# City of Grand Prairie
# Drainage Master Plan
# for Johnson Creek Corridor

**City Project No. Y#0948**

Prepared For:

Grand Prairie
TEXAS

By:

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April 2014

**RESOLUTION NO. xxxx-2014**

**A RESOLUTION APPROVING THE CITY OF GRAND PRAIRIE'S CITY•WIDE DRAINAGE MASTER PLAN FOR JOHNSON CREEK.**

**WHEREAS,** The "Drainage Master Plan for Johnson Creek Corridor" (the Plan) is about providing comprehensive, updated technical data for the management of Johnson Creek watershed; and

**WHEREAS,** the Plan addresses existing flooding, erosion, and sedimentation problems within the watershed and provides planning alternatives and design concepts to help alleviate potential flood damages; and

**WHEREAS,** the Plan provides the City of Grand Prairie with the necessary updated drainage information to coordinate future development according to the City's drainage requirements to help minimize existing and potential flood damages within Johnson Creek watershed; and

**WHEREAS,** any revisions to the floodplain and the floodways identified in these studies shall also include fully developed development conditions and shall be for the whole creek as determined in these studies and not for portions of it to ensure that there are no downstream adverse effects; required submittals to FEMA shall be for the whole creek (as determined in these studies) and not for portions of it; and

**WHEREAS,** the recommendations of this report shall be incorporated for all future development as well as CIP budget considerations.

**NOW THEREFORE, BE IT RESOLVED, BY THE CITY COUNCIL OF THE CITY OF GRAND PRAIRIE, TEXAS THAT:**

**SECTION 1.** That the City of Grand Prairie, Texas, having developed the "Drainage Master Plan for Johnson Creek Corridor" to cost-effectively manage flood or storm waters within budgeting constraints, approves and adopts the "Drainage Master Plan for Johnson Creek Corridor" thereby setting the standard for future drainage master plans, addressing existing flooding problems and providing planning recommendation, alternatives and design concepts for future development, to include CIP as well as possible developer participation projects.

**PASSED AND APPROVED BY THE CITY COUNCIL OF THE CITY OF GRAND PRAIRIE, TEXAS ON THIS THE xxx DAY OF APRIL, 2014.**

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## EXECUTIVE SUMMARY

The City of Grand Prairie (City) has tasked Jacobs Engineering Group, Inc. (Jacobs) to prepare a Drainage Master Plan (DMP) for Johnson Creek Corridor. The DMP provides updated technical data for the management of Johnson Creek watershed in the areas along the main stem of the creek. This report is intended to be a living document that can be updated as additional information becomes available. The analysis includes updated hydrologic modeling of the watershed and hydraulic modeling of the channel, observation-based field assessments of numerous drainage outfalls that discharge into Johnson Creek and several in-line hydraulic structures, a stream geomorphologic assessment, developments of alternative engineering measures, and ranking of priorities of the recommended projects.

The information presented in this report will provide the City with the necessary updated information to coordinate future projects according to the City's drainage requirements. This document can also be used to assist in minimizing existing and potential damages caused by flooding or stream instabilities. This analysis does not involve an assessment of the entire watershed including storm drain systems or other outfall drainage features that are being developed by other engineering firms.

Johnson Creek watershed within Grand Prairie is generally located east of State Highway 360 and west of the confluence with the West Fork Trinity River as shown in the vicinity map on **Figure ES-1**. Drainage within the watershed enters the main stem of the creek through networks of storm drains including subsurface pipes, constructed open channels and natural tributaries. The creek generally flows in a northeasterly direction and is composed of natural segments coupled with a channelized segment with banks armored by gabion blocks.

The limit of the study for hydraulic modeling and floodplain mapping is the main stem of Johnson Creek from its confluence with the West Fork Trinity River continuing upstream approximately 3.94 miles of which 3.72 miles are within the City of Grand Prairie. With the exception of backwater effects from West Fork Trinity River, an analysis of flooding caused by individual tributaries is not included in this study. The new hydraulic model is extended a short distance upstream to the downstream face of Lamar Boulevard within the City of Arlington to allow for a seamless tie-in with the floodplain further upstream.

In conjunction with a separate analysis being performed by the City of Arlington, the current hydrologic study involves updating the hydrologic model for Johnson Creek watershed using HEC-HMS (v3.5) program. This modeling estimates refined peak flows to determine base flood elevations (BFEs) more accurately. Approximately 25% of the entire watershed is located within the City of Grand Prairie. The remaining portion of the watershed is located within the City of Arlington. Therefore, coordination with the City of Arlington has been undertaken to ensure that the revised flows determined from this analysis can be applicable throughout the entire watershed. This revised analysis reveals that discharges through the City have increased by 4 to 16% compared to the discharges reported in the FEMA's effective (regulatory) models. The greatest increase in peak flow occurs at the confluence of Johnson Creek with the Arbor Creek. Here, the 1-percent annual chance peak flow increases from 17,310 cfs to 20,560 cfs. However, this increase from the regulatory flows has not caused any enlargement of the floodplain from the regulatory floodplain throughout this reach within the limits of the City.

A detailed hydraulic analysis is also performed by incorporating the revised peak flows into the HEC-RAS (v.4.1) model to determine revised BFEs. Several newly constructed structures, such as State Highway 161 and several pedestrian crossings near the golf course along with each of the existing aerial utility crossings have been added to this model. An increase in water surface elevation is shown at the structures that are incorporated in the new hydraulic model. Floodplain mapping is revised to reflect these revised BFEs, which shows that the top width of the newly mapped floodplain is still contained within the limits of regulatory floodplain. Improvements of the channel in areas such as the Dorchester Levee and gabion-lined channel sections have helped to contain these increases. All structures that are within the newly revised floodplain are identified. However, these structures have not been removed from the floodplain as a part of this drainage master plan development.

Johnson Creek Watershed is approximately 86.9% urbanized within the City of Grand Prairie. Therefore, the watershed is essentially fully developed since less than 3-percent of the watershed is undeveloped whereas the remaining 10-percent is zoned as either floodplain or open space. Rapid urbanization is the main underlying reason for which the Johnson Creek main channel is unstable in certain locations. The areas of these major instabilities are located within the City of Arlington. These upstream instabilities provide the portion of the channel within the City of Grand Prairie with a steady supply of sediments. This has led to deposition being the primary channel process within the City of Grand Prairie. Field investigation and the results of a bed profile analysis further indicate that deposition has occurred in the recent years with bed and bank erosions localized to approximately 10% of the overall channel length within the City limits. In areas where instabilities are occurring, measures for mitigation of the adverse impacts are suggested in this analysis.

This DMP recommends ten separate flood mitigation or stream stability projects to minimize future adverse conditions in the vicinity of the creek as shown in **Figure ES-1**. Of these ten projects, the City is directly responsible for only six projects. The remaining four projects are the responsibility of either the Great Southwest Golf Course or the North Texas Tollway Authority (NTTA). Duncan-Perry Road Bridge is potentially flooded (overtopped) during all storm events with magnitudes greater than the magnitude of a 10% annual chance storm. Therefore, a flood mitigation project has been proposed at this location to minimize road closures and to eliminate flood hazards during severe storm events.

The stream stability projects are intended to minimize the effects either erosion or deposition have on the functionality and conditions of structures along and adjacent to the creek. Three key locations have been identified as susceptible to significant scouring. In these cases, either scouring is either currently observed or is likely to occur at State Highway (SH) 161, the Union Pacific Railroad crossing and Avenue J. To mitigate the existing and future scours under SH 161, a detailed scour analysis and stabilization measures are recommended.

In addition to the scour analysis, storm drain and open channel outfall assessments along the Johnson Creek Corridor have been performed. Condition and an assessment criteria category, based on recommendations provided in the 2010 Drainage Master Plan Road Map, are assigned to thirty-six outfalls. Nine of these outfalls are in poor conditions and two have failed. These are listed in **Table ES-1** and are also identified in **Figure ES-1**.

This DMP does not include an assessment of storm drain systems within the entire watershed. No assessment of the condition of the Dorchester Flood Control Levee that is in the most downstream segment of Johnson Creek before its confluence with the West Fork Trinity River is provided with this DMP. However, the levee recertification study documentation prepared by Halff Associates, dated February 2005, is included in this study.

**Table ES-1**
**Storm Drain Outfall Assessment of Failed and Poor Condition Structures**

| Overall Ranking | Map Photo No. | Photo Date | Type | Condition | Criteria | Comments | Estimated Cost Range |
|---|---|---|---|---|---|---|---|
| 1 | 367 | 1/28/2014 | Unknown | Failure | Structural | Outfall appears to be completely covered with debris; unable to locate in field | $7,500 to $30,000 |
| 2 | 1027 | 1/28/2014 | Storm drain outfall - pipe | Failure | Structural, no headwall | Downstream most RCP joints have failed and the outfall is no longer connected to the storm drain properly; final pipe joint is obstructing flow in the channel | $20,000 to $30,000 |
| 3 | 609 | 6/8/2012 | Storm drain outfall - pipe | Poor | Siltation & aesthetic | Siltation and debris in channel blocking approximately one-third of outfall | $3,000 to $7,500 |
| 4 | 607 | 7/10/2012 | Storm drain outfall - pipe | Poor | Siltation | Heavy siltation blocking approximately one-third of outfall | $5,000 to $7,500 |
| 5 | 893b | 1/28/2014 | Storm drain outfall - pipe | Poor | No headwall, siltation, structural | Located above 893a; concrete cracks above outfall, siltation blocking bottom one-third of outfall | $7,500 to $15,000 |
| 6 | 1042 | 1/28/2014 | Storm drain outfall - pipe | Poor | Scour & aesthetic | Tree limbs in outfall, approximately 2 feet of scour downstream of outfall beginning to undermine concrete foundation | $6,000 to $10,000 |
| 7 | 938 | 7/11/2012 | Storm drain outfall - pipe | Poor | Scour, no headwall | Scour beneath and around outfall | $6,000 to $10,000 |
| 8 | 1041 | 7/9/2012 | Storm drain outfall - pipe | Poor | Structural, scour, no headwall | End joints of RCP outfall separated; slight scour downstream of outfall | $10,000 to $15,000 |
| 9 | 935 | 1/28/2014 | Storm drain outfall - pipe | Poor | No headwall, structural | Apron downstream of outfall is cracking, vegetation and leaf litter in outfall | $10,000 to $15,000 |
| 10 | 902 | 7/3/2012 | Storm drain outfall - pipe | Poor | Siltation | Siltation partially blocking outfall | $4,000 to $6,000 |
| 11 | 864 | 6/11/2012 | Storm Drain Outfall - Box | Poor | Siltation & debris | Siltation blocking approximately bottom one-foot of outfall; debris in channel downstream of outfall | $6,000 to $10,000 |

*Cost estimate ranges are based on typical installations and repairs for each of the criteria. Actual cost may exceed these estimates depending on project conditions.

The recommendations to mitigate flood hazards, erosion hazards, scouring, and structural damages are summarized in **Table ES-2**. Priorities are assigned for the capital improvement projects that are considered to be the responsibilities of the City. The projects that are not the responsibilities of the City are not given a priority number. Additional data and documentation have been provided for these projects to the City to notify the responsible parties for potential mitigation project.

**Table ES-2**
**Proposed Capital Improvement Projects to Stabilize Johnson Creek Corridor**

| Project Number | Project Description | Recommendation | CIP Priority No. | Reach | Cost |
|---|---|---|---|---|---|
| 1 | 500 LF of left bank stabilization between SH 360 and Avenue J | Stable channel design or installation of gabion blocks | 5 | 1 | $ 501,800.00 |
| 2 | Abutment repair and stabilization - Avenue J | Riprap or gabion block to reinforce abutment | 1 | 1 to 2 | $ 156,032.00 |
| 3 | Abutment repair and stabilization - Aerial Crossing #2 | Repair abutment and install reinforcement such as concrete bags or gabion block | NA | 2 | NA |
| 4 | Gabion mattress repair - Inline Structure #2 | Replace gabion mattresses | NA | 2 | NA |
| 5 | 200 LF of left bank stabilization - Immediately downstream of Inline Structure #3 | Stable channel design or installation of gabion blocks | 2 | 3 | $ 228,422.00 |
| 6 | Repair or replacement - Inline Structure #3 located northeast of the intersection of North Great Southwest Parkway and Hidden Brook Drive | Repair structure or replace with imbricated rock | NA | 3 | NA |
| 7 | 800 LF of right bank stabilization - Approx. 600 feet upstream of Duncan Perry Road | Stable channel design or installation of gabion blocks | 4 | 3 | $ 916,089.00 |
| 8 | Detailed scour evaluation and mitigation at SH 161 | Inform NTTA of potential scour issue | NA | 5 | NA |
| 9 | Inline Structure #5 located downstream of SH 161 – Replace structure, bank stabilization and sediment removal | Replace Inline Structure #5, provide dredging and sediment removal. Provide erosion protection on banks downstream approx 1300 ft. | 6 | 5 | $ 1,775,600.00 |
| 10 | Duncan-Perry Road bridge and roadway improvements | Improved bridge width and elevation or a flood warning system. | 3 | 3 | $ 4,959,273.00 |

Projects with a CTP priority are determined to be the City's responsibility. Projects that have NA priorities are the projects for which additional data and information are supplied to the responsible authorities.

Those included in the City as a CTP and their project priority is shown in **Table ES-3** as established using the Road Map ranking process. Project receiving a ranking of 3 or less in Step 1 of the ranking process are considered short term priorities, while projects receiving ranking of 4 or higher are considered to have long term priorities. There are two projects with a ranking of 3, making them small short term priority and four long term priority projects.

Based on the analyses performed in this study the following recommendations are made. Each section in this report provides additional details for these recommendations.

- The City should enforce its floodplain development standards to ensure that new flooding problems do not originate.
- Future developments near the channel should consider the erosion hazard setback procedures outlined in Section IX, which are addressed as discussed in the Drainage Manual for the City of Grand Prairie.
- The City should consider the proposed improvements projects, which have been ranked in Section XIII.
- Consideration should be given to routine inspection to find problems early and assess project priority in the future periodically.
- Maintenance of outfalls, utility crossings, and other areas can help prevent future problems and prolong the life of existing facilities until they can be addressed through the proposed projects.
- If projects cannot be completed in a timely manner, then consideration should be given to phasing the projects to allow higher priority portions to be addressed sooner.
- Storm drain systems should be evaluated in a 2D hydraulic modeling platform such as Infoworks ICM for local pluvial flooding originating from surface water ponding in the watershed.
- Levees should be included in regular maintenance schedules.

The Drainage Master Plan (DMP) for Johnson Creek Corridor provides comprehensive, updated technical data for management of the Johnson Creek watershed. This report addresses flood hazards and erosion problems within the Johnson Creek corridor and provides planning-level mitigation alternatives and design concepts to help alleviate potential damages to local residents and City infrastructure. The information presented in this report will provide the City of Grand Prairie with the necessary updated drainage information to coordinate future development and help minimize existing and potential hazards that can result from flooding and stream instability within the Johnson Creek watershed. This study is in compliance with the requirements set forth in the "City-wide Drainage Master Plan Roadmap." The City Council of Grand Prairie passed Resolution No. ____-____ approving this study on __________, 2014.

**Table ES-3**
**Ranking Process**

| | | | | | Step 1 - Initial Ranking Factor - Estimate of Probable Cost vs. # Structures Benefited [1] | | | Step 2 - Second Ranking Factor - Cost to benefit of Roadway Number of Citizens Impacted [2] | | | | | | | Step 3 - Tax Value of Benefited Property Structures [7] | | Sum of 1st, 2nd, and 3rd Factors | Step 4 - Initial Rank | Step 5 - 100-Year Ultimate Discharge at CIP Location | | Step 6 - Final Rank |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Project Number from Jacobs MDP 2014 | Capital Improvement Project Alternative | Watershed | Project Size & Short Term/Long-Term | # Structures | Cost | 1st Factor [1] | Type | Roadway Flood Event Protection | Roadway % Citizens Protected [3] | Roadway % Citizens Impacted [4] | Roadway # Citizens Impacted [5] | Cost to Benefit Roadway # Citizens Impacted [6] | 2nd Factor | Tax Value of Property Structures Benefited | 3rd Factor | Total | Rank [8] | Ultimate Q100 (cfs) | Sorting [9] | Rank [10] |
| 1 | 2 | Abutment repair and stabilization - Avenue J | Johnson Creek | Small/Short-term | 0 | $156,032 | 3 | P4D | No Protection | 0 | 100 | 8450 | $18.47 | 1 | $0 | 20 | 24 | 1 | 17,912 | 1 | 1 |
| 2 | 5 | Bank stabilization immediately downstream of Inline Structure #3 | Johnson Creek | Small/Short-term | 0 | $228,422 | 3 | - | - | - | - | - | - | 3 | $0 | 20 | 26 | 2 | 18,041 | 2 | 2 |
| 3 | 10 | Duncan Perry bridge and roadway improvements | Johnson Creek | Large/Long-term | 2* | $4,959,273 | 5 | P4D | 5-Year | 85 | 15 | 1170 | $4,238.69 | 2 | $0 | 20 | 27 | 3 | 18,233 | 3 | 3 |
| 4 | 7 | Bank stabilization approx. 600 ft upstream of Duncan Perry Road | Johnson Creek | Medium/Long-term | 0 | $916,089 | 4 | - | - | - | - | - | - | 3 | $0 | 20 | 27 | 3 | 18,041 | 4 | 4 |
| 5 | 1 | Bank stabilization between SH 360 and Avenue J | Johnson Creek | Medium/Long-term | 0 | $501,800 | 4 | - | - | - | - | - | - | 3 | $0 | 20 | 27 | 3 | 17,912 | 5 | 5 |
| 6 | 9 | Replacement of Inline Structure #5 and removal of deposited sediments | Johnson Creek | Large/Long-term | 1 | $1,775,600 | 5 | - | 10-year | - | - | - | - | 3 | $0 | 20 | 28 | 4 | 20,719 | 6 | 6 |

1 - Refer to City-Wide Drainage Master Plan Road Map, Section II.G - Implementation Plan - Step 1
2 - Refer to City-Wide Drainage Master Plan Road Map, Section II.G - Implementation Plan - Step 2
3 - Based on approximation, using logarithmic chart, with 1-year event coverage protecting 0% of traffic volume and 100-year event coverage protecting 100% of traffic volume
4 - Percent Impacted = 100% minus % of Roadway Citizens protected (approximate)
5 - Number Impacted = % Impacted multiplied by [No. Lanes * 4 hours Impacted * Hourly Volume Per Lane * Level of Service "C" Traffic Volume]
6 - Cost of CIP Divided by Roadway # Citizens Impacted
7 - Refer to City-Wide Drainage Master Plan Road Map, Section II.G - Implementation Plan - Step 3
8 - Refer to City-Wide Drainage Master Plan Road Map, Section II.G - Implementation Plan - Step 4
9 - Refer to City-Wide Drainage Master Plan Road Map, Section II.G - Implementation Plan - Step 5
10 - Refer to City-Wide Drainage Master Plan Road Map, Section II.G - Implementation Plan - Step 6

   a. Improvements by TxDOT associated with the construction of SH-161 (TxDOT CSJ # 1068-02-093 Plans are collected) that span Johnson Creek,
   b. Six aerial waterline crossings and three pedestrian bridges in the golf course area,
   c. Updated LiDAR derived topographic data and ground-based survey data throughout the remainder of Johnson Creek within the City limits.
5. Preparation of updated Flood Insurance Rate Maps (FIRM) and supporting documentation for Johnson Creek.
6. Preparation of stream geomorphologic study of Johnson Creek, detailing classification of stream evolution types, estimation of stream equilibrium conditions including stream and bank slopes and channel widths, bridge scour and analysis of unstable and problematic areas.
7. Preparation of flooding, erosion, and stream stability alternatives evaluation including conceptual opinions of probable construction cost.
8. Evaluation and prioritization of potential problem solution alternatives according to the City's Road Map.

## D. City Ordinances and Development Requirements

Johnson Creek watershed is considered highly developed at this time and proper drainage requirements and responsible development of the watershed will help prevent future flood damage and unnecessary capital improvement costs.

The City of Grand Prairie is especially progressive in their storm water management program. The City's Drainage Design Manual is updated regularly, the most current release dated January 2013 is used and is intended to "…protect the general health, safety, and welfare of the public by reducing flooding potential, controlling excessive runoff, minimizing erosion and siltation problems, and eliminating damage to public facilities resulting from uncontrolled storm water runoff."

Articles 14 and 15 of the Unified Development Code, included in the City's Drainage Design Manual, contain the City ordinances for Drainage and Floodplain Management, respectively. Requirements of Article 15 include the finished elevation of new construction have a minimum of one foot above the fully developed 100-year base flood elevation floodplain or two feet above the existing conditions floodplain, whichever is higher. Evaluation of the downstream ‘zone of influence’ is required when downstream facilities are not adequately sized to convey a design storm based on current criteria for hydraulic capacity. Post project peak flows are not allowed to exceed the existing conditions peak flows unless sufficient downstream capacity above existing discharge conditions is available. When required, detention facilities are to be designed such that peak discharges or velocities are not increased when compared to pre-project conditions for the 50%, 20%, and 1% annual chance (2-, 10- and 100-year) storm events.

The City ordinances allow for responsible development of the watershed such that flood risks to future structures can be minimized though evaluation of the downstream impacts. The ordinances also allow for protection of existing structures so that future development will not increase the flooding hazard in areas that do not have the capacity to convey increased flood discharges. Upon review of the City's Drainage Design Manual and existing development requirements, it has been determined that the

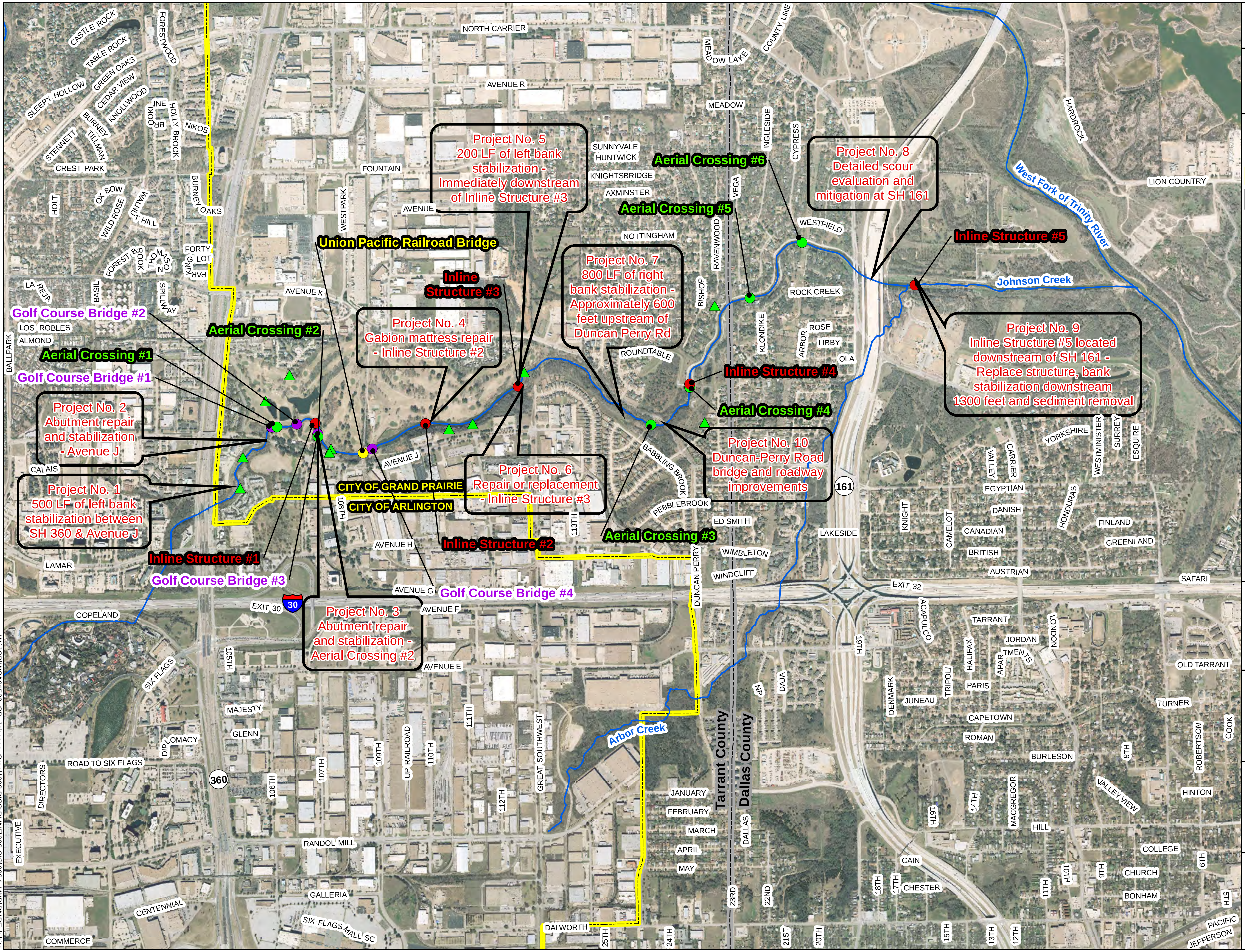



**Figure ES-1**

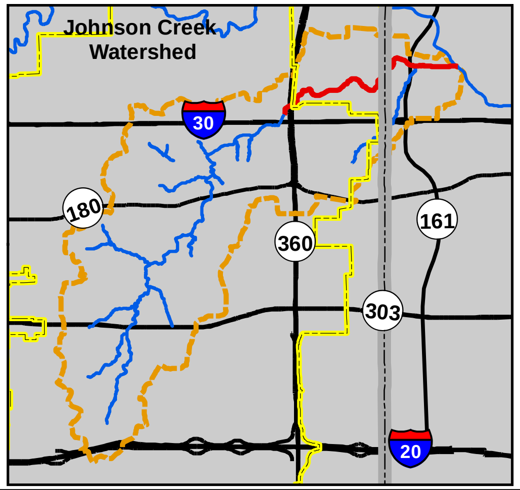
**Johnson Creek Corridor Capital Improvement Projects**

## Legend

- Stream Centerline
- City Limits
- County Boundary

**Structures**

- Aerial Crossing
- Inline Structure
- Pedestrian Bridge
- Railroad Bridge
- Outfall Location (Failed/Poor Cond)




0 1,500 3,000

Scale in Feet

JACOBS

Grand Prairie TEXAS

DMP for Johnson Creek Corridor

W:\OBWSA01500_GP_Johnson_Creek\600 DISCIPLINE\606 GIS\606.1 MXD\DMPExhibits

# I. INTRODUCTION

## A. Acknowledgements

Jacobs Engineering, Inc. (Jacobs) has developed this Drainage Master Plan for Johnson Creek Corridor with the understanding and experience the current staff has of the conditions in the watershed. Jacobs would like to acknowledge the significant contributions of all City of Grand Prairie staff in preparation of the Drainage Master Plan for Johnson Creek Corridor. In particular, the following individuals have provided invaluable input and assistance:

Romin Khavari – City Engineer
Chris Agnew – Stormwater Engineer
Mazen Kawasmi – Stormwater Utility Manager and Floodplain Administrator

## B. Authorization

The City of Grand Prairie authorized the Drainage Master Plan for the Johnson Creek Corridor and FEMA CTP Mapping Project (Y#0948) and contracted with Jacobs Engineering, Inc. for this work on January 25, 2013.

## C. Purpose of Study

This study is in compliance with the requirements set forth in the "City-Wide Drainage Master Plan Road Map." The City of Grand Prairie is a FEMA Cooperating Technical Partner (CTP). The City is committed to maintaining current flood maps as a CTP, and among other objectives, to developing comprehensive studies and a prioritized approach to flood and stormwater management, set forth in the Drainage Master Plan Road Map (Road Map). The Road Map is a well-organized approach to documentation of the modeling required for the CTP mapping process. It provides a basis for analysis of floods of various frequencies and hydraulic conditions with supportive documentation for evaluation of alternatives to specific flooding and drainage problems.

Specific objectives of the City that are addressed in this Plan include:

1. Development of a comprehensive hydrologic model for Johnson Creek Watershed. This includes coordination with the City of Arlington to prepare the hydrologic model of the entire watershed so that the City of Arlington's watershed planning is consistent with the City of Grand Prairie's plan.
2. Development of the hydrologic model is based on the US Army Corps of Engineers (USACE) hydrologic modeling program using HEC-HMS (v3.5).
3. Frequency-based design storms for the 50%, 20%, 10%, 4%, 2%, 1%, and 0.2% annual chance storms (2, 5, 10, 25, 50, 100 and 500-year storm events) for the existing conditions of watershed development are used to develop the hydrologic models.
4. Development of a hydraulic model of the Johnson Creek channel and mapping of its floodplain. The hydraulic model is based on the USACE modeling program, HEC-RAS and incorporates recent developments in the Johnson Creek system. These include

requirements in combination with the technical data provided in this report are adequate to properly manage the watershed.

## E. Watershed Description

Johnson Creek watershed is located both in Dallas and Tarrant Counties. Approximately 25% of the watershed is within the City limits of Grand Prairie, while approximately 75% of Johnson Creek watershed upstream is within City of Arlington. Arbor Creek (FEMA Stream JC-1) is the only FEMA identified tributary within City of Grand Prairie.

Johnson Creek watershed is approximately 86.9% urbanized. The main stem of Johnson Creek is crossed by roadways and pedestrian crossings at 16 locations with 6 locations at which an aerial pipe crossings. At present, industrial and commercial land uses comprise approximately 10.9% of the watershed, while transportation infrastructures make up another 57.2%. Residential development accounts for 19.2% and 13.1% of the existing watershed is undeveloped that includes parks or open space use. The watershed within Grand Prairie is essentially considered fully developed; it is estimated after review of the City's future land use database, that only 3% of the area within Grand Prairie is likely to have any future development for industrial or residential use. The entire watershed is highly urbanized under existing conditions. The upper portions of the watershed, within the City of Arlington are comprised of mostly different types of residential areas and city parks. The middle portions of the watershed are the City of Arlington's most recently developed areas encompassing the sprawling campus of the University of Texas at Arlington, the Cowboy Stadium and its expansive parking lots, Rangers Stadium, Six Flags over Texas, Downtown Arlington, and several major roadways and thoroughfares. Interstate 30 has also recently been redesigned and expanded which has the City of Arlington's area approaching full build-out. Apart from roadway crossings, much of the reach within Grand Prairie, of Johnson Creek stream channel has been modified by channelization, bank stabilization, erosion control structures, and other structural measures related to urbanization, flood control, and instabilities.

The USACE provided a channel improvement design from downstream of Duncan Perry Road to Carrier Parkway in August 1993. This project includes a drop structure at the upstream project limits. In addition, the channel is widened with gabion baskets lining the banks. A berm located on the left overbank adjacent to the drop structure protects homes from inundation by the 100-year floodplain. Repairs were made to this reach within the last two years to ensure its functionality in the near future. The Dorchester Flood Control Levee was designed to protect homes and property upstream of the confluence of Johnson Creek and West Fork of the Trinity River. This structure operates as an integrated flood control system including a levee embankment, drainage channels, underground storm drainage systems and a stormwater pump station. Halff Associates prepared an Operation and Maintenance Manual dated November 1993 which was updated February 2005 for the Levee recertification through FEMA.

## F. Principal Flooding Problems

1. Drainage Complaint Database

The drainage complaint database is reviewed. One hundred and sixty six (166) drainage complaints at one hundred and six (106) different locations have been filed with the City from 1971 to 2011 within Johnson Creek watershed. Of these complaints, fifteen (15) are erosion problems, sixteen (16) are street flooding problems, fifty-nine (59) are property flooding problems, and seventeen (17) are structure flooding problems. Three (3) complaints have been filed since April 2013 at Nottingham Place, Klondike St and Hyatt Apartments.

2. Hot Spot Locations

It is noted from the City-wide Drainage Master Plan Road Map that the following hot spot locations are previously identified and require a storm drain assessment, which is not included in this plan:

- Sunnyvale Road
- Nottingham and Duncan Perry Road
- Axminster and King Richard Drive
- Ivanhoe Circle Area

These storm drain assessments are being studied as part of a separate study by others.

3. Roadway Overtopping

Hydraulic model indicates that Duncan Perry Road overtops during any storm event greater than the 10-percent annual chance.

### G. Channel Stability Assessment

Urbanization impacts both hydrologic response and behavior of a watershed and hydraulic character and behavior of the channels within a watershed. Therefore, it changes the character and rate of response of a channel and floodplain to runoff. Even though pristine streams within pristine watersheds undergo constant change, key characteristics, such as channel slope, bank slope, and channel bottom width tend to attain a condition of dynamic equilibrium. When significant changes occur in a watershed these defining conditions of equilibrium change, often dramatically and therefore adversely affect channel stability which ultimately leads to undesirable situation of either excessive erosion or deposition or both.

Johnson Creek watershed is highly urbanized and has been like this for almost 40 years. This has allowed the Johnson Creek to reestablish a quasi-equilibrium state where instabilities are only occurring at localized levels. There are no regional instabilities evolving.

### H. Pertinent Study and Technical Data Related to Watershed Prior to Johnson Creek DMP

1. 2008 US Army Corps of Engineers Johnson Creek Watershed Study prepared by HDR Engineering, Inc. (HDR)

   HDR prepared a watershed study involving geomorphologic analysis for planning for Johnson Creek Corridor improvements. Stream flows, present conditions and predicted changes to stream conditions particularly planform geometries are analyzed using updated hydrologic and hydraulic data, and geomorphologic modeling.

2. 2011 O'Brien Engineering, Inc. (OEI) CTP TSDN for Arbor Creek

   O'Brien Engineering, Inc. (OEI) developed updated hydrologic flows and hydraulic data for existing conditions as part of a detailed study of Arbor Creek within the City of Grand Prairie, Tarrant and Dallas County, TX. Arbor Creek (FEMA named Stream JC-1) is an existing Zone AE stream, and is a major tributary of Johnson Creek.

3. 1996 Hydrologic study by the US Army Corps of Engineers (USACE)

   The USACE conducted a watershed analysis by redefining drainage areas within the watershed. The Snyder Unit Hydrograph (SUH) method for rainfall to runoff transformation and more detailed channel routing method has been incorporated in this study.

4. 2005 Halff Associates, Inc. Map Modernization Study

   Halff Associates, Inc. (Halff) completed a hydraulic study in 2005 which is provided to FEMA as part of the Map Modernization Program for the production of the Digital Flood Insurance Rate Maps (DFIRM) and revised Flood Insurance Study (FIS) for Dallas and Tarrant Counties. Cross sections of Johnson Creek for this model are developed using the City of Grand Prairie's 1999 LiDAR derived topography and field survey data obtained by Halff. Bridge and culvert data are obtained from Halff's survey, existing models, and record drawings. The Halff model is included in the FIS and DFIRM panels that became effective September 29, 2009.

# II. HYDROLOGIC STUDY

## A. General

A revised hydrologic analysis of Johnson Creek watershed is conducted for this study. The results of the analysis, along with all pertinent model parameters, are compared with those in the prior studies. These studies include the US Army Corps of Engineers (USACE) Study on hydrology of Johnson Creek (1996), the effective FEMA Flood Insurance Study (2005), and the USACE geomorphologic analysis prepared by HDR (2008) discharges and the regression equations.

The 2008 USACE used the program HEC-HMS (v. 3.1) to perform the hydrologic analysis. Halff's study completed in October 2005 which is used for FEMA Map Modifications did not include updates for hydrology of Johnson Creek Watershed. Johnson Creek watershed models developed for the present study include both existing and fully developed land use conditions, using the SCS unit hydrograph method and version 3.5 of HEC-HMS. From this DMP, the existing and fully developed conditions are equivalent as discussed in this section.

Several synthetic rainfall events are evaluated for this study including storms with return periods for the 50%, 20%, 10%, 4%, 2%, 1% and 0.2% annual exceedance probability (AEP) (2, 5, 10, 25, 50, 100 and 500-year return periods) for the fully developed land use conditions. Detailed watershed delineations, existing and fully developed land use characteristics, and distribution of the hydrologic soil type are used to develop the hydrologic models for the watershed. The City of Grand Prairie’s current Drainage Design Manual (January 2013) along with Urban Hydrology for Small Watersheds, Technical Release 55 (TR-55) Second Edition and the North Central Texas Council of Government (NCTCOG) integrated Storm Water Management (iSWM) Manual are used as guidelines for the revised hydrologic analyses.

## B. Watershed

Johnson Creek watershed within the city limits of Grand Prairie is generally located to the east of State Highway 360 and west of the West Fork Trinity River. Drainage within the watershed generally travels from west to east and north to south through storm drain pipes and culverts as well as by man-made and natural open channels. The City of Grand Prairie is only required to study approximately 3.94 miles of Johnson Creek, which stretches from its confluence with West Fork Trinity River to the corporate limits upstream, which is just downstream of State Highway 360. Only 3.72 miles of Johnson Creek are within the City limits. However, to obtain a united hydrologic model of Johnson Creek watershed, the entire watershed with a drainage area of 20.83 square mile has been studied and modeled. The City of Arlington has agreed to develop updated hydrologic data and model for the portion of the watershed that is within the limits of the City of Arlington. The catchments within the City of Arlington are located from Interstate 20 to just upstream of SH 360 at the city limits. Johnson Creek generally flows from south to north with the upper portions of the watershed lying within the City of Arlington and veers to the east as it approaches the Cowboy Stadium area and Interstate 30. Thus, the lower portions of the watershed lie within the City of Grand Prairie. The total contributing drainage area of Johnson Creek Watershed within the City Limits of Grand Prairie is 5.79 sq. miles. Within the City of Arlington the contributing drainage area of the watershed is 15.04 sq. miles. Therefore, the total watershed area of Johnson Creek is 20.83 sq miles.

Johnson Creek watershed boundary is delineated using a combination of one-foot interval contours procured by the City of Grand Prairie and two-foot interval contours provided by the City of Arlington (procured by NCTCOG). ArcGIS 10.1 has been used in the processing of watershed catchment delineations. The two topographic datasets are merged together in ESRI's shapefile format and used subsequently to create a digital elevation model using triangulated irregular network (TIN) for the entire watershed.

Catchments within this watershed are defined according to the order of the open channels and storm drains draining an area to capture the finer details of the hydrologic characteristics of these drainage units. Each catchment outlet is at the hydrologic junction of a second or third order stream or a storm drain trunk line with the stream or storm drain of the next higher order. **Figure II-1** is the Watershed Catchment Map that displays the catchments developed for Johnson Creek Watershed. Johnson Creek watershed is divided into 64 catchments with its outfall at the confluence with the West Fork Trinity River. Of those catchments, 12 are within the City of Grand Prairie limits and 52 are within the City of Arlington limits. These catchments range from 0.04 to 0.82 sq. miles. No modifications are required to the delineations within Arbor Creek Sub-watershed. The catchment boundaries within Arbor Creek Sub-watershed remain the same as submitted in the 2011 CTP submittal for this drainage unit.

### C. Land Use

Land use within Johnson Creek watershed has been evaluated for both existing and fully developed watershed conditions. These conditions are determined to be within FEMA's significance of confidence limits as described in Appendix C on Guidance for Riverine Flooding Analysis and Mapping.

The 2011 City of Arlington and Grand Prairie aerial photography, current City of Arlington and City of Grand Prairie's Landuse GIS data, both the future zoning and comprehensive plans are used to determine the existing land use. Existing land use in Johnson Creek watershed mostly consists of industrial, commercial and residential developments. The entire watershed is highly urbanized under existing conditions. The upper portions of the watershed, within the City of Arlington are comprised of mostly different types of residential areas and city parks. The middle portions of the watershed are the City of Arlington's most recently developed areas encompassing the sprawling campus of the University of Texas at Arlington, the Cowboy Stadium and its expansive parking lots, Rangers Stadium, Six Flags over Texas, Downtown Arlington, and several major roadways and thoroughfares. Interstate 30 has also recently been redesigned and expanded. The lower portions of the watershed are entirely within the City of Grand Prairie. The land uses within the City of Grand Prairie range from medium to high density residential lots, light and high industrial areas, commercial lots, and various open spaces. The land uses that are previously established for Arbor Creek, a tributary of Johnson Creek are reviewed, but not revised from those in the October 2011 TSDN for Arbor Creek. The existing conditions land use map is shown in **Figure II-2**. **Table II-1** summarizes the existing land uses present in Johnson Creek watershed. Approximately 86.9% of the watershed is developed under existing conditions.

Fully developed watershed conditions are analyzed using the existing land use map as a platform and modifying the undeveloped areas according to the Cities' of Grand Prairie and Arlington future land use and zoning maps (comprehensive plan). Open spaces designated as parks or floodplain are not modified and are kept as undeveloped land use. The fully developed (ultimate) conditions land use map is shown in **Figure II-3**. **Table II-1** summarizes the fully developed land uses found in Johnson Creek watershed. Under fully developed conditions the undeveloped land available or zoned for future development accounts for less than 3% of the watershed within Grand Prairie. With minimal development possible it is determined to be within FEMA's significance of confidence limits as described in Appendix C on Guidance for Riverine Flooding Analysis and Mapping, therefore a hydrologic restudy would not be required.

**Table II-1**
**Land Use Percentage**

| Land Use Type | Existing Area (ac) | Fully Developed Area (ac) |
|---|---|---|
| Commercial | 2,844 | 2,867 |
| High Density Residential (MF) | 1,084 | 1,147 |
| Light Industrial | 2,359 | 2,461 |
| Low Density Residential (1/4 lots) | 731 | 731 |
| Medium Density Residential (1/8 lots) | 2,576 | 2,588 |
| Mixed Use | 19 | 61 |
| Open Space/Drainage | 1,399 | 1,073 |
| Parks And Recreation | 343 | 297 |
| Pavement/Street w/ROW | 1,978 | 2,110 |
| **Total Acres** | 13,335 | 13,335 |
| **Percent** | **86.9%** | **89.7%** |

### D. Impervious Coverage

The SCS curve numbers typically account for the inherent imperviousness of a particular land use. The percent impervious for each catchment is calculated to verify curve number calculations. The impervious cover percentage is determined on the basis of land use classification. Impervious cover percentages for each of the land use types are adopted from the City's Drainage Manual (Jan. 2013), those not available are assigned through the City of Arlington's impervious data or engineering judgment. The urbanization in Johnson Creek watershed is calculated by subdividing the watershed into smaller catchments that are representative of the predominant land use they consist of. As appropriate, the interconnected impervious percentage of each catchment is assessed and factored into the calculation of the weighted CNs. **Table II-2** lists the percent imperviousness values used for typical land uses found in Johnson Creek watershed.

**Table II-2**
**Land Use Percent Impervious**

| Category | LU Code | Percent Impervious |
|---|---|---|
| Agricultural | A | 10 |
| Business | B | 85 |
| Community Service | CS | 85 |
| Festival (Commercial) | F | 85 |
| Industrial Manufacturing | IM | 72 |
| Light Industrial | LI | 72 |
| Medium Density Multi-Family | MF18-22 | 65 |
| Neighborhood Service | NS | 85 |
| Office Service | O | 85 |
| Planned Development | PD | 85 |
| Townhouse (MF) | TH | 65 |
| Commercial/Retail/Office | C/R | 85 |
| High Density Residential (MF) | HDR | 65 |
| Low Density Residential (1/4lots) | LDR | 38 |
| Medium Density Residential (1/8 lots) | MDR | 65 |
| Mixed Use | MU | 85 |
| Open Space/Drainage | OS/Drainage | 0 |
| Parks and Recreation | Park/Rec | 6 |
| Streets & Roads | Street | 90 |
| Residential (1/8 lots) | R | 65 |
| Duplex (MF) | D | 65 |
| Downtown Business | DB | 85 |
| Residential (7500sf) | R1 | 55 |
| Residential (less than 5K sq ft) | R2 | 61 |
| University | UTA | 85 |

## E. Soil Types

Soil types for the watershed are obtained from the Natural Resource Conservation Service (NRCS) Soil Survey Geographic (SSURGO) database for Tarrant and Dallas Counties. This dataset is downloaded from the NRCS website from the most recent data base. **Figure II-4** is the Soils Map that shows the distributions of the various hydrologic soil groups (HSG) within Johnson Creek watershed. About 63.7% of the watershed is covered with soils classified as Type D HSG, which represents essentially clayey soils with low infiltration rates. Approximately 26.4% of the watershed contains soils classified as Type B HSG, which represents soils with some content of gravel sand with moderate infiltration rates. About 7.4% of the watershed contains soils classified as Type C HSG, which indicates moderately fine soils with slow infiltration rates. The remaining 2.5% is Type A HSG, with the highest infiltration rate.

### F. Rainfall Losses

Infiltration losses of rainfall primarily depend on soil characteristics and land use. The Curve Number (CN) method developed by the Soil Conservation Service (now called the NRCS) is selected to evaluate the rainfall losses in this study. This uses a combination of soil conditions as described in Section E and land use to assign runoff curve numbers that represent the runoff potential of a catchment.

The curve number method accounts for incremental rainfall losses for each time step, based on a coefficient that is calculated as a weighted average of the totality of varying land uses, soil types, and impervious areas within each catchment. Table 4.1A of the Drainage Design Manual for the City of Grand Prairie (Jan. 2013) provides a list of curve numbers for the various land uses in the City; a CN from this table is assigned to each land use within each catchment. The average CN is then calculated by weighting each land use curve number proportional to the area that they represent relative to the whole. These composite curve numbers are computed for each catchment using tools available in ArcGIS. **Table II-3** includes a summary of hydrologic parameters; such as drainage area size, curve numbers and lag times. Backup data are available in **Appendix A**, **Table II-3** detailing the distribution of the weighted curve number calculated on the basis of soil and land use types.

Curve number is also determined by the moisture condition of the soil immediately prior to the storm, a factor referred to as the antecedent moisture condition (AMC). Three conditions are recognized: dry (AMC-I), moderately moist (AMC-II), and saturated (AMC-III). A condition of AMC-II is used for this study.

