RESTORING ANDSUSTAINING RIVER ENVIRONMENTS USING AN IN-STREAM TRAINING METHOD

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BACKGROUND

Historically, the Rio Grande at San Felipe used to be a braided river with a sand bed channel and a very high sediment load. Through channelization from 1932 to 1960s, the channel was straightened. As Cochiti Reservoir was built and began to impound water for flood and sediment control in 1975 which caused the reduction in flood peaks and sediment load, the channel generally became narrow and incised, and the bed material became coarser (Harvey, 2007; Shah-Fairbank et al., 2010). The reservoir traps approximately 99% of the current sediment supply to the downstream reach (Shah-Fairbank et al., 2010). The primary sediment supply to the downstream reach is from channel erosion and tributary inputs. Consequently the channel bed material size has become coarser over time, transitioning from sand to gravel, and the channel longitudinal profile has been lowered about four feet on average after the construction of the dam. The bed profile of the reach has been reasonably stable since 1992, an indication that the bed could have been armored under the post-dam flow regime. Sediment transport models suggest that significant bed mobilization would not occur at flows less than the 100-year peak of 10,000 cfs (Harvey, 2007). The reach immediately below the dam has incised and river meander bends have developed. In some areas the incision has dropped below the root zone, eroding banklines with mature cottonwood trees, and threatening infrastructure adjacent to the waterway. One specific location on the Rio Grande where this condition has occurred is immediately upstream from Angostura Diversion Dam. This location is about 21 miles downstream of Cochiti Dam. There are two meander bends where bank erosion has become a concern at three locations, RM 210.0, RM 210.1, and RM 210.3 as illustrated in Figure 1, threatening irrigation drains that border each side of the river.

In 2011, a multi-disciplinary project team was formed, including Tetra Tech, the Pueblo of San Felipe, and Bureau of Reclamation (Reclamation) staff, to initiate a process of providing bank protection at these three sites that included an initial site visit, development feasible alternatives to address the river maintenance concern at each site, selection of a preferred alternative at each site based on engineering effectiveness, ecosystem function, ease of compliance, and Pueblo preference. While individual alternative concepts were evaluated for each of the sites, the close proximity of the three sites on two consecutive bends for the downstream sites made the combination into one comprehensive project more palatable to the project team. This combination also facilitated the evaluation of adverse impacts between sites for the final design. The final design was a combination of longitudinal fill stone toe protection (LFSTP) and bendway weirs coupled with removal and/or lowering of bars.

1-DIMENSIONAL HYDRAULIC MODEL

The design of the bendway weirs and LFSTP requires inputs such as velocities, width, depth, energy slope, etc. which can be acquired from a one-dimensional (1-D) HEC-RAS hydraulic model. The outputs of the design are the ranges of bendway weir length, planform angle,

spacing, and height, and the fixed values of the LFSTP crest width and height. Different combinations of bendway weir configurations can slightly change the hydraulic conditions around the weirs that cannot be modeled by a 1-D HEC-RAS model. Therefore, a two-dimensional (2-D) hydraulic model with SRH-2D was used to assist in the process to optimize the final bendway weir configurations.

