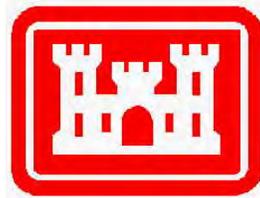


**Planning Assistance to States – Brush Creek Low Water
Crossing**

Mission Hills, Kansas



**US Army Corps of Engineers
Kansas City District**

March 2020

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1 Project Overview

Planning Assistance to State study for the city of Mission Hills, Kansas. Mission Hills has indicated the flooding that occurs at a low water crossing on State Park Road is a drain of municipality emergency response resources as well a public safety risk. The city of Mission Hills partnered with the U.S. Army Corps of Engineers and the AE firm Water Resources Solutions to conduct a planning level hydraulic analysis of the structure. A physical model and a 2D HEC-RAS model were created to investigate existing conditions as well as potential alleviative alternatives. Appendix A documents the physical model.

1.1 Study Authority

Planning Assistance to States Program (PAS/Section 22)

- Section 22 of the Water Resources Development Act of 1974 (Public Law 93-251), as amended, provides authority for the Corps to assist in the preparation of comprehensive plans for the development, utilization, and conservation of water and related land resources.
- Planning level of detail
- Analysis of existing data for planning purposes using standard engineering techniques

1.2 Previous Studies

- FEMA Flood Insurance Study, Revised 03 August 2009

1.3 Background

Mission Hills, Kansas is located in Johnson County and is part of the Kansas City Metropolitan Area. It is a relatively small municipality covering 2 square miles. The study location is centered in the Kansas City Country Club at the confluence of Rock Creek and Brush Creek. These creeks pass under a low water crossing on State Park Road. This road also forms the boundary between the city of Mission Hills and the city of Fairway.

A low water crossing is designed to overtop often. The crossing becomes a safety hazard when overtopped and must be closed by the city's emergency response personnel, such as police officers. The city's limited number of personnel combined with frequency of overtopping create a problem. Three alternatives have been investigated to alleviate this stress. The alternatives include 1) buyout option, 2) removal of the low water crossing, and 3) removal of the low water crossing with a State Park Road raise.

1.4 Existing Conditions

The attention of this study is focused on the confluence of Rock Creek and Brush Creek. These creeks are entirely urbanized and are known to be flashy. Rock Creek's headwaters are west of Mission, KS in Overland Park, KS while Brush Creek begins farther south in Overland Park, KS. Each creek has sections of natural channel bed, concrete channel, and grade control structures.

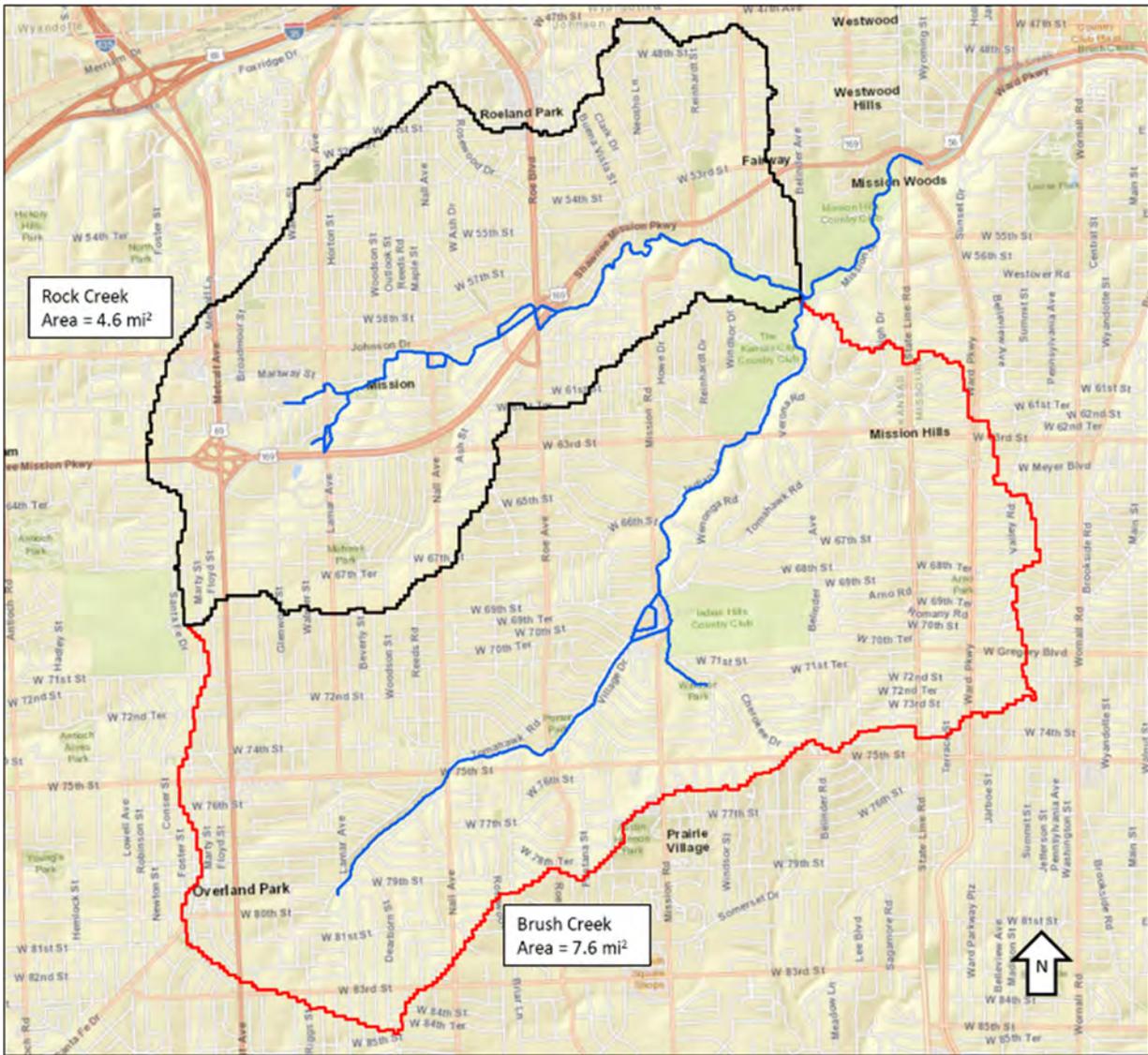


Figure 1 - Rock and Brush Creek Basins

Brush and Rock Creek are tributaries to the Blue River which is a tributary to the Missouri River. The soils in the detailed study area are primarily classified as silt or silty clay according to the NRCS soil survey. Depth to bedrock is unknown. Utilities in the proposed project area are electric, gas, communications, water, and sewer. The cities of Mission Hills and Fairway are both effected by Rock and Brush Creeks. Figure 2 is the Google Street View taken from the State Park Road Low Water Crossing.

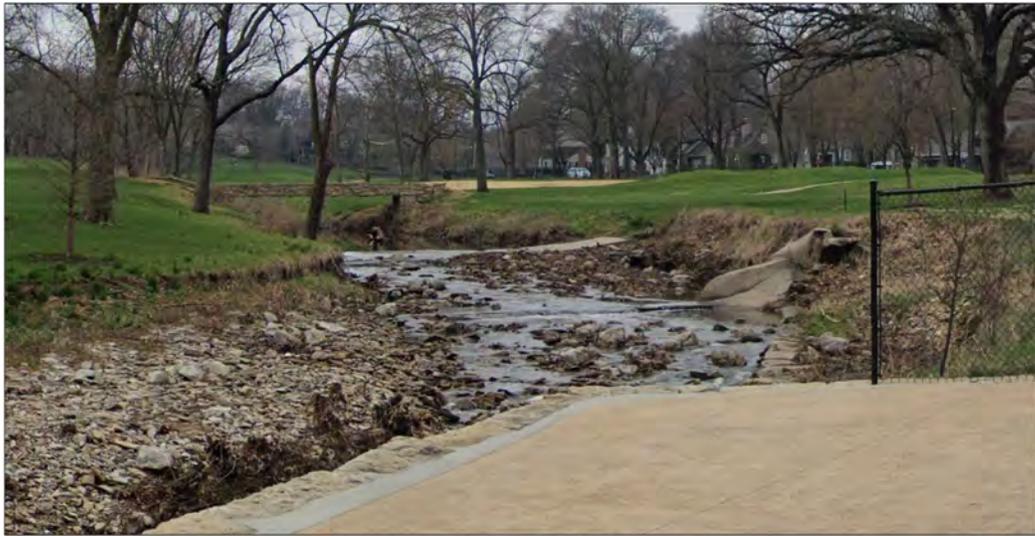


Figure 2 – Low Water Crossing Street view

The detailed study zone is outlined in Figure 3.



Figure 3 – Detailed Study Area

The channels in the detailed study area are less confined than in the upper portions of the basin. The stream banks are frequently overtopped in this portion of the basin. Numerous residential buildings are in the FEMA 10-year floodplain. There are four houses effected by the FEMA 100-year flood in this reach, three in Fairway and one in Mission Hills. One house in Fairway and the house in Mission Hills are also inundated by the 10-year FEMA flood. There are many low water crossings along both creeks that frequently overtop; some have a functioning automated warning system and others must be closed using city emergency response personnel.

1.5 Standards

- Johnson County Public Works – Street Construction and Storm Drainage Standards for Subdivisions - 2006
- Kansas Department of Transportation – Standard Specifications for State Road and Bridge Construction – 2015
- Kansas City Metro Chapter – American Public Works Association Specifications

1.6 Permits

- United States Army Corps of Engineers – Section 404 Permit
- Johnson County – Land Disturbance Permit

1.7 Utilities

Table 1 summarizes the potentially effected utilities and their contact information.

Table 1 – Utility Contacts

Utility	Company	Phone Number	Address
Electric	Evergy	888-471-5275	1200 Main St Kansas City, MO 64105
Gas	Kansas Gas Service	800-794-4780	11401 W 89th St Overland Park, KS 66214
Water	WaterOne	913-895-1800	10747 Renner Blvd Lenexa, KS 66219
Sewer	Johnson County Wastewater	913-715-8500	11811 S Sunset Dr Olathe, KS 66061
Communications	Google Fiber	877-454-6959	1814 Westport Rd Kansas City, MO 64111
Communications	AT&T	800-924-9420	9444 Nall St Overland Park, Kansas 66207
Communications	Spectrum	866-874-2389	550 Westport Rd Kansas City, MO 64111

1.8 Conformance with Watershed Studies

The hydraulic model created and analysis performed conform to USACE best engineering practices and use pre-approved USACE software.

The hydrologic data used in this study was obtained from the FEMA Flood Insurance Study model.

2 Summary of Findings

Table 2 summarizes water surface elevations from the USACE hydraulic analysis as well as water surface elevations from the FEMA effective model. Figure 4 demonstrates the output locations found in Table 2. The 100-year max flood extent is shown in Figure 5.

Table 2 – FEMA WSE vs USACE WSE

Location	10 year		50 year		100 year	
	USACE	FEMA	USACE	FEMA	USACE	FEMA
1	883.3	883	884.5	885	884.5	885
2	878.7	879.5	879.8	880	880	880
3	874.2	875	876.3	877	876.5	877
4	870.1	870	871.1	872	871.4	872
5	868.7	867.5	870.5	869	870.8	869.5
6	866	865.5	868.1	867.3	868.3	867.5
7	864.9	864.25	867.1	865.75	867.5	866.25
8	861.5	861	864.4	864	864.8	864.25

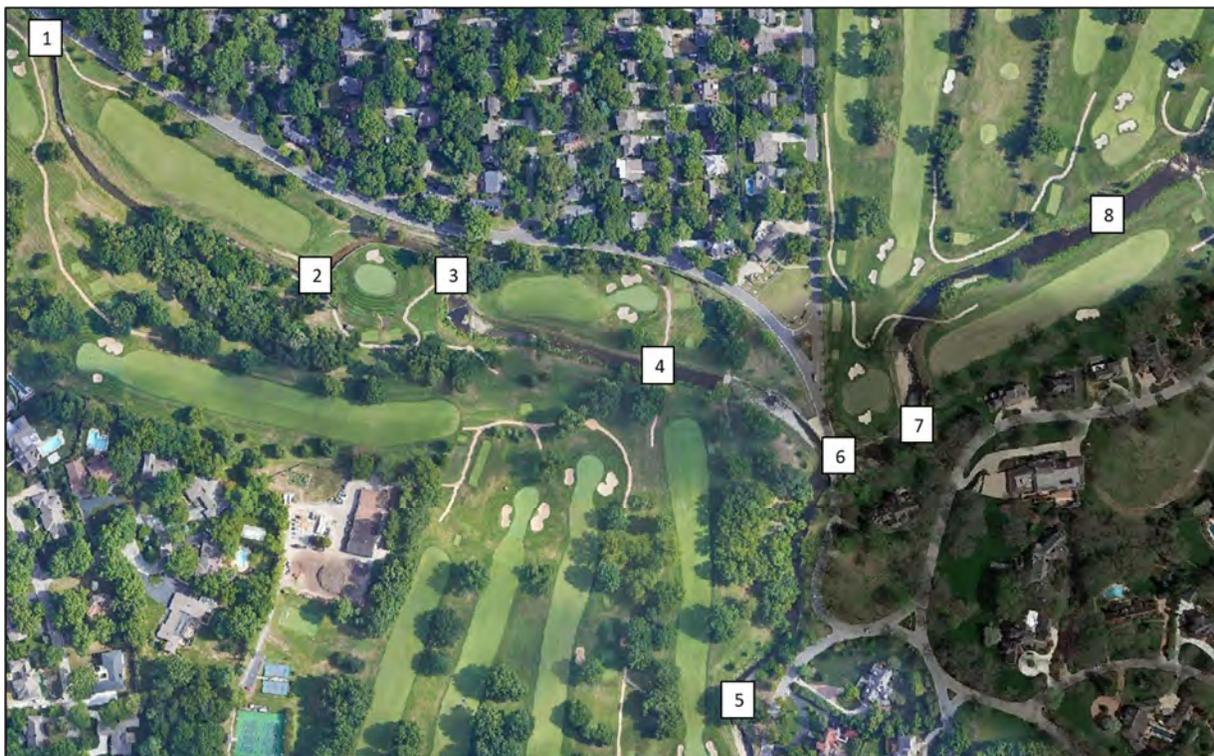


Figure 4 – FEMA vs USACE Output Locations

The general approach of a PAS study is to utilize as much existing data as possible. The pertinent data used in the USACE model are frequency flows from FEMA, a 1-foot lidar data set from Water Resource Solutions as well as a 1-meter LiDAR dataset from the State of Kansas GIS Data Access and Support Center (DASC). Using these data, the Corps created a 2D hydraulic model to further analyze the study reach. The output from the Corps model was compared to FEMA’s effective model to validate reasonableness. The findings of this analysis will help the city of Mission Hills and Johnson County Stormwater Management Advisory Council determine an appropriate path forward to mitigate the flooding concerns at the confluence of Rock and Brush Creek.



