



UPPER HAMESTRING CREEK BASIN DRAINAGE IMPROVEMENT ANALYSIS FAYETTEVILLE, AR



SEPTEMBER 3, 2021

Upper Hamestring Creek Basin
Drainage Improvement Analysis
Fayetteville, AR

Prepared for

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TABLE OF CONTENTS

1.0	INTRODUCTION	1-1
1.1	Background	1-1
2.0	HYDROLOGY	2-1
2.1	Drainage Basin Area Delineation	2-1
2.2	Modeling Methods	2-3
2.2.1	Loss Method.....	2-3
2.2.2	Transform Method	2-7
2.2.3	Routing Method	2-7
2.3	Design Storms.....	2-8
2.4	Model Calibration.....	2-9
3.0	HYDRAULIC MODELING.....	3-1
3.1	Introduction.....	3-1
3.2	1D Hydraulic Modeling	3-1
3.2.1	Geometry.....	3-1
3.2.2	Hydraulic Parameters.....	3-2
3.2.3	Structures	3-3
3.2.4	Boundary Conditions	3-4
3.2.5	Floodway.....	3-4
3.3	1D-2D Hydraulic Modeling.....	3-4
3.4	Proposed Conditions Modeling.....	3-5
4.0	RESULTS and CONCLUSIONS	4-1
4.1	Conceptual Improvement Scenarios	4-1

TABLE OF CONTENTS (CONTINUED)

LIST OF APPENDICES

APPENDIX A:	Figures 1-8 Hamestring Creek – Valley Channel Improvements
APPENDIX B:	Figures 9-12 Hamestring Creek – Storage Improvements
APPENDIX C:	Figures 13-18 Hamestring Creek Tributary HS3 – Lewis Storage Improvements
APPENDIX D:	Figures 19-24 Hamestring Creek Tributary HS3 – Hatfield Street Improvements
APPENDIX E:	Figures 25-28 South Fork Hamestring Creek Channel Improvements

LIST OF TABLES

Table 2.1	Curve Numbers	2-4
Table 2.2	Precipitation Data.....	2-8
Table 3.1	Typical Roughness Coefficients	3-2
Table 4.1	Valley Drive Channel Improvement Results	4-2
Table 4.2	Hamestring Creek Storage Results	4-3
Table 4.3	Lewis Avenue Storage Results	4-4
Table 4.4	Hatfield Street Channel Improvement Results	4-5
Table 4.5	SF Hamestring Creek Channel Improvement Results	4-6

LIST OF FIGURES

Figure 1.1	Project area map.....	1-3
Figure 2.1	Hydrologic Work Map.....	2-2
Figure 2.2	Hydrologic Soil Group Map	2-5
Figure 2.3	Land Use Map.....	2-6

1.0 INTRODUCTION

Throughout older sections of the City of Fayetteville (the City), existing roads, property and structures are subject to flooding due to events ranging in size and intensity. As part of the effort to improve local drainage in identified areas of concern, the City contracted with FTN Associates, Ltd. (FTN) to evaluate the Upper Hamestring Creek Basin by performing a drainage analysis to determine the current drainage system's conveyance capabilities and to develop conceptual level drainage scenarios for consideration to reduce local flooding issues and damage (the Project).

1.1 Background

The Project area contains all or portions of Hamestring Creek, Hamestring Creek Tributary HS3 and South Fork Hamestring Creek and is approximately 2.3 mi² in area. The Project area is generally bound by Deane Street to the north, State Highway 112 (Garland Avenue) to the east, Markham Road to the south and Betty Jo Drive to the west. Figure 1.1. Map Overview documents the location of the Project area against aerial imagery. The Project area consists primarily of established residential neighborhoods with some open space and hilly forested areas in the southern parts of the Project area.

Stormwater from the Project area ultimately drains to Hamestring Creek. The Project area contains locations that have historically experienced poor drainage, causing flooding that covers city streets and has impacted some single and multi-family residential properties.

Based on discussions with City of Fayetteville personnel, the overall goals of the Project were to prepare an up-to-date hydrologic and hydraulic drainage analysis of the Project area, which did not look at differences between surface and subsurface drainage, to aid in confirming reported areas of stormwater drainage deficiencies, identify the potential extent and severity of these deficiencies during various storm events, and to develop multiple conceptual level scenarios to attempt to improve the discharge conveyance and to reduce overall flooding for the areas of concern within the Project area.

As part of this analysis, FTN gathered available information from the City regarding storm sewer network information; obtained and processed available LiDAR topographic data;

performed field survey and reconnaissance activities to supplement data, as needed; and developed hydrologic and hydraulic models that simulated rainfall runoff over the surface (open channels, streets) to determine the efficiency of the existing drainage infrastructure.

2.0 HYDROLOGY

For the project area, a rainfall-runoff model was developed using HEC-HMS Version 4.8.0 to perform the hydrologic analyses. The SCS Curve Number, the SCS Unit Hydrograph, and the Modified Puls methods were used to determine the loss-rate, transform excess rainfall into surface runoff, and route the flow through the channel for steady-state simulations, respectively. The 10-, 2-, 1- and 0.2-percent (10-yr, 50-yr, 100-yr, and 500-yr) frequency storm depths were used for the precipitation data. The application of these methodologies is discussed in more detail in the following sections.

2.1 Drainage Basin Area Delineation

The hydrologic model inputs were developed using ESRI's ArcGIS (Version 10.5.1) software and the associated toolsets. The software and toolsets were used to delineate the watershed, to divide the watershed into subbasins, and to calculate hydrologic parameters for each subbasin. This Project involves using recently collected Light Detection and Ranging (LiDAR) elevation data collected as part of the 2015 Washington County, AR LiDAR collection gathered by FEMA and the USGS, along with the collection and use of new survey and field measurement data to develop hydrologic and hydraulic models, develop resultant floodplain mapping. The existing conditions model results were reviewed to develop proposed system improvement scenarios for identified locations throughout the Project area. The hydrologic work map for the study area is shown in Figure 2.1.

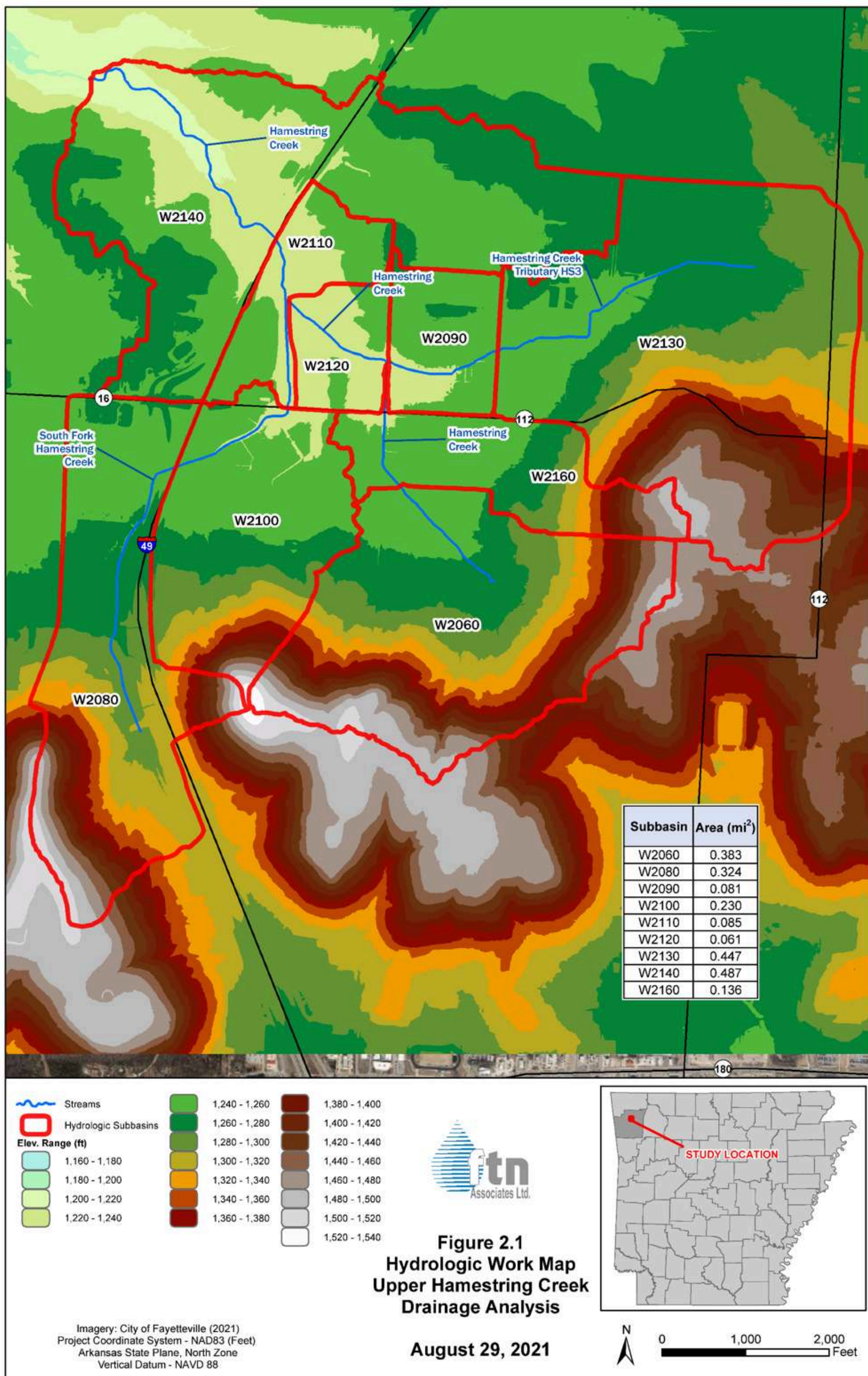


Figure 2.1. Hydrologic Work Map.

2.2 Modeling Methods

2.2.1 Loss Method

The Natural Resources Conservation Service (NRCS, formerly the Soil Conservation Service, or SCS) Curve Number method, as described in *TR-55: Urban Hydrology for Small Watersheds*, was used to calculate the rainfall loss rate for each detailed study subbasin. This method uses several parameters to determine the amount of rainfall that becomes runoff, including subbasin area, land use, soil type, depth and duration of precipitation, and initial abstraction ratio.

The soil data was taken from the NRCS Soil Survey Geographic Database (SSURGO) for the project area. The soil characteristics for the watersheds are displayed in Figure 2.2. Hydrologic Soil Group Map.

The land use for the Project area was developed using information taken from the 2011 National Land Cover Database from the US Geological Survey's (USGS) Multi-Resolution Land Characteristics Consortium (MRLC), available City of Fayetteville zoning data, and review of available aerial imagery. The following is a breakdown of the different land use types used for this study and the link to the TR-55 Runoff Curve Numbers for Urban Areas:

- Developed, Low Intensity: Lot sizes greater than 2.0 acres.
- Developed, Medium Intensity – Low: Lot sizes between 1 - 2 acres.
- Developed, Medium Intensity – Medium: Lot sizes less than 0.25 – 0.75 acres.
- Developed, Medium Intensity – High: Townhouses/Apartments.
- Developed, High Intensity: Commercial.
- Open Space: The open space land use includes open space areas in good condition; this includes pastures, grass areas, parks.

The curve number values from the City of Fayetteville Drainage Criteria Manual for each land use type and soil group are given in Table 2.1. For this study, antecedent moisture condition Type II (an average moisture condition) was assumed. The land use characteristics are displayed in Figure 2.3.

Table 2.1. Curve Numbers.