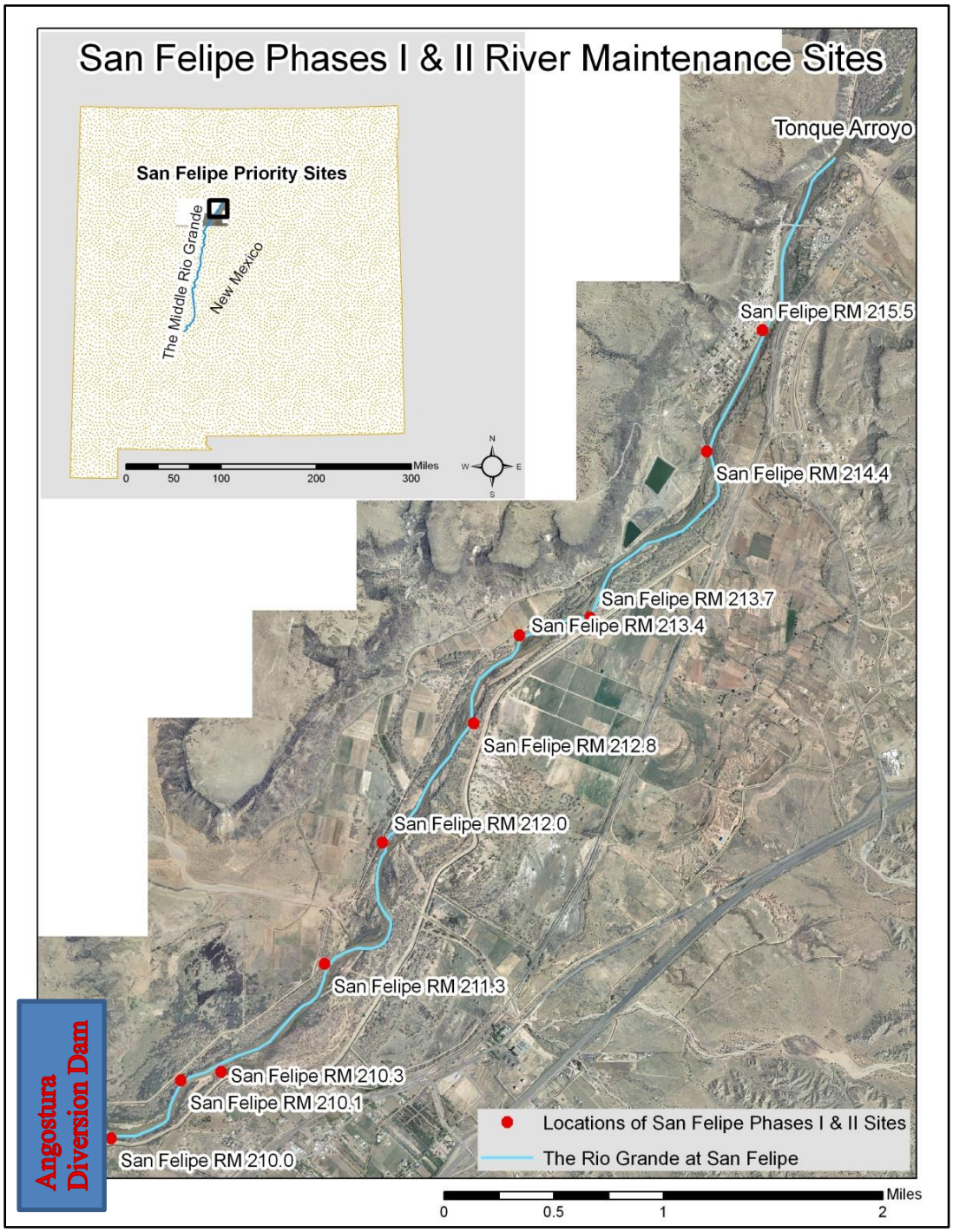

Figure 1 Map of ten priority sites along the Rio Grande at San Felipe.

There was a 1-D HEC-RAS model developed in 2007 using cross sections surveyed in 2007 to model the ten (10) priority sites illustrated in Figure 1. The floodplain elevation of the 2007 model was based on a digital terrain model (DTM) created from the 2002 photogrammetrically derived aggradation-degradation elevation. The downstream end of the model stopped just upstream from the Angostura Diversion Dam. The model was calibrated with Manning's n values gradually decreasing from upstream to downstream because the bed material is coarser upstream and becomes finer as it gets closer to the Angostura Diversion Dam. Photography of subsequent years after the model was compared with the 2007 photography and did not show denser vegetation encroachment into the channel. Therefore, it was assumed that the original Manning's n values were applicable to the updated 1-D HEC-RAS model built in 2012. The boundary of the 2012 HEC-RAS model covered from the priority site at RM 212.0 to just past the Angostura Diversion Dam.