Figure 5 – 100 Year Flood

2.1 Project Limits

The scope of this project is limited to the low water crossing on State Park road at the confluence of Rock Creek and Brush Creek. The project is located entirely in Johnson County. The effected municipalities are Mission Hills and Fairway. USACE modeling shows no significant impact upstream of downstream effects of the proposed alternatives.

2.2 Hydrology and Hydraulics

No hydrologic analysis was performed for this study. All hydrologic data was obtained from FEMA. FEMA provided USACE with the latest steady HEC-RAS model that was used to create the Flood Insurance Study report revised 03 August 2009. The spatial reference used for the hydraulics analysis was State Plane Kansas North FIPS 1501. The horizontal units were US Feet. The horizontal datum was North American Datum 1983. The vertical datum was North American Vertical Datum 1988. All elevations in the report are referenced to the NAVD 88 datum.

2.2.1 HEC-RAS

USACE created a 2D HEC-RAS model for the hydraulic analyses portion of this study. The three main components of a 2D HEC-RAS model are boundary conditions, terrain, and a computational mesh. The

2D HEC-RAS model was limited to the same extents as the physical model constructed by Water Resource Solutions. Figure 6 shows this extent.



Figure 6 – 2D Perimeter

All plans were run using the full-momentum equations and a 0.5 second time step. The model was run long enough to achieve a stable steady state solution.

2.2.1.1 Boundary Conditions

Upstream boundary conditions for the 2D model were taken from the closest flow change location in FEMA’s steady HEC-RAS model. These flow change locations and their proximity to the 2D boundary are shown in Figure 7.

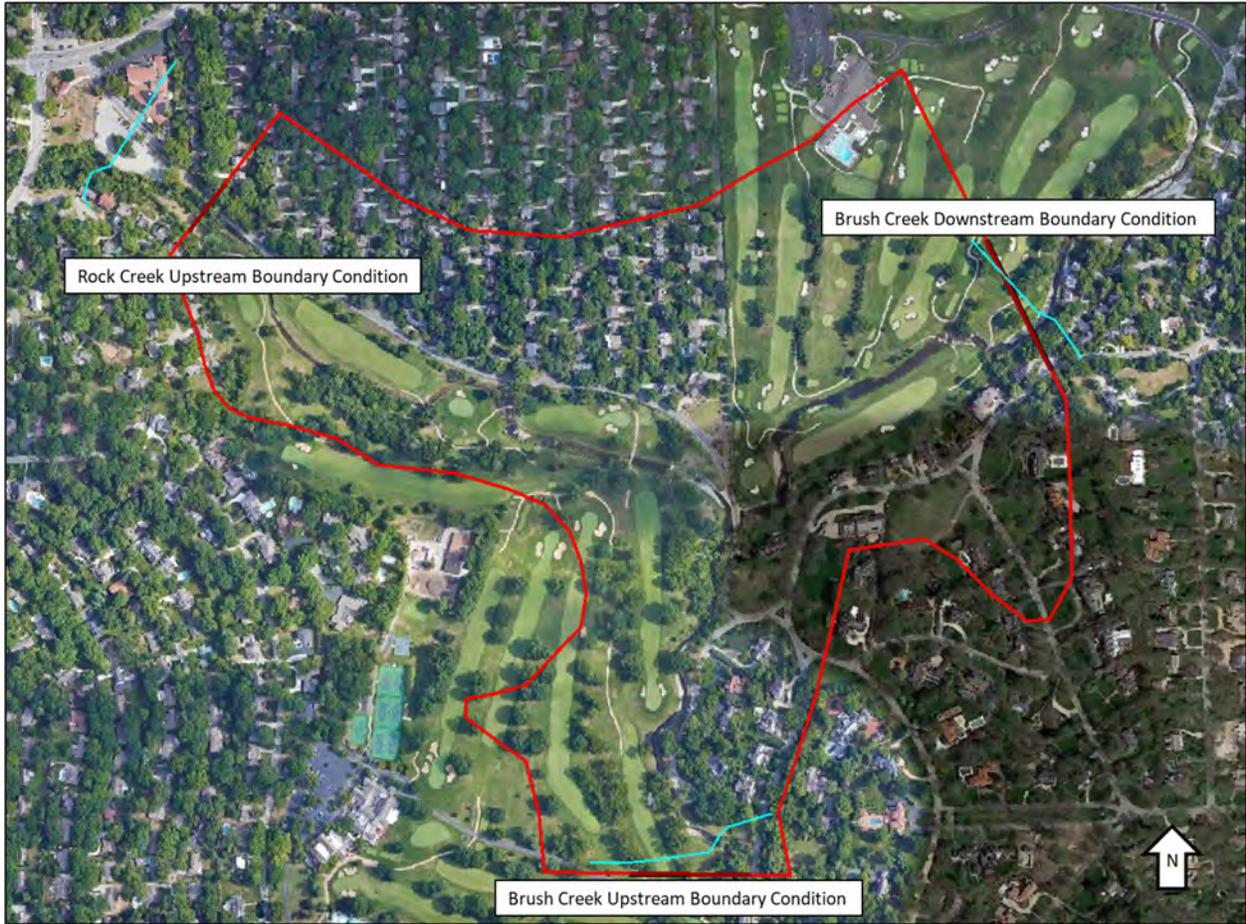


Figure 7 – 2D Model Boundary Conditions

The 10-, 50-, and 100-year flows were used for this study and are reported in Table 3.

Table 3 – FEMA Flood Flows

Flow	Rock Creek	Brush Creek
10 year	4590	5993
50 year	7318	9039
100 year	7855	9600

The rating curve used for the downstream boundary condition is shown in Figure 8.

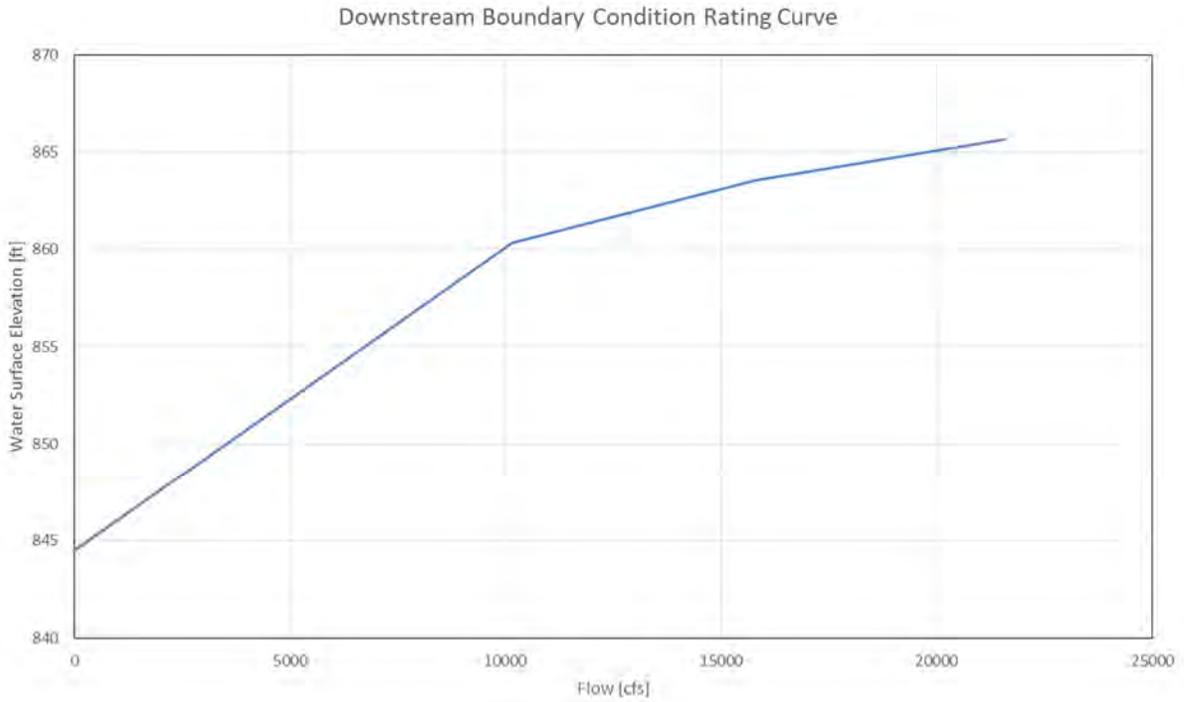


Figure 8 – Downstream Boundary Condition Rating Curve

2.2.1.2 Terrain

The terrain model is a major component of a 2D hydraulic model. The existing condition terrain model used in this study consists of a 1-foot LiDAR data set obtained from Water Resource Solutions combined with 2012 State of Kansas GIS DASC LiDAR. The elevations just upstream and downstream of the low water crossing were lowered to allow HEC-RAS to place the low water crossing at its proper invert and outlet elevations. Figure 9 shows the overall existing conditions terrain model plotted with 5-foot contours.

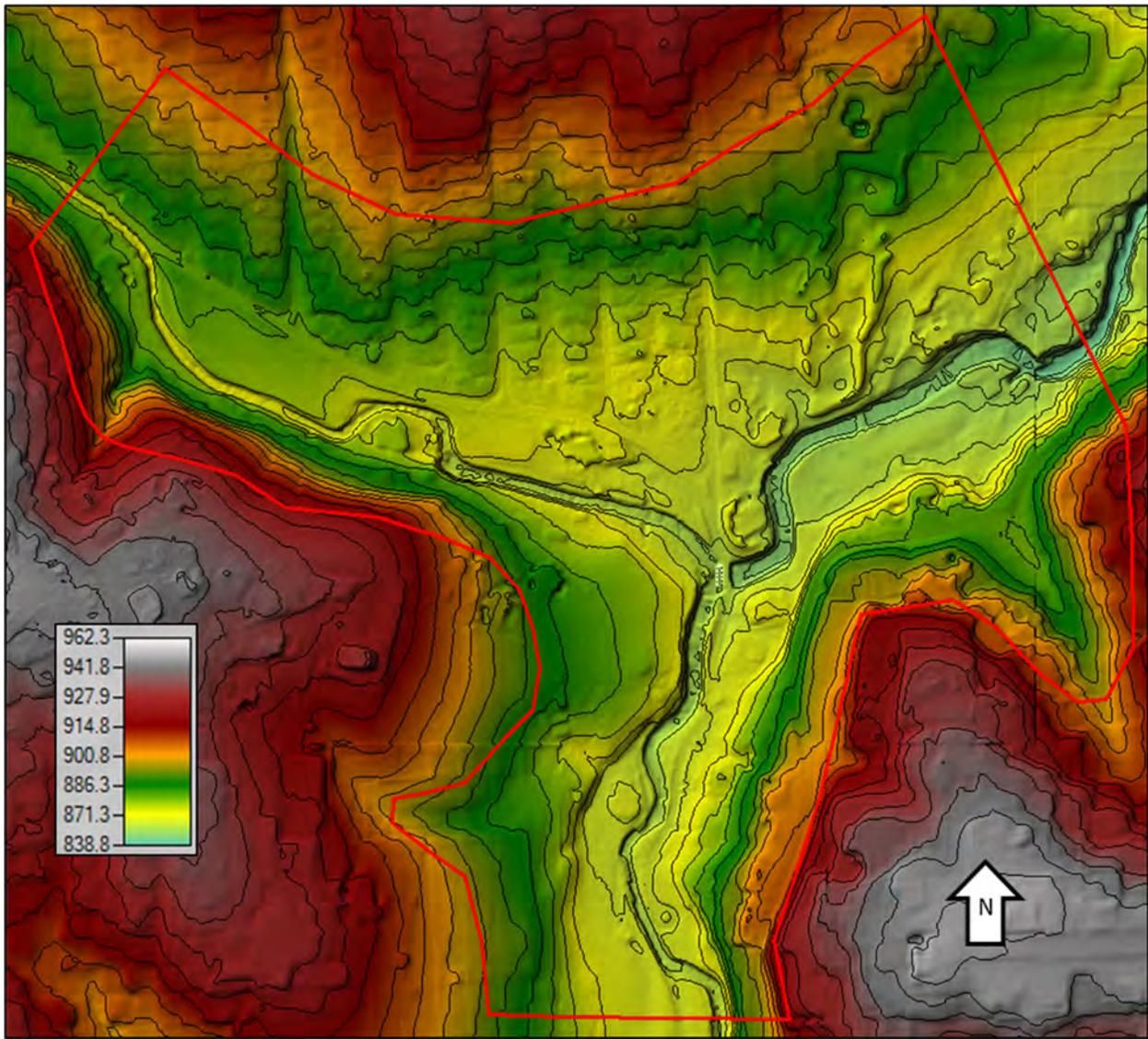


Figure 9 – Existing Condition Terrain with 5-ft contours

As part of a general investigation of the data available, the terrain used in the USACE model was compared to the cross sections used in the FEMA model. The USACE terrain and FEMA cross sections, with focus on the channel sections, are consistent with each other. Example cross sections for each reach are shown in Figure 11.

