated existing and proposed hydraulic conditions at a planning level. The resulting proposed improvement projects were assigned estimated costs based on the required structures at each location.

2.0 Design Criteria

Hydraulic structures were sized in accordance with Denison's Storm Drainage System Design Manual (Ref. 1) as follows:

- Storm drainage facilities, including street capacity, are designed to convey the 100-year storm
- Excavated channels have adequate capacity for the 25-year design flow with 2-feet of freeboard and capacity for a 100-year design flow without overtopping
- Culverts shall be designed to carry the 100-year frequency design without overtopping

3.0 Design Discharges

For a simplified, planning-level design approach, storm drain systems, culverts, and engineered channels are designed for the 100-year discharge. The design discharge for both the proposed and existing conditions are based on the simulated 100-year discharges derived from the HEC-HMS model. The design discharge is determined on an area-based unit discharge, where the ratio of peak discharge to total drainage area is calculated for the contributing sub-basin. Design discharge is calculated as the product of the unit discharge (discharge per area) and the contributing catchment area to a hydraulic structure.

$$Q_d = \frac{Q_{HMS}}{A_{HMS}} A_d$$

Where:

Q_d = Design Discharge (cfs)

Q_{HMS} = 100-year Discharge from HEC-HMS for Sub-Basin (cfs)

A_{HMS} = Total Drainage Area for HEC-HMS Sub-Basin (square mile)

A_d = Design Area Contributing to Hydraulic Structure (square mile)

Design discharge calculations are presented in Attachment 1.

4.0 Storm Drain Systems

Storm drain systems were designed such that street, inlet, and pipe capacity are not exceeded by the 100-year design flow. Gutter capacity and inlet sizing were made using design nomographs from Denison's Storm Drainage System Design Manual (Ref. 1). Storm drain sizing analysis for the proposed improvement projects are provided in Attachment 2.

4.1 Gutter Capacity

Gutter capacity was calculated for a triangular gutter. The nomograph used is reproduced in Figure 1. For all streets, the cross-slope was assumed at ¼-inch per foot, which is the standard cross slope for the City. A standard curb height of 4-inches was assumed. Based on the curb height, cross slope, and gutter slope, the capacity for each gutter is calculated in cubic feet per second (cfs).

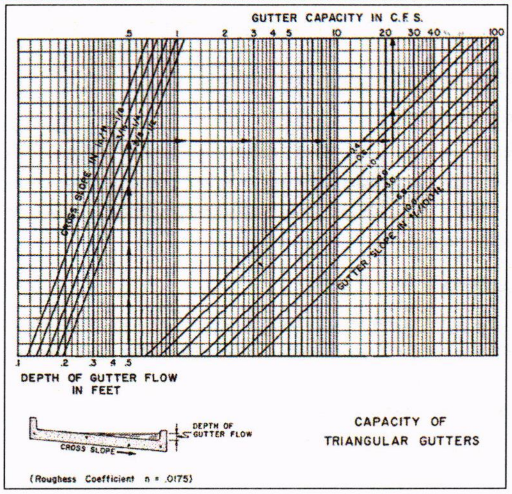


Figure 1: Gutter Capacity Nomograph for Triangular Gutters (Ref. 1)

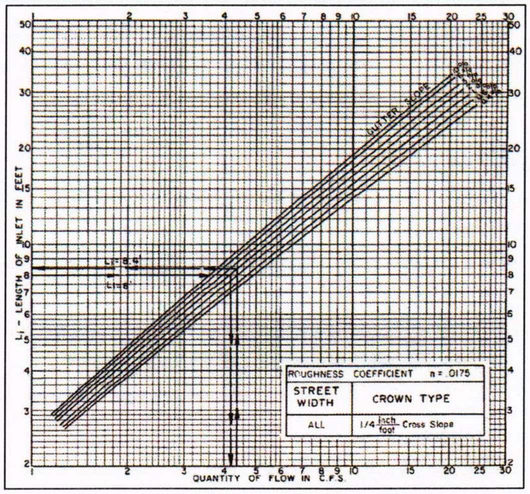


Figure 2: Standard Curb Inlet Sizing Nomograph for ¼-inch/foot Cross Slope

4.2 Inlet Sizing

Storm drain inlets were sized using a nomograph (Figure 2) for a standard curb inlet on a street with a ¼-inch per foot cross slope. Given the gutter flow calculated from Figure 1, the required inlet length can be selected. Standard inlets lengths were given in the Denison Storm Drainage System Design Manual (Ref. 1), and a combination of the standard inlets may be used to reach the length required based on this analysis. For locations where standard curb inlets are not reasonable, additional nomographs from the Drainage System Design Manual (Ref. 1) were used where appropriate.

4.3 Pipe Sizing

Pipe sizing was calculated assuming pipe capacity is equal to a full-flowing pipe with steady, uniform flow. Manning's equation was used to calculate the pipe capacity for a given nominal pipe size. For simplification, the roughness for a reinforced concrete pipe (RCP) is assumed equal to 0.013, and the longitudinal slope of the pipe is assumed to be 0.01 foot per foot (ft/ft).

$$Q_d = \frac{1.49}{n} A R^{2/3} S^{1/2}$$

Where:

Q_d = Design Discharge (cfs)

n = Roughness Coefficient = 0.013 (assumed)

A = Cross-Sectional Flow Area in Pipe (ft²)

R = Hydraulic Radius of Flow in Pipe (ft)

S = Longitudinal Slope of Pipe (ft/ft)

Pipe sizing was determined based on design flows and the capacities associated with each pipe size, as indicated in Table 1. Pipe flow is estimated based on the total contributing flow to each storm drain divided by the total number of inlets contributing to each pipe.

Table 1: Capacity for RCPs Used for Storm Drain Conduit Sizing

Nominal Diameter (inch)	Diameter (ft)	Area (ft²)	Perimeter (ft)	Hydraulic Radius (ft)	Capacity (cfs)
12	1	0.8	3.1	0.25	3.6
15	1.25	1.2	3.9	0.3125	6.5
18	1.5	1.8	4.7	0.375	10.5
24	2	3.1	6.3	0.5	22.7
30	2.5	4.9	7.9	0.625	41.1
36	3	7.1	9.4	0.75	66.9
42	3.5	9.6	11.0	0.875	100.9
48	4	12.6	12.6	1	144.0
54	4.5	15.9	14.1	1.125	197.2
60	5	19.6	15.7	1.25	261.1
66	5.5	23.8	17.3	1.375	336.7
72	6	28.3	18.8	1.5	424.6
84	7	38.5	22.0	1.75	640.6
90	7.5	44.2	23.6	1.875	769.9

5.0 Culverts

Culverts were evaluated and sized using the HY-8 program developed by the Federal Highways Administration (FHWA) (Ref. 2). HY-8 is a public domain software intended for the design of culverts based on approaches and protocols from FHWA's "Hydraulic Design of Highway Culverts" (Ref. 3). Given a design flow, existing condition and proposed conditions were evaluated to determine whether culverts have adequate capacity to convey the 100-year design discharge without overtopping to the roadway. Outputs from HY-8 for existing conditions at each proposed improvement project are provided in Attachment 3.

6.0 Engineered Channels

Engineered channels were evaluated and designed using the FHWA Hydraulic Toolbox (Ref. 4) which is a software that performs routine hydrologic and hydraulic analysis and design computations. In the program, "channel analysis" is used to evaluate the hydraulics for existing and proposed channels assuming steady uniform flow given a design discharge and channel geometry. Where existing channels were inadequate, channel improvements were proposed. Proposed channel analyses are presented in Table 2.

Table 2: Channel Evaluation Summary

Project ID	Location	Lining	Side Slopes (xH:1V)	Bottom Width (ft)	Channel Depth (ft)	Longitudinal Slope (ft/ft)	Roughness	Flow (cfs)	Flow Depth (ft)	Freeboard (ft)
A	Flora & Rivercrest	Grouted Riprap	2	10	3	0.022	0.033	318	2.3	0.7
F	Coffin & Woodlawn	Vegetation	3	5	3	0.013	0.055	178	2.9	0.1
K	Flora & Lum	Riprap D50 = 12"	3	15	3.5	0.012	0.074	266	3.0	0.5
L	Coffin & Park	Concrete	2	5	3	0.037	0.013	40	0.5	2.5

7.0 Detention Structure Design

In areas where a detention structure reduces the peak discharge, detention design using the Modified Rational Method as presented in the iSWM Hydrology Manual (Ref. 5). The allowable release rate is calculated by the Rational Equation:

$$Q_a = C_a i A$$

Where:

Q_a = Allowable Discharge Rate (cfs)

C_a = Rational Method Runoff Coefficient

i = Rainfall Intensity for the Time of Concentration (inches/hour)

A = Area (acres)

The critical duration of storm at which storage volume is maximized is calculated as:

$$T_d = \sqrt{\frac{2CAab}{Q_a}} - b$$

Where:

T_d = Critical Storm Duration (minutes)

a, b = Rainfall Factors Dependent on Location and Return Period (Table 3)

$$V_{pre} = 60\left[CAa - \sqrt{2CabAQ_a} + \left(\frac{Q_a}{2}\right)(b - t_c)\right]$$

$$V_{req} = V_{pre}\left(\frac{P_{180}}{P_{td}}\right)$$

Where:

V_{pre} = Preliminary Required Storage (ft[3])

t_c = Time of Concentration for Developed Condition (minutes)

V_{req} = Required Storage (ft[3])

P_{180} = 3-Hour Storm Depth (inches)

P_{td} = Storm Depth for the Critical Duration (inches)

Table 3: Rainfall Factors for Modified Rational Method (Ref. 5)

County	Parameter	Return Interval (Years)						
		1	2	5	10	25	50	100
Grayson	a	100.87	128.89	175.74	208.17	250.17	285.35	325.63
	b	14.086	16.567	20.006	21.751	22.993	24.027	25.322

Time of concentration was calculated according to the Rational Method:

$$t_c = 1.8(1.1 - C)D^{0.5}/S^{1/3}$$

Where:

D = Distance from the Upper End of the Watershed to Outlet (feet)

S = Average Slope Along Distance "D", in percent (ft/100-ft)

Where detention basins are used, the downstream peak discharge estimated from Section 3.0 is reduced to the allowable discharge rate. The detention basin is used to reduce the discharge to a more acceptable level for the channel and hydraulic structures downstream. Calculations for detention design are included in Attachment 4.

8.0 REFERENCES

1. City of Denison (2017). Storm Drainage System Design Manual. Prepared by Birkhoff, Hendricks & Carter, L.L.P.

2. FHWA (2007). HY-8 - Version 7.0

3. FHWA (2005). Hydraulic Design Series Number 5 Hydraulic Design of Highway Culverts, 2nd Edition. Fort Collins, CO.

4. FHWA (2020).Hydraulic Toolbox, Version 5.0. Developed by Aquaveo.

5. Integrated Stormwater Management (iSWM), *iSWM Technical Manual – Hydrology.* April 2010, Revised September 2014.

Attachment 1: Design Discharge Calculations

Attachment 1
Design Discharge Calculations

Project Number	Location	Structure	Contributing Area (mi^2)	HMS Element	HMS Area (mi^2)	HMS Discharge (cfs)	Design Discharge (cfs)	Note
A	Flora & Rivercrest	Catch Basin	0.001074	1948	0.931626	1704	2.0	
A	Flora & Rivercrest	Storm Drain Conduit	0.001074	1948	0.931626	1704	2.0	
A	Flora & Rivercrest	Upstream Channel & Detention Basin	0.279017	1948	0.931626	1704	510	
A	Flora & Rivercrest	Culvert Crossing - Flora	(see note)	(see note)	(see note)	(see note)	318	Downstream of Detention Basin
A	Flora & Rivercrest	Culvert Crossing - Rivercrest	(see note)	(see note)	(see note)	(see note)	318	Downstream of Detention Basin
A	Flora & Rivercrest	Downstream Channel	(see note)	(see note)	(see note)	(see note)	318	Downstream of Detention Basin
B	Morton & Maurice	Storm Drain Conduit - East	0.053708	700	0.792728	1683	114	
B	Morton & Maurice	Catch Basin & Gutter	0.0041	700	0.792728	1683	8.7	Contributing Area Based on Street Capacity, Inlet Spacing
B	Morton & Maurice	Storm Drain Conduit - South	0.035067	700	0.792728	1683	74	
B	Morton & Maurice	Storm Drain Conduit - West	0.006101	700	0.792728	1683	13.0	
C	Sears & Fannin	Catch Basin	0.0095	681	0.687164	1378	19.1	Contributing Area Based on Street Capacity, Inlet Spacing
C	Sears & Fannin	Gutter	0.0095	681	0.687164	1378	19.1	Contributing Area Based on Street Capacity, Inlet Spacing
C	Sears & Fannin	Storm Drain Conduit	0.12376	681	0.687164	1378	248	
D	Loy Lake Road & Waterloo Creek	Culvert Crossing	(see note)	(see note)	(see note)	(see note)	2250	Discharge from FEMA FIS
E	Loy Lake Road & Loy Creek	Culvert Crossing	0.03496	871	0.841528	1500	2237	Upstream contributing flow from R-813-820 of 2175 cfs
F	Coffin & Woodlawn	Channel	0.088663	644	0.438108	909	184	
F	Coffin & Woodlawn	Storm Drain Conduit	0.102616	644	0.438108	909	213	
G	Stream Downstream of Coffin & Park	Gabion Retaining Wall / Existing Channel	0.196186	1930	1.022744	1675	350	
H	Lillis & Crawford	Culvert	0.56652	681	0.687164	1378	1136	
I	Chandler Avenue near Munson	East Alley Lowpoint / Inlet	0.006624	820	0.138503	379	12	Low-point by homes with flooding issues
J	Dubois	Culvert Crossing	0.60829	626	0.313997	616	1193	
K	Flora & Lum	Channel	0.135574	626	0.313997	616	266	
L	Coffin & Park	Culvert under Park	0.069022	871	0.841528	1500	123	
L	Coffin & Park	Culvert along Coffin	0.022449	871	0.841528	1500	40	
L	Coffin & Park	Roadside Ditch along Coffin	0.022449	871	0.841528	1500	40	
M	Eddy & Elm	Catch Basin	0.00658	1948	0.931626	1704	12	Contributing Area Based on Street Capacity, Inlet Spacing
M	Eddy & Elm	Gutter	0.00658	1948	0.931626	1704	12	Contributing Area Based on Street Capacity, Inlet Spacing
M	Eddy & Elm	Storm Drain Conduit	0.01316	1948	0.931626	1704	24	

Attachment 2: Storm Drain Analysis

Attachment 2
Storm Drain Analysis

Catch Basins						
Project ID	Location	Drainage Area ID	Drainage Area Design Flow (cfs)	Number of Catch Basins	Catch Basin Unit Flow (cfs/inlet)	Note
A	Flora & Rivercrest	Flora Street	2250	1	2250.0	
B	Morton & Maurice	East	114	14	8.1	
B	Morton & Maurice	South	74	7	10.6	
B	Morton & Maurice	West	13	4	3.3	
C	Sears Street	Sears Street Total	266	26	10.2	
F	Coffin & Woodlawn	Coffin Drain	206 (28 Streetflow)	4	9.3	
I	Chandler & Munson	Alley Inlet	12	1	12.0	
M	Eddy Avenue	Eddy Drain	24	4	6.0	Two Catchbasins, 2 spillways

Gutter and Inlet Capacity							
Project ID	Location	Drainage Area ID	Catch Basin Unit Flow (cfs/inlet)	Cross Slope (in/ft)	Gutter Slope (%)	Gutter Capacity (cfs)	Standard Catch Basin Inlet
A	Flora & Rivercrest	Flora Street	2250.0	0.25	1	7.8	10-ft Standard Curb Inlet On Grade
B	Morton & Maurice	Total Drainage Average	8.1	0.25	1	7.8	12-ft Standard Curb Inlet On Grade
C	Sears Street	Sears Street Total	10.2	0.25	1	7.8	14-ft Standard Curb Inlet On Grade
F	Coffin & Woodlawn	Coffin Drain	9.3	0.25	1	7.8	12-ft Standard Curb Inlet On Grade
M	Eddy Avenue	Eddy Drain	6.0	0.25	1	7.8	12-ft Standard Curb Inlet On Grade

Conduits				
Project ID	Location	Conduit ID	Design Flow (cfs)	Pipe Diameter (inches)
A	Flora & Rivercrest	14	2	18
B	Morton & Maurice	12-10L	9.1	18
B	Morton & Maurice	12-10M	54.6	36
B	Morton & Maurice	12-11L	9.1	18
B	Morton & Maurice	12-11M	63.7	36
B	Morton & Maurice	12-12	167.3	54
B	Morton & Maurice	12-12L	2.8	18
B	Morton & Maurice	12-12M	69.3	42
B	Morton & Maurice	12-13L	2.8	18
B	Morton & Maurice	12-13M	172.9	54
B	Morton & Maurice	12-14	182.1	54
B	Morton & Maurice	12-14L	9.2	18
B	Morton & Maurice	12-14M	182.1	54
B	Morton & Maurice	12-15L	9.2	18
B	Morton & Maurice	12-15M	200.5	60
B	Morton & Maurice	12-16L	9.2	18
B	Morton & Maurice	12-16M	18.4	24
B	Morton & Maurice	12-1L	7	18
B	Morton & Maurice	12-1M	14	24
B	Morton & Maurice	12-2L	7	18
B	Morton & Maurice	12-2M	28	30
B	Morton & Maurice	12-3	42	36
B	Morton & Maurice	12-3L	7	18
B	Morton & Maurice	12-3M	14	24
B	Morton & Maurice	12-4L	7	18
B	Morton & Maurice	12-4M	56	36
B	Morton & Maurice	12-5	84	42
B	Morton & Maurice	12-5L	7	18
B	Morton & Maurice	12-5M	70	42
B	Morton & Maurice	12-6L	7	18
B	Morton & Maurice	12-6M	14	24
B	Morton & Maurice	12-7L	7	18
B	Morton & Maurice	12-7M	98	42
B	Morton & Maurice	12-8L	9.1	18
B	Morton & Maurice	12-8M	18.2	24
B	Morton & Maurice	12-9L	9.1	18
B	Morton & Maurice	12-9M	36.4	30
B	Morton & Maurice	12-OUT	219	60
C	Sears	15-10L	8	18
C	Sears	15-10M	16	24
C	Sears	15-11L	8	18
C	Sears	15-11M	88	42
C	Sears	15-12L	8	18
C	Sears	15-12M	104	48
C	Sears	15-13L	8	18
C	Sears	15-13M	8	18
C	Sears	15-14L	8	18
C	Sears	15-14M	16	24
C	Sears	15-15M	120	48
C	Sears	15-16M	136	48
C	Sears	15-17M	184	54
C	Sears	15-18M	192	54
C	Sears	15-19M	203	60
C	Sears	15-1L	8	18
C	Sears	15-1M	8	18
C	Sears	15-2L	8	18
C	Sears	15-2M	16	24
C	Sears	15-3L	8	18
C	Sears	15-3M	8	18
C	Sears	15-4L	8	18
C	Sears	15-4M	24	30
C	Sears	15-5L	8	18
C	Sears	15-5M	40	30
C	Sears	15-6L	8	18
C	Sears	15-6M	16	24
C	Sears	15-7L	8	18
C	Sears	15-7M	56	36
C	Sears	15-8L	8	18
C	Sears	15-8M	72	42
C	Sears	15-9L	8	18
C	Sears	15-9M	8	18
F	Coffin & Woodlawn	11-1L	9	18
F	Coffin & Woodlawn	11-1M	206	60
F	Coffin & Woodlawn	11-2L	9	18
F	Coffin & Woodlawn	11-2M	206	60
F	Coffin & Woodlawn	11-3L	9	18
I	Chandler near Munson	20-M	12	30
M	Eddy Avenue	9-1L	6	18
M	Eddy Avenue	9-1M	24	30
M	Eddy Avenue	9-2L	6	18

