

21.0 JIMMY CAMP CREEK

21.1 Watershed Description

Jimmy Camp Creek is an intermittent stream located in the eastern portion of the Fountain Creek watershed and flows southwesterly from the open plains east of Colorado Springs. The Jimmy Camp Creek watershed has a contributing drainage area of approximately 68 square miles at its confluence with Fountain Creek near downtown Fountain. The headwaters of Jimmy Camp Creek are dominated grassland on undeveloped large-acreage tracts in unincorporated El Paso County. The middle portion of the Jimmy Camp Creek watershed contains some developments with single-family homes on rural residential lots, but it is still mostly grassland on undeveloped large-acreage tracts. The lower portion of Jimmy Camp Creek contains the city of Fountain and more developed commercial, industrial, and residential areas.

21.2 Scope of Study

The portion of Jimmy Camp Creek investigated for this study is approximately 8.6 miles long starting at its confluence with Fountain Creek and extending upstream to Fountain Boulevard. A location map of the studied reach of Jimmy Camp Creek is provided in Figure 21-1.

21.3 Channel Description

Jimmy Camp Creek varies greatly within the scope of study from Fountain Boulevard to its confluence with Fountain Creek. In the upper portion of the studied reach, the channel decreases in depth and width with a sandy bed. It should be noted that within the upper portion of the studied reach, loss in flow occurs due to soils with high infiltration rates. Within the lower portion of the studied reach, Jimmy Camp Creek is a meandering channel with dense vegetation.

21.4 Hydrology

The Jimmy Camp Creek HEC-HMS model was run for both existing and future conditions applying a 24-hour storm event with 2-, 5-, 10-, 25-, 50-, 100-, and 500-year recurrence intervals. The details related to hydrologic analysis of Jimmy Camp Creek are provided in a separate study report *Fountain Creek Watershed Preliminary Hydrology Report, URS, 2005*.

Table 21-1 summarizes URS-modeled peak discharges at points of interest throughout the Jimmy Camp Creek watershed.

Table 21-1. Peak Discharges at Points of Interest Along Jimmy Camp Creek

Location	HEC-HMS Element	Area (mi ²)	Existing Peak Discharge (cfs)							Future Peak Discharge (cfs)						
			2-yr	5-yr	10-yr	25-yr	50-yr	100-yr	500-yr	2-yr	5-yr	10-yr	25-yr	50-yr	100-yr	500-yr
Downstream of CO 94	JC030	3.1	5.9	53	200	800	1,200	1,700	2,800	260	820	1,300	2,400	3,000	3,500	4,900
Upstream of Drennan Rd	RJC093	14.4	23	200	600	2,000	2,900	4,300	8,100	660	1,900	3,000	5,400	6,700	8,100	11,000
East of Fontaine Blvd	JJC260	44.5	320	1,400	3,400	9,400	13,000	17,000	28,000	1,500	5,100	8,600	17,000	22,000	27,000	38,000
At Link Rd	JJC370	62.6	310	1,800	4,100	11,000	16,000	21,000	35,000	1,500	5,400	9,200	19,000	25,000	30,000	45,000
At Railroad Bridge (USGS Jimmy Camp Creek Gauge)	JJC400	67.6	300	1,800	4,100	11,000	16,000	22,000	36,000	1,500	5,500	9,400	19,000	25,000	31,000	46,000
At confluence with Fountain Creek	JmyCmp	68.3	300	1,800	4,100	11,000	16,000	22,000	35,000	1,500	5,500	9,400	19,000	25,000	31,000	46,000

Notes:
mi² = square miles
cfs = cubic feet per second
yr = year
USGS = United States Geological Survey

21.5 Hydraulics

Hydraulic conditions and water surface profiles for Jimmy Camp Creek were determined by HEC-RAS modeling. The average channel slope for Jimmy Camp Creek was determined to be approximately 0.5%. Below is a brief description of different parameters involved in the modeling of Jimmy Camp Creek.

21.5.1 Manning's n-Values

Manning's n-values for the main channel and overbank portions of Jimmy Camp Creek were determined by field observation and aerial photographs. In the upper portion of the studied reach, the channel decreases in depth and width, with a sandy bed and minor-to-moderate vegetation along overbank areas. Therefore, for most of the upper portion of the studied reach, Manning's n-values of 0.02 and 0.08-0.12 were used for the main channel and overbank portions, respectively. Within the lower portion of the scope of study, the Jimmy Camp Creek is a meandering channel with dense vegetation along overbank areas. Therefore, for most of the lower portion of the studied reach, Manning's n-values of 0.03-0.05 and 0.08-0.14 were used for the main channel and overbank portions, respectively. Table 21-2 outlines the extent of n-values along Jimmy Camp Creek's main channel and overbank areas.

Table 21-2. Summary of Manning's Roughness n-Values Along Jimmy Camp Creek

	Manning's n-Values (Overbank Portions)	Manning's n-Values (Main Channel)
Minimum	0.08	0.02
Maximum	0.14	0.05

21.5.2 Boundary Conditions

The Jimmy Camp Creek modeling was performed under the mixed-flow regime that allows the HEC-RAS model to model both the supercritical and subcritical regimes. The downstream boundary condition for all the profiles for Jimmy Camp Creek was set at normal depth based on the downstream channel slope of 0.0001 feet/feet. The upstream boundary condition for all the profiles of the Jimmy Camp Creek was also set at the normal depth based on the upstream channel slope of 0.004 feet/feet.

