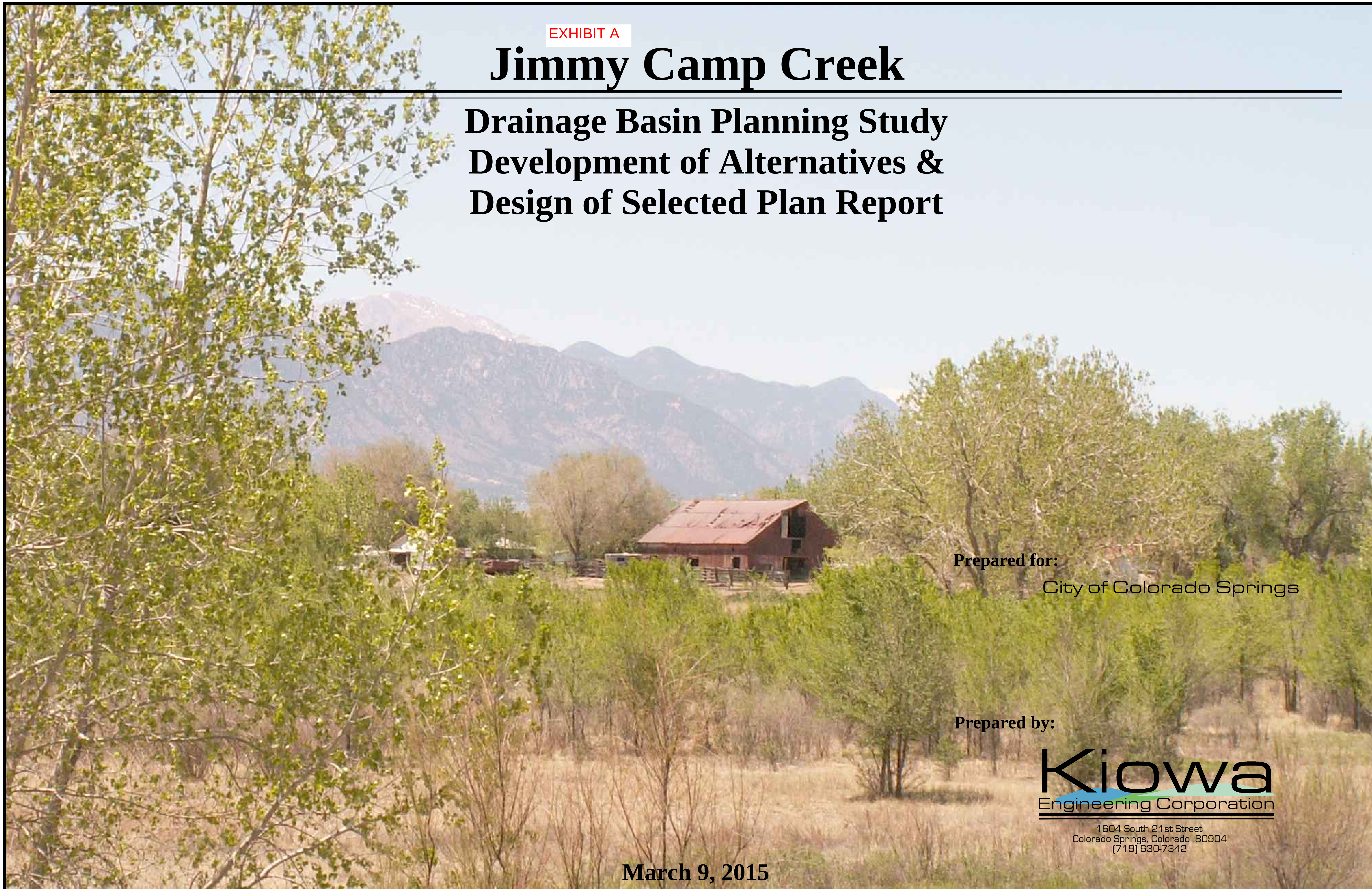
EXHIBIT A

Jimmy Camp Creek

Drainage Basin Planning Study
Development of Alternatives &
Design of Selected Plan Report



Prepared for:

City of Colorado Springs

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March 9, 2015

DEVELOPMENT OF ALTERNATIVES AND DESIGN OF SELECTED PLAN REPORT

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I. INTRODUCTION

1.1 Authorization

The Jimmy Camp Creek Drainage Basin Planning Study [DBPS] was authorized by the City of Colorado Springs under the terms of agreement between the City of Colorado Springs and Kiowa Engineering Corporation. Due to the extensive regional implications of this study, input and review to the technical scope of this project was provided by the City of Fountain and El Paso County. The area subject to study is presented on Figure I-1.

1.2 Purpose and Scope

The purpose of the study is to analyze the existing and future drainage conditions of the watershed, quantify surface runoff, define floodplains, identify drainage impacts, develop alternate solutions, and prepare a final drainage plan for implementation within the watershed. The information developed from this study will be used to regulate future development and mitigate the major drainageways within the watershed.

Specific tasks required for the study:

1. Meet with the Client and co-sponsors to obtain information, present study findings, and gain direction for future analyses.
2. Contact agencies and/or individuals that have knowledge or specific interest in the study area.
3. Inventory and compile the existing drainage system.
4. Apply the latest City/County policies and criteria.
5. Perform hydraulic and hydrologic analyses.
6. Identify existing and potential drainage and/or flooding problems.
7. Mitigate impacts on Fountain Creek.
8. Develop improvement alternatives to reduce existing and potential flooding problems, and mitigate the impact of stormwater runoff on environmentally significant areas.
9. Recommend and prepare a conceptual design for a selected alternative plan.
10. Prepare written reports for submittal to the City of Colorado Springs.
11. Apply the City Zoning Code Streamside Overlay and the Streamside Design Guidelines policies, standards and criteria, as appropriate and applicable for a drainage basin planning study.

1.3 Mapping

Project mapping for hydrologic analyses was obtained through USGS digital 7.5-minute quadrangles (20-foot contours) and supplemented with Colorado Springs Utilities FIMS mapping (2-foot contour) along the Jimmy Camp Creek channel. Additional 2-foot contour mapping was utilized for the properties of Banning Lewis Ranch and Rolling Hills Ranch. The specific quadrangles used for the study area are Falcon NW, Falcon, Elsmere, Corral Bluffs, Fountain, and Fountain NE. Revisions to the mapping vary across the quadrangles. In general, the mapping is compiled from aerial photographs taken in 1947, field checked in 1948, revised from aerial photographs taken in 1960, and field checked in 1961, and revised from aerial photographs taken in 1969 and 1975, but not field checked. Some mapping has been further revised from aerial photographs taken in 1988, field checked in 1993, and edited and published in 1994. All USGS mapping is prepared at a contour interval of twenty-feet. The horizontal control is NAD 1927 with projection zone 13 Colorado Coordinate System central and north zones. The vertical datum is 1929. In addition to the contour information, the USGS mapping provides roadway alignments and major drainage paths. This mapping was deemed suitable for the hydrologic analyses portion of this study. The USGS mapping was supplemented with 2-foot contour mapping where available. Two-foot contour mapping was used to verify watershed boundaries and evaluate drainage paths in areas where the 20-foot contours were inadequate.

1.4 Data Collection

The following maps, plans and reports were reviewed during the course of preparing this report:

Banning Lewis Ranch Master Plan, current as of August 23, 2006.

City of Colorado Springs and El Paso County Drainage Criteria Manual, prepared by HDR Infrastructure, Inc., October 1987.

City of Colorado Springs Draft Drainage Criteria Manual, Volumes I and II, prepared by Matrix Design Group, Inc., March 2014.

City of Colorado Springs Comprehensive Plan, prepared by the City of Colorado Springs, March 27, 2001.

City of Fountain Comprehensive Development Plan (Update), prepared by the City of Fountain, August 2005.

Flood Hazard Analyses, prepared by U.S. Department of Agriculture Soil Conservation Service, October 1975.

Fountain Creek Watershed Plan – Hydrology Study Final Report, prepared by URS Corporation, March 2006.

Hydrologic Modeling System HEC-HMS v. 3.0.0, prepared by US Army Corps of Engineers, December 2005.

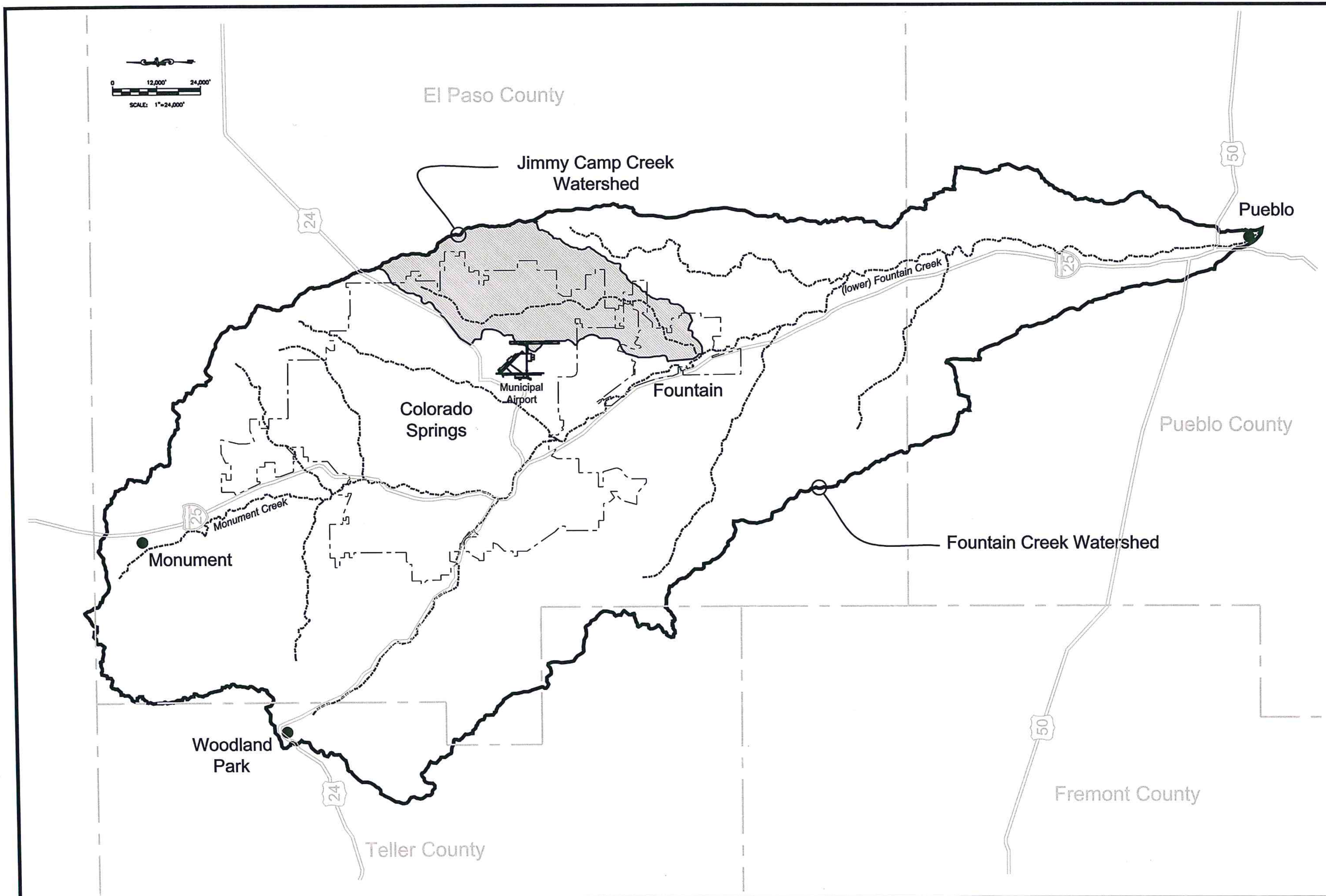
Jimmy Camp Creek Drainage Basin Planning Study, prepared by Wilson and Company, January 1987. [Note: this study was never officially adopted.]

Lorson Ranch Conceptual Development Plan.

NOAA Atlas 2, Precipitation Frequency Atlas of the Western United States, prepared by U.S. Department of Commerce, NOAA, 1973.

Norris Ranch Conceptual Development Plan.

Rolling Hills Ranch Conceptual Development Plan.



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JIMMY CAMP CREEK WATERSHED
DRAINAGE BASIN PLANNING STUDY
VICINITY MAP
CITY OF COLORADO SPRINGS

Project No.: 14008
Date: OCT 2014
Design:
Drawn: BJW
Check:
Revisions:

I-1

Soil Survey of El Paso County Area, Colorado, prepared by USDA Soil Conservation Service, June 1981.

Streamside Design Guidelines, prepared by the City of Colorado Springs.

Streamside Overlay Zone, per City of Colorado Springs City Zoning Code Section 7.3.508.

USGS 7½-minute Quadrangle Maps [Falcon NW, Falcon, Elsmere, Corral Bluffs, Fountain, Fountain NE], 1994.

West Fork Jimmy Camp Creek Drainage Basin Planning Study, prepared by Kiowa Engineering Corporation for New Generation Homes, Inc., October 2003.

Jimmy Camp Creek Drainage Basin Planning Study Hydrology Technical Addendum, prepared by Kiowa Engineering Corporation, February 2008.

Jimmy Camp Creek Drainage Basin Planning Study Storm Precipitation Analysis (SPAS) Storm Rainfall Analysis, Final Report, prepared by Applied Weather Associates, etal., December 2007.

During the preparation and subsequent review of the draft hydrology report associated with this drainage basin planning study, review comments received from the Colorado Springs Department of Utilities raised concerns regarding inconsistencies between the DBPS hydrology and the hydrology prepared as part of the Fountain Creek Watershed Plan. Specifically, peak discharge data estimated for the existing condition 2-year and 5-year recurrence intervals were significantly higher in the DBPS than those discharges estimated in the Fountain Creek Watershed Plan for the Jimmy Camp Creek basin. Because of this variance the hydrologic analysis for the DBPS included a storm rainfall analysis and a stream characterization analysis in an effort to gain more insight as to the actual nature of the high frequency storm events and into the sensitivity that hydrologic characteristics such rainfall distribution, depth and duration have upon stream flow. Based upon the results of the storm rainfall analysis the hydrologic model was calibrated so as to reflect more realistic peak flow results when compared to the stream gauge data available for the USGS gage located on Jimmy Camp Creek at Ohio Avenue. This calibration brought the results for the 2-year and 5-year recurrence intervals into much closer agreement between the DBPS and the Fountain Creek Watershed Study.

1.5 Stakeholder Review

As part of the completion of the technical analyses and the development of alternatives, individuals, major property owners and organizations with an interest in development of the long-term storm water management and major drainageway stabilization measures were contacted and routinely notified regarding their attendance at progress meetings. Six stakeholder meetings were held over a two year period between 2008 and 2010. Comments arising from these meetings were documented and addressed as part of the completion of the DBPS. A partial listing of stakeholders is presented below:

Organization
City of Colorado Springs Engineering Division
City of Colorado Springs Department of Utilities
City of Colorado Springs Planning Department
City of Colorado Springs Parks and Recreation
City of Fountain Department of Public Works
El Paso County Public Services Department
El Paso County Development Services Department
Banning Lewis Ranch Development
Colorado Centre Metropolitan District
Lorson Ranch Development
Nor'\wood Development
Rolling Hills Ranch Development
US Army Corps of Engineers
National Resource Conservation Service
El Paso County Soil Conservancy District

II. STUDY AREA DESCRIPTION

2.1 Introduction

The study area consists of the Jimmy Camp Creek watershed located in El Paso County. Jimmy Camp Creek is an east bank tributary to Fountain Creek with its outfall lying just west of Old Pueblo Road [Main Street] near the City of Fountain's historic downtown. The watershed is generally bounded by Powers Boulevard to the west, Blaney Road to the east, Old Pueblo Road to the south, and Garrett Road to the north. The Jimmy Camp Creek watershed has a drainage area of 67.1 square miles.

The topography of the study area slopes from north to south beginning near elevation 6880 feet at Garrett Road and ending near elevation 5490 feet at the outfall to Fountain Creek. The main channel of Jimmy Camp Creek has an average slope of 1.0% over a length of 24 miles.

There are nine major tributaries defined within the Jimmy Camp Creek watershed: East Fork, Franceville, Strip Mine, Corral, Marksheffel, West Fork, Ohio, C and S, and Blaney Tributaries. All of these tributaries have drainage areas greater than one square mile. The West Fork tributary was recently studied and the results have been published in a report entitled, West Fork Jimmy Camp Creek Drainage Basin Planning Study, dated October 2003. This 4.1 square mile drainage basin was studied in detail, planned, reviewed, and accepted as an approved drainage plan for the basin. The West Fork has a defined plan of drainage improvements, flood detention, and required right-of-way. Drainage basin fees have been determined and are being implemented for the West Fork Tributary. To avoid duplication of effort and complication of drainage fees, the resulting hydrologic analysis from this studied was directly input into the hydrologic models of the Jimmy Camp Creek study. This was accomplished by directly reading in the various hydrographs at the West Fork outfall to Jimmy Camp Creek for all frequencies under existing, future, and master planned (with drainage improvements) conditions.

2.2 Flood History

Throughout recorded history, the Jimmy Camp Basin has always experienced severe weather events with wide fluctuations that include drought, hail, floods and devastating snowstorms. With low population density in the basin prior to the last twenty years, endangerment of lives and damage to property was limited and rarely reported. Flooding mainly occurs in the summer months of May to August during intense rain events of several days duration when a warm, moist air mass from the Gulf of Mexico collides with a colder air mass from the north. Although frequently severe isolated summer thunderstorms rarely cause a major flood as the more frequent storms tend to be limited in area and duration.

The June 18, 1965 flood is the flood of record in El Paso County. As much as 14 inches of rain fell over several days. Hailstones near Fountain were said to be as large as tennis balls. The flow at Jimmy Camp Creek was estimated to be 124,000 cubic feet per second at a point about 4.5 miles upstream from the confluence. Considerable damage to roads and bridges occurred in the sparsely populated area. In the City of Fountain, Ohio Avenue washed out along with the railroad trestle. Santa Fe was overtopped and gullies formed on the approaches.

A large regional flood also occurred on May 30, 1935 after several days of rain. As in the 1965, the majority of damages were to agriculture, roads and bridges. In the summer of 1972, two separate flood events caused damage in the basin. The first event of July 18th, there were reports of two- to five-inches of rain in the Franceville Tributary causing about $100,000 damages to roads and bridges. State Highway 94 was closed due to bridges being washed out. Later in the summer on August 3rd, a flood did an additional $50,000 in damages to bridges and isolated eight families east of Jimmy Camp Creek on Peaceful Valley Road.

The U.S.G.S. installed a stream gage near the mouth of Jimmy Camp Creek in 1976. Review of gage records for water years 1976-2005 indicate peak flows of 4,810 cubic feet per second and 4,530 cubic feet per second for 1994 and 1995 respectively and 3,600 cubic feet per second in 1985. During the 30 years of record, the gage recorded peak flows over 1,000 cubic feet per second during seven years. Flood history clearly indicates that a potential for flash flooding is present in the Jimmy Camp Creek Basin and will increase as urbanization continues.

2.3 Land Use

Hydrologic impervious information for each sub-watershed land use was developed for input into the HEC-HMS hydrology model. The amount of impervious area within each sub-watershed was estimated for two conditions: (1) existing development and (2) anticipated maximum future development. Figures II-1 and II-3 show the various land uses applied to the hydrology models for existing and future conditions, respectively. Table II-1 describes the percent imperviousness for each of the different land use categories.

Existing Land Use

Currently, the watershed is predominantly undeveloped with a land use of pasture or open range. The pockets of existing development found within the study area are a mix of rural residential, single-family residential and commercial. The lower reach of the watershed extends into the City of Fountain where single-family residential, multi-family residential, public facilities and commercial properties are found. Impervious areas for existing conditions were compiled by examining the City's 2005 online aerial photography (2' resolution), project mapping, and by field inspections to the area. Watersheds were delineated into various land-use categories to which the imperviousness values in Table II-1 were applied. An area-weighted percent imperviousness was then computed for each sub-watershed, tributary, and total watershed. The overall watershed imperviousness for the existing condition in Jimmy Camp Creek is 4.5%.

Table II-1
Land Use Index

Category		% Impervious
Undeveloped, Open Space		2 - 5
Parks, Golf Course		5 - 10
Residential Very Low	[<1 du/ac]	10 – 20
Residential Low-Med	[1-8 du/ac]	40 – 50
Residential Med-High	[8-12 du/ac]	50 – 60
Residential High	[12-24 du/ac]	60 – 70
Industrial, Mixed Use		70 – 80
Commercial		80 – 90
Public Facilities		….. specific to each site …..

Table II-2
Existing Land Use

			% Imperviousness						Composite
Region	Area (ac)	Area (sqmi)	2	15	45	55	65	85	% Imp
East Fork Tributary	6,274	9.80	90%	10%	0%	0%	0%	0%	3.5%
Franceville Tributary	2,713	4.24	93%	7%	0%	0%	0%	0%	2.9%
Strip Mine Tributary	3,869	6.05	99%	1%	0%	0%	0%	0%	2.1%
Corral Tributary	5,649	8.83	100%	0%	0%	0%	0%	0%	2.0%
Marksheffel Tributary	3,316	5.18	80%	18%	1%	0%	0%	1%	5.2%
West Fork Tributary	2,647	4.14	88%	6%	2%	4%	0%	0%	5.8%
Ohio Tributary	767	1.20	92%	8%	0%	0%	0%	0%	3.1%
C and S Tributary	1,325	2.07	45%	9%	40%	3%	0%	3%	21.6%
Blaney Tributary	995	1.55	100%	0%	0%	0%	0%	0%	2.0%
Jimmy Camp Main	15,400	24.06	87%	8%	5%	0%	0%	0%	5.1%
	ac	sqmi							
Totals…	42,956	67.1	90%	7%	3%	0%	0%	0%	4.5%

As shown in table II-2, 90% of the watershed remains undeveloped. The primary existing development is a rural, large lot residential [RVL] development. Some low-medium density single-family housing can be found in the lower part of the watershed near the City of Fountain. There are also small areas of multi-family and commercial developments in the watershed within the City of Fountain; however, the total area amounts to less than 1% of the total watershed area. The highest density developments are found within the C and S Tributary and the lower portions of Jimmy Camp Creek. A map of the existing conditions land use is shown in Figure II-1.

Future Land Use

The future impervious cover was estimated by reviewing land use planning studies provided by the City of Colorado Springs, City of Fountain, and El Paso County. As shown in Figure II-2, over 60-percent of the watershed has detailed development planned for five major properties within the drainage basin. These developments are Banning Lewis Ranch (40%), Rolling Hills Ranch (5%), Lorson Ranch (3%), Norris Ranch (3%) and the City of Fountain (10%) 2005 Land Use Update. Each of these developments are in the early stages of development. Lorson Ranch and Banning-Lewis Ranch are at this time actively developing. This level of detailed future development in a watershed study is unusual and provides an exceptionally detailed future conditions land use map. Additional future planning information was obtained in a meeting with the El Paso County Planning Department. This meeting defined setback areas around the Corral Bluffs and lands with known dedicated uses in the watershed. Figure II-3 shows the watershed's future land use projections.

The same process that was used to quantify existing imperviousness was applied for future imperviousness. Values of impervious area were assigned to each projected land use category as described in Tables II-2 and II-3. The overall Jimmy Camp Creek watershed imperviousness for future, fully-developed conditions is 43.7% as shown in Table II-3. The predominant land use under future conditions will be low-medium single-family residential.