Attachment 3: Culvert Analysis in HY-8

Project A – Existing 1 (Flora Culvert)
Flora and Rivercrest Culvert 1

Crossing Discharge Data

Discharge Selection Method: Specify Minimum, Design, and Maximum Flow

Minimum Flow: 0.00 cfs

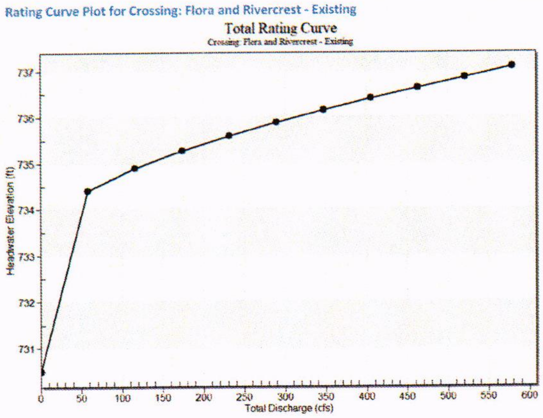
Design Flow: 579.00 cfs

Maximum Flow: 579.00 cfs

Table A1 - Summary of Culvert Flows at Crossing: Flora and Rivercrest - Existing

Headwater Elevation (ft)	Total Discharge (cfs)	Culvert 1 Discharge (cfs)	Roadway Discharge (cfs)	Iterations
730.50	0.00	0.00	0.00	1
734.40	57.90	48.96	8.94	9
734.90	115.80	55.22	60.58	6
735.27	173.70	59.37	114.32	5
735.59	231.60	62.70	168.89	5
735.88	289.50	65.54	223.95	5
736.14	347.40	68.05	279.34	5
736.39	405.30	70.32	334.96	4
736.63	463.20	72.40	390.78	4
736.86	521.10	74.33	446.77	4
737.07	579.00	76.13	502.87	4
734.20	46.24	46.24	0.00	Overtopping

Rating Curve Plot for Crossing: Flora and Rivercrest - Existing



Culvert Data: Culvert 1

Table A2 - Culvert Summary Table: Culvert 1

Total Discharge (cfs)	Culvert Discharge (cfs)	Headwater Elevation (ft)	Inlet Control Depth (ft)	Outlet Control Depth (ft)	Flow Type	Normal Depth (ft)	Critical Depth (ft)	Outlet Depth (ft)	Tailwater Depth (ft)	Outlet Velocity (ft/s)	Tailwater Velocity (ft/s)
0.00 cfs	0.00 cfs	730.50	0.00	0.000	0-NF	0.00	0.00	0.00	0.00	0.00	0.00
57.90 cfs	48.96 cfs	734.40	3.90	3.193	5-S2n	1.58	2.28	1.77	1.06	11.26	4.16
115.80 cfs	55.22 cfs	734.90	4.40	3.628	5-S2n	1.70	2.41	1.91	1.54	11.64	5.13
173.70 cfs	59.37 cfs	735.27	4.77	3.936	5-S2n	1.78	2.49	2.00	1.91	11.89	5.77
231.60 cfs	62.70 cfs	735.59	5.09	4.195	5-S2n	1.84	2.55	2.07	2.22	12.08	6.26
289.50 cfs	65.54 cfs	735.88	5.38	4.423	5-S2n	1.90	2.59	2.12	2.49	12.25	6.66
347.40 cfs	68.05 cfs	736.14	5.64	4.494	5-S2n	1.95	2.63	2.17	2.73	12.40	7.00
405.30 cfs	70.32 cfs	736.39	5.89	4.889	5-S2n	2.00	2.66	2.22	2.95	12.54	7.30
463.20 cfs	72.40 cfs	736.63	6.13	5.257	5-JS1f	2.04	2.69	3.00	3.15	10.24	7.57
521.10 cfs	74.33 cfs	736.86	6.36	5.604	5-JS1f	2.08	2.71	3.00	3.33	10.52	7.82
579.00 cfs	76.13 cfs	737.07	6.57	5.933	5-JS1f	2.12	2.73	3.00	3.51	10.77	8.04

Culvert Barrel Data

Culvert Barrel Type Straight Culvert

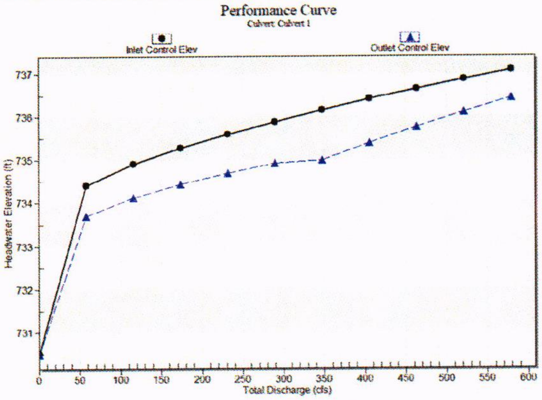
Inlet Elevation (invert): 730.50 ft,

Outlet Elevation (invert): 729.70 ft

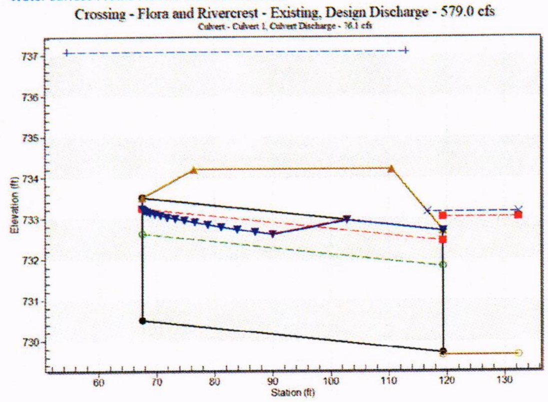
Culvert Length: 51.81 ft,

Culvert Slope: 0.0154

Culvert Performance Curve Plot: Culvert 1



Water Surface Profile Plot for Culvert: Culvert 1



Site Data - Culvert 1

Site Data Option: Culvert Invert Data

Inlet Station: 67.60 ft

Inlet Elevation: 730.50 ft

Outlet Station: 119.40 ft

Outlet Elevation: 729.70 ft

Number of Barrels: 1

Culvert Data Summary - Culvert 1

Barrel Shape: Circular

Barrel Diameter: 3.00 ft

Barrel Material: Concrete

Embedment: 0.00 in

Barrel Manning's n: 0.0120

Roadway Top Width: 34.00 ft

Culvert Type: Straight

Inlet Configuration: Square Edge with Headwall

Inlet Depression: None

Tailwater Data for Crossing: Flora and Rivercrest - Existing

Table A3 - Downstream Channel Rating Curve (Crossing: Flora and Rivercrest - Existing)

Flow (cfs)	Water Surface Elev (ft)	Velocity (ft/s)	Depth (ft)	Shear (psf)	Froude Number
0.00	729.65	0.00	0.00	0.00	0.00
57.90	730.71	1.06	4.16	1.05	0.80
115.80	731.19	1.54	5.13	1.54	0.84
173.70	731.56	1.91	5.77	1.91	0.86
231.60	731.87	2.22	6.26	2.22	0.88
289.50	732.14	2.49	6.66	2.48	0.89
347.40	732.38	2.73	7.00	2.72	0.90
405.30	732.60	2.95	7.30	2.94	0.91
463.20	732.80	3.15	7.57	3.14	0.92
521.10	732.98	3.33	7.82	3.33	0.92
579.00	733.16	3.51	8.04	3.50	0.93

Tailwater Channel Data - Flora and Rivercrest - Existing

Tailwater Channel Option: Trapezoidal Channel

Bottom Width: 10.00 ft

Side Slope (H:V): 3.00 (_:1)

Channel Slope: 0.0160

Channel Manning's n: 0.0400

Channel Invert Elevation: 729.65 ft

Roadway Data for Crossing: Flora and Rivercrest - Existing

Roadway Profile Shape: Constant Roadway Elevation

Crest Length: 34.00 ft

Crest Elevation: 734.20 ft

Roadway Surface: Paved

Project A – Design 1 (Flora Culvert)
Flora and Rivercrest Culvert 1

Crossing Discharge Data

Discharge Selection Method: Specify Minimum, Design, and Maximum Flow

Minimum Flow: 0.00 cfs

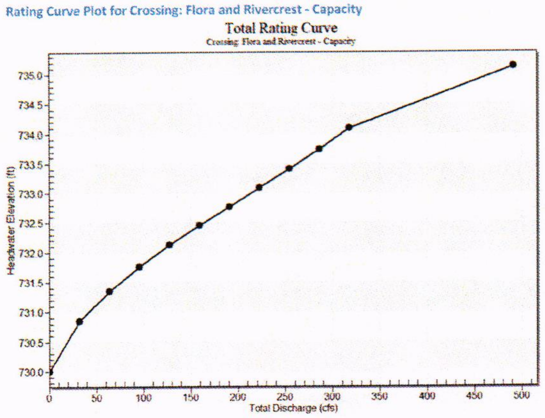
Design Flow: 318.00 cfs

Maximum Flow: 318.00 cfs

Table A4 - Summary of Culvert Flows at Crossing: Flora and Rivercrest - Capacity

Headwater Elevation (ft)	Total Discharge (cfs)	Culvert 1 Discharge (cfs)	Roadway Discharge (cfs)	Iterations
730.00	0.00	0.00	0.00	1
730.85	31.80	31.80	0.00	1
731.35	63.60	63.60	0.00	1
731.76	95.40	95.40	0.00	1
732.12	127.20	127.20	0.00	1
732.45	159.00	159.00	0.00	1
732.77	190.80	190.80	0.00	1
733.08	222.60	222.60	0.00	1
733.40	254.40	254.40	0.00	1
733.74	286.20	286.20	0.00	1
734.09	318.00	318.00	0.00	1
734.20	327.64	327.64	0.00	Overtopping

Rating Curve Plot for Crossing: Flora and Rivercrest - Capacity



Culvert Data: Culvert 1

Table A5 - Culvert Summary Table: Culvert 1

Total Discharge (cfs)	Culvert Discharge (cfs)	Headwater Elevation (ft)	Inlet Control Depth (ft)	Outlet Control Depth (ft)	Flow Type	Normal Depth (ft)	Critical Depth (ft)	Outlet Depth (ft)	Tailwater Depth (ft)	Outlet Velocity (ft/s)	Tailwater Velocity (ft/s)
0.00 cfs	0.00 cfs	730.00	0.00	0.000	0-NF	0.00	0.00	0.00	0.00	0.00	0.00
31.80 cfs	31.80 cfs	730.85	0.85	0.416	1-JS1t	0.42	0.50	0.70	0.75	2.82	3.44
63.60 cfs	63.60 cfs	731.35	1.35	0.811	1-JS1t	0.67	0.79	1.06	1.11	3.74	4.29
95.40 cfs	95.40 cfs	731.76	1.76	1.147	1-JS1t	0.88	1.03	1.34	1.39	4.45	4.85
127.20 cfs	127.20 cfs	732.12	2.12	1.464	1-JS1t	1.07	1.25	1.57	1.62	5.06	5.28
159.00 cfs	159.00 cfs	732.45	2.45	1.776	1-JS1t	1.25	1.45	1.78	1.83	5.60	5.63
190.80 cfs	190.80 cfs	732.77	2.77	2.091	1-JS1t	1.43	1.64	1.96	2.01	6.09	5.93
222.60 cfs	222.60 cfs	733.08	3.08	2.415	5-JS1t	1.60	1.82	2.13	2.18	6.55	6.19
254.40 cfs	254.40 cfs	733.40	3.40	2.750	5-JS1t	1.76	1.99	2.28	2.33	6.98	6.43
286.20 cfs	286.20 cfs	733.74	3.74	3.098	5-JS1t	1.92	2.15	2.42	2.47	7.38	6.64
318.00 cfs	318.00 cfs	734.09	4.09	3.462	5-JS1t	2.08	2.31	2.56	2.61	7.77	6.84

Culvert Barrel Data

Culvert Barrel Type Straight Culvert

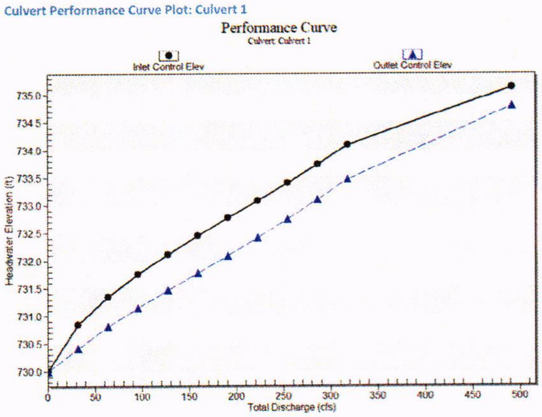
Inlet Elevation (invert): 730.00 ft,

Outlet Elevation (invert): 729.70 ft

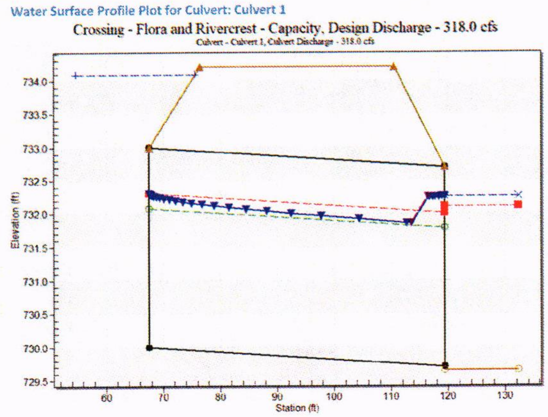
Culvert Length: 51.80 ft,

Culvert Slope: 0.0058

Culvert Performance Curve Plot: Culvert 1



Water Surface Profile Plot for Culvert: Culvert 1



Site Data - Culvert 1

Site Data Option: Culvert Invert Data

Inlet Station: 67.60 ft

Inlet Elevation: 730.00 ft

Outlet Station: 119.40 ft

Outlet Elevation: 729.70 ft

Number of Barrels: 4

Culvert Data Summary - Culvert 1

Barrel Shape: Concrete Box

Barrel Span: 4.00 ft

Barrel Rise: 3.00 ft

Barrel Material: Concrete

Embedment: 0.00 in

Barrel Manning's n: 0.0120

Culvert Type: Straight

Inlet Configuration: Square Edge (90º) Headwall (Ke=0.5)

Inlet Depression: None

Roadway Surface: Paved

Roadway Top Width: 34.00 ft

Tailwater Data for Crossing: Flora and Rivercrest - Capacity

Table A6 - Downstream Channel Rating Curve (Crossing: Flora and Rivercrest - Capacity)

Flow (cfs)	Water Surface Elev (ft)	Velocity (ft/s)	Depth (ft)	Shear (psf)	Froude Number
0.00	729.65	0.00	0.00	0.00	0.00
31.80	730.40	0.75	3.44	0.75	0.76
63.60	730.76	1.11	4.29	1.11	0.80
95.40	731.04	1.39	4.85	1.39	0.82
127.20	731.27	1.62	5.28	1.62	0.84
159.00	731.48	1.83	5.63	1.82	0.85
190.80	731.66	2.01	5.93	2.00	0.87
222.60	731.83	2.18	6.19	2.17	0.87
254.40	731.98	2.33	6.43	2.33	0.88
286.20	732.12	2.47	6.64	2.47	0.89
318.00	732.26	2.61	6.84	2.60	0.89

Tailwater Channel Data - Flora and Rivercrest - Capacity

Tailwater Channel Option: Trapezoidal Channel

Bottom Width: 10.00 ft

Side Slope (H:V): 3.00 (_:1)

Channel Slope: 0.0160

Channel Manning's n: 0.0400

Channel Invert Elevation: 729.65 ft

Roadway Data for Crossing: Flora and Rivercrest - Capacity

Roadway Profile Shape: Constant Roadway Elevation

Crest Length: 34.00 ft

Crest Elevation: 734.20 ft

Project A – Existing 2 (Rivercrest Culvert) Flora and Rivercrest Downstream Culvert 2

Crossing Discharge Data

Discharge Selection Method: Specify Minimum, Design, and Maximum Flow

Minimum Flow: 0.00 cfs

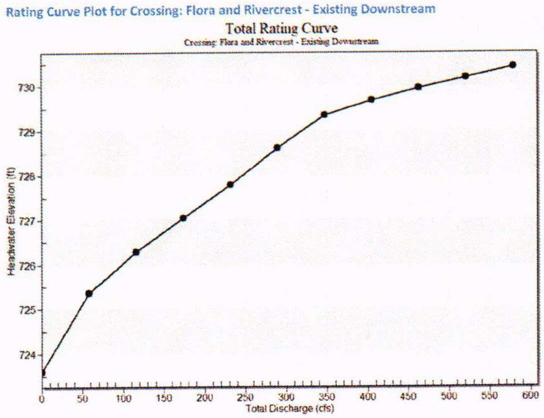
Design Flow: 579.00 cfs

Maximum Flow: 579.00 cfs

Table A7 - Summary of Culvert Flows at Crossing: Flora and Rivercrest - Existing Downstream

