Channel Maintenance Alternatives and Sediment-transport Studies for the Rio Grande Canalization Project: Final Report

Submitted to:

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EXECUTIVE SUMMARY

ES-1. Background

The United States Section of the International Boundary and Water Commission (USIBWC) Rio Grande Canalization Project (RGCP) is a narrow river corridor that extends 105.4 miles from Percha Dam at River Mile (RM) 105.4 in Sierra County, New Mexico, to American Dam at RM 0 in El Paso, Texas (Parsons, 2004). The RGCP was constructed between 1938 and 1943 to facilitate compliance with the 1906 convention between the United States and Mexico, and to properly regulate and control, to the fullest extent possible, the water supply for use of the two countries as provided by the treaty (Parsons, 2004). Based on the Act of Congress of June 4, 1936 (Public Law 648; 49 Stat. 1463), USIBWC's statutory duties within the RGCP are to provide for efficient water flows and flood protection. As a result, major elements of the project included acquisition of Right-of-Way (ROW) for the river channel and adjoining floodways (8,332 acres; Parsons, 2004), improvement of the alignment and efficiency of the river channel conveyance for water delivery (conveyance efficiency), and flood-control measures that extended through the Rincon and Mesilla Valleys of New Mexico and El Paso Valley in Texas. As part of the RGCP, a deeper main channel was dredged to facilitate water delivery for irrigation. Flood protection levees were placed along two-thirds of the length of the RGCP where the channel was not confined by hillslopes or canyon walls (e.g., Selden Canyon). In addition to a variety of dams constructed prior to the 1960s, a number of Natural Resource Conservation Service (NRCS) sediment/flood-control dams were built between 1969 and 1975 on tributary arroyos to control flooding and sediment delivery to the RGCP from about 300 square miles of drainage basin downstream of Percha Dam.

Of the many challenges that the USIBWC faces in operating the RGCP, ongoing sediment delivery from the tributary arroyos has historically been among the most significant. Sediment deposition on the alluvial fans can result in sediment plugs, island formation, and aggradation that prevents draining of irrigation return flow that could result in increased water-surface elevations and associated impacts to levee freeboard and flood conditions. The sedimentation may also be affecting the delivery of water to U.S. stakeholders and Mexico due to reductions in channel and drain return efficiencies. One of the primary requirements of the USIBWC from the 2009 Record of Decision (ROD) involved identification of methods to improve river management through an evaluation of adaptive management strategies aimed at channel maintenance activities and levee protection. As part of the adaptive management strategy approach, the USIBWC is evaluating channel maintenance alternatives to address the sediment related problems along the RGCP.

ES-2. Study Objectives

This channel maintenance alternatives and sediment-transport study is intended to build upon previously developed conceptual restoration plans (USACE, 2009) and river management plans (Parsons, 2004) to specifically address issues associated with sedimentation along the RGCP (sediment plugs, island formation, raising of the river bed, reduced irrigation drain efficiency, and increased threats to levee freeboard and flooding). These problems occur at the nine representative problem locations that are evaluated in this study (**Table ES-1**; **Figure ES-1**). The nine problem locations are:

1. Tierra Blanca Creek to Sibley Arroyo (includes vortex weir below Tierra Blanca)
2. Salem Bridge to Placitas Arroyo (includes Hatch Bridge, Thurman Arroyo, and numerous islands)
3. Rincon Siphon A Restoration Site to Rincon Siphon (includes Garcia Arroyo)

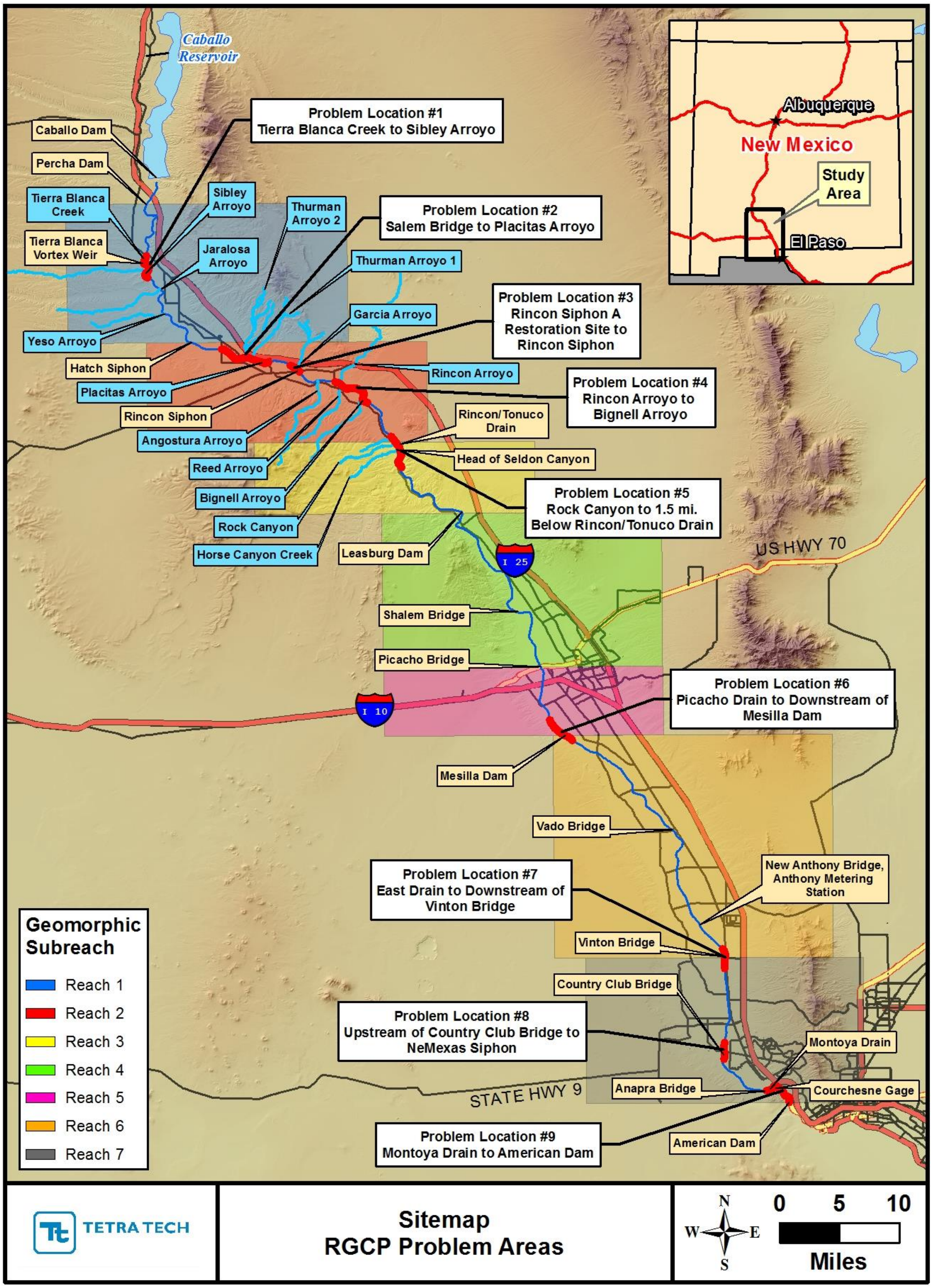

Figure ES-1. Map of the Rio Grande Canalization Project reach showing the locations of the nine problem locations that are considered in the Channel Maintenance Alternatives and Sediment-transport Studies project. Also shown are the Geomorphic Subreaches that were developed as part of the USACE (2007) study.

Table ES-1. Summary of the nine problem locations evaluated in this study.

Problem Location	Identification	Representation	Geomorphic Subreach[1]	D/S Station[2]	U/S Station[2]	RM Range[3]	Length (miles)	Comment
1	Tierra Blanca Creek to Sibley Arroyo	Vortex Weir	1	5168+00	5288+50	97.8 - 100.1	2.3	Draft Channel Maintenance Plan Low Priority Area; includes vortex weir below Tierra Blanca.
2	Salem Bridge to Placitas Arroyo	Arroyos and Islands	2	4459+10	4658+80	84.4 - 88.2	3.8	Draft Channel Maintenance Plan Low and High Priority Areas; includes Hatch Bridge, Thurman Arroyo and numerous islands.
3	Rincon Siphon A Restoration Site to Rincon Siphon	Restoration Sites and Siphon	2	4329+20	4371+40	82 - 82.8	0.8	Draft Channel Maintenance Plan Low and High Priority Areas; includes Garcia Arroyo
4	Rincon Arroyo to Bignell Arroyo	Arroyos and Islands	2	3986+60	4169+40	75.5 - 79	3.5	Draft Channel Maintenance Plan Low and High Priority Areas; includes Reed Arroyo
5	Rock Canyon to 1.4 mi below Rincon/Tonuco Drain Confluence	Drain and Mouth of Seldon Canyon	3	3643+50	3798+30	68.9 - 71.8	2.9	Draft Channel Maintenance Plan Medium Priority Area; Includes Horse Canyon Creek
6	Picacho Drain to below Mesilla Dam	Drain, Canals and Dam	5 and 6	2042+00	2167+10	38.8 - 41.2	2.4	Draft Channel Maintenance Plan Medium Priority Area; includes California Lateral
7	East Drain to below Vinton Bridge	Drain and Arroyo	6 and 7	785+70	876+00	14.8 - 16.6	1.8	Not a Draft Channel Maintenance Plan Priority Area but issues with sedimentation and flooding.
8	Upstream of Country Club Bridge to NeMexas Siphon	No Inputs, Bridge, Populated Area, Levee Encroachments	7	378+10	456+10	7.1 - 8.6	1.5	Draft Channel Maintenance Plan High Priority Area; Levee encroachment and freeboard concerns
9	Montoya Drain to American Dam	Drain	7	0	139+90	0 - 2.7	2.7	Draft Channel Maintenance Plan High Priority Area; Below Anapra Bridge

[1]From USACE (2007) Study.
[2]Station refers to the base model station line prepared for the USACE (2007) Study.
[3]Miles upstream from American Dam.

4. Rincon Arroyo to Bignell Arroyo (including Reed Arroyo)
5. Rock Creek to 1.5 miles Below Rincon/Tonuco Drain Confluence (including Horse Canyon Creek)
6. Picacho Drain to downstream of Mesilla Dam
7. East Drain to downstream of Vinton Bridge
8. Upstream of Country Club Bridge to NeMexas Siphon
9. Montoya Drain to American Dam

Results from the study provide a suite of alternatives to reduce or minimize the sediment issues at the 9 problem locations, and identify the most efficient, sustainable and environmentally beneficial methods. Once identified, the preferred alternatives can then be applied to other locations along the RGCP that have similar issues to the problem locations evaluated in this study.

ES-3. Study Approach

In general, this study evaluated five channel maintenance alternatives (CMAs) at each of the nine problem locations. At each of the problem locations except for the Mesilla Dam site, three of the CMAs are classified as "sediment-removal alternatives" and include "Channel Excavation Short", "Channel Excavation Long" and "Localized Sediment Removal" scenarios that involve excavation of sediments over varying distances and widths. At the Mesilla Dam problem location, the sediment removal alternatives only included the "short" and "long" scenarios. The remaining alternatives are classified as "Non-sediment Removal Alternatives" and vary by problem location, as discussed in Section ES-6, below.

A number of tasks were carried out as part of this assessment of CMAs for the RGCP that included a field assessment, targeted cross-section surveys, steady-state modeling of the overall RGCP, localized steady-state hydraulic modeling of the problem locations, sediment-transport modeling of the problem locations under existing and with-CMA conditions, and preparation of a benefit-cost/consequence analysis that was used to rank the alternatives and identify the two best CMAs at each location. These tasks are summarized in the following sections.

ES-4. Field Reconnaissance and Targeted Cross-section Surveys

A field reconnaissance of the Problem Locations 1 through 5 and 9 was carried out in October 2014 and Problem Locations 6 through 8 in February 2015 to assess the existing hydraulic conditions and geomorphic setting of the project reaches and preparation of this field assessment report. Sediment sampling was also conducted during the field reconnaissance to characterize the size distribution of the bed material.

The arroyos at the upper five problem locations (upstream from the Mesilla Diversion Dam problem location) have a significant effect on the hydraulic and sediment-transport conditions in the vicinity of the arroyos. Most of the arroyo fans create significant backwater effects that extend upstream over relatively long distances, and many of the fans have resulted in erosion of the opposite bank. Tributary derived sediments have accumulated downstream from the confluences and in many cases have created vegetated mid-channel or bank-attached bars that reduce conveyance efficiency.

At the lower four problem locations, the bed material is sand and the sedimentation issues have resulted in a variety of concerns. Sedimentation upstream from Mesilla Dam is affecting dam operations and is resulting in excessive sediment delivery to the Eastside and Westside Main Canals, requiring frequent maintenance by the Elephant Butte Irrigation District (EBID). At the

East Drain Problem Location, a number of west-side arroyos deliver coarse sediments to the mostly sand-bed reach, resulting in aggradation that is affecting flood conveyance as well as the efficiency of the East Drain. Aggradation at the Country Club Problem Location could result in increased water-surface elevations and associated levee freeboard encroachments that would disqualify the levee FEMA certification. The relatively low gradient at the Montoya Drain Problem Location results in low sediment-transport capacities that in turn have resulted in system-wide aggradation and the formation of numerous vegetated islands, many of which have formed or enlarged during the recent, ongoing drought period (2010–2015).

A total of 79 monumented cross sections were initially surveyed at Problem Locations 1 through 5 and 9 by the Tetra Tech team. The cross sections that were surveyed by Tetra Tech included cross sections that are in the base hydraulic model (discussed below) and were selected to be representative of the channel geometry and hydraulic controls through the study reach. Additional cross sections were surveyed at the mouths of the arroyos and drains, as well as at hydraulic controls that were identified during the field reconnaissance. As part of the surveys at Problem Locations 1 through 5 and 9, a total of 24 survey control points and 158 monumented end points were set and surveyed by Del Sur Surveying, LLC (a licensed surveyor in the State of New Mexico), and the topographic/bathymetric surveys included a total of 3,395 survey points. USIBWC conducted the cross section surveys at Problem Locations 6 through 8, and provided the data to Tetra Tech for purposes of conducting the hydraulic analysis. These surveys included a total of 79 cross sections, 49 of which were surveyed along the main channel of the RGCP, 19 of which were surveyed in the Eastside Main Canal and 11 of which were surveyed in the Westside Main Canal.

ES-5. Base Steady-state Hydraulic Modeling

The one-dimensional HEC-RAS hydraulic model that was developed for the USACE (2007) study was adopted as the base model of the RGCP for this study. This model was developed using in-channel survey data that was collected between 2004 and 2007 and 2004 LiDAR mapping in the overbanks. An updated base model of the overall RGCP was then developed by incorporating the cross-section survey data that was collected for this study at the nine problem locations. For the problem location cross sections that were not surveyed as part of this study, the geometry of the main channel was estimated by interpolating between the bounding surveyed cross sections. The overbank portions of the updated base model cross-sections that extended beyond the limits of the 2014 survey data were updated using the 2011 LiDAR mapping that was provided by USIBWC. This model was executed over a range of discharges from 500 cfs up to the 100-year peak discharge and included the discharges of interest to this study:

- Average annual spring hydrograph discharge (2,350 cfs above Mesilla Diversion Dam and 1,400 cfs downstream from the dam).
- Mid-range channel capacity (3,000 cfs)
- Upper range of channel capacity (3,500 cfs)
- 100-year routed peak discharge (ranging from 2,350 to 15,150 cfs)

A comparison of the predicted water-surface profiles from the base model and updated base model indicates that, as expected, the aggradation that has occurred along the majority of the reaches results in an increase in water-surface elevation. The largest increase in water-surface elevation occurred at Problem Location 3, where an average increase of about 3.0 feet was indicated (2,350 cfs). The smallest increase in water-surface elevation occurred at Problem Location 6 with an average increase of 0.2 feet (2,350 cfs), although this average value includes predicted decreases in water-surface elevation downstream from Mesilla Dam. Considering only the reach upstream from the dam, the average increase in water-surface elevation ranges from about 0.4 feet (3,500 cfs) to 0.6 feet (2,350 cfs). The only location where an average decrease in water-surface elevation occurs is at Problem Location 5 at the 100-year peak discharge. However, it should be noted that the original base model was developed for a range of relatively low flows (up to 6,000 cfs), whereas the updated base model was developed for a much larger range of flows up to the 100-year peak discharge (14,100 cfs at this location) and it was therefore necessary to extend some of the cross sections farther into the overbanks to contain the higher discharges. This is especially true at Problem Location 5, where the extended cross sections in the updated base model results in higher overbank flow conveyance and thus a reduction to the 100-year peak discharge water-surface elevation.

Localized base models of the problem locations were developed using the geometry for the updated base model of the overall RGCP. The downstream boundary condition for each of the models was obtained from the updated base model of the overall RGCP. These models were executed over the same steady-state discharges that were included in the updated base model of the RGCP and as such the results are identical to the updated base model results.

ES-6. Channel Maintenance Alternatives Development

The design for the sediment-removal alternatives was prepared using the existing bed profiles and information from the base and updated base hydraulic modeling. For the “Channel Excavation Long” alternatives, the up- and downstream limits of the excavation were located at the limits of the convex bed profile shape because this type of profile typically represents areas with the most significant aggradation. This generally places the upstream limit of excavation within a few hundred feet upstream of the arroyo mouth. The resulting excavation lengths under the “long” alternatives ranged from 3,900 to 11,800 feet. For the “Channel Excavation Short” alternatives, the up- and downstream limits of the excavation varied by problem location, but in general the upstream limit was set in the vicinity of the upstream limit under the “long” alternative, and a maximum target excavation length of 2,600 feet was used to set the downstream limit. The resulting excavation lengths for the “short” alternatives ranged from 900 to 2,700 feet, although an excavation length of about 4,700 feet was used at the Mesilla Dam site (Problem Location 6). The excavated bed profile under both the “short” and “long” alternatives was typically set to match the existing bed elevation at the downstream limit of the excavation with a slope that resulted in reasonable excavation depths through the excavated reach. Average excavation depths under both of these alternatives ranged from about 2.5 to 5 feet. The excavated channel width was then set such that the excavated channel has a flow capacity that restores the overall channel capacity to ~2004 conditions based on information presented in the USACE (2007) baseline study as well as the results from the 2007 baseline model. At locations where the existing channel capacity exceeds 3,500 cfs, the geometry of the excavated channel was designed to have a capacity of between 750 and 1,000 cfs since this range of discharges represents the lower regime of Caballo releases during normal operating conditions.

For the “Localized Sediment Removal” alternatives (also referred to as “Localized Excavation” or “Excavation at Mouth” alternatives), it was assumed that the excavated channel would span the entire width of channel and that the excavation profile would need to have a down-gradient slope and tie into the downstream existing bed profile to avoid creation of a pool/sediment trap. Target excavation lengths of 200 feet were used, but because the resolution of the available bed profile is derived from the modeled cross sections, which in some cases have a spacing that exceeds 300 feet, it was necessary to increase the excavation lengths beyond the target length. Excavation lengths for the “local” alternatives ranged from 80 to 690 feet.

The non-sediment removal alternatives also varied by site. At many of the sites where tributary sediment loading is the primary concern, construction of arroyo sediment traps upstream from the confluence with the RGCP could greatly reduce the coarse-grained sediment supply to the RGCP. Although it would be necessary to periodically excavate material from the sediment traps, the excavation would not occur in the bed of the active channel and loading/hauling costs could be reduced so this alternative could be less expensive than the excavations from the RGCP as historically practiced. At many of the sites where coarse-grained tributary sediments are resulting in sedimentation issues, another non-sediment removal alternative that could be employed involves construction of low-elevation spur dikes or vanes. The spurs could be designed to increase sediment-transport rates beyond the nose of the spurs while promoting deposition of coarser material between the spurs. The spurs along the bank opposite the alluvial fan would also provide the added benefit of bank protection. In addition, a number of site-specific alternatives were identified in the Statement of Work or developed during the course of this study.

ES-7. Steady-state Hydraulic Modeling of the CMAs

The localized steady-state base models were adjusted to represent with-alternative conditions to evaluate the short-term effects of the alternatives on hydraulic conditions with a specific focus on the effects on water-surface elevation. In general, the localized models for the CMAs were developed by adjusting the existing (updated base model) channel geometry, and in some cases the hydraulic roughness, to reflect elements associated with the alternatives.

The water-surface profile comparisons indicate that each of the alternatives evaluated would have at least have a localized effect on water-surface elevation. The effects of like alternatives vary by problem location, and in general show the largest effect at the lowest discharges and the smallest effect at the 100-year discharge. Many of the alternatives would result in no increase to predicted water-surface elevation over the range of modeled discharges and almost all of the alternatives (except the low-elevation spur dike alternative) would result in some localized decrease to predicted water-surface elevation. The lengths over which the alternatives would affect predicted water-surface elevation are dependent on both the longitudinal extent of the treatment as well the degree to which the treatment affects conveyance.

Based on the model results of the excavation alternatives, each scenario would result in reduced predicted water-surface elevations. None of the excavation alternatives would result in increases to predicted water-surface elevation except the short excavation alternative at Problem Locations 4 and 9, where very small, localized increases occur near the upstream limit of the excavations. As expected, the reduced conveyance area associated with the low-elevation spur dike alternative generally results in increased predicted water-surface elevations. The alternatives involving island/bar destabilization and vegetation typically result in reduced predicted water-surface elevations that are a result of the 6-inch lowering of the island and bar surfaces and the reduction to the hydraulic roughness. The comparative predicted water-surface elevations for the site-specific alternatives indicate the effects of these alternatives vary with the degree of modification. Modifications to the Tierra Blanca Vortex Weir at Problem Location 1 and installation of riprap revetment at Problem Location 8 result in very little change to predicted water-surface elevation. At Problem Location 3, replacement of the Rincon Siphon with an elevated flume and removal of the grade-control structure would have a much more significant impact, reducing the average predicted water-surface elevation at all of the modeled discharges.

ES-8. Sediment-transport Modeling of the Problem Locations

The sediment-transport modeling was performed with sediment-routing models of the problem location reaches that were developed using the mobile bed sediment-transport feature in HEC-RAS Version 4.1 (USACE 2010). At Problem Location 6, where flow splits upstream from Mesilla Dam deliver flow and sediment to the Eastside and Westside Canals, it was necessary to use the beta-test version of HEC-RAS 5.0 because Version 4.1 is not capable of modeling sediment splits at distributary junctions. The model geometry and basic structure of the models were taken from the localized hydraulic models discussed in the previous sections. In general, the mobile bed sediment-transport feature in HEC-RAS requires input to define the existing bed material, the upstream and lateral sediment supplies, the hydrologic sequence over which sediment transport is evaluated, as well as a variety of other model input that is necessary for the sediment-transport computations.

The bed material model input was developed using the gradations of the sediment samples that were collected during the field reconnaissance. Two separate hydrologic series were prepared, including a series that represents a best estimate of the hydrology during the Water Year 2005 (WY2005) to WY2014 period for input into the model validation runs, and a separate series that represents normal operating flows during a drought condition for input to the base and alternative sediment-transport simulations. The hydrologic series for the validation period was selected because cross section surveys were conducted in 2004 and in 2014/2015 for this study, and that information provided a basis for assessing the reasonableness of the sediment-transport modeling results. These hydrologic series were developed using measured discharges at the Caballo and El Paso gages, taking into account adjustments that were necessary to reflect the various inflows and outflows along the RGCP. For the hydrologic series that were used as input to the base and alternative model simulations, the measured flows at Caballo and El Paso during the 2013 irrigation season were used, after making similar adjustments to account for inflows and outflows and additional adjustments to represent “normal flow conditions” of 2,350 cfs upstream from Mesilla Dam and 1,400 cfs below the dam. These adjusted WY2013 hydrographs were then duplicated 10 times to represent a 10-year period of extreme drought conditions.

For both the base and validation model runs, the upstream bed material supply was estimated using the HEC-RAS “Equilibrium Load” option, which computes the sediment load (and gradation of the sediment load) that is in balance with the sediment-transport capacity, thereby creating an equilibrium condition at the upstream limit of the model. Bed material sediment supply from the tributaries was input assuming the mean annual bed load (USACE, 2007) is delivered during a single monsoon-season event, along with an appropriate water discharge.

Two different sediment-transport functions were used in the models, including the Meyer-Peter Müller (MPM) and Yang formulae. The MPM formula was used at Problem Locations 1 through 5, where tributaries deliver coarse sediments to the RGCP that have a significant impact on sediment-transport conditions in the river. The Yang formula was used at Problem Locations where the bed material is primarily sand. Numerous other model input that is required for the sediment-transport simulations was prepared based on an inspection of the results from initial model runs, previous experience with sediment-transport modeling of the Rio Grande, and engineering judgement. At Problem Location 6, the model input also included gate opening time series that represent operations of Mesilla Dam and the canal headworks.

Results from the validation model runs indicate that the predicted aggradation and degradation patterns match the observed (survey-based) patterns reasonably well. As such, the sediment-transport models should provide a reasonable tool for evaluating the effects of the alternatives. The localized base sediment-transport models were adjusted to reflect alternative conditions, and each were executed over the 10-year simulation period with duplicated drought condition (adjusted WY2013) annual hydrographs. Results from the modeling were used to compare the spatial and temporal effects of the alternatives on mean bed elevation change, aggradation and degradation along the modeled reach and downstream sediment deliveries. Results from the modeling of the sediment removal alternatives were also used to estimate the time over which the excavation volumes would backfill with sediment.

To evaluate the long-term effects of the alternatives on water-surface elevation, the predicted model geometry at the end of the simulation was incorporated into the localized steady-state hydraulic models. The end-of-simulation (EOS) models were then executed over the same steady-state discharges that were evaluated in the original localized hydraulic models. To provide a basis for comparison, the EOS model suite included a version of the base condition model with end-of-simulation geometry. Comparative water-surface profiles for normal operating flows of 2,350 and 1,400 cfs (above and below Mesilla Dam, respectively) and at the 100-year peak flow were then prepared to assess the long-term changes in water-surface elevation. The EOS-model water-surface profiles at the 100-year peak flow were also used to assess levee freeboard in areas where freeboard encroachments could be of concern in the long-term.

ES-9. Channel Maintenance Alternatives Evaluation

The expected benefits, costs and consequences associated with the alternatives were assessed in concert to identify the two alternatives that had the highest benefit relative to cost/consequence at each problem location in accordance with the statement of work for this study. The benefits considered in this assessment included: (1) reduction in water-surface elevation along the modeled reach, (2) reduced levee freeboard encroachments, (3) groundwater benefits, which include the benefit of increased groundwater levels in the vicinity of restoration sites as well as reduced groundwater levels elsewhere, (4) reduction in aggradation and downstream sediment loading, (5) improved irrigation drain return flows, (6) durability of the alternative, (7) restoration benefits, in addition to those benefits associated with increased groundwater levels, and (8) additional site-specific benefits. The costs and consequences considered in this assessment included: (1) annualized total cost of the alternative based on the up-front construction cost and projected O&M costs, (2) increases to water-surface elevation along the modeled reach, (3) levee freeboard encroachments, (4) groundwater consequences, which include the consequence of decreased groundwater levels in the vicinity of restoration sites as well as increased groundwater levels elsewhere, (5) increases to aggradation and downstream sediment loading, (6) increased bank erosion potential, (7) restoration consequences, in addition to those consequences associated with increased groundwater levels, and (8) additional site-specific consequences.

Construction cost estimates were developed for all alternatives within each problem location, and included the capital costs, along with annual operations and maintenance (O&M) costs for each alternative. The estimates were prepared with the best information available at this time and are considered pre-feasibility level estimates that are for comparison purposes and not for budgeting purposes. It is anticipated that many of the assumptions used within the estimates will be modified as more detailed information becomes available. To prepare the O&M cost estimates, a 50-year project life cycle was used and a maintenance period was prepared using either the results from the sediment-transport modeling and engineering judgement.

A scoring system was developed for each of the benefit and cost/consequence parameters using the results from the hydraulic and sediment-transport modeling. These systems were used to score each of the alternatives under the various scoring parameters and prepare overall benefit and cost/consequence scores. The two alternatives at each of the problem locations with the highest difference of benefit to cost/consequence were then identified and recommended for further consideration. The difference between the benefits and costs/consequences was computed by summing the individual benefit parameter scores and subtracting the summation of the cost and consequence scores. The recommended alternatives for each problem location and the difference between the net benefit and net cost/consequence scores are presented in **Table ES-2**.

Table ES-2. Summary of the two alternatives that received the highest ranking at each of the problem locations evaluated in this study, and the difference between the net benefit and net cost/consequence scores.

Problem Location	Alternative	Difference b/w Benefits and Costs/ Consequences*	Total Annualized Cost	Rank
1	Arroyo Sediment Traps	14.8	$285,000	1
	Vortex Weir	9.9	$4,100	2
2	Arroyo Sediment Traps	27.5	$90,600	1
	Island Destabilization/Vegetation Removal	19.2	$77,000	2
3	Arroyo Sediment Traps	12.4	$14,100	1
	Rincon Siphon Modifications	12.2	$100,400	2
4	Long Excavation	14.2	$653,500	1
	Island Destabilization/Vegetation Removal	7.2	$97,500	2
5	Arroyo Sediment Traps	12.1	$175,900	1
	Long Excavation	4.7	$269,500	2
6	Gate Automation	13.8	$164,200	1
	Sluiceway and Check Structures	11.1	$154,800	2
7	Arroyo Sediment Traps	26.6	$77,500	1
	Long Excavation	9.3	$164,800	2
8	Riprap	6.1	$28,300	1
	Spur Dikes	2.5	$34,200	2
9	Island Destabilization/Vegetation Removal	19.8	$32,300	1
	Long Excavation	18.1	$534,700	2

*Arithmetic difference between sum of individual benefit scores and sum of individual cost/consequence scores.

ES-10. Recommendations

Based on the findings of this study, a number of recommendations were identified for further evaluation, for incorporation into adaptive management practices, or for improving channel maintenance along the RGCP. These recommendations include:

1. Any of the recommended alternatives that are ultimately selected for implementation should be monitored locally as part of the adaptive management approach. It is recommended that this monitoring involves the establishment of monumented cross sections that are surveyed prior to and immediately after implementation to establish a base condition. Repeat surveys through time would be beneficial for evaluating channel response. These monitoring sections should also be included in the arroyos when applicable.
2. Because the sediment trap alternatives provide the highest benefit relative to costs and consequences, this alternative should be considered at all problem locations, and elsewhere along the RGCP, where tributaries deliver coarse sediment loads to the river. It is

recommended that, as part of the adaptive management approach, at least one sediment trap be constructed as laid out conceptually in this report for purposes of testing the trap efficiencies. During the testing of the sediment traps, if it is determined that it is desirable to also eliminate the finer fractions of the tributary bed material sediment supply, the trap screens could be re-designed to also trap the sand, silt, and clay classes. However, because the fine (sand, silt and clay) sediment loads delivered by the tributaries represents a relatively small portion of the overall fine sediment loads supplied and transported by the RGCP, this may not be worthwhile.

3. The tributary loading was based on a relatively simple approach of targeting a high, but realistic sediment concentration. Because tributary loads are highly variable it is recommended that arroyo sediment traps be monitored annually and enlarged if sediment removal is required too frequently.
4. Any opportunity to construct sediment traps that are larger than those evaluated in this study, which are limited in size due to the ROW constraint, should be considered and evaluated in detail. These opportunities should include the potential for construction of sedimentation basins in the upstream portions of the arroyo watershed.
5. Localized and short excavation scenarios do not appear to provide good value due to the high frequency of the excavations that would be necessary for maintenance purposes, therefore, it is recommended that any excavations be conducted over reaches that are as long as possible. Based on the long excavation alternatives evaluated in this study, a minimum length of 3,900 feet should be targeted for the excavations.
6. During the field reconnaissance of Problem Location 5, a beaver dam was identified at the mouth of the Rincon/Tonuco Drain. The dam appears to have an effect on drain efficiency and probably results in increased groundwater levels, at least during the non-irrigation season, so it is recommended that the dam be removed and the beavers be relocated. It is also recommended that beaver activity be monitored at other drains along the RGCP to ensure beaver dams are not affecting drain performance or local groundwater levels.
7. Because the sedimentation issues result in varying degrees of problems among the problem locations, it is recommended that the problem locations be prioritized during development of the implementation plan. The island/bar destabilization and vegetation removal alternative was evaluated for this study under the assumption that the full set of islands and bars that are selected for treatment be cleared and grubbed in concert. In practice, this work can be prioritized such that the largest islands and bars that have the most significant hydraulic effect receive the highest priority. Similar prioritization of the other alternatives (e.g. identification of the most problematic tributary sediment loadings for installation of the arroyo sediment traps), is also recommended prior to implementation.
8. The non-sediment removal alternatives in the vicinity of Mesilla Dam at Problem Location 6 would be affected by 2-D and 3-D flow and sediment patterns that are not taken into account in the 1-D HEC-RAS modeling conducted for this study. If it is desirable to provide more certainty to the scoring and ranking of the alternatives, the automated gate operator and check/sluiceway structure alternatives that were identified as having the highest benefit to cost/consequence difference should be evaluated further using a 2-D model platform or a physical model. The cost for conducting 2-D modeling of these alternatives is estimated to be less than 5 percent of the estimated cost of construction for each of the alternatives, while the cost for a physical model would probably be about 10 percent of the estimated cost of construction.

9. The proposed check structure and sluiceway alternative at Problem Location 6 will require relocation of the heading of the Del Rio Lateral Canal. Locating the new heading upstream from the Eastside Main Canal first check structure could result in undesirable sediment loading to the lateral if the heading were located upstream from the sluiceway. As such, the lateral canal heading should be located downstream from the sluiceway and upstream from the first check structure.

10. Many of the individual alternatives could be combined to enhance the expected relative benefits. For example, implementation of the modified Tierra Blanca Vortex Weir in addition to the sediment trap alternative may result in significant benefits for very little additional cost. Similarly, a sediment trap combined with localized excavation could produce immediate and long-term benefits. Combining of alternatives should be evaluated using similar methods to those used in this study, at a minimum.

11. The proposed modifications at the Rincon Siphon would include removal of the grade control structure below the current siphon. If this alternative were implemented, downcutting would probably occur along a significant portion of the upstream reach that could affect the foundations of the NM 154 and ATSF Railroad Bridges. It is therefore recommended that the as-built information for both bridges be reviewed to determine the depth to which the piers are buried. If it is determined that the piers are not sufficiently embedded below the channel invert elevation plus scour relative to the downstream limit of the grade control structure, the bridge foundations could be at risk to undermining. As such, it would be necessary to reconstruct the pier footings, which could result in a significant increase to the cost for this alternative that would reduce the difference between benefit and cost/consequence and potentially change the alternative ranking. All of this should be heavily considered in more detailed evaluations of this alternative.

12. Although the model validation simulations represent the no-action scenario under normal flow conditions (WY2005 to WY2014), the alternative evaluation presented in this study is based on extreme drought condition hydrology (WY2013). [Although the models were validated using the estimated actual hydrological conditions over the period from WY2005 to WY2014, the analysis and scoring of the alternatives was based on a 10-year simulation of the WY2013 irrigation release (repeated 10 times). As such, the analysis presented herein includes two separate no-action model runs, including: (1) a scenario under which the actual hydrologic conditions that occurred from WY2005 to WY2014, which was represented by the model validation simulations, and (2) a scenario under which extreme drought conditions occur over an extended (10-year) period, which were represented by the base model simulations that were used as the basis for the alternative evaluations.] It is recommended that the alternative evaluation also be conducted for normal hydrology to determine whether or not the scoring and ranking of the alternatives would change under a different flow regime. This exercise would be relatively simple because the tools that were developed for this study, primarily the sediment-transport models and alternative scoring systems, are in place.

13. The preliminary cost estimates prepared for this study may not reflect the actual construction and O&M costs, so a detailed cost analysis of the top-ranking alternatives should be carried out to better compare the two alternatives recommended at each problem location.

14. The levee freeboard analysis presented in this study uses the elevations of the levees as indicated by the 2011 LiDAR topography. It is recommended that in areas where activities are planned to improve levee freeboard conditions, the top of the levees be surveyed to ensure that the LiDAR-based levee elevations and the associated analysis presented herein are accurate. The surveys of the levees need not be extensive, but should rather include “spot” elevations at selected model cross sections for purposes of validating the 2011 LiDAR-based

levee top elevations that are reflected in the localized hydraulic modeling prepared for this study.

15. Although this study was not intended to include a detailed evaluation of the habitat benefits and consequences, the Tetra Tech team of engineers and geomorphologists understands the importance of these considerations, so the parameters were included in the scoring matrix. The scoring of the habitat benefit and consequence parameters presented in this report is somewhat subjective, so it is recommended that these parameters be re-evaluated in a separate study by an entity with appropriate expertise in riparian and aquatic ecology.
16. Each of the above recommendations should be considered as a task under a 5-year adaptive management plan, except recommendation number 8 if deemed unnecessary. In addition, it is recommended that each type of the generalized top-ranked alternatives be implemented at one of the problem locations, at a minimum, for purposes of testing under this 5-year plan. These generalized top-ranked alternatives include arroyo sediment traps, island/bar destabilization with vegetation removal and long excavation. It is also recommended that the two site specific alternatives that received the highest rank, including the installation of additional automated gate operators at Mesilla Dam (Problem Location 6, after the recommended further evaluation) and installation of riprap revetment below Country Club Bridge (Problem Location 8), be implemented as part of this 5-year plan because these alternatives appear to achieve the desired benefits with relatively low cost and consequence. As discussed in recommendation number 7, the 5-year plan should also prioritize the problem locations, and the details associated with the final design of the alternatives should be prioritized as part of the implementation plan.