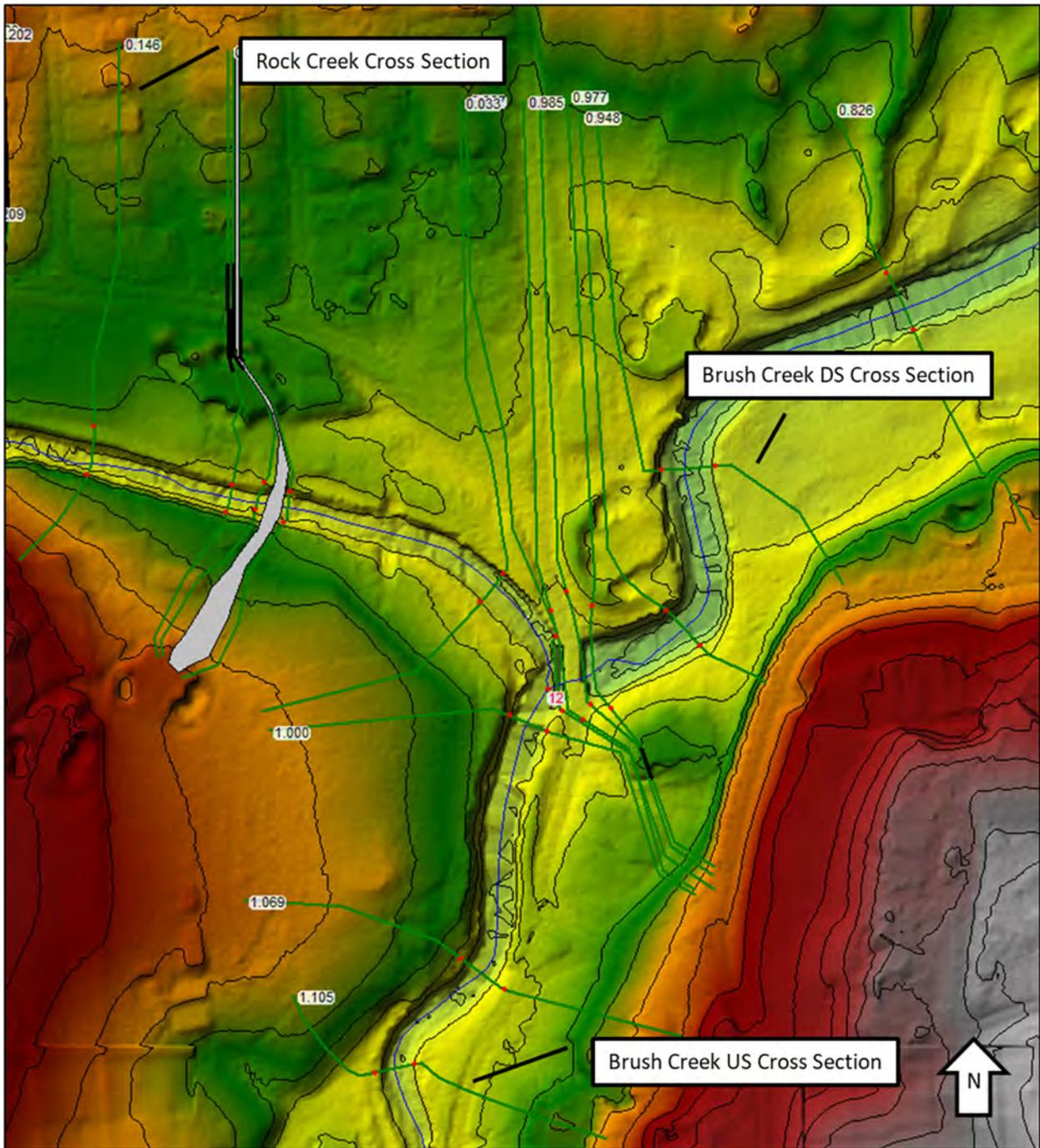


Figure 10 – FEMA Cross Sections

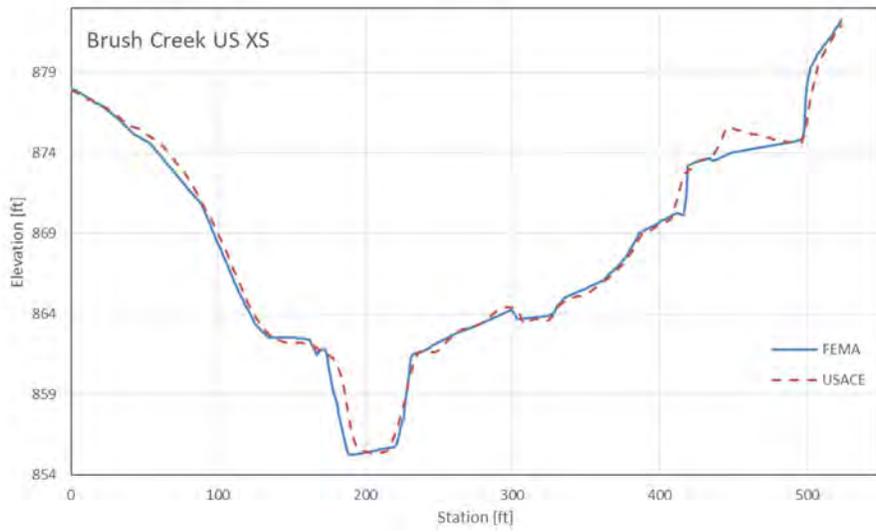
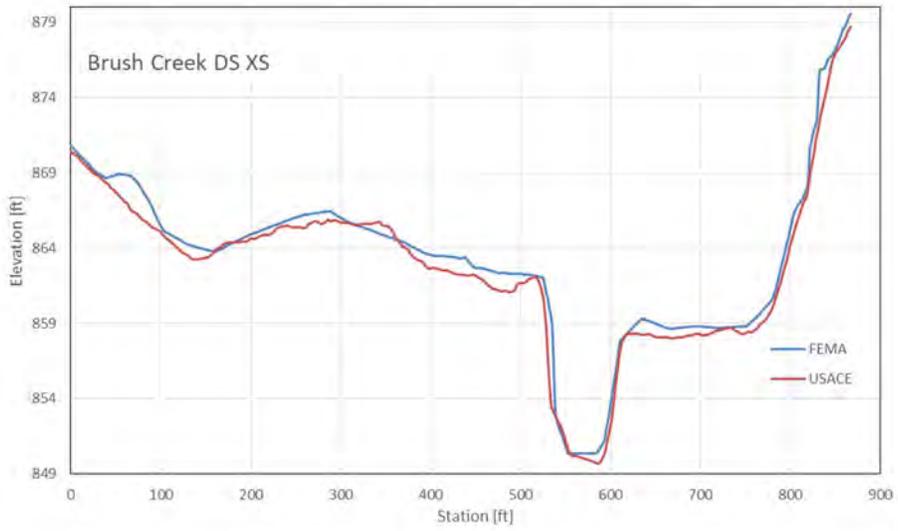
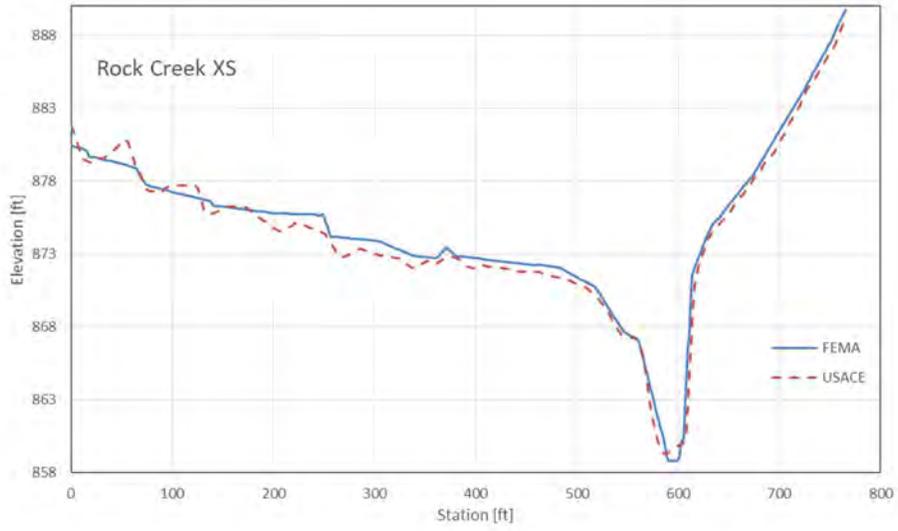


Figure 11 – FEMA Cross Section vs. USACE Terrain Comparison

A second terrain model was used to model an alternative in which the low water crossing is completely removed. Figure 12 shows the removed low water crossing as well as a profile plot of the removed crossing.

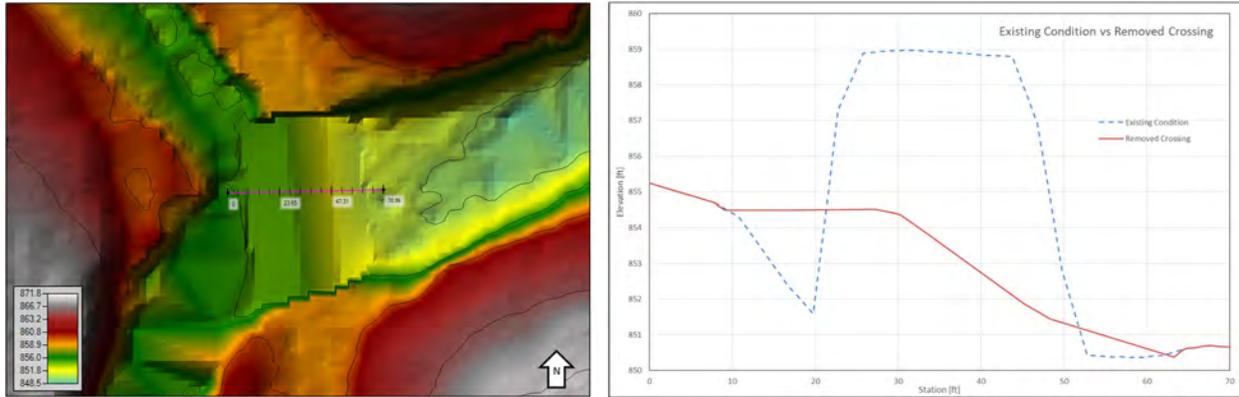


Figure 12 – Terrain showing removed Low Water Crossing

2.2.1.3 Manning's *n*

Roughness zones were delineated using aerial imagery. Roughness values were determined using engineering judgment. Calibration of roughness values was not possible due to lack of information. Figure 13 and Table 4 summarize the Manning roughness zones and values used in the 2D model.

Table 4 – Roughness Values

Roughness Zone	Value Used	Sensitivity
Channel	0.0375	0.03
Golf Course	0.045	0.035
Grass	0.05	0.0375
Residential	0.06	0.04
Road	0.02	0.01



Figure 13 – Manning Roughness Zones

In lieu of calibration data a sensitivity analysis was run on existing conditions and the removal of the low water crossing alternative. The water surface elevations for the existing condition, using the sensitivity values, were on average 0.5 feet lower than the values used for this study. The differences between the sensitivity existing conditions and the sensitivity removed low water crossing are the same, therefore the conclusions of this study and the Flood Problem Rating table would be the same. The difference in roughness had a negligible effect on inundation boundary. Velocities differed, ranging from 0.2 fps to 0.8 fps. Figure 14 is a cross section taken on top of the low water crossing, looking downstream. It shows the difference between the existing conditions and the removed low water crossing for chosen roughness values as well as the sensitivity test.

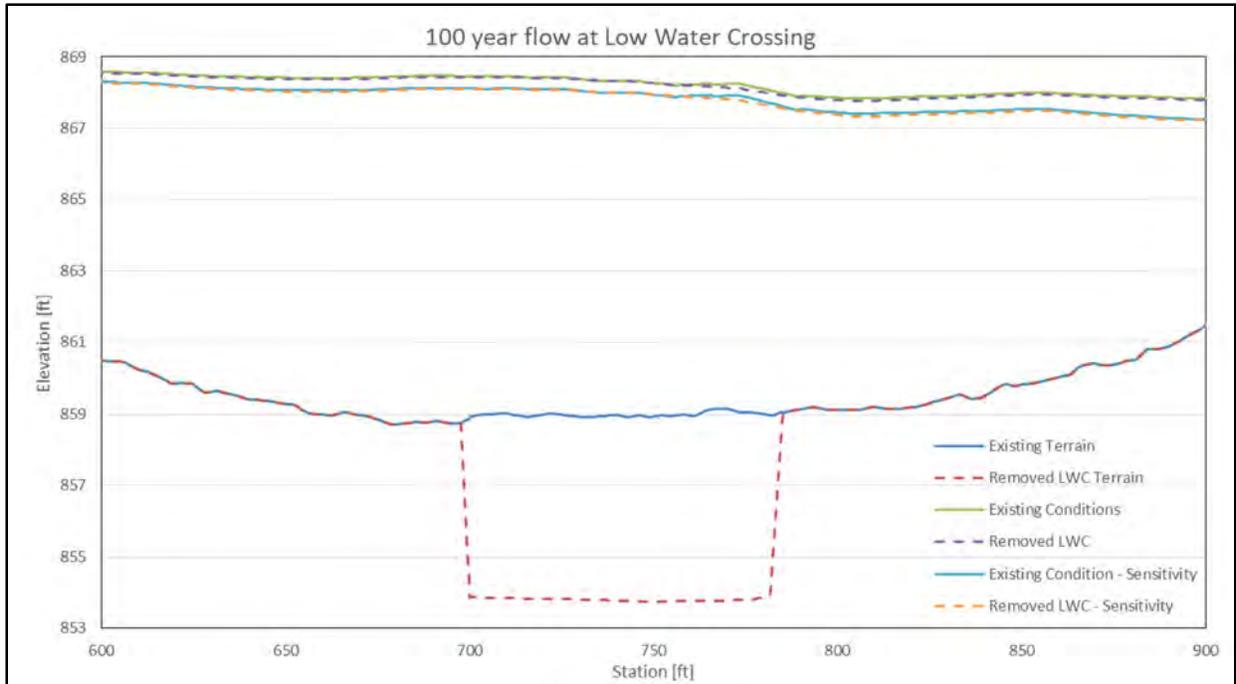


Figure 14 – Sensitivity Analysis at Low Water Crossing for 100-year flow

2.2.1.4 2D Mesh

The 2D HEC-RAS mesh consist of a 20-foot nominal cell size, breaklines and internal hydraulic structures. Breaklines were longitudinal and latitudinal to the channel. This aligns cell faces in the direction of flow which helps model stability when using the full momentum equations. Figure 15 shows an example of the mesh and breaklines along Brush Creek.



Figure 15 – 2D Mesh Example

Internal hydraulic structures are another major portion of this mesh. There are six total structures modeled. Four golf cart bridges and a low head weir that are found in the FEMA model as well as the low water crossing, the focus of this study. The internal structures were set to use the 2D equations for overtopping flow. The golf cart bridges were input as box culverts because HEC-RAS 2D does not currently support bridge structures within the 2D mesh. The dimensions of their openings were taken from the FEMA model. The average span is 40 foot and the rise is 8 foot; a 0.012 Manning's n was used as the culvert bottom. The low head weir was set to elevation 864.00 ft, the same as the FEMA model. Figure 16 shows the five structures along Rock Creek.

