Jimmy Camp Creek Drainage Basin Planning Study

Report Draft

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1.0 INTRODUCTION

1.1 Authorization

El Paso County (County), Colorado authorized Stantec Consulting Services Inc. (Stantec), in cooperation with HDR and THK Associates, to conduct the Jimmy Camp Creek Drainage Basin Planning Study under Contract 17-067-61. The performance location for this contract is the Jimmy Camp Creek Drainage Basin watershed which spans unincorporated portions of El Paso County, eastern portions of the City of Colorado Springs, and northern portions of the City of Fountain.

1.2 Purpose and Scope

The Jimmy Camp Creek Drainage Basin (Basin) is a largely undeveloped area located in the eastern portion of the Fountain Creek Watershed. Due to its large size, the development potential in the Basin (including the major development of Lorson Ranch East), and the need for a responsible drainage fee structure, the County required an updated Jimmy Camp Creek Drainage Basin Planning Study (DBPS) to help guide decision making within the drainage basin to address current problems and future development.

Stormwater management is a critical issue that requires prior planning to successfully manage growth in the County. This management is needed to mitigate the impacts of increased stormwater runoff from increased impervious surfaces, which affects the development of the community, the existing storm drainage infrastructure, and receiving channels. The most equitable way to proactively address this issue is to prevent future runoff problems and maintain consistency between infrastructure costs and benefits. The purpose of the Jimmy Camp Creek DBPS is to provide the framework for future stormwater planning and design studies in the Jimmy Camp Creek Drainage Basin within the County.

The main objectives of this DBPS are to analyze the existing and future drainage conditions of the watershed, identify corrective and future capacity improvements, and to establish Drainage and Bridge Fees. This study includes a description of the study process, basin background information, technical analysis and documentation, the proposed plan, and proposed fees. The information developed from this study, upon adoption by the County, will be used to mitigate stormwater impacts to the major drainageways within the watershed.

This DBPS is a comprehensive update of the unincorporated County area portions of the Jimmy Camp Creek DBPS published in 2015 (Kiowa Eng). The 2015 study is based on the drainage basin planning criteria in the Colorado Springs Drainage Criteria Manual (COS, 2014).

Specific phases for the updated study include the following:

Phase 1: Stakeholder Involvement and Public Collaboration Plan

a. Stakeholder Plan
b. Stakeholder Engagement Meetings

Phase 2: Problem Identification/Existing and Future Conditions

c. Basin Technical Information Gathering
d. Basin Characteristics Review
e. Hydrologic Model Development

f. Hydraulic Model Development

Phase 3: Alternative Development, Evaluation and Selection

g. Evaluation Criteria Development
h. Alternatives Development
i. Conceptual Cost Estimates
j. Alternatives Screening and Selection

Phase 4: Plan Development

k. Alternative Conceptual Design Development
l. Drainage Basin Study Report

Phase 5: Fee Development

Phase 6: Plan and Fee Adoption

1.3 Previous Studies – Related Investigations

Several drainage plans have been previously completed for the Jimmy Camp Creek Drainage Basin. This includes the West Fork Jimmy Camp Creek DBPS prepared for New Generation Homes, Inc in October 2003 (Kiowa Eng) and the updated Jimmy Camp Creek Drainage Basin Planning Study prepared in 2015 (Kiowa Eng). Certain information from these previous studies is superseded by the information in this updated DBPS.

This DBPS was based on available information from previous studies, Master Development Drainage Plans (MDDPs), Plat information, as well as other DBPSs. The following is a list of maps, plans, criteria, manuals, and reports which were reviewed while preparing this study:

- City of Colorado Springs Drainage Criteria Manual, Volume 1, March 2014, Revised January 2021.
- City of Colorado Springs Drainage Criteria Manual, Volume 2, prepared by Matrix Design Group/ Wright Water Engineers. March 2014, Revised December 2020.
- City of Fountain Comprehensive Development Plan (Update), prepared by the City of Fountain, August 2005.
- Corral Bluffs Annexation Filing No. 1, prepared by Matrix Design Group, June 17,2021.
- Draft El Paso Master Plan, prepared by Houseal Lavigne. April 23, 2021.
- El Paso County Parks Master Plan, prepared by El Paso County, June 2013.
- El Paso County, Drainage Criteria Manual. Colorado Springs: El Paso County, 1991.
- FEMA Flood Insurance Study El Paso County, Colorado and Incorporated Areas. Revised Dec 2018.
- Fountain Creek Corridor WARSSS Study, prepared by Matrix Design Group, March 2017.
- Fountain Creek Corridor Restoration Master Plan, prepared by LHK Associates and Matrix Design Group. October 2011.
- Hydrologic Soil Groups of Jimmy Camp Creek Basin, NRCS Web Soil Survey. May 2021.
- Jimmy Camp Creek Annexation Filing No. 1, prepared by Matrix Design Group. June 18, 2021.
- Jimmy Camp Creek Drainage Basin Planning Study, prepared by Kiowa Engineering, March 2015.

- Lorson Ranch East PUD Development, Preliminary Plan and Early Grading Request. Prepared by Thomas-Thomas, Revised July 2017.
- West Fork Jimmy Camp Creek Drainage Basin Planning Study, prepared by Kiowa Engineering Corporation for New Generation Homes, Inc., Oct 2003.

1.4 Summary of Obtained Data

Data used to complete the analysis for this DBPS includes: topography, aerial photography, soils, land use, stormwater infrastructure, rainfall, field survey, and U.S. Geological Survey (USGS) gage data. Most of the data was collected and utilized in a Geographic Information System (GIS) format. Existing data was used to the extent practical, including previous Jimmy Camp Creek DBPS, FEMA floodplain mapping, CDOT Bridge Data, and development submittals, etc. Table 1-1 lists the major data obtained along with the source and date received.

Table 1-1: Major Data Obtained, Data Sources, and Date Received

Data Obtained	Data Source	Date Received
Aerial Imagery	El Paso County	04/2021
2011 Topographic Contours	El Paso County	04/2021
LiDAR Data	State of Colorado (2018)	03/2021
Waterlines / Wetlands	El Paso County	04/2021
Regulatory Floodplains	El Paso County	04/2021
DOT Major Highways	El Paso County	03/2021
CDOT Structures	Colorado Department of Transportation	03/2021
Municipal Boundaries	El Paso County	03/2021
Major & Local Roads	Colorado Department of Transportation	04/2021
Future Land Use	El Paso County	07/2021
Existing Land Use	El Paso County / The 2016 National Land Cover Database (NLCD)	07/2021
Parcels	El Paso County	04/2021
Impervious Percentages	El Paso County	07/2021
Soils Data	United States Department of Agriculture (USDA) Soil Survey Geographic (SSURGO) Soil Data	04/2021
Rainfall Data	NOAA Atlas 14 Volume 8 Version 2, Precipitation-Frequency Atlas of the United States	03/2021
Stream Gage Data	United States Geological Survey (USGS) gage 07105900 Jimmy Camp Creek at Fountain, CO	03/2021
County ROW	El Paso County DPW Requested Data	04/2021
Storm Drain Information	El Paso County DPW Requested Data	04/2021

1.5 Project Coordination

Throughout the course of preparing this DBPS, project checkpoints were set up that required County concurrence before moving on to the next tasks to help manage the schedule and avoid re-work. The primary reasons for the coordination effort are to obtain technical information, confirm approaches to technical methods, and to identify concerns regarding the development of stormwater facilities within the Basin.

Approximately 55 percent of the Jimmy Camp Creek Drainage Basin lies within the incorporated cities of Fountain and Colorado Springs (Cities). The Cities did not participate in development of this DBPS. Hydraulic analyses were performed only for the portions of the drainage basin that lie in the unincorporated portions of the County. Drainage policies, plans, and drainage fees for the Cities' portions of the Jimmy Camp Creek Drainage Basin will need to be developed separately by the Cities.

The DBPS, along with all technical data and findings, was executed and completed in accordance with applicable County, State, and Federal regulations, criteria, and policies with the intent and goals described herein. Analyses of hydrology, hydraulics, and existing and proposed drainage structures were conducted in accordance with the current County Drainage Criteria Manual (1991).

The completed study will be presented at a meeting of the City of Colorado Springs/El Paso County Drainage Board, which acts as an advisory board to the City Council and the Board of County Commissioners.

1.6 Stakeholder Involvement

To promote understanding of the DBPS process and Basin Fee Development and establish a publicly acceptable drainage plan, stakeholder involvement was integrated throughout the DBPS development. The process used to develop a DBPS provided opportunities for interested parties to offer input on drainage issues, needs, and facilities within a study area. A list of basin Stakeholders was provided by the County and represented local governments, developers, neighborhood associations, non-profits, environmental groups, the Fountain Creek Watershed District, and others. A publicly available website was utilized to disseminate information to the constituents at key points in the planning process. Two meetings are planned with basin stakeholders at the following key project milestones.

The first stakeholder meeting was conducted to introduce the planning study scope and process and present the Phase 2 results and Phase 3 evaluation of preferred alternatives. The objectives of the meeting were to solicit information about the drainage conditions in the basin, identify issues to be considered and discuss possible solutions, and receive input for a selected alternative to be used for the proposed plan.

The second stakeholder meeting will be conducted to present the proposed basin plan and costs developed in Phase 4 and Phase 5. The objectives of the meeting are to receive Stakeholder input on the proposed basin plan and costs and discuss the fee calculation method and proposed fees.

1.7 Acknowledgements

During the preparation of this DBPS, government agencies and interested individuals were involved in coordination activities on an as needed basis. Representatives from the County provided valuable data resources and commentary during completion of the study. A list of the individuals and agencies involved during the preparation of this DBPS is presented below:

Name	Agency
Jennifer Irvine	El Paso County – Dept. of Public Works
Steve Jacobsen	El Paso County – Dept. of Public Works
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Joshua Palmer	El Paso County – Dept. of Public Works
Christina Prete	El Paso County – Dept. of Public Works
Glenn Reese	El Paso County – Dept. of Public Works
Jeff Rice	El Paso County – Dept. of Public Works
Nina Ruiz	El Paso County – Planning and Community Development
Jason Meyer	El Paso County – Community Services Department - Parks
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Adam Copper	City of Colorado Springs Stormwater Enterprise
Richard Mulledy	City of Colorado Springs Stormwater Enterprise
Jeff Besse	City of Colorado Springs Stormwater Enterprise
Tim Biolchini	City of Colorado Springs Stormwater Enterprise
Aaron Egbert	City of Colorado Springs Public Works
Peter Wysccki	City of Colorado Springs Planning & Community Development
Meggan Herington	City of Colorado Springs Planning & Community Development
Karen Palus	City of Colorado Springs Parks & Recreation
Kristy Martinez	City of Fountain - Planning
Brandy Williams	City of Fountain - Engineering
Silvia Huffman	City of Fountain - Parks
Kevin Binkley	Colorado Springs Utilities
Mark Shea	Colorado Springs Utilities
Tony Martinez	USACE Planning – Albuquerque District
Deric Clemons	USDA-NRCS (National Resource Conservation Service)
Keith Curtis	Regional Floodplain Administrator
Bill Banks	Fountain Creek Watershed District
Amber Shanklin	Palmer Land Conservation
Kathleen Reilly	Colorado Watershed Coordinator – Arkansas/ Rio Grande Basins
Tamara Allen	CDPHE Watershed Restoration and Protection Unit
Kevin Houck	Colorado Water Conservation Board
John Hunyadi	Colorado Dam Safety Engineer
Mitch Martin	Colorado Parks and Wildlife
David Sutley	FEMA – Region VIII

2.0 BASIN CHARACTERISTICS AND ENVIRONMENTAL RESOURCES

The information provided in this section establishes the physical setting of the Jimmy Camp Creek Drainage Basin and identifies environmental resources that were considered when developing and selecting alternatives.

2.1 Study Basin

The Jimmy Camp Creek Drainage Basin area is approximately 67.1 square miles, approximately 29.7 square miles of which lie within the unincorporated portions of the County. The watershed is generally bounded by Garrett Road to the north, Blaney Road to the east, Old Pueblo Road to the South, and Powers Boulevard to the west. The Jimmy Camp Creek Drainage Basin is part of the eastern portion of the Fountain Creek Watershed, making up approximately 7% of the watershed. Figure 2-1 shows the location of the Jimmy Camp Creek Drainage Basin within the Fountain Creek Watershed.

The Jimmy Camp Creek Drainage Basin topography generally slopes from north to south, with all flows eventually draining to Fountain Creek, with its terminus near the City of Fountain historic downtown. The Jimmy Camp Creek Drainage Basin has a maximum elevation of approximately 6,880 feet at Garrett Road in the north and a minimum elevation of approximately 5,490 feet at its confluence with Fountain Creek in the south. The average channel slope of the Jimmy Camp Creek main stem is approximately 1 percent over a length of 24 miles. Jimmy Camp Creek Drainage Basin topography and the streams considered in this DBPS are shown in Figure 2-2.

There are 9 major tributaries to Jimmy Camp Creek. Blaney, Corral, Stripmine, Franceville, East Fork, and Ohio tributaries enter Jimmy Camp from the east. Marksheffel, West Fork, and C and S tributaries enter from the west. The main stem and the tributaries considered in this DBPS are shown in Figure 2-2. Of the 9 tributaries, 3 are not included in this DBPS because they are entirely contained within the boundaries of the City of Colorado Springs. The tributaries not considered are Ohio, Marksheffel, and C and S.

Soils within the Jimmy Camp Creek Drainage Basin consist mostly of sands and loams, with the loams containing higher sand and gravel content. Jimmy Camp Creek and its tributaries carry a high sediment load, which originates from erosion of the land surface, and to a lesser extent the erosion of bed and channel banks.

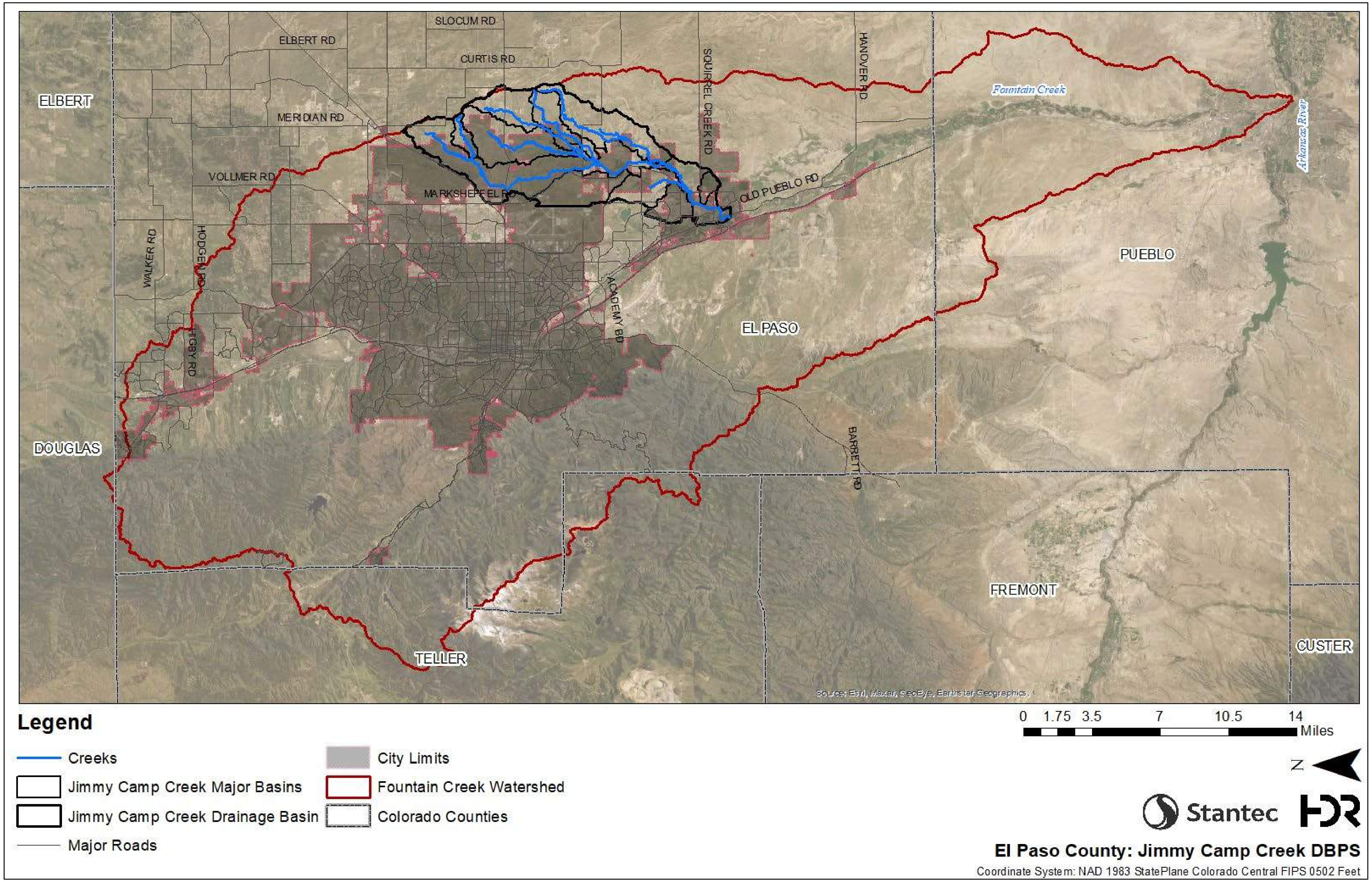


Figure 2-1. Jimmy Camp Creek Vicinity Map

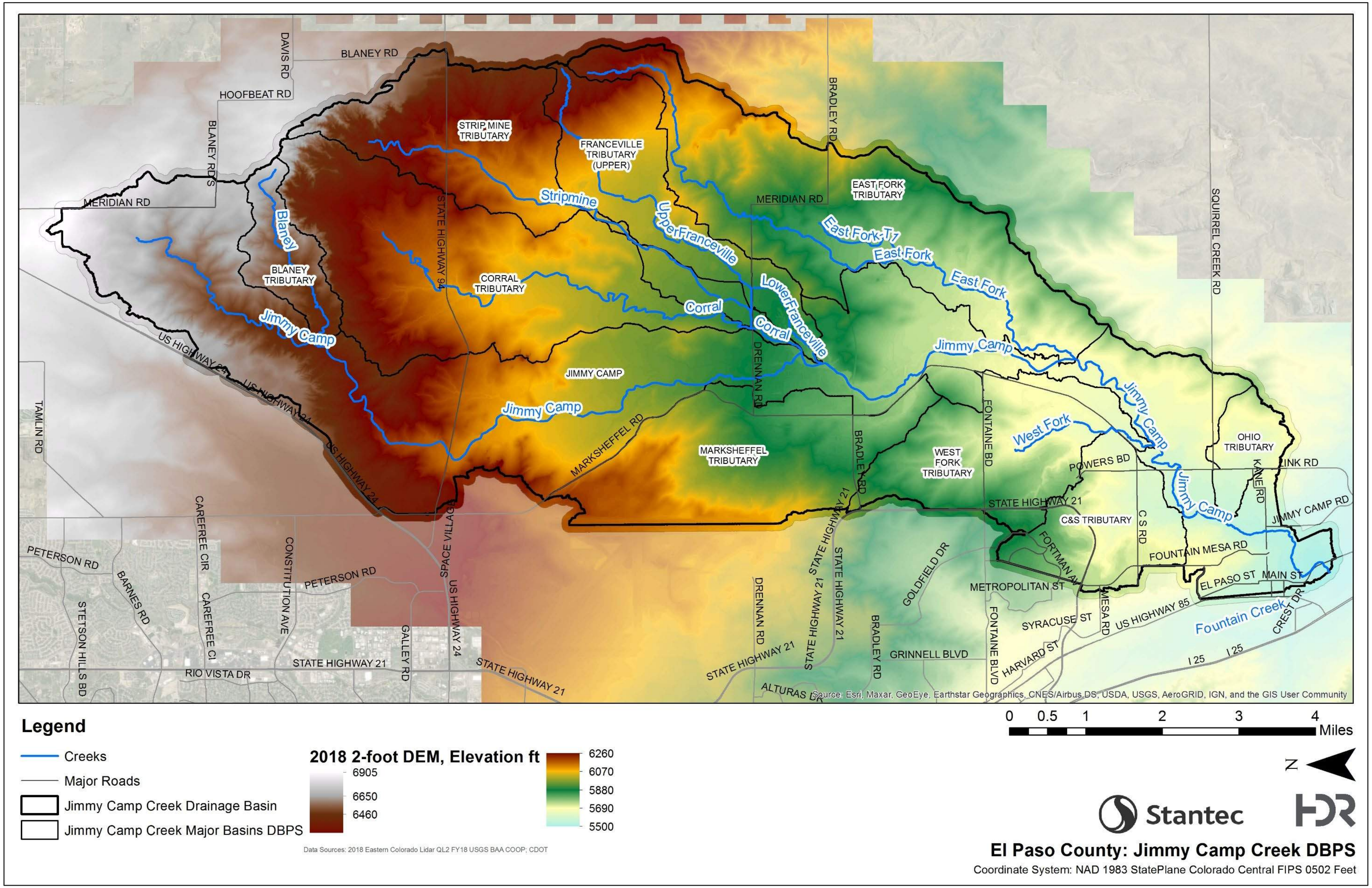


Figure 2-2. Jimmy Camp Creek Drainage Basin Topography

2.2 Climate

This area of El Paso County can be described in general as high plains, with total precipitation amounts typical of a semi-arid region. Winters are generally cold and dry. The annual average precipitation ranges from approximately from 14 to 16 inches per year, with the majority of this precipitation occurring in spring and summer in the form of rainfall. Thunderstorms are common during the summer months and are typified by quick-moving low-pressure cells which draw moisture from the Gulf of Mexico into the region. Average temperatures range from about 30°F in the winter to 75°F in the summer. The relative humidity ranges from about 25 percent in the summer to 45 percent in the winter.

2.3 Geology and Soils

Soils within the Basin vary between hydrologic soil types A through D, as identified by the U.S. Department of Agriculture, Natural Resources Conservation Service. The predominant hydraulic soil group is Type B (53% of the basin) followed by Type A (21% of the basin). Type A and B soils give this basin a lower runoff per unit area as compared to basins with soils dominated by Types C and D. The soils consist of deep, well drained soils that formed in alluvium and residium, derived from sedimentary rock. The Basin soils have high to moderate infiltration rates and are extremely susceptible to wind and water erosion where poor vegetation cover exists. The Hydrologic Soil Distribution Map for the Jimmy Camp Creek Drainage Basin is presented in Section 3.11, Figure 3-3.

2.4 Land Use

The land use information is presented to provide an understanding of the current and future development condition in the watershed. The identification of land uses abutting the drainageways is also useful in the identification of feasible plans for stabilization and aesthetic treatment of the creek. The land use in the basin consists of managed lands, suburban development, large lots or ranchettes, and rural residential (Houseal Lavigne Associates, 2021). Existing land use type is given in Figure 2-3. Most of the watershed, particularly in the upper portions, is currently undeveloped. Existing land use in developed areas consists of primarily mixed-use urban development. Lorson Ranch is the largest developed area in the basin with over 2,000 homes constructed and a planned development for over 4,000 homes, a school, and commercial areas (Lorson Ranch, 2021). Other developed areas include portions of Colorado Springs Municipal Airport, the RAM Off-Road Park and Aztec Family Raceway, Pikes Peak National Cemetery, and a portion of Peterson Air Force Base.

Future Land use is shown in Figure 2-4. The Lorson Ranch master development area is in the central portion of the drainage basin and future land use was assumed to be built out according to the expected level of development for that area. The basin encompasses part of the Colorado Springs Airport/ Peterson Air Force Base key area identified in the Draft El Paso County Master Plan (2021). This key area is primed for commercial and industrial development, in part due to the establishment of the Commercial Aeronautical Zone (CAZ), which the Board of County Commissioners approved to attract local businesses and spur development on the available land (Houseal Lavigne Associates, 2021).

Additionally, the City of Colorado Springs is planning, as of the date of this report, on annexing and rezoning two properties in the Jimmy Camp Creek basin to parkland. The first property (Jimmy Camp Creek

Annexation Filing No.1) consists of 410.3 acres located between the Blaney Rd S. and Meridian Rd. intersection and Jimmy Camp Creek in the upper watershed. The second property (Corral Bluffs Annexation Filing No. 1) consists of 920.4 acres located north of the Aztec Family Raceway in the northeast corner of the basin. These locations are shown as Open Space in the Future Land Use Map discussed below.

The County provided existing and future land uses, which are shown in Figure 2-3 and Figure 2-4. Each land use has a corresponding impervious percentage, which is used in the hydrologic analysis to predict runoff rates and volumes for the purposes of facility evaluation. Data for existing and future impervious area are presented in the Hydrology Section 3.2.

Property ownership along the major drainageways within the unincorporated areas of Jimmy Camp Creek Drainage Basin are mostly private. Along the developed reaches, drainage right-of-ways and greenbelts are maintained by the metro districts, mainly the Lorson Ranch Metropolitan District, Glen Metropolitan District, and Colorado Centre Metro District. Where development has not occurred, the drainageways generally remain under private ownership with no delineated drainage right-of-ways or easements.

There are several public parks and open spaces in the Jimmy Camp Creek Drainage Basin. In the El Paso County Master Park Plan (2013) there is a proposed 21-mile primary regional trail beginning at the confluence of Jimmy Camp Creek and Fountain Creek and continuing northeast, along Jimmy Camp Creek, until reaching the City of Colorado Springs. This trail is planned to connect to the City of Fountain's Adams Open Space, proposed Corral Bluffs Open Space, and the City of Colorado Springs's proposed Jimmy Camp Creek Open Space.

There are multiple proposed Open Spaces that would be located fully or partially in the Jimmy Camp Creek Drainage Basin. Falcon Garrett Roads Open Space would occupy the broad northeast trending ridge that separates upper Jimmy Camp Creek from the East Fork Sand Creek in the northeast headwaters of the Drainage Basin. Corral Bluffs Open Space would be connected to the southeast of Falcon Garrett Road and would provide an opportunity for a regional trail alignment linking Fountain Creek with Colorado Spring's proposed Jimmy Camp Creek Park. The proposed Fountain and Jimmy Camp Creek Open Space would protect the floodplains of both creeks and the nearby wildlife, including the globally-vulnerable Arkansas Darters that live in the spring-fed marshes adjacent to the main creek channels (EPC, 2013).

The Future Land Use industrial areas near Drennan Road and Bradley Road are part of the Highway 21 Employment Priority Development Area. This area includes the County Commercial Aeronautical Zone (CAZ) intended to attract local businesses and encourage economic opportunities tied to the Colorado Springs Airport (EPC, 2021).

Roadway and utility easements abut or cross drainageways. Primary districts operating in the unincorporated portions of the basin include Colorado Springs Utilities, Widefield Water and Sanitation District, Southern Colorado Water Conservancy, and Mountain View Electric. In general, utility and roadway crossings occur most frequently in the developed portions of the basin.

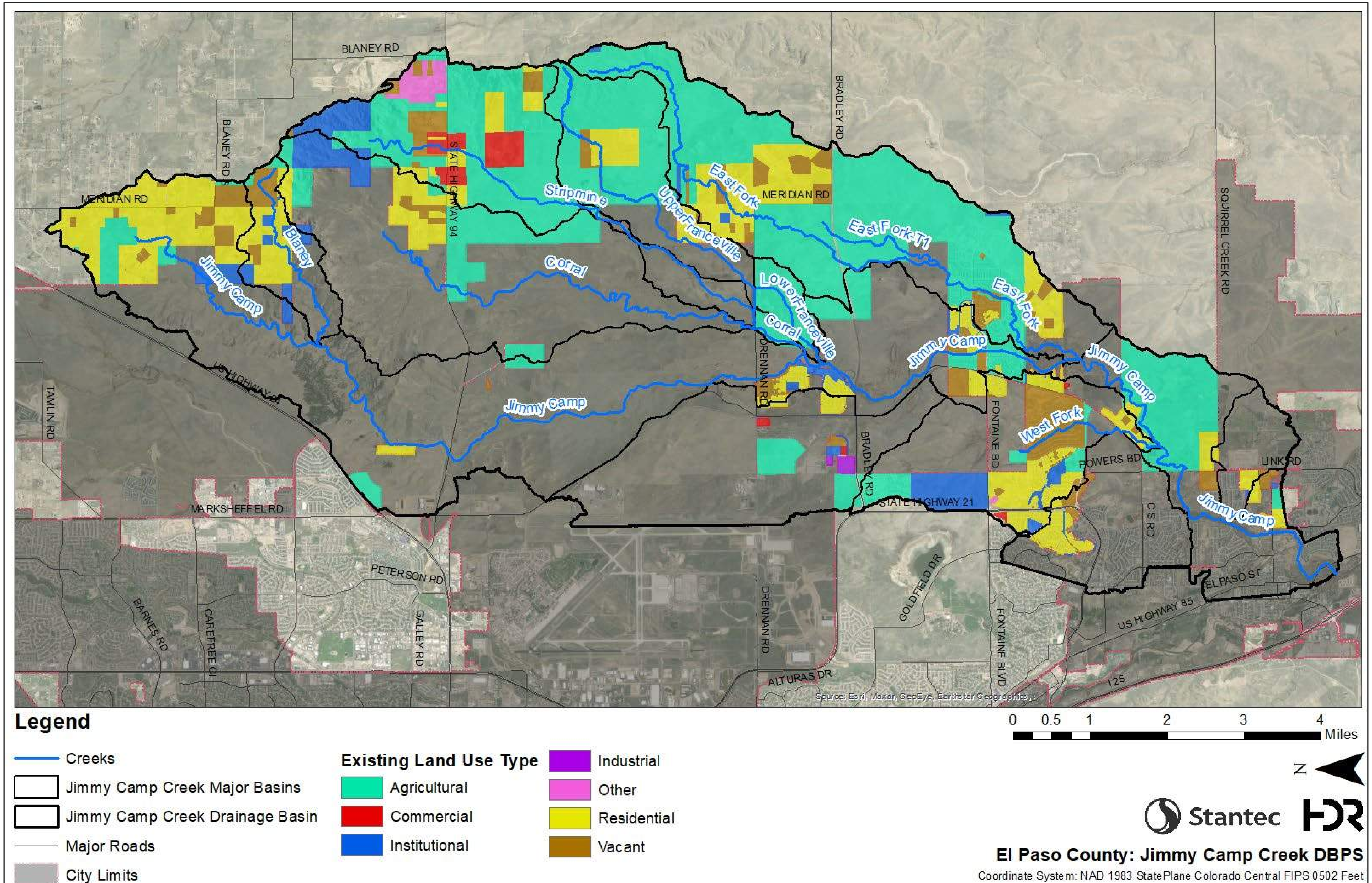


Figure 2-3. Jimmy Camp Creek Existing Land Use Map

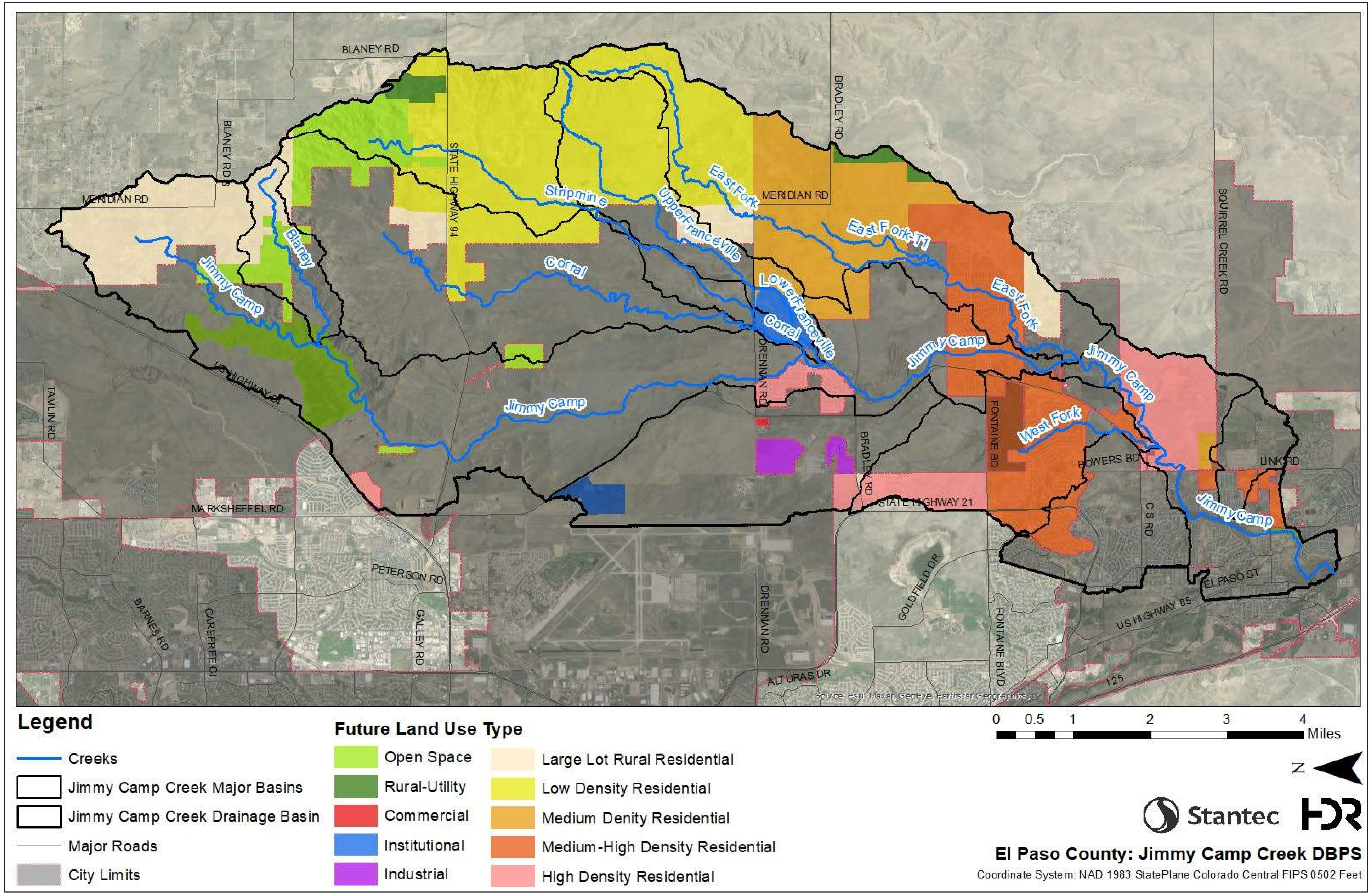


Figure 2-4. Jimmy Camp Creek Future Land Use Map

2.5 Environmental Resources

This section includes an environmental resource inventory for the drainageways in the Jimmy Camp Creek Drainage Basin, including a description of the endangered species issues, wildlife habitats, and wetland resources that may be important to consider during design and implementation of major outfall systems.

U.S. Geological Survey (USGS) topographic, Natural Resources Conservation Service (NRCS) soil survey, National Wetland Inventory (NWI) wetland maps, and the U.S. Fish and Wildlife Service (USFWS) Information for Planning and Consultation (IPaC) website were used to indicate potential resources. Aerial imagery of the Basin was also used for analysis.

2.5.1 Stream Characteristics

The Rosgen Stream Classification system is used to categorize river morphology. The advantage of the Rosgen Level I Stream Classification method is that it allows for a quick initial delineation of stream types and illustrates the distribution of these types that could be encountered within a study area. The lower sections of Jimmy Camp Creek, portions of lower East Fork Jimmy Camp Creek, and the entirety of the West Fork Jimmy Camp Creek are difficult to define using the Rosgen system because of the high level of development in the area. This development alters the river's natural system. The lower mainstem of Jimmy Camp Creek has a narrow, incised channel with a wide, highly vegetated floodplain. Due to the presence of an active floodplain, the presence of predominately hydric soils, and NWI mapped emergent wetlands in this area, there is a potential for wetland areas to occur outside the stream channel.

Jimmy Camp Creek, the Stripmine Tributary, the Corral Tributary and the northern portion of the Franceville Tributary could loosely be defined as a "C" type stream. Type "C" stream channels are located in narrow to wide valleys constructed from alluvial deposition. This stream type has well-developed, slightly entrenched floodplain and are relatively sinuous.

North of Bradley Road, Jimmy Camp Creek has a wide, shallow, and sandy stream channel connected to the surrounding floodplain. The Stripmine Tributary also has a wide, shallow, sandy stream bottom with a developed floodplain. The Franceville Tributary does not have any channelization north of Drennan Road; however, the channel becomes more defined as it travels north at South Franceville Coal Mine Road. Referencing aerial imagery, this channel has a wide, sandy stream channel. The Corral Tributary has a wide, shallow, and sandy stream channel with associated floodplain. This tributary also has a short erosion control barrier, consisting of large rocks and a retaining wall north of State Highway 94. Each of these areas could be conducive to wetland formation, due to the elevation and the presence of partially hydric soils. However, the NWI does not map any emergent wetlands in these areas.

The lower portions of the East and West Forks of Jimmy Camp Creek have ill-defined, highly vegetated channels. Previous installments of valley wide grade control, dams and large detention basins within these drainages create inconsistencies with the overall characteristics of these channels throughout these reaches. As shown in Figure 2-5 and Figure 2-6, the NWI map shows several palustrine emergent wetlands following these streams. Each reach is labeled with the NWI code as described in Cowardin et al. (1979). These alpha-numeric codes correspond to the classification nomenclature that best describes a particular wetland habitat. Wetlands could be found in these areas if hydric soil, hydric vegetation, and hydrology are present. Further studies would be needed to determine the presence of wetlands in these areas.

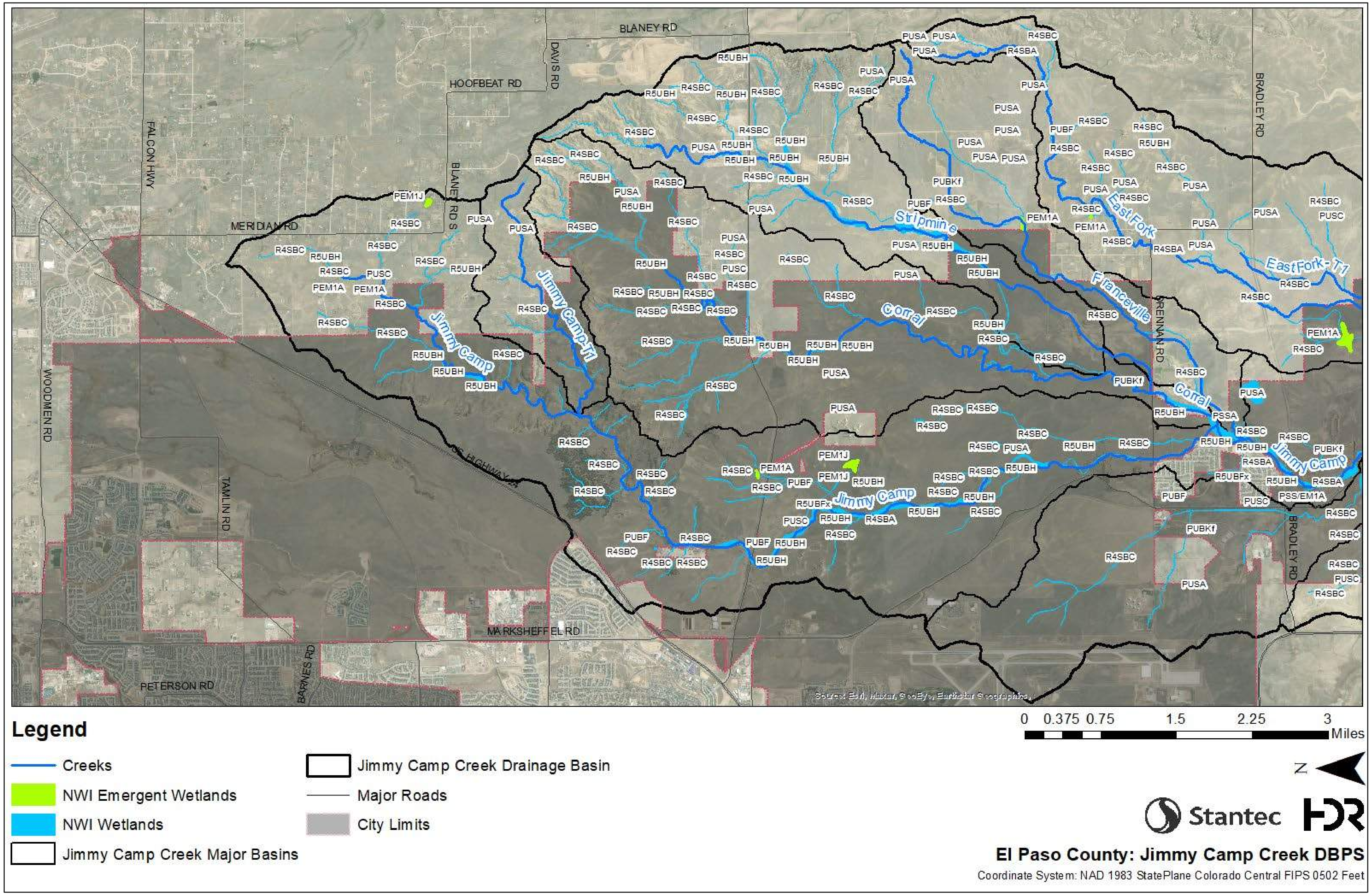


Figure 2-5. NWI Wetland Areas in Upper Jimmy Camp Creek Drainage Basin. Each reach is labeled with NWI code.

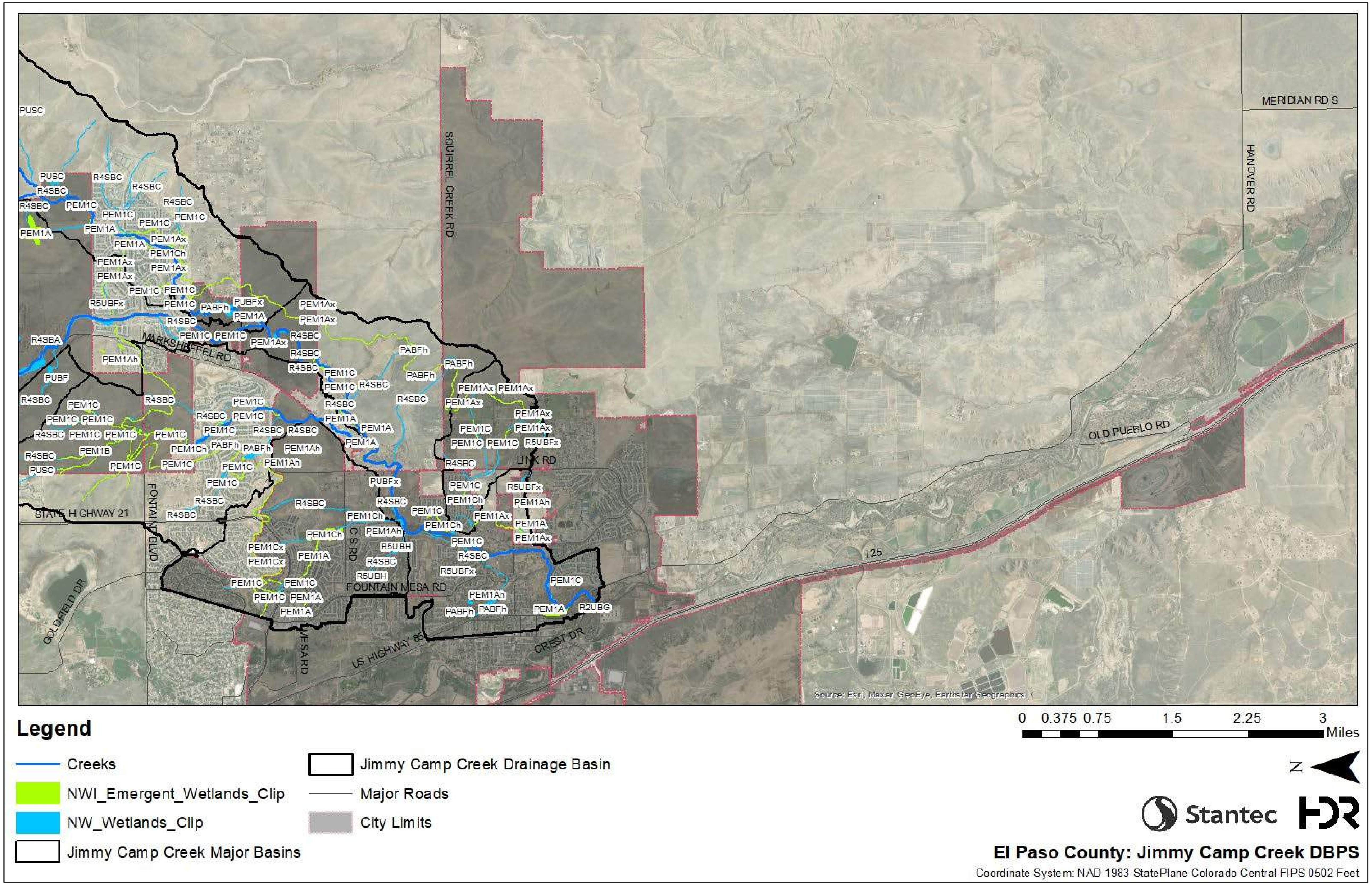


Figure 2-6. NWI Wetland Areas in Lower Jimmy Camp Creek Drainage Basin. Each reach is labeled with NWI code.

2.5.2 Geomorphic Field Assessment

The field and desktop geomorphic assessment allowed for an understanding of the sediment sources and sinks, as well as identified areas of channel and floodplain instability, providing an accurate understanding of the health and stability of the watershed given the current conditions. The geomorphic assessment methodology was derived from the Prediction Level Assessment (PLA) of the Watershed Assessment of River Stability and Supply (WARSSS) methodology (Rosgen, 2006).

A preliminary desktop analysis was conducted to identify 13 Priority Areas within the Jimmy Camp Creek Drainage Basin to be studied in greater detail. The desktop analysis involved reviewing GIS data and historical aerial imagery to identify locations of potential reference reaches and unstable reaches. Figure 2-7 shows the locations of the selected priority areas that were identified to be further investigated. The terminology of priority indicates that they were given priority to be studied. The terminology does not indicate that these reaches have a priority for improvement or are considered problem areas. The Priority Areas were selected to cover each tributary included in this DBPS and to include both potentially stable and unstable reaches.

Field walks were then conducted of the Priority Areas to identify areas of potential instability and stable reference cross sections. Locations with accessibility issues were evaluated using a modified desktop assessment (Stripmine, S1, and Corral, C2). Three Priority Areas were not assessed upon field visits as it was determined they were already improved channels and therefore were not natural reference sections. These three Priority Areas are located in West Fork and East Fork (W1, E1). Table 2-1 contains the Site IDs of the stable reference cross sections and unstable impaired reaches that were assessed, along with their corresponding Priority Area.