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CONVERSION FACTORS

SI Units	U.S. Customary Units
1 millimeter	0.0394 inches
1 meters	3.2808 feet
1 kilometer	0.6214 miles
1 hectares	2.4711 acres
1 cubic-meter/second (cms)	35.315 cubic feet/second (cfs)
1 micro-meter/second	0.02835 feet/day
U.S. Customary Units	**SI Units**
1 inch	25.4 millimeters
1 foot	0.3048 meters
1 mile	1.6093 kilometers
1 acre	0.4047 hectares
1 cubic foot/second (cfs)	0.0283 cubic-meter/second (cms)
1 foot/day	3.5273 micro-meters/second
Common Conversions	
1 gallon/day	1.5472 x 10^{-6} cubic-feet/second
1 cubic-foot/second	646,320 gallons/day
1 acre-foot/day	0.5042 cubic-feet/second
1 cubic-foot/second	1.9835 acre-feet/day

LIST OF ACRONYMS

AMAFCA	Albuquerque Metropolitan Arroyo Flood Control Authority
BOR	Bureau of Reclamation
CMA	Channel maintenance alternatives
DEM	Digital Elevation Model
DSS	Del Sur Surveyors, LLC
DTM	Digital Terrain Model
EBID	Elephant Butte Irrigation District
EIS	Environmental Impact Statement
EOS	End-of-Simulation
FLO-2D	Two-dimensional Modeling Software
GPS	Global Positioning Software
GRF	Gradient restoration facilities
HEC-RAS	Hydrologic Engineering Center – River Analysis System (computer software, a widely accepted 1-D step-backwater hydraulic model)
LiDAR	Light Detection and Ranging
MEI	Mussetter Engineering Incorporated
NAD	North American Datum
NAIP	National Agriculture Imagery Program
NAVD	North American Vertical Datum
NAVD88	North American Vertical Datum of 1988
NRCS	Natural Resources Conservation Service
O&M	Operations and Maintenance
RGCP	Rio Grande Canalization Project
RM	River Mile
ROD	Record of Decision
ROW	Right-of-Way
RTI	Resource Technology, Inc.
SCS	Soil Conservation Service
Sta	Station
URGWOPS	Upper Rio Grande Water Operations
USACE	United States Army Corps of Engineers
USIBWC	United States International Boundary and Water Commission
UTM	Universal Transverse Mercator
WY	Water Year

1 INTRODUCTION

1.1. Background

1.1.1 The Rio Grande Canalization Project

The United States Section of the International Boundary and Water Commission (USIBWC) Rio Grande section of the Rio Grande Canalization Project (RGCP) (**Figure 1**) is a narrow river corridor that extends 105.4 miles from Percha Dam at River Mile (RM) 105.4 in Sierra County, New Mexico to American Dam at RM 0 in El Paso, Texas (Parsons, 2004). The RGCP reach is contained within the Lower Bioregion (Caballo Dam, NM to Candelaria, TX) geomorphic subreach of the Rio Grande (Fullerton and Batts, 2003). The RGCP was constructed between 1938 and 1943 under the authority of an Act of Congress approved June 4, 1936 (Public Law 648; 49 Stat. 1463), to facilitate compliance with the 1906 convention between the United States and Mexico, and to properly regulate and control, to the fullest extent possible, the water supply for use of the two countries as provided by the treaty (Parsons, 2004). The 1936 Act authorized the construction, operation and maintenance of the RGCP in agreement with the Engineering Record Plan of December 14, 1935 (Baker, 1943; cited in Parsons, 2004). Based on the 1936 Act, USIBWC's statutory duties within the RGCP are to provide for efficient water flows and flood protection

Major elements of the plan included acquisition of Right-of-Way (ROW) for the river channel and adjoining floodways (8,332 acres; Parsons, 2004), improvement of the alignment and efficiency of the river channel conveyance for water delivery (conveyance efficiency), and flood-control measures that extended through the Rincon and Mesilla Valleys of New Mexico and El Paso Valley in Texas. As part of the RGCP, a deeper main channel was dredged to facilitate water delivery for irrigation. Hydraulic capacity of the dredged channel ranged from 2,500 to 3,000 cfs in the Upper Rincon Valley, to less than 2,000 cfs in the Lower Mesilla Valley (Parsons, 2001). In general, the dredged channel followed the alignment of the existing channel in most locations but reduced the in-channel length by 5 percent. This resulted in a small increase in the average river bed slope from 0.00073 (3.85 ft/mi) to 0.00074 (3.9 ft/mi). Canalization included riprapping portion of the channel banks to prevent lateral migration of the channel.

Flood protection levees, currently designed to provide a 100-year level of flood protection, were placed along two-thirds of the length of the RGCP (57 miles along the west side of the channel and 74 miles along the east side), where the channel was not confined by hillslopes or canyon walls (e.g., Selden Canyon). The width between the levees north of Mesilla Dam ranged from 750 to 800 feet, and it was a constant 600 feet downstream of Mesilla Dam. In addition to a variety of dams constructed prior to the 1960s, a number of Natural Resource Conservation Service [NRCS, formerly Soil Conservation Service (SCS)] sediment/flood-control dams were built between 1969 and 1975 on tributary arroyos to control flooding and sediment delivery to the RGCP from about 300 square miles of drainage basin downstream of Percha Dam. The NRCS dams in Broad Canyon, Green Canyon, Arroyo Cuervo and Berrenda/Jaralosa Arroyo control approximately one third of the drainage area between Percha and Leasburg Dams, and reduce the flood peak frequency by an estimated 40 percent (RTI and USACE, 1996).

A recent evaluation of the project levees by MEI and Riada (USACE, 2007) determined that the design freeboard would be encroached during the 100-year flood (i.e., the water surface would be within 3 feet of the levee crest) along 37 miles of levee in Doña Ana County and 12 miles of levee in El Paso County. The USACE (2007) study found that levee overtopping would occur during the 100-year event at several locations along the reach, with a total length of about 1 mile

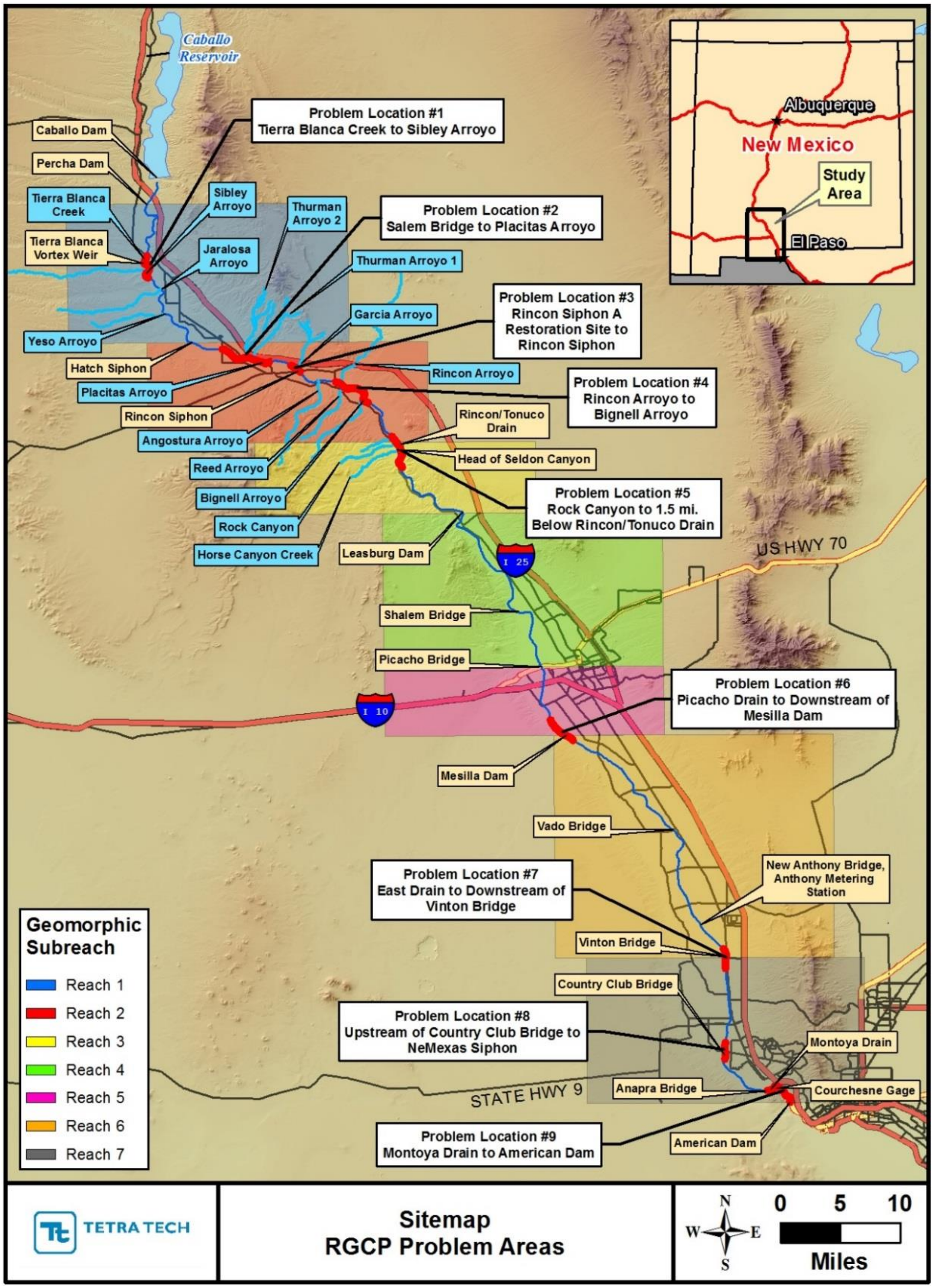

Figure 1. Map of the Rio Grande Canalization Project reach showing the locations of the nine problem locations that are considered in the Channel Maintenance Alternatives and Sediment-transport Studies project. Also shown are the Geomorphic Subreaches that were developed as part of the USACE (2007) study.

in Doña Ana County and about two miles in El Paso County. As a result of this study, USIBWC is currently in the process of raising the levees in the affected area.

1.1.2 Sedimentation Issues and Study Reasoning

Of the many challenges that the USIBWC faces in operating the RGCP, ongoing sediment delivery from the tributary arroyos has historically been among the most significant. Sediment deposition on the alluvial fans can result in sediment plugs, island formation, and aggradation that prevents draining of irrigation return flow that could result in increased water-surface elevations and associated impacts to levee freeboard and flood conditions. Although no arroyos enter the Country Club Bridge and Montoya Drain areas (Problem Locations 8 and 9, respectively), accumulation of sediment at these sites pose similar problems. The sedimentation along the RGCP may also be affecting the delivery of water to U.S. stakeholders and Mexico through reduced channel and drain return efficiencies. The alluvial fans also typically direct the flow into the opposite bank, resulting in bank erosion that could threaten the project levees or private property. One of the primary requirements of the USIBWC from the 2009 Record of Decision (ROD) involved identification of methods to improve river management through an evaluation of adaptive management strategies aimed at channel maintenance activities and levee protection. The ROD required USIBWC to update the River Management Plan (Parsons, 2004), including providing recommendations and guidelines for channel management policy, in consultation with stakeholders. It also committed USIBWC to evaluate the overall necessity of channel dredging though monitoring and modeling. It also required USIBWC to create a data collection and monitoring program to establish baseline conditions of the river, evaluate site-specific actions and impacts, and make recommendations for channel maintenance and channel stabilization or destabilization activities. As part of the adaptive management strategy approach, the USIBWC is evaluating channel maintenance alternatives to address the sediment related problems along the RGCP.

1.2. Study Objectives

This channel maintenance alternatives (CMA) and sediment-transport study is intended to build upon previously developed conceptual restoration plans (USACE, 2009) and river management plans (Parsons, 2004) to specifically address issues associated with sediment delivery from the tributaries. These problems occur at the nine representative problem locations that are evaluated in this study (**Table 1**). The nine problem locations are:

1. Tierra Blanca Creek to Sibley Arroyo (includes vortex weir below Tierra Blanca)
2. Salem Bridge to Placitas Arroyo (includes Hatch Bridge, Thurman Arroyo, and numerous islands)
3. Rincon Siphon A Restoration Site to Rincon Siphon (includes Garcia Arroyo)
4. Rincon Arroyo to Bignell Arroyo (including Reed Arroyo)
5. Rock Creek to 1.5 miles Below Rincon/Tonuco Drain Confluence (including Horse Canyon Creek)
6. Picacho Drain to downstream of Mesilla Dam
7. East Drain to downstream of Vinton Bridge
8. Upstream of Country Club Bridge to NeMexas Siphon
9. Montoya Drain to American Dam

Table 1. Summary of the nine problem locations evaluated in this study.

Problem Location	Identification	Representation	Geomorphic Subreach[1]	D/S Station[2]	U/S Station[2]	RM Range[3]	Length (miles)	Comment
1	Tierra Blanca Creek to Sibley Arroyo	Vortex Weir	1	5168+00	5288+50	97.8 - 100.1	2.3	Draft Channel Maintenance Plan Low Priority Area; includes vortex weir below Tierra Blanca.
2	Salem Bridge to Placitas Arroyo	Arroyos and Islands	2	4459+10	4658+80	84.4 - 88.2	3.8	Draft Channel Maintenance Plan Low and High Priority Areas; includes Hatch Bridge, Thurman Arroyo and numerous islands.
3	Rincon Siphon A Restoration Site to Rincon Siphon	Restoration Sites and Siphon	2	4329+20	4371+40	82 - 82.8	0.8	Draft Channel Maintenance Plan Low and High Priority Areas; includes Garcia Arroyo
4	Rincon Arroyo to Bignell Arroyo	Arroyos and Islands	2	3986+60	4169+40	75.5 - 79	3.5	Draft Channel Maintenance Plan Low and High Priority Areas; includes Reed Arroyo
5	Rock Canyon to 1.4 mi below Rincon/Tonuco Drain Confluence	Drain and Mouth of Seldon Canyon	3	3643+50	3798+30	68.9 - 71.8	2.9	Draft Channel Maintenance Plan Medium Priority Area; Includes Horse Canyon Creek
6	Picacho Drain to below Mesilla Dam	Drain, Canals and Dam	5 and 6	2042+00	2167+10	38.8 - 41.2	2.4	Draft Channel Maintenance Plan Medium Priority Area; includes California Lateral
7	East Drain to below Vinton Bridge	Drain and Arroyo	6 and 7	785+70	876+00	14.8 - 16.6	1.8	Not a Draft Channel Maintenance Plan Priority Area but issues with sedimentation and flooding.
8	Upstream of Country Club Bridge to NeMexas Siphon	No Inputs, Bridge, Populated Area, Levee Encroachments	7	378+10	456+10	7.1 - 8.6	1.5	Draft Channel Maintenance Plan High Priority Area; Levee encroachment and freeboard concerns
9	Montoya Drain to American Dam	Drain	7	0	139+90	0 - 2.7	2.7	Draft Channel Maintenance Plan High Priority Area; Below Anapra Bridge

[1]From USACE (2007) Study.
[2]Station refers to the base model station line prepared for the USACE (2007) Study.
[3]Miles upstream from American Dam.

Results from the study will provide a suite of alternatives to reduce or minimize the sedimentation issues discussed in the previous section, and identify the most efficient, sustainable and environmentally effective methods that meet mission requirements. Once identified, the alternatives can then be applied to other locations that have similar issues to the problem locations evaluated in this study.

1.3. Authorizations

This study of channel maintenance alternatives and sediment-transport along the RGCP was conducted by Tetra Tech, Inc. for the US Section of the International Boundary Commission under Contract No. IBM09D0006, Order No. IBM14T0016. The USIBWC Contracting Officer was Ms. Laura Baker. At the onset of the study, the USIBWC Contracting Officer's Representative was Dr. Padinare Unnikrishna, who was replaced by Mr. Derrick O'Hara in October 2014. The USIBWC Alternate Contracting Officer's Representative was Ms. Elizabeth Verdecchia. Tetra Tech's Program Manager for this study was Dr. Robert Mussetter, and Tetra Tech's Project Manager was Mr. Stuart Trabant. Dr. Lyle Zevenbergen performed the independent technical review as part of Tetra Tech's Quality Assurance and Quality Control (QA/QC) program.

1.4. Study Approach

In general, this study evaluated five channel maintenance alternatives (CMAs) at each of the nine problem locations (**Table 2**). At each of the sites except for the Mesilla Dam problem location, three of the CMAs are classified as "sediment-removal alternatives" and include:

1. Channel Excavation Short (also referred to as "Short Channel Excavation"). This alternative involves mechanically excavating a pilot channel within the overall main channel of the RGCP from the vicinity of the mouth of the arroyo or drain downstream over a relatively short distance. Depending on the location, the distance over which the excavation would occur generally ranges from 900 to 2,700 feet, and the pilot channel width ranges from 40 to 280 feet.
2. Channel Excavation Long (also referred to as "Long Channel Excavation"). This alternative involves mechanically excavating a pilot channel within the overall main channel of the RGCP from the mouth of the arroyo or drain downstream over a relatively long distance. Depending on the location, the distance over which the excavation would occur ranges from 0.7 to 2.2 miles, and the pilot channel width ranges from 50 to 100 feet.
3. Localized Sediment Removal (at Arroyo/Drain outlets or at Country Club Bridge). This alternative, also referred to as "Localized Excavation", involves localized mechanical excavation (i.e., at a specified area of interest or at the mouth of the arroyo or drain). Depending on the location, the distance over which the excavation would occur ranges from 80 to 500 feet, and the width of excavation varies from 120 to 300 feet.

At the Mesilla Dam Problem Location, the sediment-removal CMAs included only the Channel Excavation Short and Channel Excavation Long alternatives because the Localized Sediment Removal alternative would probably not result in the removal of sediment volumes that would be substantial enough to result in long term benefits, and because a range of non-sediment removal alternatives have been previously identified by others (EBID, 2014).

The remaining alternatives at each of the sites are classified as "Non-sediment Removal Alternatives" and vary by problem location. Specific details regarding the initial screening and development of the channel maintenance alternatives are discussed in Section 4.5.

A number of tasks were carried out as part of this assessment of CMAs for the RGCP, as outlined in the Statement of Work for the base task order contract (Problem Locations 1 through 5 and 9)

Table 2. Summary of the channel maintenance alternatives evaluated in this study.

Problem Location	Identification	Sediment-removal Alternatives	Non-Sediment Removal Alternatives
1	Tierra Blanca Creek to Sibley Arroyo	1. Channel excavation short[1,3,4] 2. Channel excavation long[1,3,4] 3. Localized channel excavation[1,3,4]	1. Construction of sediment traps in arroyos[2,4] 2. Modification of the TB Vortex Weir[2,3,4]
2	Salem Bridge to Placitas Arroyo	1. Channel excavation short[1,3,4] 2. Channel excavation long[1,3,4] 3. Localized channel excavation[1,3,4]	1. Construction of sediment traps in arroyos[2,4] 2. Island destabilization/vegetation removal[2,3,4]
3	Rincon Siphon A Restoration Site to Rincon Siphon	1. Channel excavation short[1,3,4] 2. Channel excavation long[1,3,4] 3. Localized channel excavation[1,3,4]	1. Construction of sediment trap in arroyo[2,4] 2. Replace Rincon Siphon with flume and remove GCS[2,3,4]
4	Rincon Arroyo to Bignell Arroyo	1. Channel excavation short[1,3,4] 2. Channel excavation long[1,3,4] 3. Localized channel excavation[1,3,4]	1. Island destabilization/spur dikes[2,3,4] 2. Construction of low-elev spur dikes for bank protection and gravel trapping below arroyos[2,3,4]
5	Rock Canyon to 1.4 mi below Rincon/Tonuco Drain Confluence	1. Channel excavation short[1,3,4] 2. Channel excavation long[1,3,4] 3. Localized channel excavation[1,3,4]	1. Construction of sediment trap in arroyos[2,4] 2. Construction of low-elev spur dikes in expansion zone at drain and vanes for bank protection and gravel trapping at Rock Canyon[2,3,4]
6	Picacho Drain to below Mesilla Dam	1. Channel excavation short[1,3,4] 2. Channel excavation long[1,3,4]	1. Installation of new check/sluice structures in canals[1,4] 2. Mesilla Dam gate automation[1,3,4] 3. Installation of vortex tubes or sediment collectors in canals[2,4]
7	East Drain to below Vinton Bridge	1. Channel excavation short[1,3,4] 2. Channel excavation long[1,3,4] 3. Localized channel excavation[1,3,4]	1. Construction of sediment trap in arroyos[2,4] 2. Construction of low-elev spur dikes[2,3,4]
8	Upstream of Country Club Bridge to NeMexas Siphon	1. Channel excavation short[1,3,4] 2. Channel excavation long[1,3,4] 3. Localized channel excavation[1,3,4]	1. Installation of riprap in narrow floodplain areas[1,3] 2. Construct low-elevation spur dikes to eliminate expansion in vicinity of bridge[2,3,4]
9	Montoya Drain to American Dam	1. Channel excavation short[1,3,4] 2. Channel excavation long[1,3,4] 3. Localized channel excavation[1,3,4]	1. Island destabilization/vegetation removal[2,3,4] 2. Construction of low-elevation spur dikes in expansion zone b/w Anapra Bridge and drain with island/bar destabilization[2,3,4]

[1]Alternative required for evaluation by Statement of Work
[2]Alternative identified as part of this study.
[3]Steady-state hydraulic model evaluation.
[4]Sediment-transport model evaluation.

and the Statement of Work Addendum for Modification No. 001 (Problem Locations 6 through 8; **Appendix A**[1]). These tasks included:

Task 1. Targeted Cross-section Surveys

- This task involved first performing a field reconnaissance of the nine problem locations to assess the existing hydraulic conditions and geomorphic setting of the project reaches. As discussed below, sediment sampling was also conducted during the field reconnaissance.
- The next phase of this task involved surveying selected cross sections at Problem Locations 1 through 5 and 9. Water-surface elevations were surveyed and discharge measurements were made at the time of the cross section surveys at locations where the discharge was measureable. The surveying of selected cross sections at Problem Locations 6 through 8 was performed by USIBWC.

Task 2. Hydraulic and Sediment-transport Modeling. This task involved three subtasks:

- Evaluation of the existing base (HEC-RAS) model of the RGCP (USACE, 2007), and updating the model using information collected during the targeted cross section surveys. The base model and updated base model was executed over steady-state discharges representing average annual conditions (2,350 cfs above Mesilla Diversion Dam and 1,400 cfs below the dam), representative bankfull conditions (3,000 and 3,500 cfs) and the routed, 100-year, 24-hour storm event that varies along the reach. Results from this modeling was used to evaluate the changes in water-surface elevation that have occurred since the base model condition (i.e., since the 2004 to 2007 period).
- Localized base models were developed that represent existing conditions at each of the sites. The localized base models were then adjusted to represent with-CMA conditions for the five alternatives discussed above. These models were executed over the same discharges to evaluate the effects of the CMAs on water-surface elevation.
- Sediment-transport modeling. The localized base and with-alternative HEC-RAS models were converted to mobile boundary sediment-transport models by incorporating appropriate boundary conditions and sediment-transport information (i.e., representative bed-material gradations using information from the sediment sampling conducted under Task 1). Results from this task provided estimates of the sediment-transport related benefits of the CMAs, and were also used to evaluate the sustainability and durability of the alternatives.

Task 3. Evaluate Channel Maintenance Alternatives

- Results from the hydraulic and sediment-transport modeling at each of the nine problem locations were used to evaluate the benefits associated with the five CMAs relative to the base condition. This evaluation required an assessment of the durability and sustainability of the alternative and preparation of estimated costs for the CMA implementation. Consequences of the CMA actions were also evaluated with respect to potential impacts on levee freeboard, future bank erosion (inferred from changes in hydraulic conditions and increased shear stresses predicted by the sediment-transport models), and groundwater levels. This information was used to rank the benefits of the alternatives relative to the costs and consequences, and the highest ranking two alternatives were identified at each location.

[1]The Statement of Work and Statement of Work Addendum identified Problem Location 6 as the Montoya Drain Site, Problem Location 7 as the Mesilla Dam Site, Problem Location 8 as the East Drain to below Vinton Bridge Site, and Problem Location 9 as the Country Club Bridge Site. For this study, the problem locations were renumbered sequentially in the upstream to downstream direction. The study tasks are outlined on Pages 3 through 8 of the Statement of Work and Pages 3 through 9 of the Statement of Work Addendum.

1.5. General Information about Report Content

A number of items are important to note about specific elements that are referenced in this report. All elevations refer to the North American Vertical Datum of 1988 (NAVD 88). The river stationing that was used to identify key features along the overall RGCP such as the up- and downstream limits of the problem locations is based on the base model station line that was prepared as part of the USACE (2007) study. River miles obtained from the USACE as part of that study were prepared for the 1994 Levee Study prepared by the RTI and the USACE (RTI and USACE, 1994) and therefore do not correlate with the base model river stationing. It should also be noted that the hydraulic and sediment-transport modeling conducted for this study does not include unsteady flow modeling that would be necessary to directly assess changes in conveyance efficiency. Results from the hydraulic and sediment-transport modeling were, however, used to evaluate the likely benefits associated with reduced water-surface elevations in the RGCP, which is a reasonable indicator of improved channel conveyance and drain return efficiencies. To improve readability and to simplify the table headings and figure legends, the term "Localized Excavation" is in some cases used in place of the term "Localized Sediment Removal", in which case the same terminology is used in both the text and referenced figures/tables. Similarly, the term "Short Excavation" is used interchangeably with "Channel Excavation Short", and the term "Long Excavation" is used interchangeably with "Channel Excavation Long".

2 GEOMORPHIC SUBREACH DESCRIPTIONS

The overall study reach has been previously divided into three primary subreaches: Upper (above Leasburg Dam), Middle (Leasburg Dam to Mesilla Dam) and Lower (Mesilla Dam to American Dam). The USIBWC further subdivided the RGCP reach using the seven River Management Units (RMUs) that were previously identified for the RGCP by Parsons (2003) as a guide (**Table 3**). The major geographic subreach boundaries (Rincon Valley, Selden Canyon and Mesilla Valley), coincide with the geologic structure and lithologic boundaries (Seager et al., 1975; Mack, 1997) which influence the volume and caliber of the arroyo sediment supply to the RGCP. The nine problem locations fall within Geomorphic Subreaches 1 through 3 and 5 through 7 (Table 1). A brief description of these subreaches is provided in the following sections that summarize the geomorphic setting and geologic structure as presented in USACE (2007) to provide overall context for the specific problem locations. **It should be noted that the subreach-averaged hydraulic conditions presented below are based on the 2007 version of the base model, and not the updated base model discussed below.**

Table 3. Subreach boundaries for the RGCP (from USACE, 2007).

Subreach No.	Subreach Name	Upstream Boundary (RM and Station)	Downstream Boundary (RM and Station)	Subreach Length (mi)	Upstream Location	Downstream Location
1	Upper Rincon	105.4 5576+00	92 4768+00	13.4	Percha Diversion Dam	Hatch Siphon
2	Lower Rincon	92 4768+00	72 3730+00	20	Hatch Siphon	Head of Selden Canyon
3	Selden Canyon	72 3730+00	63 3280+00	9	Head of Selden Canyon	Leasburg Diversion Dam
4	Upper Mesilla	63 3280+00	46.5 2416+00	16.5	Leasburg Diversion Dam	Picacho Bridge
5	Las Cruces	46.5 2416+00	40 2076+00	6.5	Picacho Bridge	Mesilla Diversion Dam
6	Lower Mesilla	40 2076+00	16 832+00	24	Mesilla Diversion Dam	Vinton Bridge
7	El Paso	16 832+00	0 0+00	16	Vinton Bridge	American Diversion Dam

2.1 Geomorphic Subreach 1 (Problem Location 1)

Subreach 1 extends from Percha Dam to the Hatch Siphon, a distance of 13.4 miles. The channel has historically degraded between 4 and 6 feet since the canalization in 1943 (**Figure 2**), and the bed material has coarsened as a result. The bed slope in the subreach is 0.00083 (4.4 ft/mi), and the bankfull capacity of the channel varies from 3,500 cfs to greater than 6,000 cfs. Based on the existing base model of the RGCP (USACE, 2007), at a flow of 5,000 cfs, the average channel top width is about 180 feet, the average hydraulic depth is 6.2 feet, and the average velocity is 4 fps. The distance between the levees in the subreach varies from 750 to 800 feet.

The Caballo Mountains and Rincon Hills are the most predominant sediment source for this reach of the RGCP. The headwaters of the west side arroyos primarily drain areas that are underlain by the Lower Pleistocene-age Upper Santa Fe Group (Camp Rice Fm.) that is composed of unconsolidated to poorly consolidated, erodible, and relatively fine-grained basin-fill sediments. The downstream portions of the arroyos traverse areas underlain by the Lower Santa Fe Group that include conglomeratic sediments with interbedded basalts that produce coarser-grained sediments. The eastside tributaries drain areas underlain by Paleozoic interbedded shales, sandstones and limestones that make up the lower portion of the Caballo Mountains and Rincon Hills, but the east side tributary arroyos also traverse the Camp Rice Fm. basin fill sediments.

2.2 Geomorphic Subreach 2 (Problem Locations 2, 3 and 4)

Subreach 2 extends from the Hatch Siphon to the head of Selden Canyon, a distance of 20 miles. Immediately downstream of the Hatch Siphon, the channel has historically degraded about 10 feet since 1943 (Figure 2). For the remainder of the upper part of the subreach, the degradation reduces from about 6 feet in the upstream end to about 1 foot upstream of the Rincon Siphon. Downstream of the Rincon Siphon, there has been about 9 feet of degradation, but the degradation diminishes in the downstream direction to about 2 feet. Upstream of Bignell Arroyo there has been about 2 feet of aggradation since 1943. Depending on the location within the subreach the bed material varies from sand to gravel. The bed slope in the subreach is 0.00074 (3.9 ft/mi), and the bankfull capacity of the channel varies from 3,500 to 4,500 cfs. At a flow of 4,000 cfs, the average channel top width is about 270 feet, the average hydraulic depth is 4.4 feet, and the average velocity is 3 fps. The distance between the levees in the subreach varies from 750 to 800 feet.

The important sediment source areas in the upper portion of Geomorphic Subreach 2 (upstream from Hatch) are the same as those for Subreach 1, so the types of sediment delivered by the east side and west side tributaries are also consistent with the upstream subreach. The headwaters of the more southerly tributaries in the lower Rincon Valley on the west side of the Rio Grande are located within the Sierra de las Uvas Mountains that are underlain by Tertiary-age basaltic andesites and volcanoclastic sedimentary units (Scholle, 2003). These geologic features produce coarse-grained sediments that are ultimately delivered to the RGCP by the tributary arroyos including Placitas, Reed and Bignell Arroyos.

2.3 Geomorphic Subreach 3 (Problem Location 5)

Subreach 3 extends from the head of Selden Canyon to Leasburg Diversion Dam, a distance of 9 miles. There are no comparative thalweg data for this subreach, but under low-flow conditions the bed of the channel is braided and appears to be mildly aggradational. The bed slope in the subreach is 0.00066 (3.5 ft/mi), and the bankfull capacity of the channel varies from 3,500 to 4,500 cfs. At a flow of 4,000 cfs, the average channel top width is about 230 feet, the average hydraulic depth is 4.7 feet, and the average velocity is 3.2 fps. There are no RGCP levees in the subreach. A large number of arroyos on both the east and west side of the river deliver sediment to the Rio Grande in this subreach. Because of the presence of Highway 185 on the west side of the river through the canyon, many of the west side arroyos have been channelized in the vicinity of the highway. Selden Canyon has formed where the Rio Grande cuts through the eastern portion of the Sierra de las Uvas Mountains. The canyon is bounded by the Tertiary-age basaltic andesites and volcaniclastic sedimentary units and the Lower Santa Fe Group sediments, all of which tend to produce coarse-grained sediments.

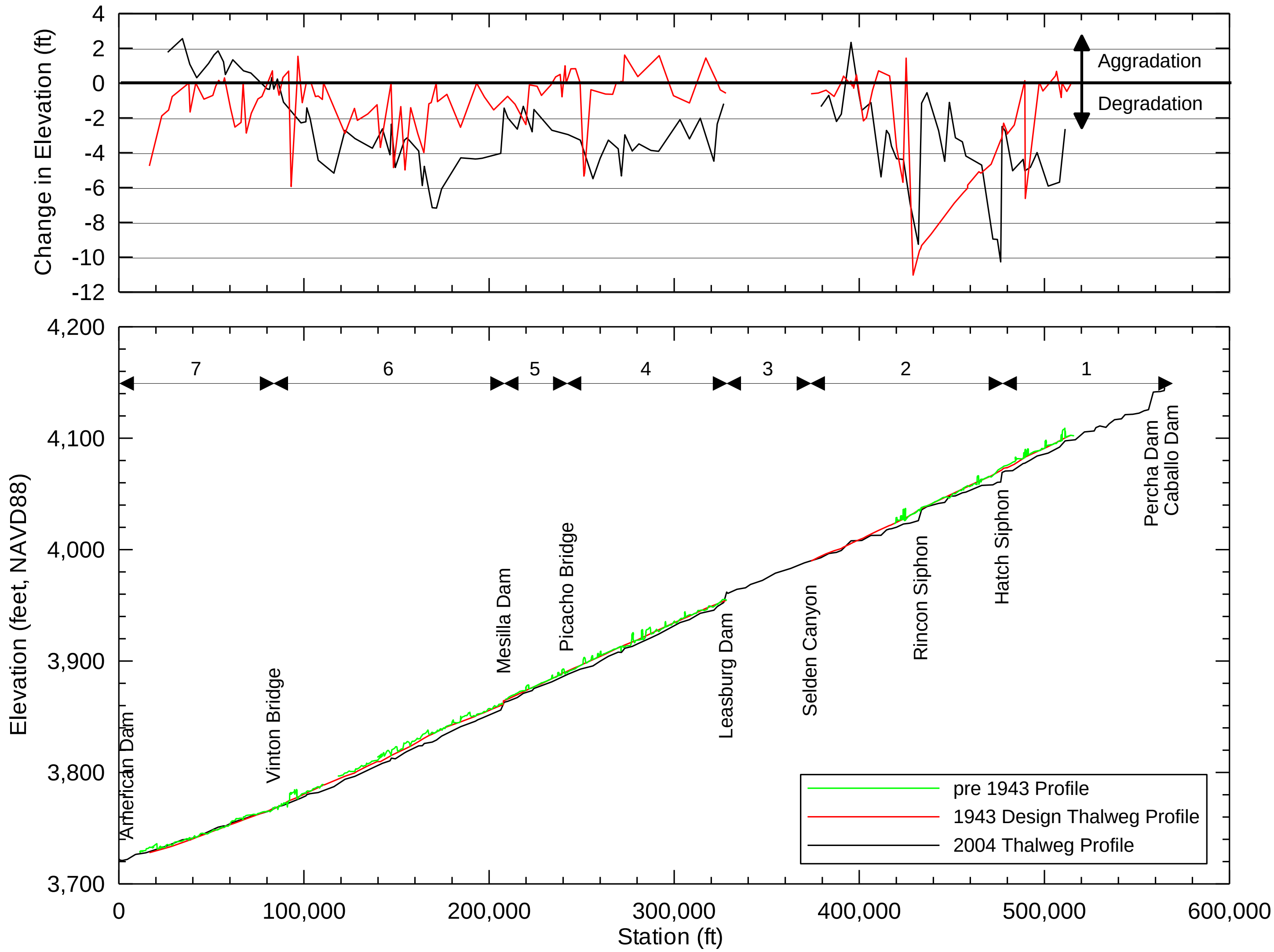

Figure 2. Pre-canalization, 1943 design and 2004 thalweg profiles of the RGCP. Also shown are the changes in elevation between the pre-canalization and 1943 profiles (green line) and between the 1943 profile and the 2004 profile (red line) [from USACE (2009)].

2.4 Geomorphic Subreach 5 (Problem Location 6)

Subreach 5 extends from the Picacho Bridge to the Mesilla Diversion Dam, a distance of 6.5 miles. The comparative thalweg data (Figure 2) indicate that there has been 2 to 3 feet of historical degradation in this subreach since 1943. However, USIBWC has historically performed extensive sediment-removal activities in the vicinity of Mesilla Diversion Dam, so the thalweg profiles do not show the aggradational tendencies at the downstream limit of this subreach. The bed slope in the subreach is 0.00074 (3.9 ft/mi), and the bankfull capacity of the channel ranges between 3,500 and 4,500 cfs. At a flow of 4,000 cfs, the average channel top width is about 360 feet, the average hydraulic depth is 3.3 feet, and the average velocity is 2.9 fps. The bed material in the subreach is sand and under low-flow conditions the bed of the channel is braided. The distance between the levees in the subreach varies from 750 to 800 feet. A considerable percentage of both banks within the subreach is revetted. A number of west side arroyos that drain the Upper Santa Fe Group deliver sediment to the Rio Grande between the Picacho Drain (~Sta 2160+00) and the Mesilla Diversion Dam.

2.5 Geomorphic Subreach 6 (Problem Locations 6 and 7)

Subreach 6 extends from the Mesilla Diversion Dam to the Vinton Bridge, a distance of 24 miles. The comparative thalweg data (Figure 2) indicate that there has been up to 8 feet of historical degradation downstream of the Mesilla Diversion Dam, but the amount of degradation diminishes in the downstream direction to about 1 foot. The bed slope in the subreach is 0.00074 (3.9 ft/mi), and the bankfull capacity of the channel is about 3,000 cfs. At this discharge, the average channel top width is about 245 feet, the average hydraulic depth is 3.3 feet, and the average velocity is 3 fps. The bed material in the subreach is sand and under low-flow conditions the bed of the channel is braided. The average distance between the levees in the subreach is 600 feet. Bank revetment along both banks is intermittent but considerable. This subreach is bounded by the Upper Santa Fe Group that consists of poorly consolidated, fine-grained, basin fill sediments of the Mesilla Bolson (Hawley, 1981). There are no significant sources of sediment delivery to the Rio Grande within the subreach, although numerous small arroyos deliver relatively small quantities of mostly fine-grained sediments.

2.6 Geomorphic Subreach 7 (Problem Locations 7, 8 and 9)

Subreach 7 extends from the Vinton Bridge to the American Diversion Dam, a distance of 16 miles. The comparative thalweg data (Figure 2) indicate that there has been up to 2 feet of aggradation since 1943. The bed slope in the subreach is 0.00056 (3 ft/mi), and the bankfull capacity of the channel is between 2,000 cfs and 2,500 cfs. At a flow of 2,000 cfs, the average channel top width is about 240 feet, the average hydraulic depth is 2.8 feet, and the average velocity is 2.5 fps. The bed material in the subreach is sand and under low-flow conditions the bed of the channel is braided. The distance between the levees in the subreach is similar to Subreach 6 (about 600 feet). Large portions of both banks within the subreach are revetted. This subreach is bounded by the Upper Santa Fe Group that consists of poorly consolidated, fine-grained, basin fill sediments of the Mesilla Bolson (Hawley, 1981). Similar to Subreach 6, there are no significant sources of sediment delivery to this subreach of the RGCP, but the smaller arroyos deliver some degree of mostly fine-grained material derived from basins composed of the Upper Santa Fe Group.

3 FIELD RECONNAISSANCE AND SITE SURVEYS

3.1 Field Reconnaissance

A field reconnaissance of Problem Locations 1 through 5 and 9 was carried out during the week of October 6, 2014, and a field reconnaissance of Problem Locations 6 through 8 was carried out during the week of February 16, 2015, to evaluate the morphological and hydraulic characteristics of the problem locations and to collect sediment samples to characterize the gradation of the bed material. Pebble counts (Wolman, 1954) were generally collected to characterize the armor material that remains at and downstream from the coarse-grained alluvial fans after reworking and coarsening by irrigations flows, whereas the bulk samples were collected from the bed of the channel or from beneath the armor layer to represent the range of sediments that have been recently transported as bed material load. A Field Assessment Report was prepared to present background information regarding the geomorphic setting of the problem locations and summarize the findings of the field reconnaissance (**Appendix B**). Representative ground photographs that were collected during the site reconnaissance are included in the field assessment report (Appendix B). Based on the field assessment, a description of the problem locations was prepared, as follows.

3.1.1 Problem Location 1

Problem Location 1 extends from the confluence with Tierra Blanca Creek downstream to Sibley Arroyo over a distance of about 1.7 miles (**Figure 3**). To update the base model of the RGCP and to develop the localized base model, the reach was extended about 1,100 feet upstream from the confluence with Tierra Blanca Creek and about 2,500 feet downstream from the confluence with Sibley Arroyo, resulting in a reach length of 2.3 miles. Garfield Bridge (New Mexico Highway 187) is located just upstream from this reach, about 1,700 feet upstream from the confluence with Tierra Blanca Creek. The Tierra Blanca Vortex Weir is located about 2,000 feet downstream from the confluence with Tierra Blanca Creek. Sediment has deposited along the main channel portion of the weir, burying the majority of the crest.

This problem location experiences sediment loading from 3 significant tributaries. Tierra Blanca Creek and Sibley Arroyo are west side tributaries and Green Arroyo is an east side tributary with its mouth across from the mouth of Tierra Blanca Creek. During recent monsoon season tributary flow events (i.e., 2006 and 2013), Tierra Blanca Creek and Sibley Arroyo delivered significant quantities of sediment to the RGCP. The damage to the existing bank protection on the east (left) bank of the river opposite the Tierra Blanca fan has not been repaired, and no sediment has been removed from the fan. The bank erosion along the left bank that is being caused by the Sibley Arroyo fan is not as significant. The fans for both of these tributaries results in significant backwater effects in the upstream reaches. The NRCS flood control and sediment detention dam on Green Arroyo has significantly reduced the amount of sediment delivered to the RGCP. The USIBWC has not performed any channel maintenance activities at this location since 2007.

