



2026
CAPITAL IMPROVEMENT PLAN
CITY OF WILLOW PARK, TEXAS

August 2026

Prepared by:



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CITY OF WILLOW PARK CAPITAL IMPROVEMENT PLAN

AUGUST 2026

I. GENERAL

A. Introduction

The City of Willow Park authorized JACOB & MARTIN, LLC to prepare an update to the capital improvement plan in November, 2025. The City has authorized an update to the plan completed in 2022 to include discussions for capital improvements to the water, sewer, street and drainage systems. Willow Park is located in an area of continued growth, along the Interstate 20 corridor, approximately 20 miles west of the City of Fort Worth.

The outlook for the City of Willow Park is for continued growth over the next 25 to 30 years. It is therefore imperative for the City to maintain a workable plan to prepare for and accommodate that growth with the least detrimental impact to the public.

B. Purpose

The purpose of the updated plan is to evaluate the City's existing water, wastewater, street, and drainage facilities and provide a plan which will allow the City to conduct orderly improvement of the water and wastewater systems to meet demands through the year 2032.

C. Scope of Work

The following areas are to be considered:

- Population & Land Use Assumptions
- Water & Wastewater Design Criteria
- Water Use Projections
- Water Supply
- Water System Infrastructure
- Recommended Water System Improvements
- Costs of Water System Improvements
- Wastewater Flows
- Wastewater System Infrastructure
- Recommended Wastewater System Improvements
- Costs of Wastewater System Improvements
- Street Inventory and Evaluation
- Recommended Street Improvements
- Drainage Inventory and Evaluation

- Recommended Drainage Improvements

D. Description of Plan Area

The plan area includes the City Limits and Extraterritorial Jurisdiction (ETJ) of the City of Willow Park. The plan area is depicted in Figure I.1.

E. Water Plan Approach

The approach to the development of the water plan involves the following steps:

1. Develop water use projections and estimate per capita use (current & future)
2. Identify and evaluate current & potential water supply sources
3. Inventory and evaluate the current infrastructure
4. Develop strategies for addressing current shortfalls & meeting future demands
5. Develop costs and financing strategies for addressing the identified needs

F. Wastewater Plan Approach

1. Develop wastewater flow projections and estimate per capita contributions
2. Identify and evaluate current & potential treatment alternatives
3. Inventory and evaluate the current infrastructure
4. Develop strategies for addressing current shortfalls & meeting future needs
5. Develop costs and financing strategies for addressing the identified needs

G. Street Plan Approach

1. Inventory and evaluate the current streets
2. Develop strategies for addressing current shortfalls & meeting future needs
3. Develop Costs and financing strategies for addressing the identified needs

H. Drainage Plan Approach

1. Inventory and evaluate the current drainage infrastructure
2. Develop strategies for addressing current shortfalls & meeting future needs
3. Develop Costs and financing strategies for addressing the identified needs

II. DEVELOPMENT AND GROWTH

A. Population Projections

According to the previous estimates of population, the City's population for 2020 and 2025 was 6,086 and 6,804, respectively. This is a population increase of approximately

12% in five years or a 2.4% increase per year. Development in the area that is underway will lead to an increase of approximately 3,450 people.

B. Land Use

As previously mentioned, the City of Willow Park is located along the Interstate 20 (I-20) Corridor approximately ten miles east of Downtown Weatherford and twenty miles west of Downtown Fort Worth. The City is a small North Central Texas community developed on mostly high and flat to steep rolling terrain in eastern Parker County. As development and growth occur within the City, the infrastructure needs to be expanded to serve existing and future residences and businesses in the service area.

A land use inventory or assessment identifies the current uses of the land throughout the planning area. An inventory of the City's land use is shown in Figure II.1 – Existing Land Use Map and the acreages are tabulated below in Table II.2 from the City's 2014 Comprehensive Plan. The inventory is a critical set of data used to create a Future Land Use Plan. The Existing Land Use Map and acreage tabulations should be updated as new building permits are issued and property tax records are changed. This will allow the City to evaluate where it is in relation to its Future Land Use Plan.

TABLE II.2

EXISTING LAND USE		
Type of Land Use	Acres	Percent
Residential - Single Family	2,675	34.3%
Residential - Two-Family	2	0.03%
Residential - Multi-Family	251	3.2%
Retail	54	0.7%
Office	18	0.2%
Commercial	445	5.7%
Industrial	4	0.1%
Public/Semi-Public	235	3.0%
Parks & Open Space	89	1.1%
Golf Course (Private)	198	2.5%
Utilities	25	0.3%
TxDOT Right-of-Way	181	2.3%
Total Developed Land	4,177	53.5%
Agriculture & Vacant	3,629	46.5%
Total Land Area	7,806	100.0%

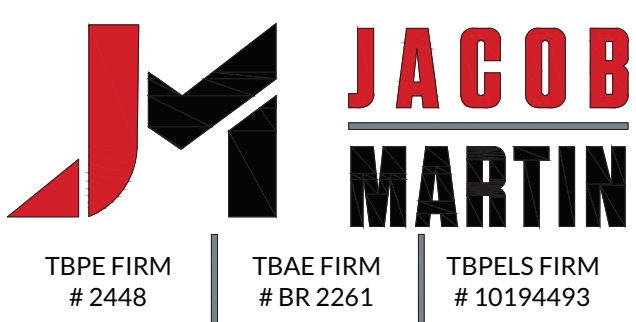
Based on the update land use plan, there is approximately 4,177 developed acres or 53.5% of the total 7,806 acres of land within the City Limits and ETJ, while the remaining acreage is vacant or used for some agricultural purpose. Currently, the most predominant land use is residential which is approximately 2,928 acres or 70% of the

total 4,177 developed acres. The future land use is expected to be predominantly residential as well. Office, retail, commercial and industrial land uses cover approximately 521 acres or 12.5% of the developed land. Most of the existing office, retail, commercial and industrial land uses in the City are located along the frontage of Interstate 20. Most of the future office, retail, commercial and industrial land uses are expected to continue along the highway frontage with minor development for these land uses occurring away from the highway. Figure II.2 - Future Land Use Plan shows the expected development areas by land use types. Table II.3 below shows the expected acreage and percentage for each land use type.

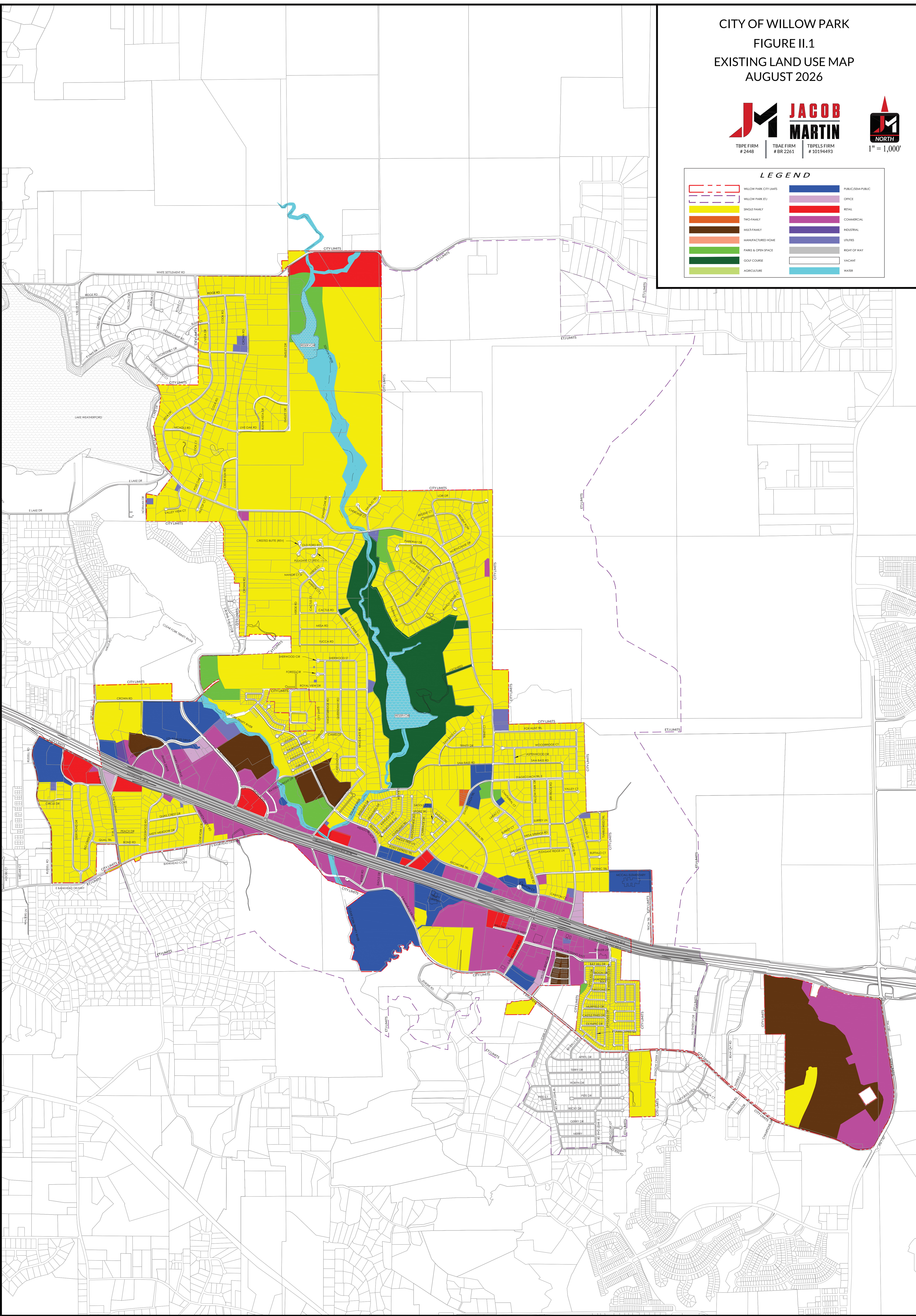
TABLE II.3

FUTURE LAND USE		
Type of Land Use	Acres	Percent
Residential - Single Family	5,115	65.5%
Residential - Two-Family	2	0.03%
Residential - Multi-Family	251	3.22%
Retail	102	1.31%
Office	18	0.23%
Commercial	579	7.42%
Industrial	109	1.40%
Public/Semi Private	260	3.33%
Parks & Open Space	89	1.14%
Golf Course (Private)	198	2.54%
Utilities	25	0.32%
TxDOT Right-of-Way	181	2.32%
Total Developed	6,929	88.8%
Undeveloped (Road & Flood Plain)	565	7.24%
Total Planning Area	7,806	100.0%

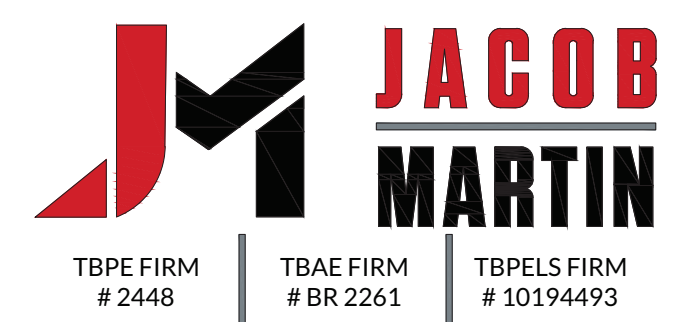
CITY OF WILLOW PARK
FIGURE II.1
EXISTING LAND USE MAP
AUGUST 2026



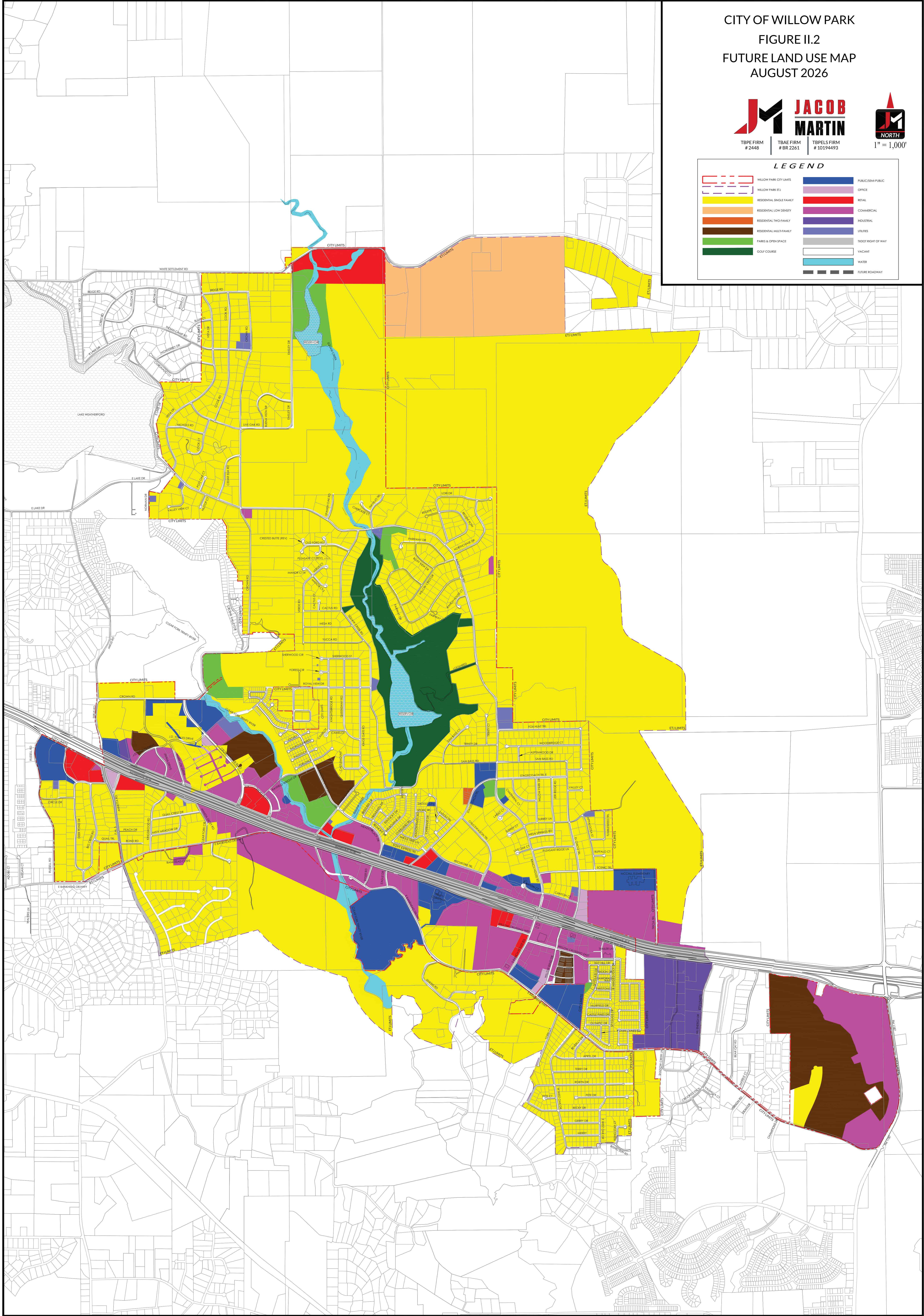
LEGEND	
	WILLOW PARK CITY LIMITS
	WILLOW PARK E/I
	SINGLE FAMILY
	TWO-FAMILY
	MULTI-FAMILY
	MANUFACTURED HOME
	PARKS & OPEN SPACE
	GOLF COURSE
	AGRICULTURE
	PUBLIC/OSM/PUBLIC
	OFFICE
	RETAIL
	COMMERCIAL
	INDUSTRIAL
	UTILITIES
	RIGHT OF WAY
	VACANT
	WATER



CITY OF WILLOW PARK
FIGURE II.2
FUTURE LAND USE MAP
AUGUST 2026



LEGEND	
	WILLOW PARK CITY LIMITS
	WILLOW PARK EJ
	RESIDENTIAL SINGLE FAMILY
	RESIDENTIAL LOW DENSITY
	RESIDENTIAL TWO FAMILY
	RESIDENTIAL MULTI-FAMILY
	PARKS & OPEN SPACE
	GOLF COURSE
	PUBLIC/GEN-PUBLIC
	OFFICE
	RETAIL
	COMMERCIAL
	INDUSTRIAL
	UTILITIES
	TRDPT RIGHT OF WAY
	VACANT
	WATER
	FUTURE ROADWAY



III. WATER SYSTEM

A. Regulations

The agency which oversees public water systems in the State of Texas is the Texas Commission on Environmental Quality (TCEQ). The TCEQ is tasked with insuring compliance with EPA and Texas regulations concerning the quality, capacity, and operation of water systems. The rules promulgated by the TCEQ for this purpose are found in 30 TAC Chapter 290. Subchapter D of the 290 rules pertains to the operational requirements and capacities for public water systems. The relevant sections of the rules for this discussion are 290.44(d) and 290.45(b)(1)(D)(iv). Section 290.44(d) states that “the system must be designed to maintain a minimum pressure of 35 psi at all points within the distribution network at flow rates of at least 1.5 gpm per connection”. This is the peak hourly demand requirement which must be met by public water systems. The TCEQ further requires, in Section 290.45(b)(1)(D)(i), that supplies of 0.6 gpm per connection be available (peak day requirement) for either well supplies or surface water sources. Section 290.45(b)(1)(D)(iv) requires 100 gallons per connection of elevated storage or a pressure tank capacity of 20 gallons per connection. Section 290.45(b)(1)(D)(ii) requires a total storage capacity of 200 gallons per connection. Each section cited herein pertains to groundwater supplies. The requirements for surface water systems are the same, for systems serving more than 250 connections, and are found in Section 290.45(b)(2) of the TCEQ regulations. The TCEQ’s regulations governing public water systems can be viewed online at www.tceq.texas.gov/rules/indxpdf.html.

B. Water Use

The City’s historical and projected water use is illustrated in Table III.1.

**TABLE III.1
METER CONNECTIONS PROJECTION**

Year	Water conn	Yearly Flow (MG/Yr)	Avg. Daily Flow		Max Daily Flow		Peak Hour gpm/conn
			MGD	gpm/conn	MGD	gpm/conn	
2020	2061	279.74	0.77	0.26	1.38	0.46	1.5
2021	2150	268.10	0.74	0.24	1.42	0.46	1.5
2025	2378	355.19	0.97	0.28	1.75	0.51	1.5
2030	2796	417.62	1.14	0.28	2.06	0.51	1.5
2035	3081	460.19	1.26	0.28	2.27	0.51	1.5
2040	3499	522.63	1.43	0.28	2.58	0.51	1.5
2045	4047	604.48	1.66	0.28	2.99	0.51	1.5
2050	5194	775.80	2.13	0.28	3.83	0.51	1.5

Historically, the City’s average-day, maximum-day, and peak-hour water demands have generally remained below the planning criteria established by the TCEQ. Between 2007 and 2021, the City’s average-day demand was approximately 0.25 gallons per minute per

connection (gpm/conn), while the maximum-day demand averaged approximately 0.55 gpm/conn. Based on the 2025 pumpage data, the City's average-day demand was approximately 0.28 gpm/conn, and the maximum-day demand was approximately 0.51 gpm/conn. The 2025 maximum-day demand remained below the TCEQ planning criterion of 0.60 gpm/conn. For planning purposes, future average-day and maximum-day water demands were projected using the 2025 per-connection demand rates of 0.28 and 0.51 gpm/conn, respectively, while the TCEQ peak-hour criterion of 1.50 gpm/conn was maintained through the year 2050.

C. Water Supply

The current water supply for the system is from twenty-five Paluxy and Trinity formation groundwater wells and a contracted treated surface water supply from the City of Fort Worth. The capacity of the City's existing wells is indicated in Table III.2. The available treated water supply from Fort Worth, by contract, is 3.5 million gallons per day (MGD).

Based on the water use projections stated above, it is expected that the City's current water supply will be sufficient beyond the year 2050. The City's wells have been decreasing in capacity over the last several years due to reduction in aquifer levels and deteriorating water quality. However, the decreased reliance on groundwater, made possible by the availability of treated surface water, is expected to extend the usable life of the City's groundwater supply.