21.5.3 Culvert Data

The information related to FHWA Chart and Scale numbers and entrance and exit loss coefficients for Jimmy Camp Creek is summarized in Table 21-3.

Table 21-3. Summary of Culvert Data for Jimmy Camp Creek

Culvert Location	FHWA Chart Number	FHWA Scale Number	Entrance Loss Coefficient	Exit Loss Coefficient
Peaceful Valley Rd	2	2	0.7	1

21.5.4 Study Results

Water surface profiles for Jimmy Camp Creek for the return intervals of 2-, 5-, 10-, 25-, 50-, 100-, and 500-years for the existing and future conditions are presented in Appendix R. Complete HEC-RAS output tables for Jimmy Camp Creek for existing and future conditions are also provided in Appendix R. Tables 21-4 and 21-5 summarize hydraulic conditions from the Jimmy Camp Creek model results. This includes average channel velocity, average flow depth, average top width, and average Froude number for existing and future conditions, respectively. The average value for each of these parameters was computed by dividing the total sum of each parameter by the total number of cross sections at Jimmy Camp Creek.

**Table 21-4. Hydraulic Conditions in Jimmy Camp Creek from HEC-RAS Results
(Existing Conditions)**

Hydraulic Parameter	10-Year Event	100-Year Event	500-Year Event
Average channel velocity (ft/sec)	7.9	12.0	13.2
Average water surface depth in channel (ft)	5.7	11.9	14.7
Average top width (ft)	563.4	1568.4	2018.4
Average Froude number	0.7	0.7	0.7

Note:
ft/s = feet per second

**Table 21-5. Hydraulic Conditions in Jimmy Camp Creek from HEC-RAS Results
(Future Conditions)**

Hydraulic Parameter	10-Year Event	100-Year Event	500-Year Event
Average channel velocity (ft/sec)	10.2	12.9	13.8
Average water surface depth in channel (ft)	8.1	13.8	16.4
Average top width (ft)	937.9	1866.8	2327.7
Average Froude number	0.7	0.7	0.7

Note:
ft/s = feet per second

22.0 SHOOKS RUN

22.1 Watershed Description

Shooks Run is divided into two separate and distinct reaches: North Shooks Run (Templeton Gap Drainage Basin) and South Shooks Run. The Templeton Gap Drainage Basin (North Shooks Run) originates in northeastern Colorado Springs in a cuplike depression, surrounded by hills. It flows southwesterly along and parallel to Templeton Gap Road, until it is intercepted by the Templeton Gap Floodway and diverted northwesterly to Monument Creek. South Shooks Run begins at the Templeton Gap Floodway and runs parallel to Templeton Gap Road for approximately 2 miles before flowing southerly for approximately 5 miles to join Fountain Creek near the southern corporate limits, approximately 0.25 mile east of U.S. highway 85-87. The watershed basin for South Shooks Run is bounded generally on the north by the Templeton Gap drainage basin; on the east by the Sand Creek drainage basin and the Spring Creek drainage basin; on the west by an unnamed drainage basin; and on the south by the Spring Creek drainage basin and an unnamed drainage basin that is a tributary to Fountain Creek. The basin slopes to the southwest, from a 6,565-foot elevation at the high point of the basin in Palmer Park to a 5,885-foot elevation at the outfall of the drainage way into Fountain Creek along the north side of I-25, about 0.25 mile east of Nevada Avenue. The stream originates in a residential area and throughout its course flows through residential or business areas.

22.2 Scope of Study

Shooks Run is divided into two separate and distinct reaches: North Shooks Run and South Shooks Run. However, for the current study, the hydraulic analysis is performed only for South Shooks Run (hereafter referenced as Shooks Run). The portion of Shooks Run investigated for this study is approximately 4.4 miles long starting at its confluence with Fountain Creek and extending upstream to a point just upstream of East LaSalle Road. A location map of the studied reach of Shooks Run is provided in Figure 22-1.

22.3 Channel Description

Shooks Run originates in a residential area and throughout its course flows through residential or business areas. Most of the main channel is a narrow, deep, eroded channel that is choked with overgrown native vegetation and significant amounts of trash and debris. The channel

banks are unstable and consist of old uncontrolled fill material that was placed as adjacent development encroached into the drainageway. Some overbank areas experience moderate-to-high vegetation. There have been some makeshift attempts to stabilize the channel with riprap and retaining structures in isolated locations. The channel meanders through a mix of land use, consisting of industrial/commercial development, residential neighborhoods, and public park areas. There are many existing buildings immediately adjacent to the top of the channel banks. The roadway and railroad crossings were constructed many years ago, and most are significantly undersized and deteriorated. Within the upstream portion of the studied reach, Shooks Run passes through a 0.25 mile reach of Petty Jewett Golf Course, adjacent to Union Boulevard. Within Petty Jewett Golf Course, Shooks Run is channelized properly and is confined in a concrete channel for nearly all of its length.

22.4 Site-Specific Issues

There currently exists a drop structure at Shooks Run approximately 0.1 mile downstream of East Espanola Street and 0.50 mile upstream of East San Miguel Street. The main channel invert for Shooks Run drops by approximately 7.4 feet at the drop structure. In order to determine a more accurate profile upstream of and through the drop structure, the drop structure was modeled in HEC-RAS model by incorporating a series of cross sections along the drop structure. The actual surveying information was available only at the upstream and the downstream ends of the drop structure. Additional cross sections were added at the slope and further downstream of the drop structure using the available topographic sources to accurately predict the location of hydraulic jump.