Table II-3
Future Land Use

			% Imperviousness						Composite
Region	Area (ac)	Area (sqmi)	5	15	45	55	65	85	% Imp
East Fork Tributary	6,274	9.80	8%	11%	65%	8%	0%	8%	42.1%
Franceville Tributary	2,713	4.24	11%	16%	68%	4%	0%	0%	35.8%
Strip Mine Tributary	3,869	6.05	23%	10%	65%	2%	0%	1%	33.4%
Corral Tributary	5,649	8.83	13%	25%	54%	6%	0%	2%	33.4%
Marksheffel Tributary	3,316	5.18	1%	18%	40%	0%	0%	42%	55.9%
West Fork Tributary	2,647	4.14	4%	6%	2%	29%	13%	46%	68.7%
Ohio Tributary	767	1.20	3%	21%	69%	0%	0%	7%	39.9%
C and S Tributary	1,325	2.07	5%	2%	77%	3%	3%	10%	46.9%
Blaney Tributary	995	1.55	5%	37%	58%	0%	0%	0%	32.0%
Jimmy Camp Main	15,400	24.06	16%	11%	37%	8%	4%	24%	45.8%
	ac	sqmi							
Totals…	42,956	67.1	12%	14%	48%	7%	2%	17%	43.7%

2.4 Soil

Soil information was obtained from the Soils Survey of El Paso County Area, Colorado, USDA Soil Conservation Service, 1981. The significance of soil type for hydrologic analysis is in the infiltration rate. Soils are classified into four hydrologic classifications; namely, Types A, B, C, and D. Initial infiltration rates range from 5.0 inches per hour for Type A soils to 3.0 inches per hour for Type C and D soils.

The study area contains all four Hydrologic Soils Group classifications. The study area is predominantly comprised of Type B soils, which constitute half of the watershed area. Type B soils can be characterized as silt loam or loam. These soils have a moderately high rate of infiltration of 4.5 inches/hour. The second most common soil type is Type C soils that have moderately low infiltration and moderately high runoff potential. These soils comprise one quarter of the watershed area. Soil Types A and D constitute the remaining one quarter (approximately one eighth each) of the watershed and are spread throughout the area.

Soil characteristics significantly influence hydrologic responses, but they are also a concern to a planning study due to the erosion and sediment potential that can develop with increased base flows and more frequent high channel velocities caused by urban development. Figure II-4 shows the soil locations by Hydrologic Soils Group classification.

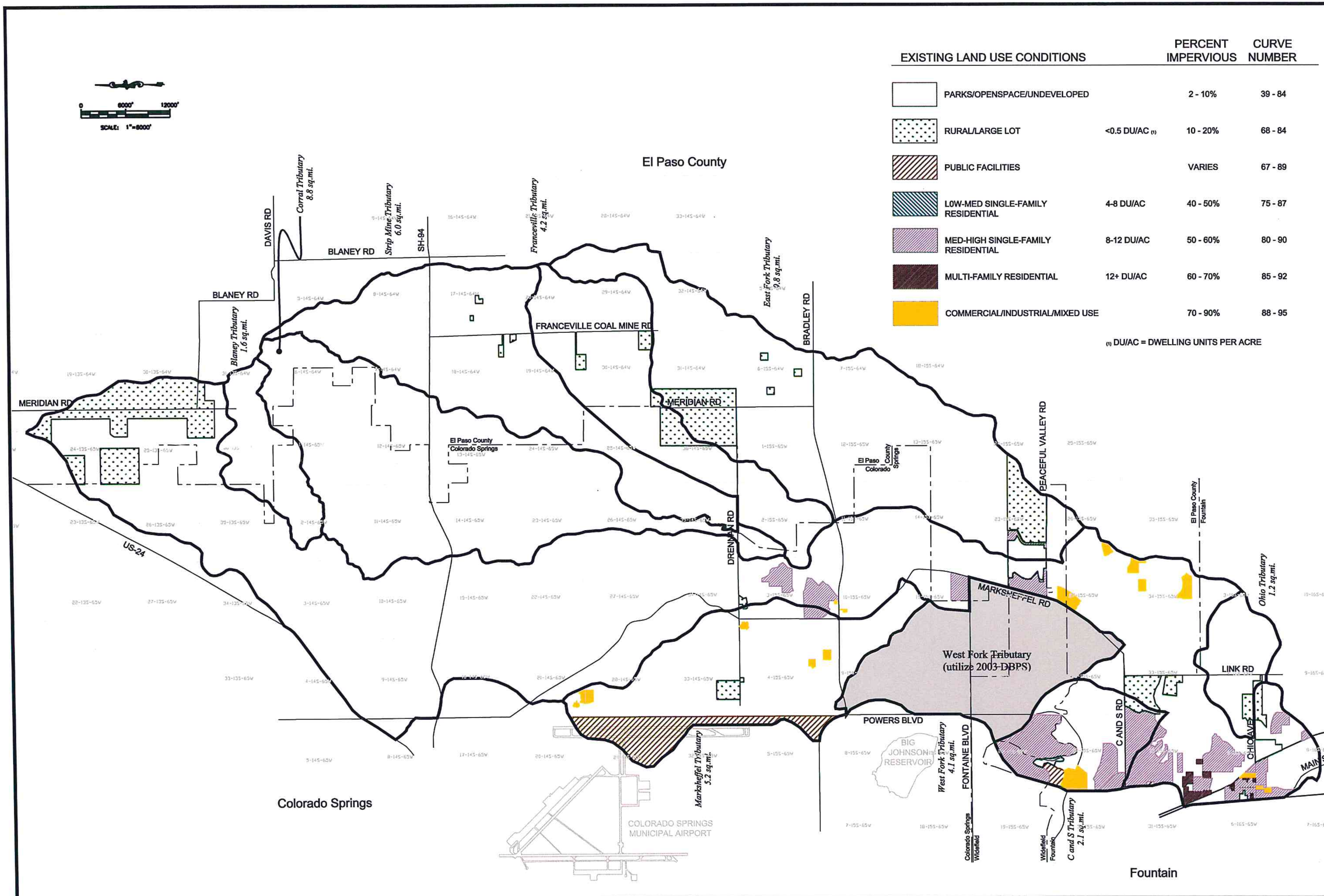
2.5 Stream Gage Data

Located near the bridge on Ohio Avenue over the mainstem of Jimmy Camp Creek, a USGS gage station exists that has 31 years of record. The U. S. Army Corps of Engineers Flood Flow Frequency Analysis (FFA) method for statistical analysis of stream flow gage data was applied to the record. Based upon this analysis the following results were obtained and are presented in Table II-4.

Table II-4
Stream gage data Analysis
Ohio Avenue at Jimmy Camp Creek
USGS Gage 0715900

	Discharge (cfs)			
	2-year	5-year	10-year	100-year
Tributary area at Gage 65.6 square miles Without 1965 peak discharge estimate	475	1,490	2,700	11,100
Tributary area at Gage 65.6 square miles With 1965 peak discharge estimate	503	2,220	5,170	50,600

The results of the flood flow frequency analysis, based on the length of record, would indicate that the 2-year and 5-year discharges could be relied upon for calibration of the hydrologic model. The result for the 100-year frequency varies considerably from past studies including the City of Colorado Springs and El Paso County Flood Insurance Study, the 1987 DBPS and the Fountain Creek Watershed Study, wherein the 100-year discharge estimated in these studies were much higher in comparison to the flood frequency analysis. Due to the period of record the gage may not yield reliable results for the 100-year frequency. Also reported on Table II-4 are the estimated peak discharges when the historical peak of 124,000 cubic feet per second associated with the 1965 flood event is input to the flood frequency analysis.



EXISTING LAND USE CONDITIONS			PERCENT IMPERVIOUS	CURVE NUMBER
	PARKS/OPENSPACE/UNDEVELOPED		2 - 10%	39 - 84
	RURAL/LARGE LOT	<0.5 DU/AC (1)	10 - 20%	68 - 84
	PUBLIC FACILITIES		VARIES	67 - 89
	LOW-MED SINGLE-FAMILY RESIDENTIAL	4-8 DU/AC	40 - 50%	75 - 87
	MED-HIGH SINGLE-FAMILY RESIDENTIAL	8-12 DU/AC	50 - 60%	80 - 90
	MULTI-FAMILY RESIDENTIAL	12+ DU/AC	60 - 70%	85 - 92
	COMMERCIAL/INDUSTRIAL/MIXED USE		70 - 90%	88 - 95

(1) DU/AC = DWELLING UNITS PER ACRE

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JIMMY CAMP CREEK WATERSHED DRAINAGE BASIN PLANNING STUDY

EXISTING CONDITIONS LAND USE MAP

CITY OF COLORADO SPRINGS

Project No.: 14008
Date: OCT 2014
Design:
Drawn: BJW
Check:
Revisions:

II-1

III. HYDROLOGIC ANALYSIS

3.1 Overview

Hydrologic analysis was conducted to determine the 2-, 5-, 10-, and 100-year peak flows for existing and future development conditions. The *Hydrologic Modeling System HEC-HMS, version 3.5,* was used to develop runoff hydrographs from individual sub-basins and to route and combine them through a model of the drainageways. A total of 356 sub-basins were developed for the 67.1 square mile study area. The sub-basins generally range in size from 70 acres to 150 acres, averaging 112 acres. The maximum sub-basin size was set below 200 acres. The watershed includes 6 large tributaries ranging in size from 4 to 10 square miles, and 3 small tributaries ranging from 1 to 2 square miles. The 6 large tributaries constitute 57% of the total watershed area and are an important factor to the Jimmy Camp Creek hydrology.

Input data was prepared using guidelines and values recommended in the City of Colorado Springs and El Paso County Drainage Criteria Manual [DCM]. Hydrologic parameters were measured from the twenty-foot contour interval USGS project mapping. Impervious values were measured from recent aerial photography and field inspections. Soil parameters were measured from the SCS Soils Survey for El Paso County. The results of the hydrologic analysis were compared to previous studies. Individual sub-basin results were evaluated by cubic feet per second/acre for reasonableness based on the applied sub-basin imperviousness. Discussions of specific hydrologic parameters and results follow.

Due to inconsistencies between the gauge data and the preliminary hydrologic model output for the frequent flood events (2-year and 5-year), additional work was undertaken to better calibrate the existing conditions model. Historical storm characteristics and channel geomorphology analyses were completed. The storm characteristics were evaluated by Applied Weather through NEXRAD data analyses. Although the record data was limited due to the infancy of the technology, it does show that basic assumptions used for conventional rainfall-runoff models are not consistent with the recorded storm data. It appears that the basin conditions prior to the more frequent storm events (2-year and 5-year) are better represented by the AMC I (drier) conditions as opposed to the AMC II (wetter) conditions as normally applied. The data used for the analyses was not sufficient to analyze more severe storms, such as the 10-year and 100-year events therefore modeling for the DBPS for these conditions was based on conventional criteria of uniform rainfall a 24-hour duration, and an AMC II condition. Further analyses of rainfall-runoff data would be required to determine if these assumptions should be revised for future studies such as DBPS updates or MDDPs.

The results of the analyses are adequate to make some preliminary adjustments for planning the basin and completing the DBPS, however, it is anticipated that further evaluation of NEXRAD data may produce a more representative design storm and better methods for applying them. Future studies in the basin, such as MDDPs, may require the application of different design storms and modeling approaches than those used in the DBPS. Calibration of the model was based on a limited number of events and additional work should be done to confirm the results of these analyses.

3.2 Storm Rainfall Analysis

A Storm Rainfall Analysis was undertaken to give more insight to the nature of storm events within the Jimmy Camp Creek watershed. This analysis was conducted using data published by the National Weather Service (NWS). The analysis was broken into two phases; Phase 1 involved the analysis of Denver and Pueblo area NEXRAD rainfall data for the storms that were associated with the peak stream flow event as gauged at the USGS stream gauge at Ohio Avenue for the period of 1994 through 2006; and Phase 2 being the development of a detailed Storm Precipitation Analysis System (SPAS) for three specific storms that produced the peak gauged flow in 2004, 2005 and 2006. The storm precipitation analysis system (SPAS) uses rain gauge data to calibrate the NEXRAD readings. The purpose of this analysis was to provide rainfall data with respect to distribution, depth and duration so that more detailed rainfall patterns could be input to the hydrologic model in an effort to calibrate the 2-year and 5-year peak discharge to better match the gauged stream flow for specific events. The majority of the results of this work are summarized in the Jimmy Camp Creek Drainage Basin Planning Study Hydrology Technical Addendum that has been prepared under a separate cover.

Metstat, Inc. and Applied Weather Associates, LLC teamed to develop a precipitation analysis software package named Storm Precipitation Analysis System (SPAS) that analyzes precipitation associated with storms. Using measured precipitation and radar data (if available) SPAS creates a detailed precipitation analyses including high spatial resolution precipitation fields that accurately quantify the spatial and temporal distribution of storm precipitation.

The analyzed precipitation fields can be used to produce hourly or sub-hourly precipitation amounts over user defined watersheds and sub-watersheds. Detailed precipitation information from SPAS has a number of valuable applications, including, but not limited to:

- Hydrologic model calibration and verification with much improved precision and reliability
- Detailed storm precipitation information to support forensic meteorology applications
- The basis for storm-centered depth-area-duration (DAD) analyses for use in site-specific probable maximum precipitation (PMP) studies

The results of the rainfall analysis were used to input rainfall data to the HEC-HMS model. A one-half square mile grid of 6-minute rainfall data was input to the HEC-HMS model in place of the standard storm distributions, depths and durations that are normally applied when modeling peak discharges and runoff volumes for watersheds. Using the storm rainfall analysis it was possible to input a storm that is specific to a gauged event. In the case of the Jimmy Camp Creek DBPS actual rainfall events that occurred in 2004, 2005 and 2006 were input to the HEC-HMS model in an effort to calibrate the hydrology model to produce peak discharges similar to those estimated at the Ohio Avenue the day of the rainfall event. Detailed results with respect to rainfall depth, duration, distribution, time interval and areal extent are presented in the final report prepared by Applied Weather Associates that is contained within the technical addendum to this DBPS.

Three storm events were chosen from the period of 1994 to 2006 when radar data was available. The three storms analyzed were the thunderstorms that occurred August 4, 2004, July 14, 2005 and August 12, 2006. These storms were chosen since the gauge results indicated that these storms produced peak discharges at the Ohio Avenue between the 2-year and 5-year frequencies according to the flood frequency analysis of the gauge data. The 2004 event storm duration was approximately 15 hours and had rainfall depths ranging from .5 inches to 3 inches. The storm covered the entire watershed but the most intense rainfall fell over the West Fork Jimmy Camp Creek sub-watershed and in the upper portions of the Jimmy Camp Creek main and Strip Mine sub-watersheds. The areas that received the highest rainfall depths and intensities covered only about 5 square miles of the entire watershed. The thunderstorm on this date and time frame produced a peak of approximately 210 cubic feet per second at the gauge that was an initial peak of a greater runoff event that caused a peak later the day of the 5th when a flow of 800 cubic feet per second was recorded at the gauge.

The 2005 event storm duration was approximately 8 hours and had rainfall depths ranging from .5 inches to 2 inches. The storm covered the entire watershed but the most intense rainfall fell over the upper portion of the Jimmy Camp Creek main sub-watershed. The area that received the highest rainfall depths and intensities covered only about 1 square mile. The thunderstorm on this date and time frame produced a peak of approximately 500 cubic feet per second at the gauge.

The 2006 event storm duration was approximately 9 hours and had rainfall depths ranging from .5 inches to 4.5 inches. The storm covered the entire watershed but the most intense rainfall fell over the lower third of the Jimmy Camp Creek watershed. The area that received the highest rainfall depths and intensities covered only about 6 square miles. The thunderstorm on this date and time frame produced a peak of approximately 698 cubic feet per second at the gauge.

SPAS is based on the sound foundation of the storm analysis procedure used by the Weather Bureau, thereby providing consistency between storms previously analyzed and those analyzed by SPAS. However, SPAS computes more precise and perhaps more accurate results by using a more sophisticated timing algorithm, a variety of base maps, a wider variety of data, fewer assumptions and more effective quality control measures. Although largely automated, SPAS has been designed to be flexible such that it can be utilized for any storm situation, account for unique meteorological conditions and accept a variety of data, including radar reflectivity. And lastly, SPAS produces reproducible results and incorporates less subjectivity than previous storm analysis studies.

SPAS provides an analysis tool for analyzing storm precipitation patterns with much improved spatial and temporal resolution that has historically been available for use in runoff model calibration and validation. The improved spatial data enables variations in soils types, infiltration rates and lag times to be associated with detailed precipitation rain rates and volumes. Additionally the hourly precipitation analyses allow for improvement in runoff timing.

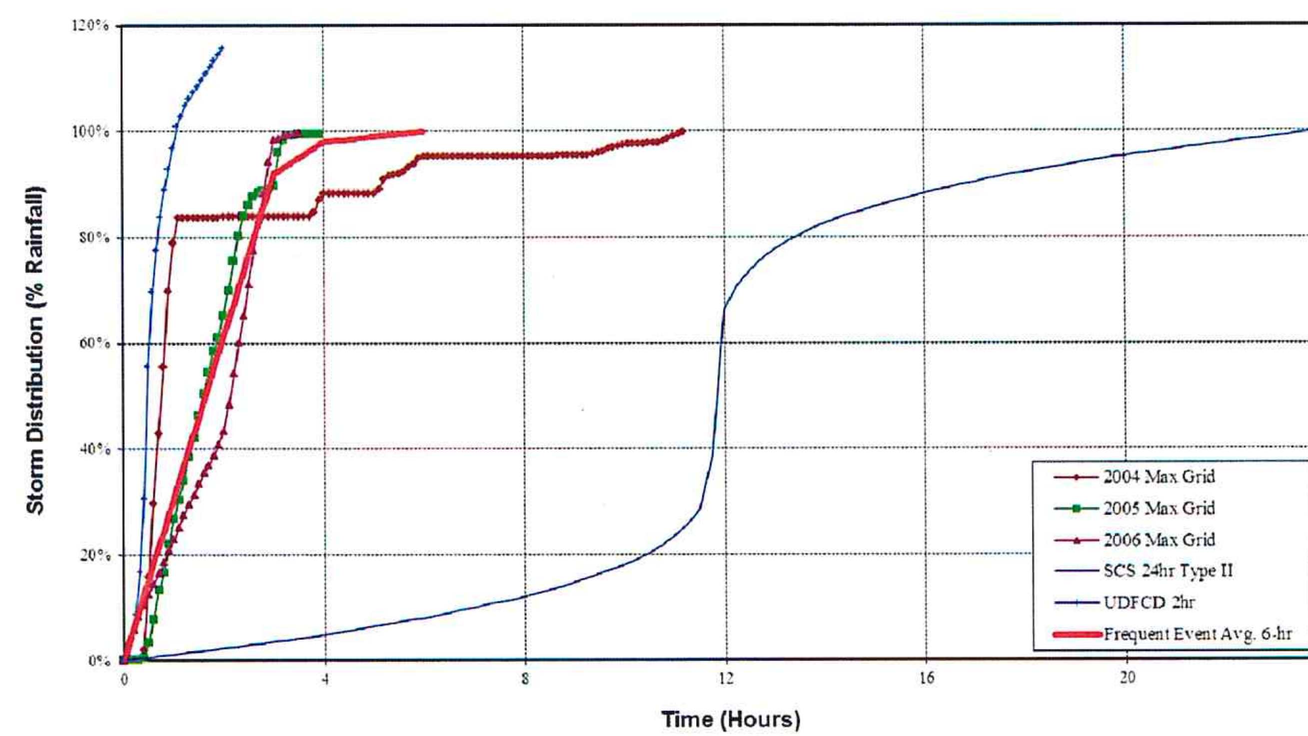
Radar Rainfall Calculation Benefits

The advantages of using radar data in a SPAS analysis outweigh the potential drawbacks associated with radar data. The major benefits are the increased spatial detail and temporal characteristics of the precipitation. While rain gauges are scattered about an area, often more concentrated in populated areas, radar precipitation calculations can be determined across the entire radar domain. Rain gauge observations have a temporal resolution of a few minutes to 24 hours (most are hourly or daily) while radar precipitation calculations have a temporal distribution of 4 to 6 minutes depending on the operating mode of the radar.

Determining a precipitation isohyetal pattern using solely rain gauge observations limits the maximum precipitation center to an observed precipitation observation location. Chances are that the maximum precipitation center is located across an area where there are no rain gauges present. Radar calculated precipitation is beneficial in identifying the true location of the maximum precipitation center and better at resolving spatial detail than spatial interpolation across the domain. Radar data also identifies enhanced precipitation depth areas due to terrain features (orographic lift due to terrain features) and identifying precipitation depths over water bodies such as oceans, lakes and reservoirs where there are no rain gauges.

Figure III-1 shows the standard storm distributions for the uniform 24-hour and 2-hour lengths. The 2004, 2005, and 2006 maximum rainfall grids, taken from the detailed Rainfall Analysis, are presented for comparison. These three specific storm events only covered a few square miles in area, not the entire JCC watershed. It is clear that neither the 24-hour or 2-hour storm distributions reflect the actual rainfall data for these events. A 6-hour storm distribution was derived from the detailed analysis of the three actual storm events and plotted against the gauges. This distribution compares very well and is recommended for use as the design storm pattern for the 2-year and 5-year frequency analyzes.

Figure III-1
2004, 2005, 2006 Storm Distributions
Calibrated NEXRAD Data



Results and Conclusions

Based upon the Storm Rainfall Analysis the following results and conclusions have been reached. The entire rainfall analysis report summarizing the results can be found in the Hydrology Technical Addendum.

1. Based upon the detailed analyses of the 2004, 2005 and 2006 storm events it has been concluded that they produced gauged discharges at Ohio Avenue in the range of 1-1/2-year to 5-year recurrence intervals. This information is summarized on Table III-1.
2. The rainfall analysis has revealed that the rainfall patterns are not uniform and vary significantly across the watershed. This more than likely explains why the 2-year and 5-year peak discharge estimates returned by the hydrology model produced initially as part of the Jimmy Camp Creek planning study are so much higher than what the gauge data and associated flood flow frequency analysis returns for these recurrence intervals. The analysis shows that there seems to be no general correlation between storm intensity, distribution, peak discharges at the gauge or location of the design storm. The analysis also shows that rainfall intensity varies significantly across the watershed and that the peak rainfall amounts can occur over very limited portions of the basin. Storms having very different characteristics can produce similar peak flow results at the Ohio Avenue gauge. Analysis of the 2004, 2005 and 2006 storms shows that maximum rainfall that resulted in gauged flows of the 5-year frequency or higher fell on less that 6 square miles of the watershed while the average rainfall fell on over 28 square miles of the watershed. The storm rainfall maps for the 2004, 2005 and 2006 events are presented on Figures III-2a, 2b and 2c. Presented on Figure III-3 are the spatial distributions for the 2004, 2005 and 2006 storm events that display the variable nature of these rainfall events that resulted in similar peak discharge results at the gauge.
3. Each of the storm events evaluated in detail were thunderstorms with total durations ranging from 8 to 15 hours. This suggests that when modeling high frequency storms a 24-hour duration is not a representative storm pattern.
4. The distribution of the three storms evaluated in detail each had a short period of high intensity rainfall, similar to the intensity predicted by the standard SCS Type II thunderstorm pattern. However, they have almost no leading leg prior to the period of intense rainfall and they have a longer period of higher intense rainfall and a much shorter trailing leg. This is shown on Figure III-1.
5. The areal distribution for each of the storms studied in detail can be used as input data to the HEC-HMS program to better calibrate the model and return a more reliable peak discharge at the Ohio Ave gauge. With actual rainfall patterns input to the model, the calibration of the HEC-HMS model can commence with parameters such as curve number, antecedent moisture condition and stream routing coefficients being adjusted in order to attempt to match the gauged hydrograph for each of the storm events.
6. The areal extent of the storms that were analyzed typically had coverage up to 40 square miles. However, the extent of the high intensity rainfall covered areas in the 2 to 6 square mile range. This suggests that high frequency storms should not be modeled using uniformly applied rainfall that covers the entire watershed.
7. The rainfall analyses completed with this DBPS considered a limited number of events and the resulting conclusions are being applied preliminarily only for the Jimmy Camp Creek basin. To develop more reliable typical storm patterns more extensive analyses should be completed. These analyses may be completed as part of the City's proposed criteria update.