Headwater Elevation (ft)	Total Discharge (cfs)	Culvert 1 Discharge (cfs)	Roadway Discharge (cfs)	Iterations
723.60	0.00	0.00	0.00	1
725.37	57.90	57.90	0.00	1
726.28	115.80	115.80	0.00	1
727.04	173.70	173.70	0.00	1
727.78	231.60	231.60	0.00	1
728.62	289.50	289.50	0.00	1
729.34	347.40	332.59	14.80	6
729.67	405.30	350.37	54.93	6
729.94	463.20	364.36	98.82	5
730.19	521.10	376.38	144.72	5
730.41	579.00	387.06	191.94	5
729.10	318.95	318.95	0.00	Overtopping

Rating Curve Plot for Crossing: Flora and Rivercrest - Existing Downstream



Culvert Data: Culvert 1

Table A8 - Culvert Summary Table: Culvert 1

Total Discharge (cfs)	Culvert Discharge (cfs)	Headwater Elevation (ft)	Inlet Control Depth (ft)	Outlet Control Depth (ft)	Flow Type	Normal Depth (ft)	Critical Depth (ft)	Outlet Depth (ft)	Tailwater Depth (ft)	Outlet Velocity (ft/s)	Tailwater Velocity (ft/s)
0.00 cfs	0.00 cfs	723.60	0.00	0.000	0-NF	0.00	0.00	0.00	0.00	0.00	0.00
57.90 cfs	57.90 cfs	725.37	1.77	0.755	1-S2n	0.93	1.29	0.99	1.06	8.01	4.16
115.80 cfs	115.80 cfs	726.28	2.68	1.506	1-S2n	1.33	1.85	1.45	1.54	9.40	5.13
173.70 cfs	173.70 cfs	727.04	3.44	2.261	1-S2n	1.65	2.29	1.83	1.91	10.33	5.77
231.60 cfs	231.60 cfs	727.78	4.18	3.074	5-S2n	1.94	2.66	2.17	2.22	11.12	6.26
289.50 cfs	289.50 cfs	728.62	5.02	3.961	5-S2n	2.22	2.98	2.47	2.49	11.82	6.66
347.40 cfs	332.59 cfs	729.34	5.74	5.081	5-S2n	2.42	3.18	2.69	2.73	12.32	7.00
405.30 cfs	350.37 cfs	729.67	6.07	5.349	5-S2n	2.51	3.26	2.78	2.95	12.52	7.30
463.20 cfs	364.36 cfs	729.94	6.34	5.566	5-S2n	2.57	3.32	2.85	3.15	12.69	7.57
521.10 cfs	376.38 cfs	730.19	6.59	5.758	5-S2n	2.63	3.36	2.91	3.33	12.83	7.82
579.00 cfs	387.06 cfs	730.41	6.81	5.740	5-S2n	2.69	3.40	2.96	3.51	12.96	8.04

Culvert Barrel Data

Culvert Barrel Type Straight Culvert

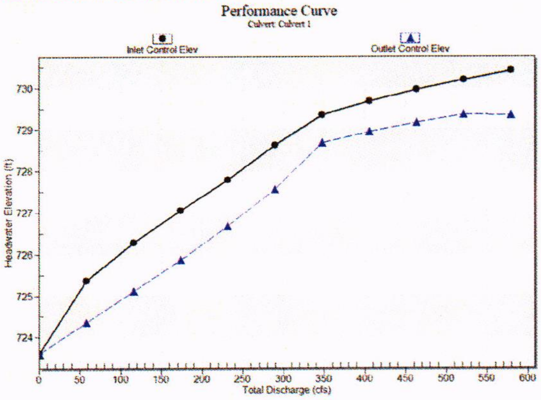
Inlet Elevation (invert): 723.60 ft,

Outlet Elevation (invert): 723.00 ft

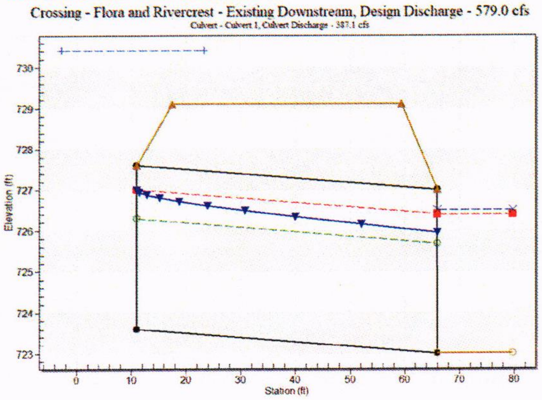
Culvert Length: 55.00 ft,

Culvert Slope: 0.0109

Culvert Performance Curve Plot: Culvert 1



Water Surface Profile Plot for Culvert: Culvert 1



Site Data - Culvert 1

Site Data Option: Culvert Invert Data

Inlet Station: 11.00 ft

Inlet Elevation: 723.60 ft

Outlet Station: 66.00 ft

Outlet Elevation: 723.00 ft

Number of Barrels: 3

Culvert Data Summary - Culvert 1

Barrel Shape: Circular

Barrel Diameter: 4.00 ft

Barrel Material: Concrete

Embedment: 0.00 in

Barrel Manning's n: 0.0120

Culvert Type: Straight

Inlet Configuration: Square Edge with Headwall

Inlet Depression: None

Roadway Top Width: 42.00 ft

Tailwater Data for Crossing: Flora and Rivercrest - Existing Downstream

Table A9 - Downstream Channel Rating Curve (Crossing: Flora and Rivercrest - Existing Downstream)

Flow (cfs)	Water Surface Elev (ft)	Velocity (ft/s)	Depth (ft)	Shear (psf)	Froude Number
0.00	723.00	0.00	0.00	0.00	0.00
57.90	724.06	1.06	4.16	1.05	0.80
115.80	724.54	1.54	5.13	1.54	0.84
173.70	724.91	1.91	5.77	1.91	0.86
231.60	725.22	2.22	6.26	2.22	0.88
289.50	725.49	2.49	6.66	2.48	0.89
347.40	725.73	2.73	7.00	2.72	0.90
405.30	725.95	2.95	7.30	2.94	0.91
463.20	726.15	3.15	7.57	3.14	0.92
521.10	726.33	3.33	7.82	3.33	0.92
579.00	726.51	3.51	8.04	3.50	0.93

Tailwater Channel Data - Flora and Rivercrest - Existing Downstream

Tailwater Channel Option: Trapezoidal Channel

Bottom Width: 10.00 ft

Side Slope (H:V): 3.00 (_:1)

Channel Slope: 0.0160

Channel Manning's n: 0.0400

Channel Invert Elevation: 723.00 ft

Roadway Data for Crossing: Flora and Rivercrest - Existing Downstream

Roadway Profile Shape: Constant Roadway Elevation

Crest Length: 42.00 ft

Crest Elevation: 729.10 ft

Roadway Surface: Paved

Project A – Design 2 (Rivercrest Culvert)
Flora and Rivercrest Downstream Culvert 2

Crossing Discharge Data

Discharge Selection Method: Specify Minimum, Design, and Maximum Flow

Minimum Flow: 0.00 cfs

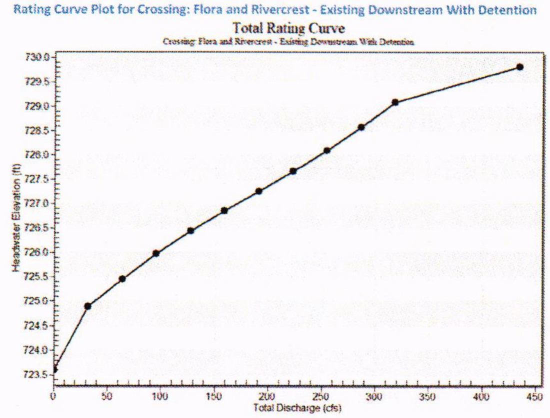
Design Flow: 318.00 cfs

Maximum Flow: 318.00 cfs

Table A10 - Summary of Culvert Flows at Crossing: Flora and Rivercrest - Existing Downstream With Detention

Headwater Elevation (ft)	Total Discharge (cfs)	Culvert 1 Discharge (cfs)	Roadway Discharge (cfs)	Iterations
723.60	0.00	0.00	0.00	1
724.89	31.80	31.80	0.00	1
725.46	63.60	63.60	0.00	1
725.97	95.40	95.40	0.00	1
726.44	127.20	127.20	0.00	1
726.86	159.00	159.00	0.00	1
727.26	190.80	190.80	0.00	1
727.66	222.60	222.60	0.00	1
728.09	254.40	254.40	0.00	1
728.56	286.20	286.20	0.00	1
729.08	318.00	318.00	0.00	1
729.10	318.96	318.96	0.00	Overtopping

Rating Curve Plot for Crossing: Flora and Rivercrest - Existing Downstream With Detention



Culvert Data: Culvert 1

Table A11 - Culvert Summary Table: Culvert 1

Total Discharge (cfs)	Culvert Discharge (cfs)	Headwater Elevation (ft)	Inlet Control Depth (ft)	Outlet Control Depth (ft)	Flow Type	Normal Depth (ft)	Critical Depth (ft)	Outlet Depth (ft)	Tailwater Depth (ft)	Outlet Velocity (ft/s)	Tailwater Velocity (ft/s)
0.00 cfs	0.00 cfs	723.60	0.00	0.000	0-NF	0.00	0.00	0.00	0.00	0.00	0.00
31.80 cfs	31.80 cfs	724.89	1.29	0.368	1-S2n	0.69	0.95	0.72	0.75	6.95	3.44
63.60 cfs	63.60 cfs	725.46	1.86	0.833	1-S2n	0.97	1.36	1.04	1.11	8.19	4.29
95.40 cfs	95.40 cfs	725.97	2.37	1.246	1-S2n	1.20	1.67	1.30	1.39	8.99	4.85
127.20 cfs	127.20 cfs	726.44	2.84	1.652	1-S2n	1.39	1.95	1.53	1.62	9.59	5.28
159.00 cfs	159.00 cfs	726.86	3.26	2.065	1-S2n	1.57	2.19	1.74	1.83	10.13	5.63
190.80 cfs	190.80 cfs	727.26	3.66	2.494	1-S2n	1.74	2.41	1.93	2.01	10.57	5.93
222.60 cfs	222.60 cfs	727.66	4.06	2.943	5-S2n	1.89	2.61	2.12	2.18	11.00	6.19
254.40 cfs	254.40 cfs	728.09	4.49	3.414	5-S2n	2.05	2.79	2.29	2.33	11.40	6.43
286.20 cfs	286.20 cfs	728.56	4.96	3.908	5-S2n	2.20	2.96	2.46	2.47	11.78	6.64
318.00 cfs	318.00 cfs	729.08	5.48	4.869	5-S2n	2.35	3.12	2.62	2.61	12.15	6.84

Culvert Barrel Data

Culvert Barrel Type Straight Culvert

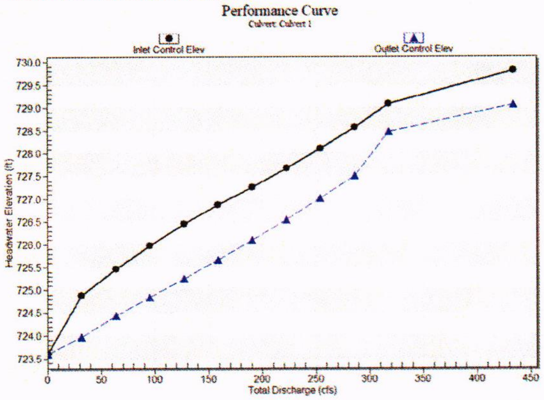
Inlet Elevation (invert): 723.60 ft,

Outlet Elevation (invert): 723.00 ft

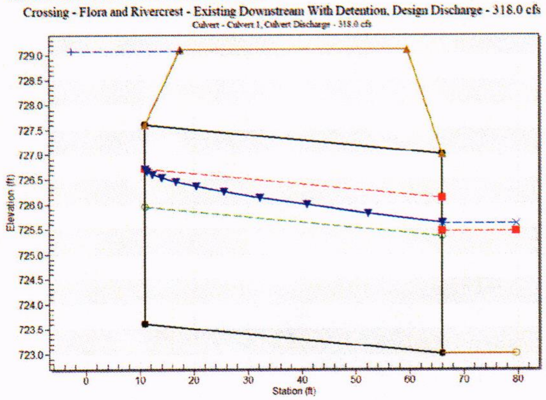
Culvert Length: 55.00 ft,

Culvert Slope: 0.0109

Culvert Performance Curve Plot: Culvert 1



Water Surface Profile Plot for Culvert: Culvert 1



Site Data - Culvert 1

Site Data Option: Culvert Invert Data

Inlet Station: 11.00 ft

Inlet Elevation: 723.60 ft

Outlet Station: 66.00 ft

Outlet Elevation: 723.00 ft

Number of Barrels: 3

Culvert Data Summary - Culvert 1

Barrel Shape: Circular

Barrel Diameter: 4.00 ft

Barrel Material: Concrete

Embedment: 0.00 in

Barrel Manning's n: 0.0120

Culvert Type: Straight

Inlet Configuration: Square Edge with Headwall (Ke=0.5)

Inlet Depression: None

Roadway Surface: Paved

Roadway Top Width: 42.00 ft

Tailwater Data for Crossing: Flora and Rivercrest - Existing Downstream With Detention

Table A12 - Downstream Channel Rating Curve (Crossing: Flora and Rivercrest - Existing Downstream With Detention)

Flow (cfs)	Water Surface Elev (ft)	Velocity (ft/s)	Depth (ft)	Shear (psf)	Froude Number
0.00	723.00	0.00	0.00	0.00	0.00
31.80	723.75	0.75	3.44	0.75	0.76
63.60	724.11	1.11	4.29	1.11	0.80
95.40	724.39	1.39	4.85	1.39	0.82
127.20	724.62	1.62	5.28	1.62	0.84
159.00	724.83	1.83	5.63	1.82	0.85
190.80	725.01	2.01	5.93	2.00	0.87
222.60	725.18	2.18	6.19	2.17	0.87
254.40	725.33	2.33	6.43	2.33	0.88
286.20	725.47	2.47	6.64	2.47	0.89
318.00	725.61	2.61	6.84	2.60	0.89

Tailwater Channel Data - Flora and Rivercrest - Existing Downstream With Detention

Tailwater Channel Option: Trapezoidal Channel

Bottom Width: 10.00 ft

Side Slope (H:V): 3.00 (_:1)

Channel Slope: 0.0160

Channel Manning's n: 0.0400

Channel Invert Elevation: 723.00 ft

Roadway Data for Crossing: Flora and Rivercrest - Existing Downstream With Detention

Roadway Profile Shape: Constant Roadway Elevation

Crest Length: 42.00 ft

Crest Elevation: 729.10 ft

Project D – Existing 1
Loy Lake Road at Waterloo Creek

Crossing Discharge Data

Discharge Selection Method: Specify Minimum, Design, and Maximum Flow

Minimum Flow: 100.00 cfs

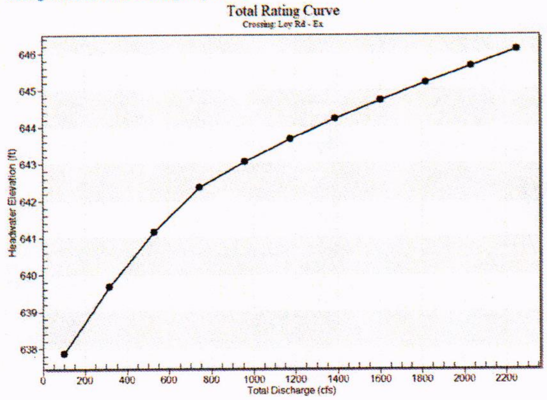
Design Flow: 2250.00 cfs

Maximum Flow: 2250.00 cfs

Table D1 - Summary of Culvert Flows at Crossing: Loy Rd - Ex

Headwater Elevation (ft)	Total Discharge (cfs)	Culvert 1 Discharge (cfs)	Roadway Discharge (cfs)	Iterations
637.87	100.00	100.00	0.00	1
639.66	315.00	315.00	0.00	1
641.14	530.00	530.00	0.00	1
642.36	745.00	687.78	57.20	6
643.07	960.00	765.59	194.40	4
643.67	1175.00	826.04	348.96	4
644.22	1390.00	876.90	513.09	4
644.73	1605.00	921.35	683.64	4
645.20	1820.00	961.12	858.87	4
645.66	2035.00	997.31	1037.69	4
646.10	2250.00	1030.72	1219.28	4
641.80	619.28	619.28	0.00	Overtopping

Rating Curve Plot for Crossing: Loy Rd - Ex



Culvert Data: Culvert 1

Table D2 - Culvert Summary Table: Culvert 1

Total Discharge (cfs)	Culvert Discharge (cfs)	Headwater Elevation (ft)	Inlet Control Depth (ft)	Outlet Control Depth (ft)	Flow Type	Normal Depth (ft)	Critical Depth (ft)	Outlet Depth (ft)	Tailwater Depth (ft)	Outlet Velocity (ft/s)	Tailwater Velocity (ft/s)
100.00 cfs	100.00 cfs	637.87	1.87	0.426	1-S2n	0.85	1.38	0.94	0.84	9.71	2.81
315.00 cfs	315.00 cfs	639.66	3.66	1.925	1-S2n	1.50	2.51	1.82	1.64	12.22	4.26
530.00 cfs	530.00 cfs	641.14	5.14	3.465	5-S2n	1.98	3.29	2.47	2.23	13.70	5.10
745.00 cfs	687.78 cfs	642.36	6.36	5.350	5-S2n	2.29	3.76	2.89	2.71	14.64	5.72
960.00 cfs	765.59 cfs	643.07	7.07	5.920	5-S2n	2.44	3.96	3.08	3.13	15.10	6.22
1175.00 cfs	826.04 cfs	643.67	7.67	6.390	5-S2n	2.55	4.09	3.22	3.51	15.46	6.64
1390.00 cfs	876.90 cfs	644.22	8.22	6.805	5-S2n	2.64	4.20	3.33	3.85	15.76	7.00
1605.00 cfs	921.35 cfs	644.73	8.73	7.181	5-S2n	2.72	4.29	3.43	4.17	16.04	7.32
1820.00 cfs	961.12 cfs	645.20	9.20	7.324	5-S2n	2.79	4.36	3.52	4.47	16.29	7.61
2035.00 cfs	997.31 cfs	645.66	9.66	7.903	5-S2n	2.86	4.42	3.59	4.76	16.52	7.88
2250.00 cfs	1030.72 cfs	646.10	10.10	8.455	5-S2n	2.92	4.47	3.66	5.03	16.74	8.12