Initial abstraction (Ia) is also a component of the curve number loss rate method. Initial abstraction is assumed to be 20 percent of the potential retention. The initial abstraction defines the amount of precipitation that must fall before surface generates runoff. It is derived using the curve number for each catchment. However, if not assigned an initial abstraction HEC-HMS uses 0.2 value. For this study all catchments are assigned an initial abstraction based on the calculated weighted curve number using the following equation (Equation II-1) from TR-55:

$$Ia = 0.2\left(\frac{1000}{CN} - 10\right) \qquad \textbf{(II-1)}$$

### G. Synthetic Unit Hydrograph

Excess rainfall (total rainfall minus losses) is transformed to direct catchment runoff using the unit hydrograph theory. The NRCS unit hydrograph option in HEC-HMS is used in this analysis to generate runoff hydrographs for each defined catchment within the watershed. The unit hydrograph method represents a hydrograph for one unit (one inch) of direct runoff, which is standard engineering practice. This method uses catchment area, curve number and lag time ($T_{LAG}$) to produce a unit hydrograph. In HEC-HMS $T_{LAG}$ is equal to the time (hours) between the center of mass of excess rainfall and the peak of the unit hydrograph (NRCS 1985). In other words, there is a delay in time after a rain event begins before the runoff reaches it maximum peak. This delay is known as lag. The lag is determined based on the time of concentration ($T_c$).

Time of concentration is calculated as the total time taken for runoff to flow from the most distant point of a catchment to its outlet. The methodology of calculating the time of concentration described in TR-55 and iSWM is used in this study. Depending on the catchment, the overland flow paths have three to four components, mainly sheet flow, shallow concentrated flow, pipe flow and/or open channel flow. Sheet flow lengths in the upper portions of a catchment are limited to 100 feet. Sheet flow after 50 to 100 feet typically becomes shallow concentrated flow and travel time is calculated using flow velocity and the distance traveled. The TR-55 method uses slope and the surface types over which the flow occurs to compute flow velocity ($v=a\sqrt{s}$). Storm drain pipe characteristics are obtained from the GIS-based datasets obtained from the cities of Arlington and Grand Prairie. Conduit travel times are calculated using Manning's equation assuming full flow conditions. This assumption is valid because travel time through storm drains in only a small portion of the overall computed time of concentration. Open channel flow travel times are also calculated using Manning's equation assuming bankfull flow conditions. The total time of concentration is an algebraic sum of each travel time component. The hydrologic parameters associated with Arbor Creek are reviewed and maintained as prepared and approved in the CTP TSDN for Arbor Creek 2011 submittal. Backup data on each of the flow length types, associated flow parameters, and calculated lag times are given in **Appendix A**, **Table II-3**. **Figure II-5**, Time of Concentration Map, outlines the flow lengths determined in this study.

Lag times, as required for HEC-HMS input, are taken as 60% of the calculated times of concentration [$T_L$=.06$T_c$], which is the relationship NRCS derived in the National Engineering Handbook Section 4, Hydrology. **Table II-3** summarizes the lag-times used for the various catchments in Johnson Creek watershed.

**Table II-3**
**Summary of Hydrologic Parameters**

| Catchment Name | Area (Sq Mi) | Curve Number | Lag-Time (min) |
|---|---|---|---|
| A1 | 0.746 | 93.6 | 15.1 |
| A2 | 0.388 | 87.9 | 16.7 |
| A3 | 0.178 | 89.5 | 16.5 |
| A4 | 0.212 | 89.6 | 20.9 |
| A5 | 0.253 | 88.5 | 25.2 |
| GP-NE-4A | 0.283 | 90.3 | 11.72 |
| GP-NE-4B | 0.740 | 86.8 | 15.70 |
| GP-NE-4C | 0.281 | 83.8 | 17.72 |
| GP-NE-6 | 0.800 | 76.8 | 14.83 |
| GP-NW-4 | 0.655 | 89.3 | 16.92 |
| GP-NW-5A | 0.697 | 88.3 | 20.13 |
| GP-NW-5B | 0.555 | 83.2 | 19.67 |
| NE-1A | 0.429 | 94.0 | 16.20 |
| NE-1B | 0.053 | 88.6 | 10.16 |
| NE-2A | 0.233 | 92.2 | 17.16 |
| NE-2B | 0.185 | 92.8 | 10.43 |

**Table II-3**
**Summary of Hydrologic Parameters**
**(continued)**

| Catchment Name | Area (Sq Mi) | Curve Number | Lag-Time (min) |
|---|---|---|---|
| NE-2C | 0.444 | 93.1 | 12.02 |
| NE-3 | 0.822 | 93.8 | 28.76 |
| NW-1A | 0.339 | 89.3 | 17.03 |
| NW-1B | 0.132 | 92.0 | 10.05 |
| NW-1C | 0.142 | 91.8 | 10.26 |
| NW-1D | 0.110 | 84.2 | 13.65 |
| NW-1-T-A | 0.194 | 89.8 | 14.20 |
| NW-1-T-B | 0.365 | 91.3 | 12.81 |
| NW-1-T-C | 0.143 | 90.4 | 9.45 |
| NW-2-T-A | 0.278 | 92.0 | 12.93 |
| NW-2-T-B | 0.240 | 92.5 | 16.06 |
| NW-2-T-C | 0.072 | 94.4 | 9.69 |
| NW-2-T-D | 0.204 | 94.1 | 13.56 |
| NW-3A | 0.245 | 93.4 | 19.64 |
| NW-3B | 0.302 | 95.5 | 11.58 |
| NW-4 | 0.595 | 95.8 | 17.00 |
| S-1 | 0.431 | 95.3 | 15.30 |
| SE-1 | 0.268 | 94.3 | 12.07 |
| SE-2 | 0.099 | 94.6 | 10.32 |
| SE-3A | 0.329 | 88.4 | 11.43 |
| SE-3B | 0.339 | 92.9 | 11.40 |
| SE-4A | 0.462 | 93.3 | 11.98 |
| SE-4B | 0.238 | 93.4 | 12.41 |
| SE-4C | 0.224 | 91.2 | 17.29 |
| SE-5 | 0.599 | 88.9 | 24.61 |
| SE-6A | 0.436 | 93.2 | 26.80 |
| SE-6B | 0.291 | 92.0 | 20.62 |
| SE-6C | 0.162 | 87.6 | 20.74 |
| SE-7 | 0.641 | 91.0 | 20.74 |
| SE-8A | 0.114 | 94.6 | 11.05 |
| SE-8B | 0.216 | 91.4 | 10.73 |
| SE-8C | 0.041 | 90.8 | 10.83 |
| SE-8D | 0.126 | 93.7 | 11.55 |
| SW-1 | 0.272 | 93.2 | 13.72 |
| SW-2 | 0.134 | 91.9 | 15.87 |
| SW-3A | 0.319 | 92.9 | 14.52 |
| SW-3B | 0.286 | 92.9 | 10.87 |
| SW-3C | 0.169 | 92.0 | 17.28 |

| Table II-3<br>Summary of Hydrologic Parameters<br>(continued) | | | |
|---|---|---|---|
| **Catchment Name** | **Area (Sq Mi)** | **Curve Number** | **Lag-Time (min)** |
| SW-4 | 0.302 | 87.2 | 15.67 |
| SW-4A | 0.339 | 90.8 | 25.24 |
| SW-4B | 0.418 | 90.1 | 35.58 |
| SW-4C | 0.483 | 87.0 | 36.88 |
| SW-5 | 0.287 | 88.2 | 21.14 |
| SW-6 | 0.173 | 93.4 | 13.54 |
| SW-6A | 0.263 | 91.8 | 15.89 |
| SW-6T-B | 0.365 | 89.3 | 19.40 |
| SW-6-T-C | 0.243 | 92.8 | 16.66 |
| SW-7 | 0.448 | 93.3 | 23.20 |

No separate calculation of times of concentration is made for fully developed watershed development due to the facts that the watershed is nearly 90% urbanized and that urbanization of the remaining 3% will not substantively change flow paths. Therefore, calculated times of concentration and lag times are identical for existing and fully developed land use conditions.

### H. Rainfall

Statistical point rainfall depths for the 50%, 20%, 10%, 4%, 2%, 1%, and 0.2% annual chance (AC) storms up to 24-hour duration are obtained from the January 2013 edition of the City of Grand Prairie Drainage Design Manual, (DDM). The rainfall depths provided in the City DDM are compiled from the data given in the National Weather Service (NWS) *Technical Paper Number 40* (TP-40) and National Oceanic and Atmospheric Administration (NOAA) *Technical Memorandum Hydro-35*. **Table II-4** is a compilation of rainfall depths for storm durations of 5 minutes to 24 hours and frequency-based point rainfall depths.

| Table II-4<br>Rainfall Data in Inches | | | | | | | |
|---|---|---|---|---|---|---|---|
| **Storm Duration** | **Recurrence Interval (Annual Chance)** | | | | | | |
| | **50%** | **20%** | **10%** | **4%** | **2%** | **1%** | **0.2%** |
| 5 min | 0.49 | 0.57 | 0.63 | 0.73 | 0.80 | 0.87 | 1.00 |
| 15 min | 1.04 | 1.22 | 1.36 | 1.56 | 1.71 | 1.87 | 2.20 |
| 60 min | 1.85 | 2.45 | 2.86 | 3.35 | 3.82 | 4.25 | 5.40 |
| 2 hrs | 2.22 | 3.00 | 3.55 | 4.15 | 4.65 | 5.20 | 6.60 |
| 3 hrs | 2.45 | 3.30 | 3.85 | 4.55 | 5.15 | 5.70 | 7.40 |
| 6 hrs | 2.91 | 3.90 | 4.65 | 5.45 | 6.20 | 6.92 | 8.80 |
| 12 hrs | 3.45 | 4.70 | 5.50 | 6.50 | 7.35 | 8.40 | 10.50 |

**Table II-4**
**Rainfall Data in Inches (continued)**

| Storm Duration | Recurrence Interval (Annual Chance) | | | | | | |
|---|---|---|---|---|---|---|---|
| | 50% | 20% | 10% | 4% | 2% | 1% | 0.2% |
| 24 hrs | 3.95 | 5.40 | 6.40 | 7.50 | 8.52 | 9.55 | 12.00 |

## I. Flood Routing

The Modified Puls method is used to route the flood hydrographs through each reach segment of Johnson Creek. This method uses storage-discharge relationships for each routing reach. The storage-discharge relationships for the reaches are generated using the USACE computer program, Hydrologic Engineering Center – River Analysis System (HEC-RAS). The HEC-RAS model has been developed using the procedures described in the Hydraulics Section III. The Modified Puls method also requires input for the number of sub-reaches within a reach of the channel. For each reach using the modified Puls method, the number of sub-reaches must also be selected so that an appropriate degree of attenuation is computed by the program. Using the reach length and average velocity, the travel time for each reach is calculated. The average velocity is taken from the existing HEC-RAS model between designated cross section of each reach and used to calculate the sub-reaches. The storage discharge curve is developed using the HEC-RAS model by modeling various discharges. At each selected discharge the volume of water between designated cross sections (or a reach) is used to create the storage discharge curve. The Modified Puls routing storage discharge curve and sub-reaches that were previously established for Arbor Creek, a tributary of Johnson Creek are not revised from that in the October 2011 TSDN for Arbor Creek. Muskingum Cunge 8-point channel routing is used for the tributaries of Johnson Creek within the City of Arlington.

## J. Detention & Diversions

There is no reservoir storage calculated for Johnson Creek watershed since no major reservoirs are present within the watershed. The storage routing with Modified Puls captured the storage that is recently created within the City of Arlington parks project area. Arbor Creek has a storage area within the model as described in the previously approved TSDN dated October 2011. The Intersate-30 road operates as a detention area due to the undersized culverts. These undersized 2-8'x8' concrete box culvert results in considerable ponding upstream of the culvert. The level pool reservoir routing method is used to account for the ponding in this area. Due to substantial differences in the level pool inundation limits, two sets of reservoir routing data are developed: one for the low flow 10% annual chance storm and one for high flows consisting of the 4%, 2%, 1% and 0.2% annual chance storms. The upstream channel routing storage-discharge relationships are adjusted accordingly.

## K. Model Calibration

In early September 2010, Tropical Storm Hermine created significant flooding throughout North Texas including areas of Arlington and Grand Prairie. This storm event is captured throughout the City of Grand Prairie with their series of stream and rain gauges. The gauges on Johnson Creek have been in service for a short period of time. The stream gauge along Johnson Creek is just downstream of the Avenue J crossing. The following procedural outline provides the steps that are followed in an attempt to validate the hydrologic model using the data collected by these gauges during the Hermine storm event.

1. Acquire gauge data from the City of Grand Prairie's gauge 6030 Johnson Creek at Avenue J with a PT (pressure transducer) sensor. This gauge location is called out on **Figure II-1** for reference.
2. Translate the raw data into a time-series table that can be directly inserted into the HEC-HMS model. This table includes the cumulative rainfall at the gauge at every 15 minutes during the storm event.
3. Run the HEC-HMS model with existing conditions parameters. The rainfall data are used to model the Hermine flood event as a time series event. The rainfall data is placed over each drainage area. No additional rainfall data is available in the upper watershed. The same storm may or may not have fallen on each catchment as shown at Avenue J gauge showed, but this is the best available rainfall data.
4. Develop a HEC-RAS Hydraulic Model for Johnson Creek using surveyed cross sections and the 2009 one-foot contours provided by the City of Grand Prairie. Simulate all culvert and bridge crossings per City's as-built plans.
5. Insert flow rates as calculated by HEC-HMS for the Hermine storm event and run in HEC-RAS model.
6. Back calculate the flows that are required to produce the max stage elevation as recorded for the Hermine event downstream of Avenue J.
7. Compare the max stage flows (Step 6) with the September 2010 flows calculated in HEC-HMS (Step 3) and determine the match level in percentage terms.
8. If percentage match is not met within 20 percent for any of the observed flows, all parameters associated with the hydrologic and hydraulic model are checked for accuracy and approximation. Such parameters included the drainage areas, lag times, curve numbers, routing methodologies, and initial abstraction values.
9. Repeat the procedures until at least the differences in flows and elevations are reached within the margin of 20 percent error. Modify the parameters as needed.
10. There is a probability that confirmation may not be achieved. Should this be the case, a subjective reason based on physical data, limitations of the modeling process, and observations in the field are made.

Prior to defining the final output of this validation method, all calculations and activities supporting the parameters for the HMS model (including the hydraulic HEC-RAS model created to compute the Modified Puls storage-discharge relationships) are re-checked and verified to ensure maximum confidence in the model. The results of the hydrologic model validation are presented in **Table II-5**.

**Table II-5**
**Hydrologic Validation Summary**

| | |
|---|---|
| Cumulative Rainfall | 6.51 in |
| Max Stage at Gauge | 501.94 ft |
| HEC-HMS Discharge Base Stage | 502.44 ft |
| HEC-HMS Discharge Output | 8,083 cfs |
| Back Calculated Discharge at Max Stage | 6,700 cfs |

**Table II-5**
**Hydrologic Validation Summary (continued)**

| | |
|---|---|
| Percent Error Discharge | 17.11 % |

The resulting percent error is based on the comparison of the HEC-HMS generated flow from the rainfall gauge data to the back calculated flow rate obtained in HEC-RAS based on the max stage in the stream gauge. A smaller percent error might be expected if more gauge data became available. Currently, there is only the single rain gauge in the watershed and single stream gauge on Johnson Creek. The gauges have also only been in operation for about 4 years and Tropical Storm Hermine is the only recent significant storm event recorded. Therefore, a percent error with a magnitude less than 20-percent can be deemed acceptable and the model calibrated. A white paper on *Tolerance limits of errors in hydrologic and hydraulic modeling* dated April 2014 is provided in **Appendix A** for reference.

## Figure II-1

## Johnson Creek Watershed Catchment Map


### Legend

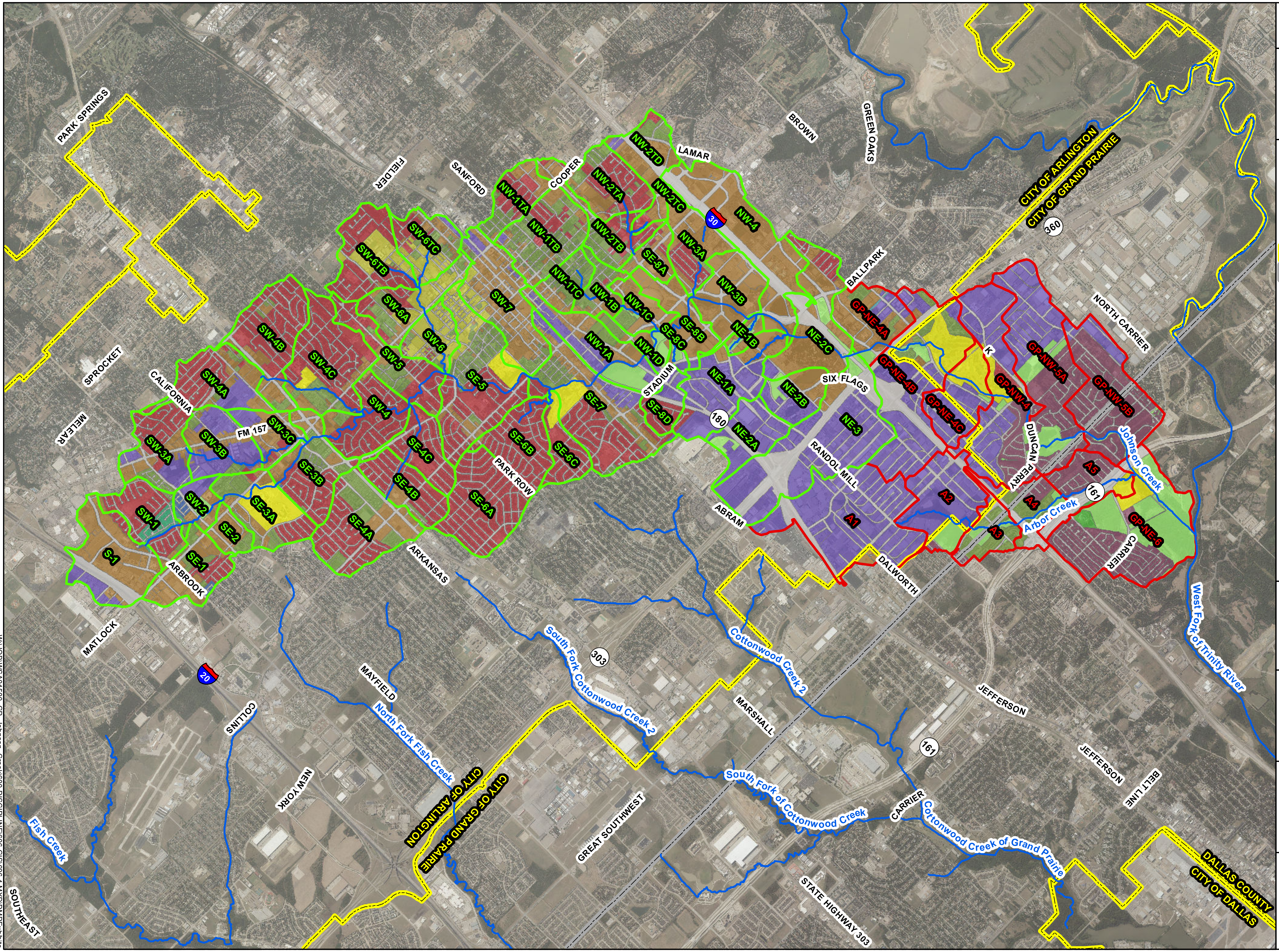
- Rain Gauge Location
- Stream Centerline
- 10' Contours
- Arlington Catchment Areas
- Grand Prairie Catchment Areas
- County Boundary
- City Limits

0 2,000 4,000 8,000 ft

JACOBS

Grand Prairie TEXAS

DMP for
Johnson Creek Corridor




# Figure II-2

## Johnson Creek Watershed Existing Conditions Land Use Map

### Legend

- Stream Centerline
- Arlington Catchment Areas
- Grand Prairie Catchment Areas
- City Limits
- County Boundary

### Land Use

- Agriculture
- Commercial
- High Density Residential (MF)
- Light Industrial
- Low Density Residential (1/4 lots)
- Medium Density Residential (1/8 lots)
- Mixed Use
- Open Space/Drainage
- Parks And Recreation
- Pavement
- Residential (A-5 less than 5000sf)
- Residential (A-7.5 7500sf)
- University

0 4,000 8,000

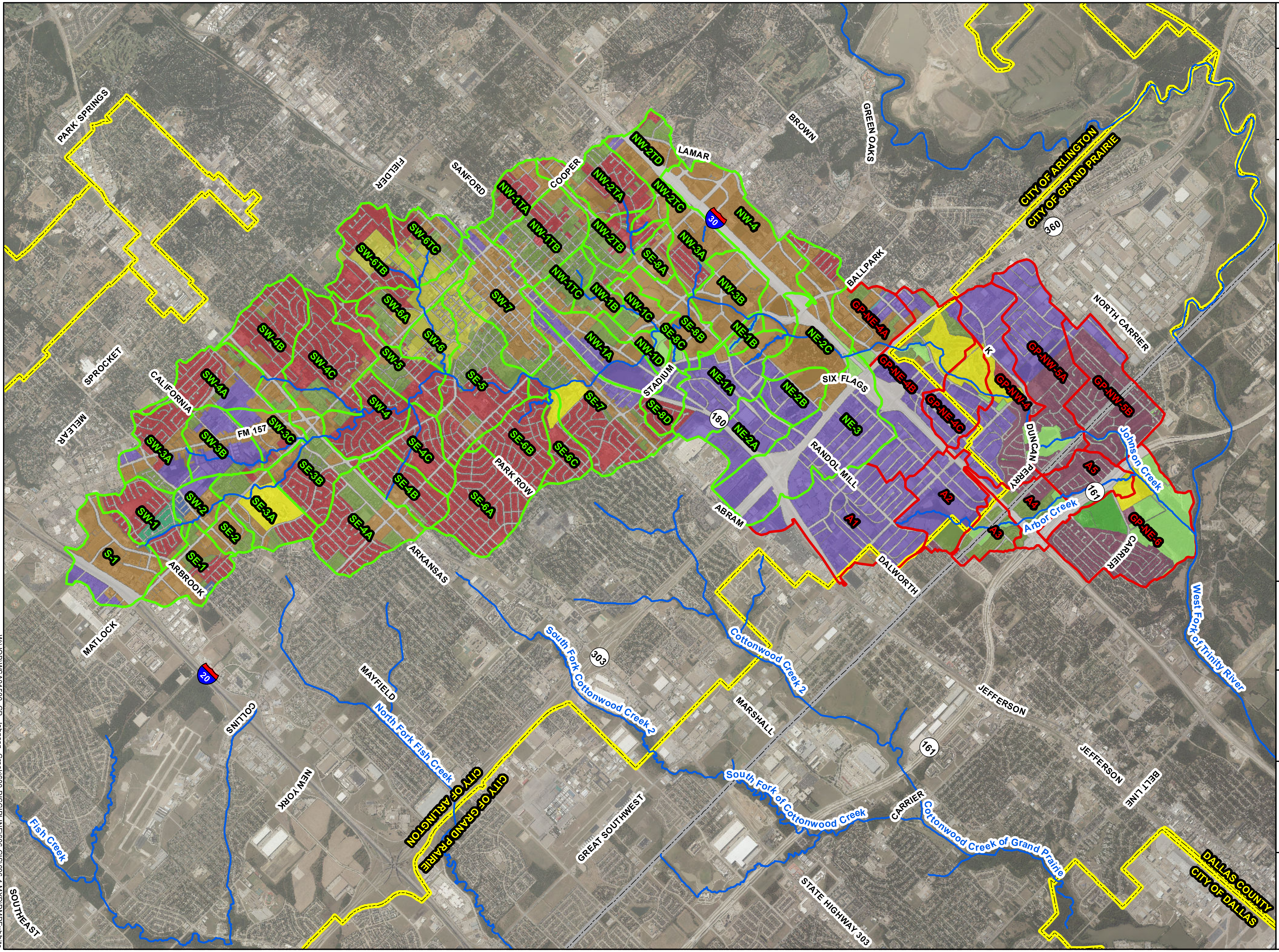
Scale in Feet

JACOBS

Grand Prairie TEXAS

DMP for Johnson Creek Corridor

W:\JOBW\WSA01500_GP_Johnson_Creek\600 DISCIPLINE\606 GIS\606.1 MXD\DMP\Exhibits




# Figure II-3

## Johnson Creek Watershed Fully Developed (Ultimate) Conditions Land Use Map

**Legend**

- Stream Centerline
- Arlington Catchment Areas
- Grand Prairie Catchment Areas
- City Limits
- County Boundary

**Land Use**

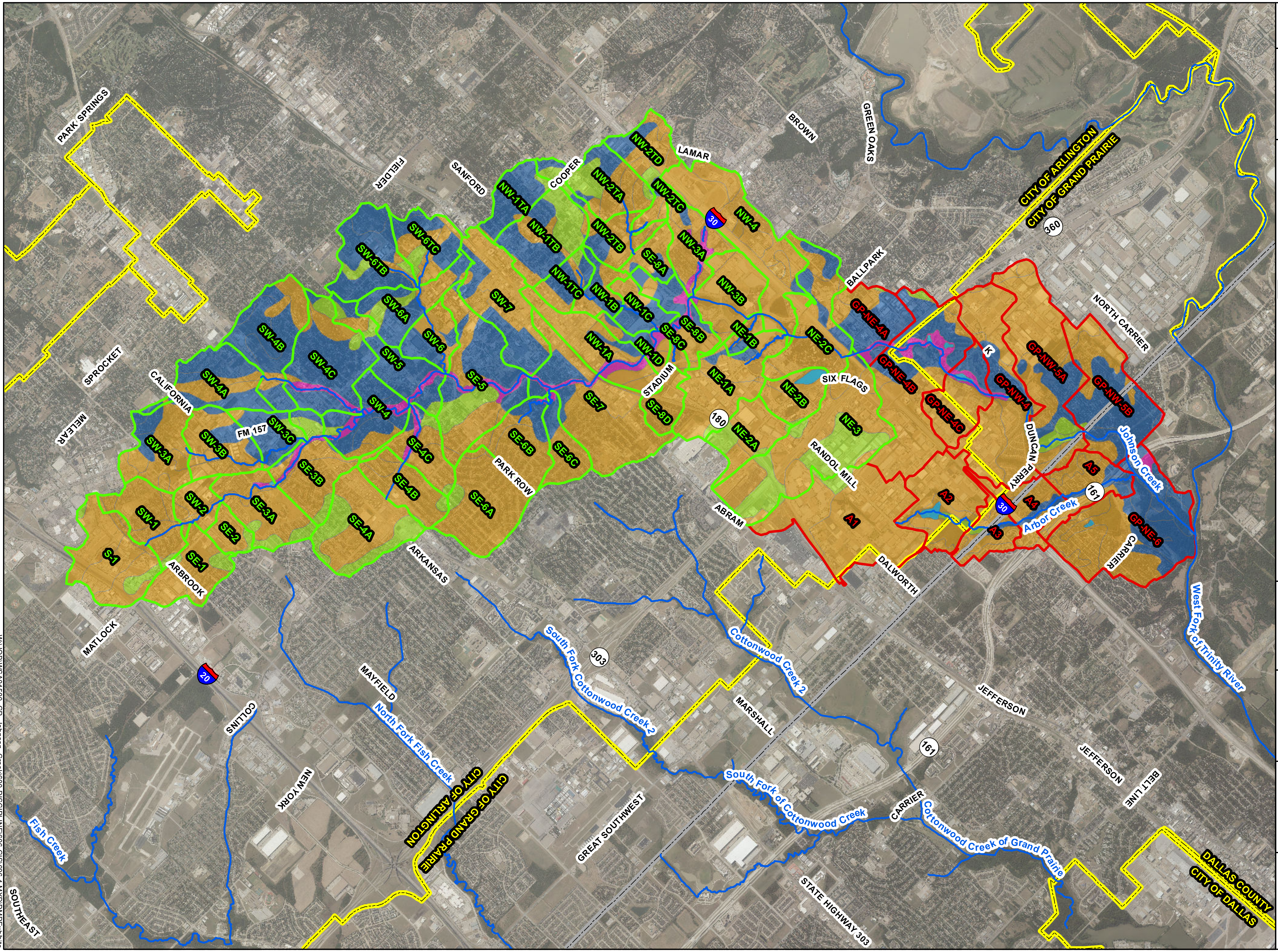
- Agriculture
- Commercial
- High Density Residential (MF)
- Light Industrial
- Low Density Residential (1/4 lots)
- Medium Density Residential (1/8 lots)
- Mixed Use
- Open Space/Drainage
- Parks And Recreation
- Pavement
- Residential (A-5 less than 5000sf)
- Residential (A-7.5 7500sf)
- University

0 4,000 8,000
Scale in Feet

JACOBS

Grand Prairie
TEXAS

DMP for
Johnson Creek Corridor




# Figure II-4

## Johnson Creek Watershed Soils Map

**Legend**

- Stream Centerline
- Arlington Catchment Areas
- Grand Prairie Catchment Areas
- City Limits
- County Boundary

**Hydrologic Soil Groups**

- A
- B
- C
- D
- Water

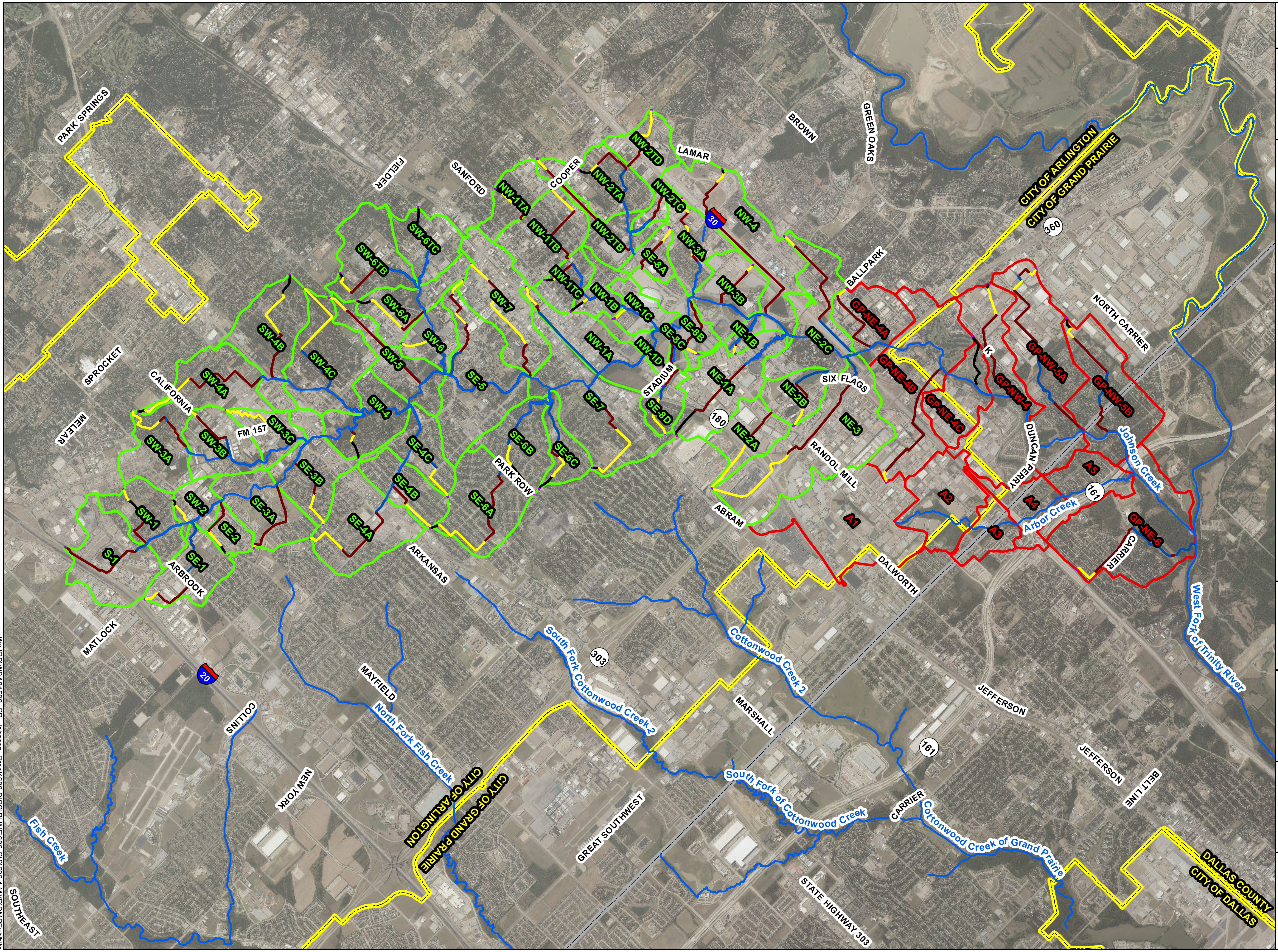
0 4,000 8,000

Scale in Feet

JACOBS®

Grand Prairie TEXAS

DMP for
Johnson Creek Corridor




# Figure II-5

## Johnson Creek Watershed Time of Concentration Map

JACOBS

Grand Prairie TEXAS

DMP for Johnson Creek Corridor

W:\JOBW\SA01500_GP_Johnson_Creek\600 DISCIPLINE\606 GIS\606.1 MXD\DMPExhibits

# III. HYDRAULIC STUDY

## A. Hydraulic Analysis

The hydraulic analysis of Johnson Creek is conducted using the HEC-RAS version 4.1 computer program. The City of Grand Prairie's current Drainage Design Manual along with FEMA's Guidelines and Specifications for Flood Hazard Mapping Partners are used as guidelines for the hydraulic analyses. The updated hydraulic model uses peak discharges computed from the HEC-HMS model for the existing 50%, 20%, 10%, 2%, 1% and 0.2% annual chance storm event (2, 5, 10, 50, 100, and 500-year) return periods. The results of the hydraulic modeling and computed water surface elevations are discussed in Section IV – Hydrologic and Hydraulic Results. The following hydraulic models were used as reference for this study:

1. 2008 US Army Corps of Engineers Johnson Creek Geomorphologic Analysis prepared by HDR Engineering, Inc. (HDR)

   HDR prepared a watershed study involving geomorphologic analysis for planning for Johnson Creek Corridor improvements. Stream flows, present conditions and predicted changes to stream conditions particularly planform geometries are analyzed using updated hydrologic and hydraulic data, and geomorphologic modeling. Additional hydraulic modeling was released from the USACE in 2010.

2. 2005 Halff Associates, Inc. Map Modernization Study

   Halff Associates, Inc. (Halff) completed a hydraulic study in 2005 which is provided to FEMA as part of the Map Modernization Program for the production of the Digital Flood Insurance Rate Maps (DFIRM) and revised FIS for Dallas and Tarrant Counties. Cross sections of Johnson Creek for this model are developed using the City of Grand Prairie's 1999 LiDAR derived topography and field survey data obtained by Halff. Bridge and culvert data are obtained from Halff's survey, existing models, and record drawings. The Halff model is included in the FIS and DFIRM panels that became effective September 29, 2009.

## B. Topography

A composite topographic shapefile consisting of field-based survey collected as points conducted in 2010 by Marshall, Lancaster & Associate, Inc. (MLAI) and one-foot interval LiDAR data collected in 2009 is used in this study. This topographic model is used as a base map for plotting all data and for floodplain delineation. From the contour shapefile, a triangulated irregular network (TIN) is created for Johnson Creek and its floodplain area to extract cross sectional geometries using the HEC-GeoRAS version 4.2.9 computer program. Each extracted cross section is evaluated against certain data external to the TIN to determine the need for augmentation of the geometrical data. Where needed, the TIN is merged with such external data, which included surveyed cross sections of the creek at selected locations, structure elevations, 'as-built' plans, and record drawings. 'As-built' plans and record drawings for all of the major crossings have been obtained from the respective agencies or municipalities to assist in defining the structural geometries.

## C. Cross Sections

1. Cross Sections Methodology

Cross sections on Johnson Creek for this study are spaced at approximately 100-200 foot intervals along the creek. A total of 127 cross sections ranging from cross section number 20784 to cross section number 424 are placed at representative locations throughout the creek. A cross section is also placed to represent flow changes, grade breaks, geometry changes, and roughness variations. In addition, four cross sections are placed to simulate the contraction and expansion of flows at the upstream and downstream faces of bridges, aerial utility crossings and drop structures. The hydraulic cross sections contained in the models are developed using a combination of the topographic information compiled into the TIN, the effective and USACE Johnson Creek hydraulic models described above, and on-the-ground field survey data provided by MLAI.

Cross section locations represented in the Johnson Creek model for this study are shown in Figure V-1 of Section V – Floodplain Mapping.

2. Structures

Detailed field surveys of roadway, trail, and utility crossings were conducted in 2010 by MLAI using GPS technology. In addition, survey had been conducted at each of the inline structures. A list of the structures surveyed is provided in **Table III-1**. No additional survey has been performed of the SH 360 crossing, TXDOT is currently under design for future improvements to this structure. Hence geometrical data for this structure is imported from the effective model and is revised based on as-built information. Also, the construction of the SH 161 crossings is completed after the MLAI survey. Therefore, as-built plans are used to define bridge deck and pier geometry for each of the SH 161 crossings. Copies of survey sketches and the as-built plans for SH 161 are included in **Appendix B**.

**Table III-1**
**Surveyed Structures**

| Structure | Location | Type |
|---|---|---|
| 500 | Avenue J Crossing | Bridge |
| 501 | Golf Course Crossing #1 | Bridge |
| 502 | Golf Course Crossing #2 | Bridge |
| 503 | Golf Course Crossing #3 | Bridge |
| 504 | Union Pacific Railroad Crossing | Bridge |
| 505 | Golf Course Crossing #4 | Bridge |
| 506 | Carrier Parkway Crossing | Bridge |
| 508 | Good Link Bike Trail Crossing | Bridge |
| 515 | 200 ft U/S of Golf Course Bridge #3 | Inline Structure |
| 516 | 1000 ft D/S of Golf Course Bridge #4 | Inline Structure |
| 517 | 200 ft NW of west end of Hidden Brook Drive | Inline Structure |
| 518 | 400 ft D/S of Good Link Bike Trail | Inline Structure |

| Table III-1 Surveyed Structures (continued) | | |
|---|---|---|
| **Structure** | **Location** | **Type** |
| 519 | At confluence with West Fork Trinity | Inline Structure |
| 520 | 850 ft D/S of Duncan Perry Road | Inline Structure |
| 537 | 100 ft D/S of Golf Course Crossing #1 | Aerial Pipe Crossing |
| 538 | 45 ft D/S of Golf Course Crossing #3 | Aerial Pipe Crossing |
| 540 | 250 ft U/S of Duncan Perry Road | Aerial Pipe Crossing |
| 541 | 60 ft U/S of Structure 520 | Aerial Pipe Crossing |
| 542 | 2500 ft U/S of Carrier Parkway | Aerial Pipe Crossing |
| 543 | 1000 ft U/S of Carrier Parkway | Aerial Pipe Crossing |
| 615 | Duncan Perry Road Crossing | Bridge |

3. Location and Layout Consideration

Cross section data for this study is obtained from a combination of (1) topographic LiDAR derived topographic data and field survey, (2) previous hydraulic models, or (3) cross sectional data geometries obtained from field surveys.

The cross section alignments from the USACE previous study conducted by USACE are imported into ArcGIS to create a polyline shapefile. The alignment and geometry of each cross section are evaluated for simulating flood conditions in the hydraulic model. Adjustments are made and additional cross sections are added to the GIS shapefile where necessary to represent the existing creek conditions not previously modeled, such as geometric transitions and structures. Each new section is developed following the procedures outlined above.

## D. Parameter Estimation

Manning’s roughness coefficients are determined from the 2011 aerial photographs and verified through field reconnaissance with reference to the guidelines outlined in the *Guide for Selecting Manning's Roughness Coefficients for Natural Channels and Flood Plains* Water-Supply Paper 2339, by the United States Geological Survey, (1989) and in *Open Channel Hydraulics* by Ven Te Chow (1959). Manning’s roughness coefficients are entered into the model on each cross section using the horizontal variation method given in HEC-RAS User's Manual. This allows for HEC-RAS to calculate the composite roughness coefficient in the channel and floodplain locations where frictional losses are highly variable along the cross section. Manning’s roughness coefficients in the channel range from 0.025 in concrete segments of the creek to 0.055 where vegetation provides a significant resistance to flow. The overbank values range from about 0.035 for new asphalt and concrete to 0.110 for dense woods and small residential lots.

**Table III-2** summarizes creek and floodplain conditions found in Johnson Creek and lists the respective Manning's roughness coefficients (“n” value) for each condition.

| Table III-2<br>Manning's Roughness Coefficients | | |
|---|---|---|
| **Description Type** | **Channel n-value** | **Overbank n-value** |
| Apartment Complex | - | 0.08 |
| Asphalt | - | 0.02 |
| Bare Earth | 0.03 | 0.035 |
| Bare Earth - Some Grass | 0.035 | 0.035 |
| Brush and Trees | - | 0.10 |
| Channel - Sandy Gravel | 0.04 | 0.05 |
| Concrete | 0.025 | 0.02 |
| Trees - Light Canopy | - | 0.07 |
| Trees - Medium Canopy | - | 0.08 |
| Trees - Dense Canopy | - | 0.10 |
| Trees - Dense Canopy and Underbrush | - | 0.11 |
| Gabions - Clean | 0.03 | 0.03 |
| Gabions - Rough | 0.045 | 0.045 |
| Mowed Grass | 0.035 | 0.035 |
| Water | - | 0.025 |
| Grass - Medium Height | 0.045 | 0.055 |
| Grass - Tall | 0.055 | 0.07 |
| Gravel | 0.04 | - |
| Outbuildings | - | 0.07 |
| Parking Lot | - | 0.05 |
| Residential (Small lots) | - | 0.11 |
| Buildings | - | 0.10 |
| Riprap | 0.06 | 0.06 |
| Wetlands | 0.06 | 0.07 |

## E. Modeling Considerations

1. Boundary Conditions and Mapping Tie-ins

FEMA specifies that a normal depth boundary condition at the downstream end of the model should be used. The downstream boundary condition for Johnson Creek is input into the HEC-RAS model with an assumed energy grade line (EGL) slope for each storm event, **Table III-3** shows the normal depth used in the model. Based on the assumed slope, HEC-RAS computes normal depth and the resulting water surface elevation (WSEL) for the cross section at the most downstream end. Under this imposed boundary condition, the WSEL (backwater) at Johnson Creek immediately upstream of its confluence point at West Fork Trinity River is compared with the WSEL at the river. At this location, the calculated WSEL for the 1% annual chance flood is 446.10 feet. This corresponds to the WSEL at the confluence of Johnson Creek and West Fork Trinity River reported in the effective FIS report. Thus, the downstream boundary condition used in the present investigation seems appropriate.

| Table III-3<br>Normal Depth | |
|---|---|
| **Annual Probability of Occurrence (%)** | **Normal Depth ($S_f$)** |
| 50 | 0.002200 |
| 20 | 0.001800 |
| 10 | 0.001160 |
| 4 | 0.000800 |
| 2 | 0.000230 |
| 1 | 0.000127 |
| 0.2 | 0.000035 |

2. Structure/Road Crossing Modeling

The segment of Johnson Creek within the City of Grand Prairie contains 16 bridges, 6 aerial crossings, and 5 drop structures. The drop structures are modeled as inline weirs with model input data that include stream distances to upstream cross sections, weir profile data, and typical broad crested weir coefficients. The bridges are modeled by entering into the model the high and low chord profile data, bridge width, and the overtopping weir coefficient. In addition, pier data for the bridges are entered into the model from ground surveys and as built plans. Energy balance or momentum balance methods are selected for low flow computation but the method that produced highest energy grade at a cross section is finally utilized. A drag coefficient of 1.2 is used to represent the flow transition around circular bridge piers. The energy balance method is selected for high flow conditions. The portion of Johnson Creek that passes through the City of Grand Prairie does not contain any crossings with culverts. Also, aerial pipe crossings are modeled as bridges with piers. The cumulative pipe diameter (in case of multiple pipes) at each crossing is input as the bridge width and the pipe heights used to determine the bridge deck thickness.

3. Islands and/or Flow Splits

Johnson Creek has been evaluated for locations where split flows may occur. There is a low flow split that occurs downstream of SH 161. In this region, the USACE designed a straight channel to convey flood flows directly to the West Fork Trinity River with low flows being diverted into Johnson Creek's original meandering channel. The inline structure located at River Station (RS) 3710 (Inline Structure #5 - D/S of PGBT) allows for flows to be diverted. However, the floodplain is not disconnected between the high and low flow channels so a split flow analysis is not viable. There is no other location where major channel splits with islands or other structures. Therefore, split flow is not considered in the hydraulic model. However, the cross sectional geometries from RS 3721 to RS 424 capture both the straight section of the channel (built in diversion) and the original meandering section of the channel.

4. Ineffective Flow Areas

Ineffective flow limits are determined using a typical 1:1 contraction ratio and a 2:1 expansion ratio and are placed in the model upstream and downstream of existing structures, buildings, and other significant obstructions. Ineffective flow is also used to represent flow changes where topographical information indicates sudden expansion or contraction of flow. Contraction and expansion coefficients of 0.1 and 0.3 respectively are used at typical cross sectional geometry transitions with 0.3 and 0.5, respectively to represent head losses at bridges. At locations where the limit of effective flow would be constricted, the contraction and expansion coefficients are increased according to recommendations provided in the HEC-RAS User's Manual to compensate for the increased head losses associated with such a transition. For example, between cross sections 14248 and 13967, the top width of the 1% annual chance flood decreases from approximately 650 feet to 180 feet, so the contraction and expansion coefficients are increased to 0.5 and 0.7, respectively to account for the head losses occurring here.

5. Supercritical Flow Condition

The HEC-RAS program at several locations, calculated the water surface elevations as critical depth. This is a typical result when the model cannot solve for depth in a given number of iterations or as a default when, during the iteration process, the calculated depth of flow goes below critical depth during subcritical flow regime specified in the computational conditions. Each location in the model, where calculated flood depths resulted in critical depth is at a bridge or structure where supercritical flow can be expected. The critical flow at Cross Section 13105 occurs in a concrete channel immediately upstream of an inline structure. The steepness and depth of drop at this inline structure indicates that supercritical flow can potentially occur at this location. However, FEMA's Guidelines and Specifications for Flood Hazard Mapping Partners in these cases (Section C.3.4.4) require to always use the subcritical flow calculation routine within HEC-RAS for simulations within natural streams. For this reason at these potential erosion deposition locations critical depths are used for WSEL calculations.