Table III-1
NEXRAD Storm Summary

Storm	Duration (hr)	Avg. Precip. (in)	Avg. Return (yr)	Max Precip. (in)	Max Return (yr)
2004	13.6	1.2	1	2.2	7
2005	3.5	0.5	<1	1.5	1.5
2006	4.2	1.7	2	4.5	25

Table III-2
Design Rainfall

Frequency:	100-yr	10-yr	5-yr	2-yr
24-hr Point Rainfall (in):	4.5	3.2	2.7	2.1
6-hr Point Rainfall (in):	3.5	2.4	2.1	1.7
Areal Reduction:	***94.4%***	***94.4%***	***94.4%***	***94.4%***
24-hr Adjusted Rainfall (in):	4.25	3.02	2.55	1.98
6-hr Adjusted Rainfall (in):	3.30	2.27	1.98	1.60

The 6-hour storm distribution for the 2-year and 5-year events should only be used to estimate undeveloped basin conditions and to set flow limits for evaluating allowable release rates from detention storage basins. Rainfall depths and durations published by NOAA Atlas 2 should continue to be applied to the design storm distributions for projected developed conditions. The 6-hour storm distribution should not be used for floodplain analyses or flood control structure design.

3.4 Sub-basins

Sub-basins were evaluated using the USGS 7½ minute quadrangle, twenty-foot contour mapping provided for the project and checked with the 2-foot contour mapping where available. Major watershed and sub-basin boundaries were established based on topographic and physical drainage boundaries, such as major roadways. Watershed boundaries were verified in the field. The watershed was divided into 356 sub-basins, with an average area of 113 acres, to convey each design storm. Larger sub-basins were used in the less developed upper reaches, and smaller sub-basins were necessary to evaluate the more complicated drainage of the more urban reaches. Consistency in sub-basin size was deemed critical to the HEC-HMS model consistency. Wide variations in sub-basin size can produce instabilities in the internal calculations and model results. A brief statistical analysis finds that the median sub-basin size is 114 acres, the lower 10^{th}-percentile is 72 acres and upper 90^{th}-percentile is 156 acres. Although efforts were made to define sub-basins of consistent size, a few were defined larger or smaller than desired due to topography, development, or required design point locations. The largest sub-basin is 192 acres that was defined between an airport runway and Drennan Road along Marksheffel Tributary. There is no development in this sub-basin and no other feature to warrant subdividing. The smallest sub-basin is 30 acres that was defined by the State Highway 94 embankment along Strip Mine Tributary. The sub-basin mapping is presented on Exhibit 1contained in the map packet of this report.

3.5 SCS Unit Hydrograph Transform

The SCS unit hydrograph transform was used to develop the runoff hydrograph from each sub-basin. This procedure requires input of a "lag time" to shape the hydrograph. The lag time was derived from the time of concentration that was computed in accordance with the SCS TR-55 manual. The parameters used in this method are travel length, slope, rainfall, and surface cover. Within each sub-basin runoff begins as sheet flow that develops into shallow concentrated flow and then to open channel flow. The travel time for each of these segments was calculated and totaled to compute the sub-basin time of concentration. The time of concentration was then reduced to 60% to produce the lag time.

Future conditions lag times are generally shorter due to more impervious areas. To simulate this, the sheet flow lengths were reduced and the land cover was adjusted from a natural vegetated cover to an urban development cover. Adjusting these factors resulted in lag times that were typically 25% shorter than those under existing conditions.

3.6 Muskingum-Cunge Routing

The HMS model reads in the storm hydrographs from each sub-basin generated. The individual hydrographs are routed through a system of defined channels, pipes, and reservoirs and combined as the system is routed downstream. Since this is a developing watershed and the future condition is expected to be fully urbanized existing stock ponds and irrigation canals in the basin were not analyzed for their influence on routing. It was assumed that these ponds and ditches would be full or not in existence as at the time of full development. Typically urban development eliminates the need for agricultural ponds and ditches and they eventually become disconnected and filled in. Some ditches can be found in urban areas as they still service a nearby agricultural need. Even in these cases, the irrigation ditches typically do not accept urban runoff into their canals. Any ditches that may remain will be assumed to require a 100-percent overflow structure such that no diversion of runoff occurs.

Separate channels were defined for each major reach in the system. The routing element input parameters include length, slope, roughness coefficient, and cross-sectional geometry. A summary of the routing reach definition is provided in the Hydrolory Technical Addendum.

The Muskingham-Cunge routing method was utilized for the Jimmy Camp Creek hydrologic models. This method employs an 8-point cross-section specific to the routing reach in which the hydrograph is being attenuated downstream. N-values specific to the channel and the overbanks are also input. The slope of the specific reach is an additional parameter. This method allows for an accurate translation specifically tailored to the variation found along each reach.

3.7 SCS Curve Number Loss

In accordance with DCM standards, for design purposes an antecedent moisture content of II (AMC II) was applied for determining runoff from a 24-hour storm. Tables 5-4 and 5-5 in the City of Colorado Springs and El Paso County Drainage Criteria Manual [DCM] were used for the definition of runoff curve numbers (CN) for various land use categories and hydrologic soil groups. A spreadsheet was developed in which each sub-basin was subdivided based on the four hydrologic soil groups. The existing conditions land use map was then overlaid on the soils map and curve numbers were calculated for each land use/soil group combination. The area of each curve number group was calculated and applied to the spreadsheet that developed a weighted average curve number for each sub-basin. This process was applied for every sub-basin in the watershed. The weighted curve number for the watershed under existing development conditions is 71 with an average percent imperviousness of 4%. Since the watershed is primarily undeveloped, these numbers are in line with expectations.

The same process was applied for future condition development assumptions. Fortunately most of this watershed has been planned in detail for future development, see Figure II-3. Information provided by the City of Fountain, City of Colorado Springs and El Paso County planning departments was vital in developing a composite map of future land development. Several large planning developments were utilized accounting for two-thirds of the watershed area. These include: Banning Lewis Ranch [27 square miles], City of Fountain Comprehensive Plan [7 square miles], West Fork Tributary 2003 DBPS [4 square miles], Rolling Hills Ranch [3 square miles], Lorson Ranch [2 square miles], and Norris Ranch [2 square miles], see Figure II-2. Future projections were made on the remaining lands with the assistance of County planners to define a highly detailed and accurate future conditions land use map. The weighted curve number for the watershed under future development conditions is 79 with an average percent imperviousness of 43%. Per the DCM standards Soil Group A was not used for areas of future urban development. Where development is planned on A soils, Soil Group B soils were applied.

Tables III-3A and III-3B describe the percent imperviousness and curve number relationship for each of the different land use categories and moisture condition. Information used in Table III-3A is based on the Drainage Criteria Manual Table 5-5 for a 24-hour storm, Antecedent Moisture Condition II. Information presented in Table III-3B was derived using methods recommended by NRCS when determining curve numbers representative of Antecedent Moisture Condition I.

In order to calibrate the HEC-HMS model to better match the stream gauge data for the 2004, 2005 and 2006 storms it was found that the AMC I moisture condition was more realistic than assuming a AMC II condition. A check of the antecedent moisture condition for the 2004, 2005 and 2006 events revealed that none of these storms was preceded by measurable rainfall in the seven days prior to the storm. It was therefore decided to utilize the AMC I moisture condition when calibrating the existing condition HEC-HMS model for the 2-year and 5-year storm events.

Table III-3A
Land Use Curve Number Index
Antecedent Moisture Condition II

Category		% Impervious	Hydrologic Soils Group			
			A	B	C	D
Undeveloped, Open Space		2 - 5	39	61	74	80
Parks, Golf Course		5 - 10	49	69	79	84
Residential Very Low	[<1 du/ac]	10 – 20	n/a	68	79	84
Residential Low-Med	[1-8 du/ac]	40 – 50	n/a	75	83	87
Residential Med-High	[8-12 du/ac]	50 – 60	n/a	80	87	90
Residential High	[12-24 du/ac]	60 – 70	n/a	85	90	92
Industrial, Mixed Use		70 – 80	n/a	88	91	93
Commercial		80 – 90	n/a	92	94	95
Public Facilities			….. specific to each site …..			

Table III-3B
Land Use Curve Number Index
Antecedent Moisture Condition I

Category		% Impervious	Hydrologic Soils Group			
			A	B	C	D
Undeveloped, Open Space		2 - 5	23	41	56	63
Parks, Golf Course		5 - 10	30	50	62	69
Residential Very Low	[<1 du/ac]	10 – 20	n/a	49	62	69
Residential Low-Med	[1-8 du/ac]	40 – 50	n/a	57	68	72
Residential Med-High	[8-12 du/ac]	50 – 60	n/a	63	72	78
Residential High	[12-24 du/ac]	60 – 70	n/a	70	78	80
Industrial, Mixed Use		70 – 80	n/a	76	79	85
Commercial		80 – 90	n/a	80	86	87
Public Facilities			….. specific to each site …..			

3.8 Calibration of the HEC-HMS Model for the 2-year and 5-year Frequencies

The result of the Storm Rainfall Analysis, as described above in Section 3.2, provided 163 "pseudo"-rain gauge stations each covering one-half square mile in area and a time increment of 6 minutes. Each of these pseudo-gauges was input in the existing conditions HEC-HMS model and the nearest sub-basins were assigned to the pseudo-gauge within that one-half square mile area. The August 2004 storm was first run with CN-values corresponding to AMC-II moisture conditions. This produced a peak flow of 1,950 cubic feet per second with a runoff volume of 378 acre-feet. This result is significantly higher than the gauge reading, similar to what was previously determined for the 2- and 5-year existing condition (un-calibrated) model results.

The first calibration adjustment was made in the Antecedent Moisture Condition [AMC]. Standard criteria calls for the use of an AMC II condition that produces curve number values, as shown in Table III-3A, based on an assumption of "average" moisture levels in the soils. The Jimmy Camp Creek watershed falls in a semi-arid climatological region. The typical soil moisture condition in this area is drier than "average" levels. Furthermore, an analysis of the previous seven-day precipitation records show that little to no rainfall occurred prior to the storm events analyzed. In the previous seven days, the 2004 storm had 0.22 inches of cumulative rainfall, the 2005 storm had 0.0 inches, and the 2006 storm had 0.49 inches. Based on this, the moisture condition was adjusted to AMC I values as depicted in Table III-3B. Lower curve numbers correlate to greater infiltration capacity of soils, which results in less runoff, both in volume and in peak flow. This adjustment seems to be consistently reliable for frequent storm events like the 2-year and 5-year storms.

Use of the NEXRAD data provides a measurable means to evaluate actual rainfall in the area. Combining measured rainfall with the measured stream gauge hydrographs allows a calibration approach to adjust specific model input parameters to target a measured result. Analysis of the AMC I calibrated 2004 storm produced similar runoff volumes to the gauge data with slightly lower peak flows. Figure III-4 shows hydrograph comparisons between the gauge reading and HEC-HMS AMC I calibrated model output. The 2004 gauge hydrograph produces a volume of 25.6 acre-feet, while the HEC-HMS model volume produces 46.6 acre-feet. This volume difference can be attributed to numerous watershed features, such as, irrigation ditch diversions, small agricultural pond storage, local surface depression storage, storage in

SCS reservoirs and inadvertent storage behind roadway crossings. It is not feasible to attempt to model all of these features within a 67 square-mile basin. In terms of average curve number value, a change in CN of one point would produce a change in runoff volume at the gauge of 21 acre-feet. The 2004 storm had broad coverage over 90% of the watershed with three distinct cells that fell over West Fork Tributary, upper Jimmy Camp Creek and the Strip Mine Tributary. The first small peak results from the West Fork Tributary. The gauge does not reflect this peak. Looking in this tributary we found sizeable agricultural ponds that may hold entire small flood events and prevent runoff from reaching the gauge station.

The second calibration refinement focused on routing adjustments. The routing parameters are based on the measured geometry of a typical cross-section in each routing reach along with measured channel slopes. Since this input was measured, it was not adjusted. The only other routing parameter lies with the roughness, Manning's n-value. This parameter is more of an engineering judgment within a defined range of acceptable values. The initial values used in the model were set in the middle of the range. By decreasing the roughness value to the lower end of the range for upper Jimmy Camp Creek, and increasing it for the upper tributaries, the model results in a single peak arriving at the gauge near the same time as shown in Figure III-4. In the 2004 event, the average watershed rainfall was 1.20" (1-year event) with a high grid of 2.23" and a low grid of 0.02". The recorded peak flow of the gauge was 215 cubic feet per second. [Note: this peak flow was a secondary peak that occurred after the primary peak on August 4th.] The return period of the storm changes greatly depending on whether the average or maximum rainfall is used. It appears that using the maximum rainfall amount is most consistent with the NOAA Atlas values and could be used as the depth for the design storm.

Figure III-4
2004 Storm – Calibrated Hydrograph Comparison

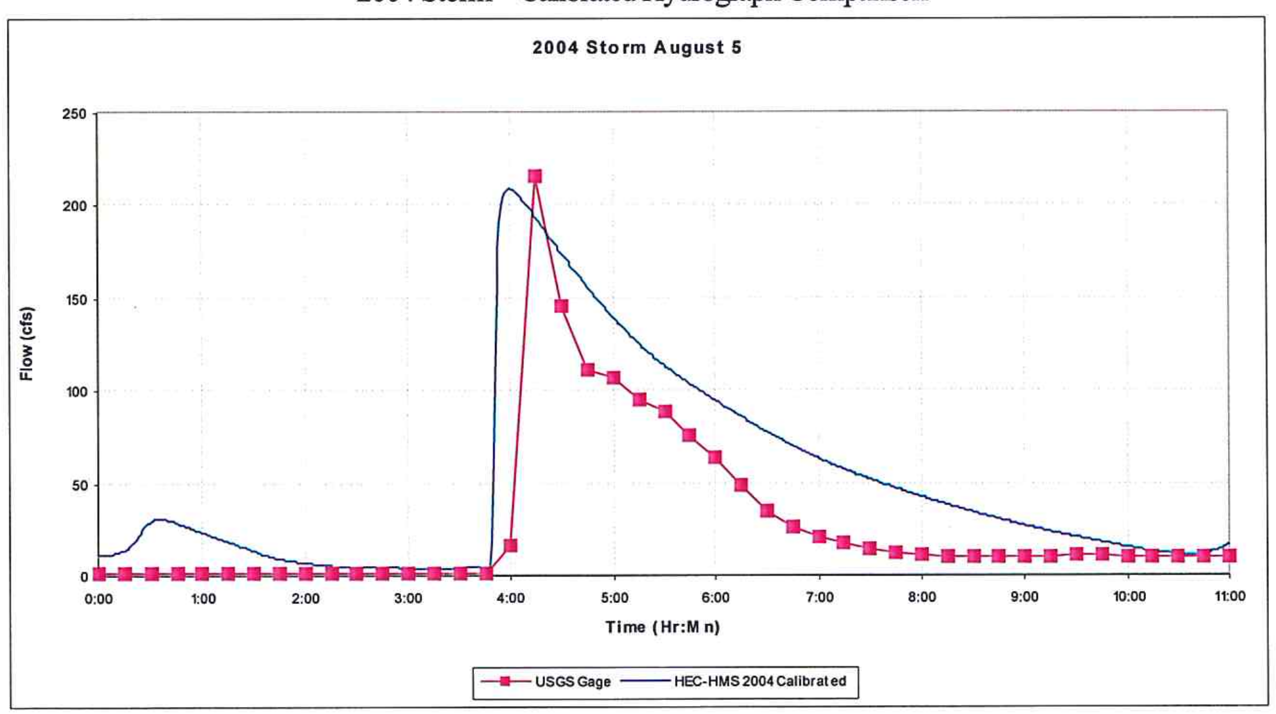


Figure III-5
2005 Storm – Calibrated Hydrograph Comparison

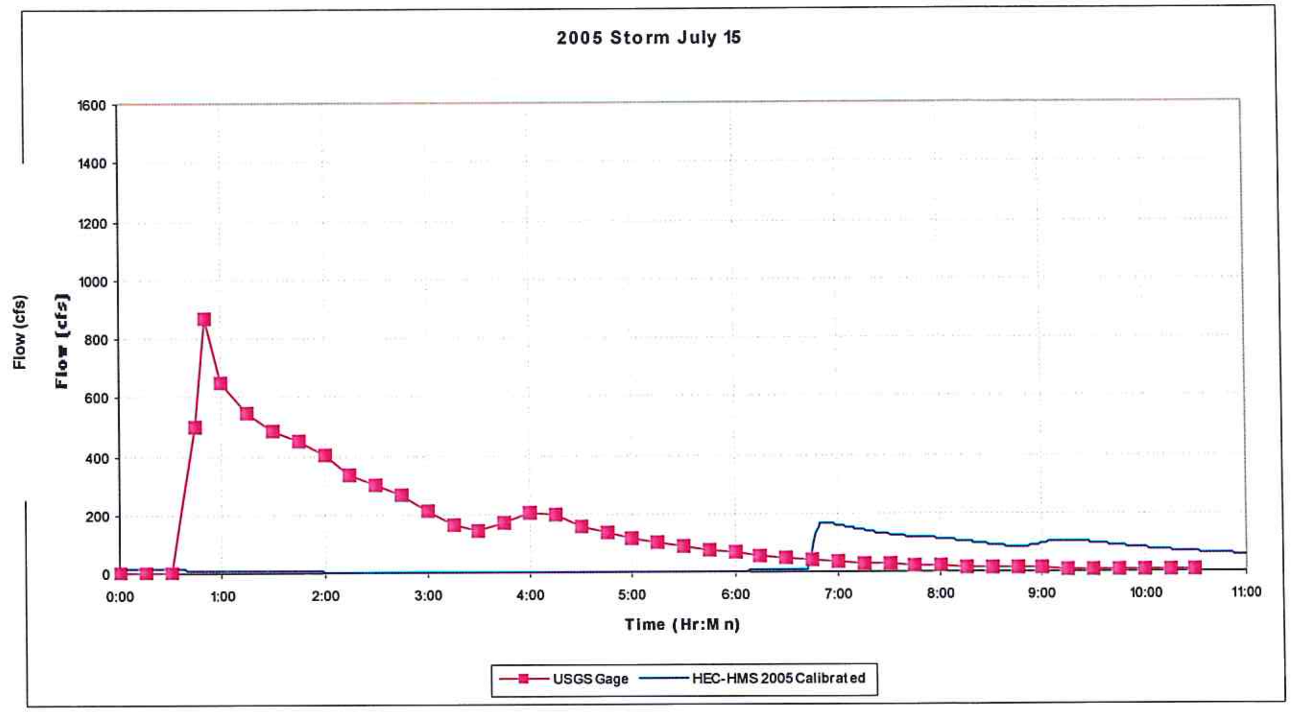
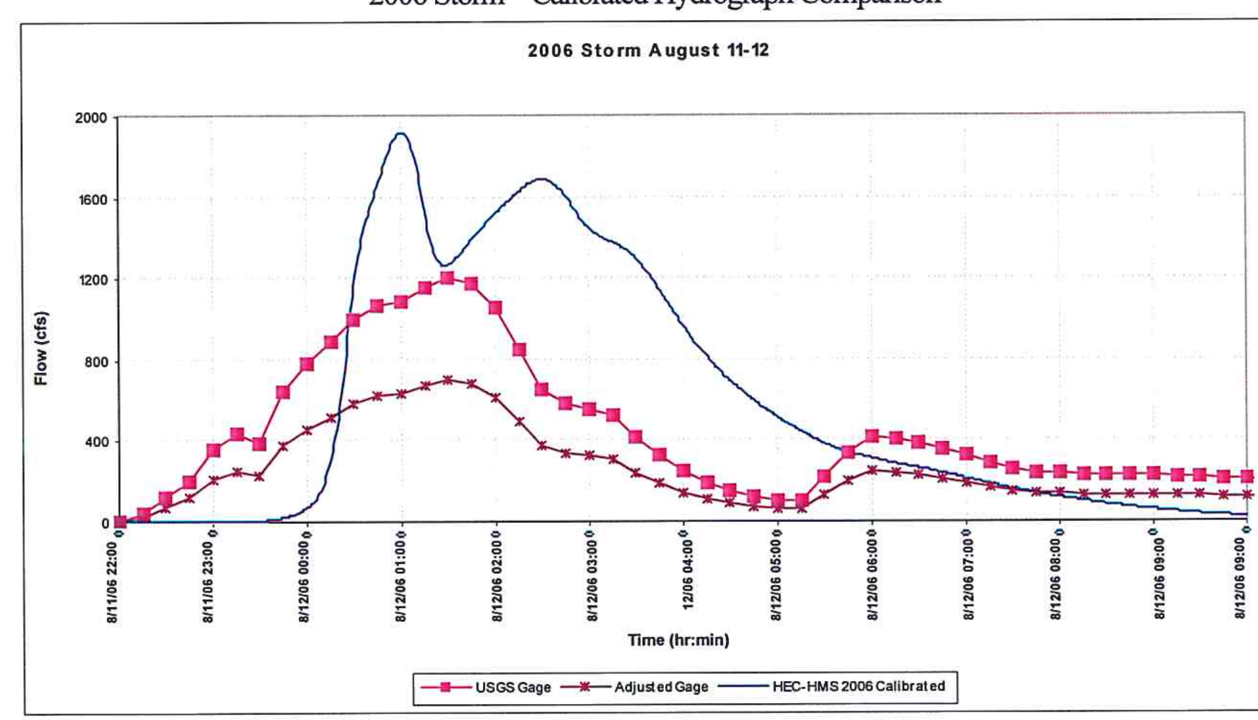


Figure III-6
2006 Storm – Calibrated Hydrograph Comparison



The 2004 calibrated model was next tested against the 2005 and 2006 storms. The 2005 storm was a small event covering only the upper third of Jimmy Camp Creek with the most intense portion of the storm centered over the northern tip of the watershed. The lack of rainfall over the lower two-thirds of the watershed did not allow this event to be properly calibrated. The average watershed rainfall was 0.50" (<1-year event) with a high grid of 1.52" and a low grid of 0.11". The recorded peak flow of the gauge was 869 cubic feet per second. The relationship between watershed rainfall and recorded gauge peak flow is not consistent for the 2005 event. The calibrated and gauge hydrographs are shown on Figure III-5. Attempts to recalibrate the 2004 model to better match the 2005 gauged hydrograph with respect to time to peak by further adjusting the roughness values in specific reaches caused the results of the 2004 calibration to diverge from the gauge data.

The 2006 storm was a larger, more widespread rainfall event that covered the lower 75% of the watershed. The most intense cells occurred in the lower watershed over East Fork Tributary and C and S Tributary. The average watershed rainfall was 1.75" (2-year event) with a high grid of 4.49" and a low grid of 0.01". The HEC-HMS output compared well with the recorded gauge. The USGS has indicated that the peak should be revised to 698 cubic feet per second instead of the 1,200 cubic feet per second as published. To achieve the peak of 698 cubic feet per second all of the gauge record was adjusted by 58% as shown in Figure III-6. The general shape, peak, and timing compare reasonably well, giving support to the calibration of the model.

The HEC-HMS calibrated model provides consistent results when looking at average watershed rainfall for each event. The calibrated storm models compare well against the stream gauge hydrographs. From this effort it was decided that using the AMC I condition most accurately defines the 2-year and 5-year storm events for Jimmy Camp Creek. Specific routing roughness factor adjustments also help to improve timing of the peaks to correlate with the gauge results. Since a specific storm pattern for Jimmy Camp Creek does not exist, the uniform rainfall procedure must continue to be used as the design storm. However, instead of applying the SCS 24-hour Type II distribution pattern, the frequent event average 6-hour distribution (see Figure III-1) will be used for analysis of the 2-year and 5-year flood events.

A comparison of results rainfall versus the recorded and calibrated peak discharges at the Ohio Avenue gauge is presented on Table III-4.