22.5 Hydrology

No hydrologic analysis was performed for Shooks Run as a part of the current study. Per a meeting with the local sponsors held in June 2004, URS used the hydrologic flows from the Drainage Basin Planning Study (DBPS) report prepared by Wilson & Company for the City of Colorado Springs in September 1993. However, the DBPS report provides only 10- and 100-year flows for the existing and future conditions. Therefore, the hydraulic analysis for Shooks Run for the current study was performed for 10- and 100-year flows for the existing and future conditions. Table 22-1 summarizes URS-modeled peak discharges at points of interest along the studied reach throughout the Shooks Run watershed.

Table 22-1. Peak Discharges at Points of Interest Along Shooks Run

Design Point/Location	Existing Condition Peak Discharge (cfs) (24-hour)		Future Condition Peak Discharge (cfs) (24-hour)	
	10-Year Event	100-Year Event	10-Year Event	100-Year Event
Downstream of Patty Jewett Golf Course	1070	3575	1105	3665
Uintah St	1135	3850	1170	3940
Cache La Poudre St	1235	4205	1275	4305
Boulder St	1520	4915	1535	5020
Bijou St	1550	4955	1550	5060
Pikes Peak Ave	3040	6650	2915	6705
Costilla St	3065	6765	2945	6820
Fountain Blvd	3100	6880	2985	6940
Fountain Creek	3050	6840	2950	6910

22.6 Hydraulics

Hydraulic conditions and water surface profiles for Shooks Run were determined by HEC-RAS modeling. The average channel slope for Shooks Run was determined to be approximately 1%. Below is a brief description of different parameters involved in the modeling of Shooks Run.

22.6.1 Manning's n-Values

Manning's n-values for the main channel and overbank portions of Shooks Run were determined by field observation and aerial photographs. Shooks Run originates in a residential area and throughout its course flows through residential or business areas in a narrow, deep, eroded channel. The main channel of the stream is choked with overgrown native vegetation and significant amounts of trash and debris. It experiences moderate-to-dense vegetation along overbank areas. In addition, the channel banks are unstable and consist of old uncontrolled fill material that was placed as adjacent development encroached into the drainageway. Therefore, for most of the studied reach, n-values of 0.045 and 0.08-0.14 were used for the main channel and overbank portions, respectively. However, within the upstream portion of the studied reach, Shooks Run passes through a 0.25 mile reach of a golf course in which it is channelized properly and is confined in a concrete channel for

nearly all of its length. Accordingly, for that portion of the stream, n-values of 0.016 and 0.035 were used for the main channel and overbank portions, respectively. Table 22-2 outlines the extent of n-values along Shooks Run's main channel and overbank areas.

Table 22-2. Summary of Manning's Roughness n-Values Along Shooks Run

	Manning's n-Values (Overbank Portions)	Manning's n-Values (Main Channel)
Minimum	0.08	0.016
Maximum	0.14	0.045

22.6.2 Boundary Conditions

The Shooks Run modeling was performed under the mixed-flow regime that allows the HEC-RAS model to model both the supercritical and subcritical regimes. The downstream boundary condition for all the profiles for Shooks Run was set at normal depth based on the downstream channel slope of 0.0054 feet/foot. The upstream boundary condition for all the profiles for Shooks Run was also set at the normal depth based on the upstream channel slope of 0.015 feet/foot.

22.6.3 Culvert Data

The information related to FHWA Chart and Scale numbers and entrance and exit loss coefficients for Shooks Run is summarized in Table 22-3.

Table 22-3. Summary of Culvert Data for Shooks Run

Culvert Location	FHWA Chart Number	FHWA Scale Number	Entrance Loss Coefficient	Exit Loss Coefficient
North Trail of Lasalle	2	3	0.9	1
E Lasalle	29	1	0.5	1
E Monroe	29	1	0.5	1
E Madison Street	29	1	0.5	1
Jefferson St	29	1	0.5	1
Paseo Rd	8	1	0.4	1
Pedestrian Bridge	34	1	0.5	1
Espanola St	8	2	0.4	1
E San Miguel St	29	1	0.5	1
E Uintah St	29	1	0.5	1
E Cache La Poudre St	29	3	0.2	1
E Willamette St	29	1	0.5	1
Pearl St	29	1	0.5	1
E Boulder St	29	1	0.5	1
E Bijou St	29	1	0.5	1
E Castilla St	29	1	0.5	1
Fountain Blvd	29	1	0.5	1
Old Railroad	30	1	0.5	1
Railroad	30	1	0.5	1
Las Vegas St	8	2	0.4	1

RAS output tables for Shooks Run for the existing and future conditions are also provided in Appendix S. Tables 22-4 and 22-5 summarize hydraulic conditions from the Shooks Run model results. This includes average channel velocity, average flow depth, average top width, and average Froude number for existing and future conditions, respectively. The average value for each of these parameters was computed by dividing the total sum of each parameter by the total number of cross sections at Shooks Run.