Bank stability and estimated erosion volumes along the impaired reaches was evaluated by conducting Bank Assessment for Non-point source Consequences of Sediment (BANCS assessment). The BANCS method utilizes two components, the Bank Erosion Hazard Index (BEHI) and Near Bank Stress (NBS). The BANCS method is discussed in more detail in Section 2.5.2.2.

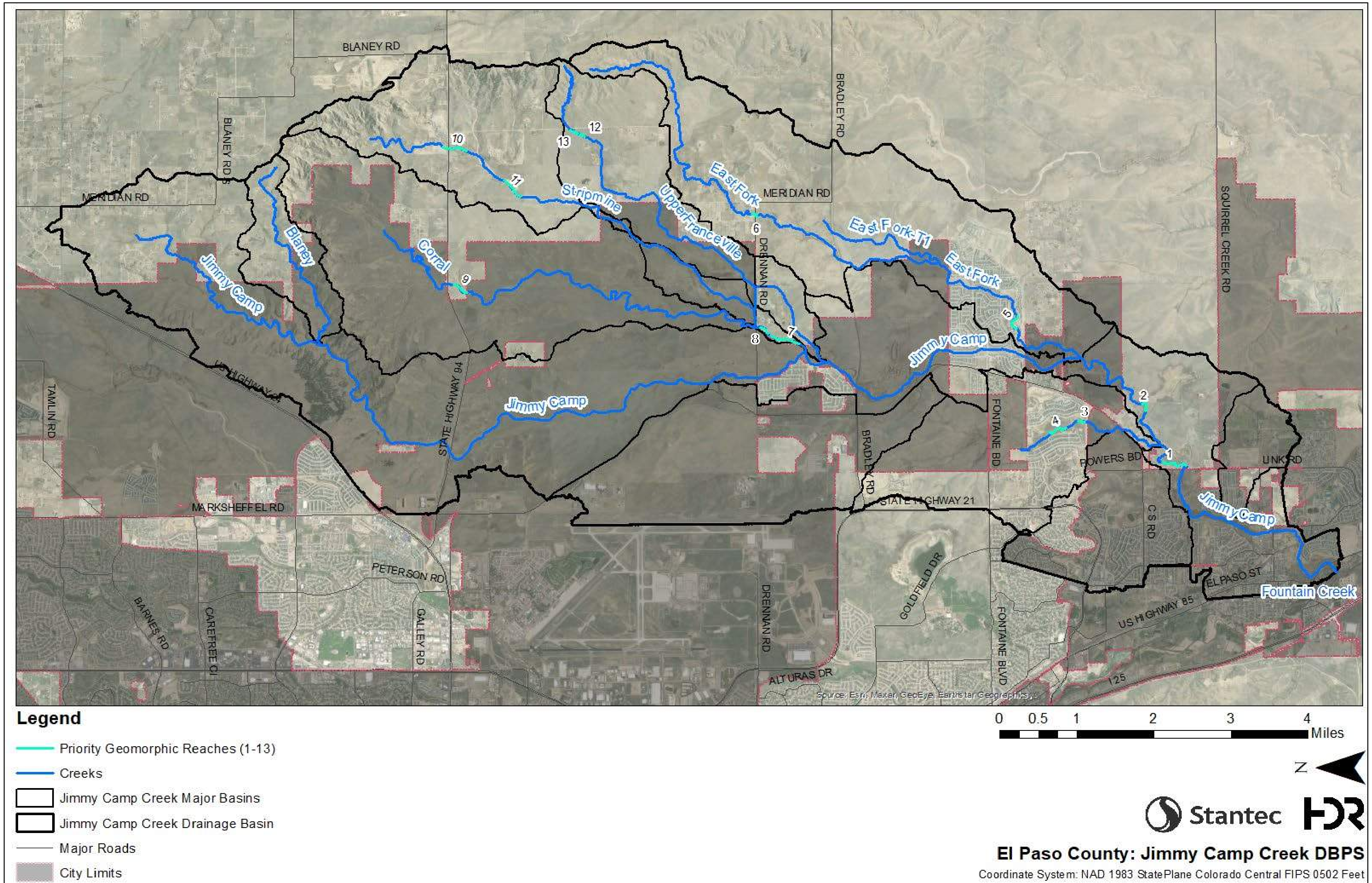


Figure 2-7. Jimmy Camp Creek Drainage Basin Geomorphic Priority Areas

Table 2-1. Summary of Geomorphic Priority Areas

Reach Name	Reach ID	Priority ID	Site ID	Stability	Assessment Type
Jimmy Camp Creek	J1	1	XS-J1-I	Impaired Reach	Field
Jimmy Camp Creek	J1	1	XS-J1-II	Stable Reference Cross Section	Field
Jimmy Camp Creek	J1	1	XS-J1-III	Stable Reference Cross Section	Field
Jimmy Camp Creek	J1	1	J1-I	Impaired Reach	Field
Jimmy Camp Creek	J2	2	J2-I	Impaired Reach	Field
Jimmy Camp Creek	J2	2	XS-J2-I	Stable Reference Cross Section	Field
West Fork Tributary	W1	3	N/A	Already Improved	Not Assessed
West Fork Tributary	W1	4	N/A	Already Improved	Not Assessed
East Fork Tributary	E1	5	N/A	Already Improved	Not Assessed
East Fork Tributary	E2	6	XS-E2-I	Stable Reference Cross Section	Field
East Fork Tributary	E2	6	XS-E2-II	Stable Reference Cross Section	Field
East Fork Tributary	E2	6	XS-E2-III	Stable Reference Cross Section	Field
East Fork Tributary	E2	6	E2-I	Impaired Reach	Field
East Fork Tributary	E2	6	E2-II	Impaired Reach	Field
East Fork Tributary	E2	6	E2-III	Impaired Reach	Field
East Fork Tributary	E2	6	E2-IV	Impaired Reach	Field
Corral Tributary	C1	7	C1-I	Impaired Reach	Field
Corral Tributary	C1	8	XS-C1-I	Stable Reference Cross Section	Field
Corral Tributary	C2	9	C2-I	Impaired Reach	Desktop
Corral Tributary	C2	9	C2-II	Impaired Reach	Desktop
Stripmine Tributary	S1	10	S1-I	Impaired Reach	Desktop
Stripmine Tributary	S1	11	S1-II	Impaired Reach	Desktop
Franceville Tributary	F1	12	XS-F1-I	Stable Reference Cross Section	Field
Franceville Tributary	F1	12	F1-I	Impaired Reach	Field
Franceville Tributary	F1	12	F1-II	Impaired Reach	Field
Franceville Tributary	F1	13	XS-F1-II	Stable Reference Cross Section	Field
Franceville Tributary	F1	13	F1-III	Impaired Reach	Field

2.5.2.1 Channel Geomorphology

A description of each of the Priority Areas that were visited in the field is given below. The reaches located in Priority Areas 9, 10, and 11 (within Reaches S1 and C2) were assessed utilizing a desktop methodology and were not assessed in the field. Priority Areas 3, 4, and 5 (within reaches E1 and W1) were not assessed as they have already been improved.

For the remaining Priority Areas (1, 2, 6, 7 ,8, 12, and 13), the channel geometry was determined. In some reaches multiple cross section measurements were taken. Because Priority Areas 7 and 8 are adjacent to each other, only one set of measurements was taken. The channel geometry of these sections is used to compare against the proposed channel cross sections used in the Alternative and Plan Development portions of the DBPS (See Section 5.4.10.2). The profile for each cross section was estimated from Light Detection and Ranging (LiDAR). Drainage areas were determined using StreamStats. StreamStats is an on-line map-based user interface developed by US Geological Survey that can be used to delineate drainage areas, get basin characteristics and estimates of flow statistics (https://www.usgs.gov/streamstats). Table 2-2 shows a summary of the reference cross section attributes.

Table 2-2. Summary of Reference Cross Section Attributes

	Franceville		East Fork			Corral	Jimmy Camp		
Priority ID	**12**	**13**	**6**	**6**	**6**	**8**	**1**	**1**	**2**
Attribute	**XS-F1-I**	**XS-F1-II**	**XS-E2-I**	**XS-E2-II**	**XS-E2-III**	**XS-C1-I**	**XS-J1-II**	**XS-J1-III**	**XS-J2-I**
Drainage Area (sq mi)	0.29	0.29	2.3	2.3	2.3	14.7	60.5	60.5	55
Bed Width (ft)	14.3	9.5	11	12.5	15	32	3	4	15
Bankfull Width (ft)	14.7	39.4	14.0	20.7	19.5	36.0	3.5	8	43.7
Maximum Bankfull Depth (ft)	0.8	1.0	2.0	1.2	3.4	0.8	1	1	0.8
Bankfull Area (sq ft)	11.2	13.5	22.9	20.7	55.4	27.2	3	6	34.6
Floodprone Width (ft)	140	58	75	56	55	96	56	112	163
Channel Slope (ft/ft)	.033	.027	.011	.009	.004	.009	.007	.005	.009

Most of the channels assessed had a well-defined active channel width, and a common feature among the stable channels is a large, well-connected floodplain. Bankfull area was less defined for the channels. For example, despite being at the end of the basin, Priority Area 1 on Jimmy Camp Creek had a relatively small channel but a large, well-connected floodplain. The channels substrate in the streams is dominated by sand and the streams have high supplies of sand that currently maintain a stable stream bed.

Priority Area 1 Description

Priority Area 1 is located on Jimmy Camp Creek in Reach J1 immediately upstream of Link Road and runs mostly parallel to Link Road for the majority of the length of the Priority Area. The bank height is not large, at approximately 4 ft, but there is evidence of an actively eroding bank. At some locations, rubble has been placed along the bank to attempt stabilization as shown in Figure 2-8. The rubble is not continuous, does not have scour protection, and is likely not effective at arresting the bank erosion. The bank is generally only protected by grasses with shallow rooting depths and is composed of erodible sand dominated soils.

In locations where the erosion is already near roadways or bridges, riprap could be used to stabilize the bank. However, large scale channel regrading and bank stabilization should not be performed as this reach contains a wide intact forested floodplain which will assist in stabilizing the stream. Large scale channel reconstruction could inadvertently destroy some of this intact forested floodplain and increase bank instability. Stabilization should only be performed to protect nearby infrastructure. A picture of a typical channel within Reach J1 is shown in Figure 2-9. Note that the main channel is quite small relative to the upstream cross sections. There are likely several reasons for this, one being that the stream is ephemeral, and flow rarely passes through this section of river, the other reasons are that the river is not incised, has ample sediment supply, and most importantly has a thick forested floodplain. The thick forested floodplain has two important features that stabilize river channels:

1. The woody vegetation extracts significant amounts of energy from the flow. Therefore, the force applied to the riverbed and soil substrate is substantially less than that in a non-forested floodplain.
2. The woody vegetation and herbaceous undercovers increase the resistance of the soil to erosion through root reinforcement and provides cover of the soil.

Removing the forested floodplain would tend to transition the channel to be more like the upstream channels that are significantly wider with more unstable banks.

Figure 2-8. Priority Area 1 on Jimmy Camp Creek Reach J1

Figure 2-9. Typical channel condition within Reach J1 on Jimmy Camp Creek

Priority Area 2 Description

Priority Area 2 is located in Jimmy Camp Creek Reach J2, directly upstream of Reach J1. The channel is similar in size and the banks are generally densely vegetated as shown in Figure 2-10 and Figure 2-11. The same general recommendations are proposed in this reach as for Reach J1.

Figure 2-10. Priority Area 2 on Jimmy Camp Creek Reach J2

Figure 2-11. Priority Area 2 on Jimmy Camp Creek Reach J2

Priority Area 6 Description

Priority Area 6 is located on East Fork Reach E2 at its crossing with Drennan Road, seen in Figures 2-12 and 2-13. This area is located much higher in the watershed where little woody vegetation exists in the Basin. The reach in this vicinity appears to have incised to some degree, but the incision is local to the bridge location. Because the incision is local to the bridge, it is most likely caused by the constriction of the bridge. There is development in the basin immediately upstream, but the development is dispersed with lot sizes of approximately 5 acres and does not appear to have significantly increased the impervious area in the drainage area.

When constructing improvements in this reach, exposed banks that are eroding should be graded back to a stable slope and sandy soil should be amended with nutrient rich topsoil to create a more suitable growing medium. The stable bank slope will be dependent upon local conditions but could be as shallow as 5H:1V to allow for vegetation to establish on the banks. Banks should then be revegetated with native woody plants and grasses that would stabilize the banks. Because of the dry climate in this area, it may be difficult to establish woody vegetation and therefore riprap may need to be used to stabilize banks near infrastructure. However, bank erosion is a natural process in streams, particularly in arid environments. We generally advise placing infrastructure sufficiently far away from eroding banks to allow this natural process and avoid costly bank stabilization and long-term maintenance.

Figure 2-12. Priority Area 6 on East Fork Reach E2

Figure 2-13. Priority Area 6 on East Fork Reach E2

Priority Area 7

Priority Area 7 is located on Corral Tributary in Reach C1 downstream of Drennan Road. Its easterly bank in this reach is almost 25 ft high (Figure 2-14). There is established vegetation along the toe of the bank, which is evidence that this bank has not moved in the last few years, but it is likely that bank erosion occurs during high flow years, as evidenced by erosion that occurred between 2011 and 2018. Based upon aerial photography from 2023, the bank appears relatively stable since 2018 as the top of bank has not moved significantly. Despite the large height of the east bank, the main channel does not appear to be incised, as evidenced of the picture of the reach immediately downstream (Figure 2-15) that shows a main channel at nearly the same elevation as the floodplain. The high banks in the reach appear to be caused by incision into a terrace that began pre-development in the region. The floodplains in this region are well connected to the main channel and grade stabilization is not recommended for existing conditions flow conditions. However, under conditions of development, where flow volumes are increased, grade control will likely be required to stabilize the reach.

Figure 2-14. Priority Area 7 on Corral Tributary Reach C1

Figure 2-15. Immediately downstream of Priority Area 7 on Corral Tributary

Priority Area 8

Priority Area 8 is located on Corral Tributary in Reach C1 downstream of Drennan Road. It is immediately upstream of the Priority Area 7 and has similar characteristics. No site photos were collected in this reach, but the channel geometry was measured with survey equipment. The bank height of the active channel was less than 1 foot, which is confirmed by the photograph of the main channel shown in Figure 2-15.

There are also high banks along the east side of this reach, but the high banks are not due to recent incision into the floodplain, but rather the channel migrating into older terraces in the area.

Priority Area 9

Priority Area 9 is located on Corral Tributary in Reach C2 downstream of State Highway 94. No site visit was conducted and instead the LiDAR and aerial photography was used to assess the reach. Based upon aerial photography, the river has similar characteristics to Priority Area 7 and 8 on Corral Tributary. There are locations where the outside bank is eroding into a high terrace, but the main channel has relatively low banks and does not show signs of being an incising stream.

Priority Area 10

Priority Area 10 is located on Stripmine in Reach S3 upstream and downstream of State Highway 94. No site visit was conducted and instead the LiDAR and Aerial photography was used to assess the reach. The river has similar geometric characteristics to Corral Tributary. There are locations where the outside bank is eroding into a high terrace, but the main channel has relatively low banks and does not show signs of being an incising stream. There is little woody vegetation along the reach and the main channel is wide and shallow with a sandy bed. This reach passes through a dirt bike track and the floodplain is largely devoid of vegetation upstream of State Highway 94.

Priority Area 11

Priority Area 11 is located on Stripmine in Reach S1 approximately 1 mile downstream of State Highway 94. No site visit was conducted and instead the LiDAR and aerial photography was used to assess the reach. The stream has similar characteristics to the Priority Area 10, but the floodplain is generally less disturbed and has herbaceous vegetation present on it.

Priority Area 12

Priority Area 12 is located on Franceville Tributary in Reach UF2 downstream of South Franceville Coal Mine Road. A picture looking downstream is shown in Figure 2-16. Banks along the reach are vegetated with herbaceous plants but no woody vegetation. Some banks are 4 to 8 feet high, but these banks are associated with boundaries of the high flows in the channel (i.e., 100-year flood boundaries). The bank heights of the base flow (flows less than 2-year flood) channel are typically less than 2 ft. Therefore, like Corral Tributary, the high banks in the reach appear to be caused by incision into a terrace that began pre-development in the region.

Figure 2-16. Priority Area 12 on Franceville Tributary Reach UF2.

2.5.2.2 Estimated Bank Erosion

Increased peak stormwater flows due to development and their impact on channel stability in the receiving systems are a well-studied phenomenon. The response of channels to changes in watershed boundary conditions (e.g. increase in storm hydrology) can be described using Simon's Channel Evolution Model illustrated in Figure 2-17. Increased peak stormwater flows result in increased shear stresses on the bed and bank of the stream causing the channel to downcut into its bed. At some point, the flows will concentrate within the incised channel and increase the shear stress on the banks and create over steepened banks that could fail. As a result, the stormwater flows will begin to erode the exposed banks widening the channel. When the channel has widened enough to result in lower energy flows, sediment deposition will begin forming a new low-flow channel with an inset bankfull bench within the limits of the incised terrace walls.

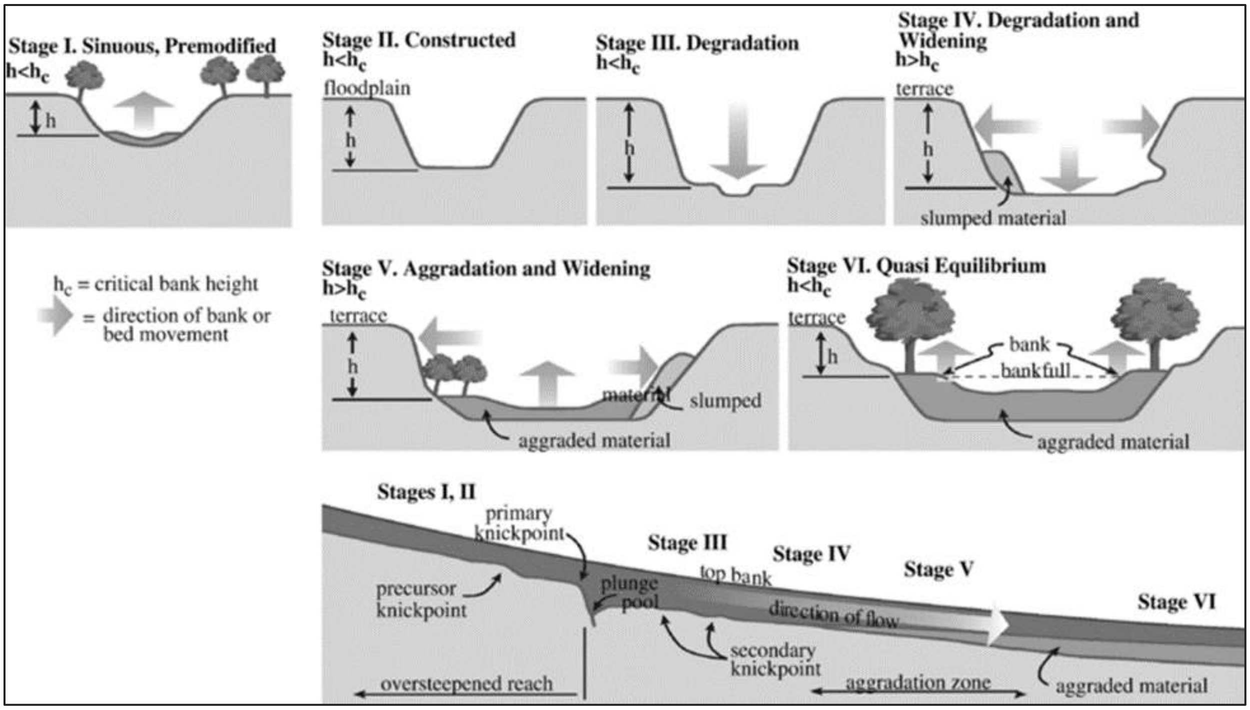


Figure 2-17. Channel Evolution Model Stages (Simon, A., Hupp, C.r., 1986)

The progress of channel evolution occurs naturally, but typically takes place over very long periods of time, with very small incremental changes. However, anthropogenic influences within watersheds can drastically accelerate the rate of change, and cause channels to rapidly change between successional states. In addition, the non-cohesive properties of the natural geology of highly erosive sandy material observed in the Jimmy Camp Creek watershed lends itself to being more impressionable to higher shear stresses. Upon field observation, the degradation stages of the channel evolution process are visibly active in some portions of the Jimmy Camp Creek watershed. Some of the studied reaches are displaying signs of instability. Given the overall undeveloped nature of the watershed, channel evolution processes observed in the Jimmy Camp Creek watershed are resulting in mostly minor bank erosion and sediment pollution at this moment.

The BANCS model was used during the field and desktop assessment to estimate an annual rate of erosion (Rosgen, 2006). The BANCS model was applied to each unstable (impaired) reach identified in Table 2-1.

The unstable reaches located in Priority Areas 9, 10, and 11 were assessed utilizing a desktop methodology and were not assessed in the field. All other unstable reaches were assessed in the field. This model uses a combination of two bank erosion estimation methods, the Bank Erosion Hazard Index (BEHI) and Near Bank Stress (NBS), to evaluate and quantify sediment sources from actively eroding stream banks. The product of the BANCS model gives an estimated annual erosion rate.

BEHI takes the following into account: bank height, rooting depth, rooting density, bank angle, surface protection, bank material, and material stratification. Ratios of bank height to bankfull height and root depth to bank depth along with the remaining variables are used to evaluate the susceptibility to erosion for multiple processes. Refer to Figure 2-18 for BEHI variables measurement diagram. After taking the previously mentioned variables into account, a BEHI rating is generated which ranges from ‘Very Low’, minimal source of erosion, to ‘Extreme’, high source of erosion. Documentation of BEHI scores and the field assessment sheets for all Priority Areas can be found in Appendix A.

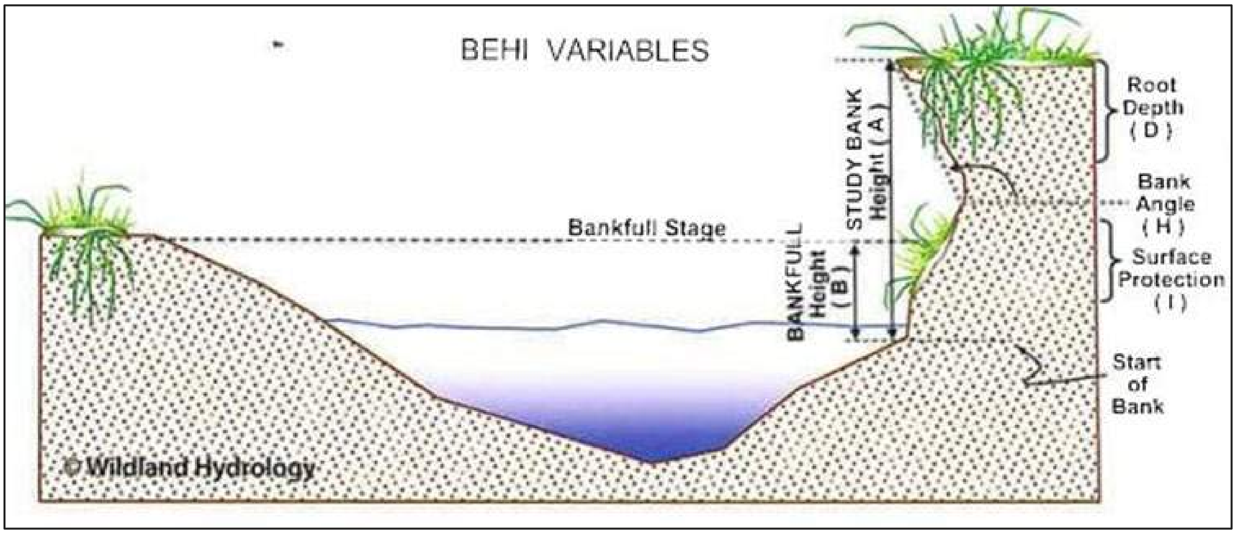


Figure 2-18. BEHI Variables

The NBS method analyzes energy distribution along the study bank. The NBS rating is higher when energy is concentrated toward the study bank. A higher rating indicates a bank that is more susceptible to erosion. There is a total of seven different methods that can be considered when generating an NBS rating. The methods utilized during the field assessment included analysis of channel pattern, transverse bar, or split channel/central bar creating near bank stress or a high velocity gradient.

Once an NBS and BEHI rating are computed for a study bank, the bank erosion rates are estimated using Figure 2-19, which is a relationship between NBS and bank erosion rate for various BEHI (Rosgen, 2006). Figure 2-19 includes curves of predicted annual streambank erosion rates created by the Colorado USDA Forest Service. The estimated volume of erosion is derived by multiplying the erosion rate produced from Figure 2-19 by the study bank height and length. Table 2-3 includes a summary of the BANCS model for each Priority Area where this assessment was completed. Appendix B provides a summary of the BANCS model for each assessed reach.

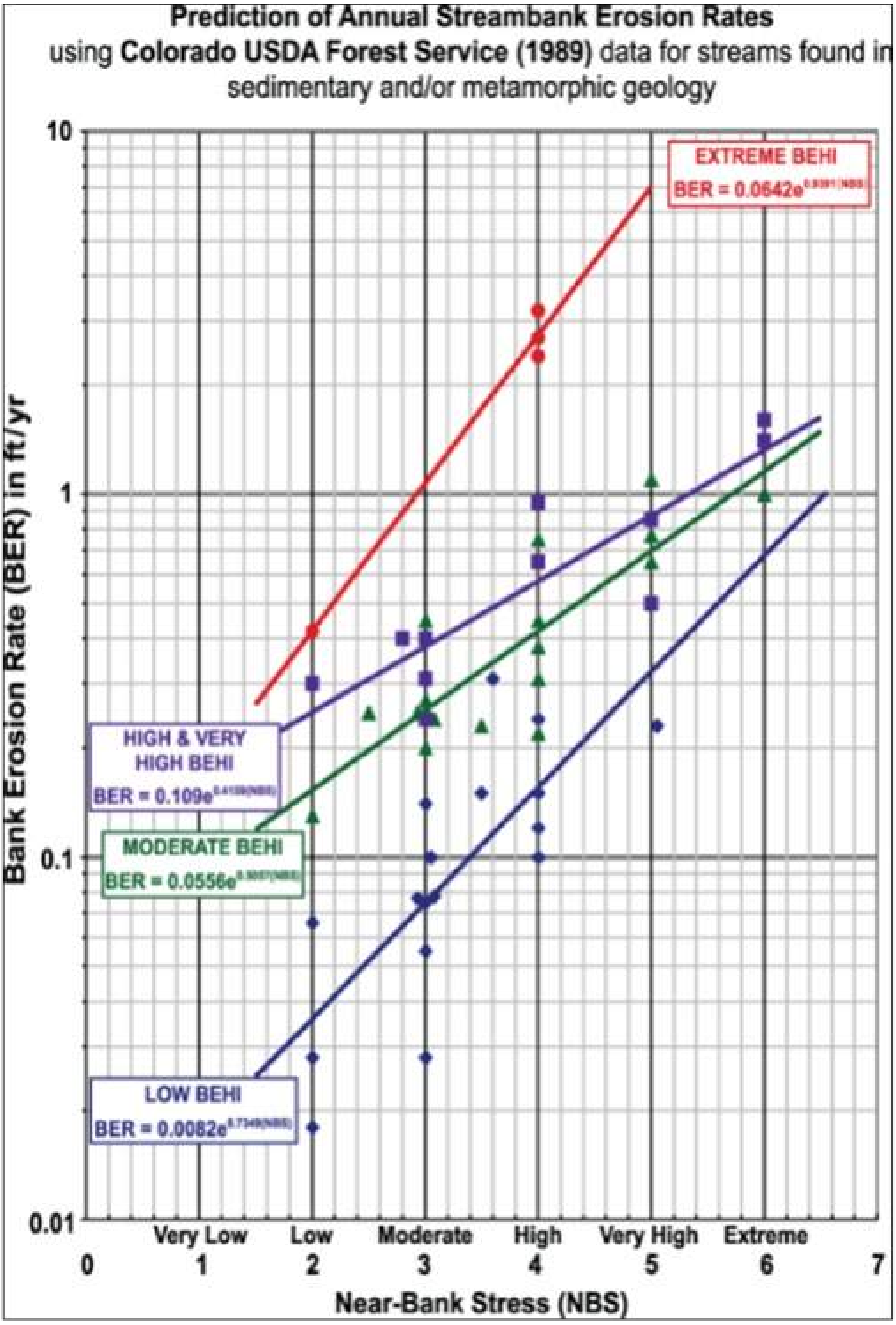


Figure 2-19. Colorado Estimated Bank Erosion Rates

Table 2-3. Jimmy Camp Creek Drainage Basin Estimated Bank Erosion

Reach Name	Priority Area	Site ID	Drainage Area (mi^2)	Length (lf)	Total Erosion (ft^3/yr)	Erosion Volume Rate (ft^3/yr/ft)
Jimmy Camp	1	J1-I	60.5	48	3,420	72
Jimmy Camp	2	J2-I	55	540	1,870	3
East Fork	6	E2-I	2.3	148	260	2
East Fork	6	E2-II	2.3	40	90	2
East Fork	6	E2-III	2.3	43	120	3
East Fork	6	E2-IV	2.3	230	230	1
Corral	7	C1-I	14.7	438	73,790	169
Corral	9	C2-I	4.19	500	24,600	49
Corral	9	C2-II	4.19	542	19,050	35
Stripmine	10	S1-I	1.44	585	4,380	7
Stripmine	11	S1-II	2.8	700	9,260	13
Franceville	12	F1-I	0.29	120	690	6
Franceville	12	F1-II	0.29	43	50	1
Franceville	12	F1-III	0.29	50	350	7

In general, under the current conditions, there is relatively minor erosion with the exception of Corral Creek Priority Area 7, which has a 24-foot-high unstable bank. However, this erosion appears to be part of a natural process of gradual erosion into a high existing terrace.

2.5.3 Wetlands

Wetlands are a key environmental resource that must be considered when planning drainage projects. Three features are needed to identify wetland types and functions: water, soil, and vegetation. These features in the Jimmy Camp Creek Drainage Basin are described below.

2.5.3.1 Wetland Hydrology

The waterways within the Basin are intermittent or ephemeral. According to EPA and US Forrest service definitions, an intermittent stream contains water for only part of the year, while an ephemeral stream flows only in response to precipitation events. There can sometimes be overlap between the two definitions and it would be possible that streams change from one to the other based upon human activities, such as groundwater pumping that depletes river flow, or irrigation that adds additional base flow. There is one active stream gage (USGS gage 07105900) within the Jimmy Camp Drainage Basin. The gage is located upstream of East Ohio Avenue near the basin outlet at Fountain Creek. Daily flow data is available for the period from January 1976 to September 2021. The chronological plot of daily flow data (Figure 2-20) shows that there is a small baseflow and short periods of high flow in response to rainfall or snowmelt events. The daily streamflow exceedance plot (Figure 2-21) indicates a baseflow of approximately 0.2 to 3 cfs.

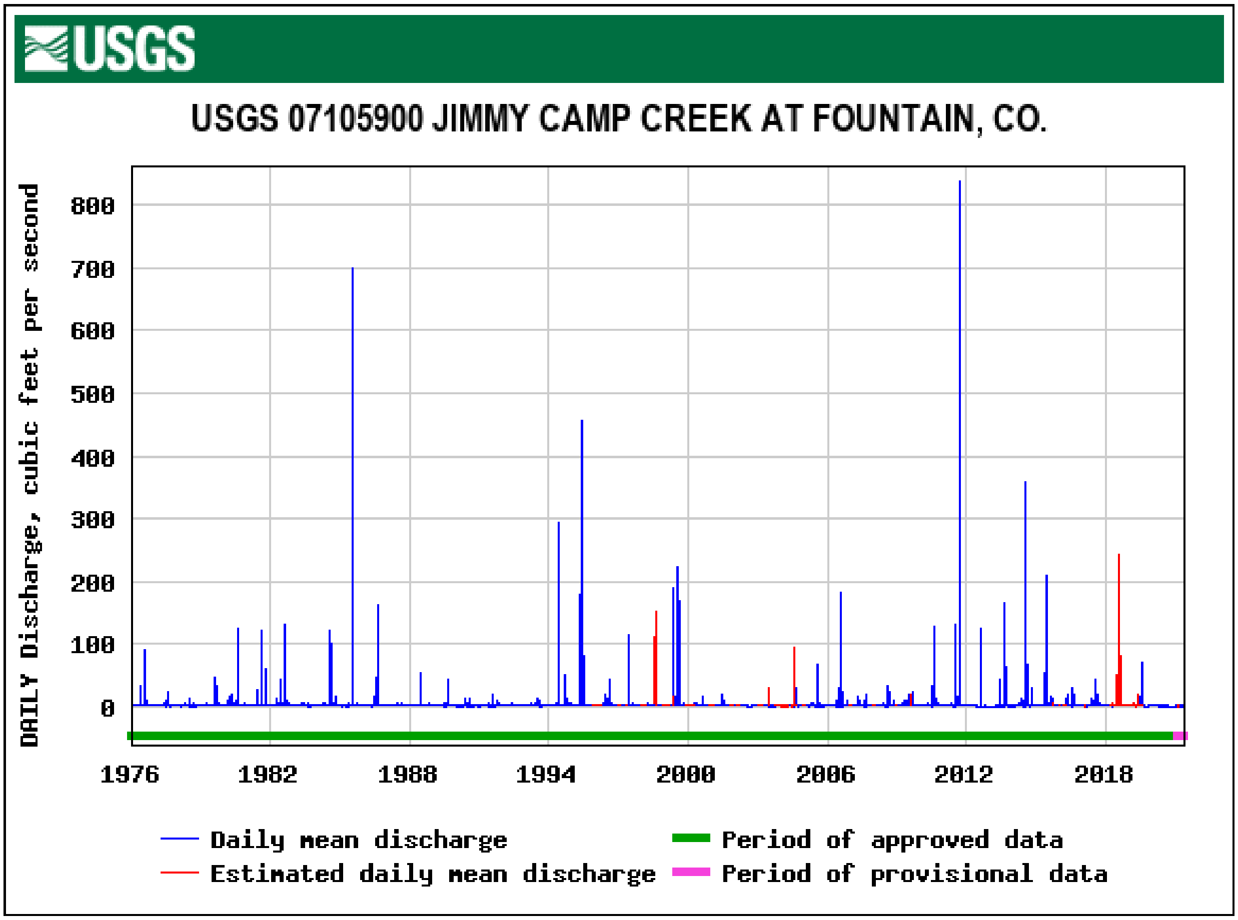


Figure 2-20. Jimmy Camp Creek at Fountain Creek Daily Streamflow, 1976-2021

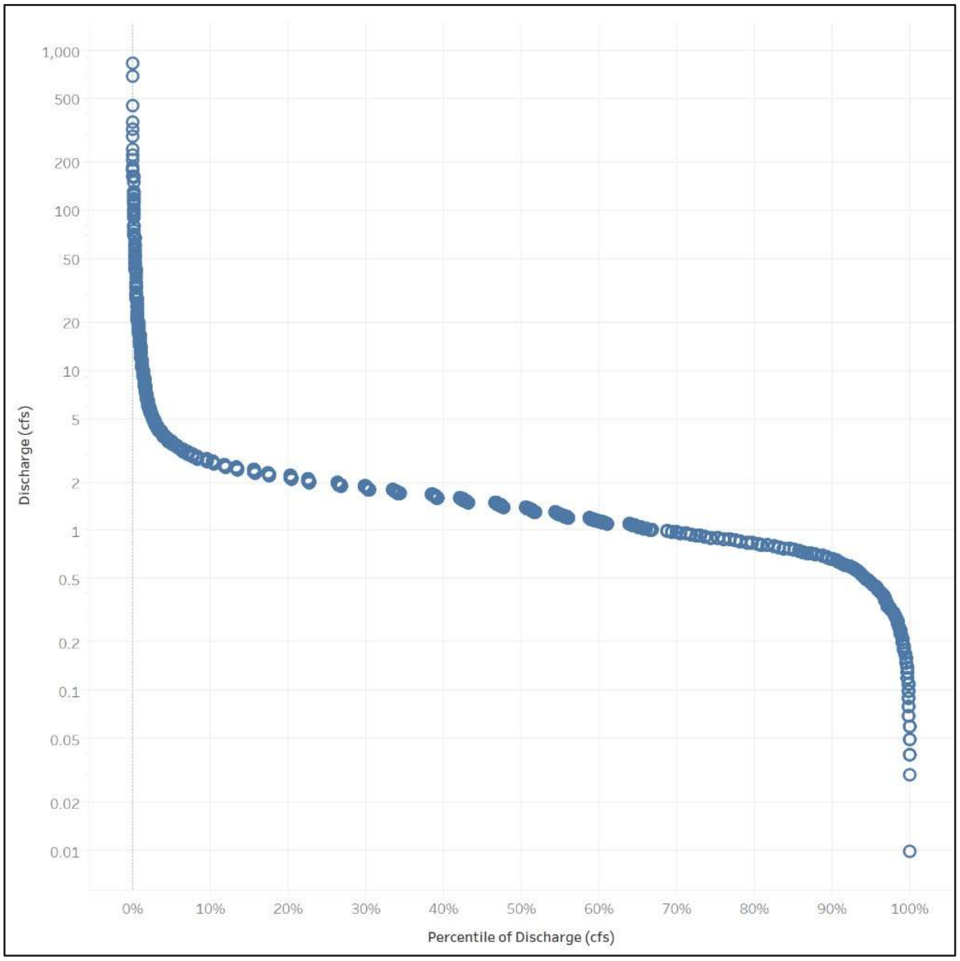


Figure 2-21. USGS 07105900 Jimmy Camp Creek at Fountain Creek Daily Streamflow Exceedance Curve, 1976-2021

Jimmy Camp Creek can be classified as an intermittent stream until its confluence at Fountain Creek. An intermittent stream may have flow when the water-table is seasonally high but ceases to flow during dry periods. The channel is narrow and more defined to the south and becomes wider and less defined north of Bradley Road. Small wetlands are located sporadically along the channel in the southern part of the Jimmy Camp Creek Drainage Basin.

Both the East and West Fork tributaries, as well as the Stripmine tributary, the Corral tributary, and the Franceville tributary are ephemeral with a sandy bottom. An ephemeral stream is located above the water-table year-round and only has flow during and shortly after rain events. Each of these tributaries have highly vegetated banks and floodplains. The ordinary high-water mark becomes less apparent to the north where the terrain flattens. Wetlands have a high potential to occur in these areas.

Two portions of the Fountain Mutual Irrigation Company (FMIC) Canal (also referred to Ditch) are located within the Jimmy Camp Creek Drainage Basin. Fountain Ditch is a 35-mile water canal system including open ditch and piped sections that diverts water from Fountain Creek and runs through the City of Colorado Springs, El Paso County, and the City of Fountain, which irrigates approximately 2,000 acres of land. Fountain Ditch has been owned and operated by Fountain Mutual Irrigation Company (FMIC) since the late 1880's. Within the Jimmy Camp Creek drainage basin, Fountain Ditch is approximately 14 miles long.

Historically the FMIC Canal delivered irrigation water for agricultural shareholders. Currently most shares are owned by municipal water providers, including Colorado Springs Utilities, that use FMIC water to augment depletions from other sources. This has reduced the typical flow of water in the FMIC Canal during the irrigation season.

The portion of FMIC Canal to the west of Jimmy Camp Creek is located mostly within developed neighborhoods and approximately the northern half of the canal can be defined as ephemeral. The channel is highly vegetated once outside the developed areas. The east portion of the Fountain Canal is mostly in a non-developed area except for the northern most portion. It has a narrow, incised, sandy bottom channel. This channel can also be defined as ephemeral.

2.5.3.2 Drainageway Soil Characteristics

There are several NRCS Soil mapping units located within the drainageways. Jimmy Camp Creek, the East Fork tributary, the Stripmine tributary, and the Corral tributary are found mostly in Ellicott loamy coarse sand (Map unit: 28), Stapleton-Bernal sandy loams (Map unit: 85), Ustic Torrifluvents, loamy (Map unit: 101), Blakeland loamy sand (Map unit: 8), Sampson loam (Map unit: 78) and the Lithic haplustepts-Rock outcrop complex (Map unit: 115). The Ellicott loamy coarse sand consists of deep, somewhat excessively drained soils found on floodplains and stream terraces and is formed from sandy alluvium. The Stapleton-Bernal sandy loam consists of deep, well drained soils found on hills and is residuum from weathered sandstone. The Ustic Torrifluvents loamy series consists of deep, well drained soils found on floodplains and stream terraces and is formed from sandy, clayey, stratified loam. The Blakeland loamy sand consists of deep, somewhat excessively drained soils found on hills and flats formed from alluvium derived from sedimentary rock and/or eolian deposits derived from sedimentary rock. The Sampson loam consists of deep, well drained soils formed from alluvium and is found on alluvial fans, terraces and depressions. The Lithic Haplustepts-Rock outcrop complex consists of deep, somewhat excessively drained soils formed from sedimentary rock and is found on scarps.

The West Fork tributary is found mostly in Nunn clay loam (Map unit: 59). The Nunn clay loam consists of deep, well drained soils found on fans and terraces and is formed from mixed alluvium. The Franceville tributary is found mostly in Limon clay (Map unit: 47), Manzanst clay loam (Map unit: 52), Nelson-Tassel fine sandy loams (Map unit: 56), Stapleton sandy loam (Map unit: 84), Truckton sandy loam (Map unit: 96), and Ustic Torrifluvents, loamy (Map unit: 101, described above). The Limon clay consists of deep, well drained soils found on flood plains and alluvial fans and is formed from clayey alluvium derived from shale. The Manzanst clay loam consists of deep, well drained soils formed from clayey alluvium derived from shale and can be found on terraces and drainageways. The Nelson-Tassel fine sandy loams consist of moderately deep, well drained soils formed from calcareous residuum weathered from interbedded sedimentary rock and can be found on hills. The Stapleton sandy loam consists of deep, well drained soils formed from sandy alluvium derived from arkose and can be found on hills. The Truckton sandy loam consists of deep, well drained soils formed from wind re-worked alluvium derived from arkose and can be found on interfluves and fan remnants.

A hydric soil is defined as soil that is formed under conditions of saturation, flooding, or ponding for a period during the growing season to develop anaerobic conditions in the upper portion of the soil’s profile. Hydric soils are a main indicator of wetlands and indicate areas of seasonally high groundwater table (within one foot of the surface). NRCS Web Soil Survey lists minor components of hydric soils in the Ellicott loamy coarse sand, Stapleton-Bernal sandy loams, Ustic Torrifluvents, loamy, Blakeland loamy sand, Nunn clay loam and Sampson loam. However, due to the coarse grained and mostly well drained sediments, there are no strong indicators of hydric soils located within the drainages outside active channels, although hydric soils may be present in some areas.

2.5.3.3 Vegetation

Land cover types within the basin include: Developed, Agriculture, Western Great Plains Shortgrass Prairie, Western Great Plains Foothill and Piedmont Grassland, Western Great Plains Riparian Woodland and Shrubland, Western Great Plains Floodplain Herbaceous Wetland, Invasive Perennial Grassland and Southern Rocky Mountain Pinyon-Juniper Woodland (Colorado GAP Landcover Data).

Emergent wetlands are present in pockets within and along southern portions of the active channels. These are likely dominated by cattails (*Typha latifolia* OBL), bulrush (*Schoenoplectus lacustris* OBL), and sedges (*Carex* sp. >FACW). These emergent wetlands are also mixed with willow dominated wetlands, commonly sandbar willow (*Salix exigua* OBL), dogwood (*Cornus* sp. >FACW) and cottonwood (*Populus deltoides* FAC).

2.5.3.4 Wetland Maps

Figure 2-5 and Figure 2-6 show the National Wetland Inventory mapping for the Jimmy Camp Creek Drainage Basin, the upper and lower basin respectively. These maps depict approximate locations where hydrology, soils and vegetation indicate the likely presence of wetlands, but they do not show locations of those wetlands that would be considered Jurisdictional Waters of the United States. The maps are intended to be used for preliminary planning purposes, but additional field delineation of wetlands should occur during project design and construction.

2.5.4 Jurisdictional Wetlands and Waterways

Jurisdictional Waters of the United States (WOTUS) are defined as any wetland or waterway hydrologically connected to navigable waters of the United States. Jurisdictional wetlands and waterways are subject to federal regulation. The mainstem and all major tributaries of Jimmy Camp Creek mapped on the USGS map will need to be assessed to determine jurisdictional status. A US Army Corps of Engineers (USACE) permit under Section 404 of the Clean Water Act may be required for projects that plan to discharge material within the ordinary high-water mark or adjacent wetlands.

Irrigation ditches are also considered jurisdictional waters if they empty into jurisdictional WOTUS. Also, any ponds or wetlands connected to those irrigation ditches are considered jurisdictional. The definition of wetlands in Colorado was recently addressed in the Colorado House Bill 24-1379, titled “Regulate Dredge & Fill Activities in State Waters”. This bill extends protection to ephemeral streams and will be important in the definition of jurisdictional wetlands in the Basin. The types of wetlands covered under Section 404 and Colorado law should be verified at the time of planned channel construction projects.

2.5.5 Potential Endangered and Threatened Species

According to the USFWS Information for Planning and Consultation (IPaC) website, El Paso County is home to multiple endangered (E) and threatened (T) species. These species include: Preble’s Meadow Jumping Mouse (*Zapus hudsonius preblei*, T), Least Tern (*Sterna antillarum*, E), Mexican Spotted Owl (*Strix occidentalis lucida*, T), Piping Plover (*Charadrius melodus*, T), Whooping Crane (*Grus americana*, E), Greenback Cutthroat Trout (*Oncorhynchus clarkii stomias*, T), Pallid Sturgeon (*Scaphirhynchus albus*, E), Pawnee Montane Skipper (*Hesperia leonardus montana*, T), Ute Ladies’-tresses (*Spiranthes diluvialis*, T), and the Western Prairie Fringed Orchid (*Platanthera praeclara*, T).

Critical habitat for the Preble's Meadow Jumping Mouse is located near the northeast side of Jimmy Camp Creek Drainage Basin. The other species have habitat requirements that are not met within the Study Area, including vertical-walled rocky cliffs, lakeshores, inland marshes, cold water lakes or streams, oxbows, unplowed calcareous prairies, and sedge meadows.

2.5.6 Environmental Resources Summary

Any channel improvement project affecting a wetland, waterway, or irrigation ditch within the Jimmy Camp Creek Drainage Basin may be subject to regulations by the USACE. Any impacts to riparian ecosystems near/within permitted activities may also need replacement. Detailed wetland delineations may be needed in areas where channel modifications and drainage outfall systems are proposed.