**TABLE III.2
WATER WELL INVENTORY**

Well #	Capacity (gpm)	Depth	Pump (HP)
1	60	182	7.5
2	36	148	7.5
3	48	166	7.5
4	24	177	7.5
5	26	158	7.5
6T	64	620	10
6P	50	160	7.5
7	37	135	7.5
9T	114	650	15
9P	85	260	7.5
10T	31	650	15
10P	58	270	7.5
11	35	215	3
El Chico T	50	650	15
El Chico P	22	221	5
14	44	290	7.5
15	97	650	30

	16T	35	624	10
	16P	26	269	3
	WWN	31	250	5
	WWS	37	255	3
	WSO-T	85	650	15
	WSO-P	38	235	5
	WSS	12	256	3
	WSN	17	256	1
	Total GPM	1,137		
D. Existing System	Total MGD	1.64		
			Water	

The City's existing water facilities are depicted in Figure III.1 – Water System Inventory Map. Approximately 43% of the well supply is conveyed directly to the City's Indian Camp pump station and elevated tank site on the north end of the system. The pump station at this site fills the adjacent elevated tank and provides supply flow to the rest of the system. The water is conveyed through the distribution system and the City's main 12-inch groundwater supply line to the Foxhunt elevated tank and pump station site. Additional groundwater is pumped into the system at this site. The Foxhunt elevated tank is on the same pressure plane as the Indian Camp tank (North Pressure Plane). Well 9, Well 10 and the El Chico well site located downstream of the Foxhunt elevated tank site, provide additional groundwater into this portion of the system as well. The Indian Camp elevated tank site provides direct service to the southwest portion of the City's system and supply flow to the Willow Springs Oaks pressure plane (South Pressure Plane) on the far southwest corner of the service area. Well 11 and the Willow Wood wells and pump station located centrally in the City's system provides additional groundwater into the distribution system prior to the South Pressure Plane. The Willow Springs Oaks pump station and elevated tank site provides direct service to the South Pressure Plane. The Willow Springs pump station and well site located on the east end of the South Pressure Plane provides additional groundwater into this portion of the system.

The Fort Worth supply is conveyed to the El Chico Pump Station site through a shared 18-inch pipeline. The water then enters the system by a direct connection to the existing 12-inch transmission line in El Chico Drive. Surface water is then conveyed to Hudson Oaks via a 14-inch shared pipeline which runs west along Interstate 20.

Schematic layouts of the City's pumping and storage facilities are included in Appendix A.

The City's distribution system is comprised of approximately 74 miles of pipe ranging in size from 2-inch to 12-inch (excluding the Fort Worth transmission line). The City's distribution system is comprised of approximately 74 miles of pipe ranging in size from 2-inch to 12-inch (excluding the Fort Worth transmission line).

An evaluation of the system identified locations containing deteriorated or undersized piping. Deteriorated pipes have led to significant water loss due to pipe breaks in the system as well as service outages. Undersized lines require higher feed pressures to maintain minimum service requirements. These higher pressures lead to even more breaks in vulnerable areas of the system. Additionally, adequate fire protection requires larger pipes to ensure sufficient flow can be maintained during a fire. A 6-inch pipe is the smallest size that is acceptable for fire coverage and there are several locations within the city where neighborhoods are served by 2-inch pipes. Additionally, adequate fire coverage requires fire hydrants spaced at 500 to 1,000 feet depending upon the area. Many areas in the city exceed even the 1,000-foot separation. The city is contemplating a project to address many of these areas. The City's system is looped, from north to south, in three locations. Additional north-south, as well as east-west, looping will be necessary to ensure adequate flow throughout the system.

E. Storage and Pumping Requirements

According to TCEQ regulations, the following minimum requirements must be provided by a community-type water system that serves more than 250 connections. Minimum residual pressure of 20 psi and a minimum normal operating pressure of 35 psi with an instantaneous use in the system of 1.5 gpm per connection are required.

*TOTAL STORAGE CAPACITY	200 Gallons per Connection
PRESSURE MAINTENANCE FACILITIES:	
A. Pressure Tank	20 Gallons per each Connection
B. Elevated Tank	100 Gallons per Connection
WATER SUPPLY FLOW RATE:	0.6 gpm per Connection
SERVICE PUMPS:	2 or more pumps with total rated capacity of 2.0 gpm per connection or 0.6 gpm if 200 gallons per connection of elevated tank capacity provided

* Pressure tank storage is not recognized.

1. Ground Storage

The City of Willow Park currently maintains nine ground storage tanks at seven locations within the system. An inventory of the ground storage facilities is included in Table III.3. Table III.4 provides the expected ground storage requirement based on the water customer projections from Section III.B. As evidenced in these tables, the City will meet

the projected regulatory requirement for ground storage volume with the existing facilities. However, from an operational standpoint, additional ground storage may be needed to meet the needs of additional supply and blending as well as insuring adequate emergency supply. Repair and replacement of ground storage tanks is also needed due to age and deterioration.

**TABLE III.3
GROUND STORAGE FACILITIES**

Site Name	Ground Elev. (Ft. above MSL)	Capacity (MG)
Indian Camp Rd.	1,012	0.500
		0.350
Well 9	896	0.06
Fox Hunt	963	0.286
Site 10 (Surry Ln)	954	0.092
El Chico Well Site	945	0.031
Willow Wood	944	0.042
Willow Springs	958	0.024
Willow Springs Oaks	975	0.042
		0.031
El Chico Pump Station	932	0.50
Total Ground Storage		1.958

**TABLE III.4
PROJECTED GROUND STORAGE REQUIREMENTS**

Year	Water Connections	Required Ground Storage (MG)
2020	2061	0.412
2025	2378	0.476
2030	2796	0.559
2035	3081	0.616
2040	3499	0.700
2045	4047	0.809
2050	5194	1.039

*Based on 200 gallons per connection.

2. High Service Pumping

The City currently maintains nine pump stations located throughout the system as well as one emergency pump station. An inventory of the pumping facilities is included in Table III.5. Based on an elevated storage requirement of 100 gallons per connection, the pumping requirement for the City would be 2.0 gallons per connection. Table III.6 provides the expected high service pump requirements based on this scenario and the water customer projections from Section III.B. The ultimate high service pump capacity under this scenario would be 10,894 gpm. While there is a deficiency by the year 2050, it

is slight. Potential improvements to address this shortfall will be examined in later sections of this report.

**TABLE III.5
HIGH SERVICE PUMP FACILITIES**

Site Name	Ground Elev. (Ft. above MSL)	Pump (HP)	Capacity (gpm)
Indian Camp Rd.	1,012	30	700
		30	700
		30	700
Well 9	896	7.5	80
		7.5	80
Fox Hunt	963	20	300
		20	300
Site 10 (Surry Ln)	954	15	350
		15	350
		15	200
El Chico	945	5	100
		10	200
		10	200
Willow Wood	944	10	175
		10	175
		10	175
Willow Springs	958	10	175
		10	175
Willow Springs Oaks	975	10	200
		10	200
		10	200
El Chico Pump Station WP	932	150	1215
		150	1215
		150	1215
El Chico Pump Station HO	932	150	468
		150	468
		150	468
Total High Service Capacity			10,784

**TABLE III.6
PROJECTED HIGH SERVICE PUMP REQUIREMENTS**

Year	Water Connections	Required High Service Pump Capacity (gpm)
2020	2061	4,122
2025	2378	4,756
2030	2796	5,592
2035	3081	6,162
2040	3499	6,998
2045	4047	8,094
2050	5194	10,388

*Based on 2.0 gpm per connection.

3. Elevated Storage

The City of Willow Park currently maintains three elevated storage tanks at the Indian Camp, Fox Hunt and Willow Springs Oaks locations. The Indian Camp and Fox Hunt tanks provide pressure maintenance for the North Pressure Plane with high water levels of 1,132 MSL. The South Pressure Plane is served by the Willow Springs Oaks tank with a high-water level of 1,102 MSL. An inventory of the elevated storage facilities is included in Table III.7. Table III.8 provides the expected elevated storage requirement based on the water customer projections from Section III.B and a requirement of 100 gallons per connection through the year 2045, as discussed in Section III.E.2. As evidenced in these tables, the elevated tank capacity is expected to be sufficient through the year 2050. However, the City has had issues with storage running low in the past. From an operational perspective, the tanks have not been adequate to maintain water supply during times of supply outages. Note also that the elevated storage in the south pressure plane may become insufficient during the planning period based on growth in that area. Additionally, the existing tanks are not expected to last through 2050 as they have deteriorated and will continue to do so. It is likely that one or all of them will require replacement in that time.

**TABLE III.7
ELEVATED STORAGE FACILITIES**

Site Name	Ground Elev. (Ft. above MSL)	Capacity (MG)
Indian Camp Rd.	1,012	0.250
Fox Hunt	963	0.250
Willow Springs Oaks	975	0.125
Total Ground Storage		0.625

**TABLE III.8
PROJECTED ELEVATED STORAGE REQUIREMENTS**

Year	Water Connections	Required Elevated Storage (MG)
2020	2,061	0.206
2025	2378	0.238
2030	2796	0.280
2035	3081	0.308
2040	3499	0.350
2045	4047	0.405
2050	5194	0.519

*Based on 100 gallons per connection.

From a regulatory standpoint, the facilities meet the projected ground storage and elevated storage needs through the planning period. Based on population projections, the pumping capacity becomes deficient sometime prior to 2050. Phased improvements will be necessary for the city to meet regulatory requirements and maintain a high level of service to its customers. Expansion into other areas and the age of the existing facilities will facilitate the need for additional elevated water storage in the future.

F. Recommended Water System Improvements

An analysis of the system has identified some areas that are or will likely become deficient by the year 2050. Elevated storage, distribution piping, transmission lines, and aging infrastructure are elements that should be addressed over the next several years and beyond. Pumping capacity will potentially become inadequate by 2050 and may be resolved with minor upgrades at that time.

1. Ground Storage

While the ground storage appears to be adequate from a regulatory standpoint, additional storage to meet emergency situations and fire flow needs is recommended. A new tank located at the El Chico Fort Worth Pump Station Site would be the most advantageous location. Additionally, Some of the City's existing ground storage tanks are deteriorating. The two ground storage tanks at the Indian Camp location should be rehabilitated with new roofs added. The ground storage tank at the El Chico Well Site Pump Station should be replaced entirely.

2. Elevated Storage

A previous Capital Improvement Plan recommended siting a new 0.25 MG elevated tank in the vicinity of the existing El Chico Well and Pump Station. This would be an

advantageous site and would address some high-pressure issues as well. An alternative would be to install a new tank at a higher elevation near the east side of the City's system where new development is taking place. It is also recommended that the Willow Springs Oaks Elevated Tank be replaced and relocated to a higher elevation and made to match the elevation of the other tanks.

3. Distribution System

Most of the water loss in the City's system is attributable to deteriorating, leaking cast iron or ductile iron waterlines. There are several additional, deteriorated lines in the system that are in need of replacement as depicted in Figure III.3. Additionally, there are a number of lines that are less than 6-inches in diameter and serve residential areas. It is recommended that these lines be upsized to 6-inch and that fire hydrants be installed in locations where the maximum separation is being exceeded.

4. Transmission Lines

It is recommended that a new 12-inch transmission line be installed from the new El Chico Pump Station to the Fox Hunt ground storage facility. This would eliminate supply flow through the distribution system, reducing system pressures, and it would be a more efficient means of filling the Fox Hunt Elevated tank. Additionally, main line loops along Crown Road, Bankhead Highway, and others would provide more reliable service at lower pressures. A tie-in from the new Willow Park/Hudson Oaks 14-inch supply line to provide surface water supply to the Willow Springs Oaks pressure plane is also recommended.

All of the proposed water distribution and storage improvements are depicted in Figure III.2.

G. Priority and Cost Estimates

The following costs and priority timelines may help in planning and budgeting for capital improvement projects. Table III.9 includes both the estimated costs of the recommended improvements and the recommended timeframes for completion of those projects.

TABLE III.9
WATER SYSTEM IMPROVEMENTS
PRIORITY AND PROJECTED COSTS

Item	Project	Location	Cost
1	West Bankhead Water Line Extension Ph. 1	El Chico/Fox Hunt	\$ 700,000.00
2	East Bankhead Water Line	Bankhead South of IH20	\$ 885,000.00
3	West Oak Water Line Replacements	West Oak Subdivision	\$ 5,000,000.00
4	Rehabilitate Indian Camp Ground Storage Tanks	Indian Camp Tank Site	\$ 500,000.00
5	Disinfection Improvements	All Well Sites	\$ 100,000.00
6	Squaw Creek & Local Water Line Replacements	Squaw Creek Road	\$ 3,750,000.00
7	Install East Side Elevated Tank	El Chico Well Site	\$ 5,000,000.00
8	West Bankhead Water Line Extension to FM 5	Bankhead Road to FM 5	\$ 1,170,000.00
9	Replace Willow Springs Oaks Elevated Tank	Willow Springs Oaks Site	\$ 5,500,000.00
10	Willow Springs Oaks Interconnect	IH20 & Crown Pointe Blvd	\$ 75,000.00
11	El Chico to Fox Hunt Transmsion Line	El Chico/Fox Hunt	\$ 3,500,000.00
12	Upsize Lines	City Wide	\$ 3,600,000.00
13	Replace Deteriorated Lines	City Wide	\$ 2,500,000.00
14	Fire Hydrant Additions	City Wide	\$ 100,000.00
15	Replace Fox Hunt Elevated Tank & Upsize	Fox Hunt Tank Site	\$ 5,750,000.00
16	Replace Willow Springs Oaks Water Lines	Willow Springs Oaks	\$ 4,500,000.00
17	Broadway Loop	Broadway Street	\$ 250,000.00
18	Russell Road Water Line	Russell Road	\$ 830,000.00
19	Replace Indian Camp Elevated Tank	City Wide	\$ 6,000,000.00
20	Fee Studies		\$ 75,000.00

The costs in Table III.9 make allowance for inflation. The priority of the line replacements and fire hydrant additions is based on the level of deterioration (number of repairs) and the City's staff recommendations. The costs have been segregated to reduce the cost impact and allow for staggering of debt service. The line replacements should be undertaken as soon as possible and on as rapid a pace as possible to reduce water loss in the system and the associated maintenance costs. The line upsizes are not demand driven and are not critical from an operational standpoint. The City must weigh this against safety concerns when attributing a priority to this item. The remainder of the recommended improvements are driven by increased demand in the system and there should be flexibility in the timing of the improvements. The pace at which development occurs may accelerate or delay the need for these items. Development may also help to offset the cost of these improvements through cost sharing agreements and impact fees.

IV. WASTEWATER SYSTEM

A. Regulations

The TCEQ is also the agency which oversees wastewater systems in the State of Texas. The TCEQ is tasked with ensuring compliance with EPA and Texas regulations concerning the effects of wastewater discharge into the waters of the State as well as public welfare and air quality. The rules promulgated by the TCEQ for this purpose are found in 30 TAC Chapter 217. Subchapter B of the 217 rules pertains to the determination of capacity and design loadings for wastewater systems. Subchapter C sets out the regulations regarding collection systems and Subchapters E through L pertain to the design and sizing of wastewater treatment systems.

The TCEQ's regulations governing wastewater systems can be viewed online at www.tceq.texas.gov/rules/indxpdf.html.

B. Wastewater Flows

Critical parameters for wastewater system planning include average daily flow, peak daily flow, and peak hour flow. Wastewater flow projections were developed using the 2025 wastewater flow records as the base year and applying projected sewer connection growth throughout the planning period. In 2025, the City served approximately 1,097 sewer connections and recorded a total annual wastewater flow of approximately 125.91 MG/Yr. The average daily flow was approximately 0.345 MGD, or 314 gallons per day per connection. The peak daily flow used for the projections was based on the highest recorded daily flow in 2025, which was approximately 0.630 MGD, or 574 gallons per day per connection. Future average-day and peak-day wastewater flows were projected by applying these per-connection flow rates to the projected number of sewer connections, while peak-hour flows were projected using the adopted peak-hour design criterion. The City has entered an agreement with the City of Hudson Oaks to treat up to 250,000 gallons per day at the new treatment facility. However, current flows and projections for Hudson Oaks would indicate a maximum contribution of 150,000 gallons per day. The projections for future capacity requirements take this into account. In addition to anticipated growth, approximately 1,281 existing water customers are not currently connected to the sewer system. As sewer facilities are extended and made available, these customers may eventually connect to the collection system; however, the timing of these connections will depend largely on future system improvements and development patterns. For planning purposes, these additional connections are assumed to begin coming online in 2050, and the wastewater connection and flow projections have been adjusted accordingly. Based on these assumptions, wastewater flows are projected to increase as new development occurs and additional existing water customers connect to the sewer system.

The City has two develops approved for construction that will significantly increase the flow to the wastewater treatment plant in addition to the Hudson Oaks contribution. These include the mixed use Beall property at the far east side of the system and the Clearion development located on the west side. The Beall property is projecting

approximately 82,700 gpd of average day flow under their first phase of construction. This would be expected to impact the system within five years. The latter phases include an additional 268,200 gpd which is expected to develop over the following five years. The Clearion development is projecting 241 single family residential connections within the next five years or an estimated 72,300 gpd.

**TABLE IV.1
WASTEWATER CONNECTIONS AND FLOW PROJECTIONS**

Year	Sewer conn	Yearly Flow (MG/Yr)	Avg. Daily Flow		Peak Hour gal/day/conn
			MGD	gal/day/conn	
2020	810	97.02	0.265	327	825
2021	870	105.62	0.289	332	825
2025	1097	125.91	0.345	314	825
2030	2114	242.60	0.665	314	825
2035	3008	345.21	0.946	314	825
2040	3316	380.56	1.043	314	825
2045	3624	415.91	1.139	314	825
2050	3911	448.89	1.230	314	825

C. Existing Wastewater System

The City's existing wastewater system is shown schematically in Figure IV.1. It consists of approximately 14 miles of 6-inch through 12-inch gravity mains and 2.7 miles of 2-inch through 6-inch force main, currently serving 1,097 customers. The wastewater system currently serves less than half the residences and businesses in Willow Park. Wastewater collected from the system is transferred by force main from four sewer lift stations. An area located on Crown Road on the west side of the City which serves eight homes utilizes individual, private lift stations. Sewer is transferred from these lift stations via a single, 2-inch force main directly to the wastewater treatment plant.

The Willow Park Village lift station serves the area generally east of Ranch House Road, south of Interstate 20, west of Hewitt Lane, and north of Bankhead Highway. The lift station was upgraded in 2015 and includes three 35 horsepower pumps rated at approximately 219 gpm each. This lift station pumps to a manhole in Willow Crossing Road through a 6-inch force main.

The Beavers Creek lift station serves the area bounded generally on the north by Scenic Drive, on the east by Tricia Trail, on the south by East Bankhead Highway, and on the west by Chuckwagon Trail. This station also receives flow from the Willow Park Village lift station. The Beavers Creek lift station was also upgraded in 2015 and consists of three 20 horsepower pumps rated at an estimated 278 gpm each and pumps via 6-inch force main to a manhole in Pitchfork Trail.

The Kings Gate lift station receives flow from the entire collection system except the few homes on Crown Lane previously mentioned. This station was upgraded along with the others and again in 2021 and currently consists of three 30 horsepower pumps rated at an estimated 450 gpm each. This lift station pumps to the wastewater treatment plant through a 10-inch force main. The lift station clogs frequently due to the nature of the sewage and is in need of a grinder to eliminate larger solids and rags.

A project has recently been completed to install a lift station on the south side of Interstate 20 on Bankhead Highway and a force main to the 12-inch gravity trunk line which feeds the Kings Gate Lift Station. The new lift station will allow service to the currently unserved areas west of Ranch House Road and South of Interstate 20. A new transmission line in this area will be required to collect and convey the sewer.

The current capacity of the wastewater treatment plant is 500,000 gpd. The plant was installed in 2021 as an interim solution to the City's overall wastewater treatment needs. However, a project is in progress to install a new 0.75 MGD wastewater treatment plant, replacing the existing interim plant. The interim facility is a conventional activated sludge wastewater plant contained in steel basins. The service area is located in the drainage basin for Lake Benbrook which has been designated as an impaired stream. Because of this, the TCEQ requires lower limits than would be typical for a plant of this size. The effluent standards in place for the plant are BOD-5 ppm, TDS-5 ppm, ammonia-nitrogen-1.8 ppm, and phosphorus-1.0 ppm. In order to meet these limits, filtration and coagulant feed are required at the plant. The City has obtained a new discharge point to dispose of up to 500,000 gpd into Reservoir No. 23 at Squaw Creek Golf Course. The effluent standards in place for the new discharge point are BOD-10 ppm, TDS-15 ppm, ammonia-nitrogen-3 ppm, and phosphorous-0.5 ppm. The City is currently constructing the new 0.75 MGD wastewater treatment facility which will utilize both of the permitted discharge locations. This plant is being constructed as a permanent solution to the City's needs with upgrade capability to 1.0 MGD within the footprint that is being built. The expectation is that an additional 0.25 MGD capacity would be added in the short term to address expected demands. Additional improvements may be added later to increase the plant capacity further, however, this would require additional disposal options and permitting.