Table 22-4. Hydraulic Conditions in Shooks Run from HEC-RAS Results (Existing Conditions)

Hydraulic Parameter	10-Year Event	100-Year Event
Average channel velocity (ft/sec)	8.6	10.8
Average water surface depth in channel (ft)	7.4	12.7
Average top width (ft)	178.3	395.4
Average Froude number	0.7	0.6

Note:
ft/s = feet per second

Table 22-5. Hydraulic Conditions in Shooks Run from HEC-RAS Results (Future Conditions)

Hydraulic Parameter	10-Year Event	100-Year Event
Average channel velocity (ft/sec)	8.8	10.9
Average water surface depth in channel (ft)	7.3	12.8
Average top width (ft)	179.3	399.6
Average Froude number	0.7	0.6

Note:
ft/s = feet per second

22.6.4 Study Results

Water surface profiles for Shooks Run for the return intervals of 2-, 5-, 10-, 25-, 50-, 100-, and 500-years for the existing and future conditions are presented in Appendix S. Complete HEC-

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23.0 SAND CREEK (MAIN STEM)

23.1 Watershed Description

The 60.8 square mile Sand Creek Watershed lies generally along the eastern edge of the developed portions of the City of Colorado Springs. Of the total, 26.7 square miles lie within the East Fork and 7.9 square miles in Peterson Field. This section of the report is specific to the Main Stem Channel. The Main Stem headwaters are in the Black Forest near the intersection of Vollmer Road and Shoup Road. Runoff flows generally in a southerly direction from the Black Forest across mainly open grassland with some 2- to 5-acre residential lot developments. The middle third of the Main Stem continues to flow south through newer residential urban development and receives runoff from the well-established urban residential areas of eastern Colorado Springs. The lower third of the watershed for the Main Stem flows southwesterly through established urban areas consisting of residential and commercial development. The confluence of the East Fork with the Main Stem is located in this lower third of the watershed. The outfall of the watershed is at the confluence with Fountain Creek in southern Colorado Springs approximately one mile north of the intersection of Interstate 25 and Academy Blvd. The confluence with Peterson Field is approximately one-quarter mile upstream of the confluence with Fountain Creek.

23.2 Scope of Study

The portion of Main Stem investigated for this study is approximately 13.7 miles long starting at its confluence with Fountain Creek and extending upstream to its headwaters. A location map of the studied reach of Sand Creek is provided in Figure 23-1.

23.3 Channel Description

In general, the Main Stem contains both natural and improved channel sections with wide connected floodplains. Within the upper portions of the watershed, located primarily on private lands, the channel appears to be a fairly wide grass swale. The channel, within the middle portions of the basin, tends to be further developed with a sandy channel bottom and light to moderate vegetation located in the floodplains. The floodplain extents tend to narrow towards the lower portions of the basin because of urban development. Where disturbed by development, the channels have become incised due to highly erodible soils when not covered by established vegetation. Head cutting downstream of the crossing structures is obvious in many locations. Numerous check structures have been constructed in the lower

reaches of the channel bottom in an effort to alleviate this problem. The road crossing structures have also acted as check structures. There are many bank erosion protection structures that have been placed in the developed areas. The channel side slopes change constantly along the Main Stem in this lower region. Within the upper and middle portions of these reaches, the side slopes can vary from a range of nearly vertical walls, to channel sections with approximate 4H:1V side slopes. The channel bottom width also varies from 3 to 5 feet wide. The lower portions of the Main Stem, located between the confluence of the East Fork and the confluence of Fountain Creek, primarily contain a straightened man-made channel with sand beds and moderately dense vegetation along the sides of the channel. Also within the lower reaches of the creek, the improved channel becomes wider, up to 300 feet, with a depth of 15 feet, and with variable side slopes that range from a vertical walled channel to one with 2H:1V side slopes. In general, the riparian vegetation in the floodplains contain mainly grass, willows and in some areas dense shrubbery.

23.4 Site Specific Issues

The following describes several areas where site specific geometry was used to generate portions of the HEC-RAS model. They are as follows:

1) Upon reviewing the field reconnaissance photos, it was determined that one recently constructed engineered channel and one major drop structure existed but the available topography did not include them. The channel is located on the Main Stem between Galley Road and Palmer Park Boulevard. The HEC-RAS model for this channel contains geometry obtained from a separate DTM created from construction plan information for the recently constructed engineering channel. The construction plan information was obtained through the Pikes Peak Regional Building Department's Floodplain Management Office. For the drop structure, located directly downstream of the Stetson Hills Boulevard bridge crossing, additional survey data was obtained and included in the HEC-RAS model.

2) At the Constitution Avenue crossing, there currently exists a large flood control facility. Constitution Avenue acts as the embankment dam for this detention pond. This structure was not included in the HEC-RAS model. However, the HEC-HMS model does contain the detention facility, and has modeled this structure. Therefore the detention facility was modeled as a flow change location at the next downstream cross-section of Constitution Avenue.

3) Upstream of the N. Carefree Rd. crossing, plans exist to construct a flood control facility, as discussed in the *Fountain Creek Watershed Preliminary Hydrology Report, URS, 2005*. The Future

Conditions HEC-HMS model, found in that report, includes this facility. As a result of the flood control facility the computed Future Conditions flowrates are less than the computed flowrates for the Existing Conditions.

23.5 Hydrology

The Sand Creek HEC-HMS model was run for the existing and future conditions for the 2-, 5-, 10-25-, 50-, 100-, and 500-year return-period storms. The details related to hydrologic analysis of Sand Creek are provided in a separate study report *Fountain Creek Watershed Preliminary Hydrology Report, URS, 2005*. The Main Stem contains various flow change locations due to existing and future detention pond sites, along with tributaries entering the creek at numerous sites. A listing of these flows can be found in a separate study report, *Fountain Creek Watershed Preliminary Hydrology Report, URS, 2005*. The Main Stem contains various flow change locations due to existing and future detention pond sites along with tributaries entering the creek at numerous sites. A listing of these flows can be found in Table 23-1.