2.5.7 Environmental Permitting Requirements

Areas identified as wetlands, Waters of the U.S., open water, and irrigation ditches may be subject to USACE Section 404 regulations. Impacts may need to be mitigated, and riparian ecosystems impacted in conjunction with permitted activities may also need replacement. Detailed wetland delineation will need to be performed in areas where drainage system improvements are proposed in potential jurisdictional areas and evaluated in relation to permitting requirements in affect at the time of construction.

Other state and local construction permits related to activities in drainage corridors will also apply. Conditions in the Jimmy Camp Creek Basin are typical of other drainageways in El Paso County, so standard permit conditions related to avoiding and mitigating impacts that have been applied to past drainage projects in the County are expected to apply in the Jimmy Camp Creek watershed as well.

2.6 Hydraulic Structures

Several existing hydraulic structures are located in the Jimmy Camp Creek Drainage Basin. In developed areas the channels are improved in some fashion, varying from widened channel cross-sections to widely spaced grade control structures. For most areas outside of these developed sites, natural channels follow their historic alignments and flows are not attenuated. Currently, there are no major regional stormwater detention basins in the watershed within the unincorporated County.

Within the unincorporated County, 16 existing road bridges and culverts cross the drainage channels in the Jimmy Camp Creek Drainage Basin, and several grade control structures were recently installed to manage downcutting and lateral channel migration as part of the Lorson Ranch development.

2.7 Stormwater Quality Considerations and Proposed Practices

Factors that will affect stormwater quality in the Jimmy Camp Creek Drainage Basin drainageways include urbanization and sedimentation/erosion processes. El Paso County addresses both factors through development requirements for application of Permanent Control Measures (PCMs) (e.g., onsite stormwater quality ponds, stormwater extended detention basins, etc.), education and outreach, system maintenance, and other programs.

The DBPS does not include specific recommendations for PBMPs such as stormwater extended detention basins in areas of new development in the Jimmy Camp Creek Drainage Basin. These will be the responsibility of developers in compliance with current criteria and policies.

The primary water quality constituent associated with the drainage corridor that could be affected by DBPS recommendations is sediment. Excessive sediment is a problem identified in the Fountain Creek watershed (THK/Matrix, 2011). A hydraulic analysis of the Jimmy Camp Creek watershed, described in detail in Section 4.8, did determine that channel erosion is a potential issue in the Jimmy Camp Creek Drainage Basin drainageways as evidenced by high velocities during peak flows. In addition, the increase in flow volumes after development would increase the rate of erosion from the stream channel. Corrective measures outlined in the DBPS to stabilize channels and reduce channel erosion will help reduce future sediment production from the Jimmy Camp Creek watershed.

3.0 HYDROLOGY

This section describes model development and analysis performed to estimate hydrologic runoff within the Jimmy Camp Creek Drainage Basin. Developing representative and appropriate hydrologic inflows on both a sub-basin and basin-wide level have a direct influence on correctly selecting and sizing stormwater management alternatives. The objective of the hydrologic analysis is to produce a defensible approach to rainfall/runoff modeling that both the County and the development community accept moving forward.

Development of hydrologic flows for the Jimmy Camp DBPS were performed using the Environmental Protection Agency's (EPA's) Storm Water Management Model (SWMM) 5 (version 5.1.015). Model development, scenario management, and model execution was performed using the Innovyze InfoSWMM software (version 14.7, Update #6). Final model deliverables are provided in the SWMM 5 software. The hydrologic modeling methods and parameters are described in this report. Hydraulic routing of runoff for use in the hydraulic model was also executed using SWMM 5 methods, which are also described in Section 3.14.

3.1 Purpose of Hydrologic Analysis

The hydrologic analysis for the Jimmy Camp Creek DBPS provided an estimate of the drainage basin's runoff and peak flow response to the 2-, 5-, 10-, 25-, 50- and 100-year recurrence interval rainfall events. The hydrologic methods applied in this DBPS involved developing suitable GIS based surface and subsurface model parameters based on various applicable documentation, including:

- City of Colorado Springs Drainage Criteria Manual, Volume 1, March 2014, Revised January 2021.
- City of Colorado Springs Drainage Criteria Manual, Volume 2, prepared by Matrix Design Group/ Wright Water Engineers. March 2014, Revised December 2020
- Drainage Criteria Manual County of El Paso, Colorado (El Paso County, 1991)
- EPA SWMM Reference Manual (EPA, 2016)
- Mile High Flood District (MHFD) (formally Urban Drainage and Flood Control District [UDFCD]) Urban Storm Drainage Criteria Manual (MHFD, 2016)

The purpose of the Jimmy Camp Creek DBPS hydrologic analysis was to develop peak flows for planning and design based on current conditions in the basin. The results of the hydrologic analysis fed into the hydraulic analysis portions of this DBPS. As such, peak flows were developed for key design points along the Jimmy Camp Creek main stem and the tributary channels within the Jimmy Camp Creek Drainage Basin. Hydraulic routing was also included in the hydrologic analysis to determine peak flows at key points in the Jimmy Camp Creek Drainage Basin for use in the hydraulic analysis, Section 4.0.

SWMM 5 model construction was performed using GIS tools to improve efficiency and apply standardized and reproducible methods for determining model input parameters. SWMM 5 methods were used to simulate both hydrologic runoff from individual sub-basins and hydraulic routing through sub-basins to Jimmy Camp Creek and its major tributaries. Detailed hydraulic modeling and analysis along Jimmy Camp Creek and its tributaries were completed using the USACE HEC-RAS model as described in Section 4.0.

This involved developing simulated water surface elevations and flood potential using the peak flows developed from the SWMM 5 model results.

3.2 Data Sources Used in Hydrologic Analysis

The hydrology for this DBPS was generated using the best available information provided by the County and acquired from public sources. Sources of information and their use include the data listed below.

- **2018 LiDAR Topography** – 2018 LiDAR Digital Elevation Model (DEM), 2 foot by 2 foot square, produced by the State of Colorado.
- **2020 El Paso County Aerial Photography (received April 2021)** – Aerial photography provided by the County in single raster dataset with a resolution of 1 foot by 1 foot square.
- **Soil Survey Geographic (SSURGO) Soil Data** – The United States Department of Agriculture (USDA) National Resources Conservation Service (NRCS) Soil Survey Geographic (SSURGO) Database was used to develop soil-based infiltration parameters.

  https://websoilsurvey.sc.egov.usda.gov/App/WebSoilSurvey.aspx
- **NOAA Atlas 14 Rainfall Data** – National Oceanic and Atmospheric Administration (NOAA) Atlas 14 Volume 8 Version 2, Precipitation-Frequency Atlas of the United States, Midwestern States was used for the source of design rainfall depths (Perica et al., 2013). http://hdsc.nws.noaa.gov/hdsc/pfds
- **County GIS Database** – County GIS database, *DPW_data.gdb*, provided on April 16, 2021 contained the feature classes listed below.

Bridge	Inlet
Channel	Junction
Culvert _Ln	Outfall
Culvert_pts	ROW
CityLimits	StormPipe
Dentention_Ponds	
EPC_Roads	

- **County GIS Shapefiles (provided on April 16, 2021)** – Floodplains.shp, Lakes.shp, Parcels.shp, WaterLine.shp, and Wetlands.shp
- **County Provided Existing and Future Impervious Percentage GIS File (provided on July 24, 2021) –** Future_LUse_JCC_SubBasins feature class
- **County Field Data for Crossing Information (provided on November 29, 2021)** – JCC_missing_crossing_data_locations_EPC.doc
- **Colorado Department of Transportation (CDOT) Inspection Structures** – Used for determining the structure size of various crossings
- **Pond Design Plan and As-Built Information** – Design plan information was available for the West Fork in Channel Pond for the Glen at Widefield Subdivision (Kiowa, 2008).
- **County GIS Data (Downloaded)** – Parcel, major highway, parks, zoning, and land use data.

3.3 Hydrologic Methods

Hydrologic and hydraulic modeling and system analyses follow the guidelines and criteria set forth in the Drainage Criteria Manual Volume 1 of El Paso County and the City of Colorado Springs DCM for SWMM 5, with the SWMM 5 model use approved by the County. Runoff hydrographs and associated peak flows were developed using the EPA SWMM Nonlinear Reservoir Method (Runoff Method) as described in the SWMM Reference Manual (EPA, 2016). This method provides a detailed hydrologic representation of the watershed and allows flexibility when used for both event-based and continuous simulation. By using the Runoff Method, the model can be directly applied to design storms of various durations and temporal distributions. The Runoff Method is used in conjunction with Horton's method for modeling infiltration. The hydrology methods and parameters are described in the subsequent sections.

3.4 Sub-basin Discretization

One of the key tasks in building a hydrologic model is to divide the study area into relatively homogeneous sub-basins and allocate flows from individual sub-basins to their respective conveyance element. In addition, the spatial arrangement between these sub-basins needs to represent ground conditions. Sub-basins were delineated to outlets (design points) within the conveyance channels, with sub-basin areas generally developed to be about 160 acres in size.

3.5 Sub-basin Delineation

Sub-basins were delineated for design points along the open channels within the project area at key locations and key land features (roads, railroads, ditches, etc.) based on both existing and proposed conditions. Sub-basin outfalls and design points correspond to any location along open channels where existing and future facilities require evaluation, including where there are pipes greater than 60-inches in diameter and at major roadway crossings.

Sub-basin boundaries were based on the 1-foot contours, aerial photography, the County's culvert GIS database, and the County's GIS storm sewer database. Sub-basins were delineated, to the extent practicable, based on maintaining consistent size, shape, and slope throughout the area. Delineations were made to design points on the main channels, with the intent of keeping the size to be about 160 acres. Considerations were made for having relatively homogenous land uses within each sub-basin so that basin parameters and resulting runoff response were correctly represented. Experience has shown that a sub-basin containing both developed and open space land uses will under-predict the runoff rates of the developed areas and over-predict the runoff rates from the open space. The same consideration was also made with soil types such that sub-basins have uniform soils to correctly estimate infiltration response. However, deviations to the above considerations were necessary, with some basins being larger due to the need for flow routing (such that there is a sufficiently channelized flow route to develop cross-sections) and some basins being smaller due to the convergence of tributaries or the requirement to add a design point.

Figure 3-1 presents the major subbasins developed for this DBPS. A detailed subbasin map for existing conditions is included in Appendix C.

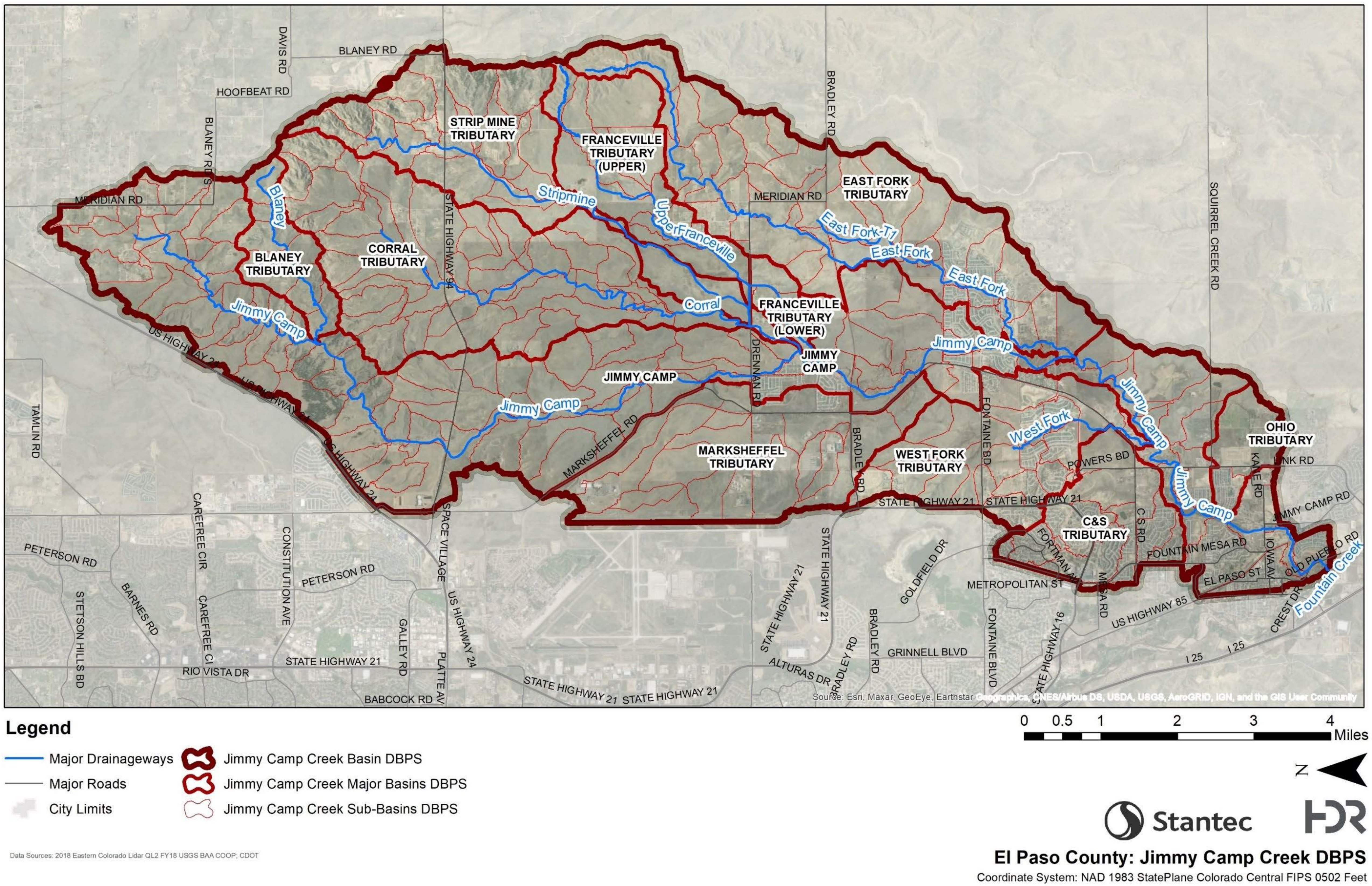


Figure 3-1. Sub-basin Map

3.6 Sub-basin Naming Convention

The SWMM model element naming was based on a convention that starts with the sub-basin names, with model nodes and then model links named accordingly to tie back to the sub-basins. Sub-basins were labeled systematically, with the labeling associating the sub-basins with their corresponding channel main stems and tributaries. Sub-basins, design points, and conveyance elements were labeled systematically and consistently to the greatest extent practical to represent their relation to each other. Sub-basins were then grouped and labeled based on their connection to the channel main stems or tributaries to identify major sub-basins. The general approach in naming modeling elements is provided in the following sections.

3.6.1 Sub-basin Naming

Sub-basin naming is based on the branch names and abbreviations (see Appendix C for the map of all subbasins and branches). A branch is defined as any model segment that is contained within a sub-basin.

Sub-basins within each branch are then named consecutively starting from the downstream end. Sub-basins are named based on the first letter of the branch name.

<u>Example:</u> If the branch is the Franceville Tributary, then the most downstream sub-basin to this branch is *F1_1*, with the next upstream being *F1_2*.

If two sub-basins drain to the same location, then the sub-basin's location in relation to the channel is added to the number. If there are more than two sub-basins contributing to a design point, then additional compass directions are used.

<u>Example:</u> On the East Fork Tributary (E1) there is an additional small tributary (T1) that has another additional small tributary (3E). The two most upstream sub-basins go to the same design point. These sub-basins are therefore named *E1-T1_3E_N* and *E1-T1_3E_S*.

3.6.2 Model Node Naming

Model node names use the sub-basin names as a base. If a node is a design point serving as an outfall to a corresponding sub-basin(s), the runoff node starts with a designation of DSNPT_ followed by the sub-basin name. If there are multiple sub-basins to a design point, the compass direction is not included in the node name.

Intermediate nodes are locations between design points that help describe conveyance geometry, including open channels and road crossings. Intermediate nodes are labeled with consecutive numbers along that reach to the next design point. If the node is a channel point, it is labeled with a CH. Nodes upstream and downstream of culverts are considered channel points.

<u>Example:</u> Assume the branch is identified as J2. If there are two channel section changes upstream of a design point, the naming would be: *DSNPT_J2*, *CH1_J2*, and *CH2_J2*.

3.6.3 Model Link Naming

Model link names are based on the upstream node name and the conveyance element type. The following examples assume the link is the first node upstream of a design point in sub-basin J1_1. The node is a channel section change (CH1).

Natural channels: NAT-

Example: NAT-CH1_ J1_1

Trapezoidal channels: TRAP-

Example: TRAP- J1_1

Circular culverts: CUL-

Example: CUL- J1_1

Box culverts: BOX-

Example: BOX- J1_1

3.7 Sub-basin Percent Imperviousness

The County provided both existing and future impervious estimates for the Jimmy Camp Creek DBPS. These feature classes provided complete coverage across the DBPS area, based on the El Paso County Master Plan (El Paso County, 2021), and were the basis for both existing and future percent imperviousness estimations used for the DBPS modeling. The resulting existing impervious coverage, or “hydrologic land use”, is presented in Figure 3-2. The percent imperviousness feature class was intersected with the sub-basins in GIS and an area weighted impervious percentage was calculated for each sub-basin. For sub-basins that are in a current developed state, percent imperviousness values were increased slightly by 5 percent based on impervious calculation performed for the Sand Creek 2021 DBPS for developments of similar densities (City of Colorado Springs, 2021). Directly connected impervious areas (DCIAs) were not evaluated separately from the percent imperviousness calculations described above given the planning level nature and scale of this DBPS.

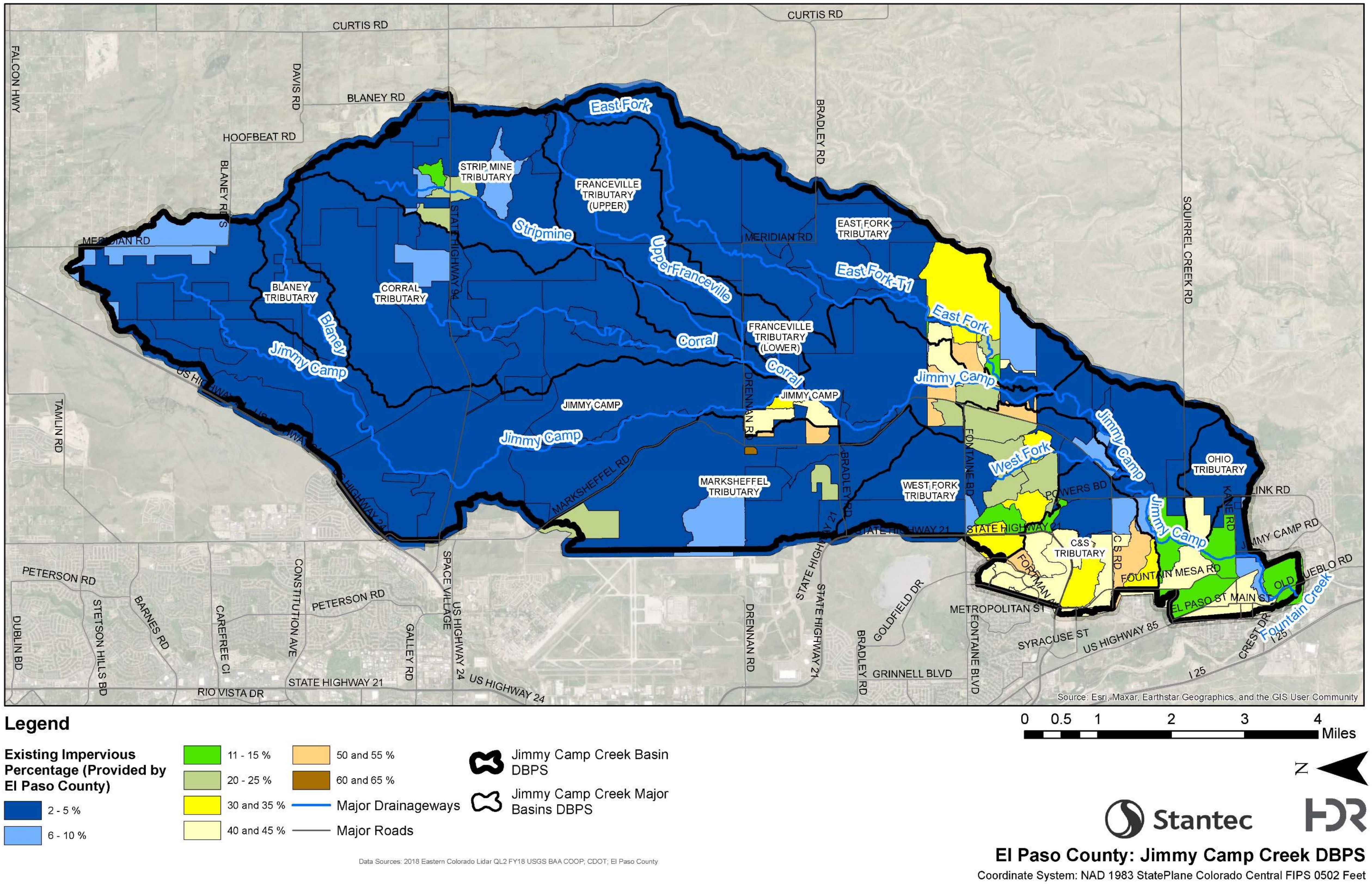


Figure 3-2. Existing Impervious Percentage Map

3.8 Sub-basin Slope

Use of the Runoff Method in SWMM 5 requires determination of the average sub-basin slope. Sub-basin slope influences the runoff travel time and resulting hydrograph shape. The average slope (ft/ft) for each sub-basin was calculated using the County's 2018 DEM data and ESRI's ArcHydro extension. The DEM is a grid format and the area-weighted average slope for each individual sub-basin was calculated by measuring the average difference in elevation between each grid cell within that sub-basin. Sub-basin slope is considered a calibration parameter and, as such, has variability in interpretation for individual sub-basins. To better reflect downstream gage data (discussed more in Section 3.16), the calculated slope values were increased 50 percent for all sub-basins (i.e., if a sub-basin had a calculated average slope of 1 percent, it was increased 50 percent to 1.5 percent).

3.9 Sub-basin Width

Sub-basin width in the Runoff Method represents the physical width of overland flow and is a variable in determining the time lag between peak precipitation and peak runoff. The process for estimating sub-basin width for this DBPS was made as straight forward and reproducible as possible.

The sub-basin widths were estimated using standard EPA-SWMM guidelines and were based on the main channel length. As with sub-basin slope, sub-basin width is considered a calibration parameter and, as such, has variability in interpretation for individual sub-basins. Because of this, two different methods, as described in the SWMM 5 reference manual (Rossman and Huber, 2016, p. 72-73), were evaluated for this study. Based on flows at the USGS stream gage on the downstream end of the basin, sub-basin width was only further modified on a skew factor (as opposed to using both a skew and shape factor).

To incorporate the use of skew factors, the following process was used to estimate the width parameters. The SWMM 5 reference manual contains a full discussion on this method (Rossman and Huber, 2016, p. 72-73).

W = estimation of watershed width

A = total sub-basin area

L = length of the sub-basin's main channel (pathway) length

A_m = larger of the two areas on each side of the main channel (pathway)

Z = skew factor, which is the ratio of areas on either side of L (the main drainage channel)

With

$Z = A_m/A$ (Eq. 3-10, p. 71)

The estimate for watershed width is then the weighted sum between the two limits of the main channel length, with twice the main channel length (2L) being the upper limit and the main channel length (L) being the lower limit. This results in the following equation:

$W = L + 2*L*(1 - Z)$ (Eq. 3-11, p. 72)

In this process, once the sub-basin areas are delineated, main channel length (L) is the only unknown. Main channel lengths for each sub-basin are determined by measuring the distance from the upper-most point in the sub-basin, through the overland and storm drain conveyance path, to the most downstream point in the sub-basin. This often correlates to the longest flow path. For the purposes of this calculation, main channels are any conveyance element that transfers runoff through the sub-basin and include open channels, streets, the main drainageway of creeks, or a combination of these.

There are four general types of sub-basins in this DBPS. These types and the associated approach to determining the main channel length for each type are described below.

Type 1: Developed (urban) sub-basins that are served by drainage pipes with the outflow point going directly into an open channel (there are no open channels within the sub-basin). The main channel length, L, is defined as the longest flow path that follows the main street and trunk storm drains to the sub-basin outlet at the channel.

Type 2: Developed (urban) sub-basins that are served by drainage pipes that outflow directly into an open channel and include the main channel itself (there is an open channel bisecting the sub-basin). The main channel length, L is defined as the longest flow path that follows the streets, the trunk storm drains, and then the open channel itself to the sub-basin outlet.

Type 3: Undeveloped (rural) sub-basins that drain directly into an open channel (there are no open channels within the sub-basin). The main channel length, L is defined as the longest flow path along the small channels as well as some sheet/shallow concentrated flow to the open channel.

Type 4: Undeveloped (rural) sub-basins that are served by small drainage channels that outflow directly into an open channel and include the main channel itself (there is an open channel bisecting the sub-basin). The main channel length, L is defined as only the main channel itself and represents the only exception to the longest flowpath approach.

3.10 Sub-basin Overland Flow Roughness

Sub-basin overland flow roughness is used in the Runoff Method and is one of the influences used for estimating the time it takes for precipitation to be transformed to runoff. Higher values of Manning's "n" represent rougher surfaces, like lawns or pastures, where runoff times will be delayed. Lower values represent impervious areas such as roads or parking lots and produce higher peak flows with little or no runoff delay. These values were estimated from Table 3-5 from the SWMM 5 reference manual (Rossman and Huber, 2016, p. 75) based on Yen (2001), which computed values based on kinematic wave analysis. Table 3-1 describes overland flow roughness values assumed for this DBPS for both pervious and impervious surfaces. These roughness assumptions tend toward being conservative (lower roughness). As with sub-basin width, it is common to consider overland flow roughness a calibration parameter (Rossman and Huber, 2016, p. 74).

Table 3-1. Assumed Overland Flow Runoff Roughness Coefficients

General Land Use Types	Assumed Land Surface	Roughness Coefficient*
All Land uses	Impervious areas: smooth surface (concrete, asphalt, gravel)	0.011
Residential, Commercial, Industrial, Institutional, Municipal, Public, Transportation Right-Of-Way, Parking Lots, Utility/Drainage Rights-Of-Ways and Easements, Military Installation	Dense residential land use	0.04
Agriculture	Pasture	0.055
* Values obtained from the EPA SWMM Reference Manual, 2016, Table 3-5, based on values from Yen (2001).		

3.11 Infiltration

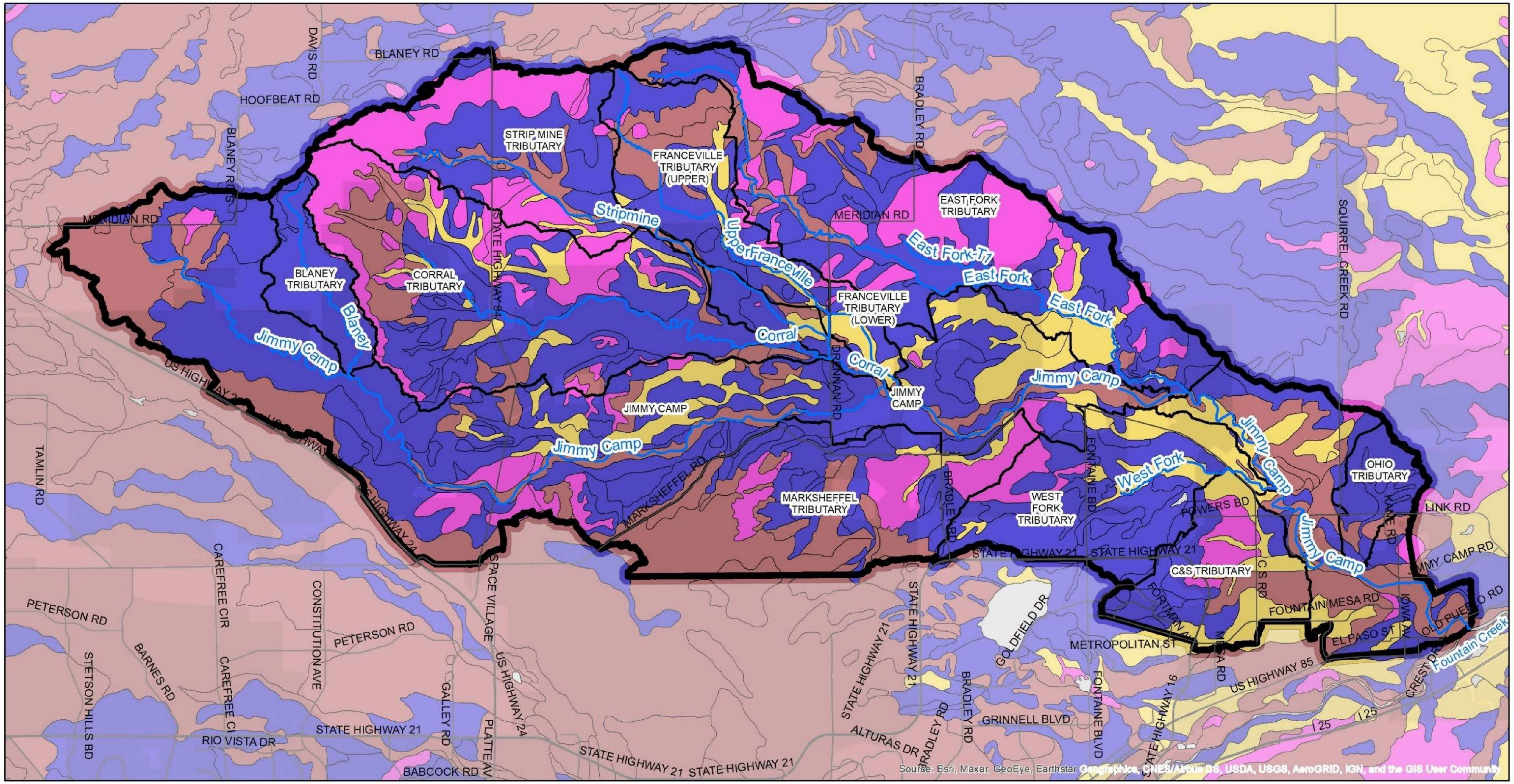
Infiltration is the process by which surface water percolates into the subsurface soil and groundwater column. Infiltration is an important hydrologic process because it governs groundwater recharge, soil moisture storage, and surface water runoff volume. As modeled in the SWMM 5 Runoff Method, soil infiltration is one of several processes that represent a withdrawal of a portion of total storm precipitation that could otherwise generate surface runoff.

Infiltration parameters were developed for the Horton infiltration method based on information for soil types and characteristics compiled and grouped from the NRCS SSURGO dataset. The Horton infiltration method was used because parameters can be estimated from existing soil surveys without performing extensive field testing. The SWMM 5 reference manual (Rossman and Huber, 2016, p. 88-99) and the Mile High Flood District (MHFD) criteria was used as a reference for determining Horton infiltration parameters using NRCS Hydrologic Soil Groups (HSGs) as a reference. HSG soils within the Basin are mapped in Figure 3-3. Hydrologic Soil Groups (HSGs) were developed for each sub-basin based on the 'dominant' condition of the soil profile using the NRCS Soil Data Viewer. The Horton infiltration parameters used for modeling, including max infiltration rate, asymptotic infiltration rate, and decay rate of infiltration, are described in Table 3-2. For this DBPS, the dominant soils are HSG A and B. Given the dominance of these soil types, high infiltration and low runoff is expected from undeveloped, pervious areas.

Table 3-2. Estimated Horton Infiltration Values

NRCS Hydrologic Soil Group	Associated Soil Textures within the Jimmy Camp Creek Drainage Basin	Percentage within the Jimmy Camp Creek Drainage Basin	Maximum (Initial) Infiltration Rate (Inches/hour)*	Minimum Infiltration Rate (Inches/hour)**	Decay rate of Infiltration (1/second)***
A	loamy sand; loamy coarse sand; gravelly loam; sandy loam, sand	21%	2.5	0.3	0.001
B	loam; sandy loam; fine sandy loam; gravelly loamy sand; silt loam	53%	2.0	0.15	0.002
C	clay loam; silt loam, Clay	10%	0.8	0.05	0.002
D	silty clay loam, clay loam, fine sandy loam	15%	0.8	0	0.002

*   Values obtained from the EPA SWMM Reference Manual, 2016, low end of Table 4-6, p. 99
** Values obtained from the EPA SWMM Reference Manual, 2016, low end of Table 4-4, p. 97
*** Values obtained from MHFD, 2016, Table 6-7



Legend

NRCS Hydrologic Soil Group

- A
- B
- C
- D
- Not rated or not available

- Major Drainageways
- Major Roads
- City Limits
- Jimmy Camp Creek Basin DBPS
- Jimmy Camp Creek Major Basins DBPS

0 0.5 1 2 3 4 Miles

N

Stantec HDR

El Paso County: Jimmy Camp Creek DBPS

Coordinate System: NAD 1983 StatePlane Colorado Central FIPS 0502 Feet

Data Sources: 2018 Eastern Colorado Lidar QL2 FY18 USGS BAA COOP; CDOT; USGS SSURGO CO625 Dataset

Figure 3-3. Jimmy Camp Creek Hydrologic Soils Map

3.12 Depression Losses

Stormwater that is collected and held in small depressions or intercepted by vegetation and does not become part of the general surface runoff is referred to as abstraction or depression loss. Depression losses include interception losses in the context of this DBPS. Most of this water eventually infiltrates or is evaporated and does not contribute to runoff. Depression losses are calculated in conjunction with infiltration and are dependent upon land use cover. Ultimately, depression loss defines the depth of rain that must fall before runoff can occur in a sub-basin. Table 3-3 describes the depression storage losses assumed for this DBPS for both pervious and impervious runoff surfaces.

Table 3-3. Assumed Depression Losses

Surface Type	Depression Loss* (in)	Types of Land Uses
Large Paved Areas	0.07	Impervious areas within all land uses
Pervious Areas	0.1	Residential, Commercial, Industrial, Institutional, Municipal, Public, Transportation Right-Of-Way, Parking Lots, Utility/Drainage Rights-Of-Ways and Easements, Military Installation
Wooded areas and open fields	0.25	Parks, Cemetery, Agriculture, Vacant Land, Airport
*Values obtained from EPA SWMM Reference Manual, 2016, p. 77 (Includes Interception Losses)		

3.13 Rainfall

This section describes the method used to determine the rainfall depths and temporal distribution for various frequencies of occurrence. These rainfall events are termed Design Storms.

3.13.1 Rainfall Depths

NOAA Atlas 14 was used to estimate rainfall depths for the simulated rainfall frequencies. Data relating to NOAA Atlas 14 is published through the Precipitation Data Server (PDS) and is correlated to specific rain gage stations throughout the country. This results in Point Precipitation Estimates that can be extracted at any location within the County. The County has two rain gage stations located in or near the Basin:

- City of Fountain (Station ID: 05-3063)
- Colorado Springs Airport (Station ID: 05-1778)

The City of Fountain gage is located near the outlet of the Basin and the Colorado Springs Airport gage is located outside of the Basin. Because these gages were located outside or on the edge of the Basin, they were not used and instead NOAA Atlas 14 Point Precipitation Estimates were extracted from three locations within the Basin, with one being in the northern location of the Basin, one being near the Basin center, and one being in a southern location of the Basin. Evaluation of these three locations showed little variation in the three data extraction locations, therefore only the North location was used for this DBPS. The process is further detailed in the El Paso County Rainfall Distribution Technical Memorandum (July 2021), provided in Appendix D.

Table 3-4 summarizes the rainfall depths extracted at the North location. Following discussions with the County, the rainfall depths in Table 3-4 were increased by 7 percent based on guidance from the Office of the State Engineer (State of Colorado, 2020) that states that "…an atmospheric moisture factor of 1.07 must be applied to account for expected future increases in temperature and associated increases in atmospheric moisture availability." Essentially, this is in reference to an empirical calculation that the atmosphere can hold 7 percent more moisture with every 1-degree Celsius increase in temperature. Provided that Atlas 14 Volume 8 was published in 2013, most of the 1-hour data used in NOAA Atlas 14 was collected prior to 2010, and that there are indications that average daily maximum temperatures in El Paso County have increased by 0.5 degrees Celsius since 2010[1], therefore this 7 percent increase in rainfall depths is warranted for this DBPS. Table 3-4 also provides the resulting rainfall depths with the 7 percent increase, as used for the current hydrologic analysis. As can be seen from Table 3-4, the 7 percent increase in rainfall depths are still within the upper limit of the 90 percent confidence interval of the NOAA Atlas 14 data.

Table 3-4. NOAA Atlas 14 Point Based Precipitation Frequency Estimates for 24-Hour Rainfall Depths for the North Location in the Jimmy Camp Creek Basin *

Rainfall Recurrence Interval	**24-Hour Rainfall Depth* (in)**	**(90% Confidence Interval)* (in)**	**24-Hour Rainfall Depth Increased by 7 Percent and Used in this DBPS (in)**
2-Year	1.92	(1.652.26)	**2.05**
5-Year	2.44	(2.092.88)	**2.61**
10-Year	2.93	(2.503.48)	**3.14**
25-Year	3.71	(3.094.65)	**3.97**
50-Year	4.38	(3.545.54)	**4.69**
100-Year	5.11	(3.976.63)	**5.47**
*http://hdsc.nws.noaa.gov/hdsc/pfds for Latitude: 38.8406° Longitude: -104.6332° Elevation (ft): 6148.29 ft			

3.13.2 Temporal Distributions

Temporal distributions define the pattern with which rainfall depth/volume is simulated in a model. It defines peak rainfall intensities and directly influences excess rainfall in relation to infiltration rates. While Atlas 14 provides Point Precipitation Estimates, it does not provide guidance on temporal distributions that can be used for engineering purposes. As part of this DBPS, the County requested a new rainfall temporal distribution be developed based on depth-duration-frequency (DDF) data from the Atlas 14 and guidance

[1] U.S. Climate Resilience Toolkit, Climate Explorer Graph Projections:https://crt-climate-explorer.nemac.org/climate_graphs/?city=Colorado+Springs%2C+CO&county=El%2BPaso%2BCounty&area-id=08041&fips=08041&zoom=7&lat=38.8338816&lon=-104.8213634

provided in Chapter 4 of the National Engineering Handbook (NEH) Part 630 Hydrology (NRCS, 2019) for a nested storm temporal distribution. This allows for a singular temporal distribution that is applicable for various watershed sizes within the Basin using procedures that are widely understood and well documented. Depth-Area Reduction Factors (DARFs) are not used given the generally uncertainty associated with rainfall frequency depths. The development of these distributions is detailed in the El Paso County Rainfall Distribution Technical Memorandum (July 2021) provided in Appendix D.

3.14 Hydraulic Routing

In addition to the development of hydrologic runoff, SWMM 5 was also used to route runoff through the drainageway channels to the outlet of Jimmy Camp Creek at its confluence with Fountain Creek. This section discusses the development of the hydraulic routing parameters used in the SWMM model analysis.

3.14.1 Routing Method

For SWMM 5, the St. Venant equations govern the translation of flows through the Basin. There are three options available in SWMM 5 for modeling a basin of this scale: steady-state, the Kinematic Wave, and the Dynamic Wave solutions, with the Kinematic and Dynamic Wave solutions using the St. Venant equations.

The Kinematic Wave solution does not solve the full St. Venant equations as it removes the inertial and pressure terms. This means that the Kinematic Wave solution does not account for downstream boundary conditions, backwater impacts, flow reversal, pressurized flow, or losses through model junctions. The Kinematic Wave solution is generally applicable to steeply-sloped conduits with shallow flows and higher velocities. It is typically used to reduce model run times compared to Dynamic Wave simulations and reduce the potential for model instabilities in cases where downstream boundary conditions would not impact model results. With the Kinematic Wave solution, only one outlet is allowed per node, meaning that diversion structures and other assumptions need to be applied for flow splits and dual drainage. The Kinematic Wave method also does not produce realistic hydraulic grade line (HGL) profiles.

The Dynamic Wave solution solves the full St. Venant equations, which accounts for downstream conditions, backwater impacts, pressurized flow, and losses through junctions. Therefore, the Dynamic Wave solution produces more theoretically accurate results as compared to the Kinematic Wave solution. The Dynamic Wave solution will also produce more theoretically accurate results and more realistic HGL profiles. For this DBPS, hydraulic routing was performed with the Dynamic Wave solution.

3.14.2 Routing Parameters

The general approach for this DBPS was to route flows in a manner to achieve reasonable results to meet the objectives of the DBPS. This section describes the routing parameters used to route hydrologic design flows through the Jimmy Camp Creek Drainage Basin.

3.14.2.1 Open Channels

The majority of model conveyance routing throughout the Jimmy Camp Creek Drainage Basin is via open channels. Most of these open channels are reflected in the SWMM 5 model as irregular conduits with associated cross section geometry taken from the 2018 DEM.

Channel alignments for modeling were based on the County's GIS data, the SIMP data (depression storage for impervious sub-areas), FEMA data, and 2018 aerial photography. Model nodes, in addition to design points and the corresponding channel segments, were used where there are appreciable changes in channel geometry, or channel slope.

Channel cross sections were cut along the channel (left to right looking downstream) to represent the irregular shape of the channel geometry. If a node was placed for a slope change, and the cross-section shape did not change between the upstream and downstream sections of channel, then the same cross section was assigned to both links.

There are also several instances of conveyance locations with undefined channels in the study area. These conveyance elements were represented as trapezoidal channels with the corresponding channel width and side slopes based on aerial photography.

Upstream and downstream channel invert elevations were extracted from the 2018 DEM and were assigned to reflect the hydraulic conditions of the channel. Invert elevations may not match the underlying DEM at specific locations due to simplification of the geometry and invert profile.

The El Paso County DCM Vol 1 (EPC, 2018) was used to select typical Manning's roughness values for channel segments. Average roughness values were used to represent entire channel reaches, with most open channels being assigned a Manning's roughness of 0.04 and concrete pipe culverts a roughness of 0.013.

3.14.2.2 Road Crossings

Jimmy Camp Creek main stem and its tributaries have numerous road crossings, with the majority being box culverts, circular culverts, or bridge road crossings. The first source of determining culvert geometry was the County stormwater GIS data. Other sources of information included County field measurements, as-built information, and CDOT inspection data. In the absence of the above information, culvert sizes were estimated from the aerial photography. Bridges were simplified and represented as either large box culverts or trapezoidal channels, whichever was more applicable.

Upstream and downstream minor conduit losses were assigned accordingly. For channels discharging into a culvert, Federal Highways Administration (FHWA) inlet types were assigned. If there are multiple culverts at a crossing, then the number of barrels in the culvert were included in the model.

Upstream and downstream invert elevations were extracted from the 2018 LiDAR DEM. For instances where preliminary model runs for 100-year flows indicated overtopping at a road crossing, overflow weirs were added to the model at the road surface so that peak flows would not be over attenuated.

3.14.3 Storage

Throughout the Basin, in both developed and undeveloped areas, there are several locations of detention that include local development ponds, stock ponds, reservoirs, and natural depressions. However, only a few locations of detention are specifically modeled as either storage elements within SWMM (requiring storage curves) or as channel cross-sections. These ponds are located either in line with or at the upstream end of modeled channel reaches. For the ponds that were modeled explicitly as storage areas, storage-elevation curves were developed from 2018 contours. Pond outlet characteristics were estimated from aerial photography and 2018 LiDAR. Detention areas that are on private property were excluded from the routing. The modeling schematic used for the Jimmy Camp Creek DBPS is presented in Figure 3-4.

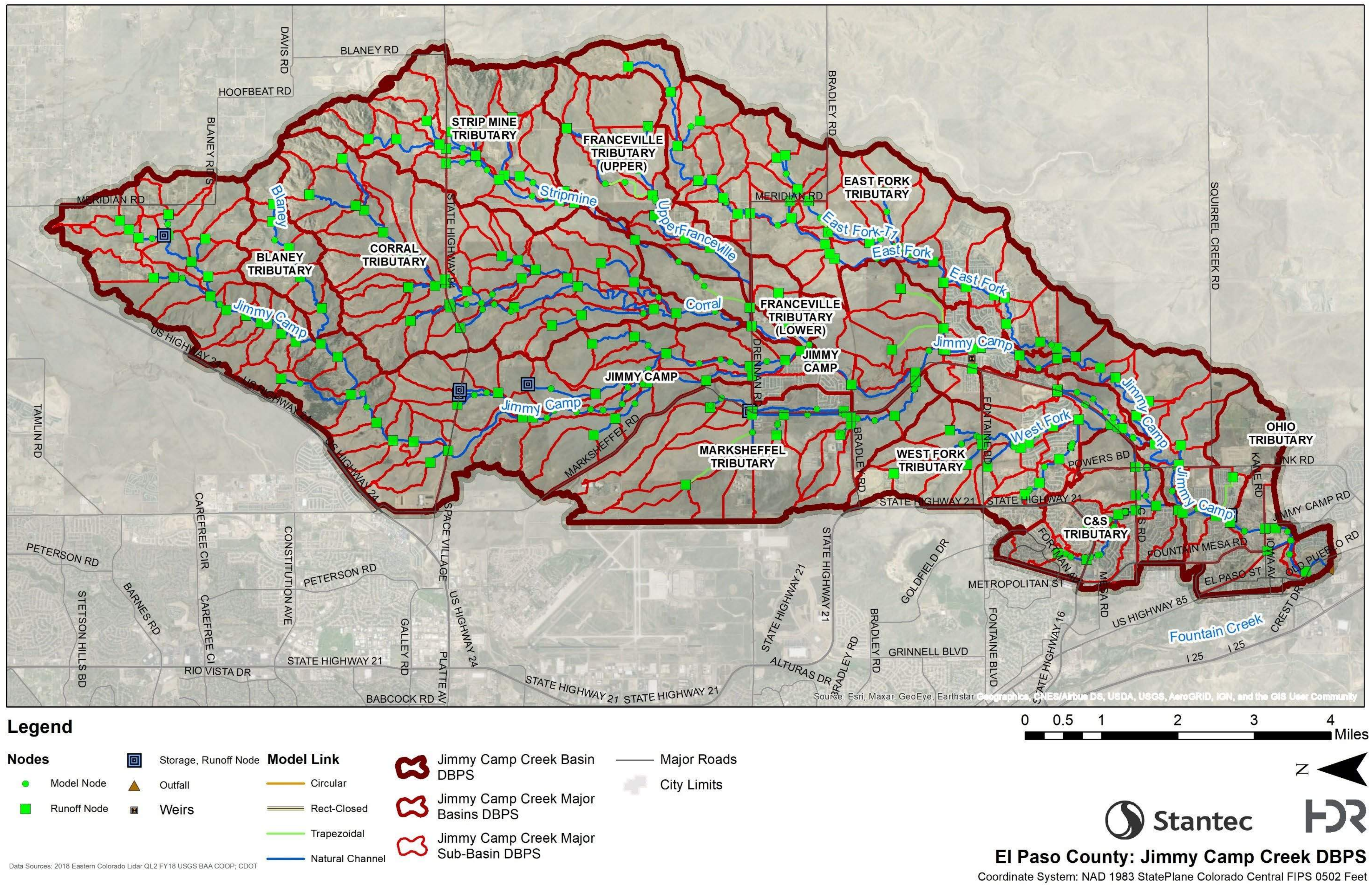


Figure 3-4. Schematic Routing Map

3.15 Existing Condition Model Results

The existing conditions model was executed for the 2-, 5-, 10-, 25-, 50-, and 100-year rainfall depths presented in Table 3-4. Existing condition sub-basin runoff model results and channel flows at key analysis points were analyzed and are provided in this section.