NLCD Description	TR-55 LANDUSE	Additional Notes	B	C	D
Developed, Open Space	Open Space (Fair)	(grass cover 50% to 75%)	69	79	84
	Open Space (Good)	(grass cover > 75%)	61	74	80
Developed, Low Intensity	Residential (2 acres)	Lots 2 acres or greater	65	77	82
Developed, Medium Intensity	Residential (1 acre)	1-2 acre lots	68	79	84
	Residential (1/2 acre)	0.25-0.75 acre lots	70	80	85
	Residential (1/8 acre or less)	Townhouses/apartments	85	90	92
Developed, High Intensity	Commercial		92	94	95
Impervious Areas	Streets and Roads	Areas of pavement	98	98	98
Pasture	Pasture (Good)		61	74	80
Open Water	Water		98	98	98
Mixed Forest	Woods / Forest (Good)		55	70	77

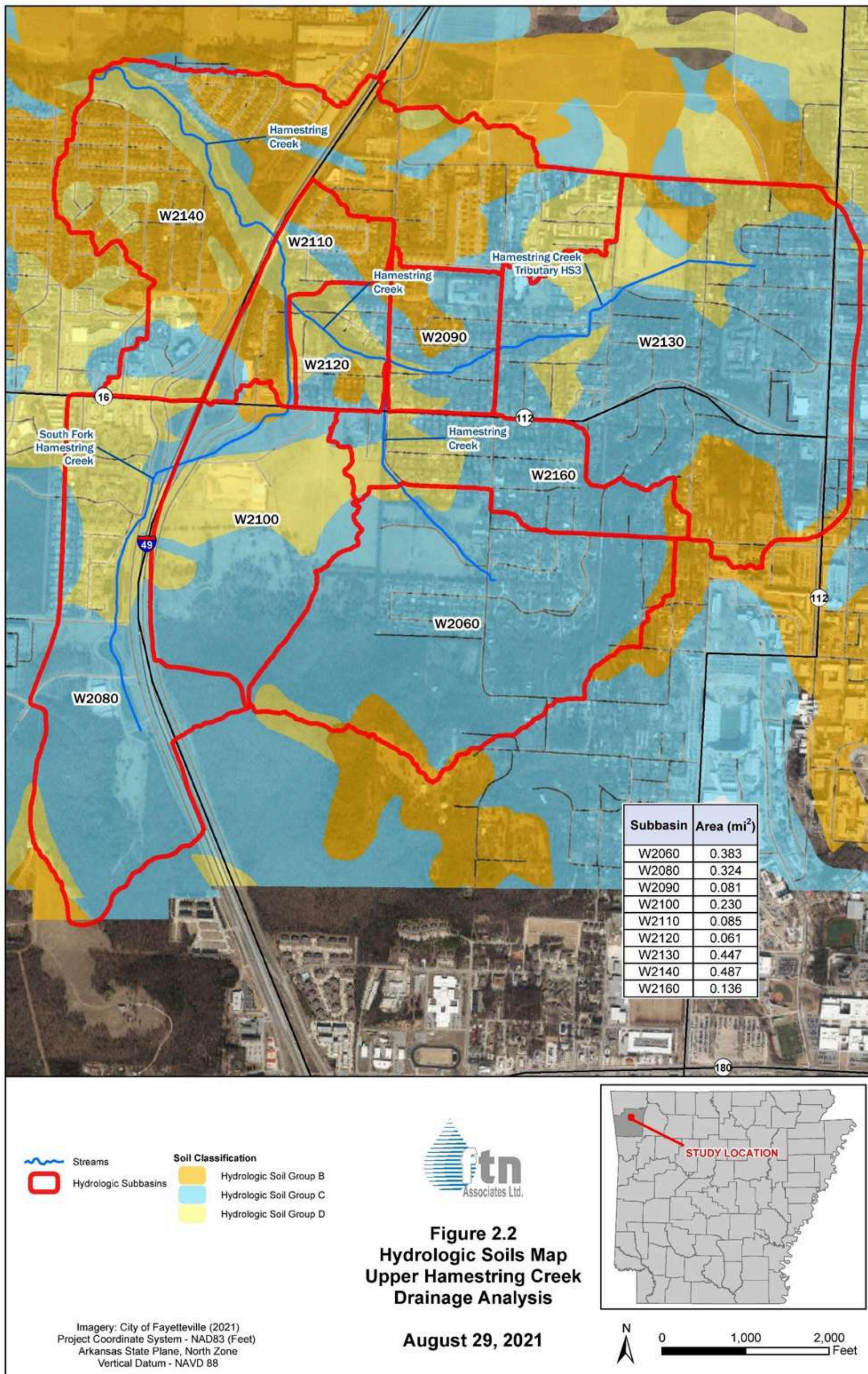


Figure 2.2. Hydrologic Soil Group Map.

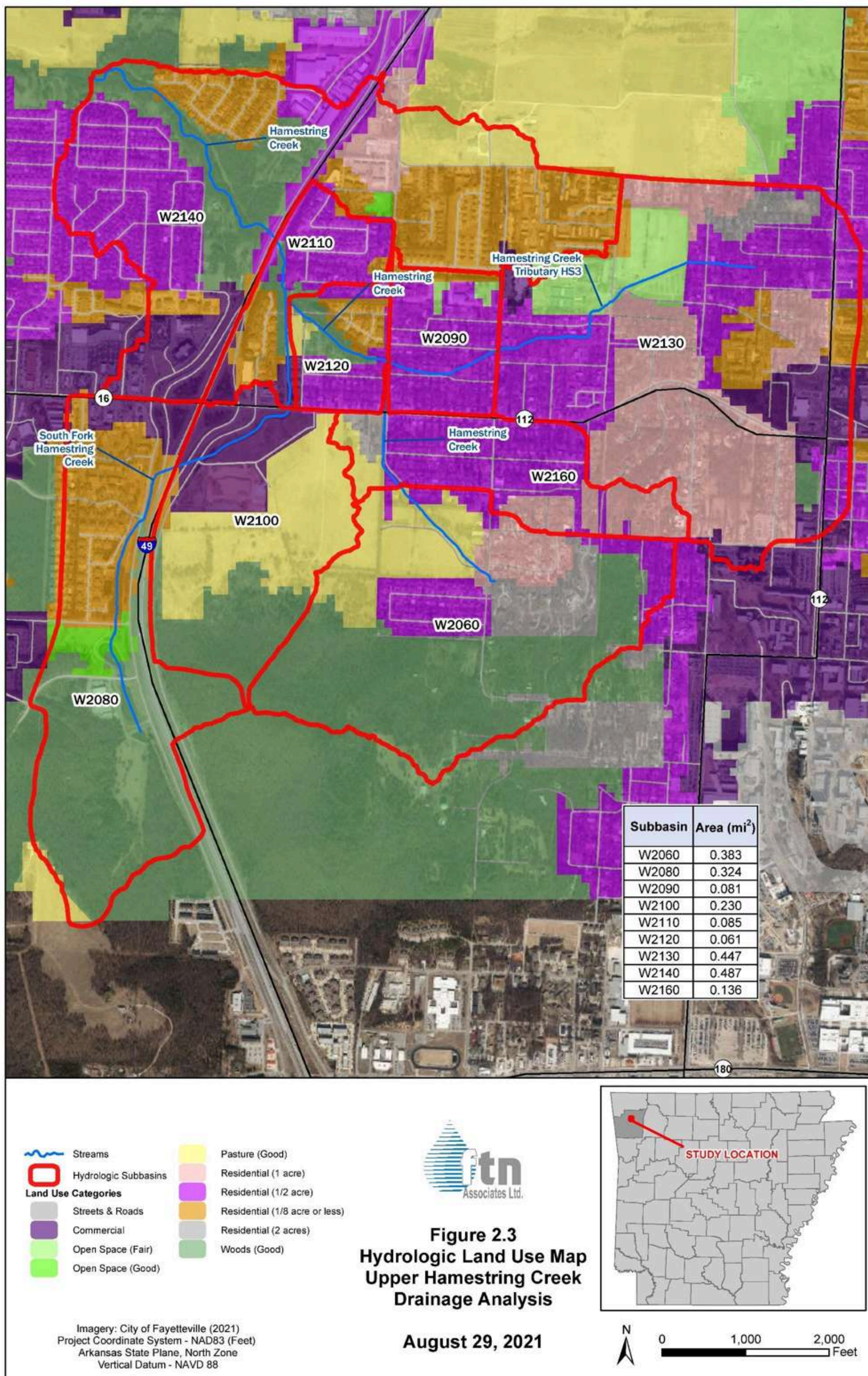


Figure 2.3. Land Use Map.

2.2.2 Transform Method

The NRCS method was also used for the transform method, which identifies the timing with which excess precipitation accumulates into peak runoff. The NRCS lag time is based on the Time of Concentration (T_c) calculated using the TR-55 method. The TR-55 method computes T_c assuming that water moves through a subbasin as sheet flow, shallow concentrated flow, open channel flow, or some combination thereof. The input variables used in the T_c calculations include flow length, slope, 2-year, 24-hour rainfall depth, and the geometric and roughness characteristics of the flow path. The flow length and slope were obtained using ArcGIS, which are based on the compiled topographic data. The 2-year, 24-hour rainfall was taken from the National Oceanic and Atmospheric Administration's (NOAA) Atlas 14 website, an updated version of the NOAA TP-40 and Hydro-35 publications identified in the City of Fayetteville Drainage Criteria Manual. The open channel characteristics used in the T_c calculations were based on preliminary hydraulic model data that was developed from HEC-HMS discharges and topography. The Manning's "n" values used in the T_c calculations were based on land use observed from aerial photography, site reconnaissance, and engineering judgment.

Upon calculating the T_c for each subbasin, the lag time (T_{Lag}) was calculated as $0.6 * T_c$. This relationship between T_{Lag} and T_c was given in the HEC-HMS Technical Reference Manual (April 2021) as the relationship suggested by the SCS (now the NRCS). This lag time (in minutes) was the input required for the transform method in HEC-HMS. A summary of the time of concentration calculations and the input variables, including the slopes, lengths, and surface characteristics, is provided in the supporting documents.

2.2.3 Routing Method

The routing method used for the detailed study reaches was the Modified Puls method. This routing method was selected because of the drainage area's tendency for overbank conveyance and structure impact on flow. HEC-GeoRAS and HEC-RAS (Version 5.0.7) were used to create a basic hydraulic model with, on average, 2-3 cross sections per basin. The cross sections were extracted from the topographic data gathered previously for this Project.

To apply the Modified Puls method, a wide range of discharges were applied to the modeled cross sections to develop the Storage-Discharge table for the reaches. The resulting storage output from the HEC-RAS model was compiled to extract the cumulative volume (storage) for each reach. This information was then entered into the HEC-HMS (Version 4.8.0) model for each reach. A summary of the routing data is provided in the supporting documents.

2.3 Design Storms

For this hydrologic analysis, the 10-, 2-, 1-, and 0.2-percent (10-, 50-, 100-, and 500-year) frequency storms were used for the meteorological models. The 24-hour event precipitation for these events were obtained from the NOAA Atlas 14 website. No baseflow was calculated based on engineering judgment of watershed characteristics. A time interval of 1 minute was used for the model run. Table 2.2. Precipitation Data is a tabular summary of frequency events used in the modeling.

For the detailed study modeling, frequency storms were used for the distribution of rainfall rather than a SCS rainfall distribution approach. This approach was chosen to provide a more accurate distribution based on historical data.

Table 2.2. Precipitation Data.

Duration	Annual Chance Flood Frequency (Depth in inches)			
	10%	2%	1%	0.2%
5 min	0.626	0.819	0.904	1.110
15 min	1.120	1.460	1.610	1.980
1 hr	2.310	3.090	3.450	4.300
2 hr	2.950	4.000	4.470	5.630
3 hr	3.380	4.620	5.180	6.600
6 hr	4.030	5.500	6.200	7.970
12 hr	4.570	6.140	6.900	8.890
24 hr	5.310	7.070	7.910	10.100

- Input type: partial duration.
- Intensity duration: 5 minutes.

2.4 Model Calibration

For this project, there were no gage locations within the Project area. Additionally, while the existing FEMA Flood Insurance Study mapping is detailed zone (Zone AE), no calibration to gage data was performed.

3.0 HYDRAULIC MODELING

3.1 Introduction

For the Project, a detailed study approach was conducted for all of South Fork Hamestring Creek and Hamestring Creek Tributary HS3, and the portion of Hamestring Creek upstream of the confluence with North Fork Hamestring Creek to its effective FEMA limit of study. The hydraulic analyses are based on unobstructed, riverine flow. The flood elevations are thus considered valid only if hydraulic structures remain unobstructed, operate properly, and do not fail. The application of these methodologies is discussed in more detail in the following sections.

3.2 1D Hydraulic Modeling

3.2.1 Geometry

Where available, FTN built upon existing FEMA effective hydraulic modeling geometries for this analysis. Additionally, the available hydraulic model geometry was imported into the CivilGEO toolset, GeoHECRAS, to develop additional geometry data, including stream centerline, flowpaths, reach lengths, and cross sections revisions, if needed. If no FEMA effective hydraulic data was available, a new hydraulic model was built using GeoHECRAS. Cross section spacing varied, but generally ranged from every 200 to 500 feet and was oriented to be as perpendicular as possible to the floodplain. Cross section elevations were based on the Washington County, AR and Incorporated Areas LiDAR data completed in 2015. All study streams were studied using the U.S. Army Corps of Engineers hydraulic computer model HEC-RAS (Version 5.0.7).

Bank stations were assigned and structures were inserted in the GeoHECRAS program (see Section 2.2.3 for more information). Engineering judgment and preliminary mapping results were used in determining areas of ineffective or obstructed flow. Common locations which required ineffective flow regions or blocked obstructions included areas where floodplains merge, low areas which are isolated from the main flowpath, low areas which were already filled with water, upstream and downstream of structures, and locations which experience dramatic changes in flow area across very short distances.

3.2.2 Hydraulic Parameters

Manning's "n" values (roughness coefficients) were developed based on land use data developed during the hydrologic analysis, aerial imagery, field survey and reconnaissance photos, and/or engineering judgment. Typical manning values for various landuses are listed in Table 2.1.

Table 3.1. Typical Roughness Coefficients.