Table 23-1. Peak Discharges at Points of Interest along the Sand Creek Main Stem.

Location	HEC-HMS Element	Area (mi ²)	Existing Peak Discharge (cfs)							Future Peak Discharge(cfs)						
			2-yr	5-yr	10-yr	25-yr	50-yr	100-yr	500-yr	2-yr	5-yr	10-yr	25-yr	50-yr	100-yr	500-yr
Downstream of Woodmen Road	JSC060	4.6	36	130	230	490	660	850	1,300	35	130	230	490	660	840	1,300
Upstream of Stetson Hills Boulevard	JSC105	9.3	64	240	410	900	1,200	1,600	2,700	190	470	700	1,300	1,600	1,900	3,000
At Carefree Circle	JSC130 or Det Prop Sky Sox	12.8	870	1,700	2,400	3,800	4,600	5,400	7,400	180	540	890	2,200	3,100	3,900	5,000
At Constitution Avenue	Detention Const	14.7	660	1,500	2,100	3,000	3,400	3,800	6,800	200	640	1,000	2,300	2,800	3,400	4,900
Downstream of Platte Avenue	JSC200	16.6	1,000	1,800	2,300	3,400	3,900	4,500	7,100	1,000	1,800	2,300	3,400	3,900	4,500	5,800
At Confluence with East Sand Creek	JSC210	22.9	3,900	6,900	9,000	13,000	16,000	18,000	23,000	3,900	6,900	9,000	13,000	16,000	18,000	23,000
At South Academy Boulevard	JSC220	51.0	5,300	9,300	12,000	18,000	22,000	25,000	34,000	5,300	9,300	12,000	19,000	22,000	26,000	34,000
At Confluence with Fountain Creek	SandCrk	60.8	6,000	10,000	14,000	21,000	25,000	29,000	38,000	5,900	10,000	14,000	21,000	25,000	29,000	38,000

Ref: Table 22-14 – Fountain Creek Watershed Hydrology Report, URS, 2005

Notes: mi² = square miles
cfs = cubic feet per second
yr = year

23.6 Hydraulics

Hydraulic conditions and water surface profiles within the main channel of the Main Stem were determined by using HEC-RAS version 3.1.3. The following sections provide a summary of the models characteristics, input data, and site specific issues encountered while creating the model.

23.6.1 Model Characteristics

Stream centerlines and cross-section cut-lines were defined for the Main Stem using Bentley Microstation version 8.05. The cross-section cut-lines were spaced approximately 500 feet apart, with additional cross-sections inserted as required near drop structures, bridges, culverts or abnormal changes in stream geometry. The stream centerlines were delineated based on the available digital contour data along with the before mentioned orthophotography and field reconnaissance photos.

The cross-section cut-lines and stream centerlines were used to extract the cross-section geometry data for import into HEC-RAS. This was done using Bentley InRoads version 8.05. The extraction utilized a digital terrain model (DTM) created from the available 10m DEM data of the watershed. Once the cross-section data was imported into the HEC-RAS model, additional cross-sections were added to the model. These cross-sections contain geometry based on the Surveyed Structures ASCII files and in several cases were combined with the DTM. The Surveyed Structures ASCII files contain the bridge, culvert, or other structure geometry. This combining of data was performed for those cross-sections, in which the surveyed geometry did not contain the 500-yr event and/or where existing structures are located. In order to ensure that the largest events were modeled by HEC-RAS, the cut-lines were extended further into the boundaries of the floodplains using data from the DTM. The data entered into the model was also checked with the field reconnaissance photos for each structure.

In some instances it was discovered that the surveyed crossing culverts at a given location were various sizes. HEC-RAS can successfully model various sized culverts, but will treat the individual culverts as a separate group. Therefore a supercritical profile cannot be determined when there is more than one culvert group at a crossing location. The result of this produces a graphical error in the HEC-RAS Profile Output plots. In order to circumvent this issue, it was determined that several culverts would need to be treated as a single culvert group with the same opening sizes, and same downstream and upstream inverts. This was performed in order to allow the HEC-RAS model to compute a uniform WSEL at the downstream end of the crossing culvert, in which no graphical error would display.

However, this was not performed for the crossing culverts located at N. Carefree Road due to the extreme variation in sizes and elevations.

For modeling purposes, the crossing culverts at N. Carefree Road were left in their original surveyed configurations. The box culverts consist of six barrels, with the two outer barrel inlet invert elevations much higher than that of the inside barrels. The outer barrels serve as a dry pathway for pedestrians and allow access to either side of N. Carefree Road during low flow events. It was determined that these culverts need to be modeled in their existing condition and not as a culvert group to correctly model actual field hydraulics. However, when modeled in this condition, the graphical error will be produced by HEC-RAS for low flows. It was determined that the inside barrels will pass the Existing Conditions 2-year and the Future Conditions 2- and 5-year flow rates, without reaching the elevation of the outer two barrels. Therefore, in order to compute a WSEL, free of the HEC-RAS graphical error, the outer two barrels were removed from the modeling. The culvert crossing was then analyzed with the inner barrels only, by a separate analysis containing the Existing Conditions 2-year event only. Similar culvert analysis was performed for the Future Conditions 2- and 5-year events also. The results of these separate analyses are included as additional data sheets containing a single water surface profile, multiple cross-sections, and a tabular data set in Appendix T.