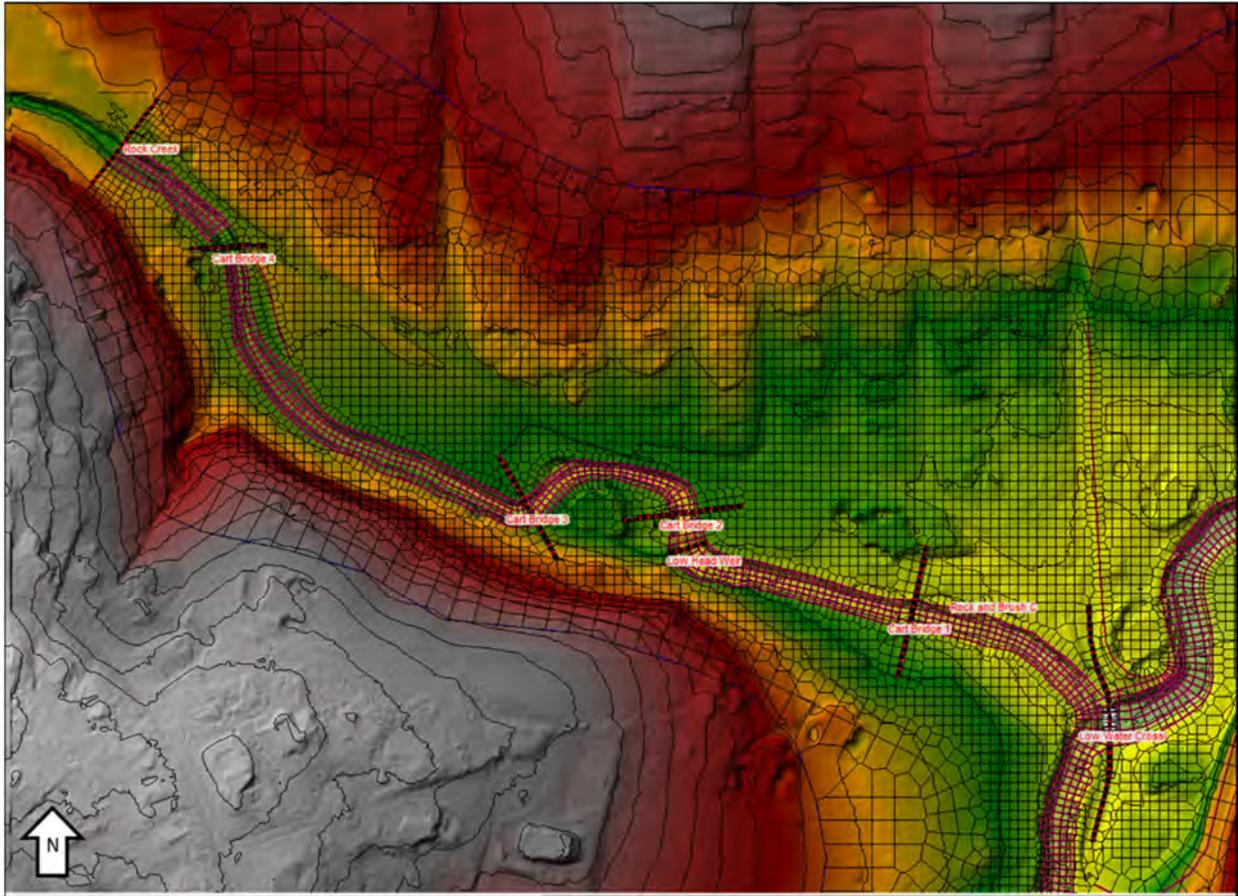


Figure 16 – 2D Internal Hydraulic Structures

The low water crossing was input using the Black and Veatch design drawings obtained from Water Resource Solutions. The culvert invert, outlet, bridge deck, and barrel size match the design drawings. The culvert roughness was set to 0.015. Figure 17 and Figure 18 show low water crossing in the mesh and internal structure editor.

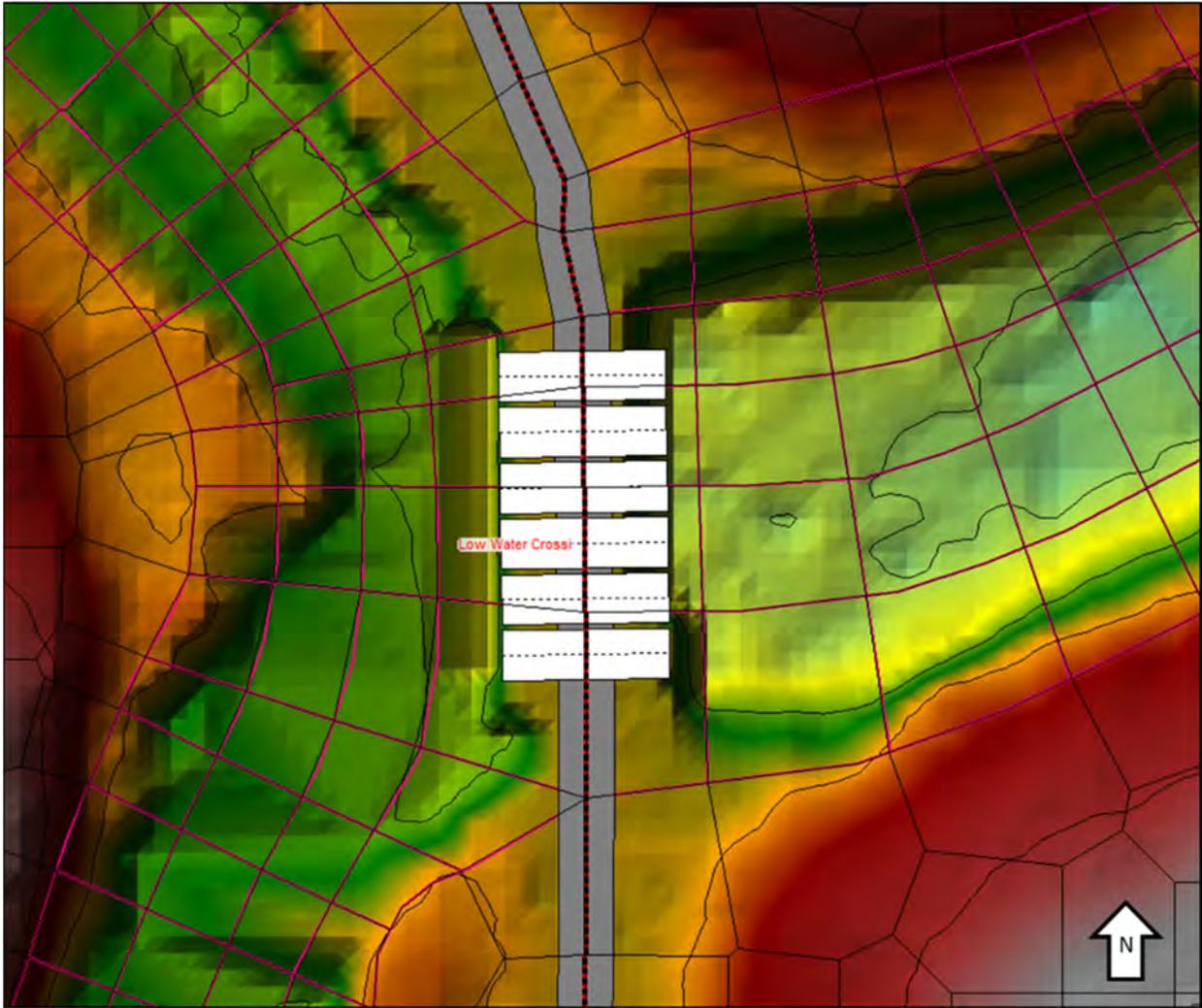


Figure 17 – Low Water Crossing in 2D mesh



Figure 18 – Internal Hydraulic Structure of Low Water Crossing

2.2.1.5 Model Limitations

There are three major limitations associated with this modeling effort. There is a lack of bathymetric data, calibration data, and a hydrograph for the study reaches. Bathymetric data is the least concern since the streams do not flow at such a great depth where the LiDAR appears overly hydro flattened. Calibration data is the most important to improve accuracy and gain confidence in modeling results. Calibration for this reach would be possible because of the gage stations on Brush Creek at Rockhill Road and Ward Parkway. The caveat is that additional LiDAR and bathymetry would need to be gathered to expand the model limits to those gages. Expanding model limits also exponentially increases the effort required to accurately model an urban reach due to all the infrastructure involved. Developing a hydrograph for this reach would also benefit the model results. A steady state model does not depict the volume of a flood; which ultimately is the defining characteristic of a flood extent and depth. Developing a hydrograph for Rock Creek and Brush Creek would be required to gain more confidence in flood extent and height.

Model boundary conditions are relatively close to the study area in question. This is not an ideal situation because the boundary conditions have a strong effect on results in their vicinity. Ideally the boundary conditions are placed farther away from the study area to mitigate their effect on results. This would require additional 1-foot LiDAR gathering and effort to build a larger model.

First floor elevation data were missing for the flooded houses. This makes confidently reporting flood elevations different for the houses in question. Depth were estimated based on LiDAR and imagery. A depth was assigned using engineering judgment based on the depths contained within the house boundary.

2.3 Improvement Alternatives

Three alternatives were investigated for this study: 1) A house buyout, 2) removal of the low water crossing, and 3) removal of the low water crossing as well as raising State Park Road. This section will contain a brief description of each alternative and their cost estimate. Figure 19 and Table 5 summarize and demonstrate the location of each ID in the Flood Problem Rating Table.

Table 5 – Flood Rating Table ID Descriptions

Rating Table ID	House / Street	City
1	5645 State Park Rd	Fairway, KS
2	5600 Mission Dr	Mission Hills, KS
3	5555 State Park Rd	Fairway, KS
4	5600 Canterbury Rd	Fairway, KS
5	Brookbank Lane	Mission Hills, KS
6	State Park Road	Mission Hills/Fairway, KS
7	Falmouth Road	Fairway, KS
8	Canterbury Road	Fairway, KS
9	Chadwick Street	Fairway, KS
10	Aberdeen Road	Fairway, KS
11	Fairway Road	Fairway, KS
12	Norwood Road	Fairway, KS
13	Belinder Avenue	Fairway, KS

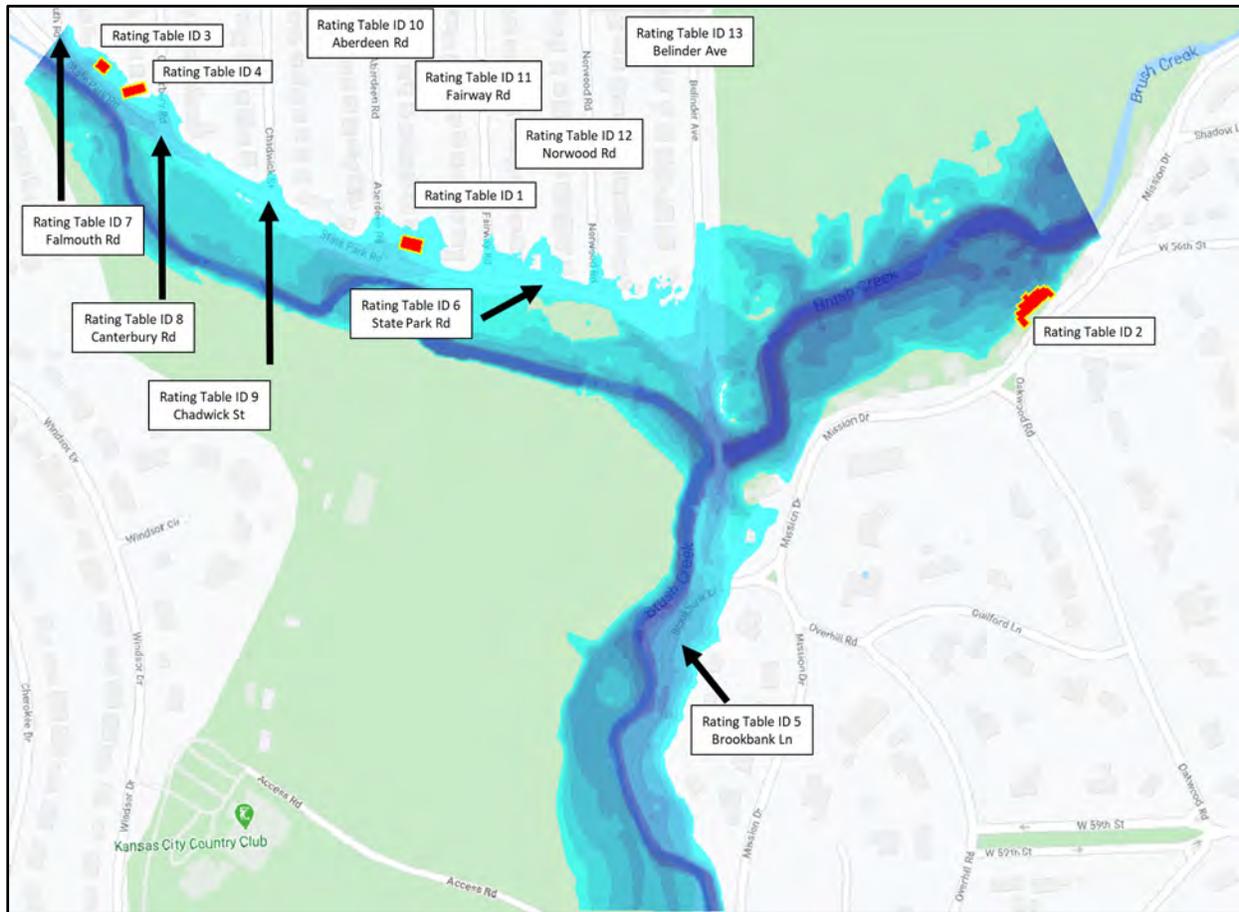


Figure 19 – Location of Buildings and Streets in Flood Rating Table

2.3.1 Alternative 1 – Buyout Flooded Houses

The Johnson County Stormwater Management Advisory Council (SMAC) requires a buyout option. In this reach two houses start inundating at the 10-year flow (5645 State Park Rd and 5600 Mission Dr) and two at the 50-year flow (5555 State Park Rd and 5600 Canterbury Rd). Table 6 shows the estimated value. No alternative makes a change to frequency of flooding for any house. These values were taken from the Johnson County’s Office of the County Appraisal website with an added 25% contingency cost. The Flood Problem Rating Table and summary are shown in Figure 20 and Table 7.