D. Recommended Wastewater System Improvements

The capacity of the wastewater system has been impacted somewhat by inflow and infiltration as well as a high rate of growth in the community. Inflow and infiltration (I&I) occur when segments of the collection system are in disrepair and allow stormwater and water from other sources to infiltrate the sewer system. The City has had significant (I&I) over the last several years which results in overflows of manholes (unauthorized discharges), overburdening of lift stations and force mains, and capacity exceedance at the wastewater treatment plant. The City has been in the process of identifying sources of I&I and has significantly improved the system in recent yearss; however, some areas of the system still need to be addressed. Additionally, the City's Kings Gate Lift Station is in need of upgrades to prevent clogging of pumps and related treatment issues. Improvements discussed in the following sections are depicted in Figure IV.2.

The greater impact on the wastewater system is growth that has been occurring and is expected to occur in the next few years. The City has made an agreement to treat up to 250,000 gallons per day of sewage from the City of Hudson Oaks in the new plant. The actual flow treated on an average day basis is not expected to exceed 150,000 however, in the short term.

1. Collection System

It is recommended that the City continue smoke testing and/or video inspection of existing sewer mains. Problem areas identified to date have been addressed, but it is expected that other areas will be identified. It is also recommended that the City adopt and enforce a pretreatment program to prevent toxic materials from entering the collection system and eventually interfering with the treatment process. The Kings Gate Lift station should be equipped with a grinder as previously stated. Other minor improvements to other lift stations are recommended.

The location and patterns of development will dictate when potential improvements will be necessary as well as the need for new lift stations and collection lines. A project is underway to install a lift station on the south side of Interstate 20 on East Bankhead Highway as well as a 10-inch gravity trunk line and force main to convey it to the existing collection system. The new lift station will allow service to the currently unserved areas east of Ranch House Road and South of Interstate 20 to the east edge of the City's service area.

2. Wastewater Treatment

As mentioned previously, the City is constructing a new 1.0 MGD wastewater treatment plant. Based on the information in Table IV.1, the current plant would only be compliant until 2028 based on the TCEQ's 75% of capacity design threshold. Flows from the committed agreements and developments will meet the threshold by that time. Therefore, the City should begin planning for additional wastewater treatment capacity as soon as possible. The new plant is being designed for 0.75 MGD with expansion capabilities to 1.0 MGD. This would be compliant to the year 2040 at which time further expansion would need to be underway. Capacity beyond 1.0 MGD would require additional permitting and most likely re-use sites (irrigation). Due to the efficiency of treatment and economy of space, sequencing batch reactor (SBR) technology has been selected for the new plant for which capacity can be increased relatively easily. Additionally, it is recommended that locations for the irrigation of effluent be identified and secured as this may be a requirement of future permits for the disposal of wastewater effluent. Golf courses, parks, and athletic fields are among the sites commonly used for irrigation. The City is currently investigating a number of sites for this purpose. Other potential options should be explored as well including a joint effort with other communities to reduce the cost of the facility.

E. Priority and Cost Estimates

The following costs may help in planning and budgeting for capital improvement projects. Table IV.2 includes the estimated costs of the recommended projects. The costs in Table IV.2 make allowance for inflation.

TABLE IV.2
WASTEWATER SYSTEM IMPROVEMENTS
PRIORITY AND PROJECTED COSTS

Item	Project	Location	Cost
1	Upsize WWTP	JD Towles at River	\$ 1,000,000.00
2	South of IH20 Sewer Transmission	South of IH 20	\$ 4,400,000.00
3	East Bankhead Lift Station	Broadway & Bankhead	\$ 600,000.00
4	Upsize WWTP	JD Towles at River	\$ 10,000,000.00
5	Effluent Irrigation	-	\$ 2,000,000.00
6	West Bankhead Extension	West Bankhead	\$ 1,000,000.00
7	Line Testing	City Wide	\$ 150,000.00
8	Lift Station Improvements	All Lift Stations	\$ 500,000.00
9	Replace Deteriorated Lines	City Wide	\$ 650,000.00
10	Fee Studies		\$ 35,000.00

The costs in Table IV.2 make allowance for inflation. The priority of the wastewater improvements is based on the condition of the existing facilities, known maintenance concerns, and the City staff's recommendations. The costs have been segregated to reduce the cost impact and allow for staggering of debt service. The collection system, lift station, and force main improvements should be undertaken as soon as possible and on as rapid a pace as possible to reduce maintenance issues in the system and improve reliability.

V. STREET SYSTEM

A. Inventory

A pavement condition analysis was recently performed in May 2025 by Infrastructure Management Services, LP (IMS) on the existing street inventory, including arterial, collector, and local streets. This inventory utilized Integrated Road Information System technology to capture images and data of the existing streets and assign an associated Pavement Condition Index (PCI) for each section of roadway based on the type, level, and severity of distress. The Pavement Condition Index serves as an objective, numerical way to compare conditions of each roadway in the city.

B. Evaluation

Unlike water and wastewater systems, streets are not regulated by any governing agency. However, street conditions can have significant impacts on the safety of the city's residents and should be treated accordingly. Whilst water and sewer systems are of the utmost importance to the underlying infrastructure of a city, the streets that make up a city are much more visible and easily noticeable to the public when there is a deficiency. Smaller cities with little growth often do not have the funding available to adequately maintain the existing inventory, let alone make significant improvements. However, as a city grows and develops it can open the city up to more financial options, such as larger street maintenance funds within the annual budget or various avenues for issuance of debt. For a city with significant growth potential such as Willow Park, planning for future roadway improvements in combination with adequate street maintenance is essential.

Based on the inventory and street analysis that was conducted by Infrastructure Management Services, LP, the PCI for each section of roadway dictates the level of improvement that would be necessary to adequately improve, rehabilitate, or maintain the street's condition. Per this street analysis by IMS, the national average pavement condition score ranges from 60-65 and the City of Willow Park averages a pavement condition score of 68. This puts Willow Park above the national average in terms of overall pavement condition across the city.

As pavement deteriorates over time, the costs associated with rehabilitation and lifetime extension of the pavement increase with the level of rehabilitation required. The pavement's condition and associated PCI value broadly determines the appropriate rehabilitation activity at that time. Therefore, when a pavement approaches the critical PCI for a given rehabilitation activity, it becomes a higher priority for a City to address, because if neglected past that critical PCI, the required rehabilitation activity jumps to the next requirement. For example, if a section of an asphalt street is beginning to see minor cracking or rutting and has a PCI of approximately 75, the required rehabilitation is simple maintenance activities such as applying a chip seal or crack sealing. However, if this same stretch of pavement is left for several years with minimal to no maintenance performed, then the PCI will most likely drop below a threshold in which chip seal or

crack sealing would be adequate and would likely require full width mill and overlay in order to properly rehabilitate the road.

The widely accepted best practice for overall pavement management is to effectively utilize a combination of the city's annual budgetary funds and debt issuance, with a logical approach to determining which roads need to be targeted by what improvement to maximize benefit to the citizens and the street system as a whole. If the City of Willow Park was to simply focus all annual funding towards reconstruction of the streets that were in the worst condition, it would not be able to keep up with the roads throughout the city that are deteriorating. This would lead to ultimately having an overall lower pavement condition index, and worse system at large. Conversely, if the City of Willow Park were to only focus on maintenance and rehabilitation of streets, the underlying issues for the streets that were in the worst condition would never be addressed, leading to the rehabilitated street deteriorating even faster than before which would result in no net benefit for the citizens or the street system as a whole. When proactive maintenance and rehabilitation is properly done on streets early in their lifecycle, the pavement life can be significantly prolonged which allows for the most optimized use of city funds.

The recommended approach for pavement management for the City of Willow Park is to utilize a dedicated portion of the City's annual street fund for maintenance and minor rehabilitation, and utilize bonds and debt issuance for larger street capital improvements. Maintenance and rehabilitation consist of, but are not limited to, crack and joint sealing, surface treatment, patching, and chip seals. Capital projects largely consists of mill and overlay or full-depth reconstruction of roadways, usually necessary due to failure of the subgrade underneath the pavement. Generally low PCI roads require reconstruction, but each street should be looked at on a case by case basis. For example, a street that is seeing significant alligator cracking and potholes, would not be best served by patching, seal coat, or even mill and overlay, but would be better addressed by full-depth reconstruction because the issues arose from the inadequate subgrade underneath the pavement. Similarly, if a street is seeing raveling, weathering, and minor cracking, then it would be ineffective use of the City's funds to reconstruct the street, and should instead explore maintenance options such as a crack sealing, surface treatment, or a thin overlay.

For streets that have been identified as being in need of major rehabilitation or reconstruction, then additional consideration should be given to the type of improvement for the street that provides the longest lifecycle in combination with being the most economical. Generally, local roads that see relatively lighter traffic in terms of volume and weight are good opportunities for asphalt overlay or reconstruction. Whereas concrete pavement would be most beneficial for arterials and specific collector roads that see larger traffic volume, as well as heavier loads. Willow Park largely consists of subgrade soils that are expansive, which was not accounted for when a large portion of the existing street system was developed and constructed. Therefore, when possible during rehabilitation and reconstruction projects, it is highly recommended to utilize subgrade stabilization to prevent premature cracking or early structural failure of the pavement.

C. Priority and Cost Estimates

Estimated costs and priorities have been assigned to the street system according to the aforementioned analysis of cost of improvement, cost of deferral, road condition, road type, etc. Table V.1 includes the recommended improvement, approximate estimated costs, and the recommended priorities for the capital projects. The costs shown for each recommendation considers inflation based on the associated year of the recommended improvement. Figure V.1 shows the City of Willow Park street inventory map with the roads identified that are on the list of priorities to address as a capital project.

a. Maintenance

In order to maintain the quality and extend the life of the streets that are currently in good shape, it is recommended that an annual seal coat program be implemented to maintain and extend the life of the streets that are not currently in poor condition or soon to be improved. A yearly operating allowance within the street department budget should be considered for this purpose and is usually approximately \$100,000 annually for a city the size of Willow Park. Table V.1 shows the recommended priority list for the seal coat maintenance program to target.

b. Capital Projects

The goal of the projects in the capital projects section is to improve the overall pavement condition of the City's street network through various methods. This can consist of targeting the streets within the city that are in the worst condition and require mill and overlay, full street reconstruction, and rehabilitation. This can also include upgrading vital thoroughfares throughout the City of Willow Park that are at critical points in their life cycle, whether that be pavement condition or traffic flow.

The City of Willow Park is also positioned to coordinate with, and work alongside, Parker County to repair certain roads that the city deems necessary as part of the City's capital projects. This provides the City the opportunity to potentially utilize its own street maintenance funds to pay for the county to construct either hot mix asphalt overlay or chip seal on the necessary roads.

The recommended priority list as shown in Table V.1 was put together primarily based on a cost of deferral method, which as discussed, strives to provide the city with the most efficient use of available funds. For example, as shown in Table V.1, Jeri Ridge Road is shown as a higher priority to address with a capital project than Hilltop Court despite Jeri Ridge Road having a current PCI of 50 compared to Hilltop Court's current PCI of 29. Jeri Ridge Road is currently in better overall condition than Hilltop Court but is recommended to be addressed sooner in part because the level of necessary improvement to rehabilitate is less. Roadway reconstruction is the most thorough repair of a street, but is also the most expensive. Generally, roads

that have a PCI notably less than 50 will likely require roadway reconstruction to properly rehabilitate the road. Mill and overlay is a cheaper option compared to roadway reconstruction, but is generally only adequate for roads with a PCI at approximately 50 or greater. Once a road dips significantly below a PCI of 50, the cost to improve begins to dramatically increase.

Whilst the PCI and cost of deferral is an important consideration when determining prioritization of streets that should be targeted by capital projects, the respective thoroughfare classification of each street needs to also be considered. Many of the roads shown as highest priority in Table V.1 are local roads, which while still important and vital to the City's overall infrastructure, may not have the same number of beneficiaries city-wide as a more critical and major thoroughfare. For example, whilst Post Oak Court is a high priority item for the city to address and improve in terms of PCI and cost of deferral, it is a short local road that serves approximately 6 residences. Whereas improving and upgrading a more major thoroughfare such as a Squaw Creek Road or East Bankhead Highway would have a much larger impact on citizens across the city.

When considering larger capital projects, it is also recommended to target areas where both street and other utilities are needing improvement and combining the projects into one, as this can reduce overall capital expenses through cost and resource efficiency. Whilst it may not be the most effective way of raising a street system's average PCI, it can have larger benefits to the city's infrastructure as whole. All of the strategies noted should be considered when the City determines which roads need to be targeted with available funds.

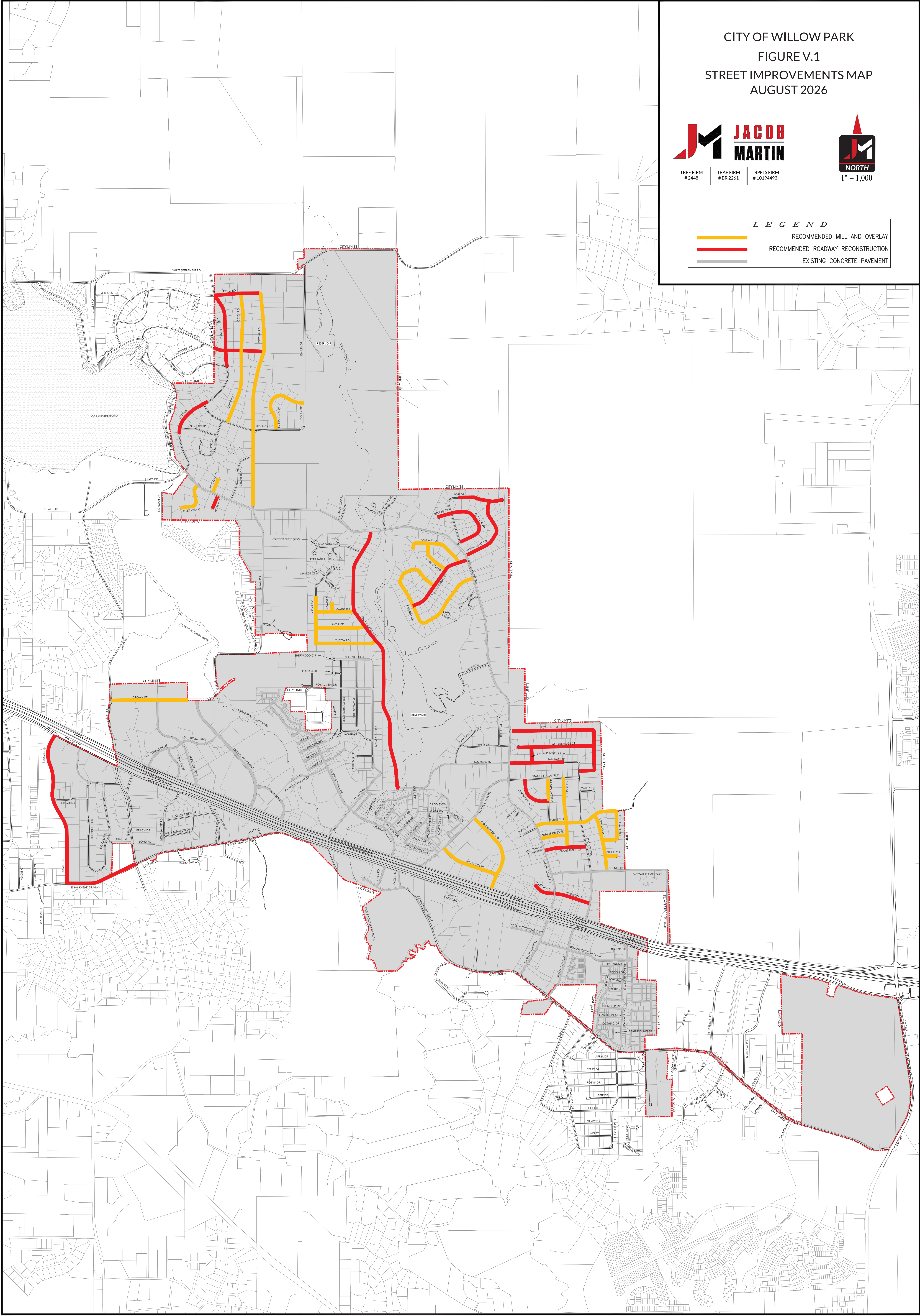
TABLE V.1
STREET SYSTEM CAPITAL PROJECTS
PRIORITY AND PROJECTED COST

Priority	Project Location	Start	End	Description	Current Avg. PCI	5 Year Avg. PCI	Linear Feet	Estimated Cost
1	Cactus Road	Verde Road	Squaw Creek Road	Mill and Overlay	47	36	1,123	\$ 300,000
2	Mesa Springs Road	Ranch House Road	Jeri Ridge Road	Mill and Overlay	47	35	766	\$ 80,000
3	Post Oak Court	Ranch House Road	End	Mill and Overlay	49	37	447	\$ 50,000
4	Aspenwood Drive	Woodbridge Court	Sam Bass Road	Road Reconstr.	25	15	469	\$ 200,000
5	Jeri Ridge Road	Pleasant Ridge Lane	Stage Coach Trail E	Mill and Overlay	50	39	2,042	\$ 170,000
6	Crown Road	Mikus Road	Crown Pointe Blvd	Mill and Overlay	51	39	2,267	\$ 186,000
7	Pleasant Ridge Lane	Ranch House Road	El Chico Trail	Road Reconstr.	26	16	1,229	\$ 500,000
8	Chaparral Court	Stage Coach Trail E	Willow Park Drive	Road Reconstr.	27	17	1,125	\$ 355,000
9	Chuckwagon Trail	Stagecoach Trail	I-20 Frontage Road	Mill and Overlay	52	40	2,462	\$ 210,000
10	Sam Bass Road	Ranch House Road	Short of El Chico Trail	Road Reconstr.	28	18	1,590	\$ 557,000
11	Crown Road	Ranch House Road	Ridge Road	Mill and Overlay	54	43	6,345	\$ 525,000
12	Regent Court	Lori Road	End	Road Reconstr.	29	19	380	\$ 231,000
13	Indian Camp Road	Crown Road	E Lake Drive	Road Reconstr.	29	17	1,459	\$ 665,000
14	Hilltop Court	Ranch House Road	End	Road Reconstr.	29	18	387	\$ 125,000
15	Woodbridge Court	Fox Hunt Trail	End	Road Reconstr.	31	21	2,161	\$ 925,000
16	Cook Road	Live Oak Road	Ridge Road	Mill and Overlay	56	46	3,847	\$ 320,000
17	Pitchfork Trail	Stagecoach Trail	Chuckwagon Trail	Mill and Overlay	57	47	1,780	\$ 155,000
18	Tumbleweed Trail	Appaloosa Street	Surrey Lane	Mill and Overlay	57	47	1,225	\$ 100,000
19	Appaloosa Street	Surrey Lane	Scenic Trail	Mill and Overlay	57	46	1,663	\$ 130,000
20	Parkway Drive	Willow Crest Drive	End	Mill and Overlay	58	47	1,688	\$ 182,000
21	Surrey Lane	Ranch House Road	Tumbleweed Trail	Mill and Overlay	58	48	2,464	\$ 211,000
22	Vista Drive	Coronado Court	Ridge Road	Road Reconstr.	34	23	2,112	\$ 1,022,000
23	Fox Hunt Trail	Ranch House Road	El Chico Trail	Road Reconstr.	34	23	3,587	\$ 1,478,000
24	Willow Crest Drive	Ranch House Road	Fairway Drive	Road Reconstr.	35	24	2,162	\$ 1,235,000
25	Russell Road	I-20 Frontage Road	E Bankhead Hwy	Road Reconstr.	37	26	4,505	\$ 2,180,000
26	Regent Row	Ranch House Road	Lori Road	Road Reconstr.	37	26	1,948	\$ 1,243,000
27	Squaw Creek Road	Sam Bass Road	Ranch House Road	Road Reconstr.	63	54	8,911	\$ 5,685,000
28	Verde Road	Start	End	Mill and Overlay	63	53	1,379	\$ 492,000
29	Canyon Court	El Chico Trail	End	Road Reconstr.	39	29	1,804	\$ 793,000
30	Ridge Road	Crest Road	Crown Road	Road Reconstr.	40	28	1,346	\$ 592,000
31	Lori Drive	Bend on Lori Drive	End	Road Reconstr.	40	30	1,326	\$ 816,000
32	Northchase Drive	Ranch House Road	Lori Drive	Road Reconstr.	41	30	2,148	\$ 1,370,000
33	Valley View Court	Ranch House Road	End	Mill and Overlay	42	31	1,045	\$ 113,000
34	Vista Drive	Live Oak Road	Nichols Road	Road Reconstr.	42	29	1,584	\$ 767,000
35	E Bankhead Hwy	FM 5	Russell Road	Road Reconstr.	43	29	2,300	\$ 1,219,000
36	Buffalo Court	Appaloosa Street	End	Mill and Overlay	65	56	318	\$ 82,000
37	Yucca Road	Verde Road	Squaw Creek Road	Mill and Overlay	66	56	1,777	\$ 587,000
38	Willow Park Drive	Chaparral Court	Stage Coach Trail E	Mill and Overlay	66	57	675	\$ 95,000
39	Buena Vista Drive	Live Oak Road	Emsley Drive	Mill and Overlay	66	57	1,944	\$ 207,000
40	Bluff View Drive	Willow Crest Drive	Parkway Drive	Mill and Overlay	66	56	1,009	\$ 113,000
41	Cactus Court	Cactus Road	End	Mill and Overlay	67	57	252	\$ 100,000
42	Mesa Road	Verde Road	Squaw Creek Road	Mill and Overlay	67	58	1,438	\$ 474,000
43	Fairway Drive	Ranch House Road	Willow Crest Drive	Mill and Overlay	68	56	4,824	\$ 414,000
Total							85,313	\$ 27,254,000

CITY OF WILLOW PARK
FIGURE V.1
STREET IMPROVEMENTS MAP
AUGUST 2026



LEGEND	
	RECOMMENDED MILL AND OVERLAY
	RECOMMENDED ROADWAY RECONSTRUCTION
	EXISTING CONCRETE PAVEMENT



VI. DRAINAGE SYSTEM

A. Inventory

The City of Willow Park's drainage system consists mainly surface drainage, roadside ditches and culverts. Some areas include curb and gutter with inlets, and storm drains. A previous inventory and analysis was prepared as part of the 2019 Master Drainage Plan by Jacob & Martin, LLC and updated in August 2022. As much of the conditions affecting drainage within the City has not changed since the date of that study, reference is made to the inventory therein. Some areas have developed since the 2019 study, however the drainage components of those developments were evaluated at the time of approval and therefore not included herein. Additionally, some improvements have been made through the City's drainage utility department over the intervening years and those improvements are accounted for in this update. For example, some culverts have been replaced and upsized in which case they have been removed from the list of structures that were determined to be under capacity. Figure V.1 shows the Culvert Inventory Map for the City of Willow Park, which was created with the 2019 study and is being utilized for this update.