Roughness Coefficients	Description
0.011	Impervious Areas (e.g., parking lots / roads)
0.035 – 0.040	Channel
0.040	Well kept lawns (e.g. golf course, ball fields, park areas)
0.050	Open Space
0.060	Pasture
0.080	Residential
0.100	Wooded areas

3.2.3 Structures

Bridge and culvert analysis is typically based on four cross sections, which aid in the determination of energy losses in and around the structures. In using the HEC-RAS program for bridges, the program automatically develops two additional cross sections inside of the bridge structure, using the geometry from cross sections 2 and 3. Cross section 1 is typically located far enough downstream as not be affected by the structure; therefore, the water is flowing fully effectively. Cross sections 2 and 3 (the adjacent sections) are typically located a short distance away from the structure; outside the influence of any elevated roadway embankments or roadside ditches. Spacing of adjacent sections from the structure face is normally based on the size of the structure opening and/or length of the culvert. Cross section 4 is typically located far enough upstream to not be affected by the structure constriction; therefore, the water is flowing fully effectively. The spacing for cross sections 1 and 4 will depend on various factors: the size of the opening, velocity and volume of flow, and the degree of the constriction. Typically, locations around the bridge and culvert crossings are based on the traditional contraction/expansion ratios.

Contraction or expansion of flow due to changes in the cross section is a common cause for energy losses between various cross sections. Therefore, various coefficients ranging from 0 – 1 are used to estimate the change in the effective cross section area. Small changes in effective cross-sectional area are represented by values of 0.1 / 0.3 (contraction/expansion), while abrupt changes such as bridges and culverts are typically represented by values of 0.3 / 0.5 to 0.6 / 0.8 (contraction/expansion).

Ineffective flow areas were also used on the upstream and downstream sides of bridges and culverts with elevations on the upstream side generally equal to the lowest top of road elevation and with elevations on the downstream side generally between the low chord and the lowest top of road elevation. In cases where a structure warranted, the ineffective flow settings were set to permanent. This latter setting was determined on a case-by-case basis.

Structure information for this analysis was either obtained from FEMA effective hydraulic models or from new survey data gathered as part of this Project.

3.2.4 Boundary Conditions

For the hydraulic modeling conducted as part of the task, the downstream boundary condition was selected to be normal depth with the slope being determined from available survey and topographic data at the downstream end of a study reach.

3.2.5 Floodway

Floodways were determined for the detailed study reaches included as part of this analysis. There are many locations throughout the study area that have floodplain mapping that is wider and/or narrower compared to the effective mapped widths. The updates to the hydrologic model have resulted in changes to the peak flow and altered the flooding extents, thus making the floodway widths vary from the effective mapped information. Differences in terrain data sources used for the effective and the new models have also contributed to changes in the modeled floodplain and floodway widths. Some of the differences can also be attributed to actual physical changes in topography due to development in overbank areas and channelization of study streams in several locations.

3.3 1D-2D Hydraulic Modeling

In addition to the 1D steady-state hydraulic modeling discussed previously, FTN also developed a 1D-2D unsteady state hydraulic model for the Project area. This hydraulic model was used to determine the impact of proposed improvement scenarios within the Project area. This model was developed using the USACE's HEC-RAS software (Version 6.0) to model both the existing and proposed improvement scenarios.

While 1D HEC-RAS can be used as an unsteady flow model, it cannot model the spread of flow (i.e., flow in both the longitudinal and lateral directions) because it uses a series of cross sections to represent the terrain surface and roughness characteristics, and it is assumed that velocities only vary in the longitudinal direction. Between these cross sections, the 1D model interpolates based on the available cross section data to perform its calculations for the area of interest. Dependent on the number of cross sections and the detail provided, the limitations of the 1D model could lead to an inaccurate representation of flooding conditions and could impact the modeled efficacy of proposed drainage improvements.

With a 2D hydraulic model, the system is modeled using a computational mesh rather than a series of cross sections along the longitudinal axis of the stream reach. The mesh consists of computational cells that have elevation ground profiles and roughness values along the cell faces that represent the topographic surface, frictional characteristics of the area, and volumetric relationships for the cell area. The use of the 2D model allows for more detailed resolution in water surface elevations, velocities, and flow patterns than is possible with a 1D model that is only capable of computing average values for three general regions at each cross section (i.e., that are averaged in the left and right overbanks and the channel). Based on engineering judgment and study goals, the decision was made to use the 1D-2D ability of HEC-RAS for the conceptual analysis.

For this model, the 2D mesh begins just upstream of the Interstate 49 crossing along Hamestring Creek and ties into the downstream 1D model section that was taken from the newly developed Hamestring Creek study previously mentioned. Additionally, breaklines were defined along roads, culverts and other significant features identified on the topography and aerial imagery. The Manning's "n" parameters were taken from land use data previously discussed. Adjustments and proposed improvements were then made to the underlying terrain information for use in proposed improvement scenarios.

3.4 Proposed Conditions Modeling

Once the existing conditions 1D-2D model was completed, FTN was tasked with modifying it to develop basic concept-level improvement scenarios for the Project area. Various scenarios were considered, including structure improvements and detention and channel enlargement along HSC; structure improvements and detention and channel modifications along HS-3; and channel improvements along SFHC. Prior to the start of the Project, City personnel identified general ideas on the location of these improvements, to see if they address the existing drainage issues. The improvement scenarios examined as part of this Project are as follows:

- Hamestring Creek: Removal of existing beaver dam downstream of Hamestring Creek's Interstate 49 crossing;
- Hamestring Creek: Adding overbank storage on existing undeveloped property upstream of Wedington Drive;

- Hamestring Creek: Improving channel capacity along Porter Road at Valley Drive;
 - (Note: channel widths of 20-, 30-, and 40-ft were examined).
- Hamestring Creek Tributary HS3: Adding overbank between Lewis Avenue and Mt. Comfort Road;
 - (Note: small and large pond volume were examined).
- Hamestring Creek Tributary HS3: Improving channel capacity along Hatfield Street; and
 - (Note: channel widths of 20- and 30-ft were examined)
- South Fork Hamestring Creek: Improving channel capacity downstream of Wedington Drive.
 - (Note: channel based on size and shape of current channelized portion between Interstate 49 and Wedington Drive).

After meeting with City personnel, channel capacity improvements would be made using similar channel characteristics as are currently being used within the watershed; while the additional storage would be consistent with typical detention ponds throughout the City.

4.0 RESULTS AND CONCLUSIONS

4.1 Conceptual Improvement Scenarios

Once the various models were completed, the modeling revealed that no single proposed improvement scenario would address all of the localized flooding issues by itself and that the flooding is primarily due to the lack of topographic relief within the developed portions of the Project area, which in turn impacts numerous homes.

Based on the proposed improvement model runs, the hydraulic model simulations showed localized areas of improvement in the computed WSEs; however, most of these results did not reveal a large improvement in the resulting floodplains due to flat sloped area. The existing conditions and proposed scenario hydraulic modeling revealed the following:

Hamestring Creek

- Beaver Dam Removal:
 - While the beaver dam that currently exists downstream of Interstate 49 does detain water through the structure, its removal does not impact the peak Water Surface Elevations (WSE) upstream of the Interstate 49 crossing significantly. It would however reduce the standing water through the Interstate 49 structure.
 - Net improvement of 0.1-ft around the Interstate 49 crossing of Hamestring Creek.
- Channel Improvement between Wedington Drive and Confluence with HS3 (Valley Drive):
 - Channel size was taken to be a vertical wall channel with flat concrete bottom. It is important to note that scenario was run to determine the maximum potential improvement that could be gained; however, it may not be possible due to future regulatory restrictions. Bottom widths of 20-, 30-, and 40-ft were utilized in an attempt to reduce channel overflows at Valley Drive and along Porter Road.
 - A vertical wall channel scenario was examined through this stretch in an attempt to reduce the overall impact to neighboring homes and property. Additionally, these scenarios will require purchasing of homes and property along the Hamestring Creek channel from the confluence with HS3 up to Wedington Drive.
 - The results from the three scenarios revealed that the construction of a 40-ft wide vertical wall concrete channel would significantly reduce the amount

of overflow into the overbank and impacting property for the 10% event scenario.

- Net improvement for the 40-ft channel width ranged from increasing the flow in the channel from 280 cfs (10% chance event) to an increase of 700 cfs (0.2% chance event), which was accompanied by a reduction to the WSEs immediately downstream of Lewis Avenue of 0.8-ft (10% chance event) to 0.25-ft (0.2% chance event).
- Table 4.1 below provides a tabular summary of the results at this location, while figures located in the Appendices section of this document (Figures 1 through 8) provide a graphical representation of the improvements for the 10- and 1% annual-chance events.

Table 4.1. Valley Drive Channel Improvement Results.

Event	Existing Conditions		20-ft Channel		30-ft Channel		40-ft Channel	
	Discharges (cfs)	Water Surface Elevation (ft)	Discharges (cfs)	Water Surface Elevation (ft)	Discharges (cfs)	Water Surface Elevation (ft)	Discharges (cfs)	Water Surface Elevation (ft)
10% Annual Chance	405	1237.3	675	1237.1	688	1236.7	688	1236.5
2% Annual Chance	507	1237.6	909	1237.4	969	1237.2	994	1237.1
1% Annual Chance	544	1237.7	990	1237.6	1,103	1237.4	1,131	1237.3
0.2% Annual Chance	655	1238.0	1,118	1237.9	1,303	1237.8	1,369	1237.7

- Storage upstream of Wedington Drive:
 - This scenario included the construction of a detention pond upstream of Wedington Drive on parcel 765-13895-000, which is currently privately owned and includes the addition of an approximate 500-ft x 200-ft pond with an average depth of 5-ft. (Approximately 11.5 Ac-ft of storage).
 - The modeling revealed that discharge from the upper segment of Hamestring Creek (upstream of Wedington Drive and the private drive crossings) is able to escape the channel prior to reaching the proposed detention pond. The topography of the area also reveals that once the flows leave the channel, they flow overland where they then flow around the proposed detention pond, which nullifies the real impact of the pond.

- Net improvement consisting of flow reductions ranging from 40 cfs (10% event) to 100 cfs (0.2% event); however, the peak WSE changes are consistently less than 0.1 ft for all events modeled.
- Table 4.2 below provides a tabular summary of the results at this location, while figures located in the Appendices section of this document (Figures 9 through 12) provide a graphical representation of the improvements for the 10- and 1% annual-chance events.

Table 4.2. Hamestring Creek Storage Results.

Event	Existing Conditions		Storage Pond	
	Discharges (cfs)	Water Surface Elevation (ft)	Discharges (cfs)	Water Surface Elevation (ft)
10% Annual Chance	360	1248.3	322	1248.2
2% Annual Chance	519	1248.4	448	1248.4
1% Annual Chance	600	1248.5	504	1248.5
0.2% Annual Chance	766	1248.7	672	1248.6

Hamestring Creek Tributary HS3

- Storage between Lewis Avenue and Mt. Comfort Road:
 - Small Storage scenario is approximately 220-ft x 250-ft in size with an average depth of 4.5-ft with no structure improvements to adjacent street crossings. (Approximately 5.5 Ac-ft of storage).
 - Large Storage scenario is approximately 220-ft x 850-ft in size with an average depth of 4.5-ft with no structure improvements to adjacent street crossings. (Approximately 19.0 Ac-ft of storage).
 - The results from the model scenarios revealed that the small detention scenario made little change in WSE or computed discharges downstream.
 - Improvements were less than 15 cfs and 0.1-ft.
 - The large detention scenario revealed that the large pond configuration provided improvements for the 10% annual-chance event and then decreased in effectiveness as the storm events grew in magnitude.
 - Improvements ranged from a reduction in flows of 250 cfs (10% chance event) to 70 cfs (0.2% chance event), which was accompanied by a reduction to the WSEs immediately downstream of Lewis Avenue of 0.5-ft (10% chance event) to 0.1-ft (0.2% chance event).
 - Table 4.3 below provides a tabular summary of the results at this location, while figures located in the Appendices section of this document (Figures 13 through 18) provide a graphical representation of the improvements for the 10- and 1% annual-chance events.

Table 4.3. Lewis Avenue Storage Results.

Event	Existing Conditions		Small Storage Pond		Large Storage Pond	
	Discharges (cfs)	Water Surface Elevation (ft)	Discharges (cfs)	Water Surface Elevation (ft)	Discharges (cfs)	Water Surface Elevation (ft)
10% Annual Chance	754	1254.4	739	1254.4	511	1253.9
2% Annual Chance	1,077	1254.7	1,074	1254.7	896	1254.5
1% Annual Chance	1,222	1254.8	1,219	1254.8	1,083	1254.7
0.2% Annual Chance	1,581	1255.1	1,578	1255.1	1,516	1255.0

- Channel Improvement along Hatfield Street:
 - Channel size was taken to be a vertical wall channel with flat concrete bottom. It is important to note that scenario was run to determine the maximum potential improvement that could be gained; however, it may not be possible due to future regulatory restrictions. Bottom widths of 20- and 30-ft were utilized in an attempt to reduce channel overflows along Hatfield Drive.
 - A vertical wall channel scenario was examined through this stretch in an attempt to reduce the overall impact to neighboring homes and property.
 - The results from the scenarios revealed that neither the 20- or 30-ft channel could contain all of the flows in HS3; thus, allowing flows to spread into the overbanks.
 - Net improvement for the 30-ft channel width had little overall impact to the flows; however, this was accompanied by a reduction to the WSEs along Hatfield Street by 0.3-ft (10% chance event) to 0.15-ft (0.2% chance event).
 - Table 4.4 below provides a tabular summary of the results at this location, while figures located in the Appendices section of this document (Figures 19 through 24) provide a graphical representation of the improvements for the 10- and 1% annual-chance events.