Table 6 – Buyout Cost

House	City	Value
5645 State Park Rd	Fairway, KS	\$ 371,625.00
5600 Mission Dr	Mission Hills, KS	\$ 4,666,375.00
5555 State Park Rd	Fairway, KS	\$ 379,875.00
5600 Canterbury Rd	Fairway, KS	\$ 489,750.00
Total		\$ 5,907,625.00

Table 7 – Flood Problem Rating Table Summary for Buyout Alternative

Asset	Weighting	Existing	Proposed	Risk
Building	20%	3.8	0.0	3.8
Street	50%	2.4	2.4	0.0
Waterway	30%	1.0	1.0	1.0
	100%	Existing Risk Score:		2.3
		Proposed Risk Score:		1.5
		Change in Risk Score:		1.1

Existing Conditions

Asset	Asset ID		Likelihood of Failure - 65% Total Risk Score					Consequence of Failure - 35% Total Risk Score						Building Risk Scores		
			50% LoF	Highest Score - 50% LoF			LoF Score	CoF Score								
	Frequency of Flooding	Depth in feet (input negative depths for freeboard)		Depth Score	Velocity in feet per second	Velocity Score		Historic Loss of Life	Impacts to Critical or Non-Critical Facility	Impacts to Historic Buildings	Affects Multiple Buildings	Reduced Economic Capacity	CoF Score (Use Maximum Score)			
Building	1	10	5.0	1.0	4.0	1.8	3.0	4.5	1.0	3.0	1.0	1.0	1.0	1.0	3.0	4.0
	1	50	4.0	1.9	4.0	3.1	4.0	4.0	1.0	3.0	1.0	3.4	1.0	3.4	4.1	
	1	100	3.0	2.0	4.0	3.3	4.0	3.5	1.0	3.0	1.0	3.4	1.0	3.4		
	2	10	5.0	0.6	3.0	1.0	3.0	4.0	1.0	3.0	1.0	1.0	1.0	3.0		3.5
	2	50	4.0	3.0	5.0	1.2	3.0	4.5	1.0	3.0	1.0	3.4	1.0	3.4		
	2	100	3.0	4.0	5.0	1.3	3.0	4.0	1.0	3.0	1.0	3.4	1.0	3.4		
	3	50	4.0	0.6	3.0	1.4	3.0	3.5	1.0	3.0	1.0	3.4	1.0	3.4	3.5	
	3	100	3.0	0.8	3.0	1.7	3.0	3.0	1.0	3.0	1.0	3.4	1.0	3.4		
	4	50	4.0	0.6	3.0	0.8	3.0	3.5	1.0	3.0	1.0	3.4	1.0	3.4		
	4	100	3.0	0.8	3.0	1.0	3.0	3.0	1.0	3.0	1.0	3.4	1.0	3.4	3.5	
	Building Score (Average all Building Risk Scores):															3.8

Street	ID	25% LoF			75% LoF			Historic Loss of Life	Loss of Street Access	Loss of Emergency Access	Affects Multiple Streets	CoF Score (Use Maximum Score)	Street Risk
		Upstream Area Characterization / Ownership	Stormwater Infrastructure Characterization	LoF Score	Impacts to Water Recreation	Stream Hydromodification	Water Quality Impairment						
	5			1.0			1.0	3.0	3.0	4.0	4.0	2.1	
	6			5.0			1.0	4.0	4.0	5.0	5.0	5.0	
	7			1.0			1.0	3.0	3.0	1.0	3.0	1.7	
	8			1.0			1.0	3.0	3.0	1.0	3.0	1.7	
	9			1.0			1.0	3.0	3.0	1.0	3.0	1.7	
	10			1.0			1.0	3.0	3.0	1.0	3.0	1.7	
	11			1.0			1.0	3.0	3.0	1.0	3.0	1.7	
	12			1.0			1.0	3.0	3.0	1.0	3.0	1.7	
	13			5.0			1.0	3.0	3.0	4.0	4.0	4.7	
Street Score (Average all Street Risk Scores):												2.4	

Waterway	ID	25% LoF			75% LoF			Impacts to Water Recreation	Stream Hydromodification	Water Quality Impairment	CoF Score (Use Maximum Score)	Waterway Risk Scores
		Upstream Area Characterization / Ownership	Stormwater Infrastructure Characterization	LoF Score	Impacts to Water Recreation	Stream Hydromodification	Water Quality Impairment					
	N/A		1.0		1.0	1	1.0	1.0	1.0	1.0	1.0	1.0
Waterway Score (Average all Waterway Risk Scores):												1.0

Buyout Houses

Asset	Asset ID		Likelihood of Failure - 65% Total Risk Score					Consequence of Failure - 35% Total Risk Score						Building Risk Scores		
			50% LoF	Highest Score - 50% LoF			LoF Score	CoF Score								
	Frequency of Flooding	Depth in feet (input negative depths for freeboard)		Depth Score	Velocity in feet per second	Velocity Score		Historic Loss of Life	Impacts to Critical or Non-Critical Facility	Impacts to Historic Buildings	Affects Multiple Buildings	Reduced Economic Capacity	CoF Score (Use Maximum Score)			
Building	1	10	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0		
	1	50	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0			
	1	100	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0			
	2	10	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0		
	2	50	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0			
	2	100	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0			
	3	50	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0		
	3	100	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0			
	4	50	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0			
	4	100	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0		
	Building Score (Average all Building Risk Scores):															0.0

Street	ID	25% LoF			75% LoF			Historic Loss of Life	Loss of Street Access	Loss of Emergency Access	Affects Multiple Streets	CoF Score (Use Maximum Score)	Street Risk
		Upstream Area Characterization / Ownership	Stormwater Infrastructure Characterization	LoF Score	Impacts to Water Recreation	Stream Hydromodification	Water Quality Impairment						
	5			1.0			1.0	3.0	3.0	4.0	4.0	2.1	
	6			5.0			1.0	4.0	4.0	5.0	5.0	5.0	
	7			1.0			1.0	3.0	3.0	1.0	3.0	1.7	
	8			1.0			1.0	3.0	3.0	1.0	3.0	1.7	
	9			1.0			1.0	3.0	3.0	1.0	3.0	1.7	
	10			1.0			1.0	3.0	3.0	1.0	3.0	1.7	
	11			1.0			1.0	3.0	3.0	1.0	3.0	1.7	
	12			1.0			1.0	3.0	3.0	1.0	3.0	1.7	
	13			5.0			1.0	3.0	3.0	4.0	4.0	4.7	
Street Score (Average all Street Risk Scores):												2.4	

Waterway	ID	25% LoF			75% LoF			Impacts to Water Recreation	Stream Hydromodification	Water Quality Impairment	CoF Score (Use Maximum Score)	Waterway Risk Scores
		Upstream Area Characterization / Ownership	Stormwater Infrastructure Characterization	LoF Score	Impacts to Water Recreation	Stream Hydromodification	Water Quality Impairment					
	N/A		1.0		1.0	1	1.0	1.0	1.0	1.0	1.0	1.0
Waterway Score (Average all Waterway Risk Scores):												1.0

Figure 20 – Flood Problem Rating Table for Buyout Alternative

2.3.2 Alternative 2 – Remove Low Water Crossing

This alternative involves removal of the low water crossing and placement of cul de sacs where the roads originally crossed Brush Creek. Belinder Avenue would no longer provide thoroughfare across Brush Creek. This alternative would completely eliminate the emergency resources required to close the crossing during floods. The risk associated with the low water crossing would be completely eliminated.

This alternative was modeled by removing the low water crossing from the terrain which allows water to flow through unimpeded by the current crossing. This geometry is detailed in the Terrain section of this report.

Table 8 contains a cost estimate for this alternative. Figure 21 and Table 9 summarize the Flood Problem Rating Table for this alternative.

Table 8 – Remove Low Water Crossing Cost Estimate

Work Item	Quantity	Unit Cost	Unit	Cost
Demo Low Water Crossing	2222	\$100.00	cy	\$222,222.22
Relocate Utilities	1	\$10,000.00	LS	\$10,000.00
Grading	29000	\$1.50	sf	\$43,500.00
Construct Cul de Sacs	250	\$400.00	ft	\$100,000.00
Site Work	1	\$25,000.00	LS	\$25,000.00
Mobilization / Demobilization		2	%	\$8,014.44
Erosion Control		3	%	\$12,021.67
Traffic Control		3	%	\$12,021.67
Administrative		5	%	\$20,036.11
Construction Subtotal				\$452,816.11
Contingency		20	%	\$90,563.22
Construction Total				\$543,379.33
Design		20	%	\$108,675.87
Project Total Cost				\$652,055.20

Table 9 – Flood Problem Rating Table Summary for Remove Low Water Crossing Alternative

Asset	Weighting	Existing	Proposed	Change in
Building	20%	3.8	3.8	0.0
Street	50%	2.4	1.9	0.5
Waterway	30%	1.0	1.0	1.0
	100%	Existing Risk Score:		2.3
		Proposed Risk Score:		2.0
		Change in Risk Score:		0.6

Existing Conditions

Asset	Asset ID		Likelihood of Failure - 65% Total Risk Score					Consequence of Failure - 35% Total Risk Score						Building Risk Scores	
			50% LoF	Highest Score - 50% LoF			LoF Score	CoF Score							
	Frequency (XX-year Event)	Frequency of Flooding	Flood Depth		Flood Velocity			Historic Loss of Life	Impacts to Critical or Non-Critical Facility	Impacts to Historic Buildings	Affects Multiple Buildings	Reduced Economic Capacity	CoF Score (Use Maximum Score)		
Depth in feet (input negative depths for freeboard)			Depth Score	Velocity in feet per second	Velocity Score										
Building	1	10	5.0	1.0	4.0	1.8	3.0	4.5	1.0	3.0	1.0	1.0	1.0	3.0	4.0
	1	50	4.0	1.9	4.0	3.1	4.0	4.0	1.0	3.0	1.0	3.4	1.0	3.4	
	1	100	3.0	2.0	4.0	3.3	4.0	3.5	1.0	3.0	1.0	3.4	1.0	3.4	
	2	10	5.0	0.6	3.0	1.0	3.0	4.0	1.0	3.0	1.0	1.0	1.0	3.0	4.1
	2	50	4.0	3.0	5.0	1.2	3.0	4.5	1.0	3.0	1.0	3.4	1.0	3.4	
	2	100	3.0	4.0	5.0	1.3	3.0	4.0	1.0	3.0	1.0	3.4	1.0	3.4	
	3	50	4.0	0.6	3.0	1.4	3.0	3.5	1.0	3.0	1.0	3.4	1.0	3.4	3.5
	3	100	3.0	0.8	3.0	1.7	3.0	3.0	1.0	3.0	1.0	3.4	1.0	3.4	
	4	50	4.0	0.6	3.0	0.8	3.0	3.5	1.0	3.0	1.0	3.4	1.0	3.4	3.5
	4	100	3.0	0.8	3.0	1.0	3.0	3.0	1.0	3.0	1.0	3.4	1.0	3.4	
	Building Score (Average all Building Risk Scores):													3.8	

Street	ID	LoF Score	Historic Loss of Life	Loss of Street Access	Loss of Emergency Access	Affects Multiple Streets	CoF Score (Use Maximum)	Street Risk
	5	1.0	1.0	3.0	3.0	4.0	4.0	2.1
6	5.0	1.0	4.0	4.0	5.0	5.0	5.0	
7	1.0	1.0	3.0	3.0	1.0	3.0	1.7	
8	1.0	1.0	3.0	3.0	1.0	3.0	1.7	
9	1.0	1.0	3.0	3.0	1.0	3.0	1.7	
10	1.0	1.0	3.0	3.0	1.0	3.0	1.7	
11	1.0	1.0	3.0	3.0	1.0	3.0	1.7	
12	1.0	1.0	3.0	3.0	1.0	3.0	1.7	
13 (LWC)	5.0	1.0	3.0	3.0	4.0	4.0	4.7	
Street Score (Average all Street Risk Scores):								2.4

Waterway	ID	25% LoF	75% LoF	LoF Score	Impacts to Water Recreation	Stream Hydromodification	Water Quality Impairment	CoF Score (Use Maximum Score)	Waterway Risk Scores
		Upstream Area Characterization / Ownership	Stormwater Infrastructure Characterization						
N/A	1.0	1.0	1	1.0	1.0	1.0	1.0	1.0	
Waterway Score (Average all Waterway Risk Scores):									1.0

Remove Low Water Crossing

Asset	Asset ID		Likelihood of Failure - 65% Total Risk Score					Consequence of Failure - 35% Total Risk Score						Building Risk Scores	
			50% LoF	Highest Score - 50% LoF			LoF Score	CoF Score							
	Frequency (XX-year Event)	Frequency of Flooding	Flood Depth		Flood Velocity			Historic Loss of Life	Impacts to Critical or Non-Critical Facility	Impacts to Historic Buildings	Affects Multiple Buildings	Reduced Economic Capacity	CoF Score (Use Maximum Score)		
Depth in feet (input negative depths for freeboard)			Depth Score	Velocity in feet per second	Velocity Score										
Building	1	10	5.0	1.0	4.0	1.8	3.0	4.5	1.0	3.0	1.0	1.0	1.0	3.0	4.0
	1	50	4.0	1.9	4.0	3.1	4.0	4.0	1.0	3.0	1.0	3.4	1.0	3.4	
	1	100	3.0	2.0	4.0	3.3	4.0	3.5	1.0	3.0	1.0	3.4	1.0	3.4	
	2	10	5.0	0.6	3.0	1.0	3.0	4.0	1.0	3.0	1.0	1.0	1.0	3.0	4.1
	2	50	4.0	3.0	5.0	1.2	3.0	4.5	1.0	3.0	1.0	3.4	1.0	3.4	
	2	100	3.0	4.0	5.0	1.3	3.0	4.0	1.0	3.0	1.0	3.4	1.0	3.4	
	3	50	4.0	0.6	3.0	1.4	3.0	3.5	1.0	3.0	1.0	3.4	1.0	3.4	3.5
	3	100	3.0	0.8	3.0	1.7	3.0	3.0	1.0	3.0	1.0	3.4	1.0	3.4	
	4	50	4.0	0.6	3.0	0.8	3.0	3.5	1.0	3.0	1.0	3.4	1.0	3.4	3.5
	4	100	3.0	0.8	3.0	1.0	3.0	3.0	1.0	3.0	1.0	3.4	1.0	3.4	
	Building Score (Average all Building Risk Scores):													3.8	

Street	ID	LoF Score	Historic Loss of Life	Loss of Street Access	Loss of Emergency Access	Affects Multiple Streets	CoF Score (Use Maximum)	Street Risk
	5	1.0	1.0	3.0	3.0	4.0	4.0	2.1
6	5.0	1.0	4.0	4.0	5.0	5.0	5.0	
7	1.0	1.0	3.0	3.0	1.0	3.0	1.7	
8	1.0	1.0	3.0	3.0	1.0	3.0	1.7	
9	1.0	1.0	3.0	3.0	1.0	3.0	1.7	
10	1.0	1.0	3.0	3.0	1.0	3.0	1.7	
11	1.0	1.0	3.0	3.0	1.0	3.0	1.7	
12	1.0	1.0	3.0	3.0	1.0	3.0	1.7	
13 (LWC Removed)	0.0	0.0	0.0	0.0	0.0	0.0	0.0	
Street Score (Average all Street Risk Scores):								1.9

Waterway	ID	25% LoF	75% LoF	LoF Score	Impacts to Water Recreation	Stream Hydromodification	Water Quality Impairment	CoF Score (Use Maximum Score)	Waterway Risk Scores
		Upstream Area Characterization / Ownership	Stormwater Infrastructure Characterization						
N/A	1.0	1.0	1	1.0	1.0	1.0	1.0	1.0	
Waterway Score (Average all Waterway Risk Scores):									1.0

Figure 21 – Flood Problem Rating Table for Remove Low Water Crossing Alternative

2.3.3 Alternative 3 – Remove Low Water Crossing and Raise State Park Road

This alternative includes the measures in the Remove Low Water Crossing alternative and adds a raise of State Park Road. In the existing condition State Park Road, and subsequently 3 houses along State Park Road, flood during the 10-year FEMA flow. In the FEMA 100-year flow these houses are inundated by about 2 feet of water. This alternative would raise State Park road by two feet to take State Park Road and 3 houses out of the 100-year FEMA flood. Raising State Park Road would also take Brookbank Lane, Falmouth Road, Canterbury Road, Chadwick Street, Aberdeen Road, Fairway Road, and Norwood Road out of the FEMA 100-year flood. An update to the stormwater drainage system is included for the aforementioned roads since they would now need to drain under the newly raised State Park Road. Figure 22 shows the potential location of the State Park Road raise. Table 10 contains a cost estimate for this alternative. Table 11 and Figure 23 summarize the Flood Problem Rating table for this alternative.