Table III-4

Comparison of Rainfall vs. Recorded Peak Flow at Ohio Avenue Gauge

Flood Event	Average Rainfall (in)	Return Period (years)	Volume (af)	Recorded Peak (cfs)	Calibrated HEC-HMS Peak (cfs)	Calibrated HEC-HMS Vol (af)
2005	0.50	0.5	129.8	869	166	35.2
2004	1.20	1	25.6	215	209	46.6
2006	1.75	2	433.9	698	1,950	573.4

3.9 Previous Studies

The Jimmy Camp Creek watershed has been studied in the past. In October 1975, the Soil Conservation Service prepared a report entitled *Flood Hazard Analyses for Portions of Jimmy Camp Creek and Tributaries*. This study includes all of Jimmy Camp Creek upstream of Link Road, as well as, Corral Tributary, Strip Mine Tributary, Franceville Tributary, and East Tributary. The hydrology from this study was used by FEMA in preparation of the 100-year regulatory floodplain. Another study was prepared by Wilson and Company in 1987 in a report entitled *Jimmy Camp Creek Master Drainage Planning Study*. This study was prepared for the City of Colorado Springs and El Paso County to formulate a mitigation plan for increased storm water resulting from development. Although the Wilson study finalized the hydrologic analysis, the overall study did not get completed and was never officially adopted. Comparisons of hydrology to both of these reports are found in the following tables. The Ohio Avenue gauge results presented in Table III-8B were obtained by applying the U.S. Army Corps of Engineers Flood Frequency Analysis statistical method to the 30 years of record that is available from the USGS at this gauge. The gauge data results presented in Table III-8B does not include the historical peak discharge in 1965 of 124,000 cubic feet per second.

1975 SCS Flood Hazard Analyses

Table III-5A

Jimmy Camp Creek Comparison to 1975 SCS Study (cubic feet per second)

Location	Area (sq. mi.)	1975 SCS Q_{100}	DBPS Q_{100}	Area (sq. mi.)	Design Point
Confluence w/ East Fork Trib	53.92	14,200	21,784	53.92	DP-J16
Peaceful Valley Rd	44.16	12,900	17,709	44.16	DP-J17
Confluence w/ Marksheffel Trib	41.99	12,600	17,361	41.99	DP-J21
Bradley Rd	36.64	11,800	16,502	36.64	DP-J22
Confluence w/ Corral Trib	31.60	10,700	15,382	31.60	DP-J24
Drennan Rd	14.84	7,100	5,881	14.84	DP-J25
SH-94	9.62	5,500	5,031	9.62	DP-J31

Table III-5B

Tributary Comparison to 1975 SCS Study (cubic feet per second)

Location	Area (sq. mi.)	1975 SCS Q_{100}	DBPS Q_{100}	Area (sq. mi.)	Design Point
East Fork Trib	9.77	5,500	4,677	9.77	DP-E1
Franceville Trib	4.23	3,500	1,515	4.23	DP-F5
Corral Trib	8.25	7,300	6,212	8.25	DP-C4
Strip Mine Trib	5.18	4,500	4,627	5.18	DP-SM2

2003 City of Fountain Study

Table III-6

Comparison to 2003 City of Fountain Study (cubic feet per second)

Location	2003 Ex Q_{100}	2003 Fu Q_{100}	DBPS Ex Q_{100}	DBPS Fu Q_{100}	Area	DP
JCC Outfall	20,805	28,338	22,094	37,986	67.11	DP-J1
Confluence w/ East Fork	19,315	26,458	21,874	32,547	53.92	DP-J16

1987 Wilson Study

Table III-7A

Comparison to 1987 Wilson Study (cubic feet per second)

Location	1987 Ex Q_{100}	1987 Fu Q_{100}	DBPS Ex Q_{100}	DBPS Fu Q_{100}	Area (sm.)	DP
JCC Outfall	21,800	31,000	22,094	31,986	67.11	DP-J1
Confluence w/ East Fork	21,400	-	21,874	32,547	53.92	DP-J16
Peaceful Valley Rd	18,100	-	17,709	26,734	44.16	DP-J17
Bradley Rd	17,800	-	16,502	23,508	36.64	DP-J22
Confluence w/ Corral Trib	15,400	-	15,832	22,741	31.60	DP-J24
Drennan Rd	6,800	-	5,881	10,248	14.84	DP-J25
SH-94	6,800	8,600	5,031	7,135	9.62	DP-J31

Table III-7B

Tributary Comparison to 1987 Wilson Study (cubic feet per second)

Location	1987 Wilson Q_{100}	DBPS Q_{100}	Area (sq.mi.)	Design Point
East Fork Trib	4,400	4,677	9.77	DP-E1
Franceville Trib	2,500	1,515	4.23	DP-F5
Corral Trib	9,600	6,212	8.25	DP-C4
Strip Mine Trib	4,000	4,627	5.18	DP-SM2

2006 Fountain Creek Watershed Study

Table III-8A

Comparison to 2006 Fountain Creek Watershed Study (FCWS) – 100-year (cubic feet per second)

Location	FCWS Existing Q_{100}	FCWS Future Q_{100}	DBPS Existing Q_{100}	DBPS Future Q_{100}	Area (sm)	DP
JCC Outfall	22,000	31,000	22,094	31,986	67.11	DP-J1
Ohio Avenue	22,000	31,000	22,139	32,149	66.11	DP-J3
Drennan Road	4,300	8,100	5,881	10,248	14.84	DP-J25
SH-94	1,700	3,500	5,031	7,135	9.62	DP-J31

Table III-8B

Comparison to Fountain Creek Watershed Study (FCWS) at Ohio Avenue Gauge

2-yr, 5-yr (cubic feet per second)

Flood Event	FCWS Uncalibrated	FCWS Calibrated	Ohio Ave. Gauge	DBPS 6-hr Calibrated	DBPS 12-hr Calibrated	2008 Uncalibrated
Q_2	7,170	302	458	113	151	2,322
Q_5	13,966	1,785	1,460	441	775	5,633

The Fountain Creek Watershed Study [FCWS] model was calibrated by adjusting the Initial Abstraction (Ia) value for all sub-watersheds. Initial Abstraction defines the amount of precipitation that must fall before surface excess results. The default model value is 0.2, and the FCWS adjusted this to 0.65. The value of 0.65 was determined by trial and error until the desired peak flows were achieved at the gauge. The goal of the FCWS focused entirely on peak flows and did not consider calibrating volumes. This method of calibration would appear to be arbitrary and has no technical justification for its application other than the removal of volume and lowering of peak flows. This method may not be appropriate in light of the 2005 storm event, which had an average watershed rainfall of only 0.50-inches and produced a 3.5-year peak flood event with a peak flow of 869 cubic feet per second. This has led the DBPS to further investigate a more definable and technically justifiable calibration procedure based upon actual rainfall data and watershed characteristics to develop a model to better reflect the recorded gauge data for more frequent flood events.

3.10 Results of Analysis

2-year and 5-year Results

Results of the Jimmy Camp Creek hydrology analysis were separated between the frequent flood events (2-year and 5-year) and the rare flood events (10-year and 100-year). The original modeling effort followed the standard procedures as outlined in the City of Colorado Springs and El Paso County Drainage Criteria Manual. Results from this effort were favorable for the 10-year and 100-year events but were determined to be high for the more frequent events when compared to the gauged peak flows at Ohio Avenue.

Since the Jimmy Camp Creek watershed has a stream gauge with 30 years of record, the calibration effort was undertaken to better match the gauge analysis for the 2-year and 5-year flood events. In summary, the calibration effort can be outlined as follows:

1. Adjusted storm duration from 24-hour to 6-hour
2. Adjusted antecedent moisture condition from AMC II to AMC I
3. Adjusted Manning's roughness coefficient specific to tributary reaches to reflect timing to the gauge

The calibration effort produced a 2-year, 5-year model that results in less runoff volume and lower peak discharges that correlate to the gauged data. With a 30-year gauge record calibration of the 10-year and 100-year flow rates could not be determined as reliable; therefore the standard engineering procedures as outlined in the City of Colorado Springs and El Paso County Drainage Criteria Manual were applied.

10-year and 100-year Results

The hydrologic results of this study are believed to be accurate for the Jimmy Camp Creek watershed. The results obtained for the hydrology modeling compare well with previous studies. Individual sub-basin 100-year peak runoff rates were further analyzed on a cubic feet per second/acre basis for reasonableness. Table III-9 provides a summary of this evaluation. Typically in undeveloped watersheds, existing 100-year runoff rates can range from 0.5 – 1 cubic feet per second/acre. In fully developed, urban watersheds this range can increase to 1 - 4 cubic feet per second/acre depending on the intensity of the development. In general large watersheds, as this one, will only increase to the 1 to 2 cubic feet per second/acre range, while smaller tributaries and individual sub-basins can increase in the 2 to 4 cubic feet per second/acre range. Any individual sub-basins that were found outside of this range were reevaluated for errors, corrected if necessary, and recalculated to ensure the results are accurate and consistent.

Table III-9
Check of Results (100-yr)

Location	Existing (cfs/ac)	Future (cfs/ac)
Outfall to Fountain Creek	0.51	0.74
Peaceful Valley Road	0.63	0.95
Bradley Road	0.70	1.00
Drennan Road	0.62	1.08
Highway 94	0.82	1.16

Other nearby watersheds were also reviewed for 100-year comparisons on a cubic feet per second/acre basis. The 2003 DBPS for West Fork Tributary produced 1.5 cubic feet per second/acre for existing conditions and 2.1 cubic feet per second/acre for future development conditions. This is a 4.1 square mile basin. A more comparable drainage basin to Jimmy Camp Creek is the Sand Creek watershed, which has a drainage area of 54.1 square miles. The Sand Creek Drainage Basin Planning Study produced 0.49 cubic feet per second/acre for existing conditions and 0.75 cubic feet per second/acre for future, undetained conditions.

Table III-10 provides a summary of peak runoff rates at key locations throughout the study area. The table includes all frequencies analyzed for both existing conditions and future conditions. Table III-11 provides a summary of volumes at key locations throughout the study.

The increase in runoff volume between the existing and future development conditions is the direct result of the increase in impervious areas attributable to the urbanization of the watershed. The increase in volume is what needs to be mitigated for by the implementation of detention storage in the watershed, either on a regional or onsite basis. The greatest incremental increase in volume is realized for the more frequent storm events such as the 2-year and 5-year recurrence intervals. Day-to-day rainfall events that produce no runoff in the existing development condition can be expected to produce measurable runoff when the land is developed. The substantial increase in the runoff volume for the 2- and 5-year frequencies is largely the cause of channel instability, particularly in drainageways that have sand bed channels, typical of Jimmy Camp Creek and its major sub-tributaries.

Figure III-7 shows the storm hydrographs for the 100-year flood events at key locations throughout the study area for both existing and future development conditions. The locations are along Jimmy Camp Creek at the outfall and major road crossings. Additional hydrographs are provided for the major tributaries in Figure III-8. Presented of Figures III-9 through III-13 provide a schematic of the HMS model with design points, routing elements, diversions, and flood detention facilities.

3.11 Further Study

Some of the results of the analyses completed for the DBPS should be considered preliminary and further study is necessary. The NEXRAD analysis was completed for a limited number of storms and some beneficial conclusions have been established, however, additional work should be done to better understand the nature of storms affecting basin runoff and to better define appropriate model parameters. Additional consideration should be given to the following issues:

a. Using additional NEXRAD Analyses the characteristics of a typical design storm should be more thoroughly evaluated. This would include detailed, calibrated analyses of many historic storms to determine the most appropriate temporal distributions and durations.
b. The assumption of uniform spatial distribution of storms has been shown to be questionable however, it is unclear from the completed study how to appropriately consider spatial variations of storms, especially for the more frequent events. Additional NEXRAD analyses should be completed to better understand the spatial characteristics of storms.
c. The adjustment of basin parameters such as Curve Number and channel roughness can be better understood by additional efforts to calibrate the models to recorded stream data.
d. Additional effort should be completed to better understand flood conditions and storms that produce less frequent events. This may require the evaluation of storms beyond the basin boundaries of Jimmy Camp Creek since no major events appear to have occurred within the basin since 1994.
e. The relationship between rainfall return period and the runoff return period is in question. It appears that due to the limited spatial extent of frequent storms the associated runoff produced depends upon where it is measured within the basin. If a uniform rainfall distribution is not applied the definition of the rainfall to runoff relationship may need to be reevaluated.
f. The relationship between the average rainfall from a storm and its maximum rainfall should be better understood to help define a typical design storm and assign a return period.

IV. HYDRAULIC ANALYSIS AND FLOODPLAIN DESCRIPTION

4.1 Overview

Hydraulic analyses were conducted to determine the extent flooding along the major drainageways of the Jimmy Camp Creek watershed during a 100-year event assuming existing basin development conditions. The hydraulic analysis also focused on determining the capacity of existing hydraulic structures that may cross over the major drainageways of the Jimmy Camp Creek watershed. Field verifications of major roadway crossings and channel conveyance improvements were conducted and the general physical condition of the structure(s) noted. Finally an effort to "characterize" the existing major drainageway channel sections with respect to environmental resources and stream stability issues was conducted and is summarized in this section of the report.

Hydraulic analyses were conducted using the U.S. Army Corps of Engineers HEC-RAS program, version 4.0. Plan and profile drawings were compiled for the main drainageways of Jimmy Camp Creek and for the Corral, East Fork Jimmy Camp Creek, Strip Mine, Franceville and Marksheffel Tributaries using 2-foot contour interval topographic mapping. The drawings show the existing channel grade, major roadway crossings, 100-year discharge data, 100-year hydraulic grade line, 100-year flood boundary, stream characterization classifications, environmental resources and roadway crossings. Cross-section data for the floodplain analysis was obtained from two-foot contour interval planimetric topographic mapping. The vertical datum for the planimetric mapping is the National Geodetic and Vertical Datum (NGVD) of 1929. The primary source of mapping along Jimmy Camp Creek was taken from the City of Colorado Springs FIMS mapping and the major drainageways within the limits of the City of Fountain and the City of Colorado Springs. Two-foot contour interval planimetric mapping for the portions of the East Fork Jimmy Camp Creek that lie in El Paso County were obtained from private sources associated with the Lorson Ranch and Rolling Hills Ranch land development projects. The capacity of the major roadway crossing structures has been estimated using the HEC-RAS water surface profile data. The hydraulic analysis for Jimmy Camp Creek was initialized by assuming a 100-year water surface at the confluence with Fountain Creek of 5499.5 as obtained from the El Paso County Flood Insurance Study profile. Manning's roughness values for use in modeling the 100-year floodplains were determined through field reviews and photographs. Representatives from the NRCS also provided comments on the roughness values as applied in the hydrologic and hydraulic modeling. The 100-year future baseline hydrologic conditions (i.e., without proposed facilities) and the 100-year existing baseline hydrologic condition profiles were compiled. The floodplain information shown on the drawings has been used primarily for the identification of flood prone areas along the major drainageways and to aid in the evaluation of alternative channel treatments. **The floodplain data contained herein is not intended to replace the information presented in the City of Fountain, City of Colorado Springs and El Paso County Flood Insurance Studies, but should be used as a planning tool for urban drainageway development projects.**

4.2 Reach Delineation

Reaches were delineated for various segments of Jimmy Camp Creek and its major tributaries. The reaches were determined based upon the existing physical condition of the low flow, floodplain, and overbanks along the drainageways. The reach limits are shown in Figure IV-1. Descriptions have been prepared for each reach by means of field visits, which were conducted to ascertain more site-specific information related to existing drainageway conditions. An environmental review of the major reaches was also conducted. The delineation of reaches was carried in order to assist in the evaluation of channel treatments and eventually in the selection of the most feasible plan(s) for long-term stability of the major drainageways within the watershed.

In some cases limits of a planning reach were determined based upon the existing roadways or jurisdictional limits or in other cases upon physical condition of the low flow, floodplain, and overbanks along the drainageways. The reach limits established for the major flow paths are as follows:

Jimmy Camp Creek

Reach J1: Fountain Creek to Link Road
Reach J2: Link Road to Confluence with East Fork Jimmy Camp Creek
Reach J3: Confluence with East Fork Jimmy Camp Creek to Corporate Limits
Reach J4A/B: Corporate Limits to Drennan Road
Reach J5: Drennan Road to SH-94
Reach J6: SH-94 to proposed Jimmy Camp Creek Reservoir Site
Reach J7: Proposed Jimmy Camp Creek Reservoir to upstream limits of floodplain delineation.

East Fork Jimmy Camp Creek

Reach EF1: Confluence of Jimmy Camp Creek to El Paso County Limits
Reach EF2: El Paso County Limits to Meridian Road
Reach EF3: Meridian Road to Upstream Limits of Floodplain Delineation

Marksheffel Tributary

Reach M-1: Confluence with Jimmy Camp Creek to Drennan Road

Franceville Tributary

Reach F1: Confluence with Jimmy Camp Creek to Drennan Road
Reach F2: Drennan Road to Meridian Road

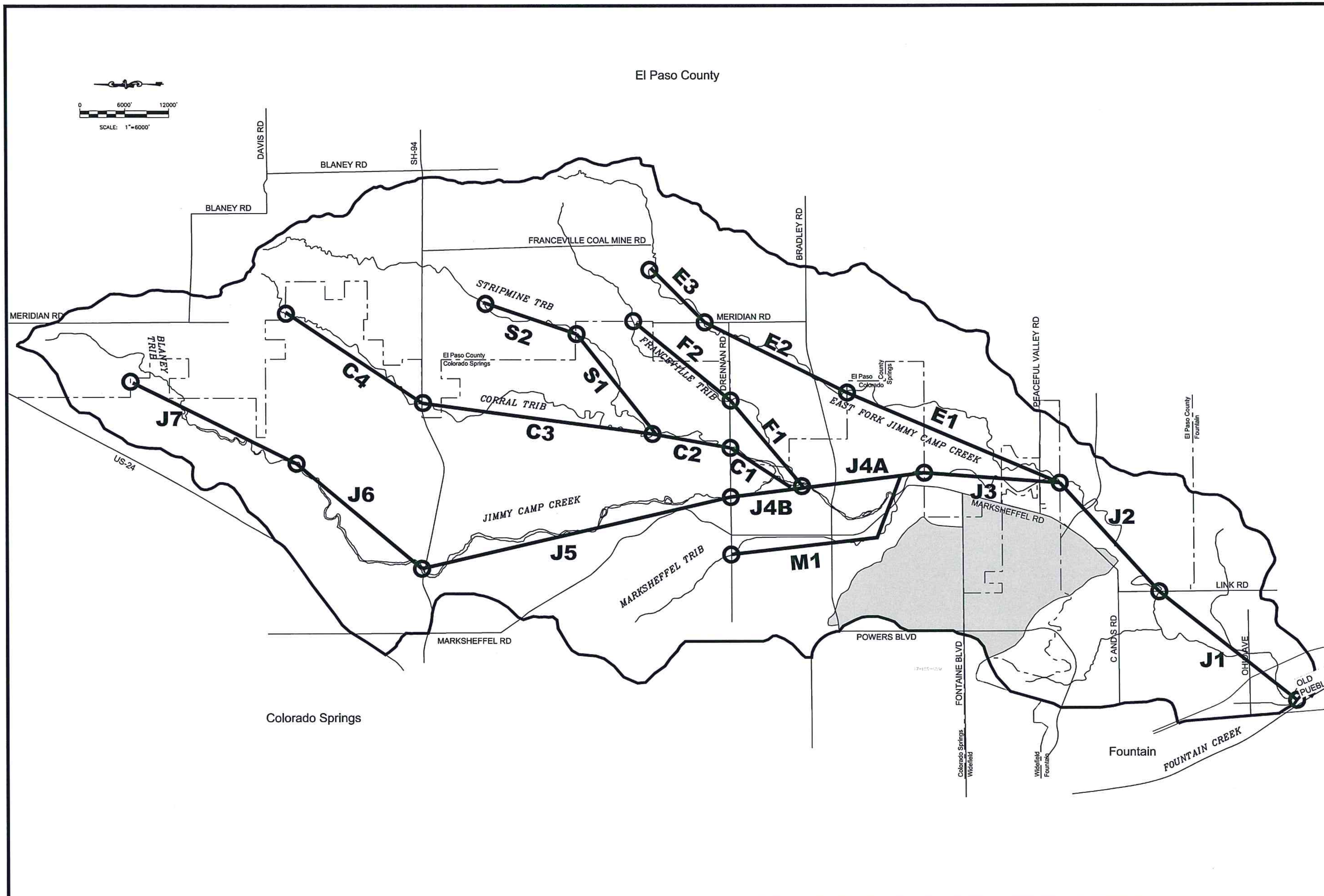
Corral Tributary

Reach C1: Confluence with Jimmy Camp Creek to Drennan Road
Reach C2: Drennan Road to Confluence with Stripmine Tributary
Reach C3: Confluence with Stripmine Tributary to SH-94
Reach C4: SH-94 to Upstream Limits of Floodplain Delineation

Stripmine Tributary

Reach S1: Confluence with Corral Tributary to El Paso County Line
Reach S2: El Paso County Line to Meridian Road

The reaches described above were used in analysis of conceptual alternatives along the major drainageways and flow paths of the Jimmy Camp Creek watershed. No reaches were delineated within the West Fork Jimmy Camp Creek sub-watershed and this area is basically fully developed at this time and has stormwater collection systems that are functioning adequately. Presented on Table IV-1 is a summary of the



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JIMMY CAMP CREEK WATERSHED
DRAINAGE BASIN PLANNING STUDY
MAJOR DRAINAGEWAY REACH DELINEATION
CITY OF COLORADO SPRINGS, COLORADO

Project No.:	14008
Date:	SEPT. 2014
Design:	
Drawn:	BJW
Check:	
Revisions:	

FIGURE
IV-1

key characteristics for each reach that has been delineated for the purposes of alternative evaluation. Drainageways serving areas of at least 100-acres will be studied in detail as part of the conceptual planning process, however the reaches determined and explained in this section were developed so that alternatives for the treatments of the major drainageways could be advanced in a systematic way. Detailed topographic mapping is available only for the major drainageways shown on Figure IV-1.

4.3 Hydraulic Structure Inventory

As part of the field investigation, the existing drainage facilities were verified and inventoried. The size, type, and general hydraulic condition were recorded for bridges, culverts, detention basins and miscellaneous drainage features that existing along the major drainageways were inventoried. Hydraulic capacities were estimated for the culverts and bridges over the major drainageways. An inventory of the major structures is presented in Table IV-2. It was assumed that the maximum hydraulic capacity of a roadway crossing was reached when the hydraulic grade line equaled the road surface.

Very limited segments of the major drainageways in the Jimmy Camp Creek watershed have been improved and most of the banks are unlined or naturally lined with vegetation. Where bank linings have been built they exist mostly at the approach and outlet sides of roadway crossings. The 100-year channel capacities were estimated using the HEC-RAS computer program.

One detention basin now exists within the watershed. The detention basin along the Marksheffel Tributary has adequate storage volume to route the 100-year existing and developed discharge downstream to the mainstem of Jimmy Camp Creek.

4.4 Watershed and Flood History

Disagreement has taken place as to the origin of the name "Jimmy Camp Creek," but a consistent thread throughout the years is that an early trapper-trader named Jimmy was killed near the spring at the headwaters of the Jimmy Camp Creek Basin. The legendary campsite was located along an ancient route that connected the Arkansas and Platte Rivers called "The Old Divide Trail," "The Trappers Trail," "The Cherokee Trail," or "Jimmy's Camp Trail" among other names. Jimmy, most likely James Daughtery, appears to date from the early 1830s. The trail and camp had long been used Native Americans by the time the trapper-traders had arrived.