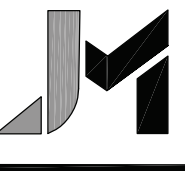
The areas within the city limits for the City of Willow Park that are currently shown to be within the FEMA floodplain are indicated in the flood maps in Figures V.2 and V.3. Most of the flood plain areas in Willow Park follow the Clear Fork of the Trinity River and Squaw Creek with minor tributaries located on the far northeast portion of the City boundary. Much of the Clear Fork flood plain is designated as Zone AE with the base flood elevation determined and floodway delineated. The Squaw Creek corridor and the other tributaries are categorized as Zone A which is defined as the area subjected to flooding in the 100-year flood event, with no base flood elevations determined.

The existing storm drainage facilities are predominantly surface drainage structures consisting of culverts, open channels, and ditches. There are also a number of storm drain structures employed by TxDOT on the I-20 drainage system which bisects the City running east and west.

The 2019 Drainage Master Plan identified numerous culverts that are under capacity as indicated in Table V.1. The corresponding list of culverts in need of replacement has been revised to remove culverts that have since been replaced and culverts for which the capacity exceedance is marginal. This resulted in 64 culverts that are under capacity and they are contained in the list in Table V.2. In specific cases of significant drainage issues, these should be addressed in the short term. Those of lesser concerns could be addressed through yearly allocation of the City's drainage utility fund and as part of larger bond projects.

Per discussions with city staff and field observation, several areas of localized flooding were identified as being of most concern. These are shown below in order of priority. First among these is a problem in the area between Sam Bass Road and Stagecoach Trail west of Ranch House Road. In this area water is backing up into yards and homes and is

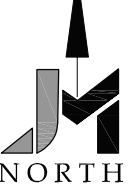
CITY OF WILLOW PARK
FIGURE V.1
CUVLERT INVENTORY MAP
MARCH 2019



**JACOB
MARTIN**

3465 CURRY LANE
ABILENE, TX 79606
325-695-1070

1508 SANTA FE DR, STE 203
WEATHERFORD, TX 76086
817-594-9680
FIRM# F-2448



NORTH
1" = 1000'

LEGEND	
	DELINEATED DRAINAGE BOUNDARY
	CITY LIMITS
	TNRIS 10-FOOT CONTOURS
	FEMA FLOODPLAIN
	RIVER OR CREEK

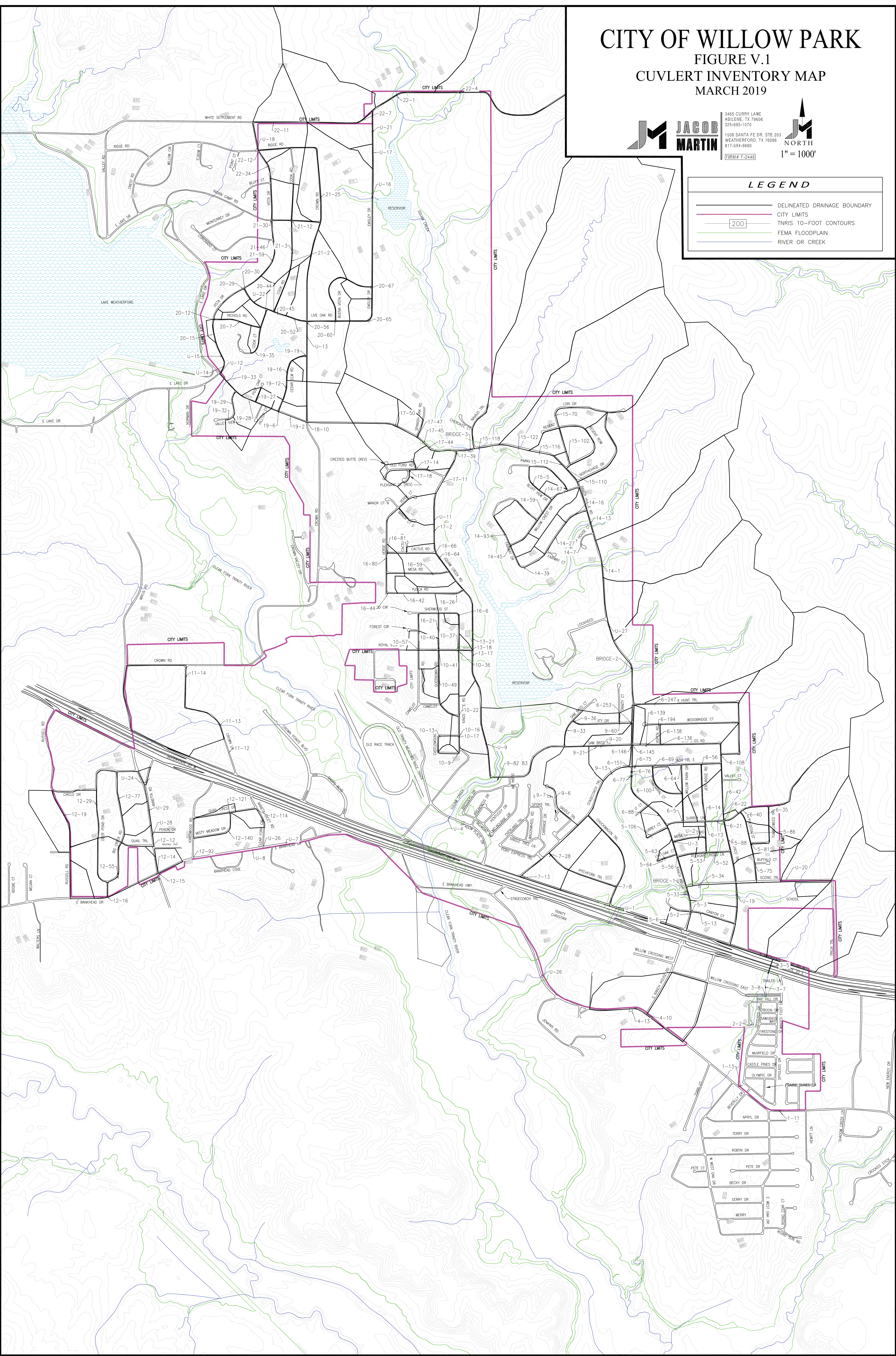


FIGURE V.2: FLOOD PLAIN MAP PANEL 48367C0300E

NOTES TO USERS

This map is for use in administering the National Flood Insurance Program. It does not necessarily identify all areas subject to flooding, particularly from local drainage sources of small size. The **community map repository** should be consulted for possible updated or additional flood hazard information.

To obtain more detailed information in areas where **Base Flood Elevations** (BFEs) and/or **floodways** have been determined, users are encouraged to consult the Flood Profiles and Floodway Data and/or Summary of Stillwater Elevations tables contained within the Flood Insurance Study (FIS) report that accompanies this FIRM. Users should be aware that BFEs shown on the FIRM represent rounded whole-foot elevations. These BFEs are intended for flood insurance rating purposes only and should not be used as the sole source of flood elevation information. Accordingly, flood elevation data presented in the FIS report should be utilized in conjunction with the FIRM for purposes of construction and/or floodplain management.

Coastal Base Flood Elevations shown on this map apply only landward of 0.3' North American Vertical Datum of 1988 (NAVD 88). Users of this FIRM should be aware that coastal flood elevations are also provided in the Summary of Stillwater Elevations table in the Flood Insurance Study report for this jurisdiction. Elevations shown in the Summary of Stillwater Elevations table should be used for construction and/or floodplain management purposes when they are higher than the elevations shown on this FIRM.

Boundaries of the **floodways** were computed at cross sections and interpolated between cross sections. The floodways were based on hydraulic considerations with regard to requirements of the National Flood Insurance Program. Floodway widths and other pertinent floodway data are provided in the Flood Insurance Study report for this jurisdiction.

Certain areas not in Special Flood Hazard Areas may be protected by **flood control structures**. Refer to Section 2.4 "Flood Protection Measures" of the Flood Insurance Study report for information on flood control structures for this jurisdiction.

The **projection** used in the preparation of this map was Texas State Plane north central zone (FIPSZONE 4202). The **horizontal datum** was NAD83, GRS1980 spheroid. Differences in datum, spheroid, projection or State Plane zones used in the production of FIRMs for adjacent jurisdictions may result in slight positional differences in map features across jurisdiction boundaries. These differences do not affect the accuracy of the FIRM.

Flood elevations on this map are referenced to the North American Vertical Datum of 1988. These flood elevations must be compared to structure and ground elevations referenced to the same vertical datum. For information regarding conversion between the National Geodetic Vertical Datum of 1929 and the North American Vertical Datum of 1988, visit the National Geodetic Survey website at <http://www.ngs.noaa.gov/> or contact the National Geodetic Survey at the following address:

NGS Information Services
NOAA, NNGS12
National Geodetic Survey
SSM-C-3, #6202
1315 East-West Highway
Silver Spring, MD 20910-3282

To obtain current elevation, description, and/or location information for **bench marks** shown on this map, please contact the Information Services Branch of the National Geodetic Survey at (301) 713-3242, or visit its website at <http://www.ngs.noaa.gov/>.

Base map information shown on this FIRM was provided in digital format by NCTCOG. This information was digitized at a scale of at least 1:12,000 from aerial photography dated 2003.

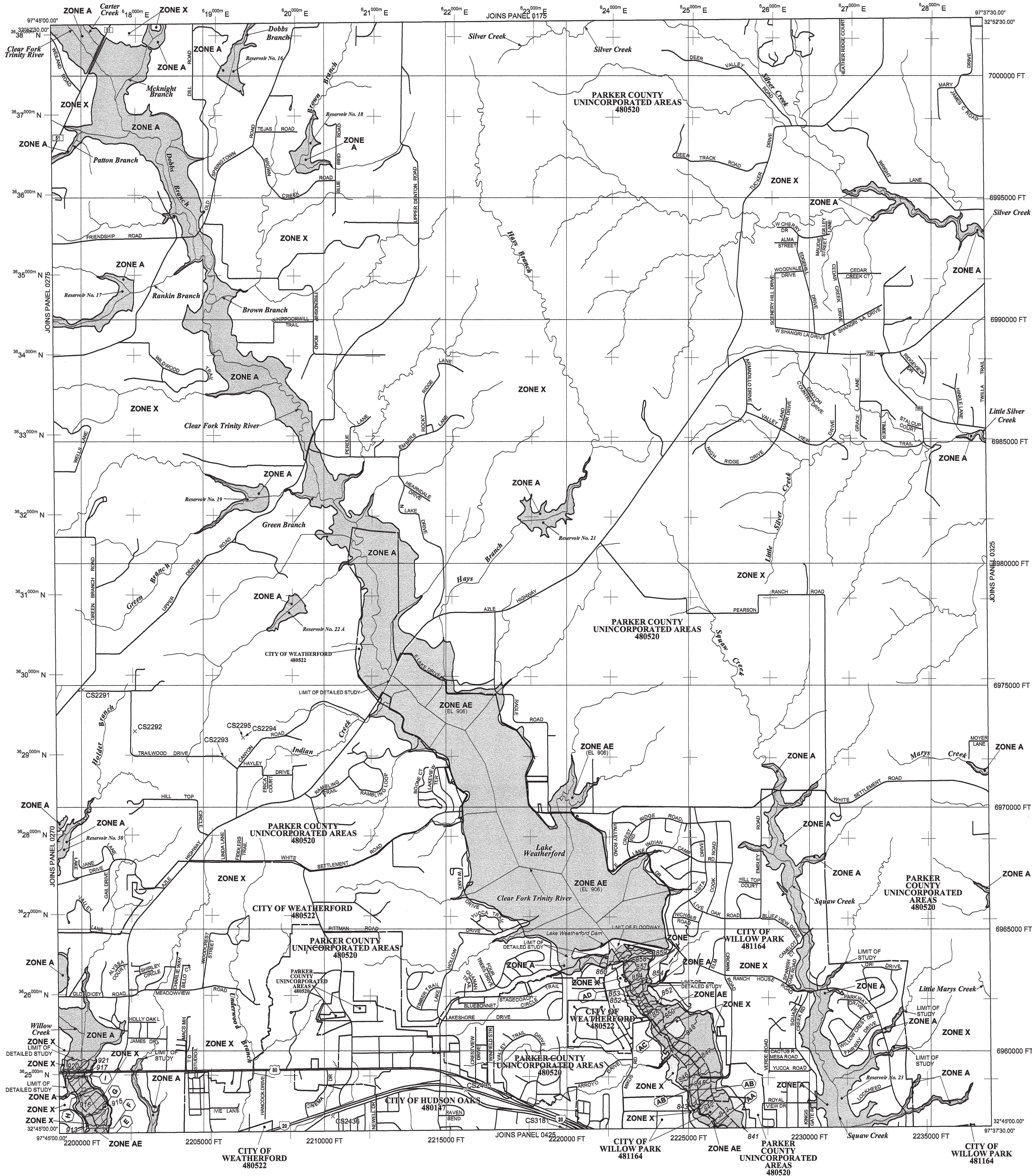
This map reflects more detailed and up-to-date **stream channel configurations** than those shown on the previous FIRM for this jurisdiction. The floodplains and floodways that were transferred from the previous FIRM may have been adjusted to conform to these new stream channel configurations. As a result, the Flood Profiles and Floodway Data tables in the Flood Insurance Study report (which contains authoritative hydraulic data) may reflect stream channel distances that differ from what is shown on this map.

Corporate limits shown on this map are based on the best data available at the time of publication. Because changes due to annexations or de-annexations may have occurred after this map was published, map users should contact appropriate community officials to verify current corporate limit locations.

Please refer to the separately printed **Map Index** for an overview map of the county showing the layout of map panels; community map repository addresses; and a Listing of Communities table containing National Flood Insurance Program dates for each community as well as a listing of the panels on which each community is located.

Contact the **FEMA Map Service Center** at 1-800-358-9616 for information on available products associated with this FIRM. Available products may include previously issued Letters of Map Change, a Flood Insurance Study report, and/or digital versions of this map. The FEMA Map Service Center may also be reached by Fax at 1-800-358-9620 and its website at <http://www.msc.fema.gov/>.

If you have **questions about this map** or questions concerning the National Flood Insurance Program in general, please call 1-877-FEMA MAP (1-877-336-2827) or visit the FEMA website at <http://www.fema.gov/>.



LEGEND

SPECIAL FLOOD HAZARD AREAS (SFHAS) SUBJECT TO INUNDATION BY THE 1% ANNUAL CHANCE FLOOD

The 1% annual chance flood (100-year flood), also known as the base flood, is the flood that has a 1% chance of being equaled or exceeded in any given year. The Special Flood Hazard Area is the area subject to flooding by the 1% annual chance flood. Areas of Special Flood Hazard include Zones A, AE, AH, AO, AR, A99, V and VE. The Base Flood Elevation is the water-surface elevation of the 1% annual chance flood.

ZONE A No Base Flood Elevations determined.

ZONE AE Base Flood Elevations determined.

ZONE AH Flood depths of 1 to 3 feet (usually areas of ponding); Base Flood Elevations determined.

ZONE AO Flood depths of 1 to 3 feet (usually sheet flow on sloping terrain); average depths determined. For areas of alluvial fan flooding, velocities also determined.

ZONE AR Special Flood Hazard Area formerly protected from the 1% annual chance flood by a flood control system that was subsequently described in Zone AR indicates that the former flood control system is being restored to provide protection from the 1% annual chance or greater flood.

ZONE A99 Area to be protected from 1% annual chance flood by a Federal flood protection system under construction; no Base Flood Elevations determined.

ZONE V Coastal flood zone with velocity hazard (wave action); no Base Flood Elevations determined.

ZONE VE Coastal flood zone with velocity hazard (wave action); Base Flood Elevations determined.

FLOODWAY AREAS IN ZONE AE

The floodway is the channel of a stream plus any adjacent floodplain areas that must be kept free of encroachment so that the 1% annual chance flood can be carried without substantial increases in flood heights.

OTHER FLOOD AREAS

ZONE X Areas of 0.2% annual chance flood; areas of 1% annual chance flood with average depths of less than 1 foot or with drainage areas less than 1 square mile; and areas protected by levees from 1% annual chance flood.

OTHER AREAS

ZONE X Areas determined to be outside the 0.2% annual chance floodplain.

ZONE D Areas in which flood hazards are undetermined, but possible.

COASTAL BARRIER RESOURCES SYSTEM (CBRS) AREAS

OTHERWISE PROTECTED AREAS (OPAs)

CBRS areas and OPAs are normally located within or adjacent to Special Flood Hazard Areas.

— Floodplain boundary
— Floodway boundary
— Zone D boundary
— CBRS and OPA boundary
— Boundary dividing Special Flood Hazard Areas of different Base Flood Elevations, flood depths or flood velocities.
— Base Flood Elevation line and value; elevation in feet*
— Base Flood Elevation value where uniform within zone; elevation in feet*
* Referenced to the North American Vertical Datum of 1988 (NAVD 88)

— Cross section line
— Transsect line
— Geographic coordinates referenced to the North American Datum of 1983 (NAD 83)
— 1000-meter Universal Transverse Mercator grid, zone 14
— 5000-foot grid : Texas State Plane coordinate system, north central zone (FIPSZONE 4202), Lambert Conformal Conic
— Bench mark (see explanation in Notes to Users section of this FIRM panel)
— M1.5 River Mile

MAP REPOSITORIES
Refer to Map Repositories list on Map Index

EFFECTIVE DATE OF COUNTYWIDE FLOOD INSURANCE RATE MAP
September 26, 2008
EFFECTIVE DATE(S) OF REVISION(S) TO THIS PANEL

For community map revision history prior to countywide mapping, refer to the Community Map History table located in the Flood Insurance Study report for this jurisdiction.

To determine if flood insurance is available in this community, contact your insurance agent or call the National Flood Insurance Program at 1-800-638-6620.