Figure 22 – Potential Location of State Park Road Raise

Table 10 – Remove Low Water Crossing and Raise State Park Road Alternative Cost Estimate

Work Item	Quantity	Unit Cost	Unit	Cost
Demo Low Water Crossing	2222	\$100.00	cy	\$222,222.22
Relocate Utilities	1	\$20,000.00	LS	\$20,000.00
Grading	65000	\$1.50	sf	\$97,500.00
Raise State Park Road 2 feet	2000	\$750.00	ft	\$1,500,000.00
Construct Cul de Sacs	250	\$400.00	ft	\$100,000.00
Update Stormwater Drainage along Raised State Park Road	1	\$20,000.00	LS	\$20,000.00
Site Work	1	\$200,000.00	LS	\$200,000.00
Mobilization / Demobilization		2	%	\$43,194.44
Erosion Control		3	%	\$64,791.67
Traffic Control		3	%	\$64,791.67
Administrative		5	%	\$107,986.11
Construction Subtotal				\$2,440,486.11
Contingency		20	%	\$488,097.22
Construction Total				\$2,928,583.33
Design		20	%	\$585,716.67
Project Total Cost				\$3,514,300.00

Table 11 – Flood Problem Rating Table Summary for Remove Low Water Crossing and Raise State Park Road Alternative

Asset	Weighting	Existing	Proposed	Change in
Building	20%	3.8	2.4	1.4
Street	50%	2.4	1.0	1.4
Waterway	30%	1.0	1.0	1.0
	100%			
		Existing Risk Score:		2.3
		Proposed Risk Score:		1.3
		Change in Risk Score:		1.3

Existing Conditions

Asset	Asset ID		Likelihood of Failure - 65% Total Risk Score						Consequence of Failure - 35% Total Risk Score						Building Risk Scores	
			50% LoF		Highest Score - 50% LoF				CoF Score							
	Frequency (XX-year Event)	Frequency of Flooding	Depth in feet (input negative depths for freeboard)	Depth Score	Velocity in feet per second	Velocity Score	LoF Score	Historic Loss of Life	Impacts to Critical or Non-Critical Facility	Impacts to Historic Buildings	Affects Multiple Buildings	Reduced Economic Capacity	CoF Score (Use Maximum Score)			
Building	1	10	5.0	1.0	4.0	1.8	3.0	4.5	1.0	3.0	1.0	1.0	1.0	3.0	4.0	
	1	50	4.0	1.9	4.0	3.1	4.0	4.0	1.0	3.0	1.0	3.4	1.0	3.4		
	1	100	3.0	2.0	4.0	3.3	4.0	3.5	1.0	3.0	1.0	3.4	1.0	3.4		
	Building	2	10	5.0	0.6	3.0	1.0	3.0	4.0	1.0	3.0	1.0	1.0	1.0	3.0	4.1
		2	50	4.0	3.0	5.0	1.2	3.0	4.5	1.0	3.0	1.0	3.4	1.0	3.4	
		2	100	3.0	4.0	5.0	1.3	3.0	4.0	1.0	3.0	1.0	3.4	1.0	3.4	
		3	50	4.0	0.6	3.0	1.4	3.0	3.5	1.0	3.0	1.0	3.4	1.0	3.4	
	Building	3	100	3.0	0.8	3.0	1.7	3.0	3.0	1.0	3.0	1.0	3.4	1.0	3.4	3.5
		4	50	4.0	0.6	3.0	0.8	3.0	3.5	1.0	3.0	1.0	3.4	1.0	3.4	
	Building	4	100	3.0	0.8	3.0	1.0	3.0	3.0	1.0	3.0	1.0	3.4	1.0	3.4	3.5
		Building Score (Average all Building Risk Scores):													3.8	

Street	ID	LoF Score	Historic Loss of Life	Loss of Street Access	Loss of Emergency Access	Affects Multiple Streets	CoF Score (Use Maximum Score)	Street Risk	
	Street	5	1.0	1.0	3.0	3.0	4.0	4.0	2.1
6 (State Park Rd)		5.0	1.0	4.0	4.0	5.0	5.0	5.0	
7		1.0	1.0	3.0	3.0	1.0	3.0	1.7	
8		1.0	1.0	3.0	3.0	1.0	3.0	1.7	
9		1.0	1.0	3.0	3.0	1.0	3.0	1.7	
10		1.0	1.0	3.0	3.0	1.0	3.0	1.7	
11		1.0	1.0	3.0	3.0	1.0	3.0	1.7	
12		1.0	1.0	3.0	3.0	1.0	3.0	1.7	
13 (LWC)		5.0	1.0	3.0	3.0	1.0	4.0	4.7	
Street Score (Average all Street Risk Scores):								2.4	

Waterway	ID	25% LoF	75% LoF	LoF Score	Impacts to Water Recreation	Stream Hydromodification	Water Quality Impairment	CoF Score (Use Maximum Score)	Waterway Risk Scores
		N/A	1.0	1.0	1	1.0	1.0	1.0	1.0
Waterway Score (Average all Waterway Risk Scores):									1.0

Proposed Conditions

Asset	Asset ID		Likelihood of Failure - 65% Total Risk Score						Consequence of Failure - 35% Total Risk Score						Building Risk Scores
			50% LoF		Highest Score - 50% LoF				CoF Score						
	Frequency (XX-year Event)	Frequency of Flooding	Depth in feet (input negative depths for freeboard)	Depth Score	Velocity in feet per second	Velocity Score	LoF Score	Historic Loss of Life	Impacts to Critical or Non-Critical Facility	Impacts to Historic Buildings	Affects Multiple Buildings	Reduced Economic Capacity	CoF Score (Use Maximum Score)		
Building	1	10	5.0	0.0	1.0	0.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.7	
	1	50	4.0	0.0	1.0	0.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0		
	1	100	3.0	0.0	1.0	0.0	1.0	2.0	1.0	1.0	1.0	1.0	1.0		
	Building	2	10	5.0	0.6	3.0	1.0	3.0	4.0	1.0	3.0	1.0	1.0	1.0	4.0
		2	50	4.0	3.0	5.0	1.2	3.0	4.5	1.0	3.0	1.0	1.0	1.0	
		2	100	3.0	4.0	5.0	1.3	3.0	4.0	1.0	3.0	1.0	1.0	1.0	
		3	50	4.0	0.0	1.0	0.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	
	Building	3	100	3.0	0.0	1.0	0.0	1.0	2.0	1.0	1.0	1.0	1.0	1.0	1.7
		4	50	4.0	0.0	1.0	0.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	
	Building	4	100	3.0	0.0	1.0	0.0	1.0	2.0	1.0	1.0	1.0	1.0	1.0	1.7
		Building Score (Average all Building Risk Scores):													

Street	ID	LoF Score	Historic Loss of Life	Loss of Street Access	Loss of Emergency Access	Affects Multiple Streets	CoF Score (Use Maximum Score)	Street Risk	
	Street	5	1.0	1.0	3.0	3.0	4.0	4.0	2.1
6 (Raise State Park Rd)		1.0	1.0	1.0	1.0	1.0	1.0	1.0	
7		1.0	1.0	1.0	1.0	1.0	1.0	1.0	
8		1.0	1.0	1.0	1.0	1.0	1.0	1.0	
9		1.0	1.0	1.0	1.0	1.0	1.0	1.0	
10		1.0	1.0	1.0	1.0	1.0	1.0	1.0	
11		1.0	1.0	1.0	1.0	1.0	1.0	1.0	
12		1.0	1.0	1.0	1.0	1.0	1.0	1.0	
13 (Remove LWC)		0.0	0.0	0.0	0.0	0.0	0.0	0.0	
Street Score (Average all Street Risk Scores):								1.0	

Waterway	ID	25% LoF	75% LoF	LoF Score	Impacts to Water Recreation	Stream Hydromodification	Water Quality Impairment	CoF Score (Use Maximum Score)	Waterway Risk Scores
		N/A	1.0	1.0	1	1.0	1.0	1.0	1.0
Waterway Score (Average all Waterway Risk Scores):									1.0

Figure 23 – Flood Problem Rating Table for Remove Low Water Crossing and Raise State Park Road

3 Recommendations

This study recommends Alternative 3, removal of the Low Water Crossing and raise State Park Road. The low water crossing on Belinder Avenue is a high life and safety risk structure and drain on City resources due to how frequent the existing structure overtops and floods. This requires a use of limited emergency response resources to close the structure during floods. Removing this low water crossing would eliminate the life and safety hazard and the need for emergency personnel to respond during flooding. Raising the elevation of State Park Road will reduce the flood risk to the surrounding roads and the at-risk houses. This alternative address all of the study's concerns, eliminating the safety hazard at the low water crossing thereby eliminating demands on the City's emergency responders. The recommended alternative provides the additional benefit of reducing the flood risk to surrounding infrastructure and personal residence both in Mission Hills and in Fairway.

To gain more detailed insight into the complexities of the Brush Creek and Rock Creek confluence, improvements and enhancements could be made to the 2D HEC-HAS model used in this study. Models have limitations in their ability to replicate actual events and predict future conditions. Yet it is possible to reduce these limitations thereby increasing the reliability of the model results. Gathering bathymetric and calibration data would improve the accuracy and confidence of model results for this study. Performing a watershed study to determine a hydrograph could also produce a more realistic result compared to a steady state model.

Appendix A

Physical Model Report
by
Water Resource Solutions



WRS Project 2019010

Brush/Rock Creek Confluence

Testing of the Physical Model

General:

The model was built to study the confluence of Brush and Rock Creek in Mission Hills, KS and consisted of a rigid scale model, with gravel streambed. Water to the model was pumped from the existing below ground tank located in Room 113 of Flarshiem Hall on the UMKC Campus. After the pump, the flow was split into two 3-inch diameter supply lines to provide water to the head boxes for Brush Creek, and Rock Creek. Flow control for each was provided with separate gate valves. Flow measurement for the Rock Creek headbox was determined by a 3"x1.5" venturi meter. For the Brush Creek Headbox a submerged orifice was built into the headbox. Submerged orifices are well described in the US Bureau of Reclamation's Water Measurement Manual. The Headbox submerged orifice deviated slightly from the standard in the measurement manual and therefore was calibrated before laboratory testing against the 3"x1.5" venturi.

The Results of this calibration are presented in Figure 1, showing the calibrated flow equation.

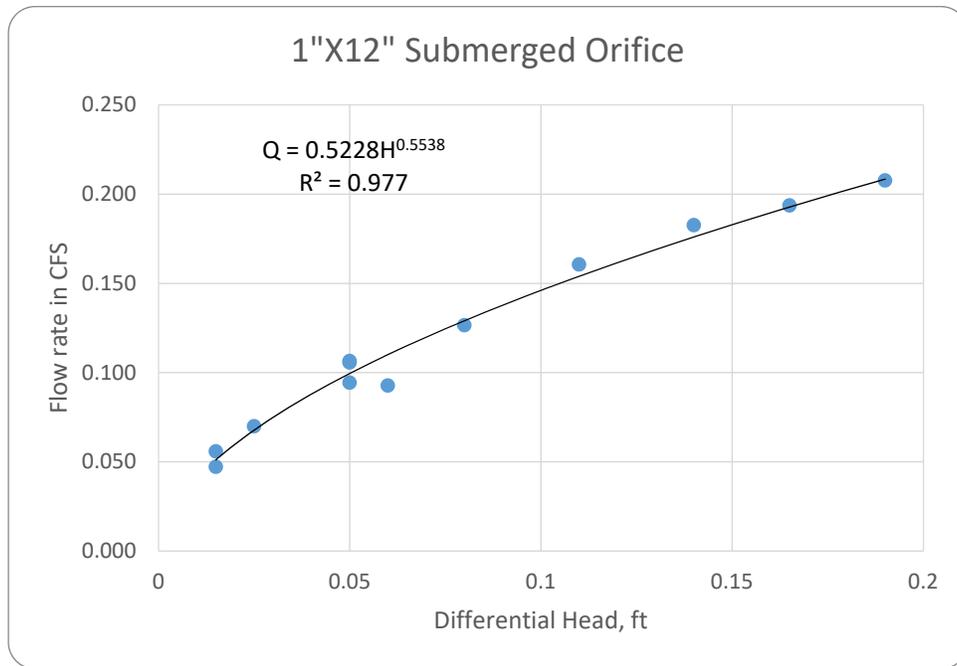


Figure 1. Flow equation for submerged orifice, Brush Creek Headbox.

Model simulations

Table 1 summarizes the model simulations that were conducted during the testing phase of this project. Observational data and photography were obtained for Runs 1, 2, 3, 5 and the un-numbered run on August 20, 2019. The first three runs were conducted to qualitatively evaluate the hydraulic performance of the weir/culvert/low water crossing performed when Rock creek contributed an average, elevated and lowered percent of the total flow. These simulations also documented how the

system performed with and without the culvert and bridge for these flows and investigated the effectiveness of providing a Rock creek bypass channel.

On August 20, and again on September 5, 2019 the model was demonstrated to various funding agencies, and stakeholders to demonstrate the flooding issues that exist at the site and test/demonstrate the viability of various options proposed to improve these hydraulic issues. Potential improvement options included alternatives proposed by the assembled observers during the visit and were not limited to previously identified alternatives.

Table 1 Summary of Testing

Run	Date	Prototype Discharges			Rock ck. as percent of total flow
		Brush ck. cfs	Rock ck. cfs	Total Flow cfs	
1	8/7/2019	2189	941	3130	30%
2	8/7/2019	2029	1016	3045	33%
3	8/7/2019	2029	172	2201	8%
-*-	8/20/2019	2220	543	2763	20%
-*-	8/20/2019	2779	543	3322	16%
4	9/3/2019	1140	486	1626	30%
5A	9/5/2019	1337	557	1893	29%
5B		2189	1052	3241	32%
6	9/10/2019	2220.03	1508	3728	40%

*No data other than photographic data was collected for Run 5 as this was a simulation to present preliminary findings to members of Mission hills.