Table 4.4. Hatfield Street Channel Improvement Results.

Event	Existing Conditions		20-Channel		30-ft Channel	
	Discharges (cfs)	Water Surface Elevation (ft)	Discharges (cfs)	Water Surface Elevation (ft)	Discharges (cfs)	Water Surface Elevation (ft)
10% Annual Chance	775	1238.1	772	1237.8	771	1237.7
2% Annual Chance	1,169	1238.4	1,165	1238.2	1,166	1238.2
1% Annual Chance	1,334	1238.6	1,331	1238.4	1,332	1238.4
0.2% Annual Chance	1,727	1238.9	1,727	1238.8	1,727	1238.7

South Fork Hamestring Creek

- Channel improvement downstream of Wedington Drive.
 - Channel size for this scenario was taken from the channel upstream of Wedington Drive, which was a 3:1 (H:V) trapezoidal channel with a 30-ft bottom width.
 - The scenario results indicate that the construction of the trapezoidal channel with dimensions consistent with the upstream channel would significantly reduce the amount of overflow into the overbank for the 10-, 2-, 1-, and 0.2% event scenario.
 - Net improvement for the trapezoidal channel had little overall impact to the flows; however, this was accompanied by a reduction to the WSEs along downstream of Wedington Drive by 1.5-ft (10% chance event) to 0.5-ft (0.2% chance event).
 - Table 4.5 below provides a tabular summary of the results at this location, while figures located in the Appendices section of this document (Figures 25 through 28) provide a graphical representation of the improvements for the 10- and 1% annual-chance events.

Table 4.5. SF Hamestring Creek Channel Improvement Results.

Event	Existing Conditions		30-ft Trapezoid Channel	
	Discharges (cfs)	Water Surface Elevation (ft)	Discharges (cfs)	Water Surface Elevation (ft)
10% Annual Chance	325	1231.4	334	1229.9
2% Annual Chance	433	1231.8	441	1230.8
1% Annual Chance	482	1231.9	491	1231.1
0.2% Annual Chance	596	1232.3	608	1231.8

APPENDICES

APPENDIX A

Figures 1-8 Hamestring Creek - Valley Channel Improvements

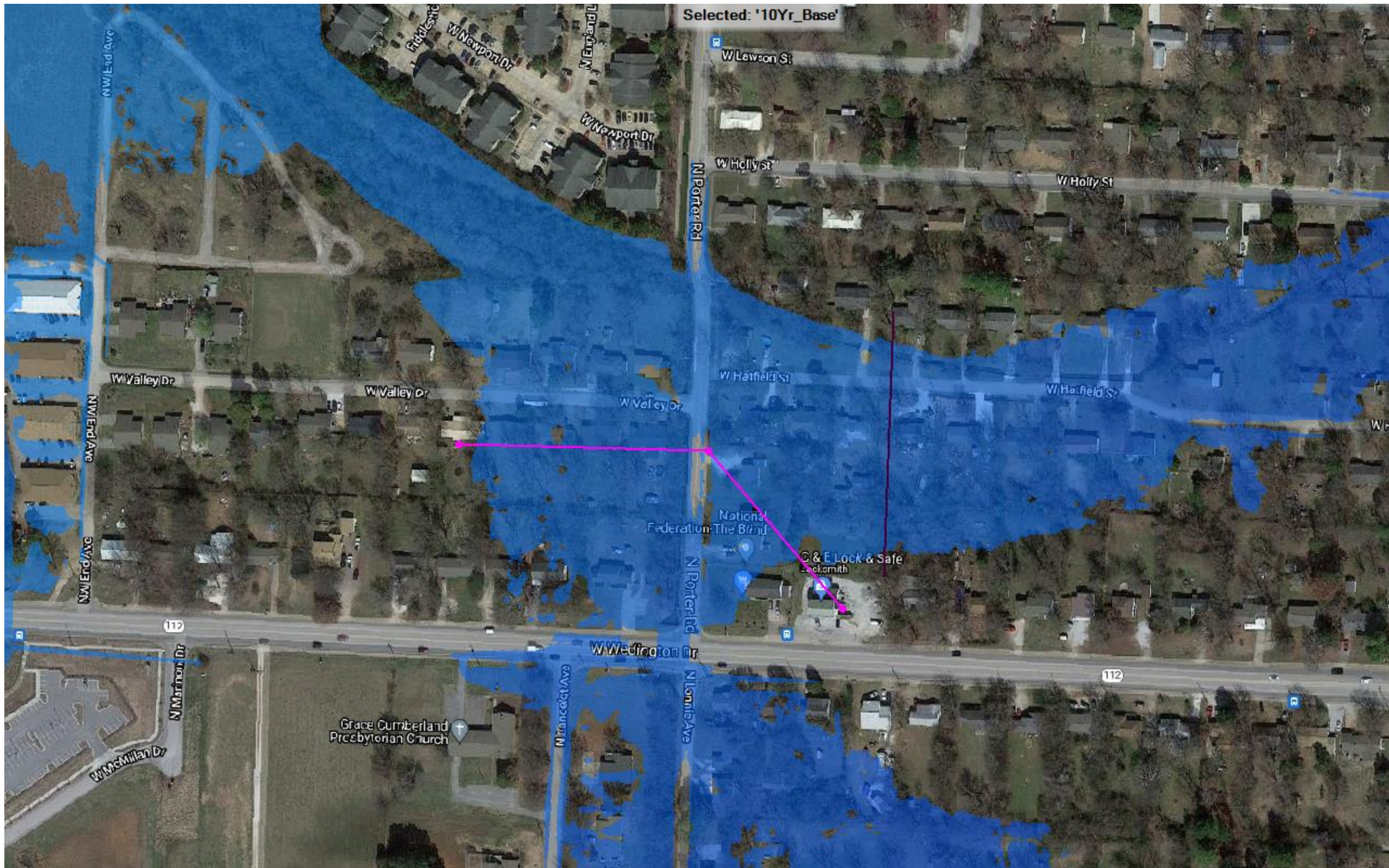


Figure 1. Existing Conditions – 10-Year Valley Drive Improvements.

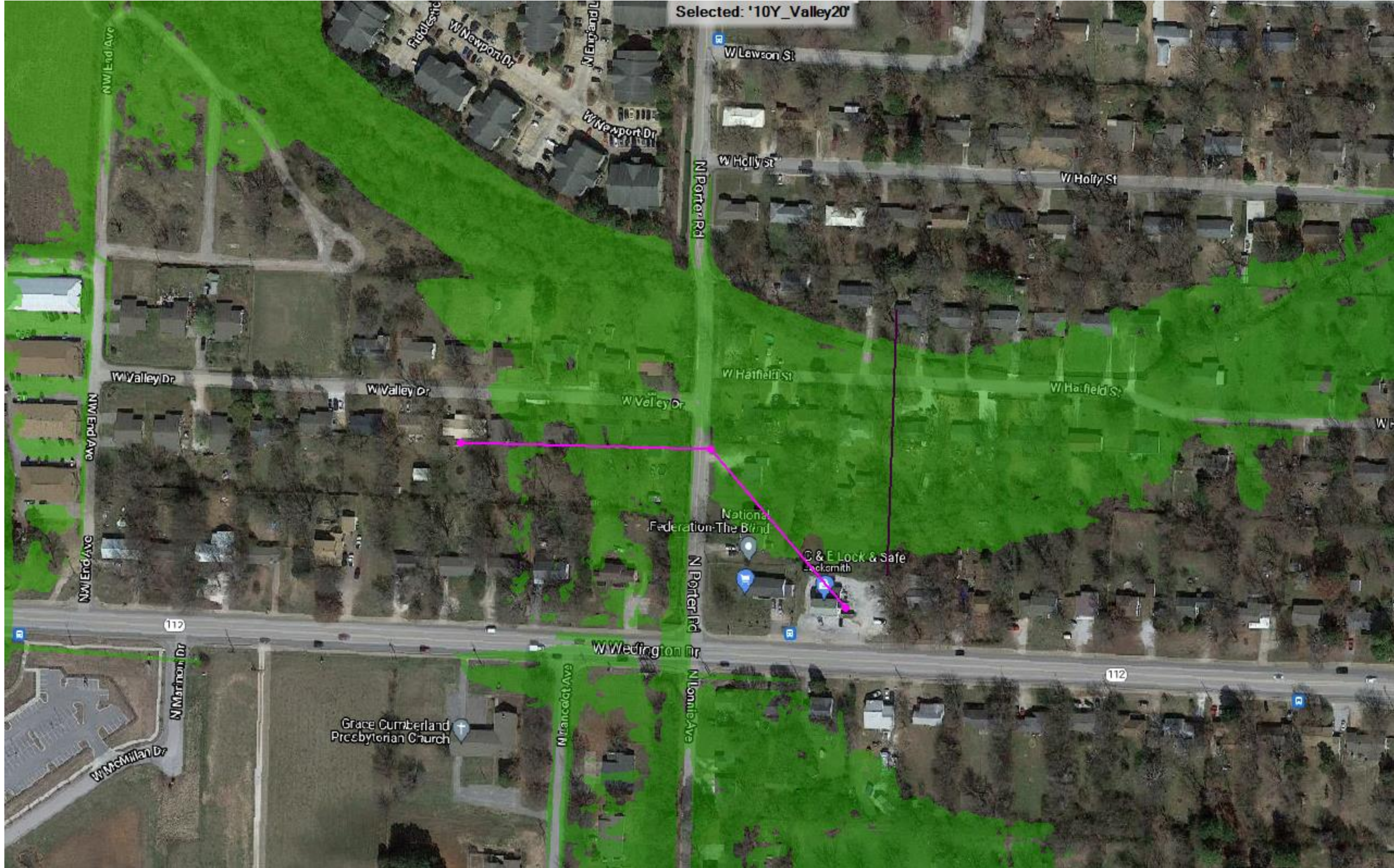


Figure 2. 20-ft Channel – 10-Year Valley Drive Improvements.

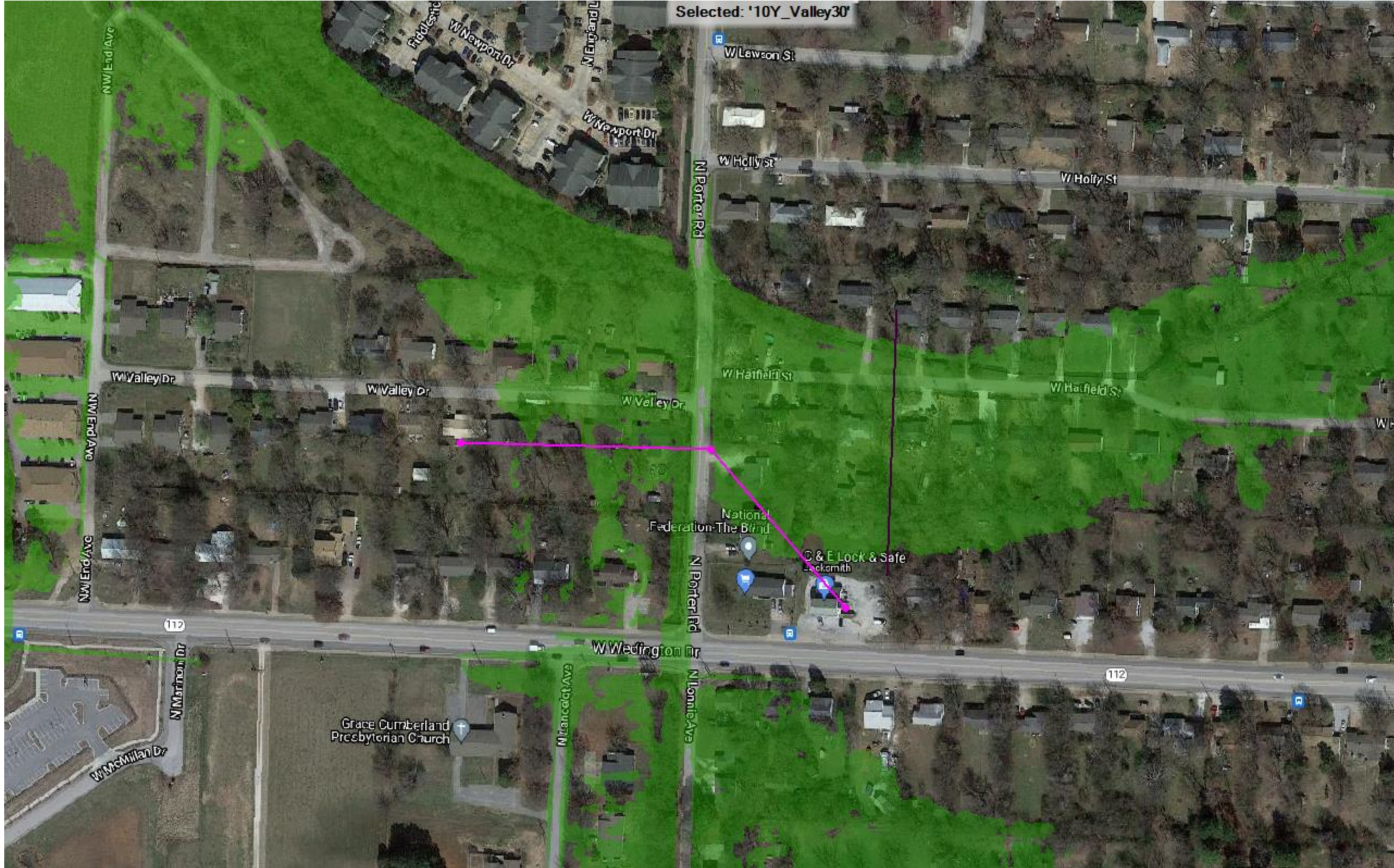


Figure 3. 30-ft Channel – 10-Year Valley Drive Improvements.