MAP SCALE 1" = 2000'
0 1000 2000 4000 FEET
0 600 1200 METERS

NATIONAL FLOOD INSURANCE PROGRAM

PANEL 0300E

FIRM
FLOOD INSURANCE RATE MAP
PARKER COUNTY, TEXAS
AND INCORPORATED AREAS

PANEL 300 OF 575
(SEE MAP INDEX FOR FIRM PANEL LAYOUT)

CONTAINS:

COMMUNITY	NUMBER	PANEL	SUFFIX
PARKER COUNTY	480520	0300	E
HUDSON OAKS, CITY OF	480147	0300	E
WEATHERFORD, CITY OF	480522	0300	E
WILLOW PARK, CITY OF	481164	0300	E

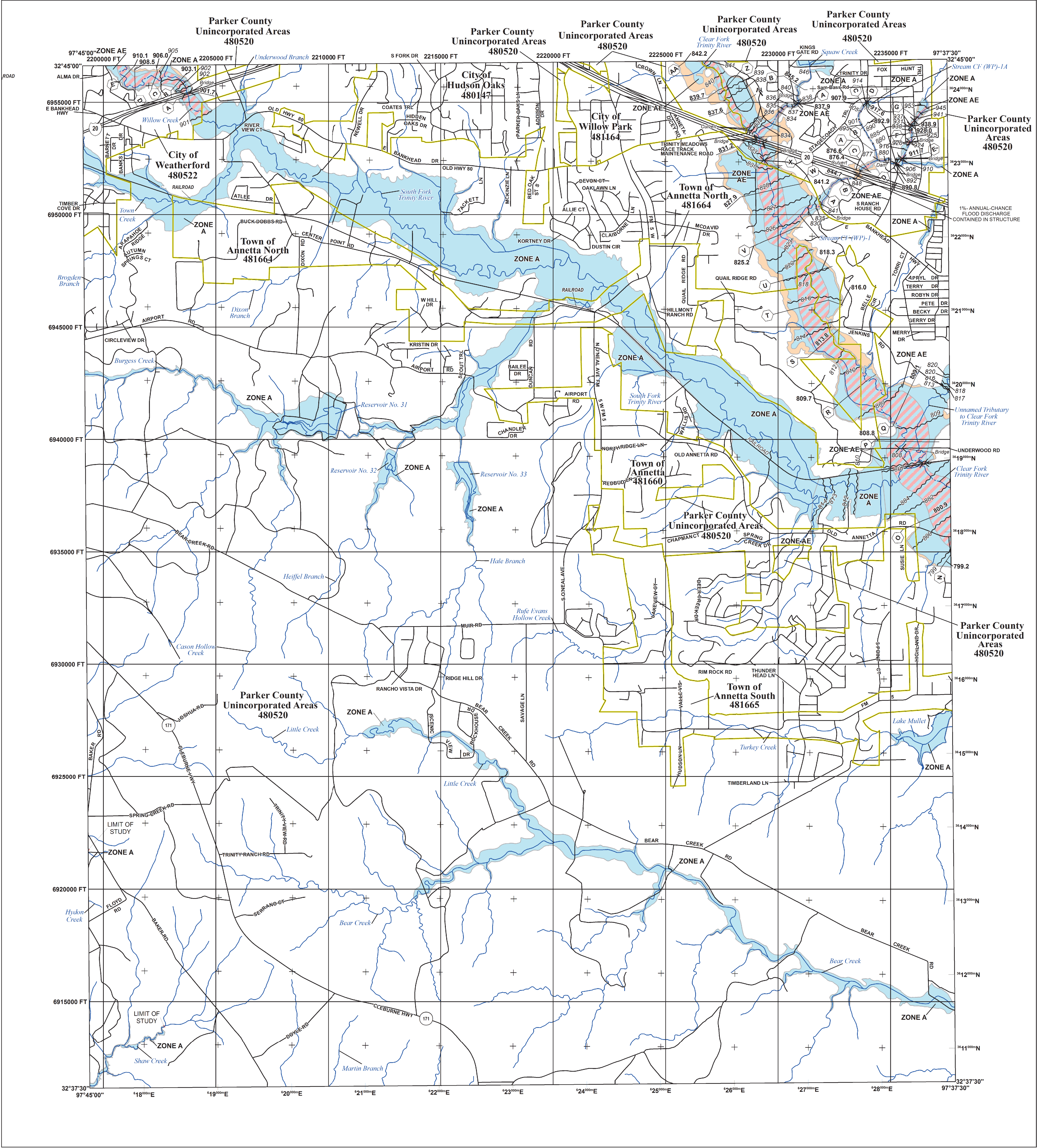
Notice to User: The Map Number shown below should be used when placing map orders; the Community Number shown above should be used on insurance applications for the subject community.

MAP NUMBER
48367C0300E

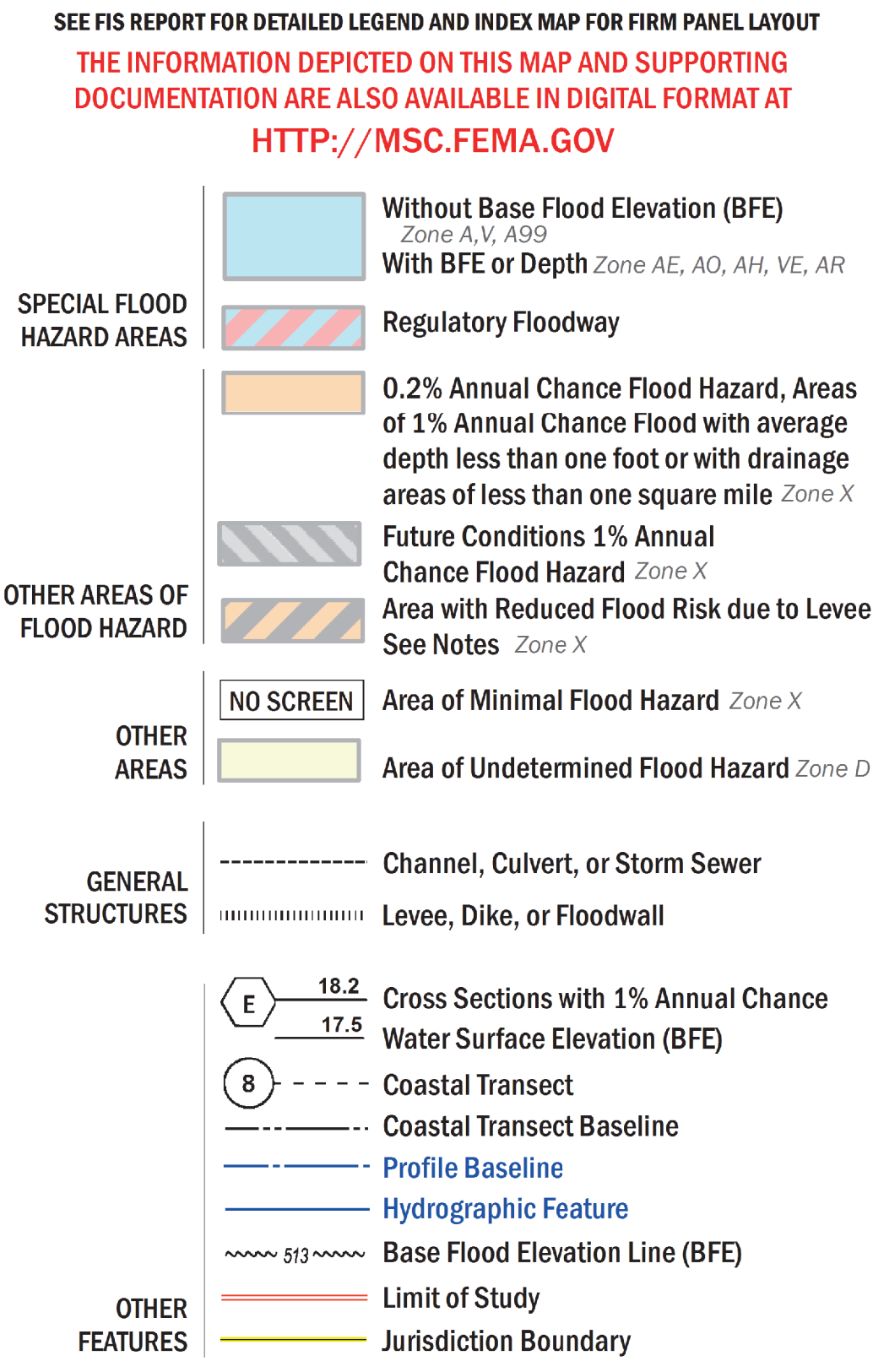
EFFECTIVE DATE
SEPTEMBER 26, 2008

Federal Emergency Management Agency

FIGURE V.2: FLOOD PLAIN MAP PANEL 48367C0425F



FLOOD HAZARD INFORMATION



NOTES TO USERS

For information and questions about this Flood Insurance Rate Map (FIRM), available products associated with this FIRM including historic versions, the current map date for each FIRM panel, how to order products or the National Flood Insurance Program in general, please call the FEMA Map Information eXchange at 1-877-FEMA-MAP (1-877-336-2627) or visit the FEMA Map Service Center website at <http://msc.fema.gov>. Available products may include previously issued Letters of Map Change, a Flood Insurance Study Report, and/or digital versions of this map. Many of these products can be ordered or obtained directly from the website.

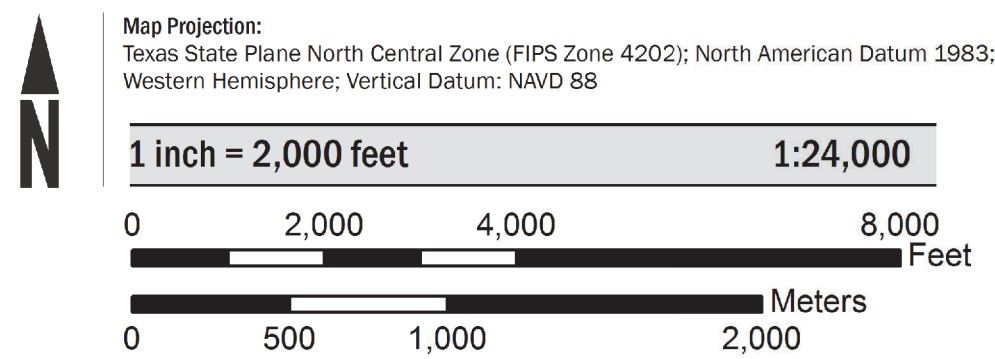
Communities annexing land on adjacent FIRM panels must obtain a current copy of the adjacent panel as well as the current FIRM index. These may be ordered directly from the Map Service Center at the number listed above.

For community and countywide map dates refer to the Flood Insurance Study report for this jurisdiction.

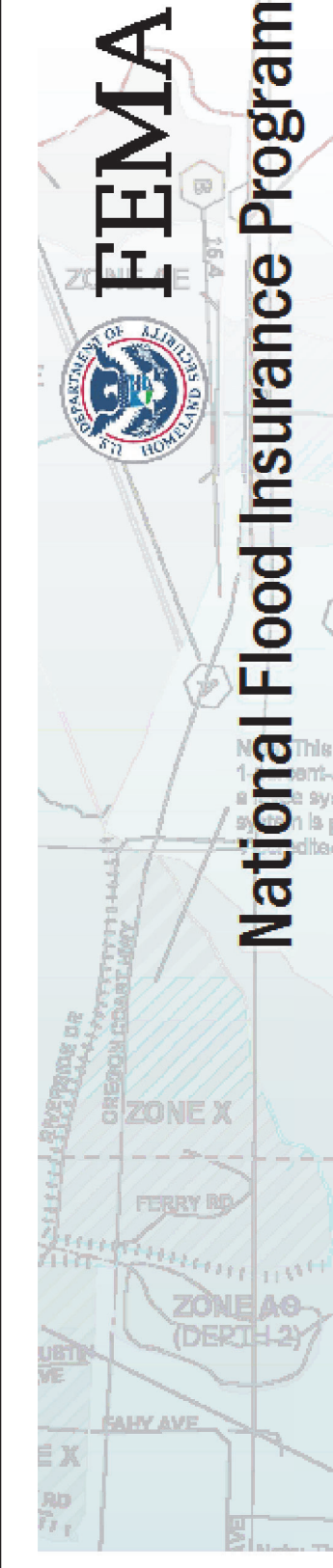
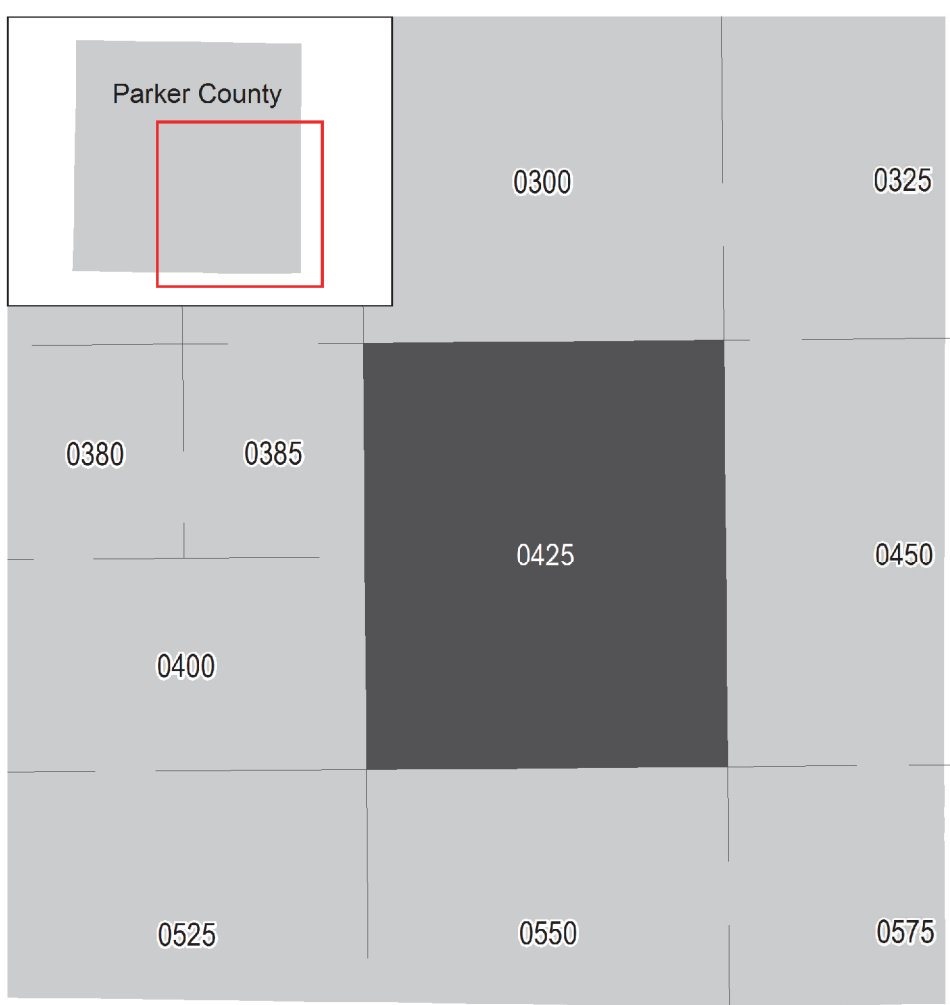
To determine if flood insurance is available in this community, contact your Insurance agent or call the National Flood Insurance Program at 1-800-638-6620.

Base map information shown on this FIRM was derived from various sources in 2009, including Texas Natural Resources Information System, North Central Texas Council of Governments, National Geodetic Survey, and United States Geological Survey.

SCALE



PANEL LOCATOR



NATIONAL FLOOD INSURANCE PROGRAM
FLOOD INSURANCE RATE MAP

PARKER COUNTY, TEXAS
And Incorporated Areas

PANEL 425 of 575

Panel Contains:			
COMMUNITY	NUMBER	PANEL	SUFFIX
ANNETTA, TOWN OF	481660	0425	F
ANNETTA NORTH, TOWN OF	481664	0425	F
ANNETTA SOUTH, TOWN OF	481665	0425	F
HUDSON OAKS, CITY OF	480147	0425	F
PARKER COUNTY	480520	0425	F
WEATHERFORD, CITY OF	480522	0425	F
WILLOW PARK, CITY OF	481164	0425	F

VERSION NUMBER

2.3.3.2

MAP NUMBER

48367C0425F

MAP REVISED

APRIL 5, 2019

permanently ponding. This is due significantly to an inadequate ditch/drainage system to direct the storm water west to Ranch House Road and a Squaw Creek Tributary.

The next area, deemed to be the second highest priority, is the culvert located at Ranch House Road and Canyon Court and crossing Canyon Court. This culvert has collapsed and will not convey adequate stormwater. The culvert is also listed in Table V.2 as needing to be upsized.

There is a section of channel that conveys a large amount of stormwater across El Chico Trail to Surrey Lane. Upstream improvements have improved drainage in this area, but the restriction in this stretch limits conveyance downstream. This is due mainly to poorly maintained ditch and poor hydraulic capacity of the natural ditch section.

The culvert at the intersection of Crown Road and Live Oak Road (crossing northeast to southwest) is insufficient to adequately convey the stormwater away from the residences on the east side. This is due to the size of the culvert and poor inlet and outlet hydraulics.

There are several areas within the City where the roadside ditches are extremely flat leading to ponding and flooding in extreme rainfall events. These include in the order of designated priority: the roadside ditch on Crown Road between Cedar Elm Road and Ranch House Road, the roadside ditches along Chuckwagon Trail (culverts on the downstream end of this area are also inadequate to convey the stormwater), the roadside ditches along the loop formed by Deer Pond Drive and Big Creek, the roadside ditches along Vista Drive between Live Oak Road and Nichols Road, the roadside ditches on Northchase Drive, and the roadside ditches and offstreet channels in the Ridge Haven Estates Subdivision.

The culvert on Live Oak near Cook Road does not convey adequately, probably due to being stopped up and/or inadequate size. This leads to flooding of the adjacent home. The culvert is also listed in Table V.2 as needing to be upsized.

The culvert outlets and grading from the Gates at Meadow Place apartments and the Reserves at Trinity subdivision to the east bank of the Clear Fork of the Trinity River have eroded and are not protected adequately from ongoing degradation.

The major channel crossings of Squaw Creek Road are inadequate to prevent street overtopping even in lower storm events, have inadequate headwall design, and are deteriorated. There are culverts along this road that are listed in Table V.2 as needing to be upsized as well.

These are the areas that have been identified as having the most frequent flooding based on observation and citizen reports. Flooding may not be observed in other areas that feed to culverts that are under capacity except under the most severe storm events, it may not reach structures because of pad elevations, or it may occur in sparsely populated areas.

B. Analysis

1. Introduction

As cities and towns develop and expand, storm water drainage becomes an increasingly serious problem. In recent years, this problem has been intensified by the public's expectations for cities to provide better drainage. The public is no longer willing to tolerate the inconvenience and delays of flooded intersections and storm water standing around their homes for days after a rain event. With limited budgets for all public works projects, city leadership and staff are under pressure to secure the best planned and most economical design for their storm drainage system.

The basic approach to the handling of storm water runoff is the same today as it was in the past, that is, to collect as much of the storm water runoff as soon as possible in an urban area and discharge it by way of a drainage system into the nearest natural draw, lake or stream. The collection and transmission of excess storm water runoff provides a means of minimizing damage to property and inconvenience to citizens that will occur otherwise from time to time. This can be accomplished by the construction of a surface drainage system, an underground drainage system, or a combination of the two in some cases.

A surface system includes transporting storm water by way of streets, either in curbs, grass or lined ditches, or channeled into natural drains and creeks. This type of system could also include temporary storm water storage in parking lots, retention ponds and possibly infiltration and evaporation basins. Subsurface or underground systems would include inlet boxes and storm sewer pipes. In some cases, pumps would be used with this type of system. Other possible components of this type of system would include infiltration trenches or dry wells.

2. Drainage Facility Planning

In past years, drainage design and improvements for newly developing areas were often corrective exercises performed subsequent to the planning process. In recent years, it has been realized that to achieve the most cost effective and efficient drainage system, the system needs to be worked out during the planning process, ideally with the aid of an engineer. A functional drainage design should support any plans for new development submitted to the City for approval. Of primary importance in the planning and design of storm drainage facilities is to establish levels of flood protection for different components of the system and possibly different areas of the City. The level of flood protection is usually expressed in terms of rainstorm intensity and corresponding runoff for which protection is to be provided.

Rainstorms can be classified by their intensity and are grouped according to the frequency with which they occur. Since rainstorm patterns differ from one region of the country to another, the National Weather Service has provided storm frequency sources for each area of the country. This system of curves shows the intensity of rainfall expressed in inches per hour for a specific duration of rainfall. A storm intensity along

with the 25-year frequency line would have a 4% probability of occurring in any given year. However, there is still the possibility that two storms of this magnitude could possibly occur in the same year or even month, but the frequency curve tells us that over a long period of time, the probability of an area having this magnitude storm would happen only once in each 25-year period.

These rainfall intensity curves form the basis and are essential for the analysis of peak rate runoff, quantities or runoff, and time distribution of flows: all of which are fundamental to the design of any drainage system.