Five sediment samples were collected in the vicinity of Problem Location 1. Pebble Count PC1 represents the fan-derived armor material along the bed of the Rio Grande and was collected from the riffle that has formed along the downstream fringe of the Tierra Blanca Creek fan and has a median grain size (D_{50}) of about 34 mm (**Figure 4**). Bulk Sample S1 (D_{50} = 2.1 mm) was collected from the subsurface materials in the vicinity of Pebble Count PC1 and includes about 50-percent gravel. Pebble Count PC2 was collected from the distal portion of the Sibley Arroyo fan and has a D_{50} of about 42 mm. Bulk Sample S2 (D_{50} = 1.8 mm; Figure 4) was collected from the subsurface materials in the vicinity of Pebble Count PC2 and has a gradation that is very similar to that of Sample S1. Bulk Sample S3 (D_{50} = 6.9 mm; **Figure 5**) was collected from the

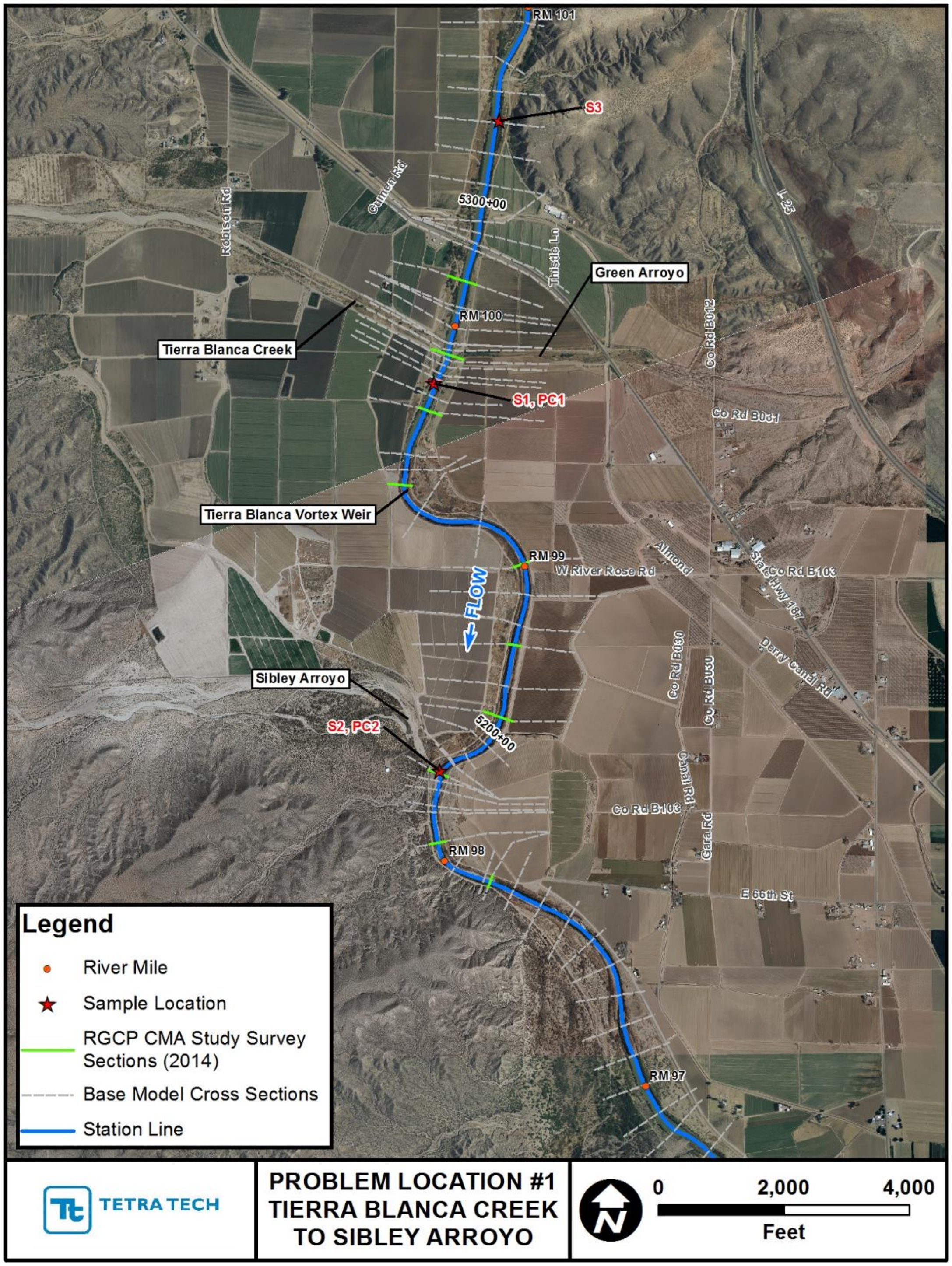

Figure 3. Aerial photograph showing key features at Problem Location 1 (Tierra Blanca Creek to Sibley Arroyo).

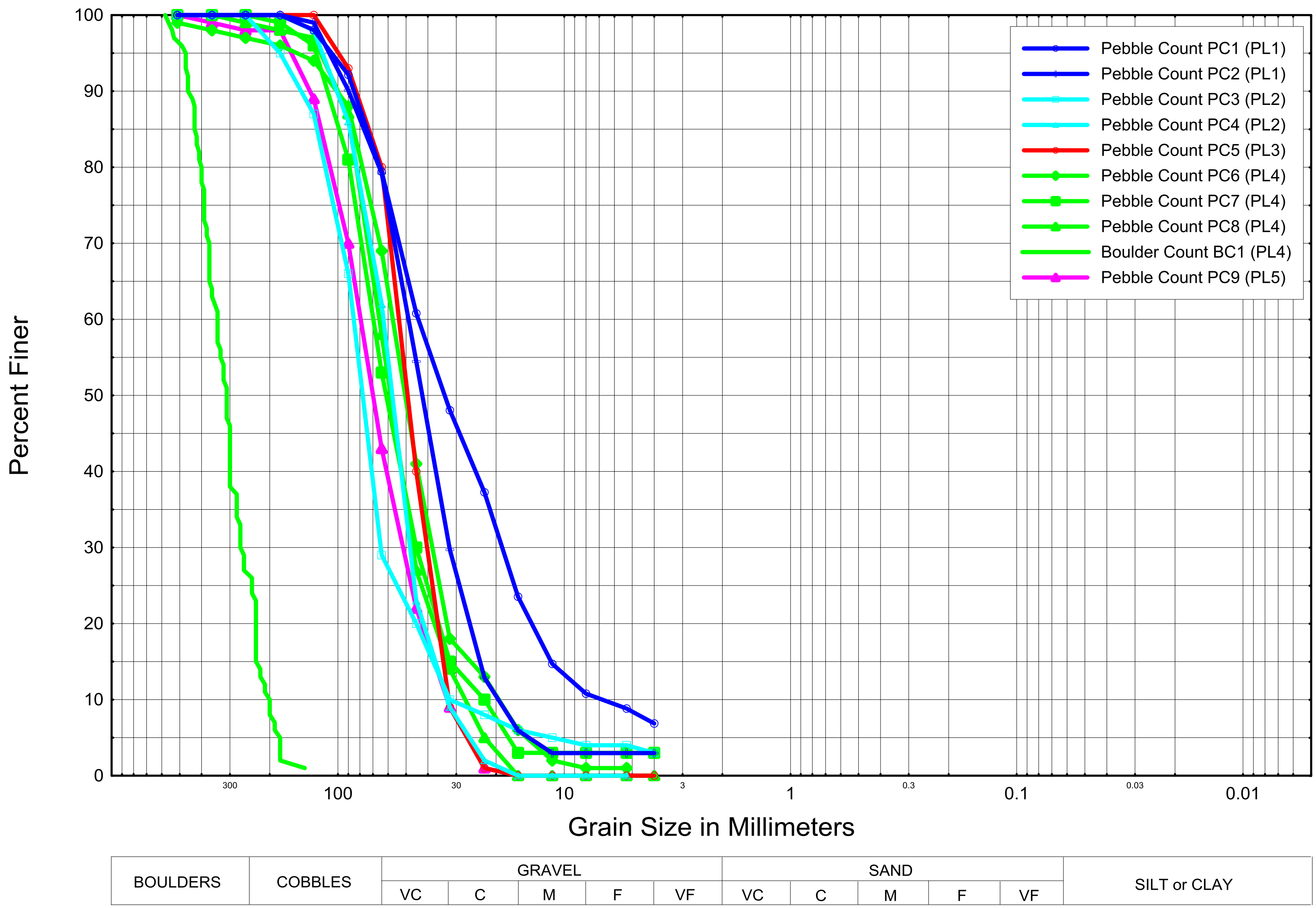

Figure 4. Gradation curves for the pebble count samples collected from the five problem locations with coarse bed material.

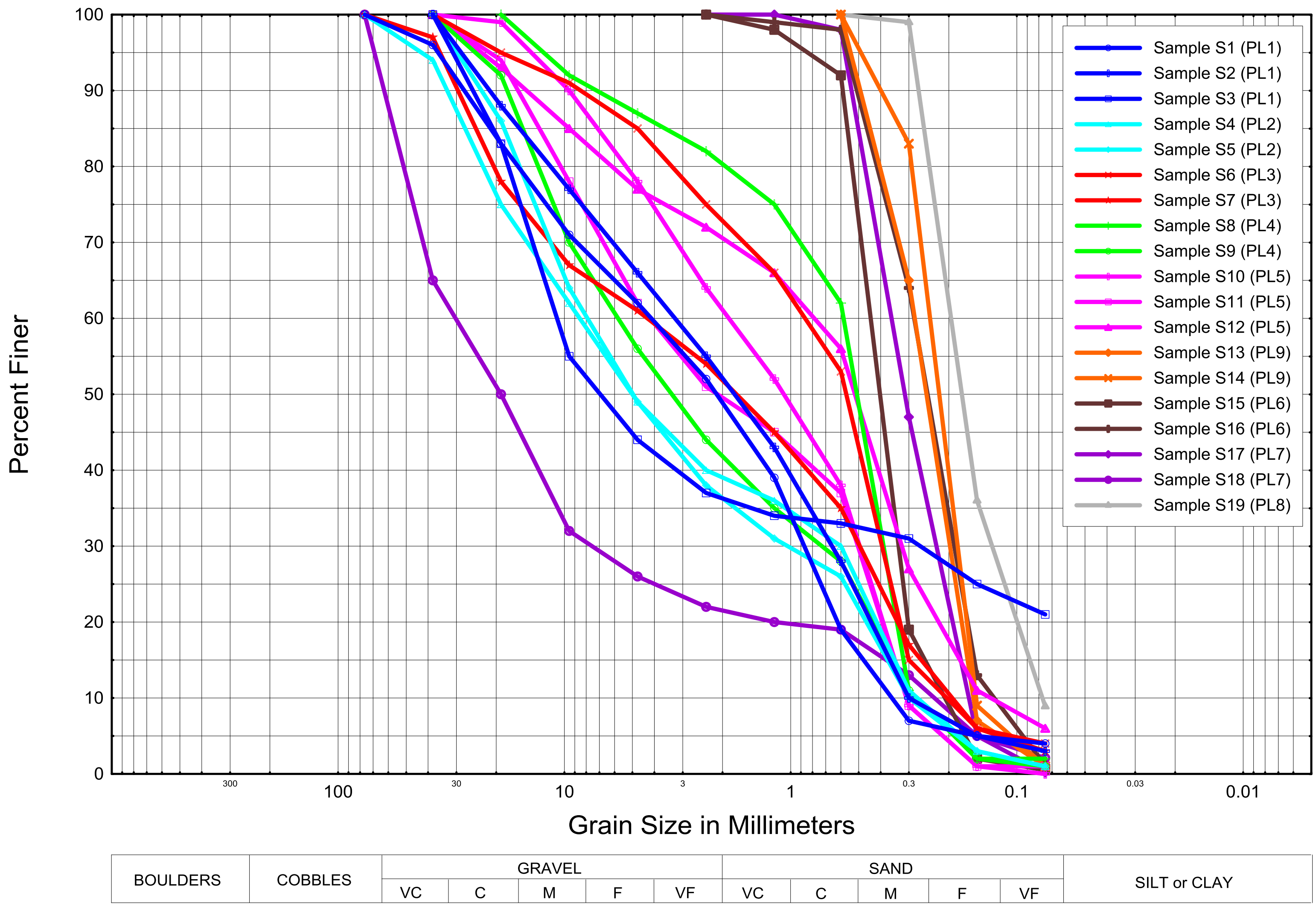

Figure 5. Gradation curves for the bulk sediment samples.

channel bed upstream from the backwater effects of the Tierra Blanca Creek fan about 2,000 feet upstream from the Garfield (New Mexico Highway 187) Bridge after first removing about 2 inches of muddy deposits, and is representative of the upstream sediment supply to the Problem Location 1 reach.

3.1.2 Problem Location 2

Problem Location 2 extends from the Salem Bridge (BB Romig Drive; New Mexico Highway 391) downstream to the confluence with Placitas Arroyo over a distance of about 3.3 miles (**Figure 6**). To update the base model of the RGCP and to develop the localized base model, the reach was extended about 1,200 feet downstream from the confluence with Placitas Arroyo with a resulting reach length of 3.8 miles based on the MEI (USACE, 2007) Station Line. In addition to the Salem Bridge, other infrastructure along the reach includes the upper Hatch Bridge (New Mexico Highway 187), and the lower Hatch Bridge (New Mexico Highway 26) is located just downstream from the problem location reach about 2,700 feet downstream from the confluence with Placitas Arroyo. Numerous vegetated islands have formed along the reach, many of which appear to have formed over the recent drought period as evidenced by the relatively young (4- to 5-year old) vegetation.

Placitas Arroyo is a very large west side tributary that enters the RGCP on the south (right) bank. This arroyo drains areas underlain by the Lower Santa Fe Group and the Tertiary-age volcanoclastic sedimentary units and delivered significant amounts of coarse material and sand during recent monsoon seasons. Although the coarse material provides the framework for the fan, the large volume of sand that overlays the coarse material results in significant backwater effects that extend over a distance of about 2,700 feet upstream from the fan. After the 2006 monsoon events and as recently as in 2013, USIBWC excavated the lower reaches of the arroyo channel and bed of the Rio Grande and the east bank of the Rio Grande was reconstructed. The relatively large amount of sand that currently resides at the mouth of the arroyo and along the fan surface indicates that this material was delivered by Placitas Arroyo relatively recently during the period since the USIBWC maintenance activities.

In addition to the west side draining Placitas Arroyo, two east side draining tributaries (Thurman I Arroyo at Sta 4524+00 and Thurman II Arroyo at Sta 4541+00) enter the RGCP on the north (left) bank. During recent monsoon season tributary flow events (i.e., 2006 and 2013), each of these tributaries delivered significant quantities of sediment to the RGCP, and appear to have delivered additional sediment since that time. After the 2006 events, USIBWC removed sediment from the river, reconstructed the opposite bank and excavated the mouth of the Thurman I Arroyo, but there does not appear to have been any work undertaken since that time. Evidence of bank protection along the right bank opposite Thurman II Arroyo suggests that similar activities were undertaken at this tributary. Islands have formed along the downstream portions of both of the Thurman Arroyo fans, along with numerous other islands and vegetated bars along the reach.

Two bulk samples were collected from the Problem Location 2 reach, including Bulk Sample S4 of the channel bed material near the Salem Bridge (New Mexico Highway 391) that represents the sediment supply to the reach (D_{50} = 5.0 mm; Figure 4) and Bulk Sample S5 (D_{50} = 5.0 mm; Figure 5) of the channel bed material near the upstream limit of the backwater effects from Placitas Arroyo. Pebble counts of the coarser material delivered by the tributaries included Pebble Count PC3 that was taken from the surface of the Thurman I Arroyo fan and Pebble Count PC4 that was taken from a riffle at the downstream fringe of the Placitas Arroyo fan.

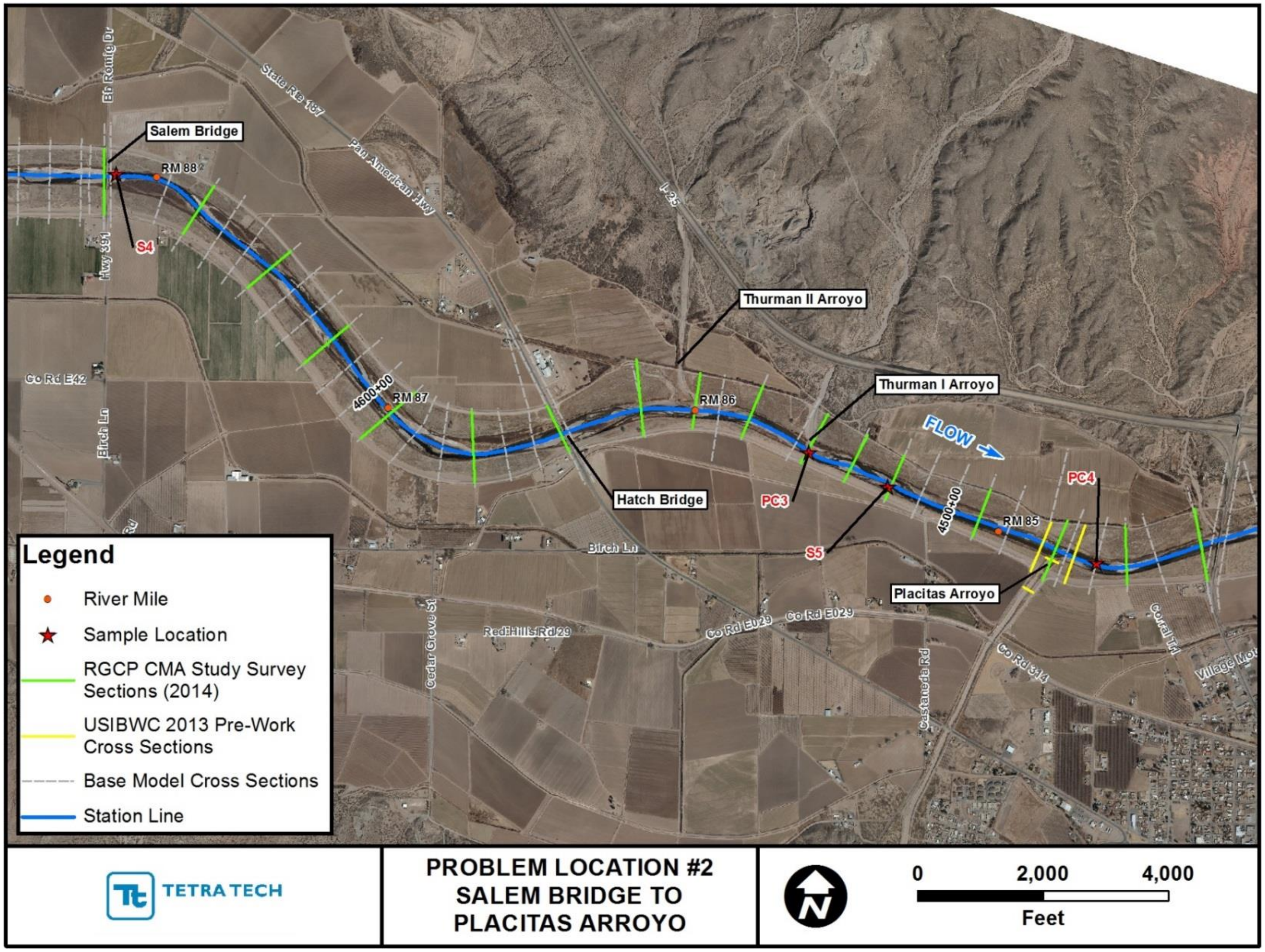

Figure 6. Aerial photograph showing key features at Problem Location 2 (Salem Bridge to Placitas Arroyo).

3.1.3 Problem Location 3

Problem Location 3 extends from the Rincon Siphon A Restoration Site downstream to Rincon Siphon over a distance of about 0.5 miles (**Figure 7**). To update the base model of the RGCP and to develop of the localized base model, the reach was extended about 1,800 feet upstream from the Rincon Siphon A Restoration Site with a resulting reach length of 0.8 miles based on the MEI (USACE, 2007) Station Line. Infrastructure along the reach includes the BNSF Railroad Bridge, the New Mexico Highway 154 Bridge, and the grade control structure at the Rincon Siphon crossing. The bridge piers for the railroad bridge are not skewed to be parallel with the direction of flow and create substantial flow blockage. The grade-control structure for the Rincon Siphon includes driven sheet pile along the downstream side of the siphon, and large riprap bed protection that extends about 200 feet downstream from the siphon crossing.

Garcia Arroyo is an east side tributary that drains the Rincon Hills. The confluence with this tributary is located about 700 feet upstream from the BNSF Railroad Bridge, and although the bulk of the fan extends about 300 feet downstream from the mouth, some fan materials are transported through the bridge structures as indicated in Bulk Sample S7 that includes about 50 percent gravel (D_{50} = 1.7 mm; Figure 5). Material sampled in Pebble Count PC5 that was collected from the surface of the fan had a D_{50} of about 50 mm (Figure 4). The upstream sediment supply to the reach that was sampled in Bulk Sample S6 (D_{50} = 0.6 mm) is somewhat finer than the supply to the more upstream reaches, likely due to the locally flatter gradient upstream from the Rincon Siphon (Figure 4).

3.1.4 Problem Location 4

Problem Location 4 extends from the confluence with Rincon Arroyo downstream to the confluence with Bignell Arroyo over a distance of about 2.9 miles (**Figure 8**). To update the base model of the RGCP and to develop of the localized base model, the reach was extended about 1,500 feet upstream from Rincon Arroyo to the New Mexico Highway 140 Bridge and about 2,000 feet downstream from the confluence with Bignell Arroyo, resulting in a reach length of 3.5 miles based on the MEI (USACE, 2007) Station Line. Other than the New Mexico Highway 140 Bridge at the upstream limit of the reach, no infrastructure is present along the reach. Numerous islands and vegetated alternating bars have formed along the reach.

Rincon Arroyo is an east side tributary that drains the Rincon Hills with its mouth located about 1,500 feet downstream from the New Mexico Highway 140 Bridge. After the 2006 monsoon season, USIBWC graded the arroyo bed and fan surface, and relocated some of the very large material in the fan along the toe of the west bank to counteract the bank erosion that has occurred over a distance of about 500 feet, and appears to be effective toe protection. Additional material was excavated in 2013 to remove sediment that has been delivered by the arroyo since that time. Remnant material in the downstream portion of the fan is very coarse. Pebble Count PC6 was conducted along the distal portion of the fan from material that represents the bi-modal (sand and gravel) fractions of the fan sediments, and has a D_{50} of about 50 mm (Figure 4). The coarsest fractions of the fan materials was measured using the boulder count method by measuring the intermediate axis of 100 randomly selected boulders from a location that was downstream from mechanical activities, and indicates the boulders have a median diameter of about 310 mm (Figure 4).

At Reed Arroyo and Bignell Arroyo the coarse tributary fans have displaced the Rio Grande towards the east and the RGCP levee. The material along the surface of both fans has a D_{50} of about 60 mm (Pebble Counts PC7 and PC8). Relatively thick vegetation has colonized the interior portions of both fans. Downstream from the Reed Arroyo fan, the bed material becomes significantly finer as indicated by Sample S9 (D_{50} = 3.3 mm; Figure 5). The gradation of the

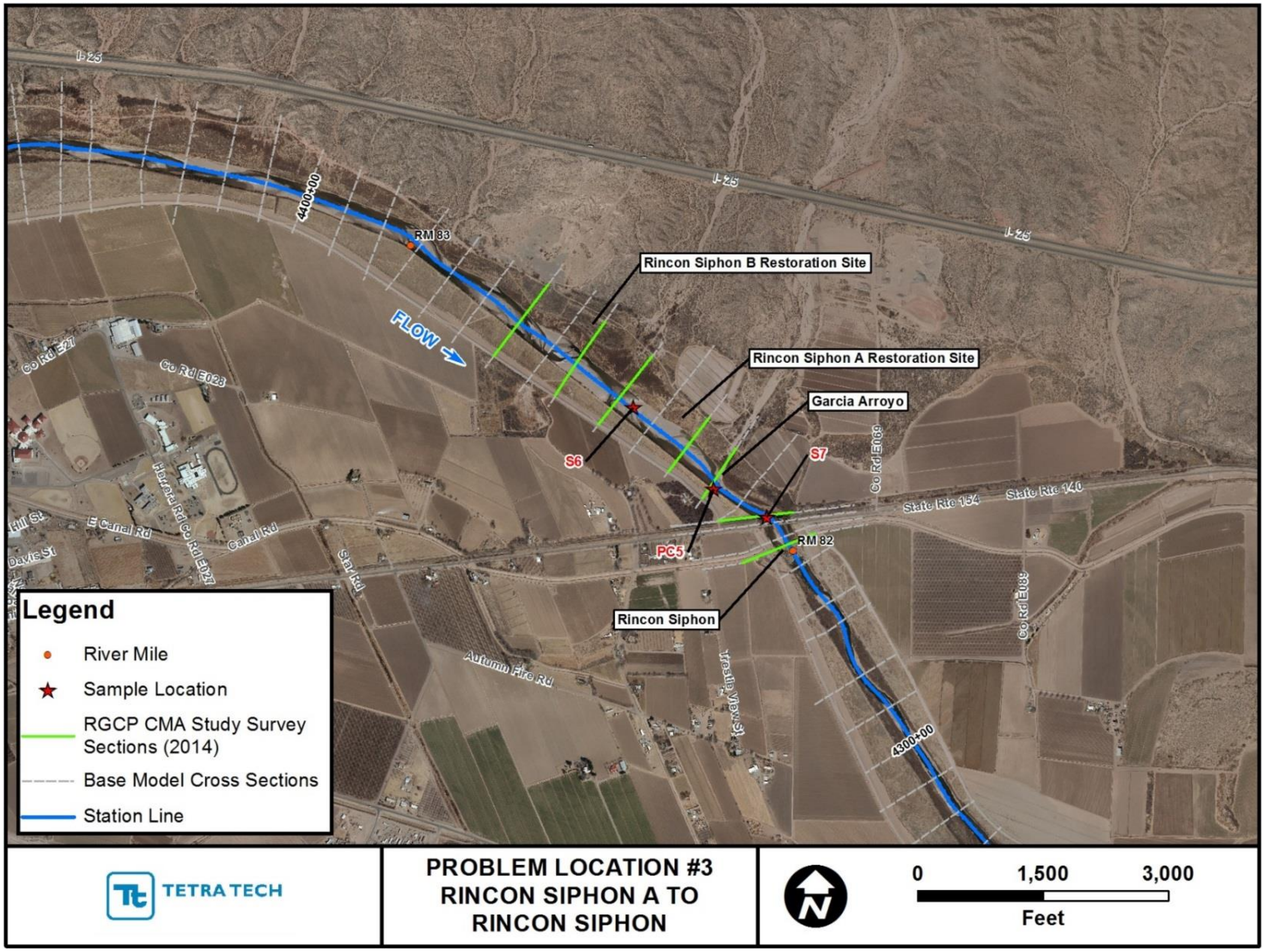

Figure 7. Aerial photograph showing key features at Problem Location 3 (Rincon Siphon A Restoration Site to Rincon Siphon).

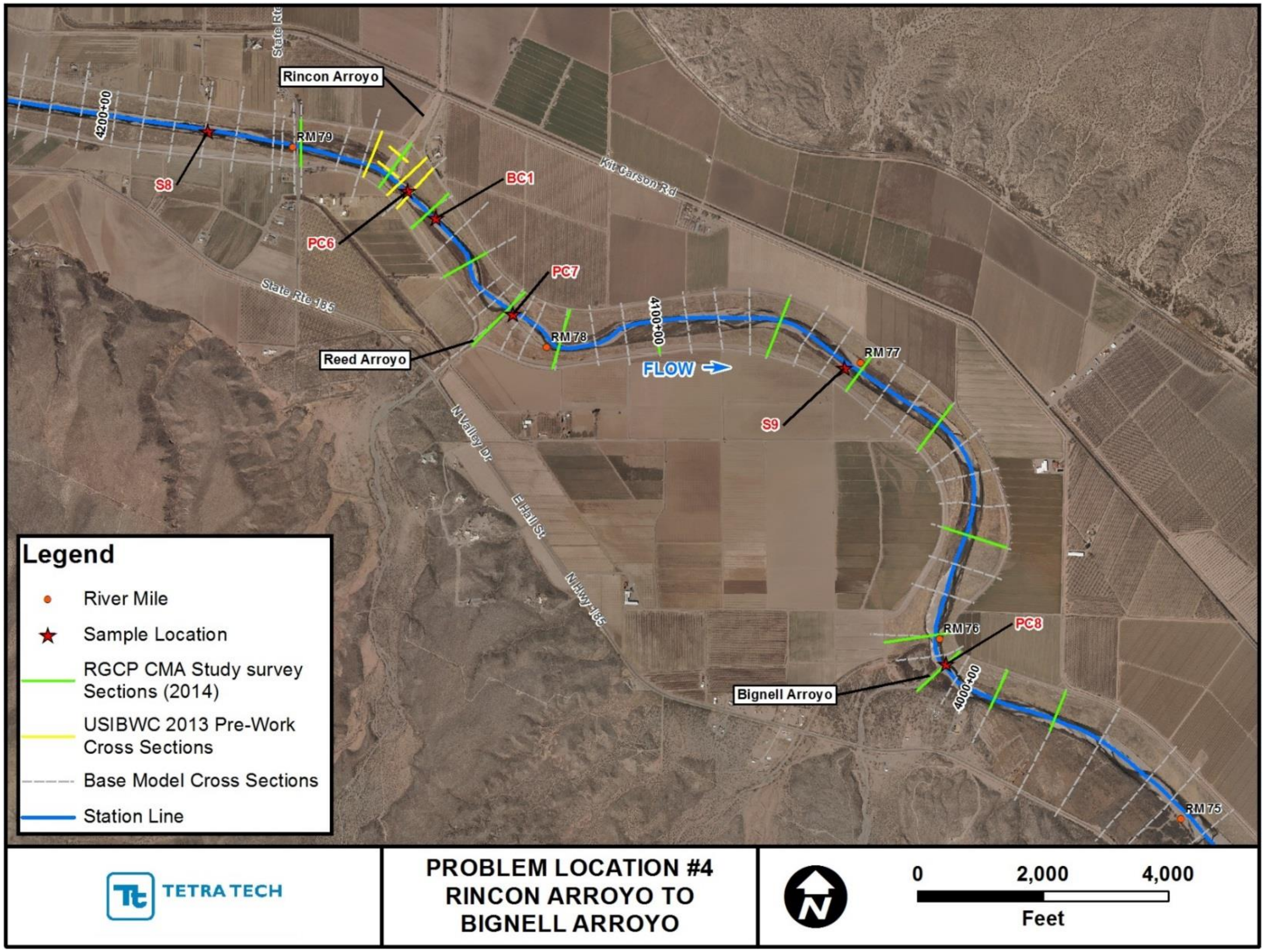

Figure 8. Aerial photograph showing key features at Problem Location 4 (Rincon Arroyo to Bignell Arroyo).

upstream sediment supply was estimated by collecting Sample S8 (D_{50} = 0.5 mm) from the channel bed about 1,500 feet upstream from the New Mexico Highway 140 Bridge near the upstream limit of the backwater zone upstream from Rincon Arroyo (Figure 5).

3.1.5 Problem Location 5

Problem Location 5 extends from the confluence with Rock Canyon downstream to a location that is about 0.8 miles below the outlet of the Tonuco/Rincon Drain over a distance of about 1.9 miles (**Figure 9**). To update the base model of the RGCP and to develop the localized base model (and as requested by USIBWC), the reach was extended about 3,000 feet upstream from the confluence with Rock Canyon and about 1.3 miles downstream from the Tonuco/Rincon Drain outlet, resulting in a reach length of 2.9 miles based on the MEI (USACE, 2007) Station Line. No infrastructure is present along the reach.

The upstream sediment supply to the reach may be affected by locally lower channel gradients that are associated with the high-flow backwater effects from Seldon Canyon. The bed material in the upstream portion of the reach that was collected in Sample S10 (D_{50} = 1.1 mm; Figure 5) is sand and gravel that likely includes gravel material delivered by the upstream tributaries such as Hersey Arroyo.

Rock Canyon Arroyo is a west side tributary to the Rio Grande that delivered large volumes of coarse sediment to the river in 2006, and has continued to do so over the past 8 years. Ongoing prograding of the fan has resulted in at least 25 feet of lateral bank erosion on the opposite east bank over a longitudinal distance of about 200 feet, resulting in significant loss to private property. Material on the Rock Canyon fan surface has a D_{50} of about 70 mm (Pebble Count PC9). The height of the fan ranges from between 4 and 5 feet, and the surface has become vegetated and appears relatively stable. Downstream from Rock Canyon near Sta 3750+00, the channel widens somewhat and the frequency of vegetated islands increases. Many of the islands have probably formed during the recent drought period as indicated by the relatively young vegetation (4 to 5 years). Although the expansion at Sta 3750+00 results in reduced sediment-transport capacities, there is still sufficient energy to transport gravel materials to the area in the vicinity of the Rincon/Tonuco Drain, as evidenced by the more than 50-percent gravel in Sample S11 (D_{50} = 2.1 mm; Figure 5).

The Rincon/Tonuco Drain is located along the east bank of the RGCP opposite the Horse Creek Canyon fan. The drain is currently blocked by a 3.5-foot high beaver dam that significantly reduces drain efficiency. Downstream from the drain, the east bank is protected by riprap along the railroad embankment that extends over a longitudinal distance of at least 1,000 feet. No discernable alluvial fan was identified at the mouth of Horse Canyon Creek, where the tributary is perched about 2 feet above the bar surface along the bed of the Rio Grande. Although this tributary does not appear to have delivered significant amounts of material to the river over the recent past, the bed material represented by Sample S12 is sandy gravel (D_{50} = 0.5 mm; Figure 5).

3.1.6 Problem Location 6

Problem Location 6 extends from just upstream from the Placitas Drain to about 3,500 feet below Mesilla Dam over a distance of about 2.4 miles (**Figure 10**). In addition to the Picacho Drain that enters along the right (west) bank, the drain for the California Lateral also enters near the upstream limit of the project reach along the left (east) bank. At the mouth of the Picacho Drain, the drain invert is perched about 2 feet above the river bed, indicating that the river has recently incised or aggradation has occurred along the downstream reaches of the drain. Considering that the California Lateral drain invert is not perched, the latter explanation appears to be the case.

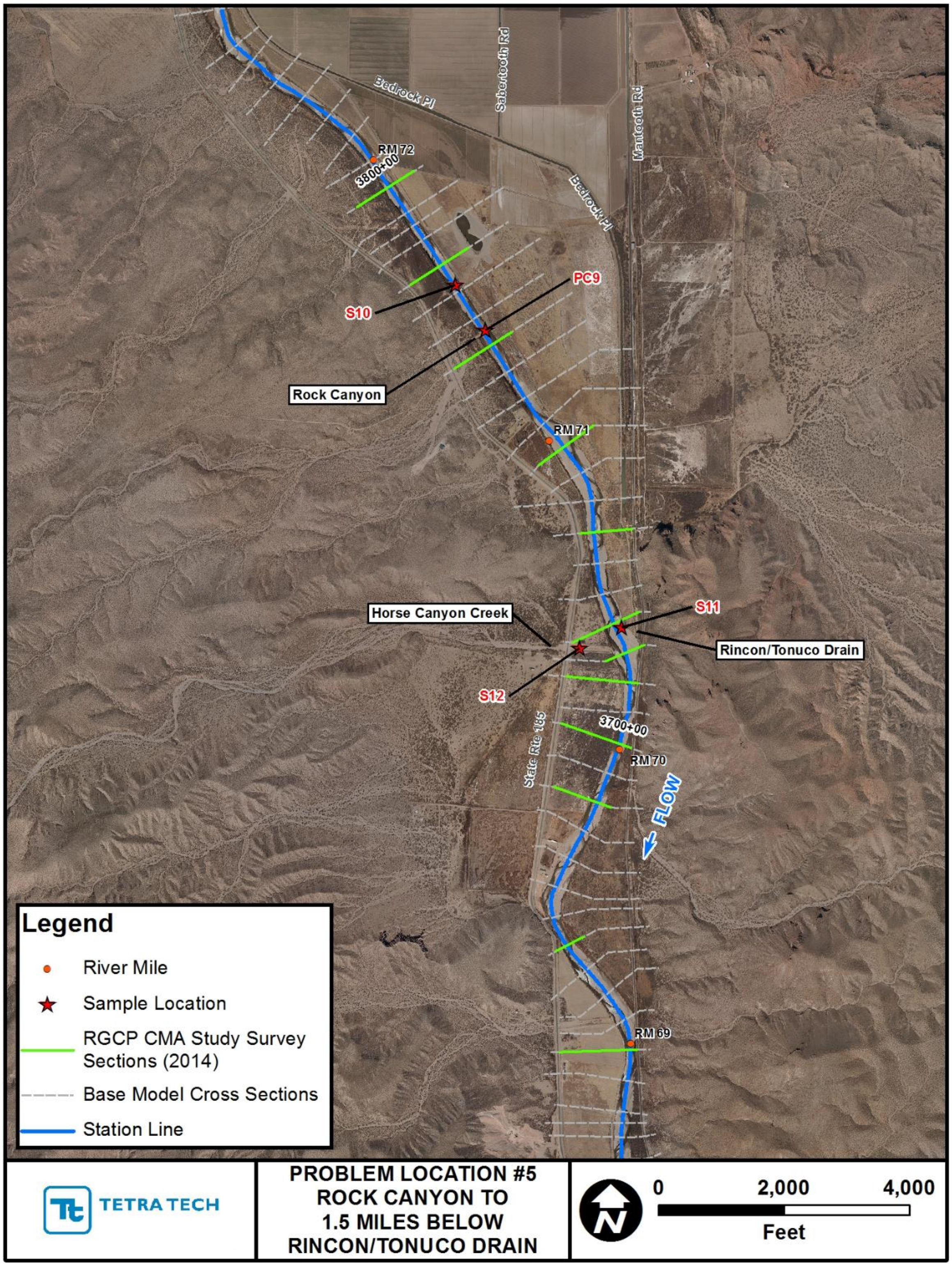

Figure 9. Aerial photograph showing key features at Problem Location 5 (Rock Canyon to below Rincon/Tonuco Drain).