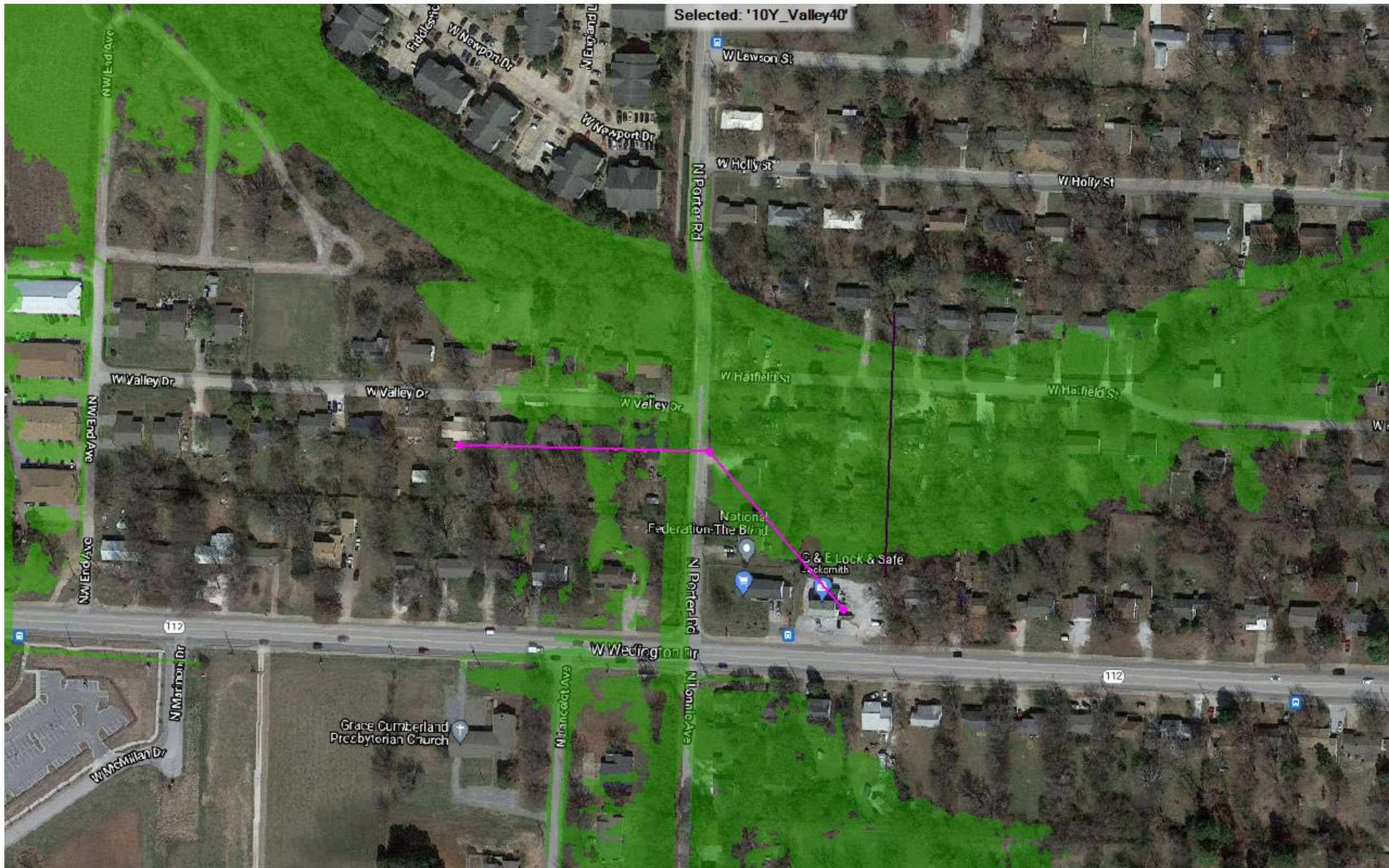


Figure 4. 40-ft Channel – 10-Year Valley Drive Improvements.

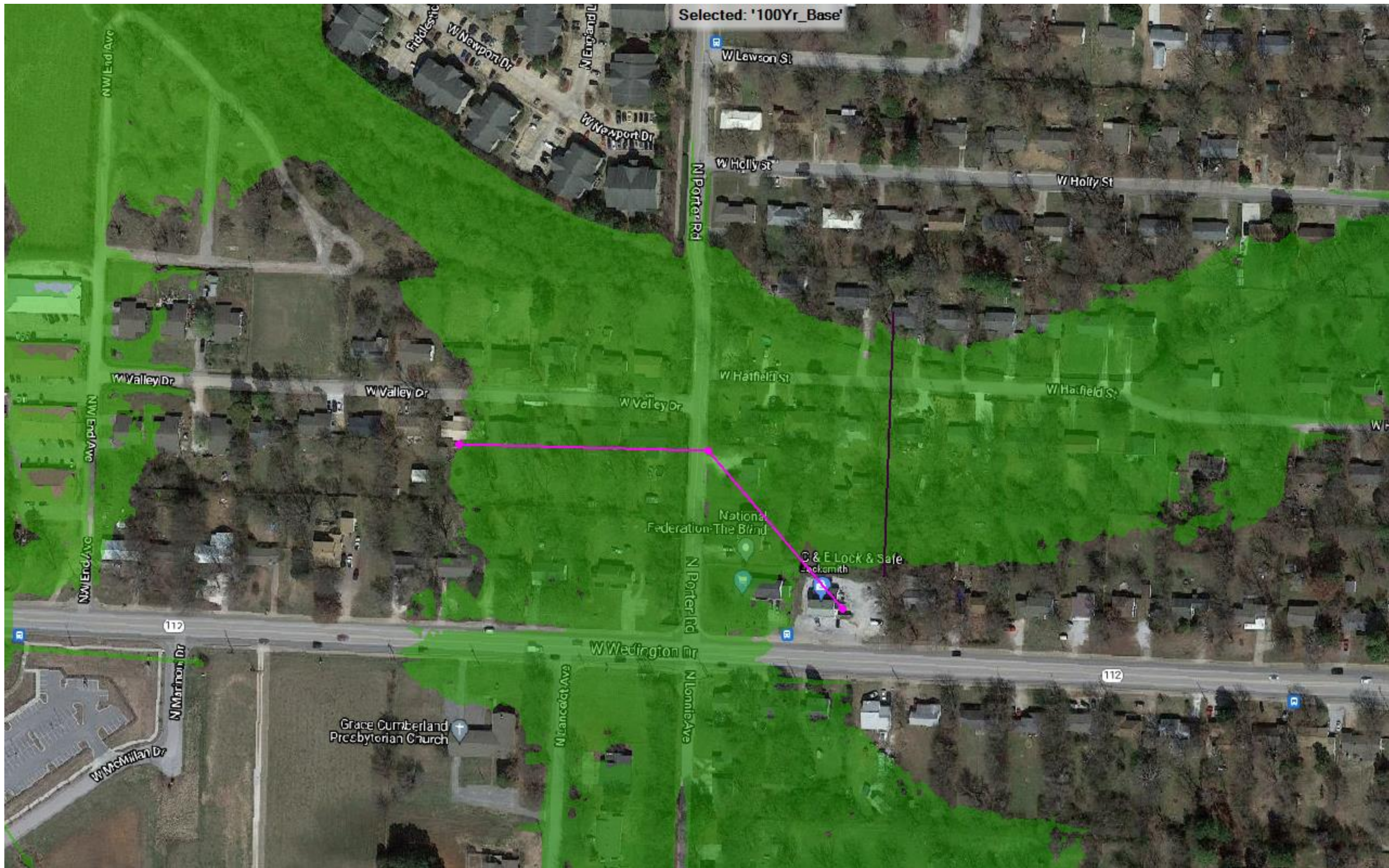


Figure 5. Existing Conditions – 100-Year Valley Drive Improvements.

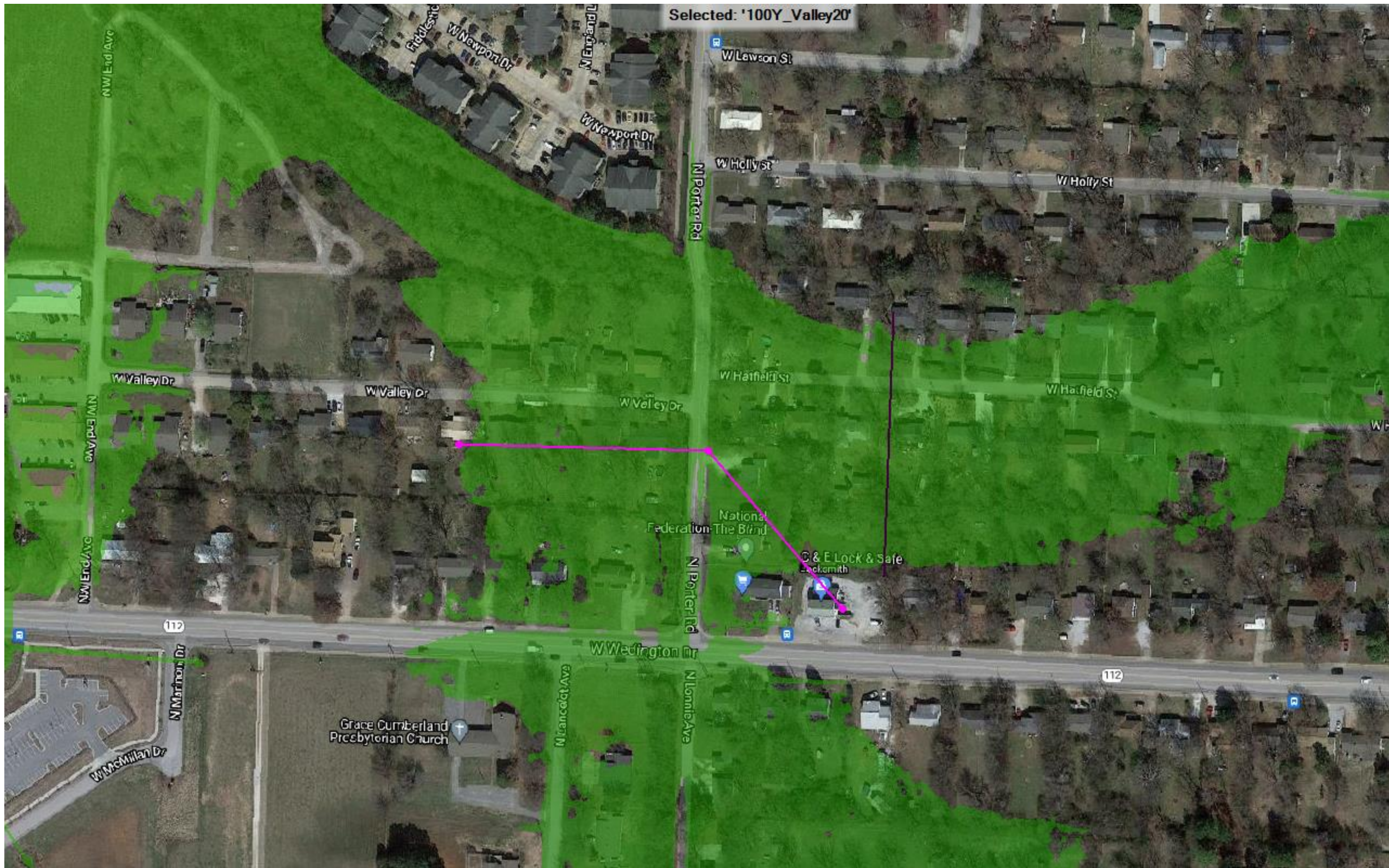


Figure 6. 20-ft Channel – 100-Year Valley Drive Improvements.