3. Facility Design Standards

a) Recommended Levels of Flood Protection for the Willow Park Area

City storm drain facilities should generally be designed to convey the 5-year, 10-year, 25-year, or 100-year storm event depending on the specific drainage facility. For storm sewer pipes with inlets on grade in residential areas, the 5-year storm event should be the minimum storm design storm frequency. For storm sewer pipes with inlets on grade in non-residential areas, the 10-year storm event should be the minimum storm design storm frequency. For storm sewer pipes draining runoff from low points, the 25-year storm event should be the minimum storm design storm frequency, provided that positive overflow is provided for the 100-year design storm event. Culverts, creeks, channels, bridges, and similar drainage facilities should be designed using a 100-year design storm frequency, as these facilities ultimately protect residents and structures from flooding. Sizing these facilities for the 100-year event protects the public and ensures that the runoff can be properly conveyed. For analysis of existing culverts, the threshold used was the 25-year event so long as positive conveyance for the 100-year event could be achieved without flooding.

Collector streets and major city thoroughfares shall have design capacities that, in the 10-year frequency storm event, maintain an 8 foot wide minimum drive lane in each driving direction that is not inundated by the spread of water in the street. For residential streets, the design shall be that the flow is not more than 1 inch above the top of curb in the 5-year frequency storm event. For all streets, the 25-year storm event shall be designed to be conveyed in and limited to the right-of-way, and the 100-year storm event shall be designed and confirmed to not encroach on any habitable structures.

b) Runoff Quantity Calculations

The rational method is a common approach for estimating runoff in urban drainage areas, in particular for smaller drainage areas, such as those found in the City of Willow Park and its surrounding areas. The City of Willow Park utilizes the rational method for any watershed's that are less than or equal to 1,000 acres. For all drainage areas considered in this study, the rational method was used to estimate storm runoff for the 2, 10, 25, and 100-year frequency storm events.

The Rational Method formula allows for the calculation of an estimate of the peak runoff rate at any location within a given drainage area as a function of the drainage area, a runoff coefficient, and the mean rainfall intensity for a duration equal to the time of concentration (the time required for water to flow from the most remote point of the drainage area to the discharge point). The Rational Method is expressed as follows:

$$Q = C \times C_a \times I \times A$$

Where:

Q	=	maximum rate of runoff in cubic feet per second
C	=	runoff coefficient representing the ratio of runoff to rainfall
C _a	=	antecedent precipitation factor (frequency factor)
I	=	average rainfall intensity for a duration equal to the time of concentration in inches per hour
A	=	drainage area in acres

The Rational Method allows for a simple and straight-forward method of calculating runoff. The runoff coefficient “C” is the variable of the Rational Method most susceptible to imprecise determination. Factors vary depending upon the amount of impervious surface (paved, concrete or roof areas) in the drainage area. The majority of the hydrologic soil groups making up the City of Willow Park tend to be Hydrologic Soil Group D. The accepted values for the runoff coefficient per the respective hydrologic soil groups in the City of Willow Park are as follows:

<u>Land Use</u>	<u>Hydrologic Soil Group</u>			
	<u>A</u>	<u>B</u>	<u>C</u>	<u>D</u>
Parks/Open Space	0.35	0.38	0.41	0.45
Single-Family Zoning	0.50	0.52	0.55	0.58
Duplex Zoning	0.60	0.63	0.66	0.70
Apartment Zoning	0.65	0.70	0.77	0.80
Commercial	0.90	0.91	0.90	0.93
Central Business District	1.00	1.00	1.00	1.00
Industrial	*As specified by design engineer and approved by city engineer*			

The antecedent precipitation factor “C_a”, also known as the frequency factor, is applicable for storms with a recurrence interval of 10-years or less. Storms with higher intensity and less frequent recurrence intervals should require adjustment to the coefficient given the smaller effect of infiltration losses on total runoff. The frequency factors “C_a” given the associated recurrence interval are as follows

<u>Recurrence Interval (Years)</u>	<u>Coefficient (Ca)</u>
10-Year or Less	1.0
25-Year	1.1
50-Year	1.2
100-Year	1.25

c) Unit Hydrograph

The unit hydrograph method is used for any watersheds that are greater than 1,000 acres. This method essentially establishes a relationship between the runoff and discharge of a watershed, and estimates peak flows and hydrographs for all design applications.

4. Channel Design

The City of Willow Park's drainage system not only includes earthen channels, storm drains, and ditches, but encompasses the use of strategically chosen streets for the collection and transmission of storm water drainage. The size of a drainage channel is determined by the amount of storm water it is to carry. With the design of channels, the Manning's Formula, which is most commonly used in open channel flow calculations, should be applied as follows:

$$Q = \frac{1.486}{n} A R^{2/3} S^{1/2}$$

Where:

- Q = flow in cubic feet per second (cfs)
- n = Manning's roughness coefficient
- A = cross-sectional area in square feet
- R = hydraulic radius in ft (cross sectional area divided by wetted perimeter)
- S = slope per foot of length

All values, with the exception of the roughness coefficient (n), are dependent on the channel section chosen and the volume of storm water at the design point. For the selection of the proper roughness coefficient, the following are typical values and guidelines:

- a. Earth channels and ditches 0.025 to 0.018
- b. Earth channels and ditches with weeds 0.033 to 0.022
- c. Concrete lined channels and rough asphalt 0.018 to 0.012

For the City of Willow Park, the following values are recommended.

- a. Earth channels and ditches 0.022
- b. Earth channels and ditches with weeds 0.027
- c. Concrete lined channels and rough asphalt 0.016

The capacities of street and gutter shall also be determined using Manning's Formula. The typical values for the roughness coefficient depending on the street type chosen for design are as follows:

- | | | |
|----|------------------|-------|
| a. | Asphalt streets | 0.015 |
| b. | Concrete streets | 0.013 |

Positive overflow should always be provided within the street if possible. In the event that positive overflow cannot be achieved, a minimum 4-foot wide, curbed concrete flume should be provided between the lots within a drainage easement at the low point of the street.

5. Terrain Features and Existing Drainage Facilities

In general, the majority of the watershed within the City of Willow Park's city limits drains to Squaw Creek or the Clear Fork of the Trinity River or to a tributary thereof.

The non-grid layout of much of the City of Willow Park less than ideal. The ditches throughout the city should be well maintained in order to maximize the efficiency of the drainage through them. Localized drainage problem areas should be improved by regrading the channel, or in some cases lining the channel with concrete and improving culverts.

The estimated design flow for each drainage area and its associated drainage structure is shown in Table V.1, along with the respective intensities based on approximate times of concentration.

The culverts shown in Table V.1 are highlighted according to two different statuses of each culvert. The culverts that currently have adequate capacity are highlighted in green. The culverts that do not have adequate capacity in that structure's respective critical storm event are highlighted in pink. Table V.2 indicates the culverts that are deemed to be in need of replacement. The culverts that coincide with priority drainage projects are highlighted and noted at the top of the list.

TABLE V.1
MASTER PLAN CULVERT LIST
AND
DRAINAGE CALCULATIONS

Structure ID	Upstream Structure(s)	C/CN	Lov (ft)	Yov (%)	Lch (ft)	Ych (ft/ft)	Tc (min)	I10 (in/hr)	I25 (in/hr)	I50 (in/hr)	I100 (in/hr)	Area (ac)	Flow from Area (cfs)				Cumulative Flow at Outlet (cfs)				Existing Size (in)	Material	HW (ft)	HW/D	Entrance Type	Flow Capacity (cfs)	Sufficient Capacity (Y/N)
													Q 10-yr	Q 25-yr	Q 50-yr	Q 100-yr	Q 10-yr	Q 25-yr	Q 50-yr	Q 100-yr							
19.12		0.45	538.47	0.019			25.97	4.33	5.06	5.82	6.61	6.33	12.3	14.4	16.6	18.8	12.3	14.4	16.6	18.8	12	rcp	3	3	projecting	7.25	N
6.56		0.45			536.1	0.017	10.00	7.13	8.26	9.5	10.8	3.47	11.1	12.9	14.8	16.9	11.1	12.9	14.8	16.9	12	rcp	3	3	projecting	7.25	N
6.100		0.45			898.68	0.026	10.00	7.13	8.26	9.5	10.8	5.96	19.1	22.2	25.5	29.0	19.1	22.2	25.5	29.0	12	rcp	3	3	projecting	7.25	N
6.5		0.45	156.34	0.038	496.87	0.012	17.38	5.45	6.34	7.3	8.29	3.21	7.9	9.2	10.5	12.0	7.9	9.2	10.5	12.0	12	rcp	2.2	2.2	projecting	6	N
6.88	6.5	0.45			1124.56	0.02	10.00	7.13	8.26	9.5	10.8	6.27	20.1	23.3	26.8	30.5	28.0	32.5	37.3	42.4	12	rcp	3	3	projecting	7.25	N
5.81		0.45	719.8	0.056	453.19	0.046	25.81	4.35	5.08	5.84	6.64	7.12	13.9	16.3	18.7	21.3	13.9	16.3	18.7	21.3	12	rcp	2.1	2.1	projecting	5.5	N
19.6		0.45	176.76	0.023	227.73	0.004	18.86	5.21	6.07	6.98	7.93	1.23	2.9	3.4	3.9	4.4	2.9	3.4	3.9	4.4	12	cmp	2.2	2.2	projecting	4.5	Y
5.64	5.56,u-3	0.45			694.16	0.024	10.00	7.13	8.26	9.5	10.8	2.49	8.0	9.3	10.6	12.1	33.3	38.7	44.5	50.6	12	cmp	2.5	2.5	projecting	4.75	N
21.2	21.12	0.45			1325.41	0.017	9.42	7.13	8.26	9.5	10.8	11.6	37.1	43.0	49.4	56.2	44.6	51.7	59.4	67.6	12	cmp	3.5	2.3	headwall	6	N
21.3																					18			3.5		6.75	
5.3		0.45			1431.07	0.02	10.00	7.13	8.26	9.5	10.8	7.57	24.3	28.1	32.4	36.8	24.3	28.1	32.4	36.8	12	cmp	3	3	headwall	6	N
21.12		0.45	117.36	0.03	783.72	0.009	19.17	5.16	6.02	6.92	7.86	3.22	7.5	8.7	10.0	11.4	7.5	8.7	10.0	11.4	15	rcp	2.9	2.3	projecting	6	N
19.19		0.45			573.96	0.019	4.76	7.13	8.26	9.5	10.8	2.2	7.1	8.2	9.4	10.7	7.1	8.2	9.4	10.7	15	rcp	3	2.4	projecting	6	N
19.27		0.45			383.23	0.034	2.80	7.13	8.26	9.5	10.8	3.04	9.8	11.3	13.0	14.8	9.8	11.3	13.0	14.8	15	rcp	3.3	2.6	projecting	7.25	N
6.108		0.45			1072.98	0.007	11.67	6.65	7.72	8.88	10.09	4.73	14.2	16.4	18.9	21.5	14.2	16.4	18.9	21.5	15	rcp	3.1	2.4	groove w/headwall	11	N
20.65		0.45	489.88	0.01	377.41	0.003	33.81	3.68	4.31	4.95	5.63	5.95	9.9	11.5	13.3	15.1	9.9	11.5	13.3	15.1	15	cmp	2.6	2.1	projecting	7.8	N
10.40	10.57	0.45			822.95	0.021	10.00	7.13	8.26	9.5	10.8	3.75	12.0	13.9	16.0	18.2	20.1	23.2	26.7	30.4	15	cmp	3	2.4	projecting	7.8	N
10.36	10.41,10.49	0.45	460.68	0.009	894.29	0.029	34.57	3.63	4.25	4.88	5.55	11.5	18.8	22.0	25.3	28.8	47.0	54.8	63.0	71.7	15	cmp	3.1	2.5	projecting	7.8	N
10.9		0.45			443.86	0.023	10.00	7.13	8.26	9.5	10.8	1.56	5.0	5.8	6.7	7.6	5.0	5.8	6.7	7.6	15	cmp	2.9	2.3	projecting	7.8	Y
U-2		0.45	181.88	0.022	141.05	0.05	16.15	5.66	6.59	7.58	8.61	0.97	2.5	2.9	3.3	3.8	2.5	2.9	3.3	3.8	15	cmp	3.3	2.6	projecting	7.8	Y
12.114		0.45	360.5	0.039	217.89	0.032	19.96	5.05	5.89	6.77	7.69	4.52	10.3	12.0	13.8	15.6	10.3	12.0	13.8	15.6	15	cmp	3	2.4	projecting	7.8	N
17.39		0.45	448.32	0.051	465.58	0.047	21.64	4.82	5.63	6.47	7.35	5.13	11.1	13.0	14.9	17.0	11.1	13.0	14.9	17.0	15	cmp	3.1	2.5	headwall	9.25	N
16.21		0.45			801.52	0.02	6.06	7.13	8.26	9.5	10.8	5.17	16.6	19.2	22.1	25.1	16.6	19.2	22.1	25.1	15	cmp	2.9	2.3	headwall	9.25	N
15.110		0.45			2453.17	0.016	15.50	5.79	6.73	7.74	8.79	17	44.3	51.5	59.3	67.3	44.3	51.5	59.3	67.3	15	cmp	6	4.8	headwall	14.5	N
6.146	6.253	0.45			466.63	0.034	10.00	7.13	8.26	9.5	10.8	0.35	1.1	1.3	1.5	1.7	2.3	2.7	3.1	3.5	15	cmp	2.8	2.2	headwall	9.25	Y
10.41		0.45			850.41	0.025	5.84	7.13	8.26	9.5	10.8	4.38	14.1	16.3	18.7	21.3	14.1	16.3	18.7	21.3	15	iron	2.7	2.2	projecting	10.25	N
10.37	10.57,10.40,16.21	0.45			987.27	0.04	10.00	7.13	8.26	9.5	10.8	7.3	23.4	27.1	31.2	35.5	60.1	69.6	80.0	91.0	15	iron	3.6	2.9	projecting	11.5	N
10.49		0.45	366.65	0.025	254.57	0.008	23.91	4.55	5.31	6.11	6.94	6.92	14.2	16.5	19.0	21.6	14.2	16.5	19.0	21.6	15	iron	2.6	2.1	projecting	10.25	N
10.16	10.17	0.45			447.32	0.018	10.00	7.13	8.26	9.5	10.8	1.99	6.4	7.4	8.5	9.7	8.4	9.7	11.2	12.7	15	iron	3.1	2.5	projecting	10.25	Y
10.17		0.45			363.25	0.011	10.00	7.13	8.26	9.5	10.8	0.62	2.0	2.3	2.7	3.0	2.0	2.3	2.7	3.0	15	iron	2.8	2.2	projecting	10.25	Y
10.13		0.45			364.82	0.033	10.00	7.13	8.26	9.5	10.8	1.51	4.8	5.6	6.5	7.3	4.8	5.6	6.5	7.3	15	iron	3	2.4	projecting	10.25	Y
B-2		77.7	4789.1	3.278			65.92	2.34	2.76	3.17	3.61	231	173.7	240.5	310.4	389.6	173.7	240.5	310.4	389.6	2-15"	rcp	4.5	4	headwall	24	N
																					21"		4.5	2.9		22	
																					2-36"		4.5	1.7		126	
21.30		0.45	504	0.04	773.17	0.003	34.27	3.65	4.27	4.91	5.58	9.94	16.3	19.1	22.0	25.0	16.3	19.1	22.0	25.0	18	rcp	3.2	2.1	projecting	15.5	N
20.30		0.45	366.39	0.02	437.56	0.003	30.46	3.93	4.6	5.29	6.01	4.98	8.8	10.3	11.9	13.5	8.8	10.3	11.9	13.5	18	rcp	3.6	2.4	projecting	15.5	Y

TABLE V.1
MASTER PLAN CULVERT LIST
AND
DRAINAGE CALCULATIONS

													Flow from Area (cfs)				Cumulative Flow at Outlet (cfs)										
Structure ID	Upstream Structure(s)	C/CN	Lov (ft)	Yov (%)	Lch (ft)	Ych (ft/ft)	Tc (min)	I10 (in/hr)	I25 (in/hr)	I50 (in/hr)	I100 (in/hr)	Area (ac)	Q 10-yr	Q 25-yr	Q 50-yr	Q 100-yr	Q 10-yr	Q 25-yr	Q 50-yr	Q 100-yr	Existing Size (in)	Material	HW (ft)	HW/D	Entrance Type	Flow Capacity (cfs)	Sufficient Capacity (Y/N)
20.29		0.45			833	0.011	7.91	7.13	8.26	9.5	10.8	4.75	15.2	17.7	20.3	23.1	15.2	17.7	20.3	23.1	18	rcp	3.6	2.4	projecting	15.5	N
20.7		0.45			505.13	0.004	7.92	7.13	8.26	9.5	10.8	8.2	26.3	30.5	35.1	39.9	26.3	30.5	35.1	39.9	18	rcp	5.5	3.7	projecting	21.5	N
20.44		0.45			1006.08	0.019	7.38	7.13	8.26	9.5	10.8	5.99	19.2	22.3	25.6	29.1	19.2	22.3	25.6	29.1	18	rcp	3.1	2.1	projecting	15.5	N
20.45	20.44	0.45	235.97	0.05	1084.79	0.015	22.54	4.71	5.5	6.32	7.18	6.77	14.3	16.8	19.3	21.9	14.3	16.8	19.3	21.9	18	rcp	4.1	2.7	projecting	18	Y
20.60		0.45	127.12	0.04	1095.71	0.018	19.07	5.18	6.04	6.94	7.88	7.58	17.7	20.6	23.7	26.9	17.7	20.6	23.7	26.9	18	rcp	4	2.7	projecting	18	N
19.32		0.45	550.96	0.047	279.35	0.1	22.53	4.71	5.5	6.32	7.18	5.38	11.4	13.3	15.3	17.4	11.4	13.3	15.3	17.4	18	rcp	5.2	3.5	projecting	20	Y
18.10		0.45	296.87	0.01	883.27	0.02	29.17	4.03	4.72	5.43	6.17	17.6	31.9	37.3	42.9	48.8	31.9	37.3	42.9	48.8	2-18	rcp	2.8	1.9	projecting	31	N
16.81		0.45	203.32	0.034			14.25	6.04	7.02	8.08	9.17	0.59	1.6	1.9	2.1	2.4	1.6	1.9	2.1	2.4	18	rcp	3.4	2.3	projecting	15.5	Y
13.21	16.6,16.26	0.45			1104.12	0.034	10.00	7.13	8.26	9.5	10.8	13	41.8	48.4	55.7	63.3	56.7	65.8	75.6	86.0	18	rcp	3.3	2.2	projecting	15.5	N
6.22		0.45			351.86	0.02	10.00	7.13	8.26	9.5	10.8	1.19	3.8	4.4	5.1	5.8	3.8	4.4	5.1	5.8	18	rcp	3.1	2.1	projecting	15.5	Y
6.14		0.45			795.57	0.025	10.00	7.13	8.26	9.5	10.8	4.26	13.7	15.8	18.2	20.7	13.7	15.8	18.2	20.7	18	rcp	3.1	2.1	projecting	15.5	N
6.13		0.45	77.01	0.026	469.79	0.004	16.96	5.52	6.43	7.39	8.39	1.15	2.9	3.3	3.8	4.3	2.9	3.3	3.8	4.3	18	rcp	3.2	2.1	projecting	15.5	Y
5.53	U-2	0.45			470.37	0.019	10.00	7.13	8.26	9.5	10.8	1.69	5.4	6.3	7.2	8.2	7.9	9.2	10.5	12.0	18	rcp	3.1	2.1	projecting	15.5	Y
5.86		0.45	551.79	0.042			21.72	4.81	5.61	6.46	7.33	4.91	10.6	12.4	14.3	16.2	10.6	12.4	14.3	16.2	18	rcp	3.3	2.2	projecting	15.5	Y
12.19		0.3	647.44	0.012	402.66	0.03	34.20	3.65	4.27	4.92	5.59	17.6	19.3	22.5	26.0	29.5	19.3	22.5	26.0	29.5	18	rcp	3.5	2.3	projecting	15.5	N
16.66		0.45	302.79	0.003	619.74	0.003	39.81	3.31	3.88	4.46	5.07	2.85	4.2	5.0	5.7	6.5	4.2	5.0	5.7	6.5	18	rcp	3.3	2.2	groove w/headwall	16	Y
6.145		0.45			1104.17	0.025	10.00	7.13	8.26	9.5	10.8	4.88	15.7	18.1	20.9	23.7	15.7	18.1	20.9	23.7	18	rcp	4.2	2.8	groove w/headwall	17.25	N
9.21		0.45	574.25	0.037	639.17	0.005	31.71	3.83	4.48	5.16	5.86	6.52	11.2	13.1	15.1	17.2	11.2	13.1	15.1	17.2	2-18	rcp	3.5	2.3	groove w/headwall	34.5	Y
19.35	20.7	0.45	389.13	0.04	216.64	0.088	19.76	5.08	5.92	6.81	7.73	5.62	12.8	15.0	17.2	19.5	39.2	45.5	52.3	59.4	18	cmp	3.2	2.1	projecting	12	N
10.57		0.45			617.28	0.008	7.01	7.13	8.26	9.5	10.8	2.5	8.0	9.3	10.7	12.2	8.0	9.3	10.7	12.2	18	cmp	3.1	2.1	projecting	12	Y
10.22		0.45			490.13	0.008	10.00	7.13	8.26	9.5	10.8	3.43	11.0	12.7	14.7	16.7	11.0	12.7	14.7	16.7	18	cmp	3	2	projecting	11	N
5.56	u-3	0.45			840.38	0.015	10.00	7.13	8.26	9.5	10.8	4.32	13.9	16.1	18.5	21.0	25.3	29.4	33.8	38.5	18	cmp	3.7	2.4	projecting	12	N
U-28		0.45			941.54	0.003	13.91	6.11	7.11	8.17	9.28	4.89	13.4	15.6	18.0	20.4	13.4	15.6	18.0	20.4	18	cmp	3.5	2.3	projecting	12	N
12.121		0.45	377.77	0.013	454.71	0.031	27.15	4.22	4.93	5.67	6.44	2.13	4.0	4.7	5.4	6.2	4.0	4.7	5.4	6.2	18	cmp	3.5	2.3	projecting	12	Y
12.140		0.45	200.23	0.02	961.41	0.017	23.56	4.59	5.36	6.16	7	8.84	18.3	21.3	24.5	27.8	18.3	21.3	24.5	27.8	2-18	cmp	3.2	2.1	projecting	12	N
16.6		0.45			328.16	0.046	2.22	7.13	8.26	9.5	10.8	1.77	5.7	6.6	7.6	8.6	5.7	6.6	7.6	8.6	18	cmp	3.4	2.3	headwall	13.5	Y
15.5		0.45			1454.01	0.014	11.07	6.81	7.9	9.09	10.33	5.92	18.1	21.0	24.2	27.5	18.1	21.0	24.2	27.5	18	cmp	4.2	2.8	headwall	17	N
14.67		0.45			603.74	0.033	10.00	7.13	8.26	9.5	10.8	3.24	10.4	12.0	13.9	15.7	10.4	12.0	13.9	15.7	18	cmp	3.2	2.1	headwall	13.5	Y
14.45		0.45			510.73	0.01	10.00	7.13	8.26	9.5	10.8	4.1	13.2	15.2	17.5	19.9	13.2	15.2	17.5	19.9	18	cmp	3.8	2.5	headwall	13.5	N
14.16		0.45			788.87	0.037	10.00	7.13	8.26	9.5	10.8	5.08	16.3	18.9	21.7	24.7	16.3	18.9	21.7	24.7	18	cmp	4	2.7	headwall	17	N
14.39		0.45	297.28	0.017	431.45	0.009	25.20	4.41	5.15	5.93	6.73	3.76	7.5	8.7	10.0	11.4	7.5	8.7	10.0	11.4	18	cmp	4.8	3.2	headwall	17	Y
6.64	6.56	0.45			905.88	0.02	10.00	7.13	8.26	9.5	10.8	6.72	21.6	25.0	28.7	32.7	32.7	37.9	43.6	49.5	18	cmp	3.4	2.3	headwall	13.5	N
9.60		0.45			589.92	0.036	10.00	7.13	8.26	9.5	10.8	2	6.4	7.4	8.6	9.7	6.4	7.4	8.6	9.7	18	cmp	3.5	2.3	headwall	13.5	Y
6.253		0.45			391.96	0.031	10.00	7.13	8.26	9.5	10.8	0.37	1.2	1.4	1.6	1.8	1.2	1.4	1.6	1.8	18	cmp	3.4	2.3	headwall	13.5	Y
19.2	19.6	0.45			523.57	0.008	6.32	7.13	8.26	9.5	10.8	4.19	13.4	15.6	17.9	20.4	16.3	16.3	18.9	21.8	18	Iron	3.5	2.3	projecting	15.5	N