A CD-ROM containing copies of all hydraulic computer models, GIS shapefiles, and figures used in preparation of this report is included in **Appendix E**.

# IV. HYDROLOGIC AND HYDRAULIC STUDY RESULTS

## A. Hydrologic Study Results

As previously discussed in Section II Hydrologic Study, undeveloped or open space land contributes to approximately 13% of the watershed area under existing conditions and approximately 10% of the watershed area under fully developed conditions as available area for further development. Consequently, fully developed urbanization of the watershed would result in average increases in peak flood discharges across the watershed less than 1%, which in this case is determined to be within FEMA's significance of confidence limits as described in FEMA's Guidance for Riverine Flooding Analysis and Mapping (Appendix C). **Table IV-1** is a summary of peak flood discharges for Johnson Creek at various locations in the watershed within the City of Grand Prairie for storms with various return periods.

**Table IV-1**
**Summary of Discharges**

| Location | Hydrologic Node | Drainage Area (sq mi) | Peak Discharges (cfs) Recurrence Interval (Annual Chance) | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | | | 50% | 20% | 10% | 4% | 2% | 1% | 0.2% |
| Confluence with West Fork | Junction-26 | 20.84 | 7,701 | 12,118 | 14,509 | 16,810 | 19,283 | 20,719 | 24,577 |
| Confluence with Arbor Creek | Junction-25 | 20.04 | 7,815 | 12,199 | 14,545 | 16,641 | 19,130 | 20,560 | 24,392 |
| US of Carrier Pkway | Junction-24 | 18.26 | 7,456 | 10,398 | 12,696 | 14,739 | 17,067 | 18,405 | 22,336 |
| 2400' DS of Ave K | Junction-23 | 17.70 | 7,419 | 10,307 | 12,589 | 14,601 | 16,902 | 18,233 | 22,314 |
| US of Ave K | Junction-22-2 | 17.00 | 7,368 | 10,184 | 12,445 | 14,424 | 16,688 | 18,041 | 22,143 |
| US N Great Southwest Pkwy | Junction-22 | 16.35 | 7,331 | 10,121 | 12,335 | 14,276 | 16,561 | 17,926 | 22,024 |
| URS Railroad | Junction-21-3 | 16.07 | 7,318 | 10,095 | 12,307 | 14,209 | 16,530 | 17,912 | 22,015 |
| US HWY 360 | Junction-21-2 | 15.33 | 7,295 | 10,160 | 12,390 | 14,088 | 16,711 | 17,979 | 22,245 |
| DS I-30 | Junction-21 | 15.04 | 7,280 | 10,134 | 12,369 | 14,077 | 16,996 | 18,344 | 22,242 |

The results of the 1% annual chance peak discharges for Johnson Creek are compared to the computed discharges in the HDR 2008 USACE Study as well as to those listed in the FEMA September 2009 FIS. **Table IV-2** provides a comparison of the peak discharges for each of the studied events along with the discharges and drainage area size obtained from the USGS regional regression equation. The USGS regression equations are an additional source for estimating peak discharges in the Dallas-Fort Worth area as defined in the NCTCOG iSWM Design Manual for Site Development. These equations are developed for urban streams between 3 and 40 square miles, which is ideal for this watershed comparison. The standard error of estimate of the regression equations is approximately 30 percent. **Figure IV-1**, Hydrologic Junctions Map, lays out these junctions discharge points.

The HDR 2008 USACE Study used larger catchment areas and the rainfall model is accurate to the SCS 24 hour Type II storm rainfall data of 9.5 inches. The revised discharges are modeled with a storm frequency for multiple events and more defined catchments within the upper watershed for a more detailed analysis. Also, the watershed has had some improvements made by the USACE along Johnson Creek in areas that provided additional storage and I-30 improvements in the watershed contributed to further better defined catchments and discharge locations. The time of concentrations are carefully evaluated for the sub-divided catchments. Therefore, the more detailed watershed data and storm analysis shows a lower or more detailed discharge than in the previous study.

Compared to the discharges reported in the effective FEMA FIS increases in peak discharge occur. This is expected as much of the watershed has been further developed. However, with additional storage areas provided in areas of the upper reaches of the watershed through new developments discharges show minimal increases as compared to the 1996 USACE Hydrologic study as referenced in Section I-H of this plan.

**Table IV-2**
**Discharge Comparison with Effective Data**

| Flooding Source and Approximate Location | Drainage Area (sq mi) | Revised Discharges (cfs) | HDR Study (cfs) | 1996 Discharges (cfs) | FEMA Discharges (cfs) | Regression Equation (cfs) |
|---|---|---|---|---|---|---|
| US of Hwy 360 | 15.33 | 17,979 | - | - | 17260 | 16,544 |
| UP Railroad | 16.07 | 17,912 | - | - | - | - |
| 200 feet US of 111th Street | 16.35 | 17,926 | 20,004 | 21,170 | - | - |
| US of Duncan Perry Road | 17.00 | 17,979 | 20,004 | - | 17,220 | - |
| 2400' DS of Ave K | 17.70 | 18,233 | 18,396 | 20,170 | - | - |
| US of Stream JC-1 (Arbor Creek) | 18.26 | 18,405 | 18,756 | 20,300 | - | - |
| DS of Stream JC-1 (Arbor Creek) | 20.04 | 20,560 | 18,756 | 20,300 | 17,310 | - |
| Confluence with Trinity River | 20.84 | 20,719 | 22,300 | 23,390 | 18,130 | 21,134 |

## B. Hydraulic Study Results

The computed peak discharges are used for calculation of water surface elevations resulting from the 50%, 20%, 10%, 4%, 2%, 1%, and 0.2% annual chance storm events for existing conditions. **Table IV-3** compares the differences in computed water surface elevations given in the FEMA FIS effective study and the present investigation in the City of Grand Prairie, called Revised Preliminary WSEls for the 1% annual chance flood. Several factors cause the differences in WSEls at certain cross sections in the effective model and in the present study. Differences in water surface elevations can be attributed to new peak discharges reflecting most recent urban developments in the watershed, new structures along the channels including SH 161, and incorporation of finer details in the cross sectional geometries and model parameters in the current updated hydraulic models.

In addition, the City of Grand Prairie requested that the six aerial utility crossings be simulated. These structures provide a significant obstruction to flow due to the presence of four or more piers in each crossing and pipe thicknesses that range from approximately one to five feet in diameter. The aerial utility crossings create significant backwater during the 1% annual chance flood flows at these locations. This backwater propagates upstream to a considerable distance from each structure. These aerial utility crossings are not included in the FEMA effective models and are one of the major causes that result in the significant differences in the calculated WSELs in the present study as compared to the FEMA effective discharges. These differences in water surface elevations difference will be shown in the mapping update. The hydraulic modeling is prepared in such a way to allow for tying in with the effective FEMA floodplain using FEMA's criteria of no greater than 0.5 feet above or below the effective WSEL. Additional care is being provided for the tie-in analysis as the City of Arlington begins their remapping process of the upper portion of the creek. The increases shown in **Table IV-3** at the confluence are contributed to the downstream boundary condition differences. This revised study implemented the downstream boundary condition with the backwater from the West Fork Trinity River. The floodplain mapping delineates this backwater elevation. The difference between the effective 100-year floodplain and the revised floodplain mapping is shown on **Figure IV-2**. In most areas, due to more accurate topography and revised watershed hydrologic analysis since the effective mapping, the floodplain top width was narrowed even with increased water surface elevations. The USACE gabion lined section (RAS cross sections 9558 to 5113) was designed with minimal freeboard.

**Appendix B** contains a complete listing of computed flood water surface elevations for all flood profiles on Johnson Creek. Flood profiles are included in **Appendix B** of this report for all the previously specified flood profiles.

A CD-ROM included in **Appendix E** contains all of the hydraulic models and mapping shape files developed as part of this report.

**Table IV-3**
**Water Surface Elevation Comparison**

| Effective Cross Section | Effective WSEl (ft) | Revised Preliminary FIS Section | Revised WSEl Preliminary FIS (ft) | Difference Revised and Effective (ft) |
|---|---|---|---|---|
| 20930 | 517.30 | 20784 O | 517.00 | -0.30 |
| 20635 | 517.43 | 20476 | 516.19 | |
| 20275 | 516.93 | 20159 | 515.60 | |
| 19925 | 516.46 | 20023 | 513.50 | |
| 19850 | 514.22 | 19980 | SH 360 SB Frontage Rd | |
| 19818 | - | 19941 | 512.94 | |
| 19785 | 513.19 | 19925 | 512.99 | |
| 19745 | 513.09 | 19850 | State Highway 360 | |
| 19685 | - | 19770 | 512.03 | |
| 19625 | 512.23 | 19758 | 512.23 | |

**Table IV-3**
**Water Surface Elevation Comparison (continued)**

| Effective Cross Section | Effective WSEl (ft) | Revised Preliminary FIS Section | Revised WSEl Preliminary FIS (ft) | Difference Revised and Effective (ft) |
|---|---|---|---|---|
| 19565 | 512.14 | 19700 | SH 360 NB Frontage Rd | |
| 19532 | - | 19645 | 511.81 | |
| 19500 | 511.24 | - | 511.26 | -0.04 |
| 19440 | 511.26 | 19396 N | 510.76 | |
| 19340 | 511.00 | - | 510.06 | 1.06 |
| 19097 | 510.42 | 19085 | 510.15 | |
| 18977 | 510.11 | - | 508.49 | |
| 18841 | 509.95 | - | Avenue J Bridge | |
| 18701 | 509.03 | 18756 M | 506.79 | |
| 18611 | 508.56 | 18423 | 506.96 | |
| 18375 | 508.60 | 18128 | 507.10 | |
| 18081 | 508.40 | 18080 | Golf Course Bridge #1 | |
| 18008 | 507.20 | 18034 | 507.06 | 1.76 |
| 17940 | - | 17926 | 507.11 | |
| 17878 | 505.99 | 17845 | Aerial Pipe Crossing #1 | |
| 17827 | 506.27 | 17835 | 507.09 | |
| 17741 | 506.09 | - | 507.07 | |
| 17711 | 505.53 | - | 507.02 | |
| 17697 | - | - | Golf Course Bridge #2 | |
| 17683 | 505.11 | - | 506.97 | |
| 17652 | 505.26 | 17827 L | 506.92 | |
| 17374 | 505.00 | 17768 | 506.88 | |
| 17329 | 504.96 | 17750 | Inline Structure #1 | |
| 17283 | 504.79 | 17736 | 506.87 | |
| 17270 | - | 17568 | 506.55 | |
| 17257 | 504.68 | 17425 | 506.52 | |
| 17222 | 504.65 | 17407 | Golf Course Bridge #3 | |
| 17133 | 504.43 | 17391 | 506.42 | |
| 16960 | 504.31 | 17219 | 506.44 | |
| 16934 | - | 17075 | Aerial Pipe Crossing #2 | |
| 16907 | 504.30 | 17055 | 506.39 | |
| 16760 | 504.22 | 17044 | 506.39 | |
| 16755 | - | 16923 | 506.39 | |
| 16739 | 503.79 | 16892 | 506.28 | |
| 16705 | 503.71 | 16876 | 505.85 | 3.35 |
| 16541 | 503.48 | 16863 | Union Pacific Railroad | |

| Table IV-3<br>Water Surface Elevation Comparison (continued) | | | | |
|---|---|---|---|---|
| **Effective Cross Section** | **Effective WSEl (ft)** | **Revised Preliminary FIS Section** | **Revised WSEl Preliminary FIS (ft)** | **Difference Revised and Effective (ft)** |
| 16039 | 503.16 | 16845 | 500.76 | |
| - | - | 16830 | 500.72 | |
| - | - | 16816 | 497.87 | -0.13 |
| - | - | 16596 | Golf Course Bridge #4 | |
| - | - | 16489 | 495.39 | |
| - | - | 16215 | 493.33 | |
| 15881 | 502.45 | 16029 K | 493.38 | |
| 15810 | - | 15972 | 493.04 | |
| 15738 | 498.74 | 15916 | 492.58 | |
| 15708 | 498.32 | 15857 | 491.07 | 0.37 |
| 15671 | 498.04 | 15829 J | Inline Structure #2 | |
| 15660 | - | 15804 | 491.07 | |
| 15649 | 496.07 | 15780 | 490.13 | |
| 15558 | 493.72 | 15623 | 489.81 | 0.51 |
| - | - | 15472 | 486.79 | |
| 15047 | 491.21 | 15246 I | 486.43 | 0.43 |
| - | - | 15053 | 483.04 | |
| 14702 | 490.65 | 14839 H | 482.84 | |
| 14685 | - | 14828 | 482.90 | |
| 14664 | 490.59 | 14816 | 482.31 | -1.29 |
| - | - | 14540 | Inline Structure #3 | |
| 14180 | 489.32 | 14248 G | 482.31 | |
| - | - | 13967 | 482.20 | |
| 13720 | 486.00 | 13662 F | 482.03 | |
| 13321 | 484.47 | 13419 | 481.48 | |
| - | - | 13345 | 481.37 | |
| - | - | 13260 | 480.59 | 1.79 |
| 12968 | 483.58 | 13105 E | 478.40 | |
| 12946 | - | 13100 | 478.14 | |
| 12924 | 481.58 | 13073 | 477.55 | |
| - | - | 12931 | 476.79 | 0.89 |
| 12727 | 481.32 | 12781 | 475.98 | |
| - | - | 12639 | 474.30 | |
| - | - | 12423 | 473.82 | 0.32 |
| 12020 | 478.83 | 12201 D | 472.89 | |
| - | - | 11939 | 472.97 | |
| - | - | 11715 | Aerial Pipe Crossing #3 | |

| Table IV-3<br>Water Surface Elevation Comparison (continued) | | | | |
|---|---|---|---|---|
| **Effective Cross Section** | **Effective WSEl (ft)** | **Revised Preliminary FIS Section** | **Revised WSEl Preliminary FIS (ft)** | **Difference Revised and Effective (ft)** |
| 11496 | 477.74 | 11477 | 472.01 | |
| 11042 | 475.92 | 11247 C | 471.41 | |
| - | - | 10998 | 471.27 | |
| - | - | 10809 | Duncan Perry Road | |
| 10523 | 473.55 | 10591 B | 471.37 | |
| - | - | 10375 | 466.94 | -0.76 |
| - | - | 10261 | 465.80 | |
| - | - | 10240 | 465.68 | |
| - | - | 10217 | 465.47 | |
| 10004 | 470.48 | 10139 | Aerial Pipe Crossing #4 | |
| 9886 | 470.08 | 10018 | 464.80 | |
| 9850 | - | 9992 | 464.80 | |
| 9815 | 469.25 | 9969 | Inline Structure #4 | |
| 9741 | 467.56 | 9735 A | 463.90 | |
| 9239 | 467.09 | 9558 | 463.88 | |
| - | - | 9310 | 464.04 | |
| - | - | 9207 | 463.82 | |
| - | - | 9197 | 461.79 | |
| 9092 | 467.27 | 9187 | 462.02 | |
| 9016 | 464.18 | 9177 | 461.62 | |
| 9008 | - | 9152 | 461.39 | |
| 9001 | 463.50 | 9116 | 459.90 | |
| 8903 | 463.40 | 9059 | 458.01 | |
| 8786 | 463.22 | 8921 | Aerial Pipe Crossing #5 | |
| 8709 | 463.09 | 8808 | 458.16 | |
| 8476 | 461.98 | 8535 | 457.61 | |
| 8368 | 461.54 | 8399 | 456.68 | |
| 8256 | 461.60 | 8149 | 456.29 | |
| 8025 | 461.34 | 7887 | 456.52 | |
| 7658 | 460.62 | 7598 | 455.85 | |
| 7332 | 458.74 | 7233 | 455.38 | |
| 7259 | 458.89 | 7220 | 455.37 | |
| 7009 | 458.27 | 7206 | 455.16 | |
| 6758 | 457.80 | 6998 | 454.64 | |
| 6714 | 456.84 | 6873 | 453.36 | |
| 6671 | 456.25 | 6715 | Aerial Pipe Crossing #6 | |
| 6615 | 456.69 | 6566 | 452.78 | |

| Table IV-3<br>Water Surface Elevation Comparison (continued) | | | | |
|---|---|---|---|---|
| Effective Cross Section | Effective WSEl (ft) | Revised Preliminary FIS Section | Revised WSEl Preliminary FIS (ft) | Difference Revised and Effective (ft) |
| 6569 | 456.77 | 6473 | 452.67 | |
| 6530 | 454.12 | 6371 | 452.59 | |
| 6493 | 454.70 | 6339 | 451.63 | |
| 6442 | 455.31 | 6215 | 451.72 | |
| 6396 | 455.52 | 6084 | 451.77 | |
| 6335 | 455.26 | 5755 | 451.90 | |
| 6292 | 454.47 | 5745 | North Carrier Parkway SB | |
| 6257 | 454.46 | 5738 | 451.42 | |
| 6229 | 453.49 | 5673 | 451.41 | |
| 5975 | 452.52 | 5499 | North Carrier Parkway NB | |
| 5531 | 450.12 | 5291 | 451.27 | |
| 5468 | 450.25 | 5113 | 451.09 | |
| 5328 | 449.90 | - | State Highway 161 SB - PGBT | |
| 5272 | 450.53 | - | 450.88 | |
| 5099 | 447.49 | - | 450.80 | |
| 5046 | 448.33 | - | State Highway 161 - PGBT | |
| 4864 | 448.15 | - | 450.12 | |
| - | - | 4932 | 450.08 | |
| - | - | 4809 | State Highway 161 NB - PGBT | |
| - | - | 4773 | 449.57 | |
| 4741 | 448.85 | 4738 | 449.48 | |
| 4683 | 448.94 | 4730 | Good Link Bike Trail | |
| 4600 | 0.00 | 4698 | 449.31 | |
| 4508 | 448.59 | 4666 | 448.68 | |
| 4393 | 447.94 | 4613 | 448.61 | |
| 4172 | 447.20 | 4578 | 448.30 | |
| - | - | 4544 | Inline Structure #5 | |
| - | - | 4518 | 448.28 | |
| - | - | 4404 | 447.20 | |
| - | - | 4291 | 446.54 | |
| - | - | 4276 | 446.39 | |
| - | - | 4245 | 446.24 | |
| - | - | 4215 | 446.16 | |
| - | - | 4172 | 446.10 | |
| - | - | 4151 | 517.00 | -0.30 |
| - | - | 4131 | 516.19 | |
| - | - | 3973 | 515.60 | |

**Table IV-3**
**Water Surface Elevation Comparison (continued)**

| Effective Cross Section | Effective WSEl (ft) | Revised Preliminary FIS Section | Revised WSEl Preliminary FIS (ft) | Difference Revised and Effective (ft) |
|---|---|---|---|---|
| 3669 | 446.30 | 3807 | 513.50 | |
| 3595 | 445.86 | 3721 | SH 360 SB Frontage Rd | |
| 3560 | 0.00 | 3710 | 512.94 | |
| 3524 | 445.58 | 3660 | 512.99 | |
| 3276 | 445.09 | 3048 | State Highway 360 | |
| 3008 | 444.52 | - | 512.03 | |
| 2815 | 442.88 | - | 512.23 | |
| 2561 | 442.07 | - | SH 360 NB Frontage Rd | |
| 2346 | 442.15 | 2372 | 511.81 | |
| 2249 | 441.52 | - | 511.26 | -0.04 |
| 1474 | 441.14 | 1873 | 510.76 | |
| 1281 | 440.64 | 1388 | 510.06 | 1.06 |
| 780 | 439.39 | 873 | 510.15 | |
| 421 | 438.69 | 424 | 508.49 | |

[1] FEMA Effective water surface elevations from the effective FIS
[2] CTP Study water surface elevations that will revise the effective data once approved by FEMA
[3] Cross sections are compared at sections at approximately the same location, the difference is approximate and used just for reference.

The accuracy of the hydraulic model has been evaluated by using the information of the high water mark created during the Tropical Storm Hermine flooding event. (Refer to Section II-K) Gauging station 6030 located downstream of Avenue J is used in determining the accuracy of the model. The peak flows obtained from the rainfall data for this storm event as calculated in the HEC-HMS model are inputted into the HEC-RAS model to calculate WSELs that can be compared to the high water marks noted during field investigation. The maximum water surface elevation at the gage station is 501.94 feet during the Hermine event. The HEC-RAS model calculates a WSEL of 502.34 feet using the peak flows calculated for this event using the HEC-HMS model. Thus, the difference between observed and calculated WSEL is 0.4 feet. Unfortunately, due to a single gauging station and rainfall data available for only a few years, a true calibration of the hydraulic model cannot be performed. The resulting difference between the observed and calculated WSEL is within a range that can be considered reasonable for simulation of other flooding events.

## C. Floodway

Floodway analysis has been performed in HEC-RAS according to FEMA's guidelines for the 1% annual chance flood flows. The floodway encroachment stations in the model are determined first by measuring the floodway widths and locations found on the effective FIRM panel relative to each cross section. The approximate encroachment location is then modified at locations where the floodway surcharge is greater than 1.0 feet or less than 0.0 feet. In cases where the encroachment stations fall inside the creek bank, the station is moved at least to the top of bank. Most modifications to the effective regulatory floodway are due to the topographic changes that have occurred along the stream, more accurate ground elevation data, and differences in peak flood discharges. The floodways developed in this study are shown with yellow dashed lines as indicated in **Figure IV-2**, Floodway Comparison Map. **Table IV-4** contains details of the floodway and base flood elevations for the lettered cross sections that will be identified on the revised FIRM panels.

**Table IV-4**
**Floodway Data Table**

| FLOODING SOURCE | | FLOODWAY | | | BASE FLOOD WATER SURFACE ELEVATION | | | |
|---|---|---|---|---|---|---|---|---|
| **CROSS SECTION** | **DISTANCE[1]** | **WIDTH (FEET)** | **SECTION AREA (SQUARE FEET)** | **MEAN VELOCITY (FEET PER SECOND)** | **REGULATORY** | **WITHOUT FLOODWAY** | **WITH FLOODWAY** | **INCREASE** |
| Johnson Creek | | | | | | | | |
| A | 9,735 | 333 | 1,524 | 12.08 | 466.95 | 466.95 | 467.01 | 0.1 |
| B | 10,591 | 218 | 2,118 | 8.52 | 473.82 | 473.82 | 473.99 | 0.2 |
| C | 11,247 | 188 | 2,267 | 7.96 | 476.79 | 476.79 | 477.32 | 0.5 |
| D | 12,201 | 216 | 2,858 | 6.31 | 480.59 | 480.59 | 481.00 | 0.4 |
| E | 13,105 | 414 | 2,292 | 7.87 | 482.31 | 482.31 | 482.83 | 0.5 |
| F | 13,662 | 253 | 2,332 | 7.74 | 486.43 | 486.43 | 486.57 | 0.2 |
| G | 14,248 | 214 | 2,486 | 7.21 | 489.81 | 489.81 | 489.87 | 0.1 |
| H | 14,839 | 468 | 2,888 | 7.96 | 491.07 | 491.07 | 491.40 | 0.3 |
| I | 15,246 | 394 | 2,686 | 6.67 | 493.04 | 493.04 | 493.16 | 0.1 |
| J | 15,829 | 185 | 1,482 | 12.10 | 497.87 | 497.87 | 497.91 | 0.0 |
| K | 16,029 | 158 | 2,647 | 6.77 | 505.85 | 505.85 | 505.87 | 0.0 |
| L | 17,827 | 372 | 3,508 | 5.11 | 507.06 | 507.06 | 507.68 | 0.6 |
| M | 18,756 | 166 | 2,207 | 8.12 | 510.06 | 510.06 | 510.45 | 0.4 |
| N | 19,396 | 164 | 2,109 | 8.49 | 511.26 | 511.26 | 511.64 | 0.4 |
| O | 20,784 | 135 | 2,510 | 7.16 | 517.00 | 517.00 | 517.40 | 0.4 |

[1] Feet above West Fork Trinity River along profile base line

| TABLE | FEDERAL EMERGENCY MANAGEMENT AGENCY DENTON COUNTY, TEXAS AND INCORPORATED AREAS | FLOODWAY DATA |
|---|---|---|
| | | Johnson Creek |

## D. Quality Assurance / Quality Control

Quality assurance/quality control (QA/QC) for the 2013 hydrologic and hydraulic studies is performed by a third party reviewer (Halff Associates) in March 2014. The QA/QC comments and responses are included electronically in **Appendix E**.

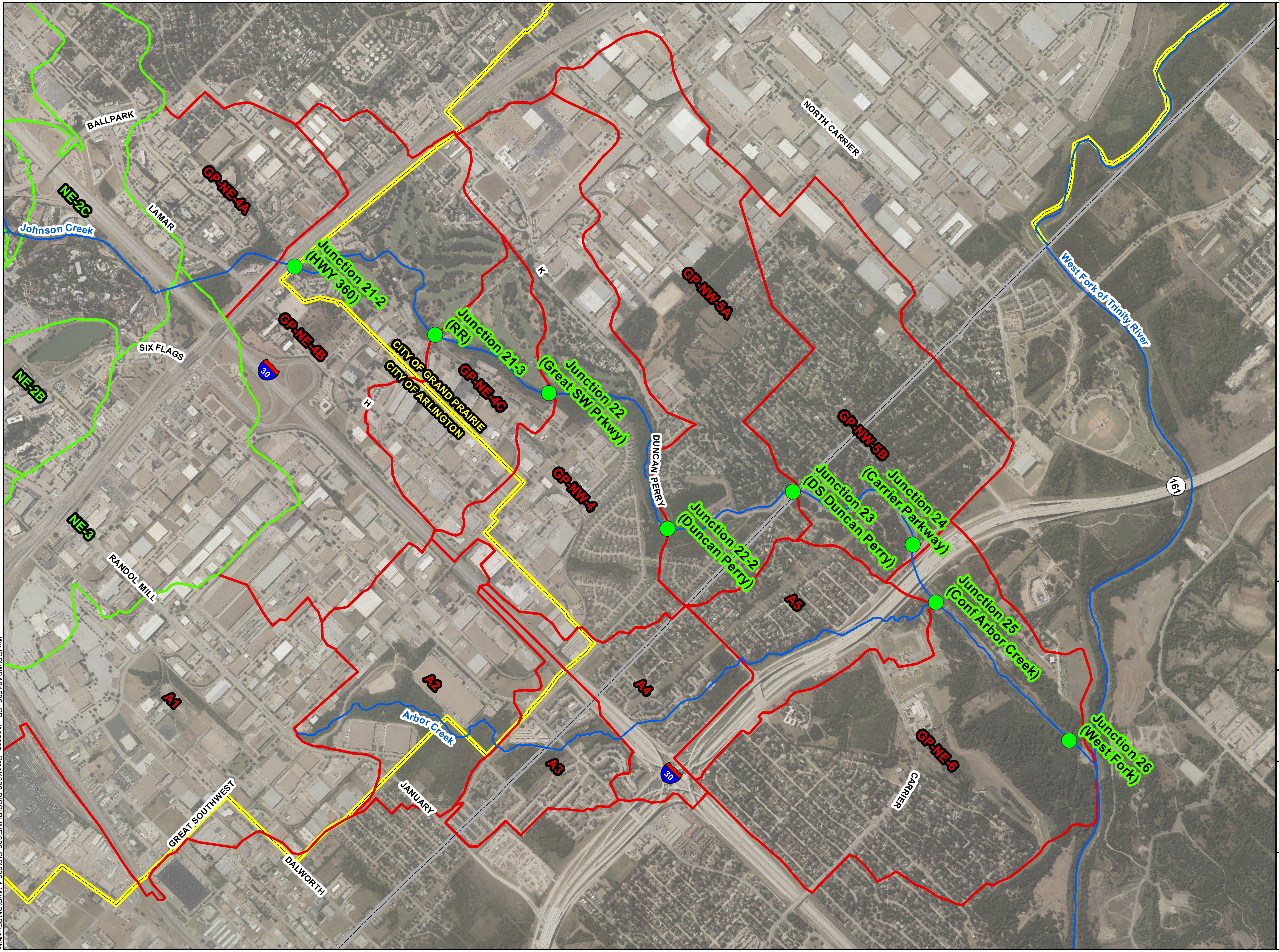



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**Figure IV-1**

**Johnson Creek Hydrologic Junctions Map**

**Legend**

- Hydrologic Junctions
- Stream Centerline
- Arlington Catchment Areas
- Grand Prairie Catchment Areas
- City Limits
- County Boundary

0 1,500 3,000

Scale in Feet

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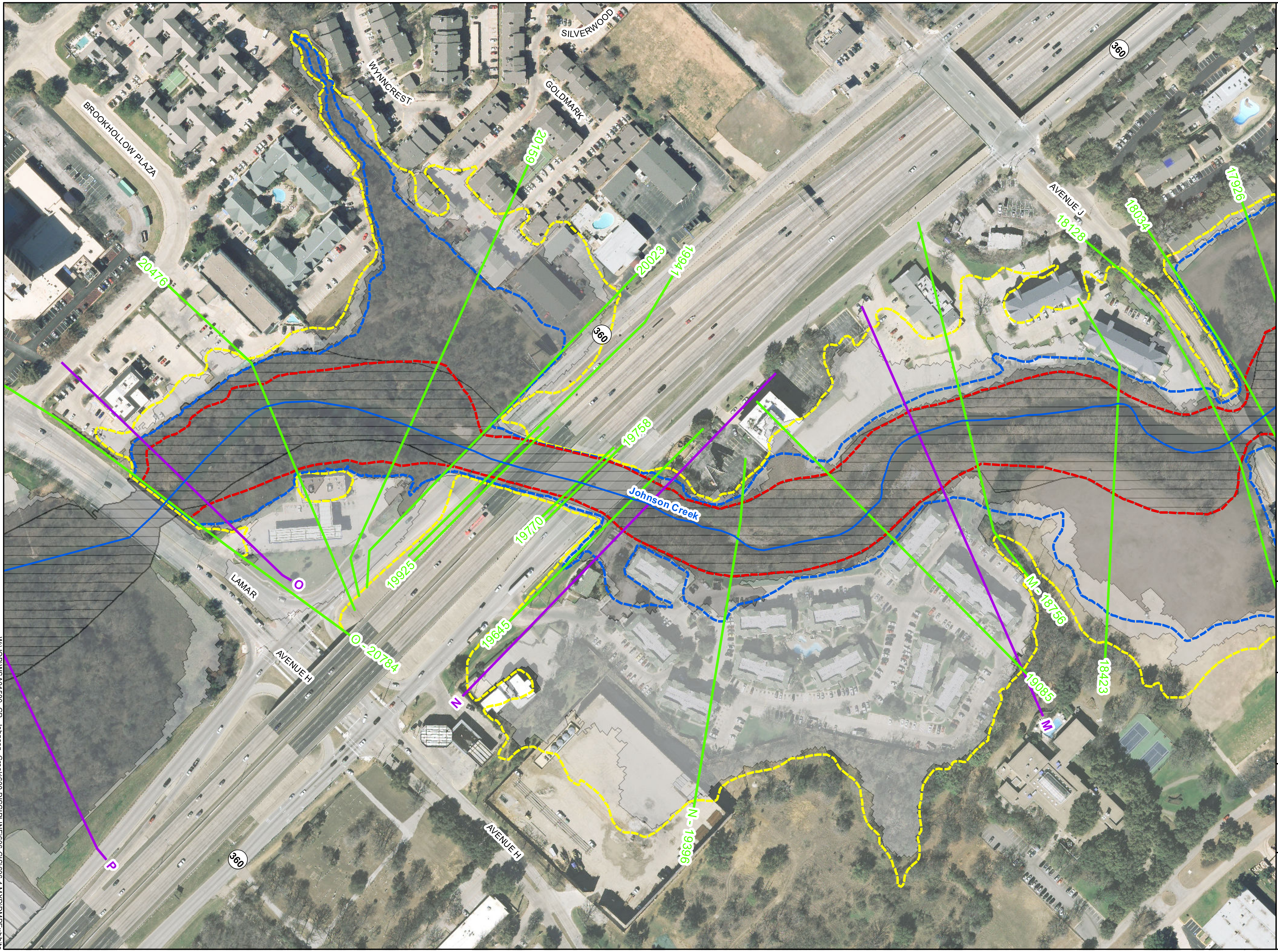



# Figure IV-2

## Johnson Creek Floodzone and Floodway Comparison Map (1 of 8)

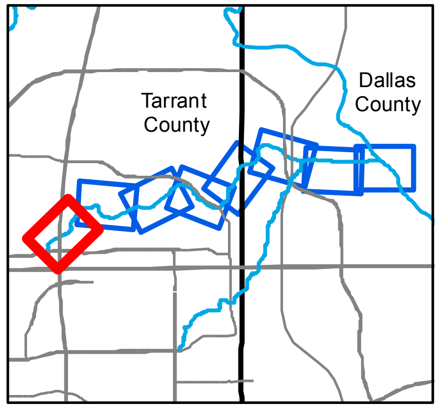
**Legend**

- Stream Centerline
- Cross Sections
- FEMA Cross Sections
- County Boundary
- Revised Floodway
- Revised 1% AC Floodplain
- Revised 0.2% AC Floodplain

**FEMA DFIRM**

- Floodway
- 1% AC Floodplain
- 0.2% AC Floodplain

AC = Annual Chance




0 200 400

Scale in Feet

JACOBS

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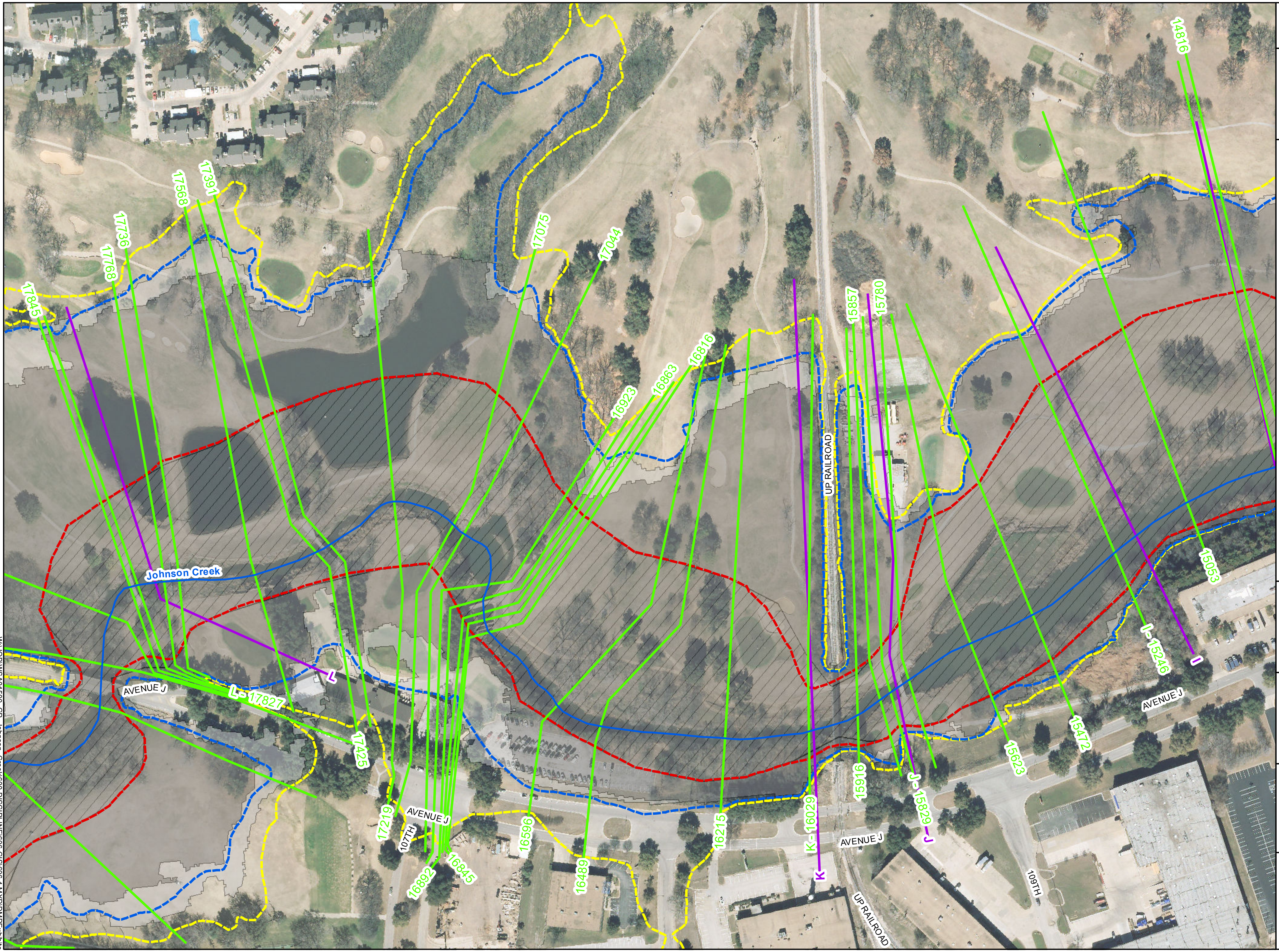
DMP for Johnson Creek Corridor




# Figure IV-2

## Johnson Creek Floodzone and Floodway Comparison Map (2 of 8)

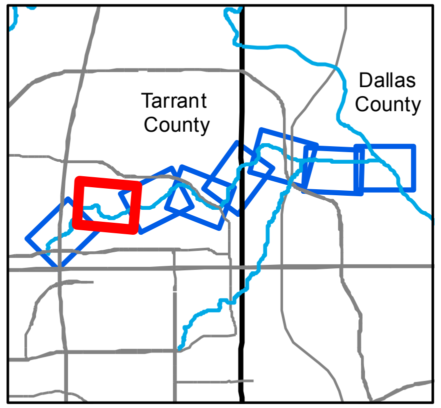
**Legend**

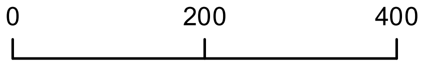
- Stream Centerline
- Cross Sections
- FEMA Cross Sections
- County Boundary
- Revised Floodway
- Revised 1% AC Floodplain
- Revised 0.2% AC Floodplain

**FEMA DFIRM**

- Floodway
- 1% AC Floodplain
- 0.2% AC Floodplain

AC = Annual Chance

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# Figure IV-2

## Johnson Creek Floodzone and Floodway Comparison Map (3 of 8)

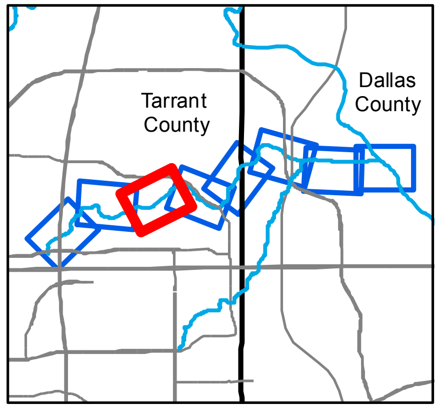
**Legend**

- Stream Centerline
- Cross Sections
- FEMA Cross Sections
- County Boundary
- Revised Floodway
- Revised 1% AC Floodplain
- Revised 0.2% AC Floodplain

**FEMA DFIRM**

- Floodway
- 1% AC Floodplain
- 0.2% AC Floodplain

AC = Annual Chance




0 200 400

Scale in Feet

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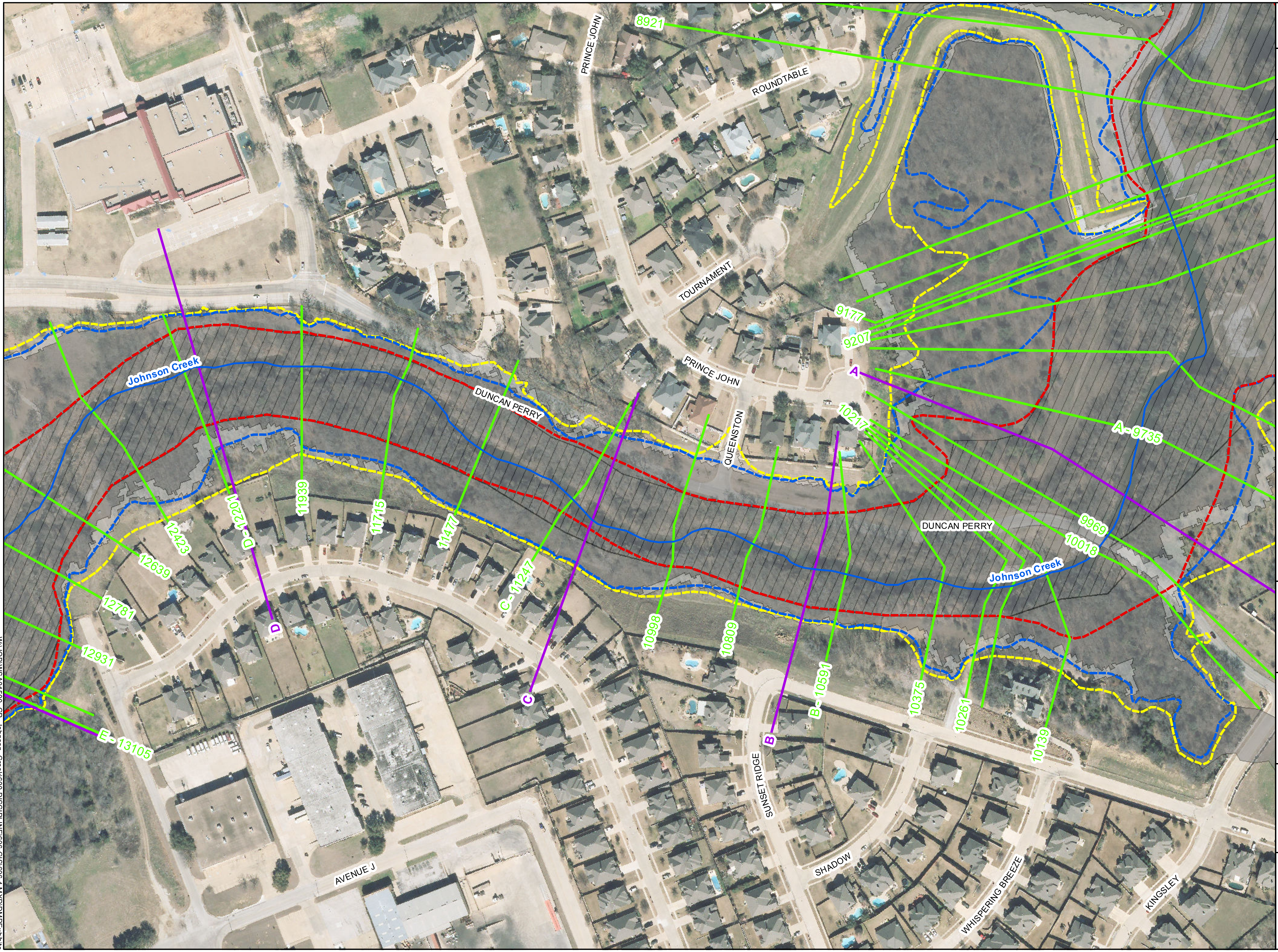
# Figure IV-2

## Johnson Creek Floodzone and Floodway Comparison Map (4 of 8)



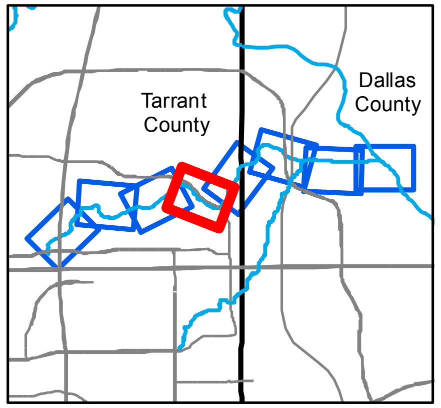

**Legend**

- Stream Centerline
- Cross Sections
- FEMA Cross Sections
- County Boundary
- Revised Floodway
- Revised 1% AC Floodplain
- Revised 0.2% AC Floodplain