3.15.1 Existing Condition Sub-Basin Model Results

Sub-basin existing condition peak runoff flows and total runoff volumes are provided on a sub-basin map contained in Appendix C.

3.15.2 Existing Condition Flow Rates through the DBPS

Analysis points throughout the entire DBPS area were chosen for the presentation of flow rates. These analysis points are presented in Figure 3-5. Flows for these corresponding analysis points are provided in Table 3-5.

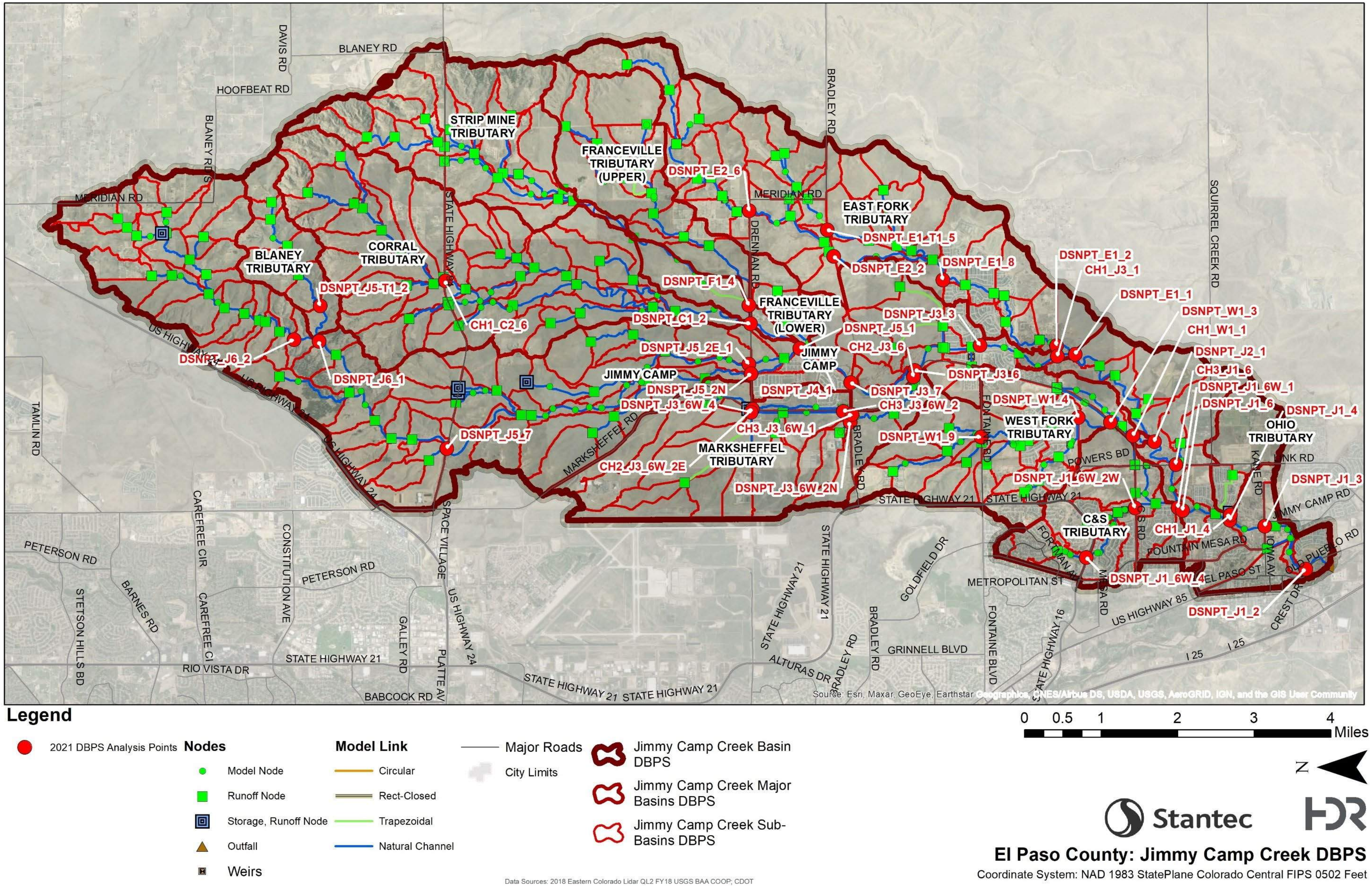


Figure 3-5. Analysis Points used in the Jimmy Camp Creek DBPS for the Presentation of Peak Flow Rates

Table 3-5. Existing Conditions Peak Flow Rates at Analysis Points

Major Drainageway	Model Node ID	Location Description	Contributing Area (mi^2)	100-Year Flow (cfs)	50-Year Flow (cfs)	25-Year Flow (cfs)	10-Year Flow (cfs)	5-Year Flow (cfs)	2-Year Flow (cfs)	Notes
Jimmy Camp Creek	DSNPT_J5_2E_1	Drennan Rd (East)	2.24	388	239	122	42	20	11	
	DSNPT_J6_2	Upstream of confluence with Blaney Trib	4.82	913	467	242	122	82	56	
	DSNPT_J6_1	Confluence with Blaney	6.51	1,290	623	304	148	99	68	
	DSNPT_J5_7	State Highway 94	9.51	1,392	646	307	145	96	64	
	DNSPT_J5_2N	Drennan Rd (West)	13.31	1,514	766	366	150	95	62	
	DSNPT_J5_1	Confluence with Corral Trib	33.99	6,644	3,989	2,212	914	462	215	
	DSNPT_J4_1	Confluence with Franceville Trib	34.70	6,712	4,002	2,221	920	465	217	
	DSNPT_J3_7	Bradley Rd	35.47	6,570	4,013	2,211	925	470	235	
	DSNPT_J3_6	Confluence with Marksheffel	41.75	7,124	4,343	2,352	1,027	520	249	
	DSNPT_J3_3	Fountain Blvd	43.97	7,248	4,424	2,390	1,049	538	360	
	CH1_J3_1	Peaceful Valley Rd	44.29	7,161	4,388	2,366	1,034	533	303	
	DSNPT_E1_1	Confluence with East Fork	53.57	8,036	4,923	2,643	1,028	529	305	Long, flat, rough slope to next design point
	DSNPT_J2_1	Confluence with West Fork	59.32	7,935	4,851	2,639	1,033	639	372	
	CH3_J1_6	Link Rd	60.22	7,874	4,792	2,523	923	483	268	
	DSNPT_J1_6	Confluence with C and S Trib	63.25	8,745	5,350	2,817	1,081	578	395	
	DSNPT_J1_3	Ohio Ave	65.36	8,715	5,295	2,798	1,087	582	365	
	DSNPT_J1_2	Outfall to Fountain Creek	66.51	8,731	5,204	2,642	1,088	586	367	
East Fork Tributary	DSNPT_E1_T1_5	Bradley Rd (East)	0.30	172	121	78	34	15	4	
	DSNPT_E2_6	Drennan Rd	2.45	486	215	97	66	49	34	
	DSNPT_E2_2	Bradley Rd (West)	4.42	568	267	112	51	33	22	
	DSNPT_E1_8	At City of Colorado Springs Boundary	6.89	699	432	252	104	52	25	
	DSNPT_E1_2	Upstream of Confluence with JCC (Peaceful Valley Rd)	8.92	1,087	660	308	85	27	19	
West Fork Tributary	DSNPT_W1_9	Fountain Blvd	1.23	271	162	92	40	21	15	
	DSNPT_W1_4	Mesa Ridge Pkwy	3.44	881	689	538	383	297	209	
	DSNPT_W1_3	S Marksheffel Rd	3.94	701	425	260	175	127	104	Tailwater condition due to Marksheffel culvert impacting peak flow rates
	CH1_W1_1	Upstream of Confluence with JCC	4.33	134	118	105	86	71	55	The culvert under Marksheffel Road is 24"and does not have the capacity to carry the flow. Overflows to this culvert continue along the Marksheffel ditch to the southwest and crosses under Marksheffel Rd at the crossing just east of Link Rd (within in the C&S Tributary).
Corral Tributary	CH1_C2_6	State Highway 94	3.93	2,719	1,861	1,119	500	267	133	
	DSNPT_C1_2	At confluence with Stripmine Trib (Upstream of confluence with JCC)	18.00	4,876	3,140	1,764	753	374	156	

Table 3-5. Existing Conditions Peak Flow Rates at Analysis Points (continued)

Major Drainageway	Model Node ID	Location Description	Contributing Area (mi^2)	100-Year Flow (cfs)	50-Year Flow (cfs)	25-Year Flow (cfs)	10-Year Flow (cfs)	5-Year Flow (cfs)	2-Year Flow (cfs)	Notes
Stripmine Tributary	DSNPT_S1_7	State Highway 94	1.40	1,972	1,492	1,070	630	380	179	
Franceville Tributary	DSNPT_F1_4	Confluence of Franceville and Stripmine	8.18	2,236	1,432	820	359	180	68	
Marksheffel Tributary	DSNPT_J3_6W_2N	Bradley Rd (West)	0.67	794	576	398	228	150	90	Small, separate west tributary at Bradley Rd; Connects with main trib. approximately 360' downstream of CH3_J3_6W_1
	CH2_J3_6W_2E	Drennan Rd (East)	1.31	335	183	94	34	12	5	
	CH3_J3_6W_2	Marksheffel Rd (North of Bradley Rd)	1.64	268	210	165	119	94	67	Flows reduced by Marksheffel Rd crossing
	DSNPT_J3_6W_4	Drennan Rd (West)	1.93	430	264	157	83	58	38	Not on the same tributary as CH3_J3_6W_2
	CH3_J3_6W_1	Bradley Rd (East)	4.58	933	587	377	229	156	96	
	CH2_J3_6	Upstream of Confluence with JCC	5.88	1,355	1,063	723	363	261	144	
Blaney Tributary	DSNPT_J5-T1_2	Upstream of Confluence with JCC	1.32	459	263	145	70	45	32	
C&S Tributary	DSNPT_J1_6W_4	Mesa Ridge Pkwy	0.50	648	595	473	335	257	183	
	DSNPT_J1_6W_2W	C and S Rd	1.50	617	474	342	247	198	144	Flows controlled by a large detention pond upstream (Cross Creek Park pond)
	DSNPT_J1_6W_1	Upstream of Confluence with JCC	2.74	1,595	1,288	1,022	724	549	379	
Ohio Trib.	CH1_J1_4	Upstream of Confluence with JCC	1.06	119	107	103	118	114	46	Flows controlled by a large detention pond upstream

3.16 Hydrologic Model Comparison to other studies

The Jimmy Creek Drainage Basin has been studied in the past. There are four known sources of published flow rates available for the Jimmy Creek Drainage Basin:

- The Federal Emergency Management Agency (FEMA) Flood Insurance Study (FIS) for El Paso County (FEMA, 2018).
- The 2015 Jimmy Creek Drainage Basin Planning Study (Kiowa, 2015).
- The USGS Paleoflood Investigations to Improve Peak-Streamflow Regional-Regression Equations for Natural Streamflow in Eastern Colorado (USGS, 2016).
- The USGS streamgage on Jimmy Camp Creek at Fountain, Colorado

  (https://waterdata.usgs.gov/co/nwis/uv?site_no=07105900).

This section describes the updated existing conditions flow rates in the context of this published data.

3.16.1 2018 FEMA FIS

Existing condition flow rates from the current model were compared to the published flow rates in the FEMA FIS (FEMA, 2018). According to the FIS, these flows were obtained from previous studies. The FIS states that hydrologic analyses used for Jimmy Camp Creek and its tributaries came from the United States Department of Agriculture, the Soil Conservation Service (SCS), and Flood Hazard Analysis reports. It also states that the SCS report used the SCS hydrologic method to develop peak flows for the 10-, 50-, and 100-year recurrence interval rainfall events with the 5-year obtained by a log-probability extrapolation from the other events. SCS hydrologic methods were used due to inadequate streamflow data and it complied with state statutory requirements at that time. The FIS bibliography references are listed below:

- U.S. Department of Agriculture, Soil Conservation Service, Colorado Water Conservation Board, Flood Hazard Analyses, Portions of Jimmy Camp Creek and Tributaries, El Paso County, Colorado, October 1975.
- U.S. Department of Agriculture, Soil Conservation Service, in Cooperation with the Colorado Water Conservation Board, Flood Hazard Analysis, Sand Creek, City of Colorado Springs and El Paso County, Colorado, July 1973.

The FIS further states that for flows below Peaceful Valley Road, the peak flow rates were developed by the USACE from the following report:

- U.S. Department of the Army, Corps of Engineers, Albuquerque District, Report on Hydrologic Investigations for the Flood Insurance Study of Colorado Springs and El Paso County, Colorado, December 1976.

Flow rates from the current modeling are lower than the FIS in all modeled tributaries within the Jimmy Camp Creek basin for 10-, 50-, and 100-year flows. Table 3-6 compares the FIS flows to the current DBPS model flows. The primary reasons for the differences are that this DBPS includes detailed routing (more refined cross-section and crossing data), more robust routing methods (dynamic wave currently being used that better represents in-channel storage), more detailed infiltration modeling (impervious percentage and

Horton infiltration modeling versus Curve Numbers), and different runoff computations (non-linear reservoir routing versus unit hydrograph).

Table 3-6. Flow Rates for the Current DBPS Compared to the 2018 FEMA FIS.

Drainageway	Location	FEMA Drainage Area (mi^2)	Current DBPS Drainage Area (mi^2)	10-Year Maximum Flow				50-year Maximum Flow				100-Year Maximum Flow				Location Notes
				FEMA FIS (cfs)	Current DBPS Existing Conditions Model (cfs)	Difference (cfs)	% Difference ***	FEMA FIS (cfs)	Current DBPS Existing Conditions Model (cfs)	Difference (cfs)	% Difference ***	FEMA FIS (cfs)	Current DBPS Existing Conditions Model (cfs)	Difference (cfs)	% Difference ***	
Jimmy Camp Creek	At Confluence with Fountain Creek	66.4	66.51	8,500	1088	-7,412	-155%	12,400	5,204	-7,196	-82%	16,000	8,731	-7,269	-59%	
East Tributary	At Confluence with Jimmy Camp Creek	9.2	8.92	2,800	85	-2,715	-188%	4,600	660	-3,940	-150%	5,500	1,087	-4,413	-134%	Location is upstream of confluence
West Tributary	At Confluence with Jimmy Camp Creek	3.93	4.33	1,160	86	-1,074	-172%	2,280	118	-2,162	-180%	2,780	134	-2,646	-182%	Location is upstream of confluence
Franceville Tributary*	At Confluence with Jimmy Camp Creek	4.1	3.03	1,700	359	-1,341	-130%	2,800	1,432	-1,368	-65%	3,500	2,236	-1,264	-44%	Downstream area diverted to the Strip Mine Tributary for the present analysis
Corral Tributary**	At Confluence with Jimmy Camp Creek	15.9	18	3,800	753	-3,047	-134%	6,000	3,140	-2,860	-63%	7,300	4,876	-2,424	-40%	Corral Tributary includes the Strip Mine and Franceville Tributaries for the present analysis

*Franceville Tributary does not have a means to cross Drennan Road; Topography indicates that it now drains to the Strip Mine tributary, which then drains to the Corral Tributary

**Corral Tributary now includes the Strip Mine and Franceville Tributaries

*** % difference = Difference between the flow rates divided by the average of the flow rates

3.16.2 2015 Jimmy Camp Creek DBPS

Existing condition flow rates from the current DBPS model were compared to the published flow rates from the 2015 DBPS developed for Colorado Springs (Kiowa, 2015). The 2015 DBPS used the SCS curve number method with a SCS Type II rainfall distribution, with hydraulics routed in HEC-HMS. Table 3-7 compares the 2015 DBPS design point flows to the current DBPS model flows. Flow rates from the current study are lower than the corresponding flow rates presented in the 2015 DBPS. Similar to the FEMA results, the primary reasons for the differences are that this DBPS includes more detailed routing (more refined cross-section and crossing data), more robust routing (dynamic wave currently being used that better represents in-channel storage versus Muskingum-Cunge), more detailed infiltration modeling (impervious percentage and Horton infiltration modeling versus Curve Numbers), and different runoff computations (non-linear reservoir routing versus unit hydrograph).

Table 3-7. Flow Rates for the Current DBPS Compared to the 2015 DBPS

Drainageway*	Location	2015 DBPS Drainage Area (mi^2)	Current DBPS Drainage Area (mi^2)	10-Year Maximum Flow: 2015 DBPS Existing (cfs)	10-Year Maximum Flow: Current DBPS Existing Conditions Model (cfs)	10-Year Maximum Flow: Difference (cfs)	10-Year Maximum Flow: % Difference**	100-Year Maximum Flow: 2015 DBPS Existing 100-year (cfs)	100-Year Maximum Flow: Current DBPS Existing Conditions Model (cfs)	100-Year Maximum Flow: Difference (cfs)	100-Year Maximum Flow: % Difference**	Location Notes
Jimmy Camp Creek	at Fountain Creek	67.11	66.51	9,443	1,088	-8,355	-159%	22,094	8,731	-13,363	-87%	Outfall to Fountain Creek
	at Ohio Ave	66.11	65.36	9,447	1,087	-8,360	-159%	22,139	8,715	-13,424	-87%	Ohio Ave
	at Link Rd	60.93	60.22	9,310	923	-8,387	-164%	21,878	7,874	-14,004	-94%	Link Rd
	at West Fork	59.77	59.32	9,296	1,033	-8,263	-160%	21,875	7,935	-13,940	-94%	Confluence with West Fork
	at East Fork	53.92	53.57	9,243	1,028	-8,215	-160%	21,784	8,036	-13,748	-92%	Confluence with East Fork
	at Peaceful Valley Rd	44.16	44.29	7,731	1,034	-6,697	-153%	17,790	7,161	-10,629	-85%	Peaceful Valley Rd
	at Marksheffel Trib	41.99	41.75	7667	1,027	-6,640	-153%	17,361	7,124	-10,237	-84%	Confluence with Marksheffel
	at Bradley Rd	36.64	35.47	7153	925	-6,228	-154%	16,502	6,570	-9,932	-86%	Bradley Rd
	at Franceville Trib	36.19	34.7	7,116	920	-6,196	-154%	16,422	6,712	-9,710	-84%	Confluence with Franceville Trib
	at Corral Trib	31.6	33.99	6,834	914	-5,920	-153%	15,382	6,644	-8,738	-79%	Confluence with Corral Trib
	at Drennan Rd	14.84	13.31	2,509	150	-2,359	-177%	5,881	1,514	-4,367	-118%	Drennan Rd (West)
	at State Highway 94	9.62	9.51	2,300	145	-2,155	-176%	5,031	1,392	-3,639	-113%	State Highway 94
	at Blaney Trib	6.39	6.51	1,959	148	-1,811	-172%	4,107	1,290	-2,817	-104%	Confluence with Blaney
	U/S of Blaney	4.67	4.82	1,254	122	-1,132	-165%	2,773	913	-1,860	-101%	Upstream of confluence with Blaney Trib
Jimmy Camp Creek Corral Trib.	at Jimmy Camp Creek	8.25	18	2885	753	-2,132	-117%	6,212	4,876	-1,336	-24%	At confluence with Stripmine Tributary (Upstream of confluence with JCC)
Jimmy Camp Creek East Fork Trib.	at Jimmy Camp Creek	9.77	8.92	2030	85	-1,945	-184%	4,677	1,087	-3,590	-125%	Upstream of Confluence with JCC (Peaceful Valley Rd)
Jimmy Camp Creek Marksheffel Trib.	at Jimmy Camp Creek	5.18	5.88	832	363	-469	-78%	1,916	1,355	-561	-34%	Upstream of Confluence with JCC
Jimmy Camp Creek Stripmine Trib.	at Jimmy Camp Creek	5.18	N/A	2451	N/A	N/A	N/A	4,627	N/A	N/A	N/A	Stripmine Tributary combines with Franceville Tributary for the present analysis. There is not a separate design point that represents Stripmine Trib

Table 3-7. Flow Rates for the Current DBPS Compared to the 2015 DBPS (continued)

Drainageway*	Location	2015 DBPS Drainage Area (mi^2)	Current DBPS Drainage Area (mi^2)	10-Year Maximum Flow				100-Year Maximum Flow				Location Notes
				2015 DBPS Existing (cfs)	Current DBPS Existing Conditions Model (cfs)	Difference (cfs)	% Difference**	2015 DBPS Existing 100-year (cfs)	Current DBPS Existing Conditions Model (cfs)	Difference (cfs)	% Difference**	
Jimmy Camp Creek Franceville Trib.	at Jimmy Camp Creek	4.32	8.18	640	359	-281	-56%	1,515	2,236	721	38%	Confluence of Franceville and Stripmine, Stripmine Tributary combines with Franceville Tributary for the present analysis
Jimmy Camp Creek C&S Trib.	at Jimmy Camp Creek	2.07	2.74	898	724	-174	-21%	1,770	1,595	-175	-10%	Upstream of Confluence with JCC
Jimmy Camp Creek Blaney Trib.	at Jimmy Camp Creek	1.55	1.32	1102	70	-1,032	-176%	1,927	459	-1,468	-123%	Upstream of Confluence with JCC
Jimmy Camp Creek Ohio Trib.	at Jimmy Camp Creek	1.22	1.06	268	118	-150	-78%	661	119	-542	-139%	Upstream of Confluence with JCC

*West Fork Tributary not included in DBPS 2015 and is not included here for comparison

** % difference = Difference between the flow rates divided by the average of the flow rates

3.16.3 USGS Flood Frequency Analysis

In 2016, the USGS, in cooperation with the Colorado Department of Transportation, developed regional regression equations for Eastern Colorado (Kohn et al, 2016). As part of this Scientific Investigations Report (SIR), flood frequency statistics were produced for the one known stream gage in the Jimmy Camp Creek Drainage Basin, which is the USGS gage 07105900 located at the downstream end of Jimmy Camp Creek near its mouth to Fountain Creek. Table 3-8 provides the published results of this analysis at the gage as developed by the USGS. The Expected Moments Algorithm (EMA) with the multiple Grubbs-Beck (MGB) test method (Grubbs and Beck, 1972) was used to compute Log-Pearson Type III exceedance-probability estimates for the gage. Compared to this SIR, the current modeling produced peak flows that are less than the SIR published flow rates for 2-, 5-, 10-, 25-, 50-, and 100-year flood frequencies. The primary cause for this discrepancy in peak flows is that the SIR used a flood that occurred in 1965 that was outside the period of record of the USGS Gage 07105900, The 1965 estimated peak flow was 124,000 cfs and this flood is discussed in the next section.

Table 3-8. USGS Bulletin 17B Gage Analysis for Jimmy Camp Creek at USGS Gage 07105900

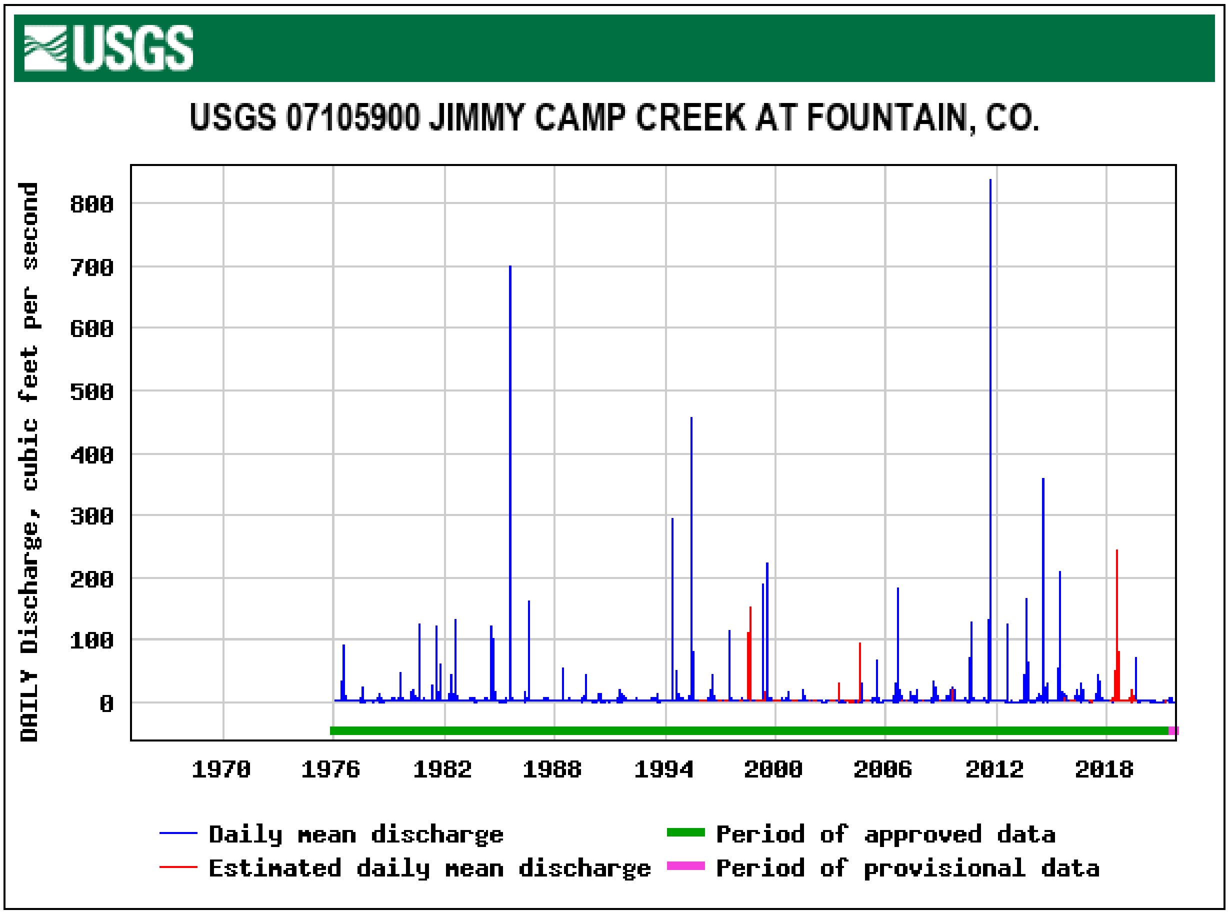
	USGS Flow Statistics Flow Rate (cfs)*	USGS Confidence Interval**		Current DBPS Existing Conditions Model – Jimmy Camp Creek at Ohio Avenue
		Lower 95% Confidence Limit	Upper 95% Confidence Limit	
2 Year Peak Flood (50 Percent Annual Chance)	516	305.7	900.8	365
5 Year Peak Flood (20 Percent Annual Chance)	2,050	1,141	4,365	582
10 Year Peak Flood (10 Percent Annual Chance)	4,470	2,292	12,140	1,087
25 Year Peak Flood (4 Percent Annual Chance)	10,800	4,789	45,120	2,798
50 Year Peak Flood (2 Percent Annual Chance)	19,500	7,661	121,300	5,295
100 Year Peak Flood (1 Percent Annual Chance)	33,900	11,640	325,500	8,715

*Source: Kohn et. al., 2016, Appendix 5

**Source: USGS StreamStats Data-Collection Station Report, https://streamstatsags.cr.usgs.gov/gagepages/html/07105900.htm#300

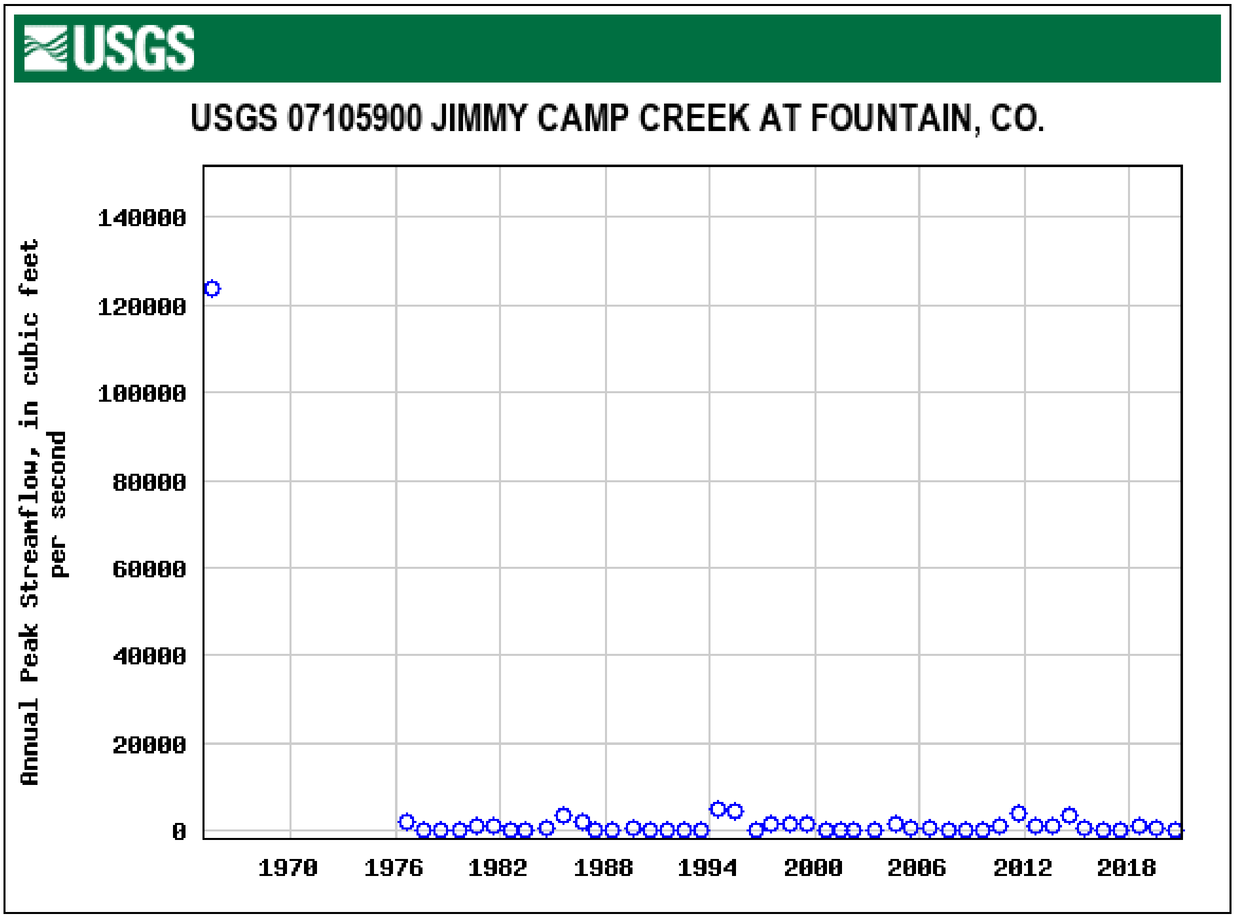
3.16.4 USGS Gage 07105900

As stated in Section 3.16.3, there is one known stream gage in the Jimmy Camp Creek Drainage Basin, (USGS gage 07105900). Gage 07105900 has been in operation since 1976. However, the annual statistics include the flood event of 1965. Figure 3-6 presents the flow data corresponding to this gage over its entire period of record (1976 through 2021). Figure 3-7 provides the graph of the annual peaks associated with the gage, which includes the estimated discharge associated with the 1965 flood event. The discharge in 1965 was calculated by a post flood analysis by USGS who performed a two-section slope-area measurement by U.S. Geological Survey (Costa and Jarrett, 2008).



Source:
https://nwis.waterdata.usgs.gov/co/nwis/uv/?cb_00060=on&cb_00065=on&format=gif_stats&site_no=07105900&period=&begin_date=1960-01-01&end_date=2021-09-23 .

Figure 3-6. Flow Data at the USGS gage 07105900 from 1976 – 2021.



Source: https://nwis.waterdata.usgs.gov/co/nwis/peak/?site_no=07105900&agency_cd=USGS

Figure 3-7. Annual Peak Flow Data at the USGS Gage 07105900 from 1965 – 2021

It should be noted that the annual peak flow associated with the gage in 1965 is a significant outlier with a peak flow of 124,000 cfs. The next closest peak flow occurred in 1994 with a peak flow of 4,810 cfs. It should also be noted that within Fountain Creek, at USGS gage 0710600 downstream of the Jimmy Camp Creek confluence, the highest recorded peak flow is 22,100 cfs, which was recorded in 1940. There have also been peaks of 20,100 cfs (1999), 21,000 cfs (2015), and 20,300 cfs (2023) at the Fountain Creek gage, at which time the peaks on Jimmy Camp Creek were 1,710 cfs (1999), 882 cfs (2015), and 2,040 cfs (2023). The Fountain Creek gage has been in operation from 1939 – 1955 and 1985 – 2021, so unfortunately, the 1965 event was not captured. The 124,000 cfs measurement taken on Jimmy Camp Creek is well above any peak flow recorded both within Jimmy Camp Creek and its receiving water of Fountain Creek, both of which have significant periods of record. The 1965 flood event is described in detail by the USGS in SIR 2008-5164, An Evaluation of Selected Extraordinary Floods in the United States Reported by the U.S. Geological Survey and Implications for Future Advancement of Flood Science (Costa and Jaret, 2008). This report reviews the methodology and reliability of the1965 peak flow rate of 124,000 cfs and gives this measurement a ‘poor’ rating. Annual peak flows with the 1965 event excluded is presented in Figure 3-8.

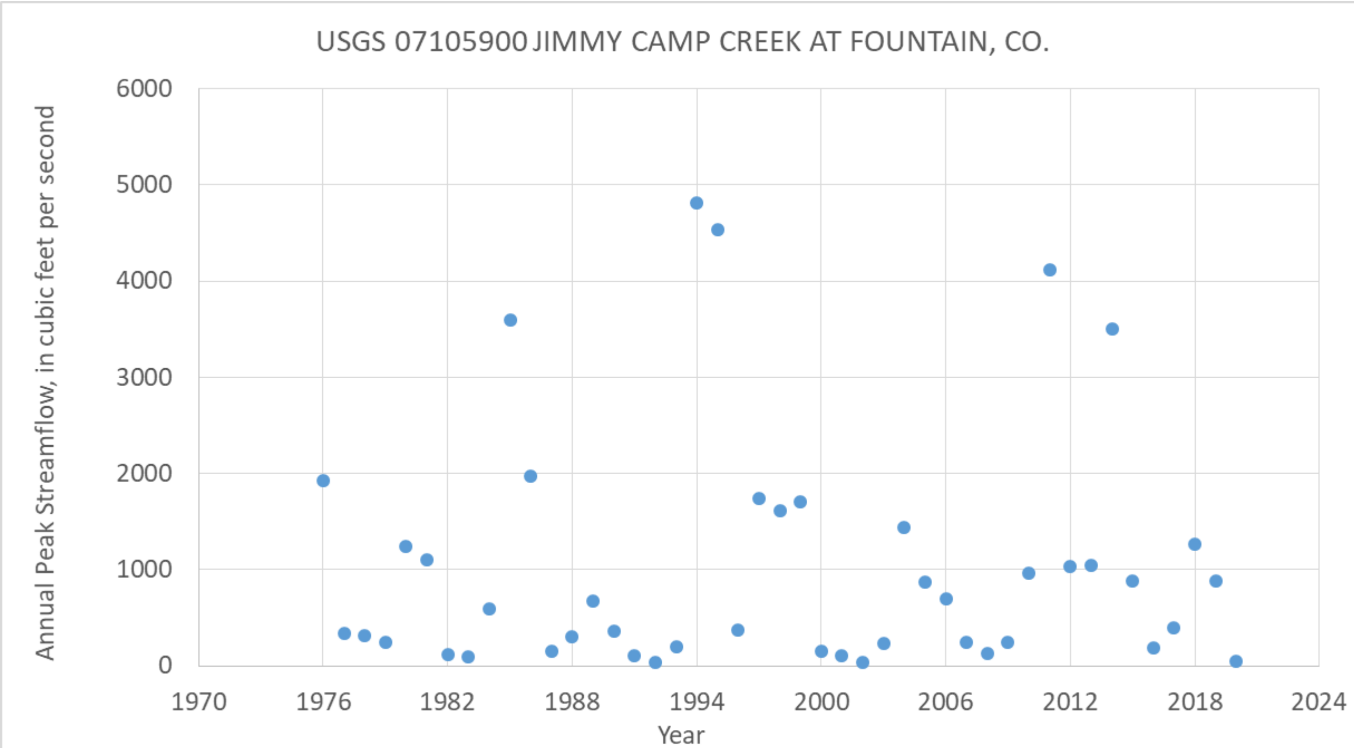


Figure 3-8. Annual Peak Flow Data at the USGS Gage 07105900 from 1976– 2021

Given how the 124,000 cfs flow impacts the USGS flood frequency statistics, a Bulletin 17 Log-Pearson Type II frequency analysis was performed to exclude the 1965 event. The analysis was performed using the United States Army Corps of Engineers (USACE) Hydrologic Engineering Center's (HEC) Statistical Software Package (HEC-SSP) (version 2.2) based on the flow data on record at the gage. HEC-SSP is used to develop flood flow frequency analysis based on Bulletin 17B (Interagency Advisory Committee on Water Data, 1982) and Bulletin 17C (England, et al., 2018). Both Bulletin 17B (consistent with the USGS analysis presented in Section 3.16.3) and Bulletin 17C analyses (more refined analysis) were performed on the gage data and are presented in Table 3-9. Bulletin 17B and 17C produced similar results for the computed flows, with the differences being in how confidence limits are calculated. Results of this analysis indicate that the currently modeled flow rates are close to what is indicated by the Bulletin 17 analysis for the 100-year flow rates. Rainfall frequencies do not always produce the same flood frequencies, especially for more frequent rainfall and flood events. Because the gage is located near the outlet of the Basin there are many factors that can influence the flow rates at this gage at its location at the downstream end of the watershed. These factors include the timing of rainfall, the distribution of rainfall over the watershed, antecedent moisture conditions, culvert blockages, etc. The daily rainfall amounts were compared to the measured peak stream flows that occurred that same day, and it was observed that there are numerous instances where the approximate 2- to 5- year 24-hour rainfall depths produced flow rates under 500 cfs.

Table 3-9. Results of a Bulletin 17 Analysis of the USGS Gage 07105900 Accounting for Only the Period of Gaged Flow

Percent Chance Exceedance	Rainfall Return	Bulletin 17B Flows (Using Years 1976 - 2021)			Bulletin 17C Flows (Using Years 1976 - 2021)			Current DBPS Existing Conditions Modeled Flows– Jimmy Camp Creek at Ohio Avenue
		Computed Curve	90% Confidence Interval		Computed Curve	90% Confidence Interval		
			Lower 95% Confidence Limit	Upper 95% Confidence Limit		Lower 95% Confidence Limit	Upper 95% Confidence Limit	
		Flow (cfs)	Flow (cfs)		Flow (cfs)	Flow (cfs)		Flow (cfs)
50	2-year	523	377	725	523	367	742	365
20	5-year	1,532	1,082	2,329	1,532	1,080	2,261	582
10	10-year	2,634	1,782	4,322	2,634	1,817	4,311	1,087
5	20-year	4,077	2,643	7,180	4,077	2,701	7,733	N/A
2	50-year	6,592	4,051	12,625	6,592	4,006	15,484	5,295
1	100-year	9,021	5,341	18,298	9,022	5,041	25,059	8,715

HEC-SSP was also used to perform a volume frequency analysis on USGS gage 07105900 as part of this DBPS. The results of this analysis are presented in Table 3-10. Results of this analysis indicate that the currently modeled 24-hour flow volumes are greater than what is indicated by the volume frequency analysis for all modeled flow rates.

Table 3-10. Results of a Volume Frequency Analysis of the USGS Gage 07105900 (1976 - 2021)

Percent Chance Exceedance	Rainfall Return	Volume Frequency Curve for a 24-Hour Period (ac-ft)	Current DBPS Existing Conditions Modeled Volumes – Jimmy Camp Creek at Ohio Avenue (ac-ft)
50	2-year	110	386
20	5-year	358	577
10	10-year	644	829
5	20-year	1,030	N/A
2	50-year	1,719	1,976
1	100-year	2,396	2,810

3.17 Future Condition Modeling

For this DBPS, future conditions modeling focuses on the changes in sub-basin impervious percentages due to projected build out conditions in undeveloped areas. Changes in sub-basin boundaries or slopes due to development grading are not included. Future stormwater infrastructure such as closed conduits, open channels, or detention ponds are also not included.

3.17.1 Future Land Use Development

As described in Section 3.7, the County provided future impervious estimates for the Jimmy Camp Creek DBPS. These future imperviousness estimates were developed by the County based on the El Paso County Master Plan currently being developed. The resulting future impervious coverage, or “hydrologic land use”, is presented in Figure 3-9. As with the existing conditions analysis, the future impervious feature class was intersected with the sub-basins in GIS and an area weighted impervious percentage was calculated for each sub-basin.

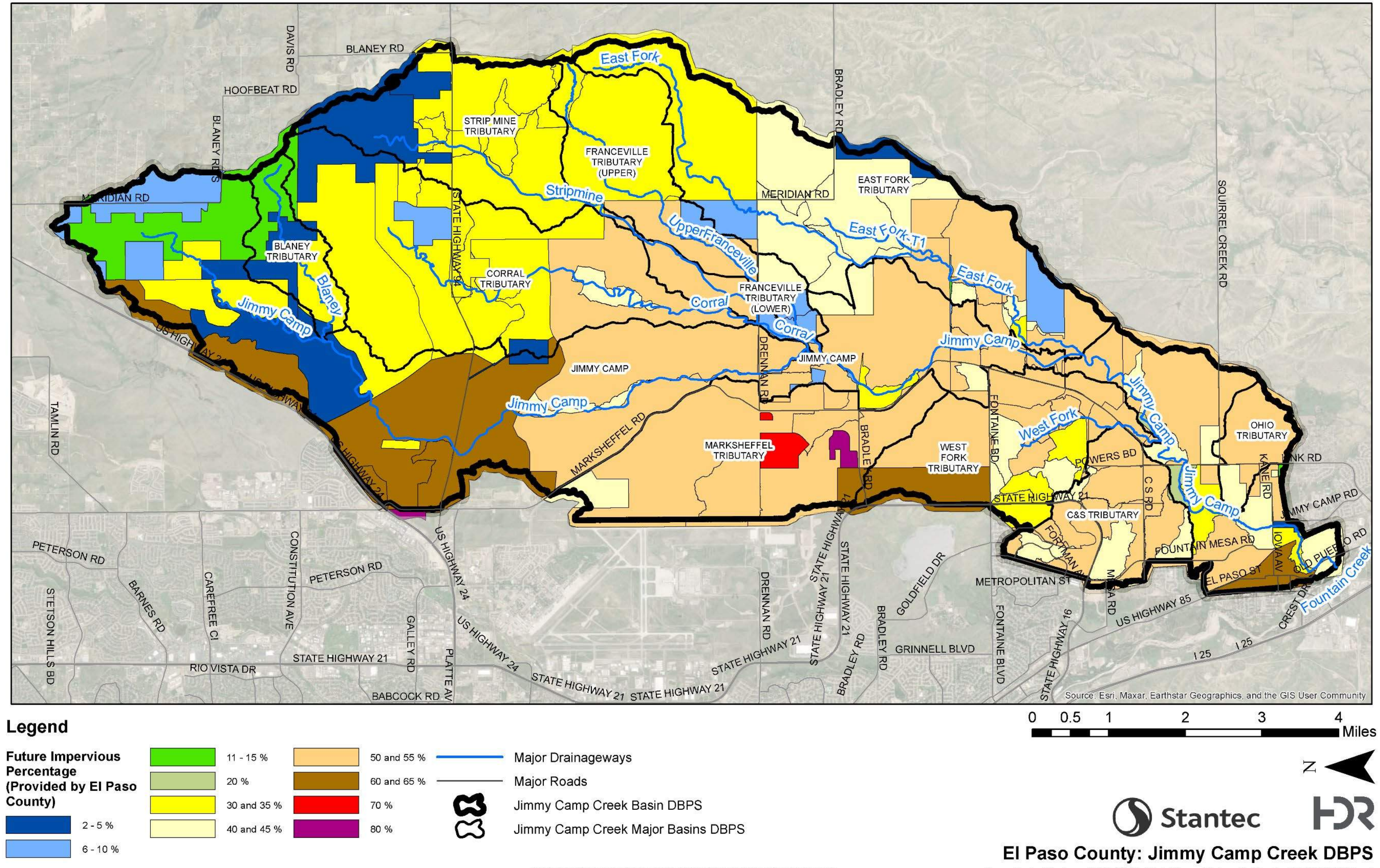


Figure 3-9. Future Impervious Percentage Map

3.17.2 Future Condition Model Results

The future conditions model was executed for the 2-, 5-, 10-, 25-, 50-, and 100-year rainfall depths presented in Table 3-4. The future conditions model was then evaluated for the 2-, 5-, 10-, 25-, 50-, and 100-year recurrence intervals.

The peak runoff flow and total runoff volumes from the Future Conditions Sub-Basin Model are provided in Appendix E. As with the Existing Conditions Sub-basin Model figure, this future conditions figure also provides model results at the sub-basin level throughout the DBPS area. Future conditions flow rates corresponding to the analysis points presented in Figure 3-5 are provided in Table 3-11. A comparison of existing and future 100-year flows is presented in Table 3-12.