Comanche, Kiowa, Arapahoe, Cheyenne, and Sioux tribes are thought to have lived in the area at times. On-going archaeological excavations by the University of Colorado at Colorado Springs have uncovered evidence documenting prehistoric use during the Developmental Period with radiocarbon dates of about 655 A.D., 1650 A.D. and a third in the range of 1270 A.D. to 1400 A.D. Early hunters migrating into North America may have used the ancient route along the watercourse for thousands of years.

One of the earliest published reports along the trail was by Rufus Sage in 1842 who stated in his journal during his northward travel that "we reached an affluent of Fontaine qui Bouitte, called Daugherty's creek...Our place of stay is a sweet little valley enclosed by piney ridges...the creek derives its name from Daugherty, a trader who was murdered upon it several years ago." Subsequent to Sage's journal entry, many other parties were documented to use the route up the basin. Among them are Lt. John C. Fremont (1843), Francis Parkman (1846), a band of Mormon emigrants (1846-7), bands of Cherokees (1849 and 1850), the Loring-Marcy Expedition (1857-58), numerous cattle drives such as the Goodnight-Loving, and the many gold seekers of 1858-59. In general, many people made use of the availability of wood, water and grass on the easiest crossing of the Platte-Arkansas Divide.

Settlement during the homesteading era produced many farms and ranches in the basin. Prior to the fencing movement, an annual round-up known as the "Jimmy Camp Round-up" occurred and herded the cattle toward Corral Bluffs on the east edge of the basin to separate the cattle. Well into the 1900s, farmers and ranchers traveling to Colorado Springs from eastern El Paso County would camp on their way into and out of the city where an old county highway passed the historic springs and calling their camp "Farmer's Rest." Many of the ranches too small to be viable became abandoned and were commingled into larger, more viable spreads such as the Banning-Lewis Ranch.

In addition to ranching and dairy farms, coal, sand and gravel mining have occurred in the basin. Railroads traversed the basin to support the dairy, ranching and mining industries, with spurs such as the one to the Franceville Coal Mine. The Fountain Mutual Irrigation Canal and Chilcotte Ditch No. 27 originating at Fountain Creek supplied irrigation water to fields around the City of Fountain.

Currently, a large portion of the basin has been annexed into the City of Colorado Springs and will be converted to mixed urban uses. A similar situation is predicted to occur in the City of Fountain at the downstream end of the basin. The area remaining in El Paso County will be subject to urbanization, however some of the upper reaches that lie within El Paso County will retain rural residential uses

Throughout recorded history, the Jimmy Camp Basin has always experienced severe weather events with wide fluctuations that include drought, hail, floods and devastating snowstorms. With low population density in the basin prior to the last twenty years, endangerment of lives and damage to property was limited and rarely reported. Infrequent yet potentially dangerous precipitation events need to be kept in mind while planning for development in this basin.

Flooding is mainly occurs in the summer months of May to August during intense rain events of several days duration when a warm, moist air mass from the Gulf of Mexico collides with a colder air mass from the north. Although frequently severe, isolated summer thunderstorms rarely cause major flooding as they tend to be limited in area and duration.

Heavy snowstorms and rainstorms are caused by similar meteorological patterns, but snowstorms do not typically cause floods as peak flows are attenuated by snowmelt. A few early accounts snowstorms will be conveyed here to illustrate the intermittent, but severe events that have taken place in the past. During the Loring-Marcy military expedition of 1858, a snowstorm started on April 29 on "a mild and pleasant spring day, with no appearance of bad weather, but as night approached it became cloudy, and about dark a snowstorm set in accompanied by a violent gale of wind from the north, which increased until it became a perfect tempest, and continued without cessation for sixty hours." By May 1, one man froze to death and over three hundred mules and horses stampeded with many dead or missing. Twenty years later, a similar storm struck during the Jimmy Camp Roundup of 1878 and "snow was eleven feet deep in the Corral (Bluffs), and sheep were dug out alive after being buried for two and even three weeks."

The June 18, 1965 flood is the flood of record in El Paso County. As much as 14 inches of rain fell over several days. Hailstones near Fountain were said to be as large as tennis balls. The flow at Jimmy Camp Creek was estimated to be 124,000 cubic feet per second at a point about 4.5 miles upstream from the confluence with Fountain Creek, however no stream gage recordings are available for this event. Considerable damage to roads and bridges occurred in the sparsely populated area. In the City of Fountain, Ohio Avenue washed out along with the railroad trestle. Santa Fe Avenue was overtopped and gullies formed on the approaches.

A large regional flood also occurred on May 30, 1935 after several days of rain. As in the 1965, the majority of damages were to agriculture, roads and bridges. In the summer of 1972, two separate flood events caused damage in the basin. The first event of July 18th, there were reports of two- to five-inches of rain in the Franceville Tributary causing about $100,000 damages to roads and bridges. State Highway 94 was closed due to bridges being washed out. Later in the summer on August 3rd, a flood did an additional $50,000 in damages to bridges and isolated eight families east of Jimmy Camp Creek on Peaceful Valley Road.

The U.S.G.S. installed a stream gage near the mouth of Jimmy Camp Creek in 1976. Reported in the Hydrology chapter of this report were the results of the statistical analysis of the USGS gage data at Ohio Avenue. Review of gage records for water years 1976-2006 indicate peak flows of 4,810 cubic feet per second and 4,530 cubic feet per second for 1994 and 1995 respectively and 3,600 cubic feet per second in 1985. During the 31 years of record, a peak recorded peak flow of over 1,000 cubic feet per second occurred seven times. Flood history clearly indicates that a potential for flash flooding is present in the Jimmy Camp Creek Basin and will increase as urbanization continues.

4.5 Floodplains

The location of the 100-year floodplain is important since it denotes the limit of allowable encroachment. Often times the 100-year floodplain contains the higher quality riparian and wetland habitat areas. These areas are desirable areas to preserve when focusing on the alternative planning process. It is recommended that the land which contains the main channels of Jimmy Camp creek watershed have the 100-year floodplain limits verified at the time of development, using the hydrology summarized herein as part of the initial steps of land development planning. For areas where no floodplains have been delineated, either in this report or in the Flood Insurance Study, the 100-year floodplain should be required to be determined using methods similar to those applied in this study.

Several studies have been completed within the watershed and have been used for flood hazard information and floodplain management. These studies include:

Floodplain Information Report, Fountain and Jimmy Camp Creeks, Colorado Springs and Fountain Colorado, U.S. Army Corps of Engineers, March 1973.

Flood Hazard Analysis, Jimmy Camp Creek, East Fork Jimmy Camp Creek, Franceville, Corral and Strip Mine Tributaries, City of Fountain and El Paso County, prepared by the USDA Soil Conservation Service, 1975.

Floodplains for the 100-year existing condition discharge have been delineated for Jimmy Camp Creek, the East Fork Jimmy Camp Creek and the Corral, Franceville, Stripmine, and Marksheffel tributaries. The floodplain was estimated in order to assess where hydraulic inadequacies exist along the major drainageways. The analysis assumed rigid boundary conditions to exist along the channel cross-sections. The field inventory supplied roughness and bridge opening data for use in the HEC-RAS modeling.

The most significant areas of the existing flood hazard occur along the mainstem Jimmy Camp Creek in reaches 1, 2, and 3. The floodplain is wide and shallow for the most part, with the most extreme velocities occurring in the transitions in and out of roadway crossings. The crossing at Peaceful Valley Road is not of sufficient capacity to keep the 100-year discharge from overtopping the roadway. This combined with the unlined banks downstream of Peaceful Valley Road causes an extremely wide (1,500 to 2,000 feet) and shallow floodplain to result. Wide floodplains also occur between the Ohio Avenue and D&RGW railroad crossing. The limited channel capacity in this segment of Jimmy Camp Creek forces the 100-year discharge out of the low flow area of the drainageway. Upstream of Peaceful Valley Road single-family residential structures encroach very close to the 100-year floodplain, otherwise there are a very limited number of habitable structures that are presently lying within the existing condition 100-year floodplain of Jimmy Camp Creek.

Due to the limited channel capacity of the Stripmine Tributary flow split occurs in the lower reach of this drainageway. Historic photographs and geologic information indicate that the cause of the flow split has been from flood flows heavily laden with sediment. Sediment .that is carried by the drainageway drops out as the Stripmine Tributary nears its confluence with the Corral Tributary. This causes a very wide, shallow and uncontrolled floodplain with two distinct outfall points to the Corral Tributary.

At Drennan Road the Franceville Tributary has been diverted from its historic path to the Corral Tributary. Since no crossing under Drennan Road was every constructed to carry the Franceville Tributary along its historic path, a flow spit occurs and the majority of the flow will travel west along the north side of Drennan Road and enter the Corral Tributary. Some residual flow is predicted to pass over Drennan Road and travel south in a wide and shallow uncontrolled manner and eventually outfalling to Corral Tributary downstream of Drennan Road.

The floodplain of the East Fork Jimmy Camp Creek is also very wide and shallow in some places, particularly in the segment of the drainageway south of Drennan Road. In this location the historic channel is very poorly defined and at some location no perceptible low flow area can be seen. This is the case through the Rolling Hills Ranch and portion of the Lorson Ranch properties. Near the confluence with Jimmy Camp Creek, the East Fork passes through and over the embankment of an existing lake used for irrigation of a golf course. Similar to Jimmy Camp Creek, there are presently no habitable structures that lie within the 100-year floodplain of the East Fork.

The roadway crossings over the major tributaries of the Jimmy Camp Creek watershed have adequate capacity to convey the estimated 100-year discharge under the roadway. There are however several exceptions to this. The crossing at Peaceful Valley Road at Jimmy Camp Creek and over the East Fork do not have sufficient capacity to convey the 100-year discharge and it is predicted that the roadway would be overtopped. The existing culvert at Bradley Road and the East Fork convey only 85 percent of the estimated 100-year discharge, however improvements to the channel approach and outlet transitions would increase the capacity of this culvert and result in the preventing the roadway from being overtopped. The exiting culvert under Meridian Road and the East Fork are also inadequate and should be upgraded if Meridian Road is improved in the future. Finally the culverts under Marksheffel Road that carry the Marksheffel Tributary to the Jimmy Camp Creek drainageways are under capacity and the roadway would be overtopped if a 100-year release from the existing detention basin was to occur.

4.6 Environmental Resource Review

Presented in this section is an environmental resource inventory for the major drainageways in the basin including a description of the wetland resources, wildlife habitats and endangered species issues that may be relevant during design and implementation of major outfall systems.

Topographic, soil survey and wetland inventory maps were used to indicate potential wetland resources prior to field visits in the summer and fall of 2006 to verify the current condition of the vegetation and hydrology. Aerial photography was also used to evaluate areas where access was prohibited. Environmental resources were mapped on the FIMS database obtained from the City of Colorado Springs Utility Department.

Information presented is for planning purposes only. Prior to construction of proposed outfall systems, detailed wetland delineation will need to be done to determine the precise boundaries of jurisdictional wetlands and waters of the U.S. that will be subject to regulation by the Army Corps under Section 404 of the Clean Water Act.

Basin Description

The Jimmy Camp Creek Basin drainage system is composed of the mainstem along with four major tributaries: the West Fork, the East Fork, the Franceville and the Stripmine Tributaries. The mainstem of Jimmy Camp Creek is about 21 miles long starting at an elevation of about 6900 feet of elevation and outfalling to Fountain Creek at an elevation of about 5500 feet. The terrain of the basin is predominately gently rolling hills formed of wind blown sediments and small areas of forested sandstone outcrops in the headwaters. Topography varies from moderately sloping in areas where shallow wind blown sediments overlay shale to steeply sloping topography where the sandstone outcrops.

Stream classification

With the exception of the downstream portion of the basin near the City of Fountain, the principal land use in the basin is grazing. As the majority of the basin is in an undeveloped state, the stream classification for natural rivers can be applied per Rosgen (1994). Level II classification is shown on each drawing. The significance of applying a geomorphological classification is that if parameters such as sinuosity, entrenchment, stream gradient, etc. of the natural condition are replicated in the proposed channel design, the channel will likely be stable.

The Jimmy Camp Creek and tributaries can be classified as the "C" type. The "C" stream is typically located in valleys formed of alluvial deposits with a well-developed, slightly entrenched floodplain. Width-to-depth ratios and sinuosity are moderate to high. The stream gradient is low and bed material is typically comprised of coarse sand. Point bars are characteristic in the stream. Channels of the "C" type stream de-stabilize rapidly when cumulative changes are made that alter bank stability, flow regimes and watershed conditions.

In the upper reaches of the mainstem and Corral Tributary where sandstone outcrops are encountered, the stream classification changes to a "B" type stream. Compared to a "C" type steam, the "B" type stream has a narrower valley that limits the development of a wide floodplain. Entrenchment is greater with a "B" type stream and sinuosity is lower. The stream gradient is also significantly steeper. The "B" type stream tends to be more stable than the "C" type.

Wetland Hydrology

Jimmy Camp Creek is a perennial waterway up to about Link Road after which it becomes an ephemeral stream for the most part. A U.S.G.S. stream gage is located one and a half miles above the mouth with 31 years of record. According to the gage, the mean flow rate on a typical winter day is less than one cubic foot per second. On a typical summer day, the mean flow rate at the gage is about three cubic feet per second. For the years of record, several times peak flows of over 4,000 cubic feet per second have been recorded. It should be kept in mind that Jimmy Camp Creek has a history of severe flooding with a record flood on June 17, 1965 estimated at 124,000 cubic feet per second.

From Link Road to Highway 94, the flow of Jimmy Camp Creek is generally ephemeral, i.e. it flows only in response to precipitation events. Along this stretch, the channel is a dry, unvegetated wash and lacks a surficial water table necessary to support wetlands. In places along this segment, there is a shallow water table adequate to support riparian ecosystems.

Above Highway 94, there is an intermittent flow emanating from the historic springs of the headwaters. The sandstone outcrops function as a reservoir rock to provide a small baseflow to the creek, adequate in places to support an emergent wetland channel.

Two irrigation canals dating back to the 1800s, the Fountain Mutual Irrigation Canal and the Chilcotte Ditch No. 27, originate at Fountain Creek and traverse the lower portion of the watershed. Historically, the ditch waters were used to irrigate hay meadows. The water rights associated with the ditches are being converted to domestic water supply and golf course irrigation. Return flows from irrigated areas have enhanced the natural flow of the lower portion of the basin, as has residential lawn irrigation. When the basin becomes developed, base flow to the creek can be expected to increase when imported and ground water is used to irrigate landscaping.

The major tributaries, Stripmine, Franceville, Marksheffel and the East Fork are all ephemeral. They typically have a clearly defined channel with an ordinary high water mark in the upper reaches where steeper gradients and more stable bedrock are encountered. Where the terrain flattens out and the substrate becomes less consolidated, the channels are unclear. Most of these tributaries have no clear connection to the mainstem and display evidence of lateral migration within historic times.

Drainageway Soil Characteristics

The soils of the drainageways are in three SCS mapping units. The majority of the length of the mainstem is in the Ellicott loamy sand series with the upper stretches on the Stapleton Bernal sandy loam series. The Stripmine and East Fork Tributaries are also located in the Ellicott series. The Ellicott soil is a deep, somewhat excessively drained found on terraces and floodplains and formed from coarse sands derived from the Laramie Fox Hills Sandstone of the headwaters. The Stapleton Bernal unit of the upper reaches is deep and well-drained soil also derived from Arkosic sandstone. The Ustic Torrifluvents unit is present on the Corral Tributary and a small portion of the main stem. This unit is a well-drained soil of terraces and floodplains

Hydric soils are defined as a soil that is formed under conditions of saturation, flooding or ponding for a suitable period of time during the growing season to develop anaerobic conditions in the upper portion of the soil's layer. The significance of hydric soils along the drainageway is that they are indicators of wetlands and also indicate areas of seasonally high groundwater table (within one-foot of the surface). None of the soil types found in the basin are hydric according to the El Paso County list of hydric soils. Areas of hydric inclusions of Pleasant soils in depressions or inclusions of fluvaquentic haplaquolls in drainage swales may be present in small, localized areas. Due to the nature of the course grained and thus well-drained sediments there are no strong indicators of hydric soils in the drainages, although small areas may be present.

Vegetation

Five categories of native vegetation were found in the study area: western short grass prairie, emergent wetlands, willow wetlands, riparian woodland and pine/juniper woodlands. By and large, the most common native vegetation found is the western short grass prairie dominated by blue grama grass (*Bouteloua gracilis*). Associated graminoid and herbaceous species are sideoats grama (*Bouteloua curtipendula*), prairie sandreed (*Calamovilfa longifolia*), inland saltgrass (*Distichlis stricta*), little bluestem (*Schizachyrium scoparium*), big bluestem (*Andropogon gerardii*), annual sunflower (*Helianthus annuus*), and wild gourd (*Cucurbita foetidissima*). Weed species are kochia (*Kochia scoparia*), flixweed (*Descurania sp.*), musk thistle (*Carduus nutans*), Russian thistle (*Salsola collina*), hoary cress (*Cardaria draba*), Canada thistle (*Cirsium arvense*), mullein (*Verbascum thaspus*) and teasel (*Dipsacus sylvestris*).

Small areas of emergent wetlands are present. These are dominated by cattails (*Typha latifolia* OBL) with minor amounts of bulrush (*Schoenoplectus lacustris* OBL), spikerush, (*Eleocharis sp.*), scouring rush (*Hippichaete laevigata* FACW), three-square (*Schoenoplectus pungeons* OBL), rush (*Juncus balticus* OBL), sedges (*Carex sp.* >FACW) and curly dock (*Rumex crispus* FACW). The emergent wetlands are commonly intermixed with willow wetlands dominated by sandbar willow (*Salix exigua* OBL). These plant communities are found within a few vertical feet of the stream channel, low floodplains and terraces or swales where irrigation water is abundant.

The riparian woodlands present in the basin are dominated by the plains cottonwood (*Populus deltoides*) along with native shrubs such as chokecherry (*Prunus virginiana*), wildrose (*Rosa woodsii*), snowberry (*Symphoriocarpus occidentalis*) and golden current (*Ribes aureum*). Introduced species such as tamarisk (*Tamarisk ramossima*), Russian olive (*Elaeagnus angustifolia*), and Siberian elm (*Ulmus pumila*) are commonly found in this riparian ecosystem. Tamarisk in particular is an extremely noxious weed species capable of replacing all other native species by inhibiting regeneration of the native canopy. Tamarisk also lowers the water table and negatively impacts wetlands.

Ponderosa pine/juniper woodlands inhabit the sandstone outcrops of the headwaters. Associated understory shrub species are mountain mahogany and snowberry, with grass cover dominated by blue grama grass.

Qualitatively, the vegetation of the basin varies from high quality to low quality. On the Banning-Lewis Ranch, most of the rangeland is of high quality and covered with native grasslands, indicating good rangeland management over the years, although there are some degraded areas near homesteads where livestock were concentrated. In the lower portions of the basin that were subdivided into smaller acreages, the vegetation condition is typically very poor due to overgrazing and human use. By and large, the greatest degradation to the vegetative cover is the widespread presence of tamarisk in the riparian areas of the lower basin. Tamarisk is the primary non-native phreatophyte of concern in Colorado. In fact, former Governor Bill Owens issued Executive Order #D00203 in 2004 on the comprehensive removal of tamarisk and restoration of Colorado's native riparian ecosystems. Other weed species that have been mentioned previously are subject to the State of Colorado weed control regulations and can also be expected to proliferate when the ground is disturbed.

Jurisdictional Wetland and Waterways

The mainstem and all major tributaries of Jimmy Camp Creek mapped on the floodplain drawings are "blue lines" on the U.S.G.S. map and will need to be evaluated in regards to regulation of jurisdictional waters of the U.S. and adjacent wetlands by the Corps of Engineers. Plans to discharge dredged or fill material within the ordinary high water mark or adjacent wetlands may require a Department of the Army Permit under Section 404 of the Clean Water Act.

Irrigation ditches that empty into jurisdictional waters are considered jurisdictional waters of the U.S. subject to regulations, as are ponds and wetlands fed by canals. Drainage separation structures in the vicinity of the canals may also need a Department of the Army permit.

Potential ESA Issues

In regards to potential endangered species issues the current recommendation of the United States Fish and Wildlife Service U.S.F.W.S, is to compare the habitat of the study area with that required for the federally listed endangered (E) and threatened (T) species on the El Paso County Endangered Species List. The list currently contains the six following species: bald eagle (*Haliaeetus leucocephalus* T), black-footed ferret (*Mustela nigripes* E), greenback cutthroat trout (*Salmo clarki stomias* T), Mexican spotted owl (*Strix occidentalis lucida* T), Preble's meadow jumping mouse (Zapus hudsonius preblei T), and Ute ladies tress orchid (*Spiranthes diluvialis* T).

With the exception of the Preble's meadow jumping mouse, each one of these species have special habitat requirements that are not met in the study area, such as open lake shorelines, perennial water, moist wet meadows, riverine sandbars or mudflats, high altitude habitat, cliffs, forested vegetation, thick riparian vegetation, or lake or river systems. The range of the Preble's meadow jumping mouse has never been found to extend this far south although habitat suitable does in very limited and isolated places. Consistent with the U.S.F.W.S endangered species habitat requirements, no endangered threatened species is likely to occur in the area.

Wildlife Habitat

Wildlife species observed during field visits turkey, antelope, rabbits, skunks, and raptors. There was also evidence of coyote and fox. Additional species such as migratory songbirds, deer, and various rodents would be expected to also be present. All of the native ecosystems within the study site provide wildlife habitat dependant on the requirements of each species, but more valuable are the wetlands and riparian communities due to their smaller size. Revegetation with native species similar to those present today is important to preserving and increasing wildlife opportunities.

Conclusions

As shown on the accompanying Floodplain, Environmental Resources and Stream Classification Maps, areas marked as wetlands, waters of the U.S., open water, and irrigation ditches may be subject to U.S.A.C.E. regulations. Riparian ecosystems impacted in conjunction with permitted activities may also need replacement. Detailed wetland delineation will need to be done in areas where drainage outfall systems are proposed in potential jurisdictional areas and evaluated in relation to permitting requirements in affect at the time of construction.

4.7 Stream Characterization

Mussetter Engineering, Inc. (MEI), under subcontract to Kiowa Engineering Corporation, conducted this initial assessment of bankfull channel capacity at 10 locations within the Jimmy Camp Creek watershed as depicted on Figure IV-2). The purpose of the assessment was to determine whether there is a consistent relationship between the channel capacity and a flow of a specific frequency (1- to 2-year recurrence interval) or duration (Leopold et al., 1964; Dunne and Leopold, 1978; Moody et al., 2003). However, in arid and semi-arid regions of the U.S., there is less likely to be a direct correlation between the channel capacity and a flow of a given recurrence interval (Baker, 1977; Schumm, 1977; Wolman and Gerson, 1978; Williams, 1978; Graf, 2002) because of the absence of continuous interaction between the flows and the channel boundary materials.

Hydrology

Flood-frequency and flow-duration curves were developed from the annual peak flow data (1976-2006) and the mean daily flow records (1976-2006), respectively, from the Jimmy Camp Creek at Fountain, Colorado (USGS Gage No. 07105900) that is located immediately downstream of the Ohio Street crossing. At the gauging station, the contributing drainage basin area of Jimmy Camp Creek is approximately 66.4 square miles. The results of the flood flow frequency analysis are summarized in chapter 3 of this report.