Water level and velocity data was collected for the existing condition and with the culvert and roadway removed (the weir structure was not modified). Water level data was estimated by observation of the water levels in relation to the 2-ft contours on the model corresponding to the cross-sections associated with the Existing HEC-RAS Model. Local velocity was measured during runs 4 and 6 at and in the vicinity of the weir/culvert/low water crossing for low flow (Run 4) and overtopping flow (Run 6).

Spot elevations were collected using a hand-held pitot tube. Velocities were measured at HEC-RAS sections upstream of the weir on Brush and Rock creek and immediately downstream of the weir. Three velocities were measured at mid-channel, and half way between the bank and mid channel on both left and right side of the channel. Additional spot velocities were measured at other locations of interest.

For all runs still photography (*.jpg) and (*.MP4) video was obtained. Only representative still photography is presented in this report, however, the digital images and video are available. For most of the videos, a tracer consisting of surface chads or dye is used to highlight the flow patterns. These data can be used to compare to numeric model results to identify locations of high velocity and turbulence. For the purposes of this report, any discussion of the photographic data will be cited by run number and filename.

Summary of Results

Results of all the Hydraulic testing are summarized separately in this section. Digital MP4 videos are referenced by file name in these sections to further document model observations.

Qualitative Analysis

Run 1 Q=3100 cfs (overtopping) with 30% from Rock Creek

Table 2 summarizes the *.mp4 videos that were obtained during this run.

Table 2 List of Video documentation for Run 1

File Name	Conditions	Perspective	Observations
Run1a.mp4	Existing conditions and Overtopping flow	Looking downstream at confluence	Dye used to illustrate flow. Brush creek flow forms weak hydraulic jump creating backwater up Rock creek.
Run1b.mp4	without culvert/Road	Looking downstream at confluence	Water level at weir reduced. Strong velocity jets at both abutments.
Run1c.mp4	without culvert/road	Looking downstream at confluence	Chads used to show flow.
Run1d.mp4	Existing conditions and Overtopping flow	Looking downstream at confluence	Chads used to illustrate flow.
Run1e.mp4	Existing conditions and Overtopping flow	Looking downstream at confluence	Chads used to illustrate flow.
Run1f.mp4	Rock Creek Bypass channel	Looking downstream at confluence	Rock Creek Bypass conveyed only a modest flow.

For existing conditions the flow approaching the Brush creek flow is supercritical as evidenced by cross-wave patterns and a weak hydraulic jump that extends perpendicular from middle of the weir upstream to the adjacent left bank. This jump is locally unsteady. Placid (sub-critical water exists on the right side of Brush creek immediately upstream of the bridge. The Hydraulic jump backs water up Rock creek as evidenced by the relatively calm water to the left of the hydraulic jump. This forces Rock Creek water to flow diagonally over the approach road. Dye and chads clearly show that flow from rock creek are forced far left and that Brush creek flows dominate the flow through the culvert cells.

With the weir and road removed it is clear that the water levels immediately upstream of the weir were lower, and that Rock creek did not flow diagonally over the approach. The weak hydraulic jump noted for existing conditions remained but was lower and extended downstream of the weir reducing backwater on Rock creek. Significant separation and drawdown along both abutments was noted indicating the existence of high velocity and low pressure adjacent to the abutments. Although the removal of road and culvert appeared to lower water levels in the immediately upstream of the confluence on Rock creek, it appears that water levels further upstream on both tributaries were unaffected.

As an alternative scenario, a 12 foot wide bypass channel to route Rock creek flows downstream around the road/culvert (Run1f.mp4). The results of this test showed that the bypass flows were not sufficiently large enough to significantly modify the flow at the culvert. The primary reason for this is that the invert of the bypass at the upstream end was too high and would need to be lower. However lowering the invert would result in significant additional excavation of a channel without significantly increasing flow capture. Flow capture of the Rock Creek bypass is also limited because the flow at the entrance to the bypass is near critical flow. Because of this, the momentum of the flow in the main channel of Rock creek is difficult to turn into the bypass channel.

Although not videoed, during the presentation to the client. The Rock creek channel was completely dammed forcing all Rock Creek flow down the bypass. Observation of the flow at the weir/culvert indicated that the flow conditions at the weir/culvert were not improved.

Table 3 presents the water surface measurements for Run 1. The results indicate that removal of the culvert and roadway did not significantly alter water levels.

Table 3 Water Elevations Run 1		
Cross Section	Water Surface El., ft.	
	w/road	w/o road
BC 971	858	858
BC 1127	858	860
BC 1252	860	860
BC 1380	860	860
BC 1440	860	860
BC 1669	861	861
BC 1747	861	861
BC 1823	862	862
BC 1969	-	-
RC 330	862	862
RC 260	862	862
RC 200	861	861
RC 147	861	861
RC 104	860	856
RC 62	861	860
RC 37	861	860

Run 2 Q=3000cfs with 33% from Rock Creek

The total discharge for this run was similar to Run 1, however the contribution from Rock creek was increased to investigate how the flow conditions would change with a larger flow contribution from Rock creek. Table 4 summarizes the *.mp4 videos that were obtained during this run.

Table 4 List of Video documentation for Run 2

File Name	Conditions	Perspective	Observations
Run2a.mp4	Existing conditions and Overtopping flow	Looking downstream at confluence	Overtopping flows push across entire road and form jump to the right of the right abutment.
Run2b.mp4	Existing conditions and Overtopping flow	Looking downstream at confluence	Rock creek chads illustrate flow along length of road. Not Hydraulic jump on Right approach.
Run2c.mp4	Without culvert/road	Looking downstream at confluence	Strong mixing and high velocities downstream of the weir.
Run2d.mp4	Without culvert/road	Looking downstream at confluence	Strong mixing and high velocities downstream of the weir. Chads show location of lateral hydraulic jump.
Run2e.mp4	Without culvert/road	Looking downstream at confluence	Similar to Run 2d.mp4
Run2f.mp4	Rock Creek Bypass channel	Looking upstream	Rock Creek Bypass conveyed only a modest flow.

With a larger contribution of flow from Rock creek, the flow characteristics are significantly different. In this case supercritical Rock creek flows formed a hydraulic jump near where the hydraulic jump formed in Run1 however the sub- and supercritical flows were reversed. The Rock creek flow was super critical and Brush creek was subcritical. This is opposite of what happened when Brush creek was more dominant. Instead of flowing diagonally over the left approach to the bridge, a portion of the flow from Rock creek flowed parallel over the bridge from the near the left abutment to an area beyond the right abutment (Figure 2). A hydraulic jump perpendicular to the long axis of the bridge is clearly visible in the right approach roadway. Chads sprinkled on the flow clearly show this behavior (Run2b.mp4).

Table 5 presents the water surface measurements for Run 2 for existing conditions.

Cross Section	WSE ft.
BC 971	856
BC 1127	856
BC 1252	859
BC 1380	858
BC 1440	858
BC 1669	859
BC 1747	859
BC 1823	860
BC 1969	860
RC 330	862
RC 260	861
RC 200	860

RC 147	858
RC 104	858
RC 37	858

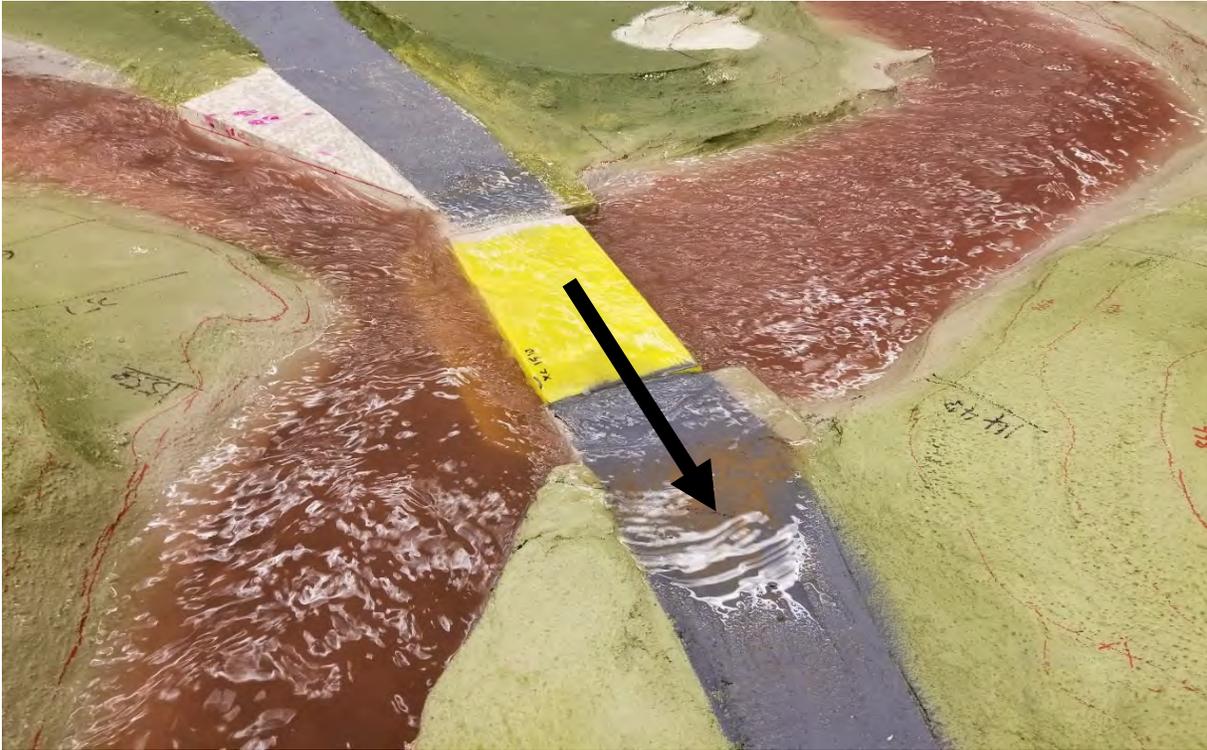


Figure 2 With Rock creek, dominate water overtops and runs east on road forming a hydraulic jump (near head of arrow) on the right roadway approach.

With the Road and culvert removed, overtopping of the approaches is nearly non-existent. However, there is a strong drawdown and high velocities near the left abutment. Between the two abutments a lateral hydraulic jump between the supercritical Rock creek flow, and subcritical brush creek exist approximately $2/3^{\text{ds}}$ to the right of the left abutment. As was noted in Run 1, water levels in the immediate vicinity of the weir were lowered by removal of the culvert and road, but had little influences further up or downstream of the weir.

Run 3 Q=2000 cfs with 8% from Rock Creek

The total discharge for this run was similar to Run 1, however the contribution from Rock creek was decreased to nearly zero to investigate how the flow conditions would change with Brush creek flow dominating. Table 6 summarizes the *.mp4 videos that were obtained during this run.

Table 6 List of Video Documentation for Run 3

File Name	Conditions	Perspective	Observations
Run3a.mp4	Existing conditions without overtopping	Looking downstream at confluence	Brush creek flow sweep past weir parallel to crest. Water backs up rock creek to exposed sewer line. Most flow is concentrated in left culvert barrel.
Run3b.mp4	Without road and culvert.	Looking downstream at confluence	Flow jet from Brush creek aimed at center of left abutment wall with high velocity. Right approach.

With the culvert and road the flow from Brush creek sweeps parallel to the weir as the weir acts more like a side channel spillway. Most of the flow is concentrated in the culvert barrel furthest to the left of the structure. Downstream of the culvert on the right side there is a large area of slack water (Figure 3).



Figure 3. With no flow from Rock creek the flow from Brush creek sweeps across weir and flows through the left culvert barrels. Downstream there is dead water on the right side of the channel.

With the Road and culvert removed the flow is similar but the flow jet from Brush creek impinges nearly perpendicular in the left abutment. It would be anticipated that large erosional potential would occur there (Figure 4). As before in Runs 1 and 2 water levels in the immediate vicinity of the structure are lower but not significantly different elsewhere upstream or downstream. As before, the downstream channel is largely ineffective flow area on the right side of the channel.

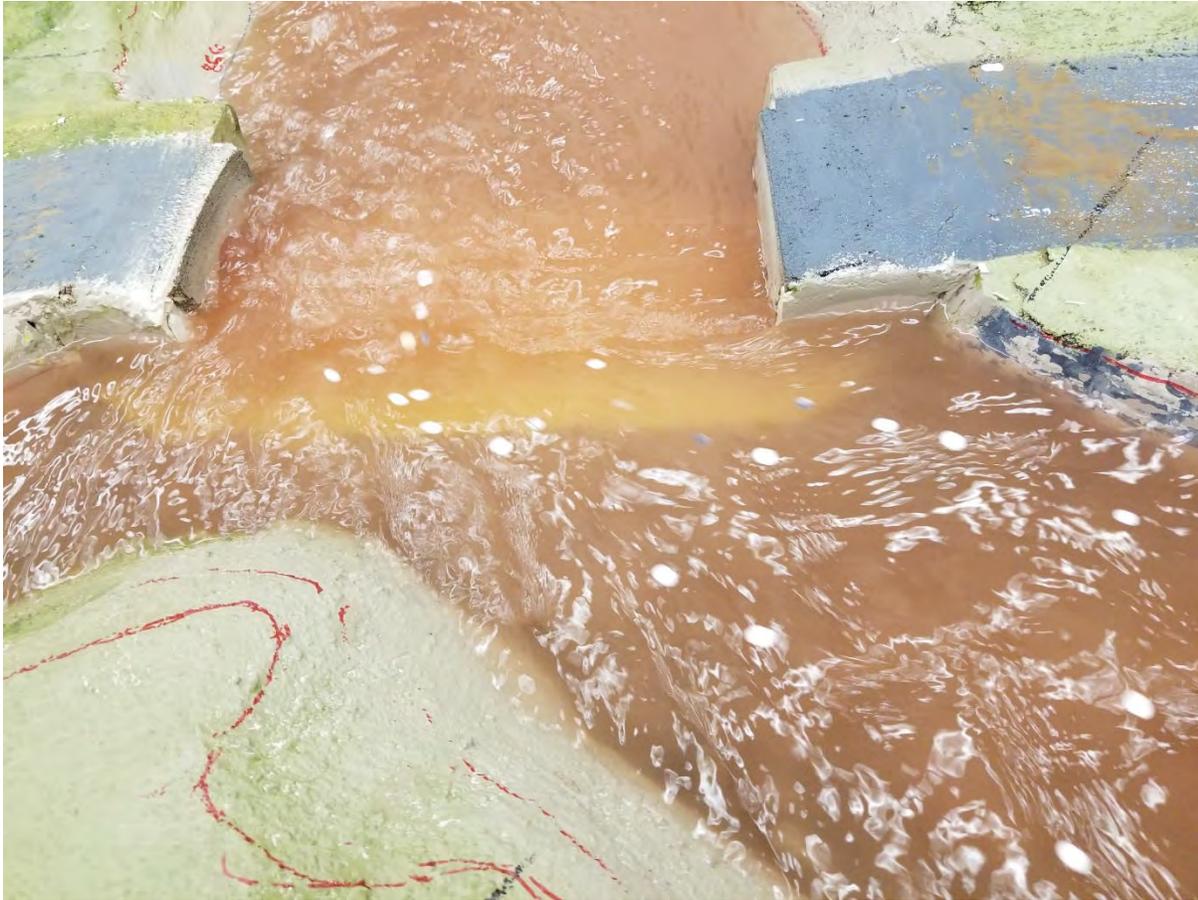


Figure 4. Without the culvert and road a strong jet impacts the left abutment.