Ineffective flow areas and obstructions were used in the geometry to model bendway weirs, areas in between weirs, and the LFSTP. Since the design of bendway weir spacing was in the range from 70 feet to 80 feet and the spacing of cross sections ranged from 150 feet to 500 feet, interpolated cross sections were required to model 12-foot wide bendway weirs. It is likely that errors inherent from the difference in bed elevation of the interpolated model and the actual river channel are present. The bendway weirs and LFSTP were modeled as obstruction objects. Areas of recirculation flow between weirs were modeled as ineffective flow areas (Sclafani et al., 2012).

In the process of riprap sizing, maximum velocities of the design flow at cross sections were required. However, this parameter cannot be obtained accurately from a 1-D hydraulic model. Instead of using a 2-D model at an early design stage that required a lot of effort, flow distribution was used to run the 1-D model to provide velocities for sub-sections of a channel cross section. These velocities are not the true 2-D velocity field, but they are interpolated velocities based on channel depth values of those sub-sections.

The pre-project 1-D simulation provided the hydraulic parameters for scour calculation, riprap sizing, and the design of the bendway weirs and LFSTP. The post-project 1-D simulation was performed for one design configuration. Then it was decided that it was more efficient to adjust design parameters using AutoCAD Civil 3D and GIS and to analyze results using a 2-D hydraulic model, so different design configuration combinations were tested with a 2-D model.

DESIGN CALCULATIONS

Immediately upstream of the Angostura Diversion Dam, the bend on the right side of the river is referred to as the upstream bend, and the bend on the left side of the river is referred to as the downstream bend in this report. The priority site RM 210.3 is immediately upstream from the upstream bend and on the opposite bank. The upstream bend passes through RM 210.1. RM 210.0 is located at the downstream end of the downstream bend.

Scour Computations: Expected scour depth was estimated as the first step of the analysis. An extensive, but not exhaustive, review of scour equations was conducted to identify a number of methods that would be appropriate for the project site. Long-term scour computation used

Yang's, Shields', Lane's, and Peter-Myer-Mueller's approaches, and competent velocity method. The selected scour equations for general and bend scour can be categorized as mean velocity equations, regime equations, and bend equations. The bend equations are based on the degree of tightness of a bend that is responsible for a transverse bed slope with significantly increased depth near the outer bank. The total scour depths developed under the 100 year-return flow of 10,000 cfs for two (2) bends are summarized in Table 1.

Table 1 Calculations of total scour

Scour Components		Velocity Equations			Regime Equations			Bend Equations		
		Limiting Velocity	Mean Velocity	Zeller[1] (1989)	Neill (1973)	Lacey (1930)	Blench (1969)	Maynord[1] (1996)	Zeller[1] (1989)	Thorne (1997)
General	Upstream	included	incl.	incl.	incl.	incl.	incl.	incl.	1.7	incl.
	Downstream								2.2	
Bend	Upstream	included	incl.	0.0	incl.	incl.	incl.	incl.	incl.	incl.
	Downstream			0.0						
Thalweg Formation	Upstream	included	incl.	1.0	incl.	incl.	incl.	incl.	1.0	incl.
	Downstream			1.0					1.0	
Long term scour	Upstream	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04
	Downstream	0.18	0.18	0.18	0.18	0.18	0.18	0.18	0.18	0.18
Average Total Scour	Upstream	5.9	6.1	2.7	6.5	6.2	6.6	6.6	2.7	8.1
	Downstream	0.7	3.7	3.3	5.1	2.0	5.7	5.4	3.3	7.5
Avg Total Scour with Factor of Safety = 1.1	Upstream	6.5	6.7	3.0	7.2	6.8	7.2	7.2	3.0	8.9
	Downstream	0.8	4.0	3.7	5.7	2.2	6.3	5.9	3.7	8.3
Design Scour Depth for Upstream Bend =			7.0 ft		(Max =	8.9 ft		Min =	3.0 ft)	
Design Scour Depth for Downstream Bend =			5.0 ft		(Max =	8.3 ft		Min =	0.8 ft)	