**FEMA DFIRM**

- Floodway
- 1% AC Floodplain
- 0.2% AC Floodplain

AC = Annual Chance

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Scale in Feet

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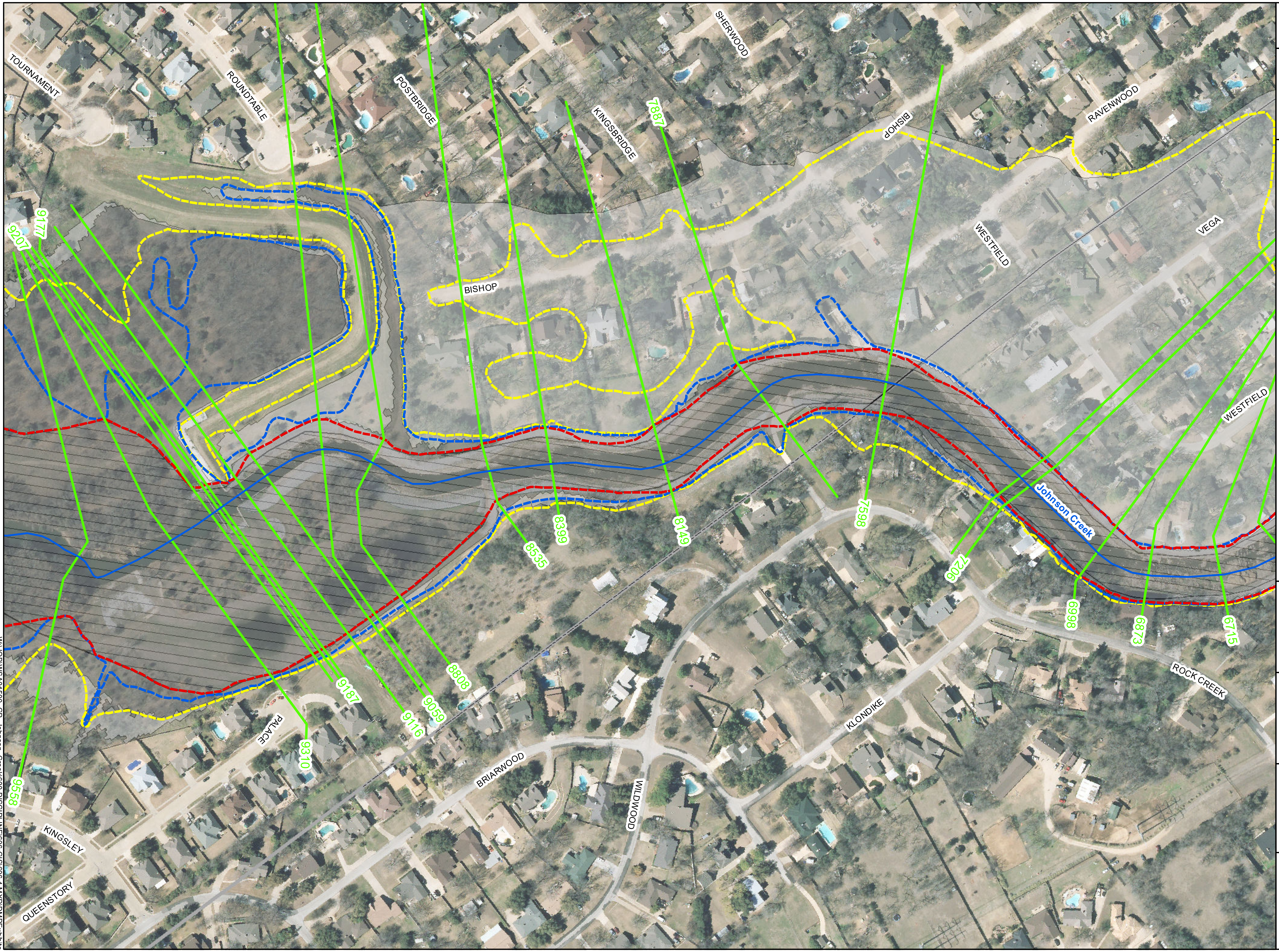


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# Figure IV-2

## Johnson Creek Floodzone and Floodway Comparison Map (5 of 8)

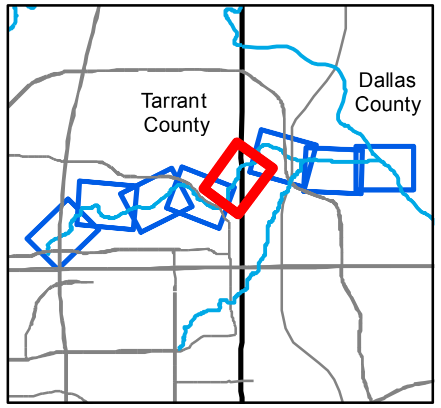
**Legend**

- Stream Centerline
- Cross Sections
- FEMA Cross Sections
- County Boundary
- Revised Floodway
- Revised 1% AC Floodplain
- Revised 0.2% AC Floodplain

**FEMA DFIRM**

- Floodway
- 1% AC Floodplain
- 0.2% AC Floodplain

AC = Annual Chance




0 200 400

Scale in Feet

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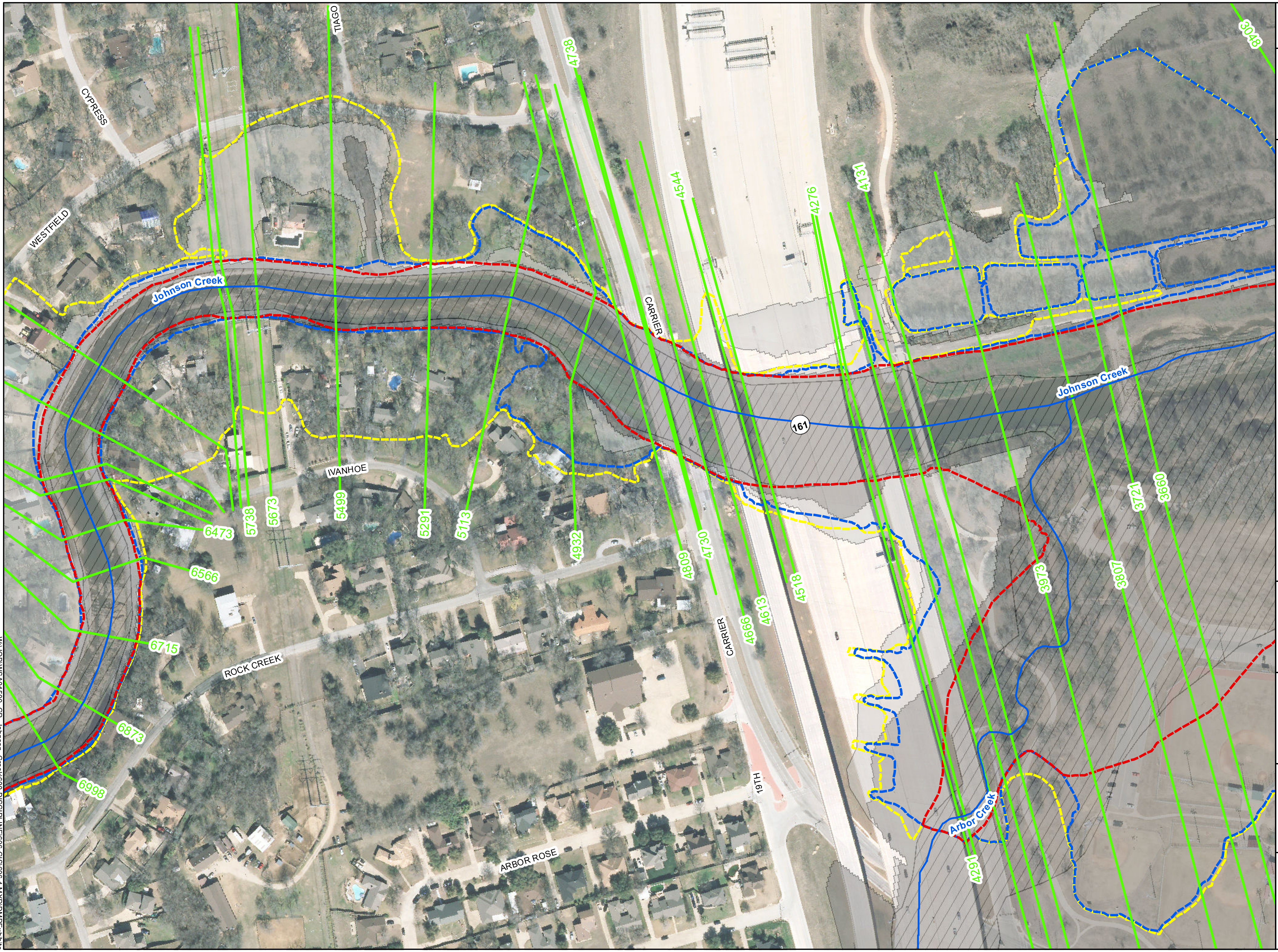



# Figure IV-2

## Johnson Creek Floodzone and Floodway Comparison Map (6 of 8)

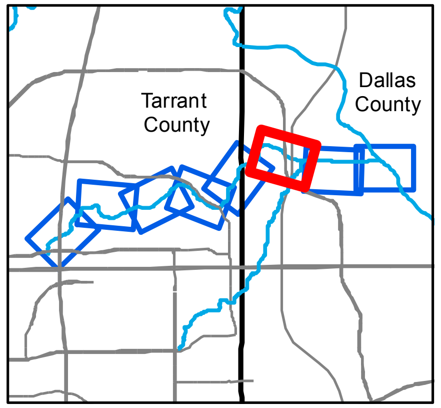
**Legend**

- Stream Centerline
- Cross Sections
- FEMA Cross Sections
- County Boundary
- Revised Floodway
- Revised 1% AC Floodplain
- Revised 0.2% AC Floodplain

**FEMA DFIRM**

- Floodway
- 1% AC Floodplain
- 0.2% AC Floodplain

AC = Annual Chance




0 200 400
Scale in Feet

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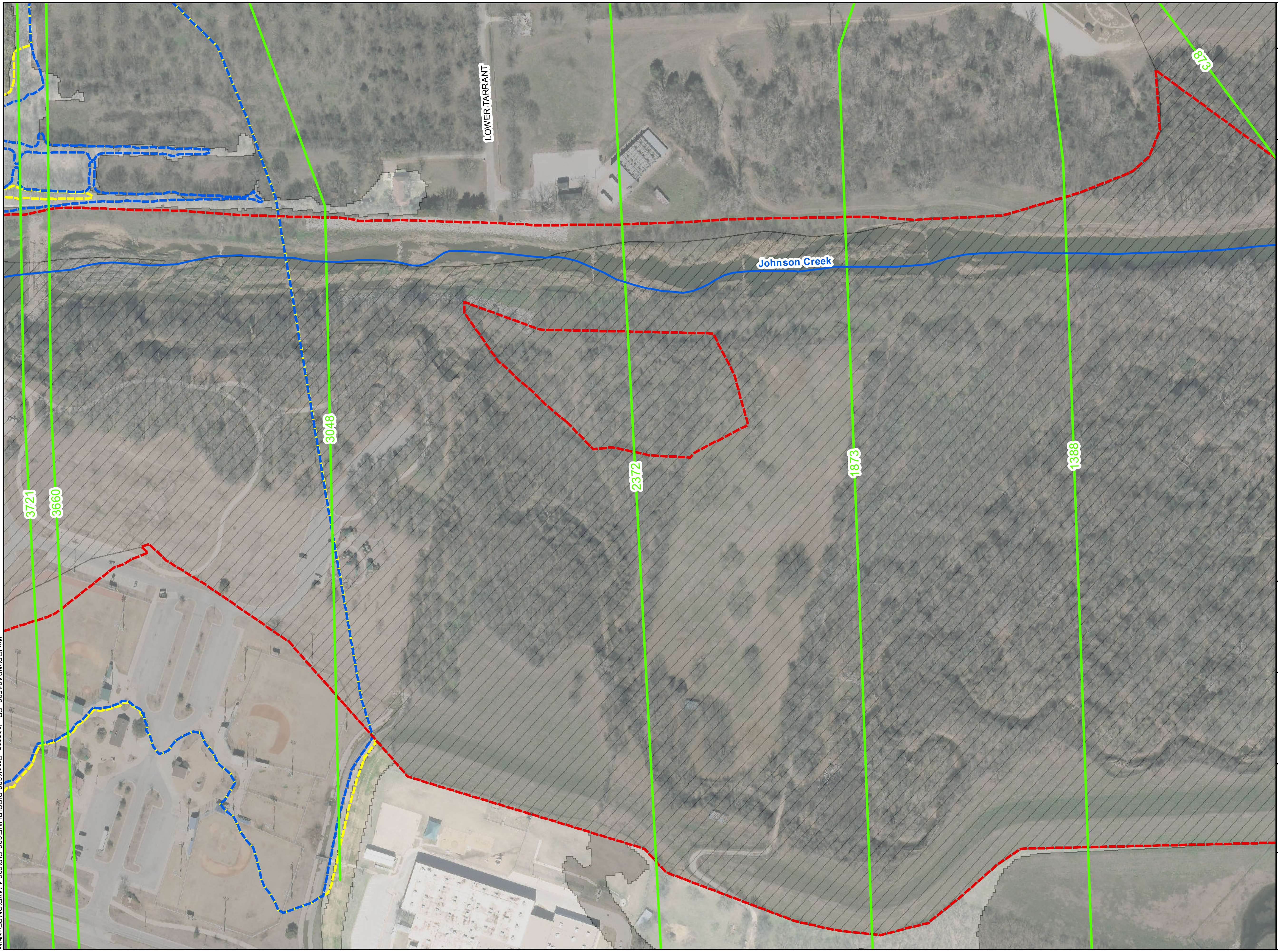
Johnson Creek Corridor




# Figure IV-2

## Johnson Creek Floodzone and Floodway Comparison Map (7 of 8)

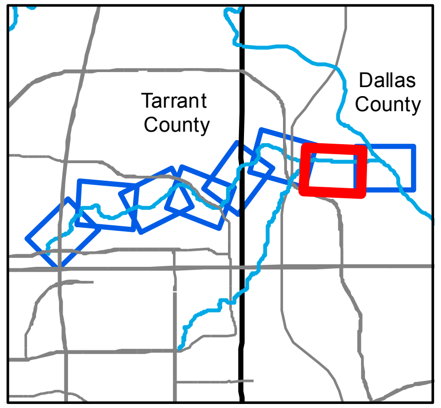
**Legend**

- Stream Centerline
- Cross Sections
- FEMA Cross Sections
- County Boundary
- Revised Floodway
- Revised 1% AC Floodplain
- Revised 0.2% AC Floodplain

**FEMA DFIRM**

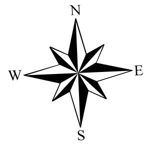
- Floodway
- 1% AC Floodplain
- 0.2% AC Floodplain

AC = Annual Chance




0 200 400

Scale in Feet

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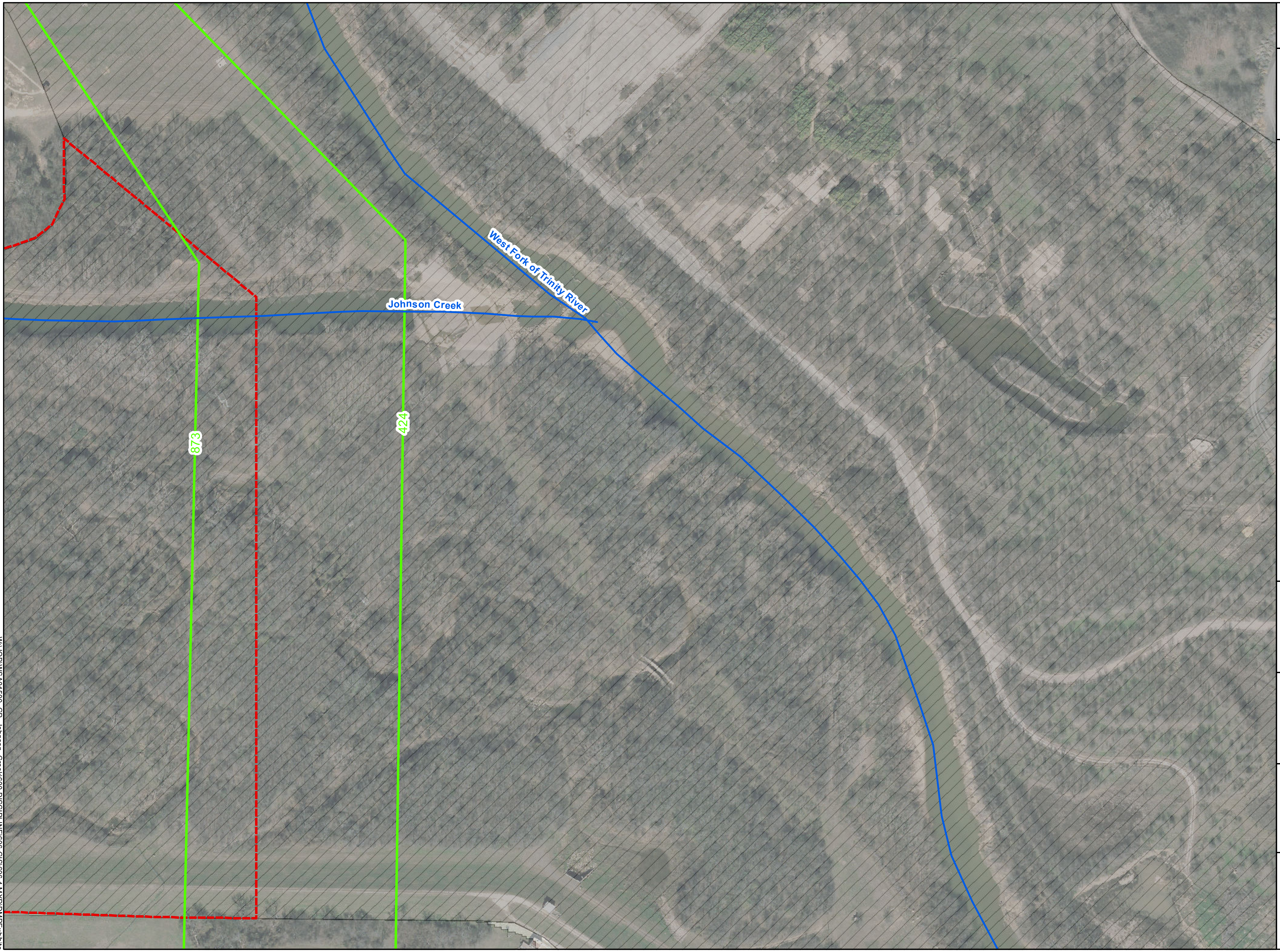



# Figure IV-2

## Johnson Creek Floodzone and Floodway Comparison Map (8 of 8)

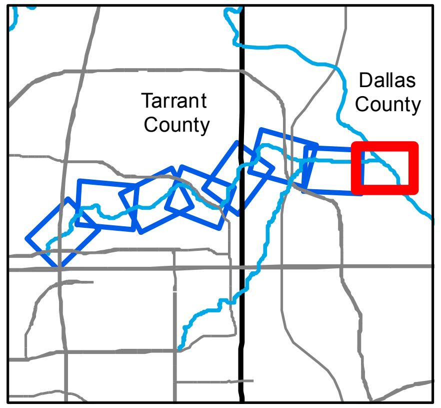
**Legend**

- Stream Centerline
- Cross Sections
- FEMA Cross Sections
- County Boundary
- Revised Floodway
- Revised 1% AC Floodplain
- Revised 0.2% AC Floodplain

**FEMA DFIRM**

- Floodway
- 1% AC Floodplain
- 0.2% AC Floodplain

AC = Annual Chance




0 200 400
Scale in Feet

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# V. FLOODPLAIN MAPPING

## A. General

As part of the Drainage Master Plan for Johnson Creek Corridor, Jacobs remapped the floodplain of Johnson Creek using topographic data obtained by the City of Grand Prairie in 2009 developed using LiDAR technology. Mapping includes delineations for the floodway and the fully developed floodplains resulting from the 1% annual chance and 0.2% annual chance storm events. The effective FEMA flood zone designation for Johnson Creek is Zone AE and includes floodways. The flood zone will remain designated as a Zone AE and will include modifications to both the floodway and floodplain. **Figure V-1** illustrates the detailed floodplain delineation, floodway, base flood elevations, and model cross section locations.

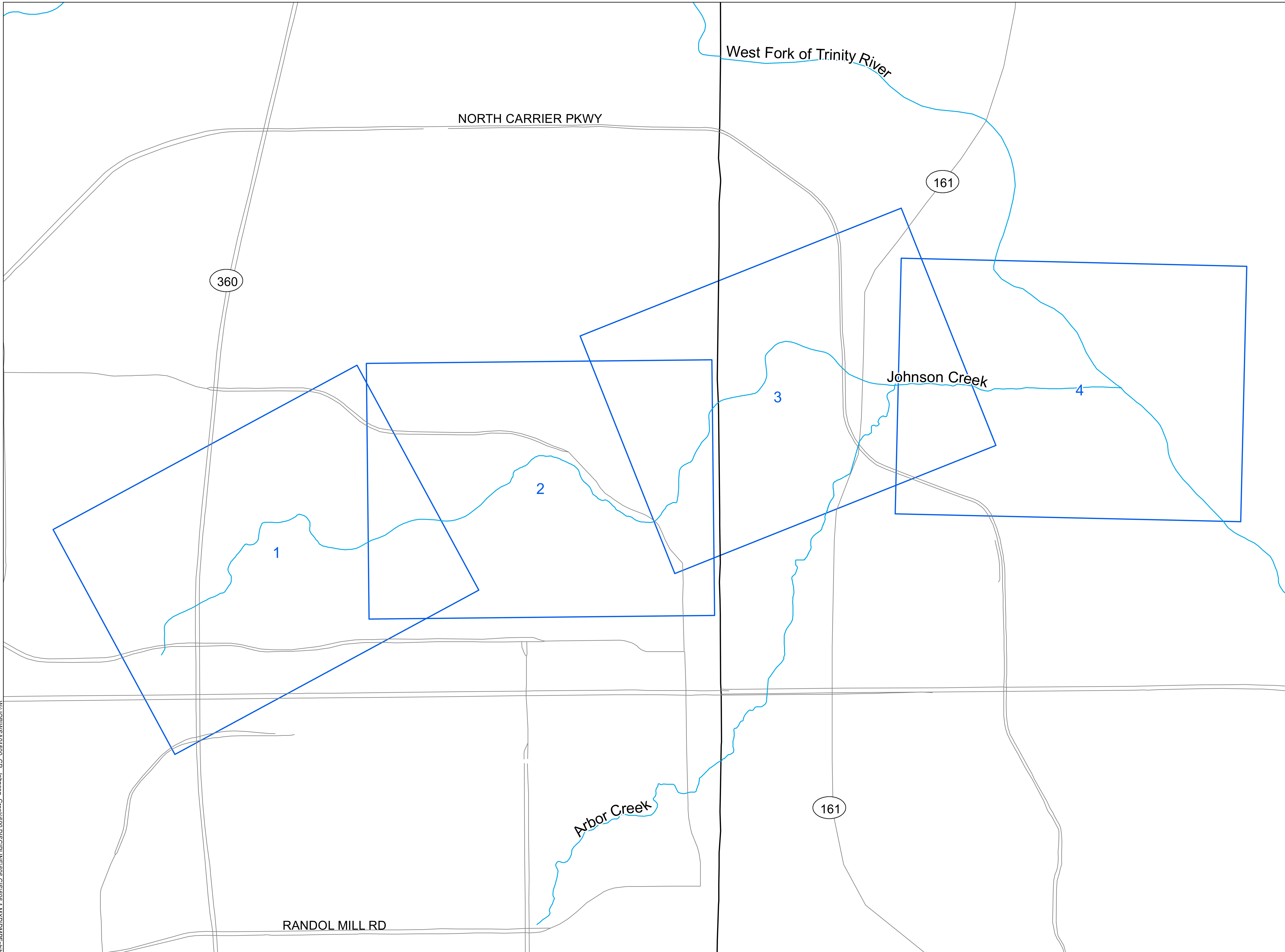
Floodplain delineations and other mapping and model shapefiles are included in the CD-ROM in **Appendix E**.

## B. FEMA Map Revisions

A separate study and report has been prepared providing the hydrologic modeling, hydraulic modeling, floodplain mapping, Digital Flood Insurance Rate Map (DFIRM) production, and revised Flood Insurance Study (FIS) for FEMA. This study is conducted by the City of Grand Prairie under the Cooperating Technical Partners (CTP) Program MAS agreement #3. Existing conditions models created for and utilized in both the CTP study and this DMP are identical. The affected FEMA map panels that include Johnson Creek will be revised with the CTP process and have not been approved as a part of this DMP.

# Figure V-I

## Johnson Creek Floodplain Delineation Index Map

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## Legend

Stream Centerline

Map Index Grid

County Boundary

0 1,500 3,000

Scale in Feet

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# Figure V-I

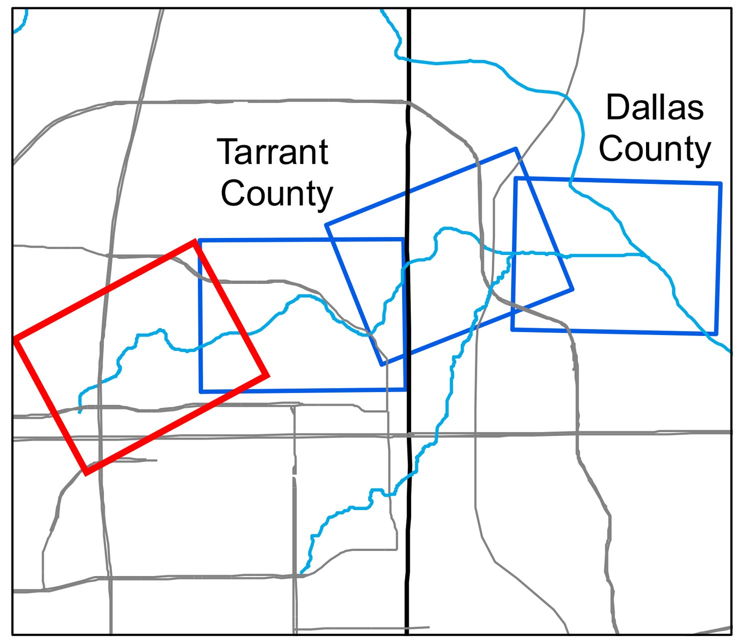
## Johnson Creek Floodplain Delineation Map

(1 of 4)

### Legend

- Stream Centerline
- Cross Sections
- Base Flood Elevation
- County Boundary
- Map Index Grid
- Revised Floodway
- Revised 1% AC Floodplain
- Revised 0.2% AC Floodplain

AC = Annual Chance




0 200 400 800

Scale in Feet

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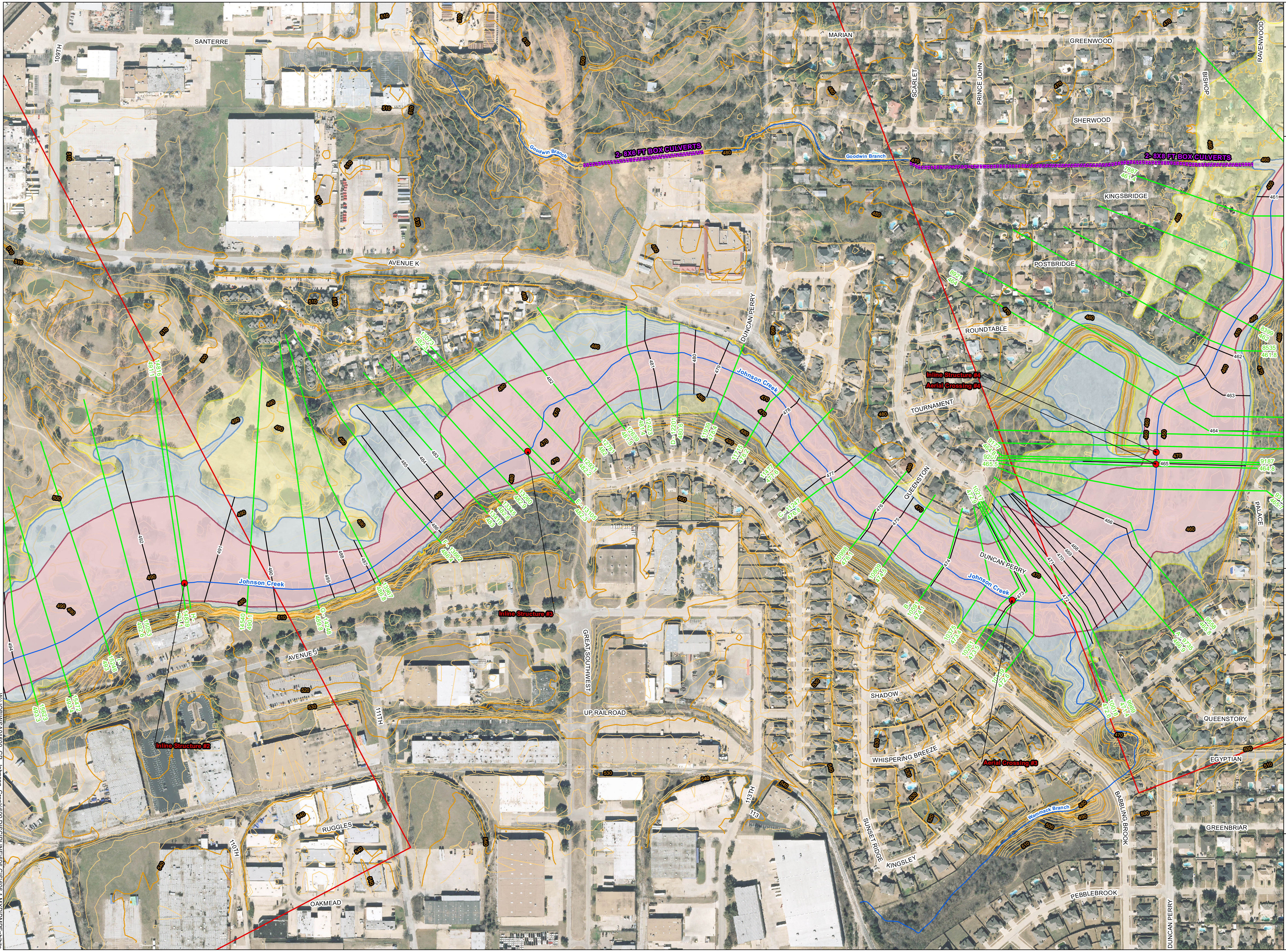


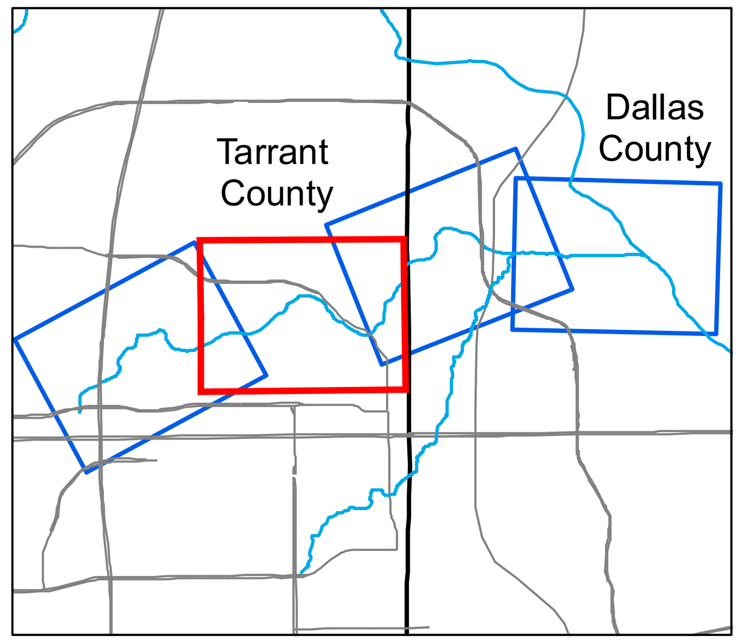

# Figure V-I

## Johnson Creek Floodplain Delineation Map (2 of 4)

### Legend

- Stream Centerline
- Cross Sections
- Base Flood Elevation
- County Boundary
- Map Index Grid
- Revised Floodway
- Revised 1% AC Floodplain
- Revised 0.2% AC Floodplain

AC = Annual Chance




0 200 400 800

Scale in Feet

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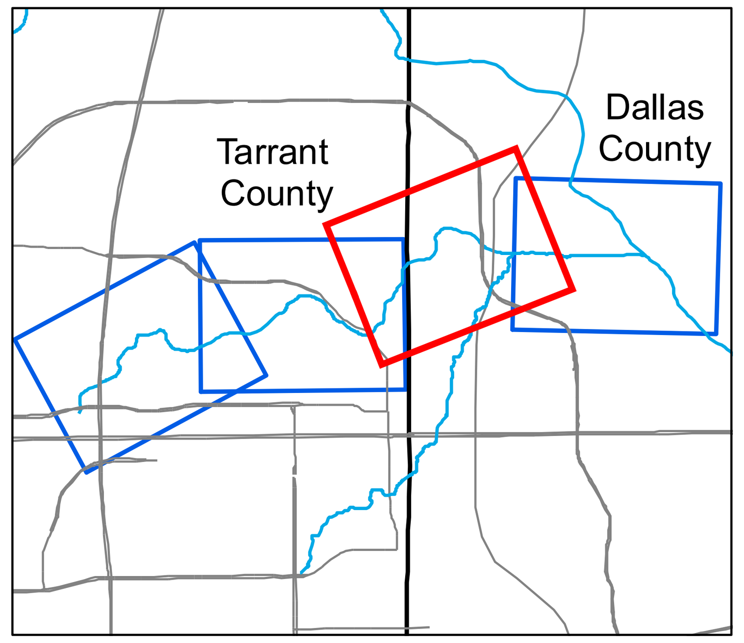
# Figure V-I

## Johnson Creek Floodplain Delineation Map (3 of 4)

### Legend

- Stream Centerline
- Cross Sections
- Base Flood Elevation
- County Boundary
- Map Index Grid
- Revised Floodway
- Revised 1% AC Floodplain
- Revised 0.2% AC Floodplain

AC = Annual Chance




0 200 400 800

Scale in Feet

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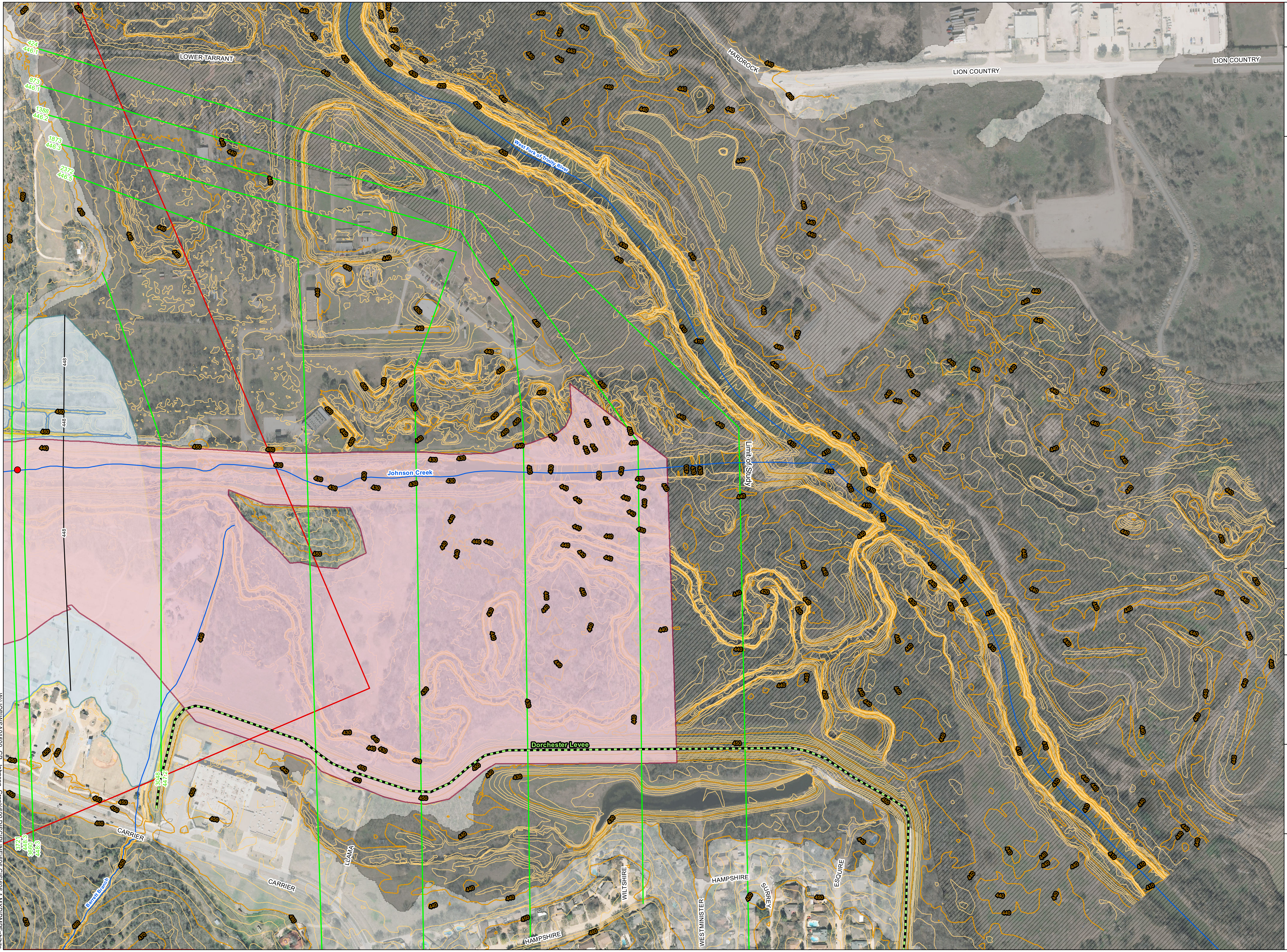


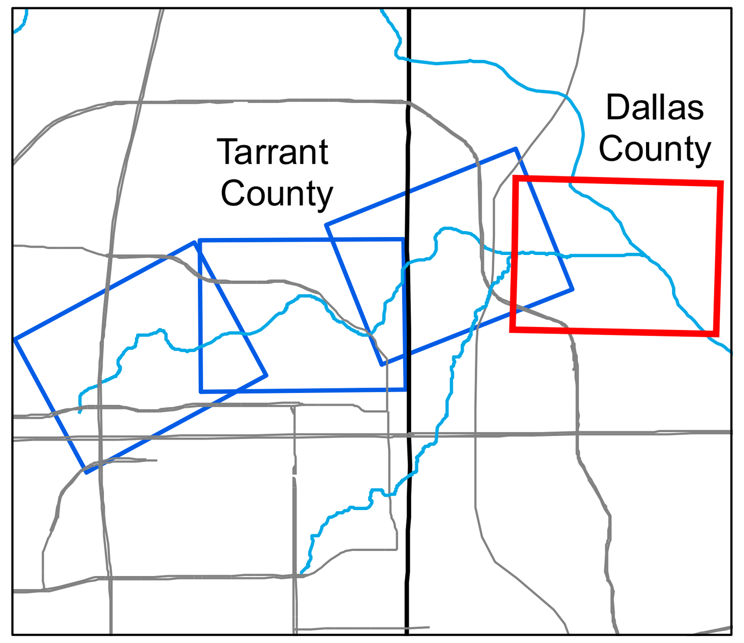

# Figure V-I

## Johnson Creek Floodplain Delineation Map (4 of 4)

**Legend**

- Stream Centerline
- Cross Sections
- Base Flood Elevation
- County Boundary
- Map Index Grid
- Revised Floodway
- Revised 1% AC Floodplain
- Revised 0.2% AC Floodplain

AC = Annual Chance




0 200 400 800

Scale in Feet

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# VI. ROADWAY CROSSINGS

## A. Evaluation of Existing Roadway Crossings

Existing roadway crossings of Johnson Creek have been evaluated to determine the potential of overtopping of flood water during 50%, 10%, 2% or 1% annual chance storm events under fully developed watershed conditions. **Table VI-1** lists the road crossings, associated river station, minimum top of the roadway elevations and peak water surface elevations (WSEL) for various frequency-based design floods. The WSELs are reported at the upstream face of the structure from output files in **Appendix B**. Velocities are also located in this output data. Duncan Perry roadway crossing was shaded to show which storm events are overtopped.

**Table VI-1**
**Existing Roadway Crossing Summary**

| River Station | Roadway Crossing | Min. Top of Road Elev. | 50% Event WSEL | 10% Event WSEL | 2% Event WSEL | 1% Event WSEL |
|---|---|---|---|---|---|---|
| 19700 | SH 360 Frontage Road NB | 516.40 | No 505.56 | No 509.09 | No 511.50 | No 512.23 |
| 18080 | Avenue J | 511.00 | No 502.38 | No 505.39 | No 507.66 | No 508.49 |
| 17835 | Golf Course Bridge #1 | 501.40 | No 501.01 | Yes 503.77 | Yes 506.30 | Yes 507.10 |
| 17407 | Golf Course Bridge #2 | 499.76 | Yes 500.33 | Yes 503.64 | Yes 506.20 | Yes 507.02 |
| 16876 | Golf Course Bridge #3 | 494.80 | Yes 499.24 | Yes 502.84 | Yes 505.66 | Yes 506.52 |
| 15972 | Union Pacific (Great Southwest) Railroad | 509.73 | No 498.66 | No 502.24 | No 504.99 | No 505.85 |
| 15804 | Golf Course Bridge #4 | 491.14 | Yes 494.85 | Yes 496.26 | Yes 497.52 | Yes 497.87 |
| 9992 | Duncan Perry Road | 470.56 | No 465.33 | No 470.16 | Yes 472.16 | Yes 472.52 |
| 4773 | N. Carrier Parkway SB | 458.32 | No 446.04 | No 449.45 | No 451.40 | No 451.90 |
| 4698 | N. Carrier Parkway NB | 459.19 | No 445.60 | No 448.98 | No 450.91 | No 451.41 |
| 4578 | PGBT (SH161) SB FR | 478.50 | No 445.38 | No 448.71 | No 450.60 | No 451.08 |
| 4404 | PGBT (SH161) Main Lanes | 470.50 | No 445.19 | No 448.45 | No 450.31 | No 450.79 |
| 4245 | PGBT (SH161) NB FR | 458.00 | No 444.41 | No 447.68 | No 449.59 | No 450.07 |
| 4151 | Good Link Bike Trail | 454.70 | No 443.88 | No 447.08 | No 449.00 | No 449.47 |

"Yes" indicates top of road is lower than the calculated water surface elevation, therefore overtopping i.e, flooding occurs. "No" indicates top of road is higher than the calculated water surface elevation, therefore no overtopping i.e, flooding occurs.

Pedestrian bridges are included in the table for evaluation; none call for a significant reason to upgrade the structure at these locations. The present analysis shows only Duncan Perry Road would be overtopped by floods resulting from the less frequent 2-, 1-, and 0.2-percent annual chance of occurrence storm events. For vehicular and pedestrian safety, consideration should be given to raising Duncan Perry Road and increasing the conveyance capacity of the crossing. **Table VI-2** shows the improvement alternative that provides the level of service necessary for protection against the 1% annual chance storm events. **Section VII** provides more detailed descriptions of the proposed (conceptual) road crossing improvement.

**Table VI-2**
**Existing Roadway and Proposed Alternatives**

| Roadway Name | 1% Annual Chance Flow cfs | Existing Crossing Description | Minimum Top of Road Elevation | | Proposed Improvement | 1% Annual Chance Existing WSEL ft | 1% Annual Chance Proposed WSEL ft | Difference WSEL |
|---|---|---|---|---|---|---|---|---|
| | | | Existing | Concept Analysis | | | | |
| Duncan-Perry Rd | 18233 | Bridge with 90 ft. opening | 470.5 | 472.3 | widening the bridge by 100 ft. on the right side and raising the bridge deck by 1.80 ft and adjusting the approach | 471.29 | 467.87 | 3.42 |

## B. Evaluation of Proposed and Future Roadway Crossings

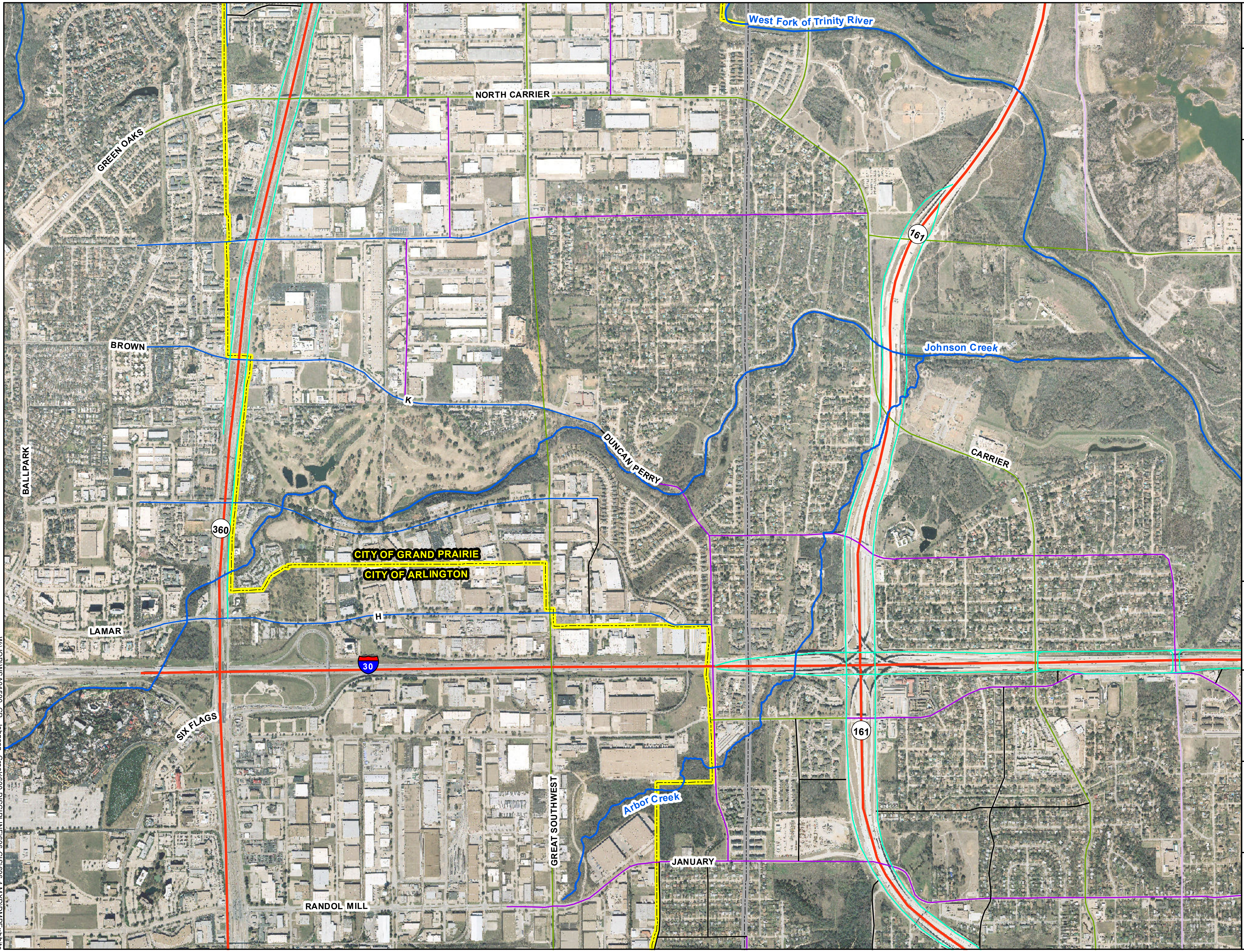
The City of Grand Prairie's Master Thoroughfare Plan indicates that new or improved roads are planned in Johnson Creek watershed. Most of these are associated with the SH-161 project and have been completed. Of the planned road improvements that cross Johnson Creek, the Great Southwest Parkway extension has not yet been designed or gone through the planning process. The planned improvements have not determined the extents of the roadway alignment. **Figure VI-1** shows the Master Thoroughfare Plan roadway within Johnson Creek watershed showing existing and future crossings over Johnson Creek. TXDOT is under design for future improvements to State Highway 360.