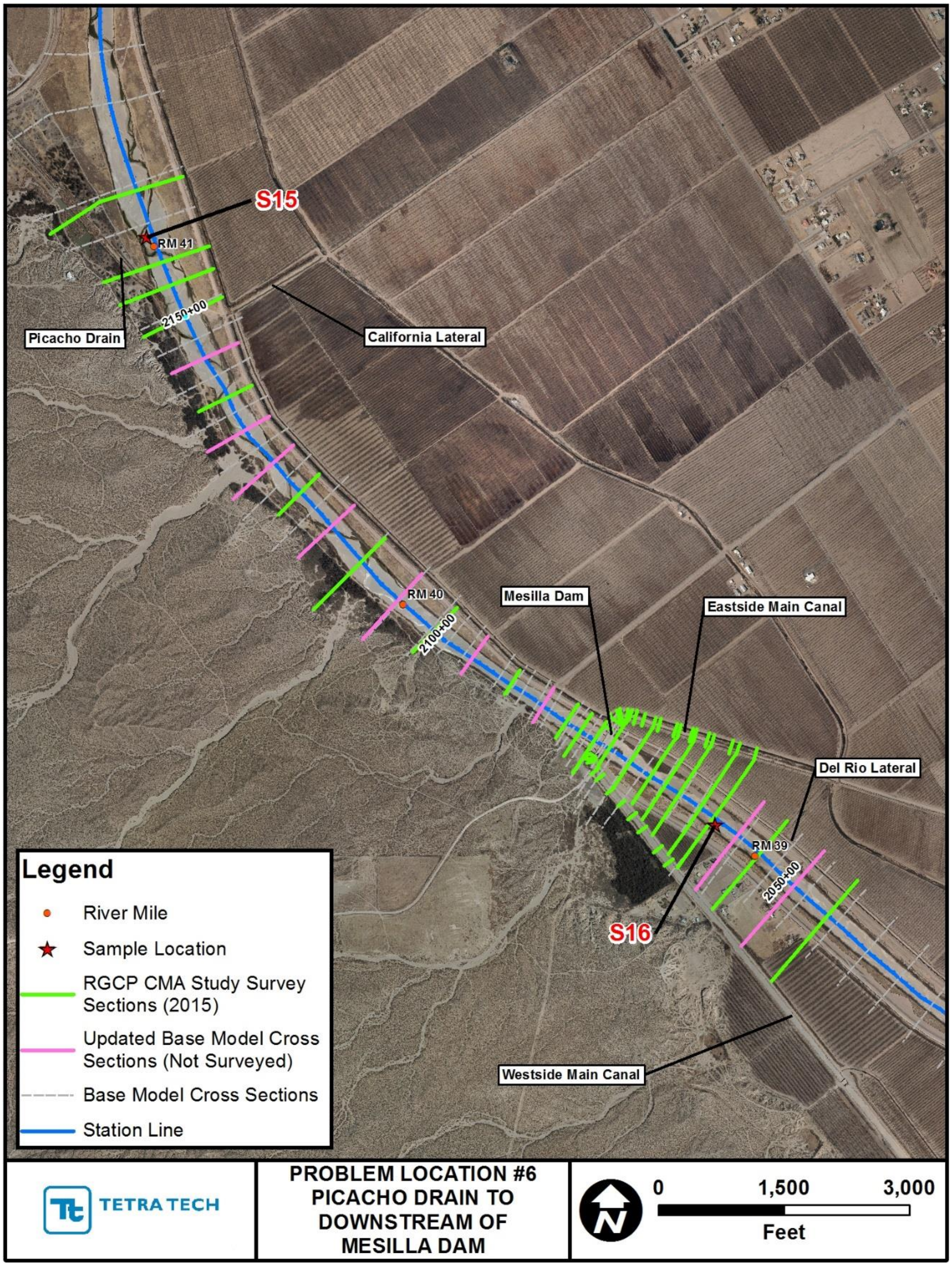

Figure 10. Aerial photograph showing key features at Problem Location 6 (Picacho Drain to below Mesilla Dam).

The infrastructure at Mesilla Dam includes a single gate that delivers flow to the Del Rio Lateral, the three bays with two gates each at the headworks for the Eastside Main Canal, the thirteen radial gates at the dam that deliver flows to the downstream river, and the four bays with two gates each at the headworks for the Westside Main Canal. The metering stations for the Eastside and Westside main canals are located a short distance downstream from the headgates (less than 900 feet based on the canal station lines), and the river gage ("River Below Mesilla Dam") is located about 2 miles downstream from the dam.

Sample S15, located near the upstream limit of the site and representative of the upstream sediment supply to the reach, is primarily sand with D_{50} = 0.40 mm (Figure 5). Trace gravels are present in the bed material. Numerous relatively small arroyos and one larger arroyo drain the non-leveed west bank upstream from the dam and deliver sand and gravel to the reach. The backwater zone upstream from Mesilla Dam does not extend over a distance of more than 2,000 feet, at least under recent flow and operating conditions, and has resulted in the deposition of significant amounts of material. The surface depositional material appears to progressively fine in the downstream direction in the backwater zone above the dam. Based on Sample S16 (D_{50} = 0.25 mm; Figure 5) that was collected downstream from the dam, the sediment-trapping effects of the dam results in some minor fining of the bed material below the dam. Upstream from the dam, the banks are at least intermittently protected by riprap over significant distances, and in many cases several feet of bank accretion has buried the riprap. Except for the relatively large, vegetated mid-channel bar immediately downstream from the dam, the reach is mostly void of vegetated bar surfaces.

3.1.7 Problem Location 7

Problem Location 7 extends about 1.8 miles from just upstream from the mouth of the East Drain to about 5,000 feet below the Vinton Bridge (**Figure 11**). The only infrastructure along this reach is the Vinton Bridge, although a metering station is located about 2,000 feet from the bridge. The mouth of the East Drain has become overgrown with cattails where backwater from the RGCP extends up the drain to the control structure. The invert elevation of the drain is consistent with the elevation of the river bed, so any aggradation along the river at this location has resulted in similar depths of deposition in the drain. At the drain-control structure, about 8 inches of deposition has occurred based on the difference between the invert of the control structure and the bed of the drain immediately downstream from the control structure. Progressively lower drain bank heights in the downstream direction indicate that the degree of deposition also increases in the downstream direction.

Sample S17, collected from the bed of the channel near the upstream limit of the problem location reach, indicates the bed-material sediment supply is primarily sand with a median grain size of 0.3 mm (Figure 5). The widest main channel sections occur along the approximately 2,300-foot-long reach at and downstream from the East Drain and along the approximately 1,300-foot-long reach below Vinton Bridge. A few low-elevation, grassy mid-channel bars have formed below the expansion zones in the wider reaches. Four relatively small arroyos enter the reach from the left bank along the reach. Each of the arroyos has been channelized to various degrees at some point along their course and have subsequently incised. The arroyos drain the eastern lying Franklin Mountains that deliver mostly fine-grained materials, although the sediment-loading includes some gravels. Sample S18, collected from the subsurface of the fan of the arroyo at Sta 786+50, is representative of the materials delivered by each of the arroyos and has a median grain size of 19.1 mm (Figure 5). Although the material supplied by the arroyos is significantly coarser than the dominant sediment supply, the volume of material does not appear to be significant as evidenced by the relatively localized sediment deposits in the fans. Surface gravels persist for less than 200

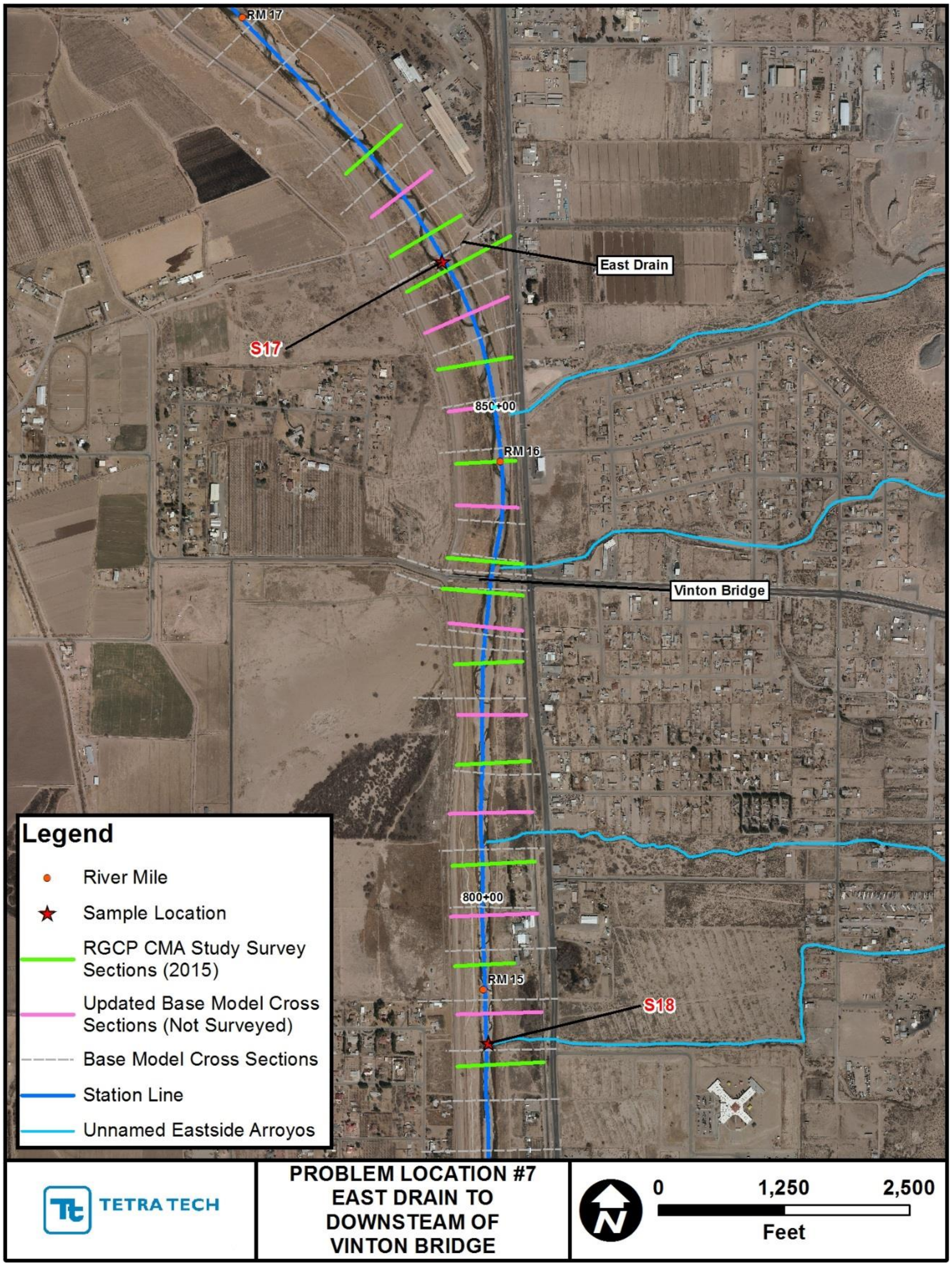

Figure 11. Aerial photograph showing key features at Problem Location 7 (East Drain to below Vinton Bridge).

feet downstream from the mouths of the arroyos at Sta 786+50 and Sta 834+00, while there was no evidence of recent loading from the arroyos at Sta 806+50 and Sta 849+50.

3.1.8 Problem Location 8

Problem Location 8 extends from about 4,200 feet upstream from the Country Club Bridge to just downstream from the Nemexas Siphon crossing over a distance of about 1.5 miles (**Figure 12**). Country Club Bridge is skewed to the main channel by about 20 degrees. Both the bridge abutments and bridge piers are also skewed by this amount; however, because each of the six pier sets are made up of ten 16-inch-wide columns, the effective pier width is about 13.3 feet. Vegetated islands are forming upstream from Pier Sets Nos. 2, 3 and 4 (Photo B.19). Some minor bank erosion along the left (west) bank has occurred under the bridge deck as a result of flows being defected away from debris that collects at the face of Pier Set No. 2. Other than this localized bank erosion, the banks appear to be stable along the remaining extent of the reach. Vegetation along the banks varies along the reach, with intermittent grass and light shrubs along the upstream portion of the reach and more dense, older woody vegetation near the Nemexas Siphon, especially along the right (east) bank.

The bed material is almost entirely sand and does not vary along the project reach. Sample S19 was collected from the channel bed near the upstream limit of the project and has a median grain size of 0.17 mm (Figure 5). The main channel top width is about 210 feet along the upstream 2,000 feet of the reach, decreasing to about 180 feet above the bridge and further decreasing to about 170 feet over the downstream 3,000 feet of the reach, with a locally wider reach below the bridge where the main channel top width increases to as much as 250 feet. The width of the floodplain (i.e., the width between the levees) generally increases in the downstream direct from about 470 feet in the upper reach to more than 600 feet in the lower portion of the reach, although the floodplain is locally narrower (~500 feet) in the vicinity of the siphon. Riprap bank protection was identified along the left (east) main channel bank at the siphon crossing, but it was not possible to determine if other areas are protected because of the bank vegetation and unknown degrees of bank accretion. Bank heights measured from the thalweg vary from 5 to 6 feet.

3.1.9 Problem Location 9

Problem Location 9 extends from the Montoya Drain outlet downstream to American Dam over a distance of about 2.2 miles (**Figure 13**). To update the base model of the RGCP and to develop the localized base model, the reach was extended about 3,000 feet upstream to Anapra Bridge (Racetrack Drive), resulting in a reach length of 2.7 miles based on the MEI (USACE, 2007) Station Line. Infrastructure along the reach includes Anapra Bridge, Courchesne Bridge (McNutt Road), the two Southern Pacific Railroad Bridges above American Dam and American Dam. The USIBWC Rio Grande at El Paso Gage (also referred to as the Courchesne Gage; USIBWC Gage No. 08-3640.00) is located just upstream from the Courchesne Bridge, and includes the automated gage house and gaging cable.

The upstream sediment supply to the reach, represented by Sample S13, is much finer than the sediment supply to the upstream problem areas (Problem Locations 1 through 5) and is sand material (D_{50} = 0.3 mm; Figure 5). This material appears to be consistent along the entire reach and is not significantly affected by materials delivered by the Montoya Drain, as evidenced by Sample S14 (D_{50} = 0.2 mm; Figure 5). A number of large, vegetated islands and alternate bars are present along the reach. Based on the relatively young vegetation that is present along the up- and downstream sides of the larger and older islands, the size of many of the islands has likely increased over the past 5 years. Of particular concern is the very large island that has formed upstream from the outlet of the Montoya Drain. USIBWC has expressed interest in formulating a plan for removal of this island (pers. comm., Derrick O'Hara, USIBWC). Aggradation

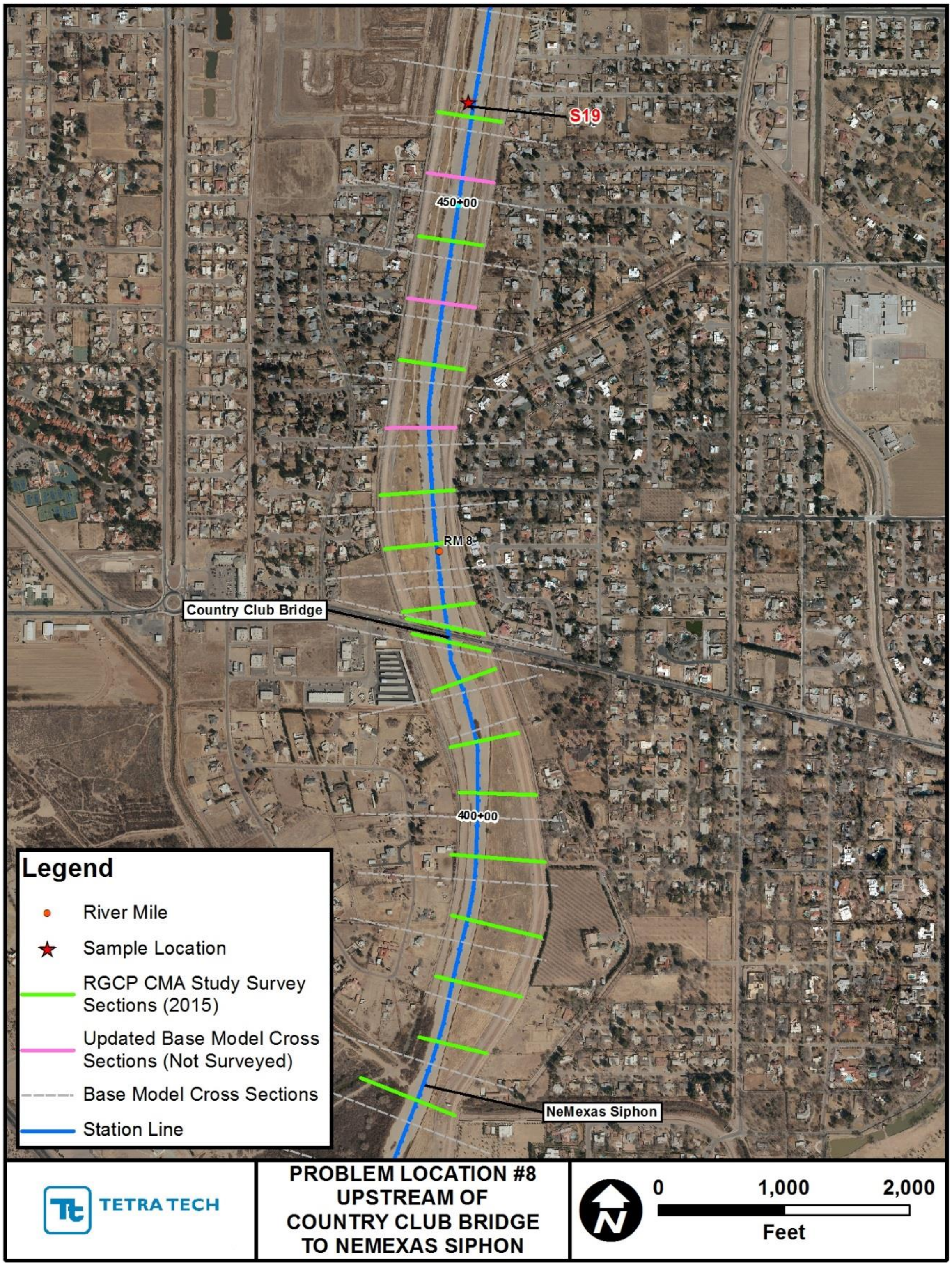

Figure 12. Aerial photograph showing key features at Problem Location 8 (Above Country Club Bridge to Nemexas Siphon).

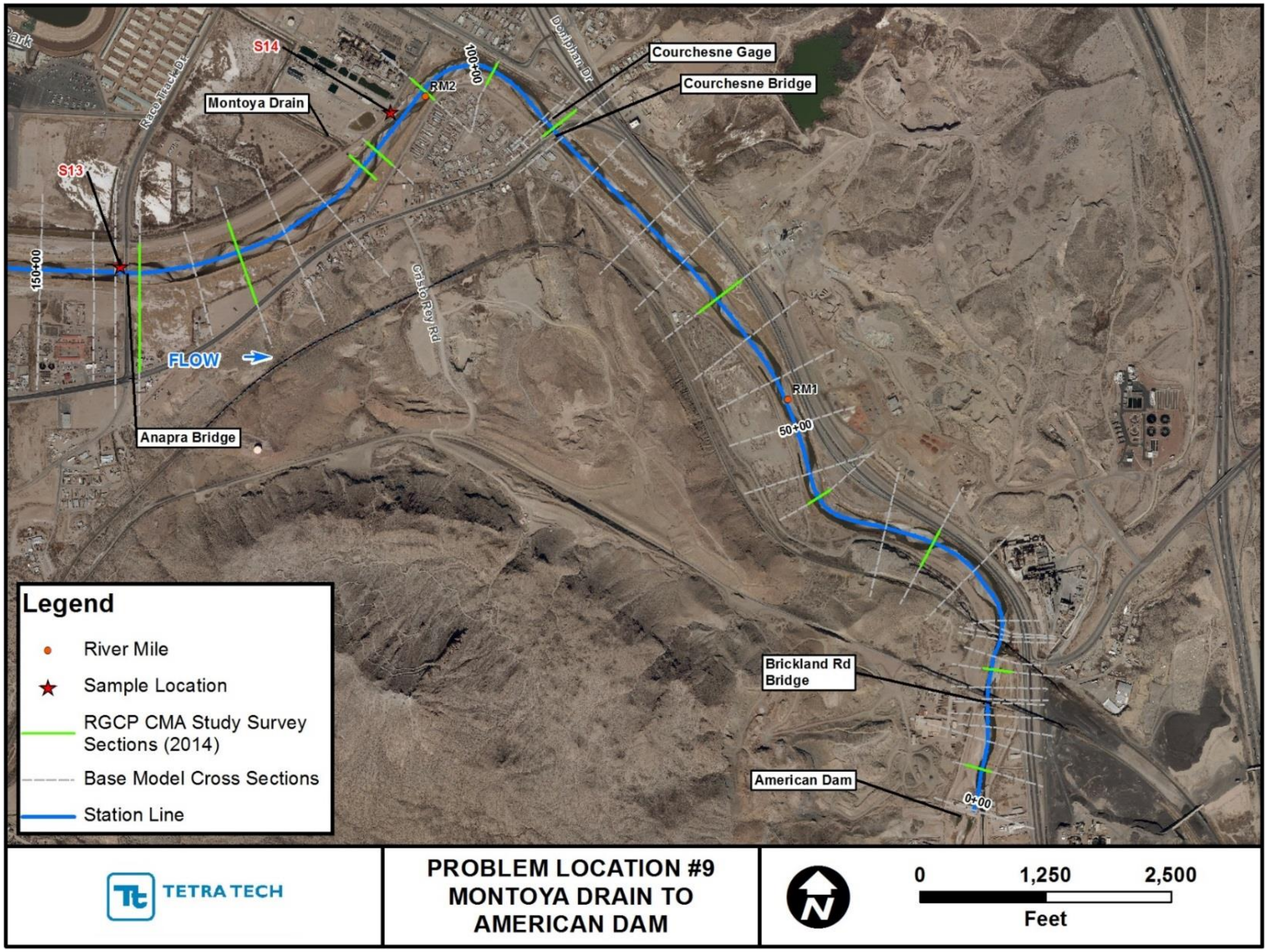

Figure 13. Aerial photograph showing key features at Problem Location 9 (Montoya Drain to American Dam).

along the reach appears widespread, and has likely resulted in reduced efficiency of the Montoya Drain. Riprap bank protection lines both banks along the entirety of the reach, although in many locations the riprap has become buried as a result of bank accretion and subsequently overgrown with vegetation. Backwater effects from American Dam extend at least 1,200 feet upstream from the dam during low-flow conditions, but probably extend much farther during high-flow periods.

3.2 Site Surveys at Problem Locations 1 through 5 and 9

Topographic and bathymetric surveys Problem Locations 1 through 5 and 9 were carried out in October 2014 to define the current geometry of the main channel and to evaluate the degree of aggradation or degradation that has occurred since the time that the 2007 base model (based primarily on cross sections that were surveyed in 2004) of the RGCP was developed. The surveys were conducted in two general phases.

The first phase involved monumentation (rebar and cap) and surveying of survey control points and cross section endpoints at Problem Locations 1 through 5 and 9. Prior to this surveying effort, Tetra Tech laid out the cross section alignments at these problem locations based on the following considerations:

1. Re-survey the 2013 pre-work cross sections, where available. The 2013 pre-work cross sections were surveyed by USIBWC and represent conditions prior to mechanical excavation of materials from the RGCP in the vicinity of Placitas and Rincon arroyos.
2. Re-survey representative sections surveyed by EBID in 2007 in the vicinity of Green/Tierra Blanca, Sibley and Reed Arroyos that are currently in the 2007 base model.
3. Re-survey representative sections that were surveyed by Tetra Tech in 2004 and are currently in the 2007 base model.
4. Locate cross sections in the Rio Grande at the mouth of the arroyos.
5. Locate cross sections at the hydraulic or geomorphic controls.

After the cross section alignments were submitted to USIBWC for review, the general location of the survey control and the precise alignment of the cross sections were provided to Del Sur Surveyors, LLC (DSS), who are licensed surveyors in the State of New Mexico and were subcontracted to conduct this phase of the surveying. DSS then set and surveyed the survey control points and endpoints, and provided the survey data to Tetra Tech for use in the second phase of the surveying. **Appendix C** includes the DSS Surveyor's Report that summarizes the methods, instrumentation, and other details of their surveys.

The second phase of the surveying involved collection of the topographic and bathymetric survey data that was used to define the channel geometry at Problem Locations 1 through 5 and 9. Tetra Tech conducted these surveys under the oversight of the licensed DSS surveyors using a Leica Viva-series Real Time Kinetic (RTK) survey-grade GPS system, with a horizontal and vertical positioning accuracy of approximately 0.05 feet. Tetra Tech collected a total of 3,395 survey data points at the 84 cross sections (including 4 additional cross-section resurveys within Placitas and Rincon Arroyos) along the reaches of Problem Locations 1 through 5 and 9 (**Table 4**). The survey data are presented in the DSS Surveyor's Report (Appendix C), and included in spreadsheet format on the digital data disc (**Appendix M**). Aerial imagery showing the location of the monumented survey control points, monumented cross-section endpoints, and the topographic/ bathymetric survey data are presented in **Appendix D**.

Table 4. Summary of the cross sections surveyed by Tetra Tech/Del Sur Surveying at Problem Locations 1 through 5 and 9.

Problem Location	Identification	Representation	Number of Survey Sections	D/S Cross Section	U/S Cross Section	Actual Length (miles)	Average Survey Section Spacing (ft)	Number of Sections in Base Model	Number of 2013 Pre-Work Sections
1	Tierra Blanca Creek to Sibley Arroyo	Vortex Weir	12	516800.1[a] 516402.8[b] 511526.8[c]	528852.2[a,b] 536423.8[c]	2.28[a] 2.36[b] 4.72[c]	1100	36	0
2	Salem Bridge to Placitas Arroyo	Arroyos and Islands	20	445906.5[a] 445728.0[b] 443140.0[c]	465884.8[a,b,c]	3.78[a] 3.82[b] 4.31[c]	1050	36	4
3	Rincon Siphon A Restoration Site to Rincon Siphon	Restoration Sites and Siphon	7	432920.8[a] 430594.6[b] 423584.0[c]	437135.2[a,b] 444895.1[c]	0.80[a] 1.24[b] 3.64[c]	700	12	0
4	Rincon Arroyo to Bignell Arroyo	Arroyos and Islands	19	398658[a,b] 395630.2[c]	416941.9[a,b] 419954.8[c]	3.46[a,b] 4.03[c]	1020	37	5
5	Rock Canyon to 0.8 mi below Rincon/Tonuco Drain Confluence	Drain and Mouth of Seldon Canyon	12	364349.5[a] 363853.0[b] 359912.1[c]	379830.9[a,b] 381328.9[c]	2.93[a] 3.03[b,c]	1410	33	0
9	Montoya Drain to American Dam	Drain	13	0.5[a,b,c]	13994.19[a,b] 18475.3[c]	2.65[a,b] 3.50[c]	1170	25	0

[a]Survey sections and lengths
[b]Localized steady-state hydraulic model sections and lengths
[c]Localized sediment-transport model sections and lengths

Table 5. Summary of the cross sections surveyed by USIBWC at Problem Locations 6 through 8.

Problem Location	Identification	Representation	Number of Survey Sections	Downstream Cross Section	Upstream Cross Section	Actual Length (miles)	Average Survey Section Spacing (ft)	Number of Sections in Updated Base Model	Number of 2013 Pre-Work Sections
6	Picacho Drain to below Mesilla Dam	Drain, Canals and Dam	51*	204340.4[a,b,c]	216618.6[a,b,c]	2.33[a,b,c]	610**	60***	0
7	East Drain to below Vinton Bridge	Drain and Arroyo	12	78425.3[a,b] 71396.7[c]	87862.2[a,b] 98397.8[c]	1.79[a,b] 5.11[c]	860	21	0
8	Upstream of Country Club Bridge to NeMexas Siphon	No Inputs, Bridge, Populated Area, Levee Encroachments	16	37744.7[a,b] 34422.0[c]	45715.9[a,b] 49071.1[c]	1.51[a,b,c]	530	19	0

*Includes 21 sections in RGCP, 19 sections in Eastside Main Canal and 11 sections in Westside Main Canal.
**Average survey section spacing in RGCP (does not include sections surveyed in the Eastside and Westside Main Canals).
***Includes 30 sections in RGCP, 19 sections in Eastside Main Canal and 11 sections in Westside Main Canal.
[a]Survey sections and lengths.
[b]Localized steady-state hydraulic model sections and lengths.
[c]Localized sediment-transport model sections and lengths.

3.3 Site Surveys at Problem Locations 6 through 8

USIBWC conducted the cross section surveys at Problem Locations 6 through 8 during the period from December 2014 to February 2015. A summary of the USIBWC cross section surveys is presented in **Table 5**. These surveys were also conducted using an RTK survey-grade GPS system, and included a total of 79 cross sections, 49 of which were surveyed along the main channel of the RGCP, 19 of which were surveyed in the Eastside Main Canal and 11 of which were surveyed in the Westside Main Canal. It is not known how many control points were established or used as part of the USIBWC surveys, but the topographic data included 1,440 survey points at Problem Location 6, 469 survey points at Problem Location 7, and 534 survey points at Problem Location 8, resulting in a total of 2,443 survey points. The survey data are included in spreadsheet format on the digital data disc (Appendix M) and aerial imagery showing the location of the topographic/bathymetric survey data are presented in Appendix D.

3.4 Survey Data Reduction and Cross-section Development

Once the surveys were complete, Tetra Tech post-processed the survey data to generate the cross-sectional geometry that was used as input to the updated base model of the RGCP. The raw survey data was first reviewed to ensure the horizontal and vertical accuracy was within reasonable tolerance levels (0.1 feet in the horizontal and 0.05 feet in the vertical). The cross sections were developed by transposing the individual topographic/bathymetric survey data points onto a planview line drawn between the cross-section endpoints and then developing the station versus elevation data based on the cross-section station relative to the left endpoint of the cross section in the base hydraulic model (in most cases, the extents of the model cross sections extend beyond the limits of the cross-section surveys). It should be noted that many of the cross sections that were surveyed by USIBWC were not surveyed perpendicular to the direction of flow (see Appendix D). At these cross sections, the planform line was laid out perpendicular to the main channel and overbank flow paths, and the surveyed points were then transposed onto the lines. The maximum distance that the points were translated onto the cross-section line was 73 feet, and the longest distances for translation occurred in the overbank areas, so the resulting station-elevation data are believed to be an accurate representation of the true cross-sectional geometry. The resulting cross-section plots and station-elevation data are provided in **Appendix E** and provided in digital format in Appendix M.

3.5 Pre-Work and Post-Work Cross-section Comparisons

At two of the problem locations, the USIBWC performed sediment removal activities following the 2013 monsoon events that delivered significant quantities of sediment to the RGCP. These activities were performed at the Placitas Arroyo (Problem Location 2) and Rincon Arroyo (Problem Location 4) tributary mouths. Prior to the sediment removal, the USIBWC surveyed “Pre-Work” cross sections within the main channel of the RGCP and in the arroyos at the mouths of Placitas and Rincon Arroyos. At Placitas Arroyo, two cross sections were surveyed in the main channel and two cross sections were surveyed in the arroyo (Figure 6), and at Rincon Arroyo, three cross sections were surveyed in the main channel and two cross sections were surveyed in the arroyo (Figure 8). The cross section survey data were provided to Tetra Tech, and the sections were re-surveyed during the 2014 surveys (referred to as “Post-Work” cross sections) to assess the effects of the excavations.

A comparison of the Pre-Work and Post-Work cross sections in the vicinity of Placitas Arroyo indicates that, despite the sediment removal, aggradation along the fan surface has continued with about 1 foot of deposition (Cross Section 448023.2; **Figure 14**). The aggradation has resulted in increased backwater effects and associated deposition upstream from the fan (Cross Section 448572.3; **Figure 15**), albeit not as significant as that which has occurred along the fan surface.

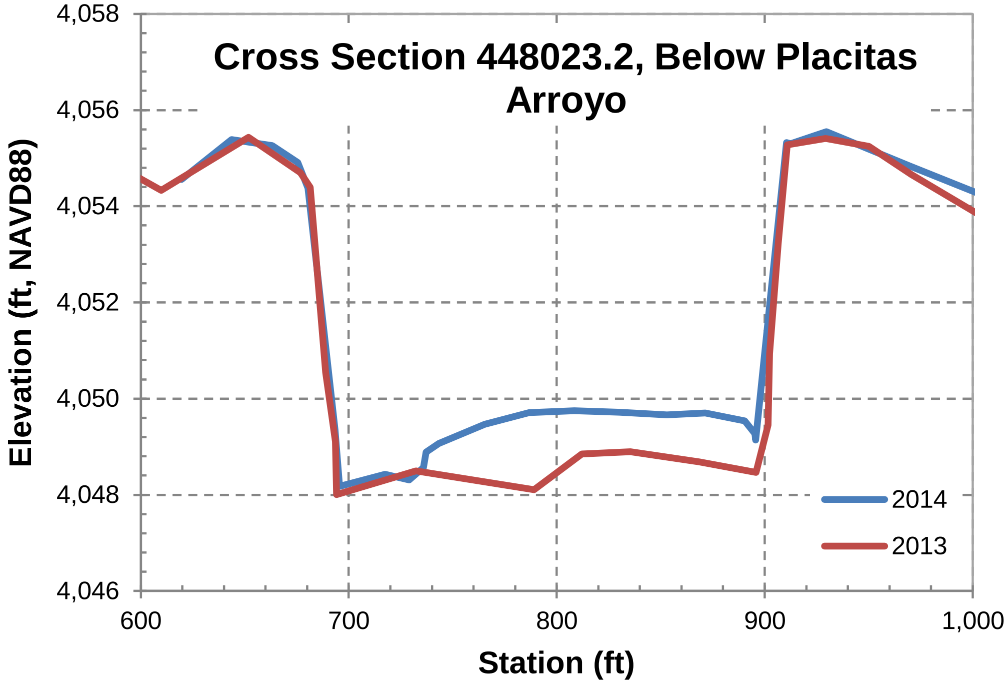

Figure 14. Comparison of 2013 (Pre-Work) and 2014 (Post-Work) cross sections at Section 448023.2 located just downstream from the confluence with Placitas Arroyo.

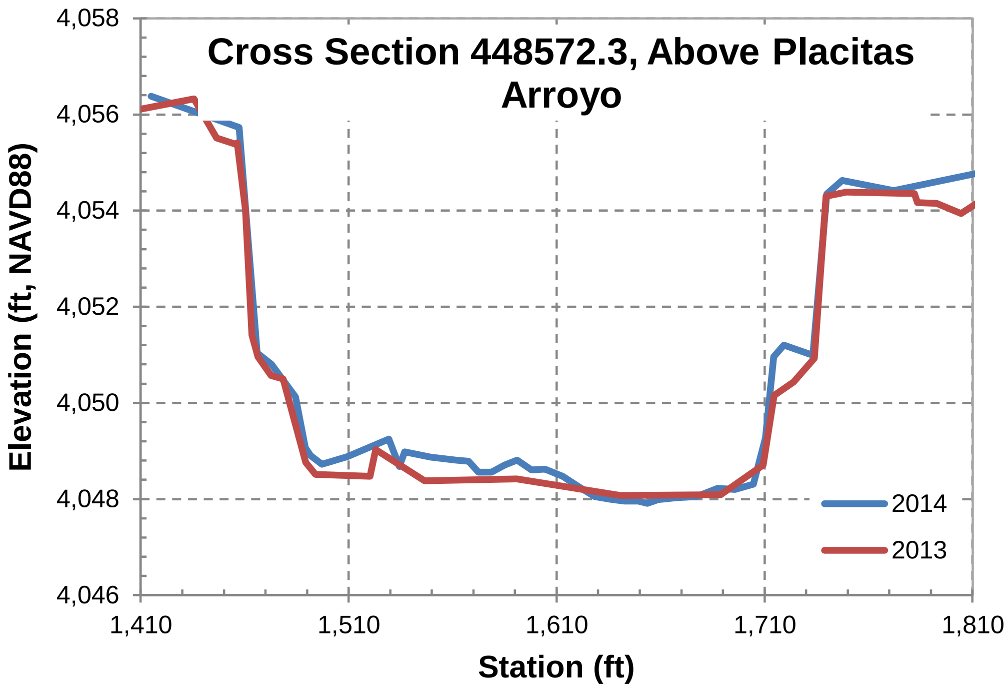

Figure 15. Comparison of 2013 (Pre-Work) and 2014 (Post-Work) cross sections at Section 448572.3 located just upstream from the confluence with Placitas Arroyo.

The cross-section comparison plots within Placitas Arroyo indicate that very little change occurred between the Pre-Work and Post-Work surveys at the upstream cross section (**Figure 16**), and that either the channel bed was graded to be essentially flat or a combination of aggradation and degradation occurred at the downstream cross section (**Figure 17**).

The cross-sectional geometry that resulted from the excavation at the confluence with Rincon Arroyo appears to be more sustained during the period between the Pre-Work and Post-Work surveys. Upstream from the arroyo fan, the existing channel bed is about 1 foot below the Pre-Work bed (Cross Section 415831.8; **Figure 18**), suggesting that the excavation has resulted in reduced backwater effects and lowered rates of aggradation. At the mouth of the arroyo and on the downstream fan surface, the existing channel bed is between 1 and 2 feet below the Pre-Work bed (Cross Sections 415256.5 and 415013.3; **Figures 19** and **20**). The excavated bed surface in Rincon Arroyo is also lower than the Pre-Work bed, with lowered bed elevations that exceed 2 feet at the upstream cross section and exceed 4 feet at the downstream cross section (**Figures 21** and **22**, respectively).

3.6 Discharge Measurements

Discharge measurements were made at the time of the topographic and bathymetric surveys for Problem Locations 1 through 5 and 9 to assign a discharge to the surveyed water-surface profiles. Because the surveys occurred during the non-irrigation season, the discharge at Problem Locations 1 through 5 was too low to accurately measure and was estimated at less than 1 cfs at each of the locations. At Problem Location 9, the discharge was measureable, and the measurement was made at the USIBWC Rio Grande at El Paso, TX gage (USIBWC Gage No. 08-3640.00) on October 20, 2014, between 9:45am and 10:15am. The discharge measurement was conducted using a Marsh McBirney “Flo-Mate”, an electromagnetic flowmeter capable of measuring velocities to an accuracy of ±2 percent. The flow measurements were conducted using U.S. Geological Survey (USGS)-approved methods (Carter and Davidian, 1968). At the time of the measurement, the average discharge reported by the gage was 31.8 cfs, and very similar to the reported gage discharge of 32.9 cfs, indicating the current stage-discharge rating curve that is used at the *El Paso* gage is reasonably accurate over this range of relatively low flows.

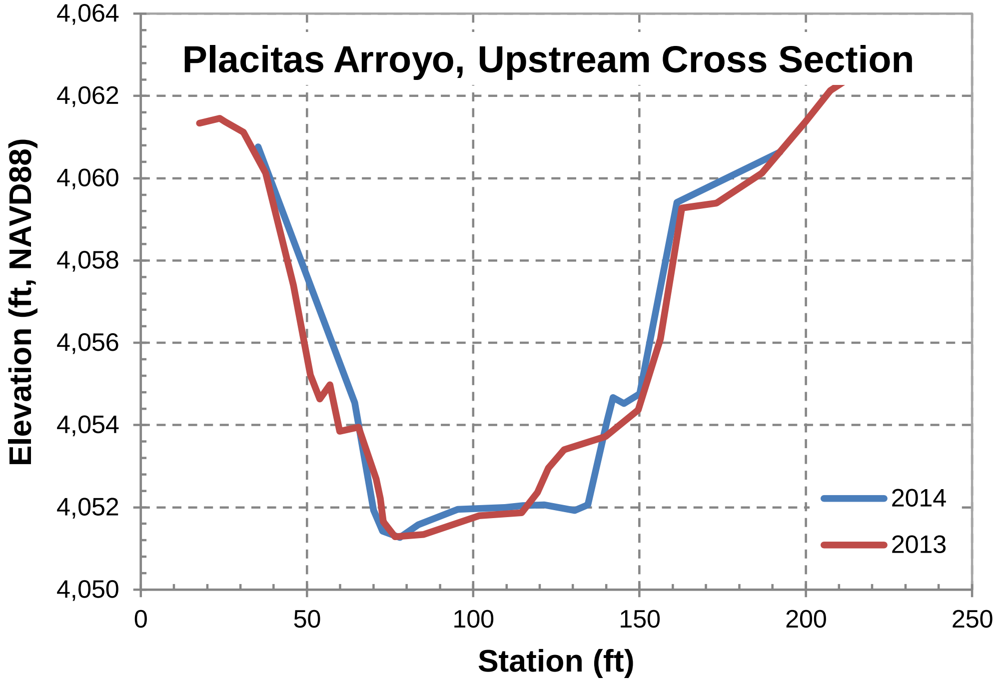

Figure 16. Comparison of 2013 (Pre-Work) and 2014 (Post-Work) cross sections at the upstream cross section in Placitas Arroyo.