[1]Developed for sand bed rivers, assumed conservative for gravel bed

Riprap Sizing: There are many approaches that are used in the professional engineering practice for sizing revetment riprap. NCHRP 568 (Lagasse, 2006) evaluated seven of the most commonly used revetment riprap sizing equations and recommended the EM 1601 equation (USACE, 1994) on the basis of discriminating between stable or failed riprap, bank and bend correction factors, and the reasonableness of safety/stability factors. The EM 1601 calculated D_{50} size can be used as the first approximation of the minimum riprap size needed for the design of the LFSTP at the upstream bend which is 12 inches. This riprap size ensures the longitudinal structure stability for discharges up to 100-year return flow of 10,000 cfs. The proposed correction factor for cross-stream features recommended by de Almeida and Martin-Vide (de Almeida and Martín-Vide, 2009) is about 2, which results in a transverse D_{50} size of 24 inches for both the upstream and the downstream bends to withstand the 100-year return flow of 10,000 cfs.

Design of Longitudinal Fill Stone Toe Protection (LFSTP): LFSTP is continuous riprap revetment with a trapezoidal cross section placed along the toe of a bankline to increase the roughness at the toe of the bank against erosion forces from incoming sustained high flows and to adapt to scour holes near the outer bank developed under sustained high flow conditions (Martin-Vide et al., 2010). Additional critical functional elements of the LFSTP are the keys and

tie-backs buried in the floodplain to prevent high flows from flanking the structure (McCullah and Gray, 2005). The LFSTP crest elevation was designed to be approximately 0.5 ft higher than the point bar at the upstream bend so that the hydraulically rougher stone toe will protect the toe of the outer bank until the hydraulic pressure is released as water overtops the high point of the opposite point bar. The width of the LFSTP was designed to provide rock that would self-launch into scour holes developed under sustained high flows along the toe of the stone structure which consequently protects the right bankline (Martin-Vide et al., 2010). As rock self-launches to the estimated scour depth, the crest width of the LFSTP is reduced to the minimum width of $2*D_{100}$ which is 4 ft (Derrick, 2011) keeping the original crest height unchanged. Since there is uncertainty in how the rock self-launches, a factor of 1.5 was added to the quantity of the self-launching rock that lands in potential scour holes (Derrick, 2011; Maynord et al., 1989). Tie-backs and keys are the LFSTP components that extend from the bankline into the floodplain. Keys are oriented perpendicular to the bankline while tie-backs are located at the upstream and downstream ends of the LFSTP and oriented at an angle from 20° to 30° with the bankline (Derrick, 2011). Assessing this area to be relatively, it is necessary to key the stone structure into the stable ground beyond the 1935-1962 channel boundaries, which gives the key length of 100 feet and a tie-back length as 600 feet.