Upstream of the Link Road crossing, Jimmy Camp Creek and its tributaries (Corral, Strip Mine, Franceville, East Fork Jimmy Camp Creek and Marksheffel tributaries) have ephemeral flow channels that can generally be described as sand bed, aggrading with a tendency to braid. Locally, areas of accumulation of sediment in valley floor fans have incised and are exporting the stored sediments downstream where they are being deposited in new valley bottom fans that have braided channels (e.g., Franceville Tributary upstream of Drennan Road). Downstream of Link Road, Jimmy Camp Creek tends to be a single-thread sand-bed channel that is incised and has perennial flow, primarily due to the presence in the lower basin of the Fountain Mutual and Chilcotte Ditches. Seepage losses from the ditches and tailwater discharge are the causes of the perennial flow in the lower basin and are most likely responsible for the presence of the dense riparian vegetation along the channel downstream of Link Road. Based on the flow-duration curve at the gage, 10 cubic feet per second is equaled or exceeded less than 1 percent of the time (approximately 4 days per year) and for greater than 50 percent of the time, the flow is less than 1.6 cubic feet per second as shown on Figure IV-3. At the Ohio Avenue gauging station, based on the flood-frequency curve the 2-year peak flow is about 500 cubic feet per second the 5-year peak flow is about 1,500 cubic feet per second, and the 10-year peak flow is about 2,800 cubic feet per second as shown on Figure IV-4. In contrast, HEC-HMS modeling of the basin as summarized in the Fountain Creek Watershed Hydrology study (2006) developed 2-, 5- and 10-year undeveloped peak flows at the gauging station of 300, 1,800 and 4,100 cubic feet per second, respectively. The significantly reduced magnitude of the 10-year peak flow might well be due to the ability of the ditches to abstract significant amounts of flood flows from the channel of Jimmy Camp Creek. Upstream of the USGS gauging station, there are no gages, and therefore, drainage basin area was used as a surrogate for flow (Dunne and Leopold, 1978). Drainage areas upstream of each of the 10 measurement sites are provided in Table IV-3.

Data collection

The 10 sites within the Jimmy Camp Creek drainage basin were selected to encompass a reasonable distribution of drainage areas in order to test the hypothesis that the capacity of the channels was related to a particular flow magnitude that was in turn related to the contributing drainage area. Site selection was constrained to some extent by land access, but as shown in Table IV-3, the drainage areas upstream of the selected sites range in size from 0.7 to 67.2 square miles. At each of the sites, a straight, single-channel reach with a reasonably well-defined channel cross section that was likely to contain the full range of low to moderate flows was selected for survey. Reach lengths varied from 100 feet (Site 8) to 417 feet (Site 7). Prior to surveying the site, channel cross sections were identified and the top-of-bank stations on both sides of the channel were identified and the top-of-bank stations on both sides of the channel were identified with pin flags. Topographic (bank heights, materials, angles and continuity) and botanical (lower limits of perennial vegetation species) criteria were used to establish the top-of-bank stations at each cross section. The channel capacity is equivalent to the term "bankfull capacity" but there is no a-prior assumption of return period associated with use of the term. A typical cross-section depicting how the bankfull capacity was defined is presented on Figure IV-5. A thalweg profile and at least four cross sections were surveyed at each site with a Leica Model 1230 RTK-GPS rover unit that has a nominal accuracy of 0.3 feet both vertically and horizontally. The cross-section profiles were extended beyond the pin flags to encompass topographically high elevations that would contain higher magnitude flows. A reach photograph and the surveyed cross sections with water-surface elevations derived from HEC-RAS models for each site are provided in the Technical Addendum to this DBPS.

Data analysis

The surveyed cross sections at each of the ten sites were coded into individual un-calibrated HEC-RAS models and a range of flows was run for each of the sites. Normal-depth downstream boundary conditions were assumed for each model. Bed slopes ranged from about 0.01 (53 ft/mi) at the upstream sites to 0.003 (16 ft/mi) just upstream of the Fountain Creek confluence (Table IV-3). Manning's n-values that were used for the channel varied from 0.028 to 0.032 based on previous experience with similar channels. Thalweg, field-identified top-of-bank and water-surface profiles for the 10 sites are provided in the Technical Addendum to this DBPS.

At each site the discharge associated with the water-surface profile that best matched the field-identified top-of banks profiles was considered to represent the channel capacity. Computed channel capacity values ranged from 40 cubic feet per second at Site 9 to 525 cubic feet per second at Site 7 (Table IV-3) and site-averaged cross section areas ranged from 11 square feet (Site 2) to 94 square feet (Site 7).

The relationship between the average cross-sectional area of the channel below the field-identified top-of-banks at each site and the contributing drainage area is shown in Figure IV-6. Inclusion of all 10 sites into the arithmetic relationship provides a very weak positive relationship (R^2 = 0.0052). Exclusion of the two outliers (Sites 1 and 7) does improve the strength of the relationship to some extent (R^2 = 0.22), but semi-logarithmic and logarithmic functions do not improve the strength of the relationship. The two outlier sites have a common characteristic—both sites locally have perennial flow conditions that have encouraged the establishment of grasses and sedges along the channel margins. Perennial flows at Site 1 in the upper reaches of Jimmy Camp Creek may be due to the presence of springs, whereas at Site 7 the flows are due to runoff from a housing development on the west bank of the creek immediately upstream of Bradley Road.

The arithmetic relationship between the channel bankfull capacities at the field-identified top-of-bank at each site and the contributing drainage area is shown in Figure IV-7. A very weak inverse relationship (R^2 = 0.0091) appears to exist between the channel capacity and the drainage area, which is contrary to the expected positive form of the relationship (Leopold et al., 1964; Dunne and Leopold, 1978; Moody et al., 2003). The most probable cause of the inverse relationship is the varied flow regime in the watershed that is due to the combined effects of the local geology and soils that have very high infiltration rates in the upper part of the basin and the presence of base flows in the lower basin. The upstream sites, with the exception of Sites 1 and 7 are ephemeral flow channels where the form of the channel is greatly influenced by the most recent flow events (Graf, 2002). In contrast, the downstream sites (9, 10, and 11) are perennial flow channels that are heavily vegetated. It is also likely that the downstream sites have a lower sediment supply since there appears to be a discontinuity in the sediment supply-transport relationship at about Link Road. Upstream of Link Road, the valley floor is heavily vegetated and there are a number of small discontinuous channels that is a characteristic of a sheet flooding and an aggrading reach. In contrast, the channel of Jimmy Camp Creek downstream of Link Road has incised about 8 feet, and there are multiple head-cuts on the valley floor immediately upstream of the bridge. The discontinuity in the sediment supply-transport relationship could be due to the valley floor contraction created by the Link Road Bridge and its abutments.

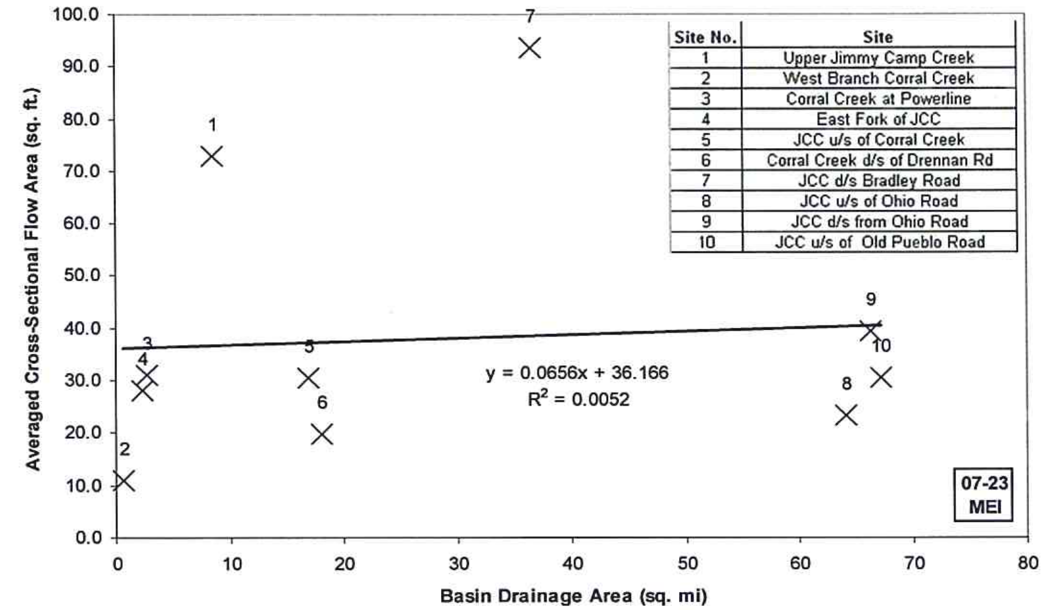

Figure IV-6: Plot of average channel cross-section area at the field-identified top-of-banks against contributing drainage area for the 10 sites.

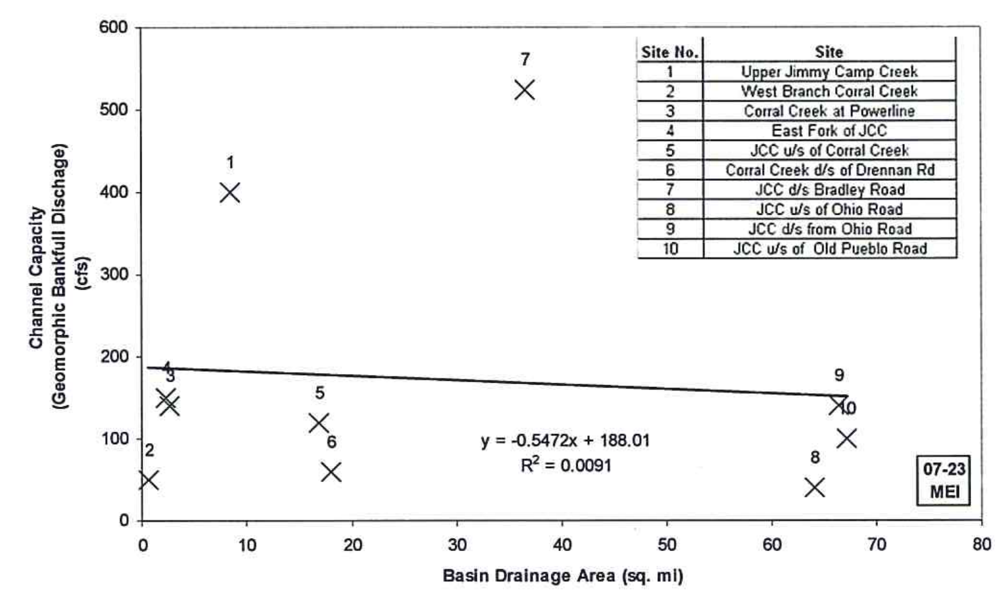

Figure IV-7: Plot of channel capacity at the field-identified top-of-banks against contributing drainage area for the 10 sites.

Exclusion of Sites 1 and 7 from the arithmetic relationship between drainage basin area and channel capacity does not significantly improve the strength of the relationship, nor does it alter the inverse form of the relationship. Logarithmic and semi-logarithmic functions also do not alter the form or the strength of the relationship between channel capacity and drainage area.

It is of interest to note that the computed 140-cfs channel capacity at the USGS gage (Site 10) has a recurrence interval of about 1.25 years and an exceedence value on the flow- duration curve (Figure 2) of 0.1 percent (less than 1 day per year). An independent assessment of the bank-full discharge at the Jimmy Camp Creek gage as summarized in the Fountain Creek Watershed Hydrology Study (2006) indicated that the recurrence interval was about 1.42 years. This suggests that the criteria that were being used to identify the top-of-banks and the channel capacity (bank-full discharge) at the 10 sites within the Jimmy Camp Creek basin were appropriate and were consistently applied. The very low channel capacity at Site 8 (40 cubic feet per second) probably represents a depositional reach that is very heavily vegetated both on the banks and in the channel where cattails were growing.

Analysis of the unit discharges represented by the channel capacities at each of the sites may help to explain the inverse form of the drainage basin area-channel capacity relationship. The results are summarized in Table IV-4. In the ephemeral channels in the upper part of the basin (Sites 1, 2, 3, 4) the unit discharges range from about 47 to 76 cubic feet per second per square mile. In the middle part of the basin (Sites 5, 6, 7) the unit discharges range from about 3 to 14 cubic feet per second per square mile. In contrast to the upper reaches where the flow is ephemeral, the unit discharges in the perennial flow lower part of the basin range from about 1.5 to 2 cubic feet per second per square mile (Sites 9, 10). The unit discharge data suggest that intense, relatively short duration thunderstorms in the upper part of the basin are capable of producing high runoff from the Cretaceous and Late Pleistocene units that crop out in the upper basin (Madole and Thorson, 2003). The middle part of the basin is underlain by highly permeable Late and Middle Holocene deposits (Madole and Thorson, 2003), and it is likely that the flows developed in the upper and middle part of the basin are lost to infiltration in the highly permeable units in the middle part of the basin, thereby leading to the much lower unit discharges and smaller channels. The very low unit discharges in the lower part of the basin are also probably due to the loss of flows to the highly permeable soils.

Conclusions

The results of this initial assessment of channel capacity in the Jimmy Camp Creek watershed indicate that there is not a statistically valid relationship between the capacity of the channel and the contributing drainage area that can be used to evaluate the magnitude of the low magnitude high frequency peak flows in the watershed. The primary reason for the lack of a relationship is most likely the fact that the flow regime at the sites located upstream of Link Road is ephemeral, whereas at the sites located downstream of Link Road the flow regime is perennial. The spatial distribution of the geologic and soil conditions within the elongated watershed appear to affect the magnitude of the flows and hence the size of the channels. The upper basin sites have relatively high unit discharges because of the presence of less permeable geologic and soil units, whereas the middle and lower basin sites where the soils are highly permeable have much lower unit discharges that are reflected in the smaller sizes of the channels. Given these conditions within the basin it is highly unlikely that the addition of more data will improve the relationship.

Table IV-4: Summary of unit discharges at the 10 surveyed sites.

Site No.	Site	Drainage Area (sq mi)	Channel Capacity (cfs)	Unit Discharge at Channel Capacity (cfs/sq mi)
1	Upper Jimmy Camp Creek	8.54	400	46.8
2	West Branch Corral Trib	0.66	50	75.8
3	Corral Tributary at Powerline	2.70	140	51.9
4	East Fork Jimmy Camp Creek	2.26	150	66.4
5	JCC u/s of Corral Tributary	16.91	120	7.1
6	Corral Tributary d/s of Drennan Rd	18.08	60	3.3
7	JCC d/s Bradley Road	36.52	525	14.4
8	JCC u/s of Ohio Avenue	64.11	40	0.62
9	JCC d/s from Ohio Avenue	66.36	140	2.1
10	JCC u/s of Old Pueblo Road	67.22	100	1.5

The lack of a strong correlation between frequency and bank-full capacity is somewhat expected given the results of the rainfall analysis presented in Chapter 3 of this report. It was found that the higher frequency rainfall and runoff events (i.e., the 2- and 5-year recurrence intervals) are caused by highly localized storm that can occur within a larger basin-wide storm or independently anywhere within the basin, and usually covering only a few square miles. This means that the bank-full capacity at any given location measured as defined herein is more a function of specific storm characteristics such as location, spatial coverage, rainfall intensity and duration. This can explain why bank-full discharges do not increase with area. However the bank-full capacities estimated at each of the ten locations measured provide useful data with respect to what should be expected along receiving drainageways during a high frequency event such as a 2-year or 5-year storm. Even with a significant number of additional gaged sites within the basin, due to the random nature of runoff producing rainfall events, it would not be anticipated that any stronger correlation between frequency and bank-full capacity would be achieved.

The measured bank-full capacities can provide guidance on the acceptable release rates from new development to better maintain historic channel characteristics. If the bank-full capacities as determined in this analysis are maintained then the existing channel sections can be preserved even in the developed basin condition. This could lead to significant savings in terms of future channel improvements, however grade control will still be required to maintain the longitudinal invert gradients to stable levels.

V. DEVELOPMENT OF ALTERNATIVES

5.1 Introduction

Alternative concepts have been examined that address the existing and future stormwater management needs of the basin. Alternatives have been identified for the major drainageway and flow paths within the major sub-watersheds. Quantitative and qualitative comparisons are presented, and a recommendation made as to which concepts are most feasible to advance to preliminary design and eventually implementation.

The general planning goals to be achieved during the alternative evaluation phase are:

1. Identify stormwater management methods and facilities that will reduce flood hazards and damages;

2. Identify stormwater management methods and measures that will prevent future flooding within the watershed and within in future urbanized areas.

3. Provide stormwater management within developing areas of the basin in order to reduce the detrimental effects of urban runoff;

4. Provide stormwater facilities that preserve and/or enhance the existing drainageway and areas adjacent to the drainageway that provide valuable environmental resource in the area;

5. Identify facilities which will minimize future operations and maintenance costs; and

6. Provide stormwater management facilities that will at least maintain and/or enhance the water quality characteristics of the basin.

7. Provide for stormwater conveyance facilities that are consistent with the intent of the City of Colorado Springs streamside ordinance so that the relationship of the stream to the development occurring adjacent to the major drainageways of the Jimmy Camp Creek watershed will provide multiple use and open space benefits to the future residents of the City.

The City/County Drainage Criteria Manual was used as a guide in the conceptual sizing of facilities. Planning goals were developed through the agency/individual coordination process. Common and/or mutual goals of the interested agencies were identified prior to the initiation of the alternative evaluation phase.

5.2 Technical Findings and Background

As part of developing the alternatives for storage and channel treatments to be recommended for the Jimmy Camp Creek basin, hydrology and hydraulic analyses were conducted. The results of these analyses produced conclusions that are beneficial in focusing the alternative development process. A few of the key findings of the hydrologic and hydraulic analyses were:

1. The rainfall analysis conducted for selected storms in the Jimmy Camp Creek basin shows that the higher frequency events such as the 2- and 5-year storms, are highly random in their coverage, location and duration. A wide array of storms can occur over the basin that can produce 2- and 5-year level rates of runoff as measured at the Ohio Avenue stream gage.
2. The calibration of the hydrologic model and associated analysis indicates that the higher frequency storms need to be evaluated using an antecedent moisture condition of I in order to achieve a reasonable correlation between the hydrologic model and gage data at Ohio Avenue for peak and volume. The calibration effort also shows that a shorter duration storm needs to be considered when evaluating the higher frequency events, as the 24-hour duration is not supported by the rainfall data analysis or the stream gage data. Using an antecedent moisture condition of I will result in approximately a 14-point reduction in the Curve Number as compared to an antecedent moisture condition of II.
3. The stream characterization analysis revealed that there is not a strong correlation between storm frequency and bankfull capacity mainly due to the physical nature of the watershed and its major drainageways. However the measured bankfull capacity can help to identify the discharges associated with the 2- to 5-year year events and associated sizing of low flow conveyance parameters for the major drainageways.

5.3 Evaluation Parameters

Stakeholder meetings were held throughout the planning process in order to discuss the overall goals of the study and to solicit specific concerns from governmental agencies, individuals, major landowners and private community groups. One result of this coordination effort was the development of the following list of parameters that should be considered when evaluating alternative storm water management concepts:

-Flood hazard management	*-Open Space/recreation/trails*
-Flood control	*-Land use impact*
-Operation and maintenance	*-Stormwater quality*
-Sustainability	*-Environmental/habitat impacts*
-Right-of-way acquisition	*-Administration/ implementation*

By reviewing the relative impact of future storm water runoff upon the major drainageways, the evaluation parameters were ranked by importance relative to each other. A minor importance ranking resulted from the absence of concerns related to the impact of urban storm water runoff within the basin as a whole. A moderate ranking resulted from the fact that urban storm water runoff must be handled within the watershed but should be able to be addressed using conventional and more common storm water handling measures and facilities. A high importance ranking resulted from information gathered in the field, technical calculations and from feedback from stakeholders and sponsors where it became obvious that a high level of concern exists regarding the handling of urban storm water runoff. Presented on Table V-1 is a summary of the relative importance of each evaluation parameter with explanatory comments.

As a result of the development and review of the evaluation parameters, the parameters viewed as being of high importance were flood control, opens space/recreation and trails, operations and maintenance, stormwater quality, environmental impact and sustainability. Those that have moderate importance were land use and administration and implementation. As such the high and moderate importance parameters will be used to screen each concept's relative impact upon each parameter and allow for the selection of the most feasible concept to pursue within the various reaches of the watershed.

5.4 Watershed Storage System Alternatives

A review of the various methods to limit the impact of urbanization upon the rates of stormwater runoff were evaluated with respect to the key planning parameters as listed above. Based upon the technical work, field visits, and meetings with the interested agencies and individuals, the alternative storage concepts were developed.

Detention Concepts

As presented in the hydrology chapter of this report, it has been estimated that peak discharges and volumes will increase along all of the major drainageways of the watershed as a result of urbanization. A key impact that urbanization will have upon the basin hydrology is that "everyday" rainfall events will result in runoff that formerly would not have increased the peak discharge, the frequency, and the duration or the runoff event. Most of the major drainageways are now unlined and natural in their section. Increases in runoff peak and volume for the higher frequency storms will create greater instability in the existing natural sections. In combination with the decrease in the natural sediment supply caused by urbanization, the increase in the rates and duration of the higher frequency events will cause the major drainageways to become unstable. For the major events such as the 50- or 100-year, the impact of urbanization will be to significantly increase the rates of discharge over existing conditions. This will cause significant increases in the velocity of the flood flows, and in areas where there is little definition to the main channel, the width of the floodplains will increase. This in turn will cause an increase in the potential of flood hazards and damages. It is clear that in the Jimmy Camp Creek watershed that flood storage facilities must be sized to control both the low and high frequency runoff events if degradation of the major drainageways is to be minimized. Detention schemes were analyzed in the alternative planning process in order to address this situation. Three sub-categories of detention storage were considered to be feasible within the Jimmy Camp Creek basin. These were:

Sub-Regional and Regional Detention
Onsite detention
Full-spectrum detention

Sub-regional and Regional detention: This concept involves the provision of large storage basins able to maintain discharges to at or below historic rates from substantial portions of a watershed. Regional and sub-regional storage volumes that are necessary to control the 100-year storm event to existing levels can range from 100-acre feet to over 500-acre feet. Storage facilities of this size would serve more than one developing area and/or ownership. This concept works best in basins that have major downstream capacity constraints but are developed to the point where insufficient developable land exists such that onsite or full-spectrum storage would not have the desired hydrologic impact. Drawbacks of this concept are that the drainageways that carry the runoff to regional basins need to be designed to convey fully developed rates of runoff, implementation issues with respect to securing the land needed for such a facility, and timing issues related to when a regional facility can be afforded by the level of development in a given watershed. Also, from the hydrologic analysis it was found that the provision of sub-regional or regional detention storage will not reduce rates of runoff at the watershed's outfall point if they are sited below Drennan Road. Therefore the sub-regional or regional detention storage analyzed in this study would be located all north of Drennan Road. Presented on Figures V-1 and V-2 are a sub-regional and regional detention storage concepts for the Jimmy Camp Creek watershed. Both of these concepts result in maintaining the 100-year peak discharges at Fountain Creek to existing levels. Peak discharges for the sub-regional and regional detention basin concepts in comparison to the future and existing discharges are presented on Table V-2.

Another drawback to the sub-regional and regional storage concept is that these facilities do little to control the higher frequency events. As found in the rainfall analysis, higher frequency storms are relatively small in spatial extent, many times covering less than ten square miles. It is possible that a storm of this nature could occur over a portion of the watershed that does not have a storage facility within it. In this case the runoff would move un-detained down the major drainageways and create the potential degradation to stream banks and inverts. Another concern related to the implementation of a sub-regional or regional detention concept is that initial land development activities in the Jimmy Camp Creek basin will more than likely be situated far offsite from the site of the detention facilities. As such temporary detention schemes may have to be considered while the basin develops to a point where hydrologically the installation of the sub-regional or regional detention basin is warranted and until such time the construction of the facility is economically feasible. In the interim period the receiving major drainageways would collect and convey developed runoff that could adversely impact the stability of the natural channel sections.

Regional or sub-regional detention manages the increase in runoff volume due to urbanization at relatively few locations and therefore many segments of the receiving drainageways will carry un-detained discharges. While none of the detention concepts can reduce the total volume of runoff, regional or sub-regional detention if implemented will require that the major drainageways be protected from the detrimental effects of the increase in volume and associated peak discharges for all frequencies.