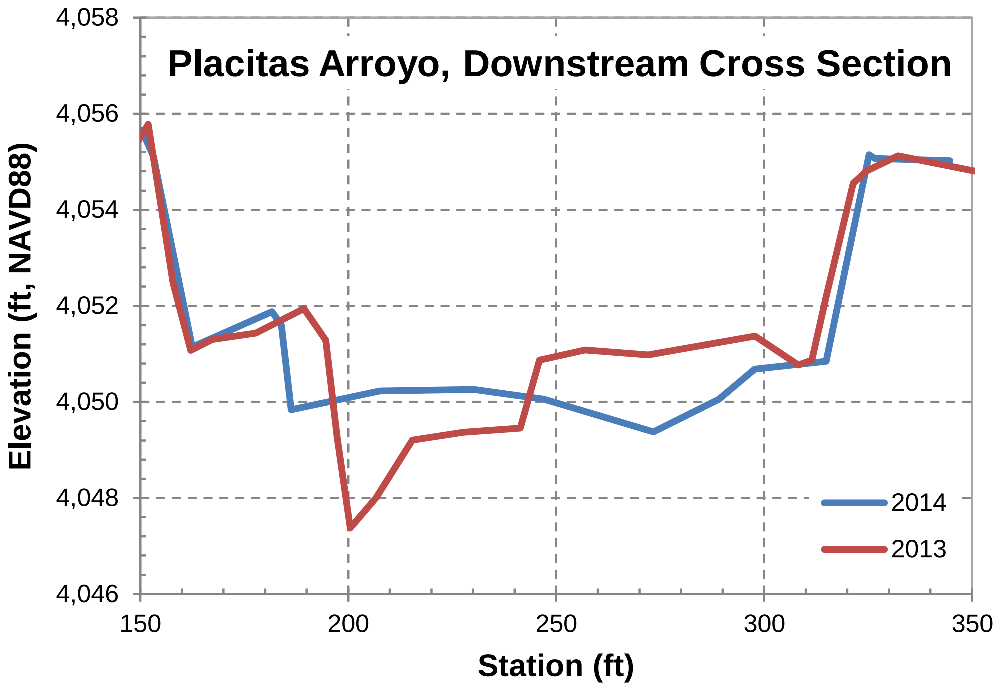

Figure 17. Comparison of 2013 (Pre-Work) and 2014 (Post-Work) cross sections at the downstream cross section in Placitas Arroyo.

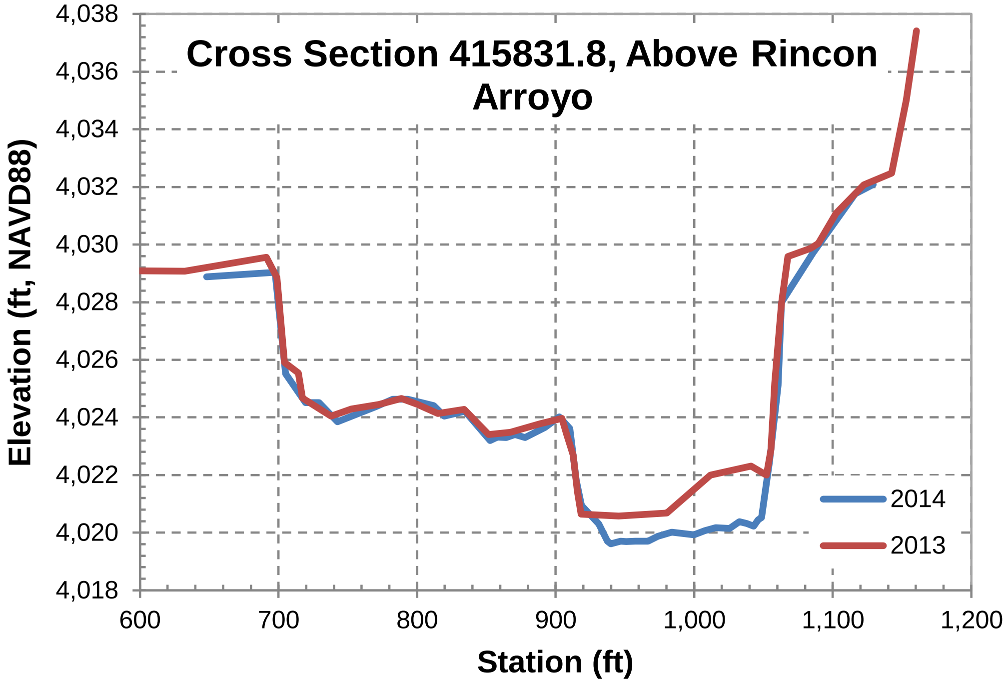

Figure 18. Comparison of 2013 (Pre-Work) and 2014 (Post-Work) cross sections at Section 415831.8 located just upstream from the confluence with Rincon Arroyo.

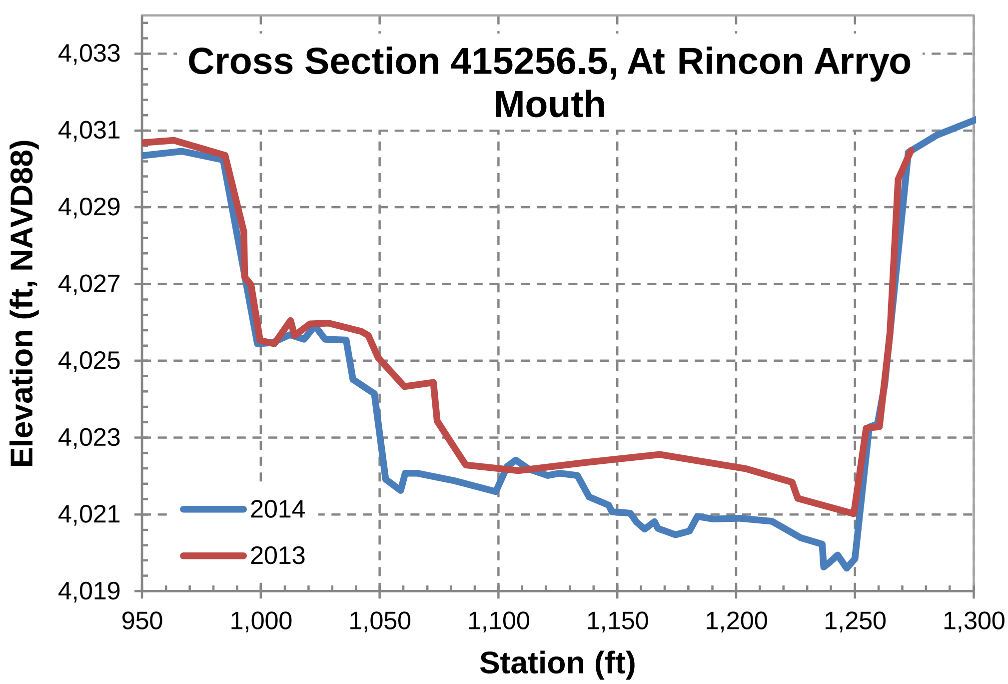

Figure 19. Comparison of 2013 (Pre-Work) and 2014 (Post-Work) cross sections at Section 415256.5 located along the Rincon Arroyo fan.

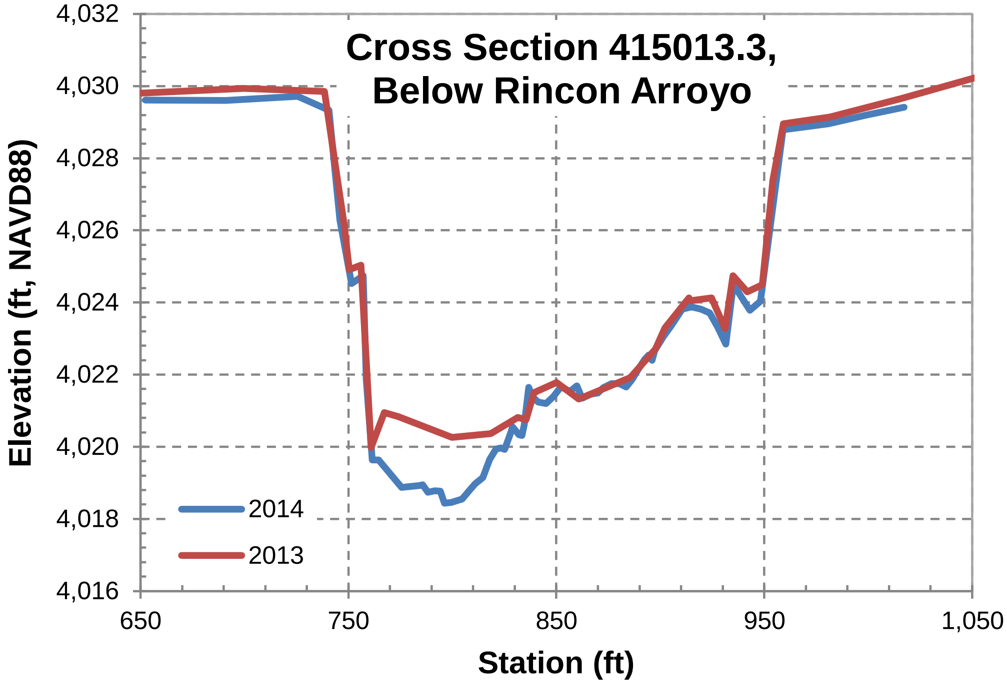

Figure 20. Comparison of 2013 (Pre-Work) and 2014 (Post-Work) cross sections at Section 415013.3 located along the downstream portion of the Rincon Arroyo fan.

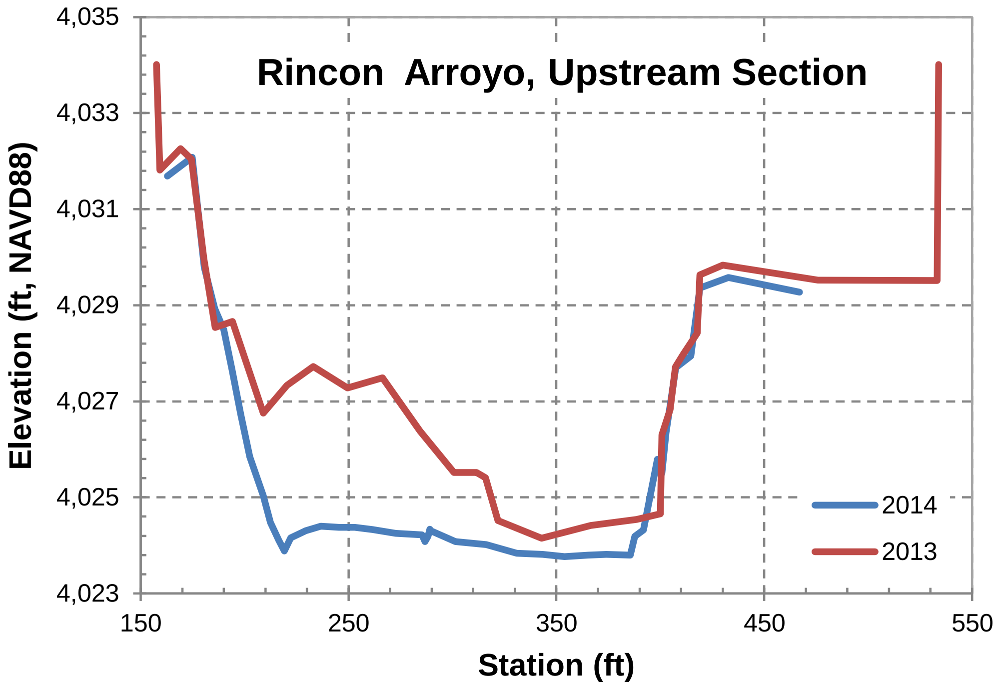

Figure 21. Comparison of 2013 (Pre-Work) and 2014 (Post-Work) cross sections at the upstream cross section in Rincon Arroyo.

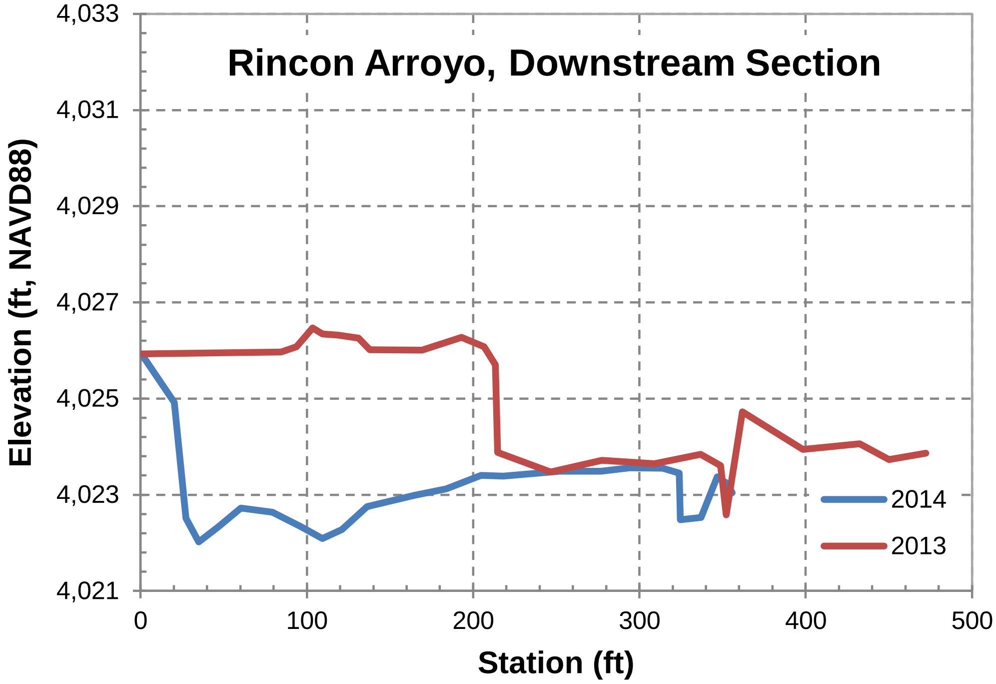

Figure 22. Comparison of 2013 (Pre-Work) and 2014 (Post-Work) cross sections at the downstream cross section in Rincon Arroyo.

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4 HYDRAULIC MODELING AND ALTERNATIVE DEVELOPMENT

A wide range of steady-state hydraulic models were developed and used for this study. First, the previously developed base model of the overall RGCP was obtained and reviewed. This model was then updated using the 2014 survey data described above. The updated base model was used to prepare localized base models at each of the nine problem locations, and these updated base models were then modified to represent with-CMA conditions to evaluate the short-term effects of the alternatives on water-surface elevation. All hydraulic modeling was performed using the USACE HEC-RAS computer software, Version 4.1 (USACE, 2010), a widely accepted 1-D step-backwater hydraulic model. The following sections summarize the development and results from the hydraulic modeling, and outline the development of the site-specific alternatives for channel maintenance.

4.1 Existing Base Model of the RGCP

The most up-to-date version of the base model of the RGCP was developed by Mussetter Engineering, Inc. (now a part of Tetra Tech and responsible for this study) as part of the USACE baseline study (USACE, 2007). In general, this model involved making adjustments to a version of the RGCP model that was developed by Tetra Tech (Tetra Tech, 2005). The adjustments involved incorporation of 145 main channel cross sections that were surveyed by Tetra Tech in 2004 and 20 main channel cross sections that were surveyed by the Elephant Butte Irrigation District (EBID) in 2007. The geometry of the overbanks was developed using a 2004 LiDAR-based digital terrain model (DTM) that was prepared for the Doña Ana County Flood Commission. Supplemental cross sections were then added between the survey sections by first cutting the sections from the 2004 DTM and then making appropriate adjustments to the subaqueous portions of the sections. Other updates to the model included adjustments to the geometry and relevant model input for numerous bridges and hydraulic structures. The model was calibrated, to the extent possible, by adjusting the Manning's *n* roughness values such that the predicted water-surface elevations matched measured water-surface profiles that were surveyed in 2007 as part of that study. The existing base model extends over a distance of about 107 miles from about 1.3 miles above Percha Dam to American Dam, and includes 1,178 cross sections, 28 bridges and 3 inline structures. The calibrated model was executed over a range of flows from 500 cfs up to 6,000 cfs. Although the model was primarily developed for this range of relatively low flows, it was also executed using the routed 100-year peak discharge predicted by the 2007 FLO-2D model that was also developed as part of that project (**Table 6**; USACE, 2007). The specific details of the development of this model are summarized in a Draft Technical Memorandum that was prepared by MEI as a part of that project (MEI, 2007a).

4.2 Updated Base Model of the RGCP

The previously developed base model of the RGCP was updated using information from the targeted cross-sectional surveys discussed above. The updates were only made at the nine problem locations and were carried out by taking the following steps:

1. The model cross sections within each of the problem location reaches were first cut from the most recent available (2011) LiDAR-based mapping. This mapping included bare-earth, 1-meter DEM topography that was developed by Tetra Tech for the USIBWC in August 2011. The 1-meter digital elevation model (DEM) was originally provided in the UTM coordinate system and was re-projected to New Mexico State Plane NAD83 coordinates. The original elevations were in meters (NAVD88) and were converted to feet (NAVD88). The 2011 topography covered the area from the Doña Ana County boundary south to American Dam

Table 6. Routed 100-year, 24-hour Peak Discharge (modified from USACE, 2007).

Location	HEC-RAS River Station (ft)	Routed Peak Flow (cfs)
Caballo Dam release	564,639	2,350
Trujillo Canyon	543,623	4,880
Montoya Arroyo	536,424	8,470
Green Canyon/Tierra Blanca Creek	527,575	11,600
Sibley Arroyo	518,915	12,970
Berrenda Arroyo	510,891	14,900
Arroyo Cuervo	492,837	15,150
Placitas Arroyo	448,155	14,690
Angostura Arroyo	423,306	14,300
Rincon Arroyo	415,055	14,070
Reed Arroyo	412,284	14,110
Broad Canyon	356,831	11,690
Faulkner Canyon	333,942	10,990
Leasburg Diversion Dam	328,017	12,060
Shalem Bridge	271,159	13,120
Mesilla Diversion Dam	207,726	12,870
Vinton, Texas	83,171	12,110
Nuway, Texas	74,569	13,130
Canutillo, Texas	67,078	13,090
Borderland, Texas	56,502	11,170
Courchesne Bridge	8,865	9,790
American Diversion Dam	418	10,990

(River Sta 5259+28 to Sta 0+00.457). Because the 2011 topography did not cover the RGCP reach from Caballo Dam south to the Doña Ana County line (River Sta 5646+39.1 to Sta 5259+28), the 2004 LiDAR-based mapping was used to cut the sections near the upstream limit of Problem Location 1. Cross sections not located within the problem location reaches were not modified in the updated base model (except for the bathymetric adjustments to the cross sections located between Problem Locations 2 and 3, as discussed below).

2. The cross-sectional geometry obtained from the 2014 surveys was then spliced into the LiDAR-based cross sections and the resulting geometry was inserted into the model.
3. For model cross sections located within problem location reaches that were not surveyed in 2014/2015, the portion of the cross section that was underwater at the time of the 2011 LiDAR surveys was estimated using the HEC-RAS cross-section interpolation routine. Care was taken to ensure that all mid-channel and bank-attached bar features that were captured in the LiDAR mapping were preserved in the updated base model. The interpolated, bathymetric portions of the cross sections were then spliced into the LiDAR-based cross sections and the resulting geometry inserted into the model.
4. Once the cross-sectional geometry of the model was inserted, minor adjustments to the channel bank stations and ineffective flow areas were made to accurately represent main-channel and overbank conveyance conditions. Incorporation of ineffective flow areas into the cross sections located at the drain/arroyo mouths was especially important in that the surveyed cross sections often include the bed geometry along the arroyo that do not effectively convey RGCP flows.
5. The Manning's *n* (roughness) values along the channel bed and margins were also adjusted to be more representative of the existing vegetation and channel boundary material characteristics.

A comparison of the 2004 and 2014 cross section surveys showed about 2 feet of aggradation at the downstream limit of Problem Location 2 and at the upstream limit of Problem Location 3. As such, the relatively short portion of the base model between Problem Locations 2 and 3 was adjusted to reflect the aggradation by adjusting the bed elevation in the updated base model to ensure that an appropriate downstream boundary condition could be prepared for the localized model for Problem Location 2. The bed elevations were adjusted by interpolating the thalweg (minimum bed) elevation between the downstream survey section at Problem Location 2 and the upstream survey section at Problem Location 3. The bed geometry was then adjusted by raising the thalweg elevation to match the interpolated elevation, and a distance weighted scheme was used such that progressively smaller increases were made proceeding outward toward the bank stations.

In most cases, the planform alignment and extent of the cross sections did not change significantly between the original and updated base models. However, some of the 2014 survey sections did not coincide with the original base model sections to ensure that specific features (i.e., hydraulic controls such as the Tierra Blanca Vortex Weir, the Pre-Work cross sections surveyed by USIBWC, etc.) were captured by the surveys. These cross sections were included in the updated base model, and if the sections were located relatively close to an existing model section, the existing model section was removed from the updated base model because maintaining a generally consistent cross section spacing is computationally important to the sediment-transport modeling. Because the surveyed cross sections at Problem Locations 6 through 8 did not coincide with any of the base model sections, all of the existing base model cross sections were removed and replaced with either the 2014/2015 survey section or the LiDAR/interpolation-based cross sections. It should be noted that the original base model was developed for a range of relatively low flows (up to 6,000 cfs), whereas the updated base model was developed for a much larger range of flows up to the 100-year peak discharge. It was therefore necessary to extend some of the cross sections farther into the overbanks to contain the higher discharges. While the extended cross sections did not significantly affect the flow conveyance in the overbanks at most locations, the wider cross sections at Problem Location 5 did result in increased overbank flow conveyance. The implications of this refinement are discussed in more detail below.

4.3 Comparison of Base Model and Updated Base Model Results

A comparison of the results from the original base model and updated base model was made to assess the effects of aggradation, degradation, or channel maintenance activities that have occurred over recent years. The comparison was made at each of the nine problem locations for the following four flow scenarios:

- Average annual spring hydrograph discharge (2,350 cfs above Mesilla Diversion Dam and 1,400 cfs downstream from the dam).
- Mid-range channel capacity (3,000 cfs).
- Upper range of channel capacity (3,500 cfs).
- 100-year routed peak discharge (ranging from 2,350 to 15,150 cfs; Table 6).

Results from the comparison generally indicate that, as expected, the aggradation that has occurred along the majority of the reaches results in an increase in water-surface elevation (**Table 7**; comparative water-surface profile and change plots for each problem location are presented in **Appendix F**). At Problem Location 1, the average increase in water-surface elevation ranges from 0.7 feet (100-year peak discharge) to 1.7 feet (2,350 cfs), and the maximum increase of about 2.9 feet (2,350 cfs) occurs in the vicinity of the confluence with Tierra Blanca Creek/Green Arroyo. About 1.2 feet of average increase in water-surface elevation occurs over the range of flows at Problem Location 2 and the maximum increase of about 2 feet (2,350 to 3,500 cfs) occurs in between Thurman I and Thurman II Arroyos. The largest increase in water-surface elevation among the sites occurred at Problem Location 3, where an average increase of about 3.0 feet was indicated (2,350 cfs) and a maximum increase of 4.3 feet (2,350 cfs) predicted near the Rincon Siphon. However, the original base model used estimated main channel bathymetry in the vicinity of the grade-control structure for the siphon and does not appear to accurately represent the elevations along the crest of the siphon. As a result, the predicted increase in water-surface elevation is probably not accurate along the reach between the siphon and the 2004 BC-42 survey section (Sta 4337+04 to Sta 4329+20), but appear reasonable along the upstream portion of the reach.

About 1 foot of average increase in water-surface elevation occurs over the range of flows at Problem Location 4 and a maximum increase of about 1.8 feet (2,350 to 3,500 cfs) is indicated upstream from Bignell Arroyo. Relatively small increases of less than 0.7 feet occur at Problem Location 5 at the lower discharges, and an average decrease change in water-surface is indicated at the 100-year event and is a result of the use of wider cross sections in the updated base model at this problem location. The maximum increase in water-surface elevation at this problem location occurs in the vicinity of the Rock Canyon fan. The smallest increase in water-surface elevation occurred at Problem Location 6 with an average increase of 0.2 feet (2,350 cfs), although this average value includes predicted decreases in water-surface elevation downstream from Mesilla Dam. Considering only the reach upstream from the dam, the average increase in water-surface elevation ranges from about 0.4 feet (3,500 cfs) to 0.6 feet (2,350 cfs). Relatively small increases in water-surface elevation of between 0.2 to 0.4 feet, on average, occur at Problem Location 7 despite the aggradation that has occurred along the thalweg of this reach.

Table 7. Summary of change in predicted water-surface elevations between the original and updated base models for the RGCP.

Problem Location	Type of Change	Thalweg Elev. Change (ft)	Change in Predicted Water-surface Elevation (ft)			
			2,350/ 1,400 cfs*	3,000 cfs	3,500 cfs	100-year Peak
1	Average	2.68	1.67	1.55	1.46	0.74
	Maximum	8.05	2.85	2.63	2.44	1.18
	Minimum	-1.34	-0.32	-0.25	-0.19	-0.13
2	Average	0.45	1.18	1.23	1.28	1.08
	Maximum	2.38	2.01	2.06	2.08	1.68
	Minimum	-2.26	-0.02	0.13	0.23	0.47
3	Average	3.54	3.01	2.92	2.80	1.45
	Maximum	5.14	4.35	4.23	4.13	2.31
	Minimum	1.95	1.39	1.46	1.43	0.05
4	Average	1.01	0.97	1.05	1.08	1.09
	Maximum	3.90	1.78	1.82	1.79	1.54
	Minimum	-0.45	0.02	0.03	0.03	0.03
5	Average	0.18	0.67	0.65	0.64	-0.21
	Maximum	1.09	1.25	1.25	1.24	0.26
	Minimum	-1.25	0.01	0.01	0.00	-1.32
6	Average	1.82	0.20	0.20	0.15	0.21
	Maximum	3.84	0.95	0.95	0.90	0.61
	Minimum	-0.87	-0.79	-0.77	-1.46	-0.29
7	Average	1.57	0.38	0.24	0.22	0.20
	Maximum	2.51	0.69	0.36	0.32	0.42
	Minimum	-0.59	0.01	-0.06	-0.08	-0.23
8	Average	1.99	0.95	0.64	0.62	0.56
	Maximum	3.52	1.46	0.92	0.89	0.85
	Minimum	0.78	-0.06	-0.03	-0.05	-0.04
9	Average	0.20	0.79	0.67	0.63	0.53
	Maximum	2.10	1.44	1.21	1.17	1.01
	Minimum	-1.69	0.00	0.00	0.00	0.00

*2,350 cfs upstream from Mesilla Dam; 1,400 cfs below Mesilla Dam. Reported values at Problem Location 7 are at 2,350 cfs.

Water-surface elevations at Problem Location 8 have increased by between 0.6 feet (100-year peak discharge) to 1.0 feet (1,400 cfs), on average, but have increased by as much as 1.5 feet along the portion of the reach between Country Club Bridge and the Nemexas Siphon crossing. Average increases in water-surface elevation range from 0.6 to 0.8 feet at Problem Location 9, with a maximum increase of 1.4 feet (1,400 cfs) indicated below Courchesne Bridge.

4.4 Localized Base Modeling of Problem Locations

The updated base model of the RGCP was used to prepare the localized base models for each of the problem locations. The predicted water-surface elevations from the updated base model of the overall RGCP was used to generate the downstream boundary condition for the localized base models. These models were executed over the same range of flows that were included in the updated base models of the overall RGCP. Because the localized base models include the model geometry and downstream boundary conditions from the overall base model, the model results are identical to the results from the overall updated base model of the RGCP along the problem location reaches.

The updated base model of the RGCP includes the inline structure for Mesilla Dam, but does not include the Eastside and Westside Canals. The localized base model at Problem Location 6 was therefore revised to include both of the canals and the canal headworks. Two versions of the localized base model were prepared. In the first version, a junction was located upstream from Mesilla Dam that represents the point where the flow splits into the canals and into the downstream river. This version of the model does not include any flow optimization at the junction but instead uses specified flows in the canals that preserve the flow change that was assumed at the structure in the updated base model. The second version of the model was prepared to provide the necessary inputs to the sediment-transport model discussed below. This version of the model is somewhat more complicated in that the canal headworks were coded into the model such that flow optimization at the junction could be used and the effects of the various gate configurations could be assessed. The canal headworks were coded in as inline structures on the canal reaches (rather than lateral structures along the mainstem reach). Artificially flat cross sections that span the width of the structures were inserted at the upstream end of the canal reaches, since HEC-RAS requires that inline structures be bound by up- and downstream cross sections. Use of this second version of the model, and the model results, are discussed in the sediment-transport modeling section below.

4.5 Alternative Development

Five separate channel maintenance alternatives were evaluated at each of the nine problem locations. The alternatives included “sediment removal” alternatives that involved the mechanical excavation of accumulated sediment to varying degrees, and “non-sediment removal” alternatives that included a variety of options that did not directly involve excavation within the RGCP.

4.5.1 Solutions Applied on Other Streams and Rivers Similar to the RGCP

A review of treatments that have been applied to streams and rivers with sedimentation and channel maintenance issues similar to those on the RGCP was carried out to identify a range of alternatives that could be considered in this study. The review was focused on areas in the arid southwestern United States to ensure the identified alternatives would be most applicable to the study area.

4.5.1.1 Sediment Traps

The 2011 Las Conchas Fire was the largest in recorded history for the state of New Mexico. The fire moderately to severely burned approximately 1,600 acres of the Santa Clara Pueblo, much of it in the Santa Clara Creek watershed. Forty-five percent of the watershed was burned, altering the surface hydrologic characteristic of the watershed. Since the Las Conchas fire, there have been few large rainfall events, but there have been a significant number of debris flows in the burned basins. Sediment supply from the tributary channels is of particular concern due to the steep gradient of the channels coupled with the loss of vegetative cover and hydrophobic soils. The Santa Clara Pueblo, working with the U.S. Army Corps of Engineers, installed a number of sediment-trapping debris basins upstream from the mouths of the tributaries to control the sediment delivery into Santa Clara Creek. Although mechanical excavation of the trapped materials has been periodically required, the basins have successfully reduced tributary sediment loading to the mainstem. However, impoundments on the mainstem of Santa Clara Creek, although very effective sediment traps, have resulted in widespread and systematic downstream channel erosion.

The Albuquerque Metropolitan Arroyo Flood Control Authority (AMAFCA) operates numerous flood control facilities on various arroyos that are within the Authority's jurisdiction and are tributary to the Rio Grande. The facilities not only successfully reduce downstream flood flows but also trap significant amounts of sediment and therefore require periodic maintenance. Many of these facilities include water quality structures designed to clean stormwater of debris, sediment and refuse. The structures constructed to date have operated as designed, trapping large quantities of sediment and debris. In some cases, AMAFCA has worked with the U.S. Fish and Wildlife Service to ensure the water quality structures provide the maximum possible habitat benefits.

The Los Angeles County Department of Public Works has installed and maintained a large number of sediment-trapping debris basins as part of their flood control program. The debris basins are typically located at the mouths of canyons and are intended to capture tributary sediment delivered during storm events; if this sediment was otherwise conveyed into the flood control system, it could result in blockage and subsequent increased flood conditions, as well as increased surface wear on the system's concrete channels and drains. The debris basins have proven effective at trapping sediments and mitigating the downstream impacts, but frequent maintenance to remove the trapped material is required.

4.5.1.2 In-channel Treatments

The Bureau of Reclamation (BOR) has constructed various in-channel structures within the Middle Rio Grande for a large number of reasons. BOR constructed gradient restoration facilities (GRFs) to raise the channel bed, provide fish passage, reduce lateral migration, and reconnect the river to the abandoned floodplain. The GRF design typically includes a pilot channel and grading in the floodplain to improve sediment continuity during low flows and enhance hydrologic connectivity between the channel and floodplain during high flows. The facilities have generally functioned as intended. BOR constructed buried rock vanes and riprap revetment bank protection at various locations that have successfully deterred lateral channel migration and bank erosion. BOR installed bio-engineered bank protection consisting of a rock toe and several layers of coir fabric encapsulated soil planted with native vegetation. These structures have performed reasonably well, although there is some evidence of erosion into the upper portion of the bank. The BOR's bendway weirs have (1) reduced bank erosion by directing flow away from the bankline and (2) improved fish habitat by providing increased hydraulic diversity. BOR has excavated pilot channels along reaches where sediment plugs tend to form. These channels are intended to limit the formation of the plugs that tend to result in significant lateral migration around the plugged areas. BOR recently experimented with destabilization of bank-attached bars and islands, but the success of these activities are not currently known. While many of the in-channel treatments that BOR has installed on the Middle Rio Grande are intended to improve aquatic and riparian habitat, most of the treatments are also intended to counteract excessive erosion. Only the pilot channels are intended to specifically address sedimentation issues.

4.5.1.3 Floodplain and Overbank Treatments

BOR has conducted out-of-channel work along the Middle Rio Grande for a wide variety of reasons, most of which are centered on the goal of improving riparian habitat. Levee setbacks have been constructed in areas where lateral migration of the channel was threatening the levees, and these projects provide the added benefit of improved habitat by increasing the area of the floodplain. For purposes of experimenting with habitat improvement treatments, BOR constructed bankline benches, overbank terraces and offline channels in the floodplain, and BOR removed lateral confinements at some locations. Monitoring of these localized projects is ongoing so the degree of success has not yet been established. BOR experimented with a variety of floodplain vegetation management techniques including invasive species control, floodplain revegetation, wetland management, and enhanced willow swales. These experimental management techniques are currently being monitored so the success of each is still being determined.

4.5.1.4 Other Solutions

Since its construction in 1964, The Glen Canyon Dam on the Colorado River has historically trapped sediments, reducing the sediment supply to downstream reaches by 95 percent of pre-dam conditions. The operation of the dam has also reduced downstream flood flows that historically mobilized, transported and distributed sediments delivered by downstream tributary channels. Over the past 20 years, the BOR has been experimenting with flood flow releases from the dam in an effort to simulate pre-dam flood conditions to determine if the higher flows would transport and distribute the accumulated sediments at the tributary alluvial fans. Results from the experiments have indicated that the simulated floods have restored sandbars and reduced vegetation encroachment, but that the improved habitat has yet to benefit native fish.

Although the Platte River in Central Nebraska is not located within the arid southwestern United States, experiments with artificial floods have been undertaken to assess a variety of objectives. This river has experienced significant changes in channel geometry and riparian habitat as a result of the altered hydrologic regime associated with upstream reservoirs. In addition, invasive species (primarily phragmites) and other vegetation have colonized bars causing further loss of nesting habitat for target endangered species (Least Tern, Piping Plover and Whooping Crane). The Central Nebraska Public Power and Irrigation District, in cooperation with the Platte River Recovery Implementation Program and the Nebraska Department of Natural Resources, have released large discharges from Kingsley Dam (Lake McConaughy) to determine if the high flows would have sufficient hydraulic energy to scour the vegetation and reduce the height of the bars. Due to the scour resistant nature of the vegetation, the artificial floods have not met project objectives.

4.5.2 Sediment Removal CMAs

At each of the problem locations except for the Mesilla Dam site (Problem Location 6), three of the CMAs are identified as sediment-removal alternatives. As indicated in Table 2, the three sediment-removal CMAs include:

1. Channel Excavation Long,
2. Channel Excavation Short, and
3. Localized Channel Excavation.

At the Mesilla Dam site, the sediment-removal CMAs included only the Channel Excavation Short and Channel Excavation Long alternatives. Mapping showing the extents of the excavation, along with comparative profile and typical cross section plots for the sediment-removal alternatives are presented in **Appendix G**.

For the Channel Excavation Long alternatives, the up- and downstream limits of the excavation were located at the limits of the convex bed profile shape because this type of profile typically represents areas with the most significant aggradation. The resulting excavation lengths under the "long" alternatives ranged from 3,900 to 11,800 feet. For the Channel Excavation Short alternatives, the up- and downstream limits of the excavation varied by problem location, but in general, the upstream limit was set in the vicinity of the upstream limit under the "long" alternative, and a maximum target excavation length of 2,600 feet was used to set the downstream limit. The resulting excavation lengths for the "short" alternatives generally ranged from 900 to 2,700 feet. At the Mesilla Dam site (Problem Location 6), where the degree of substantial aggradation extends over longer distances than at the other problem locations, the "short" excavation length increased to 4,700 feet. The excavated bed profile under both the "short" and "long" alternatives was typically set to match the existing bed elevation at the downstream limit of the excavation with a slope that resulted in reasonable excavation depths through the excavated reach. At the Mesilla Dam site, the downstream elevation of the excavated bed profile was set to match the lowest sill elevation at the dam that corresponds with the sills of the leftmost and rightmost gates. Average excavation depths under both the "short" and "long" alternatives ranged from about 2.5 to 5 feet. The excavated channel width was then set such that the excavated channel has a flow capacity that restores the overall channel capacity to ~2004 conditions based on information presented in the USACE (2007) baseline study as well as the results from the 2007 baseline model. At locations where the existing channel capacity exceeds 3,500 cfs, the geometry of the excavated channel was designed to have a capacity of between 750 and 1,000 cfs since this range of discharges represent the lower regime of Caballo releases during normal operating conditions.

For the "Localized Sediment Removal" alternatives, it was assumed that the excavated channel would span the entire width of channel and that the excavation profile would need to have a down-gradient slope and tie into the downstream existing bed profile to avoid creation of a pool/sediment trap. Channel bottom widths over which the excavation would occur range from 120 to 300 feet. Target excavation lengths of 200 feet were used, but because the resolution of the available bed profile is derived from the modeled cross sections, which in some cases have a spacing that exceeds 300 feet, it was necessary to increase the excavation lengths beyond the target length. Excavation lengths for the "local" alternatives ranged from 80 to 500 feet, and the resulting average excavation depths range from 2.5 to 5.0 feet.

4.5.3 Non-Sediment Removal CMAs

The remaining alternatives at each of the sites are classified as "Non-sediment Removal Alternatives" and vary by the problem location. A number of potential alternatives that were identified in the Statement of Work were initially eliminated from consideration for a variety of reasons. These potential alternatives, and the reason for not considering them further, are as follows:

- Incorporation of standard and enhanced vortex weirs: Vortex weirs[2] are v-shaped weirs constructed with boulders, similar to the existing weir below the mouth of Tierra Blanca Creek. While vortex weirs do tend to create localized areas of scour in the immediate vicinity of the

[2] The statement of work for this study uses the term "vortex weir", so that terminology is used in this report for purposes of consistency.

weir downstream from the crest, the weirs would provide grade-control to maintain the upstream channel gradient that could cause further deposition. The weirs would also limit potential downcutting by preventing the upstream migration of headcuts.