TABLE V.1
MASTER PLAN CULVERT LIST
AND
DRAINAGE CALCULATIONS

													Flow from Area (cfs)				Cumulative Flow at Outlet (cfs)											
Structure ID	Upstream Structure(s)	C/CN	Lov (ft)	Yov (%)	Lch (ft)	Ych (ft/ft)	Tc (min)	I10 (in/hr)	I25 (in/hr)	I50 (in/hr)	I100 (in/hr)	Area (ac)	Q 10-yr	Q 25-yr	Q 50-yr	Q 100-yr	Q 10-yr	Q 25-yr	Q 50-yr	Q 100-yr	Existing Size (in)	Material	HW (ft)	HW/D	Entrance Type	Flow Capacity (cfs)	Sufficient Capacity (Y/N)	
16.44		0.45			580.19	0.019	4.82	7.13	8.26	9.5	10.8	2.4	7.7	8.9	10.3	11.7	7.7	8.9	10.3	11.7	21	rcp	3.8	2.2	projecting	21.25	Y	
16.42		0.45	477.89	0.027			22.45	4.72	5.51	6.34	7.2	3.84	8.2	9.5	11.0	12.4	8.2	9.5	11.0	12.4	21	rcp	3.8	2.2	projecting	21.25	Y	
16.26		0.45	160.86	0.056	825.6	0.01	19.59	5.1	5.95	6.84	7.77	4.03	9.2	10.8	12.4	14.1	9.2	10.8	12.4	14.1	21	rcp	4.1	2.3	projecting	21.25	Y	
16.59		0.45	556.62	0.018			26.58	4.27	4.99	5.74	6.52	6.92	13.3	15.5	17.9	20.3	13.3	15.5	17.9	20.3	21	rcp	6	3.4	square w/headwall	26	Y	
22.34		0.45	309.53	0.03	274.86	0.051	19.46	5.12	5.97	6.87	7.8	12	27.6	32.1	37.0	42.0	27.6	32.1	37.0	42.0	24	rcp	5	2.5	projecting	32.5	Y	
21.25		0.45			1380.25	0.014	10.42	7	8.12	9.34	10.61	16.5	52.0	60.3	69.3	78.8	52.0	60.3	69.3	78.8	24	rcp	6	3.0	projecting	36.5	N	
21.59	21.2	0.45	401.29	0.03	975.46	0.022	27.09	4.22	4.94	5.68	6.45	14.2	27.0	31.6	36.3	41.2	71.6	83.3	95.8	108.8	24	rcp	4.8	2.4	projecting	32.5	N	
17.50		0.45	874.24	0.024	878.34	0.019	37.24	3.46	4.05	4.66	5.29	25.5	39.6	46.4	53.4	60.6	39.6	46.4	53.4	60.6	24	rcp	5.3	2.7	projecting	36.5	N	
17.44	17.18,17.14	0.45	189.02	0.011	648.31	0.04	22.11	4.76	5.56	6.39	7.26	4.2	9.0	10.5	12.1	13.7	25.4	29.6	34.0	38.7	24	rcp	4.4	2.2	projecting	32.5	Y	
17.18		0.45			659.42	0.006	8.25	7.13	8.26	9.5	10.8	3.01	9.7	11.2	12.9	14.6	9.7	11.2	12.9	14.6	24	rcp	4.4	2.2	projecting	32.5	Y	
6.69	6.64,6.56	0.45			1264.33	0.016	10.00	7.13	8.26	9.5	10.8	3.01	9.7	11.2	12.9	14.6	42.4	49.1	56.4	64.2	24	rcp	4	2	projecting	29.5	N	
5.106	6.88,6.5	0.45	673.6	0.028			26.14	4.31	5.04	5.8	6.59	5.85	11.3	13.3	15.3	17.3	39.3	45.7	52.6	59.8	24	rcp	4.5	2.3	projecting	32.5	N	
16.80	16.81	0.45	619.74	0.006			35.55	3.56	4.17	4.8	5.45	3.87	6.2	7.3	8.4	9.5	7.8	9.1	10.5	11.9	24	rcp	3.4	1.7	square w/headwall	22	Y	
5.2		0.45			1065.8	0.026	10.00	7.13	8.26	9.5	10.8	12	38.5	44.6	51.3	58.3	38.5	44.6	51.3	58.3	2-24	rcp	5.5	2.8	square w/headwall	63.5	Y	
7.28		0.45			1635.26	0.014	12.01	6.56	7.62	8.76	9.95	14.9	44.0	51.1	58.7	66.7	44.0	51.1	58.7	66.7	24	rcp	4.6	2.3	square w/headwall	26	N	
4.10		0.45			2417.16	0.015	15.71	5.75	6.69	7.69	8.73	29.9	77.3	90.0	103.4	117.4	77.3	90.0	103.4	117.4	2-24	rcp	4	2	square w/headwall	52	N	
22.7		0.3	1456.83	0.03	1210.9	0.027	43.34	3.13	3.67	4.22	4.8	48.5	45.5	53.3	61.3	69.8	45.5	53.3	61.3	69.8	24	cmp	4.5	2.3	projecting	25	N	
7.8		0.3			1551.52	0.023	10.00	7.13	8.26	9.5	10.8	21.9	46.7	54.1	62.3	70.8	46.7	54.1	62.3	70.8	24	cmp	5	2.5	projecting	25	N	
U-29		0.45	853.68	0.02			31.68	3.83	4.48	5.16	5.86	8.53	14.7	17.2	19.8	22.5	14.7	17.2	19.8	22.5	24	cmp	4	2	projecting	23	Y	
12.16		0.45	643.05	0.016	1189.03	0.003	47.62	2.94	3.44	3.97	4.51	12.8	16.9	19.8	22.9	26.0	16.9	19.8	22.9	26.0	2-24	cmp	4.3	2.2	projecting	50	Y	
11.13		0.3	731.5	0.023	2128.23	0.028	39.75	3.31	3.88	4.47	5.08	69.7	69.2	81.1	93.5	106.2	69.2	81.1	93.5	106.2	24	cmp	4	2	projecting	23	N	
15.102		0.45			1294.16	0.026	10.00	7.13	8.26	9.5	10.8	11	35.2	40.8	46.9	53.3	35.2	40.8	46.9	53.3	30	cmp	6.6	3.3	headwall	65	Y	
14.59		0.45			1062.79	0.025	10.00	7.13	8.26	9.5	10.8	9.07	29.1	33.7	38.8	44.1	29.1	33.7	38.8	44.1	24	cmp	5.2	2.6	headwall	32.5	N	
14.27		0.45	293.93	0.01	436.97	0.011	27.24	4.21	4.92	5.66	6.43	5.96	11.3	13.2	15.2	17.2	11.3	13.2	15.2	17.2	24	cmp	4.8	2.4	headwall	32.5	Y	
9.36	9.60	0.45			1255.58	0.014	10.00	7.13	8.26	9.5	10.8	5.96	19.1	22.2	25.5	29.0	25.5	29.6	34.0	38.7	24	cmp	5	2.5	headwall	32.5	Y	
6.151	6.253,6.146	0.45			458.42	0.028	10.00	7.13	8.26	9.5	10.8	0.49	1.6	1.8	2.1	2.4	3.9	4.5	5.2	5.9	24	cmp	5	2.5	headwall	32.5	Y	
22.1		75.8	18201.65	1.02			364.37	0.65	0.77	0.89	1.02	1549	433.2	580.0	734.3	907.6	433.2	580.0	734.3	907.6	3-24	Iron	5.3	2.7	projecting	109.5	N	
U-16	21.25	0.3	538.64	0.01	2460.9	0.015	46.61	2.98	3.49	4.02	4.57	81.3	72.7	85.1	98.0	111.5	135.5	158.0	181.9	206.7	24	Iron	4	2.0	projecting	29.5	N	
12.55		0.45			1352.75	0.004	16.18	5.66	6.59	7.57	8.6	10.7	27.2	31.7	36.4	41.3	27.2	31.7	36.4	41.3	2-27	cmp	5.1	2.3	projecting	68	Y	
20.56	20.60	0.45			2106.08	0.02	12.64	6.41	7.44	8.56	9.72	25.7	74.2	86.1	99.1	112.5	91.9	106.7	122.7	139.4	30	rcp	6.3	2.5	projecting	61.5	N	
5.88	5.86,5.81	0.45	603.1	0.027	163.16	0.006	27.98	4.14	4.84	5.57	6.32	10.9	20.3	23.7	27.3	31.0	44.9	52.4	60.3	68.4	30	rcp	6.1	2.4	projecting	61.5	Y	
9.13	9.21	0.45			1467.46	0.012	11.91	6.59	7.65	8.8	9.99	13.2	39.1	45.4	52.2	59.3	50.4	58.6	67.4	76.5	2-30	rcp	6.1	2.4	square w/headwall	100	Y	

TABLE V.1
MASTER PLAN CULVERT LIST
AND
DRAINAGE CALCULATIONS

													Flow from Area (cfs)				Cumulative Flow at Outlet (cfs)										
Structure ID	Upstream Structure(s)	C/CN	Lov (ft)	Yov (%)	Lch (ft)	Ych (ft/ft)	Tc (min)	I10 (in/hr)	I25 (in/hr)	I50 (in/hr)	I100 (in/hr)	Area (ac)	Q 10-yr	Q 25-yr	Q 50-yr	Q 100-yr	Q 10-yr	Q 25-yr	Q 50-yr	Q 100-yr	Existing Size (in)	Material	HW (ft)	HW/D	Entrance Type	Flow Capacity (cfs)	Sufficient Capacity (Y/N)
7.13	7.28	0.45			1349.76	0.033	10.00	7.13	8.26	9.5	10.8	11	35.3	40.9	47.1	53.5	79.3	92.0	105.8	120.2	2-30	rcp	5.6	2.2	square w/headwall	98	Y
15.70		0.45			1273.45	0.027	10.00	7.13	8.26	9.5	10.8	9.55	30.6	35.5	40.8	46.4	30.6	35.5	40.8	46.4	30	cmp	6	2.4	headwall	45	Y
14.93	14.45	0.45			1017.19	0.021	10.00	7.13	8.26	9.5	10.8	12.9	41.2	47.8	54.9	62.5	54.4	63.0	72.5	82.4	30	cmp	5.1	2	headwall	45	N
9.33		0.45			1609.31	0.024	10.00	7.13	8.26	9.5	10.8	11.3	36.1	41.9	48.1	54.7	36.1	41.9	48.1	54.7	2-30	cmp	9.8	3.9	headwall	140	Y
9.6		0.45	690.47	0.032	527.19	0.034	29.26	4.03	4.71	5.42	6.16	35.4	64.2	75.1	86.4	98.2	64.2	75.1	86.4	98.2	30	cmp	8.6	3.4	headwall	70	N
19.29	19.28	0.45	503.85	0.062	539.51	0.035	22.59	4.7	5.49	6.32	7.17	9.5	20.1	23.5	27.0	30.7	35.4	41.2	47.4	53.8	36	rcp	6.7	2.2	projecting	85	Y
17.45	17.50	0.45			1715.34	0.021	10.68	6.93	8.03	9.24	10.49	22.3	69.4	80.4	92.6	105.1	109.0	126.8	145.9	165.7	36	rcp	7.5	2.5	projecting	85	N
17.11		0.45	752.23	0.016	301.36	0.056	33.37	3.71	4.34	4.99	5.67	11.9	19.8	23.2	26.7	30.3	19.8	23.2	26.7	30.3	36	rcp	9.5	3.2	projecting	99	Y
17.2		0.45	502.15	0.022	660.43	0.047	27.94	4.14	4.84	5.57	6.33	16.4	30.6	35.7	41.1	46.7	30.6	35.7	41.1	46.7	36	rcp	7.3	2.4	projecting	85	Y
U-9	10.22,10.16	0.45			1478.43	0.041	10.00	7.13	8.26	9.5	10.8	18	57.6	66.7	76.7	87.2	75.0	86.9	99.9	113.6	36	rcp	5	1.7	projecting	72.5	N
15.116		0.45			900.18	0.027	10.00	7.13	8.26	9.5	10.8	7.11	22.8	26.4	30.4	34.6	22.8	26.4	30.4	34.6	36	rcp	9.3	3.1	projecting	99	Y
15.112	15.102,15.110	0.45			308.63	0.029	10.00	7.13	8.26	9.5	10.8	1.75	5.6	6.5	7.5	8.5	85.2	98.8	113.7	129.1	36	rcp	9	3	projecting	99	Y
6.247		0.45	997.93	0.049	1786.01	0.008	43.66	3.11	3.65	4.2	4.77	16.9	23.6	27.7	31.8	36.2	23.6	27.7	31.8	36.2	36	rcp	7	2.3	groove w/headwall	79	Y
6.194	6.247	0.45			2407.88	0.01	18.47	5.27	6.14	7.06	8.02	23.5	55.8	65.0	74.7	84.8	79.3	92.6	106.5	121.0	2-36	rcp	7.7	2.6	groove w/headwall	158	Y
6.136		0.45			2223.67	0.011	16.59	5.58	6.5	7.48	8.49	17.3	43.5	50.7	58.3	66.2	43.5	50.7	58.3	66.2	36	rcp	6.9	2.3	groove w/headwall	79	Y
6.139	6.136,6.194,6.247	0.45			1051.95	0.015	10.00	7.13	8.26	9.5	10.8	4.36	14.0	16.2	18.6	21.2	136.8	159.5	183.5	208.4	3-36	rcp	6.6	2.2	groove w/headwall	237	Y
6.75	6.145,6.139,6.136,6.194,6.247	0.45			2603.12	0.015	16.60	5.58	6.5	7.47	8.49	22.1	55.4	64.6	74.2	84.4	207.9	242.2	278.6	316.5	4-36	rcp	7.3	2.4	groove w/headwall	316	Y
6.76	6.69,6.100,6.64,6.56,6.75,6.145,6.139,6.136,6.194,6.247	0.45			1458	0.023	10.00	7.13	8.26	9.5	10.8	13.3	42.7	49.5	56.9	64.7	312.1	363.0	417.4	474.3	5-36	rcp	7.5	2.5	groove w/headwall	395	Y
12.14	12.55,12.16	0.45			1351.18	0.015	10.17	7.08	8.2	9.43	10.72	18.5	58.9	68.2	78.5	89.2	103.1	119.7	137.7	156.5	3'x5'	rcb	7.4	2.5	90d	170	Y
12.15	12.14,12.55	0.45			574.58	0.002	10.89	6.86	7.96	9.16	10.4	5.65	17.4	20.2	23.3	26.4	120.5	140.0	161.0	183.0	3'x6'	rcb	8.2	2.7	90d	219	Y
17.14	17.18	0.45	156.09	0.026	775.69	0.006	22.64	4.7	5.48	6.31	7.16	3.2	6.8	7.9	9.1	10.3	16.4	19.1	22.0	24.9	36	cmp	6.2	2.1	projecting	64	Y
13.18	10.37,10.40,10.57,16.21	0.45			608.32	0.039	10.00	7.13	8.26	9.5	10.8	1.55	5.0	5.8	6.6	7.5	65.0	75.3	86.7	98.5	36	cmp	6.2	2.1	projecting	64	N
5.6	5.2,5.3	0.45			578.21	0.016	10.00	7.13	8.26	9.5	10.8	4.53	14.5	16.8	19.4	22.0	77.3	89.6	103.0	117.1	40	cmp	9.3	2.8	headwall	110	Y
6.35		0.3	497.3	0.03	2069.31	0.038	32.13	3.8	4.45	5.11	5.81	45.1	51.4	60.2	69.1	78.6	51.4	60.2	69.1	78.6	42	rcp	9.5	2.7	projecting	140	Y