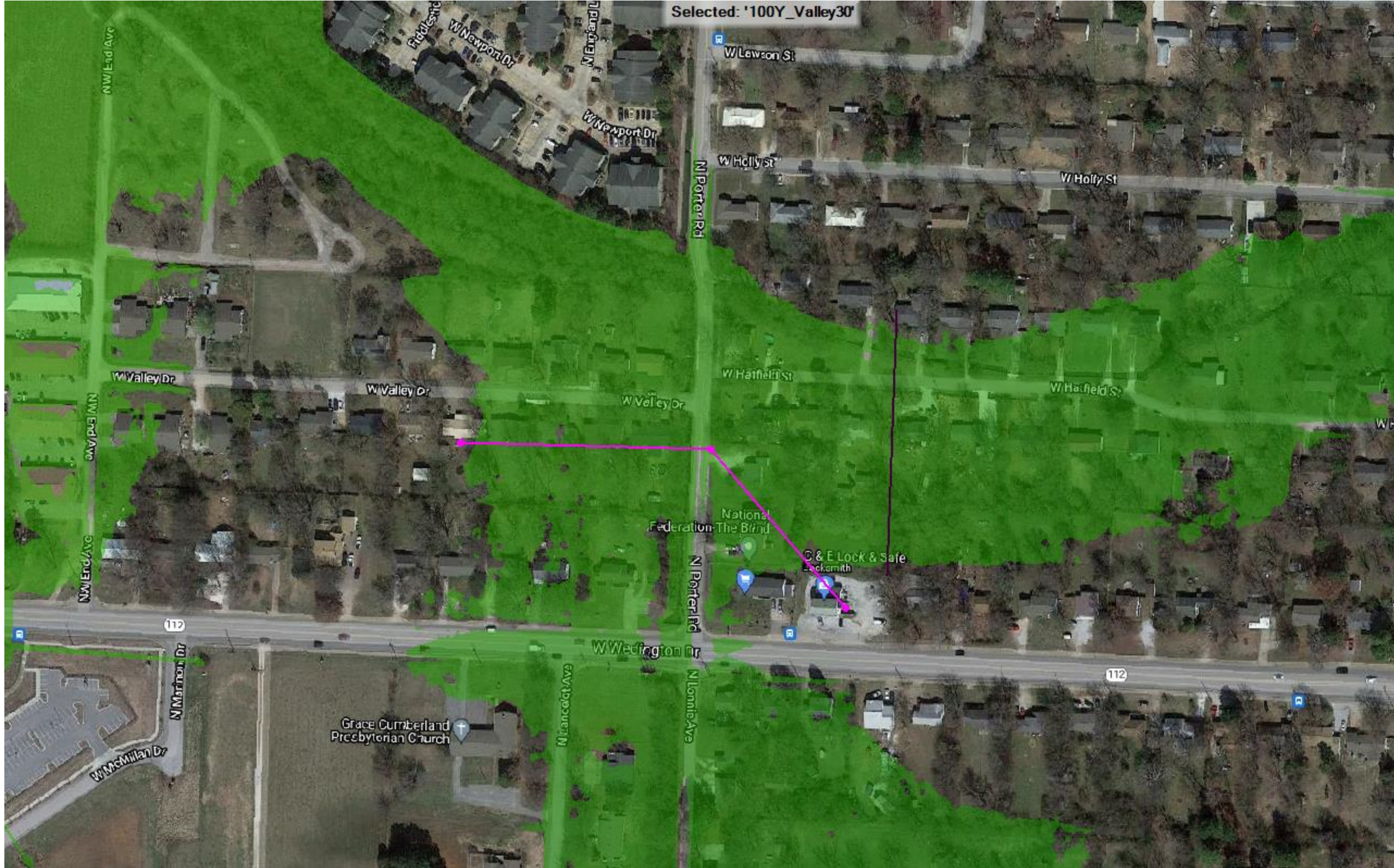


Figure 7. 30-ft Channel – 100-Year Valley Drive Improvements.

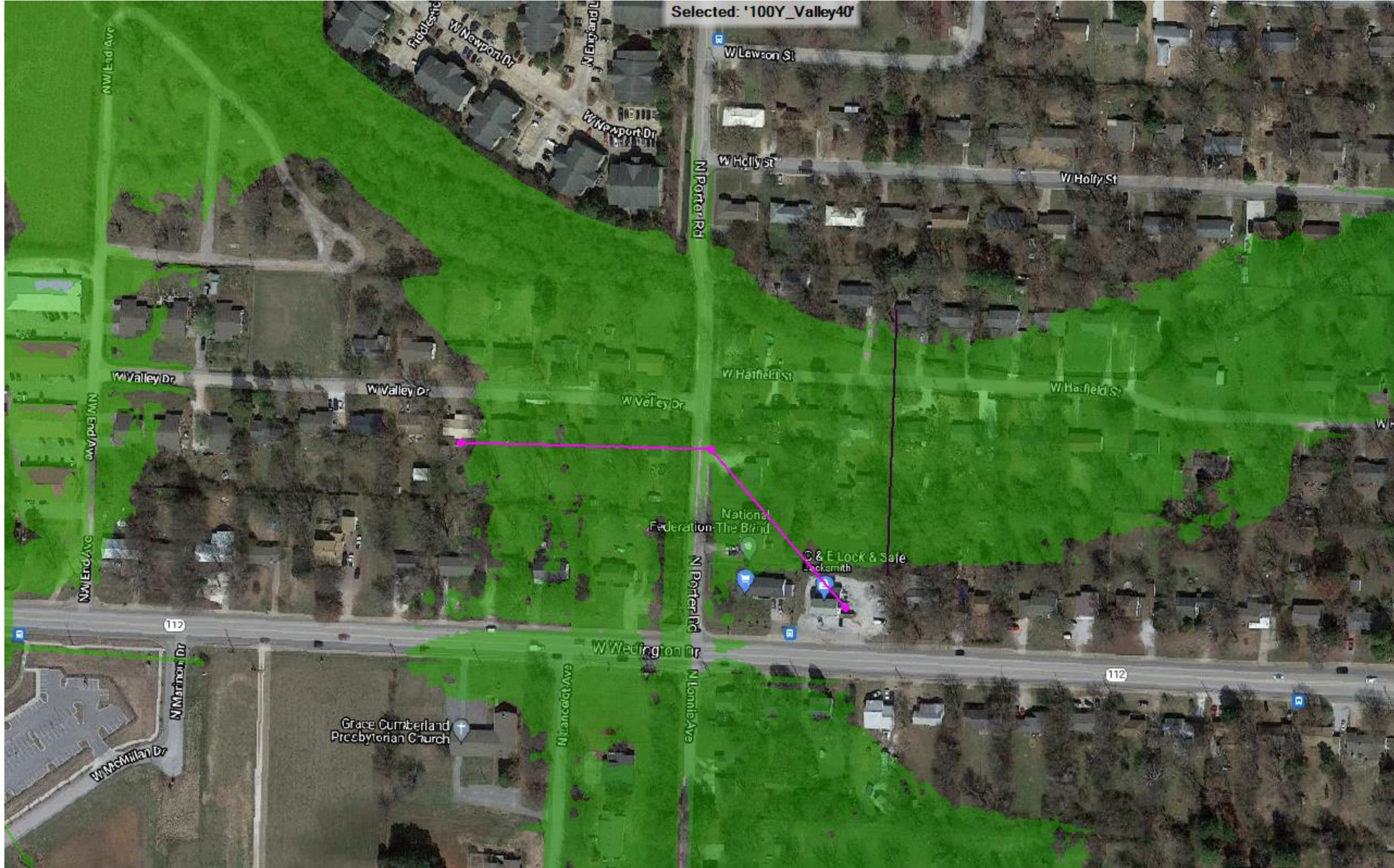


Figure 8. 40-ft Channel – 100-Year Valley Drive Improvements.

APPENDIX B

Figures 9-12 Hamestring Creek – Storage Improvements

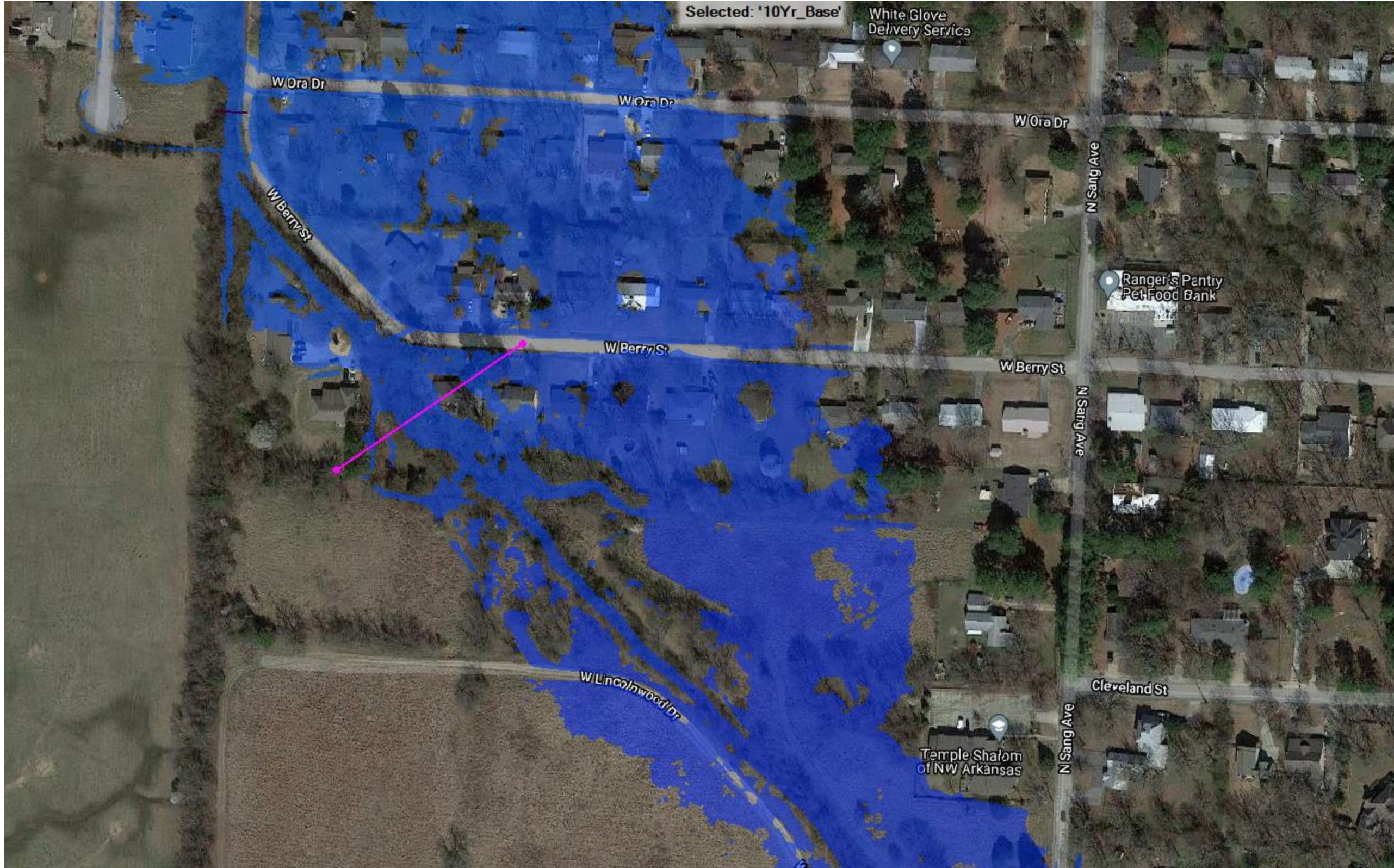


Figure 9. Existing Conditions – 10-Year Hamstring Creek Storage.



Figure 10. 10-Year Hamstringing Creek Storage.



Figure 11. Existing Conditions – 100-Year Hamstring Creek Storage.



Figure 12. 100-Year Hamstring Creek Storage.

APPENDIX C

Figures 13-18 Hamestring Creek Tributary HS3 – Lewis Storage Improvements



Figure 14. Small Storage Pond – 10-Year Lewis Storage.



Figure 15. Large Storage Pond – 10-Year Lewis Storage.



Figure 16. Existing Conditions – 100-Year Lewis Storage.



Figure 18. Large Storage Pond – 100-Year Lewis Storage.