Table 3-11. Future Conditions Peak Flow Rates at Analysis Points

Major Drainageway	Model Node ID	Location Description	Contributing Area (mi^2)	100-Year Flow (cfs)	50-Year Flow (cfs)	25-Year Flow (cfs)	10-Year Flow (cfs)	5-Year Flow (cfs)	2-Year Flow (cfs)	Notes
Jimmy Camp Creek Mainstem	DSNPT_J5_2E_1	Drennan Rd (East)	2.24	1,898	1,530	1,221	886	680	485	
	DSNPT_J6_2	Upstream of confluence with Blaney Trib	4.82	2,604	1,951	1,468	1,034	782	555	
	DSNPT_J6_1	Confluence with Blaney	6.51	3,602	2,661	2,001	1,377	1,020	727	
	DSNPT_J5_7	State Highway 94	9.51	5,925	4,490	3,430	2,340	1,755	1,256	
	DNSPT_J5_2N	Drennan Rd (West)	13.31	8,494	6,665	5,066	3,419	2,552	1,815	
	DSNPT_J5_1	Confluence with Corral Trib	33.99	21,838	16,944	12,601	8,623	6,414	4,443	
	DSNPT_J4_1	Confluence with Franceville Trib	34.70	22,038	17,074	12,672	8,664	6,139	4,452	
	DSNPT_J3_7	Bradley Rd	35.47	22,100	17,122	12,738	8,710	6,072	4,499	
	DSNPT_J3_6	Confluence with Marksheffel	41.75	26,498	20,788	15,718	10,766	7,701	5,715	
	DSNPT_J3_3	Fountain Blvd	43.97	26,998	21,007	15,465	11,077	7,951	5,874	
	CH1_J3_1	Peaceful Valley Rd	44.29	26,919	20,925	15,403	10,977	7,849	5,879	
	DSNPT_E1_1	Confluence with East Fork	53.57	29,722	23,080	17,157	12,156	8,669	6,362	Long, flat, rough slope to next design point
	DSNPT_J2_1	Confluence with West Fork	59.32	29,494	22,755	16,895	12,034	8,695	6,322	
	CH3_J1_6	Link Rd	60.22	29,423	22,666	16,831	11,982	8,684	6,295	
	DSNPT_J1_6	Confluence with C and S Trib	63.25	32,409	24,974	18,447	13,051	9,403	6,698	
	DSNPT_J1_3	Ohio Ave	65.36	30,358	24,729	18,430	13,107	9,492	6,754	
	DSNPT_J1_2	Outfall to Fountain Creek	66.51	27,655	24,197	18,373	13,101	9,505	6,696	
East Fork Tributary	DSNPT_E1_T1_5	Bradley Rd (East)	0.30	289	221	157	103	70	43	
	DSNPT_E2_6	Drennan Rd	2.45	1,593	1,305	1,042	754	576	404	
	DSNPT_E2_2	Bradley Rd (West)	4.42	1,645	1,472	1,255	992	783	581	
	DSNPT_E1_8	At City of Colorado Springs Boundary	6.89	2,540	1,942	1,507	1,035	763	541	
	DSNPT_E1_2	Upstream of Confluence with JCC (Peaceful Valley Rd)	8.92	3,630	2,813	1,990	1,243	873	593	
West Fork Tributary	DSNPT_W1_9	Fountain Blvd	1.23	1,385	1,067	815	593	460	330	
	DSNPT_W1_4	Mesa Ridge Pkwy	3.44	2,722	2,118	1,579	1,069	786	483	
	DSNPT_W1_3	S Marksheffel Rd	3.94	2,759	2,104	1,524	985	740	449	Tailwater condition due to Marksheffel culvert impacting peak flow rates
	CH1_W1_1	Upstream of Confluence with JCC	4.33	145	136	124	108	98	80	The culvert under Marksheffel Road is 24"and does not have the capacity to carry the flow. Overflows to this culvert continue along the Marksheffel ditch to the southwest and crosses under Marksheffel Rd at the crossing just east of Link Rd (within in the C&S Tributary).
Corral Tribu-tary	CH1_C2_6	State Highway 94	3.93	4,471	3,295	2,518	1,625	1,224	876	
	DSNPT_C1_2	At confluence with Strip Mine Trib (Upstream of confluence with JCC)	18.00	11,590	9,033	6,611	4,466	3,282	2,233	
Stripmine Tributary	DSNPT_S1_7	State Highway 94	1.40	2,122	1,636	1,187	714	445	221	

Table 3-11. Future Conditions Peak Flow Rates at Analysis Points (continued)

Major Drainageway	Model Node ID	Location Description	Contributing Area (mi²)	100-Year Flow (cfs)	50-Year Flow (cfs)	25-Year Flow (cfs)	10-Year Flow (cfs)	5-Year Flow (cfs)	2-Year Flow (cfs)	Notes
Franceville Tributary	DSNPT_F1_4	Confluence of Franceville and Strip Mine	8.18	4,925	3,884	2,902	1,929	1,394	911	
Marksheffel Tributary	DSNPT_J3_6W_2N	Bradley Rd (West)	0.67	2,146	1,862	1,577	1,197	941	669	Small, separate west tributary at Bradley Rd; Connects with main trib. approximately 360' downstream of CH3_J3_6W_1
	CH2_J3_6W_2E	Drennan Rd (East)	1.31	1,149	1,173	1,168	884	701	513	
	CH3_J3_6W_2	Marksheffel Rd (North of Bradley Rd)	1.64	1,185	1,062	851	608	513	399	Flows reduced by Marksheffel Rd crossing
	DSNPT_J3_6W_4	Drennan Rd (West)	1.93	2,744	2,211	1,752	1,262	982	708	Not on the same tributary as CH3_J3_6W_2
	CH3_J3_6W_1	Bradley Rd (East)	4.58	4,257	3,582	2,934	2,257	1,802	1,316	
	CH2_J3_6	Upstream of Confluence with JCC	5.88	4,768	4,011	3,160	2,316	1,866	1,377	
Blaney Tributary	DSNPT_J5-T1_2	Upstream of Confluence with JCC	1.32	852	618	467	326	249	183	
C and S Tributary	DSNPT_J1_6W_4	Mesa Ridge Pkwy	0.50	651	606	485	344	265	189	
	DSNPT_J1_6W_2W	C and S Rd	1.50	667	520	383	259	209	154	Flows controlled by a large detention pond upstream (Cross Creek Park pond)
	DSNPT_J1_6W_1	Upstream of Confluence with JCC	2.74	3,462	2,461	1,802	1,089	736	511	
Ohio Trib.	CH1_J1_4	Upstream of Confluence with JCC	1.06	320	174	163	151	139	133	Flows controlled by a large detention pond upstream

Table 3-12. Existing Condition Versus Future Conditions Peak Flow Rates at Analysis Points

Major Drainage Way	Model Node ID	Location Description	Contributing Area (mi^2)	Existing 100-Year Flow (cfs)	Future 100-Year Flow (cfs)
Jimmy Camp Creek	DSNPT_J5_2E_1	Drennan Rd (East)	2.24	388	1,898
	DSNPT_J6_2	Upstream of confluence with Blaney Trib	4.82	913	2,604
	DSNPT_J6_1	Confluence with Blaney	6.51	1,290	3,602
	DSNPT_J5_7	State Highway 94	9.51	1,392	5,925
	DNSPT_J5_2N	Drennan Rd (West)	13.31	1,514	8,494
	DSNPT_J5_1	Confluence with Corral Trib	33.99	6,644	21,838
	DSNPT_J4_1	Confluence with Franceville Trib	34.70	6,712	22,038
	DSNPT_J3_7	Bradley Rd	35.47	6,570	22,100
	DSNPT_J3_6	Confluence with Marksheffel	41.75	7,124	26,498
	DSNPT_J3_3	Fountain Blvd	43.97	7,248	26,998
	CH1_J3_1	Peaceful Valley Rd	44.29	7,161	26,919
	DSNPT_E1_1	Confluence with East Fork	53.57	8,036	29,722
	DSNPT_J2_1	Confluence with West Fork	59.32	7,935	29,494
	CH3_J1_6	Link Rd	60.22	7,874	29,423
	DSNPT_J1_6	Confluence with C and S Trib	63.25	8,745	32,409
	DSNPT_J1_3	Ohio Ave	65.36	8,715	30,358
	DSNPT_J1_2	Outfall to Fountain Creek	66.51	8,731	27,655
East Fork Tributary	DSNPT_E1_T1_5	Bradley Rd (East)	0.30	172	289
	DSNPT_E2_6	Drennan Rd	2.45	486	1,593
	DSNPT_E2_2	Bradley Rd (West)	4.42	568	1,645
	DSNPT_E1_8	At City of Colorado Springs Boundary	6.89	699	2,540
	DSNPT_E1_2	Upstream of Confluence with JCC (Peaceful Valley Rd)	8.92	1,087	3,630
West Fork Tributary	DSNPT_W1_9	Fountain Blvd	1.23	271	1,385
	DSNPT_W1_4	Mesa Ridge Pkwy	3.44	881	2,722
	DSNPT_W1_3	S Marksheffel Rd	3.94	701	2,759
	CH1_W1_1	Upstream of Confluence with JCC	4.33	134	145
Corral Tributary	CH1_C2_6	State Highway 94	3.93	2,719	4,471
	DSNPT_C1_2	At confluence with Strip Mine Trib (Upstream of confluence with JCC)	18.00	4,876	11,590
Stripmine Tributary	DSNPT_S1_7	State Highway 94	1.40	1,972	2,122

Table 3-12. Existing Condition Versus Future Conditions Peak Flow Rates at Analysis Points (continued)

Major Drainage Way	Model Node ID	Location Description	Contributing Area (mi^2)	Existing 100-Year Flow (cfs)	Future 100-Year Flow (cfs)
Franceville Tributary	DSNPT_F1_4	Confluence of Franceville and Strip Mine	8.18	2,236	4,925
Marksheffel Tributary	DSNPT_J3_6W_2N	Bradley Rd (West)	0.67	794	2,146
	CH2_J3_6W_2E	Drennan Rd (East)	1.31	335	1,149
	CH3_J3_6W_2	Marksheffel Rd (North of Bradley Rd)	1.64	268	1,185
	DSNPT_J3_6W_4	Drennan Rd (West)	1.93	430	2,744
	CH3_J3_6W_1	Bradley Rd (East)	4.58	933	4,257
	CH2_J3_6	Upstream of Confluence with JCC	5.88	1,355	4,768
Blaney Tributary	DSNPT_J5-T1_2	Upstream of Confluence with JCC	1.32	459	852
C&S Tributary	DSNPT_J1_6W_4	Mesa Ridge Pkwy	0.50	648	651
	DSNPT_J1_6W_2W	C and S Rd	1.50	617	667
	DSNPT_J1_6W_1	Upstream of Confluence with JCC	2.74	1,595	3,462
Ohio Trib.	CH1_J1_4	Upstream of Confluence with JCC	1.06	119	320

4.0 HYDRAULICS

This section describes the methodology and results of the hydraulic analysis performed for the Jimmy Camp Creek drainage basin. The open channel hydraulic models were prepared using the USACE HECRAS modeling software, version 6.3. In addition to the hydraulic analysis of open channels, existing storm sewer trunk lines 60-inches in diameter and greater were analyzed for hydraulic capacity.

4.1 Purpose of Hydraulic Analysis

The purpose of the hydraulic analysis for the Jimmy Camp Creek DBPS was to identify existing and future deficiencies in major drainageways and large storm sewer trunk lines within the Basin. The hydraulic analysis aimed to document existing hydraulic deficiencies to identify the need for future feasible stormwater and flood control solutions.

Another goal of the hydraulic analysis was to identify locations where the existing conditions 100-year floodplain differs significantly from the effective FEMA floodplain shown on the Flood Insurance Rate Maps (FIRMs). The existing conditions 100-year floodplain was delineated from the hydraulic model results and compared to the regulatory floodplain to identify areas of inconsistency.

4.2 Hydraulic Model Reach Naming

The reach naming convention used within the HEC-RAS model is based on seven branches of Jimmy Camp Creek, identified as Jimmy Camp Creek Mainstem, West Fork Tributary, East Fork Tributary, Franceville Tributary, Stripmine Tributary, Corral Tributary, and Blaney Tributary. For reach naming purposes, the 7 branches were abbreviated to J, W, E, F, S, C, and B, respectively. Each reach name begins with the branch letter followed by a number, starting from the most downstream reach and increasing in the upstream direction. For example, J5 refers to reach 5 of the Jimmy Camp Creek channel. The reach names are shown in Figure 4-1. Two smaller side branches of Stripmine Tributary and one side branch of East Fork Tributary were also included in the model. The tributaries to Stripmine Tributary are termed S1-T1, S2-T1 and the tributary to East Fork Tributary is termed E1-T1.

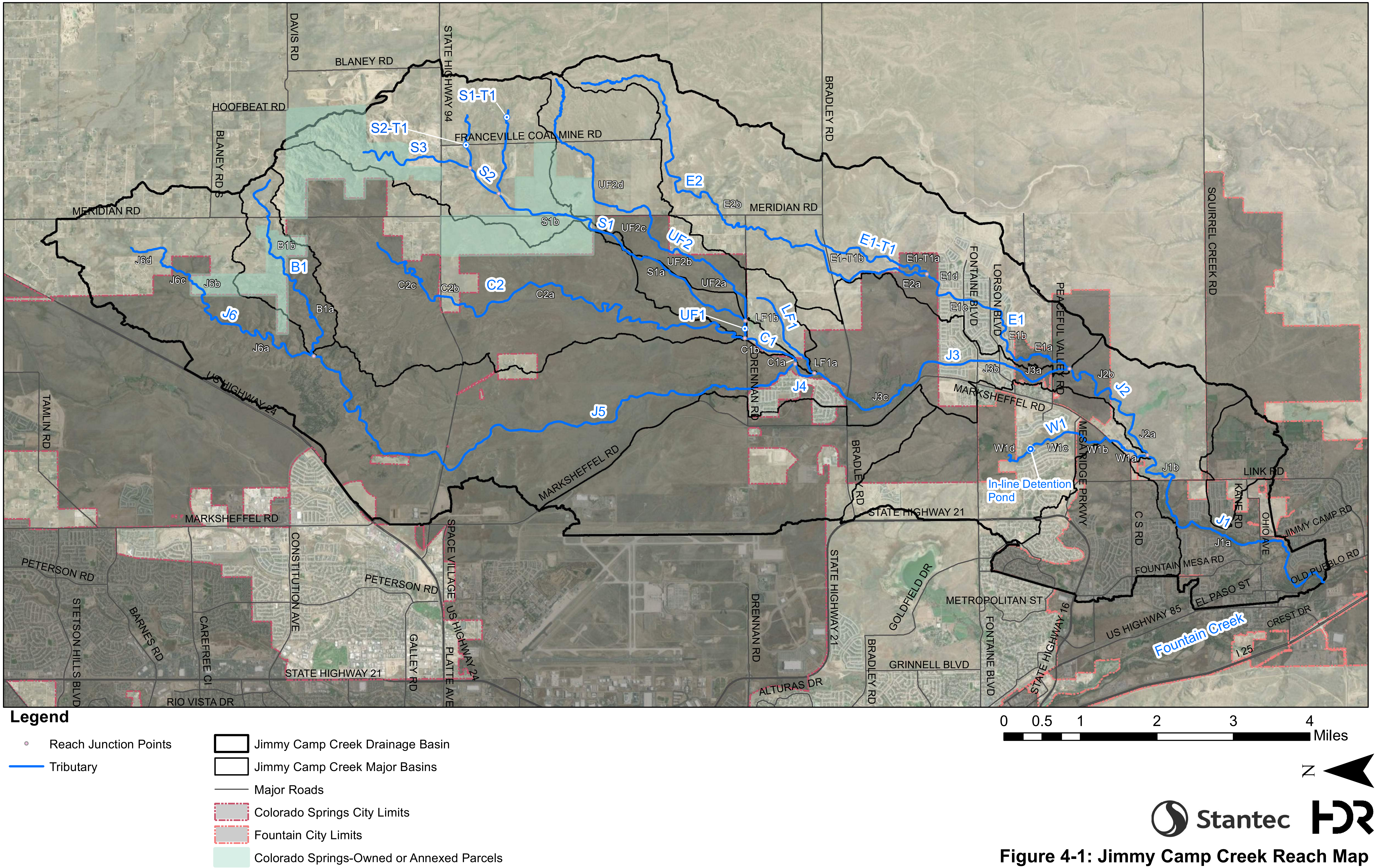


Figure 4-1: Jimmy Camp Creek Reach Map

Coordinate System: NAD 1983 StatePlane Colorado Central FIPS 0502 Feet

4.3 Reach Delineation

4.3.1 Included Drainageways

Channel centerlines were developed from the waterline GIS shapefile data provided by EPC and the 2018 LiDAR-based Digital Elevation Model (DEM) data collected by the State of Colorado and obtained through the City of Colorado Springs. The centerlines from the original data files were adjusted to more closely follow the stream thalweg reflected in the 2018 DEM data as well as more current aerial photography. The extents of the channel centerlines were compared to flowlines in the National Hydrography Dataset (NHD). Based on this comparison, the centerlines for all the reaches in the upstream end of the drainage basin (J6, B1, C2, S3, UF2, and E2) were extended up to 10,000 feet further upstream where the DEM and aerial imagery indicated well defined channels.

Drainageways in the Jimmy Camp Creek basin run through unincorporated EPC, the City of Colorado Springs, and the City of Fountain. Table 4-1 lists all the modeled reaches, their total length, and the reach length that lies within unincorporated EPC, which is the focus of this study. Some reaches (J4, J5, and UF1) are entirely outside of the unincorporated EPC boundaries.

Franceville Tributary is split into two parts at Drennan Road, Upper (UF) and Lower (LF) Franceville. A culvert buried by sediment at Drennan Road disconnects the historic flow path and prevents Lower Franceville from receiving flows from Upper Franceville unless the flow overtops the road. The buried culvert is owned by the City of Colorado Springs and is assumed to remain non-functional and is not represented in the hydraulic model. The hydraulic model does not show flows in Upper Franceville overtopping Drennan Road. All discharge from Upper Franceville is routed westward along the north side of Drennan Road to the Corral Tributary. The Corral Tributary then crosses Drennan Road and runs south for approximately 0.8 miles to the main stem of Jimmy Camp Creek. Lower Franceville joins the main stem of Jimmy Camp Creek a short distance downstream, approximately 1.2 miles south of Drennan Road.

The effective FIS does, however, show flows in Upper Franceville overtopping Drennan Road and continuing downstream in Lower Franceville, which runs along the east side of the Pikes Peak National Cemetery (PPNC). As of September 2023, there is a LOMR in process for the improved Lower Franceville channel that PPNC constructed to alleviate flooding on the site. Since it is based on the effective FIS, the LOMR assumes that Lower Franceville receives flows from Upper Franceville.

Table 4-1. Major Drainageway and Reach Length Summary

Drainageway	Reach ID	Total Reach Length [miles]	Unincorp. EPC Reach Length* [miles]
Jimmy Camp Creek	J1	4.3	0.8
	J2	2.3	1.2
	J3	4.0	1.0
	J4	0.4	0.0
	J5	8.4	0.0
	J6	4.0	0.9
Blaney Tributary	B1	3.2	0.9
Corral Tributary	C1	0.8	0.7
	C2	7.3	0.3
Lower Franceville Tributary	LF1	1.5	1.2
Upper Franceville Tributary	UF1	0.2	0.0
	UF2	5.4	3.4
Stripmine Tributary	S1	4.0	0.3
	S2	0.5	0.5
	S3	1.8	0.9
South Tributary to Stripmine Tributary	S1-T1	1.2	1.2
North Tributary to Stripmine Tributary	S2-T1	0.8	0.8
East Fork Tributary	E1	2.9	1.8
	E2	7.3	6.8
Tributary to East Fork Tributary	E1-T1	2.0	1.5
West Fork Tributary	W1	2.4	1.7
TOTAL		64.9	26.0

*Unincorporated EPC reach length does not include parcels owned or annexed by City of Colorado Springs

4.3.2 Excluded Drainageways

4.3.2.1 Marksheffel Tributary

The Marksheffel Tributary sub-basin was included in the hydrologic analysis but the drainageway is not included in the hydraulic analysis because the main channel lies entirely within the City of Colorado Springs.

4.3.2.2 C&S Tributary

The C&S Tributary sub-basin was included in the hydrologic analysis but the drainageway is not included in the hydraulic analysis because the main channel lies entirely within the City of Fountain.

4.3.2.3 Ohio Tributary

The Ohio Tributary sub-basin was included in the hydrologic analysis but the drainageway is not included in the hydraulic analysis because the main channel lies entirely within the City of Fountain.

4.3.2.4 Chilcotte Canal Number 27

Chilcotte Canal Number 27 is an agricultural irrigation canal located in the City of Fountain. It runs generally north to south on the east side of Jimmy Camp Creek near Ohio Avenue before continuing south and leaving the basin. The canal has a negligible effect on drainage and is not included in either the hydrologic or hydraulic analysis.

4.3.2.5 Fountain Ditch

Fountain Ditch is a 35-mile water canal system including open ditch and piped sections that diverts water from Fountain Creek and runs through the City of Colorado Springs, El Paso County, and the City of Fountain, which irrigates approximately 2,000 acres of land. Fountain Ditch has been owned and operated by Fountain Mutual Irrigation Company (FMIC) since the late 1880's. Within the Jimmy Camp Creek drainage basin, Fountain Ditch is approximately 14 miles long.

Over the years, several development projects occurred in the vicinity of Marksheffel Road and Fontaine Boulevard where Fountain Ditch historically laid. Based on the drainage plan review of development projects in the area, Fountain Ditch receives no apparent stormwater discharge from existing developments except Cottonwood Meadows, which consists of approximately 46.2 acres of land bounded by Fontaine Boulevard to the south, Marksheffel Road to the east, and undeveloped land to the north and west. Per the approved *Final Drainage Report Cottonwood Meadows Filing No. 1* dated October 1999, FMIC's Drainage District accepts historic runoff within the existing irrigation ditch and maintains ditch improvements adjacent to Cottonwood Meadows Filing No. 1 and Jimmy Camp Creek. Fountain Ditch was included in the hydrologic analysis but is not included in the hydraulic analysis.

4.4 Cross Sections

The hydraulic model contains 65 miles of channel center lines and 22 roadway crossings. Approximately 26 miles of channel and 16 roadway crossings are within unincorporated EPC. The remainder lie within the City of Colorado Springs or the City of Fountain. Hydraulic model cross sections were placed at a maximum 400 feet spacing along each of the channel reaches that are within unincorporated EPC. Reaches that are within the City of Colorado Springs and the City of Fountain were not modeled in detail. All reaches within unincorporated EPC lie upstream of City of Colorado Springs and City of Fountain reaches. At each location where a reach enters unincorporated EPC from one of the cities, at least 4 cross sections were placed downstream of the County boundary so that the hydraulic model could stabilize and establish a downstream water surface elevation for the County reach. This methodology is acceptable because the existing channel slopes are generally too steep to maintain a subcritical flow regime over an extended distance and the model frequently defaults to critical depth. At each location (11 in total) where a reach enters unincorporated EPC from one of the cities, no obvious flow constrictions were found in the downstream reach that would cause backwater conditions in the upstream reach.

The effective 100-year FEMA floodplain delineation was used as a guide to determine an approximate width of each cross section with widths extending at least 50 feet outside of the FEMA designated floodplain. Additional cross sections were placed at all major hydraulic controls, including drop structures, bridge and culvert crossings, and areas with significant change in channel geometry or slope.

4.4.1 Bridge and Culvert Cross Sectional Placement

Bridge and culvert crossings were modeled per guidance found in the *HEC-RAS 6.0 Hydraulic Reference Manual*. A total of four cross sections were placed at each bridge and culvert crossing in the model as shown in Figure 4-2. Per HEC-RAS guidance, the first cross section was placed at a location downstream of the bridge or culvert where the constricted flow from the crossing has fully expanded to the typical channel width (Cross Section 1, as shown on Figure 4-2). The distance downstream of the bridge or culvert varied depending on the degree of the constriction and the characteristics of the flow in that area. The flow transition line was drawn from the downstream edge of the bridge or culvert opening using an expansion ratio (ER) to help identify the location of the first cross section. These expansion ratios varied from 1:1 to 2:1. The expansion ratio was based on channel slope, degree of constriction, and ratio of overbank to channel roughness.

The second cross section was placed a short distance downstream from the crossing to represent the natural ground downstream of the bridge or culvert (Cross Section 2, as shown on Figure 4-2). This cross section was typically placed near the toe of the roadway embankment, as recommended in the HEC-RAS manual.

The third cross section was placed a short distance upstream from the crossing to represent the natural ground upstream of the bridge or culvert (Cross Section 3, as shown on Figure 4-2). This cross section was typically placed near the toe of the roadway embankment, as recommended in the HEC-RAS manual.

The fourth cross section for each bridge and culvert crossing was placed at a point far enough upstream of the crossing to represent the full channel width before flow contracts through the crossing (Cross Section 4, as shown in Figure 4-2). Similar to the first cross section, the fourth cross section was placed utilizing a flow transition line based upon a contraction ratio (CR) of 1:1, based upon guidance in Appendix B of the

HEC-RAS manual. Flow transitions occur in a shorter distance when contracting as opposed to expanding, which is reflected in the fourth cross section as it is located closer to the modeled crossing than the first cross section.

A similar approach was used to model each of the drop structures. Cross sections were placed at the crest and toe of the drop structure and additional cross sections were placed a sufficient distance upstream and downstream.

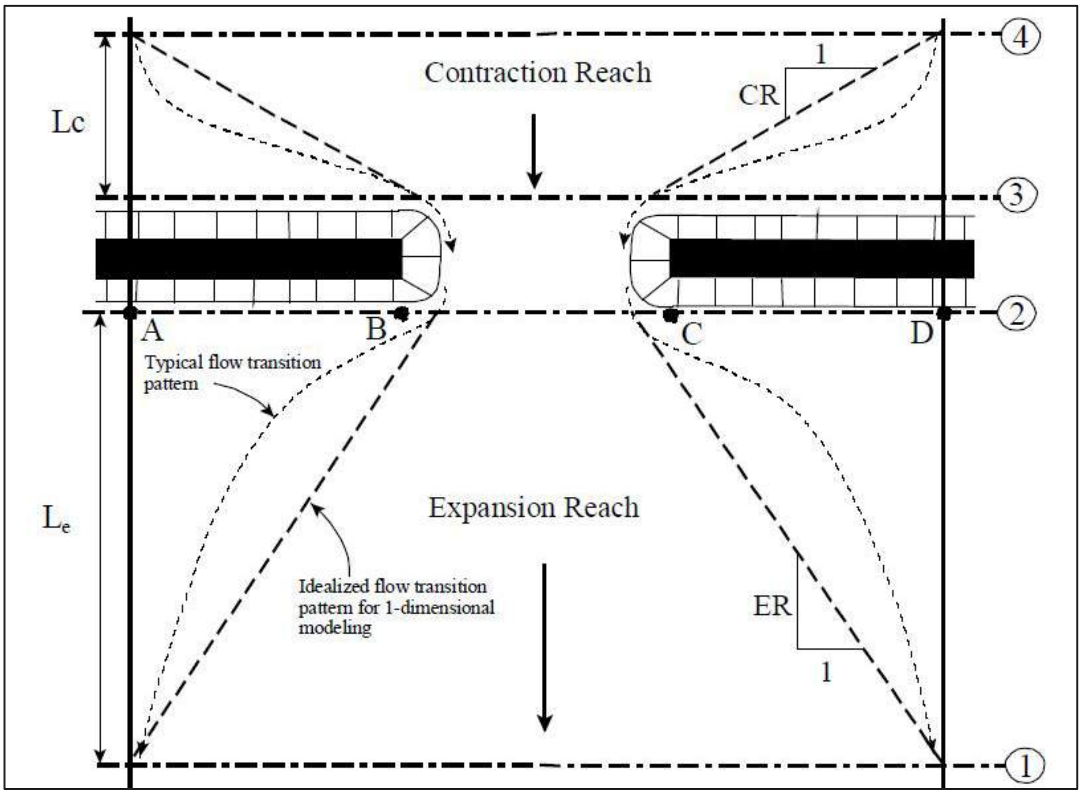


Figure 4-2. Cross Section Locations for Hydraulic Crossings

4.4.2 Manning's n Values

The 2016 National Land Cover Database (NLCD) was used to create a Land Use overlay for specifying channel and overbank roughness (Manning's n values) in the hydraulic model. The NLCD was used as a starting point and then initial assessments were verified using aerial photography. The Land Use overlay layer was used to provide roughness values on the model overbanks. Roughness values were assigned to the channel between the model bank stations.

Six channel conditions and 6 overbank conditions were selected to provide a range of representative land cover conditions within the study area. A Manning's n value was assigned to each of the 12 land cover conditions. These values were based on the El Paso County Drainage Criteria Manual (DCM) (2014), Colorado Springs DCM Volume 1 (2021), *Open-Channel Hydraulics* by Ven Te Chow (1959), and equations found in the Urban Storm Drainage Criteria Manual (USDCM) (2018). Table 4-2 shows typical roughness values for natural channels from the HEC-RAS *Hydraulic Reference Manual* that are excerpted from Chow's

Open-Channel Hydraulics. Table 4-3 and Table 4-4 list the 12 land cover conditions and associated Manning's n values used in the model as well as the source and assumptions.

Table 4-2. Typical Manning's n Values for Natural Channels

Type of Channel and Description	Minimum	Normal	Maximum
A. *Natural Streams*			
1. **Main Channels**			
a. Clean, straight, full, no rifts or deep pools	0.025	0.030	0.033
b. Same as above, but more stones and weeds	0.030	0.035	0.040
c. Clean, winding, some pools and shoals	0.033	0.040	0.045
d. Same as above, but some weeds and stones	0.035	0.045	0.050
e. Same as above, lower stages, more ineffective slopes and sections	0.040	0.048	0.055
f. Same as "d" but more stones	0.045	0.050	0.060
g. Sluggish reaches, weedy. deep pools	0.050	0.070	0.080
h. Very weedy reaches, deep pools, or floodways with heavy stands of timber and brush	0.070	0.100	0.150
2. **Flood Plains**			
a. Pasture no brush			
1. Short grass	0.025	0.030	0.035
2. High grass	0.030	0.035	0.050
b. Cultivated areas			
1. No crop	0.020	0.030	0.040
2. Mature row crops	0.025	0.035	0.045
3. Mature field crops	0.030	0.040	0.050
c. Brush			
1. Scattered brush, heavy weeds	0.035	0.050	0.070
2. Light brush and trees, in winter	0.035	0.050	0.060
3. Light brush and trees, in summer	0.040	0.060	0.080
4. Medium to dense brush, in winter	0.045	0.070	0.110
5. Medium to dense brush, in summer	0.070	0.100	0.160
d. Trees			
1. Cleared land with tree stumps, no sprouts	0.030	0.040	0.050
2. Same as above, but heavy sprouts	0.050	0.060	0.080
3. Heavy stand of timber, few down trees, little undergrowth, flow below branches	0.080	0.100	0.120
4. Same as above, but with flow into branches	0.100	0.120	0.160
5. Dense willows, summer, straight	0.110	0.150	0.200
3. **Mountain Streams, no vegetation in channel, banks usually steep, with trees and brush on banks submerged**			
a. Bottom: gravels, cobbles, and few boulders	0.030	0.040	0.050
b. Bottom: cobbles with large boulders	0.040	0.050	0.070

Source: USACE, HEC-RAS Hydraulic Reference Manual, Version 6.0, December 2020, Excerpt of Table 3-1.

Table 4-3. Hydraulic Modeling Manning's n Values for Channels

Description	"n" Value	Source/Assumptions
Sand-Silt	0.032	Colorado Spring DCM Table 12-2 and USDCM Table 8-5
Vegetated Streambed, light to medium	0.040	Colorado Springs DCM Table 12-2
Vegetated Streambed, medium to dense	0.060	Colorado Springs DCM Table 12-2
Boulder Drops	0.070	USDCM Figure 9-3 (Using approximate values)
Riprap Lining	0.040	Average roughness value of riprap. Based on USCDM equation $n=0.0395\ D_{50}^{1/6}$
Concrete Lining	0.013	Open-Channel Hydraulics

Table 4-4. Hydraulic Modeling Manning's n Values for Overbanks

Description	"n" Value	Source/Assumptions
Scattered Brush, heavy weeds	0.050	Open-Channel Hydraulics
Light brush and trees	0.060	Open-Channel Hydraulics
Medium to dense brush	0.100	Open-Channel Hydraulics
Short grass pasture	0.030	Open-Channel Hydraulics
Residential	0.100	Value to include obstructions of residential lots in lieu of creating blockages, HEC-RAS 5.0 2D Modeling User's Manual
Pavement	0.016	Open-Channel Hydraulics

Aerial photographs were used to estimate channel conditions in order to assign Manning's n values between the model bank stations. The following figures show examples of aerial images for the 4 channel conditions found in the drainageways in the hydraulic model ("sand-silt", "vegetated streambed, light to medium", "vegetated streambed, medium to dense", and "boulder drops"). The majority of the modeled drainageways are either "sand-silt channel" or "vegetated streambed, light to medium".

(located on Corral Tributary, reach C2)

Figure 4-3. Representative Image of Sand-Silt Channel, n = 0.032

(located on Jimmy Camp Creek, reach J3, downstream of Lorson Ranch)

Figure 4-4. Representative Image of Vegetated Streambed, Light to Medium, n=0.040

(located on Jimmy Camp Creek, reach J2, near Bonnie Cap Lane)

Figure 4-5. Representative Image of Vegetated Streambed, Medium to Dense, n=0.060

(located on Jimmy Camp Creek, reach J3, in Lorson Ranch)

Figure 4-6. Representative Image of Boulder Drop, n=0.070

4.4.3 Bank Stations

Once the channel centerline station and elevations were obtained from the DEM, bank stations were assigned. The bankfull channel is the deepest part of a cross section and often has a lower roughness value than the vegetated overbank terraces. In much of the Jimmy Camp Creek drainage basin, there is often a lack of a defined low flow channel, a common observation in sandy systems. Therefore, to have a consistent basis, model bank stations were set at the existing conditions 5-year water surface elevation.

4.4.4 Contraction and Expansion Coefficients

Contraction and expansion coefficients were assigned to each cross section based on transitions in cross sectional geometry. Typically, cross sections along the open channel sections of a reach have gradual transitions and contraction and expansion coefficients of 0.1 and 0.3 respectively. At bridge and culvert sections, where the transition is usually more abrupt, higher contraction and expansion values of 0.3 and 0.5, respectively, were used.

4.4.5 In Line Detention Ponds

As described in the Hydrology section, there were multiple detention ponds that were modeled in the SWMM hydrology model. Only one of these ponds was on a reach that was included in the HEC-RAS hydraulic model, on the West Fork upstream of Mesa Ridge Parkway. Only the dam embankment was included in the hydraulic model. The outfall structure of the pond was not included since the pond is included in the hydrology model. Cross sections were placed in the pond reach to model the general flow pattern through the pond. Flow rates upstream and downstream of the pond were taken from the hydrology model. The location of the in-line detention pond is shown on Figure 4-1.

4.5 Hydraulic Structure Data and Inventory

Data that was used to input bridge and culvert crossings into the model came from a variety of sources. Construction plans were utilized when available since they contain the most detailed information. Construction plans were located for 9 of the 22 roadway crossings in the hydraulic model. The plans were obtained from El Paso County Development Review and CDOT Staff Bridge archives. For the other crossings, where construction plans were not available, structure information was obtained from the CDOT Off-System Bridge Inspection database, EPC GIS data, or field measurements performed by Stantec or EPC.

The source of the information for each bridge or culvert structure is noted in the “Source of Data” column in Table 4-7. Jimmy Camp Creek Structure Evaluation Summary. The roadway elevations at bridges and culverts were taken directly from the DEM. The culvert invert elevations were set to match the DEM channel elevation unless more accurate elevation data was available. Structure overtopping is discussed in Section 4.8, Hydraulic Deficiencies.

4.6 Flow Data and Boundary Conditions

Flow rates for the hydraulic model were obtained from the SWMM hydrologic model. Flow rates were input for the 2-, 5-, 10-, 25-, 50-, and 100-year storm events. The 100-year storm event was used to identify hydraulic deficiencies and delineate floodplains. Floodway modeling was not included in the hydraulic analysis. Selected SWMM design points along major drainageways were used as flow change locations in the HEC-RAS model. There was no interpolation of calculated flows between SWMM design points.

The HEC-RAS model utilized a 1D steady flow regime in subcritical flow mode. The downstream boundary condition was based on Fountain Creek water surface elevations shown in the FIS flood profiles at the confluence with Jimmy Camp Creek. For storm events not evaluated in the FIS, such as the 2-year and 5-year storm events, the normal depth at the channel slope was used as the downstream boundary condition. The 25-year storm event was also not analyzed in the FIS study, therefore the 10-year water surface elevation in Fountain Creek was used as the downstream boundary condition for the 25-year storm event.

The size of the Fountain Creek drainage area is significantly larger than the Jimmy Camp Creek drainage area, therefore it was necessary to consider the coincidental probability of a given storm recurrence interval occurring at the same time in each drainage basin to select the appropriate downstream boundary water surface elevation in Fountain Creek. The Federal Highway Administration (FHWA) HEC22 manual gives some guidance on selecting appropriate storm- frequencies when two drainage basins are different in size.

Table 4-5 shows the table from HEC-22 that correlates storm frequencies for coincidental occurrence based on the area ratio of the drainage basins.

In this case, the Jimmy Camp Creek drainage basin is approximately 61 square miles, while the Fountain Creek drainage basin at the confluence with Jimmy Camp Creek is approximately 606 square miles. This is an approximately 10 to 1 size difference between the basins. According to HEC-22, the 10-year storm is coincident in both basins, however, a 100-year storm at Jimmy Camp Creek corresponds to a 50-year storm at Fountain Creek. Table 4-6 lists the FIS water surface elevation at Fountain Creek for each storm event as well as the downstream boundaries used in the HEC-RAS model.

Table 4-5. Storm Frequencies for Coincidental Occurrence from HEC-22

Area Ratio	Frequencies for Coincidental Occurrence			
	10-Year Design		100-Year Design	
	Main Stream	Tributary	Main Stream	Tributary
10,000 to 1	1 10	10 1	2 100	100 2
1,000 to 1	2 10	10 2	10 100	100 10
100 to 1	5 10	10 5	25 100	100 25
10 to 1	10 10	10 10	50 100	100 50
1 to 1	10 10	10 10	100 100	100 100

Source: FHWA, Hydraulic Engineering Circular No. 22, Third Edition, August 2013, Table 7-3.

Table 4-6. Jimmy Camp Creek Downstream Boundary Conditions

Storm Event	Fountain Creek Water Surface Elevation (WSEL)	Model Downstream Boundary Condition
2-Year	N/A	Normal Depth at Channel Slope
5-Year	N/A	Normal Depth at Channel Slope
10-Year	5499.3	5499.3 (10-Year Fountain Creek WSEL)
25-Year	N/A	5499.3 (10-Year Fountain Creek WSEL)
50-Year	5502.5	5502.5 (50-Year Fountain Creek WSEL)
100-Year	5503.1	5502.5 (50-Year Fountain Creek WSEL)

4.7 Hydraulic Modeling Results Summary

The HEC-RAS model results are shown in the hydraulic data tables included in Appendix F. Large format hydraulic exhibit maps are included as an attachment to this report.

Comparison of the existing conditions 100-year floodplain with the regulatory FEMA floodplain shows that in general, the existing conditions floodplain is smaller than the regulatory FEMA floodplain. This is primarily because the flow rates from the SWMM hydrologic model are lower than the flow rates shown in the FIS, as discussed in the Hydrology section.

Another reason for differences between the existing conditions 100-year floodplain and the regulatory FEMA floodplain is the updated topographic mapping used for this study. LiDAR-based DEM data prepared in 2018 was used for this study. The FIS states that the regulatory FEMA floodplain for Jimmy Camp Creek and its tributaries is based on topographic mapping prepared from aerial photographs taken by the Soil Conservation Service (SCS) in 1973. The effective floodplain boundaries are based on 2 reports, a 1973 USACE Flood Plain Information Report, and a 1975 SCS Flood Hazard Analysis. The FIS bibliography references are listed below:

- U.S. Department of Agriculture, Soil Conservation Service, Colorado Water Conservation Board, Flood Hazard Analyses, Portions of Jimmy Camp Creek and Tributaries, El Paso County, Colorado, October 1975.
- U.S. Department of the Army, Corps of Engineers, Albuquerque District, Flood Plain Information, Fountain and Jimmy Camp Creeks, Colorado Springs. Fountain, El Paso County, Colorado, March 1973.

On the East Fork Tributary, reach E2 and side tributary E1-T1 run through a wide, relatively flat valley with no defined channel and many shallow drainage paths. This drainage pattern begins north of Bradley Road and continues south of Bradley Road. Additionally, a portion of the Upper Franceville Tributary downstream of Franceville Coal Mine Road exhibits the same shallow, undefined drainage pattern. Both areas are shown on Figure 4-8. It is difficult to accurately model flow paths in this type of terrain with a one-dimensional hydraulic model. The floodplain has been delineated to include all of the area that could be inundated by meandering flows. The floodplain extents are similar to the regulatory FEMA floodplain.

There are 2 areas where the existing conditions floodplain significantly exceeds the regulatory FEMA floodplain:

- West Fork Tributary upstream of Mesa Ridge Parkway
- Jimmy Camp Creek at Peaceful Valley Road.

Upstream of Mesa Ridge Parkway, the West Fork Tributary has been channelized and runs through an inline detention pond adjacent to The Glen at Widefield residential development. It is unknown how the design criteria used for the detention pond compares to those used for this study. The hydraulic model shows that the inline detention pond overtops to the east. The one-dimensional hydraulic model cannot quantify the amount of overtopping flow or the extents of inundation, so the floodplain has been drawn to include all of the area that possibly could have areas of shallow flow or ponding from the overtopping flow.

At Peaceful Valley Road, Jimmy Camp Creek overtops the roadway and also overtops a low point on the west bank approximately 600 feet upstream the road. The floodplain on the west side of the creek has been drawn to include all areas that could be inundated by shallow flow or ponding from the overtopping flow.

4.8 Hydraulic Deficiencies

Hydraulic deficiencies were identified that include channel stability concerns, overtopped roadway crossings, and large diameter storm sewer surcharges. This section describes how these deficiencies were determined and where they are located in the drainage basin.

4.8.1 Overtopped Roadway Crossings

The hydraulic model contains 22 roadway crossings. Sixteen of these are within unincorporated EPC. The remainder are located within the City of Colorado Springs or the City of Fountain. Roadway crossings overtopped by the 100-year storm were defined as deficiencies. The hydraulic model was used to determine which crossings are overtopped by the 100-year flood. Crossings are labeled as deficient if any part of the modeled roadway is overtopped, even if the roadway low point is not located directly above the structure. Table 4-7 summarizes the hydraulic analysis results for all 22 roadway crossings included in the hydraulic model.

Table 4-7. Jimmy Camp Creek Structure Evaluation Summary. Future Conditions assumes no on-site detention.

Drainage	Reach Name	Location	Structure Description	Source of Data	Existing 100-Year (cfs)	Future 100-Year (cfs)	Structure Capacity with Existing Flows	Structure Capacity with Future Flows	Jurisdiction
Jimmy Camp	J1	Ohio Ave.	4 Span Bridge	Bridge Inspection Sketch	8,719	30,363	Adequate	Adequate	City of Fountain
Jimmy Camp	J1	Link Rd.	3 Span Bridge	Bridge Inspection Sketch	7,990	29,627	Adequate	Overtopped	City of Fountain
Jimmy Camp	J3	Peaceful Valley Rd.	4 - 30" CMP	EPC field data	7,241	26,990	Overtopped	Overtopped	City of Fountain
Jimmy Camp	J3	Lorson Blvd.	2 Span Bridge	Construction plans	7,241	26,990	Adequate	Adequate	El Paso County
Jimmy Camp	J3	Fontaine Blvd.	2 Span Bridge	Construction plans	7,241	26,990	Adequate	Adequate	El Paso County
Jimmy Camp	J3	Bradley Rd.	3 Span Bridge	Construction plans	6,570	22,100	Adequate	Adequate	City of Colorado Springs
West Fork	W1	Furlong Cir.	54" CMP	EPC GIS data	141	156	Overtopped	Overtopped	El Paso County
West Fork	W1	Ingle Ln.	2 - 36" CMP	EPC GIS data	141	156	Overtopped	Overtopped	El Paso County

Table 4-7. Jimmy Camp Creek Structure Evaluation Summary (continued).

Drainage	Reach Name	Location	Structure Description	Source of Data	Existing 100-Year (cfs)	Future 100-Year (cfs)	Structure Capacity with Existing Flows	Structure Capacity with Future Flows	Jurisdiction
West Fork	W1	Marksheffel Rd.	24" RCP	Stantec field data	141	156	Overtopped*	Overtopped*	City of Fountain
West Fork	W1	Mesa Ridge Pkwy.	2 Span Bridge	1-sheet construction drawing	1,183	2,833	Adequate	Adequate	El Paso County
Corral	C1	Drennan Rd.	2 Span Bridge	Bridge Inspection Sketch	4,876	11,591	Adequate	Adequate	City of Colorado Springs
Corral	C2	SH-94	1-11'x14' + 2-11'x10' CBC	Construction plans	2,720	4,475	Adequate	Adequate	El Paso County
Stripmine	S3	SH-94	2-12'x12' CBC	Construction plans	2,206	2,435	Adequate	Adequate	El Paso County
Stripmine South Tributary	S1-T1	Franceville Coal Mine Rd.	Single Span Bridge	EPC provided data	268	828	Adequate	Adequate	El Paso County
Stripmine North Tributary	S2-T1	Franceville Coal Mine Rd.	3 – 60” RCP	EPC provided data	617	1,155	Adequate	Overtopped	El Paso County
Upper Franceville	UF2	Franceville Coal Mine Rd.	2 - 36" CMP	EPC GIS data	182	561	Overtopped*	Overtopped*	El Paso County

Table 4-7. Jimmy Camp Creek Structure Evaluation Summary (continued).

Drainage	Reach Name	Location	Structure Description	Source of Data	Existing 100-Year (cfs)	Future 100-Year (cfs)	Structure Capacity with Existing Flows	Structure Capacity with Future Flows	Jurisdiction
East Fork	E1	Lorson Blvd.	48' wide Conspan concrete arch culvert	Construction plans	1,830	3,673	Adequate	Adequate	El Paso County
East Fork	E1	Fontaine Blvd.	48' wide Conspan concrete arch culvert	Construction plans	1,272	3,077	Adequate	Adequate	El Paso County
East Fork Tributary	E1-T1	Bradley Rd.	2 - 66" RCP	Construction plans	424	779	Overtopped*	Overtopped*	El Paso County
East Fork	E2	Bradley Rd.	2 - 8'x12' CBC	Construction plans	601	2,187	Adequate	Adequate	El Paso County
East Fork	E2	Drennan Rd.	2 Span Bridge	Bridge Inspection Sketch	488	1,605	Adequate	Adequate	El Paso County
East Fork	E2	Meridian Rd.	2 - 36"x48" HERCP + 2 - 36" RCP	EPC GIS data	507	2,049	Overtopped*	Overtopped*	El Paso County

Notes:

* Hydraulic model shows overtopping but headwater can also be diverted away from crossing in roadside ditch. More detailed modeling is required to assess conditions at the crossing.