- Re-establishment of oxbow embayments or creation of new embayments: Unless the embayment were located within a tributary or drain that delivers significant quantities of sediment to the RGCP (i.e., the “arroyo sediment trap” alternative discussed below), the embayments would probably not capture significant quantities of the bed-material load because the eddy that would tend to form at the mouth of the embayment would likely only entrain the portion of the bed material load that travels along the bank.
- Grading to create inset floodplains, terraces or lowered overbanks: Grading to create lowered overbanks in a traditional sense would likely result in increased deposition due to the loss of flow and associated energy along the floodplain or overbank, which could also affect conveyance efficiency. Grading to create out-of-bank terraces would likely only affect sediment-transport conditions at flows that exceed the channel capacity, and unless the terraces were constructed in a manner that constricted the flow over significantly long reaches, the terraces could result in increased deposition at these higher flows. Creation of an inset floodplain within the currently active channel could result in increased energies and sediment-transport rates along the main channel at low flows. However, it may be more cost efficient to construct spur dikes in a manner that allows the low-elevation floodplain to form naturally (i.e., the “low-elevation spur dike” alternative discussed below).
- Mechanical destabilization of banks or native material bank stabilization: Destabilization of the banks may ultimately result in a wider channel geometry that would likely have reduced hydraulic energies and thus reduced sediment-transport rates. The wider channel geometry would also likely reduce conveyance efficiency, so this alternative was not further considered.
- Installation of riprap revetments, toe revetment plantings, or other bio-engineered bank stabilization methods: Significant reaches of bank instabilities were not identified at any of the problem locations. At some of the tributary confluences, evidence of previous, localized erosion of the opposite bank was identified, but actions have been taken since to at least partially protect the eroding bank at most locations (i.e., by mechanically shifting the coarsest fan material onto the toe of the eroded bank). Protecting the locally eroding banks may be necessary in the future to protect the levees or other infrastructure and the erosion should be monitored to that end, but any form of bank protection would not address the current sedimentation issues. Furthermore, toe revetment plantings or other bio-engineered bank stabilization methods may reduce conveyance efficiency due to increased evapotranspiration rates. Per the Statement of Work, installation of riprap revetments at Problem Location 8 is included as a non-sediment removal alternative at this site. To protect the eroding banks that are opposite the mouths of some of the tributary alluvial fans (i.e., the Rincon Arroyo Fan at Problem Location 4 and the Rock Canyon alluvial fan at Problem Location 5), vanes are recommended in lieu of riprap revetment (discussed below).
- Mechanical removal or reduction of lateral confinements: Other than the alluvial fans that have formed at some of the arroyo mouths, no significant lateral confinements that could be mechanically removed were identified at any of the problem locations. Removal of the lateral confinements at the alluvial fans is considered as part of the sediment-removal alternatives. Lateral confinements associated with infrastructure (i.e., bridges) would be relatively expensive to remove. As a result, removal or reduction of lateral confinements was not considered further in this study.

Other potential non-sediment removal alternatives that were initially considered but ultimately ruled out include:

- Installation of sediment ejection systems within the RGCP: Recently developed sediment ejection systems such as Streamside Technology's Sediment Collector™ use the energy of the flowing water to move bed-load sediment into a hopper where the sediment is pumped to a disposal site. While these systems have been laboratory and field tested, they have not been tested in a channel the size of the RGCP and the performance levels are not known. Costs associated with the pumping of sediment from the collector's hopper would likely be significant, and the pumping would also remove unknown quantities of water from the RGCP. As such, this alternative was not further considered as an option in the RGCP.
- Beaver activity maintenance: Beaver activity at the mouth of the Rincon/Tonuco Drain (Problem Location 5) is currently affecting drain efficiency. Removal and relocation of the beavers would probably enhance drain efficiency at a very low cost, and is therefore a recommended action. However, because this alternative will not benefit channel maintenance along the RGCP and because this alternative is currently practiced, it was not recommended as a channel maintenance alternative.

After the initial screening of the alternatives, the final non-sediment removal CMAs were selected for each site (Table 2). The final non-sediment removal CMAs were selected based on the review of successful mitigation actions implemented on streams and rivers in other semi-arid environments, previous experience with similar sediment related and/or channel maintenance issues, engineering and geomorphic judgement, and by order of elimination of the alternatives that were not considered based on the reasoning presented above. It is important to note that the selected alternatives varied by problem location, depending on the specific problems and constraints at each site. It should also be noted that construction of the majority of the non-sediment removal alternatives would require Section 404 permits to be in compliance with the Clean Water Act, since the work would require construction activities in ephemeral or perennial waters of the United States. The following paragraphs summarize the alternatives in general; specific details regarding the design of the alternatives at each problem location are outlined in Section 4.6.

At many of the sites where tributary sediment loading is the primary concern, construction of sediment traps within the arroyo upstream from the confluence with the RGCP would greatly reduce coarse-grained sediment supply to the RGCP. (Estimated mean annual bed material volumes delivered by the tributaries are discussed in the alternative design specifics section, below.) The arroyo sediment traps could be designed in series in a manner that traps the coarsest material in the upstream trap and progressively finer material proceeding toward the mouth (**Figure 23**). The lowest trap could be designed as an embayment to the RGCP and may provide habitat benefits as a lower velocity, off-channel refuge area with vegetative cover. Although it would be necessary to periodically excavate material from the sediment traps, the excavation may not be required as frequently and loading/hauling costs may be reduced, so this alternative would probably be less expensive than the excavations from the RGCP as historically practiced. One potential concern with this type of alternative is the potential need to acquire private land to create sediment traps that would have enough volume to store materials delivered during tributary flooding events. As a result, the sediment-traps were designed within the USIBWC ROW, where possible.

Another non-sediment removal alternative that could be employed at many of the sites where coarse-grained tributary sediments are resulting in sedimentation issues involves construction of low-elevation spur dikes or vanes (**Figure 24**). The low-elevation spur dikes are similar to bendway weirs, but are not specifically designed for river training or bank stabilization purposes.

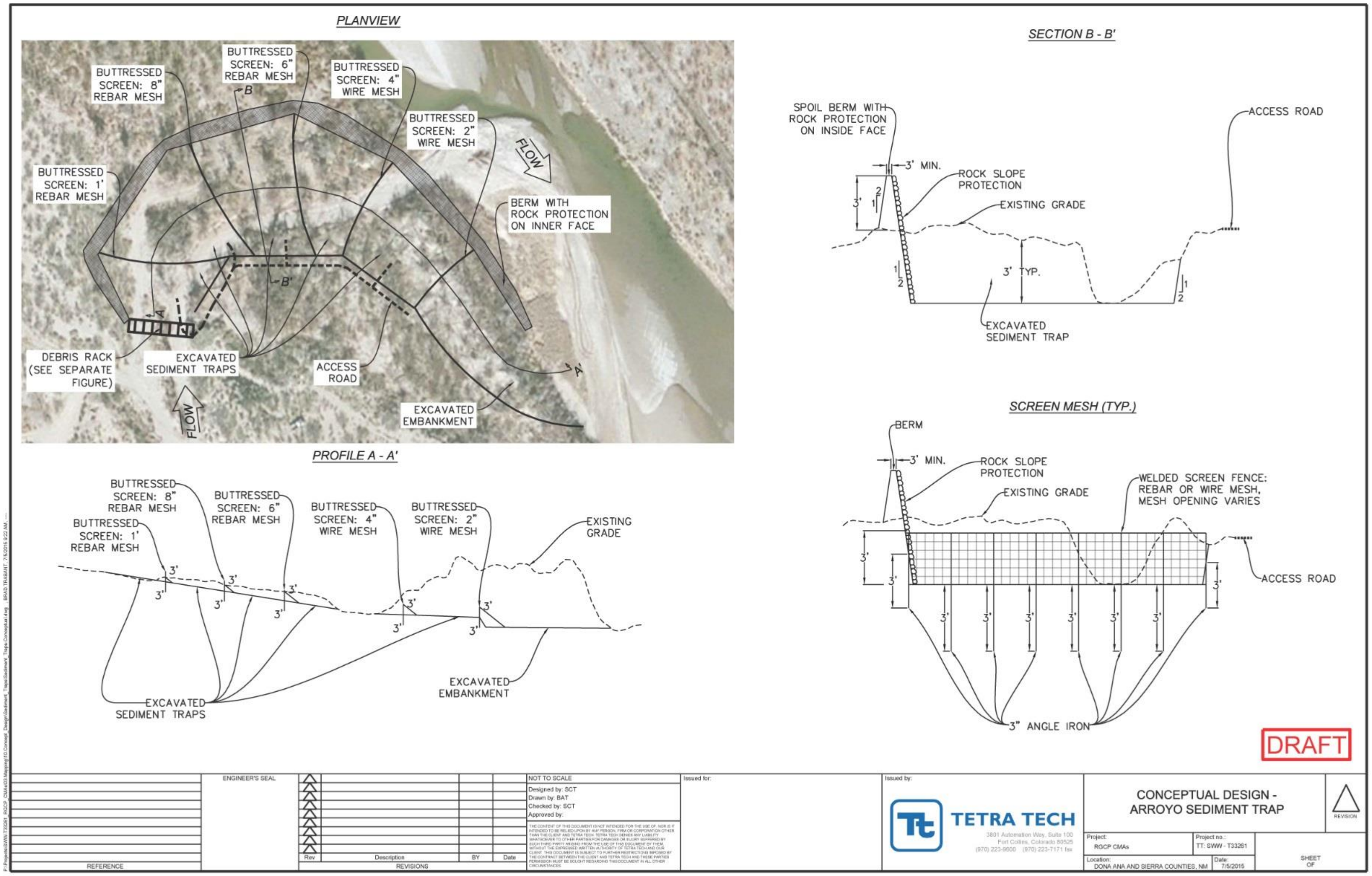

Figure 23. Example of conceptual planview drawing showing the layout of a typical arroyo sediment trap/ habitat feature (Rock Canyon at Problem Location 5).

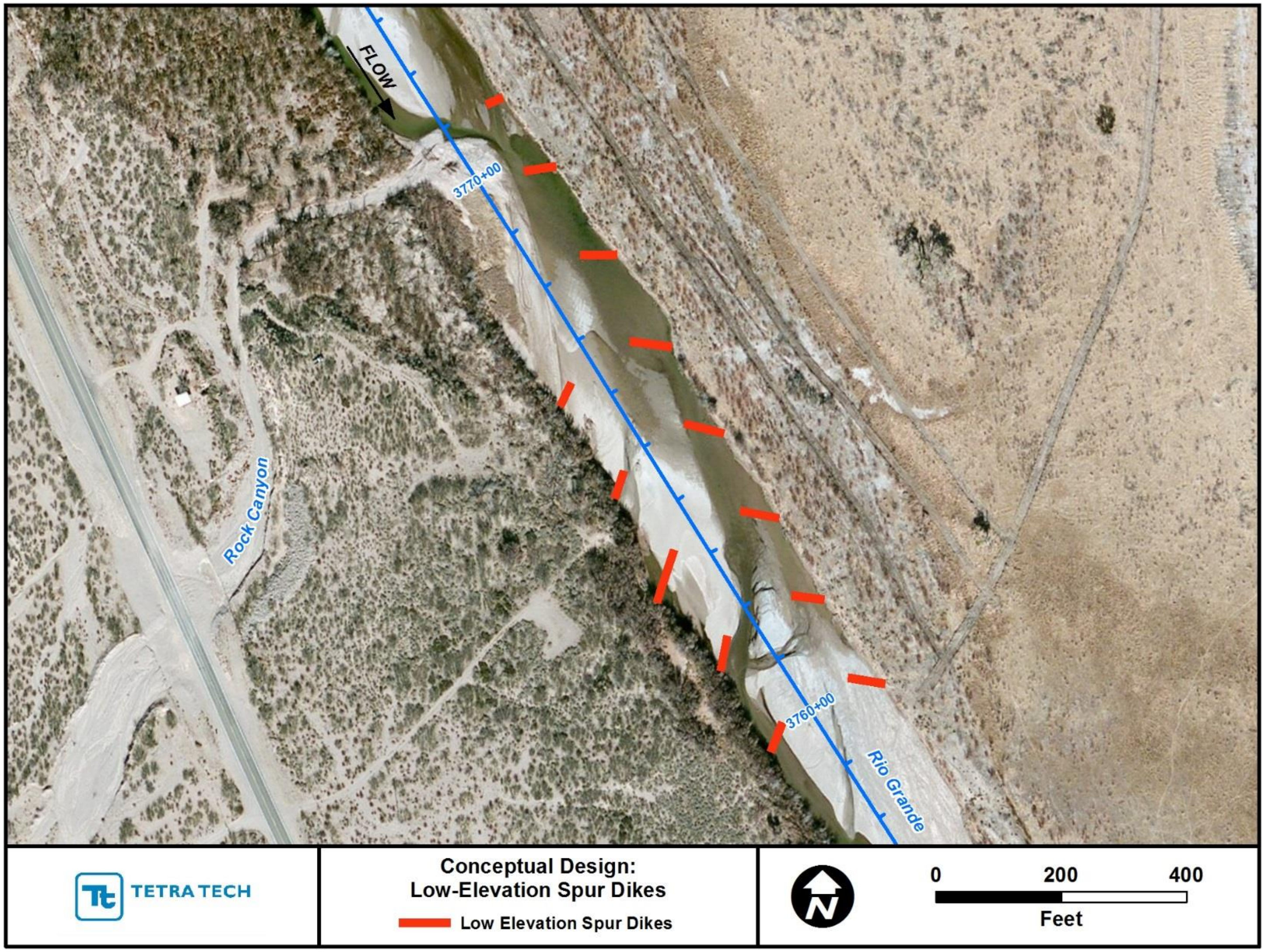

Figure 24. Example of conceptual planview drawing showing the layout of a typical low-elevation spur dike and vain configuration (Rock Canyon at Problem Location 5).

The spurs are constructed with rock riprap or boulders, laid out with a slight upstream orientation, and have a top elevation that is less than the top of bank elevation to control bedload movement. The spurs could be designed to constrict the flows and increase the velocity and energy between the spur nose and opposite bank, thereby increasing sediment-transport rates. If properly designed, the coupled erosion along the nose of the spur and deposition upstream from the next lower spur would manage the deposition of the coarser (gravel and cobble) fractions in a desirable manner, leaving the sand-sized fractions available for downstream transport. Downstream from the tributary fans, the vanes could be constructed in a manner that would be similar to a vortex weir, but instead of spanning the entire width of the channel would have a gap in the middle to allow for incision between the vanes. This alternative may require some degree of maintenance in that it might be necessary to periodically remove the trapped material between the vanes. The spurs along the bank opposite the alluvial fan would also likely provide the added benefit of bank protection, especially if designed with an upstream orientation that would direct overtopping flows away from the bank. The top of the spur should be either flat or have a mild downward slope (~50H:1V) away from the bank, and should be designed to be overtopped at relatively low discharges (less than 1,000 cfs).

The third non-sediment removal alternative that could be considered at a number of the sites where sedimentation has resulted in the formation of large, vegetated islands or bars involves mechanical destabilization of the features and vegetation removal. The islands and bars create backwater, reduce conveyance efficiency and induce upstream deposition. Vegetation on the bars creates hydraulic roughness that results in further deposition and continued growth of the bars. Bar growth has been especially rapid during the recent, ongoing drought period. Without high flows, natural scour and vegetation removal is unlikely, so mechanical destabilization of the bar surfaces and removal of the vegetation would be necessary to promote scour. It is important to note that this alternative does not consider mechanical excavation of the bar materials, since that action is considered as part of the short and long excavation (sediment removal) alternatives. Instead, this alternative involves clearing and grubbing, leaving the majority of the bar/island sediment in place. Considering the unknowns regarding future hydrologic conditions and the potential for the current drought to persist over many years, the maintenance activities may need to be performed somewhat frequently.

A number of site-specific alternatives were identified in the Statement of Work or developed during the course of this study that included:

- Modifications to the Tierra Blanca Vortex Weir (Problem Location 1). The weir could be modified to remove the central portion of the crest which would allow incision through the weir, thereby reducing the potential for upstream deposition. The remaining portion of the weir would likely trap coarser bed materials in a manner similar to the low-elevation vanes discussed above but allow the sand-size bed material to pass downstream. Periodic maintenance to remove the trapped material may be necessary.

- Modifications to the Rincon Siphon and grade-control structure (Problem Location 3). The sheet pile and rock riprap grade-control structure at the Rincon Siphon (Problem Location 3) appears to result in significant deposition of sediment upstream from the siphon, including material delivered by Garcia Arroyo and the upstream sediment supply. Removal of the grade-control structure and replacement of the siphon with a flume crossing would probably reduce the amount of deposition at and upstream from this site. It is recognized that the construction costs associated with this alternative would be very high; nevertheless this alternative was considered due to the relatively significant effects of the grade-control structure on hydraulic and sediment-transport conditions at and upstream from the siphon.

- Construction of low-elevation spur dikes to narrow the channel in the vicinity of the Rincon/Tonuco Drain and to protect the east bank and control aggradation at Rock Canyon (Problem Location 5). The channel expansion that occurs just upstream from the Rincon/Tonuco Drain results in reduced energies and flow deposition at and downstream from the mouth of the drain. Construction of a series of spur dikes may result in deposition of material between the dikes and the ultimate formation of an inset floodplain. The constricted channel in between the nose of the spur dikes and the opposite bank would likely more efficiently convey the upstream sediment supply, thereby reducing sediment deposition. Construction of vanes along the east bank at and downstream from the confluence with Rock Canyon would likely not only protect this bank from further erosion but also assist in controlling the erosion and deposition of the gravel materials delivered by Rock Canyon, thereby reducing downstream deposition.
- Installation of sluice structures on the Westside and Eastside Main Canals at the Mesilla Dam site (Problem Location 6). These sluice structures were outlined as an alternative in the Statement of Work and initially proposed in the Elephant Butte Irrigation District's (EBID) River Sediment Management Alternatives (EBID, 2014), and would be constructed as outlined in that document. The structures would be installed at the canal headings and include a check structure and sluicing structure designed to capture sediment for return to the river.
- Installation of additional automated gate operators at Mesilla Dam (Problem Location 6). This alternative was outlined as an alternative in the Statement of Work and initially proposed in EBID's River Sediment Management Alternatives (EBID, 2014), and would be constructed as outlined in that document. Currently, Mesilla Dam Gates 2 and 12 are automated for purposes of sluicing sediment that accumulates upstream from the dam, and have proven effective at maintaining head and flow control into the canals. However, because the automated gates are located near the lateral extents of the dam and are the most frequently operated, aggradation tends to occur along the middle nine gates. EBID has therefore proposed to install additional automated gate operators at Gates 5 and 9 to aid in sluicing sediment during times of automatic control.
- Installation of a vortex tube or similar sediment removal system in both the Eastside Main Canal and Westside Main Canal (Problem Location 6). Vortex tubes are devices designed to extract sediment from canals and consist of a tube laid horizontally across the canal bed with an intake slot along the top longitudinal edge. Flow and sediment from near the canal bed is drawn into the system by the vortex that forms inside the tube. The ejected sediment could be returned to the river, or delivered to a settling basin that would allow for clear water return to the canal. Vortex tubes are relatively inexpensive, easy to design and construct, and have proven effective at removing sediment for a wide range of applications. Although sediment ejection systems such as Streamside Technology's Sediment Collector™ (discussed above) were ruled out as potential alternatives in the RGCP, these types of systems may be effective in the canals. However, because costs associated with the pumping of sediment from the collector's hopper would likely be significant, the vortex tube alternative appears to be the most viable alternative.
- Installation of 2,300 feet of riprap revetment in the vicinity of Country Club Bridge (Problem Location 8; identified as a required alternative in Statement of Work). Although no evidence of recent bank erosion was identified during the field reconnaissance of Problem Location 8 and a review of aerial photography indicates that there has been no significant bank erosion since at least 1991, the right (west) bank of the river is very close to the east levee along the reach between Sta 385+00 and Sta 408+00. As such, about 2,300 feet of riprap revetment would be required to protect the levee at this location.

- Construction of spur dikes to narrow the channel between Anapra Bridge and the Montoya Drain with island/bar destabilization and vegetation removal (Problem Location 9). The channel expansion that occurs downstream from Anapra Bridge results in reduced energies and flow deposition at and upstream from the drain. Construction of a series of spur dikes would probably result in deposition of material between the dikes and the ultimate formation of a floodplain. The constricted channel in between the nose of the spur dikes and the opposite bank would tend to more efficiently convey the upstream sediment supply, thereby reducing sediment deposition. Because many vegetated islands and bars have formed along the spur reach, island and bar destabilization and vegetation removal was included as part of this alternative.

4.6 Site-specific Alternative Design Variations

Although many of the alternatives that were evaluated at numerous problem locations were similar in form and the general application would not vary from location to location, there are subtle differences between the implementation techniques for a specified alternative at each location. Specific elements of the design for each of the alternatives that were necessary to develop the modeling and to prepare the cost estimates are summarized in the following sections.

4.6.1 Sediment Removal CMAs

The short and long channel excavation alternatives were included at each of the nine problem locations, and the localized channel excavation alternative was evaluated at all of the problem locations except Problem Location 6 (Mesilla Dam Site). At the problem locations where localized channel excavation was considered, it was assumed that this excavation would also be included in the short and long channel excavation alternatives, essentially resulting in a compound channel excavation that cuts the short and long channel through the localized excavation section. For each of the excavation alternatives, it was assumed that the excavated material would be hauled off site. Mapping showing the extents of the excavations along with comparative longitudinal thalweg profiles and typical cross sections showing the channel bed under existing and with-excavation conditions are presented in Appendix G. **Table 8** summarizes the excavation lengths, average excavation bottom width and depth, and excavated volume for each of the excavation alternatives, by problem location.

At Problem Location 1, localized excavation was evaluated at the mouth of Tierra Blanca Creek/Green Arroyo and at the mouth of Sibley Arroyo. Localized excavation at the mouths of Tierra Blanca Creek/Green Arroyo would occur over a length of 130 feet and would remove 1,570 cubic yards (CY) of material and would occur over 120 feet at the mouth of Sibley Arroyo, removing 4,180 CY. Short excavation at Tierra Blanca Creek/Green Arroyo would extend from Sta 5269+30 to Sta 5278+30 over a distance of 900 feet, removing 7,250 CY of material. At Sibley Arroyo, short excavation would extend from Sta 5179+20 to Sta 5193+10 over a distance of about 1,400 feet, removing 13,300 CY of material. Long excavation at Tierra Blanca Creek/Green Arroyo are limited by the Tierra Blanca Vortex Weir and would extend from Sta 5249+20 to Sta 5288+50 over a distance of 3,400 feet, removing about 21,780 CY of material. At Sibley Arroyo, long excavation would extend from Sta 5164+00 to Sta 5204+20 over a distance of 4,020 feet, removing 26,740 CY of material. No vegetated islands or bars are present at Problem Location 1, so none of the excavation alternatives would require cuts through these types of features.

At Problem Location 2, localized excavation was evaluated at three locations, including the mouths of Thurman II Arroyo, Thurman I Arroyo and Placitas Arroyo, removing about 30,160 CY, 8,340 CY and 7,680 CY of material, respectively. Localized excavation at the mouth of Thurman II Arroyo would remove the vegetated island that has formed downstream from this arroyo at Sta

Table 8. Summary of excavation parameters for the sediment-removal alternatives at each of the nine problem locations.

Problem Location	Designation	Alternative	Vicinity	Excavated Length (ft)	Avg. Excavated Depth (ft)	Avg. Excavated Width (ft)	Excavated Volume (CY)	Comment
1	Tierra Blanca Creek to Sibley Arroyo	Channel Excavation Short	Tierra Blanca Creek	900	3.5	60	7,250	-
			Sibley Arroyo	1,400	2.7	90	13,300	-
			Total/Average*	**2,300**	**3.1**	**75**	**20,550**	-
		Channel Excavation Long	Tierra Blanca Creek	3,420	4.5	60	21,780	Excavated length limited by vortex weir.
			Sibley Arroyo	4,020	3.4	60	26,740	-
			Total/Average*	**7,440**	**4.0**	**60**	**48,520**	-
		Localized Sediment Removal	Tierra Blanca Creek	130	2.8	120	1,570	-
			Sibley Arroyo	120	2.9	140	4,180	-
			Total/Average*	**250**	**2.9**	**130**	**5,750**	-
2	Salem Bridge to Placitas Arroyo	Channel Excavation Short	Thurman Arroyos I&II	3,400	2.8	250	71,580	Continuous excavated channel.
			Placitas Arroyo	1,190	2.5	80	13,000	-
			Total/Average*	**4,590**	**2.7**	**165**	**84,580**	-
		Channel Excavation Long	Thurman to Placitas	**9,500**	**3.0**	**110**	**126,890**	Continuous excavated channel.
		Localized Sediment Removal	Thurman II Arroyo	390	3.0	240	30,160	-
			Thurman I Arroyo	490	2.5	200	8,340	-
			Placitas Arroyo	190	2.6	220	7,680	-
			Total/Average*	**1,070**	**2.7**	**220**	**46,180**	-
3	Rincon Restoration Site A to Rincon Siphon	Channel Excavation Short	Garcia Arroyo	2,280	2.1	90	17,220	Excavated length limited by siphon grade control structure and bridge ROW.
		Channel Excavation Long	Garcia Arroyo	3,780	2.5	110	36,370	Extended excavation u/s from arroyo.
		Localized Sediment Removal	Garcia Arroyo	410	2.5	150	11,330	-

*Total length, average depth, average top width.

Table 8. Summary of excavation parameters for the sediment-removal alternatives at each of the nine problem locations (con't).

Problem Location	Designation	Alternative	Vicinity	Exca-vated Length (ft)	Avg. Exca-vated Depth (ft)	Avg. Exca-vated Width (ft)	Exca-vated Volume (CY)	Comment
4	Rincon Arroyo to Bignell Arroyo	Channel Excavation Short	Rincon Arroyo	1,230	3.4	110	24,690	-
			Reed Arroyo	1,280	3.2	60	16,050	-
			Bignell Arroyo	1,180	4.0	40	24,550	
			Total/Average*	**3,690**	**3.5**	**70**	**65,290**	-
		Channel Excavation Long	Rincon/Reed Arroyos	6,210	3.9	60	66,940	Continuous excavated channel.
			Bignell Arroyo	9,940	4.7	60	154,650	
			Total/Average*	**16,150**	**4.3**	**60**	**221,590**	-
		Localized Sediment Removal	Rincon Arroyo	220	3.6	220	13,200	-
			Reed Arroyo	250	2.7	160	6,730	-
			Bignell Arroyo	280	3.9	200	18,110	-
			Total/Average*	**750**	**3.4**	**193**	**38,040**	-
5	Rock Canyon to below Rincon/ Tonuco Drain Outlet	Channel Excavation Short	Rock Canyon	1,500	4.2	90	44,950	-
			Rincon/Tonuco Drain	1,490	3.5	60	55,970	-
			Total/Average*	**2,990**	**3.9**	**75**	**100,920**	-
		Channel Excavation Long	Rock Canyon	6,000	4.5	60	114,430	-
			Rincon/Tonuco Drain	7,480	3.9	80	107,450	-
			Total/Average*	**13,480**	**4.2**	**70**	**221,880**	-
		Localized Sediment Removal	Rock Canyon	500	5.0	270	24,670	-
			Rincon/Tonuco Drain	500	3.5	360	46,570	-
			Total/Average*	**1,000**	**4.3**	**315**	**71,240**	-
6	Picacho Drain to Below Mesilla Dam	Channel Excavation Short	Mesilla Dam	4,710	3.9	50	35,540	-
		Channel Excavation Long	Mesilla Dam	8,860	4.1	50	58,170	-

*Total length, average depth, average top width.

Table 8. Summary of excavation parameters for the sediment-removal alternatives at each of the nine problem locations (con't).

Problem Location	Designation	Alternative	Vicinity	Exca-vated Length (ft)	Avg. Exca-vated Depth (ft)	Avg. Exca-vated Width (ft)	Exca-vated Volume (CY)	Comment
7	East Drain to Below Vinton Bridge	Channel Excavation Short	East Drain	1,880	4.2	120	22,720	-
			Vinton Bridge	1,660	4.2	80	11,470	Includes east bank tributary above bridge.
			Unnamed Lower Trib.	1,490	4.1	40	3,860	
			Total/Average*	**5,030**	**4.2**	**80**	**38,050**	
		Channel Excavation Long	East Drain to Vinton Brg	4,730	4.4	70	33,410	Includes east bank tributary above bridge.
			Below Vinton Brg	4,190	4.2	50	14,750	Includes east bank tributary below bridge.
			Total/Average*	**8,920**	**4.3**	**60**	**48,160**	
		Localized Sediment Removal	East Drain	450	3.4	200	4,330	-
8	Above Country Club Bridge to Nemexas Siphon	Channel Excavation Short	Country Club Bridge	1,620	4.4	70	21,520	-
		Channel Excavation Long	Country Club Bridge	5,970	4.5	60	43,000	Above Bridge to Nemexas Siphon
		Localized Sediment Removal	Country Club Bridge	370	4.2	300	8,770	-
9	Montoya Drain to American Dam	Channel Excavation Short	Montoya Drain	2,600	2.4	280	38,130	-
		Channel Excavation Long	Montoya Drain	11,530	3.7	100	176,250	Excavated downstream to American Dam.
		Localized Sediment Removal	Montoya Drain	210	2.5	200	15,650	-

*Total length, average depth, average top width.

4541+70. Short excavation at Thurman I and Thurman II Arroyos extends from Sta 4545+60 to Sta 4511+60 over a distance of 3,420 feet, removing 84,580 CY of material. Short excavation at Placitas Arroyo extends from Sta 4471+50 to Sta 4483+40 over a distance of 1,190 feet and removes 76,370 CY of material. Long excavation that would encompass the fans for all three arroyos extends from Sta 4459+10 to Sta 4554+20 over a distance of 9,520 feet, removing 102,000 CY of material. Both the short and long excavations cut through numerous vegetated islands and bank-attached bars.

Localized excavation at Problem Location 3 would be focused on the Garcia Arroyo alluvial fan, excavating about 11,330 CY of material over a distance of about 400 feet. For the short and long excavation alternatives, it was assumed that the excavation could not occur within the Rincon Siphon ROW nor within the ROW of the bridges, so the downstream limit of the excavations was set a short distance upstream from the BNSF Railroad Bridge. Short excavation would extend from Sta 4333+60 to Sta 4356+40 over a distance of about 2,280 feet, removing about 17,220 CY of material. Long excavation would extend from Sta 4333+60 to Sta 4371+40 over a distance of about 3,780 feet, removing about 36,370 CY of material. excavation alternative Although there are no significant vegetated islands at Problem Location 3, all of the excavation alternatives would involve a cut through the bank-attached bar that has formed along the left bank downstream from Garcia Arroyo.

At Problem Location 4, localized excavation was evaluated at the mouths of Rincon Arroyo, Reed Arroyo and Bignell Arroyo, and would result in removal of 13,200 CY, 6,730 CY and 18,110 CY of material, respectively. Localized excavation would involve removal of the bank-attached bars below each of the arroyos. Short excavation at Rincon Arroyo would extend from Sta 4146+00 to Sta 4158+30 over a distance of 1,230 feet and would remove 24,690 CY of material. Short excavation at Reed Arroyo would extend from Sta 4115+00 to Sta 4127+80 over a distance of 1,280 feet and remove 16,050 CY of material. Short excavation at Bignell Arroyo would extend from Sta 3996+80 to Sta 4008+60 over a distance of 1,180 feet and remove 24,550 CY of material. Long excavation would extend from above Rincon Arroyo to below Bignell Arroyo and would be steeper and shallower in the upstream reach (Sta 4098+70 to Sta 4158+30) than in the downstream reach (Sta 3996+80 to Sta 4093+70). Both the short and long excavations would involve cuts through numerous vegetated islands and bank-attached bars.

At Problem Location 5, localized excavation was evaluated at the mouth of Rock Canyon and at the mouth of the Rincon/Tonuco Drain, removing about 500 CY of material at both locations. Localized excavation would remove significant portions of the alluvial fan and bank-attached bar at the mouth of Rock Canyon. Localized excavation at the mouth of the Rincon/Tonuco Drain would also include removal of fan materials delivered by Horse Canyon Creek and the large expansion bar that has formed at this location. Short excavation would remove about 44,950 CY of material at Rock Canyon from Sta 3758+30 to Sta 3773+30 (1,500 feet) and would remove about 55,970 CY of material from the Rincon/Tonuco Drain area from Sta 3708+50 to Sta 3723+40 (1,490 feet). Short and long excavations would involve cuts through some of the recently formed islands that have relatively young vegetation.

Short and long excavation alternatives were evaluated at Problem Location 6 and would involve an excavated channel that starts at Mesilla Dam and extend some distance upstream. Localized excavation was not considered at this location. Short excavation at this site would extend from Sta 2077+60 to Sta 2124+70 (4,710 feet) and remove 35,540 CY of material. Long excavation would cover the same area as the short excavation but would be somewhat deeper and extend farther upstream to Sta 2166+20 (8,860 feet), increasing the volume of excavated sediment to 58,170 CY. No significant vegetated islands or bank-attached bars are present in the modeled reach above the dam, so the short and long excavation alternatives would not involve any island/bar cuts.

Localized excavation at Problem Location 7 was evaluated at the mouth of the East Drain, where about 4,330 CY would be removed. Short excavation reaches extended from Sta 784+30 to Sta 799+20 (1,490 feet) at the downstream unnamed tributary, from Sta 827+90 to Sta 844+60 (1,660 feet) in the vicinity of Vinton Bridge, and from Sta 854+60 to Sta 873+40 (1,880 feet) at the mouth of the East Drain. At these three locations, short excavation would remove 3,860 CY, 11,470 CY and 22,720 CY, respectively. Short excavation would include cuts through the recently formed mid-channel bar with young vegetation near the mouth of the East Drain and the alluvial fans from the arroyos at Sta 786+00 and Sta 834+00. Long excavation would extend along nearly the entire Problem Location 7 reach from Sta 784+30 to Sta 873+40 over a distance of 8,920 feet. The downstream portion of the long excavation would be steeper and narrower than the upper reach, extending from Sta 784+30 to Sta 826+10 (4,190 feet) and removing 14,750 CY of material. The upstream portion of the long excavation extend from Sta 826+10 to Sta 873+40 (4,730 feet), removing 33,410 CY of material. Long excavation would include cuts through four recently formed mid-channel bars with young vegetation and the alluvial fans at all four tributaries that enter the RGCP within this problem location.

At Problem Location 8, localized excavation was evaluated over about 370 feet at Country Club Bridge, removing 8,770 CY of material. Short excavation would extend from Sta 406+70 to Sta 422+90 (1,620 feet), removing about 21,520 CY, and include a cut through the vegetated island downstream from Country Club Bridge. Long excavation would extend from Sta 382+50 to Sta 442+20 (5,670 feet), removing about 43,000 CY of material, and would also include a cut through the island downstream from Country Club Bridge.

Localized excavation at Problem Location 9 that would remove about 15,650 CY of material was evaluated over about 210 feet at the mouth of Montoya Drain. The localized excavation would include complete removal of the vegetated bar that has formed at the mouth of the drain. Short excavation would extend from the mouth of the Montoya Drain at Sta 88+60 to the upstream face of Courchesne Bridge at Sta 114+60 (2,600 feet) and remove about 38,130 CY of material. Short excavation would cut through a number of vegetated and non-vegetated mid-channel and bank-attached bars. Long excavation would extend from a short distance upstream of American Dam at Sta 7+00 to a short distance above Montoya Drain at Sta 122+40 (11,530 feet) and remove 176,250 CY of material and cut through many vegetated and non-vegetated mid-channel and bank-attached bars.

4.6.2 Arroyo Sediment Traps

Arroyo sediment traps were evaluated at Problem Locations 1, 2, 3, 5 and 7. In general, each of the traps are similar in design (Figure 23). Conceptual layouts of the traps are presented in **Appendix H**. All of the sediment traps would include a series of trapping features (rock check structures, piles or fence screens) designed to trap the coarse material and allow a portion of the finer (sand, silt and clay) fractions to pass to reduce maintenance and manage the most problematic coarse material. [During the recommended adaptive management testing of the sediment traps discussed below, if it is determined that it is desirable to also eliminate the finer fractions of the tributary bed material sediment supply, the trap screens could be re-designed to also trap the sand, silt, and clay classes. However, because the fine (sand, silt and clay) sediment loads delivered by the tributaries represents a relatively small portion of the overall fine sediment loads supplied and transported by the RGCP, this may not be advisable.] All of the sediment traps would also include an embayment at the downstream end connecting to the Rio Grande. A debris rack would be necessary at the upstream entrance to the trap to capture floating debris that could affect the performance of the trapping features (**Figure 25**). The sediment traps were

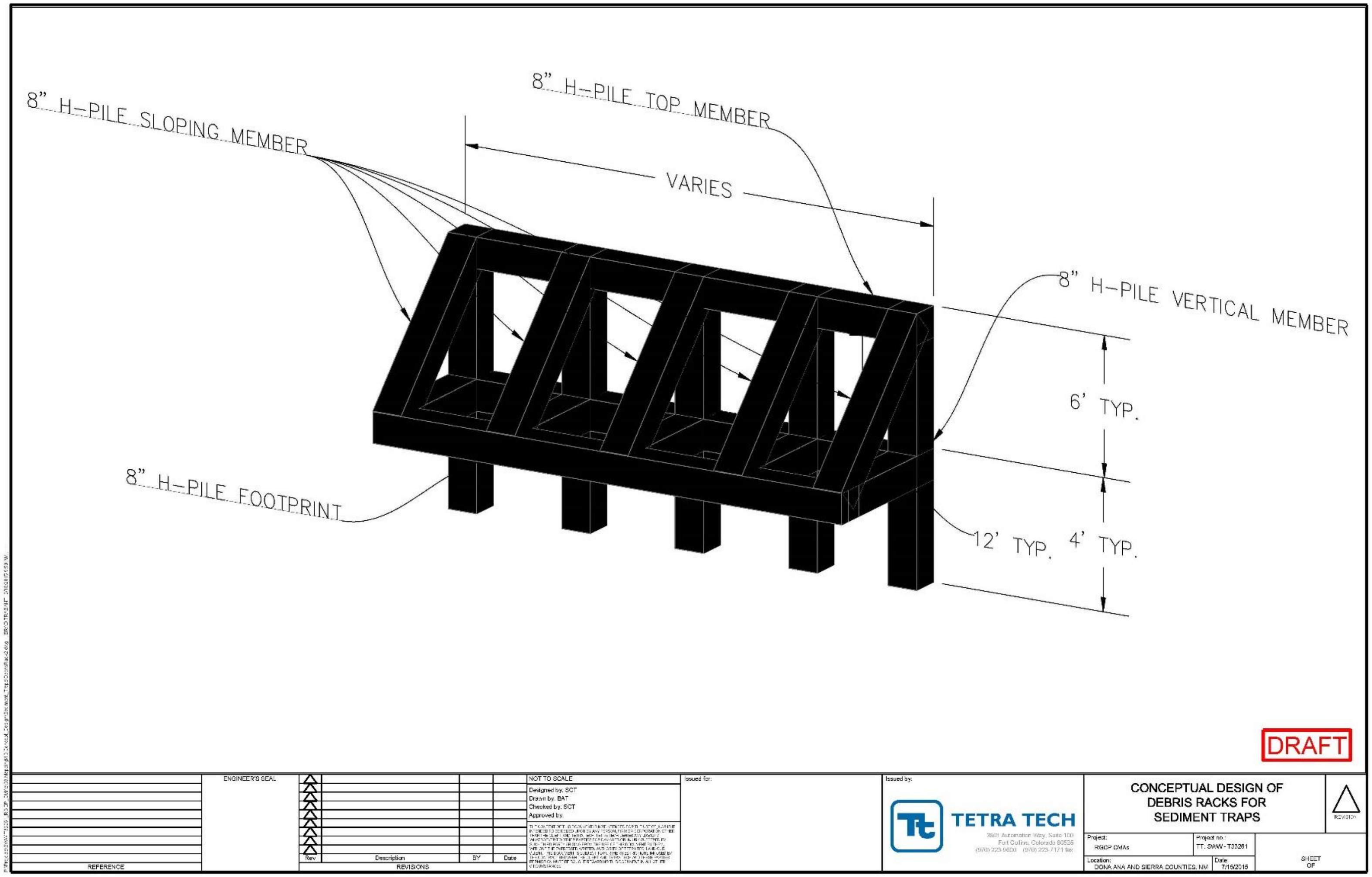

Figure 25. Conceptual 3-D rendering of the debris racks proposed for the sediment trap alternatives.

designed to have a trapping volume that exceeds the average annual bed-load yield from the tributaries (USACE, 2007; **Table 9**)[3]. Where possible, the footprint of the trap was laid out within the USIBWC ROW to avoid the need to purchase private property. Surface areas of the traps ranged from 0.3 acres to 4.4 acres, and the average depth of the trap ranged from 3 feet to 4 feet (**Table 10**). At many locations, the resulting trap volume exceeds the total average annual sediment load (Table 10). Most of the sediment traps would require excavation into the floodplain to obtain the required volume.