Culvert Barrel Data

Culvert Barrel Type Straight Culvert

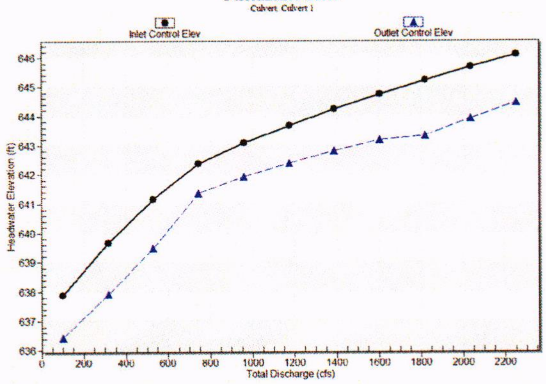
Inlet Elevation (invert): 636.00 ft,

Outlet Elevation (invert): 635.00 ft

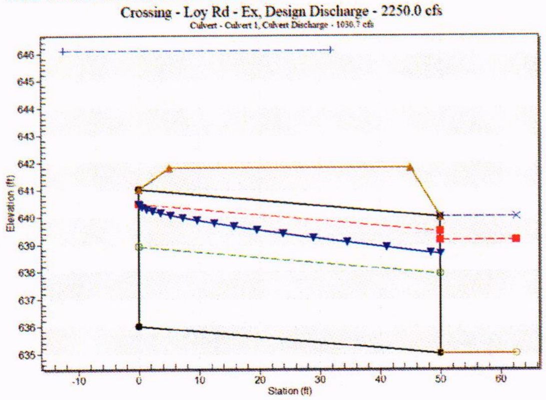
Culvert Length: 50.01 ft,

Culvert Slope: 0.0200

Culvert Performance Curve Plot: Culvert 1



Water Surface Profile Plot for Culvert: Culvert 1



Site Data - Culvert 1

Site Data Option: Culvert Invert Data

Inlet Station: 0.00 ft

Inlet Elevation: 636.00 ft

Outlet Station: 50.00 ft

Outlet Elevation: 635.00 ft

Number of Barrels: 4

Culvert Data Summary - Culvert 1

Barrel Shape: Circular

Barrel Diameter: 5.00 ft

Barrel Material: Concrete

Embedment: 0.00 in

Barrel Manning's n: 0.0120

Culvert Type: Straight

Inlet Configuration: Square Edge with Headwall

Inlet Depression: None

Roadway Top Width: 40.00 ft

Tailwater Data for Crossing: Loy Rd - Ex

Table D3 - Downstream Channel Rating Curve (Crossing: Loy Rd - Ex)

Flow (cfs)	Water Surface Elev (ft)	Velocity (ft/s)	Depth (ft)	Shear (psf)	Froude Number
100.00	635.84	0.84	2.81	0.52	0.56
315.00	636.64	1.64	4.26	1.03	0.62
530.00	637.23	2.23	5.10	1.39	0.64
745.00	637.71	2.71	5.72	1.69	0.66
960.00	638.13	3.13	6.22	1.95	0.68
1175.00	638.51	3.51	6.64	2.19	0.69
1390.00	638.85	3.85	7.00	2.40	0.70
1605.00	639.17	4.17	7.32	2.60	0.70
1820.00	639.47	4.47	7.61	2.79	0.71
2035.00	639.76	4.76	7.88	2.97	0.72
2250.00	640.03	5.03	8.12	3.14	0.72

Tailwater Channel Data - Loy Rd - Ex

Tailwater Channel Option: Trapezoidal Channel

Bottom Width: 40.00 ft

Side Slope (H:V): 3.00 (_:1)

Channel Slope: 0.0100

Channel Manning's n: 0.0450

Channel Invert Elevation: 635.00 ft

Roadway Data for Crossing: Loy Rd - Ex

Roadway Profile Shape: Constant Roadway Elevation

Crest Length: 45.00 ft

Crest Elevation: 641.80 ft

Roadway Surface: Paved

Project D – Design 1
Loy Lake Road at Waterloo Creek

Crossing Discharge Data

Discharge Selection Method: Specify Minimum, Design, and Maximum Flow

Minimum Flow: 160.00 cfs

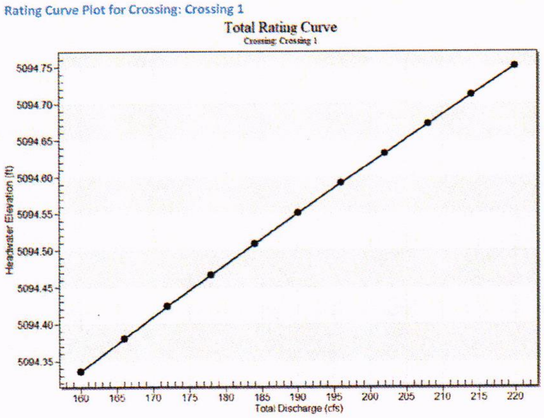
Design Flow: 160.00 cfs

Maximum Flow: 220.00 cfs

Table D4 - Summary of Culvert Flows at Crossing: Crossing 1

Headwater Elevation (ft)	Total Discharge (cfs)	Culvert 1 Discharge (cfs)	Culvert 2 Discharge (cfs)	Roadway Discharge (cfs)	Iterations
5094.34	160.00	151.72	8.27	0.00	7
5094.38	166.00	157.11	8.88	0.00	4
5094.42	172.00	162.50	9.49	0.00	4
5094.47	178.00	167.89	10.09	0.00	4
5094.51	184.00	173.28	10.71	0.00	4
5094.55	190.00	178.66	11.33	0.00	4
5094.59	196.00	184.04	11.94	0.00	4
5094.63	202.00	189.43	12.55	0.00	4
5094.67	208.00	194.81	13.17	0.00	4
5094.71	214.00	200.20	13.79	0.00	4
5094.75	220.00	205.58	14.41	0.00	4
5101.00	1155.91	1080.53	75.38	0.00	Overtopping

Rating Curve Plot for Crossing: Crossing 1



Culvert Data: Culvert 1

Table D5 - Culvert Summary Table: Culvert 1

Total Discharge (cfs)	Culvert Discharge (cfs)	Headwater Elevation (ft)	Inlet Control Depth (ft)	Outlet Control Depth (ft)	Flow Type	Normal Depth (ft)	Critical Depth (ft)	Outlet Depth (ft)	Tailwater Depth (ft)	Outlet Velocity (ft/s)	Tailwater Velocity (ft/s)
160.00 cfs	151.72 cfs	5094.34	2.09	2.337	2-M2c	1.58	1.53	1.53	0.86	5.96	3.74
166.00 cfs	157.11 cfs	5094.38	2.13	2.381	2-M2c	1.61	1.56	1.56	0.87	6.02	3.80
172.00 cfs	162.50 cfs	5094.42	2.17	2.425	2-M2c	1.64	1.58	1.58	0.89	6.08	3.85
178.00 cfs	167.89 cfs	5094.47	2.21	2.467	2-M2c	1.67	1.61	1.61	0.91	6.14	3.90
184.00 cfs	173.28 cfs	5094.51	2.25	2.509	2-M2c	1.69	1.64	1.64	0.93	6.20	3.95
190.00 cfs	178.66 cfs	5094.55	2.28	2.551	2-M2c	1.72	1.66	1.66	0.95	6.25	4.00
196.00 cfs	184.04 cfs	5094.59	2.32	2.592	2-M2c	1.75	1.69	1.69	0.97	6.30	4.05
202.00 cfs	189.43 cfs	5094.63	2.36	2.633	2-M2c	1.78	1.72	1.72	0.99	6.36	4.10
208.00 cfs	194.81 cfs	5094.67	2.39	2.673	2-M2c	1.80	1.74	1.74	1.00	6.41	4.15
214.00 cfs	200.20 cfs	5094.71	2.43	2.712	2-M2c	1.83	1.77	1.77	1.02	6.46	4.19
220.00 cfs	205.58 cfs	5094.75	2.46	2.752	2-M2c	1.86	1.79	1.79	1.04	6.51	4.24

Culvert Barrel Data

Culvert Barrel Type Straight Culvert

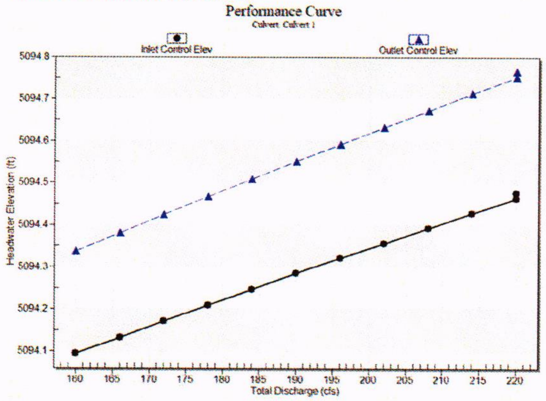
Inlet Elevation (invert): 5092.00 ft,

Outlet Elevation (invert): 5090.86 ft

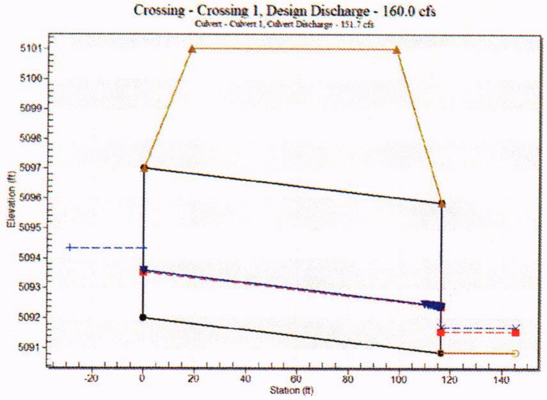
Culvert Length: 116.31 ft,

Culvert Slope: 0.0098

Culvert Performance Curve Plot: Culvert 1



Water Surface Profile Plot for Culvert: Culvert 1



Site Data - Culvert 1

Site Data Option: Culvert Invert Data

Inlet Station: 0.00 ft

Inlet Elevation: 5092.00 ft

Outlet Station: 116.30 ft

Outlet Elevation: 5090.86 ft

Number of Barrels: 5

Culvert Data Summary - Culvert 1

Barrel Shape: Circular

Barrel Diameter: 5.00 ft

Barrel Material: Corrugated Steel

Embedment: 0.00 in

Barrel Manning's n: 0.0240

Culvert Type: Straight

Inlet Configuration: Square Edge with Headwall

Inlet Depression: None

Culvert Data: Culvert 2

Table D6 - Culvert Summary Table: Culvert 2

Total Discharge (cfs)	Culvert Discharge (cfs)	Headwater Elevation (ft)	Inlet Control Depth (ft)	Outlet Control Depth (ft)	Flow Type	Normal Depth (ft)	Critical Depth (ft)	Outlet Depth (ft)	Tailwater Depth (ft)	Outlet Velocity (ft/s)	Tailwater Velocity (ft/s)
160.00 cfs	8.27 cfs	5094.34	0.99	1.336	7-A2c	-1.00	0.71	0.71	0.86	4.11	3.74
166.00 cfs	8.88 cfs	5094.38	1.03	1.380	7-A2c	-1.00	0.74	0.74	0.87	4.20	3.80
172.00 cfs	9.49 cfs	5094.42	1.08	1.424	7-A2c	-1.00	0.77	0.77	0.89	4.28	3.85
178.00 cfs	10.09 cfs	5094.47	1.12	1.466	7-A2c	-1.00	0.79	0.79	0.91	4.36	3.90
184.00 cfs	10.71 cfs	5094.51	1.16	1.509	7-A2c	-1.00	0.82	0.82	0.93	4.44	3.95
190.00 cfs	11.33 cfs	5094.55	1.20	1.550	7-A2c	-1.00	0.84	0.84	0.95	4.52	4.00
196.00 cfs	11.94 cfs	5094.59	1.25	1.591	7-A2c	-1.00	0.86	0.86	0.97	4.60	4.05
202.00 cfs	12.55 cfs	5094.63	1.28	1.632	7-A2c	-1.00	0.89	0.89	0.99	4.67	4.10
208.00 cfs	13.17 cfs	5094.67	1.32	1.672	7-A2c	-1.00	0.91	0.91	1.00	4.74	4.15
214.00 cfs	13.79 cfs	5094.71	1.36	1.711	7-A2c	-1.00	0.93	0.93	1.02	4.81	4.19
220.00 cfs	14.41 cfs	5094.75	1.40	1.751	7-A2c	-1.00	0.95	0.95	1.04	4.88	4.24

Culvert Barrel Data

Culvert Barrel Type Straight Culvert

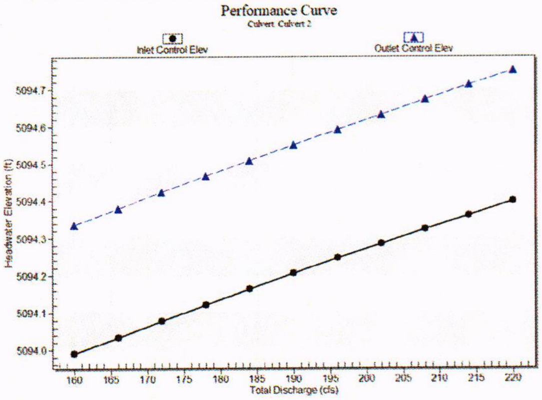
Inlet Elevation (invert): 5093.00 ft,

Outlet Elevation (invert): 5093.20 ft

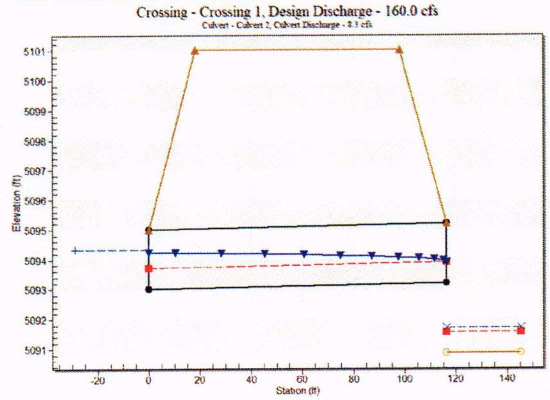
Culvert Length: 116.30 ft,

Culvert Slope: -0.0017

Culvert Performance Curve Plot: Culvert 2



Water Surface Profile Plot for Culvert: Culvert 2



Site Data - Culvert 2

Site Data Option: Culvert Invert Data

Inlet Station: 0.00 ft

Inlet Elevation: 5093.00 ft

Outlet Station: 116.30 ft

Outlet Elevation: 5093.20 ft

Number of Barrels: 2

Culvert Data Summary - Culvert 2

Barrel Shape: Circular

Barrel Diameter: 2.00 ft

Barrel Material: Concrete

Embedment: 0.00 in

Barrel Manning's n: 0.0120

Culvert Type: Straight

Inlet Configuration: Square Edge with Headwall

Inlet Depression: None

Tailwater Data for Crossing: Crossing 1

Table D7 - Downstream Channel Rating Curve (Crossing: Crossing 1)

Flow (cfs)	Water Surface Elev (ft)	Velocity (ft/s)	Depth (ft)	Shear (psf)	Froude Number
160.00	5091.72	0.86	3.74	0.53	0.71
166.00	5091.73	0.87	3.80	0.55	0.72
172.00	5091.75	0.89	3.85	0.56	0.72
178.00	5091.77	0.91	3.90	0.57	0.72
184.00	5091.79	0.93	3.95	0.58	0.72
190.00	5091.81	0.95	4.00	0.59	0.72
196.00	5091.83	0.97	4.05	0.60	0.73
202.00	5091.85	0.99	4.10	0.62	0.73
208.00	5091.86	1.00	4.15	0.63	0.73
214.00	5091.88	1.02	4.19	0.64	0.73
220.00	5091.90	1.04	4.24	0.65	0.73

Tailwater Channel Data - Crossing 1

Tailwater Channel Option: Rectangular Channel

Bottom Width: 50.00 ft

Channel Slope: 0.0100

Channel Manning's n: 0.0350

Channel Invert Elevation: 5090.86 ft

Roadway Data for Crossing: Crossing 1

Roadway Profile Shape: Constant Roadway Elevation

Crest Length: 100.00 ft

Crest Elevation: 5101.00 ft

Roadway Surface: Paved

Roadway Top Width: 80.00 ft

Project E – Existing 1
Loy Lake Road Culvert 1

Crossing Discharge Data

Discharge Selection Method: Specify Minimum, Design, and Maximum Flow

Minimum Flow: 0.00 cfs

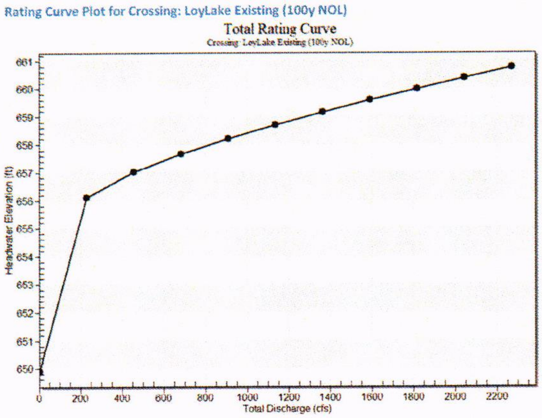
Design Flow: 2271.00 cfs

Maximum Flow: 2271.00 cfs

Table E1 - Summary of Culvert Flows at Crossing: LoyLake Existing (100y NOL)

Headwater Elevation (ft)	Total Discharge (cfs)	Culvert 1 Discharge (cfs)	Roadway Discharge (cfs)	Iterations
649.90	0.00	0.00	0.00	1
656.10	227.10	208.12	18.91	13
657.02	454.20	233.21	220.99	4
657.66	681.30	249.07	432.21	4
658.21	908.40	261.69	646.69	4
658.70	1135.50	272.44	863.05	4
659.15	1362.60	281.90	1080.69	4
659.58	1589.70	290.45	1299.24	4
659.99	1816.80	298.27	1518.52	4
660.37	2043.90	305.53	1738.36	4
660.75	2271.00	312.31	1958.69	4
655.88	201.41	201.41	0.00	Overtopping

Rating Curve Plot for Crossing: LoyLake Existing (100y NOL)