When possible, future thoroughfare crossings, or improved crossings, should be designed to pass 1% annual chance storm event under fully developed conditions without creating adverse impacts on the upstream or downstream properties. It is desirable for these roadway crossings to span the entire 1% annual chance floodplain; however, in most developed floodplains, such as this one, it is not typically feasible or even economically beneficial as there is already substantial encroachment into the natural floodplain by the existing crossings and adjacent properties.

As discussed in the previous section, Duncan Perry Road is the only roadway crossing on Johnson Creek that is not adequately sized, at present, to pass flood flows resulting from a storm event with 1% annual exceedance probability without overtopping the roadway. The Duncan Perry Road improvement that is needed to mitigate this inadequacy is discussed in the previous section.

City staff has indicated that Great Southwest Parkway road extension is not likely to be designed in the foreseeable future; therefore, no consideration has been given to the effects of this future roadway on the floodplain or floodway at this time. The current alignment shown on the Master Thoroughfare Plan is likely to not be accepted by current property owners. The current alignment would require a large bridge design that would not impact the existing floodplain and additional storage grading to maintain existing discharges in the creek. Further investigation is recommended as the future design and location of the roadway is approved.

State Highway 360 is under design by TXDOT with URS Corporation (URS). The current status is the design will detain any additional discharges that will be allowed through the larger bridge opening. The design engineer is provided the current watershed analysis created from the CTP submittal to FEMA as discussed in Section II of this report. The City of Grand Prairie should remain in communication with the future design and construction of SH 360.




# Figure VI-1

## Johnson Creek Master Thoroughfare Map

**Legend**

- City Limits
- County Boundary

**2010 Master Thoroughfare Plan**

**Street Classification**

- HWY
- P7U
- P6D
- P4D
- P3U (one-way)
- M5U
- M4U
- M3U (two-way)
- C2U

0 1,500 3,000

Scale in Feet

N E S W

JACOBS

Grand Prairie TEXAS

DMP for Johnson Creek Corridor

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# VII. ALTERNATIVES FOR STREAMS AND OPEN CHANNELS

## A. Summary of Flooding Issues

1. Known Flooding Issues

A drainage complaint database has been developed by the City of Grand Prairie. This database contains no recent riverine flooding issues within the Johnson Creek corridor. The drainage complaint database has been reviewed. One-hundred and sixty-six (166) drainage complaints at one-hundred and six (106) different locations have been filed with the City from 1971 to 2012 within Johnson Creek watershed. Of these complaints, fifteen (15) are erosion problems, sixteen (16) are street flooding problems, fifty-nine (59) are property flooding problems, and seventeen (17) are structure flooding problems. Three (3) complaints have been filed since April 2013 at Nottingham Place, Klondike St, and Hyatt Apartments.

2. Roadway Flooding Issues

As discussed in Section VI, Duncan Perry Road is the only road that would be overtopped in case of flood flows resulting from a 1% annual exceedance storm event. Alternatives for improving this crossing are discussed in Section B of this Section.

3. Structure Flooding Issues

To evaluate the potential for flooding of structures, the floodplain is evaluated against available structure data. The City of Grand Prairie has provided LiDAR derived topographic data collected in 2009 that includes building outlines. These building outlines are overlain on the floodplain resulting from the 1% annual chance storm event to identify buildings that may potentially have finished floor elevations below the BFE.

In this manner, a total of eight structures have been identified near the floodplain; of these structures, seven are not considered within the revised floodplain as mapped in this study. One structure is a non-habitable structure. **Figure VII-1** displays these structures with the 2013 Aerial Photo. The City plans to provide elevation certificates for each of the habitable structures identified on Figure VII-1. These BFEs and updated floodplain mapping will become effective once FEMA approves the floodplain maps developed under the current CTP study.

No habitable structures have been determined to be within the floodplain mapped from 100-year storm event. There has been no reported flooding problem with these structures, but only reports of flooding in the yards and nearby areas of these structures. Therefore, further consideration has not been given to any flood reduction measures or buy-out alternatives for these structures.

## B. Alternatives Analysis

Conceptual alternatives are considered for only one existing roadway crossing that appears to have a reasonable likelihood of being impacted by flooding. These concept alternatives are discussed below. Some of these alternatives are introduced in Section VI and preliminary opinion of probable construction costs for each flood control alternative can be found in Section XII of this report. Refer to **Table VI-2** for a summary of proposed bridge crossing improvements. Total annual costs, including construction and design, are based on a 50-year project life and a 7% discount rate.

Any improvement in the FEMA floodway that cause an increase in the BFE will require a Conditional Letter of Map Revision (CLOMR) from FEMA. Before approving the CLOMR, FEMA will want to see the following information:

- An evaluation of alternatives that would not result in a BFE increase above that permitted and a demonstration of why these alternatives are not feasible;
- Notification of affected property owners explaining the impact of the proposed project on their property;
- Concurrence of the Chief Executive Officer (CEO) and any other communities affected by the proposed actions; and
- Certification that no structures are located in areas that would be impacted by any increased base flood elevations.

Johnson Creek and any adjacent wetlands would be considered waters of the United States; therefore, any construction that impacts the channel and associated wetlands would require permitting through the U.S. Army Corps of Engineers (USACE) under Section 404 of the Clean Water Act. Depending on the nature of the improvements, such as bridge improvements can typically be permitted under Nationwide Permit 14 (NWP 14) for "Linear Transportation Crossings" to satisfy the USACE requirements. Other improvements such as streambank armoring, maintenance of existing structures and aquatic habitat enhancements can be covered under nationwide permits. A delineation of waters of the US is required to assess the feasibility of claiming a particular Nationwide Permit.

1. Duncan Perry Road (cross section 9992)

The existing bridge along Duncan Perry Road is a two-lane roadway and has a span length of 90 feet. The current bridge opening does not have the capacity to pass flood flows resulting from a 2% annual chance storm event without causing roadway overtopping. The depth of overtopping is about two feet for a flood flow that can result from a 1% annual chance storm event.

As summarized in **Table VI-2**, the alternative was laid out for Duncan Perry Roadway improvements. This alternative includes elevating the roadway deck. This will provided one-foot of freeboard from the low chord of the bridge resulting from a storm event with the 1% annual exceedance probability to comply with current design standards of the City. Raising the roadway deck by 1.80 feet would require about 1100 feet of Duncan Perry Road to be elevated for an existing grade tie-in of the approaches. The elevated roadway profile would remove the existing road from the floodplain. Grading in the overbanks is considered to lower the water surface elevation and compensate for loss of valley storage in this reach. In turn less flooding on the left overbank through this reach of Johnson Creek will be achieved.

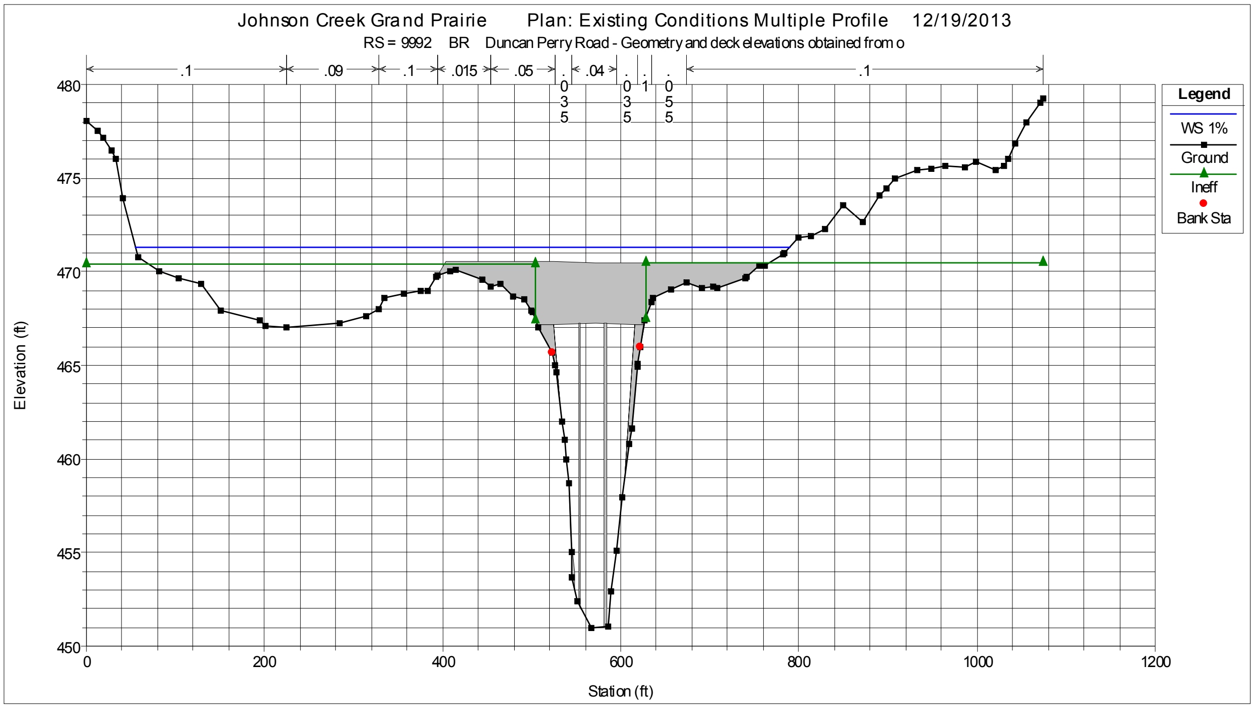
**Figure VII-2** shows the concept of the Duncan Perry Road alternative with the existing and proposed cross section view of the bridge. **Figure VII-3** shows a plan view of the Duncan Perry Road alternative with a conceptual plan with grading areas.

A summary of the opinion of probable construction costs is shown in **Table VII-1**. **Section XII** provides a more detailed breakdown of the probable construction costs. If this alternative is implemented, Duncan Perry Road probably would not be overtopped by a flood event resulting from the 1% annual exceedance probability. These improvements cause an increase in the BFE upstream of Duncan Perry Road if additional storage area is not provided. Because the construction would occur in the floodway, a CLOMR will be required before construction. After construction, a Letter of Map Revision (LOMR) will be necessary to incorporate any revisions to the floodplain and floodway mapping.

In addition this alternative, a flood warning system at Duncan Perry Road has been considered. This would consist of a flashing light that functions on a battery backup powered devise to provide warning when the water rises to a specified elevation. The stage height is usually measured by a transducer that sends the signal to the warning device. Several models can be considered for this location. The flood warning system option is an alternative to the structural improvements considered above.

| **Table VII-1<br>Probable Construction Cost for Proposed Improvements of the Bridge along Duncan Perry Road over Johnson Creek** | |
|---|---|
| Construction Subtotal | $3,449,929 |
| Approximately 25% Contingency | $ 862,482 |
| Construction Total | $4,312,411 |
| Approx. 15% for Engineering and Survey | $ 646,862 |
| **Total** | $4,959,273 |

## Existing Conditions Duncan Perry Road




## Duncan Perry Road Alternative Cross Section View

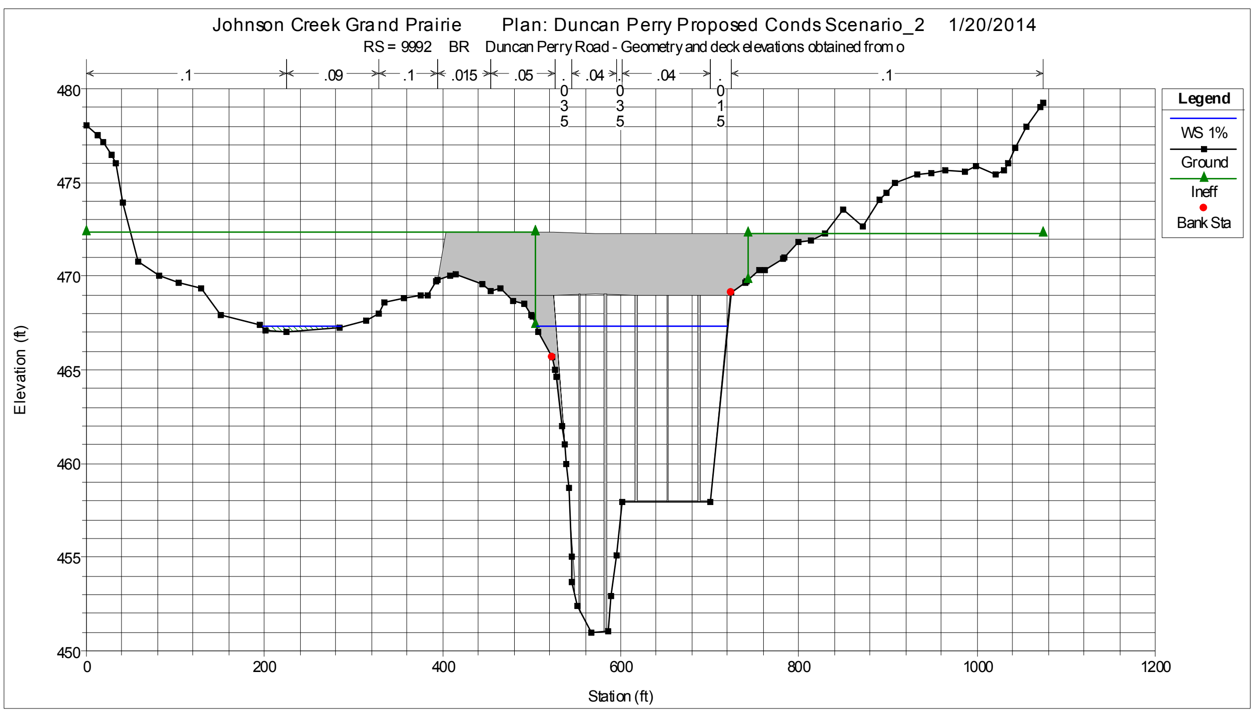
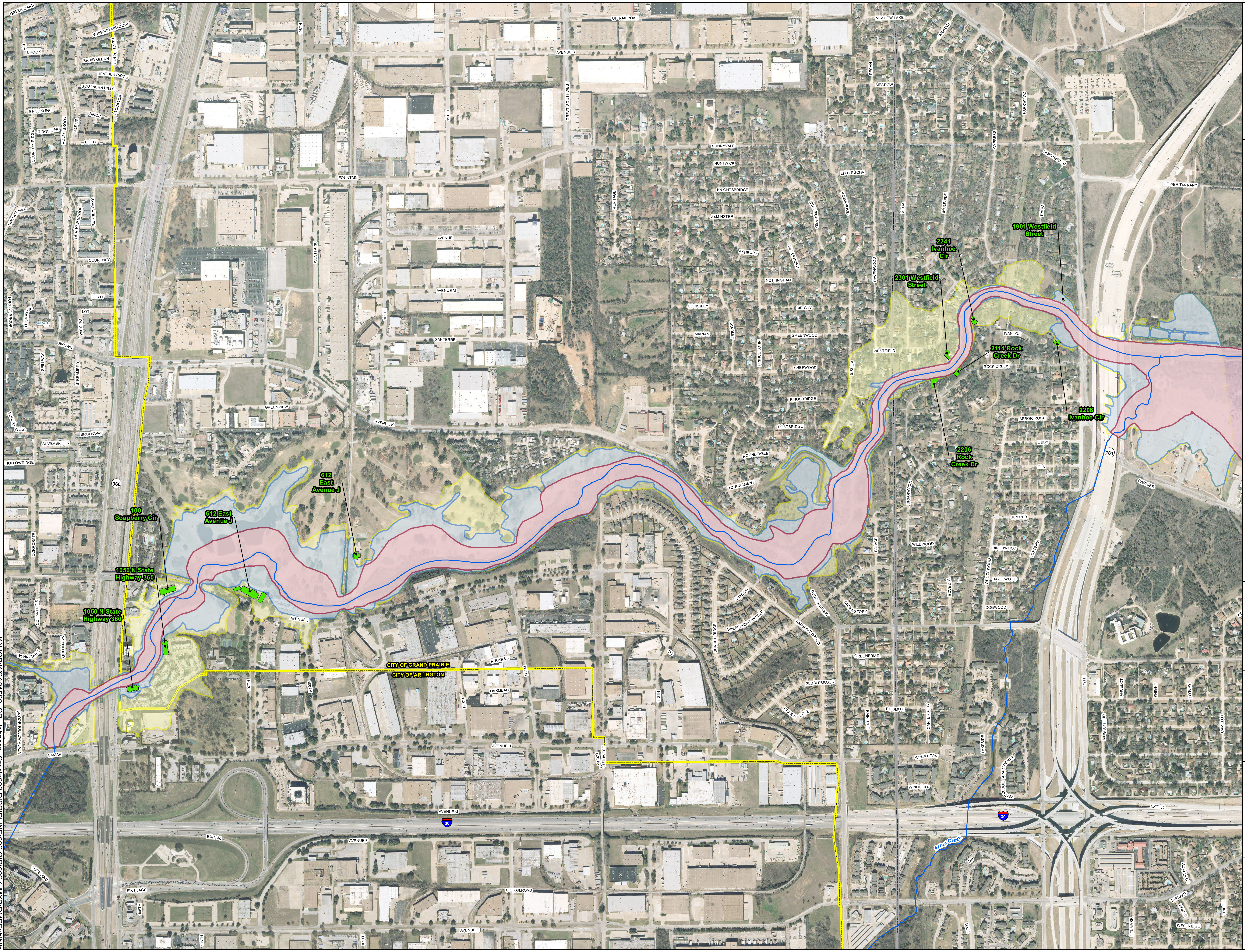



**Figure VII-2. Duncan Perry Cross Sections for the 1% Annual Chance Flood Capacity**




# Figure VII-1

## Johnson Creek Location of Properties Near The Floodplain

**Legend**

- Stream Centerline
- Properties Identified Near Floodplain
- City Limits
- County Boundary
- Revised Floodway
- Revised 1% AC Floodplain
- Revised 0.2% AC Floodplain

AC = Annual Chance

0 500 1,000 2,000

Feet

JACOBS

Grand Prairie TEXAS

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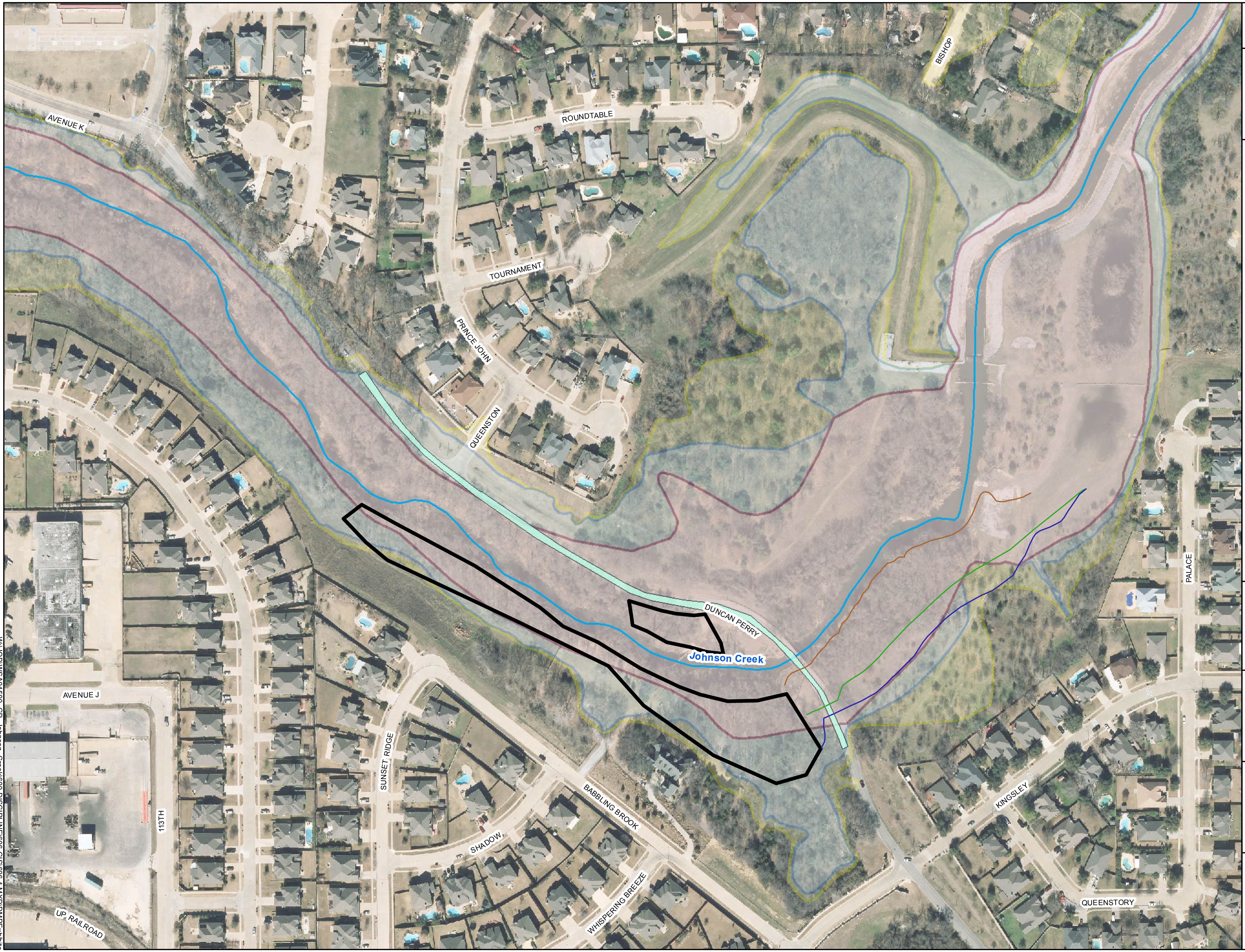
**Figure VII-3**

## Johnson Creek Duncan Perry Road Alternative




### Legend

**Grading Schematic**

- Beginning of the bench
- End of the bench bed, grade break
- Tie-in to the natural ground
- Grading Storage Area
- Roadway In Floodplain
- Revised Floodway
- Revised 1% AC Floodplain
- Revised 0.2% AC Floodplain

AC = Annual Chance

0 200 400
Scale in Feet

N
W E
S

JACOBS

Grand Prairie
TEXAS

DMP for
Johnson Creek Corridor

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# VIII. STORM WATER INFRASTRUCTURE ANALYSIS

Analysis of the storm water infrastructure in Johnson Creek watershed is not included as part of this update of the Drainage Master Plan. This will be provided by others.

# IX. CHANNEL STABILITY ASSESSMENT & EROSION HAZARD ANALYSIS

## A. Introduction

Over the past 60 years, development within Johnson Creek watershed has led to significant instabilities within the main channel in direct response to increased runoff. Also, channelization has greatly reduced the sinuosity index or the deviations from a path defined by the direction of maximum downslope of the creek from 1.89 in 1942 to 1.55 in 2013. In turn, steeper slopes have developed in the main channel forcing the creek to adjust to a renewed state of equilibrium. Highly disturbed streams such as Johnson Creek first undergo degradation of the channel bottom, then widening as the banks erode, and lastly aggradation as the channel attempts to re-stabilize itself. In general, healthy streams are well connected to their floodplains while unstable streams are channelized with limited ability to allow flows into the floodplain. The essence of the study of applied geomorphology is determining how a stream responds to disturbances such as channelization and increased runoff.

This section is divided into three parts. First, the methodologies of the assessments are introduced. Next, the findings of the analysis are discussed, focusing on the current evolutionary state of Johnson Creek and locations of important geomorphologic features observed in the channel, including local areas of concern. Last, recommendations for mitigation of channel instabilities are presented.

## B. Channel Stability Concepts

The geomorphology of Johnson Creek has been assessed in the field by Jacobs Engineering during the months of October and November 2013. This evaluation included a visual assessment of the stream's condition and locating potential areas of concern for erosion and deposition where mitigation may be required to ensure infrastructure along and adjacent to Johnson Creek is not threatened by natural channel processes. Particular emphasis is placed on evaluating the condition of each of the roadway, railroad, and aerial utility crossings to identify evidence of scour that could threaten the future stability of the structures. Also, local failures at and near grade control and inline structures are noted. For clarity of the analysis, Johnson Creek has been divided into five geomorphologic reaches as shown in **Figure IX-1**. Care has been taken to ensure that each reach is bounded on the upstream and downstream ends by hard grade controls, both natural and anthropomorphic. **Table IX-1** describes the reach limits.

**Table IX-1**
**Reach Descriptions**

| Reach Number | U/S Boundary | D/S Boundary |
|---|---|---|
| 1 | SH 360 | Avenue J |
| 2 | Avenue J | Inline Structure #3 |
| 3 | Inline Structure #3 | Inline Structure #4 |
| 4 | Inline Structure #4 | Carrier Parkway |
| 5 | Carrier Parkway | Confluence with West Fork Trinity |

Areas of concern are documented and photographed with field notes to support each observation. Field investigations of the structures are performed using the methodologies outlined in Hydrologic Engineering Circular No. 18, Evaluating Scour at Bridges, Fifth Edition (HEC-18, 2012), Johnson et al. (1999), and Lagasse, et al. (2001). Geomorphologic features are identified using methodologies developed by Leopold, et al. (1964).

**Rapid Channel Assessment**

In an effort to systematically assess the stability of each reach, this study employs the rapid assessment of channel stability as presented by Lagasse et al. (2001). The rapid assessment of stream stability uses 13 differently weighted stability indicators to categorically rate the overall stability of the stream as excellent, good, fair, or poor. Reach by reach, each stability indicator is rated using a similar scoring system with values categorized as excellent, good, fair, or poor with three possible values for each category. The ratings are based on descriptors outlined in **Table IX-2** (Johnson et al., 1999; Lagasse et al., 2001). Once the rating value is assigned, the rating value is weighted using a pre-determined multiplier as shown in **Table IX-3**. Then, the weighted ratings are summed and used to determine the stability rating of the reach as shown in **Table IX-4**.

**Table IX-2**
**Stability Indicators, Ratings, and Descriptions (Johnson et al., 1999; Lagasse et al., 2001)**

| STABILITY INDICATOR | RATINGS | | | |
|---|---|---|---|---|
| | EXCELLENT (1-3) | GOOD (4-6) | FAIR (7-9) | POOR (10-12) |
| 1. Bank soil texture and coherence | Clay and silty clay; cohesive material | Clay loam to sandy clay loam | Sandy clay to sandy loam | Loamy sand to sand; non-cohesive material |
| 2. Average bank slope angle | Bank slopes <3H:1V (18° or 33%) on both sides | Bank slopes up to 2H:1V (27° or 50%) on one or occasionally both banks | Bank slopes to 1.7H:1V (31° or 60%) common on one or both banks | Bank slopes over 60% common on one or both banks |
| 3. Vegetative bank protection | Wide bank of woody vegetation with at least 90% density and cover. Primarily hard wood, leafy, deciduous trees with mature, healthy, and diverse vegetation located on the bank. Woody vegetation oriented vertically | Medium bank of woody vegetation with 70-90% plant density and cover. A majority of hard wood, leafy, deciduous trees with maturing, diverse vegetation located on the bank. Woody vegetation oriented 80-90° from horizontal with minimal root exposure | Small bank of woody vegetation with 50-70% plant density and cover. A majority of soft wood, piney, coniferous trees with young or old vegetation lacking in diversity located on or near the top of bank. Woody vegetation oriented at 70-80° from horizontal often with evident root exposure. | Woody vegetation bank may vary depending on age and health with less than 50% plant density and cover. Primary soft wood, piney, coniferous trees with very young, old and dying, and/or monostand vegetation oriented at less than 70° from horizontal with extensive root exposure. |

**Table IX-2**
**Stability Indicators, Ratings, and Descriptions (Johnson et al., 1999; Lagasse et al., 2001) (continued)**

| STABILITY INDICATOR | RATINGS | | | |
|---|---|---|---|---|
| | EXCELLENT (1-3) | GOOD (4-6) | FAIR (7-9) | POOR (10-12) |
| 4. Bank cutting | Little or nonevident. Infrequent raw banks less than 15cm (5.9 in) high generally | Some intermittently along channel beds and at prominent constrictions. Raw banks may be up to 30 cm (11.8 in) high | Significant and frequent. Cuts 30-60 cm (11.8-23.6 in) high. Root mat overhangs. | Almost continuous cuts, some over 60 cm (23.6 in) high. Undercutting, sod-root overhangs, and side failures frequent. |
| 5. Mass wasting or bank failure | No or little evidence of potential or very small amounts of mass wasting. Uniform channel width over the entire reach. | Evidence of infrequent and/or minor mass wasting. Mostly headed over with vegetation. Relatively constant channel width and minimal scalloping of banks. | Evidence of frequent and/or significant occurrences of mass wasting that can be aggravated by higher flows, which may cause undercutting and mass wasting of unstable banks. Channel width quite irregular with scalloping banks. | Frequent and extensive mass wasting. Potential for bank failure, evidenced by tension cracks, massive under-cutting, and bank slumping, is considerable. Channel width highly irregular and scalloped banks. |
| 6. Bar development | Bars are mature, narrow relative to stream width at low flow, well vegetated, and composed of coarse gravel to cobbles. | Bars may have vegetation and/or be composed of coarse gravel to cobbles, but minimal recent growth of bar evident by lack of vegetation on portions of the bar. | Bar widths tend to be wide and composed of newly deposited coarse sand to small bobbles and/or may be sparsely vegetated | Bar widths generally greater than 1/2 the stream width at low flow. Bars are composed of extensive deposits of fine particles up to coarse gravel with little to no vegetation. |
| 7. Debris jam potential | Debris or potential for debris in channel is negligible | Small amounts of debris present. Small jams could be formed. | Noticeable accumulations of all sizes. Moderate downstream debris jam potential possible | Moderate to heavy accumulations of various size debris present. Debris jam potential significant. |
| 8. Obstruction, flow deflectors, and sediment traps | Rare or not present | Present, causing cross currents and minor bank and bottom erosion | Moderately frequent and occasionally unstable obstructions cause noticeable erosion of the channel. Considerable sediment accumulations behind obstructions | Frequent and often unstable causing a continual shift of sediment and flow. Traps are easily filled causing channel to migrate and/or widen. |

**Table IX-2**
**Stability Indicators, Ratings, and Descriptions (Johnson et al., 1999; Lagasse et al., 2001) (continued)**

| STABILITY INDICATOR | RATINGS | | | |
|---|---|---|---|---|
| | EXCELLENT (1-3) | GOOD (4-6) | FAIR (7-9) | POOR (10-12) |
| 9. Channel bed material consolidation and armoring | Assorted sizes tightly packed, overlapping, and possibly imbricated. Most material >4 mm (0.16in). | Moderately packed with some overlapping. Very small amounts of material <4 mm (0.16in). | Loose assortment with no apparent overlap. Small to medium amounts of material < 4 mm (0.16in). | Very loose assortment with no packing. Large amounts of material < 4mm (0.16in). |
| 10. Shear stress ratio | $\tau_0/\tau_c < 1.0$ | $1.0 \le \tau_0/\tau_c < 1.5$ | $1.5 \le \tau_0/\tau_c < 2.5$ | $\tau_0/\tau_c \ge 2.5$ |
| 11. High flow angle of approach to bridge or culvert | 0° ≤ a ≤ 5° | 5° < a ≤ 10° | 10° < a ≤ 30° | a > 30° |
| 12. Bridge or culvert distance from meander impact point | Dm > 115ft | 66< Dm ≤ 115ft | 33 < Dm ≤ 66ft | 0 < Dm ≤ 33ft |
| 13. Percentage of channel constriction | 0-5% | 6-25% | 26-50% | >50% |

**Table IX-3**
**Stability Indicator Weights (Johnson et al., 1999; Lagasse et al., 2001)**

| STABILITY INDICATOR | WEIGHT |
|---|---|
| 1. Bank soil texture and coherence | 0.6 |
| 2. Average bank slope angle | 0.6 |
| 3. Vegetative bank protection | 0.8 |
| 4. Bank cutting | 0.4 |
| 5. Mass wasting or bank failure | 0.8 |
| 6. Bar development | 0.6 |
| 7. Debris jam potential | 0.2 |
| 8. Obstructions, flow deflectors, and sediment traps | 0.2 |
| 9. Channel bed material consolidation and armoring | 0.8 |
| 10. Shear stress ratio | 1.0 |
| 11. High flow angle of approach to bridge or culvert | 0.8 |
| 12. Bridge or culvert distance from meander impact point | 0.8 |
| 13. Percentage of channel constriction | 0.8 |

**Table IX-4**
**Total Stability Ratings and Associated Overall Ratings**
**(Johnson et al., 1999; Lagasse et al., 2001)**

| OVERALL RATING | TOTAL RATING |
|---|---|
| Excellent | R < 32 |
| Good | 32 ≤ R <55 |
| Fair | 55 ≤ R <78 |
| Poor | R ≥ 78 |

Stability indicators 1-9 are scored by the field team using field notes, stream data sheets, and photos. Shear stress ratio (stability indicator 10) is determined using the following equations:

$$\tau_o = \gamma RS \quad \textbf{(IX-1)}$$

and

$$\tau_c = K_s(\gamma_s - \gamma)d_{50} \quad \textbf{(IX-2)}$$

Where, $\tau_o$ = reach averaged shear stress, $R$ = hydraulic radius, $S$ = channel slope, $\tau_c$ = critical shear stress, $K_s$ = Shield's parameter (0.047 for sand-bed channels), $\gamma_s$ = specific weight of sediment, $\gamma$ = specific weight of water, and $d_{50}$ = the particle size such that fifty percent of bed material is finer and passes through a sieve of this size. The shear stress, channel slope, and hydraulic radius are obtained from the HEC-RAS model output under bankfull conditions. The sediment size is obtained from field analysis of bed samples. The ratio, $\tau_o/\tau_c$, is equal to unity at the initiation of movement for the majority of bed sediment. If this ratio is greater than one, most of the sediment along the bed is mobile.

Ratings for stability indicators 11 and 12 are assessed using GIS and aerial photographs to measure the high flow angle of approach to bridges or culverts, and the distance of bridges or culverts to meander impact points. The percentage of channel constriction (stability indicator 13) is determined by comparing the averaged bankfull width of the reach and the local bankfull width at multiple locations within the reach, and totaling the areas with bank full widths less than the reach average. The field team consisted of two members who collaborated to determine the appropriate ratings for each category within each reach. Once the scoring is completed, the ratings are weighted and summed. Based on these numbers, each reach is assigned an overall stability rating of excellent, good, fair, or poor.

Reach 4 has been omitted from the Rapid Channel Assessment due to the large amount of armoring and limited potential for natural channel processes to influence the stability indicators.

**Channel Evolution Model**

Both stream classification systems and channel evolution models indicate the present condition of a stream reach under investigation, but characterization of additional reach upstream and downstream of the investigated reach usually provide an understanding of the overall trend of the stream. This is mainly because the most widely used channel evolution model, such as the ones proposed by Schumm et al. (1984) and Simon (1989); assume channel evolution due to bank collapse based on a “space for time” substitution. In other words, downstream conditions are interpreted as preceding (in time) the immediate location of interest and upstream conditions are interpreted as following (in time) the immediate location of interest. Thus, the channel evolution model describes a predictable sequence of changes in time that a stream undergoes after certain kinds of perturbation in the system such as channelization or change in land use. **Figure IX-2** shows the channel evolution model proposed by Simon (1989). It consists of six stages. Identification of the evolutionary stage of a reach of Johnson Creek has aided in the prediction of the future stage of that reach. **Table IX-5** shows the processes at work in each of Simon’s stages.

An important aspect of the Channel Evolution Model is that stream power is directly correlated to the sediment transport capacity of a stream as defined in Equation IX-3.

$$\gamma QS \propto Q_S d_{50} \qquad \textbf{(IX-3)}$$

Where, $\gamma$ = the unit weight of water, $Q$ = water discharge, $S$ = bed or energy slope, $Q_s$ = sediment discharge, and $d_{50}$ = the median sediment size. An implication of this relationship is that reducing the channel slope will reduce the sediment transport capacity of the stream and lead to aggradation for a particular sediment size. Conversely, increasing the slope will lead to a more degradational system.

**Table IX-5**
**Diagnostic Hillslope (Overland) and Fluvial Processes in the Various Stages of Channel Evolution**

| **Class** | | **Dominant Processes** | | **Characteristic Forms** | **Geobotanical Evidence** |
|---|---|---|---|---|---|
| **Number** | **Name** | **Fluvial** | **Hillslope** | | |
| I | Pre-modified | Sediment transport - mild aggradation; basal erosion on outside bends; deposition on inside bends | | Stable, alternate channel bars; convex top-bank shape; flow line high relative to top bank; channel straight or meandering | Vegetated banks to flow line |
| II | Constructed | | | Trapezoidal cross section; linear bank surfaces; flow line lower relative to top bank | Removal of vegetation |
| III | Degradation | Degradation; basal erosion on banks | Pop-out Failures | Heightening and steepening of banks; alternate bars eroded; flow line lower relative to top bank | Riparian vegetation high relative to flow line and may lean toward channel |
| IV | Threshold | Degradation; basal erosion on banks | Slab, rotational and pop-out failures | Large scallops and bank retreat; vertical face on upper-bank surfaces; failure blocks on upper bank; some reduction in bank angles; flow line very low relative to top bank | Riparian vegetation high relative to flow line and may lean toward channel |

**Table IX-5**
**Diagnostic Hillslope (Overland) and Fluvial Processes in the Various Stages of Channel Evolution (continued)**

| Class | | Dominant Processes | | Characteristic Forms | Geobotanical Evidence |
|---|---|---|---|---|---|
| **Number** | **Name** | **Fluvial** | **Hillslope** | | |
| V | Aggradation | Aggradation; Development of meandering thalweg; initial deposition of alternate bars; reworking of failure material on lower banks | Slab, rotational and pop-out failures; low angle slides of pervious failed material | Large scallops and bank retreat; vertical face on upper-bank surfaces with a slough line; flattening of bank angles; flow line low relative to top bank; development of new floodplain | Tilted and fallen riparian vegetation; reestablishing vegetation on slough line; deposition of material above root collars of slough line vegetation |
| VI | Restabilization | Aggradation; further development of meandering thalweg; further deposition of alternate bars; reworking of failed material; some basal erosion on outside bends; deposition on floodplain and bank surfaces | Low-angle slides; some pop-out failures near flow line | Stable, alternate channel bars; convex top-bank shape; vertical face on top bank; flattening of bank angles; development of new floodplain; flow line high relative to top bank | Reestablishing vegetation extends up to slough line and upper bank; deposition of material above root collars of slough line and upper bank vegetation; some vegetation establishing on bars |

### Scour Analysis Methodology

A scour analysis is performed for the 100-yr flood event in accordance with HEC-18 (Lagasse, et al. 2012). The intent of HEC-18 is to establish methods for estimating various scour components for use in conjunction with engineering judgment to determine the total potential scour depth. This analysis assumes that the scour components develop independently. The required hydraulic parameters for calculating scour depths are obtained from the HEC-RAS and computed internally from the steady state modeling results. Three types of scour are evaluated in this analysis, contraction, abutment and pier. **Appendix C** contains the preliminary scour analysis computations.

Contraction scour is caused by channel width constriction at a bridge crossing. This type of scour occurs when the area of flow is decreased, resulting in increased velocities and bed shear stress in the contracted area. Laursen's equation is used to determine the mode of bed transport. This equation revealed that the mean velocity of the flow in the upstream main channel is greater than the critical velocity of the bed material at each of the structures analyzed, indicating that live-bed conditions exist in Johnson Creek during the 100-year event. Thus based on the definition of live-bed scour, contraction scour will occur at the proposed bridge location if the flow in the section has an increasing capacity to transport sediment, assuming that the approach flows to the section have attained equilibrium transport conditions. HEC-18 recommends the modified version of Laursen's 1960 relationship (Equation IX-4) for estimating live-bed contraction scour as presented here:

$$\frac{Y_2}{Y_1} = \left(\frac{Q_2}{Q_1}\right)^{6/7} \left(\frac{W_1}{W_2}\right)^{k_1} \quad \textbf{(IX-4)}$$

where $Y_1$ = average depth in the upstream main channel, $Y_2$ = average depth in the contracted section, $W_1$ = bottom width of the upstream main channel, $W_2$ = bottom width of the contracted section, $Q_1$ = flow in the upstream channel transport sediment, $Q_2$ = flow in the contracted channel, and $k_1$ = coefficient of bed material transport. From this, $Y_s = Y_2 - Y_1$, the average contraction scour depth.

Local scour is the result of water flowing around a pier or an abutment. These obstructions induce the formation of vortex systems caused by the acceleration of the flow around the obstruction. A horseshoe vortex is formed by water hitting the upstream surface of the obstruction. In addition, vertical vortices, referred to as wake vortices, occur downstream of the obstruction. Both vortices remove material from the base of the obstruction. However, the intensity of the vortices diminishes downstream from the obstruction. In calculating local scour at abutments, Froehlich's equation is utilized. Froehlich (TRB 1989) analyzed 170 live-bed scour measurements in laboratory flumes by regression analysis to obtain the following equation:

$$\frac{Y_s}{Y_a} = 2.27 K_1 K_2 \left(\frac{L'}{Y_a}\right)^{0.43} Fr^{0.61} + 1 \quad \textbf{(IX-5)}$$

where $K_1$ = coefficient for abutment shape, $K_2$ = coefficient for angle of embankment to flow, $L'$ = length of active flow obstructed by the embankment, $Y_a$ = average flow depth in the approach region, $Fr$ = upstream Froude Number and $Y_s$ = scour depth.

Pier scour is calculated using the HEC-18 equation (based on the CSU equation) for both clear water and live-bed conditions (Richardson, 1990). This equation predicts maximum pier scour depths and is defined as follows:

$$Y_s = 2.0 K_1 K_2 K_3 K_4 a^{0.65} Y_1^{0.35} Fr_1^{0.43} \quad \textbf{(IX-6)}$$

where $Y_s$ = depth of scour, $K_1$ = correction factor for pier nose shape, $K_2$ = correction factor for angle of attack of flow, $K_3$ = correction factor for bed condition, $K_4$ = correction factor for armoring of bed material, $a$ = pier width, $Y_1$ = flow depth directly upstream of the pier and $Fr_1$ = Froude Number directly upstream of the pier. The correction factors have been assigned based on pier geometry and bed conditions as observed in the field and obtained from as-built plans. The results of the local scour calculations (pier or abutment) are added to the contraction scour depth to obtain the total scour at each location.

## C. Findings of Channel Assessment

### Rapid Channel Assessment

**Table IX-6** provides a summary of the results of the assessment and the scoring. After compiling the scores, Reaches 1 and 5 are rated as Good with Reach 2 and 3 rated Fair. Reach 3 has the highest score with an overall rating of 68.8 followed closely by Reach 2 at 65.6. These scores fall close to the middle of the range for a Fair rating. With an overall rating of 54.0 in Reach 1, the channel in this region barely qualifies for a Good rating. The rating for Reach 5 is slightly better with an overall score of 51.4.

**Table IX-6**
**Rapid Channel Assessment Results and Overall Ratings for Each Reach**

| Stability Indicator | REACH 1 | | REACH 2 | | REACH 3 | | REACH 5 | |
|---|---|---|---|---|---|---|---|---|
| | Rating | Weighted Rating | Rating | Weighted Rating | Rating | Weighted Rating | Rating | Weighted Rating |
| 1. Bank soil texture and coherence | 7 | 4.2 | 7 | 4.2 | 7 | 4.2 | 7 | 4.2 |
| 2. Average bank slope angle | 7 | 4.2 | 6 | 3.6 | 8 | 4.8 | 7 | 4.2 |
| 3. Vegetative bank protection | 5 | 4.0 | 6 | 4.8 | 7 | 5.6 | 6 | 4.8 |
| 4. Bank cutting | 8 | 3.2 | 7 | 2.8 | 10 | 4.0 | 6 | 2.4 |
| 5. Mass wasting or bank failure | 7 | 5.6 | 8 | 6.4 | 9 | 7.2 | 5 | 4 |
| 6. Bar development | 5 | 3.0 | 8 | 4.8 | 9 | 5.4 | 9 | 5.4 |
| 7. Debris jam potential | 4 | 0.8 | 7 | 1.4 | 7 | 1.4 | 6 | 1.2 |
| 8. Obstructions flow deflectors, and sediment traps | 5 | 1.0 | 7 | 1.4 | 5 | 1.0 | 6 | 1.2 |
| 9. Channel bed material consolidation and armoring | 8 | 6.4 | 8 | 6.4 | 6 | 4.8 | 4 | 3.2 |
| 10. Shear stress ratio | 12 | 12.0 | 9 | 9.0 | 12 | 12.0 | 8 | 8 |

| Table IX-6<br>Rapid Channel Assessment Results and Overall Ratings for Each Reach (continued) | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Stability Indicator | REACH 1 | | REACH 2 | | REACH 3 | | REACH 5 | |
| | Rating | Weighted Rating | Rating | Weighted Rating | Rating | Weighted Rating | Rating | Weighted Rating |
| 11. High flow angle of approach to bridge or culvert | 2 | 1.6 | 6 | 4.8 | 5 | 4.0 | 10 | 8 |
| 12. Bridge or culvert distance from meander impact point | 4 | 3.2 | 10 | 8.0 | 10 | 8.0 | 2 | 1.6 |
| 13. Percentage of channel constriction | 6 | 4.8 | 10 | 8.0 | 8 | 6.4 | 4 | 3.2 |
| **Overall Rating/ Weighted Totals** | **Good** | **54.0** | **Fair** | **65.6** | **Fair** | **68.8** | **Good** | **51.4** |

The lower scores for Reaches 2 and 3 are predominantly related to the shear stress ratio, bar development, bank cutting and structure distance to meander impact point. The high shear stress ratios are generally due to the median sediment size which is observed to be relatively fine grained material. Therefore, only minimal bed shear stress leads to a mobile bed. Also, Reaches 2, 3 and 5 have well vegetated bars along with newly formed bars with little to no vegetation leasing to poorer ratings. Local bank erosion, cutting and failures in Reach 3 are the main reason has the poorest overall score.