Bank stations, reach lengths, Manning's n-values, potential levees and ineffective flow areas were input to the HEC-RAS model based on topography, orthophotography, and the field photos. The hydraulic structures were modeled as if they were free of any major obstructions. This would reflect properly maintained conditions and is generally valid for most bridges on the project streams. However, some of the culverts have reduced capacities due to sedimentation, subsequent vegetation growth, and the accumulation of debris. Cleaning and maintaining these culverts is imperative to restore their flood flow capacities.

23.6.2 Manning's 'n'

Manning's roughness n-values for the main channel and overbank portions of Main Stem were determined by field observation and aerial photographs. The n-values for the Main Stem vary from 0.02 to 0.1 for the main channel portion and from 0.05 to 0.07 for the overbank areas. Table 23-2 outlines the extent of n-values along the Main Stem of Sand Creek's main channel and overbank areas.

Table 23-2. Summary of Manning's Roughness n-Values Along Sand Creek (Main Stem)

	Manning's n-Values (Overbank Portions)	Manning's n-Values (Main Channel)
Minimum	0.05	0.02
Maximum	0.07	0.10

23.6.3 Boundary Conditions

The hydraulic modeling for Main Stem was performed under the mixed-flow regime that allows the HEC-RAS model to model both the supercritical and subcritical regimes. The downstream boundary condition for all the profiles for Main Stem was set at normal depth based on the downstream channel slope of 0.014 feet/feet. The upstream boundary condition for all the profiles for Main Stem was also set at the normal depth based on the upstream channel slope of 0.015 feet/feet.

23.6.4 Culvert Data

The information related to the FHWA Chart and Scale numbers and entrance and exit loss coefficients for the culverts on the Main Stem are provided in Table 23-3.

Table 23-3. Summary of Structure Locations for Sand Creek (Main Stem)

Structure Location	FHWA Chart Number	FHWA Scale Number	Entrance Loss Coefficient	Exit Loss Coefficient
Hancock Ave culvert	8	2	0.4	1
Chelton Rd culvert	8	1	0.4	1
Powers Blvd culvert	8	1	0.5	1
Waynoka Rd culvert	8	1	0.5	1
N Carefree Rd culvert	8	1	0.4	1

23.6.5 Study Results

Water surface profiles and cross-sections for the return intervals of 2-, 5-, 10-, 25-, 50-, 100- and 500-years for the existing and future conditions of the Main Stem are presented in Appendix T. Complete HEC-RAS output tables for the existing and future conditions are also provided in Appendix T. Tables 23-4 and 23-5 summarize hydraulic conditions for the Sand Creek Main Stem model results. This includes average channel velocity, average flow depth, average top width, and average Froude number

for existing and future conditions, respectively. The average value for each of these parameters was computed by dividing the total sum of each parameter by the total number of cross sections at Sand Creek.

Table 23-4. Hydraulic Conditions in the Sand Creek (Main Stem) from the HEC-RAS Results (Existing Conditions)

Hydraulic Parameter	10-Year Event	100-Year Event	500-Year Event
Average channel velocity (ft/sec)	11.8	14.8	15.9
Average water surface depth in channel (ft)	3.6	6.0	7.1
Average top width (ft)	142.3	227.6	275.3
Average Froude number	1.3	1.3	1.3

Note:
ft/s = feet per second

Table 23-5. Hydraulic Conditions in the Sand Creek (Main Stem) from the HEC-RAS Results (Future Conditions)

Hydraulic Parameter	10-Year Event	100-Year Event	500-Year Event
Average channel velocity (ft/sec)	12.0	14.8	15.9
Average water surface depth in channel (ft)	3.8	6.0	7.4
Average top width (ft)	145.9	235.3	287.3
Average Froude number	1.3	1.3	1.2

Note:
ft/s = feet per second

The water surface profiles, cross sections, and tabular data sections also include additional data sets that depict the results of the separate 2-yr Existing Conditions flow rate analysis. Also included are additional data sets for the separate 2- and 5-yr Future Conditions flow rate analysis. These additional profiles, cross-sections, and tabular data sections can be found on the page(s) immediately following the River Station (RS) number of the N. Carefree Road crossing, located in each respective Appendix section. The additional data has been limited to the following RS numbers, and they are as follows:

1. Existing Conditions (2-yr only analysis):

- Water Surface Profile: RS 44231.03 through 47738.84
- Cross Sections: RS 45725.10 through 46940.07 BR
- Tabular Data: RS 46271.06 through 46882.60

2. Future Conditions (2-yr and 5-yr only analysis):

- Water Surface Profile: RS 45725.10 through 49316.31
- Cross Sections: RS 45725.10 through 46940.07 BR
- Tabular Data: RS 46249.70 through 46817.88

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24.0 EAST FORK SAND CREEK

24.1 Watershed Description

The East Fork headwaters are located near the southern edge of the Black Forest south of Arroya Lane. As previously stated, the 26.7 square mile East Fork Watershed is located in the Sand Creek watershed. The upper third of the East Fork flows through open grassland. The middle third of the East Fork flows through both open grassland and manmade channels. The lower third of the watershed for the East Fork flows southwesterly through established urban areas consisting of residential and commercial development. The confluence of the East Fork Sand Creek and the Sand Creek Main Stem is located upstream of Fountain Boulevard.