CMP = Corrugated Metal Pipe

RCP = Reinforced Concrete Pipe

CBC = Concrete Box Culvert

HERCP = Horizontal Elliptical Reinforced Concrete Pipe

4.8.2 Storm Sewer Surcharges

Existing storm sewer trunk lines 60-inches in diameter and greater were analyzed for hydraulic capacity. There are 3 locations within the study area that contain existing large diameter storm sewer pipes. The locations are shown in Figure 4-7. The analysis of the sewer trunk lines was completed using the Bentley FlowMaster program. The FlowMaster program does not incorporate backwater effects including the impact of Jimmy Camp Creek water surface elevations on the storm sewer capacity. Deficiencies were defined as pipe capacities insufficient to contain the 100-year flow without surcharging. None of the pipelines analyzed had adequate capacity. Table 4-8 shows the results of the large storm sewer evaluation.

Figure 4-7. Jimmy Camp Creek Storm Sewer Locations

Table 4-8. Jimmy Camp Creek 60" Storm Sewer Evaluation

Description	Material	Shape	Size (in)	Design 100-YR[1] (cfs)	Existing 100-YR[2] (cfs)	Max Capacity[3] (cfs)	Existing Structure Capacity[4]
Fontaine Blvd (Old Glory Dr Tract A and D)	HDPE	Elliptical	83 x 53	305	655	160	Inadequate
Carriage Meadows Dr (Outfall Tract A)	RCP	Round	60	245	568	282	Inadequate
Peaceful Ridge Dr (Tract C & F to Outfall)	RCP	Round	60	184	360	178	Inadequate

Notes:
[1] The design 100-YR flow as determined in the drainage study for the subdivision.
[2] The existing 100-YR flow from the hydrology model for this DBPS.
[3] The maximum capacity of the pipe as determined in this DBPS.
[4] The adequacy of the pipe to convey the existing 100-YR flow from this DBPS without surcharging.

The flow rates shown in Table 4-8 that were used to evaluate the pipes are from the hydrology prepared for this DBPS. The pipes were actually designed using flow rates determined in the drainage study prepared for each subdivision.

The Final Drainage Report (FDR) for Pulte at Lorson Ranch (Pentacor, 2006) shows the Fontaine Boulevard pipe discharging into a detention pond on the northeast corner of Fontaine Boulevard and Jimmy Camp Creek. The StormCAD pipe design output tables show the maximum 100-year flow rate in the pipe to be 305 cfs, which surcharges the pipe. The pipe is shown to have a full flow capacity of 165 cfs in the FDR.

As described in the FDR for Carriage Meadows at Lorson Ranch Filing No. 1 (Core Engineering Group, 2006), the Carriage Meadows Drive pipe conveys flow from the FMIC ditch to Jimmy Camp Creek. The report states that the 100-year flow rate in the pipe is 245 cfs under developed conditions. The pipe is shown to have a maximum capacity of approximately 270 cfs in the FDR. This DBPS is not accounting for the FMIC diversion, on-site detention, or flows allowed to overtop pipes (street flows).

The FDR for Carriage Meadows at Lorson Ranch Filing No. 1 (Core Engineering Group, 2006) shows that the Peaceful Ridge Drive pipe will run along the north boundary of the Carriage Meadows subdivision and will convey runoff from the future Peaceful Ridge subdivision to Jimmy Camp Creek. The 100-year flow rate in the pipe is 184 cfs under developed conditions per the FDR, which surcharges the pipe. Detention has been provided by the Carriage Meadows development south of Fontaine Boulevard. Details of the detention and how it relates to the subject pipes is not accounted for in this DBPS.

4.8.3 Channel Deficiencies

Open channel deficiencies were defined as flow velocities greater than 5 ft/s or shear stress greater than 0.6 lb/sf. These criteria are based on National Resource Conservation Service (NRCS) allowable channel velocity information presented in the National Engineering Handbook. A detailed discussion is presented in Section 5.4.1, Assumptions for Alternative Development.

The modeled reaches are described below. Values in the summary tables are only reflective of areas within unincorporated EPC. Figure 4-8 and Figure 4-9 (located at the end of this section) highlight areas of high velocity and/or excessive shear stress that exceed the defined limits for existing and future conditions. In areas where the channel has been improved, these exceedances do not necessarily indicate a stability issue because flow over grade control structures will have high velocity and shear stress values. Improved channel reaches are identified in the following discussions of each drainageway.

4.8.3.1 Jimmy Camp Creek Main Branch

The main branch of Jimmy Camp Creek begins at the confluence with Fountain Creek in the City of Fountain near Old Pueblo Road and Hidden Prairie Parkway. The modeled drainageway runs upstream for approximately 23.4 miles to a point near Meridian Road at Partridge Lane. The creek was divided into 6 reaches, J1 through J6, in the hydraulic model and includes 6 bridged roadway crossings, as shown in Table 4-7. A summary of channel velocities and shear stresses for this branch is shown in Table 4-9.

Reach 1 of Jimmy Camp Creek (J1) begins at the confluence of Jimmy Camp Creek and Fountain Creek. It runs upstream for approximately 4.3 miles to the confluence with the West Fork Tributary. The only part of this reach that is within unincorporated EPC is 0.8 miles that begins at Link Road and runs upstream to the confluence with the West Fork Tributary. The maximum velocity or shear criteria are exceeded throughout the reach. Only 2 of the cross sections in the existing conditions model show velocity and shear values that do not exceed the defined limits.

Reach 2 of Jimmy Camp Creek (J2) begins at the confluence with the West Fork Tributary and runs upstream for approximately 2.3 miles to the confluence with the East Fork Tributary, which is located approximately 1,200 feet south of Peaceful Valley Road. The only part of this reach that is within unincorporated EPC is 1.2 miles at the downstream end from the confluence with the West Fork Tributary to the City of Fountain boundary line. The maximum velocity or shear criteria are exceeded throughout the reach. Only 2 of the cross sections in the existing conditions model show velocity and shear values that do not exceed the defined limits.

Reach 3 of Jimmy Camp Creek (J3) begins at the confluence with the East Fork Tributary and runs upstream for approximately 4.0 miles to the confluence with the Lower Franceville Tributary. The only part of this reach that lies within unincorporated EPC is approximately 1.0 mile that runs through Lorson Ranch. This section of the creek has been channelized and stabilized with grade control structures. It is unknown how the design criteria used for the improvements compares to those used for this study. The existing conditions model shows velocity and shear values that exceed the defined limits throughout the reach.

Reach 4 of Jimmy Camp Creek (J4) begins at the confluence with the Lower Franceville Tributary and runs upstream for approximately 0.4 miles to the confluence with the Corral Tributary. No part of the main channel in this reach lies within unincorporated EPC, although unincorporated parcels border the west side of the reach.

Reach 5 of Jimmy Camp Creek (J5) begins at the confluence with the Corral Tributary and runs upstream for approximately 8.4 miles to the confluence with the Blaney Tributary. No part of the main channel in this reach lies within unincorporated EPC, although unincorporated parcels border the west side of the lower end of this reach.

Reach 6 of Jimmy Camp Creek (J6) begins at the confluence with the Blaney Tributary and runs upstream for approximately 4.0 miles to the upstream study limit near Meridian Road and Partridge Lane. Only about 0.9 miles of the upstream end of this reach lie within unincorporated EPC. The maximum velocity and shear criteria are exceeded throughout the reach, except within a small impoundment created by an earthen embankment across the channel located approximately 0.5 miles from the upstream study limit.

Table 4-9. Jimmy Camp Creek 100-Year Velocity and Shear Stress Summary. Future Conditions assumes no on-site detention.

Reach ID	Maximum Velocity (ft/s)		Maximum Shear Stress (lb/ft^2)	
	Existing Conditions	Future Conditions	Existing Conditions	Future Conditions
J1	10.6	17.4	3.6	6.5
J2	12.6	16.7	4.1	5.7
J3	12.1	16.7	8.4	11.0
J6	11.4	12.9	2.4	2.7
Note: Open channel deficiencies were defined as flow velocities greater than 5 ft/s or shear stress greater than 0.6 lb/sf				

4.8.3.2 Blaney Tributary

Blaney Tributary lies in mostly undeveloped land east of Highway 24, 1.2 miles east of its intersection with Constitution Avenue. The modeled drainageway is approximately 3.2 miles long and runs from its confluence with Jimmy Camp Creek to Corral Bluffs View east of Meridian Road. There are no modeled roadway crossings. Only the upstream 0.9 miles of this reach lie within unincorporated EPC. A summary of channel velocities and shear stresses for this tributary is shown in Table 4-10. The maximum velocity or shear criteria are exceeded throughout the reach, except within a small impoundment created by an earthen embankment across the channel about 300 feet east of Meridian Road. Outside of the impoundment, only 1 cross section in the existing conditions model shows both velocity and shear values that do not exceed the defined limits.

Table 4-10. Blaney Tributary 100-Year Velocity and Shear Stress Summary. Future Conditions assumes no on-site detention.

Reach ID	Maximum Velocity (ft/s)		Maximum Shear Stress (lb/ft^2)	
	Existing Conditions	Future Conditions	Existing Conditions	Future Conditions
B1	9.1	10.1	1.7	2.0

4.8.3.3 West Fork Tributary

The West Fork Tributary begins at the confluence with Jimmy Camp Creek located east of Link Road and south of C&S Road. The modeled drainageway runs upstream for approximately 2.4 miles to the upstream study limit in undeveloped land north of The Glen at Widefield. Two parts of this reach lie within unincorporated EPC, from Jimmy Camp Creek to Marksheffel Road and from Mesa Ridge Parkway to the upstream study limit, for a total length of 1.7 miles. The model includes 1 bridged roadway crossing and 2 culvert crossings, as shown in Table 4-7. A summary of channel velocities and shear stresses for this branch is shown in Table 4-11. Between Jimmy Camp Creek and Marksheffel Road, the velocity and shear values are generally within the acceptable range.

North of Mesa Ridge Parkway, the stream has been channelized and an inline detention pond is located near the upstream end. It is unknown how the design criteria used for the improvements compares to those used for this study. Within this improved reach, the existing conditions model shows velocity or shear values that exceed the defined limits for approximately 2,400 feet downstream of the detention pond.

Table 4-11. West Fork Tributary 100-Year Velocity and Shear Stress Summary. Future Conditions assumes no on-site detention.

Reach ID	Maximum Velocity (ft/s)		Maximum Shear Stress (lb/ft^2)	
	Existing Conditions	Future Conditions	Existing Conditions	Future Conditions
W1	7.2	11.1	1.9	3.3

4.8.3.4 Corral Tributary

The Corral Tributary begins at the confluence with Jimmy Camp Creek located south of Drennan Road and southwest of Pikes Peak National Cemetery. The modeled drainageway runs upstream for approximately 8.1 miles to the upstream study limit in undeveloped land north of SH-94 and west of Corral Valley Road. The drainageway was divided into 2 reaches, C1 and C2, in the hydraulic model and includes 2 roadway crossings, as shown in Table 4-7. A summary of channel velocities and shear stresses for this tributary is shown in Table 4-12.

Reach C1 begins at the upstream end of reach J4 and runs upstream for approximately 0.8 miles to the confluence with the Upper Franceville Tributary on the north side of Drennan Road. This reach lies almost entirely within unincorporated EPC. The maximum velocity and shear criteria are exceeded throughout the reach in both existing and future conditions.

Reach C2 begins at Drennan Road and runs upstream for approximately 7.3 miles to the upstream study limit north of SH-94. Only 0.3 miles of this reach immediately south of the SH-94 crossing lie within unincorporated EPC. The maximum velocity and shear criteria are exceeded throughout the reach downstream of SH-94.

Table 4-12. Corral Tributary 100-Year Velocity and Shear Stress Summary. Future Conditions assumes no on-site detention.

Reach ID	Maximum Velocity (ft/s)		Maximum Shear Stress (lb/ft^2)	
	Existing Conditions	Future Conditions	Existing Conditions	Future Conditions
C1	11.7	15.6	2.4	3.6
C2	10.3	12.0	2.0	2.5

4.8.3.5 Franceville Tributary

As discussed in Section 3.4, the Franceville Tributary was split into upper and lower reaches due to a culvert buried in sediment rendering it non-functional at Drennan Road that disconnects the historic flow path and prevents the lower reach from receiving flows from the upper reach.

Lower Franceville Tributary (LF1) begins at the confluence with Jimmy Camp Creek located north of Bradley Road and east of Marksheffel Road. The modeled drainageway runs upstream for approximately 1.5 miles to the south side of Drennan Road. This reach lies almost entirely within unincorporated EPC. The upstream end of this reach has been channelized for approximately 4,200 feet where it borders the Pikes Peak National Cemetery. Within the channelized section, the existing conditions model shows velocity or shear values that exceed the defined limits at all but 2 cross sections. In the unimproved section of the reach, the maximum velocity or shear criteria are exceeded at all cross sections except the most downstream one.

Upper Franceville Tributary begins at the confluence with the Corral Tributary on the north side of Drennan Road and runs upstream for approximately 5.6 miles to the upstream study limit in undeveloped land east of Franceville Coal Mine Road. Upper Franceville Tributary was divided into 2 reaches, UF1 and UF2, in the hydraulic model and includes 1 roadway culvert crossing, as shown in Table 4-7.

Reach UF1 begins at the confluence with the Corral Tributary and runs eastward along the north side of Drennan Road for 0.2 miles to the confluence with the Stripmine Tributary. No part of this reach lies within unincorporated EPC.

Reach UF2 begins at the confluence with the Stripmine Tributary on the north side of Drennan Road and runs upstream for approximately 5.4 miles to the upstream study limit in undeveloped land east of Franceville Coal Mine Road. Approximately 3.4 miles of the upstream end of this reach lie within unincorporated EPC. The maximum velocity and shear criteria are exceeded throughout the reach upstream of Franceville Coal Mine Road. For approximately 7,000 feet downstream of Franceville Coal Mine Road, the maximum velocity or shear values are above the defined limits. Downstream of this section, approximately 3,000 feet of channel show acceptable velocity and shear values until the stream leaves unincorporated EPC and enters Colorado Springs.

A summary of channel velocities and shear stresses for Franceville Tributary is shown in Table 4-13.

Table 4-13. Franceville Tributary 100-Year Velocity and Shear Stress Summary. Future Conditions assumes no on-site detention.

Reach ID	Maximum Velocity (ft/s)		Maximum Shear Stress (lb/ft^2)	
	Existing Conditions	Future Conditions	Existing Conditions	Future Conditions
LF1	7.3	9.4	1.9	2.8
UF2	9.4	10.5	1.9	2.1

4.8.3.6 Stripmine Tributary

The Stripmine Tributary begins on the north side of Drennan Road at the confluence with Upper Franceville Tributary. The modeled drainageway runs upstream for approximately 6.4 miles to the upstream study limit in undeveloped land approximately one mile north of SH-94. The main branch of the drainageway was divided into 3 reaches in the hydraulic model (S1, S2 and S3) in order to include 2 side tributaries (S1-T1 and S2-T1). The main branch includes 1 roadway crossing and each of the side branches has 1 roadway crossing, as shown in Table 4-7. A summary of channel velocities and shear stresses for this tributary is shown in Table 4-14.

Reach S1 begins at the upstream end of UF1 on the north side of Drennan Road and runs upstream for approximately 4.0 miles to the confluence with the South Tributary (S1-T1) in undeveloped land near the Pikes Peak Gun Club shooting range. Approximately 0.3 miles of the upstream end of this reach lies within unincorporated EPC on land not owned by the City of Colorado Springs. The maximum velocity and shear criteria are exceeded throughout the reach.

Reach S2 begins at the confluence with the South Tributary (S1-T1) and runs upstream for approximately 0.5 miles to the confluence with the North Tributary (S2-T1). This reach lies entirely within unincorporated EPC. The maximum velocity and shear criteria are exceeded throughout the reach.

Reach S3 begins at the confluence with the North Tributary (S2-T1) and runs upstream for approximately 1.8 miles to the upstream study limit north of SH-94. Approximately 0.9 miles of the reach lies within unincorporated EPC. The maximum velocity and shear criteria are exceeded throughout the reach except at 2 cross sections immediately upstream of SH-94.

The South Tributary (S1-T1) begins at the upstream end of S1 and runs eastward for approximately 1.2 miles to the upstream study limit in undeveloped land east of Franceville Coal Mine Road. This reach lies entirely within unincorporated EPC. The maximum velocity and shear criteria are exceeded throughout the reach upstream of Franceville Coal Mine Road. Downstream of Franceville Coal Mine Road, the maximum velocity or shear values are above the defined limits at almost all cross sections.

The North Tributary (S2-T1) begins at the upstream end of S2 and runs eastward for approximately 0.8 miles to the upstream study limit in undeveloped land east of Franceville Coal Mine Road. This reach lies entirely within unincorporated EPC. The maximum velocity and shear criteria are exceeded throughout the reach except at 2 cross sections immediately upstream of Franceville Coal Mine Road.

Table 4-14. Stripmine Tributary 100-Year Velocity and Shear Stress Summary. Future Conditions assumes no on-site detention.

Reach ID	Maximum Velocity (ft/s)		Maximum Shear Stress (lb/ft^2)	
	Existing Conditions	Future Conditions	Existing Conditions	Future Conditions
S1	10.6	11.4	2.1	2.3
S2	9.7	10.7	1.8	2.3
S3	11.3	11.4	2.4	2.4
S1-T1	8.2	10.9	1.4	2.4
S2-T1	8.3	10.3	1.6	2.1

4.8.3.7 East Fork Tributary

The East Fork of Jimmy Camp Creek begins at the confluence with Jimmy Camp Creek south of Peaceful Valley Road and east of Marksheffel Road. The modeled drainageway runs upstream for approximately 10.2 miles to the upstream study limit in undeveloped land east of Franceville Coal Mine Road. It was divided into 2 reaches, E1 and E2, and also has a side tributary, E1-T1. The model includes 6 roadway crossings, as shown in Table 4-7. A summary of channel velocities and shear stresses for this tributary is shown in Table 4-15.

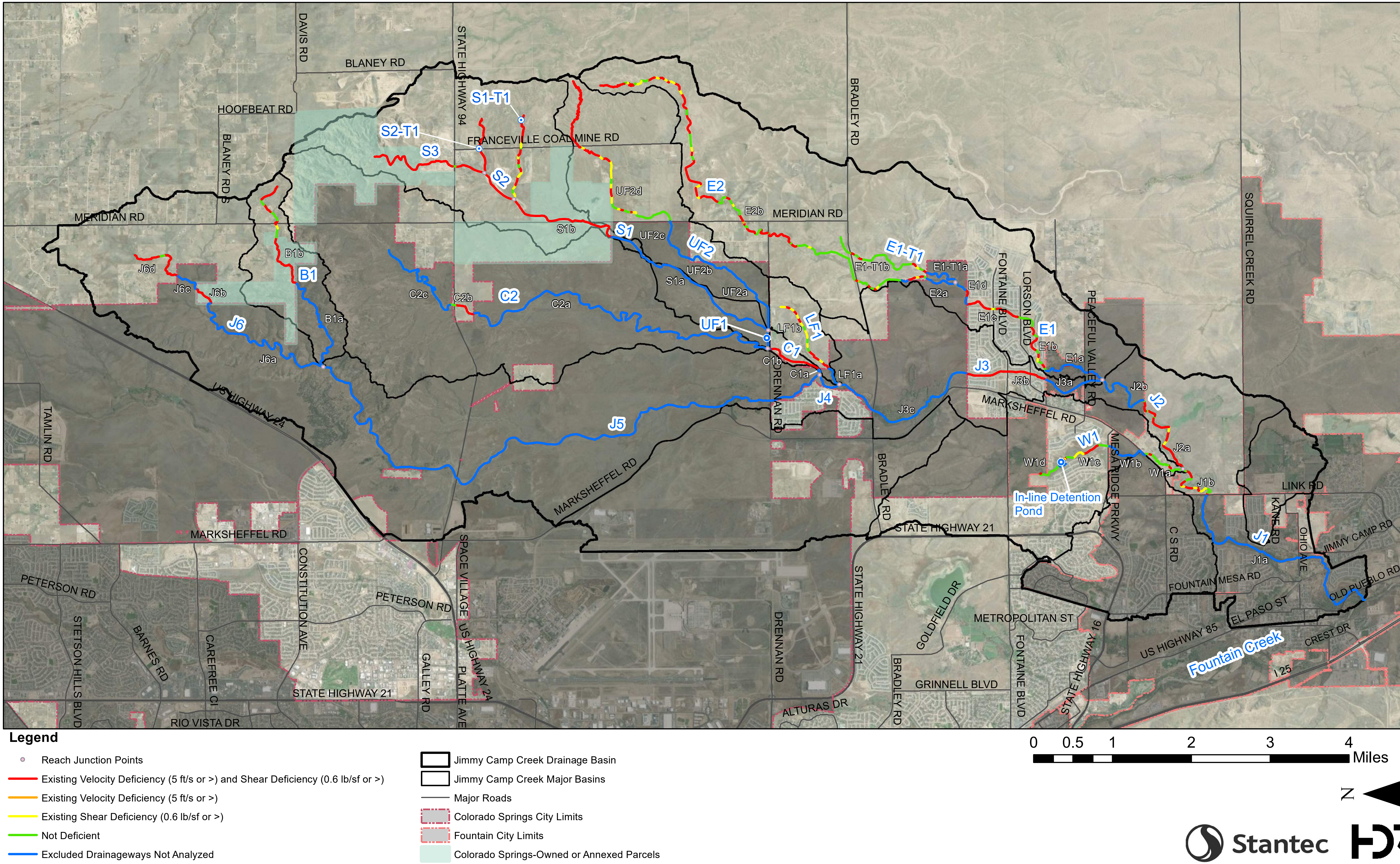
Reach E1 begins at Jimmy Camp Creek and runs upstream for approximately 2.9 miles to the confluence with side tributary E1-T1 on the north side of Lorson Ranch. Approximately 1.8 miles of this reach lies within unincorporated EPC where it runs through Lorson Ranch. Most of this section of the reach has been channelized and stabilized with grade control structures. It is unknown how the design criteria used for the improvements compares to those used for this study. The existing conditions model shows velocity and shear values that exceed the defined limits throughout the reach except for approximately 1,100 feet at the Lorson Boulevard crossing.

Reach E2 begins at the confluence with side tributary E1-T1 and runs upstream for approximately 7.3 miles to the upstream study limit in undeveloped land east of Franceville Coal Mine Road. This reach lies almost entirely within unincorporated EPC. Upstream of Drennan Road, the existing conditions model shows velocity or shear values that exceed the defined limits throughout most of the reach except for some isolated areas where the channel goes through natural depressions or wide sandy flats. South of Drennan Road, the defined main channel disappears, and stream flows meander through a wide, relatively flat valley with many shallow drainage paths. This drainage pattern begins north of Bradley Road and continues south of Bradley Road to the El Paso County / Colorado Springs boundary. Most of the areas where the channel is undefined show acceptable velocity and shear values because the flow is wide and shallow.

East Fork Tributary (E1-T1) begins at the upstream end of E1 and runs northward for approximately 2.0 miles to the upstream study limit north of Bradley Road. Approximately 1.5 miles of the upstream end of this reach lies within unincorporated EPC. Most of this reach is flowing through the same wide, relatively flat area as described above for reach E2, and shows acceptable velocity and shear values until it enters a defined channel approximately 1,300 feet upstream of the El Paso County / Colorado Springs boundary. Flows in the channel exceed the maximum velocity and shear criteria.

Table 4-15. East Fork Tributary 100-Year Velocity and Shear Stress Summary. Future Conditions assumes no on-site detention.

Reach ID	Maximum Velocity (ft/s)		Maximum Shear Stress (lb/ft^2)	
	Existing Conditions	Future Conditions	Existing Conditions	Future Conditions
E1	11.9	15.1	6.6	9.9
E2	10.5	12.5	2.4	2.9
E1-T1	7.6	9.1	1.2	1.8



Stantec HDR

Figure 4-8: Jimmy Camp Creek - Existing Deficiencies

Coordinate System: NAD 1983 StatePlane Colorado Central FIPS 0502 Feet

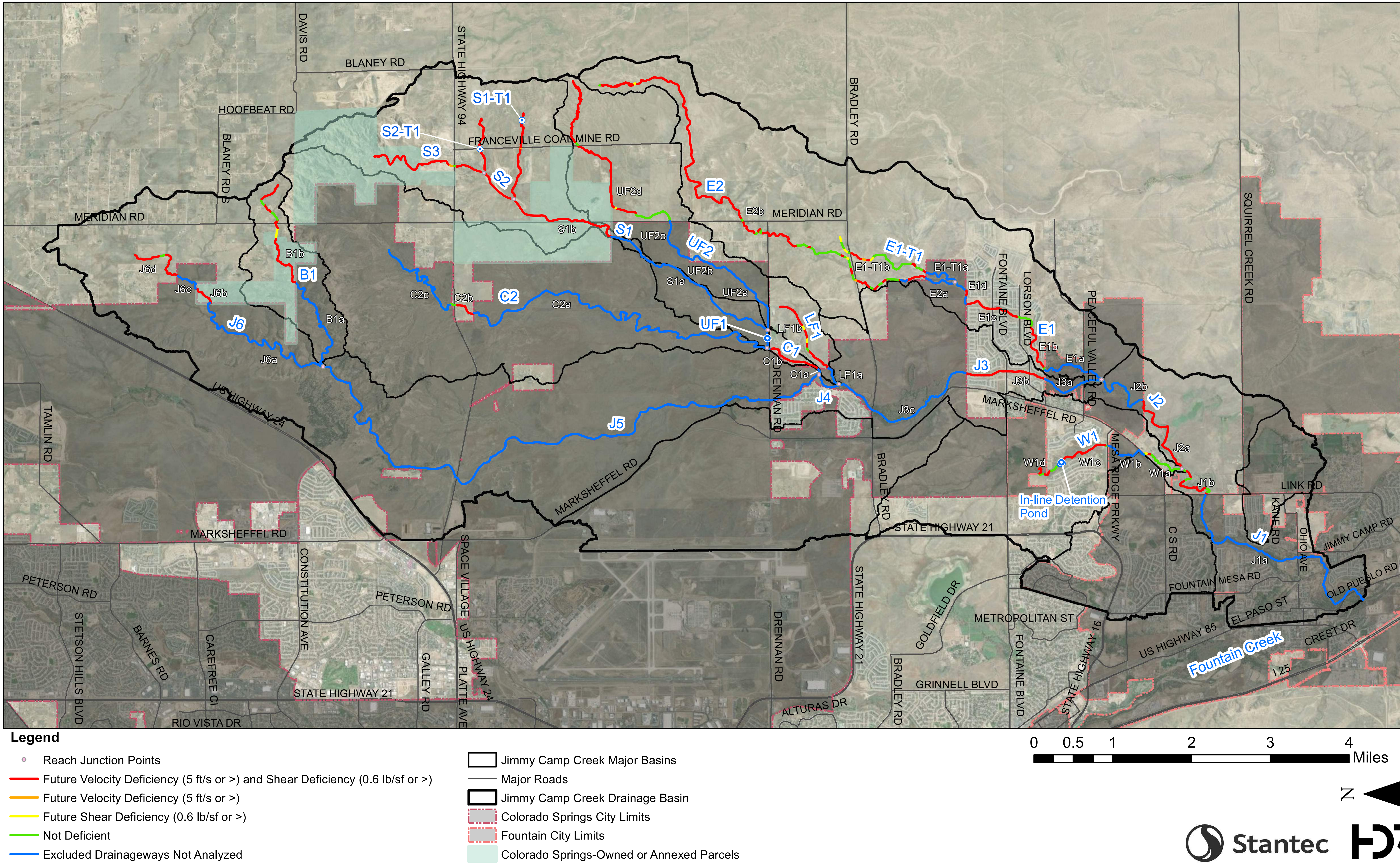


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Figure 4-9: Jimmy Camp Creek - Future Deficiencies

Coordinate System: NAD 1983 StatePlane Colorado Central FIPS 0502 Feet

5.0 ALTERNATIVE EVALUATION

5.1 Introduction

This chapter describes the development and evaluation of drainage alternatives in the Jimmy Camp Creek drainage basin that were designed to address existing and future problem areas. Drainage problem areas in the basin were identified based on the hydrologic and hydraulic analyses described in previous chapters and the geomorphic assessments and other information described in this chapter.

Drainage alternatives represent comprehensive solutions to current and future flooding and channel stability issues in the overall Jimmy Camp Creek drainage basin. They consist of a collection of individual options for specific locations that provide a consistent approach to drainage management from the upper end of the watershed to the outfall at Fountain Creek.

5.2 Hydraulic Deficiencies and Existing Conditions

The Basin Characteristics chapter details a field and desktop geomorphic assessment that identified sediment sources and sinks, as well as potential areas of channel and floodplain instability, providing a general understanding of the health and stability of the watershed given the current conditions. Documentation of existing observed problem areas demonstrated that there are currently limited drainage system issues in the Jimmy Camp Creek drainage basin as it is largely undeveloped and the impacts associated with increased runoff, decreased sediment supply and stream encroachment have not yet occurred in most of the upper basin. The DBPS alternatives are focused on maintaining this stability in the channels and preserving current channel infrastructure while at the same time maintaining current channel capacity under existing and future conditions. There are locations where channelization may be necessary if development in the reach is desired, such as areas without a defined main channel where the flood flows spread across a large portion of the valley. These areas include the East Fork tributary (E2) and side tributary to East Fork (E1-T1) extending from north of Bradley Road to South of Bradley Road and a portion of the Upper Franceville tributary downstream of Franceville Coal Mine Road.

In addition, the hydraulic analysis detailed in the Hydraulics Chapter 4 identified locations where excessive velocity or shear stress is present that could create channel erosion problems under a developed scenario.

The alternatives were developed to address areas in the Jimmy Camp Creek drainage basin that have experienced historical problems with flooding or channel stability, or that are anticipated to experience problems in the future based on anticipated land use and hydrology changes.

5.3 Evaluation Criteria

Multiple evaluation criteria categories were used to evaluate alternatives and select a preferred alternative that best meets the various objectives of the plan. This section describes the development and purpose of the criteria to evaluate and compare Jimmy Camp Creek DBPS alternatives. Application of the evaluation criteria is described in Section 5.4.

5.3.1 Description of Evaluation Criteria

The evaluation criteria adopted for the Jimmy Camp Creek DBPS were based on goals to define different aspects of project success. These goals are defined in Table 5-1 and are organized in four categories: Channel and Floodplain Goals, Environmental Goals, Multiple Benefit Goals, and Cost Goals. Alternatives were compared using the evaluation criteria through a semi-quantitative process.

Environmental goals are related to maintaining a naturally functioning stream, reducing channel construction to the extent possible, reducing excess sediment transport, and reducing permitting requirements. There were no ecological assessments or surveys conducted to determine presence or absence of sensitive or protected species within the Jimmy Camp Creek basin. These surveys may be necessary before detailed design and construction. The costs for these surveys and the potential mitigation measures are not included in this report.

Table 5-1. Jimmy Camp Creek DBPS Goals and Evaluation Criteria

Goal Category	Goals and Evaluation Criteria
Channel and Floodplain Goals	• Remove insurable structures from 100-year floodplain by reducing 100-year peak discharge or relocating structures from the floodplain; note the DBPS itself will not modify the regulated FEMA floodplain • Reduce impact upon major thoroughfares and utilities, existing and future, by improving channel and bridge/culvert capacity • Improve channel stability by reducing or eliminating areas of channel scour, downcutting and lateral migration through creation of stable slopes, grade control, or bank stabilization measures • Minimize the need for intergovernmental negotiations due to jurisdictional boundaries
Environmental Goals	• Improve environmental resources by approximating naturally functioning systems: channels with active floodplains, efficient low flow channels, natural channel and floodplain vegetation, and minimize need for grade control structures • Improve Fountain Creek water quality by reducing the discharge of potential pollutants, primarily in the form of excess sediment from Jimmy Camp Creek to Fountain Creek • Minimize regulatory issues (e.g., wetlands permitting)
Multiple Benefit Goals	• Provide open space and trail opportunities by allowing stream corridors to be used for multiple public recreation benefits • Reduce peak flows by using detention or land management to reduce 10-year and 100-year peak flows to as close to pre-development conditions as possible as required of new development under the El Paso County Drainage Criteria Manual (EPC DCM)
Cost Goals	• Minimize cost for construction and property/right-of-way acquisition • Minimize cost for maintenance, repair, and replacement

5.4 Alternative Development

This section describes the process and basic information used to develop Jimmy Camp Creek DBPS alternatives and the resulting three alternatives developed for the DBPS.

5.4.1 Assumptions for Alternative Development

At the beginning of the alternative development process, a number of assumptions were adopted to focus the effort and avoid exploring alternatives that would ultimately not meet EPC's objectives for the DBPS. The key assumptions framing the alternative development process are listed below:

- **Include effect of onsite detention.** The EPC DCM requires new developments to install extended detention basins or other permanent control measures to maintain post-development runoff rates at pre-development conditions and to mitigate impacts of land development on receiving water quality. Therefore, the peak flows from the existing conditions hydrology were used to develop DBPS alternatives. The flow volumes, however, will be significantly increased by development and it was assumed that the volumes of runoff would be equal to the future condition hydrology computed in Section 3.17. The DBPS assumes that Green Infrastructure (as described in the Colorado Springs Green Infrastructure Guidance Manual; COS 2022) is not incorporated into the developed parcels, however, Green Infrastructure should be considered as it can significantly reduce runoff volumes and reduce the need for channel stabilization.
- **Adopt stable channel slope for planning.** The stable channel slope used for planning was based upon criteria given in the Colorado Springs Drainage Criteria Manual (City DCM). Figure 5-1 shows design slope guidance for sand bed channels that is presented in Figure 12-4 of the City DCM. The stable slope is intended to approximate the slope at which flow velocities and shear stresses allow for a balanced sediment transport to avoid excessive channel erosion. While the current channel under the current flow regime is predominantly stable, development will increase flow volumes and likely decrease coarse (sand and larger) sediment supply. These factors will cause channel and bank erosion and require channel stabilization measures.

  There are two reasons why increased flow volumes can cause channel instability:

  1. The increase in flow volume increases the total sediment transport capacity of the system. Without a commensurate increase in sediment supply in response to the expanded capacity, the imbalance causes erosion of the channel bed.
  2. Vegetation and cohesive material can destabilize during extended flow durations. An example of this destabilization is given in Figure 5-2 (Note vertical scale in Figure is in m/s). In this figure, the 5 ft/s velocity criteria used to identify hydraulic deficiencies is shown as the blue dotted line (note that the figure vertical scale is in m/s). Based upon the intersection of the hydraulic deficiency line with the channel lining categories in Figure 5-2, a channel with good grass cover would be stable for a flow duration greater than 3.5 hrs at 5 ft/s. Currently, large portions of the Jimmy Camp Creek basin channels exceed a 5 ft/s flow velocity; with increasing duration of these velocities, channel erosion could occur. Additionally, the City DCM recommends a maximum design velocity of 5 ft/s and a maximum shear stress of 0.6 lb/ft^2 during the 100-year

flood for natural unlined channels (such as those existing in the Jimmy Camp Creek basin (Table 5-2)). These are the criteria used to determine if channel stabilization is necessary.

It is recommended that before specific stabilization measures are implemented into a reach, a more comprehensive sediment transport analysis be performed where bed material data is collected in each tributary and hydrographs developed to determine sediment loads. The City DCM also discusses interim channel designs for situations where development will not immediately change the existing sediment balance. Because development will occur gradually, the impacts to the channel will occur gradually and the channel improvements could be staged based upon observed channel response.

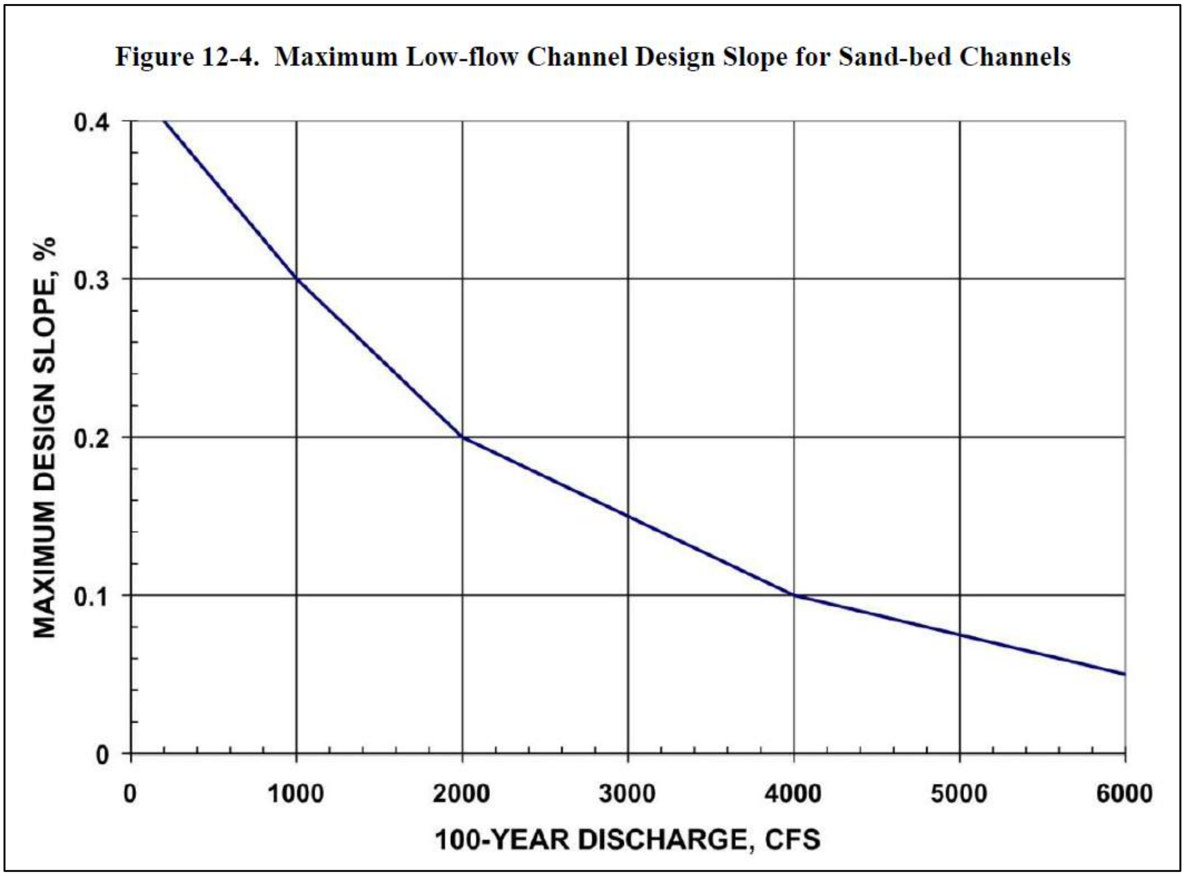


Figure 5-1. Stable Slope Relationship used in JCC DBPS. Taken from City DCM Vol. 1.

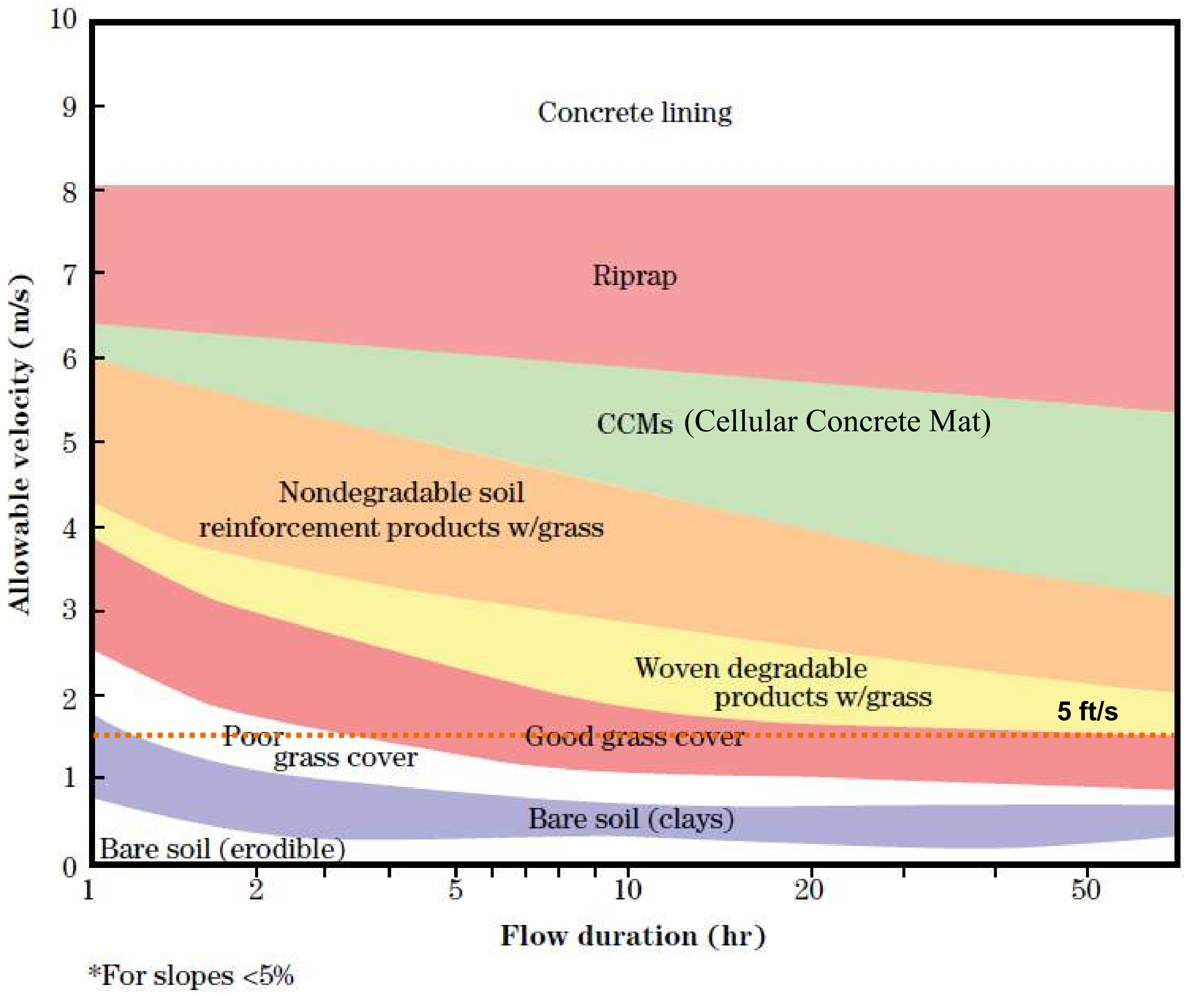


Figure 5-2. Effect of flow duration on allowable velocities for various channel linings from NRCS (2007)

Table 5-2. Hydraulic Design Criteria for Natural Unlined Channels for from City DCM Table 12-3 from Chapter 12.

Design Parameter	Erosive Soils or Poor Vegetation	Erosion Resistant Soils and Vegetation
Maximum Low-Flow Velocity (ft/s)	3.5	5.0
Maximum 100-year Velocity (ft/s)	5.0	7.0
Maximum Froude No. Low Flow	0.5	0.7
Maximum Shear Stress for 100-year (lb/ft^2)	0.60	1.0
[1]Velocities, Froude Number and Shear Stress are average values for the cross section. [2]Erosion Resistant soils are those with 30% or greater clay content.		

- **Incorporate existing improvements to the maximum extent practical.** Some stream reaches in the Jimmy Camp Creek drainage basin have been improved through installation of grade control structures and bank stabilization measures. These improvements will be incorporated into any alternatives, except in the case where the improvements exhibit signs of failure and would have to be replaced. No improvements are included where existing measures are performing as intended.
- **Detention for Water Quality.** It is assumed that the effect of water quality attenuation features is negligible to 100-year flows.
- **Grade Control Design.** Maximum grade control height will be 6 ft for Constructed Channel drops and 4 ft for natural and Constructed Natural channels. This is based upon the maximum height per City DCM Section 4.2.2 Constructed Channel Drop Structures and Table 12-7 Maximum Grade Control Structure Drop Heights.
- **Online Detention**. It was assumed that online detention will not be permitted and that the maximum height of the embankments of detention structures will be 10 ft. This was to prevent the structure from becoming a "Jurisdictional Dam", which is a dam that impounds water above the elevation of the natural surface of the ground creating a reservoir with a capacity of more than 100 acre-feet, or a reservoir with a surface area in excess of 20 acres at the high-water line, or exceeds 10 feet in height measured vertically from the elevation of the lowest point of the natural surface of the ground where that point occurs along the longitudinal centerline of the dam up to the flowline crest of the emergency spillway of the dam. For reservoirs created by excavation, the vertical height shall be measured from the invert of the outlet.

5.4.2 Channel Types

Based on the consideration of the evaluation criteria, existing channel segments were categorized into four types depending on if the channel is improved and if the channel would experience capacity or stability problems based on the existing or future flows. Table 5-3 shows the definition of the four channel categories. Channel types designated for each channel segment in the Jimmy Camp Creek drainage basin are shown on Figure 5-3. The convention used for reach designation consists of three parts. The first is the identifies the stream (e.g., J = Jimmy Camp Creek, and E = East Fork). The second identifies the reach number as determined by tributary junctions (e.g., 1 is the most downstream reach). The third part is a letter to define the reach portion being inside/outside the City boundary. If the portion is outside the City boundary, it can be further separated by its improved/unimproved channel type. This means that when whenever the stream crosses from the city to the County, the next letter in the alphabet is assigned to the reach. The letter is also incremented when the stream changes from improved or unimproved. As an example, Reach J2a is on Jimmy Camp Creek, is the second reach from its mouth as defined by tributary junctions, and is an unimproved reach located outside the City boundaries. Reach J2b would be the reach upstream located within the City boundaries.

Figure 5-3 also shows parcels that were recently annexed by the City since the start of this DBPS and they are designated separately from parcels located within the City before the start of the DBPS. These recently annexed parcels are included in the hydraulic model and are included in the costs estimate for this DBPS.