**Channel Evolution**

The portion of Johnson Creek that flows through the City of Grand Prairie is the downstream section of the overall channel. According to Simon's Channel Evolution Model, in disturbed systems such as Johnson Creek, the downstream end of the system is expected to be in a late stage in its evolution with a prevailing quasi-equilibrium condition. **Table IX-7** summarizes the evolution stage of the identified reaches in Johnson Creek.

| Table IX-7<br>Evolution Classification of Johnson Creek | |
|---|---|
| **Reach ID** | **Evolution Class** |
| 1 | Late Class V |
| 2 | Late Class V |
| 3 | Class V |

**Table IX-7**
**Evolution Classification of Johnson Creek (continued)**

| Reach ID | Evolution Class |
|---|---|
| 4 | Not applicable with a gabion lined channel |
| 5 | Late Class V |

Depositional features are quite evident in Reach 1, including alternating point bars in previously straight regions. These point bars are allowing for the development of a meandering thalweg and the channel attempts to minimize its stream power by lowering its slope. Also, while deposition is observed on portions of the right bank, there is some evidence of widening on the left bank. Also, there is evidence that the creek is close to being in a state of re-stabilization (Class VI) with mature woody vegetation becoming established near the top of the banks.

Reach 2 is characterized by established riffle-pool sequences with only slight widening of the channel occurring upstream of the Union Pacific Railroad. Downstream of this structure, the channel widens considerably and deposition on the bed and banks is the principal process. However, the channel is considered a late Class V because a new floodplain has not yet reestablished itself. In addition, grade controls maintain low slopes in this region which inhibit the formation of a meandering thalweg. Based on continued maintenance from the golf course, a transition to Class VI is likely to be considerably slower here than in adjacent reaches because excessive deposition would likely be naturally removed from the channel bottom by movement of bed load due to the shear stress ratio being greater than one. In addition, since this reach is located in a well maintained golf course, sustained anthropomorphic impacts to the creek are expected in the future with reduced ability for natural processes to continue the evolution of this reach.

Reach 3 appears to be undergoing a widening based on observations of cut banks and limited vegetation. However, point bar and mid channel bar formation is occurring with evidence of a meandering thalweg developing. The higher occurrence of cut banks is more indicative of a channel that is earlier in its evolutionary stage than the other reaches analyzed with deposition primarily confined to the channel bed.

Reach 4 is a gabion basked lined reach of the channel with no significant applicability to process through any class of channel evolution.

Upstream of the diversion weir in Reach 5, the channel and banks are depositional with little widening expected in the future. This region is close to Class VI, and is therefore generally stable as new banks and a floodplain become established. However, downstream of the diversion weir, numerous mid channel bars have formed and the channel is experiencing some bank erosion indicative of a Class V stage. In addition, especially in the region immediately downstream of the diversion weir, a meandering thalweg is developing. Closer to the confluence with the West Fork Trinity River, the size of the mid channel bars decreases. Also, wood vegetation has developed on both banks extending almost to the baseflow elevation in the channel. Therefore, the stream becomes closer to quasi-equilibrium at its further downstream location.

## D. Areas of Concern and Importance Geomorphologic Features

Field observations indicated that the dominant process within the segment of Johnson Creek that flows through the City of Grand Prairie is aggradation. This is supported by the observation of fine sand on the banks and evidence of reestablishing floodplains. In addition, in relatively undisturbed portions of the creek such as Reaches 1 and 3, vegetation has become reestablished to the top of the banks of the creek. However, despite evidence of late stage evolution in the majority of Johnson Creek within the City, obvious signs of localized erosion is observed in the field. Generally, this erosion is isolated and does not appear to be threatening any structure on the overbanks.

The following summarizes the important geomorphologic features observed in Johnson Creek within the limits of the City of Grand Prairie according to each of the respective geomorphologic reaches, as identified in Section B. Maps indicating the location of each of these features are included with a photo labeled for the number described below.

***Reach 1***

1. A rock outcrop is located across the channel approximately 100 feet downstream of the Northbound SH 360 Frontage Road. This indicates that the bed is cut to bedrock and is a hard point that ensures any streambed instabilities downstream cannot propagate upstream. The photo labeled 123 on **Figure IX-3.1** shows this rock outcrop.
2. Beginning approximately 180 feet downstream of the rock outcrop, two distinct features are noted. On the left bank, erosion had cut to rock, limiting any further erosion in this region. On the right overbank, continuing approximately 500 feet downstream, the floodplain has become reestablished with deposition of medium to fine sand as shown in photo 128 on **Figure IX-3.1**.
3. From cross section 19085 to immediately upstream of the Avenue J crossing at cross section 18128, the Johnson Creek channel begins a riffle-pool sequence with an average riffle to pool distance of 125 feet.
4. Significant abutment scour is observed along the Avenue J crossing in the location of photo 130 on **Figure IX-3.1**.

***Reach 2***

5. A general riffle-pool sequence continues downstream of the Avenue J crossing with an average riffle to pool distance of 175 feet to cross section 17044. Scour is not observed at any of the structures in this region, including the aerial utility crossing located downstream of cross section 17768 shown in photo 004 in **Figure IX-4.1**.
6. A constructed on-channel pond is located between cross sections 17044 and 16923 as shown on **Figure IX-4.1**. The bottom width of the channel at this location is twice as large as upstream and downstream sections. Significant deposition including the formation of a point bar, an alluvial deposit that forms by gradual accumulation on the inner side of a meander, is observed in the eastern side of this feature, indicative of the reduced sediment transport capacity in the location as shown in **Photograph IX-1**. However, the formation of vegetation on the point bar indicates that the feature is relatively stable.

**Photograph IX-1: Photograph of Point Bar Located Downstream of Cross Section 17044.**

7. A golf course bridge and aerial crossing are located immediately downstream of the on channel pond from cross section 16892 to 16816 as shown on **Figure IX-4.1**. No scour is observed along the bridge, but significant abutment scour including concrete slab failures are observed on the right side of the aerial crossing. **Photograph IX-2** shows the slope and concrete failures.

**Photograph IX-2: Slope and Concrete Failure at Aerial Crossing #2**

8. Beginning immediately downstream of the aerial utility crossing at cross section 16816 and continuing to the Union Pacific Railroad Bridge, riffle-pool sequencing becomes reestablished in the channel with an average riffle to pool spacing of approximately 140 feet as shown in **Figure IX-4.2** photo 008.
9. Despite the large contraction, significant scour is not observed in the field at the Union Pacific Railroad Bridge. However, a deep scour pool is evident on aerial photographs downstream of this feature shown in **Figure IX-4.2** photo 009.
10. Approximately 160 feet downstream of the Union Pacific Railroad Bridge, a golf cart path crosses the creek at cross section 15829. Scour is not observed here, but a significant portion of the downstream end of the concrete beam spanning the bridge is damaged as shown in the photograph in **Photograph IX-3**.

**Photograph IX-3: Concrete Span Damage at Golf Course Bridge #4**

11. Between cross sections 15780 and 14839, a wide and deep pool is formed by an inline grade control structure made of gabion blocks at the downstream end of this feature as shown on photo 011 on **Figure IX-4.2**.
12. Significant failure of gabion mattresses is observed on the left bank continuing approximately 225 feet downstream of the inline structure from cross section 14839 as shown in **Photograph IX-4** below. **Figure IX-4.3** outlines the length of the gabion failure on the aerial photo.

**Photograph IX-4: Gabion Mattress Failure at Inline Structure #2**

13. At the downstream end of the reach is a second inline grade control structure at cross section 13105 as shown on **Figure IX-4.3**. Deep and wide pools have formed in the area bounding the two grade controls.

***Reach 3***

14. The concrete grade control structure located at the upstream end of this reach at cross section 13073 as shown on **Figure IX-5.1** has a large downed tree on its left side which is causing debris to accumulate. In addition, structural failures are occurring on the right side of its downstream face as shown in **Photograph IX-5**.

**Photograph IX-5: Concrete Failure on Downstream Side of Inline Structure #3**

15. Immediately downstream of the inline structure at cross section 13073 a large point bar shown on photo 15 with well established vegetation has developed. The point bar limits are estimated as shown on **Figure IX-5.1**. Flow within the main channel of Johnson Creek is diverted around this feature with the majority of it flowing rapidly down its left side. In this location, a nearly vertical cut bank has developed and is one of the few locations of erosion observed.
16. Downstream of the point bar, a riffle-pool sequence becomes the dominant feature in the main channel. This continues downstream to cross section 11715 as shown in photo 16 on **Figure IX-5.1**.
17. A rock outcrop with a scour pool downstream is located approximately 40 feet downstream of cross section 11715 shown on **Figure IX-5.1**. This marks the upstream location of a short riffle pool sequence which continues to cross section 10998 with an average riffle to pool spacing of 170 feet.
18. Between cross sections 10998 and 10261, further channel instabilities are occurring as shown in **Figure IX-5.1** as cut bank. On the left side of the channel, there is evidence of the recent formation of a point bar, with cut and nearly vertical banks on the right side of the channel as shown in **Photograph IX-6**.

**Photograph IX-6: Bank Cuts on the Right Bank in the Middle of Reach 3**

19. Riffle-pool sequencing becomes the prominent channel feature downstream of this location, continuing to a rock outcrop located approximately 100 feet upstream of Duncan Perry Road as located on **Figure IX-5.1**. The mean riffle to pool spacing in this location is approximately 75 feet.
20. Limited scour is observed on either the piers or abutments under Duncan Perry Road as shown in photo 020 on **Figure IX-5.1** and various photos included in the photo log in **Appendix C**.
21. Downstream of Duncan Perry Road continuing to the end of the reach, sediment laden water with high turbidity is observed within the main channel. However, this portion of the reach is a stable deep and wide pool with mature vegetation to the banks on both sides of the channel as shown in photo 021 of **Figure IX-5.1**.

***Reach 4***

22. The main channel within this reach from cross section 9116 to 4809 is trapezoidal in shape and consists entirely of gabion lined banks as shown in photo 022 on **Figure IX-6.1**.
23. A plunge pool is constructed at the downstream end of the weir located at the upstream end of the reach. Immediately downstream of this location, a small point bar between 9059 and 8921 on **Figure IX-6.1** has formed on the left side of the channel. In addition, relatively clear water is observed flowing downstream of this location indicating that much of the sediment within the channel upstream is deposited between sections 8808 to 8535. A comparison of the turbidity of the water upstream and downstream of the weir is shown in **Photographs IX-7** and **IX-8**.
24. A point bar developed on the left side of the channel between cross sections 8149 and 7887 shown on **Figure IX-6.1**. Poorly graded rock is the dominant type of

material indicating that the gabion walls could be the main source of deposition shown on photo 024.

25. The aerial utility crossing at cross section 7233 is not impacted by scour or channel processes as viewed from a site visit.
26. A rock outcrop is located between cross sections 6998 and 6566 as shown on **Figure IX-6.1**. The reason for its formation is unclear as the material is quite large and uncharacteristic of the predominant sediment within this reach or Johnson Creek as a whole.
27. An aerial utility crossing and high transmission power lines crossing the creek in the vicinity of cross section 5738 are not impacted by channel processes during this analysis as shown in photo 027.

**Photograph IX-7: Sediment Laden Water Upstream of Weir at Cross Section 9116**

**Photograph IX-8: Relatively Clear Water Flowing Downstream**

28. Between cross section 6566 and the downstream end of the reach, scattered rock deposition is observed with more significant deposition occurring on the right bank upstream of Carrier Parkway as shown on **Figure IX-6.1** photo 028 downstream of cross section 5113. The source of this sediment appears to be from sheet flow on the overbanks.

***Reach 5***

29. The deposition on the right side of the main channel observed at the downstream end of Reach 4 continues downstream to the Southbound State Highway 161 Service Road. Here, the potential source becomes the bridge abutment itself, as the material becomes much finer and more characteristic of the earth embankments as shown in photo 029 on **Figure IX-7.1**.

30. Significant scour is observed along the piers located closest to the channel under SH 161 and its service roads as shown in **Photograph IX-9**. In addition, flow from the road aggressively seeps through the overhead highway on the left banks and has caused substantial cuts in the earthen abutments. Erosion is also observed under the outlets of the numerous outflow pipes in this area. The erosion from these outfalls continues perpendicular to the creek and adds to the scour observed at the piers. Pier scour limits are defined on **Figure IX-7.1**. Of particular concern is significant erosion at the base of the concrete abutments bounding the left side of the main lane bridge. Approximately five feet of erosion has occurred at this location. The erosion caused by the runoff from the roadway is shown in **Photograph IX-10**.

**Photograph IX-9: Pier Scour at SH 161 Caused Footing to Become Exposed**

31. No scour is observed at the Good Links Bike Trail crossing as shown in photo 031 of the photo log **Appendix C**.

32. Downstream of the bike trail, a deep pool has been established which is controlled by the concrete inline structure located at cross section 3721 shown in photo 032. The inline structure downstream of cross section 3807 as shown in **Figure IX-7.1** allows low flows to be diverted into the original Johnson Creek channel while high

flows continue over the dam and further down along a straight wide bottom channel to West Fork Trinity River. The structure consists of a concrete dam with a water control valve in the middle and energy dissipater and low-flow weirs downstream. The structure is in need of maintenance with the clogged water control valve. The function of the water control valve is to control base flow through it. This water control valve is shown in **Photograph IX-11**. Upstream section of the valve and inline concrete structure is filled with sediments. Downstream of the structure, rip raps, placed on both banks has become displaced. The banks, particularly the north bank is undergoing sever erosion and toe scouring.

33. A large and vegetated point bar straddles the middle of the channel downstream of the inline structure from cross section 3360 to 3048. This feature is over six feet higher than the surrounding channel. Flows are diverted to the left and right side of this feature with eroding banks on both sides. The banks continue to erode on each overbank and progressing downstream for approximately 600 feet is existing rock rip rap with some displacement.

**Photograph IX-10: Erosion Caused by Roadway Runoff Seeping through SH 161 Crossing**

**Photograph IX-11: Water control Valve on Downstream Side of Inline Structure #5. Valve is fully opened but is clogged with sediment**

34. Smaller point bars have formed downstream and continue to the region between cross sections 1873 and 1388 as shown on **Figure IX-7.2**.
35. The final 1,400 feet of stream length continuing to the confluence is characterized by deep pools with vegetated side banks shown on aerial photo in **Figure IX-7.2**. A large concrete drop structure approximately 15 feet high is located immediately upstream of the confluence and is not threatened by erosion at the present time.

**Table IX-8** summarizes the most significant problem areas observed during the geomorphologic investigation and provides the item number as listed above. These locations can also be seen on Figure IX-3.1 thru Figure IX-7.2.

| **Table IX-8 Key Locations of Channel Instabilities in Johnson Creek** | |
|---|---|
| **List No.** | **Areas of Significant Identified Problems** |
| 4 | Significant abutment scour along the Avenue J crossing |
| 12 | Significant failure of gabion mattresses on the left bank continuing approximately 225 feet downstream of Inline Structure #2 |
| 14 | Downed tree on Inline Structure #3 located at the upstream end of Reach 3 on its left side |
| 14 | Structural failures on the right side of its downstream face of Inline Structure #3 located northeast of the intersection of North Great Southwest Parkway and Hidden Brook Drive |
| 18 | Bank cutting on right banks between cross sections 10998 and 10261 |

**Table IX-8**
**Key Locations of Channel Instabilities in Johnson Creek (continued)**

| List No. | Areas of Significant Identified Problems |
|---|---|
| 30 | Pier and abutment scour under SH 161 |
| 33 | Damaged 12-inch water control valve on Inline Structure #5 and upstream sediment blocking the valve |
| 34 | Downstream of Inline Structure #5 is excessive deposition of sediments and bank erosion with displacements of the rock rip rap |

### E. Non-Structural Measures (Erosion Hazard Setbacks)

Some areas of existing or potential bank instability may not warrant immediate attention, as there are no structures or infrastructure threatened by bank failures. In these areas, the City of Grand Prairie has expressed a desire to focus on non-structural measures to address potential future damage. Erosion hazard setbacks can be an effective way to protect structures and infrastructure by ensuring that future construction is located far enough away from the channel banks that it will not be threatened by channel erosion, widening, or bank instability. Through field investigation, two specific areas of bank instability are identified: the left bank near cross section 12931 and right bank at cross section 10809. These areas are shown on **Figure IX-8**.

The Drainage Design Manual provides a procedure for determining such setbacks. It should be considered that the necessary setback distance will vary based on the existing bank heights, anticipated channel down cutting, slopes of the surrounding area, and anticipated final channel width. **Figure IX-9** and **Figure IX-10** shows a schematic of the setback determination at the two locations discussed above.

As discussed previously, stable side slopes for Johnson Creek have been estimated to be 4:1, or about 14 degrees. To determine the setback, a 4:1 line should be drawn from the expected final toe location to where it meets the existing grade. An additional 10 feet should be added to allow for maintenance.

Some sites will require additional considerations; particularly those that have the potential for local scour or are on the outer bends of a meander. Local scour can substantially increase the bank height and require additional setbacks. Since meander bends can migrate 1 to 2 feet in a typical year, setbacks should be set with an adequate buffer to allow for the expected migration. Due to the unique situation for each location, the setback distances will require site-specific analysis.

The data and recommendations contained in this DMP for Johnson Creek Corridor should be taken into account before designing or approving erosion hazard setback distances. Some land parcels may be overly burdened by a potential setback distance; elsewhere, meander rates and directions or other factors may be indeterminate. In these and similar instances, structural measures may be warranted to limit the setback distance or improve confidence in the final result.

### F. Structural Measures

**Table IX-9** summarizes the maximum contraction, abutment and pier scour at each Johnson Creek crossing that can potentially result from flood flows originating from the 1% annual chance storm event over the fully developed watershed conditions. The HEC-RAS generated scour analysis plots for each structure are provided in **Appendix C**.

**Table IX-9**
**1% Annual Chance (100-Yr) Scour Analysis Results**

| Location | Type | 1% Annual Chance Contraction Scour | | | 1% Total Left Abutment Scour | 1% Right Abutment Scour | 1% Maximum Pier Scour |
|---|---|---|---|---|---|---|---|
| | | LOB [ft] | Channel [ft] | ROB [ft] | [ft] | [ft] | [ft] |
| SH 360 Southbound Frontage Road | Bridge | 8.7 | 6.7 | 0.5 | 27.5 | 11.7 | 13.8 |
| SH 360 Main Lanes | Bridge | 1.0 | 0.0 | 0.8 | n/a | n/a | n/a |
| SH 360 Northbound Frontage Road | Bridge | 0.0 | 0.0 | 0.0 | 2.8 | 0.8 | 6.8 |
| Avenue J | Bridge | 1.9 | 16.1 | 0.7 | 42.3 | 32.4 | 39.0 |
| Golf Course Crossing #1 | Bridge | 4.8 | 0.0 | 0.8 | n/a | n/a | n/a |
| 100 ft D/S of Golf Course Crossing #1 | Aerial Pipe Crossing | 0.8 | 0.0 | 1.9 | n/a | n/a | 4.8 |
| Golf Course Crossing #2 | Bridge | 1.7 | 0.0 | 0.0 | n/a | n/a | n/a |
| Golf Course Crossing #3 | Bridge | 7.4 | 0.0 | 1.0 | n/a | n/a | n/a |
| 45 ft D/S of Golf Course Crossing #3 | Aerial Pipe Crossing | 1.5 | 0.0 | 0.9 | 15.2 | 12.2 | 4.8 |
| Union Pacific Railroad | Bridge | 7.8 | 26.5 | 14.6 | 37.9 | 33.7 | 23.4 |
| Golf Course Crossing #4 | Bridge | 2.2 | 5.3 | 6.9 | n/a | n/a | n/a |
| 250 ft U/S of Duncan Perry Road | Aerial Pipe Crossing | 1.0 | 0.0 | 0.9 | n/a | n/a | 10.8 |
| Duncan Perry Road Crossing | Bridge | 2.6 | 0.0 | 3.5 | 20.8 | 19.6 | 7.5 |
| 800 ft D/S of Duncan Perry Road | Aerial Pipe Crossing | 0.6 | 0.0 | 1.8 | n/a | n/a | 5.3 |

**Table IX-9**
**1% Annual Chance (100-Yr) Scour Analysis Results (continued)**

| Location | Type | 1% Annual Chance Contraction Scour | | | 1% Total Left Abutment Scour | 1% Right Abutment Scour | 1% Maximum Pier Scour |
|---|---|---|---|---|---|---|---|
| | | LOB [ft] | Channel [ft] | ROB [ft] | [ft] | [ft] | [ft] |
| North Carrier Parkway Southbound | Bridge | 0.0 | 0.0 | 0.0 | 10.2 | 9.2 | 12.6 |
| North Carrier Parkway Northbound | Bridge | 0.0 | 0.6 | 0.0 | 5.7 | n/a | 14.5 |
| PGBT Southbound Frontage Road | Bridge | 0.1 | 1.2 | 0.0 | n/a | n/a | 16.2 |
| PGBT Main Lanes | Bridge | 0.0 | 0.6 | 0.2 | n/a | n/a | 14.1 |
| PGBT Northbound Frontage Road | Bridge | 0.0 | 0.4 | 0.0 | n/a | n/a | 15.6 |
| Good Link Bike Trail Crossing | Bridge | 0.0 | 0.0 | 0.0 | n/a | n/a | 7.3 |

The highest scour depths are located against the abutments of Avenue J, the Union Pacific Railroad and Duncan Perry Road. A contraction scour depth of 26.5 feet within the channel at Avenue J has caused the high scour depth here. The field observations indicate that the concrete abutments at this crossing have been eroded, especially on the right bank downstream of the structure.

Although no significant scour or erosion is observed at the Union Pacific Railroad crossing, there is the potential for deep scour at this location. Contraction scour is limited at Duncan Perry Road. However, abutment scour is quite significant due to the projected embankment length into effective flow.

Field observations indicate that the most significant scour occurs along the piers under SH 161. Field observations also indicate that a significant amount of this is caused by local runoff from the highway causing erosion in a lateral direction perpendicular to the flow of Johnson Creek. However, scour is also observed parallel to the direction of flow. Scour depths resulting in the 100-year storm event at the three SH 161 bridges range from 14.1 feet to 16.2 feet. Based on the as-built information, the piers are approximately 20 feet deep. Therefore, pier scour from the creek coupled with the transverse erosion caused by local runoff is a concern.

**Mitigation Recommendations**

Based on the geomorphologic evaluation of Johnson Creek, measures to increase the stability of the stream are recommended at several key locations as shown in the overall Mitigation Alternatives **Figure IX-11**. **Figures 11-1.1, 11-2.1, 11-3.1, 11-4.1**, and **11-5.1** provide a detailed location of these recommendations and are summarized as follows.

- On the eroded left bank beginning approximately 250 feet downstream of SH 360 in **Reach 1**, one suggestion is to install gabion block to reduce the potential for mass wasting to the extents shown on **Figure 11-1.1**. However, this portion of the channel is close to quasi-equilibrium, and would greatly benefit from stable channel design to allow it to remain in a more natural state. Also, the abutment failure at Avenue J should be remediated by installing gabions or riprap to reinforce the banks.
- Limited instabilities are observed in **Reach 2**, so all recommended mitigation is local in nature and only to repair damaged existing structures. Primarily, it is recommended to repair and reinforce the right abutment at Aerial Crossing #2 at cross section 16845 shown on **Figure 11-2.1**. Also, several gabion mattresses have been dislodged at Inline Structure #2 at cross section 14839 which will require replacement. The Union Pacific Railroad crossing will also require inspections to ensure scour does not become a hazard.
- The large vegetated point bar located at the upstream end of **Reach 3** indicates the stream in this area is beginning to re-stabilize. However, an erosion setback area is noted on **Figure 11-3.1** to indicate that bank cuts on the left side of the channel require some stabilization. The inline structure at cross section 13105 also requires maintenance and repair. Concrete failures have occurred on the right side and a large tree has fallen onto the left side at its crest. A more holistic approach would be to replace this entire structure with imbricated rock to allow for a more aesthetic appearance to the grade control while maintaining functionality. The erosion setback hazards near the middle of the reach are due to vertical banks observed on the right side of the channel. Gabion blocks can be placed here as local mitigation. However, since this region is adjacent to high value residential lots, a more comprehensive approach may be needed such as stable channel design with a more aesthetic approach to mitigate the unstable banks. Also, periodic inspections of Duncan Perry Road should place emphasis on scour potential at piers and abutments. There are several important utilities in this vicinity, including a 72-inch Trinity River Authority (TRA) wastewater interceptor.
- Action items on **Reach 4** are limited to periodic maintenance to ensure the channel maintains its capacity to ensure proper flood control. This includes cleaning out accumulated sediment and ensuring local gabion failures on the banks do not occur, **Figure 11-4.1** shows this reach.
- While the channel in **Reach 5** is generally stable, the pier and abutment scour under SH 161 is in need of mitigation to the extents shown in **Figure 11-5.1**. Therefore, it is recommended that a detailed scour analysis be performed and presented to TXDOT to ensure scour does not affect structural stability and integrity of the bridge. The large vegetated point bar downstream of Inline Structure #5 at cross section 3721 should be dredged. However, subsequent flooding events may likely lead to additional deposition at this location since the channel is quite wide in this location. This area is gradually falling into a poorer condition. Downstream of the structure both sedimentation and erosion are causing problems. A great amount of sediment deposition is occurring in the middle of the channel forming a middle bar that is now virtually an island that not only obstruct flows but also has substantially reduced the conveyance capacity of flood flows. Erosion on the north bank is encroaching limits of Waggoner City Park. Furthermore, sediments have accumulated upstream of the inline structure and are blocking low flows that should go through the control valve (mostly seepage flows are now passing through it). The 12-inch water control valve in the grade control structure is no longer functioning properly and should be replaced. The valve appears to be used to keep the downstream channel with

'environmental flows'. Also, accumulated sediment within the valve must be removed to ensure its proper functionality. Replacing the valve and the inline structure at this location is recommended. In addition, to ensure proper functionality in the future, accumulated sediment upstream and downstream of the structure should be removed.

In addition to the mitigation strategies outlined above, the highly depositional nature of Johnson Creek requires periodic investigation to ensure flooding does not increase. This is especially true in Reach 4 where slight increases in the water surface elevation could lead to substantial residential flooding on the left overbank. In general, Johnson Creek is not highly unstable within the City of Grand Prairie and large-scale channel rebuild in excess of 1,000 feet in length is not needed. However, future considerations need to ensure local instabilities do not increase in size or cause structural failures. A summary of the proposed projects is provided in **Table IX-10**. Cost is provided in this table, but is expanded on in Section XII of this document.

**Table IX-10**
**Proposed Capital Improvement Projects to Stabilize Johnson Creek**

| Project Number | Project Description | Recommendation | CIP Priority No. | Reach | Cost |
|---|---|---|---|---|---|
| 1 | 500 LF of left bank stabilization between SH 360 and Avenue J | Stable channel design or installation of gabion blocks | 5 | 1 | $ 501,800.00 |
| 2 | Abutment repair and stabilization - Avenue J | Riprap or gabion block to reinforce abutment | 1 | 1 to 2 | $ 156,032.00 |
| 3 | Abutment repair and stabilization - Aerial Crossing #2 | Repair abutment and install reinforcement such as concrete bags or gabion block | NA | 2 | NA |
| 4 | Gabion mattress repair - Inline Structure #2 | Replace gabion mattresses | NA | 2 | NA |
| 5 | 200 LF of left bank stabilization - Immediately downstream of Inline Structure #3 | Stable channel design or installation of gabion blocks | 2 | 3 | $ 228,422.00 |
| 6 | Repair or replacement - Inline Structure #3 located northeast of the intersection of North Great Southwest Parkway and Hidden Brook Drive | Repair structure or replace with imbricated rock | NA | 3 | NA |

**Table IX-10**
**Proposed Capital Improvement Projects to Stabilize Johnson Creek (continued)**

| Project Number | Project Description | Recommendation | CIP Priority No. | Reach | Cost |
|---|---|---|---|---|---|
| 7 | 800 LF of right bank stabilization - Approx. 600 feet upstream of Duncan Perry Road | Stable channel design or installation of gabion blocks | 4 | 3 | $ 916,089.00 |
| 8 | Detailed scour mitigation evaluation at SH 161 | Inform NTTA of potential scour issue | NA | 5 | NA |
| 9 | Inline Structure #5 located downstream of SH 161 –Replace structure, bank stabilization downstream 1300 feet and sediment removal | Replace Inline Structure #5, provide dredging and sediment removal. Provide erosion protection on banks. | 6 | 5 | $ 1,775,600.00 |
| 10 | Duncan-Perry Road bridge and roadway improvements | Improved bridge width and elevation or a flood warning system. | 3 | 3 | $ 4,959,273.00 |

Projects with a CTP priority are determined to be the City's responsibility. NA projects are provided additional data and information to provide to property owners.

### G. Inspection and Maintenance

The City of Grand Prairie should consider implementation of a routine field inspection program to monitor the status of the stream stability issues discussed herein. Such a program should focus on problem areas noted. These inspections can be used to help prioritize the necessary improvements and provide the City staff with more detailed knowledge of the stream and impacted facilities. It may be possible to temporarily address or prevent stream stability issues through maintenance, when those issues are detected early through routine inspections.

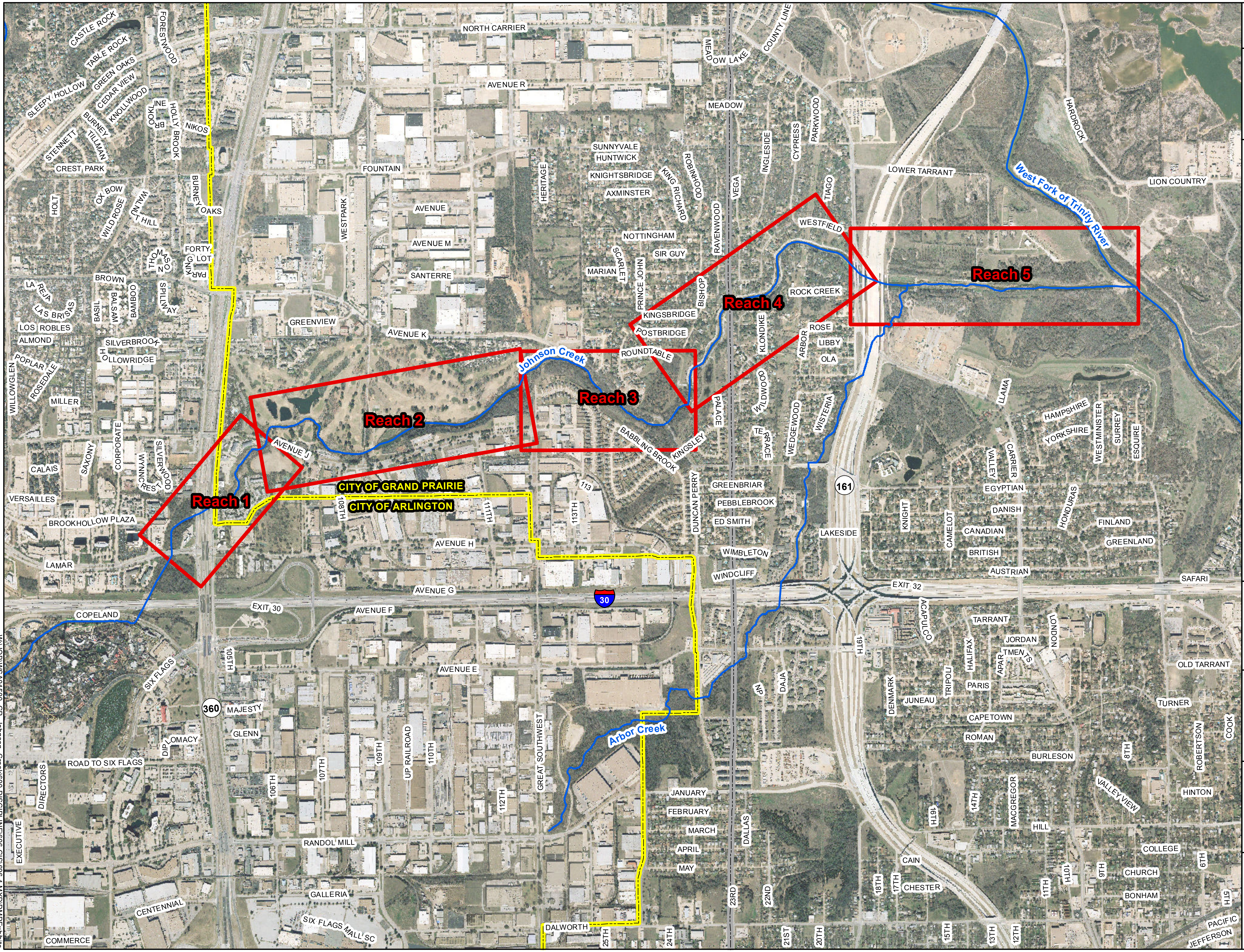
Most of the areas requiring structural measures described above are too degraded to be addressed by maintenance measures alone; however, there are a number of structures in fair to good condition for which routine inspection and maintenance should substantially minimize degradation. Such maintenance may include removal of vegetation, sealing of concrete joints, clearing of accumulated debris, or adding riprap.

Areas of potential sedimentation should be inspected and cleaned out as appropriate.

Existing and future gabion walls should be inspected for debris damage and repaired according to the manufacturer's recommendations. Major damage, such as spilled rocks or misaligned baskets, will require an engineer's involvement.

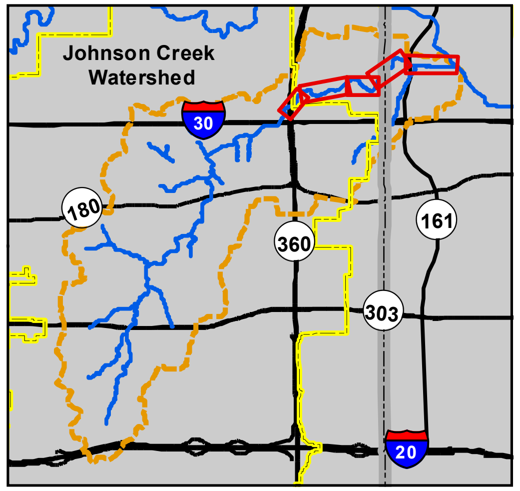
# Figure IX-1

## Johnson Creek Reach Index Map




## Legend

- Stream Centerline
- City Limits
- County Boundary
- Map Index




0 1,500 3,000

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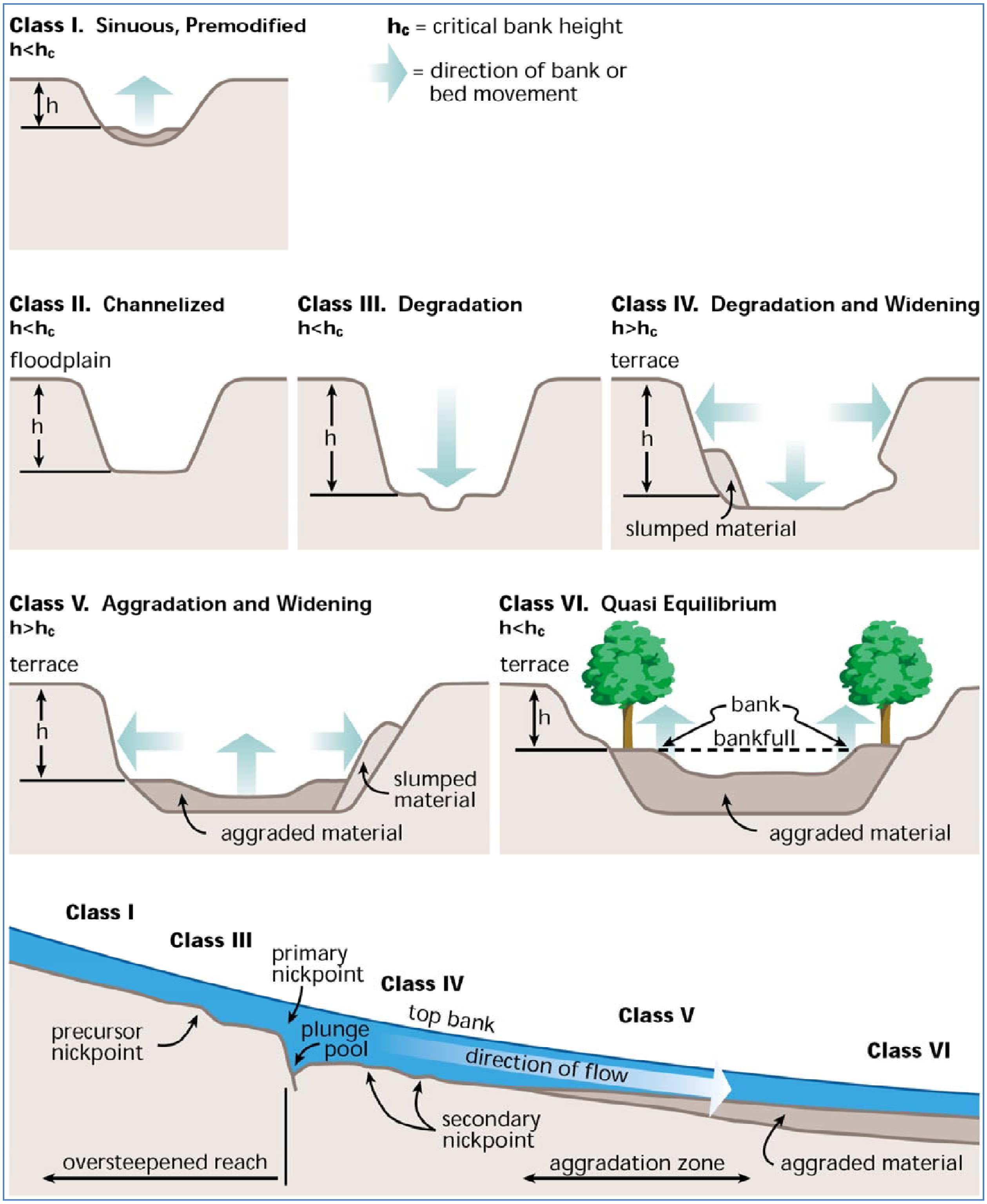
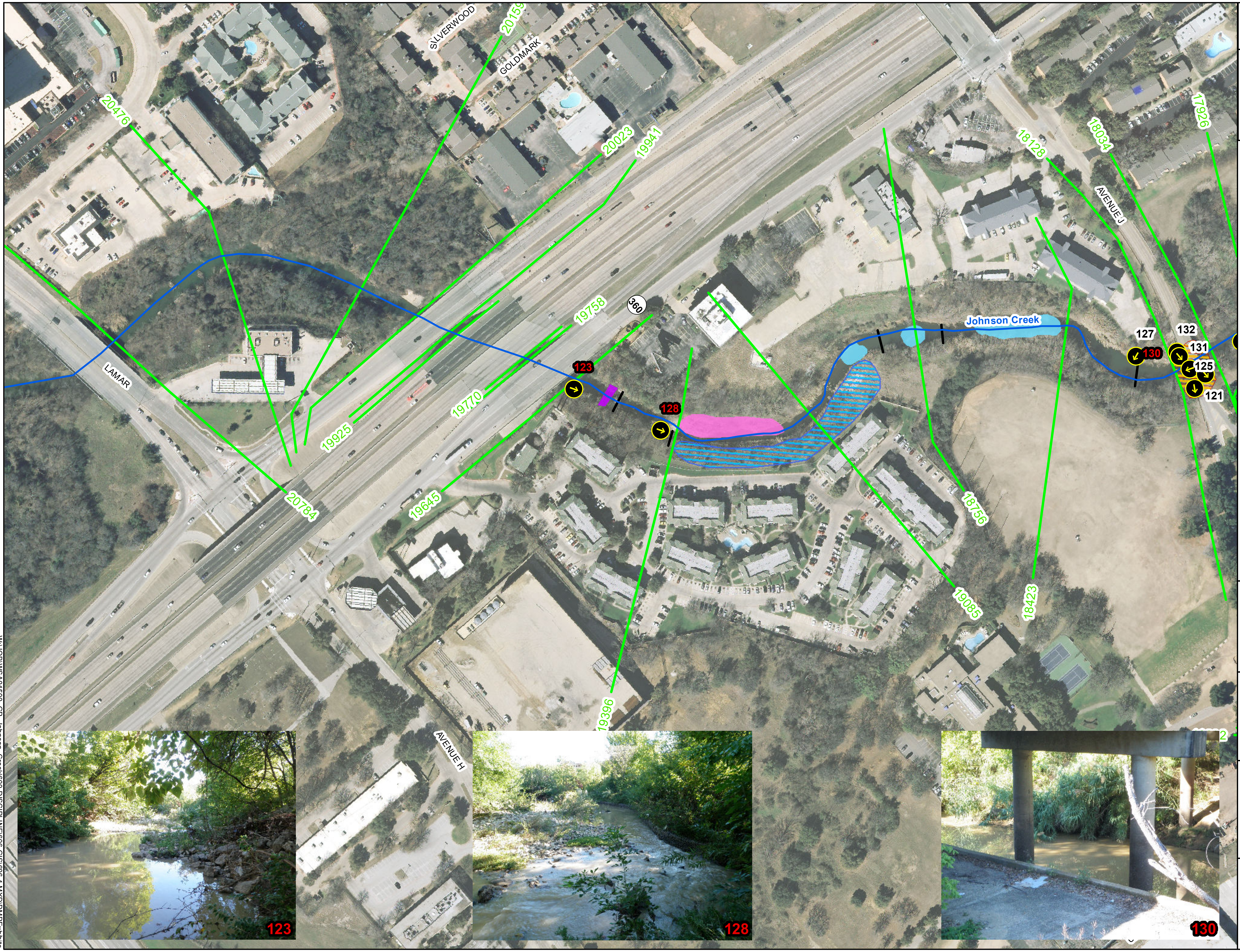



**Figure IX-2: Channel Evolution Model (after Simon, 1989)**

# Figure IX-3.1

## Johnson Creek Reach 1




### Legend

- Photo Locations
- Stream Centerline
- Cross Sections

### Features

- Concrete Grade Control
- Deposition on Bank
- Pool
- Rocky Bank
- Scour Under Bridge
- Riffles

* All photos located in Appendix C

0 200 400
Scale in Feet

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**Figure IX-4.1**

**Johnson Creek Geomorphology Reach 2**

## Legend

- Photo Locations
- Stream Centerline
- Cross Sections

## Features

- Point Bar
- Pool
- Scour Under Bridge
- Riffles

* All photos located in Appendix C

0 100 200

Scale in Feet

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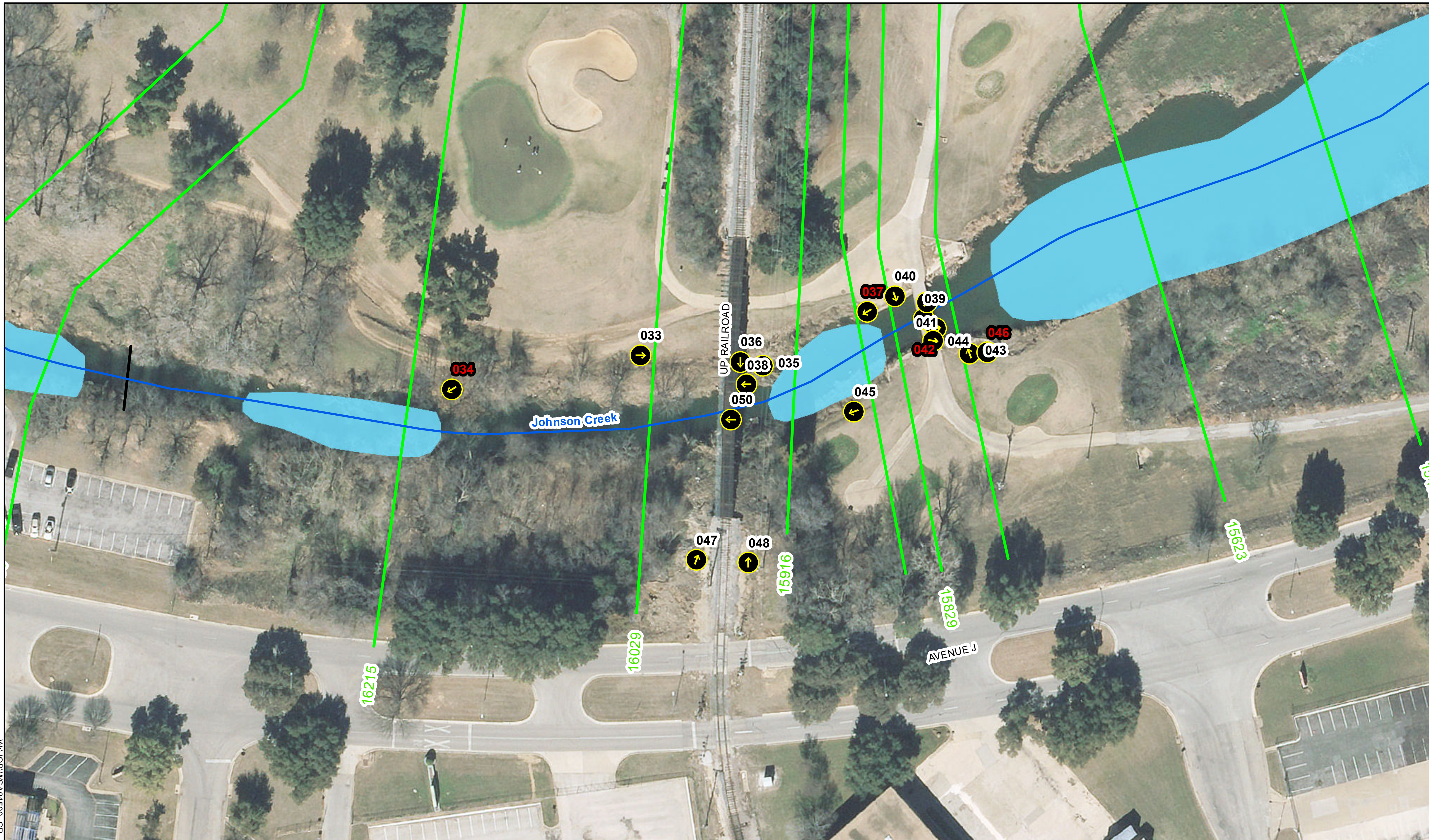
# Figure IX-4.2