TABLE V.1
MASTER PLAN CULVERT LIST
AND
DRAINAGE CALCULATIONS

													Flow from Area (cfs)				Cumulative Flow at Outlet (cfs)										
Structure ID	Upstream Structure(s)	C/CN	Lov (ft)	Yov (%)	Lch (ft)	Ych (ft/ft)	Tc (min)	I10 (in/hr)	I25 (in/hr)	I50 (in/hr)	I100 (in/hr)	Area (ac)	Q 10-yr	Q 25-yr	Q 50-yr	Q 100-yr	Q 10-yr	Q 25-yr	Q 50-yr	Q 100-yr	Existing Size (in)	Material	HW (ft)	HW/D	Entrance Type	Flow Capacity (cfs)	Sufficient Capacity (Y/N)
6.40		0.45			898.39	0.029	10.00	7.13	8.26	9.5	10.8	5.04	16.2	18.7	21.5	24.5	16.2	18.7	21.5	24.5	42	rcp	9.3	2.7	projecting	140	Y
16.64	16.59	0.45			1035.38	0.019	7.48	7.13	8.26	9.5	10.8	10	32.1	37.2	42.8	48.6	45.4	52.7	60.7	69.0	48	rcp	17	4.3	projecting	275	Y
13.17	10.41,10.49,10.36,13.18,10.37,10.40,10.57,16.21	0.45			1132.84	0.041	10.00	7.13	8.26	9.5	10.8	7.12	22.8	26.5	30.4	34.6	134.9	156.7	180.1	204.8	48	rcp	10	2.6	projecting	200	Y
6.42	6.40,6.35,6.108	77.8	6601.57	2.075			106.79	1.65	1.95	2.25	2.56	227	138.2	188.3	241.6	299.3	219.9	283.7	351.2	423.8	2-48	rcp	9	2.3	projecting	355	N
6.21	6.13,6.14,6.42,6.22,6.40,6.35,6.108	0.45			1214.35	0.019	10.00	7.13	8.26	9.5	10.8	10.4	33.2	38.5	44.2	50.3	273.5	345.8	422.6	505.0	2-48	rcp	8.8	2.2	projecting	355	N
5.52	6.13,6.21,5.53,5.88,6.22,6.42,6.40,6.35,5.86,5.81,u-2	0.3			1007.3	0.016	10.00	7.13	8.26	9.5	10.8	8.1	17.3	20.1	23.1	26.2	315.7	395.1	479.4	569.4	2-48	rcp	12	3	projecting	400	N
12.77	12.19,12.29	0.45	199.06	0.01	968.96	0.026	25.21	4.41	5.15	5.93	6.73	18.7	37.2	43.4	50.0	56.8	75.7	88.5	102.0	115.8	48	rcp	12	3	projecting	200	Y
U-19		0.3	366.68	0.016	1141.69	0.028	29.34	4.02	4.7	5.41	6.14	50.2	60.5	70.7	81.4	92.4	60.5	70.7	81.4	92.4	2-48	rcp	6	1.5	square w/headwall	220	Y
9.82	Res. 23,9.84,10.2																				2-48	rcp		3	square w/headwall	390	Y
9.83	2,10.16,9.33,	0.3			3053.79	0.029	14.61	5.96	6.94	7.98	9.06	59.8	106.9	124.4	143.1	162.4	288.5	339.7	373.1	418.4	2-72		12	2	w/headwall	800	
12.29	12.19	0.3	588.98	0.003	837.81	0.021	46.47	2.98	3.5	4.03	4.58	21.5	19.2	22.6	26.0	29.5	38.5	45.1	51.9	59.0	48	cmp	9.4	2.4	projecting	141	Y
15.122	15.70	0.3	628.81	0.067	5246.83	0.026	43.83	3.1	3.64	4.19	4.76	145	134.9	158.4	182.3	207.1	165.5	193.9	223.2	253.6	54	rcp	6.5	1.4	projecting	160	N
U-4	9.82-83,u-9,9.33,9.36	0.3			1907.81	0.051	10.00	7.13	8.26	9.5	10.8	30.2	64.5	74.8	86.0	97.8	489.7	572.8	641.2	723.2	54	cmp		2	headwall	200	N
																					84		9	1.5		460	
5.34	5.53,5.52,5.88,u-2,a-1,u-20,6.13,6.14,6.21,6.22,5.86,5.81,6.35,6.40,6.42,6.108	0.3			1170.01	0.027	10.00	7.13	8.26	9.5	10.8	15.7	33.5	38.8	44.6	50.7	445.6	546.2	653.3	767.1	2-55	rcp	11	2.3	square w/headwall	436	N
22.4		77	7277.35	1.79			127.44	1.45	1.71	1.97	2.25	258	138.0	187.3	239.7	298.7	138.0	187.3	239.7	298.7	3-60	cmp	11	2.3	mitered	750	Y
U-24	12.77,12.29,12.19	0.45			1147.96	0.035	6.45	7.13	8.26	9.5	10.8	11.5	36.7	42.6	48.9	55.6	93.1	108.6	124.9	141.9	60	smooth metal	7	1.4	square w/headwall	185	Y

Structure ID	Upstream Structure(s)	C/CN	Lov (ft)	Yov (%)	Lch (ft)	Ych (ft/ft)	Tc (min)	I10 (in/hr)	I25 (in/hr)	I50 (in/hr)	I100 (in/hr)	Area (ac)	Flow from Area (cfs)				Cumulative Flow at Outlet (cfs)				Existing Size (in)	Material	HW (ft)	HW/D	Entrance Type	Flow Capacity (cfs)	Sufficient Capacity (Y/N)
													Q 10-yr	Q 25-yr	Q 50-yr	Q 100-yr	Q 10-yr	Q 25-yr	Q 50-yr	Q 100-yr							
19.33	20.15,20.12,20.7,19.35,U-22,20.45,20.52,20.56,U-13,19.16,19.12	0.3	398.6	0.015	2188.41	0.027	35.41	3.57	4.18	4.81	5.47	63	67.5	79.0	91.0	103.4	281.4	327.8	377.1	428.5	66	rcp	15	2.7	projecting	405	N
4.13	4.10	0.45			1489.01	0.013	11.37	6.73	7.81	8.98	10.21	26.3	79.6	92.4	106.2	120.8	156.9	182.3	209.6	238.2	8'x6'	rcb	14	2.4	30d to 75d	800	Y
U-1	5.6,B-1,5.64,5.106,6.76,6.151	0.3			4184.34	0.012	26.16	4.31	5.04	5.8	6.58	92.6	119.7	140.0	161.1	182.7	1137.9	1352.0	1580.2	1820.0	18'x9.5'	rcb	12	1.2	30d to 75d	1908	Y
B-3	22.4,22.1,22.7,u-21,u-16,20.65,17.47,17.50,u-22,u-23, U-17	75.6	11239.84	1.851			184.70	1.1	1.3	1.5	1.71	663	273.1	371.3	475.6	590.0	1080.8	1414.5	1767.0	2157.0	90'x20'	bridge				83768.7	Y
B-1	U-19,A-1,A-3,5.34	0.3			1368.39	0.023	10.00	7.13	8.26	9.5	10.8	11.3	24.1	28.0	32.2	36.6	585.6	709.3	840.9	980.2	60'x2.5'	bridge				1941.4	Y

**TABLE V.2
CULVERTS IN NEED OF REPLACEMENT**

Structure ID	Cumulative Flow at Outlet (cfs)				Existing Size (in)	Material	Flow Capacity (cfs)	Sufficient Capacity (Y/N)	% Over Capacity	
	Q 10-yr	Q 25-yr	Q 50-yr	Q 100-yr						
6.69	42.4	49.1	56.4	64.2	24	rcp	29.5	N	66.4%	Sam Bass & Stagecoach
6.100	19.1	22.2	25.5	29.0	12	rcp	7.25	N	205.6%	
6.145	15.7	18.1	20.9	23.7	18	rcp	17.25	N	5.2%	
5.3	24.3	28.1	32.4	36.8	12	cmp	6	N	369.0%	Ranch House & Canyon Court
20.56	91.9	106.7	122.7	139.4	30	rcp	61.5	N	73.5%	Crown Rd & Live Oak
7.8	46.7	54.1	62.3	70.8	24	cmp	25	N	116.6%	Chuckwagon & Pitchfork
20.44	19.2	22.3	25.6	29.1	18	rcp	15.5	N	43.6%	Live Oak & Cook Raod
U-9	75.0	86.9	99.9	113.6	36	rcp	72.5	N	19.9%	Squaw Creek Road
13.18	65.0	75.3	86.7	98.5	36	cmp	64	N	17.7%	
13.21	56.7	65.8	75.6	86.0	18	rcp	15.5	N	324.5%	
6.64	32.7	37.9	43.6	49.5	18	cmp	13.5	N	180.7%	
17.39	11.1	13.0	14.9	17.0	15	cmp	9.25	N	40.5%	
4.10	77.3	90.0	103.4	117.4	2-24	rcp	52	N	73.0%	
5.106	39.3	45.7	52.6	59.8	24	rcp	32.5	N	40.6%	
5.34	445.6	546.2	653.3	767.1	2-55	rcp	436	N	25.3%	
5.56	25.3	29.4	33.8	38.5	18	cmp	12	N	145.0%	
5.64	33.3	38.7	44.5	50.6	12	cmp	4.75	N	714.7%	
5.81	13.9	16.3	18.7	21.3	12	rcp	5.5	N	195.9%	
6.108	14.2	16.4	18.9	21.5	15	rcp	11	N	49.4%	
6.5	7.9	9.2	10.5	12.0	12	rcp	6	N	52.6%	
6.56	11.1	12.9	14.8	16.9	12	rcp	7.25	N	77.9%	
6.88	28.0	32.5	37.3	42.4	12	rcp	7.25	N	348.3%	
7.28	44.0	51.1	58.7	66.7	24	rcp	26	N	96.4%	
10.22	11.0	12.7	14.7	16.7	18	cmp	11	N	15.9%	
10.36	47.0	54.8	63.0	71.7	15	cmp	7.8	N	602.6%	
10.37	60.1	69.6	80.0	91.0	15	iron	11.5	N	505.2%	
10.40	20.1	23.2	26.7	30.4	15	cmp	7.8	N	197.4%	
10.41	14.1	16.3	18.7	21.3	15	iron	10.25	N	58.8%	
10.49	14.2	16.5	19.0	21.6	15	iron	10.25	N	61.3%	
12.114	10.3	12.0	13.8	15.6	15	cmp	7.8	N	53.6%	
12.140	18.3	21.3	24.5	27.8	2-18	cmp	12	N	77.7%	
12.19	19.3	22.5	26.0	29.5	18	rcp	15.5	N	45.4%	
14.16	16.3	18.9	21.7	24.7	18	cmp	17	N	11.1%	
14.45	13.2	15.2	17.5	19.9	18	cmp	13.5	N	12.9%	
14.59	29.1	33.7	38.8	44.1	24	cmp	32.5	N	3.7%	
14.93	54.4	63.0	72.5	82.4	30	cmp	45	N	40.0%	

Rows highlighted indicate creek culverts that are insufficiently sized to convey the 100-yr flow.

TABLE V.2
CULVERTS IN NEED OF REPLACEMENT

	Cumulative Flow at Outlet (cfs)								
15.122	165.5	193.9	223.2	253.6	54	rcp	160	N	21.2%
15.5	18.1	21.0	24.2	27.5	18	cmp	17	N	23.8%
16.21	16.6	19.2	22.1	25.1	15	cmp	9.25	N	107.8%
17.45	87.7	101.8	117.1	132.9	36	rcp	85	N	19.7%
17.50	39.6	46.4	53.4	60.6	24	rcp	36.5	N	27.1%
18.10	31.9	37.3	42.9	48.8	2-18	rcp	31	N	20.4%
19.12	12.3	14.4	16.6	18.8	12	rcp	7.25	N	98.8%
19.19	7.1	8.2	9.4	10.7	15	rcp	6	N	36.3%
19.35	29.4	34.2	39.3	44.7	18	cmp	12	N	184.9%
20.29	15.2	17.7	20.3	23.1	18	rcp	15.5	N	13.9%
20.60	17.7	20.6	23.7	26.9	18	rcp	18	N	14.5%
20.65	9.9	11.5	13.3	15.1	15	cmp	7.8	N	47.9%
20.7	26.3	30.5	35.1	39.9	18	rcp	21.5	N	41.8%
21.12	7.5	8.7	10.0	11.4	15	rcp	6	N	45.4%
21.2	53.4	62.1	71.4	81.1	12	cmp	6	N	934.5%
21.25	52.0	60.3	69.3	78.8	24	rcp	36.5	N	65.2%
21.30	16.3	19.1	22.0	25.0	18	rcp	15.5	N	23.2%
21.59	53.2	62.1	71.4	81.1	24	rcp	32.5	N	91.0%
22.1	433.2	580.0	734.3	907.6	3-24	Iron	109.5	N	429.7%
22.7	45.5	53.3	61.3	69.8	24	cmp	25	N	113.4%
B-2	173.7	240.46	310.41	389.572	2-36"		126	N	90.8%
U-16	135.5	158.0	181.9	206.7	24	Iron	29.5	N	435.6%
U-28	13.4	15.6	18.0	20.4	18	cmp	12	N	30.4%

Rows highlighted indicate creek culverts that are insufficiently sized to convey the 100-yr flow.

6. Storm Drainage Problems

For each of the areas identified as experiencing localized flooding, the alternative actions that could be taken are as follows.

- For the channel issues between Sam Bass Road and Stagecoach Trail, the ditches should be regraded and lined with concrete so that flat grades that cannot be overcome, may better convey the runoff.
- For the channel between El Chico and Surrey Lane, the natural channel should be graded so as to be better defined, stabilized with a lasting liner material, and a concrete pilot channel be installed to prevent low frequency storm event erosion.
- The culvert at Crown Road and Live Oak Road should be replaced and adequate headwall and endwall installed with better hydraulic capacity.
- The roadside ditches along Crown Road between Clear Elm Road and Ranch House Road, Chuckwagon Trail, Deer Pond and Big Creek Road, Vista Drive, Northchase Drive, and the Fox Hunt Trail Development should be lined with concrete to improve the hydraulic efficiency of the conveyances. Driveway and intersection culverts should be addressed as necessary.
- The culvert at Ranch House Road and Canyon Court should be replaced.
- The culvert on Live Oak near Cook Road should be upsized and the adjacent ditch regraded.
- The area east of the Clear Fork of the Trinity River along the Gates at Meadow Place apartments and the Reserves at Trinity subdivision should be regraded and the headwalls upgraded.
- The culverts crossing Squaw Creek Road should be replaced and upsized as necessary to improve stormwater conveyance.

C. Priority and Cost Estimates

Stormwater drainage problems can lead to damage to public infrastructure, significant private property damage, and loss of life. These drainage issues can compound and significantly worsen over time due to erosion if left unaddressed.

The City of Willow Park assesses a stormwater rate that is intended to address maintenance needs for the stormwater system on a yearly basis. The City should ensure that these funds are allocated for drainage system improvement as intended. The City of Willow Park also assesses stormwater impact fees on new development that can be used

to finance capital projects that are larger in nature. These fees however are not likely to be sufficient to cover all of the capital projects described in this report, but rather the debt service associated with financing the projects. The City will therefore likely need to fund larger projects through general obligation bonds or tax notes. The city should to allocate funding to do the following:

- Utilize a maintenance program for culverts, ditches, inlets, or any other minor storm drainage issues that can be addressed. This can also include various culvert upsizing and replacement for those that are not experiencing localized flooding, but are undersized based on the amount of flow coming to that culvert. Additionally, regrading ditches and clearing debris from culverts is a relatively inexpensive and easy way to optimize the efficiency of existing storm drain facilities.
- Begin capital improvements projects by 2028.

The table below details the 10 year capital improvements that are recommended. Included in the table is a preliminary estimated cost for each project. An average inflation rate of 3% should be considered per year past the date of this report.

**TABLE V.3
STORM DRAINAGE
PRIORITY AND PROJECTED COST**

Item	Project	Location	Cost
1	Sam Bass to Stagecoach Channel Improvements	Sam Bass to Stagecoach	\$ 150,000.00
2	El Chico to Surrey Lane Channel Improvements	El Chico to Surrey	\$ 130,000.00
3	Replace Culvert at Crown Road & Live Oak Road	Crown & Live Oak	\$ 75,000.00
4	Crown Road Ditch Upgrades	Cedar Elm to Ranch House	\$ 940,000.00
5	Chuckwagon Trail Ditch Upgrades	Stagecoach to IH20	\$ 1,500,000.00
6	Deer Pond Ditch Upgrades	Deer Pond & Big Creek	\$ 3,250,000.00
7	Vista Drive Ditch Upgrades	Live Oak to Nichols	\$ 250,000.00
8	Replace Culvert on Live Oak near Cook Road	Live Oak & Cook	\$ 50,000.00
9	Northchase Drive Ditch Upgrades	Northchase Drive	\$ 562,500.00
10	Regrade Area East of the Clear Fork & Upgrade Headwalls	East Bank of Clear Fork	\$ 350,000.00
11	Upgrade Foxhunt Trails Development Ditches & Culverts	Fox Hunt Trail Subdivision	\$ 3,000,000.00
12	Replace Culvert at Ranch House and Canyon Court	Ranch House & Canyon Ct	\$ 60,000.00
13	Replace Squaw Creek Major Crossings	Squaw Creek Road	\$ 450,000.00
14	Impact Fee and Capital Improvements Studies		\$ 75,000.00
			\$ 10,842,500.00

Culverts indicated as needing replacement in Table V.2 should be included where possible in bond projects, however, it is recommended that an annual program be implemented to regrade ditches and replace culverts. A yearly operating allowance within the drainage utility department budget should be considered for this purpose and should be approximately \$100,000 annually for a city the size of Willow P

III. CONCLUSIONS AND RECOMMENDATIONS

Based on our evaluation of the City of Willow Park's infrastructure, we offer the following conclusions and recommendations to assist the City with decisions related to capital improvements through the year 2050

A. Water System

The City has recently completed a project that should insure adequate water supply through 2050. However, water conveyance and distribution should remain a priority for the immediate future. The following is recommended toward that end:

- Replace old deteriorated water lines and other facilities
- Provide storage in key locations
- Complete a surface water supply transmission line to convey surface water to the Foxhunt site.
- Complete system loops in critical flow areas.

These projects would resolve issues with tank refill and allow backup options in times of shortage.

The reduction of water loss and improving pressure maintenance and providing key loops within the system while serving expected growth should be the next priority:

- Install East and West Bankhead extension loops
- Rehabilitate the Indian Camp ground storage tanks
- Replace Squaw Creek water lines
- Install chloramine disinfection facilities
- Complete replacement of old, poorly installed and deteriorated lines by 2035.

These projects would protect the water supply and insure that expected growth could be accommodated.

Long term supply and demand service should be considered early in the planning period to eliminate shortages in the future:

- Replace an existing elevated tanks
- Upsize existing small distribution lines.
- Install fire hydrants in various locations
- Install Fox Hunt Transmission Line
- Complete loops in key portions of the system

These improvements would allow complete distribution of each water source throughout the service area.

B. Wastewater System

The upgrade of the City's wastewater treatment system should be the highest priority along with necessary transmission facilities. The City should continue with the projects that are underway to address this concern as well as:

- Upgrade WWTP to 1.0 MGD
- Complete the South of IH20 Transmission Line and Lift Station

In order to provide capacity for future growth:

- Begin collection system inspection and testing program by 2028
- Upsize WWTP to 1.5 MGD
- Install irrigation facilities
- Complete lift station upgrades
- Complete West Bankhead extension

C. Streets

A focus on the upgrade of the City's streets should be maintained to improve the PCI of the system as a whole. The City should continue with the projects that are underway to address this concern:

- Complete priority 1-22 street improvements by 2030
- Begin a yearly seal coat program by 2027

In order to ensure a safe and adequate street system other lower grade streets should be upgraded:

- Begin priority 22 and beyond street improvements by 2030
- Continue yearly maintenance program

D. Drainage

Inadequate drainage can result in significant property damage, damage to infrastructure and even loss of life. Additionally, drainage issues tend to worsen over time as erosion occurs. The City should allocate funds to address significant drainage issues:

- Begin and continue a bond program for scheduled capital projects to include priorities 1-20
- Continue maintenance program through the planning period.

IV. FINANCING OF IMPROVEMENTS

The water and wastewater improvement projects can be funded by low interest loans and possibly loan forgiveness from the Texas Water Development Board, USDA Rural Utility Services, special federal allocations, and local bond issues. The larger, future recommended improvements should be funded by the Texas Water Development Board as the terms of their funding is very favorable. Smaller projects should be funded by a combination of yearly budgeting and bond issues. Community Development Block Grants should be considered as well for areas that are eligible.

Additionally, capital improvement fees should be evaluated periodically. These fees can be used to fund much of the improvements needed in the system. As development increases in the City, impact fees will also serve to offset potential rate increases. Regardless of the financial vehicles used to fund the recommended projects, it is likely that water and sewer rates will have to increase as well.

Street projects are not generally ranked well enough to be funded through state and federal agencies. The City should consider allocating in the yearly budget for as much as possible and fund larger projects through general obligation bonds, certificates of obligation, or tax notes.

The drainage projects are expected to be funded through the newly instituted Stormwater Utility Rate and impact fees. The larger projects are intended to be funded by a bond issue with these new fees covering the debt service. The Stormwater Utility and impact fees should cover yearly maintenance and smaller projects as well.

V. CLOSURE

The City should continue to evaluate this plan on a five year basis. As development occurs, the need and timing for various improvements will be more clearly defined. The City should continue to encourage development in accordance with the most recent comprehensive plan and this document.