Onsite detention: This concept involves the provision of small storage areas that serve individual parcels or developments so that discharges to downstream land or drainageways are maintained at historic rates. This concept works best in small sub-watersheds where no regional sites are available or wherever there may be capacity constraints in existing downstream stormwater systems. This concept manages the increase in runoff volume due to urbanization at its point of origin. Accordingly a major drawback to this concept is that the collective impact of numerous onsite storage basins is to create higher discharges than existing rates within the receiving drainageways and cause degradation of the receiving drainageway either by bank erosion or invert erosion. This phenomenon has been experienced in many urbanized areas and has been documented by hydrologic studies prepared for urban watersheds throughout the United States. An advantage to this concept is that it addresses the timing issues that face the implementation of sub-regional and regional concepts in that onsite storage basin can be built economically within very small parcels and usually involving not more than 40 acres in tributary area. El Paso County has required onsite detention when downstream conveyances are not available, a common occurrence in rural and developing urban areas or where concepts embodied in a regional planning study have not as yet been implemented. The City of Colorado Springs has required onsite detention to be implemented where downstream conveyances and capacity is not adequate, but in general does not encourage onsite detention storage.

Full Spectrum Detention (FSD): This concept has recently come to the forefront as a method system for urban storm water management. This concept addresses the problem outlined above under Onsite Detention with respect to the negative impact upon the receiving major drainageways. These facilities can serve small parcels as well as act on more of a regional basis. Full spectrum detention manages the increase in runoff due to urbanization by holding the increase volume over an extended period of time so as to not cause the release from each individual full spectrum basin to accumulate to peak levels greater than existing conditions. Depending upon land use full spectrum detention storage cannot practically serve tributary areas greater that around 300 acres. Tributary areas above this size cause the embankment of such facilities to fall under the jurisdiction of the Office of the State Engineer dam safety regulations. Since there would be a significant number of FSDs that will be needed in a watershed the size of Jimmy Camp Creek, embankments falling under the jurisdiction of the State were considered to be undesirable from the standpoint of the implementation and administration parameters. By releasing runoff from the storage pool at very low rates, the additive nature of releases similar to that from onsite basins can be mitigated for since the outlet hydrograph mimics the existing hydrograph in peak for all frequencies of runoff events, not only the 5-year and 100-year events. A drawback of this concept is that most times the FSD facility needs to be built off-stream so that runoff carried by the receiving drainageway can continue along its historic path and within the existing channel section. An advantage to a FSD system is that water quality storage can be provided for within the overall capacity of a FSD. Water quality storage would have to be provided for separately in a sub-regional or regional detention concept. Fountain, Colorado Springs and El Paso County have each adopted criteria for FSD and is requiring that FSD be implemented on future land development projects. As with onsite detention ongoing maintenance is needed to assure proper functioning of the outlet structure and sediment storage pools.

The hydrology related to the impact that FSD can have on a watershed was analyzed as part of the development of the DBPS. Full spectrum storage facilities were analyzed for the 1.6 square mile Blaney Tributary in order to determine if FSD could in fact reduce peak discharges to historic rates for all frequencies. A multiple FSD system was modeled for the Blaney Tributary in order to assess the capability of a FSD system to maintain developed rates runoff to pre-development conditions for all frequencies. The results of this analysis are presented in the Hydraulic Technical Addendum to this DBPS. A methodology of sizing a FSD was developed that is based upon the City/County DCM. It was found that a multiple facility FSD detention system was able to maintain developed rates of runoff to at or below pre-development conditions for all frequencies. A unit storage requirement of .066 acre-feet per acre was calculated assuming an average percent imperviousness of 57.5. It was also determined that the methodology used to size the FSD produced storage volumes similar to what would be estimated using the full spectrum spreadsheets contained within Volume II of the City of Colorado Springs and El Paso County DCM.

The analysis of FSD completed for this study provided confidence that it is a feasible concept and some guidance about how it could be implemented. However additional analysis being completed with the City's assessment of its stormwater management practices may require that procedures for implementing this concept be revised.

5.5 Preliminary Matrix of Detention Storage Alternatives

Feasible concepts were developed for the storage of urbanized runoff for each reach of the major drainageways and were evaluated as to each concept's compatibility or impact upon each of the evaluation parameters listed above. Relative impact was assigned to each concept as to low, neutral and high. A low impact (-1) was determined wherever a concept's viability for handling urbanized runoff was considered to cause little physical change with respect to a specific evaluation parameter. Neutral impact (0) was determined wherever a concept's viability for storing urbanized runoff was considered to be manageable and any potential negative impact could be planned and mitigated for as the stormwater management system is developed and implemented. High impact (1) was determined wherever the concept's viability for storing urbanized runoff would render the physical characteristics of the existing drainageways unsuitable with respect to capacity, unstable with respect to erosion control, or generally achieving or not achieving the goal of a particular evaluation parameter. Impact upon a given parameter could be judged to be either negative or positive as well. Relative impacts have been judged between each for the three storage concepts.

Presented on Table V-3 is a matrix related to the evaluation of each of the three storage concepts. Based upon this qualitative ranking, FSD was found to be the most viable solution in addressing the impact of urbanized runoff within the Jimmy Camp Creek watershed. This concept has the fewest negative high impacts and the greatest number of positive high impacts. Though not part of the evaluation process, it is likely that the FSD concept would be the easiest to implement and would work well in a phased development scenario as urbanization within the watershed proceeds over a multi-year time period.

5.6 Cost Comparison of Storage Alternatives

Though the cost to construct and acquire land for detention basins was not one of the parameters used to evaluate the relative impacts of each alternative storage concept, a cost comparison between a sub-regional, regional and FSD storage has been prepared. In order to compare the three storage concepts with respect to cost and land acquisition actual construction costs for regional and sub-regional detention storage basins was developed using data for seven detention basin ranging size from 11 to 205 acre feet. This data is summarized on Table V-4. Three of the storage basins were jurisdictional. From the cost analysis of the seven facilities unit storage costs were developed on a dollar per acre-foot basis. For the seven basins analyzed unit storage costs s of $23,762 per acre-foot and $24,353 per acre-foot for the regional and sub-regional storage basins, respectively. Unit land requirements were also estimated using the parcel data for each of the seven storage basins. From the analysis a unit land requirement of .203 acres per acre-foot and .285 acres per acre-foot for the regional and sub-regional storage basins, respectively.

The cost attributable to water quality for those regional and sub-regional detention basins where water quality storage was provided was taken out of the overall unit cost estimate. Since water quality storage is required in the City/County DCM, the total volume of water quality storage that would have to be provided offsite from the sub-regional or regional detention basins needs to be estimated. The average developed percent imperviousness for the Jimmy Camp Creek watershed was calculated to be 57.5 percent using the land use data presented Chapter II. Using the average percent imperviousness the unit water quality storage requirements (acre-feet per acre of developed land) was estimated by calculating the water quality storage requirements for a hypothetical 100-acre parcel. The method outlined in Volume II of the City County DCM was used to estimate the required the water quality capture volume for the hypothetical basin. From this calculation a unit water quality storage volume of .024 acre-feet per acre was estimated for an average percent imperviousness of 57.5. Applying the unit water quality storage volume over the developable acreage within the watershed of 31,500, a total of 760 acre-feet of water quality storage was estimated The total cost of providing water quality storage was then determined using the sub-regional unit cost of $24,353 per acre-foot for the estimated 760 acre-feet of water quality. A total cost of $18,508,300 was calculated and then added to the storage costs for the regional and sub-regional alternatives.

For the regional detention concept a total storage of 1,172 acre-feet (1,139 acres and 33 acres regional and sub-regional sized detention basins, respectively) was used to develop the total storage cost for the regional concept presented on Figure V-2. For the sub-regional detention concept a total of 1,146 acre-feet (540 acres and 606 acres regional and sub-regional sized detention basins, respectively) was used to develop the total storage costs for the sub-regional concept presented on Figure V-1. Using the unit costs and volumes described above the results were:

Regional system with off-site water quality storage:	**$43,121,500**
Sub-regional system with off-site water quality storage:	**$44,227,700**

In order to estimate the total storage required for the FSD system, the analysis prepared for the City and described previously in this report was used to determine the total cost of a FSD system. Since FSD storage basins provide water quality there is no need to account for the cost of offsite water quality storage when estimating the total cost for a FSD system. It was estimated that a total of 1,859 acre-feet of FSD storage would be required for the watershed by applying the unit full spectrum storage of .066 acre-feet per acre. Applying the unit cost estimated for sub-regional detention basins, the results obtained were:

FSD system:	**$45,272,200**

As can be seen the results show that the FSD may be approximately 10 percent greater in cost than a regional or sub-regional system. Land required for a FSD system would be greater as well. Presented on Table V-5 is the estimated acreage that would be necessary to accommodate the 2,100 acre-feet of full spectrum storage. This estimate was developed using the detention basin data presented on Table V-4. The land required for a regional, sub-regional and FSD system were estimated at 458, 500 and 598 acres, respectively. While FSD may be costlier and require more land as compared to the regional and sub-regional concepts, the following circumstances need to be taken into consideration:

1. Because FSD manages the discharge of urban runoff to the major drainageways in such a way that resembles the pre-development condition, there will be less need to provide horizontal and vertical stabilization along the receiving drainageways as compared to the other storage concepts. Both the regional and sub-regional systems will require that extensive reaches of the major drainageways within the Jimmy Camp Creek watershed be enlarged and horizontally and vertically stabilized since they will be conveying fully developed runoff up to and between the detention basins. The 10 percent cost difference between the FSD and regional/sub-regional detention schemes will be exceeded by costs required to enlarge the channel and stabilize the banks along receiving drainageways in the regional/sub-regional detention scheme, costs that will not be incurred in a FSD system. While grade stabilization in the form of checks and drop structures is necessary along the drainageways for any of the storage concepts it is anticipated that the total cost of grade control structures would be reduced for the FSD. An estimate of the difference in channel costs between the sub-regional, regional and FSD storage systems is provided below in the discussion of alternative channel concepts.
2. One of the major disadvantages of a regional/sub-regional system is that the storage facilities often lie offsite from the where development may be occurring, especially in the early stages of the development. This situation can cause extreme problems in the phasing of the infrastructure, and in the financing of the construction of an offsite facility. This can cause significant delays in the implementation of regional/sub/regional facilities and in the interim can subject the receiving drainageways to urbanized flows. This in turn forces the need to enlarge and stabilize drainageways that may also be offsite from the area of development and many times on ownerships lying downstream of the developing parcels.
3. The land requirement for of FSD is around 33 percent and 9 percent greater for the FSD concept as compared to the regional and sub-regional systems, respectively, the parcels associated with FSD will be much smaller in general (say 5 to 20 acres) than the parcels that may be needed for a regional or sub-regional facility (20 to 80 acres). The sites for regional and sub-regional sites are limited to relatively few locations within the watershed whereas FSD sites can be integrated within or very close by the location of development. This is particularly advantageous in the earlier stages of urbanization. Since regional and sub-regional systems have the inherent problems associated with phasing and implementation, establishing a timeframe for land acquisition is extremely difficult and the future cost of the land cannot be accurately determined in the context of a DBPS. Land for FSD facilities would be able to be acquired or dedicated through normal land development processes.
4. A regional or sub-regional system will almost certainly require that a detention storage and land acquisition fee be established for the basin. This is because a regional or sub-regional system will collect runoff from varying types of land uses and significant numbers of property owners. This may lead to property owner and developer concerns related to the establishment of an equitable fee system. It could be argued that since a FSD system serves much smaller parcels and watersheds, and much fewer overall properties, it is not necessary to spread the costs over the entire watershed in the form of a storage or land acquisition fee.
5. Should FSD be fully implemented and the result is that discharges remain at existing levels there may not need to be the need to revise those segments of Jimmy Camp Creek and its major tributaries that have had detailed flood plain studies that are presently shown in the City of Colorado Springs, City of Fountain and El Paso County flood insurance studies.
6. Because there is very limited impact upon peak discharges that would result from the implementation of a FSD storage concept, the existing environmental resources along the major drainageways will not be adversely impacted.
7. Although none of the proposed storage schemes reduce runoff volumes from developed areas, FSD provides some mitigation of increased runoff volumes by releasing the excess volume over an extended period of time and at less erosive flow rates.
8. Eventually development will significantly reduce the area from which sediment is made available for transport by the drainageways no matter which storage scheme is applied. However FSD will increase the likelihood that sediment transport rates will continue at pre-development conditions over a longer period of time.
9. The City of Colorado Springs presently has a MS4 permit with the State of Colorado. To be in compliance with its MS4 permit the City requires that water quality storage be achieved off-stream. FSD basins can be sited in most cases off-stream whereas regional detention storage cannot. Water quality storage would be required onsite in the regional or sub-regional detention alternatives.

5.7 Major Drainageway Conveyance Alternatives

At this time the majority of the major drainageway reaches of Jimmy Camp Creek and its major sub-tributaries are unimproved. Where channel stabilization measures have been constructed, they occur mostly at the approaches and exits at roadway crossings. As determined in the hydraulic analysis of the existing floodplains there are several locations, mostly within reaches J1 through J3, where the existing channel banks are not of sufficient height to contain the 100-year discharge without overtopping and causing areas of extremely wide floodplains. While this is not a problem for the watershed at present, these wide uncontrolled floodplains will have to be addressed as the land develops. Since it has been concluded that FSD is the most viable storage alternative to pursue, existing condition hydrology can be assumed when the sizing of major channel conveyances are carried out. As such it has been assumed that FSD storage will be implemented and it is under this assumption that viable channelization concepts have been evaluated. Since existing rates of runoff would be maintained natural or unimproved channels may be feasible if their physical characteristics such a depth and velocity of the flow are non-erosive. Accordingly these two major drainageway concepts have been evaluated.

Floodplain preservation: This concept involves leaving the floodplains along the receiving drainageways un-encroached and in their natural cross-section with stabilization of the low flow channel. The viability of this concept depends heavily upon the stability of a drainageways' existing section that is in turn related to the natural floodplain's width, velocity and depth of flow. This concept shall be the default approach to be applied throughout the watershed. The use of other concepts must be justified and shown to provide sufficient benefits, such as flood damage reduction, to be allowed.

In the case of Jimmy Camp Creek the floodplains within reaches J1 through J3 vary significantly in their width and depth. At several locations shallow overbank flooding could occur and force the floodplain widths to exceed 2,000 feet. This is most prevalent in reach J1 however in reach J3 in the vicinity of Peaceful Valley Road the lack of culvert capacity forces the 100-year runoff to move over land along the right bank (as oriented facing downstream) and does not rejoin the main channel for several 1,000 feet downstream of Peaceful Valley Road. The low flow area of the Jimmy Camp Creek drainageway is well defined in most reaches of Jimmy Camp Creek however in reaches J4 and J5 the low flow area of the drainageway is poorly defined.

For the major sub-tributaries the 100-year floodplains are generally narrow (300 feet and less) with exception of East Fork Jimmy Camp Creek in the lower portion of reach E1 and reach E2 below Drennan Road where the floodplain widths exceed 2,000 feet. Along Reach F1 of the Franceville Tributary, a wide shallow floodplain exists upstream of Drennan Road caused by the lack of culvert capacity to carry flood flows to the historic channel that lies downstream of Drennan. The lack of culvert capacity at this location forces the Franceville Tributary to join the Corral Tributary north of Drennan Road. In reach S1 of the Stripmine Tributary the lack of channel capacity near station 82+00 causes flood flows to leave the main flow path and force runoff overland and flood plains widths to exceed feet. A typical floodplain preservation concept has been presented on Figure V-3.

Channelization: This concept involves reconfiguring the natural section to convey in a conventional trapezoidal channel the 2-year through 100-year rates of runoff through the watershed and outfall to Fountain Creek. Since the low flow area of the major drainageways is generally well defined, a benched trapezoidal section appears to be a feasible section to implement. This type of conveyance would be required to be configured to avoid or minimize the disturbance of existing vegetation. Where disturbances occur riparian habitat can be introduced on the benches of the channel section. In order to determine the geometry of benched channel sections the general criteria of subcritical flow (i.e., Froude number less than 0.8) was assumed. This assumption in combination with depth of flow limitations in the low flow area of the drainageways will allow for softer treatments along the banks such as grass-lining or engineering vegetation. A typical detail of the benched channel concept is presented on Figure V-4.

In the upper reaches of Jimmy Camp Creek and it major sub-tributaries the channels grow more incised making the implementation of a benched section less feasible, however the proposed conveyance sections will have to be analyzed on a reach-by-reach basis as the conceptual design plans are developed. This concept will only be applied where it can be shown to be significantly beneficial, such as by reducing flood damages. Increasing the amount of developable land is not sufficient justification to reduce storage capacity in natural floodplains or to damage or remove wetland or riparian habitat.

Grade control will be needed along all reaches in order to maintain a maximum longitudinal slope of approximately 0.5 percent. The spacing of grade controls will be dictated by the location of hydraulic structures such as bridges and culverts and as the gradient increases, most notably in the upper segments of Jimmy Camp Creek (reaches J6 and J7) and in reaches E3, S2 and C4. The grade control will help to maintain velocities at non-erosive levels or to levels that require only light riprap protection. Various types of grade control structures are available for implementation.

Presented on Table V-6 is a matrix presenting qualitative evaluations of the floodplain preservation and channelization concepts.

5.8 Drainageway System Alternatives Conclusions

Based upon the alternative evaluation process it is recommended that the both of the channel concepts be advanced for further consideration. The floodplain preservation concept should be considered the default alternative so that the beneficial effects of the floodplain preservation concept, such as flood storage and habitat preservation, are maintained and assured. In this regard the implementation of a floodplain preservation concept does not constitute a loss of developable land since developing within the flood fringe areas will reduce the potential for natural flood storage that could negatively impact the watershed in areas below such encroachments. The channelization concept should only be applied in those drainageways segments where flood damages could now occur and where the 100-year floodplain is wide and uncontrolled such as in the vicinity of Peaceful Valley Road. The benched channel in this type of reach can be used to reduce the floodplain width significantly from the existing 1,500 to 2,000 foot wide floodplain that now exists in this area of the watershed. A more defined low flow channel could be created as well. In the reaches where a benched channel section is proposed, the channel improvements would need to be designed such that the environmental qualities of the floodplain can be avoided or enhanced compared to the existing conditions. The implementation of a benched channel section should not be advanced simply for the purposes of creating more developable land. The benched portion of the channel section should be used for the protection, replacement or restoration of wetland or riparian resources. The Manning's roughness values applied in the design of the benched channel section needs to take into account the vegetative habitat that may exist now or in the future. The benched channel concept is also recommended for the lower reaches of the Franceville and Stripmine tributaries where these tributaries will be redirected to join the Corral Tributary. Finally the benched channel section should be considered at the transitions in and out of roadway crossings.

The floodplain preservation concept is most applicable in the upper segments of Jimmy Camp Creek and the major sub-tributaries. Floodplains in these segments are much narrower and confined. As development proceeds adjacent to floodplains, it may be necessary to stabilize existing banks at outside bends to prevent lateral migration.

Both of these concepts are feasible because of the establishment of FSD in the basin. The FSD concept will maintain peak discharges at existing levels thereby reducing the overall width of the floodplain or benched channel sections. The base flow within the major drainageways will increase over time due to the urbanization of the watershed. The increase in base flow will be a benefit to existing vegetative habitat along the low flow thread of the stream and will not only help to sustain existing riparian and wetland species but promote the spread of these same species over time.

For both of the conveyance concepts grade control in the form of drop structures will need to be implemented. The sediment load from sub-watersheds tributary to the major receiving drainageways will decrease as urbanization proceeds that in turn will introduce instability along the low flow area of the drainageways. This will create the need to provide vertical stabilization of the low flow area of the channel throughout the watershed. The intent of the grade control for all of the major receiving drainageways is to establish or maintain a longitudinal gradient of approximately 0.5 percent. Longitudinal slopes of this gradient will promote subcritical flow conditions in the channels and keep velocities to levels where grass-lined banks are feasible or to the level where moderately sized riprap (12-inch D50) can be used for stabilization of the low flow channel or for banks at outside bends of the drainageways. Various types of grade control structures could be implemented including soil cement drops, vertical concrete or grouted sloping boulder drops. The use of soil cement to create artificial rock outcrops to check the vertical degradation of the invert may provide a highly sustainable method of grade control and reduce the need for importation of riprap, which in the case of the Jimmy Camp Creek basin would have to be supplied from quarries far offsite from the watershed. The total number of grade control structures can be reduced by the introduction of meanders along the low flow area of a drainageway. Finally it may be necessary to phase the construction of grade control structures so that the stream slope is not flattened too much in the early stages of urbanization. Reducing the gradient of the stream in the early stages of development may cause sediment to accumulate within the low flow channel and reduce the carrying capacity of channel.

5.9 Drainageway Conveyance Cost Comparisons

As mentioned above in the discussion of the storage concepts, the cost of a sub-regional or regional storage system has been estimated to be approximate 10 percent less in total cost than the FSD concept assuming that all of the storage is constructed at the same time. The ability to incrementally construct FSD will reduce costs over time. The difference in cost between the storage concepts will be more than offset by the reduction in channel conveyance and grade control costs that would be afforded through the implementation of FSD. In order to assess the conveyance cost reduction associated with the FSD reach J5 of the Jimmy Camp Creek drainageway was analyzed for the regional detention and FSD alternatives. A 10,700-foot segment of Jimmy Camp Creek reach J5 between design points J28 and J31 was hydraulically investigated to confirm whether or not a savings in the cost of channel conveyance and grade control could be achieved for the FSD with floodplain preservation as compared to a regional detention alternative with a 100-year capacity benched channel section. The benched channel concept was applied in this segment using the hydrology for the regional detention and FSD alternatives. The 100-year discharge ranged from 4,500 cubic feet per second to 7,500 cubic feet per second for the regional detention alternative, and from 5,200 cubic feet per second (without areal adjustment) to 6,000 cubic feet per second segment for the FSD alternative. The 5-year discharge ranged from 1,800 cubic feet per second to 3,000 cubic feet per second for the regional detention alternative. The 5-year discharge for the FSD alternative is constant at around 150 cubic feet per second. A slope of 0.5 percent was assumed for the purposes of the analysis and it was also assumed that the capacity of the low flow was set at the 5-year discharge. The assumption of the 0.5 percent slope results in Froude numbers less than 0.8.

Based upon the above assumptions and discharge rates for the segment under analysis a total channel conveyance cost of $1.87 million was estimated for the benched channel section with regional detention alternative compared to $1.34 million for the floodplain preservation with FSD alternative. The cost comparison is presented in Table V-7. Using the above totals a cost savings of approximately $50 per lineal foot could be achieved if the FSD alternative is implemented. Applying the unit cost savings over the approximately 23.7 miles of the main stem of Jimmy Camp Creek a cost savings of approximately $6.3 million is estimated. Applying the same savings per foot for the other major drainageways and additional $5.5 million reduction in channel conveyance costs can be estimated bringing the total to $11.7 million for the entire watershed. This savings alone would offset the cost of providing the FSD storage volume over and above the volume required for the regional detention system.

The cost of providing grade control for each alternative was analyzed as well. For the floodplain preservation concept with FSD, the low flow area of the creek that is the portion of the floodplain that is to convey the 5-year existing condition flow of 150 cubic feet per second. A channel slope of 0.5 percent and a maximum drop height of 3-feet were assumed for the segment of Jimmy Camp Creek between design points J28 and J31. For the benched channel section with regional detention, sloping boulder drops with sheet pile cut-off walls were assumed, a channel slope of 0.5 percent and a maximum drop height of 6-feet. The major difference between the grade control structure that are required for each of the two channel conveyance alternatives is that the floodplain preservation concept would require that the drop stabilize only the low flow area of the floodplain, while the a grade control for the benched channel would be required to span the entire width of the benched channel. For the reach under analysis, the low flow area of the creek is typically 20- to 25-feet wide, while the top width for the benched channel ranges from 165 feet to 220 feet. For the subject reach, while twice as many low flow grade controls are needed for the floodplain preservation alternative, the width of a benched channel drop is significantly wider at the crest.