APPENDIX D

Figures 19-24 Hamestring Creek Tributary HS3 – Hatfield Street Improvements

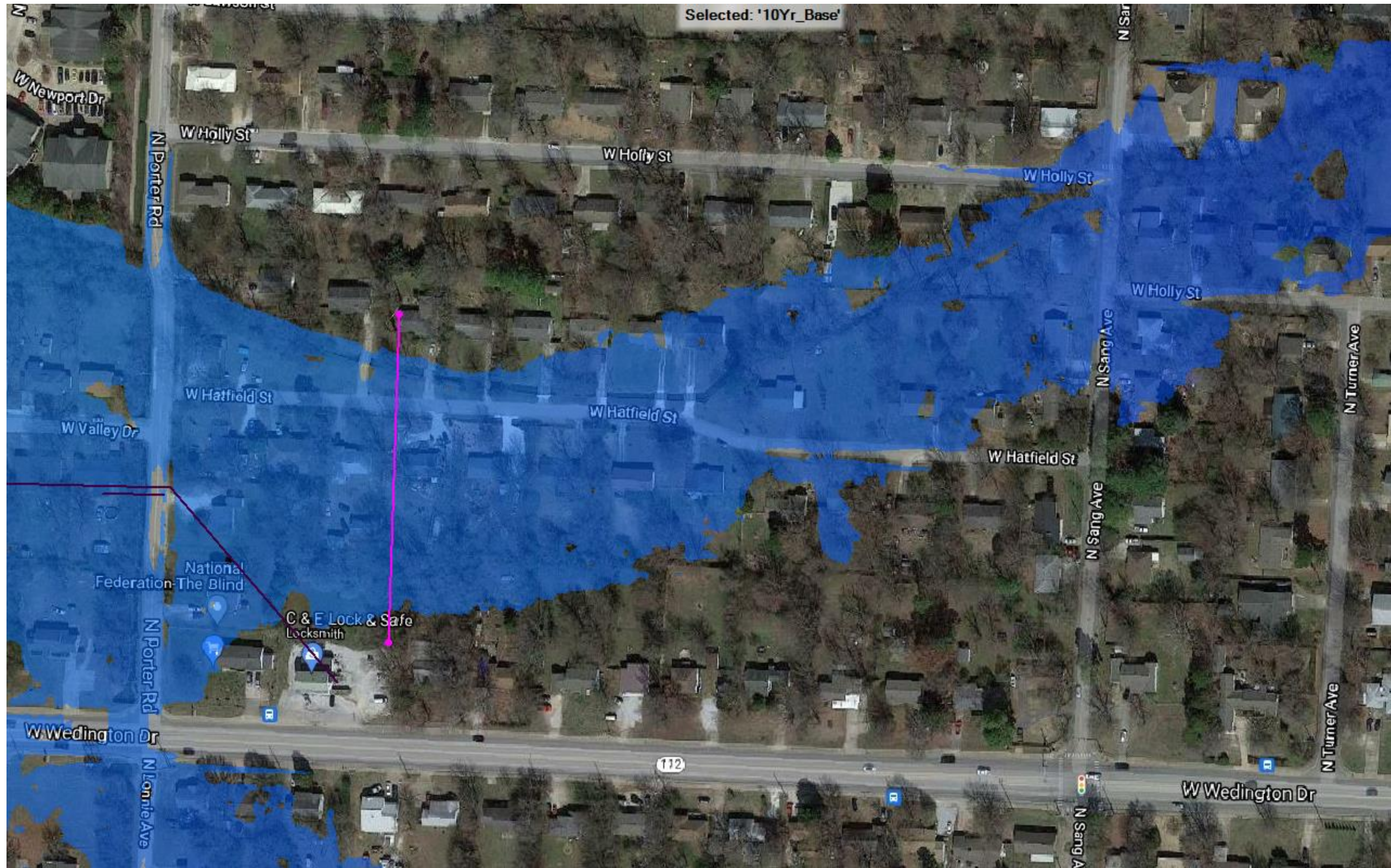


Figure 19. Existing Conditions – 10-Year Hatfield Street Improvements.

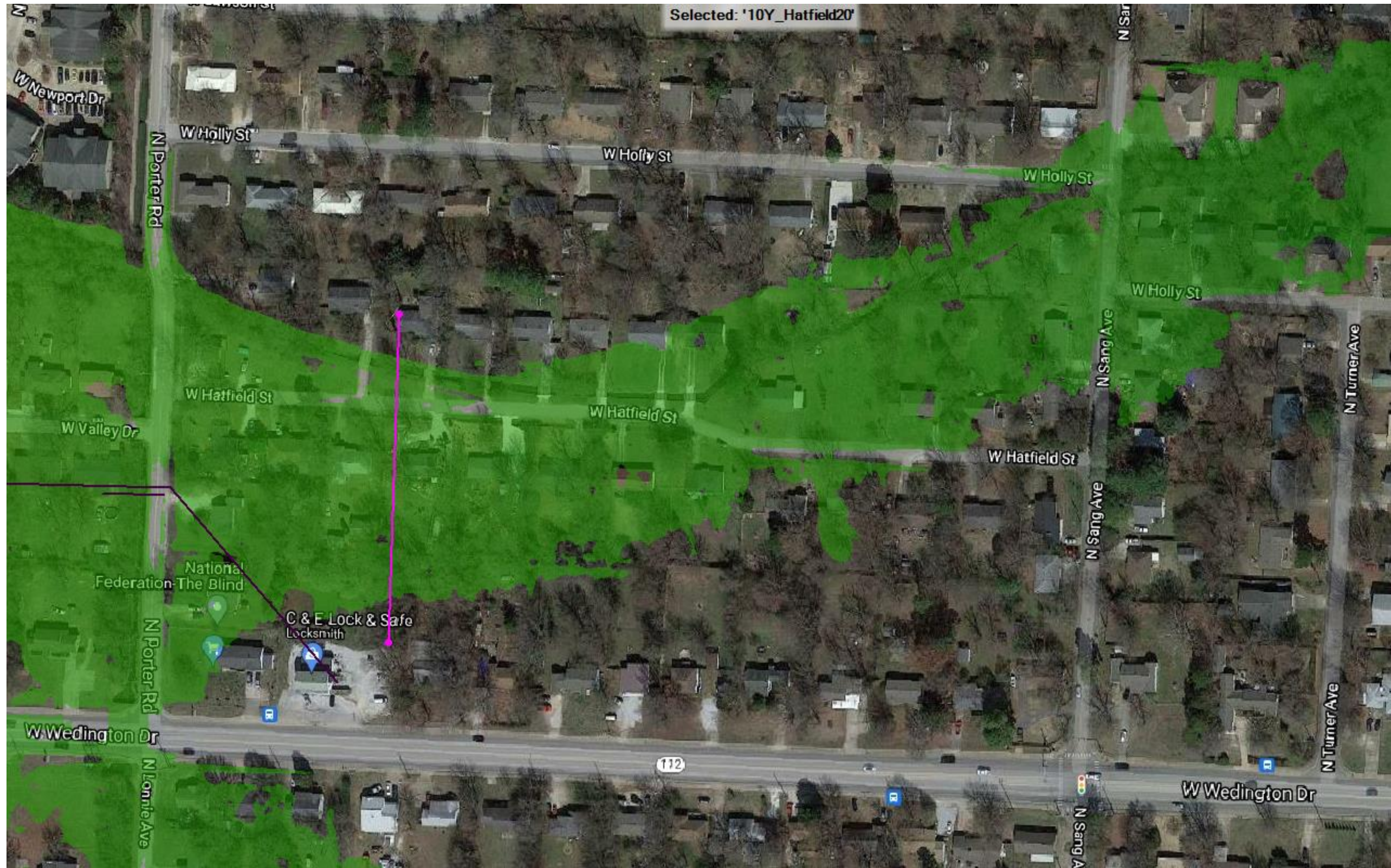


Figure 20. 20-ft Channel – 10-Year Hatfield Street Improvements.

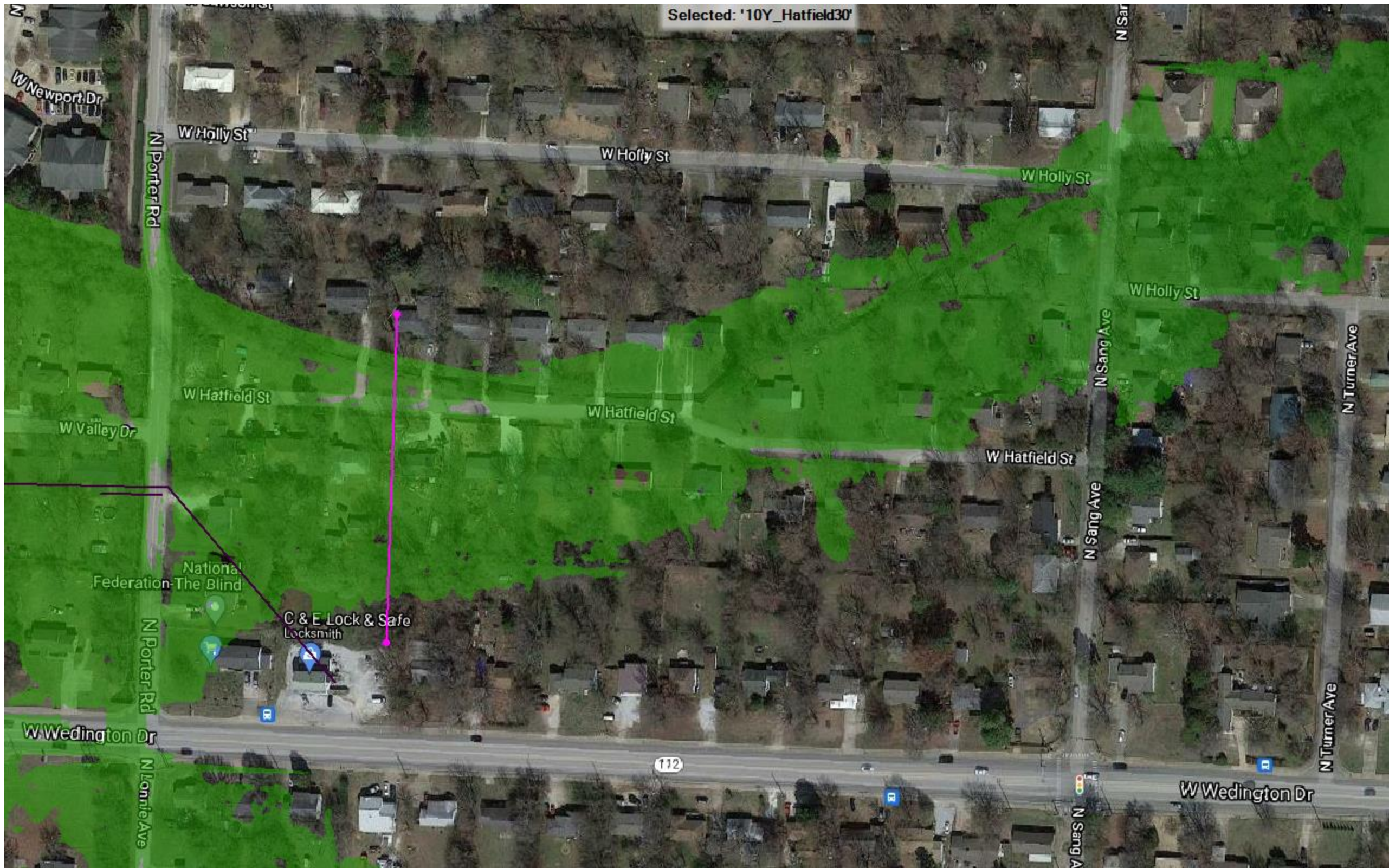


Figure 21. 30-ft Channel – 10-Year Hatfield Street Improvements.

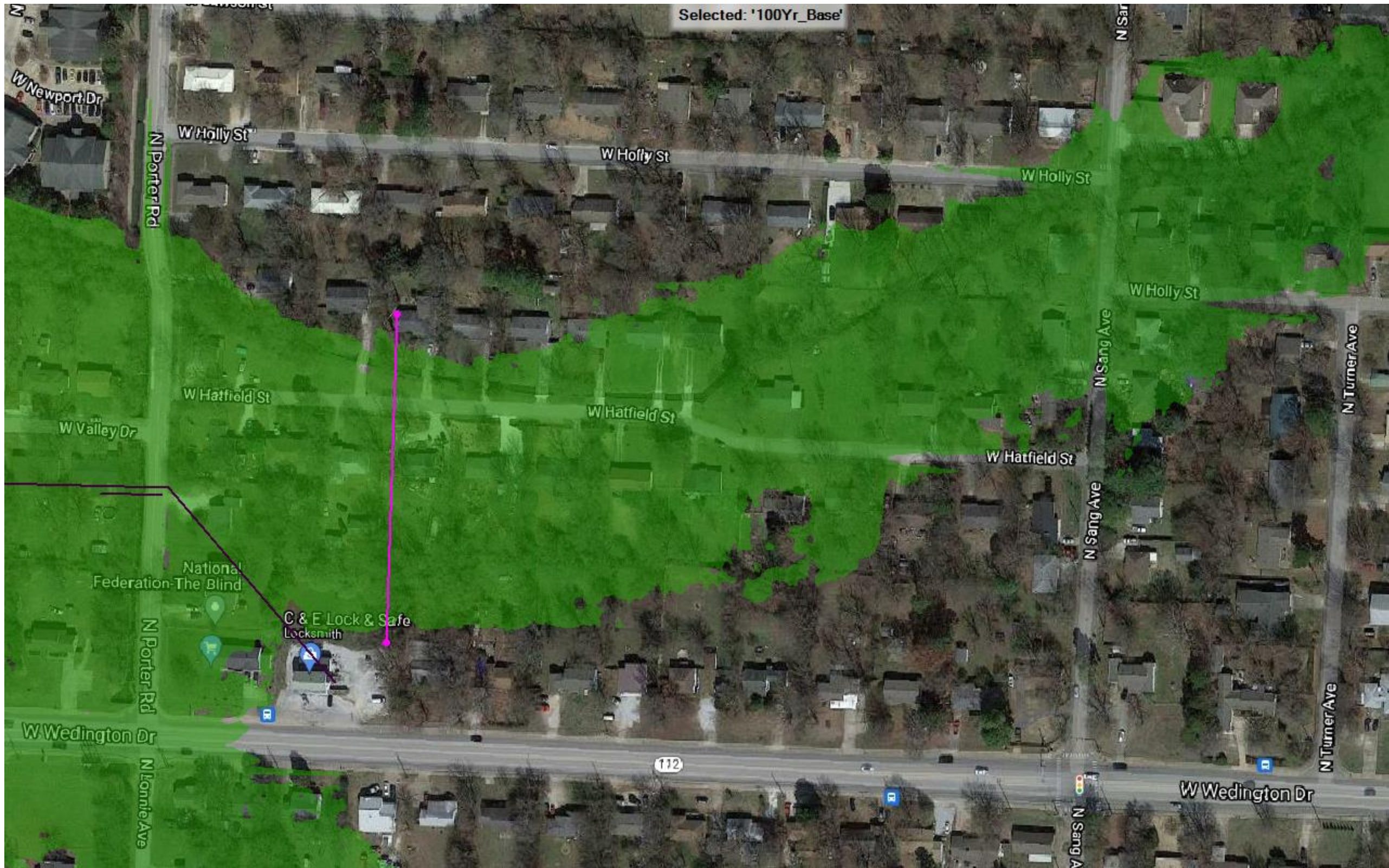


Figure 22. Existing Conditions – 100-Year Hatfield Street Improvements.