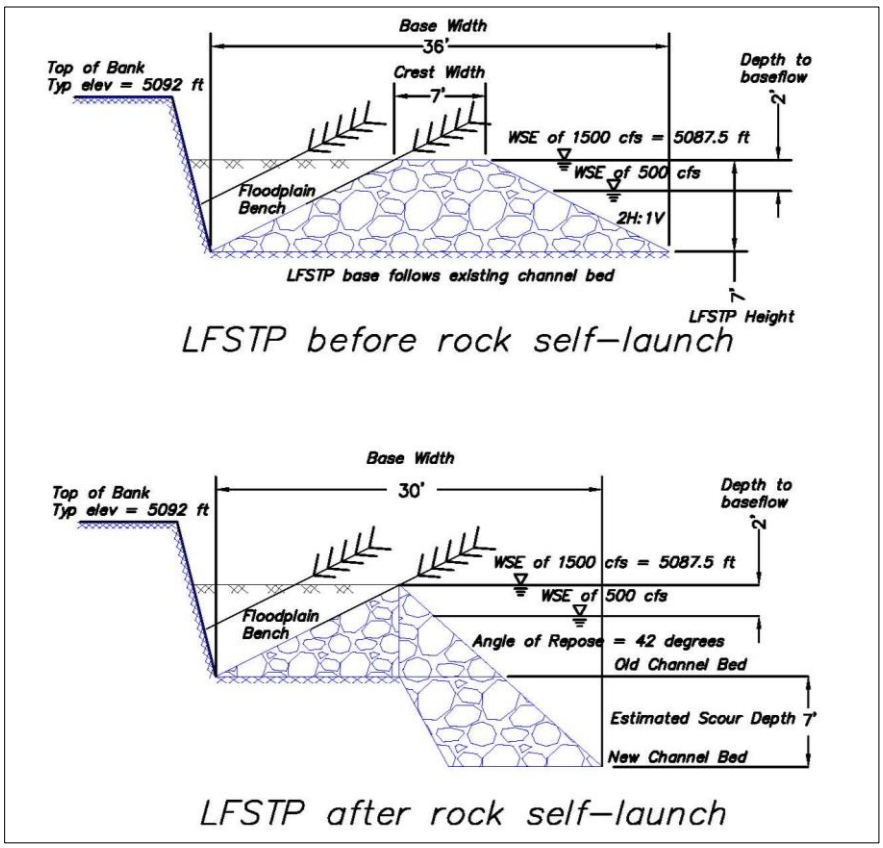

Figure 1 Process of LFSTP riprap launching into scour holes

<u>Design of Bendway Weirs:</u> Bendway weirs are low and upstream angled rock in-stream structures that help to lower velocities along a bankline and redirect the thalweg path away from the bankline (Kinzli and Thornton, 2010). The design references for bendway weirs are HEC-23

(FHWA, 2011), NCHRP 544 Report (McCullah and Gray, 2005), and Reclamation's modeling recommendations for transverse features such as spur dikes and vanes (Scurlock et al., 2012). From literature (FHWA, 2011; McCullah and Gray, 2005) the entire ranges of recommended bendway weir design parameters were derived. These ranges were narrowed down to a reasonable set of weir length, height, spacing, and planform angle. This was done based on literature documentation that cautioned against certain combinations (FHWA, 2011; Jia et al., 2005; Kinzli and Thornton, 2010). The narrowed ranges of the design parameters were combined into 18 combinations for the upstream bend and 37 combinations for the downstream bend. These sensitivity tests were used to determine the sensitivities of design parameters to velocities at the outer bank and at the centerline of the channel. After the sensitivity test, three design combinations were selected for each bend and a 2-dimensional (2-D) model was developed to test the post-weir baseline design parameters scenarios with the three design combinations at each bend. The scenarios modeled with a 2-D model are summarized in Table 2.

Table 2 Design parameters matrix tested with a 2-D model

Weir Parameters	Simulation ID	Weir Length (ft)	Weir Angle (degree)	Weir Spacing (ft)	Crest Elevation (ft)
Upstream Bend	Post-weir Baseline Design Parameters	65	60	85	5085
	MOD 2	65	60	85	5085.5
	MOD 3	60	70	80	5085
	MOD 4	70	70	90	5085
Downstream Bend	Post-weir Baseline Design Parameters	70	60	70	5084
	MOD 2	70	60	70	5084.5
	MOD 3	70	60	70	5083.5
	MOD 4	70	60	70	5083

FINALIZE DESIGN WITH A 2-D MODEL

SRH 2-D was selected as the hydraulic modeling software for this analysis because it was written and has been maintained by Reclamation so that the design team could get prompt technical support from Reclamation's current software developer.