24.2 Scope of Study

The portion of East Fork Sand Creek investigated for this study is approximately 12.4 miles long starting at its confluence with Sand Creek and extending upstream to its headwaters. A location map of the studied reach of East Fork Sand Creek is provided in Figure 24-1.

24.3 Channel Description

The East Fork contains both natural and improved channel sections with wide connected floodplains. The upper portions of the channel appear to be a fairly wide grass swale. Within the middle portions of the basin, the channel contains a sandy channel bottom with light to moderate vegetation located in the floodplains. Where altered by development, the channels have become incised due to the erodibility of the existing soils. As in the Main Stem, numerous check structures have been constructed in the middle and lower reaches of the East Fork at the channel bottom to alleviate this problem. The lower reaches contain primarily a straightened channel with a sandy bottom. The channel side slopes change constantly along the East Fork. Within the upper and middle portions of these reaches, the side slopes can vary from a range of nearly vertical walls to channel sections with approximate 4H:1V side slopes. The channel bottom width also varies from 3 to 5 feet wide. Within the lower reaches of the East Fork, the improved channel becomes wider, up to 50 feet, with a depth up to 20 feet, and variable side slopes that range from a vertical walled channel to 2H:1V side slopes. In general, the riparian vegetation in the floodplains contains mainly grass, willows, and in some areas fairly dense shrubbery.

24.4 Hydrology

The East Fork Sand Creek HEC-HMS model was run for the existing and future conditions for the 2-, 5-, 10-, 25-, 50-, 100-, and 500-year return-period storms. A general summary of the modeled flow for the East Fork at its confluence with the Main Stem is provided in Table 24-1.

The East Fork contains various flow change locations due to existing and future detention pond sites, along with tributaries entering the creek at numerous sites. A listing of these flows can be found in Table 24-1. A separate analysis was performed in order to determine the flows for input into a separate subreach along the East Fork. This will be discussed in Section 24.4.4.

Table 24-1. Peak Discharges at Points of Interest Along the East Fork Sand Creek

Location	HEC-HMS Element	Area (mi ²)	Existing Peak Discharge (cfs)							Future Peak Discharge (cfs)						
			2-yr	5-yr	10-yr	25-yr	50-yr	100-yr	500-yr	2-yr	5-yr	10-yr	25-yr	50-yr	100-yr	500-yr
Downstream of Constitution Ave	JES140	14.0	27	160	350	1,200	1,800	2,500	4,400	700	1,200	1,600	2,400	2,800	3,200	4,300
At Marksheffel Rd	JES240	20.6	130	380	630	1,800	2,600	3,600	6,300	1,100	1,900	2,500	3,900	4,600	5,400	7,100
At Hwy	JES250	23.7	400	860	1,200	2,100	2,900	4,000	6,900	1,200	2,100	2,800	4,500	5,400	6,300	8,600
Downstream of Powers Blvd	JES270	24.4	430	940	1,300	2,300	2,900	4,000	6,900	1,200	2,100	2,900	4,500	5,400	6,400	8,700
At confluence with Sand Creek	ESandCrk	26.7	1,100	2,000	2,700	4,500	5,800	7,000	9,900	1,300	2,300	3,100	4,900	6,100	7,300	10,000

Ref: Table 22-15 – Fountain Creek Watershed Hydrology Report, URS, 2005

Notes:

- mi² = square miles
- cfs = cubic feet per second
- yr = year

24.5 Hydraulics

Hydraulic conditions and water surface profiles for the East Fork of Sand Creek were determined by using HEC-RAS. The following sections provide a summary of characteristics, input data, and site-specific issues encountered while creating the model.

24.5.1 Model Characteristics

The same methods were used in extracting the stream centerlines and cross-section cut-line data for the East Fork Reach as for the Main Stem Reach. Also, the Surveyed Structure ASCII file data were input in the same manner as in the Main Stem. In order to ensure that the largest events were modeled by HEC-RAS along the East Fork, the cut-lines were extended further into the boundaries of the floodplains using the DTM. The data entered into the model were also checked with the field reconnaissance photos for each structure.

In general, the upstream and downstream boundary conditions for the East Fork were calculated for normal depth flow based on their respective upstream and downstream channel slope.

As in the Main Stem, it was discovered that the surveyed crossing culverts contained various sizes. HEC-RAS will successfully model the culverts, but will treat the individual culverts as a separate group. The result produces a graphical error in the HEC-RAS Profile Output plots. In order to circumvent this issue, it was determined that several culverts would need to be treated as a uniform culvert with the same opening sizes, and the same downstream and upstream inverts. This was performed in order to allow the HEC-RAS model to compute a uniform WSEL at the downstream end of the crossing culvert, in which no graphical error would display. However, this was not performed for the crossing culverts located at Peterson Road (RS 34712) and Marksheffel Road (RS 42639) due to their extreme variation in sizes and elevations.

24.5.2 Manning's n-Values

Manning's roughness n-values for the overbank portions and the main channel were determined by field observation and aerial photographs. The n-values for the East Fork vary from 0.02 to 0.1 for the main channel portion and from 0.05 to 0.07 for the overbank areas. Table 24-2 outlines the extent of n-values along East Fork Sand Creek's main channel and overbank areas.