Figure 23. 20-ft Channel – 100-Year Hatfield Street Improvements.



Figure 24. 30-ft Channel – 100-Year Hatfield Street Improvements.

APPENDIX E

Figures 25-28 South Fork Hamestring Creek Channel Improvements

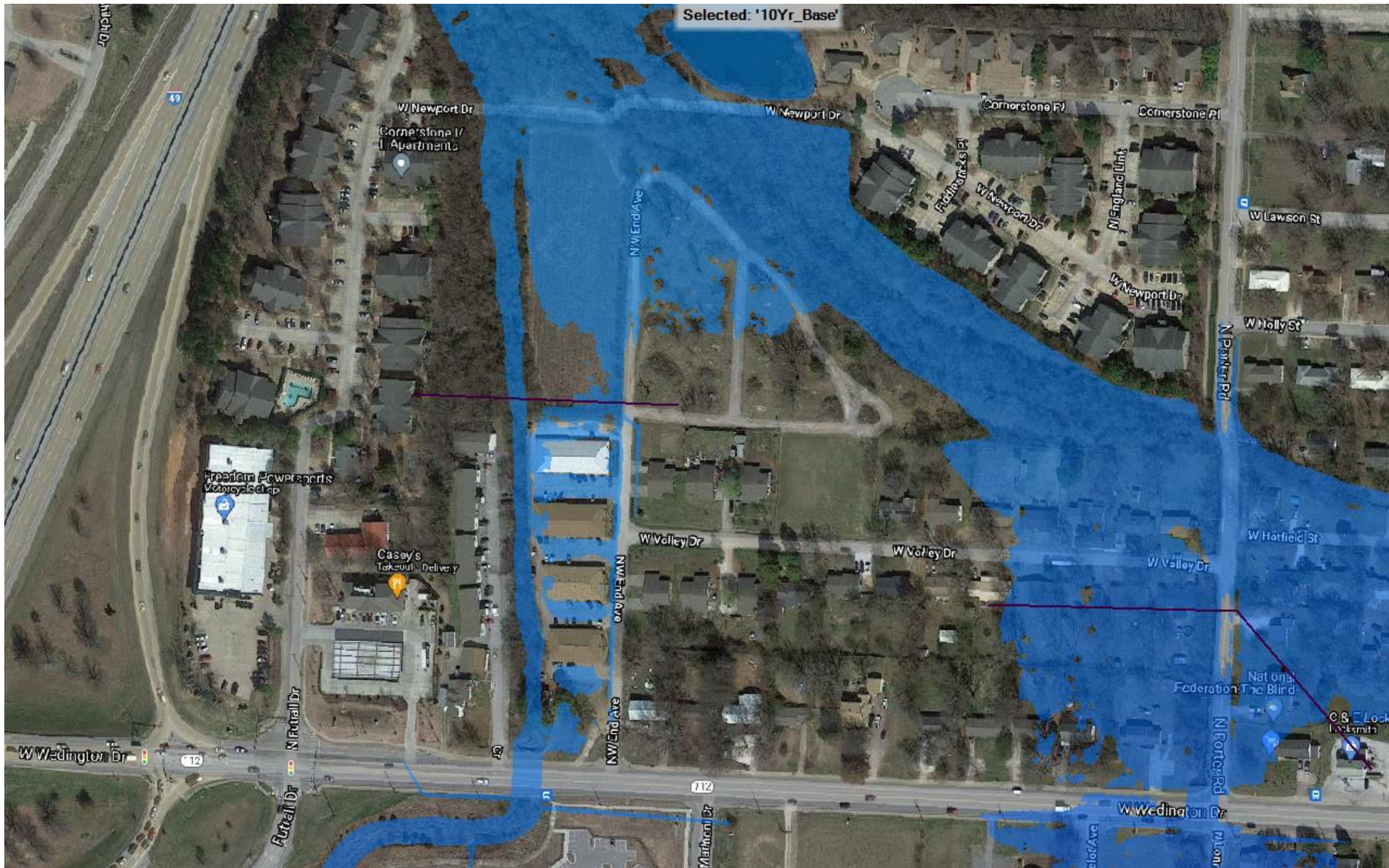


Figure 25. Existing Conditions – 10-Year SF Hamstrung Creek Improvements.

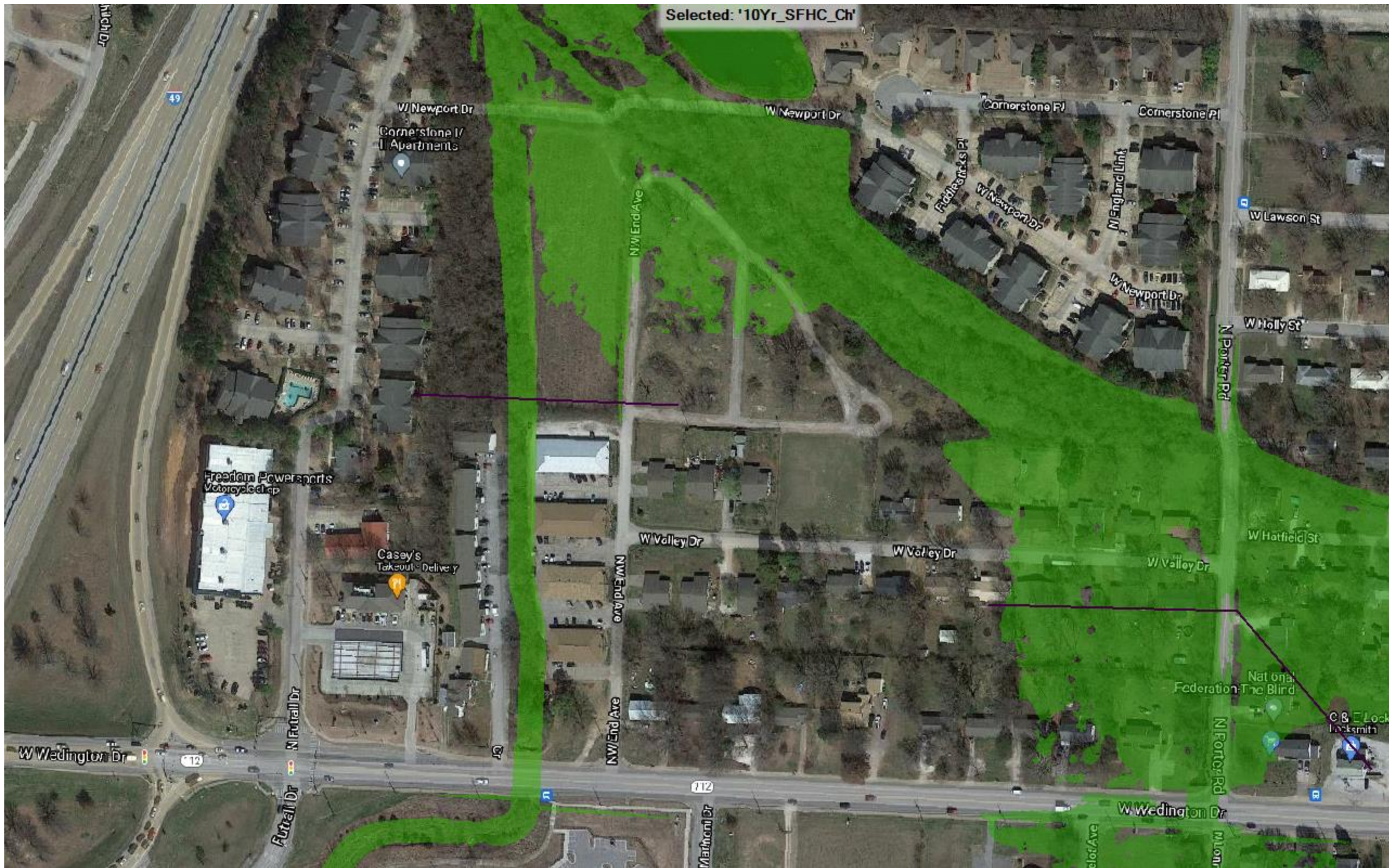


Figure 26. 30-ft Trapezoidal Channel – 10-Year SF Hamstring Creek Improvements.

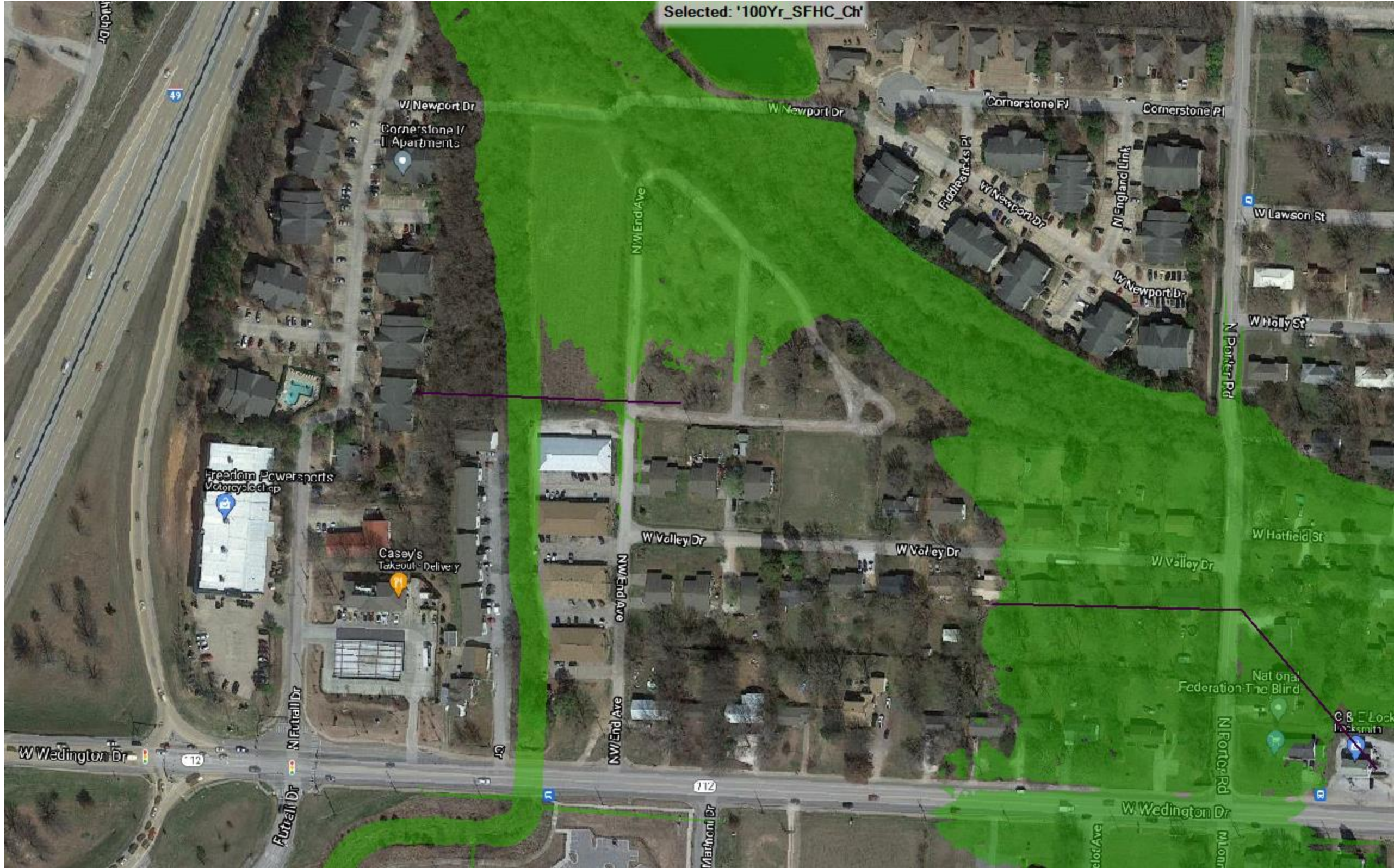


Figure 28. 30-ft Trapezoidal Channel – 100-Year SF Hamestrung Creek Improvements.