A comparison of grade control costs is presented on Table V-8. For the reach under consideration, the unit grade control cost for the floodplain preservation concept was determined to be $24.50 per lineal foot of drainageway channel. Unit grade control for the benched channel concept was determined to be $297 per lineal foot of channel. For the segment analyzed the savings in grade control costs afforded by the implementation of the FSD concept is estimated at $3.19 million. Applying the unit savings over the entire length of the major drainageways, if was estimated that a $69 million cost difference could be expected between the two channel conveyance concepts. This magnitude of savings confirms that there is a significant cost effectiveness associated with the implementation of a FSD system that far offsets the additional cost of the storage for the FSD scenario necessary to maintain the rates of runoff to pre-development conditions. The combined savings of conveyance and grade control costs just for the segment subject to this analysis is estimated at $3.47 million.

For the floodplain preservation concept it may be possible to leave the low flow area of the channel in its present form and thereby reduce the cost of stabilizing the low flow channel. Most of the cost savings is derived from the reduction in the construction cost for the low flow channel and in the considerable difference in the costs of a low flow grade controls versus a benched channel sloping boulder drop. The regional detention concept produces much higher 5-year rates of runoff than if the FSD concept is assumed. This finding is to be expected since the FSD releases the higher frequency flows at a much lower rate into the receiving drainageways as compared to the regional or regional storage concept. For the segment of Jimmy Camp Creek analyzed, in order to convey the 5-year discharge in a low flow channel of the benched channel concept, the magnitude of the five-year flows in the regional concept requires a trapezoidal section with a top width of 160 feet for a depth of three-feet which is the maximum recommended low flow depth per UDFCD criteria. By comparison the low flow section in the FSD concept is only three feet deep and has bottom widths ranging from 10- to 20-feet. The cost estimate for the floodplain preservation concept assumed that the low flow would have to be excavated. Wherever a stable low flow section exists this excavation would not be necessary, further lowering the cost of the floodplain preservation concept could be expected.

The impact upon conveyance right-of-ways was also assessed for each of the storage alternatives. For the segment of Jimmy Camp Creek under analysis, the total acreage needed for a benched channel section regional detention was estimated at 50 acres. The floodplain acreage in this segment was estimated at and 85 acres for the FSD storage concepts. While a significant reduction in acreage could be afforded by the use of a benched channel section, the cost of earthwork associated with forming a benched section could drive the unit cost of a benched channel significantly higher as well.

Based upon the analysis described above, if FSD is implemented with floodplain preservation and low flow channel stabilization, the additional storage costs associated with FSD will be more than offset by the savings in major drainageway conveyance and grade control costs as compared to the regional detention scenarios.

VI. CONCEPTUAL DESIGN OF SELECTED PLAN

6.1 Introduction

The results of the conceptual design analysis are summarized in this section. The alternative improvements have been qualitatively evaluated, and presented to the project sponsors, stakeholders interested agencies and individuals through periodic public and technical progress meetings. Field review of specific areas of concern has been conducted in order to refine the channel treatments suggested for use along the major drainageways and flow paths. The conceptual plan for the recommended alternative is shown on the drawings contained at the rear of this report.

6.2 Criteria

Past and current versions of the City of Colorado Springs and El Paso County Drainage Criteria Manual were used in the development of the conceptual sections and plans for the major drainageways within the Basin. The criteria and methods summarized City/County Drainage Criteria Manual was supplemented by various other manuals. These were:

1. Urban Storm Drainage Criteria Manual, Volumes I, II, and III prepared by the Urban Drainage and Flood Control District.
2. City of Fountain Department of Public Works Standard Specifications and Subdivision Criteria Manual.

6.3 Hydrology

Presented in Chapter 3 was the hydrology analysis and results obtained for the existing and developed basin conditions. Presented on Table III-10, peak discharges for the 2-, 5-, 10- and 100-year recurrence intervals. The peak flow data for the existing development conditions were used to determine the extent of the 100-year floodplains and to size drainageway conveyances and road crossings. The discharges summarized on Table III-10 for the 5- and 100-year frequencies are presented on the profile of the conceptual design plans contained at the rear of this report. The 2- and 5-year recurrence interval discharges were determined using a 6-hour Type IIA storm pattern and an antecedent moisture condition of AMC-I. The 10 through 100-year discharges were determined using a Type II storm distribution. Finally, an areal adjustment factor was applied for all design points have a tributary area greater than 10 square miles. Estimation of existing condition flow rates at additional design points may need to be determined as more detailed studies are prepared in support of land development activities. The sub-basins, reaches and design points associated with the hydrology analysis are shown on Exhibit 1 contained at the rear of this report. Contained in the Jimmy Camp Creek Drainage Basin Planning Study Hydrology Technical addendum is a complete listing of peak discharges for all the sub-basins, stream segments and design points as well as calculations spreadsheets and HEC-1 input and output.

6.4 Detention Storage

The recommended conceptual plan for storage of urbanized runoff for the Jimmy Camp Creek basin is to provide full spectrum detention (FSD) basins. The storage facilities will have a wide range in storage volume, however based upon the analysis a storage volume of 50 acre-feet and a tributary area of approximately 150 acres, depending upon the proposed land use within a FSD watershed, are considered as maximum parameters for planning purposes. Approximately 2,100 acre-feet of storage will be needed within the watershed at full build-out of the basin. These basins will be capable of providing water quality capture volume, storage of the "excess urban runoff volume" (EURV), and storage and routing of the 5-year and 100-year flood events to the receiving drainageways. The location of the facilities will be refined as land development activities dictate. Planning for the locations of FSD storage basins needs to be addressed during the master development drainage plan phase of a project. At that time a more comprehensive analysis of the size and location of FSD basins can be conducted using more detailed topographic, environmental resource mapping, refined land development plans and drainage criteria. The 100-year release rate will also have to be refined during the master and final drainage planning phases for sites that will incorporate a FSD.

The rational for recommending that FSD be implemented in the Jimmy Camp Creek watershed was summarized in Chapter V. Methods for the sizing of FSD's range from generalized methods to very detailed hydrograph methods. Three general methods are described as follows:

1. Contained within the most current version of Volume II of the City/County Storm Drainage Criteria Manual (DCM) is a spreadsheet method for determining the EURV volume has been developed. The method requires an estimate of the percent imperviousness of the contributing watershed, the tributary area and rainfall data. This spreadsheet determines the EURV using the design storms that are typically used for the analysis of urban storm water management systems in the Denver metropolitan area. The EURV obtained using the Volume II spreadsheet was very comparable to the EURV obtained using the methods described below. The City of Fountain has adopted the methods described in the DCM and requires it use when sizing FSD's for developments within the City of Fountain.
2. A generalized method for obtaining the based in the use of the SCS curve numbers as tabulated in the "***Procedures for Determining Peak Flows in Colorado***" prepared by the United States Department of Agriculture, Soil Conservation Service (now NRCS), March 1977. Curve numbers for the proposed development with an AMC II moisture condition and for the existing development with an AMC I moisture condition need to be tabulated for the watershed proposed to drain to the FSD basin. The five-year, 6-hour rainfall needs to be estimated using the rainfall data contained in the City/County Drainage Criteria Manual or from the ***Precipitation-Frequency Atlas of the Western United States, Volume III, Colorado***. The runoff in inches can be determined using the existing and proposed condition. The difference in runoff will be the five-year EURV.
3. Using the HEC-HMS hydrograph package or the USACOE HEC-1 Hydrograph Model, the five-year EURV can be estimated by determining the difference in volume between and five-year fully developed condition AMC II and the five-year existing development condition AMC I hydrology for the sub-basin that will be tributary to the FSD. The 5-year 6-hour Type IIA storm would be for

both development conditions. The curve number option needs to be used when applying the HEC-HMS or HEC-1 models.

Contained with the conceptual design drawings is the layout of a typical FSD. The outlet structure needs to be sized so as to release the EURV within a 60 to 70-hour period. The perforated plates used to control the discharge of the EURV can be sized using the method explained in Volume II of the DCM. The outlet structure also needs to be sized to limit the 5- and 100-year discharges to the existing development condition. The final layout and design for a FSD will be dependent upon the location of future roadways and the layout of major land developments. It should be encouraged that FSD basins be sited so that the design may take advantage of roadway embankments, natural depressions and sump areas and existing wetland and/or riparian areas. They should be sited whenever possible so that the can be comingled with open spaces within future master planned land developments and park sites. It is recommended that all of the FSD basins that will be constructed in the watershed become publically or quasi-publically (e.g., metropolitan districts) owned and operated as these structures form such a critical element of the stormwater management plan for the watershed.

Though the implementation of a FSD system addresses the increase in rates of runoff affected by the development of the watershed, there is still a significant increase in the total volume of runoff compared to existing conditions. To address the change in the volume of runoff, low impact development (LID) measures could be implemented to reduce some of the increase in volume due to development. Porous paving systems, green or open space buffers and onsite water harvesting within parking and landscaped areas can be used to assist in percolation of runoff. These measures need to be identified early on in the land development process for any given parcel. The incentive to provide LID measures that manage the volume of stormwater produced by residential and commercial development would lie in the fact that the EURV from a watershed can be reduced, thereby reducing the size of the storage facility itself.

6.5 Major Receiving Drainageways

In general, the floodplain preservation concept has been selected as the primary conveyance system for Jimmy Camp Creek and it major sub-tributaries. This conveyance system would encourage the preservation of the floodplains as depicted on the conceptual design plan and profiles. The floodplain shown on the conceptual plans was determined using the 100-year existing condition hydrology as summarized in Table III-8. Selective locations such as at outside bends of the floodplain and at approaches and exits of roadway crossings may need to be protected with soil/riprap bank linings. The location of selective bank lining has been shown on the conceptual plans. Typical bank lining details have been provided. The low flow area of the drainageway, the portion of the floodplain that conveys the five-year discharge, will need to be armored per the typical low flow section presented in the conceptual design plans. At some locations, particularly in the lower reaches of Jimmy Camp Creek, the low flow channel is well defined and stable. Bank linings in these cases need to be placed so as to minimize disturbance to vegetation that may be acting to stabilize the invert and banks of the low flow channel.

At some locations a benched channel section has been proposed in order to transition the drainageway through bridges and culverts, or to eliminate wide uncontrolled shallow flooding such as is present in the vicinity of Peaceful Valley Road. A typical benched channel section has been presented on the conceptual design plans. Benched channels should be sited so as to avoid disturbance or take advantage existing riparian or wetland resources within the floodplain or along the low flow area of the drainageway. The type of soil/riprap linings are presented on the typical sections provided with the conceptual design plans. Where velocities can be shown to be non-erosive for the 100-year event, the overbank lining may be grass with a soil reinforcement mat on the benches. The low flow area of the benched section has been sized to carry the 5-year existing condition rate of runoff. The design of the channel should achieve a 100-year Froude Number of 0.8 or less so that normal flow conditions can be maintained during flood events. Wherever possible the low flow portion of the benched channel section should follow the alignment of the existing invert. Keeping a moderate sinuosity along the low flow channel will help to reduce the amount of vertical grade control structures.

The major drainageway improvements for Jimmy Camp Creek and its major sub-tributaries are presented on the Conceptual Design Plan and Profile drawings located at the rear of this report. Typical details are also provided for the measures shown on the plans.

6.6 Sub-drainageways

The conceptual planning for the watershed also included the evaluation of sub-drainageways, those drainageways that are not shown on the Conceptual Design Plan and Profiles and those drainageways that collect and convey runoff from sub-basins greater than 100 acres. Summarized on Tables VI-1 through VI-5 is design data for each sub-drainageway that collect and convey runoff from areas generally greater than 100 acres. The sub-drainages will almost always lie downstream of a FSD storage basin. Peak discharge data for the existing development condition was used to size the channel sections summarized on the Tables.

6.7 Grade Control

Grade control structures have been conceptually sited along the major drainageways and appear on the Conceptual Design plan and profiles. These structures are required to achieve and/or maintain the design slope, or to maintain the invert of a channel that is proposed to remain natural. Grade control may be needed at approaches to roadway crossings in order to gain headroom for the culvert as it passes beneath the roadway. Sloping drops are recommended and should be constructed out of grouted boulders. Maximum drop height for the stabilization of the low flow channel associated with the floodplain preservation concept was limited to three feet. The maximum drop height for a benched channel section was set at 6 feet. Typical details for a low flow drop, with and without a plunge pool, and for a sloping bounder drop are contained with the Conceptual Design Plan and Profiles.

6.8 Water Quality

Improvement of urban stormwater quality has become an important issue in drainage basin planning. Many pollutants are naturally associated with sediments that enter sensitive receiving waters. The pollutants are naturally occurring compounds that are carried to the drainageways in storm runoff. Other pollutants are the result of urbanization such as lawn chemicals, oil and grease, pet feces, lawn clippings and other items. Many pollutants can be limited by programs such as erosion control at construction sites, educational programs to inform the public as to the proper use of lawn chemicals, oil recycling programs and street sweeping programs. Even with these programs in place, erosion along the drainageways can generate large quantities of sediment that can settle out along the downstream channel bottoms.

The primary active water quality measure identified in this DBPS will be a capture pool inside each of the FSD basins. An advantage of the FSD basin is that it combines the water quality capture along with the EURV storage pool. The EURV should be determined using the methods outlined above and should have an outlet structure that will release the EURV volume over a 70-hour period.

6.9 Trails

As mentioned previously providing multi-use trails along the drainageways is desirable especially along the main stem of Jimmy Camp Creek and its major sub-tributaries. While providing access to the channels for maintenance, these trails could provide access to the other regionally planned trails, provide linkages through open spaces between smaller parks and opens spaces, and provide linkages between the opens spaces created by the FSD storage basins. Accordingly, a maintenance trail has been shown on the typical benched channel wherever this section may have been proposed. Trails alongside or within a floodplain will need to be located so as to provide maintenance access to the low flow but will need to be planned so that they minimize or avoid impact to riparian vegetation that may exist within the floodplain subject to preservation. The layout of a trail along a drainageway should be carried out taking into account hydraulic considerations, utilities in the area, access to dedicated parks and roadway crossings. Trails can meander within the floodplain or channel benches as well. They should be constructed out of asphalt or concrete when they are on the bench or where a trail approaches the low flow area of the floodplain or at the approach to a roadway crossing.

6.10 Maintenance and Re-vegetation

Maintenance of drainageway facilities is essential in preventing long term degradation of the creek and overbank areas. Along the drainageways, clearing of debris and dead vegetation should be considered within the low flow area of the creek and its tributaries. Trimming and thinning of shrubs and trees should be carried out if greater visual and physical access to the floodplain and low flow area is desired. On the overbanks, limited maintenance of the existing vegetative cover is recommended. Yearly clearing of trash and debris at roadway crossings is also recommended to ensure the design capacity of the crossing, and to enhance the crossings for trail users if a trail exists. In reaches that are to be selectively lined or the floodplain is to be preserved maintenance activities should be carried out while minimizing the disturbances to native vegetation.

The maintenance of the appurtenances within FSD basins should be carried out twice a year at a minimum to assure proper functioning of the EURV outlet structure. Trash racks and perforated plates should be cleared of debris. Sediment that has accumulated in the micro-pool and pre-sedimentation basins should be removed bi-annually as well. It is recommended that the full spectrum detention basins if built in accordance with the design standards and criteria should become the long-term responsibility of a public or quasi-public entity. Proper function of the FSD's is a critical element of the overall plan for stormwater management within the Jimmy Camp Creek basin.

The City of Colorado Springs has developed standard operation procedures for inspection and maintenance of storage facilities that would include FSD basins. The procedures manual outlines the requirements for access and easements to storage sites. The requirements for personnel, equipment, safety, maintenance activities, restoration and rehabilitation of storage facilities are all identified in the procedures manual. Each FSD basin will need to have an operations and maintenance manual prepared at the time that the final drainage report and construction plans prepared as part of the land development approval process.

6.11 Right-of-way

For the most part the main channels within the watershed that pass through the developed portions of the basin should be contained within dedicated drainage tracts, easements or right-of-ways. For FSD basins the right-of-ways or tracts should at a minimum encapsulate the 100-year storage pool. The land underlying the facility should be dedicated to the appropriate public agency so that maintenance access is assured. 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 more narrow dedicated right-of-way along the low flow area of the drainageway should be obtained through the land development process.

VII. IMPLEMENTATION OF SELECTED PLAN

7.1 General

The results of the analyses summarized in Chapter 6 represent a concept level design process. The selected plan improvements shown on the conceptual design drawings will be subject to refinement as the development of the land within the Jimmy Camp Creek Basin commences. The size and location of the channel conveyances will have to be determined based upon a higher level of engineering analysis that is typically carried out during the preparation of the master development drainage and final drainage planning reports. It is an underlying intent of the selected to plan to preserve to the greatest extent practical the existing condition 100-year floodplain and environmental resources that exist therein. It will be important that the major drainageway channel conveyances that have been identified in this DBPS be followed and major deviations from the concepts presented herein should be discouraged when land development applications are made to the City of Colorado Springs.

With respect to FSD as presented in this DBPS, the location of future FSD basins will be refined during the land development process. Guidelines for locating FSD's have been provided in previous sections of the DBPS. If implemented, FSD will result in the limitation of peak discharges released from developing areas to pre-development conditions. As such, the future major drainageway conveyances and road crossings need only to be designed to be able to carry the pre-development condition discharges. Consolidation of FSD sites should be encouraged in order to limit long-term maintenance costs so long as the intent of the FSD system is achieved. Implementation of the concepts in this DBPS will reduce the level of planning and engineering that will be required during later drainage planning phases associated with the land development process.

7.2 Cost Estimates

Presented on Table VII-1 are the costs estimates for the major drainageway conveyances for Jimmy Camp Creek and its major sub-tributaries within the City of Colorado Springs. Presented on Table VII-2 are conveyance costs for sub-drainageways for the City of Colorado Springs. There has been no cost estimate made for local storm sewer systems. An estimate for the cost to replace roadway crossings found to be deficient when the hydraulic analysis was prepared has also not been made in this DBPS. Unit costs applied when calculating the conveyance costs are prepared on the tables. Engineering design costs have been estimated at 10 percent of the construction. A contingency allowance of 10 percent off the construction has been assumed. No allowance for the relocation of utilities has been assumed when developing the conveyance cost estimates.

Presented on tables within the DBPS are costs estimates for the major drainageway conveyances for Jimmy Camp Creek and its major sub-tributaries within the City of Colorado Springs. There has been no cost estimate made for local storm sewer systems. An estimate for the cost to replace roadway crossings found to be deficient when the hydraulic analysis was prepared has also not been made in this DBPS. Unit costs applied when calculating the conveyance costs are prepared on the tables. The estimated cost of the FSD basins was presented in Chapter 5 of the DBPS. The cost and acreage data associated with FSD has been provided in the DBPS and used in the development of a storage fee. Since the effect of implementing the FSD alternative is to maintain rates of runoff to be conveyed by the receiving drainageways to pre-development conditions it is has been concluded to be reasonable to spread only the cost of the major drainage conveyances in amongst all un-platted property within Colorado Springs.

The total cost for future roadway culverts and bridges has not been made in this DBPS. This is primarily because the number and location of the future roadway crossing cannot be accurately determined at this time. All future roadway crossings should be sized to convey the pre-development condition discharge. Because runoff will be controlled to existing peak discharges, there is no additional costs for culverts and bridges associated with providing capacity because of increased runoff due to development.

7.3 Unplatted Acreage

Presented on Figure VII-1 are the jurisdictional limits and corresponding acreage of the three governmental entities in the Jimmy Camp Creek watershed. Presented on Figure VII-2 are the un-plattable acreage that lies within the City of Colorado Springs, City of Fountain and El Paso County. Using El Paso County Tax Assessor maps, plats and ownership records the amount of un-platted and developable acreage was estimated. From these records the following total un-platted acreages were determined:

City of Colorado Spring outside BLR	**148 acres**
City of Colorado Spring inside BLR	**13,341acres**
City of Colorado Springs Total	**13,489 acres**
El Paso County	**14,018 acres**
City of Fountain	**664 acres**

The unplatted acreage shown on Figure VII-2 excludes the existing 100-year floodplains, large regional parks, school sites and public utility easement corridors Land that is already platted has not been accounted for in the estimate of the plattable acreage unless the platted parcel exceeded 15 acres in size. Most of these large acreage platted parcels occur within the County. The un-platted acreage listed in the report is the land that is considered developable and would be subject to drainage and storage fees.

The weighted percent imperviousness was estimated for the entire watershed. Based upon the land use planning information accumulated and applied in this DBPS, the weighted percent imperviousness for the watershed was determined to be 57.5 percent.

7.4 Unit Drainage Costs

Presented on Table VII-3 of the DBPS and this Executive Summary are the unit major drainageway and FSD storage fee calculations for the City of Colorado Springs. All of the improvements that were used in the calculation of the unit drainage costs are considered public facilities subject to maintenance by the Colorado Springs in accordance with this DBPS and applicable drainage criteria. The unit drainage costs can

Table VII-3: Jimmy Camp Creek Major Drainageway and FSD Storage Fees
Jimmy Camp Creek Drainage Basin Planning Study

City of Colorado Springs

Major Drainageway Unit Fee		
Major Drainageway Conveyances	$	63,160,818
Sub-drainageway Conveyances	$	24,772,830
Total	$	87,933,648
Un-platted acreage		13489
Major drainageway unit fee: $/acre	$	6,519

FSD Unit Storage Fee		
FSD Basin Costs w/15% engr and contigency	$	59,872,220
Total plattable acreage in basin		28171
Total plattable acreage in Colorado Springs		13489
Ratio of total plattable acreage in Colorado Springs		0.48
City of Colorado Springs Share of storage co	$	28,668,360
FSD Storage fee: $/acre	$	2,125

be used to structure a fee system for the Jimmy Camp Creek watershed to replace the present fee system that has been established using the 1987 Wilson DBPS. It is recommended that a drainage fee be established within each of the jurisdictions to cover the capital improvement costs associated with the stabilization of the major and sub-drainageways identified in this DBPS. Since FSD is the selected storage option for the watershed, it may be possible to have the fees associated with the unit drainage costs accumulate during the initial phases of land development until such time that major drainageway or sub-drainageway stabilization is needed. Having the drainage fund accumulate by not requiring a developer to install major drainageway improvements during the initial phase of the land development process will help the keep the drainage fund from becoming immediately in debt. It will also give the City time and some greater flexibility in focusing the capital improvement funds generated by the fee system. Managing the fees system in this way may also help the land development process by not front-end loading the very initial phases of development with the costs of major and sub-drainageway improvements that could very well be offsite from the land development activity itself.

The FSD storage cost can be used to develop a FSD storage fee. The unit storage fee can be assessed at the time of platting if the parcel subject to platting is so limited in size as to not to be feasible to site a regional FSD. In developing the FSD unit storage fee 15 percent has been added to the unit acre-foot construction cost presented on Table V-4 of the DBPS to bring the unit storage cost to 2014 dollars. Fees that accumulate in the FSD storage fund could later be used to reimburse a property owner that would be required because of its size to construct and FSD. It is however preferable to construct the regional FSD's at the earliest possible time during the development of a sub-watershed so that the impact of develop runoff on the receiving drainageway is mitigated.

Because the land area within the watershed and the land that is within the City is controlled by one major land owner it may be feasible to "close" the basin to fees. This would then end the need to collect drainage and FSD fees at the time of platting land. Accordingly, no reimbursement for any public major drainageway or FSD facilities would occur.

A bridge fee has not been calculated for this watershed. This is primarily because the number and location of bridges cannot be accurately determined, and the fact that any bridge or major roadway crossing would only have to be sized to convey pre-development condition discharges. In this regard, the cost of a bridge or culvert associated with a future road is based on the need for transportation and not storm water conveyance. It may be necessary to establish some form of interim fee to cover the cost of reimbursements already established under the present Jimmy Camp Creek bridge fee system.