Culvert Data: Culvert 1

Table E2 - Culvert Summary Table: Culvert 1

Total Discharge (cfs)	Culvert Discharge (cfs)	Headwater Elevation (ft)	Inlet Control Depth (ft)	Outlet Control Depth (ft)	Flow Type	Normal Depth (ft)	Critical Depth (ft)	Outlet Depth (ft)	Tailwater Depth (ft)	Outlet Velocity (ft/s)	Tailwater Velocity (ft/s)
0.00 cfs	0.00 cfs	649.90	0.00	0.000	0-NF	0.00	0.00	0.00	2.00	0.00	0.00
227.10 cfs	208.12 cfs	656.10	6.20	4.996	5-S2n	2.80	3.09	2.80	2.00	11.08	0.00
454.20 cfs	233.21 cfs	657.02	7.12	5.937	5-S2n	3.08	3.26	3.08	2.00	11.25	0.00
681.30 cfs	249.07 cfs	657.66	7.76	6.576	5-S2n	3.28	3.35	3.28	2.00	11.28	0.00
908.40 cfs	261.69 cfs	658.21	8.31	7.210	7-M2c	3.51	3.42	3.42	2.00	11.44	0.00
1135.50 cfs	272.44 cfs	658.70	8.80	7.404	7-M2c	4.00	3.47	3.47	2.00	11.75	0.00
1362.60 cfs	281.90 cfs	659.15	9.25	7.889	7-M2c	4.00	3.52	3.52	2.00	12.04	0.00
1589.70 cfs	290.45 cfs	659.58	9.68	8.353	7-M2c	4.00	3.56	3.56	2.00	12.30	0.00
1816.80 cfs	298.27 cfs	659.99	10.09	8.764	7-M2c	4.00	3.59	3.59	2.00	12.55	0.00
2043.90 cfs	305.53 cfs	660.37	10.47	9.123	7-M2c	4.00	3.62	3.62	2.00	12.78	0.00
2271.00 cfs	312.31 cfs	660.75	10.85	9.476	7-M2c	4.00	3.64	3.64	2.00	13.00	0.00

Culvert Barrel Data

Culvert Barrel Type Straight Culvert

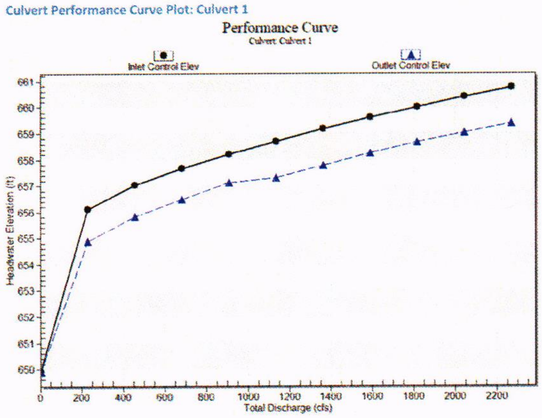
Inlet Elevation (invert): 649.90 ft,

Outlet Elevation (invert): 648.00 ft

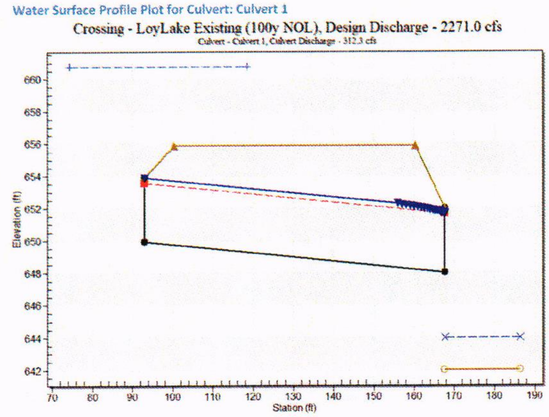
Culvert Length: 74.72 ft,

Culvert Slope: 0.0254

Culvert Performance Curve Plot: Culvert 1



Water Surface Profile Plot for Culvert: Culvert 1



Site Data - Culvert 1

Site Data Option: Culvert Invert Data

Inlet Station: 93.00 ft

Inlet Elevation: 649.90 ft

Outlet Station: 167.70 ft

Outlet Elevation: 648.00 ft

Number of Barrels: 2

Culvert Data Summary - Culvert 1

Barrel Shape: Circular

Barrel Diameter: 4.00 ft

Barrel Material: Corrugated Steel

Embedment: 0.00 in

Barrel Manning's n: 0.0240

Culvert Type: Straight

Inlet Configuration: Thin Edge Projecting (Ke=0.9)

Inlet Depression: None

Tailwater Data for Crossing: LoyLake Existing (100y NOL)

Table E3 - Downstream Channel Rating Curve (Crossing: LoyLake Existing (100y NOL))

Flow (cfs)	Water Surface Elev (ft)	Depth (ft)
0.00	644.00	2.00
227.10	644.00	2.00
454.20	644.00	2.00
681.30	644.00	2.00
908.40	644.00	2.00
1135.50	644.00	2.00
1362.60	644.00	2.00
1589.70	644.00	2.00
1816.80	644.00	2.00
2043.90	644.00	2.00
2271.00	644.00	2.00

Tailwater Channel Data - LoyLake Existing (100y NOL)

Tailwater Channel Option: Enter Constant Tailwater Elevation

Constant Tailwater Elevation: 644.00 ft

Roadway Data for Crossing: LoyLake Existing (100y NOL)

Roadway Profile Shape: Constant Roadway Elevation

Crest Length: 60.00 ft

Crest Elevation: 655.88 ft

Roadway Surface: Paved

Roadway Top Width: 60.00 ft

Project E – Design 1
Loy Lake Road Culvert 1

Crossing Discharge Data

Discharge Selection Method: Specify Minimum, Design, and Maximum Flow

Minimum Flow: 0.00 cfs

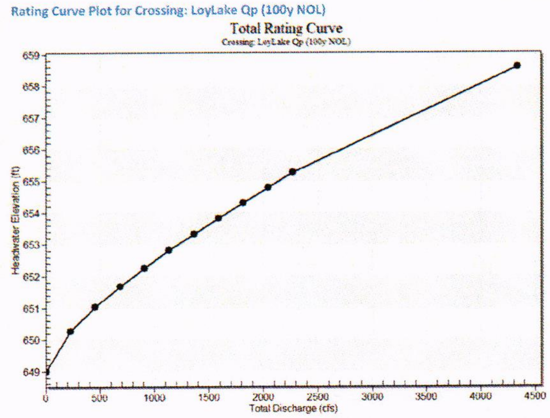
Design Flow: 2271.00 cfs

Maximum Flow: 2271.00 cfs

Table E4 - Summary of Culvert Flows at Crossing: LoyLake Qp (100y NOL)

Headwater Elevation (ft)	Total Discharge (cfs)	Culvert 1 Discharge (cfs)	Roadway Discharge (cfs)	Iterations
649.00	0.00	0.00	0.00	1
650.28	227.10	227.10	0.00	1
651.03	454.20	454.20	0.00	1
651.66	681.30	681.30	0.00	1
652.24	908.40	908.40	0.00	1
652.80	1135.50	1135.50	0.00	1
653.32	1362.60	1362.60	0.00	1
653.82	1589.70	1589.70	0.00	1
654.30	1816.80	1816.80	0.00	1
654.78	2043.90	2043.90	0.00	1
655.28	2271.00	2271.00	0.00	1
655.88	2533.09	2533.09	0.00	Overtopping

Rating Curve Plot for Crossing: LoyLake Qp (100y NOL)



Culvert Data: Culvert 1

Table E5 - Culvert Summary Table: Culvert 1

Total Discharge (cfs)	Culvert Discharge (cfs)	Headwater Elevation (ft)	Inlet Control Depth (ft)	Outlet Control Depth (ft)	Flow Type	Normal Depth (ft)	Critical Depth (ft)	Outlet Depth (ft)	Tailwater Depth (ft)	Outlet Velocity (ft/s)	Tailwater Velocity (ft/s)
0.00 cfs	0.00 cfs	649.00	0.00	0.000	0-NF	0.00	0.00	0.00	2.00	0.00	0.00
227.10 cfs	227.10 cfs	650.28	1.28	0.0*	1-S2n	0.33	0.89	0.33	2.00	14.55	0.00
454.20 cfs	454.20 cfs	651.03	2.03	0.0*	1-S2n	0.50	1.41	0.56	2.00	16.80	0.00
681.30 cfs	681.30 cfs	651.66	2.66	0.0*	1-S2n	0.65	1.84	0.77	2.00	18.32	0.00
908.40 cfs	908.40 cfs	652.24	3.24	0.0*	1-S2n	0.78	2.23	0.98	2.00	19.41	0.00
1135.50 cfs	1135.50 cfs	652.80	3.80	0.0*	1-S2n	0.90	2.59	1.17	2.00	20.20	0.00
1362.60 cfs	1362.60 cfs	653.32	4.32	0.0*	1-S2n	1.02	2.93	1.36	2.00	20.86	0.00
1589.70 cfs	1589.70 cfs	653.82	4.82	0.0*	1-S2n	1.12	3.24	1.55	2.00	21.43	0.00
1816.80 cfs	1816.80 cfs	654.30	5.30	0.0*	1-S2n	1.23	3.54	1.73	2.00	21.91	0.00
2043.90 cfs	2043.90 cfs	654.78	5.78	0.047	1-S2n	1.33	3.83	1.91	2.00	22.35	0.00
2271.00 cfs	2271.00 cfs	655.28	6.28	0.611	5-S2n	1.43	4.11	2.08	2.00	22.75	0.00

* Full Flow Headwater elevation is below inlet invert.

Culvert Barrel Data

Culvert Barrel Type Straight Culvert

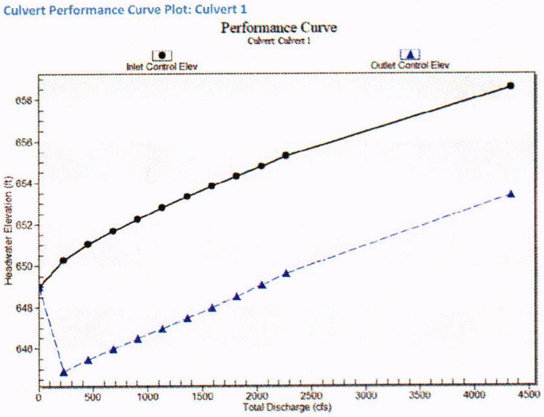
Inlet Elevation (invert): 649.00 ft,

Outlet Elevation (invert): 644.00 ft

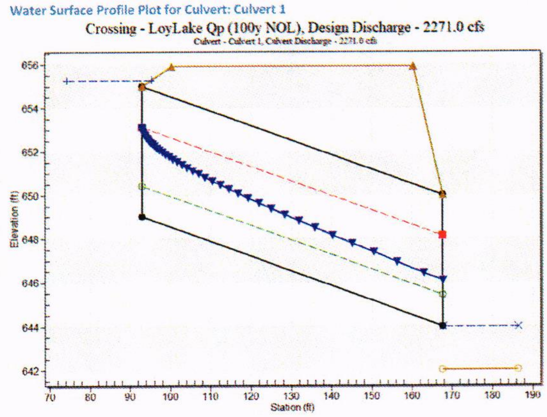
Culvert Length: 74.87 ft,

Culvert Slope: 0.0669

Culvert Performance Curve Plot: Culvert 1



Water Surface Profile Plot for Culvert: Culvert 1



Site Data - Culvert 1

Site Data Option: Culvert Invert Data

Inlet Station: 93.00 ft

Inlet Elevation: 649.00 ft

Outlet Station: 167.70 ft

Outlet Elevation: 644.00 ft

Number of Barrels: 6

Culvert Data Summary - Culvert 1

Barrel Shape: Concrete Box

Barrel Span: 8.00 ft

Barrel Rise: 6.00 ft

Barrel Material: Concrete

Embedment: 0.00 in

Barrel Manning's n: 0.0120

Culvert Type: Straight

Inlet Configuration: Square Edge (30-75º flare) Wingwall (Ke=0.4)

Inlet Depression: None

Tailwater Data for Crossing: LoyLake Qp (100y NOL)

Table E6 - Downstream Channel Rating Curve (Crossing: LoyLake Qp (100y NOL))

Flow (cfs)	Water Surface Elev (ft)	Depth (ft)
0.00	644.00	2.00
227.10	644.00	2.00
454.20	644.00	2.00
681.30	644.00	2.00
908.40	644.00	2.00
1135.50	644.00	2.00
1362.60	644.00	2.00
1589.70	644.00	2.00
1816.80	644.00	2.00
2043.90	644.00	2.00
2271.00	644.00	2.00

Tailwater Channel Data - LoyLake Qp (100y NOL)

Tailwater Channel Option: Enter Constant Tailwater Elevation

Constant Tailwater Elevation: 644.00 ft

Roadway Data for Crossing: LoyLake Qp (100y NOL)

Roadway Profile Shape: Constant Roadway Elevation

Crest Length: 60.00 ft

Crest Elevation: 655.88 ft

Roadway Surface: Paved

Roadway Top Width: 60.00 ft

Project F – Existing
Coffin and Woodlawn Culvert

Crossing Discharge Data

Discharge Selection Method: Specify Minimum, Design, and Maximum Flow

Minimum Flow: 0.00 cfs

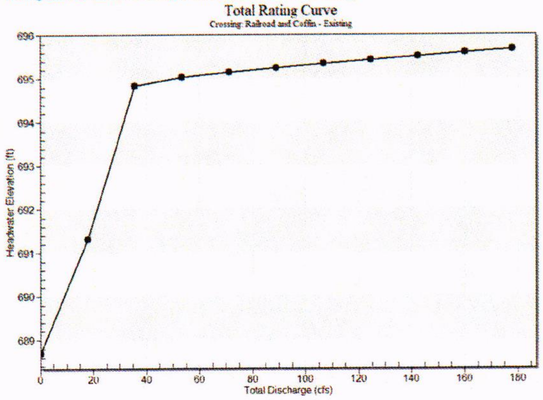
Design Flow: 178.00 cfs

Maximum Flow: 178.00 cfs

Table F1 - Summary of Culvert Flows at Crossing: Railroad and Coffin - Existing

Headwater Elevation (ft)	Total Discharge (cfs)	Culvert 1 Discharge (cfs)	Roadway Discharge (cfs)	Iterations
688.70	0.00	0.00	0.00	1
691.31	17.80	17.80	0.00	1
694.84	35.60	34.53	0.99	36
695.03	53.40	35.15	18.20	8
695.15	71.20	35.55	35.61	6
695.25	89.00	35.88	53.09	5
695.35	106.80	35.69	71.09	5
695.43	124.60	35.37	89.22	5
695.51	142.40	35.09	107.30	4
695.59	160.20	34.82	125.37	4
695.66	178.00	34.58	143.42	4
694.81	34.42	34.42	0.00	Overtopping

Rating Curve Plot for Crossing: Railroad and Coffin - Existing



Culvert Data: Culvert 1

Table F2 - Culvert Summary Table: Culvert 1

Total Discharge (cfs)	Culvert Discharge (cfs)	Headwater Elevation (ft)	Inlet Control Depth (ft)	Outlet Control Depth (ft)	Flow Type	Normal Depth (ft)	Critical Depth (ft)	Outlet Depth (ft)	Tailwater Depth (ft)	Outlet Velocity (ft/s)	Tailwater Velocity (ft/s)
0.00 cfs	0.00 cfs	688.70	0.00	0.000	0-NF	0.00	0.00	0.00	0.00	0.00	0.00
17.80 cfs	17.80 cfs	691.31	2.61	2.301	5-S2n	1.40	1.52	1.40	0.97	7.56	2.31
35.60 cfs	34.53 cfs	694.84	6.14	5.946	7-M2c	2.00	1.87	1.87	1.39	11.30	2.80
53.40 cfs	35.15 cfs	695.03	6.33	6.135	7-M2c	2.00	1.84	1.84	1.69	11.64	3.13
71.20 cfs	35.55 cfs	695.15	6.45	6.225	7-M2t	2.00	1.82	1.95	1.95	11.40	3.38
89.00 cfs	35.88 cfs	695.25	6.55	6.505	4-FFf	2.00	1.81	2.00	2.16	11.42	3.58
106.80 cfs	35.69 cfs	695.35	6.49	6.646	4-FFf	2.00	1.82	2.00	2.36	11.36	3.76
124.60 cfs	35.37 cfs	695.43	6.39	6.731	4-FFf	2.00	1.83	2.00	2.53	11.26	3.91
142.40 cfs	35.09 cfs	695.51	6.31	6.813	4-FFf	2.00	1.84	2.00	2.69	11.17	4.05
160.20 cfs	34.82 cfs	695.59	6.23	6.890	4-FFf	2.00	1.85	2.00	2.84	11.08	4.17
178.00 cfs	34.58 cfs	695.66	6.16	6.963	4-FFf	2.00	1.87	2.00	2.98	11.01	4.28

Culvert Barrel Data

Culvert Barrel Type Straight Culvert

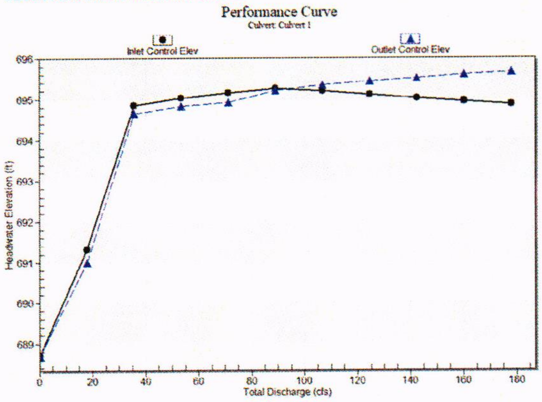
Inlet Elevation (invert): 688.70 ft,

Outlet Elevation (invert): 688.00 ft

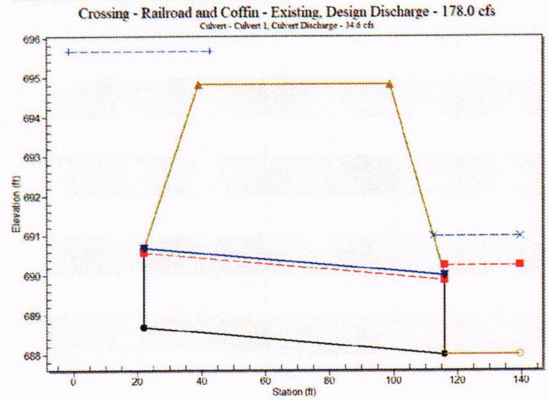
Culvert Length: 94.01 ft,

Culvert Slope: 0.0074

Culvert Performance Curve Plot: Culvert 1



Water Surface Profile Plot for Culvert: Culvert 1



Site Data - Culvert 1

Site Data Option: Culvert Invert Data

Inlet Station: 22.00 ft

Inlet Elevation: 688.70 ft

Outlet Station: 116.01 ft

Outlet Elevation: 688.00 ft

Number of Barrels: 1

Culvert Data Summary - Culvert 1

Barrel Shape: Circular

Barrel Diameter: 2.00 ft

Barrel Material: Concrete

Embedment: 0.00 in

Barrel Manning's n: 0.0120

Roadway Top Width: 60.00 ft

Culvert Type: Straight

Inlet Configuration: Square Edge with Headwall

Inlet Depression: None

Tailwater Data for Crossing: Railroad and Coffin - Existing

Table F3 - Downstream Channel Rating Curve (Crossing: Railroad and Coffin - Existing)