## Johnson Creek Geomorphology Reach 2








### Legend

- Photo Locations
- Stream Centerline
- Cross Sections

### Features

- Pool
- Riffles

* All photos located in Appendix C

0 80 160

Scale in Feet

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# Figure IX-4.3

## Johnson Creek Geomorphology Reach 2

### Legend

- Photo Locations
- Stream Centerline
- Cross Sections

### Features

- Concrete Grade Control
- Gabion Failure
- Point Bar
- Pool
- Riffles

* All photos located in Appendix C

0 150 300
Scale in Feet

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# Figure IX-5.1

## Johnson Creek Geomorphology Reach 3

### Legend

- Photo Locations
- Stream Centerline
- Cross Sections

### Features

- Concrete Grade Control
- Cut Bank
- Point Bar
- Pool
- Rock Out Crop
- Slight Deposition
- Stable Riffle Pool
- Riffles

* All photos located in Appendix C

0 230 460

Scale in Feet

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# Figure IX-6.1

## Johnson Creek Geomorphology Reach 4

### Legend

- Photo Locations
- Stream Centerline
- Cross Sections

### Features

- Clear Water
- Deposition on Bank
- Pier Scouring
- Point Bar
- Pool
- Rock Out Crop
- Slight Deposition
- Riffles

* All photos located in Appendix C

0 280 560

Scale in Feet

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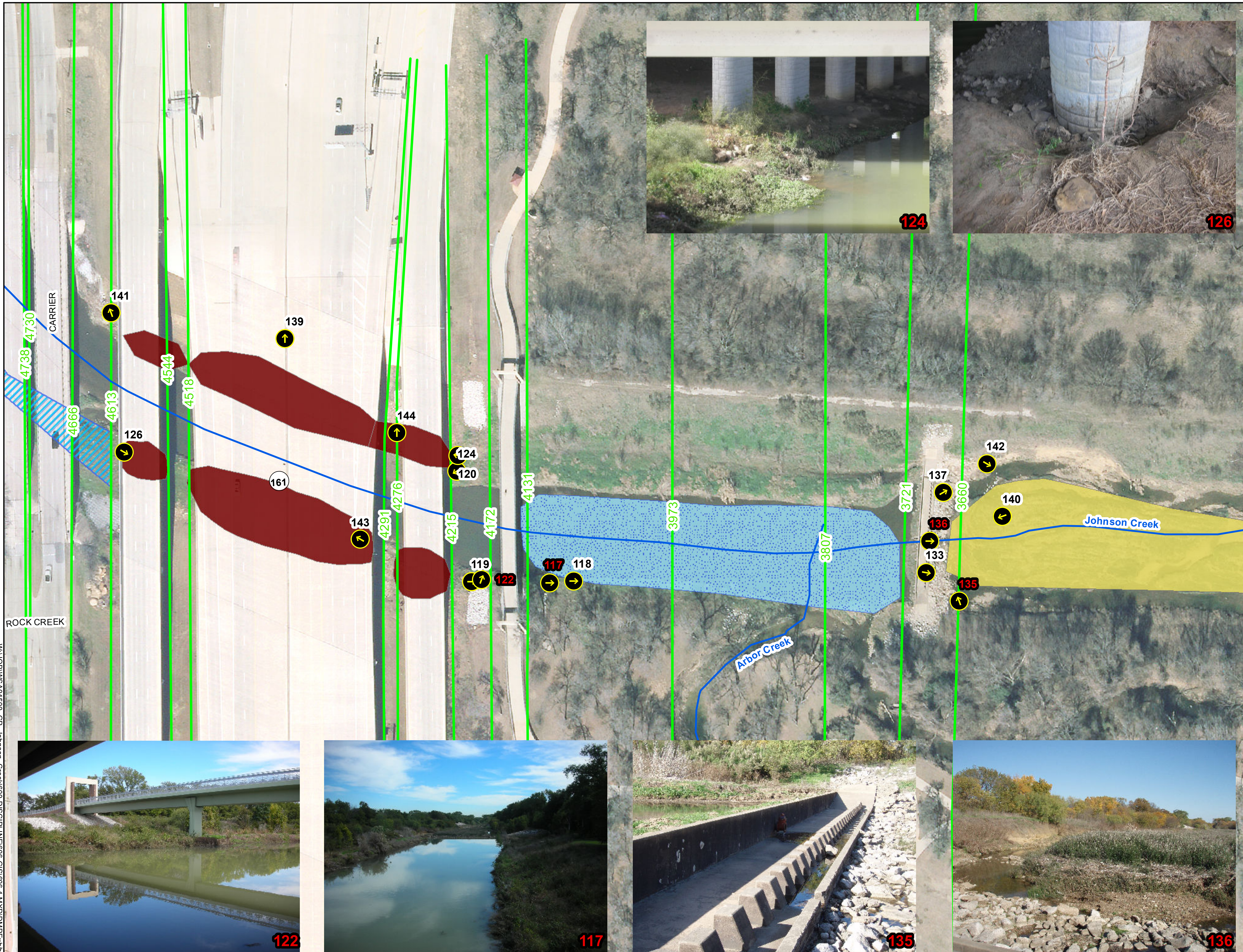
# Figure IX-7.1

## Johnson Creek Geomorphology Reach 5




### Legend

- Photo Locations
- Stream Centerline
- Cross Sections

### Features

- Deposition on Bank
- Deposition Pond
- Pier Scouring
- Point Bar

* All photos located in Appendix C

0 100 200

Scale in Feet

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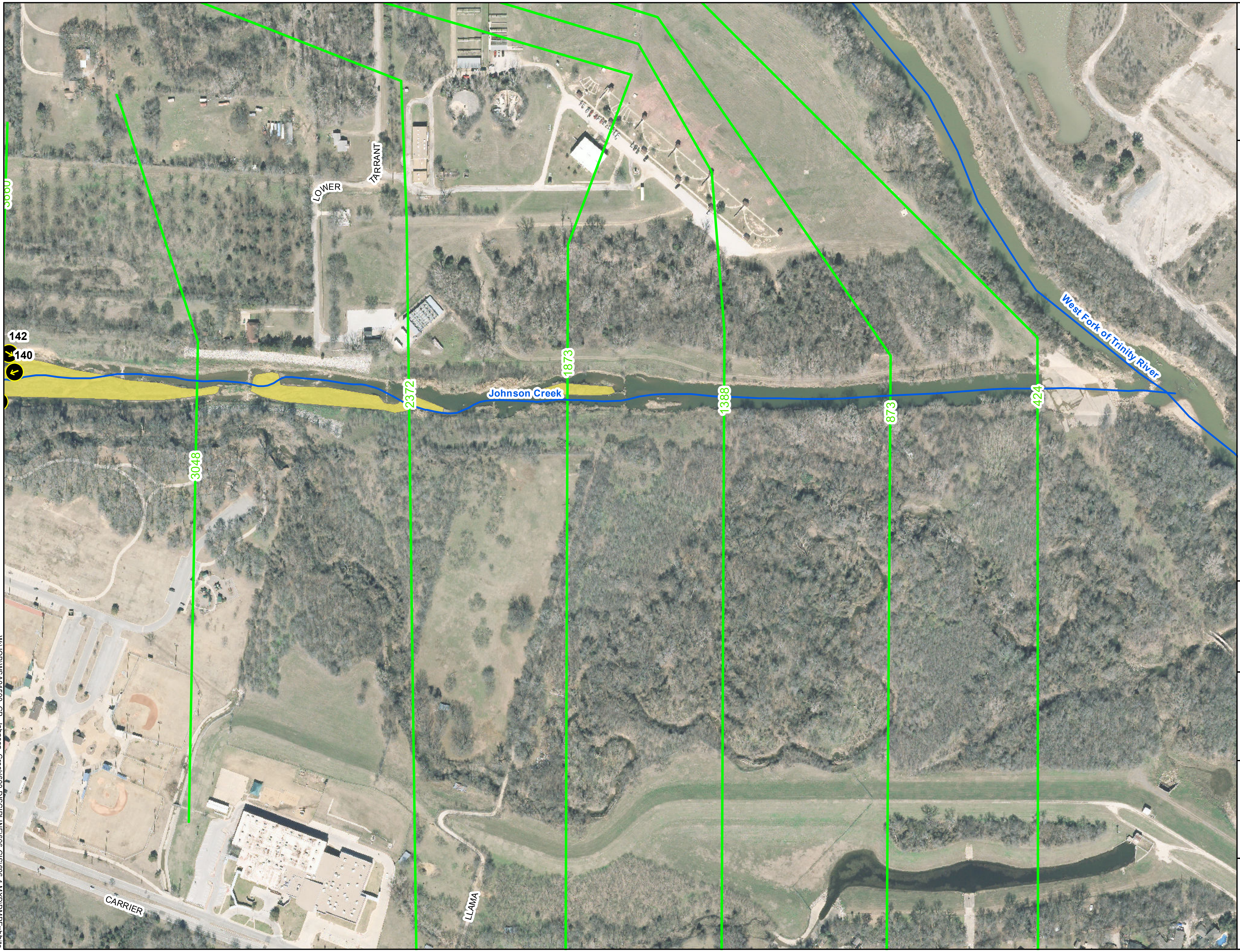


**Figure IX-7.2**

**Johnson Creek Geomorphology Reach 5**

## Legend

- Photo Locations
- Stream Centerline
- Cross Sections

## Features

- Point Bar

* All photos located in Appendix C

0 280 560
Scale in Feet

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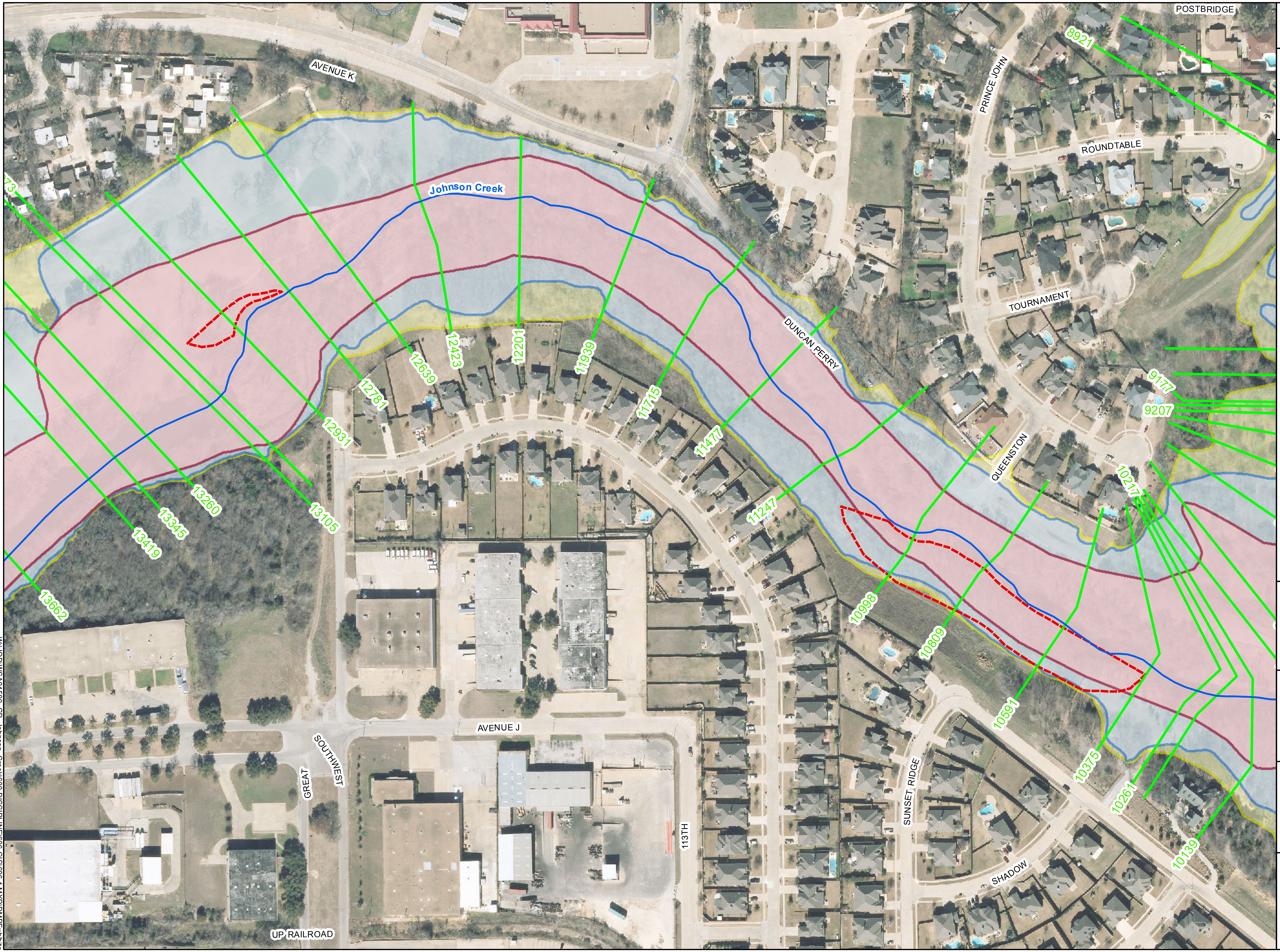


# Figure IX-8

## Johnson Creek Erosion Hazard Setbacks

**Legend**

- Stream Centerline
- Cross Sections
- Erosion Hazard Setbacks
- Revised Floodway
- Revised 1% AC Floodplain
- Revised 0.2% AC Floodplain

AC = Annual Chance

0 200 400

Scale in Feet

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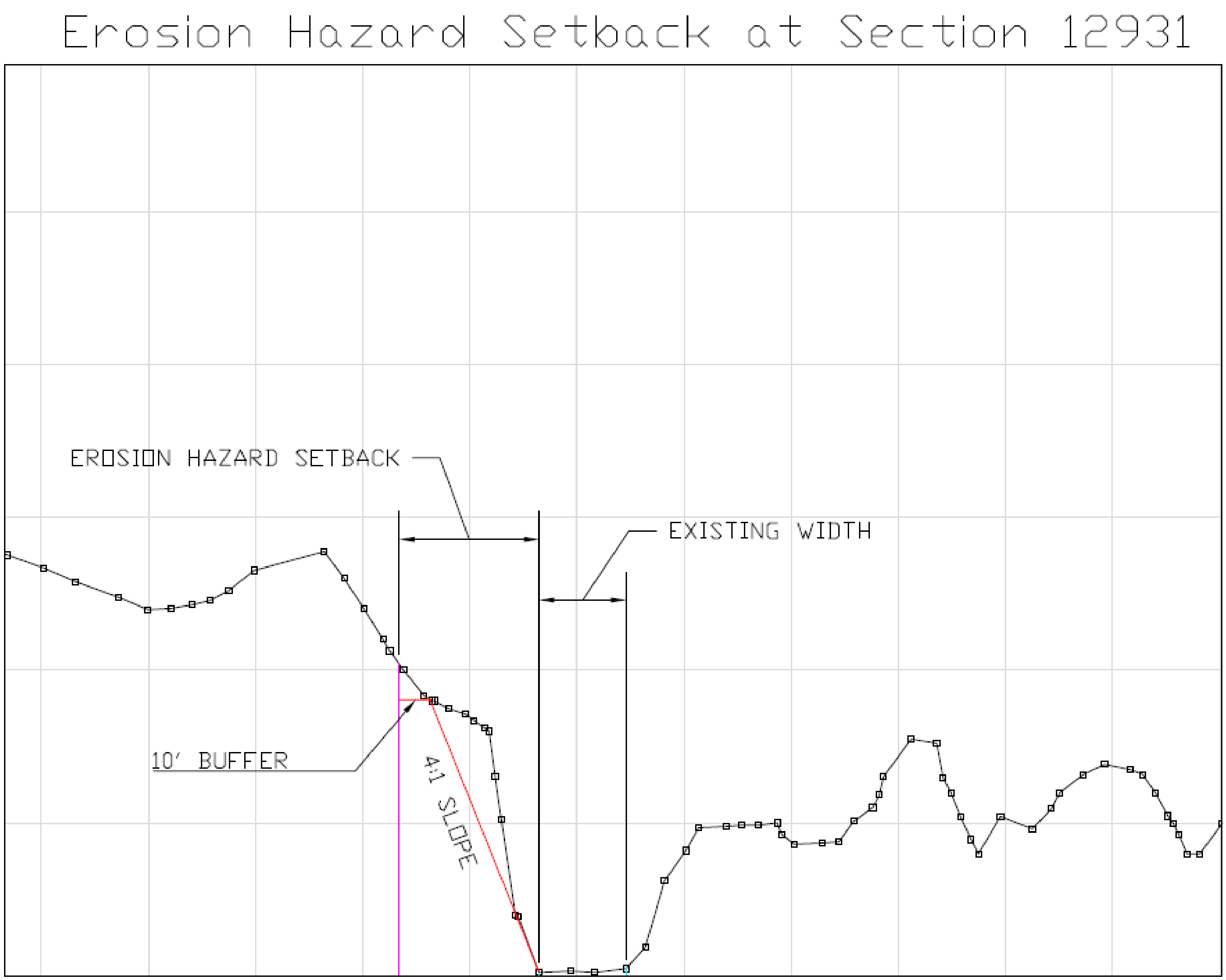



**Figure IX-9. Determination of Erosion Hazard Setback**

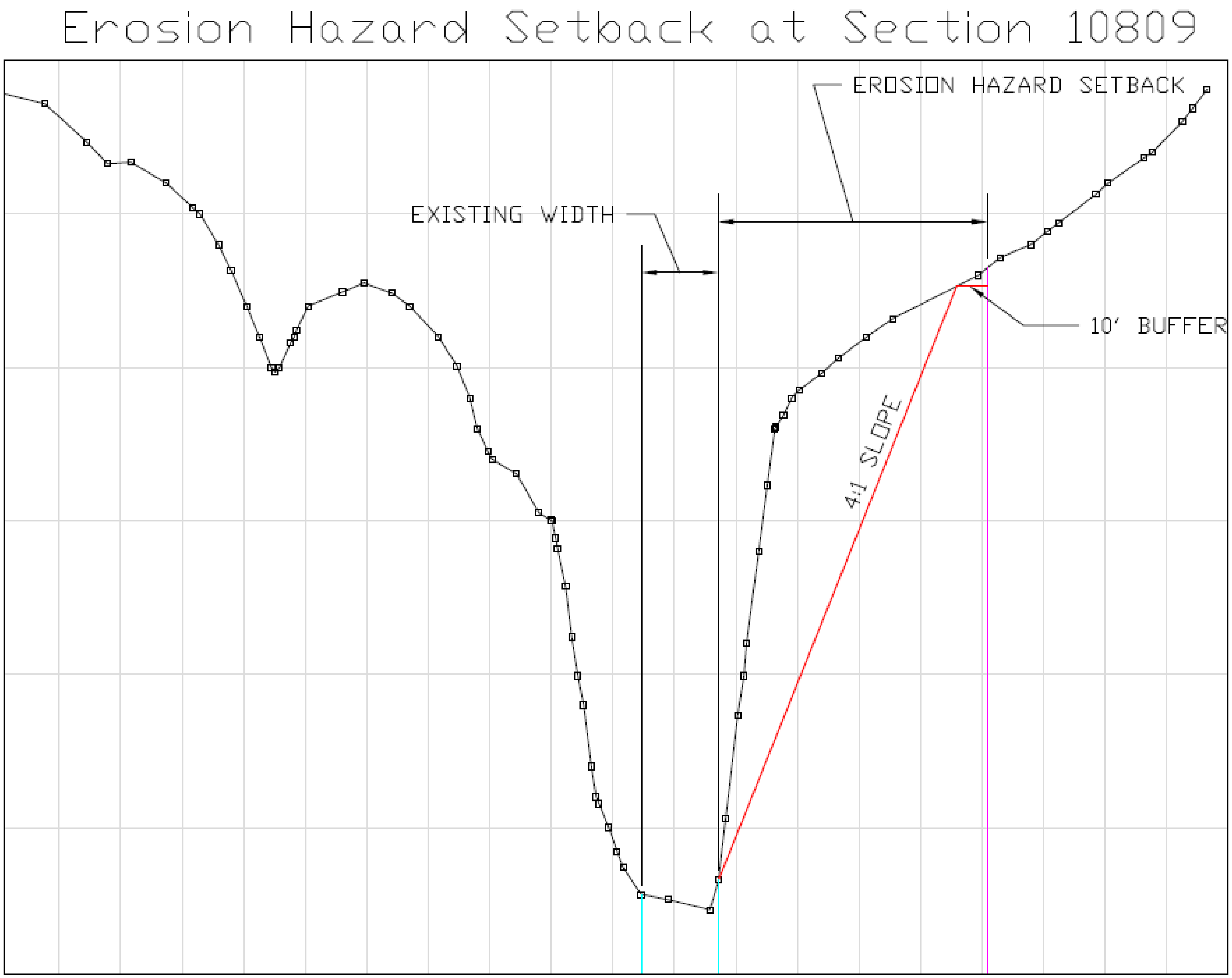
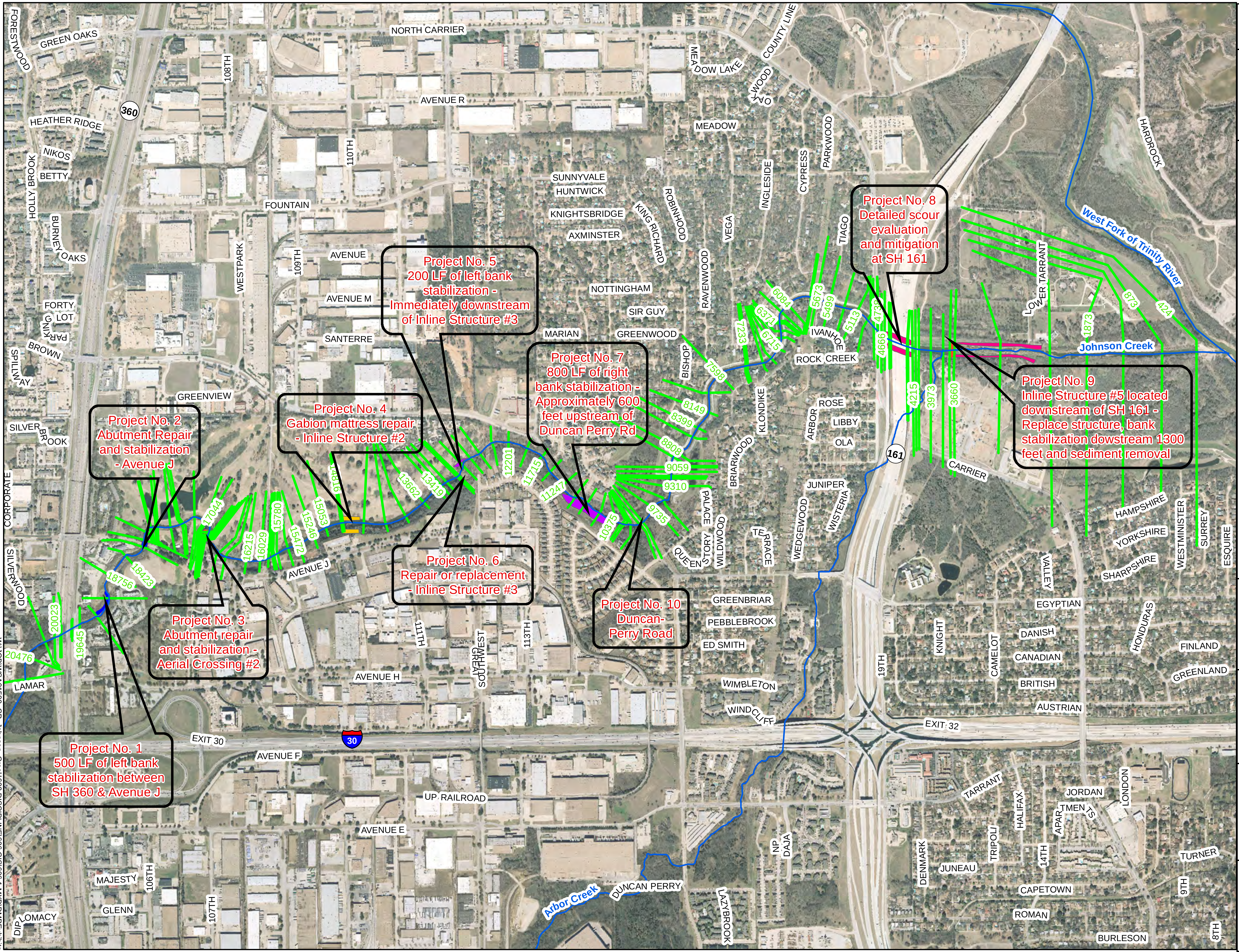



**Figure IX-10. Determination of Erosion Hazard Setback**




# Figure IX-11

## Johnson Creek Mitigation Alternatives

### Legend

- Stream Centerline
- Cross Sections
- Gabion Block
- Gabion Matress
- Rock Rip Rap
- Structural Repair

0 1,200 2,400
Scale in Feet

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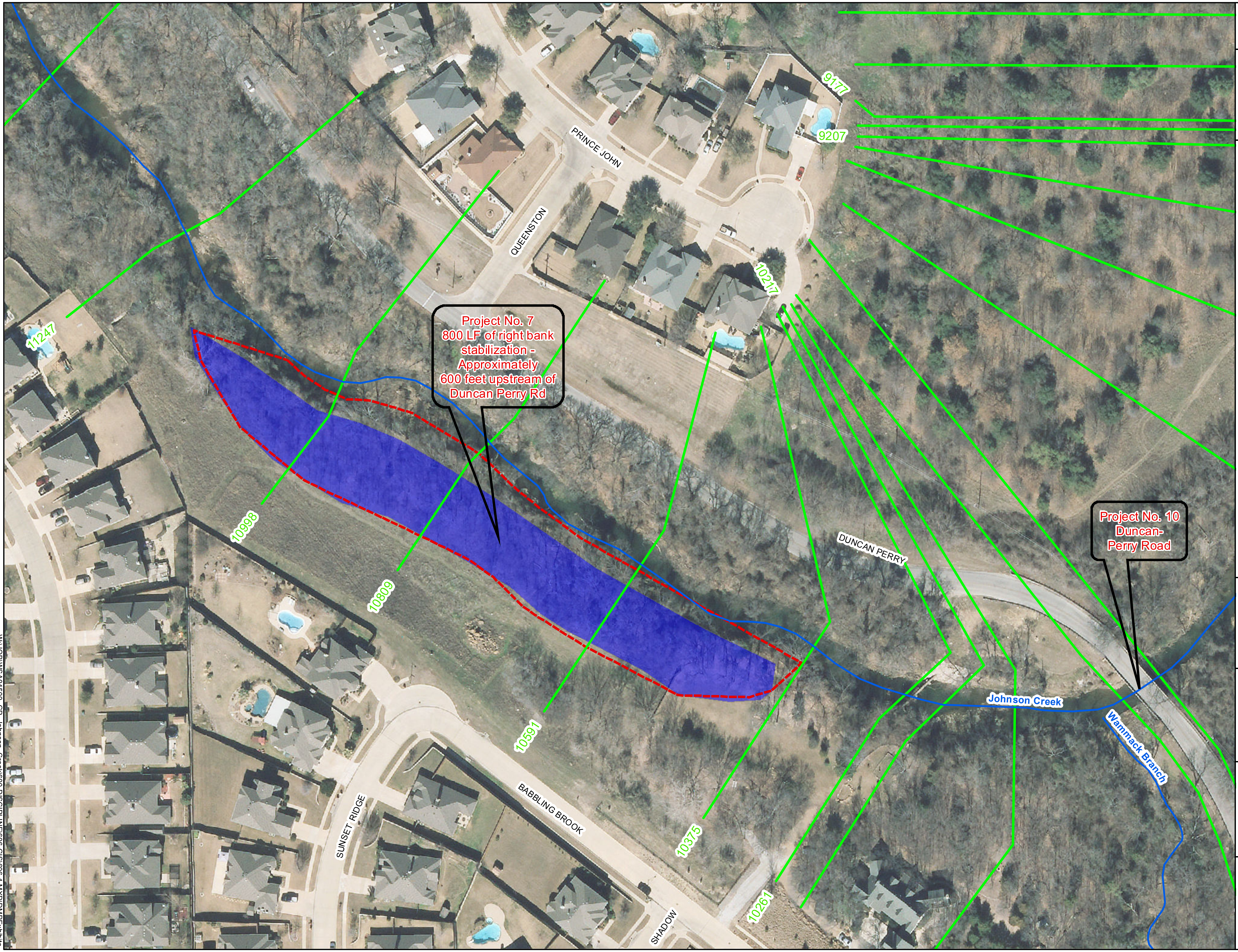
# Figure IX-11-1

## Johnson Creek Mitigation Alternatives Reach 1




### Legend

- Stream Centerline
- Cross Sections
- Erosion Hazard Setback
- Gabion Block

0 100 200

Scale in Feet

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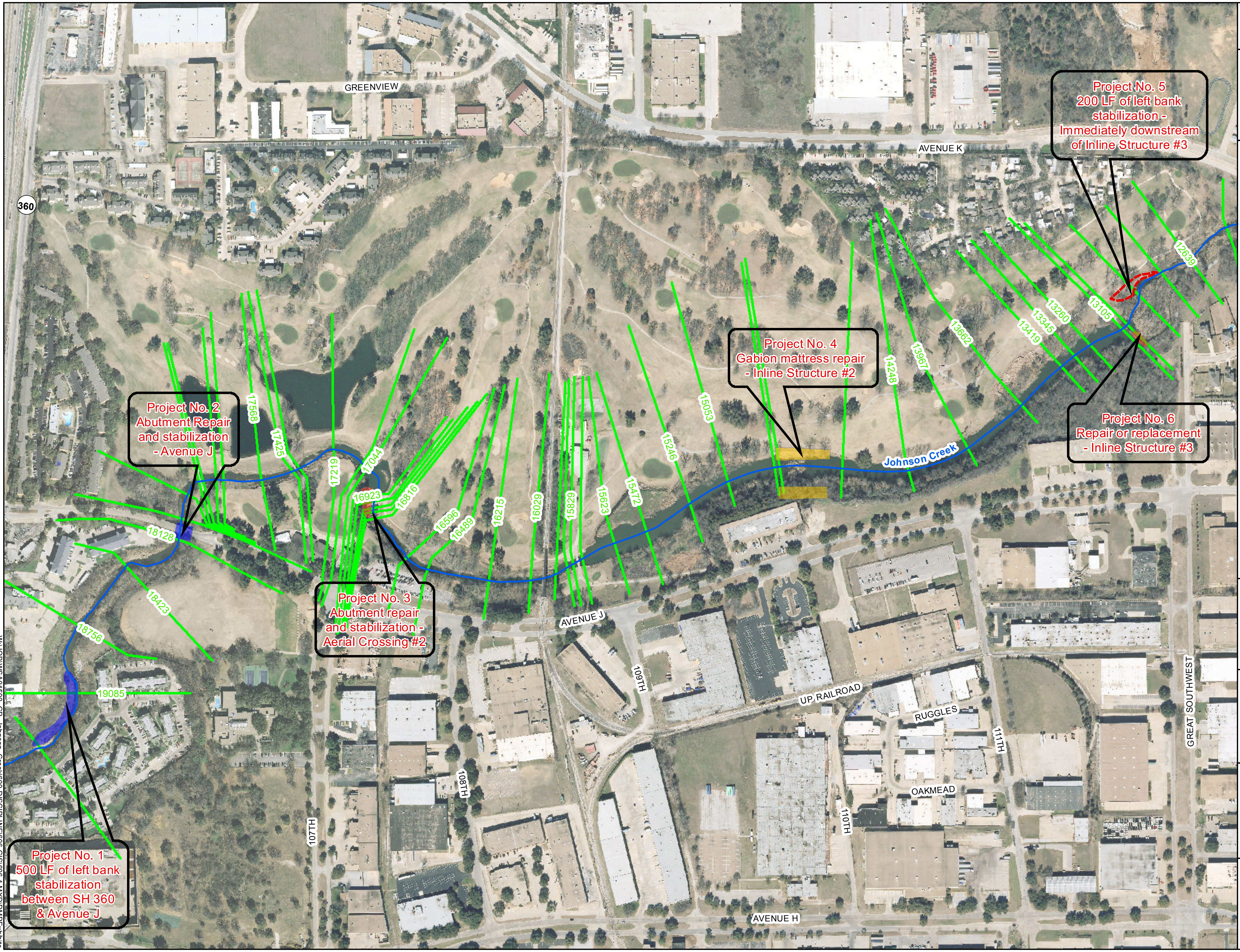


**Figure IX-11-2**

**Johnson Creek**
**Mitigation Alternatives**
**Reach 2**

**Legend**

- Stream Centerline
- Cross Sections
- Erosion Hazard Setback
- Gabion Block
- Gabion Matress
- Structural Repair

0 400 800
Scale in Feet

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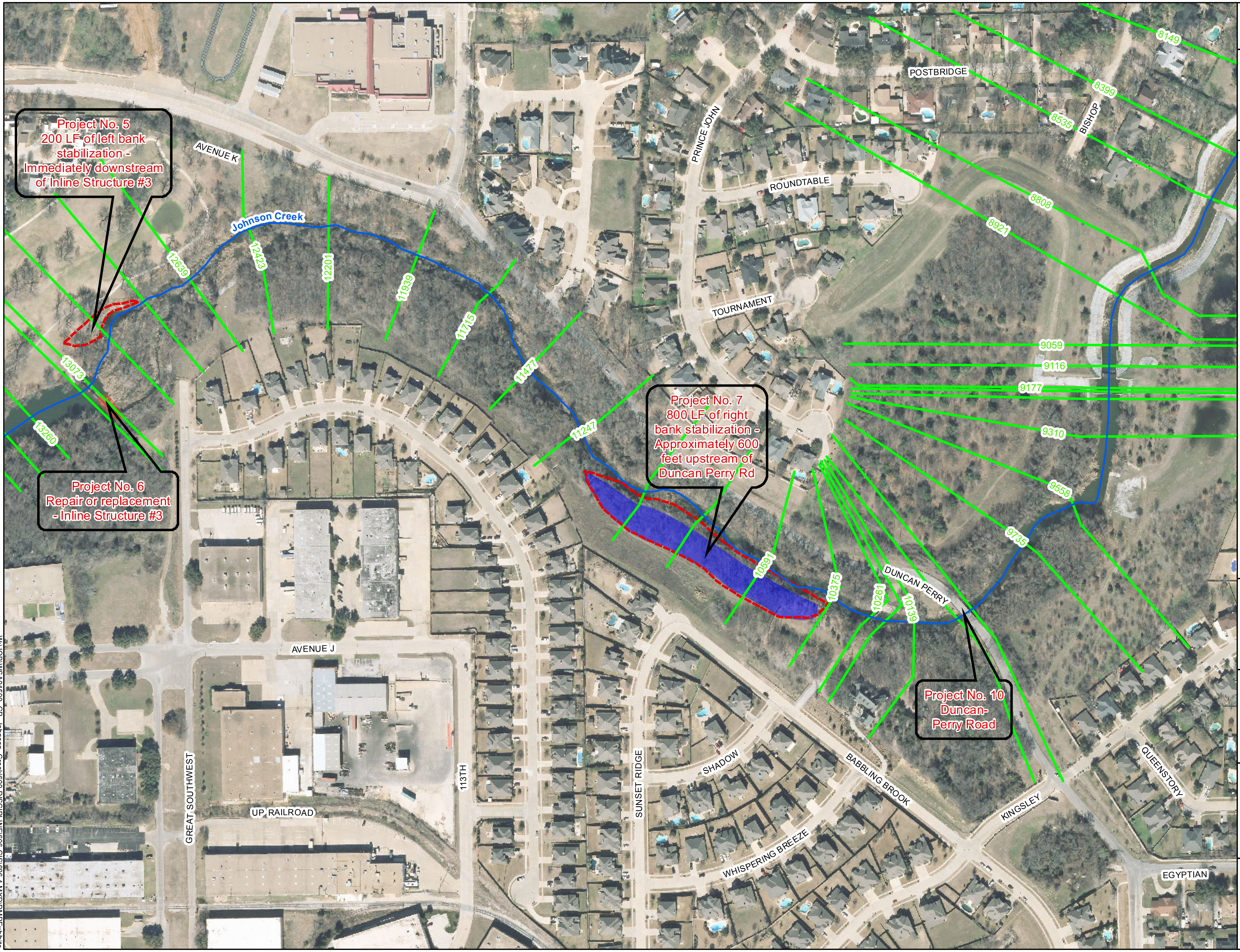
# Figure IX-11-3

## Johnson Creek Mitigation Alternatives Reach 3




**Legend**

- Stream Centerline
- Cross Sections
- Erosion Hazard Setback
- Gabion Block
- Structural Repair

0 250 500

Scale in Feet

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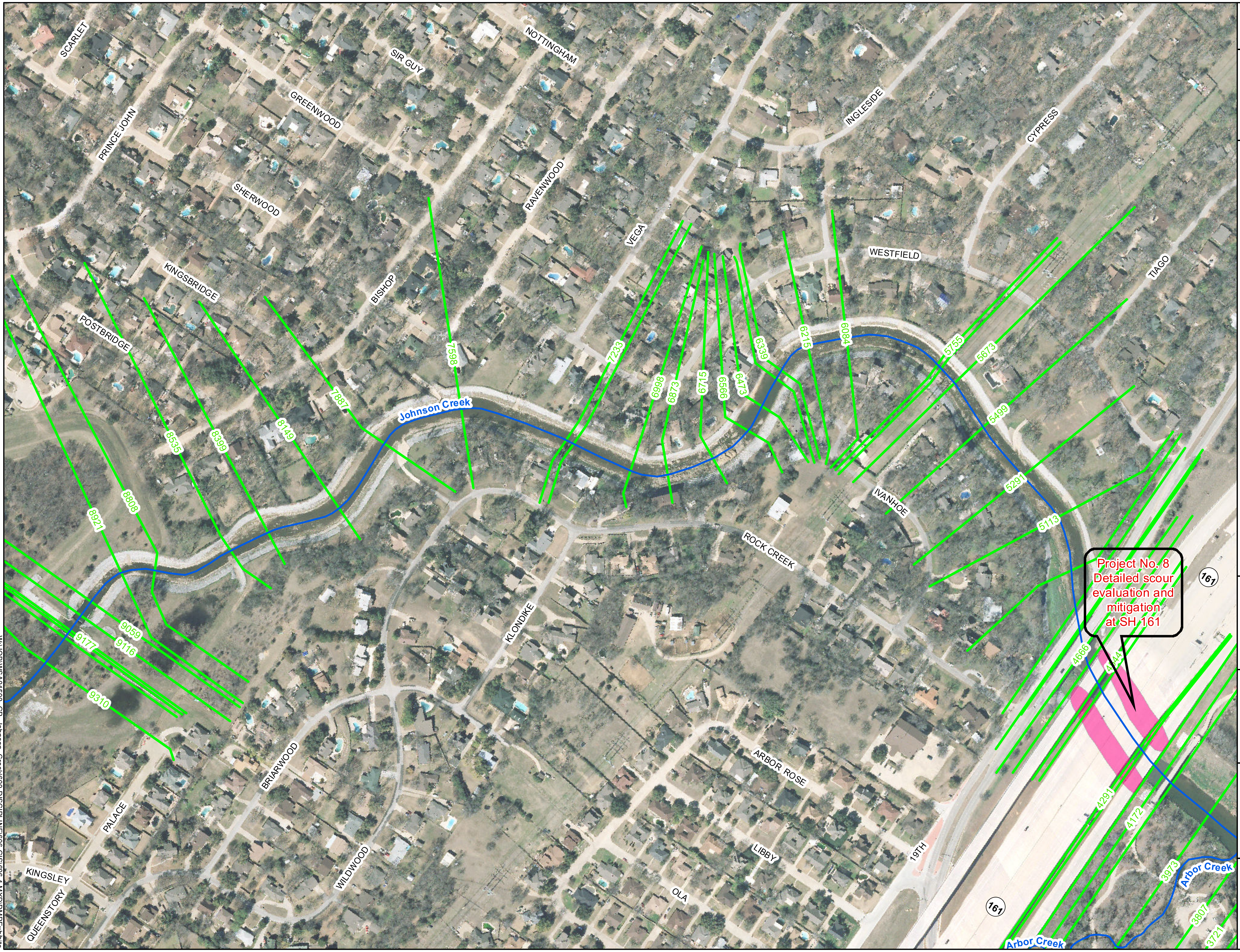


# Figure IX-11-4

## Johnson Creek Mitigation Alternatives Reach 4

**Legend**

- Stream Centerline
- Cross Sections
- Rock Rip Rap

0 300 600

Scale in Feet

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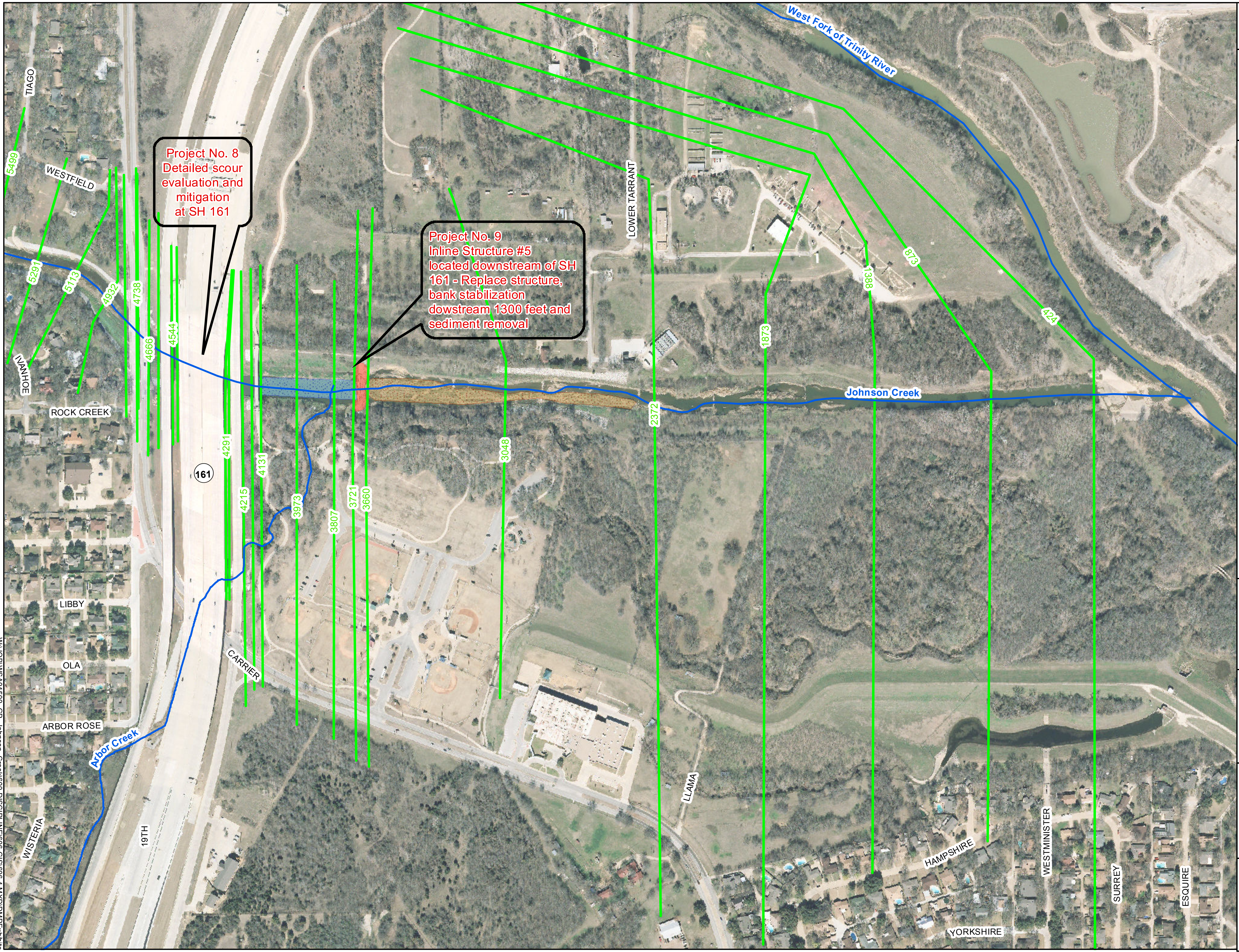


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**Figure IX-11-5**

**Johnson Creek**
**Mitigation Alternatives**
**Reach 5**

**Legend**

- Stream Centerline
- Cross Sections
- Sediment Dredge
- Sediment Removal
- Structural Repair

0 400 800

Scale in Feet

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# X. DAMS / LEVEES / DETENTION / DRAINAGE REVIEWS

## A. Dams, Levees, and Detention Facilities

Inspections of Dorchester Levee and the levee located at the upstream end of the USACE channel in the left overbank (Berm) are not included in this study. The USACE channel plans with the Berm design are also included in **Appendix B** with other hydraulic data. Halff provided the previous Dorchester Levee re-certification documentation dated February 2005 which is included in **Appendix E**.

## B. Drainage Reviews

No drainage reviews are included in the study report. No detention facilities are known to be present in this watershed within the City of Grand Prairie.

# XI. STORM DRAIN OUTFALL ASSESSMENT

There are 36 storm drains that outfall directly into Johnson Creek within the City of Grand Prairie. The majority of these outfalls are circular pipes. Box culverts and flumes also drain into Johnson Creek. The condition of each of these outfalls is assessed and ranked according to the urgency at which concerns need to be addressed. The results of this assessment are listed in **Table XI-1** providing each outfall's condition, assessment criteria category, and ranking. **Figure XI-1**, the Storm Drain Outfall Location Map displays the location of each structure. Photographs of the outfall are contained in **Appendix D** (Storm Drain Outfalls).