Table 5-3. Description of Existing Jimmy Camp Creek Channel Types

Channel Types	Description
Type 1	Improved, and no existing or future problems anticipated during peak flows or longer duration high flows • No additional improvements needed • Focus on maintaining existing improvements
Type 2	Improved, but existing or future problems anticipated during peak flows or longer duration high flows • Unless existing improvements are failing or undersized, existing improvements will be maintained to minimize cost • Additional improvements needed to stabilize existing channels and protect existing infrastructure
Type 3	Unimproved, with existing or future problems anticipated during peak flows or longer duration high flows • Extensive improvements will be needed
Type 4	Unimproved, and no existing or future problems anticipated during peak flows or longer duration high flows • No additional improvements needed

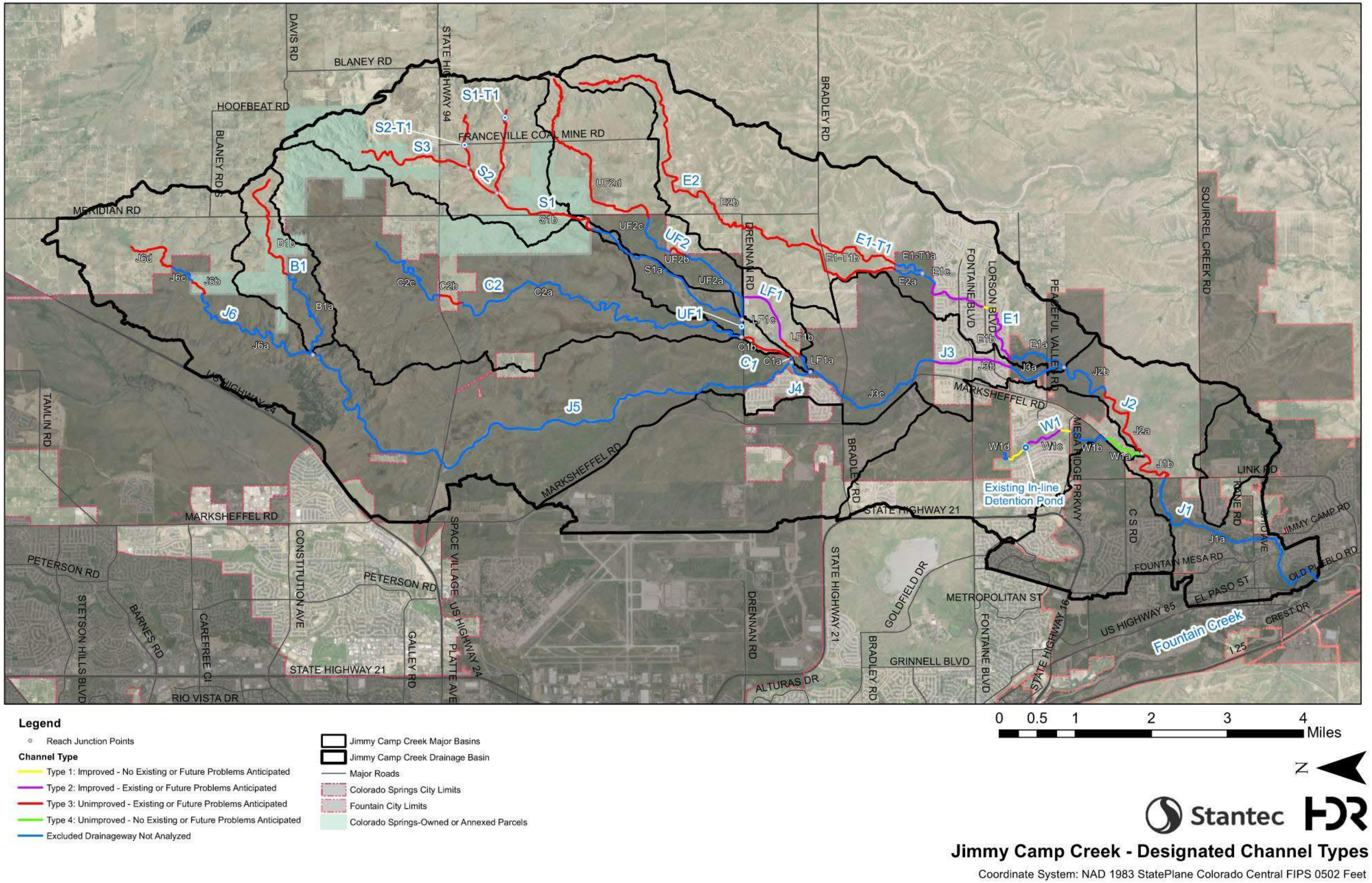


Figure 5-3. Jimmy Camp Creek Drainage Basin Designated Channel Types.

5.4.3 Channel Improvement Themes

5.4.3.1 Description of Channel Improvement Themes

Based on the channel improvement options that were acceptable to the County and the existing channel types, channel improvement themes were created to organize and standardize improvement decisions. Because the Jimmy Camp Creek drainage basin analysis included approximately 65 stream miles, standard channel improvement themes were needed to simplify the development of conveyance options. Table 5-4 shows the definition of the three themes adopted for the Jimmy Camp Creek DBPS. Figure 5-4 shows the themes assigned to each of the channel segments in the basin. Table 5-5 shows a summary of the channel type and channel theme for each reach in the basin. A brief description of the reasoning for each theme is given in the table as well.

Additional explanation for Reach J1 and J2 of Jimmy Camp Creek is warranted given that Maintenance Only is recommended. Both Reach J1 and J2 of Jimmy Camp are densely vegetated with a small main channel and well-connected floodplain. An assessment of these reaches is given in Chapter 2 (Basin Characteristics), and it was determined that maintenance is the preferred method. The flow volumes are expected to increase in this reach as well, but the soils are more cohesive, the vegetation much denser and the upstream channels will supply sediment to this reach even if the upstream hillslope production is decreased.

Table 5-4. Channel Improvement Themes

Channel Theme	Description
Maintenance Only	For some locations, improvements have been made over time or the channel does not need to be modified for future conditions. Proper maintenance and minor localized improvements may be needed but are not included in project costs.
Constructed Channel	For some reaches a constructed channel will be necessary to contain flood flows because there is no defined channel currently. A balanced engineered solution with a terraced floodplain will be used in conjunction with grade control, allowing for some restoration of ecological value within the existing limitations of the right-of-way.
Constructed Natural Channel	In most of the upper portion of the basin, the channels are unimproved as development has not occurred on a large scale. Most of these channel segments fall into Type 3 (see Table 5-3). As flows increase, these channels will experience additional flow and may begin to erode if no stabilization measures are used. A balanced solution allowing some natural stream processes to occur within a defined corridor is preferred.

Table 5-5. Jimmy Camp Creek Channel Types

Drainageway	Reach ID	Channel Type	Channel Theme / Comments
Jimmy Camp	J1b	3	Maintenance / Floodplain preservation because of dense riparian corridor
	J2a	3	Maintenance / Floodplain preservation because of dense riparian corridor
	J3b	2	Maintenance / Reach through Lorson Ranch has been improved.
	J6b	3	Constructed Natural Channel / Grade Stabilization
	J6d	3	Constructed Natural Channel / Grade Stabilization
Blaney	B1b	3	Constructed Natural Channel / Grade Stabilization
West Fork	W1a	4	Maintenance / Reach downstream of Marksheffel Rd runs through large lot development. No structures in floodplain and velocities generally acceptable.
	W1c	1 or 2	Maintenance / Reach upstream of Mesa Ridge Pkwy has been improved.
Corral	C1b	3	Constructed Natural Channel / Grade Stabilization
	C2b	3	Constructed Natural Channel / Grade Stabilization
Stripmine	S1b	3	Constructed Natural Channel / Grade Stabilization
	S2	3	Constructed Natural Channel / Grade Stabilization
	S3	3	Constructed Natural Channel / Grade Stabilization
Stripmine South Tributary	S1-T1	3	Constructed Natural Channel / Grade Stabilization
Stripmine North Tributary	S2-T1	3	Constructed Natural Channel / Grade Stabilization
Lower Franceville	LF1b	3	Constructed Natural Channel / Grade Stabilization
	LF1c	2	Maintenance / Upper part of reach has already been channelized in Pikes Peak National Cemetery.

Table 5-5. Jimmy Camp Creek Channel Types (continued)

Drainageway	Reach ID	Channel Type	Channel Theme / Comments
Upper Franceville	UF2b	3	Maintenance Only / Short reach that runs through large lot development.
	UF2d	3	Lower portion: Constructed Channel / Grade Stabilization. Upper portion: Constructed Natural Channel / Grade Stabilization. Sub-regional Detention also a possibility.
East Fork	E1b	1 or 2	Maintenance / Reach through Lorson Ranch has been improved.
	E2b	3	Lower portion: Constructed Channel / Grade Stabilization. Upper portion: Constructed Natural Channel / Grade Stabilization. Sub-regional Detention also a possibility.
East Fork Tributary	E1-T1b	3	Constructed Channel / Grade Stabilization.

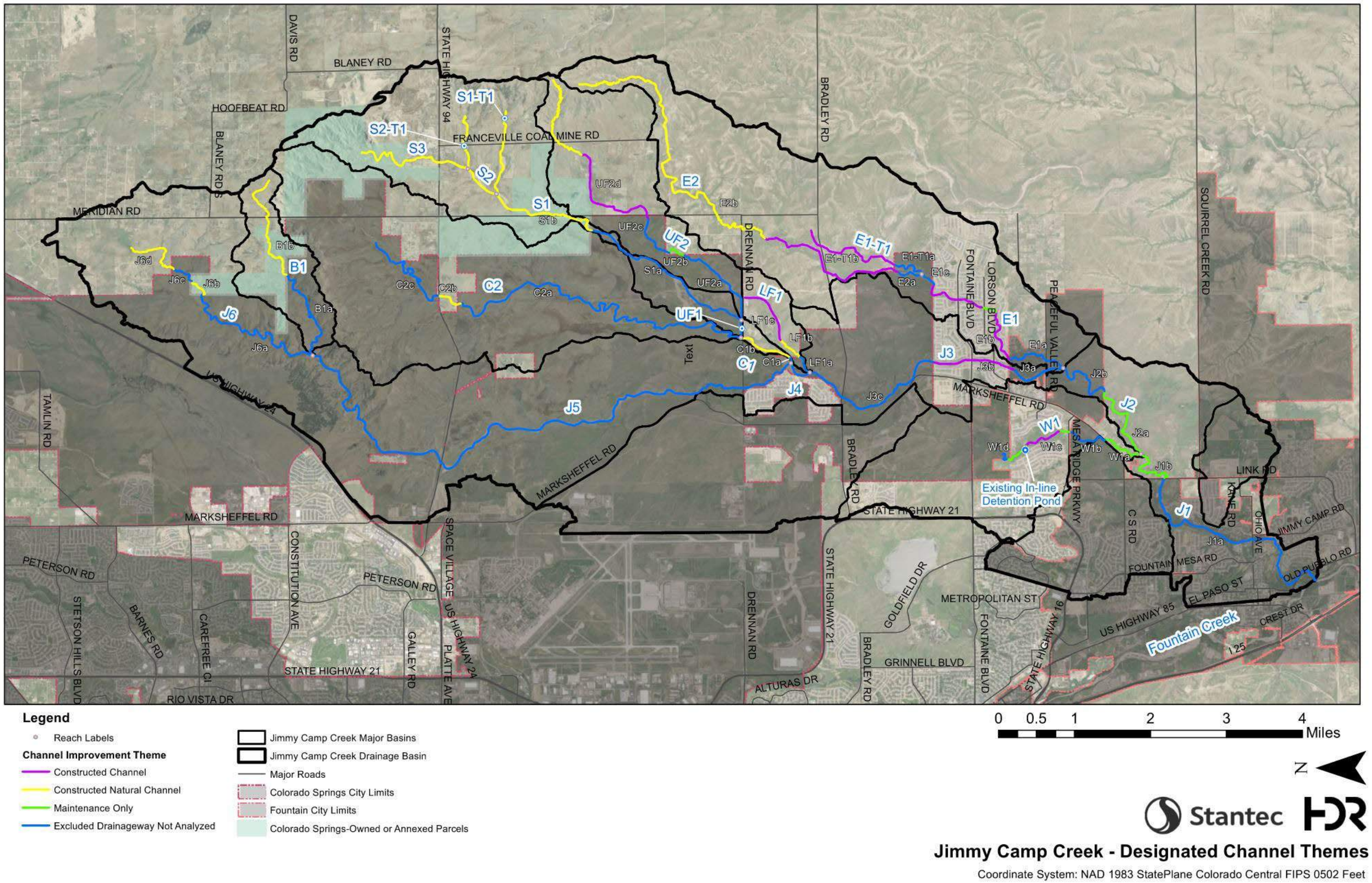


Figure 5-4. Jimmy Camp Creek Drainage Basin Designated Channel Themes.

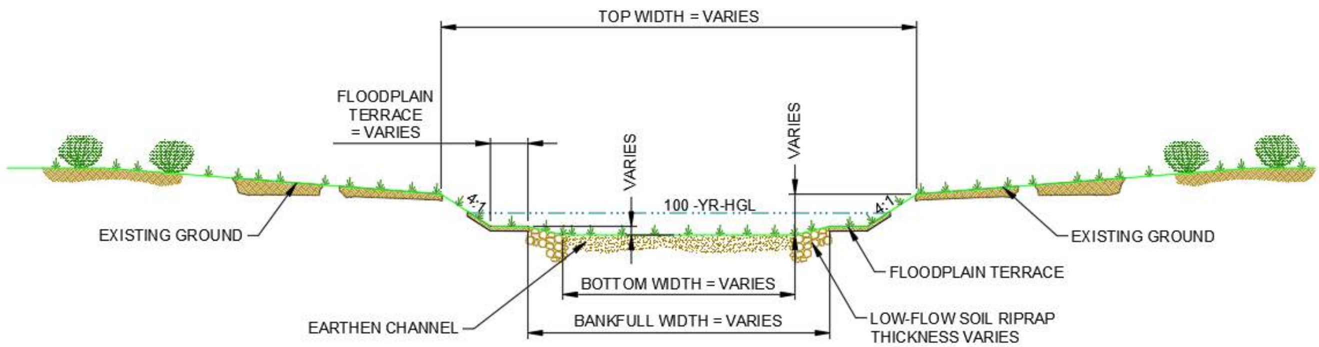
5.4.3.2 Typical Cross Sections

Typical cross sections were designed as guidelines for both the Constructed Channel and the Constructed Natural Channel themes to help with cost estimating and projection of right-of-way requirements. These typical cross sections are based on assumptions of the estimated stable slope and geomorphic parameters of the two themes. Additional geomorphic and sediment transport data will be needed to properly design any specific channel improvement project within these segments.

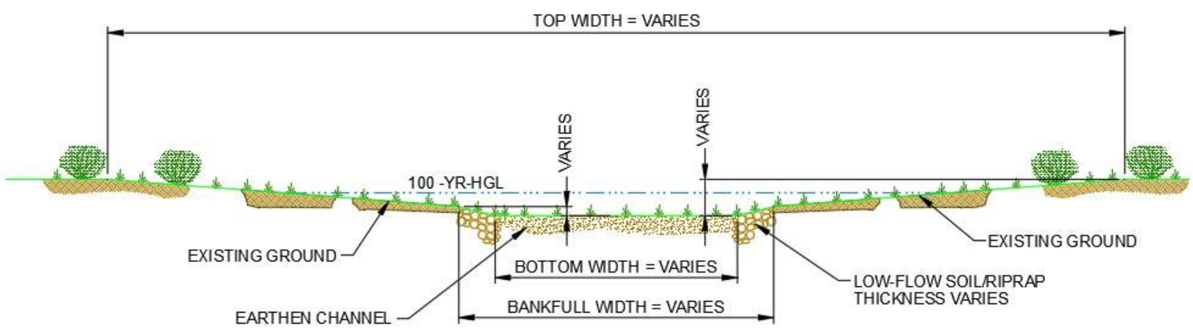
The low flow (defined as bankfull for the purposes of this report) channel dimensions for each stream reach were estimated using guidance from the City DCM. Chapter 12 Section 3.1.1 of the City DCM contains equations for calculating the low flow cross sectional area, width, and depth. These equations are based on analyses of channels in the Jimmy Camp Creek drainage basin. The design cross sectional area of the low flow channel is based on the size of the contributing drainage area. The estimated low flow dimensions for each stream reach have widths ranging from 20 ft to 48 ft and depths ranging from 0.7 ft to 1.6 ft.

As described in Section 2.5.2, Geomorphic Field Assessment, estimates of the low flow (bankfull) dimensions were also developed from the geomorphic assessment that was completed for this study. Reference cross sections were identified at various locations within the basin. Table 2-2 shows a summary of the reference cross section attributes. The estimated low flow dimensions of the reference sections have widths ranging from 3.5 ft to 43.7 ft and depths ranging from 0.8 ft to 3.4 ft. Examination of Table 2-2 shows that the narrowest width values and highest depth value appear to be outliers of the overall dimension results. The range of widths and depths estimated using the City DCM guidance is in general agreement with the reference section bankfull attributes.

Figure 5-5 shows the typical cross sections for Constructed Channels and Constructed Natural Channels. The Constructed Natural Channel incorporates low flow stabilization and full floodplain preservation to provide natural channel functions. The Constructed Channel has a stabilized low flow and overbank floodplain terraces. However, the Constructed Channel is entirely graded in a general trapezoidal shape with no preservation of the existing natural floodplain. The design intent should be to provide sufficiently wide floodplain terraces that limit maximum flow depth and, in combination with grade controls, result in flow velocities that do not require a fully lined channel section.



CONSTRUCTED CHANNEL SECTION



CONSTRUCTED NATURAL CHANNEL SECTION

Figure 5-5. Typical Channel Sections

5.4.4 Channel Stabilization Options

5.4.4.1 Grade Control Options

Due to more runoff and lower sediment yields, long-term stable low-flow channel slopes are expected to be significantly flatter than existing channel slopes. To achieve the desired stable condition, grade control structures are proposed to mitigate steeper channel sections and stabilize the stream reach. The proposed channel is assumed to keep the existing alignment.

The maximum height per the City DCM is 6 ft for Constructed Channels and the maximum drop height for Constructed Natural Channels is 4 ft based upon the City DCM, Table 12-7. The assumed risk of undue maintenance and potential for excessive scour downstream of the 6 ft drop structures was deemed too high for the sand bedded systems of Jimmy Camp Creek. It was assumed that smaller drop structures would result in less maintenance and less interruption of natural sediment processes, so drop structures with a height of 1.5 ft were preferred. If the structure spacing required for a drop height of 1.5 ft became less than 200 ft, the drop height was increased to 2.5 ft, then to 4 ft, as necessary. This was done to avoid an excessive number of drops in short channel reaches. However, in some steep reaches, the structure spacing had to be decreased to less than 200 ft to meet the stabilization goals (Table 5-2) with a maximum drop height of 4 ft.

Figure 5-6 shows the typical drop structure design for steeper channels.

In final design, grade control structures may consist of void filled riprap in a natural configuration or grouted boulders with different heights based on the local features. It is not intended that the final design match the sizes and spacing shown in the alternatives. The final design should provide adequate channel stabilization while incorporating aesthetic design characteristics. The grade control structures may also need to be modified to satisfy fish passage criteria. It is assumed that this will not significantly increase the cost of the grade control structure because most of the grade control structures are 2.5 ft or less.

Any modification to wetlands/Waters of the State could require permitting and mitigation. The recently implemented Colorado Mitigation Procedures (COMP), Colorado Stream Quantification Tool (CSQT) and mitigation banking are used by the U.S. Army Corps of Engineers Regulatory Office to analyze permit applications under Section 404 of the Clean Water Act. The COMP, developed by Colorado regulatory offices, provides regulatory specialists with a framework to objectively evaluate a wetland or a stream’s functional condition by providing a measurable and repeatable method of calculating debits and credits for wetland and waterway impacts caused by permitted activities. These procedures utilize the CSQT, also developed by the Colorado regulatory offices in partnership with the U.S. Environmental Protection Agency, to evaluate a stream’s hydrology, hydraulics, geomorphology, chemistry, and biology. The tool uses a combination of metrics based on watershed data as well as common survey and field measurements, such as width-depth ratios and bank erosion.

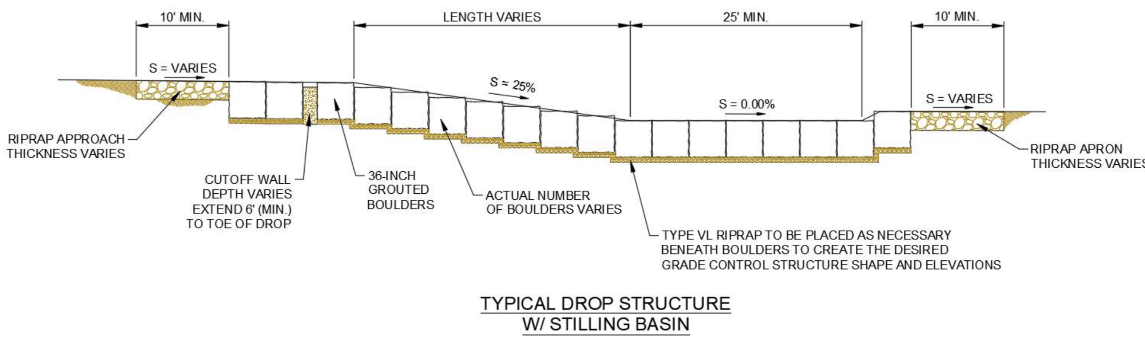


Figure 5-6. Typical Grouted Boulder Drop Structure.

5.4.4.2 Bank Stabilization Options

Channel improvements proposed in the DBPS will alter the existing grade of channels in the areas identified for improvements and reshape the channel and active floodplain to provide additional stability within the system. Bank stabilization measures are included in the Constructed Channel and the Constructed Natural Channel templates to ensure that the grade stabilization features are not flanked by the stream. No significant bank stabilization measures are recommended under existing conditions based upon the geomorphic assessment given in Chapter 2 (Basin Characteristics and Environmental Resources). In some locations, however, additional bank stabilization could be required to protect critical infrastructure that is built in the future.

Bank stabilization could consist of one or more of the following techniques specified in the City DCM Chapter 12.

3. Reduction of Bank Slopes: Reducing bank slopes to 6H:1V or flatter in locations with sufficient right-of-way (ROW) and channel width will assist with vegetation establishment and overall stability. Steeper slopes may be required where site constraints do not allow for shallower slopes, with a maximum of 3H:1V being allowed with appropriate slope protection for the sandy soils present in the basin. This option would also involve revegetation to stabilize regraded banks.
4. Riprap/Boulder Protection: Large riprap or boulder bank protection can be used at locations where ROW conditions limit shallower bank slopes. Riprap or boulder protection should be designed using the method and as defined in the City DCM Section 10.10 Riprap. Riprap bank protection may also be designed to be buried and revegetated to improve channel aesthetics. The decision about whether to use riprap or natural boulders will be based upon cost and aesthetic considerations.
5. Bioengineered Bank: In places where establishment of vegetation is feasible, bioengineered channel banks can provide stability with a more natural look and feel than other armoring techniques. This option would involve use of surface stabilization measures (straw mats, geotextile fabrics, log toes, root wads, etc.) in combination with strategic revegetation selected for the specific application. Bioengineered banks could be used throughout the basin, provided that an appropriate design and plant species are used. In the upper basin, it may be difficult to establish woody species and the

design may have to rely upon herbaceous plants with limited rooting depths and therefore may not be advisable for tall banks. In many areas, however, the bank heights are small and the bankfull depth is less than 1.5 feet in many of the streams (see Table 2-2). The final selection of appropriate bank stabilization techniques is dependent upon several factors including proximity of infrastructure, climate, soils, water table, and hydraulic conditions (NRCS Technical Supplement 14I, Streambank Soil Bioengineering).

5.4.5 Improvements to Existing Hydraulic Structures

Existing culverts and bridges with inadequate capacity for existing conditions are listed in Table 5-6 and shown on Figure 5-. Roadway crossings overtopped by the 100-year storm were defined as deficiencies. The hydraulic model was used to determine which crossings are overtopped by the 100-year flood. Crossings are labeled as deficient if any part of the modeled roadway is overtopped, even if the roadway low point is not located directly on top of the structure.

New hydraulic structures included in the alternatives are listed in Sections 5.4.10 and 5.4.11. Necessary improvements to these structures were determined by sizing a new structure to carry the 100-year peak flow without causing pressure flow in the new structure. A structure that doesn't allow pressure flow will more likely pass debris and will cause less scour downstream of the structure.

Table 5-6. Evaluation of Existing Structures

No.	Drainage	Reach Name	Location	Structure Description	Source of Data	Existing 100-Year Flow (cfs)	Existing Structure Capacity	Jurisdiction
1	Jimmy Camp	J3	Peaceful Valley Rd.	4 – 30” CMP	EPC field data	7,241	Overtopped	City of Fountain
2	West Fork	W1	Furlong Cir.	54" CMP	EPC GIS data	141	Overtopped	El Paso County
3	West Fork	W1	Ingle Ln.	2 - 36" CMP	EPC GIS data	141	Overtopped	El Paso County
4	West Fork	W1	Marksheffel Rd.	24" RCP	Stantec field data	141	Overtopped*	City of Fountain
5	Upper Franceville	UF2	Franceville Coal Mine Rd.	2 - 36" CMP	EPC GIS data	182	Overtopped*	El Paso County
6	East Fork Tributary	E1-T1	Bradley Rd.	2 - 66" RCP	Construction plans	424	Overtopped*	El Paso County
7	East Fork	E2	Meridian Rd.	2 - 36"x48" HERCP + 2 - 36" RCP	EPC GIS data	507	Overtopped*	El Paso County

Notes:

* Hydraulic model shows overtopping but headwater can also be diverted away from crossing in roadside ditch. More detailed modeling is required to assess conditions at the crossing.

CMP = Corrugated Metal Pipe

RCP = Reinforced Concrete Pipe

CBC = Concrete Box Culvert

HERCP = Horizontal Elliptical Reinforced Concrete Pipe

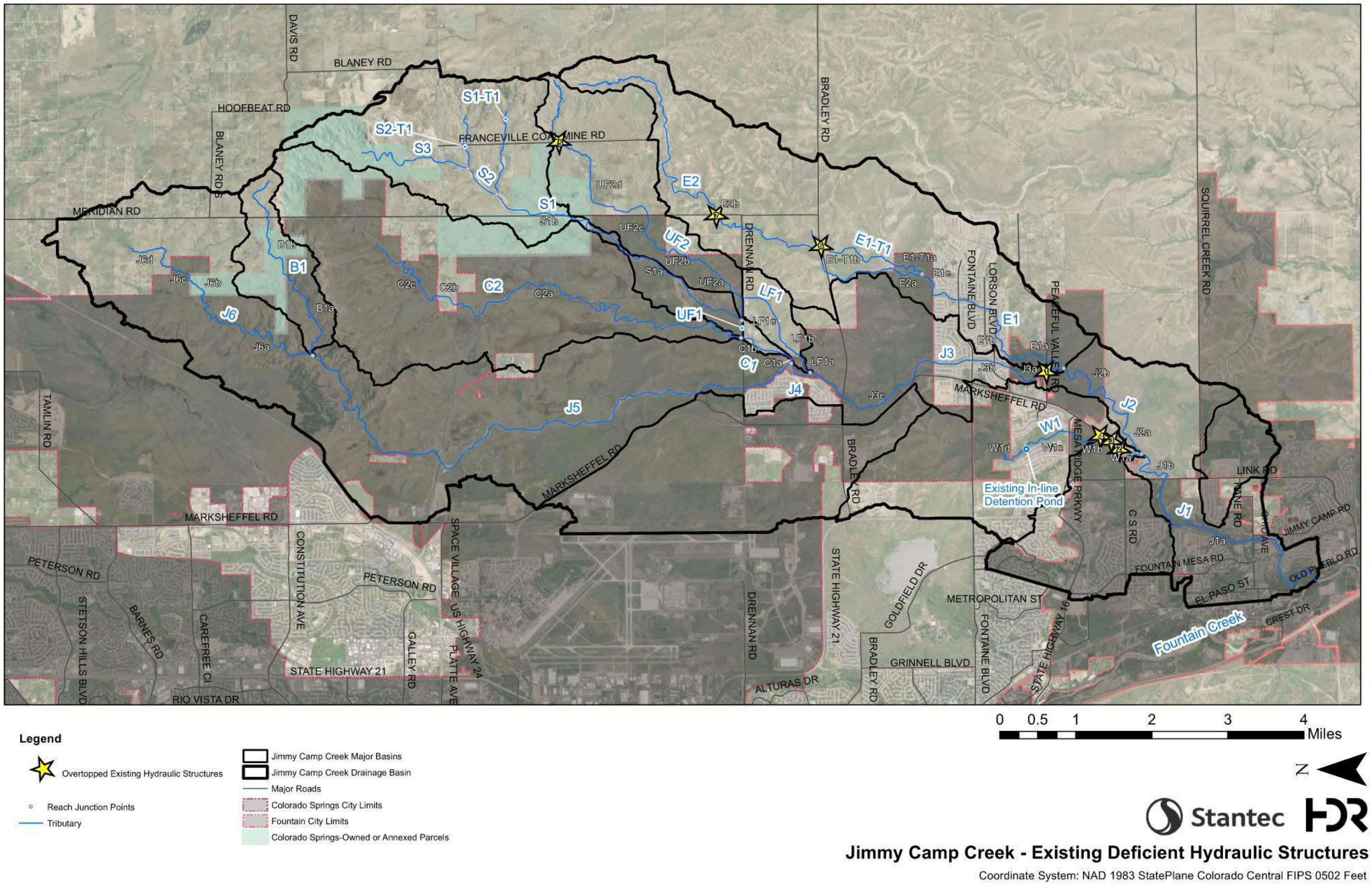


Figure 5-7. Existing Deficient Hydraulic Structures Map.

5.4.6 Regional Detention Options

As part of the alternatives analysis, regional detention alternatives were evaluated. The objectives of regional detention were the following:

- Reduce channel velocities to less than 5 ft/s
- Reduce number of structures affected by flooding
- Reduce areas flood extents, particularly where the 100-year flood plain is much larger than the main channel.

There are many considerations when designing detention structures. As mentioned previously, to prevent it from becoming a dam under the jurisdiction of the state, the embankment height must be less than 10 feet. Online detention basins are not considered feasible alternatives because of the water quality regulations that prevent degradation of water quality and the fact that these online detention basins would trap significant amounts of sediment and require sediment excavation. The regulation can be found in Colorado Department of Public Health and Environment (CDPHE) water quality Regulation No. 31 – The Basic Standards and Methodologies for Surface Water 5 CCR 1002-31 (https://cdphe.colorado.gov/water-quality-control-commission-regulations). Online or offline detention basins also have the potential to expose the project to water right liabilities as defined under Senate Bill 15-212 / Colorado Revised Statute (CRS) §37-92-602 (8) which states that the operation of the detention facility will not cause a reduction to the natural hydrograph as it existed prior to the upstream development.

Another constraint on a detention alternative is that the facility must be within unincorporated El Paso County and provide significant benefit to parcels in the unincorporated areas. For example, a previous detention analysis for the Jimmy Camp Creek basin had detention facilities further down in the watershed where there would be limited benefits to County parcels, and most of the benefit was realized in City owned parcels. Based upon this constraint, only the East Fork, Upper Franceville, and Stripmine tributaries have significant lengths of stream within the County that could benefit from detention.

The stream velocities in East Fork, Upper Franceville, and Stripmine were analyzed to determine how much the 100-year flow would have to be reduced to have channel velocities below 5 ft/s. In Upper Franceville and East Fork, it was found that the flow would have to be reduced to the 25-year flood to have channel velocities less than 5 ft/s. In Stripmine, it was found that the 100-year flow would have to be reduced down to the 2-year flood level to have channel velocities less than 5 ft/s. This would equate to a flow reduction from 2,074 cfs to 177 cfs in Stripmine at Highway 94. The 100-year floodplain is also generally well contained within the channel for the majority of its length in the County. For these reasons, it was determined that detention within Stripmine is not feasible.

Potential locations for detention along East Fork and Upper Franceville were determined by placing the detention in the upper portion of the watershed to have the maximum potential benefit to parcels within the County.

The three regional detention locations identified for evaluation in the DBPS are presented on Figure 5-. Two locations are on the Upper Franceville tributary, and the third location is on the East Fork tributary. These locations result in 4 separate scenarios as defined in Table 5-7.

Table 5-7. Regional Detention Scenarios

Scenario	Name	Note
Scenario 1	Location 1 on the Upper Franceville tributary	Single regional detention location on the Upper Franceville tributary at Location 1.
Scenario 2	Location 2 on the Upper Franceville tributary	Single regional detention location on the Upper Franceville tributary at Location 2.
Scenario 3	Location 1 and Location 2 on Upper Franceville tributary	Two regional detention locations on the Upper Franceville tributary. Location 1 is sized similar to Scenario 1, with a reduction in size of Location 2.
Scenario 4	Location 1 on East Fork tributary	Single regional detention location on the East Fork tributary.

The objectives of regional detention are to attenuate existing and future 100-year peak flow rates down to existing 25-year peak flow rates. The 25-year flow rate was chosen because that is approximately the flow rate when the flow velocities were limited to less than 5 ft/s downstream of the detention basin.

County regulations and design criteria require post-development 100-year peak flow rates to be mitigated to existing conditions. Given that the future conditions hydrologic modeling only assumed future imperviousness and that modeling onsite detention is beyond the scope of this DBPS, a SWMM based modeling solution is not available to estimate detention requirements with respect to future flow rates. However, the future conditions modeling is representative of future volumes and, therefore, a spreadsheet model was used to estimate regional detention requirements to mitigate flows to existing 25-year rates. Table 5-8 summarizes the future 100-year flow volumes and existing 25-year flow rates used as metrics in this analysis.

The spreadsheet model was developed to limit the channels downstream of the conceptual regional detention locations to existing 25-year maximum flow rates. Based on these existing 25-year maximum flow restrictions coupled with the 100-year total volumes associated with future land use impervious percentages, the regional detention volume estimations are provided in Table 5-9, along with estimations of the corresponding areas required to accommodate these volumes. Given that area requirements are highly variable depending upon how the site is graded, the accommodation of access roads, and other area considerations, ranges of values are provided. The lower value in the area range is calculated based on a flat pond bottom, a square footprint, 4-foot horizontal to 1-foot vertical side slopes, and a 30 percent increase to accommodate access roads and easements. The maximum height of the embankment is assumed to be 10 ft to avoid state jurisdictional dam status. The upper value in the area range is calculated based on grading plans associated with pond designs that have much less volume at the lower depths due to water quality features, trickle channels, pond bottom slopes, etc. If regional detention is pursued, area requirements should be designed by developing site-specific grading plans based on County criteria.

Table 5-8. Regional Detention Volume and Flow Metrics

Scenario	Location 1 on the Upper Franceville Tributary		Location 2 on the Upper Franceville Tributary		Location 1 on East Fork Tributary	
	Existing 25-year Peak Flow Rate	Future 100-Year Total Flow Volume	Existing 25-year Peak Flow Rate	Future 100-Year Total Flow Volume	Existing 25-year Peak Flow Rate	Future 100-Year Total Flow Volume
Scenario 1	53 cfs	24 ac-ft	N/A	N/A	N/A	N/A
Scenario 2	N/A	N/A	61 cfs	46 ac-ft	N/A	N/A
Scenario 3*	53 cfs	24 ac-ft	61 cfs	46 ac-ft	N/A	N/A
Scenario 4	N/A	N/A	N/A	N/A	87 cfs	131 ac-ft
* Location 1 unchanged between Scenarios 1 and 3 due to the same flow rates and subsequent attenuation objectives						

Table 5-9. Regional Detention Size Requirements

Scenario	Location 1 on the Upper Franceville Tributary		Location 2 on the Upper Franceville Tributary		Location 1 on East Fork Tributary	
	Approximate Volume Requirement	Approximate Grading Area Requirement	Approximate Volume Requirement	Approximate Grading Area Requirement	Approximate Volume Requirement	Approximate Grading Area Requirement
Scenario 1	10.5 ac-ft	2.3 – 3.0 acres	N /A	N/A	N/A	N/A
Scenario 2	N/A	N/A	22 ac-ft	4.5 – 5.7 acres	N/A	N/A
Scenario 3*	10.5 ac-ft	2.3 – 3.0 acres	13.5 ac-ft	3.0 – 3.5 acres	N/A	N/A
Scenario 4	N/A	N/A	N/A	N/A	80 ac-ft	13.8 – 19.5 acres
* Location 1 unchanged between Scenarios 1 and 3 due to the same flow rates and subsequent attenuation objectives						

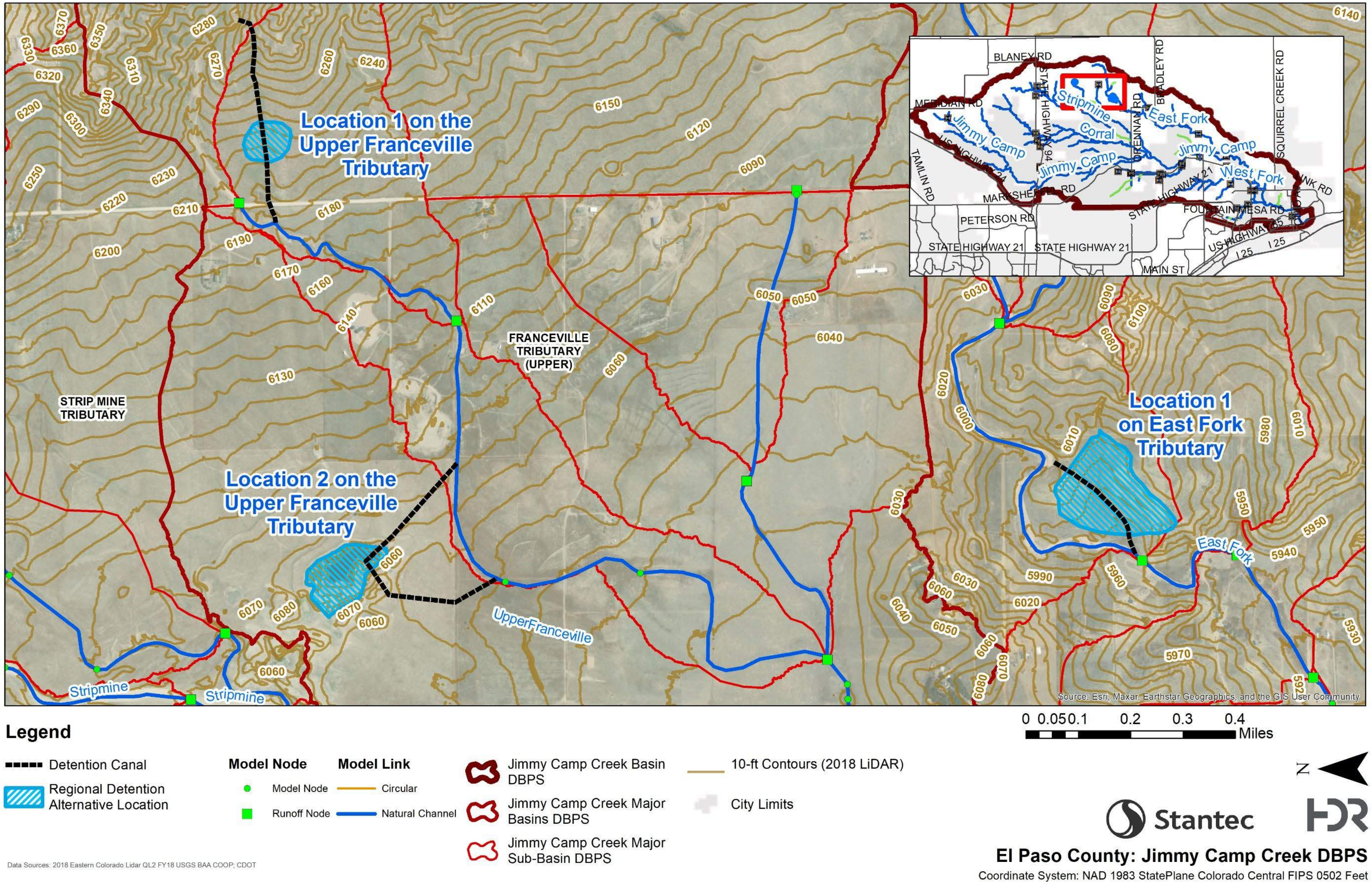


Figure 5-8. Analyzed Locations for Conceptual Regional Detention

5.4.7 Storm Drain Improvements

Select storm sewers were evaluated as discussed in Section 4.8.2 Hydraulics. No costs for storm drain improvements were incorporated into this DBPS and therefore they were not included in the alternative analysis. The DBPS did not have the site-specific design conditions nor the level of detail necessary to recommend improvements, and therefore, no storm system improvements at the studied locations are recommended.

5.4.8 Overview of Alternatives

Alternatives were developed to address areas in the Jimmy Camp Creek drainage basin that have experienced historical problems with flooding or channel stability, or that are anticipated to experience problems in the future based on anticipated land use and hydrology changes. Three alternatives were considered for the basin:

- No Action
- Alternative 1 – Conveyance Improvements with No Regional Detention
- Alternative 2 – Conveyance Improvements with Maximum Feasible Regional Detention.

The only significant difference between Alternatives 1 and 2 is the inclusion of regional detention in Alternative 2. The primary objective of detention within Alternative 2 was to decrease 100-year flow velocities downstream of the detention basins to less than 5 ft/s to reduce costs of channel improvements. The stream stabilization measures in the East Fork and Upper Franceville tributaries could be reduced because of the reduction in peak flows. In all other stream segments, the channel improvements (typical cross sections and grade control structures) are the same for both alternatives. The roadway crossings and storm sewer improvements required for existing and future proposed facilities are also the same in both alternatives.

The Alternatives development did not consider potential impacts of the Colorado Mitigation Procedures (COMP) on design and costs of stream restoration. The COMP was developed by Colorado regulatory offices in response to the 2008 Compensatory Mitigation Rule (33 CFR 332), as discussed previously in Section 5.4.4.1.

5.4.9 No Action Alternative

5.4.9.1 No Action Hydrology

The No Action Alternative would allow for development but contains no channel improvements. The peak flows under the No Action Alternative would be consistent with the existing conditions peak flows. As described previously, the EPC DCM requires new developments to install extended detention basins or other permanent control measures to maintain post-development runoff rates at pre-development conditions and to mitigate impacts of land development on receiving water quality. Therefore, the peak flows from the existing conditions hydrology were used to develop DBPS alternatives. However, the increase in impervious area would increase the volumes of runoff and runoff volumes would be similar to the future conditions hydrology. The assumed percent impervious for the majority of the basins is above 30% (see Figure 3-9)) and this will have significant impacts on runoff volumes. A plot of the 10-year flood volumes under existing and future conditions is shown in Figure 5-9. The flow volumes increased by an average factor of approximately 6 for drainage areas over 10 sq mi.

5.4.9.2 No Action Channel Response

This large increase in flow volume without a commensurate increase in sediment supply will result in significant erosion of the stream channels. No future channel improvements are assumed to occur under the No Action Alternative and therefore erosion is expected to occur throughout the stream system under the No Action Alternative. The erosion would result first in vertical incision followed by bank failure as the bank height increases to unstable heights. The bank failure will result in channel widening and potential loss of property. The most common conceptual model of channel evolution resulting from an increase in flow volume was given in Section 2.5.2.2. Because the stream beds are primarily composed of erodible sandy material, the erosion response will likely be rapid and relatively large. The Sand Creek watershed is a nearby example of what will occur in the Jimmy Camp Creek watershed. After development within the Sand Creek watershed, the increase in flow volumes resulted in extensive erosion of stream channels that continues to occur. One example of channel incision is given in Figure 5-10, and it is further documented in the Sand Creek Drainage Basin Planning Study (January 2021). A similar response is expected in the Jimmy Camp Creek watershed after development without adequate stream channel design and planning.

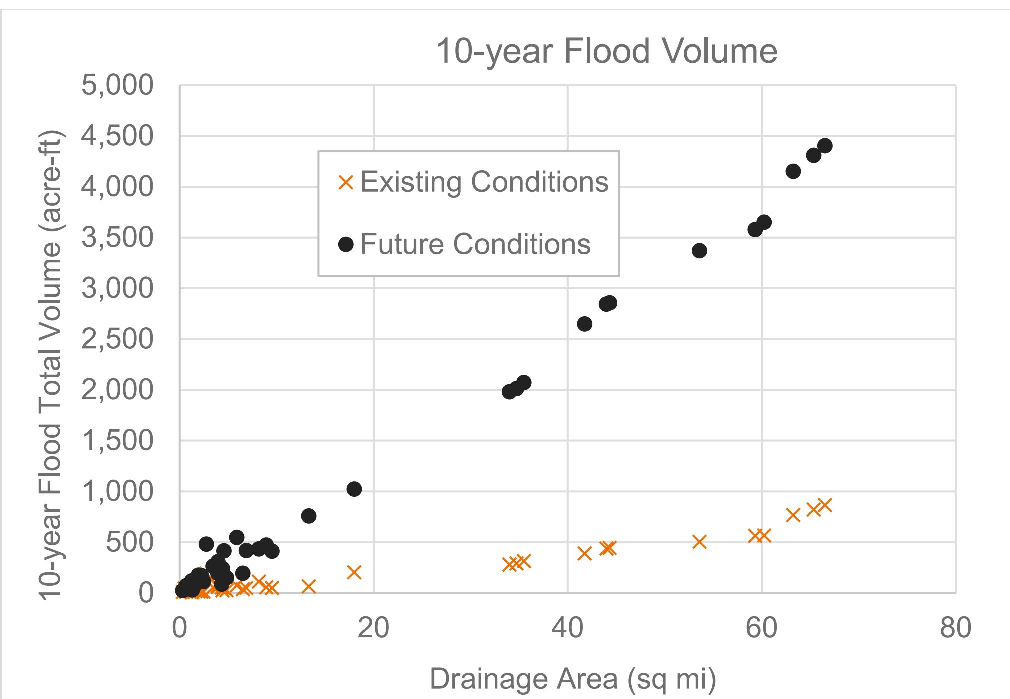


Figure 5-9. Change in flow volumes due to future development based upon Existing and Future Conditions Hydrologic Model

Figure 5-10. Example of degradation in Sand Creek near Barnes Road Bridge in Colorado Springs.

5.4.10 Alternative 1 – Conveyance Improvements with No Regional Detention

5.4.10.1 Alternative 1 Hydrology

Hydrology for Alternative 1 was based on the existing conditions peak flows, but the volume of flow would be consistent with future conditions. As described previously, the EPC DCM requires new developments to install extended detention basins or other permanent control measures to maintain post-development runoff rates at pre-development conditions and to mitigate impacts of land development on receiving water quality. Therefore, the peak flows from the existing conditions hydrology were used to develop DBPS alternatives. It was assumed that the volumes of runoff would be similar to the future conditions hydrology.

The existing conditions peak flows are given in Table 3-5 Hydrologic Analysis. Alternative 1 was developed with no assumption of any hydrologic adjustment for effects of proposed conveyance system improvements such as altered cross section geometry or flatter channel slopes. This was due to the proposed installation of grade control structures to maintain maximum allowable slopes.