Flow (cfs)	Water Surface Elev (ft)	Velocity (ft/s)	Depth (ft)	Shear (psf)	Froude Number
0.00	688.00	0.00	0.00	0.00	0.00
17.80	688.97	0.97	2.31	0.73	0.48
35.60	689.39	1.39	2.80	1.04	0.51
53.40	689.69	1.69	3.13	1.27	0.52
71.20	689.95	1.95	3.38	1.46	0.53
89.00	690.16	2.16	3.58	1.62	0.54
106.80	690.36	2.36	3.76	1.76	0.54
124.60	690.53	2.53	3.91	1.90	0.55
142.40	690.69	2.69	4.05	2.02	0.55
160.20	690.84	2.84	4.17	2.13	0.56
178.00	690.98	2.98	4.28	2.23	0.56

Tailwater Channel Data - Railroad and Coffin - Existing

Tailwater Channel Option: Trapezoidal Channel

Bottom Width: 5.00 ft

Side Slope (H:V): 3.00 (_:1)

Channel Slope: 0.0120

Channel Manning's n: 0.0550

Channel Invert Elevation: 688.00 ft

Roadway Data for Crossing: Railroad and Coffin - Existing

Roadway Profile Shape: Constant Roadway Elevation

Crest Length: 60.00 ft

Crest Elevation: 694.81 ft

Roadway Surface: Paved

Project H – Existing
Lillis & Crawford Culvert

Crossing Discharge Data

Discharge Selection Method: Specify Minimum, Design, and Maximum Flow

Minimum Flow: 0.00 cfs

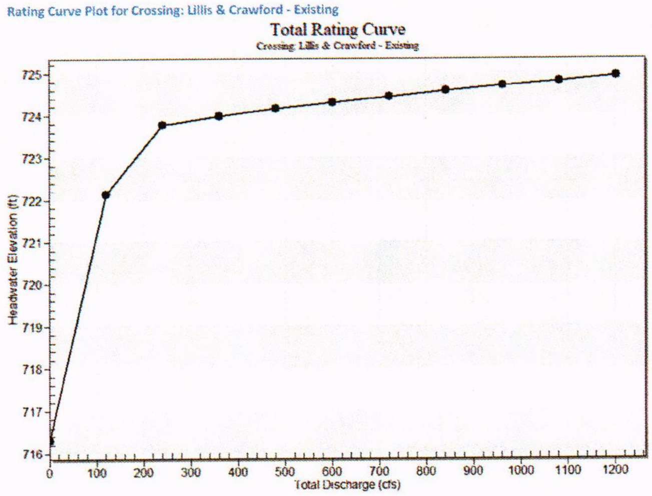
Design Flow: 1203.00 cfs

Maximum Flow: 1203.00 cfs

Table H1 - Summary of Culvert Flows at Crossing: Lillis & Crawford - Existing

Headwater Elevation (ft)	Total Discharge (cfs)	Culvert 1 Discharge (cfs)	Roadway Discharge (cfs)	Iterations
716.30	0.00	0.00	0.00	1
722.14	120.30	120.30	0.00	1
723.77	240.60	155.92	84.64	9
723.98	360.90	159.93	200.82	6
724.15	481.20	163.15	318.00	6
724.30	601.50	165.96	435.42	5
724.44	721.80	168.49	553.27	5
724.57	842.10	170.82	671.27	5
724.69	962.40	172.98	789.38	4
724.81	1082.70	175.01	907.68	4
724.92	1203.00	176.93	1025.98	3
723.50	150.48	150.48	0.00	Overtopping

Rating Curve Plot for Crossing: Lillis & Crawford - Existing



Culvert Data: Culvert 1

Table H2 - Culvert Summary Table: Culvert 1

Total Discharge (cfs)	Culvert Discharge (cfs)	Headwater Elevation (ft)	Inlet Control Depth (ft)	Outlet Control Depth (ft)	Flow Type	Normal Depth (ft)	Critical Depth (ft)	Outlet Depth (ft)	Tailwater Depth (ft)	Outlet Velocity (ft/s)	Tailwater Velocity (ft/s)
0.00 cfs	0.00 cfs	716.30	0.00	0.000	0-NF	0.00	0.00	0.00	0.50	0.00	0.00
120.30 cfs	120.30 cfs	722.14	5.84	1.531	5-S2n	2.34	3.68	2.43	0.50	16.49	0.00
240.60 cfs	155.92 cfs	723.77	7.47	4.134	5-S2n	2.88	4.38	3.00	0.50	17.30	0.00
360.90 cfs	159.93 cfs	723.98	7.68	4.377	5-S2n	2.94	4.45	3.07	0.50	17.38	0.00
481.20 cfs	163.15 cfs	724.15	7.85	4.575	5-S2n	2.99	4.51	3.12	0.50	17.45	0.00
601.50 cfs	165.96 cfs	724.30	8.00	4.751	5-S2n	3.03	4.56	3.16	0.50	17.51	0.00
721.80 cfs	168.49 cfs	724.44	8.14	4.912	5-S2n	3.07	4.61	3.20	0.50	17.55	0.00
842.10 cfs	170.82 cfs	724.57	8.27	5.061	5-S2n	3.10	4.65	3.24	0.50	17.59	0.00
962.40 cfs	172.98 cfs	724.69	8.39	5.201	5-S2n	3.14	4.69	3.27	0.50	17.63	0.00
1082.70 cfs	175.01 cfs	724.81	8.51	5.334	5-S2n	3.17	4.73	3.30	0.50	17.66	0.00
1203.00 cfs	176.93 cfs	724.92	8.62	5.461	5-S2n	3.20	4.76	3.33	0.50	17.70	0.00

Culvert Barrel Data

Culvert Barrel Type Straight Culvert

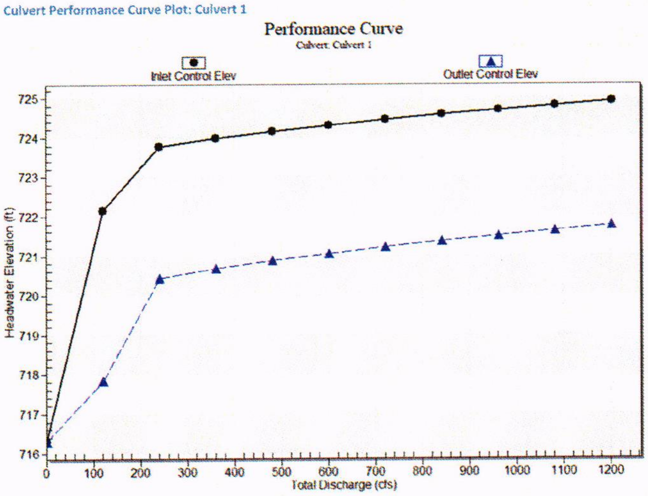
Inlet Elevation (invert): 716.30 ft,

Outlet Elevation (invert): 711.80 ft

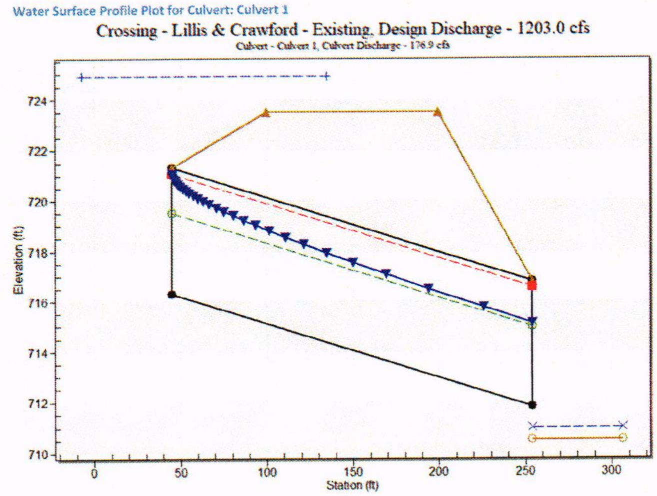
Culvert Length: 209.05 ft,

Culvert Slope: 0.0215

Culvert Performance Curve Plot: Culvert 1



Water Surface Profile Plot for Culvert: Culvert 1



Site Data - Culvert 1

Site Data Option: Culvert Invert Data

Inlet Station: 45.00 ft

Inlet Elevation: 716.30 ft

Outlet Station: 254.00 ft

Outlet Elevation: 711.80 ft

Number of Barrels: 1

Culvert Data Summary - Culvert 1

Barrel Shape: Concrete Box

Barrel Span: 3.00 ft

Barrel Rise: 5.00 ft

Barrel Material: Concrete

Embedment: 0.00 in

Barrel Manning's n: 0.0120

Culvert Type: Straight

Inlet Configuration: Square Edge (30-75º flare) Wingwall

Inlet Depression: None

Tailwater Data for Crossing: Lillis & Crawford - Existing

Table H3 - Downstream Channel Rating Curve (Crossing: Lillis & Crawford - Existing)

Flow (cfs)	Water Surface Elev (ft)	Depth (ft)
0.00	711.00	0.50
120.30	711.00	0.50
240.60	711.00	0.50
360.90	711.00	0.50
481.20	711.00	0.50
601.50	711.00	0.50
721.80	711.00	0.50
842.10	711.00	0.50
962.40	711.00	0.50
1082.70	711.00	0.50
1203.00	711.00	0.50

Tailwater Channel Data - Lillis & Crawford - Existing

Tailwater Channel Option: Enter Constant Tailwater Elevation

Constant Tailwater Elevation: 711.00 ft

Roadway Data for Crossing: Lillis & Crawford - Existing

Roadway Profile Shape: Constant Roadway Elevation

Crest Length: 200.00 ft

Crest Elevation: 723.50 ft

Roadway Surface: Paved

Roadway Top Width: 100.00 ft

Project H – Design
Lillis & Crawford Culvert

Crossing Discharge Data

Discharge Selection Method: Specify Minimum, Design, and Maximum Flow

Minimum Flow: 0.00 cfs

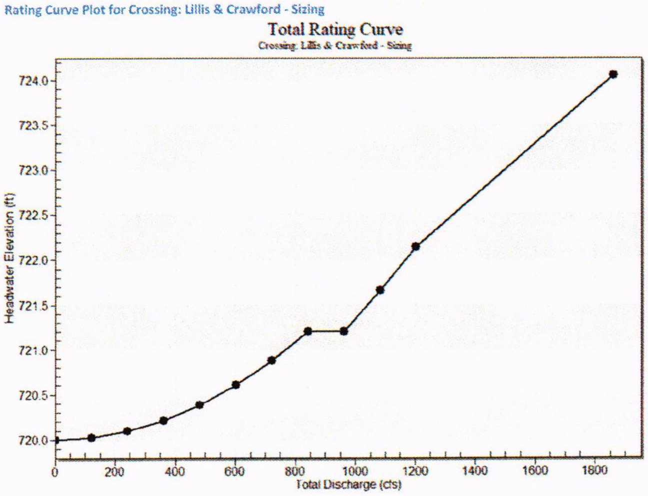
Design Flow: 1203.00 cfs

Maximum Flow: 1203.00 cfs

Table H4 - Summary of Culvert Flows at Crossing: Lillis & Crawford - Sizing

Headwater Elevation (ft)	Total Discharge (cfs)	Culvert 1 Discharge (cfs)	Roadway Discharge (cfs)	Iterations
720.00	0.00	0.00	0.00	1
720.02	120.30	120.30	0.00	1
720.10	240.60	240.60	0.00	1
720.21	360.90	360.90	0.00	1
720.38	481.20	481.20	0.00	1
720.60	601.50	601.50	0.00	1
720.87	721.80	721.80	0.00	1
721.20	842.10	842.10	0.00	1
721.20	962.40	962.40	0.00	1
721.66	1082.70	1082.70	0.00	1
722.14	1203.00	1203.00	0.00	1
723.50	1504.84	1504.84	0.00	Overtopping

Rating Curve Plot for Crossing: Lillis & Crawford - Sizing



Culvert Data: Culvert 1

Table H5 - Culvert Summary Table: Culvert 1

Total Discharge (cfs)	Culvert Discharge (cfs)	Headwater Elevation (ft)	Inlet Control Depth (ft)	Outlet Control Depth (ft)	Flow Type	Normal Depth (ft)	Critical Depth (ft)	Outlet Depth (ft)	Tailwater Depth (ft)	Outlet Velocity (ft/s)	Tailwater Velocity (ft/s)
0.00 cfs	0.00 cfs	720.00	0.00	3.700	0-NF	0.00	0.00	5.00	9.50	0.00	0.00
120.30 cfs	120.30 cfs	720.02	1.19	3.724	1-S1f	0.43	0.79	5.00	9.50	0.80	0.00
240.60 cfs	240.60 cfs	720.10	1.90	3.795	1-S1f	0.67	1.26	5.00	9.50	1.60	0.00
360.90 cfs	360.90 cfs	720.21	2.48	3.915	1-S1f	0.88	1.65	5.00	9.50	2.41	0.00
481.20 cfs	481.20 cfs	720.38	3.03	4.083	1-S1f	1.07	2.00	5.00	9.50	3.21	0.00
601.50 cfs	601.50 cfs	720.60	3.53	4.301	1-S1f	1.25	2.32	5.00	9.50	4.01	0.00
721.80 cfs	721.80 cfs	720.87	4.00	4.573	1-S1f	1.41	2.62	5.00	9.50	4.81	0.00
842.10 cfs	842.10 cfs	721.20	4.45	4.905	1-S1f	1.58	2.90	5.00	9.50	5.61	0.00
962.40 cfs	962.40 cfs	721.20	4.90	3.700	1-S2n	1.73	3.17	1.86	9.50	17.23	0.00
1082.70 cfs	1082.70 cfs	721.66	5.36	5.356	5-JS1f	1.89	3.43	5.00	9.50	7.22	0.00
1203.00 cfs	1203.00 cfs	722.14	5.84	5.745	5-JS1f	2.04	3.68	5.00	9.50	8.02	0.00

Culvert Barrel Data

Culvert Barrel Type Straight Culvert

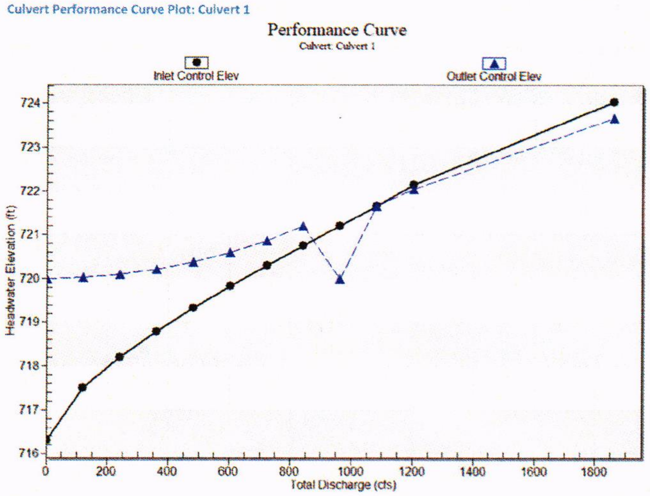
Inlet Elevation (invert): 716.30 ft,

Outlet Elevation (invert): 711.80 ft

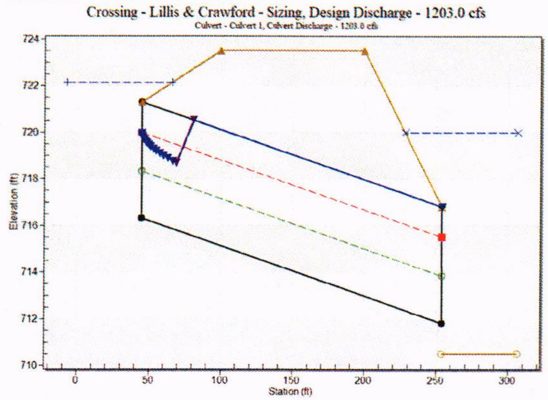
Culvert Length: 209.05 ft,

Culvert Slope: 0.0215

Culvert Performance Curve Plot: Culvert 1



Water Surface Profile Plot for Culvert: Culvert 1



Site Data - Culvert 1

Site Data Option: Culvert Invert Data

Inlet Station: 45.00 ft

Inlet Elevation: 716.30 ft

Outlet Station: 254.00 ft

Outlet Elevation: 711.80 ft

Number of Barrels: 6

Culvert Data Summary - Culvert 1

Barrel Shape: Concrete Box

Barrel Span: 5.00 ft

Barrel Rise: 5.00 ft

Barrel Material: Concrete

Embedment: 0.00 in

Barrel Manning's n: 0.0120

Culvert Type: Straight

Inlet Configuration: Square Edge (30-75º flare) Wingwall

Inlet Depression: None

Tailwater Data for Crossing: Lillis & Crawford - Sizing

Table H6 - Downstream Channel Rating Curve (Crossing: Lillis & Crawford - Sizing)

Flow (cfs)	Water Surface Elev (ft)	Depth (ft)
0.00	720.00	9.50
120.30	720.00	9.50
240.60	720.00	9.50
360.90	720.00	9.50
481.20	720.00	9.50
601.50	720.00	9.50
721.80	720.00	9.50
842.10	720.00	9.50
962.40	720.00	9.50
1082.70	720.00	9.50
1203.00	720.00	9.50