For this study, a number of the design parameters were assumed. Design of the specific type of trapping feature is beyond the scope of this study and would need to consider at a minimum the stability of the structures, sediment-trap efficiency by size fraction, debris collection mitigation, and cost. It was assumed for this study that the trapping features would be constructed with rebar and wire screens with progressively finer mesh openings in the downstream direction. The fence screens would be constructed by driving 3-inch angle iron “fence posts” to a depth of 3 feet with a spacing of 12 feet, and depending on the location extend between 3 to 4 feet above the floor of the trap. Angle iron buttressing supports would be required for the fence posts to stabilize the structure. The screens would be constructed by welding #4 rebar (0.5-inch diameter) or 1/8-inch solid-core, galvanized iron wire mesh to the fence posts. Rebar screen fences were used for mesh openings of 6 inches and larger and wire fences were used for openings of 4 inches and smaller. Most of the traps would require a berm to direct flow and sediment into the traps and many would also require construction of an access road for maintenance purposes. The berm would be constructed to a height of 3 feet above the existing grade and be constructed using spoil material from the excavations. The inside face of the berm would require rock slope protection and it was assumed that the stone could be derived from materials excavated from the traps. It was also assumed that no maintenance would be required in the excavated embayments since the mostly fine material deposited in this portion of the trap would likely be flushed by the eddy that would tend to occur in the embayments. At locations where material has deposited in the river at the location of the embayment, it may be necessary to excavate a small pilot channel to connect the embayment to the river bed; however, because the extent and layout of the bars at the time of construction it is not known, this aspect of the design was not considered. It should be noted that the ROW information (provided by USIBWC) may not be correct or up-to-date at some of the problem locations; best judgement was used in these areas.

At Problem Location 1, sediment-traps were evaluated at the mouths of Tierra Blanca Creek, Green Arroyo, and Sibley Arroyo (Table 10 and Appendix H). All three sediment traps were designed within USIBWC ROW. Considering the relatively coarse sediment that these tributaries deliver, five mesh fences would be required at each of the three traps with mesh openings ranging from 2 inches to 1 foot. To increase the area and volume for the trap at Tierra Blanca Creek and to avoid the need to reroute any existing roads, the trap was oriented along the Rio Grande floodplain in the upstream direction and would therefore require excavation and a 3-foot high berm. This trap has a surface area of 4.4 acres, average depth of 4 feet and volume of 17.7 acre-feet (about the total average annual sediment yield and about 2.7 times the average annual bed-load yield). The trap for Green Arroyo is oriented along the left Rio Grande floodplain in the downstream direction and would therefore require excavation. This trap has a surface area of 2.7 acres, average depth of 4 feet and volume of about 11 acre-feet (about the total average annual sediment yield and about 3 times the average annual bed-load yield). The Sibley Arroyo trap was

[3]The USACE (2007) bed-load yield estimates were based on studies of tributary arroyos in the San Acacia Reach upstream from Elephant Butte Reservoir and the Rectification Reach downstream from El Paso (MEI, 2004 and 2007b) that indicated the average bed-load yield was about 35 percent of the total yield.

Table 9. Summary of mean annual tributary total sediment yield and mean annual tributary bed-load yield for the tributaries considered in this study (from USACE, 2007). Also shown are the corresponding water discharges and dates of the annual events that were assumed for purposes of the sediment-transport modeling.

Problem Location	Watershed Name	Station (ft)	Basin Drainage Area (mi^2)	Mean Annual Sediment Yield (ac-ft)	Mean Annual Bed Load Yield (ac-ft)	Assumed Corres-ponding Flow (cfs)	Assumed Date of Loading
1	Tierra Blanca Creek	5276+50	68.2	18.95	6.63	1,070	7/31
	Green Canyon	5276+00	35.6	10.13	3.55	570	8/31
	Sibley Arroyo	5190+20	27.2	7.99	2.80	450	8/31
2	Thurman II Arroyo	4545+00	6.0	1.98	0.69	210	7/31
	Thurman I Arroyo	4526+50	3.4	1.12	0.39	120	7/31
	Placitas Arroyo	4483+10	34.6	9.87	3.45	520	7/31
3	Garcia Arroyo	4341+00	3.5	1.22	0.43	60	7/31
4	Rincon Arroyo	4154+60	124.7	36.20	12.67	1,900	7/1
	Reed Arroyo	4126+30	9.6	3.52	1.23	180	8/1
	Bignell Arroyo	4006+80	3.9	1.10	0.38	60	9/1
5	Rock Canyon	3770+70	3.7	1.03	0.36	50	7/31
	Horse Canyon	3714+00	3.7	1.04	0.36	60	8/15
6	Subarea 24	2087+50	4.2	1.98	0.69	110	7/31
7	Subarea 101	849+60	2.9	1.55	0.54	80	7/31
	Subarea 102 (U/S)	833+90	4.0	1.66	0.58	90	7/31
	Subarea 102 (D/S)	806+40	2.5	1.02	0.36	50	7/31
	Subarea 103 (U/S)	786+20	3.1	1.37	0.48	70	7/31

Table 10. Summary of Sediment Trap Conceptual Designs.

Problem Location	Tributary Name	Station (ft)	Surface Area (acres)	Average Depth (ft)	Trap Volume (ac-ft)	Percent of Annual Total Yield Trapped	Percent of Annual Bed-load Trapped
1	Tierra Blanca Creek	5276+50	4.4	4	17.7	94%	267%
	Green Canyon	5276+00	2.7	4	11.0	108%	309%
	Sibley Arroyo	5190+20	2.7	3	8.2	102%	292%
2	Thurman II Arroyo	4545+00	1.0	3	2.9	148%	424%
	Thurman I Arroyo	4526+50	1.4	3	4.1	364%	1041%
	Placitas Arroyo	4483+10	3.5	4	14.0	142%	405%
3	Garcia Arroyo	4341+00	0.6	3	1.7	140%	401%
5	Rock Canyon	3770+70	1.7	3	5.2	501%	1430%
	Horse Canyon	3714+00	1.2	3	3.6	347%	992%
7	Subarea 101	849+60	0.3	4	1.0	67%	192%
	Subarea 102 (U/S)	833+90	0.5	4	2.0	118%	337%
	Subarea 102 (D/S)	806+40	1.4	3	4.1	398%	1136%
	Subarea 103 (U/S)	786+20	0.6	3	1.8	129%	369%

also oriented along the Rio Grande floodplain in the upstream direction and would therefore require excavation. This trap has a surface area of 2.7 acres, average depth of 3 feet and volume of about 8.2 acre-feet (about the total average annual sediment yield and about 2.9 times the average annual bed-load yield). Small access roads stemming from the existing roads would be required at each of the trap bays for maintenance purposes at the Tierra Blanca Creek and Green Arroyo traps. The existing road at the Sibley Arroyo trap would need to be rerouted around the north and west side of the trap, and would also require access roads to each trap bay.

Three sediment-traps were also evaluated at Problem Location 2 at the mouths of Thurman II Arroyo, Thurman I Arroyo and Placitas Arroyo (Table 10 and Appendix H). All three sediment traps were designed within USIBWC ROW. Five mesh fences would be required at each of the three traps with mesh openings ranging from 2 inches to 1 foot due to the relatively coarse sediment that these tributaries deliver. The traps for Thurman I and Thurman II Arroyos would be oriented along the existing arroyo alignment so would not require any berms but would require some excavation into the arroyo banks to obtain the desired width. (To avoid hauling and disposal costs, spoil material from the excavations could be used to create a berms that would increase the volume of the traps.) Traps for both arroyos would require access roads stemming from the existing levee road. The trap for Thurman II Arroyo has a surface area of about 1 acre, average depth of 3 feet and volume of about 2.9 acre-feet (about 1.5 times the total average annual sediment yield and about 4 times the average annual bed-load yield). The trap for Thurman I Arroyo has a surface area of about 1.4 acres, average depth of 3 feet and volume of about 4.1 acre-feet (about 1.5 times the total average annual sediment yield and about 4 times the average annual bed-load yield). The trap for Placitas Arroyo would be located within the existing arroyo so no berms would be required and would only require excavation through the embayment portion of the trap. This trap has a surface area of 3.5 acres, average depth of 4 feet and volume of about 14 acre-feet (about 1.4 times the total average annual sediment yield and about 4 times the average annual bed-load yield). Only the embayment portion of this trap would pass through a USIBWC restoration site. Access points to each of the bays for the Placitas trap could stem from the existing roads on either the north or south sides of the arroyo.

A single trap was evaluated at Problem Location 3 for Garcia Arroyo (Table 10 and Appendix H). To increase the area and volume for the trap and to avoid impacts to the USIBWC restoration site, the trap was oriented along the Rio Grande floodplain in the upstream direction and would therefore require excavation and a 3-foot high berm. Only the embayment portion of this trap would pass through USIBWC restoration sites (Rincon A and Rincon C). Although Garcia Arroyo delivers relatively coarse sediments, the available area allows for only four mesh fences with mesh openings ranging from 2 to 8 inches. This trap has a surface area of 0.6 acres, average depth of 3 feet and volume of 1.7 acre-feet (about 1.4 times the total average annual sediment yield and about 4 times the average annual bed-load yield). An access road stemming from the east levee road would need to be constructed for maintenance purposes.

At Problem Location 5, sediment-traps were evaluated at the mouths of Rock Canyon and Horse Canyon Creek (Table 10 and Appendix H). Both sediment traps are outside of USIBWC ROW so acquisition of private property would be necessary. Considering the relatively coarse sediment that are delivered by Rock Canyon, five mesh fences would be required for this trap with mesh openings ranging from 2 inches to 1 foot. This trap generally follows the alignment of Rock Canyon but extends some distance into the Rio Grande floodplain to increase the trap area and volume, and would therefore require excavation and a 3-foot high berm. This trap has a surface area of 1.7 acres, average depth of 3 feet and volume of 5.2 acre-feet (about 5 times the total average annual sediment yield and about 14 times the average annual bed-load yield). Sediments delivered by Horse Canyon Creek are somewhat finer, so only four mesh fences would be used at this trap with mesh openings ranging from 2 to 8 inches. This trap would be constructed within the existing arroyo so berms would not be necessary and excavation would only be required through the downstream embayment. Construction of access roads stemming from Highway 185 would be required at both of the sediment traps.

Four sediment-traps were evaluated at the mouths of the four unnamed left bank arroyos that enter Problem Location 7 (Table 10 and Appendix H). The three contributing watersheds for these four arroyos were identified in Tetra Tech (2004) and in subsequent reports (i.e., USACE, 2007) from upstream to downstream as Subarea 101, Subarea 102 and Subarea 103. Subarea 101 entirely drains to the upstream arroyo (identified as Subarea 101 arroyo). Sixty two percent of Subarea 102 delivers flow and sediment to the next downstream arroyo [identified as Subarea 102 (U/S) arroyo] and 38 percent of this watershed contributes to the third arroyo [identified as Subarea 102 (D/S)]. Only the upstream portion of Subarea 103 delivers flow to the downstream most tributary [identified as Subarea 103 (U/S)], which represents about 59 percent of this watershed. All four sediment traps were designed within USIBWC ROW. Relatively fine sediments are delivered by these tributaries, so four mesh fences would be required at each of these traps with mesh openings ranging from 1 to 6 inches. To increase the area and volume for the trap at the upstream three traps, these traps were oriented along the Rio Grande floodplain and would therefore require excavation and a 3-foot-high berm. The trap for the downstream tributary would be oriented along the existing arroyo alignment so would not require any berms but would require some excavation into the arroyo banks to obtain the desired width. (To avoid hauling and disposal costs, spoil material from the excavations could be used to create a berms that would increase the volume of the traps.) The trap for the Subarea 101 arroyo has a surface area of 0.3 acres, average depth of 4 feet and volume of 1 acre-feet that represents about 70 percent of the total average annual sediment yield and about twice the average annual bed-load yield. The trap for the Subarea 102 (U/S) arroyo has a surface area of 0.5 acres, average depth of 4 feet and volume of 2 acre-feet (about 120 percent of the total average annual sediment yield and about 3.4 times the average annual bed-load yield). The trap for the Subarea 102 (D/S) arroyo has a surface area of 1.4 acres, average depth of 3 feet and volume of 4.1 acre-feet (about 4 times the total average annual sediment yield and about 11 times the average annual bed-load yield). The trap for the Subarea 103 (US) arroyo has a surface area of 0.6 acres, average depth of 3 feet and volume of 1.8 acre-feet (slightly larger than the total average annual sediment yield and about 3.7 times the average annual bed-load yield). For the traps at the Subarea 101 and 102 (U/S) arroyos, it was assumed that maintenance equipment could use the pedestrian trail to access the traps, so relatively short access roads would be required. Construction of access roads stemming from the existing dirt road would be necessary for the trap at the Subarea 102 (D/S) arroyo, and rerouting of the existing dirt road would be required. Construction of a similar access road would also be required for the trap at the Subarea 103 (U/S) arroyo.

4.6.3 Low-elevation Spur Dikes

Low-elevation spur dikes were evaluated at Problem Locations 4, 5, 7, 8 and 9. To ensure the spurs would not have significant negative impacts on levee freeboard, they were designed to have an elevation that is consistent with the 500-cfs water-surface elevation. The spurs are very similar in concept to bendway weirs in that they have low elevations relative to the channel banks and are oriented about 30 degrees in the upstream direction (**Figure 26, Appendix I**). It should be noted that a detailed scour analysis that is beyond the scope of this study would be necessary to determine the spur dimensions, stone size, toe down depth and bank key length that would be required for a stable design. To obtain reasonable estimates for use in the conceptual design, results from the hydraulic modeling at a representative location (near the confluence with Rincon Arroyo at Problem Location 4) were used to prepare bend scour, contraction scour and impinging wall scour calculations and to size the stone for the spurs. The bend scour calculations were performed for a discharge of 3,000 cfs to represent the upper limit of the RGCP conveyance

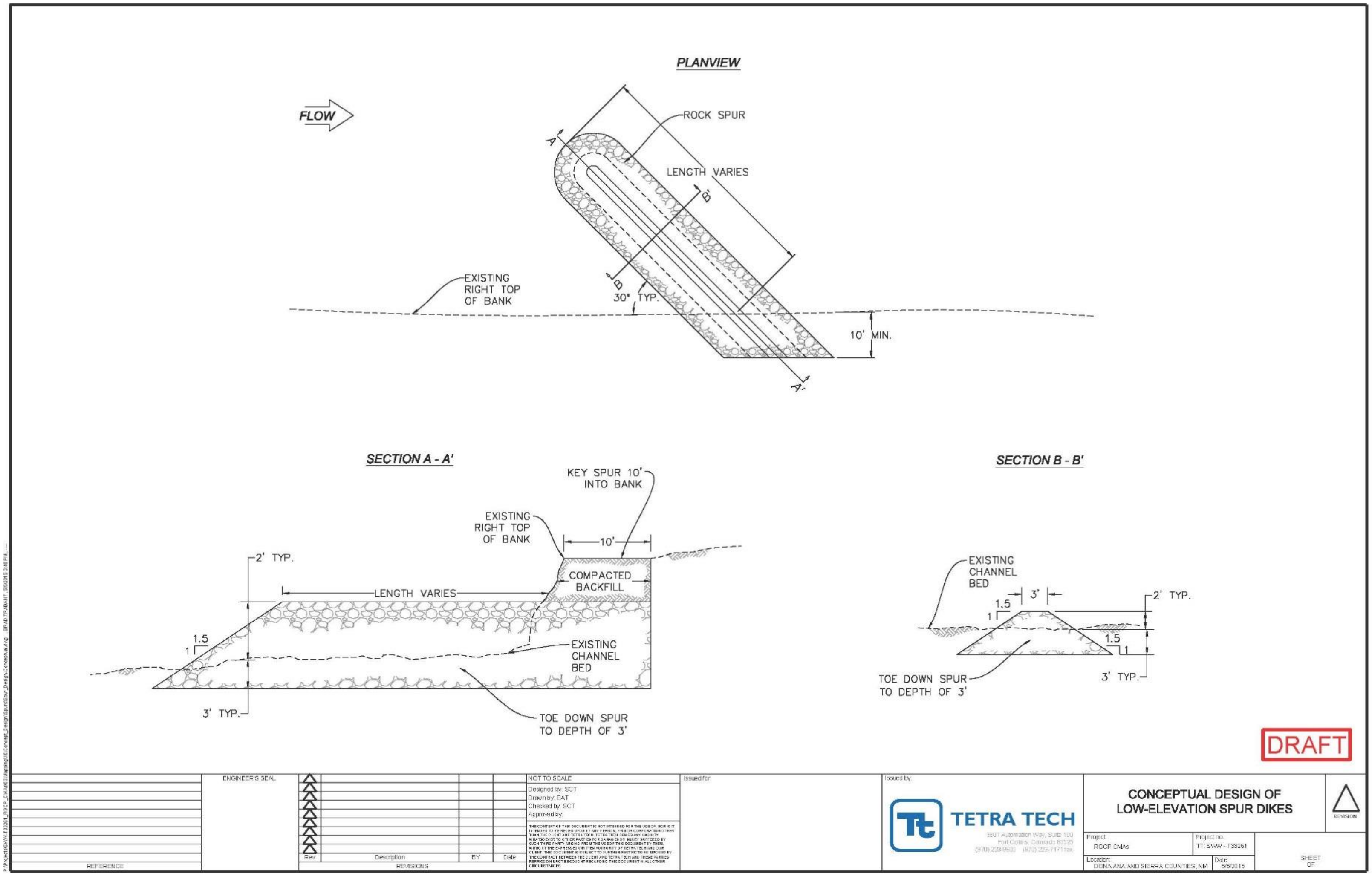

Figure 26. Conceptual plan, profile and section drawings of typical low-elevation spur dike.

capacity and indicate bend scour would range from 0.4 to 0.6 feet. Contraction scour and impinging wall scour calculations were performed at a discharge of 500 cfs since this discharge was used to set the elevation of the spurs. The calculations indicate that contraction scour would be relatively minor (less than 0.2 feet) due to the low spur elevation, and that impinging wall scour would be about 1.8 feet. A spur toe-down depth of 3 feet was therefore used for all spurs. Riprap stone sizing calculations were made using procedures outlined in the U.S. Army Corps of Engineers "Hydraulic Design of Flood Control Channels" (USACE, 1994) and results from the hydraulic modeling at the 100-year routed peak discharge. The calculations indicate the stone should have a median diameter of up to 10 inches; thus, it was conservatively assumed that the spurs would be constructed using 12-inch angular stone. It was also assumed that the spurs would have a top width of 3 feet, would need to be keyed into the banks over a distance of 10 feet, and would have 1.5H:1V sideslopes (Figure 26). A summary of the spur layouts at each site is presented in **Table 11**, and plan view layouts of the spurs are included in Appendix I.

Table 11. Summary of low-elevation spur dike designs.

Problem Location	Number of Spurs	Average Spur Length (ft)	Maximum Spur Length (ft)	Total Length of Spurs (ft)	Average Spur Height (ft)
4	36	65	105	2,340	1.8
5	37	103	727	3,820	1.6
7	36	68	101	2,450	1.9
8	21	57	107	1,200	1.9
9	14	72	117	1,010	1.6

Three separate groupings of spurs were evaluated at Problem Location 4. The spur grouping in the vicinity of Rincon Arroyo were laid out to protect the eroding bank opposite the alluvial fan and to evaluate erosion of the fan materials and downstream bars. At Reed Arroyo, spurs were located along the right bank upstream from the arroyo to prevent further bank erosion at this location, and along the left bank at and downstream from the alluvial fan. The spurs in the vicinity of Bignell Arroyo were laid out along the right bank opposite the fan and along the left bank downstream from the arroyo to evaluate the potential for eroding the bank-attached bar that has formed below the fan. Separate groupings were evaluated at this location to reduce construction costs and to assess the effects of the spurs on downstream sediment loading in the intermediate areas.

Two separate groupings of spurs were evaluated at Problem Location 5. The upstream grouping extends over a distance of about 3,200 feet from the confluence with Rock Canyon through the downstream expansion zone. This grouping was laid out to assess the potential for eroding material in the Rock Canyon fan and to determine if the spurs would reduce aggradation and bar formation in the expansion zone. The downstream grouping is located in the vicinity of the Rincon/Tonuco Drain and the confluence with Horse Canyon Creek and was designed to erode sediment that has accumulated in the abrupt expansion zone at this location (**Figure 27**). To ensure that the spurs do not affect drain efficiency, a longitudinal dike was employed that would extend from the left bank upstream from the drain over a distance of about 800 feet through the widest section of the expansion. It was assumed that the longitudinal dike would have a height that is consistent with the left bank (about 5 feet in height) to reduce or eliminate sedimentation behind the dike and below the drain, and would be oriented parallel with the right bank. A series

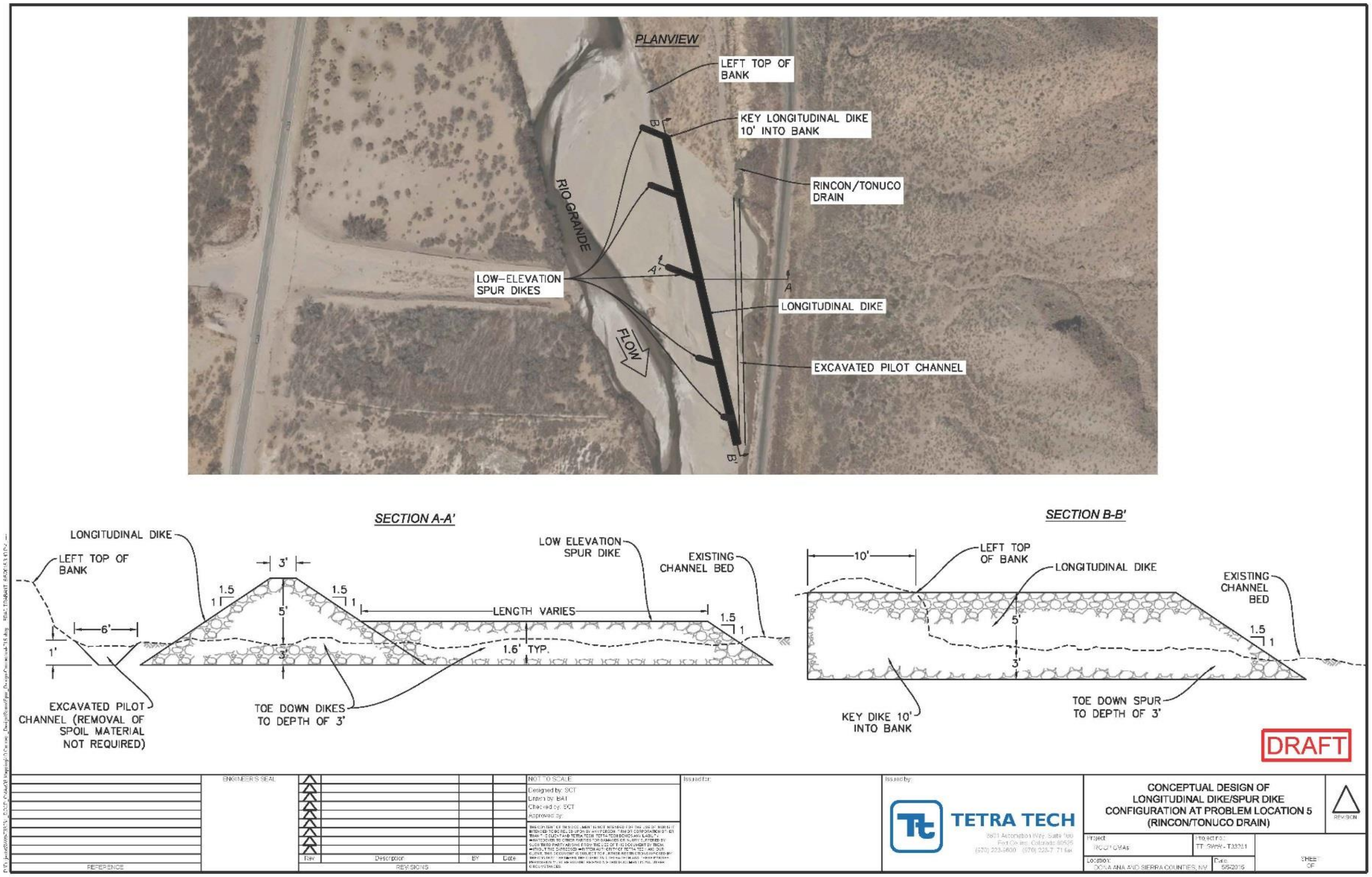

Figure 27. Conceptual plan and section drawings of longitudinal dike/low-elevation spur dike configuration at Problem Location 5 near the mouth of the Rincon/Tonuco Drain.

of five low-elevation spurs that would tie into the longitudinal dike were then laid out to constrict the flows and increase sediment-transport rates along the right side of the channel. Depending on the location of the low-flow channel at the time of construction, it would likely be necessary to excavate a small pilot channel that would extend from the mouth of the drain to the downstream limit of the longitudinal dike for purposes of improving drain efficiency.

The spurs at Problem Location 7 extend along nearly the entire reach from above the mouth of the East Drain at Sta 869+00 to the confluence with the downstream unnamed tributary at Sta 784+00. The upstream three spurs were located along the right bank to increase sediment-transport rates at the mouth of the East Drain. Between the mouth of the drain and the confluence with the upstream unnamed tributary, the spurs were located along the left bank to protect the outside of the bend. The spurs along the remainder of the reach were located on the right bank to evaluate the potential for increased erosion of the alluvial fan materials.

At Problem location 8, spurs were laid out along the left bank upstream from Country Club Bridge along the reach where the left bank is relatively close to the east levee and along the right bank downstream from the bridge for similar purposes of protecting the west levee. Both sets of spurs would also likely also result in increased sediment-transport rates and reduce aggradation in the vicinity of the bridge. Because the spurs along the right bank downstream from the bridge are located on the inside of the bend, riprap bank protection is recommended along the left bank over a distance of about 650 feet to protect the outside of the bend. This riprap was designed in a manner similar to the riprap alternative that was evaluated at this site as discussed in the site-specific alternatives discussed below, and would require about 780 CY of riprap.

Spurs were evaluated at Problem Location 9 along the expansion zone between Anapra Bridge and the mouth of the Montoya Drain, where significant aggradation has resulted in the formation of large, vegetated islands and bank-attached bars. The spurs were laid out over a distance of about 2,800 feet to constrict the channel to a width that is more consistent with the up- and downstream reaches. Because many of the spurs would intersect the islands and bars, and to determine if the spurs could be used to manage island and bar formation, island and bar destabilization and vegetation removal was also considered as part of this alternative along the spur reach. (Details regarding the island/bar destabilization and vegetation removal are discussed below.)

4.6.4 Island and Bar Destabilization with Vegetation Removal

Island and bar destabilization with vegetation removal was evaluated at the problem locations where vegetated islands and bars are a dominant feature (Problem Locations 2, 4 and 9). Because this alternative represents a non-sediment removal alternative, the island and bar destabilization was designed to evaluate the potential for erosion of these features after mechanical removal of the vegetation. (As indicated above, a number of the sediment-removal alternatives included excavation through the islands and bars.) This alternative involves clearing, grubbing, and disposal of herbaceous and woody vegetation from the islands and bars. For purposes of the modeling, it was assumed that the grubbing process would reduce the elevation of the selected features by 6 inches. It was also assumed that the hydraulic roughness would be reduced to the bare-ground roughness in the bounding non-vegetated channels. The islands and bars that were selected for treatment are identified in the mapping presented in **Appendix J**. A total of 16, 13, and 22 features were treated at Problem Locations 2, 4 and 9, respectively. The surface area of the treated features at Problem Locations 2, 4 and 9 was 34.7, 43.9, and 14.5 acres, respectively.

4.6.5 Site-specific Non-sediment Removal Alternatives

Site-specific alternatives identified in the scope of work or identified during the course of this study were evaluated at a number of problem locations. These alternatives are intended to address concerns with specific features that are unique to the problem location reach. The site-specific alternatives include:

- Problem Location 1:
  - Modifications to Tierra Blanca Vortex Weir.
- Problem Location 3:
  - Replace Rincon Siphon with flume crossing.
- Problem Location 6:
  - Installation of check/sluiceway structures in the Eastside and Westside Main Canals.
  - Installation of automated gate controllers for Mesilla Dam Gates 5 and 9 for sluicing purposes.
  - Installation of vortex tube sediment extractors in the Eastside and Westside Main Canals.
- Problem Location 8:
  - Riprap bank protection below Country Club Bridge.

For the Problem Location 1 Tierra Blanca Vortex Weir modifications (**Figure 28**), it was assumed that the middle portion of the weir would be removed over a distance of 30 feet. Because the weir was buried in sediment at the time of the field reconnaissance, the size of rock used along this portion of the weir and the height of the weir are not known. For this study, it was assumed that the rock in this portion of the weir are a single layer of 16-inch diameter boulders (i.e., the boulders are not stacked). As such, the modifications would result in a 30-foot wide by 16-inch deep notch through the weir. Downstream from the weir, the channel gradient is very flat over a distance of about 5,000 feet, so it is recommended that a pilot channel be excavated over this reach to reduce the likelihood for sediment accumulations in the weir notch (**Figure 29**). The pilot channel would be relatively small (8 feet wide by about 1.5 feet deep) and the spoil would be left along the fringes of the channel in the bed of the Rio Grande (Figure 29).

Modifications to the Rincon Siphon at Problem Location 3 involve removal of the sheet pile and rock grade control structure at the siphon and replacing the siphon with a flume crossing over the Rio Grande (**Figure 30**). Removal of the grade control structure would require removal of the two sheetpile walls at the up- and downstream ends of the structure as well as the 4.5-foot deep layer of riprap (about 12,370 CY). Removal of the siphon would require removal of the sheet pile walls along the up- and downstream sides of the siphon, the headwall, and the barrel with concrete casing. It was assumed that a 20-foot wide concrete-box would be used for the flume, so some grading of the canal transitions into and out of the flume would be required. The flume would tie into the canal bed at the east and west limits of the existing siphon and it was assumed that the floor and walls would have a thickness of 10 inches. The inside height of the flume (5.6 feet measured from the floor to the top of the flume) was set to match the existing height of the canal at the western entrance point plus 1 foot of freeboard, so the total height of the flume (including the 10-inch thick floor) would be 6.1 feet.

Three site-specific non-sediment removal alternatives were evaluated at Problem Location 6 in the vicinity of Mesilla Dam. Two of the alternatives were identified by EBID and are discussed in the River Management Alternatives Report (EBID, 2014). These two alternatives include (1) installation of automated gate operators at Mesilla Dam Gates 5 and 9, and (2) installation of check structures with sluiceways in the Eastside and Westside canals. Design considerations associated with these two alternatives are presented in EBID (2014), pertinent excerpts of which are included in **Appendix K**. The third non-sediment removal alternative at this problem location

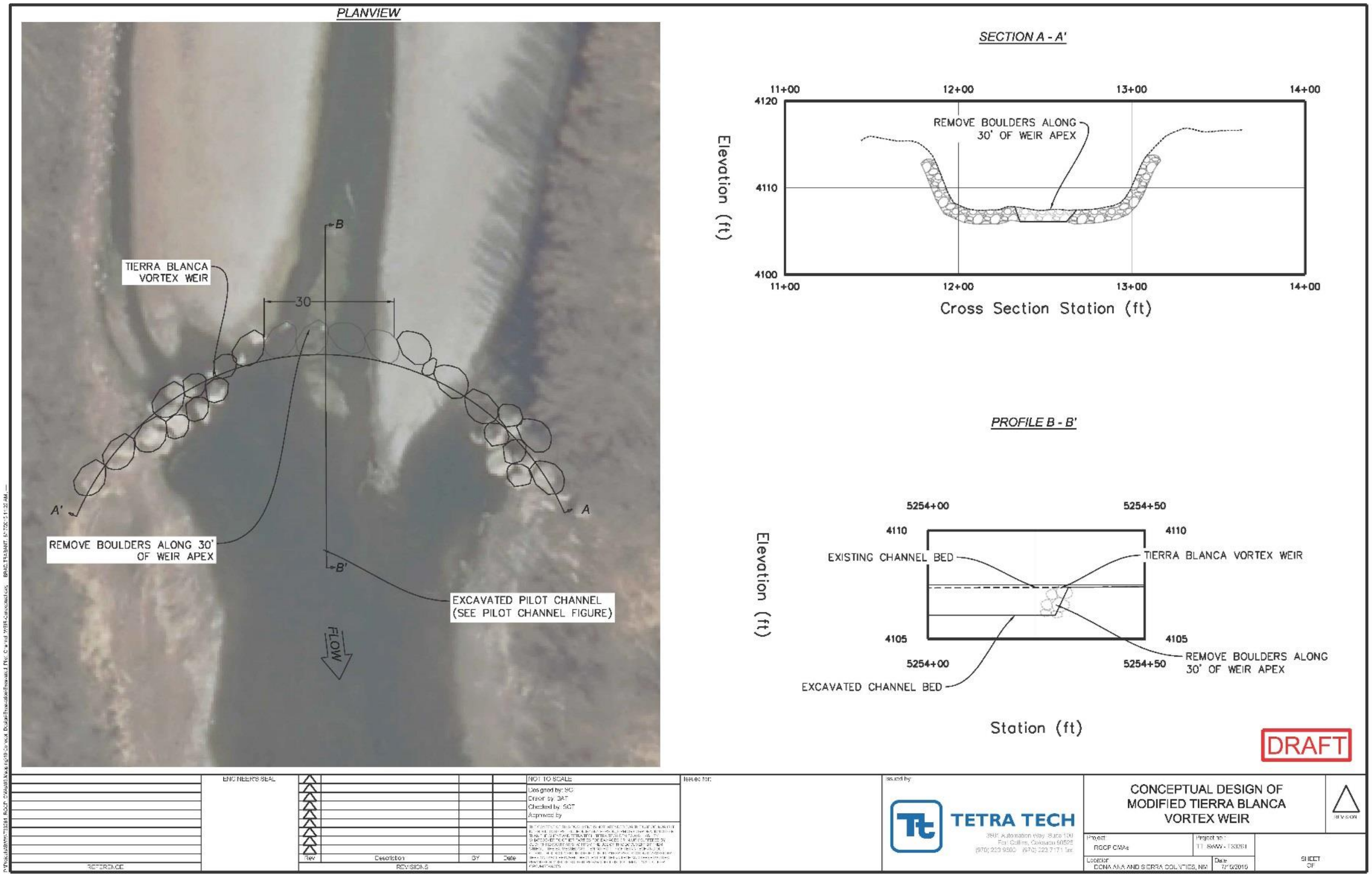

Figure 28. Conceptual layout of the modified Tierra Blanca Vortex Weir as proposed as part of that alternative.

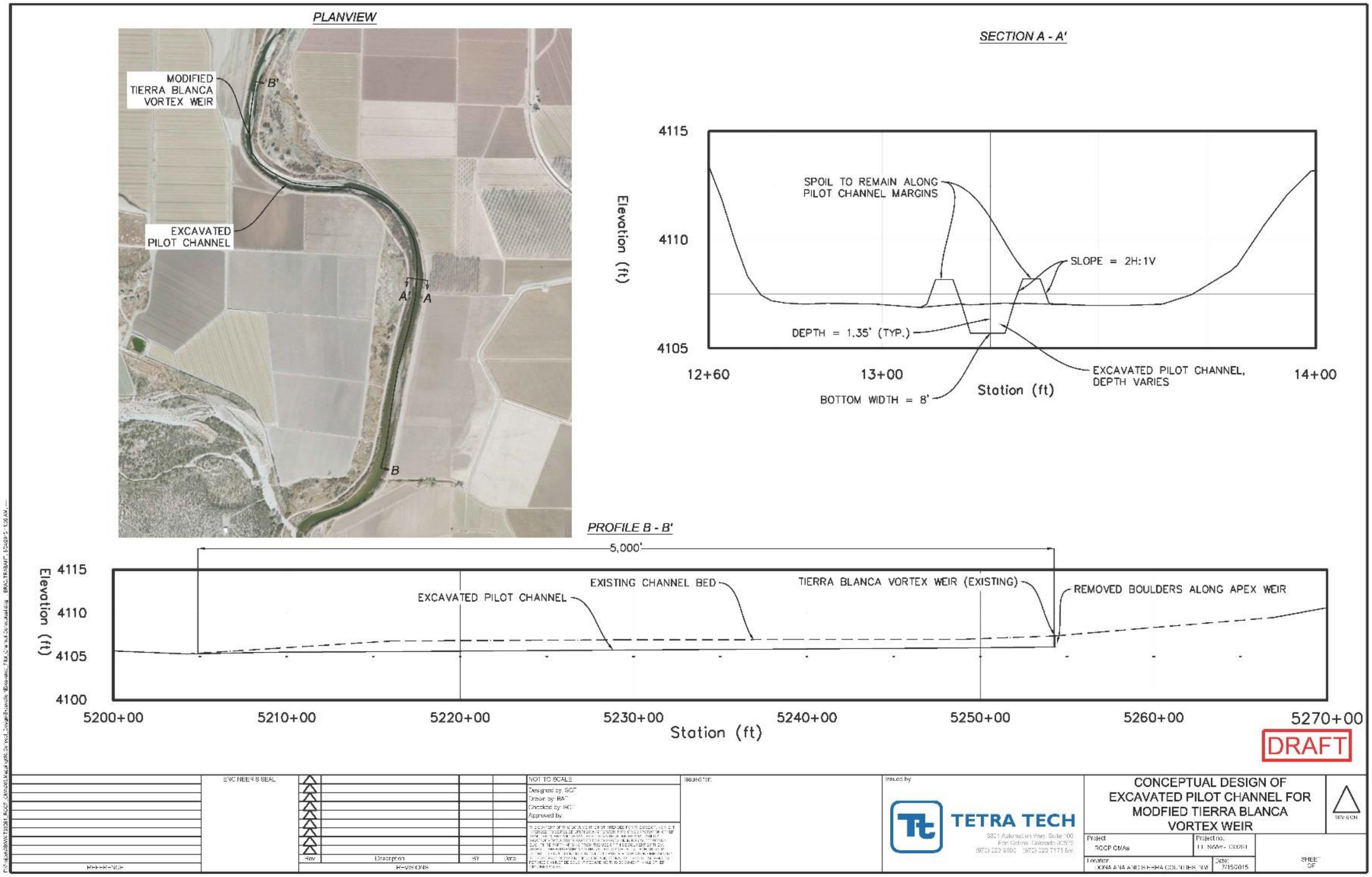

Figure 29. Conceptual layout of the excavated pilot channel downstream from the Tierra Blanca Vortex Weir that is proposed as part of the modified Tierra Blanca Vortex Weir alternative.