5.4.10.2 Alternative 1 Channel Improvements

The channel improvements associated with Alternative 1 were based on the channel themes described in Section 4.4.3. Maintenance Only, Constructed Channel, and Constructed Natural Channel themes were applied to each stream reach based on existing conditions and future development.

Alternative 1 includes grade control structures in every channel reach in which the current slope exceeds the stable slope, with the exception of reaches J1 and J2, where dense vegetation is present along banks and the floodplain. As discussed in Chapter 2.0 Basin Characteristics and in Section 4.4.3, reach J1 and J2 are densely vegetated with a small main channel and well-connected floodplain. These factors reduce flow energy and stabilize soils. Table 5-10 gives a summary of the channel and grade control improvements in each stream reach.

Table 5-10. Alternative 1 Conveyance Improvements

Drainageway	Reach ID	Unincorp. EPC Reach Length [ft]	Reach Length to be Improved and Included in Cost Estimate [ft]	Channel Type	Selected Channel Theme	Assumed Stable Slope	Low Flow Channel Geometry		# of Grade Control Structures		
							Width [ft]	Depth [ft]	1.5 ft	2.5 ft	4 ft
Jimmy Camp Creek	J1b	4,447	0	Type 3 – Unimproved – Existing or Future Problems	Maintenance Only	---	---	---	---	---	---
	J2a	6,460	0	Type 3 – Unimproved – Existing or Future Problems	Maintenance Only	---	---	---	---	---	---
	J3b	5,464	0	Type 2 – Improved – Existing or Future Problems	Constructed Channel	---	---	---	---	---	---
	J6	6,147	6,147	Type 3 – Unimproved – Existing or Future Problems	Constructed Natural Channel	0.30%	30	1.0	5	31	5
Blaney	B1b	8,783	8,783	Type 3 – Unimproved – Existing or Future Problems	Constructed Natural Channel	0.35%	23	0.8	0	7	52
Corral	C1b	3,679	3,679	Type 3 – Unimproved – Existing or Future Problems	Constructed Natural Channel	0.06%	41	1.4	14	3	0
	C2b	1,737	1,528	Type 3 – Unimproved – Existing or Future Problems	Constructed Natural Channel	0.15%	32	1.1	5	2	0
Lower Franceville	LF1b	2,191	2,191	Type 3 – Unimproved – Existing or Future Problems	Constructed Natural Channel	0.35%	22	0.7	0	10	0
	LF1c	4,281	0	Type 2 – Improved – Existing or Future Problems	Constructed Channel	---	---	---	---	---	---
Upper Franceville	UF2b	1,281	0	Type 3 – Unimproved – Existing or Future Problems	Maintenance Only	0.35%	---	---	---	---	---
	UF2d	9,617	8,336	Type 3 – Unimproved – Existing or Future Problems	Constructed Channel	0.35%	22	0.7	0	4	24
		8,093	8,093	Type 3 – Unimproved – Existing or Future Problems	Constructed Natural Channel	0.35%	20	0.7	0	0	60
Stripmine	S1b	7,981	7,981	Type 3 – Unimproved – Existing or Future Problems	Constructed Natural Channel	0.10%	33	1.1	1	33	0
	S2	2,763	2,763	Type 3 – Unimproved – Existing or Future Problems	Constructed Natural Channel	0.10%	30	1.0	0	14	0
	S3	9,567	9,388	Type 3 – Unimproved – Existing or Future Problems	Constructed Natural Channel	0.15%	27	0.9	0	24	22
Stripmine South Tributary	S1-T1	6,417	6,417	Type 3 – Unimproved – Existing or Future Problems	Constructed Natural Channel	0.35%	24	0.8	0	1	36
Stripmine North Tributary	S2-T1	4,141	3,921	Type 3 – Unimproved – Existing or Future Problems	Constructed Natural Channel	0.30%	21	0.7	0	2	21
East Fork	E1b	4,117	0	Type 2 – Improved – Existing or Future Problems	Constructed Channel	0.20%	---	---	---	---	---
	E1b	970	0	Type 1 – Improved – No Existing or Future Problems	Maintenance Only	0.20%	---	---	---	---	---
	E1b	4,438	0	Type 2 – Improved – Existing or Future Problems	Constructed Channel	0.20%	---	---	---	---	---
	E2b	11,226	11,226	Type 3 – Unimproved – Existing or Future Problems	Constructed Channel	0.30%	33	1.1	25	9	2
	E2b	25,089	25,089	Type 3 – Unimproved – Existing or Future Problems	Constructed Natural Channel	0.30%	29	1.0	21	34	66
East Fork Tributary	E1-T1b	7,698	7,698	Type 3 – Unimproved – Existing or Future Problems	Constructed Channel	0.30%	23	0.8	20	7	0
West Fork	W1c	8,929	0	Type 2 – Improved – Existing or Future Problems	Constructed Channel	---	---	---	---	---	---
TOTAL		155,516	113,240								

5.4.10.3 Alternative 1 Improvements to Existing Hydraulic Structures

Based upon the structure evaluations in the hydraulic analysis, the required roadway crossing improvements are shown in Table 5-11. For the purposes of the alternative analysis, it was assumed that a box culvert would be used to convey water through the crossing. The proposed structure is designed to convey the 100-year flood with no overtopping, which may be in excess of design criteria that allows overtopping of collector and local roads.

Table 5-11. Proposed Improvements to Hydraulic Structures

Drainage	Reach Name	Location	Existing Structure Description	Source of Data	Existing 100-Year Flow (cfs)	Proposed Structure
West Fork	W1	Furlong Cir.	54” CMP	EPC GIS data	141	1 – 8’ x 4’ CBC
West Fork	W1	Ingle Ln.	2 – 36” CMP	EPC GIS data	141	1 – 8’ x 4’ CBC
Upper Franceville	UF2	Franceville Coal Mine Rd.	2 – 36” CMP	EPC GIS data	182	1 – 10’ x 4’ CBC
East Fork Tributary	E1-T1	Bradley Rd.	2 – 66” RCP	Construction plans	424	1 – 10’ x 8’ CBC
East Fork	E2	Meridian Rd.	2 – 36”x48” HERCP + 2 – 36” RCP	EPC GIS data	507	3 – 12’ x 4’ CBC

5.4.10.4 Alternative 1 Regional Detention Improvements

Alternative 1 does not include any regional detention.

5.4.10.5 Alternative 1 Storm Sewer Improvements

The hydraulic analysis included a capacity assessment of large diameter storm sewer pipes at select locations (see Section 3.9.2). However, based on site-specific design conditions at a level of detail not included in this DBPS, no storm system improvements at the studied locations are recommended.

5.4.11 Alternative 2 – Conveyance Improvements with Regional Detention

5.4.11.1 Alternative 2 Hydrology

Hydrology for Alternative 2 was based on the existing conditions peak flows, but assuming future conditions volumes. As described previously, the County DCM requires new developments to install extended detention basins or other permanent control measures to maintain post-development runoff rates at pre-development conditions and to mitigate impacts of land development on receiving water quality. Therefore, the peak flows from the existing conditions hydrology were used to develop DBPS alternatives, while the volume of runoff was taken from the future condition's hydrology.

Alternative 2 includes all three proposed detention structures (scenario 3 and 4 as defined in Section 4.4.6) on the East Fork and Upper Franceville Tributaries. As described previously, the conceptual regional detention facilities were designed to limit peak flows downstream of the ponds to the existing conditions 25-year flow rates in order to limit flow velocities to approximately 5 ft/s downstream of the detention basin. Table 5-12 summarizes the peak flows for Alternative 2 where they are reduced from the Alternative 1 flows. The 100-year flows downstream of the detention basins for Alternative 2 were computed by assuming that the reduction in flow for the 100-year flood immediately downstream of the basin is constant throughout the tributary.

Table 5-12. Locations where Alternative 2 Peak Flows are reduced from Alternative 1

Major Drainageway	Reach ID	SWMM Model Node ID	Location Description	Existing		Alternative 2	
				100-Year Flow (cfs)	25-Year Flow (cfs)	100-Year Flow (cfs)	25-Year Flow (cfs)
East Fork	E2	DSNPT_E2_6	Drennan Rd	486	97	97	97
	E2	DSNPT_E2_2	Bradley Rd (West)	568	112	179	112
	E2	DSNPT_E1_8	At City of Colorado Springs Boundary	699	252	310	252
	E1	DSNPT_E1_2	Upstream of Confluence with JCC (Peaceful Valley Rd)	1,087	308	698	308
Upper Franceville	F1	DSNPT_F1_11	S Franceville Coal Mine Rd	182	73	73	73
	F1	DSNPT_F1_10	D/S of S Franceville Coal Mine Rd	207	76	76	76
	F1	DSNPT_F1_9	Confluence of Upper Franceville and Tributary	349	134	217	134
	F1	DSNPT_F1_6	Near Mocking Bird Ln	402	106	271	106

On the East Fork, the 100-year flood peak is decreased from 486 to 97 cfs immediately downstream of the proposed detention basin on East Fork. The reduction in the 100-year flow rate (a decrease of 389 cfs) is assumed constant from the detention basin to the confluence with Jimmy Camp Creek.

Downstream of the confluence with Jimmy Camp Creek, the effect of attenuation within East Fork is considered insignificant because the 100-year flow in Jimmy Camp Creek is approximately 8 times the 100-year flow in East Fork. The reduction in velocity due to the detention is shown in Figure 5-11 for East Fork. The reduction in velocity is substantial and the velocities are reduced to below 5 ft/s for all of East Fork reach upstream of the tributary (E1-T1). Downstream of this tributary, the velocities are not reduced below the 5 ft/s threshold. The channel velocity shown in the figure is a moving average of the 5 nearest cross sections. The averaging is done to see the effect of the alternative more easily, and to more accurately understand the effect on reach averaged sediment transport rates.

The detention structures would not significantly affect flow volumes during storms but only temporarily store water during the peak flows, and then release it after the peak has passed. The structures would, however, interrupt the natural sediment processes. Water and the sediment that the water is carrying would be diverted into the detention structures during high flows. Most of the sediment would settle out in these structures and then not be released to the downstream channel. Over time, the detention structure would fill with sediment and the sediment would need to be excavated out to maintain functionality of the structure. In addition, the sediment would not be delivered to the downstream channel and the decrease in sediment supply could increase the rate of erosion of sediment from the channel. If detention alternative is pursued, these impacts and maintenance needs would need to be addressed.

On Upper Franceville, the 100-year flood is reduced from 182 to 73 cfs immediately downstream of the most downstream detention basin on Upper Franceville. This reduction in the 100-year flow rate (109 cfs) is assumed constant from the detention basin until the confluence with Jimmy Camp Creek. The reduction in velocity due to detention on Upper Franceville is shown in Figure 5-12. The magnitude of reduction is less than in East Fork. The velocities are reduced for the majority of the stream below the detention except for the lower end because the effect of detention is less, and because this section of Upper Franceville has been somewhat channelized by development. Below the confluence with Stripmine the detention does not decrease the velocities below the 5 ft/s threshold.

These results do not reflect any hydrologic adjustment for effects of proposed conveyance system improvements such as altered cross section geometry or flatter channel slopes due to the proposed installation of grade control structures to maintain maximum allowable slopes.

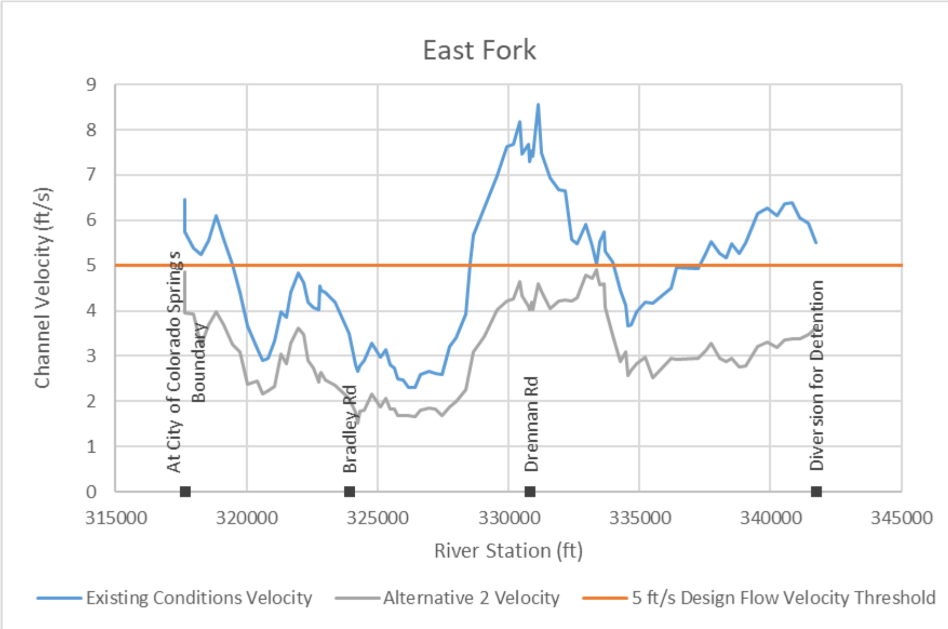


Figure 5-11. Reduction in Velocity in East Fork due to Alternative 2 detention for 100-year flood

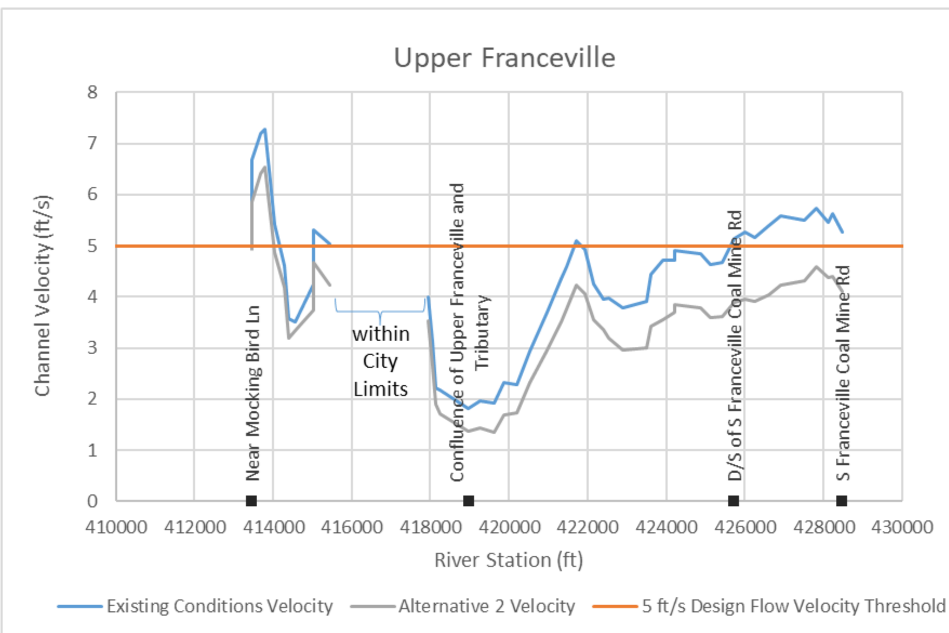


Figure 5-12. Reduction in Velocity in Upper Franceville due to Alternative 2 detention for 100-year flood

5.4.11.2 Alternative 2 Channel Improvements

Channel cross sections, grade control structures and bank stabilization measures are the same for Alternative 2 as they were for Alternative 1 with the exception of the locations where peak flows are reduced on the Upper Franceville and East Fork tributaries. The reduction in peak flows allows for smaller constructed channels and fewer grade control structures. The same templates and typical cross sections would be used, but dimensions would be fit to the Alternative 2 hydrology. The same channel lengths will still need to be improved, but the number of grade control structures in the Upper Franceville and East Fork tributaries will be substantial reduced for the Constructed Natural Channel as shown in Table 5-13. The table shows the number of grade control structures for each drop height required for Alternative 2. It also shows the reduction in the number of grade control structures relative to Alternative 1. Only Upper Franceville and East Fork tributaries are shown in the table because detention is only present in these two tributaries for Alternative 2.

Table 5-13. Number of Grade Control Structures for Alternative 2

Drainageway	Reach ID and Channel Theme	Number of Grade Control Structures for Alternative 2			Reduction in Number and Percent of Grade Control Structures relative to Alternative 1		
		1.5 ft drop	2.5 ft drop	4 ft drop	1.5 ft drop	2.5 ft drop	4 ft drop
Upper Franceville	UF2d Constructed Channel	0	0	0	0 (0%)	4 (100%)	24 (100%)
	UF2d Constructed Natural Channel	0	0	46	0 (0%)	0 (0%)	14 (23%)
East Fork	E2b Constructed Channel	0	0	0	25 (100%)	9 (100%)	2 (100%)
	E2b Constructed Natural Channel	0	28	59	21 (100%)	6 (18%)	7 (11%)

5.4.11.3 Alternative 2 Regional Detention Improvements

Alternative 2 includes detention on the Upper Franceville and East Fork Tributaries. The approximate requirements for these detention facilities are provided in Table 5-14. The assumed locations of the facilities are shown in Figure 5-8.

Regional detention Scenarios 3 and 4 were combined for the purposes of analyzing Alternative 2 (see Table 5-9). This combination of detention Scenarios includes all identified potential detention locations and will provide the maximum benefit of detention.

In addition to construction of the detention basins, the following facilities will need to be constructed to divert and convey water to and from the detention basins as they are located off-stream:

- Weir at the entrance to the canal leading to the basin
- Canals with sufficient capacity leading to and from detention facility
- Spillway for volumes that exceed basin and outlet capacity

Table 5-14. Assumed Detention Size Requirements for Alternative 2

Scenario	Upper Franceville Tributary				East Fork Tributary	
	Location 1		Location 2		Location 1	
	Volume Requirement	Grading Area Requirement	Volume Requirement	Grading Area Requirement	Volume Requirement	Grading Area Requirement
Scenario 3	10.5 ac-ft	2.3 – 3.0 acres	13.5 ac-ft	3.0 – 3.5 acres	-	-
Scenario 4	-	-	-	-	80 ac-ft	13.8 – 19.5 acres
Note: Volume and Grading Area Requirements are approximate						

5.4.11.4 Alternative 2 Storm Sewer Improvements

As stated in Alternative 1 Section 5.4.10.5, storm sewer improvements are not recommended as part of this project.

5.4.12 Alternative Costs

Alternative costs were developed using the Mile High Flood District UD-MP Cost Estimator tool Version 2.2. The unit costs (Table 5-15) used in the tool were developed based on numerous sources, including the Urban Drainage and Flood Control District (now MHFD) Bid Tabs Program, the Colorado of Transportation's Cost Data Book, bid tab data from the City and County of Denver, and the City and County of Denver's Storm Drainage Master Plan. The tool uses unit costs from 2012 as a baseline and escalates costs using the Colorado Construction Cost (CCI) Index. The CCI Index for Calendar Year 2022 4th Quarter was used for the inflation index. This results in costs that are 190% of calendar year 2012. The soil excavation unit costs were modified using recommendations from EPC, which were based upon a review of recent channel improvement projects. The low-range unit cost for excavation recommended by EPC was $9/cubic yard, versus the $21/cubic yard as determined by the MHFD tool.

The costs by reach for Alternative 1 and 2 are provided in Table 5-16 and Table 5-17, respectively. The difference in costs between Alternative 1 and 2 is provided in Table 5-18. Note that the only reaches where Alternative 1 and 2 differ are Reaches UF2 and E2 (where detention facilities are present in Alternative 2). The total cost of Alternative 1 is $154,900,000 and the total cost of Alternative 2 is $166,600,000 (rounded to the nearest $100,000), thus the total cost of Alternative 2 is $11,700,000 more than the total costs of Alternative 1. Alternative 2 does substantially reduce the cost of grade control within the reaches with detention, but the costs of constructing the detention facilities is greater than the reduction in grade control costs. Considering the acreage of the Jimmy Camp Creek drainage basin within El Paso County is 14,018 acres results in a per unit acre cost of $11,100/acre for Alternative 1 and $11,900/acre for Alternative 2. The detailed cost estimates by reach are included in Appendix G.

The costs were compared to the 2015 DBPS. The 2015 costs included the costs for major drainageway improvements and major sub-tributaries within the City of Colorado Springs. The 2015 DBPS did not estimate local sewer improvements, roadway crossings, or utility relocation, which is consistent with the estimates included in this DBPS, except that roadway crossings are included in this DBPS. The costs for the 5 improved roadway crossings shown in Table 5-11 are negligible in comparison to the drainageway improvements. The unplatted acreage of land within the City of Colorado Springs that was considered developable was estimated to be 13,489 acres in the 2015 DBPS. The Capital Cost per acre from the study was $6,519. It is not clear how price escalation was determined for the study, or if it was considered. For example, the 2015 study assumed unit storage costs of $23,762 and $24,353 per acre-foot for regional and sub-regional detention, respectively. These costs were based upon construction costs from previous costs of detention basins in the area, but there was no escalation of costs to the current year. For comparison, the unit cost from the MHFD cost estimator was $86,749 per acre-ft escalated to 2023 dollars. This is an increase in costs of 250%. As another example, the 2015 DBPS assumed a unit cost of Type M soil riprap was $60 per cubic yard, which is less than the 2012 cost of $70 per cubic yard in the MHFD cost estimator. With escalation to 2023, $133 per cubic yard was assumed, an increase in costs of 121% relative to the 2015 DBPS. It was estimated that the 2015 DBPS assumed unit costs were an average of 25% less than 2012 costs recommended by the MHFD cost estimator, based upon comparison of riprap and boulder costs.

Because of the large difference in unit cost assumptions, it is difficult to directly compare the cost from the 2015 study to the current study. The 2015 study also assumed 10% engineering and 10% contingency costs. These are significantly less than recommended in the MHFD cost estimator. A summary of the major cost assumption differences between the 2015 DBPS and the current study is shown in Table 5-19.

Table 5-15. Summary of Assumed Unit Costs

Item	Unit	Adjusted Unit Cost
		2022 Q4
Grouted Boulders, 36-inch	C.Y.	$361
Soil Riprap, Type M	C.Y.	$133
Excavation, Complete-in-Place	C.Y.	$21
Bedding, Granular Type II	C.Y.	$110
Grout	C.Y.	$457
Check Structure, Concrete	L.F.	$514
6-inch Riprap, Type VL	C.Y.	$86
9-inch Riprap, Type L	C.Y.	$105
12-inch Riprap, Type M	C.Y.	$114
18-inch Riprap, Type H	C.Y.	$152
24-inch Riprap, Type VH	C.Y.	$162
Soil Riprap, Type VL	C.Y.	$95
Soil Riprap, Type L	C.Y.	$114
Soil Riprap, Type M	C.Y.	$133
Soil Riprap, Type H	C.Y.	$152
Soil Riprap, Type VH	C.Y.	$171
Excavation, Low Range	C.Y.	$9*
Excavation, Mid Range	C.Y.	$21*
Excavation, High Range	C.Y.	$40*
Detention (Complete-in-Place)	AC-FT	$86,749
Reclamation & seeding (native grasses)	ACRE	$1,902
Concrete	C.Y.	$1,141
Steel	LB.	$2.00
Note: All unit costs are from the MHFD calculator except for: * Modified based upon input from EPC		

Table 5-16. Summary of Alternative 1 costs.

Drainageway	Reach ID	Channel Theme	Improved Reach Length in Cost Estimate [ft]	Channel Improvements [$]	Grade Control [$]	Detention [$]	Other Costs * [$]	Sub-Total [$]	Cost/LF [$]
Jimmy Camp Creek	J6b, J6d	natural	6,147	2,170,698	5,096,431	0	738,011	8,005,140	1,823
Blaney	B1b	natural	8,783	3,069,315	6,110,077	0	930,702	10,110,094	1,612
Corral	C1b	natural	3,679	1,333,374	2,772,044	0	419,957	4,525,375	1,722
	C2d	natural	1,528	542,112	925,603	0	149,909	1,617,624	1,482
Lower Franceville	LF1b	natural	2,191	764,268	791,937	0	158,549	1,714,754	1,096
Upper Franceville	UF2d	constructed	8,336	3,448,098	2,225,267	0	608,135	6,281,500	1,055
		natural	8,093	2,813,997	4,836,535	0	836,597	8,487,129	1,468
Stripmine	S1, S2, S3	natural	20,132	7,110,684	11,921,757	0	1,940,275	20,972,716	1,458
Stripmine South Tributary	S1-T1	natural	6,417	2,245,089	3,357,810	0	569,913	6,172,812	1,347
Stripmine North Tributary	S2-T1	natural	3,921	1,366,293	2,317,703	0	373,631	4,057,627	1,449
East Fork	E2b	constructed	11,226	4,838,637	3,322,154	0	876,752	9,037,543	1,127
		natural	25,089	8,858,466	12,234,116	0	2,535,776	23,628,358	1,318
East Fork Tributary	E1-T1b	constructed	7,698	3,198,357	1,894,115	0	826,577	5,919,049	1,076
West Fork Tributary	W1	maintenance only	0	0	0	0	130,570	130,570	---
Capital Costs							--->	110,660,291	---
Engineering							15%	16,599,044	---
Contingency							25%	27,665,073	---
Total								154,924,407	1,368

* Other costs include roadway crossings, revegetation, mobilization @ 5%, and stormwater management/erosion control @ 5%
Reaches J1, J2, J3, and E1 have already been improved or do not need improvements and are not included in cost estimate
Reach W1 only has roadway crossing improvements

Table 5-17. Summary of Alternative 2 costs.

Drainageway	Reach ID	Channel Theme	Improved Reach Length in Cost Estimate [ft]	Channel Improvements [$]	Grade Control [$]	Detention [$]	Other Costs * [$]	Sub-Total [$]	Cost/LF [$]
Jimmy Camp Creek	J6b, J6d	natural	6,147	2,170,698	5,096,431	0	738,011	8,005,140	1,823
Blaney	B1b	natural	8,783	3,069,315	6,110,077	0	930,702	10,110,094	1,612
Corral	C1b	natural	3,679	1,333,374	2,772,044	0	419,957	4,525,375	1,722
	C2d	natural	1,528	542,112	925,603	0	149,909	1,617,624	1,482
Lower Franceville	LF1b	natural	2,191	764,268	791,937	0	158,549	1,714,754	1,096
Upper Franceville	UF2d	constructed	8,336	3,448,098	2,225,267	0	608,135	6,281,500	1,055
		natural	8,093	2,813,997	3,708,010	4,531,976	1,176,941	12,230,924	2,116
Stripmine	S1, S2, S3	natural	20,132	7,110,684	11,921,757	0	1,940,275	20,972,716	1,458
Stripmine South Tributary	S1-T1	natural	6,417	2,245,089	3,357,810	0	569,913	6,172,812	1,347
Stripmine North Tributary	S2-T1	natural	3,921	1,366,293	2,317,703	0	373,631	4,057,627	1,449
East Fork	E2b	constructed	11,226	4,838,637	3,322,154	0	876,752	9,037,543	1,127
		natural	25,089	8,858,466	9,024,286	7,414,920	2,956,286	28,253,958	1,577
East Fork Tributary	E1-T1b	constructed	7,698	3,198,357	1,894,115	0	826,577	5,919,049	1,076
West Fork Tributary	W1	maintenance only	0	0	0	0	130,570	130,570	---
Capital Costs							--->	119,029,686	---
Engineering							15%	17,854,453	---
Contingency							25%	29,757,422	---
Total								166,641,560	1,472

* Other costs include roadway crossings, revegetation, mobilization @ 5%, and stormwater management/erosion control @ 5%
Reaches J1, J2, J3, and E1 have already been improved or do not need improvements and are not included in cost estimate
Reach W1 only has roadway crossing improvements

Table 5-18. Cost of Alternative 1 subtracted from Alternative 2 for Reaches UF2 and E2. All other reaches have identical costs.

Reach	Channel Improvements [$]	Grade Control [$]	Detention [$]	Other Costs * [$]	Sub-Total [$]
UF2	0	-1,128,525	4,531,976	340,344	3,743,795
E2	0	-3,209,830	7,414,920	420,510	4,625,600
Capital Costs					8,369,395
Engineering @ 15%					1,255,409
Contingency @ 25%					2,092,349
Total					11,717,153
* Other costs include roadway crossings, revegetation, mobilization @ 5%, stormwater management/erosion control @ 5%					

Table 5-19. Summary of Escalation and Non-Contract Costs for Current Study and 2015 DBPS.

Item	Current study	2015 DBPS
Escalation relative to MHFD 2012 unit costs	90%	-25%
Engineering	15%	10%
Contingency	25%	10%
Total Percentage Increase from 2012 Construction Costs	**130%**	**-5%**

5.5 Evaluation and Selection of Alternatives

This section evaluates the alternatives in terms of their ability to meet the project goals as defined in Section 4.3. The two DBPS alternatives and the No Action alternative were scored using the evaluation criteria described previously. All alternatives, including the No Action alternative, are based on the watershed hydrology modeling performed for the DBPS. All alternatives assume existing conditions peak flows as discussed in Section 5.4.10.1, except where detention is proposed on Upper Franceville and East Fork for Alternative 2.

Alternatives were evaluated using a semi-quantitative approach whereby each alternative was given a score from 1 to 5, with 5 being best, for each evaluation criterion based on the combined knowledge and experience of the Stantec and EPC. The breakdown of the score is as follows: 1 (worst), 2 (bad), 3 (average), 4 (good), and 5 (best). Results are shown in Table 5-20. Because Alternative 1 and 2 differ only in the magnitude of improvements needed in the Upper Franceville and East Fork mainstem downstream of the detention basins, their scores are very similar. Both alternatives have superior evaluations compared to the No Action alternative.

The primary difference between Alternative 1 and 2 is capital cost and maintenance. Alternative 1 is approximately $11,700,000 less costly than Alternative 2 and the average cost per linear foot of channel is reduced by $104. The maintenance activities for Alternative 2 would be significantly more than Alternative 1 as the detention facilities and the diversion canals used to operate the detention facilities will require significant maintenance.

Alternative 1 is recommended based on review of the alternative evaluation matrix, input from stakeholders, and the goals for sediment management in the Jimmy Camp Creek drainage basin and the overall Fountain Creek watershed. Alternative 1 will be used to develop the drainage basin fees and proposed costs for improvements. However, Alternative 2 and other detention alternatives are potentially feasible alternatives and could be pursued when the final designs of the reach are performed.

Table 5-20. Evaluation of DBPS Alternatives

Evaluation Criterion	Alternative Score (1 to 5), higher is better			Comments
	No Action	Alternative 1 (Channel Improvements)	Alternative 2 (Detention and Channel Improvements)	
Channel and Floodplain Goals				
Remove insurable structures from 100-year floodplain	3	3	3	No Action: based on DBPS 100-year existing condition flows. There are few structures currently in 100-year floodplain. Alternative 1: peaks flows based on DBPS 100-year existing condition flows. Alternative 2: peaks flows based on DBPS 100-year existing condition flows. Except for Upper Franceville and East Fork DBPS itself does not change regulated FEMA floodplain
Improve channel stability	1	5	5	No Action: Development will increase flow volumes and potentially reduce sediment delivery to streams. Alternative 1 and 2: maintain stable slope to reduce bed and bank erosion.
Reduce impact upon major thoroughfares and utilities, existing and future, by improving channel and bridge/culvert capacity	1	5	5	No Action: No Improvements to hydraulic structures. Alternative 1 and 2: Alternatives improve hydraulic structures.
Environmental Goals				
Approximate naturally functioning system	2	4	4	No Action: erosion in unstable upper and middle basin segments will create incised or overly wide channels. Alternatives: preserve stable grade and semi-active controlled overbank area.
Improve environmental resources	1	4	4	No Action: erosion in unstable upper and middle basin segments will create incised channels with limited ecological value. Alternatives prevent further environmental degradation. Environmental benefits are associated with improved channel stability and sediment control.
Improve Fountain Creek water quality	1	4	4	No Action: more sediment produced from channels after development. Alternatives: less upstream sediment production from channels.
Minimize regulatory issues	4	3	1	No Action does not require permitting for channel modification or detention basins. Alternative 1 requires permitting for channel modification. Alternative 2 requires permitting for channel modification and detention basins.
Multiple Benefit Goals				
Reduce peak flows to pre-development conditions	4	4	5	All Alternatives: The County DCM requires new developments to install extended detention basins or other permanent control measures to maintain post-development runoff rates at pre-development conditions and to mitigate impacts of land development on receiving water quality. Alternative 2 would further reduce flows in Upper Franceville and East Fork.
Provide open space and trails opportunities	1	3	2	No Action: No proposed channel improvements and lack of improvements could limit access to drainage paths. Alternative 1: Channel alternatives in upper undeveloped areas could accommodate open space and trails. Alternative 2: The space required for detention facilities and their diversion canals could reduce open space relative to Alternative 1.

Table 5-20. Evaluation of DBPS Alternatives (continued)

Evaluation Criterion	Alternative Score (1 to 5), higher is better			Comments
	No Action	Alternative 1 (Channel Improvements)	Alternative 2 (Detention and Channel Improvements)	
Cost Goals				
Minimize construction cost	5	3	1	No action: No channel construction, grade control or crossings. Alternative 1 proposes channel improvements throughout basin. Alternative 2 proposes channel improvements throughout the basin and also proposes construction of off-line detention facilities. The detention facilities reduce costs of some channel improvements, but the cost of the detention is significantly greater than the reduction in costs of channel improvements.
Minimize maintenance cost	1	4	2	No Action: substantial annual maintenance, increasing as upper basin develops. Includes reconstruction of failing facilities. Alternative 1 will reduce maintenance compared to No Action, though channels will still require some maintenance. Alternative 2 will reduce maintenance compared to No Action, though channels will still require some maintenance. The detention facilities will likely require maintenance activities at diversions, diversion canals, and within the basins themselves.
Total Score	**24**	**42**	**36**	

6.0 CONCEPTUAL DESIGN OF SELECTED PLAN

The purpose of this section is to describe the basis of estimating capital costs and the drainage fee per acre for the recommended DBPS stormwater improvements.

6.1 Summary of the DBPS process

The objective of this DBPS was to analyze the existing and future drainage conditions of the watershed in the unincorporated County, identify corrective and future capacity improvements, and to establish drainage and bridge fees for future development. This study includes a description of the study process, basin background information, technical analysis and documentation, and the proposed plan. The information developed from this study, upon adoption by the County, will be used to mitigate stormwater impacts to the major drainageways within the watershed.

This DBPS is a comprehensive update of the unincorporated County area portions of the Jimmy Camp Creek DBPS published in 2015 (Kiowa Eng). The 2015 study is based on the drainage basin planning criteria in the Colorado Springs Drainage Criteria Manual (COS, 2014).

6.2 Criteria

The following manuals were used in the development of the hydrologic and hydraulic analysis as well as in the development of the conceptual design and plans for the major drainageways within the Basin. The principle manuals were:

- COS. (2021). *Drainage Criteria Manual.* Colorado Springs: City of Colorado Springs.
- COS. (2022). *Green Infrastructure Guidance Manual.* City of Colorado Springs Stormwater Enterprise
- EPC. (1991). *Drainage Criterial Manual.* Colorado Springs: El Paso County
- Mile High Flood District (MHFD) (Formerly Urban Drainage and Flood Control District [UDFCD]). (January 2016). *Urban Storm Drainage Criteria Manual: Volume 1 Management, Hydrology, and Hydraulics*. Urban Drainage and Flood Control District. Denver, Colorado. www.udfcd.org

6.3 Hydrology

Development of hydrologic flows for the Jimmy Camp DBPS were performed using the Environmental Protection Agency's (EPA's) Storm Water Management Model (SWMM) 5 (version 5.1.015). Model development, scenario management, and model execution was performed using the Innovyze InfoSWMM software (version 14.7, Update #6).

The purpose of the Jimmy Camp Creek DBPS hydrologic analysis is to develop peak flows for planning and design based on current and future conditions in the basin. The results of the hydrologic analysis feed into the hydraulic analysis portions of this DBPS. As such, peak flows were developed for key design points along the Jimmy Camp Creek main stem and the tributary channels within the Jimmy Camp Creek Drainage Basin. Hydraulic routing was also included in the hydrologic analysis to determine peak flows at key points in the Jimmy Camp Creek Drainage Basin for use in the hydraulic analysis.

The hydrologic results presented in the DBPS are not intended to be a replacement for the FEMA regulatory floodplain and do not supersede it. The peak flows developed for the DBPS do not include the 1965 flood in the analyses, which was an event that was more than 20 times larger than next highest recorded peak flow. The FEMA analysis generally had 100-yr peak flows that were significantly higher than the 100-yr peak flows estimated in this DBPS. As a result, the FEMA regulatory floodplain generally covers a significantly greater area than the existing floodplain shown in the DBPS.

The EPC DCM requires new developments to install extended detention basins or other permanent control measures to maintain post-development runoff rates at pre-development conditions and to mitigate impacts of land development on the water quality of receiving waters. Therefore, the peak flows from the existing conditions hydrology were used to develop DPBS alternatives. Development is assumed to increase flow volumes and it was assumed that the volumes of runoff would be similar to the future condition hydrology. The DPBS assumes that Green Infrastructure (as described in the Colorado Springs Green Infrastructure Guidance Manual; COS 2022) is not incorporated into the developed parcels, however, Green Infrastructure should be considered as it can significantly reduce runoff volumes and reduce the need for downstream channel stabilization.

6.4 Hydraulics and Channel Assessment

The purpose of the hydraulic analysis for the Jimmy Camp Creek DBPS was to identify existing and future deficiencies in major drainageways and large storm sewer trunk lines within the Jimmy Camp Creek Drainage Basin. The hydraulic analysis aimed to document existing hydraulic deficiencies to identify the need for future feasible stormwater and flood control solutions.

The hydraulic analysis also identified locations where the existing conditions 100-year floodplain differs significantly from the effective FEMA floodplain shown on the Flood Insurance Rate Maps (FIRMs). Comparison of the existing conditions 100-year floodplain with the regulatory FEMA floodplain shows that in general, the existing conditions floodplain is smaller than the regulatory FEMA floodplain. This is primarily because the flow rates from the SWMM hydrologic model are lower than the flow rates shown in the FIS, as discussed in the Hydrology section. Another reason for differences between the existing conditions 100-year floodplain and the regulatory FEMA floodplain is the updated topographic mapping used for this study. LiDAR-based DEM data prepared in 2018 was used for this study.

6.5 Channel Stabilization Measures

The channel design represents comprehensive solutions to current and future flooding and channel stability issues in the unincorporated areas of the Jimmy Camp Creek drainage basin. The measures consist of a collection of individual options for specific locations that provide a consistent approach to drainage throughout the Basin in El Paso County.

Due to more runoff and lower sediment yields, long-term stable low-flow channel slopes are expected to be significantly flatter than existing channel slopes. To achieve the desired stable condition, grade control structures are proposed to mitigate steeper channel sections and stabilize the stream reach. The proposed channels are assumed to keep the existing alignments.

In final design, grade control structures may consist of void filled riprap in a natural configuration or grouted boulders with different heights based on the local features. It is not intended that the final design match the sizes and spacing shown in the alternatives. The final design should provide adequate channel stabilization while incorporating aesthetic design characteristics. The grade control structures may also need to be modified to satisfy fish passage criteria if applicable.

Any modification to wetlands/Waters of the State could require permitting and mitigation. The recently implemented Colorado Mitigation Procedures (COMP), Colorado Stream Quantification Tool (CSQT) and mitigation banking are used by the U.S. Army Corps of Engineers Regulatory Office to analyze permit applications under Section 404 of the Clean Water Act. The COMP, developed by Colorado regulatory offices, provides regulatory specialists with a framework to objectively evaluate a wetland or a stream's functional condition by providing a measurable and repeatable method of calculating debits and credits for wetland and waterway impacts caused by permitted activities. These procedures utilize the CSQT, also developed by the Colorado regulatory offices in partnership with the U.S. Environmental Protection Agency, to evaluate a stream's hydrology, hydraulics, geomorphology, chemistry, and biology. The tool uses a combination of metrics based on watershed data as well as common survey and field measurements, such as width-depth ratios and bank erosion.

Specific bank erosion control measures were not recommended as part of this DBPS other than those that stabilize the low flow channel in using the two themes for the channel 1) Constructed Channels and 2) Constructed Natural Channels. There are some banks that are currently eroding, but there did not appear to be any infrastructure at risk and there were no significant bank protection measures suggested in the tributaries included in this study. A potential strategy that can be considered is to allow the bank erosion to continue, rather than build relatively expensive bank protection measures that require future maintenance. We suggest that development can be placed sufficiently far from eroding banks so that it will not be at risk. Large bank stabilization measures should not be required other than those provided by the Constructed and Constructed Natural Channels. However, in addition to understanding the floodplain extent, we suggest that the fluvial hazard zone be mapped as part of the development process. The intention of the fluvial hazard zone mapping is to identify future extent of deposition and erosion hazards in the stream corridor. Guidance on the Colorado Hazard Mapping Program (CHAMP) is available from the Colorado Water Conservation Board.

6.6 Floodplain preservation

The Constructed Natural Channel theme incorporates low flow stabilization and full floodplain preservation to provide natural channel functions. The Constructed Channel has a stabilized low flow and overbank floodplain terraces. However, the Constructed Channel is entirely graded in a general trapezoidal shape with limited preservation of the existing natural floodplain. The majority of the channels in this DBPS are Constructed Natural Channel and therefore intended to incorporate floodplain preservation. Maintaining existing floodplains will reduce future flood risks, reduce concentration of flood flows thereby reducing erosion risks, reduce long term maintenance costs of stabilization measures, and increase ecological benefits.

6.7 Description of Selected Alternative

The evaluation criteria adopted for the Jimmy Camp Creek DBPS were based on goals to define different aspects of project success. These goals were organized in four categories: Channel and Floodplain Goals, Environmental Goals, Multiple Benefit Goals, and Cost Goals.

Two Alternatives were developed and analyzed as part of this DBPS. The first alternative (Alternative 1) did not include regional detention and the second alternative (Alternative 2) did include regional detention. For Alternative 2, the regional detention was placed locations that were undeveloped and located close to the stream channels. Both alternatives assumed that development does not increase peak flows relative to existing conditions. However, Alternative 2 would decrease peak flows downstream of the detention facilities due to the storage volumes present.

Both alternatives included channel improvement as described previously, but because of detention, the requirements for channel improvements were less for Alternative 2 than 1. However, the cost savings in channel improvement for Alternative 2 were less than the cost of the detention facilities.

The two Alternatives were compared against the No Action Alternative, which would assume no channel improvements. They were compared using the evaluation criteria through a semi-quantitative process where ratings were assigned to each criterion. Alternative 1 is recommended based on review of the alternative evaluation matrix, input from stakeholders, and the goals for sediment management in the Jimmy Camp Creek drainage basin and the overall Fountain Creek watershed. Alternative 1 will be used to develop the drainage basin fees. The plan and profile of the proposed conceptual design is given in Appendix H: Conceptual Design.

6.8 Water Quality

Factors that will affect stormwater quality in the Jimmy Camp Creek Drainage Basin drainageways include urbanization and sedimentation/erosion. El Paso County addresses both factors through development requirements for application of Permanent Control Measures (PCMs) (e.g., onsite stormwater quality ponds, stormwater extended detention, etc.), education and outreach, system maintenance, and other programs. The EPC DCM requires new developments to install extended detention basins or other permanent control measures to maintain post-development runoff rates at pre-development conditions and to mitigate impacts of land development on receiving water quality.

For the analyses herein, the DPBS assumes that Green Infrastructure (as described in the Colorado Springs Green Infrastructure Guidance Manual; COS 2022) is not incorporated into the developed parcels, however, Green Infrastructure should be considered as it can significantly reduce runoff volumes, reduce the need for channel stabilization, and improve water quality.

6.9 Trails

Multi-use trails can be incorporated into the drainageway design to provide multiple benefits. The trails can be used for recreational use, provide linkages to a broader regional network of trails, parks and open spaces, and provide access for channel maintenance. They should be located outside of low flow channel within the maintenance access and should be located to minimize the impact to riparian vegetation along the low flow channel that acts to stabilize the channel banks. The specific design and construction of these trails is not included in the conceptual design or in the cost estimates in this DBPS.

The layout of a trail along a drainageway should account for hydraulic considerations, utilities in the area, and access to dedicated parks and roadway crossings. The trail may need to have asphalt or concrete surfacing if water velocities during floods are expected to significant or if the trail is expected to be inundated frequently.

6.10 Maintenance

The EPC DCM requires new developments to install extended detention basins or other permanent control measures to maintain post-development runoff rates at pre-development conditions and to mitigate impacts of land development on receiving water quality and peak flows. The designs and channel improvements in this DBPS assumes these facilities are built and maintained indefinitely into the future. Details on the EPC drainage facilities maintenance policy are included in EPC Drainage Criteria Manual (EPC, 1991).

The maintenance of grade control structures is expected to be minimal, but inspection and maintenance of at select grade control structures is possible after high flow events. The grade control design should be such that slight movement of boulders does not destabilize the structure, but if excessive scour or bank erosion causes substantial damage to a structure, then that structure would need to be repaired.

Vegetation maintenance may be required if a Constructed Channel drainageway becomes excessively overgrown but given the arid nature of this area and relatively poor soils, significant clearing of vegetation is unlikely to be needed. In general, vegetation establishment should be encouraged as this will be the most efficient method to maintain bank stability and prevent erosion of floodplain soils. In reaches where floodplain preservation is recommended, no clearing of native vegetation is recommended.

Annual clearing of trash and debris at roadway crossings is recommended to ensure the design capacity of the crossing, and to enhance the crossings for trail users if a trail exists.

Maintenance activities are not included in the cost estimate in this DBPS.

6.11 Right of Way

The main channels within the watershed that pass through the developed portions of the basin should be contained within dedicated drainage tracts, easements, or rights-of-way. For those segments of the drainageway where floodplain preservation is the recommended plan, a combination of open space dedication (such as parklands and greenbelts), in combination with a narrower dedicated right-of-way along the low flow area of the drainageway should be obtained through the land development process.

Right-of-way acquisitions are not included in the cost estimate in this DBPS.

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APPENDIX A
BANK EROSION HAZARD INDEX (BEHI) VARIABLES AND AN OVERALL BEHI RATING

APPENDIX B
SUMMARY FORM OF ANNUAL STREAMBANK EROSION ESTIMATES

APPENDIX C

JIMMY CAMP CREEK DBPS

EXISTING CONDITION SUB-BASIN MODEL

APPENDIX D

EL PASO COUNTY

RAINFALL DISTRIBUTION TECHNICAL MEMO

APPENDIX E

JIMMY CAMP CREEK DBPS

FUTURE CONDITION SUB-BASIN MODEL

APPENDIX F
HYDRAULIC RESULTS

APPENDIX G

DETAILED COST ESTIMATES BY REACH

APPENDIX H
CONCEPTUAL DESIGN