As the 2-D model required very dense terrain data and there were no surveyed channel data points within areas in between the surveyed cross sections, the reach elevation was interpolated with cross sections six feet apart in HEC-RAS and exported to GIS to build the terrain for the 2-D model. As the 2-D model would be used to visually compare velocity distributions of different combinations of weir design parameters, the systematic errors caused by the interpolation of the terrain were thought to be similar for all tested bendway weir parameter combinations at the design discharge (2-year return, 5,600 cfs), which should not affect the overall final design selection. Values of WSE, velocities, shear stresses, etc. from the 2-D model would not be used specifically in the design process but rather to finalize a preferred design. Therefore, when it was assumed that the terrain systematic errors were constant for all combination simulations, the 2-D

model could assist the process to finalize the bendway weir design parameters. All 2-D simulation runs were performed with the assumption of a fixed bed and bank.

The post-weir baseline design parameters in Table 2 were initially considered to be the final parameter selection because they had a good balance between moderately reducing the outer bank maximum velocities and moderately increasing the centerline and inner bank average velocities. Therefore, they served as the post-weir design parameter baselines so that other parameter modifications should be compared to them. MOD 2 slightly changed the baseline condition by increasing all bendway weir height by 0.5 ft to investigate how higher weirs would benefit the project areas. MOD 3 has a different combination of weir design parameters at the upstream bend and a lower weir crest elevation with all other parameters unchanged at the downstream bend. MOD 4 tested another combination of bendway weir design parameters at the upstream bend as well as another lower weir crest elevation at the downstream bend with all other parameters unchanged. As the crests of the bendway weirs at the downstream bend were lowered to 5083 ft for MOD 4, the elevation of the bar near the southern end was excavated to 5082 ft, 1 ft lower than the bendway weir crest elevation at the downstream bend to release some hydraulic pressure from the high velocity field along the water intake structure. For all other modifications and baseline simulations, this bar elevation (5083 ft) was always lower than the bendway weir field at the downstream end. After the first four simulations, it was determined that MOD 3 was the best option for the upstream bend because it created a more distinct thalweg path immediately off the tips of the bendway weirs. Also the post-weir baseline simulation had the best combination for the downstream bend from the sensitivity analysis while other combinations did not show any significant difference in reducing near bank velocities. Besides it was found that lowering the bar at the southern end to 5082 feet would help to release the hydraulic pressure against the water intake structure. Therefore, this bar lowering feature was added to MOD 3 for the upstream bend and to the post-weir design parameter baseline for the downstream bend as the final weir design parameters that are summarized in Table 3.

Table 3: Final bendway weir design parameters

Weir Parameters	Bar Elevation (ft)	Weir Length (ft)	Weir Angle (degrees)	Weir Spacing (ft)	Crest Elevation (ft)
Upstream Bend	5082	60	70	80	5085
Downstream Bend	5082	70	60	70	5084

EXPECTED RESPONSE

The post-design 2-D model results showed that the velocities against the outer bank along the weirs at a bend were significantly reduced while area with higher velocities were moved to the tops of the weirs and toward the opposite point bank as in Figure 4. The model also showed that areas in between weirs had a variety of low to moderate velocities as in Figure 4. The primary hydraulic benefits of this project design are the relocation of the erosive force away from the bankline so that the river can still effectively deliver water and dissipate energy through its bends under high flow conditions within a confined space. The project's geomorphic benefits include the development of variable depth environments as the river scours along weir tips and opposite

point bars and deposits sediment in between weirs as in Figure 5. This provides bank protection, while still maintaining the current channel width and allows some geomorphic adjustments in a confined environment. The environmental benefits are potential establishment of vegetation in deposited areas between weirs and behind the LFSTP, plus a variable velocity and depth environment as in Figure 6 that potentially increases the opportunity for aquatic habitat. Therefore, it is hoped that this in-stream river training method will restore and sustain the ability of the reach to transport water and sediment, protect infrastructure along the waterway, and encourage ecosystem diversity within a confined and altered fluvial environment.