## A. Assessment Resources

The initial ranking of each outfall is determined by the following resources:

- City of Grand Prairie Drainage Design Manual (Jan. 2013), which notes City requirements for storm drain outfalls.
- The City database of field-checked storm drains outfalls, which provides information about the condition of the documented outfalls.
- Photos obtained from field observations performed in January 2014. In addition, Alan Plummer Associates, Inc. provided photographs of 25 of the outfalls which are obtained during the Summer of 2012.
- Jacobs Engineering, Inc. Site visit during January 2014 provided additional field observations of some structures identified from review of the Alan Plummer data.

## B. Condition and Criteria

Each storm drain outfall is assigned a condition and an assessment criteria category based on recommendations provided in the 2010 Drainage Master Plan Road Map. The four conditions include 1) Good (requires no remedial measures/continue normal inspections, 2) Fair (may require some remedial measures that may not immediate), 3) Poor (requires immediate remedial measures), and 4) Failure (requires design and/or construction in order to correct the problem).

The outfall deficiency criteria provided by the City was assigned to each outfall from one or more of the following: 1) structural, 2) no headwall, 3) scour, 4) siltation, or 5) aesthetics. After each storm drain outfall is assessed based on condition and criteria, a numerical ranking is assigned based on the need for repair (1 being the highest priority). A brief description of each category is given below.

- Outfalls assigned to the structural criteria category have experienced a structural failure or visibly significant degradation of the structure, including large cracks and spalls or exposed steel.
- The scour criteria category is given to outfalls experiencing erosion or scour either from storm water draining from the storm drain system or from flows in the receiving creek.
- Outfalls under the siltation criteria category have excessive amounts of sediment deposits that are reducing its conveyance capacity.

- Outfalls are assigned to the no headwall criteria category if the outfall is constructed without a headwall on the outfall pipe. The City Drainage Design Manual requires all inlets and outfalls on closed conduits to be constructed with City standard or TxDOT standard headwalls.
- For outfalls assigned to the aesthetics criteria category, the appearance of the structure is negatively impacted, requiring maintenance. Some examples of this included downed trees near the outfall, trash or leaf litter or signs of vandalism such as graffiti.

### C. Field Check

Field observations of each outfall are made in the summer of 2012 and in January 2014 to take photographs and document current conditions. **Table XI-1** is a summary of the condition assessment of each outfall.

**Table XI-1**
**Storm Drain Outfall Assessment**

| Overall Ranking | Map Photo No. | Photo Date | Type | Condition | Criteria | Comments | Estimated Cost Range* |
|---|---|---|---|---|---|---|---|
| 1 | 367 | 1/28/2014 | Unknown | Failure | Structural | Outfall appears to be completely covered with debris; unable to locate in field | $7,500 to $30,000 |
| 2 | 1027 | 1/28/2014 | Storm drain outfall - pipe | Failure | Structural, no headwall | Downstream most RCP joints have failed and the outfall is no longer connected to the storm drain properly; final pipe joint is obstructing flow in the channel | $20,000 to $30,000 |
| 3 | 609 | 6/8/2012 | Storm drain outfall - pipe | Poor | Siltation & aesthetic | Siltation and debris in channel blocking approximately one-third of outfall | $3,000 to $7,500 |
| 4 | 607 | 7/10/2012 | Storm drain outfall - pipe | Poor | Siltation | Heavy siltation blocking approximately one-third of outfall | $5,000 to $7,500 |
| 5 | 893b | 1/28/2014 | Storm drain outfall - pipe | Poor | No headwall, siltation, structural | Located above 893a; concrete cracks above outfall, siltation blocking bottom one-third of outfall | $7,500 to $15,000 |

**Table XI-1**
**Storm Drain Outfall Assessment (continue)**

| Overall Ranking | Map Photo No. | Photo Date | Type | Condition | Criteria | Comments | Estimated Cost Range* |
|---|---|---|---|---|---|---|---|
| 6 | 1042 | 1/28/2014 | Storm drain outfall - pipe | Poor | Scour & aesthetic | Tree limbs in outfall, approximately 2 feet of scour downstream of outfall beginning to undermine concrete foundation | $6,000 to $10,000 |
| 7 | 938 | 7/11/2012 | Storm drain outfall - pipe | Poor | Scour, no headwall | Scour beneath and around outfall | $6,000 to $10,000 |
| 8 | 1041 | 7/9/2012 | Storm drain outfall - pipe | Poor | Structural, scour, no headwall | End joints of RCP outfall separated; slight scour downstream of outfall | $10,000 to $15,000 |
| 9 | 935 | 1/28/2014 | Storm drain outfall - pipe | Poor | No headwall, structural | Apron downstream of outfall is cracking, vegetation and leaf litter in outfall | $10,000 to $15,000 |
| 10 | 902 | 7/3/2012 | Storm drain outfall - pipe | Poor | Siltation | Siltation partially blocking outfall | $4,000 to $6,000 |
| 11 | 864 | 6/11/2012 | Storm Drain Outfall - Box | Poor | Siltation & debris | Siltation blocking approximately bottom one-foot of outfall; debris in channel downstream of outfall | $6,000 to $10,000 |
| 12 | 610 | 6/8/2012 | Storm drain outfall - pipe | Fair | Aesthetic | Debris and vegetation downstream of outfall | $1,000 to $1,500 |
| 13 | 747 | 6/11/2012 | Storm drain outfall - pipe | Fair | Structural | Concrete cracks on downstream end of outfall; structure appears to be functioning properly | $1,000 to $1,500 |

**Table XI-1**
**Storm Drain Outfall Assessment (continue)**

| Overall Ranking | Map Photo No. | Photo Date | Type | Condition | Criteria | Comments | Estimated Cost Range* |
|---|---|---|---|---|---|---|---|
| 14 | 1182 | 7/9/2012 | Storm drain outfall - pipe | Fair | Aesthetic, scour, no headwall | Slight scour downstream of outfall with some vegetation overgrowth; structure not threatened | $1,000 to $1,500 |
| 15 | 936 | 7/9/2012 | Storm drain outfall - pipe | Fair | Scour, no headwall | Significant scour downstream of outfall; structure not threatened | $1,000 to $1,500 |
| 16 | 1181 | 7/9/2012 | Storm drain outfall - pipe | Fair | Scour, no headwall | Slight scour outside of outfall | $1,000 to $1,500 |
| 17 | 461 | 1/28/2014 | Storm drain outfall - pipe | Fair | Aesthetic, scour | Graffiti above headwall; slight scour under headwall | $1,000 to $1,500 |
| 18 | 1180 | 7/14/2012 | Storm drain outfall - pipe | Fair | Scour, no headwall | Moderate scour below outfall; structure not threatened | $2,000 to $4,000 |
| 19 | 643 | 6/6/2012 | Storm drain outfall - pipe | Fair | No headwall | Headwall installation needed | $2,500 to $5,000 |
| 20 | 865 | 6/11/2012 | Storm drain outfall - pipe | Fair | Debris | Debris in channel downstream of outfall | $1,000 to $1,500 |
| 21 | 667 | 6/11/2012 | Storm drain outfall - box | Fair | Siltation | Minor siltation on bottom of outfall | $2,000 to $4,000 |
| 22 | 666 | 6/11/2012 | Storm drain outfall - box | Fair | Siltation | Minor siltation on bottom of outfall | $1,000 to $1,500 |
| 23 | 349 | 6/18/2012 | Storm drain outfall - flume | Fair | Aesthetic | Vegetation and minor siltation in outfall | $1,000 to $1,500 |

| Table XI-1<br>Storm Drain Outfall Assessment (continue) | | | | | | | |
|---|---|---|---|---|---|---|---|
| **Overall Ranking** | **Map Photo No.** | **Photo Date** | **Type** | **Condition** | **Criteria** | **Comments** | **Estimated Cost Range*** |
| 24 | 893a | 1/28/2014 | Storm drain outfall - pipe | Fair | Aesthetic | Debris and loose vegetation at outfall | $1,000 to $1,500 |
| 25 | 561 | 7/3/2012 | Storm drain outfall - pipe | Fair | Aesthetic | Slight vegetation overgrowth around outfall | $1,000 to $1,500 |
| 26 | 892 | 1/28/2014 | Storm drain outfall - pipe | Fair | Aesthetic | Debris and loose vegetation at outfall | $1,000 to $1,500 |
| 27 | 560 | 7/10/2012 | Storm drain outfall - pipe | Fair | Aesthetic | Debris and minor amounts of vegetation overgrowth | $1,000 to $1,500 |
| 28 | 571 | 6/8/2012 | Storm drain outfall - box | Fair | Aesthetic | Vegetation and downed limbs in outfall | $1,000 to $1,500 |
| 29 | 672 | 1/28/2014 | Storm drain outfall - pipe | Fair | Aesthetic | Minor overgrown vegetation | $1,000 to $1,500 |
| 30 | 953 | 7/9/2012 | Storm drain outfall - pipe | Fair | Aesthetic | Some vegetation overgrowth; structure not threatened | $1,000 to $1,500 |
| 31 | 569 | 6/8/2012 | Storm drain outfall - pipe | Good | - | - | - |
| 32 | 608 | 7/10/2012 | Storm drain outfall - pipe | Good | - | - | - |
| 33 | 345 | 6/8/2012 | Storm drain outfall - pipe | Good | - | - | - |
| 34 | 1090 | 6/11/2012 | Storm drain outfall - | Good | - | - | - |

**Table XI-1**
**Storm Drain Outfall Assessment (continue)**

| Overall Ranking | Map Photo No. | Photo Date | Type | Condition | Criteria | Comments | Estimated Cost Range* |
|---|---|---|---|---|---|---|---|
| | | | flume | | | | |
| 35 | 128 | 7/3/2012 | Storm drain outfall - pipe | Good | - | - | - |
| 36 | 1093 | 1/28/2014 | Storm drain outfall - pipe | Good | - | - | - |

*Cost estimate ranges are based on typical installations and repairs for each of the criteria. Actual cost may exceed these estimates depending on project conditions.

### D. Outfall Summary and Recommendations

The outfall (Map Number 367) requiring the highest priority is located approximately 550 feet upstream of the Union Pacific Railroad on the south side of the channel. While the City outfall inventory indicates an outfall at this location, field observers could not locate it. However, a substantial amount of concrete debris and broken slabs are observed at this location indicating that perhaps the outfall is underneath this debris. If this is the case, the outfall is non-functional and can become a cause of flooding in the upstream storm drain system. The debris at this location requires clearing to ensure proper conveyance from this outfall, or the structure needs to be reconstructed.

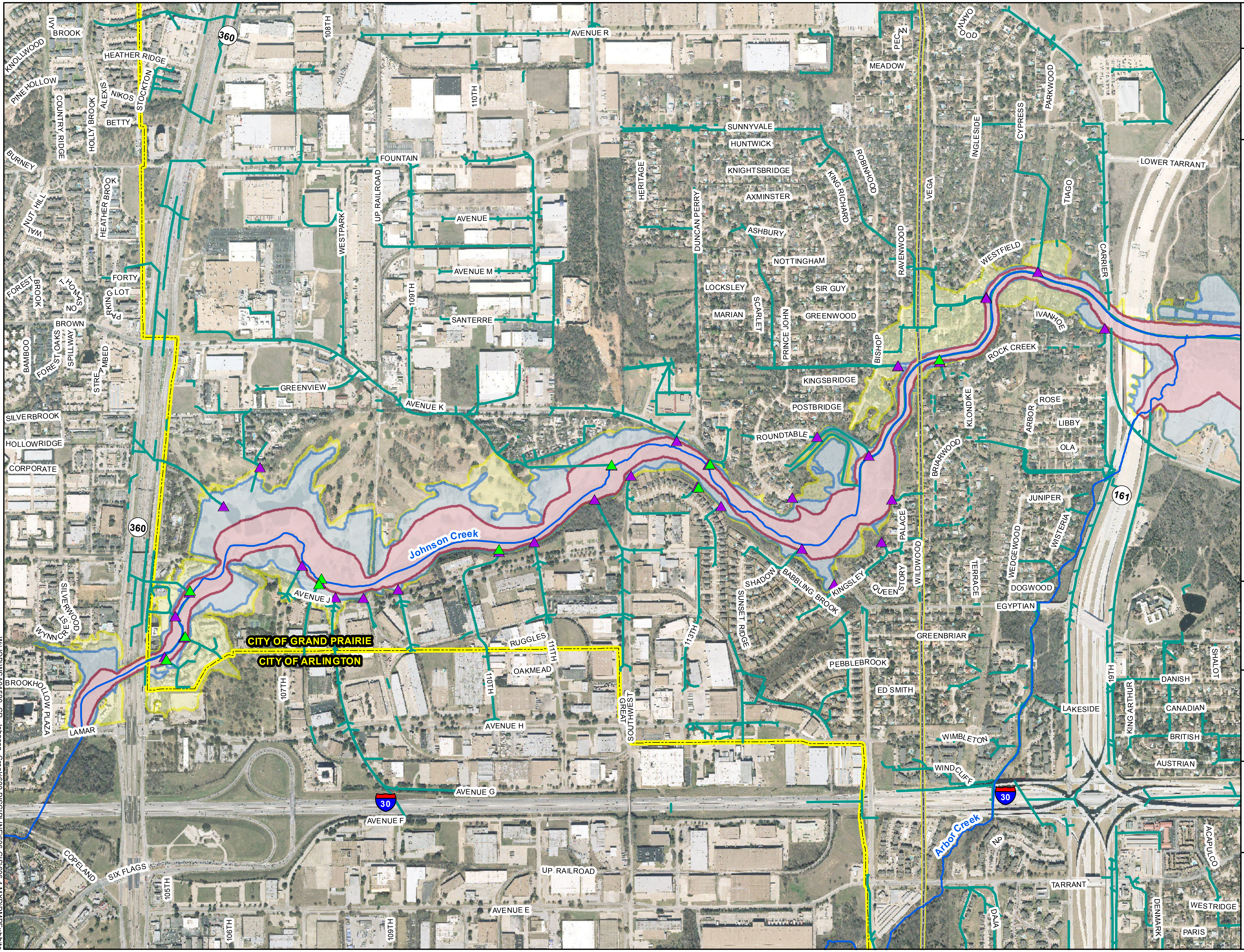
The storm drain outfall (Map Number 1027) located approximately 250 feet downstream of Inline Structure #3 has also failed. The last pipe segment has become completely detached and it is lying on the north side of the channel blocking conveyance in the channel. It is recommended that either this section is removed and a new headwall is built or that the entire outfall is reconstructed.

The City should proceed with maintenance of the remaining outfalls along Johnson Creek classified as 'Poor' (eight, in total). These structures appear to be at risk of either structural damage or reduced conveyance capacity of flood flows due to significant silt deposition and structural damage. Outfalls rated as ‘Fair’ require remedial maintenance and continued field inspections to ensure that the condition does not worsen. Outfalls with a ‘Good’ rating do not require maintenance at this time, but regular inspections need to occur to ensure that the rating does not deteriorate in the future.

The following lists the recommended maintenance required for each of the outfall deficiency criteria.

- For outfalls requiring structural maintenance, evaluate necessary repairs to determine whether outfall replacement is necessary. Restore outfall to adequate operable condition and install erosion protection to prevent future or additional undermining. Design of any replacement structure or structure repairs should be in accordance with the City of Grand Prairie standards. The estimated cost is $7,500 to $30,000.

- Siltation blocking or materials restricting flow from the outfall should be removed. Scour protection should be designed to adequately protect structural integrity of the outfall and to prevent erosion and siltation downstream. Design guidelines for protection around outfalls can be found in the North Central Texas Council of Governments iSWM Technical Manual Section 4.0 and the City of Grand Prairie Drainage Manual. The estimated cost for erosion control and siltation removal range from $1,000 to $7,500 per outfall.
- Outfalls that are under the Aesthetic condition criteria will require the removal of accumulated debris including vegetation, downed trees and garbage from the outfall structure and nearby. Repair superficial defects to the outfall structure such as displaced riprap, vandalism or overgrown vegetation. The estimated cost for aesthetic maintenance is $1,000 to $1,500 per outfall.
- All outfalls, whether already repaired, scheduled for repair, or categorized as 'Good' in this report should be monitored on a regular basis as scheduled by the City.




# Figure XI-1

## Johnson Creek Location of Storm Drain Outfalls

**Legend**

- Outfall Locations Verified by City
- Outfall Locations Verified by Jacobs
- Stream Centerline
- Storm Drain Lines
- City Limits
- County Boundary
- Revised Floodway
- Revised 1% AC Floodplain
- Revised 0.2% AC Floodplain

AC = Annual Chance

0 1,000 2,000
Scale in Feet

N
E
S
W

JACOBS

Grand Prairie
TEXAS

DMP for
Johnson Creek Corridor

W:\JOBWSA01500_GP_Johnson_Creek\600 DISCIPLINE\606 GIS\606.1 MXD\DMP Exhibits

## XII. PRELIMINARY QUANTITIES/ESTIMATES OF PROBABLE COST

Opinions of probable construction costs are prepared for the alternatives discussed in Section VII and for the proposed structural measures for channel stability discussed in Section IX. For the structural measures to ensure stream stability, costs are grouped by reach, as discussed in Section IX.

The following opinions about probable construction costs for project numbers 1, 2, 5, 7, 9, and 10 are in **Tables XII-1** through **XII-6**, respectively, are based on recent bid tabulations, discussions with contractors, and experience with similar projects in this area. These quantities are based on conceptual designs, which will require additional analysis and permitting before final design and construction documents can be prepared. This process is likely to change the final design and thus the actual construction costs which will be developed as “Engineer’s opinion about probably construction cost”.  Project numbers 3, 4, 6 and 8 are not given a cost as each project lies with privately maintained property and are not the City’s responsibility. These were noted projects as part of this analysis, but no costs are provided. Each property owner was made aware by the City of Grand Prairie of the current conditions evaluated.

All construction cost estimates include a 25 percent contingency cost. Industry standard for 30% design is a contingency of 20 percent and this is in study or feasibility phase therefore, 25% is used. Design fees are based on percentages of the construction cost with contingencies. The design fee percentage used is 15% based on the project cost. This could vary due to smaller projects may require a higher percentage for design than the larger projects. Costs are prepared for year 2014 dollars.

| ITEM No. | DESCRIPTION | QUANTITY | UNIT | UNIT PRICE | TOTAL |
|---|---|---|---|---|---|
| | **Table XII-1<br>Probable Construction Cost** | | | | |
| | DATE: January 2014 | | | | |
| | **City of Grand Prairie Master Drainage Plan for Johnson Creek** | | | | |
| | **Bank Stabilization Between SH 360 and Avenue J** | | | | |
| | **Project #1** | | | | |
| 1 | Mobilization | 1 | LS | $ 15,000.00 | $ 15,000.00 |
| 2 | Temporary access road | 350 | LF | $ 25.00 | $ 8,750.00 |
| 3 | Erosion control | 1 | LS | $ 6,000.00 | $ 6,000.00 |
| 4 | Divert water | 1 | LS | $ 5,000.00 | $ 5,000.00 |
| 5 | Clear and grub | 1500 | SY | $ 3.00 | $ 4,500.00 |
| 6 | Tree removal | 15 | EA | $ 1,500.00 | $ 22,500.00 |
| 7 | Gabions with tie-backs | 60 | CY | $ 375.00 | $ 22,500.00 |
| 8 | Gabions without tie-backs | 800 | CY | $ 300.00 | $ 240,000.00 |
| 9 | Grading - Fill | 715 | CY | $ 20.00 | $ 14,300.00 |
| 10 | Grading- Cut | 202 | CY | $ 15.00 | $ 3,028.00 |
| 11 | Hydromulch | 2,500 | SY | $ 3.00 | $ 7,500.00 |
| | Construction Subtotal | | | | $ 349,078.00 |
| | Approximate 25% Contingency | | | | $ 87,270.00 |
| | Construction Total | | | | $ 436,348.00 |
| | | | | | |
| | Engineering, Survey, and Environmental for Design | | | Approximately 15% | $ 65,452.00 |
| | | | | | |
| | Project Total | | | | **$ 501,800.00** |
| | | | | | |
| | Total Annual Cost (4% Interest) | | | | $ 20,072.00 |

Quantities are based on a concept design and are subject to plan revisions and field conditions. Costs are in year 2014 dollars.

Unit prices shown herein are from recent bid tabulations of projects in the general area of the subject project and input from contractors with experience in this type of work. Because of the size and nature of this project, unit prices (and therefore the total cost) are subject to substantial variation, dependant on market conditions, and current availability of qualified, interested contractors, as well as other typical factors.

This document is released for the purpose of providing a concept opinion of probable construction cost. It is not to be used for construction, bidding, or permit purposes.

**Table XII-2**
**Probable Construction Cost**

DATE: January 2014

**City of Grand Prairie Master Drainage Plan for Johnson Creek**

## Abutment Repair and Stabilization - Avenue J

**Project #2**

| ITEM No. | DESCRIPTION | QUANTITY | UNIT | UNIT PRICE | TOTAL |
|---|---|---|---|---|---|
| 1 | Mobilization | 1 | LS | $ 15,000.00 | $ 15,000.00 |
| 2 | Temporary access road | 75 | LF | $ 25.00 | $ 1,875.00 |
| 3 | Erosion control | 1 | LS | $ 6,000.00 | $ 6,000.00 |
| 4 | Traffic control | 60 | DAY | $ 75.00 | $ 4,500.00 |
| 5 | Divert water | 1 | LS | $ 6,000.00 | $ 6,000.00 |
| 6 | Gabions with tie-backs | 72 | CY | $ 375.00 | $ 27,000.00 |
| 7 | Gabions without tie-backs | 152 | CY | $ 300.00 | $ 45,600.00 |
| 8 | Grading - Fill | 125 | CY | $ 20.00 | $ 2,509.00 |
| 9 | Grading- Cut | 4 | CY | $ 15.00 | $ 60.00 |
| | Construction Subtotal | | | | $ 108,544.00 |
| | Approximate 25% Contingency | | | | $ 27,136.00 |
| | Construction Total | | | | $ 135,680.00 |
| | | | | | |
| | Engineering, Survey, and Environmental for Design | | | Approximately 15% | $ 20,352.00 |
| | | | | | |
| | Project Total | | | | **$ 156,032.00** |
| | | | | | |
| | Total Annual Cost (4% Interest) | | | | $ 6,241.00 |

Quantities are based on a concept design and are subject to plan revisions and field conditions. Costs are prepared in year 2014 dollars.

Unit prices shown herein are from recent bid tabulations of projects in the general area of the subject project and input from contractors with experience in this type of work. Because of the size and nature of this project, unit prices (and therefore the total cost) are subject to substantial variation, dependant on market conditions, and current availability of qualified, interested contractors, as well as other typical factors.

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**Table XII-3**
**Probable Construction Cost**

DATE: January 2014

**City of Grand Prairie Drainage Master Plan for Johnson Creek Corridor**

## Bank Stabilization JP#5

### Project # 5 - Immediately downstream of Inline Structure #3

| ITEM No. | DESCRIPTION | QUANTITY | UNIT | UNIT PRICE | TOTAL |
|---|---|---|---|---|---|
| 1 | Mobilization | 1 | LS | $ 15,000.00 | $ 15,000.00 |
| 2 | Temporary access road | 75 | LF | $ 25.00 | $ 1,875.00 |
| 3 | Erosion control | 1 | LS | $ 6,000.00 | $ 6,000.00 |
| 4 | Divert water | 1 | LS | $ 5,000.00 | $ 5,000.00 |
| 5 | Clear and grub | 2700 | SY | $ 7.00 | $ 18,900.00 |
| 6 | Gabions with tie-backs | 48 | CY | $ 375.00 | $ 18,000.00 |
| 7 | Gabions without tie-backs | 300 | CY | $ 300.00 | $ 90,000.00 |
| 8 | Grading - Fill | 128 | CY | $ 20.00 | $ 2,560.00 |
| 9 | Grading- Cut | 4 | CY | $ 15.00 | $ 67.00 |
| 10 | Hydromulch | 500 | SY | $ 3.00 | $ 1,500.00 |
| | Construction Subtotal | | | | $ 158,902.00 |
| | Approximate 25% Contingency | | | | $ 39,726.00 |
| | Construction Total | | | | $ 198,628.00 |
| | | | | | |
| | Engineering, Survey, and Environmental for Design Approximately 15% | | | | $ 29,794.00 |
| | | | | | |
| | Project Total | | | | **$ 228,422.00** |
| | | | | | |
| | Total Annual Cost (4% Interest) | | | | $ 9,137.00 |

Quantities are based on a concept design and are subject to plan revisions and field conditions.

Unit prices shown herein are from recent bid tabulations of projects in the general area of the subject project and input from contractors with experience in this type of work. Because of the size and nature of this project, unit prices (and therefore the total cost) are subject to substantial variation, dependant on market conditions, and current availability of qualified, interested contractors, as well as other typical factors.

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**Table XII-4**
**Probable Construction Cost**

DATE: January 2014

**City of Grand Prairie Drainage Master Plan for Johnson Creek Corridor**

## Bank Stabilization JP*#7

### Project #7 - Approx. 600 ft upstream of Duncan Perry Road

| ITEM No. | DESCRIPTION | QUANTITY | UNIT | UNIT PRICE | TOTAL |
|---|---|---|---|---|---|
| 1 | Mobilization | 1 | LS | $ 15,000.00 | $ 15,000.00 |
| 2 | Temporary access road | 350 | LF | $ 25.00 | $ 8,750.00 |
| 3 | Erosion control | 1 | LS | $ 6,000.00 | $ 6,000.00 |
| 5 | Divert water | 1 | LS | $ 15,000.00 | $ 15,000.00 |
| 6 | Clear and grub | 1500 | SY | $ 3.00 | $ 4,500.00 |
| 7 | Tree removal | 15 | EA | $ 1,500.00 | $ 22,500.00 |
| 8 | Gabions with tie-backs | 80 | CY | $ 375.00 | $ 30,000.00 |
| 9 | Gabions without tie-backs | 1,669 | CY | $ 300.00 | $ 500,667.00 |
| 10 | Grading - Fill | 1,052 | CY | $ 20.00 | $ 21,046.00 |
| 11 | Grading- Cut | 146 | CY | $ 15.00 | $ 2,191.00 |
| 12 | Hydromulch | 3,875 | SY | $ 3.00 | $ 11,625.00 |
| | Construction Subtotal | | | | $ 637,279.00 |
| | Approximate 25% Contingency | | | | $ 159,320.00 |
| | Construction Total | | | | $ 796,599.00 |
| | | | | | |
| | Engineering, Survey, and Environmental for Design Approximately 15% | | | | $ 119,490.00 |
| | | | | | |
| | Project Total | | | | **$ 916,089.00** |
| | | | | | |
| | Total Annual Cost (4% Interest) | | | | $ 36,644.00 |

Quantities are based on a concept design and are subject to plan revisions and field conditions.

Unit prices shown herein are from recent bid tabulations of projects in the general area of the subject project and input from contractors with experience in this type of work. Because of the size and nature of this project, unit prices (and therefore the total cost) are subject to substantial variation, dependant on market conditions, and current availability of qualified, interested contractors, as well as other typical factors.

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**Table XII-5**
**Probable Construction Cost**
DATE: January 2014
**City of Grand Prairie Drainage Master Plan for Johnson Creek Corridor**

## Inline Structure #5 Repair, Deposition Removal and Bank Stabilization

**Project #9**

| ITEM No. | DESCRIPTION | QUANTITY | UNIT | UNIT PRICE | TOTAL |
|---|---|---|---|---|---|
| 1 | Mobilization | 1 | LS | $ 25,000.00 | $ 25,000.00 |
| 2 | Temporary access road | 1300 | LF | $ 25.00 | $ 32,500.00 |
| 3 | Temporary bridge | 1 | LS | $ 15,000.00 | $ 15,000.00 |
| 4 | Erosion control | 1 | LS | $ 15,000.00 | $ 15,000.00 |
| 5 | Divert water | 1 | LS | $ 50,000.00 | $ 50,000.00 |
| 6 | Clear and grub | 12,000 | SY | $ 3.00 | $ 36,000.00 |
| 7 | Tree removal | 10 | EA | $ 1,500.00 | $ 15,000.00 |
| 8 | Riprap stone (12-in median diameter) | 3,000 | CY | $ 120.00 | $ 360,000.00 |
| 9 | Remove inline structure | 1 | EA | $ 30,000.00 | $ 30,000.00 |
| 10 | Remove 12-inch water control valve | 1 | EA | $ 1,200.00 | $ 1,200.00 |
| 11 | Remove stilling basin | 1 | EA | $ 20,000.00 | $ 20,000.00 |
| 12 | Install 12-inch gate valve | 1 | EA | $ 8,500.00 | $ 8,500.00 |
| 13 | Install concrete inline structure | 1 | EA | $ 200,000.00 | $ 200,000.00 |
| 14 | Install stilling basin | 1 | EA | $ 75,000.00 | $ 75,000.00 |
| 15 | Sediment dredge | 3,000 | CY | $ 69.00 | $ 207,000.00 |
| 16 | Grading - Cut - Channel | 5,800 | CY | $ 25.00 | $ 145,000.00 |
| | Construction Subtotal | | | | $ 1,235,200.00 |
| | Approximate 25% Contingency | | | | $ 308,800.00 |
| | Construction Total | | | | $ 1,544,000.00 |
| | | | | | |
| | Engineering, Survey, and Environmental for Design | | | Approximately 15% | $ 231,600.00 |
| | | | | | |
| | Project Total | | | | **$ 1,775,600.00** |
| | | | | | |
| | Total Annual Cost (4% Interest) | | | | $ 71,024.00 |

Quantities are based on a concept design and are subject to plan revisions and field conditions. Costs are in 2014 dollars.

Unit prices shown herein are from recent bid tabulations of projects in the general area of the subject project and input from contractors with experience in this type of work. Because of the size and nature of this project, unit prices (and therefore the total cost) are subject to substantial variation, dependant on market conditions, and current availability of qualified, interested contractors, as well as other typical factors.

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**Table XII-6**
**Probable Construction Cost**

DATE: January 2014

**City of Grand Prairie Drainage Master Plan for Johnson Creek Corridor**

## Duncan Perry Road 100-Yr Bridge

**Project #10**

| ITEM No. | DESCRIPTION | QUANTITY | UNIT | UNIT PRICE | TOTAL |
|---|---|---|---|---|---|
| 1 | Proposed Bridge Construction | 10,000 | SF | $ 120.00 | $ 1,200,000.00 |
| 2 | Utility Relocation | 1 | LS | $ 500,000.00 | $ 500,000.00 |
| 3 | Traffic Control | 1 | LS | $ 48,000.00 | $ 48,000.00 |
| 4 | Construction of four-lane undivided roadway (M4U) | 6,111 | SY | $ 54.00 | $ 329,994.00 |
| 5 | Fill for the Construction of four-lane undivided roadway (M4U) | 22,011 | CY | $ 15.00 | $ 330,165.00 |
| 6 | Pedestrian Guard Rail | 300 | LF | $ 80.00 | $ 24,000.00 |
| 7 | Cuts to provide valley storage to mitigate the impact of fill | 22666 | CY | $ 15.00 | $ 339,990.00 |
| 8 | Cuts downstream of the bridge to accommodate bridge widening | 31852 | CY | $ 15.00 | $ 477,780.00 |
| 9 | Construction Mobilization | 1 | LS | $ 200,000.00 | $ 200,000.00 |
| | Construction Subtotal | | | | $ 3,449,929.00 |
| | Approximate 25% Contingency | | | | $ 862,482.00 |
| | Construction Total | | | | $ 4,312,411.00 |
| | | | | | |
| | Engineering, Survey, and Environmental for Design | | Approximately 15% | | $ 646,862.00 |
| | | | | | |
| | Project Total | | | | **$ 4,959,273.00** |
| | | | | | |
| | Total Annual Cost (4% Interest) | | | | $ 198,371.00 |

Quantities are based on a concept design and are subject to plan revisions and field conditions.

Unit prices shown herein are from recent bid tabulations of projects in the general area of the subject project and input from contractors with experience in this type of work. Because of the size and nature of this project, unit prices (and therefore the total cost) are subject to substantial variation, dependant on market conditions, and current availability of qualified, interested contractors, as well as other typical factors.
Utility relocation is estimated as lump sum based on possible relocation of 6’ and 54’ sewer lines and 18’ water line and Right of Way Costs and Easement Acquisition.

This document is released for the purpose of providing a concept opinion of probable construction cost. It is not to be used for construction, bidding, or permit purposes.

# XIII. EVALUATION & PRIORITIZATION/PHASING & IMPLEMENTATION

## A. Evaluation & Prioritization

Ten improvement alternatives have been developed for Johnson Creek Corridor to address issues such as roadway overtopping and stream instability. Four of the ten projects are determined not to be within the City's jurisdiction, but the data will be shared with the property owners. The first alternative is for improvements to Duncan Perry Road that would raise the road and increase the conveyance capacity of the bridge opening such that the crossing can handle 1% annual chance storm without being overtopped. The other five alternatives are structural measures to address stream instability or maintenance issues.

Each alternative is ranked based on the process described in Section II.G of the City of Grand Prairie Drainage Master Plan Road Map. **Table XIII-1** shows a summary of the ranking process. The Step 4 initial ranking process produced several ties. Step 5 in the ranking process is designed to break these ties, but is not helpful for deciding between two or more tying projects on the same reach of stream and it gives no weight to varying urgencies. Ties in the initial rankings, therefore, are broken by factors including cost and exigency. Also, some stream stability issues need repairs sooner than others, which fact is not accounted for in the standard Road Map method.

## B. Phasing & Implementation

1. Final Short-term Priorities Implementation

The City Road Map suggests that short-term priority Capital Improvement Projects (CIPs) could generally be described as those projects with an initial ranking factor of 1, 2, or 3. Two of the projects considered here meet the requirement of 3 and two projects with a 4 for medium size projects. Other considerations are warranted, particularly the likelihood of damage to infrastructure caused by an extended delay in consideration. It is recommended that the potential for damage caused by delay be considered in ranking projects. In order to fully develop the projects discussed, each will need to be compared to those in the Drainage Master Plans for the other watersheds to fully and properly prioritize them. It is recommended that the projects with a final ranking of one through four be given short-term consideration.

2. Final Long-term Plan Implementation

The projects with a final ranking of five or greater can be delayed in implementation and should therefore be given a long-term priority. These two projects should be monitored to determine the need to adjust its priorities. Phasing of portions of some of these projects may be warranted, particularly to protect utility crossings, when the full project cannot be immediately implemented. Duncan Perry Roadway was ranked as a priority 3 and a flood warning system may be warranted at this project location, which could be made priority as the cost would be much lower. Project number 6 is the replacement of inline structure #5 downstream of SH 161, which includes three sections; the upstream dredging and structure replacement, the downstream sedimentation removal and bank stabilization. These three tasks could be phased.

The following considerations should be given to projects that cannot be implemented in the short-term:

- Consider use of the flood warning system to protect citizens from road overtopping events until adequate funding can be obtained for road crossing improvements.
- Consider buy-outs of structures threatened by bank instability.
- Maintenance of threatened utilities to help mitigate damage.
- Routine inspection of facilities in and near the channel to detect potential problems and avert failure.
- Consider removal of certain facilities threatened by stream instability rather than implementing structural measures to protect them.

**Table XIII-1**
**Ranking Process**

| City Ranking | Project Number from Jacobs MDP 2014 | Capital Improvement Project Alternative | Watershed | Project Size & Short Term/Long-Term | Step 1 - Initial Ranking Factor - Estimate of Probable Cost vs. # Structures Benefited [1] | | | Step 2 - Second Ranking Factor - Cost to benefit of Roadway Number of Citizens Impacted [2] | | | | | | | Step 3 - Tax Value of Benefited Property Structures [7] | | Sum of 1st, 2nd, and 3rd Factors | Step 4 - Initial Rank | Step 5 - 100-Year Ultimate Discharge at CIP Location | | Step 6 - Final Rank |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | # Structures | Cost | 1st Factor [1] | Type | Roadway Flood Event Protection | Roadway % Citizens Protected [3] | Roadway % Citizens Impacted [4] | Roadway # Citizens Impacted [5] | Cost to Benefit Roadway # Citizens Impacted [6] | 2nd Factor | Tax Value of Property Structures Benefited | 3rd Factor | Total | Rank [8] | Ultimate Q100 (cfs) | Sorting [9] | Rank [10] |
| 1 | 2 | Abutment repair and stabilization - Avenue J | Johnson Creek | Small/Short-term | 0 | $156,032 | 3 | P4D | No Protection | 0 | 100 | 8450 | $18.47 | 1 | $0 | 20 | 24 | 1 | 17,912 | 1 | 1 |
| 2 | 5 | Bank stabilization immediately downstream of Inline Structure #3 | Johnson Creek | Small/Short-term | 0 | $228,422 | 3 | - | - | - | - | - | - | 3 | $0 | 20 | 26 | 2 | 18,041 | 2 | 2 |
| 3 | 10 | Duncan Perry bridge and roadway improvements | Johnson Creek | Large/Long-term | 2* | $4,959,273 | 5 | P4D | 5-Year | 85 | 15 | 1170 | $4,238.69 | 2 | $0 | 20 | 27 | 3 | 18,233 | 3 | 3 |
| 4 | 7 | Bank stabilization approx. 600 ft upstream of Duncan Perry Road | Johnson Creek | Medium/Long-term | 0 | $916,089 | 4 | - | - | - | - | - | - | 3 | $0 | 20 | 27 | 3 | 18,041 | 4 | 4 |
| 5 | 1 | Bank stabilization Between SH 360 and Avenue J | Johnson Creek | Medium/Long-term | 0 | $501,800 | 4 | - | - | - | - | - | - | 3 | $0 | 20 | 27 | 3 | 17,912 | 5 | 5 |
| 6 | 9 | Replacement of Inline Structure #5 and removal of deposited sediments | Johnson Creek | Large/Long-term | 1 | $1,775,600 | 5 | - | 10-year | - | - | - | - | 3 | $0 | 20 | 28 | 4 | 20,719 | 6 | 6 |

1 - Refer to City-Wide Drainage Master Plan Road Map, Section II.G - Implementation Plan - Step 1
2 - Refer to City-Wide Drainage Master Plan Road Map, Section II.G - Implementation Plan - Step 2
3 - Based on approximation, using logarithmic chart, with 1-year event coverage protecting 0% of traffic volume and 100-year event coverage protecting 100% of traffic volume
4 - Percent Impacted = 100% minus % of Roadway Citizens protected (approximate)
5 - Number Impacted = % Impacted multiplied by [No. Lanes * 4 hours Impacted * Hourly Volume Per Lane * Level of Service "C" Traffic Volume]
6 - Cost of CIP Divided by Roadway # Citizens Impacted
7 - Refer to City-Wide Drainage Master Plan Road Map, Section II.G - Implementation Plan - Step 3
8 - Refer to City-Wide Drainage Master Plan Road Map, Section II.G - Implementation Plan - Step 4
9 - Refer to City-Wide Drainage Master Plan Road Map, Section II.G - Implementation Plan - Step 5
10 - Refer to City-Wide Drainage Master Plan Road Map, Section II.G - Implementation Plan - Step 6

# XIV. SHORT TERM PRIORITIES & LONG TERM PLAN

## A. Short-Term Priorities Implementation

Of the projects listed in **Table XIII-1**, two are given lower priority in the Road Map methodology. However, all of the projects considered for Johnson Creek Corridor could be crucial to public safety. Many projects resulted in tie-in using the Road Map to prioritize, which required evaluation of the relative cost of the repair (e.g. an inexpensive repair is ranked higher than the costlier one), and the amount of public benefit.

Based on this analysis, the projects with final rankings of one through two should be considered short-term projects. The costs of these projects are likely to grow, if not addressed in the short-term, as additional damage occurs. If it is not feasible to complete these projects, as proposed, on a short-term basis, then consideration should be given to other means of accomplishing them or to removing the need of the project altogether. These options could include removal or relocation of the infrastructure in question, phasing the project, or regular maintenance to address deterioration on an ongoing basis.

If improvements are not feasible immediately to Duncan Perry Road a flood warning system should be considered to provide citizens protection during an impending flood event. A flood warning system cost is not determined in the ranking. This cost would rank the project priority differently with a much lower cost.

## B. Long-Term Plan Implementation

The proposed projects with final rankings of four through eight can be considered long-term projects. They should still need to be addressed within a reasonable time frame. Routine inspection and maintenance of these areas should be considered. This will also provide early indication of accelerating degradation and the need for adjusting priorities. Some projects may need to be pursued in phases or split up if the situation worsens or increases.

The lowest ranked alternatives are Duncan Perry bridge, bank stabilization approximately 600 ft upstream of Duncan Perry Road and bank stabilization between SH 360 at Avenue J and the replacement of Inline Structure #5 with sedimentation removal. These received lowest priorities due to cost and the critical infrastructure being impacted. There is serious channel degradation that should not be taken lightly. These alternatives should be considered in coordination with other relevant City departments to determine an appropriate course of action to protect City infrastructure.

As with short-term projects, consideration should be given to routine inspection that would allow for early detection of immediate threats to infrastructure. Immediate threats may warrant remedial maintenance, increasing the project's priority, or consideration of phasing the project to address the more immediate needs. Other approaches may eliminate the need for certain portions of the projects, such as buy-outs of structures or relocation of infrastructure.

## XV. MASTER PLAN STUDY WRAP-UP & RECOMMENDATIONS

The purpose of this Drainage Master Plan for Johnson Creek Corridor is to evaluate the hydrology of the entire watershed and assess conditions relating to flooding and channel instability along the main stem of Johnson Creek. The objective is to suggest measures to address and mitigate problem areas. This Plan provides details of the watershed and modeling results including delineation of the floodplain that can be generated from a store event with a 100-year (1% annual chance) return period under fully developed conditions of the watershed. A geomorphologic study is completed based on the hydraulic modeling, and measurements and observations made during field investigation. Results from these analyses are utilized to develop a set of recommendation improvements to address issues related to roadway overtopping, scouring, and channel instability. These recommendations, when combined with those from the other watersheds, can be used as the basis for a capital improvement program. The following sections summarize the recommendations given in the report.

### A. Streams and Open Channels

Floodplain maps developed in this study show that no habitable structures are in the 100-year floodplain. The under sizing of the bridge contributes to the overtopping of Duncan Perry Road, even during flood events that can result for a 50-year storm event. Consideration has been given to improvements to this area to reducing flooding and for mitigating the overtopping of Duncan Perry Road. These improvements are discussed in Section VII.

### B. Stream Stability

Section IX discusses stream instability issues and protective measures in greater detail.

### C. Improvement Project Prioritization

The proposed stream stability and bridge improvement project are ranked based on criteria discussed in the Drainage Master Plan Road Map. Details of these rankings are provided in Section XIII.

### D. Storm Drain Outfalls

Storm drain outfalls are assessed based on the current condition and then prioritized based on their maintenance requirements. Section XI provides more information on this process and the results.

### E. Other Drainage Facilities

Detention ponds, dams, and levees are not assessed with this study along the Johnson Creek corridor. Analysis of the storm sewer systems is not included in this study.

## F. Recommendations

Based on the analyses performed in this study the following recommendations are made. Each section in this report provides additional details for these recommendations.

- The City should enforce its floodplain development standards to ensure that new flooding problems do not originate.
- Future developments near the channel should consider the erosion hazard setback procedures outlined in Section IX, which are addressed as discussed in the Drainage Manual for the City of Grand Prairie.
- The City should consider the proposed improvements projects, which have been ranked in Section XIII.
- Consideration should be given to routine inspection to find problems early and assess project priority in the future periodically.
- Maintenance of outfalls, utility crossings, and other areas can help prevent future problems and prolong the life of existing facilities until they can be addressed through the proposed projects.
- If projects cannot be completed in a timely manner, then consideration should be given to phasing the projects to allow higher priority portions to be addressed sooner.
- Storm drain systems should be evaluated in a 2D hydraulic modeling platform such as Infoworks ICM for local pluvial flooding originating from surface water ponding in the watershed.
- Levees should be included in regular maintenance schedules.

## G. Master Drainage Plan Maintenance

The Drainage Master Plan for Johnson Creek Corridor should be maintained to keep it relevant, accurate and up to date. Future field assessments, flood studies, LOMRs, detention ponds, storm drain studies, improvements, and the like should be incorporated or added as appropriate. The electronic documentation of the study is included in **Appendix E**.

**GLOSSARY**

**AC** - Annual Chance

**AE** - Approximate Elevation

**BFE** - Base Flood Elevation

**BMP** - Best Management Practice

**CSJ** - Control Section Job

**CTP** - Cooperating Technical Partner

**DFIRM** - Digital Flood Insurance Rate Map

**DMP** - Drainage Master Plan

**FEMA** - Federal Emergency Management Agency

**FIRM** - Flood Insurance Rate Map

**FIS** - Flood Insurance Study

**GIS** - Geographic Information System

**HDR** - HDR Engineering, Inc.

**HEC-HMS** - Hydrologic Engineering Center Hydrologic Modeling System (USACE)

**HEC-RAS** - Hydrologic Engineering Center River Analysis System (USACE)

**iSWM** - integrated Stormwater Management

**LiDAR** - Light Detection and Ranging

**NB** – North Bound

**NCTCOG** - North Central Texas Council of Governments

**OEI** - O'Brien Engineering, Inc.

**PGBT** - President George Bush Turnpike

**PT** - Pressure Transducer Sensor

**QA** - Quality Assurance

**SB** - South Bound

**$S_f$** - Friction Slope

**GLOSSARY (continued)**

**SUH** - Synthetic Unit Hydrograph

**Tc** - Time of Concentration

**TCEQ** - Texas Commission on Environmental Quality

**TSDN** - Technical Support Data Notebook

**TPWD** - Texas Parks and Wildlife Department

**TXDOT** - Texas Department of Transportation

**USACE** - United States Army Corps of Engineers

**WSEl** - Water Surface Elevation

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# APPENDIX A
# HYDROLOGIC DATA

# APPENDIX B
# HYDRAULIC DATA

# APPENDIX C
# JOHNSON CREEK CHANNEL ASSESSMENT

# APPENDIX D
# STORM DRAIN OUTFALLS

# APPENDIX E
# CD-ROM