Table 7 presents the water surface measurements for Run 3.

Table 7 Water surface Elevations Run 3		
Cross Section	WSE ft.	
BC 971	856	
BC 1127	857	
BC 1252	857	
BC 1380	857	
BC 1440	856	
BC 1669	859	
BC 1747	859	
BC 1823	860	
BC 1969	860	
RC 330	859	
RC 260	858	

RC 200	858		
RC 147	858		
RC 104	858		
RC 37	858		

Run 4 Q=1600 cfs with 30% from Rock Creek

This was another low-flow run conducted primarily for the purpose of collecting water level and velocity measurements. The flow rate was less than for Run 3 but the percentage of the flow from Rock creek was 30% which is approximately 10% less than the flow splits used for nots of the design discharges published by FEMA. Table 8 summarizes the *.mp4 videos that were obtained during this run.

Table 8 List of Video documentation for Run 4

File Name	Conditions	Perspective	Observations
Run4a.mp4	Existing conditions without overtopping	Looking downstream at confluence	Chads tracked for Rock creek
Run4b.mp4	Existing conditions without overtopping	Looking downstream at confluence	Chads on Brush Creek. Most flow through central barrel.
Run4c.mp4	Road /culvert removed without overtopping	Looking downstream at confluence	Chads tracked for Rock creek
Run4d.mp4	Road /culvert removed without overtopping	Looking downstream at confluence	Chads on Brush Creek. Most flow through central barrel.

For this series of runs it was observed that both the Brush and Rock creek flows converged and flow through the weir structure nearly perpendicular to the weir (Figure 5). This observation was observed both with and without the road and culverts. As before removal of the bridge had minimal influence on water levels upstream and downstream of the structure. Downstream, the flow in the channel is fairly uniform with only a small area along the right bank where the flow area is ineffective.



Figure 5. At low flow water flows over the weir nearly perpendicular to the crest of the weir.

The Hydraulics of Run 4 represent a conditions that is desirable for this type of structure in that the flow over the weir is nearly perpendicular and the flow is directed parallel into the culvert barrels. This makes maximum use of the flow area of the culvert structure.

Spot velocity measurements using a hand-held pitot-tube were collected in the approach reach of Brush and Rock Creek. Velocity measurements downstream of the confluence/weir were also made. The location of cross-sections where velocities were measured corresponded to the cross-sections from the 1-dimensional HEC-RAS model for the site. Velocities at each section were collected at three verticals by roughly dividing the channel width into 4 equal width areas and measuring the velocity near the center of each sub area. These verticals were denoted as left, center, and right corresponding to hydraulic convention looking downstream. These measurements are summarized in Table 9.

Table 9 Prototype Velocity Data for Run 4

Test 4 A Existing Conditions

Location of Measurement Reach	X-section	Velocities in f/s		
		Left	Center	Right
Upstream Weir	RC 147	8.0	9.0	6.9
	RC 104	9.8	8.0	5.7
	RC 62	10.2	8.0	6.9
	Weir crest	8.0	11.0	9.8
	BC 1669	9.8	11.3	10.6
	BC 1558	10.6	12.7	11.3
Downstream Weir	BC 1440	8.0	6.9	5.7
	BC 1380	6.9	6.9	5.7

Test 4 B Without Road and Culvert

Upstream Weir	RC 147	9.0	8.0	5.7
	RC 104	9.8	8.0	6.9
	RC 62	10.6	8.0	6.9
	Weir crest	9.0	11.3	9.8
	BC 1669	9.0	9.8	9.0
	BC 1558	11.3	13.9	11.3
Downstream Weir	BC 1440	8.0	6.9	5.7
	BC 1380	6.9	6.9	5.7

Note: X-section number from the HEC-RAS Model

Table 10 presents the water surface measurements for Run 4 for existing conditions and for the case where the culvert and roadway are removed.

Table 10 Run 4 Water Surface Elevations		
	W/Road	W/o Road
Cross Section	WSE ft.	WSE ft.
BC 971	856	856
BC 1127	855	857
BC 1252	858	858
BC 1380	858	857
BC 1440	858	857
BC 1669	860	860
BC 1747	860	860
BC 1823	860	860
BC 1969	860	860
RC 330	860	860

RC 260	860	860
RC 200	858	858
RC 147	858	858
RC 104	858	858
RC 37	858	858

Run 5 Q=1900 cfs 29% from Rock Creek and Q= 3200 cfs 32% from Rock Creek

This run was conducted on September 5 to demonstrate the performance of the model to the Mission Hills city Council. In this a lower and higher flood event was simulated. Conditions with and without the culvert. No data was collected but the qualitative observations from previous runs were validated qualitatively. A photographic record of this meeting was also recorded by Mike Smith WRS.

Run 6 Q=3728 cfs 40% from Rock Creek

This run closely matched the flow conditions for the 1-year flood both in magnitude and flow split. Table 11 is a list of the videos available describing the flow conditions for this run.

Table 11 List of Video documentation for Run 6

File Name	Conditions	Perspective	Observations
Run6a.mp4	Existing conditions	Looking downstream at confluence	High velocities and insular flow conditions. Chads tracked for Rock creek
Run6b.mp4	Existing conditions	Looking downstream at confluence	High velocity near sewer pipe.
Run6c.mp4	Existing conditions	Looking downstream at confluence	Chads tracked for Rock creek
Run6d.mp4	Existing conditions	Looking downstream at confluence	Chads on Brush Creek. High velocity over road.
Run6e.mp4	Existing conditions	Looking downstream at confluence	Dye showing flow.

Significant overtopping of the roadway was observed for this case. The flow in the approach and over the road is unsteady and time variant and standing waves (Figure 6).

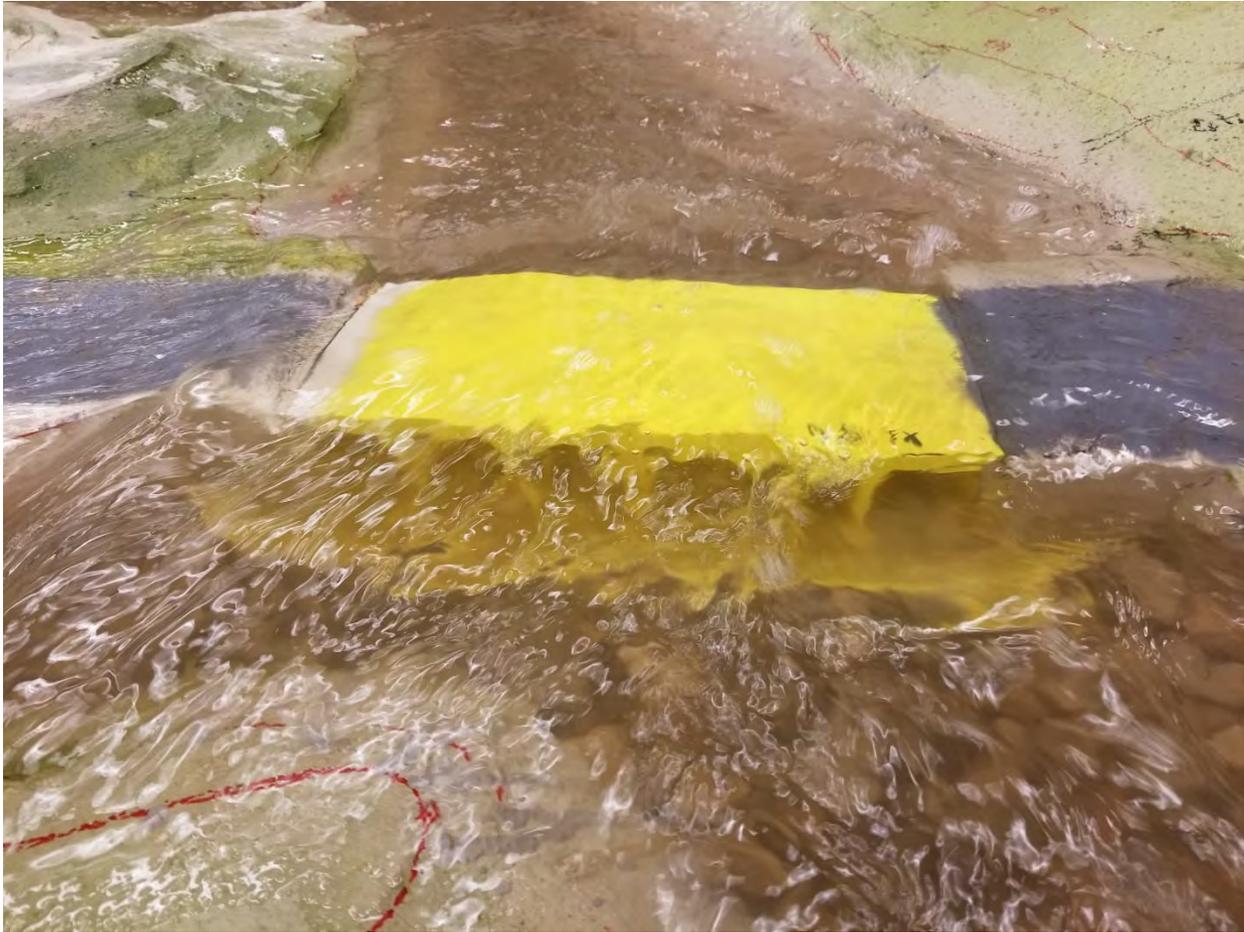


Figure 6. Run 6 for existing conditions shows a complex time variant overtopping flow with multiple locations having standing waves.

Velocities in the channel and at the structure are high with some locations having velocities nearing 20 f/s as show in Table 12.

Table 12 Prototype Velocity Data for Run 6

Test6 A Existing Conditions		Velocities in f/s		
Location of Measurement				
Reach	X-section	Left	Center	Right
	RC 200	16.0	16.0	13.9
Upstream Weir	RC 147	13.9	13.3	11.3
	RC 104	13.3	11.3	6.9
	RC 62	13.9	11.3	8.0
	Weir crest	11.3	8.0	9.8
Roadway overtopping		10.6	8.0	8.0
Left Approach Overtopping		9.8	13.9	13.3

	BC 1669	12.0	12.0	9.0
	BC 1558	9.0	12.0	5.7
Downstream Weir	BC 1440	8.0	10.6	9.0
	BC 1380	8.0	8.0	8.0
Test 6 B Without Road and Culvert				
Upstream Weir	RC 147	6.0	12.0	9.8
	RC 104	3.0	10.6	6.9
	RC 62	3.0	14.5	6.9
	Weir crest	5.0	6.9	9.0
Left Approach Overtopping		10.0	12.7	12.7
	BC 1669	8.0	12.7	11.3
	BC 1558	4.0	12.7	8.0
Downstream Weir	BC 1440	--	--	--
	BC 1380	--	--	--
Additional Velocity Data				
RC 200 near wall Downstream of Sewer		18.4	f/s	
RC 62 Near wall Upstream Weir		17.5	f/s	
Left abutment without Culvert		16.0	f/s	
Note: X-section number from the HEC-RAS Model				

Water surface elevations for this flow are presented in Table 13, however it should be noted that the measurements were difficult to collect due to the unsteady nature of the flow.

Table 13 Water Elevations Run 1		
Cross Section	Physical WSE ft.	
	w/road	w/o road
BC 971	858	858
BC 1127	858	860
BC 1252	860	860
BC 1380	860	860
BC 1440	860	860
BC 1669	861	861
BC 1747	861	861
BC 1823	862	862
BC 1969	-	-
RC 330	862	862
RC 260	862	862
RC 200	861	861
RC 147	861	861
RC 104	860	856
RC 62	861	860
RC 37	861	860

Summary of results

The data presented in this report provides valuable information and data to support the feasibility study for this location. The measured hydraulic data (velocity and flow depths) will be useful for validating and verifying numerical 1 2 and 3-dimensional studies that may be conducted to refine the hydraulics for various options for this site. Moreover the observational/qualitative data obtained from the physical model study is also a benefit. The video and still photography can be review by subsequent analysts working to provide recommendations to improve the flooding issues at this location.

The following is a summary of conclusions that were made by operating and documenting the model operation. Other interested parties reviewing the information may refine these opinions or form different opinions based on the presented results based on new information obtained during subsequent studies.

For the early tests it was noted that the bypass channel was relatively ineffective at conveying significant Rock Creek Flows. During the two presentations to stakeholders the consensus opinion was that the benefits of a bypass were minimal and no further testing of that option was conducted. This is why no additional study of the bypass other than the original qualitative assessment was conducted.

It is clear the confluence of Brush and Rock Creek is hydraulically poor. These two flow converge at a large obtuse angle. At very low Froude numbers such a convergence would be not create a significant problem, but in this case, where the Froude numbers are near or slightly large than unity, the flow patterns are unsteady, highly turbulent and vary significantly with the flow discharges and relative flow split between Rock and Brush creek. As an example when Brush creek flows were dominant, super critical flow caused a jump to sub critical flow on the Rick Creek side. This situation was reverse when the Rock Creek flows were dominant. As the flow split during a flood is unpredictable, this leads to a highly unpredictable flow condition immediately upstream of the weir.

The high Froude numbers that occur also result in the hydraulic structures (the culvert, weir and roadway) to not perform as theoretically intended. When flows are near Froude unity the momentum of the flow cannot be ignored. As a consequence flow can no longer be considered to be steady and gradually varied. Even though the discharges may be constant, the flow conditions become locally unsteady as standing waves and eddies and large-scale turbulence dominates the flow.

The above issues with this location were not significantly mitigated by removal of the culvert and road. The removal of the culvert and rod did change the flow patterns at the weir, but the new patterns that formed were as problematic (or more) that with the road and culvert. This, one again is due to the elevated Froude numbers of the two flow.

Finally, removal of the culvert and road locally modified the water levels in the immediate vicinity of the structure, but had no significant influence on water levels upstream or downstream of the weir. Once again this is due to the high velocities and Froude numbers.