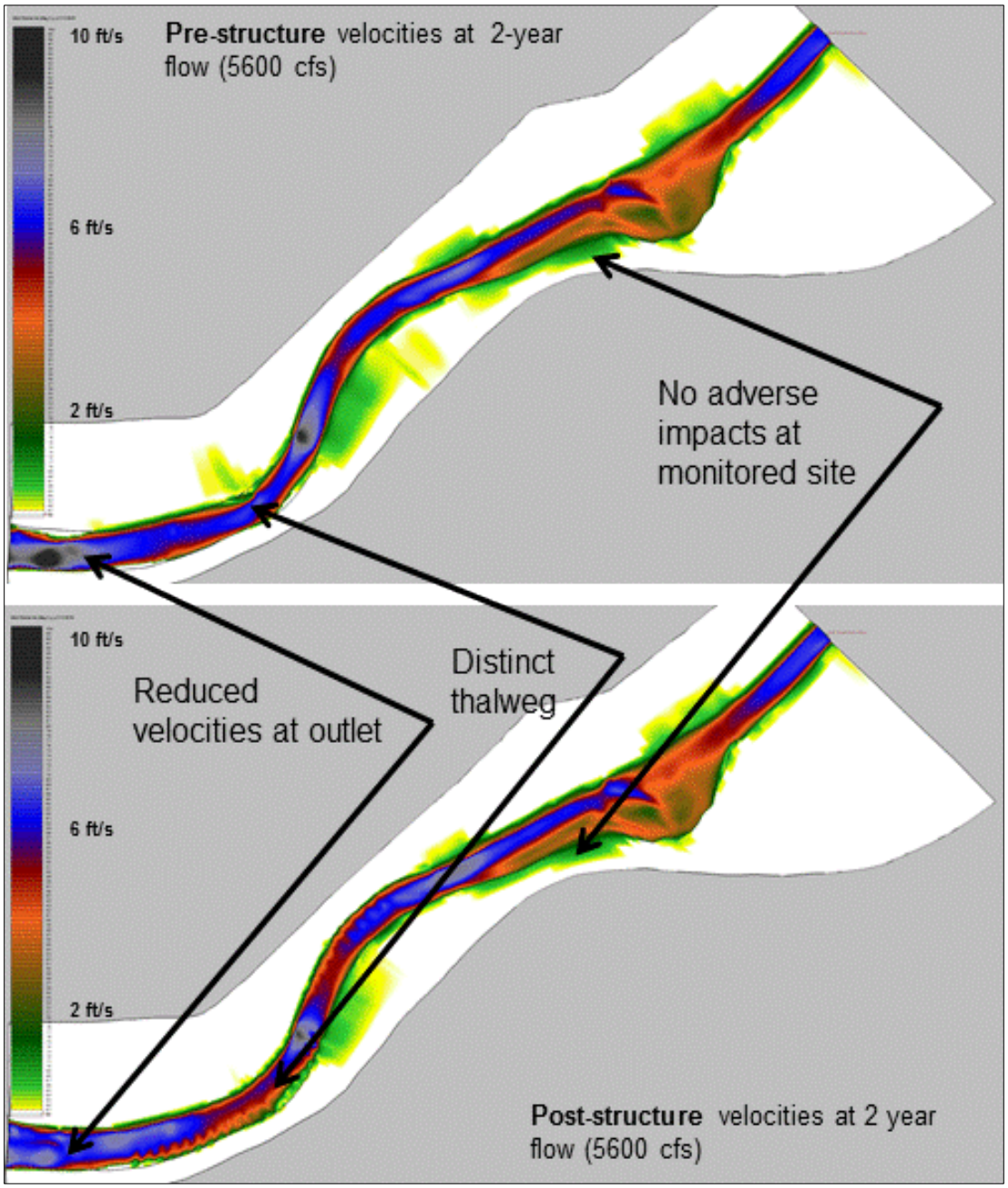

Figure 4 Hydraulic effects of in-stream structures

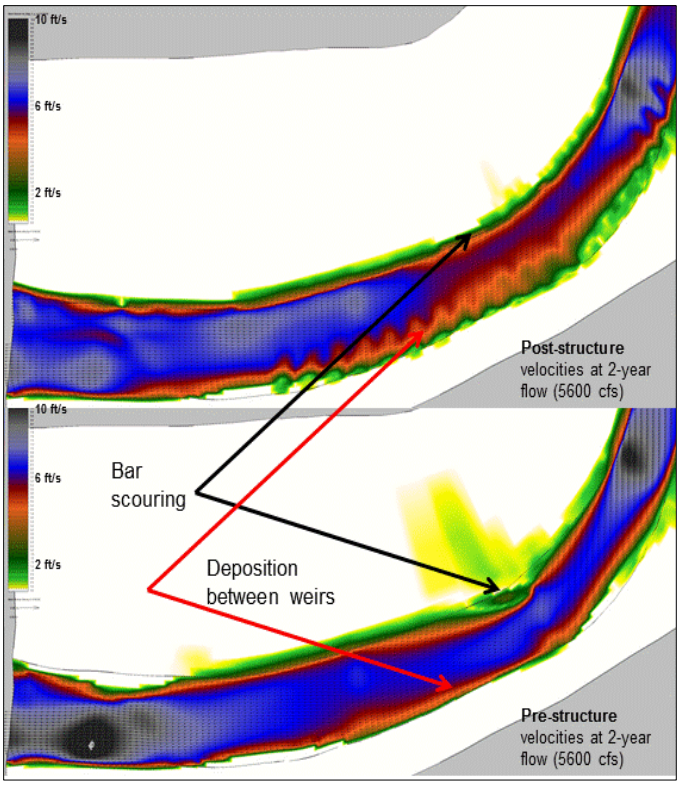

Figure 5 Geomorphic effects of in-stream structures