Tailwater Channel Data - Lillis & Crawford - Sizing

Tailwater Channel Option: Enter Constant Tailwater Elevation

Constant Tailwater Elevation: 720.00 ft

Roadway Data for Crossing: Lillis & Crawford - Sizing

Roadway Profile Shape: Constant Roadway Elevation

Crest Length: 200.00 ft

Crest Elevation: 723.50 ft

Roadway Surface: Paved

Roadway Top Width: 100.00 ft

Project J – Existing 1
Dubois Between Scullin and Barrett Culvert 1

Crossing Discharge Data

Discharge Selection Method: Specify Minimum, Design, and Maximum Flow

Minimum Flow: 0.00 cfs

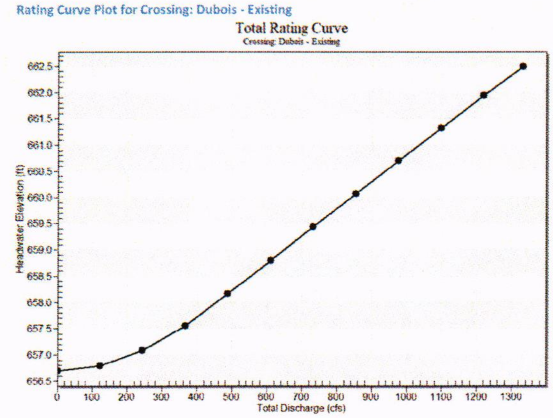
Design Flow: 1220.00 cfs

Maximum Flow: 1220.00 cfs

Table J1 - Summary of Culvert Flows at Crossing: Dubois - Existing

Headwater Elevation (ft)	Total Discharge (cfs)	Culvert 1 Discharge (cfs)	Roadway Discharge (cfs)	Iterations
656.70	0.00	0.00	0.00	1
656.79	122.00	122.00	0.00	1
657.08	244.00	244.00	0.00	1
657.55	366.00	366.00	0.00	1
658.16	488.00	480.10	7.90	3
658.79	610.00	574.02	35.99	3
659.43	732.00	656.04	75.95	3
660.07	854.00	728.23	125.77	3
660.70	976.00	793.37	182.62	3
661.32	1098.00	852.88	245.13	4
661.93	1220.00	907.82	312.18	3
657.80	416.20	416.20	0.00	Overtopping

Rating Curve Plot for Crossing: Dubois - Existing



Culvert Data: Culvert 1

Table J2 - Culvert Summary Table: Culvert 1

Total Discharge (cfs)	Culvert Discharge (cfs)	Headwater Elevation (ft)	Inlet Control Depth (ft)	Outlet Control Depth (ft)	Flow Type	Normal Depth (ft)	Critical Depth (ft)	Outlet Depth (ft)	Tailwater Depth (ft)	Outlet Velocity (ft/s)	Tailwater Velocity (ft/s)
0.00 cfs	0.00 cfs	656.70	0.00	7.200	0-NF	0.00	0.00	4.00	7.30	0.00	0.00
122.00 cfs	122.00 cfs	656.79	2.02	7.295	4-FFf	1.39	1.46	4.00	7.30	1.94	0.00
244.00 cfs	244.00 cfs	657.08	3.11	7.578	4-FFf	2.04	2.09	4.00	7.30	3.88	0.00
366.00 cfs	366.00 cfs	657.55	4.04	8.051	4-FFf	2.65	2.59	4.00	7.30	5.83	0.00
488.00 cfs	480.10 cfs	658.16	5.01	8.664	4-FFf	3.35	2.97	4.00	7.30	7.64	0.00
610.00 cfs	574.02 cfs	658.79	5.97	9.292	4-FFf	4.00	3.23	4.00	7.30	9.14	0.00
732.00 cfs	656.04 cfs	659.43	6.97	9.933	4-FFf	4.00	3.42	4.00	7.30	10.44	0.00
854.00 cfs	728.23 cfs	660.07	7.97	10.568	4-FFf	4.00	3.56	4.00	7.30	11.59	0.00
976.00 cfs	793.37 cfs	660.70	8.98	11.197	4-FFf	4.00	3.66	4.00	7.30	12.63	0.00
1098.00 cfs	852.88 cfs	661.32	9.98	11.819	4-FFf	4.00	3.73	4.00	7.30	13.57	0.00
1220.00 cfs	907.82 cfs	661.93	10.96	12.433	4-FFf	4.00	3.78	4.00	7.30	14.45	0.00

Culvert Barrel Data

Culvert Barrel Type Straight Culvert

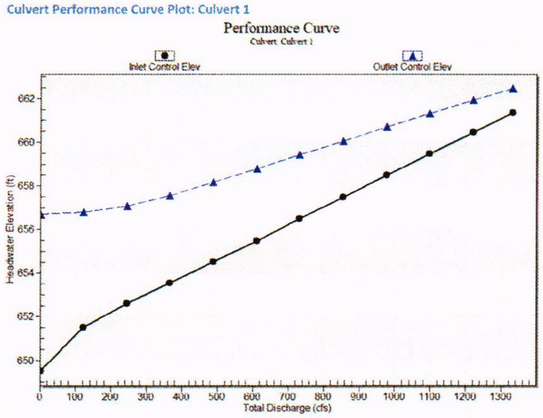
Inlet Elevation (invert): 649.50 ft,

Outlet Elevation (invert): 649.40 ft

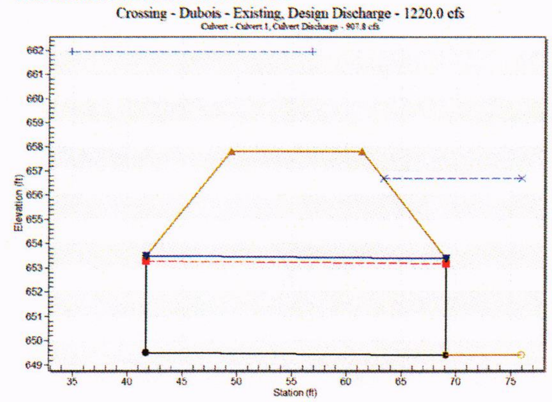
Culvert Length: 27.40 ft,

Culvert Slope: 0.0036

Culvert Performance Curve Plot: Culvert 1



Water Surface Profile Plot for Culvert: Culvert 1



Site Data - Culvert 1

Site Data Option: Culvert Invert Data

Inlet Station: 41.70 ft

Inlet Elevation: 649.50 ft

Outlet Station: 69.10 ft

Outlet Elevation: 649.40 ft

Number of Barrels: 5

Culvert Data Summary - Culvert 1

Barrel Shape: Circular

Barrel Diameter: 4.00 ft

Barrel Material: Concrete

Embedment: 0.00 in

Barrel Manning's n: 0.0120

Culvert Type: Straight

Inlet Configuration: Square Edge with Headwall

Inlet Depression: None

Tailwater Data for Crossing: Dubois - Existing

Table J3 - Downstream Channel Rating Curve (Crossing: Dubois - Existing)

Flow (cfs)	Water Surface Elev (ft)	Depth (ft)
0.00	656.70	7.30
122.00	656.70	7.30
244.00	656.70	7.30
366.00	656.70	7.30
488.00	656.70	7.30
610.00	656.70	7.30
732.00	656.70	7.30
854.00	656.70	7.30
976.00	656.70	7.30
1098.00	656.70	7.30
1220.00	656.70	7.30

Tailwater Channel Data - Dubois - Existing

Tailwater Channel Option: Enter Constant Tailwater Elevation

Constant Tailwater Elevation: 656.70 ft

Roadway Data for Crossing: Dubois - Existing

Roadway Profile Shape: Constant Roadway Elevation

Crest Length: 12.00 ft

Crest Elevation: 657.80 ft

Roadway Surface: Paved

Roadway Top Width: 12.00 ft

Project J – Design 1
Dubois Between Scullin and Barrett Culvert 1

Crossing Discharge Data

Discharge Selection Method: Specify Minimum, Design, and Maximum Flow

Minimum Flow: 0.00 cfs

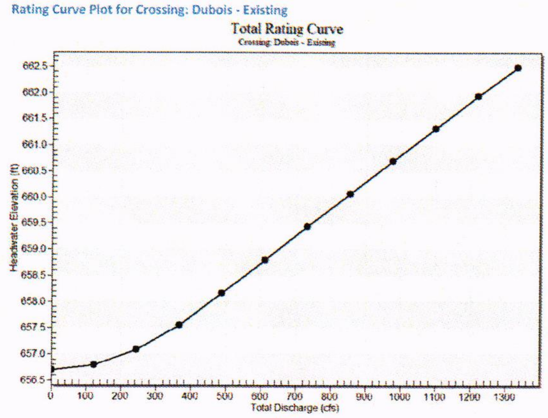
Design Flow: 1220.00 cfs

Maximum Flow: 1220.00 cfs

Table J4 - Summary of Culvert Flows at Crossing: Dubois - Existing

Headwater Elevation (ft)	Total Discharge (cfs)	Culvert 1 Discharge (cfs)	Roadway Discharge (cfs)	Iterations
656.70	0.00	0.00	0.00	1
656.79	122.00	122.00	0.00	1
657.08	244.00	244.00	0.00	1
657.55	366.00	366.00	0.00	1
658.16	488.00	480.10	7.90	3
658.79	610.00	574.02	35.99	3
659.43	732.00	656.04	75.95	3
660.07	854.00	728.23	125.77	3
660.70	976.00	793.37	182.62	3
661.32	1098.00	852.88	245.13	4
661.93	1220.00	907.82	312.18	3
657.80	416.20	416.20	0.00	Overtopping

Rating Curve Plot for Crossing: Dubois - Existing



Culvert Data: Culvert 1

Table J5 - Culvert Summary Table: Culvert 1

Total Discharge (cfs)	Culvert Discharge (cfs)	Headwater Elevation (ft)	Inlet Control Depth (ft)	Outlet Control Depth (ft)	Flow Type	Normal Depth (ft)	Critical Depth (ft)	Outlet Depth (ft)	Tailwater Depth (ft)	Outlet Velocity (ft/s)	Tailwater Velocity (ft/s)
0.00 cfs	0.00 cfs	656.70	0.00	7.200	0-NF	0.00	0.00	4.00	7.30	0.00	0.00
122.00 cfs	122.00 cfs	656.79	2.02	7.295	4-FFf	1.39	1.46	4.00	7.30	1.94	0.00
244.00 cfs	244.00 cfs	657.08	3.11	7.578	4-FFf	2.04	2.09	4.00	7.30	3.88	0.00
366.00 cfs	366.00 cfs	657.55	4.04	8.051	4-FFf	2.65	2.59	4.00	7.30	5.83	0.00
488.00 cfs	480.10 cfs	658.16	5.01	8.664	4-FFf	3.35	2.97	4.00	7.30	7.64	0.00
610.00 cfs	574.02 cfs	658.79	5.97	9.292	4-FFf	4.00	3.23	4.00	7.30	9.14	0.00
732.00 cfs	656.04 cfs	659.43	6.97	9.933	4-FFf	4.00	3.42	4.00	7.30	10.44	0.00
854.00 cfs	728.23 cfs	660.07	7.97	10.568	4-FFf	4.00	3.56	4.00	7.30	11.59	0.00
976.00 cfs	793.37 cfs	660.70	8.98	11.197	4-FFf	4.00	3.66	4.00	7.30	12.63	0.00
1098.00 cfs	852.88 cfs	661.32	9.98	11.819	4-FFf	4.00	3.73	4.00	7.30	13.57	0.00
1220.00 cfs	907.82 cfs	661.93	10.96	12.433	4-FFf	4.00	3.78	4.00	7.30	14.45	0.00

Culvert Barrel Data

Culvert Barrel Type Straight Culvert

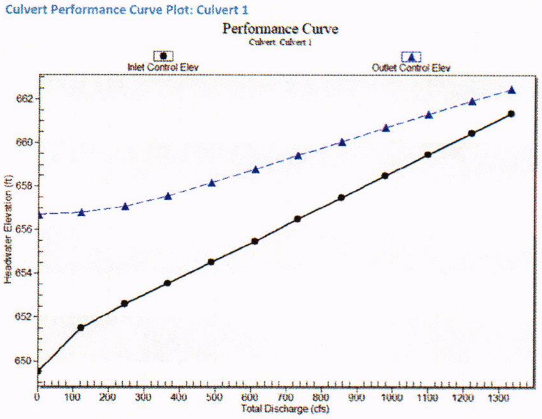
Inlet Elevation (invert): 649.50 ft,

Outlet Elevation (invert): 649.40 ft

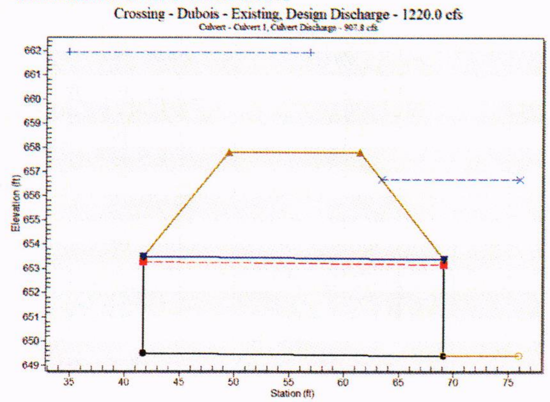
Culvert Length: 27.40 ft,

Culvert Slope: 0.0036

Culvert Performance Curve Plot: Culvert 1



Water Surface Profile Plot for Culvert: Culvert 1



Site Data - Culvert 1

Site Data Option: Culvert Invert Data

Inlet Station: 41.70 ft

Inlet Elevation: 649.50 ft

Outlet Station: 69.10 ft

Outlet Elevation: 649.40 ft

Number of Barrels: 5

Culvert Data Summary - Culvert 1

Barrel Shape: Circular

Barrel Diameter: 4.00 ft

Barrel Material: Concrete

Embedment: 0.00 in

Barrel Manning's n: 0.0120

Culvert Type: Straight

Inlet Configuration: Square Edge with Headwall

Inlet Depression: None

Tailwater Data for Crossing: Dubois - Existing

Table J6 - Downstream Channel Rating Curve (Crossing: Dubois - Existing)

Flow (cfs)	Water Surface Elev (ft)	Depth (ft)
0.00	656.70	7.30
122.00	656.70	7.30
244.00	656.70	7.30
366.00	656.70	7.30
488.00	656.70	7.30
610.00	656.70	7.30
732.00	656.70	7.30
854.00	656.70	7.30
976.00	656.70	7.30
1098.00	656.70	7.30
1220.00	656.70	7.30

Tailwater Channel Data - Dubois - Existing

Tailwater Channel Option: Enter Constant Tailwater Elevation

Constant Tailwater Elevation: 656.70 ft

Roadway Data for Crossing: Dubois - Existing

Roadway Profile Shape: Constant Roadway Elevation

Crest Length: 12.00 ft

Crest Elevation: 657.80 ft

Roadway Surface: Paved

Roadway Top Width: 12.00 ft

Project L – Existing 1 (Coffin/Park Intersection) Coffin Street and Park Avenue Culvert 1

Crossing Discharge Data

Discharge Selection Method: Specify Minimum, Design, and Maximum Flow

Minimum Flow: 0.00 cfs

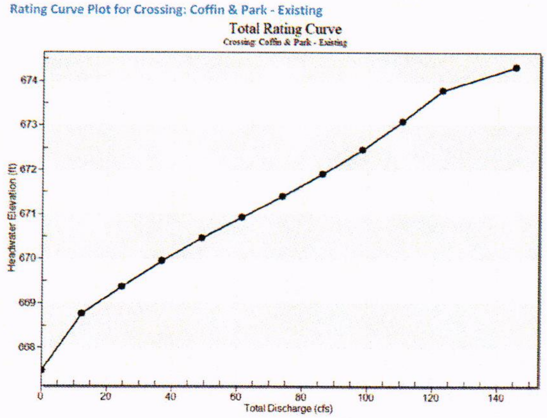
Design Flow: 123.00 cfs

Maximum Flow: 123.00 cfs

Table L1 - Summary of Culvert Flows at Crossing: Coffin & Park - Existing

Headwater Elevation (ft)	Total Discharge (cfs)	Culvert 1 Discharge (cfs)	Roadway Discharge (cfs)	Iterations
667.50	0.00	0.00	0.00	1
668.76	12.30	12.30	0.00	1
669.37	24.60	24.60	0.00	1
669.95	36.90	36.90	0.00	1
670.46	49.20	49.20	0.00	1
670.92	61.50	61.50	0.00	1
671.39	73.80	73.80	0.00	1
671.90	86.10	86.10	0.00	1
672.45	98.40	98.40	0.00	1
673.08	110.70	110.70	0.00	1
673.79	123.00	123.00	0.00	1
674.10	128.03	128.03	0.00	Overtopping

Rating Curve Plot for Crossing: Coffin & Park - Existing



Culvert Data: Culvert 1

Table L2 - Culvert Summary Table: Culvert 1

Total Discharge (cfs)	Culvert Discharge (cfs)	Headwater Elevation (ft)	Inlet Control Depth (ft)	Outlet Control Depth (ft)	Flow Type	Normal Depth (ft)	Critical Depth (ft)	Outlet Depth (ft)	Tailwater Depth (ft)	Outlet Velocity (ft/s)	Tailwater Velocity (ft/s)
0.00 cfs	0.00 cfs	667.50	0.00	0.100	0-NF	0.00	0.00	2.87	5.60	0.00	0.00
12.30 cfs	12.30 cfs	668.76	1.36	0.127	1-JS1t	0.55	1.02	2.87	5.60	1.27	0.00
24.60 cfs	24.60 cfs	669.37	1.97	0.208	1-JS1t	0.77	1.46	2.87	5.60	2.55	0.00
36.90 cfs	36.90 cfs	669.95	2.55	0.343	1-S2n	0.95	1.81	1.06	5.60	13.92	0.00
49.20 cfs	49.20 cfs	670.46	3.06	0.532	1-S2n	1.10	2.10	1.25	5.60	14.68	0.00
61.50 cfs	61.50 cfs	670.92	3.52	0.775	1-S2n	1.23	2.36	1.43	5.60	15.28	0.00
73.80 cfs	73.80 cfs	671.39	3.99	1.072	1-S2n	1.35	2.60	1.59	5.60	15.81	0.00
86.10 cfs	86.10 cfs	671.90	4.50	1.422	5-S2n	1.47	2.81	1.75	5.60	16.27	0.00
98.40 cfs	98.40 cfs	672.45	5.05	2.460	5-S2n	1.58	3.01	1.90	5.60	16.71	0.00
110.70 cfs	110.70 cfs	673.08	5.68	3.006	5-S2n	1.68	3.18	2.05	5.60	17.13	0.00
123.00 cfs	123.00 cfs	673.79	6.39	3.595	5-S2n	1.79	3.33	2.18	5.60	17.54	0.00