Table 24-2. Summary of Manning's Roughness n-Values Along East Fork of Sand Creek

	Manning's n-Values (Overbank Portions)	Manning's n-Values (Main Channel)
Minimum	0.05	0.02
Maximum	0.07	0.10

24.5.3 Boundary Conditions

The hydraulic modeling for East Fork was performed under the mixed-flow regime that allows the HEC-RAS model to model both the supercritical and subcritical regimes. The downstream boundary condition for all the profiles for East Fork was set at normal depth based on the downstream channel slope of 0.009 feet/feet. The upstream boundary condition for all the profiles for East Fork was also set at the normal depth based on the upstream channel slope of 0.021 feet/feet.

24.5.4 Culvert Data

The information related to the FHWA Chart and Scale numbers and entrance and exit loss coefficients for the culverts on the East Fork is included in Table 24-3.

Table 24-3. Summary of Structure Locations for the East Fork

Culvert Location	FHWA Chart Number	FHWA Scale Number	Entrance Loss Coefficient	Exit Loss Coefficient
Highway 24 culvert	8	1	0.4	1
Peterson Rd. culvert	34	1	0.9	1
Marksheffel Rd. culvert	55	1	0.9	1
Constitution Ave culvert	8	1	0.4	1

24.5.5 Site-Specific Issues

The following describes several areas where site-specific geometry was used to generate portions of the HEC-RAS model.

1) The geometry of the East Fork drop structure, located between Power Boulevard and Stewart Road, does not permit the 500-year event computed WSEL to be fully contained within the surveyed geometry. Since the available topographic data do not show the correct contours for the drop

structure, the HEC-RAS model was allowed to vertically extend the cross-section geometry approximately 0.83 foot in order to contain this event.

2) A separate subreach was added to the East Fork near Peterson Road. This was done to represent the flowpaths for each of the events modeled as defined by the FEMA Flood Insurance Rate Maps (FIRMs). The FIRMs referenced are Map Numbers 08041C0752F and 08041C0754F. These maps show that the 100-year floodplain extends south and parallels Peterson Road, until it is intercepted by a drainage swale. This drainage swale parallels Highway 24 and directs the floodplain west until it reconnects to the main channel of the East Fork. This subreach for the East Fork required special inputs for its flow data. A special analysis was performed using the existing geometry of the area directly upstream of Peterson Road. This analysis utilized the existing geometry of HEC-RAS XS 34807.77. The XS was delineated along the Left Bank and extended east. This allowed the program to model a split flow at the LOB region. This computed LOB flow was then used as the input for the given events in the flow data editor of HEC-RAS for the existing and future conditions data files. A summary of these flows for the subreach of the East Fork at Peterson Road are provided in Table 24-4.

Table 24-4. Computed Flow Rates for Peterson Road Subreach

Return Period	LOB Discharge (cfs)	
	Existing Conditions	Future Conditions
	(cfs)	(cfs)
2-year	11	78
5-year	44	207
10-year	82	320
25-year	209	579
50-year	316	576
100-year	494	765
500-year	881	1541

for existing and future conditions, respectively. The average value for each of these parameters was computed by dividing the total sum of each parameter by the total number of cross sections at East Fork Sand Creek.

Table 24-5. Hydraulic Conditions in the East Fork of Sand Creek from HEC-RAS Results (Existing Conditions)

Hydraulic Parameter	10-Year Event	100-Year Event	500-Year Event
Average channel velocity (ft/sec)	5.8 ft/s	9.5 ft/s	11.2 ft/s
Average water surface depth in channel (ft)	2.1 ft	4.1 ft	5.2 ft
Average top width (ft)	129.8 ft	201.9 ft	253.1 ft
Average Froude number	1.0	1.0	1.1

Note:
ft/s = feet per second

Table 24-6. Hydraulic Conditions in the East Fork of Sand Creek from HEC-RAS Results (Future Conditions)

Hydraulic Parameter	10-Year Event	100-Year Event	500-Year Event
Average channel velocity (ft/sec)	8.4 ft/s	10.6 ft/s	11.7 ft/s
Average water surface depth in channel (ft)	3.3 ft	4.8 ft	5.5 ft
Average top width (ft)	168.4 ft	235.4 ft	265.6 ft
Average Froude number	1.0	1.0	1.1

Note:
ft/s = feet per second

24.5.6 Study Results

Water surface profiles and cross sections for the return intervals of 2-, 5-, 10-, 25-, 50-, 100-, and 500-years for the existing and future conditions of the East Fork are presented in Appendix U. Complete HEC-RAS output tables for the existing and future conditions are also provided in Appendix U. Tables 24-5 and 24-6 summarize the hydraulic conditions for the East Fork model results. This includes average channel velocity, average flow depth, average top width, and average Froude number

25.0 REFERENCES

- 1) Chow, V.T. 1959. *Open Channel Hydraulics*.
- 2) Jarret, Robert D. 1985. *Determination of Roughness Coefficients for Streams in Colorado*.
- 3) Simons, D. B.; E.V. Richardson, M. Albertson, and Robert Kodoatie. 2004. *Geomorphic, Hydrologic, Hydraulic and Sediment Transport Concepts Applied to Alluvial Rivers*.
- 4) U.S. Army Corps of Engineers. 2002. HEC-RAS Hydraulic Reference Manual- Version 3.1.
- 5) URS Corporation. December 2003. *United States Air Force Academy Floodplain Revisions – Phase II Final Report*
- 6) URS Corporation. September 2005. *Fountain Creek Watershed Preliminary Hydrology Report, URS, 2005*.
- 7) City of Pueblo, Colorado, Pueblo County. September 1986. *Flood Insurance Study Report*.

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