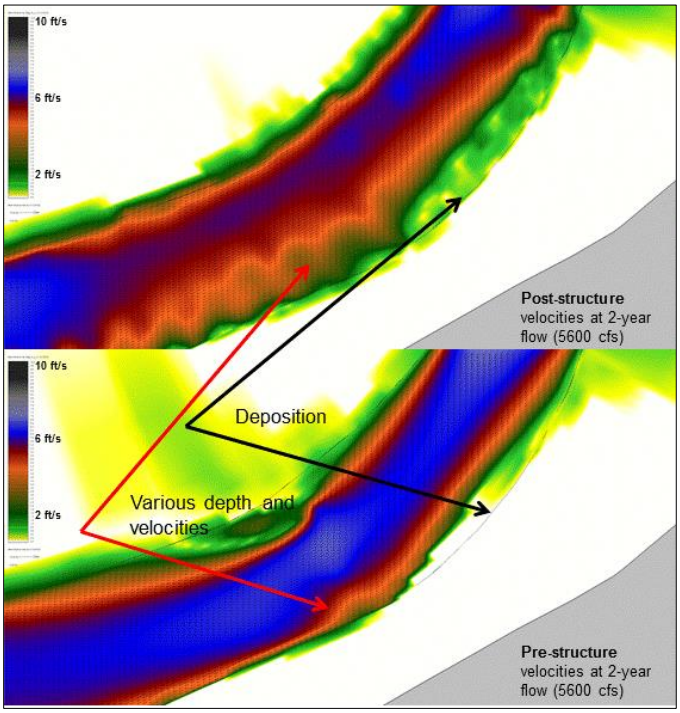

Figure 6 Environmental Effects of In-stream Structures

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