Culvert Barrel Data

Culvert Barrel Type Straight Culvert

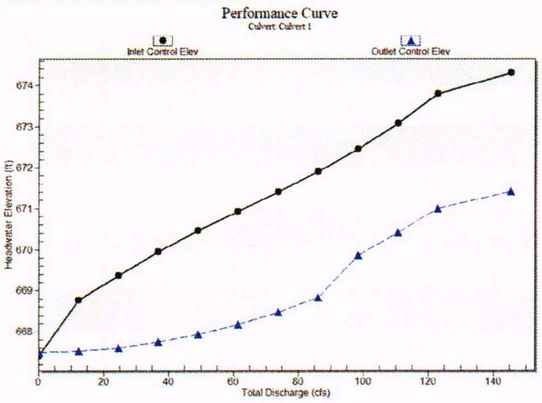
Inlet Elevation (invert): 667.40 ft,

Outlet Elevation (invert): 664.63 ft

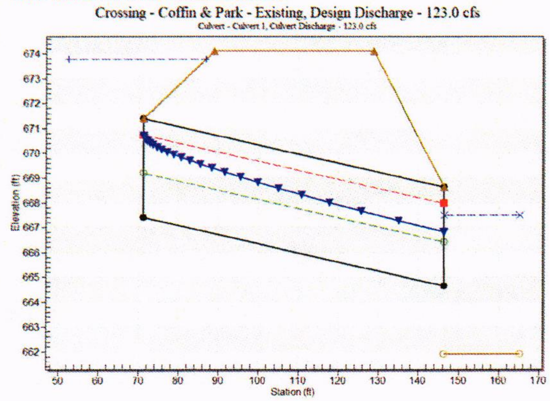
Culvert Length: 75.20 ft,

Culvert Slope: 0.0369

Culvert Performance Curve Plot: Culvert 1



Water Surface Profile Plot for Culvert: Culvert 1



Site Data - Culvert 1

Site Data Option: Culvert Invert Data

Inlet Station: 71.25 ft

Inlet Elevation: 667.40 ft

Outlet Station: 146.40 ft

Outlet Elevation: 664.63 ft

Number of Barrels: 1

Culvert Data Summary - Culvert 1

Barrel Shape: Circular

Barrel Diameter: 4.00 ft

Barrel Material: Concrete

Embedment: 0.00 in

Barrel Manning's n: 0.0120

Culvert Type: Straight

Inlet Configuration: Square Edge with Headwall

Inlet Depression: None

Tailwater Data for Crossing: Coffin & Park - Existing

Table E3 - Downstream Channel Rating Curve (Crossing: Coffin & Park - Existing)

Flow (cfs)	Water Surface Elev (ft)	Depth (ft)
0.00	667.50	5.60
12.30	667.50	5.60
24.60	667.50	5.60
36.90	667.50	5.60
49.20	667.50	5.60
61.50	667.50	5.60
73.80	667.50	5.60
86.10	667.50	5.60
98.40	667.50	5.60
110.70	667.50	5.60
123.00	667.50	5.60

Tailwater Channel Data - Coffin & Park - Existing

Tailwater Channel Option: Enter Constant Tailwater Elevation

Constant Tailwater Elevation: 667.50 ft

Roadway Data for Crossing: Coffin & Park - Existing

Roadway Profile Shape: Constant Roadway Elevation

Crest Length: 50.00 ft

Crest Elevation: 674.10 ft

Roadway Surface: Paved

Roadway Top Width: 40.00 ft

Project L – Existing 2 (Coffin Roadside Culvert) Coffin Street Roadside Culvert 2

Crossing Discharge Data

Discharge Selection Method: Specify Minimum, Design, and Maximum Flow

Minimum Flow: 0.00 cfs

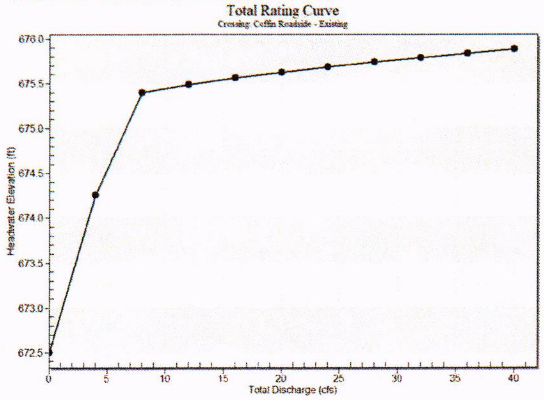
Design Flow: 40.00 cfs

Maximum Flow: 40.00 cfs

Table L4 - Summary of Culvert Flows at Crossing: Coffin Roadside - Existing

Headwater Elevation (ft)	Total Discharge (cfs)	Culvert 1 Discharge (cfs)	Roadway Discharge (cfs)	Iterations
672.50	0.00	0.00	0.00	1
674.26	4.00	4.00	0.00	1
675.40	8.00	5.81	2.17	11
675.49	12.00	5.94	6.04	6
675.56	16.00	6.04	9.95	5
675.63	20.00	6.10	13.89	5
675.68	24.00	6.18	17.81	4
675.74	28.00	6.25	21.75	4
675.79	32.00	6.31	25.68	4
675.84	36.00	6.38	29.62	4
675.88	40.00	6.44	33.56	3
675.30	5.68	5.68	0.00	Overtopping

Rating Curve Plot for Crossing: Coffin Roadside - Existing



Culvert Data: Culvert 1

Table L5 - Culvert Summary Table: Culvert 1

Total Discharge (cfs)	Culvert Discharge (cfs)	Headwater Elevation (ft)	Inlet Control Depth (ft)	Outlet Control Depth (ft)	Flow Type	Normal Depth (ft)	Critical Depth (ft)	Outlet Depth (ft)	Tailwater Depth (ft)	Outlet Velocity (ft/s)	Tailwater Velocity (ft/s)
0.00 cfs	0.00 cfs	672.50	0.00	0.000	0-NF	0.00	0.00	0.05	0.10	0.00	0.00
4.00 cfs	4.00 cfs	674.26	1.69	1.756	7-M2c	1.00	0.85	0.85	0.10	5.64	0.00
8.00 cfs	5.81 cfs	675.40	2.84	2.896	7-M2c	1.00	0.95	0.95	0.10	7.54	0.00
12.00 cfs	5.94 cfs	675.49	2.94	2.988	7-M2c	1.00	0.95	0.95	0.10	7.69	0.00
16.00 cfs	6.04 cfs	675.56	3.01	3.061	7-M2c	1.00	0.95	0.95	0.10	7.82	0.00
20.00 cfs	6.10 cfs	675.63	3.07	3.125	7-M2c	1.00	0.94	0.94	0.10	7.99	0.00
24.00 cfs	6.18 cfs	675.68	3.14	3.182	7-M2c	1.00	0.92	0.92	0.10	8.16	0.00
28.00 cfs	6.25 cfs	675.74	3.19	3.236	7-M2c	1.00	0.91	0.91	0.10	8.30	0.00
32.00 cfs	6.31 cfs	675.79	3.25	3.287	7-M2c	1.00	0.91	0.91	0.10	8.43	0.00
36.00 cfs	6.38 cfs	675.84	3.31	3.335	7-M2c	1.00	0.90	0.90	0.10	8.56	0.00
40.00 cfs	6.44 cfs	675.88	3.36	3.381	7-M2c	1.00	0.88	0.88	0.10	8.80	0.00

Culvert Barrel Data

Culvert Barrel Type Straight Culvert

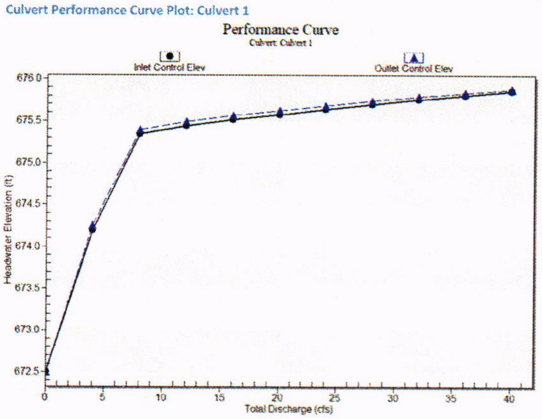
Inlet Elevation (invert): 672.50 ft,

Outlet Elevation (invert): 672.35 ft

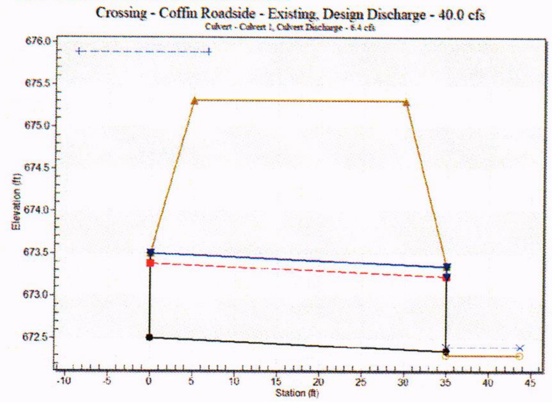
Culvert Length: 35.00 ft,

Culvert Slope: 0.0043

Culvert Performance Curve Plot: Culvert 1



Water Surface Profile Plot for Culvert: Culvert 1



Site Data - Culvert 1

Site Data Option: Culvert Invert Data

Inlet Station: 0.00 ft

Inlet Elevation: 672.50 ft

Outlet Station: 35.00 ft

Outlet Elevation: 672.35 ft

Number of Barrels: 1

Culvert Data Summary - Culvert 1

Barrel Shape: Circular

Barrel Diameter: 1.00 ft

Barrel Material: Concrete

Embedment: 0.00 in

Barrel Manning's n: 0.0120

Culvert Type: Straight

Inlet Configuration: Square Edge with Headwall (Ke=0.5)

Inlet Depression: None

Tailwater Data for Crossing: Coffin Roadside - Existing

Table L6 - Downstream Channel Rating Curve (Crossing: Coffin Roadside - Existing)

Flow (cfs)	Water Surface Elev (ft)	Depth (ft)
0.00	672.40	0.10
4.00	672.40	0.10
8.00	672.40	0.10
12.00	672.40	0.10
16.00	672.40	0.10
20.00	672.40	0.10
24.00	672.40	0.10
28.00	672.40	0.10
32.00	672.40	0.10
36.00	672.40	0.10
40.00	672.40	0.10

Tailwater Channel Data - Coffin Roadside - Existing

Tailwater Channel Option: Enter Constant Tailwater Elevation

Constant Tailwater Elevation: 672.40 ft

Roadway Data for Crossing: Coffin Roadside - Existing

Roadway Profile Shape: Constant Roadway Elevation

Crest Length: 25.00 ft

Crest Elevation: 675.30 ft

Roadway Surface: Paved

Roadway Top Width: 25.00 ft

Project L – Design 2 (Coffin Roadside Culvert) Coffin Street Roadside Culvert 2

Crossing Discharge Data

Discharge Selection Method: Specify Minimum, Design, and Maximum Flow

Minimum Flow: 0.00 cfs

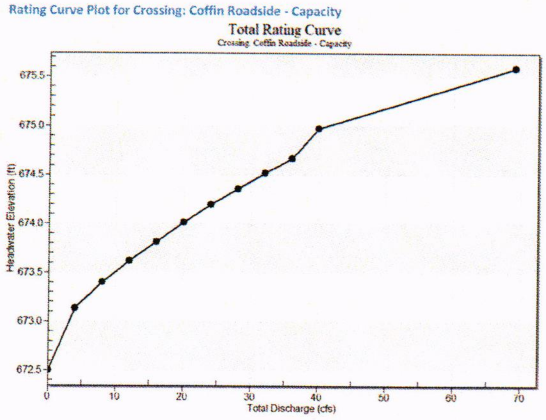
Design Flow: 40.00 cfs

Maximum Flow: 40.00 cfs

Table L7 - Summary of Culvert Flows at Crossing: Coffin Roadside - Capacity

Headwater Elevation (ft)	Total Discharge (cfs)	Culvert 1 Discharge (cfs)	Roadway Discharge (cfs)	Iterations
672.50	0.00	0.00	0.00	1
673.13	4.00	4.00	0.00	1
673.40	8.00	8.00	0.00	1
673.61	12.00	12.00	0.00	1
673.81	16.00	16.00	0.00	1
674.01	20.00	20.00	0.00	1
674.19	24.00	24.00	0.00	1
674.36	28.00	28.00	0.00	1
674.51	32.00	32.00	0.00	1
674.67	36.00	36.00	0.00	1
674.97	40.00	40.00	0.00	1
675.30	49.33	49.33	0.00	Overtopping

Rating Curve Plot for Crossing: Coffin Roadside - Capacity



Culvert Data: Culvert 1

Table L8 - Culvert Summary Table: Culvert 1

Total Discharge (cfs)	Culvert Discharge (cfs)	Headwater Elevation (ft)	Inlet Control Depth (ft)	Outlet Control Depth (ft)	Flow Type	Normal Depth (ft)	Critical Depth (ft)	Outlet Depth (ft)	Tailwater Depth (ft)	Outlet Velocity (ft/s)	Tailwater Velocity (ft/s)
0.00 cfs	0.00 cfs	672.50	0.00	0.000	0-NF	0.00	0.00	0.05	0.10	0.00	0.00
4.00 cfs	4.00 cfs	673.13	0.63	0.316	1-S2n	0.44	0.46	0.44	0.10	3.39	0.00
8.00 cfs	8.00 cfs	673.40	0.90	0.526	1-S2n	0.63	0.66	0.63	0.10	4.15	0.00
12.00 cfs	12.00 cfs	673.61	1.11	0.701	1-S2n	0.77	0.81	0.77	0.10	4.66	0.00
16.00 cfs	16.00 cfs	673.81	1.31	0.864	1-S2n	0.90	0.94	0.90	0.10	5.05	0.00
20.00 cfs	20.00 cfs	674.01	1.51	1.021	1-S2n	1.01	1.06	1.01	0.10	5.37	0.00
24.00 cfs	24.00 cfs	674.19	1.69	1.176	1-S2n	1.12	1.16	1.12	0.10	5.63	0.00
28.00 cfs	28.00 cfs	674.36	1.86	1.333	1-S2n	1.22	1.26	1.22	0.10	5.86	0.00
32.00 cfs	32.00 cfs	674.51	2.01	1.493	1-S2n	1.32	1.35	1.32	0.10	6.08	0.00
36.00 cfs	36.00 cfs	674.67	2.17	1.657	1-S2n	1.42	1.44	1.42	0.10	6.25	0.00
40.00 cfs	40.00 cfs	674.97	2.32	2.475	2-M2c	1.52	1.52	1.52	0.10	6.41	0.00

Culvert Barrel Data

Culvert Barrel Type Straight Culvert

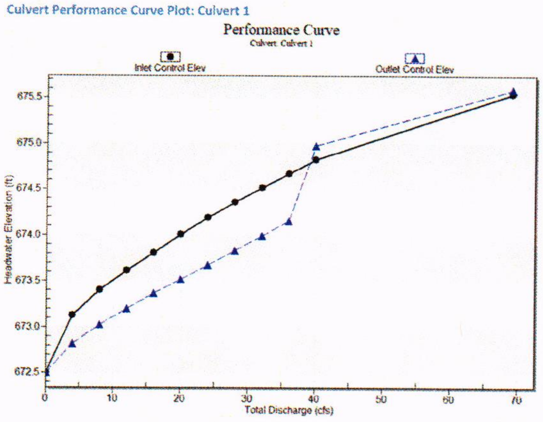
Inlet Elevation (invert): 672.50 ft,

Outlet Elevation (invert): 672.35 ft

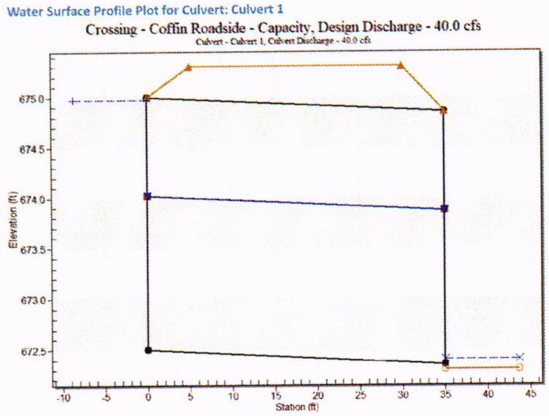
Culvert Length: 35.00 ft,

Culvert Slope: 0.0043

Culvert Performance Curve Plot: Culvert 1



Water Surface Profile Plot for Culvert: Culvert 1



Site Data - Culvert 1

Site Data Option: Culvert Invert Data

Inlet Station: 0.00 ft

Inlet Elevation: 672.50 ft

Outlet Station: 35.00 ft

Outlet Elevation: 672.35 ft

Number of Barrels: 2

Culvert Data Summary - Culvert 1

Barrel Shape: Circular

Barrel Diameter: 2.50 ft

Barrel Material: Concrete

Embedment: 0.00 in

Barrel Manning's n: 0.0120

Culvert Type: Straight

Inlet Configuration: Square Edge with Headwall (Ke=0.5)

Inlet Depression: None

Tailwater Data for Crossing: Coffin Roadside - Capacity

Table L9 - Downstream Channel Rating Curve (Crossing: Coffin Roadside - Capacity)

Flow (cfs)	Water Surface Elev (ft)	Depth (ft)
0.00	672.40	0.10
4.00	672.40	0.10
8.00	672.40	0.10
12.00	672.40	0.10
16.00	672.40	0.10
20.00	672.40	0.10
24.00	672.40	0.10
28.00	672.40	0.10
32.00	672.40	0.10
36.00	672.40	0.10
40.00	672.40	0.10

Tailwater Channel Data - Coffin Roadside - Capacity

Tailwater Channel Option: Enter Constant Tailwater Elevation

Constant Tailwater Elevation: 672.40 ft

Roadway Data for Crossing: Coffin Roadside - Capacity

Roadway Profile Shape: Constant Roadway Elevation

Crest Length: 25.00 ft

Crest Elevation: 675.30 ft

Roadway Surface: Paved

Roadway Top Width: 25.00 ft

Attachment 4: Detention Design

Objective - Reduce Peak Discharge for the 100-year to the capacity of the culvert crossing at Rivercrest near Flora

Project ID	A
Location	Flora & Rivercrest

a	325.63
b	25.322
Contributing Area, A (acre)	178.57

Rational C	0.55
Allowable Release, Qa (cfs)	318

Length of Travel, D (ft)	4500
Slope, S (%)	1.1
Time of Concentration, Tc (min)	64.3
Critical Duration, Td (min)	46.0

Preliminary Required Storage, V Preliminary (ft^3)	185018
Preliminary Required Storage, V Preliminary (acre-ft)	4.25

3-hour Precipitation, P180 (inch)	5.663
Critical Duration Precipitation, Ptd (inch)	3.147

Required Storage, Vreq (acre-ft)	7.64

Precipitation Iron Ore Creek Basin	
Duration	100-year Depth (inch)
5min	1.015
15min	2.009
1h	3.658
2h	4.85
3h	5.663
6h	7.067
12h	8.421
24h	9.806

County	Parameter	Return Interval (Years)						
		1	2	5	10	25	50	100
Grayson	a	100.87	128.89	175.74	208.17	250.17	285.35	325.63
	b	14.086	16.567	20.006	21.751	22.993	24.027	25.322