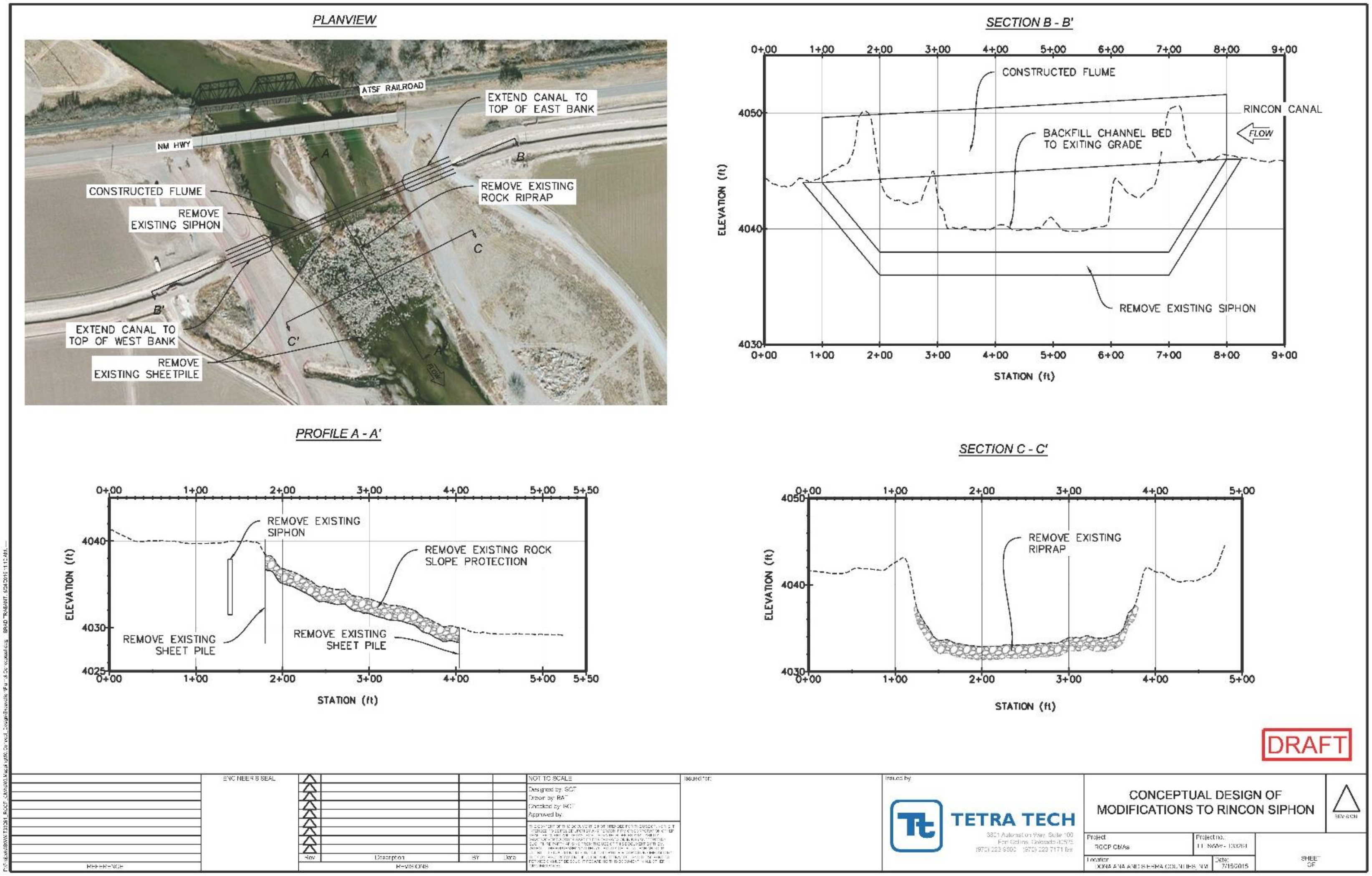

Figure 30. Conceptual layout of the modifications to the Rincon Siphon as proposed under the modified Rincon Siphon alternative.

involves the installation of sediment-ejecting vortex tubes within the bed of the two canals (**Figure 31**). Vortex tubes (Atkinson, 1994a and 1994b) are typically oriented between 45 and 60 degrees with the direction of flow and often are laid out in series. It was assumed for this study that both canals would be outfitted with two 8-inch diameter tubes in series, oriented 60 degrees to the direction of flow. The tubes could be located just upstream from the metering stations to avoid the need to relocate the gages and to limit the distance of the escape channel or tube that would need to extend from the riverward side of the canal to the banks of the Rio Grande (**Figure 32**). It is recommended that the vortex tubes be constructed within a concrete sill and include a control gate to control flow loss during periods of low sediment loading. It was assumed that escape channels would be used in lieu of tubes for maintenance purposes, consisting of a 1-foot deep concrete channel with a bottom width of 16 inches and 1H:1V sideslopes. The escape channels would need to pass through culverts at the existing levee roads, and it was assumed that 1-foot diameter corrugated metal pipes would be used for the culverts. While none of these alternatives would have direct, significant effects on the hydraulic conditions within the Rio Grande, both would likely affect sediment-transport conditions in both the river and the canals. Hydraulic and sediment-transport modeling of these alternatives is discussed below.

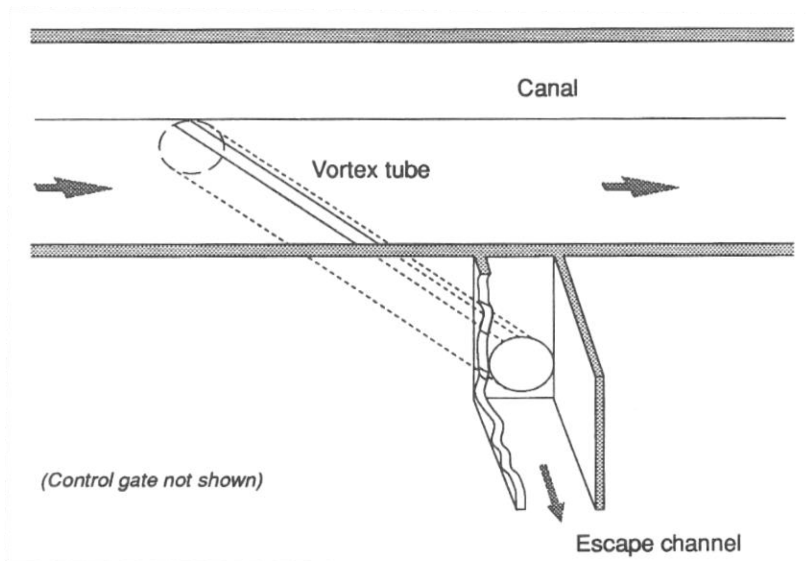

Figure 31. Conceptual 3-D layout of a vortex tube sediment extractor (modified from Atkinson, 1994a).

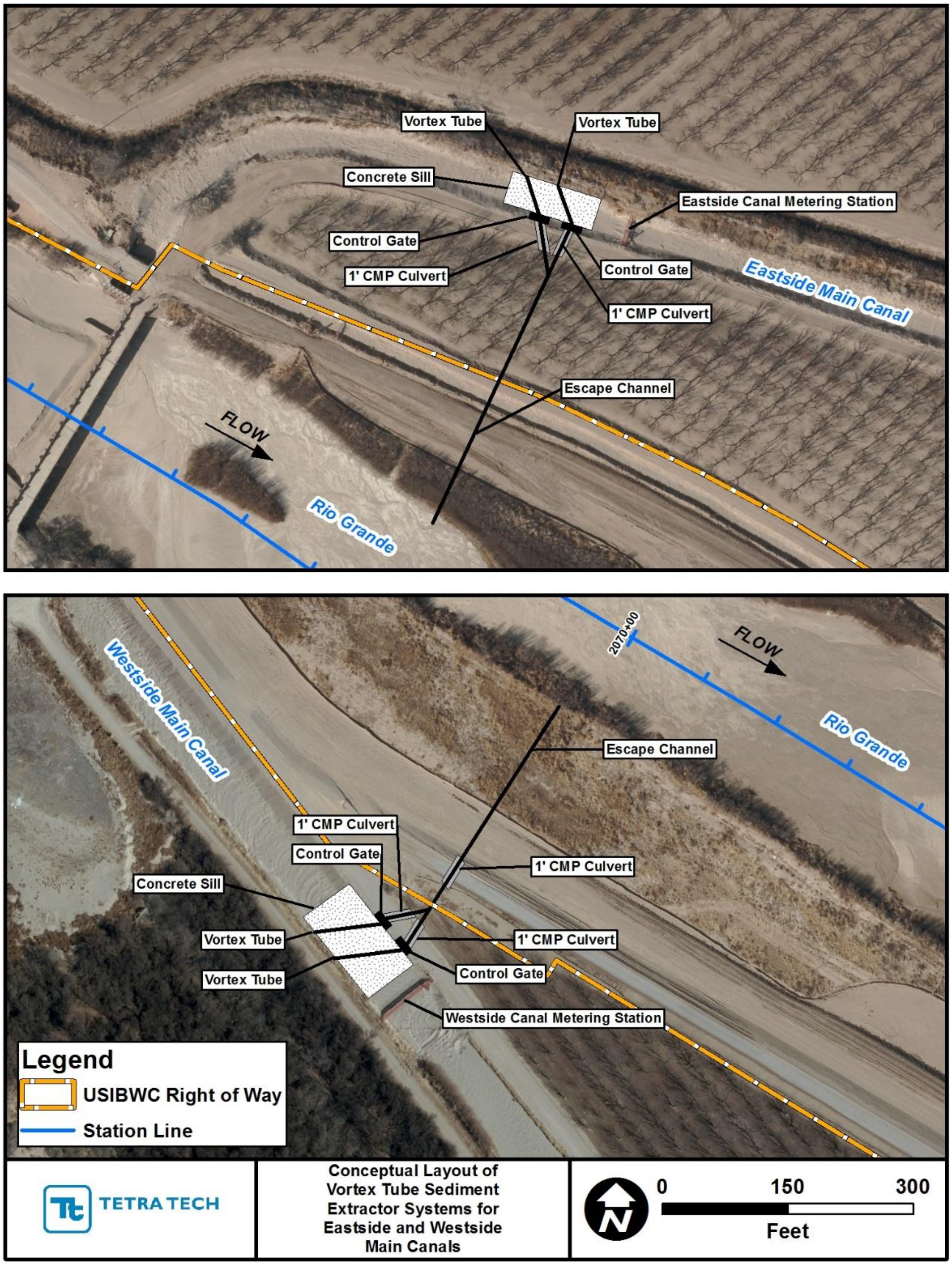

Figure 32. Conceptual layout of the vortex tube sediment extractor system at the Eastside and Westside Main Canals.

Riprap revetment at Problem Location 8 was an alternative included in the Statement of Work for this study. The riprap revetment is required along the right bank downstream from the bridge, extending along the inside of the bend at Sta 408+00 through the straight reach to just above the NeMexas Siphon at Sta 385+00 over a distance of 2,300 feet (**Figure 33**; pers. comm. Derrick O'Hara, April 2015). Although the riprap is required along the inside of a bend or straight bank line where there does not appear to be any active bank erosion, the right bank is very close to the west levee along this reach. In some places, the right top of bank is less than 25 feet from the toe of the east levee, and USIBWC has expressed concern that the vegetated island that has formed downstream from the Country Club Bridge could direct flows into the right bank, causing bank erosion that would further reduce the buffer distance to the levee (pers. comm. Elizabeth Verdecchia, April 2015). The riprap was designed to have a 1.5H:1V sideslope and included bank key-in distances of 10 feet at the up- and downstream limits of the revetment. The riprap was sized using procedures outlined in USACE (1994) and results from the hydraulic modeling at the 100-year routed peak discharge. Results from the calculations indicate the stone used for the riprap would need to have a median diameter of 9 inches; stone with a median diameter of 12 inches was used in the design to be consistent with the existing riprap at the NeMexas Siphon. Although bend scour is not applicable to the reach, general scour calculations were performed using results from the hydraulic modeling to determine the necessary toe-down for the riprap, and indicate about 4.8 feet of general scour could occur at the 100-year event. The revetment was thus designed to have a toe-down depth of 5 feet. The riprap would extend up to the existing top of bank, have a thickness of 2 feet (twice the median stone diameter), and would require about 2,760 CY of riprap. A 6-inch layer of granular filter material would be required for the riprap bedding.

4.7 Localized Modeling of the CMAs

4.7.1 Localized CMA Model Development

The localized steady-state base models were adjusted to represent with-alternative conditions to evaluate the short-term effects of the alternatives on hydraulic conditions with a specific focus on the effects on water-surface elevation. In general, the localized models for the CMAs were developed by adjusting the existing (updated base model) channel geometry, and in some cases the hydraulic roughness, to reflect elements associated with the alternatives. Localized hydraulic models of the CMAs are included as separate plans in the HEC-RAS projects for each problem location on the digital data disc (Appendix M).

The channel geometry for the sediment-removal alternatives was adjusted by inserting the appropriate excavated channel geometry (see Appendix G) using the HEC-RAS Channel Modification editor. At locations where the excavated channel cut through vegetated islands or bars, the hydraulic roughness was adjusted to have Manning's *n*-values that are consistent with the adjacent non-vegetated channel bottom *n*-values.

Island and bar destabilization and vegetation removal was modeled by first adjusting the hydraulic roughness to match the non-vegetated channel bed *n*-values in the bounding channels. The surface of the islands and bars was then lowered by 6 inches to reflect the material that would be removed as a part of the grubbing process. To ensure that the extents of the islands and bars that were selected for treatment reflect existing conditions, the most recent available aerial photography [National Agriculture Imagery Program (NAIP), 2014] was used to delineate the features presented in Appendix J.

Figure 33. Extents of riprap revetment for riprap bank protection alternative considered at Problem Location 8.

For locations where low-elevation spur dikes were considered, the spur geometry was coded directly into the cross-sectional geometry, and a Manning's *n*-value of 0.04 was used along the length of the spur. Because the spurs are oriented about 30 degrees with the cross sections, the effective spur length was used instead of the actual length of the spur. At locations where the spurs did not fall directly on a cross section but would have an effect on the hydraulic conditions at the cross section, the effective spur geometry was transposed onto the cross section. For the model at Problem Location 9, once the model was adjusted to represent the spurs, the island and bar destabilization was added into the model.

Modifications to the Tierra Blanca Vortex Weir were modeled by adding the 30-foot wide and 18-inch deep notch to the section at the weir as well as the pilot channel along the 5,000-foot long reach below the weir. The pilot channel was added using the HEC-RAS Channel Modification editor and spoil berms with equal volume to the cuts were then manually inserted into the model geometry. No changes to the hydraulic roughness were necessary for the modeling of this alternative.

At Problem Location 3, removal of the Rincon Siphon and grade-control structure was modeled by removing the inline structure that represents the upstream sheet pile wall and then lowering the section at the crest of the grade-control structure, which is the only model section located within the extents of the structure. Even though the bottom of the riprap at the crest is at an elevation of 4035.7 feet, the section at the grade control structure was lowered to an elevation of 4037 feet to be consistent with the surveyed bed elevations immediately upstream from the structure. The flume crossing was then coded into the model as a bridge using the HEC-RAS bridge editor.

The riprap bank protection alternative at Problem Location 8 would probably have a relatively insignificant effect on the overall channel hydraulics and water-surface elevations; however, this alternative was included in the steady-state modeling since it would need to be evaluated in the sediment-transport modeling. The revetment was inserted into the cross sectional geometry for the sections where riprap is recommended. Installation of the revetment would require excavation of material for toe-down purposes and it would be possible to grade and compact the spoil material to provide the foundation for the riprap. Considering this along with the recommended riprap thickness of 2 feet plus 6 inches of filter material, the outside face of the revetment top was located 3 feet inside the existing top of bank, extending at a 1.5H:1V sideslope to the channel bed. A Manning's *n*-value of 0.04 was used for the riprap.

A number of the non-sediment removal alternatives were not evaluated using the steady-state hydraulic models because the effects of the alternatives would be insignificant or because it is not possible to know how the alternative would affect hydraulic conditions and water-surface elevations. For example, the arroyo sediment traps would not have any effect on computed steady-state water-surface elevations since the traps are located in the overbanks and would not convey effective flow.

At Problem Location 6, both the check/sluiceway and vortex tube alternatives would likely have a small effect on water-surface elevations between the dam and the sediment returns because the discharge delivered to the headgates would need to be increased to account for the return flows. As such, flows in the river between the dam and the sediment returns would be slightly lower than under existing conditions, whereas flows in the canals would be slightly larger than under existing conditions. However, because the change in headgate discharge required to account for the return flows depends on actual operations of the headgates and gates for the sluicing structures or vortex tubes, it is not possible to determine the associated changes in water-surface elevation. It should be noted that the change in headgate discharge would probably be relatively small compared to the overall river discharge, so changes in water-surface elevation would also probably be relatively small.

Lastly, it is not possible to determine changes in water-surface elevation associated with the automated gate alternative at Mesilla Dam since any change would be entirely attributed to actual gate operations. However, one potential scenario was considered to determine the range of possible changes. Consistent with the original base model of the RGCP, the gates in the updated base model were set to have a 1-foot opening except Gate 1 that was set to 4 feet and Gate 13 that was set to 6 feet. It should be noted that sediment accumulations upstream from the gates eliminate or greatly reduce the effective gate opening (**Figure 34**). To assess one potential change in water-surface elevation that could result from operational changes, the gate configuration was adjusted to include 6-foot openings at Gates 5 and 9 with the remaining gates closed. Results from this modeling is discussed below.

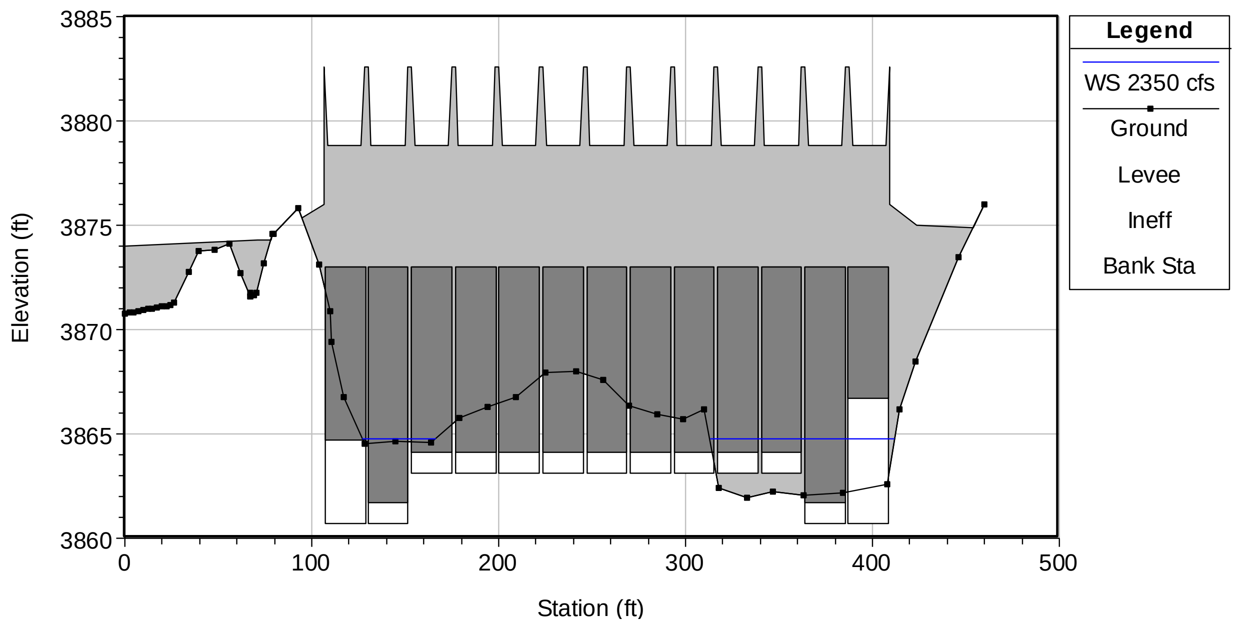

Figure 34. Model cross section at the upstream face of Mesilla Dam showing the gate openings and ground elevation used in the updated base model and localized base model for Problem Location 6.

4.7.2 Localized CMA Model Results

Results from the localized CMA modeling was used to evaluate the short-term changes in water-surface elevation that would occur as a result of alternative implementation. It should be noted that these results only represent the predicted changes to water-surface elevation that would occur immediately after implementation of the alternatives and do not reflect the anticipated channel adjustments that would result from the alternatives. The longer-term changes in water-surface elevation are discussed in the sediment-transport modeling section below. The predicted water-surface elevation profiles from the existing conditions (localized base model) and the CMA models are presented along with plots showing the predicted change in water-surface elevation relative to the base modeling in **Appendix L**. **Tables 12** through **20** summarize the predicted changes.

Table 12. Summary of predicted changes in water-surface elevation for the alternatives modeled at Problem Location 1.

Alternative	Average Change in Water-surface Elevation (ft)*				Length Affected (ft)			
	2,350 cfs	3,000 cfs	3,500 cfs	100-yr Q	2,350 cfs	3,000 cfs	3,500 cfs	100-yr Q
Localized Excavation	-0.3	-0.3	-0.3	-0.1	9,940	9,940	9,940	9,940
Short Excavation	-1.0	-0.9	-0.8	-0.2	10,940	10,940	10,940	10,940
Long Excavation	-1.7	-1.5	-1.4	-0.5	12,050	12,050	12,050	12,050
Modified Tierra Blanca Vortex Weir	0.0	0.0	0.0	0.0	5,870	5,870	6,650	6,010
Alternative	Maximum Increase in Water-surface Elevation (ft)*				Maximum Decrease in Water-surface Elevation (ft)*			
	2,350 cfs	3,000 cfs	3,500 cfs	100-yr Q	2,350 cfs	3,000 cfs	3,500 cfs	100-yr Q
Localized Excavation	0.0	0.0	0.0	0.3	-1.0	-1.0	-0.9	-0.3
Short Excavation	0.0	0.0	0.0	0.0	-1.9	-1.7	-1.6	-0.5
Long Excavation	0.0	0.0	0.0	0.0	-3.1	-2.9	-2.7	-1.2
Modified Tierra Blanca Vortex Weir	0.0	0.0	0.0	0.0	-0.1	-0.1	-0.1	0.0

*Change in predicted water-surface elevation relative to the localized base model.

Table 13. Summary of predicted changes in water-surface elevation for the alternatives modeled at Problem Location 2.

Alternative	Average Change in Water-surface Elevation (ft)*				Length Affected (ft)			
	2,350 cfs	3,000 cfs	3,500 cfs	100-yr Q	2,350 cfs	3,000 cfs	3,500 cfs	100-yr Q
Localized Excavation	-0.2	-0.2	-0.2	-0.2	16,700	17,730	17,730	17,730
Short Excavation	-0.5	-0.4	-0.4	-0.3	17,700	18,730	18,730	18,730
Long Excavation	-0.8	-0.7	-0.7	-0.5	18,190	19,230	19,980	19,230
Island/Bar Destab. & Vegetation Removal	-0.1	-0.1	-0.1	-0.2	19,980	19,980	19,980	19,980
Alternative	**Maximum Increase in Water-surface Elevation (ft)***				**Maximum Decrease in Water-surface Elevation (ft)***			
	2,350 cfs	3,000 cfs	3,500 cfs	100-yr Q	2,350 cfs	3,000 cfs	3,500 cfs	100-yr Q
Localized Excavation	0.0	0.0	0.0	0.1	-0.9	-0.8	-0.8	-0.6
Short Excavation	0.0	0.0	0.0	0.0	-1.8	-1.7	-1.6	-1.0
Long Excavation	0.0	0.0	0.0	0.0	-2.2	-2.1	-2.0	-1.2
Island/Bar Destab. & Vegetation Removal	0.1	0.1	0.1	0.0	-0.6	-0.6	-0.7	-1.0

*Change in predicted water-surface elevation relative to the localized base model.

Table 14. Summary of predicted changes in water-surface elevation for the alternatives modeled at Problem Location 3.

Alternative	Average Change in Water-surface Elevation (ft)*				Length Affected (ft)			
	2,350 cfs	3,000 cfs	3,500 cfs	100-yr Q	2,350 cfs	3,000 cfs	3,500 cfs	100-yr Q
Localized Excavation	-0.7	-0.8	-0.8	-0.5	3,430	3,430	3,020	3,430
Short Excavation	-0.8	-0.8	-0.9	-0.5	3,430	3,430	3,020	3,430
Long Excavation	-1.1	-1.1	-1.1	-0.6	3,430	3,430	3,020	3,430
Remove Rincon Siphon & GCS, Add Flume	-1.2	-1.0	-1.1	-0.4	780	780	1,200	4,210
Alternative	**Maximum Increase in Water-surface Elevation (ft)***				**Maximum Decrease in Water-surface Elevation (ft)***			
	2,350 cfs	3,000 cfs	3,500 cfs	100-yr Q	2,350 cfs	3,000 cfs	3,500 cfs	100-yr Q
Localized Excavation	0.0	0.0	0.0	0.0	-1.3	-1.3	-1.2	-0.6
Short Excavation	0.0	0.0	0.0	0.0	-1.3	-1.3	-1.3	-0.7
Long Excavation	0.0	0.0	0.0	0.0	-1.3	-1.4	-1.3	-0.7
Remove Rincon Siphon & GCS, Add Flume	0.0	0.0	0.0	0.0	-3.4	-3.5	-3.5	-2.3

*Change in predicted water-surface elevation relative to the localized base model.

Table 15. Summary of predicted changes in water-surface elevation for the alternatives modeled at Problem Location 4.

Alternative	Average Change in Water-surface Elevation (ft)*				Length Affected (ft)			
	2,350 cfs	3,000 cfs	3,500 cfs	100-yr Q	2,350 cfs	3,000 cfs	3,500 cfs	100-yr Q
Localized Excavation	-0.3	-0.3	-0.3	-0.2	16,570	16,570	16,570	16,570
Short Excavation	-0.4	-0.4	-0.4	-0.3	17,260	17,260	17,260	17,260
Long Excavation	-1.7	-1.5	-1.3	-0.7	17,260	17,260	17,260	17,260
Island/Bar Destab. & Veg. Removal	0.0	0.0	-0.1	-0.3	17,260	17,260	17,260	17,260
Low-elev. Spur Dikes	0.5	0.5	0.5	0.4	4,910	4,910	4,910	4,660
Alternative	Maximum Increase in Water-surface Elevation (ft)*				Maximum Decrease in Water-surface Elevation (ft)*			
	2,350 cfs	3,000 cfs	3,500 cfs	100-yr Q	2,350 cfs	3,000 cfs	3,500 cfs	100-yr Q
Localized Excavation	0.0	0.0	0.0	0.0	-1.7	-1.5	-1.4	-0.7
Short Excavation	0.0	0.0	0.0	0.0	-1.7	-1.6	-1.5	-0.8
Long Excavation	0.0	0.0	0.0	0.0	-2.1	-2.0	-1.8	-1.0
Island/Bar Destab. & Veg. Removal	0.2	0.2	0.1	0.0	-0.2	-0.3	-0.3	-0.7
Low-elev. Spur Dikes	1.1	1.0	1.0	0.7	0.0	0.0	0.0	-0.1

*Change in predicted water-surface elevation relative to the localized base model.

Table 16. Summary of predicted changes in water-surface elevation for the alternatives modeled at Problem Location 5.

Alternative	Average Change in Water-surface Elevation (ft)*				Length Affected (ft)			
	2,350 cfs	3,000 cfs	3,500 cfs	100-yr Q	2,350 cfs	3,000 cfs	3,500 cfs	100-yr Q
Localized Excavation	-0.2	-0.2	-0.2	-0.1	8,500	8,500	8,500	8,500
Short Excavation	-0.6	-0.6	-0.6	-0.2	8,980	8,980	8,980	8,980
Long Excavation	-1.4	-1.3	-1.2	-0.3	15,480	15,480	15,480	15,480
Low-elev. Spur Dikes	0.5	0.5	0.3	0.1	8,500	8,500	8,500	8,500
Alternative	Maximum Increase in Water-surface Elevation (ft)*				Maximum Decrease in Water-surface Elevation (ft)*			
	2,350 cfs	3,000 cfs	3,500 cfs	100-yr Q	2,350 cfs	3,000 cfs	3,500 cfs	100-yr Q
Localized Excavation	0.0	0.0	0.0	0.0	-0.6	-0.6	-0.6	-0.3
Short Excavation	0.0	0.0	0.0	0.0	-1.4	-1.3	-1.3	-0.4
Long Excavation	0.0	0.0	0.0	0.0	-2.7	-2.6	-2.5	-0.7
Low-elev. Spur Dikes	1.0	1.0	0.7	0.2	0.0	0.0	0.0	0.0

*Change in predicted water-surface elevation relative to the localized base model.

Table 17. Summary of predicted changes in water-surface elevation for the alternatives modeled at Problem Location 6.

Alternative	Average Change in Water-surface Elevation (ft)*				Length Affected (ft)			
	2,350 cfs	3,000 cfs	3,500 cfs	100-yr Q	2,350 cfs	3,000 cfs	3,500 cfs	100-yr Q
Short Excavation	-0.8	-0.8	-0.8	-0.1	8,860	8,860	8,860	8,860
Long Excavation	-1.1	-1.0	-1.0	-0.2	8,860	8,860	8,860	8,860
Alternative	Maximum Increase in Water-surface Elevation (ft)*				Maximum Decrease in Water-surface Elevation (ft)*			
	2,350 cfs	3,000 cfs	3,500 cfs	100-yr Q	2,350 cfs	3,000 cfs	3,500 cfs	100-yr Q
Short Excavation	0.0	0.0	0.0	0.0	-1.8	-1.7	-1.6	-0.2
Long Excavation	0.0	0.0	0.0	0.0	-1.8	-1.6	-1.6	-0.3

*Change in predicted water-surface elevation relative to the localized base model.

Table 18. Summary of predicted changes in water-surface elevation for the alternatives modeled at Problem Location 7.

Alternative	Average Change in Water-surface Elevation (ft)*				Length Affected (ft)			
	1,400 cfs	3,000 cfs	3,500 cfs	100-yr Q	1,400 cfs	3,000 cfs	3,500 cfs	100-yr Q
Localized Excavation	-0.1	-0.1	-0.1	0.0	1,400	1,400	1,400	1,000
Short Excavation	-0.4	-0.2	-0.2	-0.1	8,930	8,930	8,930	8,930
Long Excavation	-0.7	-0.4	-0.4	-0.2	8,930	8,930	8,930	8,930
Low-elev. Spur Dikes	0.5	0.4	0.4	0.3	8,930	8,930	8,930	8,930
Alternative	Maximum Increase in Water-surface Elevation (ft)*				Maximum Decrease in Water-surface Elevation (ft)*			
	1,400 cfs	3,000 cfs	3,500 cfs	100-yr Q	1,400 cfs	3,000 cfs	3,500 cfs	100-yr Q
Localized Excavation	0.0	0.0	0.0	0.0	-0.1	-0.1	-0.1	0.0
Short Excavation	0.0	0.0	0.0	0.0	-0.9	-0.5	-0.4	-0.2
Long Excavation	0.0	0.0	0.0	0.0	-1.0	-0.6	-0.5	-0.3
Low-elev. Spur Dikes	0.6	0.5	0.4	0.3	0.0	0.0	0.0	0.0

*Change in predicted water-surface elevation relative to the localized base model.

Table 19. Summary of predicted changes in water-surface elevation for the alternatives modeled at Problem Location 8.

Alternative	Average Change in Water-surface Elevation (ft)*				Length Affected (ft)			
	1,400 cfs	3,000 cfs	3,500 cfs	100-yr Q	1,400 cfs	3,000 cfs	3,500 cfs	100-yr Q
Localized Excavation	-0.1	-0.1	-0.1	-0.2	4,220	4,220	4,220	4,220
Short Excavation	-0.3	-0.3	-0.3	-0.3	5,050	5,050	5,050	5,050
Long Excavation	-0.8	-0.5	-0.5	-0.4	7,470	7,470	7,470	7,470
Riprap	0.1	0.0	0.0	0.0	6,970	7,970	7,480	7,970
Low-elev. Spur Dikes	0.3	0.2	0.2	0.2	6,970	6,970	6,970	6,970
Alternative	Maximum Increase in Water-surface Elevation (ft)*				Maximum Decrease in Water-surface Elevation (ft)*			
	1,400 cfs	3,000 cfs	3500 cfs	100-yr Q	1,400 cfs	3,000 cfs	3,500 cfs	100-yr Q
Localized Excavation	0.0	0.0	0.0	0.0	-0.3	-0.3	-0.3	-0.3
Short Excavation	0.0	0.0	0.0	0.0	-0.6	-0.6	-0.6	-0.5
Long Excavation	0.0	0.0	0.0	0.0	-1.2	-0.8	-0.8	-0.6
Riprap	0.1	0.1	0.1	0.1	0.0	0.0	0.0	0.0
Low-elev. Spur Dikes	0.4	0.3	0.3	0.3	0.0	0.0	0.0	0.0

*Change in predicted water-surface elevation relative to the localized base model.

Table 20. Summary of predicted changes in water-surface elevation for the alternatives modeled at Problem Location 9.

Alternative	Average Change in Water-surface Elevation (ft)*				Length Affected (ft)			
	1,400 cfs	3,000 cfs	3,500 cfs	100-yr Q	1,400 cfs	3,000 cfs	3,500 cfs	100-yr Q
Localized Excavation	-0.2	-0.2	-0.2	-0.2	2,750	2,750	2,750	2,750
Short Excavation	-0.4	-0.4	-0.3	-0.4	5,130	5,130	5,130	5,130
Long Excavation	-1.9	-1.7	-1.6	-1.4	13,290	13,290	13,290	13,290
Island/Bar Destab. & Vegetation Removal	-0.3	-0.3	-0.4	-0.4	13,580	13,580	13,580	13,580
Low-elev. Spur Dikes	0.1	0.0	0.0	-0.1	2,750	2,750	5,040	2,750
Alternative	Maximum Increase in Water-surface Elevation (ft)*				Maximum Decrease in Water-surface Elevation (ft)*			
	1,400 cfs	3,000 cfs	3,500 cfs	100-yr Q	1,400 cfs	3,000 cfs	3,500 cfs	100-yr Q
Localized Excavation	0.0	0.0	0.0	0.0	-0.4	-0.3	-0.3	-0.2
Short Excavation	0.1	0.1	0.1	0.0	-0.9	-0.7	-0.6	-0.5
Long Excavation	0.0	0.0	0.0	0.0	-2.7	-2.2	-2.1	-2.0
Island/Bar Destab. & Vegetation Removal	0.0	0.0	0.0	0.0	-0.5	-0.6	-0.6	-0.7
Low-elev. Spur Dikes	0.1	0.0	0.0	0.0	0.0	-0.1	-0.1	-0.1

*Change in predicted water-surface elevation relative to the localized base model.

The water-surface profile comparisons indicate that each of the alternatives evaluated would have at least have a localized effect on water-surface elevation. The effects of like alternatives vary by problem location, and in general show the largest effect at the lowest discharges and the smallest effect at the 100-year discharge. Many of the alternatives would result in no increase to predicted water-surface elevation over the range of modeled discharges and almost all of the alternatives (except the low-elevation spur dike alternative) would result in some localized decrease to predicted water-surface elevation. The lengths over which the alternatives would affect predicted water-surface elevation are dependent on both the longitudinal extent of the treatment as well the degree to which the treatment affects conveyance.

Based on the model results of the excavation alternatives, each scenario would result in reduced predicted water-surface elevations. The reduction in predicted water-surface elevation associated with the excavation alternatives is, as expected, smallest under the localized excavation alternative and largest under the long excavation alternative. At the lower discharges (1,400 cfs below Mesilla Dam and 2,300 cfs above Mesilla Dam), the localized excavation alternative would result in average reductions to predicted water-surface elevation of between 0.1 feet (Problem Location 7) to 0.7 feet (Problem Location 3). At these same discharges, the short excavation alternative would result in average reductions to predicted water-surface elevations of between 0.3 feet (Problem Location 8) and about 1 foot (Problem Location 1) and the long excavation alternative would reduce the average predicted water-surface elevation by between 0.7 feet (Problem Location 7) and nearly 2 feet (Problem Location 9). At the 100-year peak discharge, the localized excavation alternative would result in average reductions to predicted water-surface elevation of between less than 0.1 feet (Problem Location 7) to 0.5 feet (Problem Location 3), the short excavation alternative would result in average reductions of between 0.1 feet (Problem Locations 6 and 7) and 0.5 feet (Problem Location 3), and the long excavation alternative would result in average reductions of between 0.2 feet (Problem Locations 6 and 7) and 1.4 feet (Problem Location 9). None of the excavation alternatives would result in increases to predicted water-surface elevation except the short excavation alternative at Problem Locations 4 and 9, where very small, localized increases occur near the upstream limit of the excavations. The length over which the excavation alternatives affect predicted water-surface elevation within the modeled reaches ranges from as little as 1,400 feet for the localized excavation alternative at Problem Location 7 to nearly 4 miles under the long alternative at Problem Location 2 over the range of modeled flows[4].

As expected, the reduced conveyance area associated with the low-elevation spur dike alternative generally results in increased predicted water-surface elevations. At the lower discharges (1,400 cfs below Mesilla Dam and 2,300 cfs above Mesilla Dam), the spur dike alternative would result in average predicted water-surface elevation increases of between about 0.1 feet (Problem Location 9) to 0.5 feet (Problem Locations 4, 5 and 7). At the 100-year peak discharge, the spur dike alternative would result in average predicted water-surface elevation increases of less than 0.4 feet (Problem Location 4), and would result in an average reduction to predicted water-surface elevation at Problem Location 9 due to the effects of the island/bar destabilization and vegetation removal treatments that were built into the spur dike alternative at this location. Localized maximum increases to predicted water-surface elevation are as much as 1.1 feet for the lower three discharges, reducing to about 1 foot during the 100-year peak discharge (Problem Location 4). Average decreases in predicted water-surface elevation range from 0.2 feet (Problem Location 2) to nearly 0.5 feet (Problem Location 9). The length over which the spur dikes would affect

[4]The lengths reported herein are based on the extents of the localized models as outlined in the statement of work for this study and may not represent the affected area upstream from the upstream limit of the localized models.

predicted water-surface elevation ranges from about 0.5 miles (Problem Location 9) to about 1.7 miles (Problem Location 7).

The alternatives involving island/bar destabilization and vegetation generally result in reduced predicted water-surface elevations that are a result of the 6-inch lowering of the island and bar surfaces and the reduction to the hydraulic roughness. In some cases, the model results indicate a slight increase in predicted water-surface elevation that is caused by the removal of the HEC-RAS ineffective flow areas that were included in the base modeling, which when removed may cause a small increase to the conveyance weighted hydraulic roughness of the overall channel. At the lowest modeled discharges (1,400 cfs below Mesilla Dam and 2,350 cfs above the dam), average changes in predicted water-surface elevation range from essentially no change (Problem Location 4) to a decrease of about 0.3 feet (Problem Location 9). Average decreases in predicted water-surface elevation at the 100-year peak flow range from 0.2 feet (Problem Location 2) to 0.5 feet (Problem Location 9). Localized maximum increases in predicted water-surface elevation are less than 0.2 feet and occur at the lower modeled discharges at Problem Location 4. The lengths over which the treatments would affect predicted water-surface elevation range from about 2.6 miles (Problem Location 9) to about 3.8 miles (Problem Location 2).

The comparative predicted water-surface elevations for the site-specific alternatives indicate the effects of these alternatives vary with the degree of modification. Modifications to the Tierra Blanca Vortex Weir at Problem Location 1 and installation of riprap revetment at Problem Location 8 result in very little change to predicted water-surface elevation. At Problem Location 1, the notching of the Tierra Blanca Vortex Weir and associated downstream pilot channel would reduce the average predicted water-surface elevation by less than 0.1 feet at the lower discharges and have essentially no effect at the 100-year peak flow. The riprap revetment at Problem Location 8 would increase the average predicted water-surface elevation by less than 0.5 feet over the range of modeled flows. At Problem Location 3, replacement of the Rincon Siphon with an elevated flume and removal of the grade-control structure would have a much more significant impact, reducing the average predicted water-surface elevation by between 1.1 feet and 1.2 feet at the three lower discharges but by only 0.4 feet at the 100-year peak flow. The reduced predicted water-surface elevations would occur over a distance of about 800 feet at a discharge of 2,350 cfs, increasing to a distance of about 4,200 feet at the 100-year peak flow.

Results from the modeling at Problem Location 6 that was carried out to assess one potential change that could result from operational changes at Mesilla Dam indicate that the operation of the gates could have a significant effect on predicted water-surface elevations upstream from the dam, as expected (**Figure 35**). The results from the modeling at a discharge of 2,350 cfs with 6-foot openings at Gates 5 and 9 have a predicted water-surface elevation at the dam that is more than 2 feet higher than the original base model gate configuration (4-foot opening at Gate 1, 6-foot opening at Gate 13, and 1-foot openings at the remaining gates). The increased predicted water-surface elevations extend over a distance of about 2,500 feet upstream from the dam.

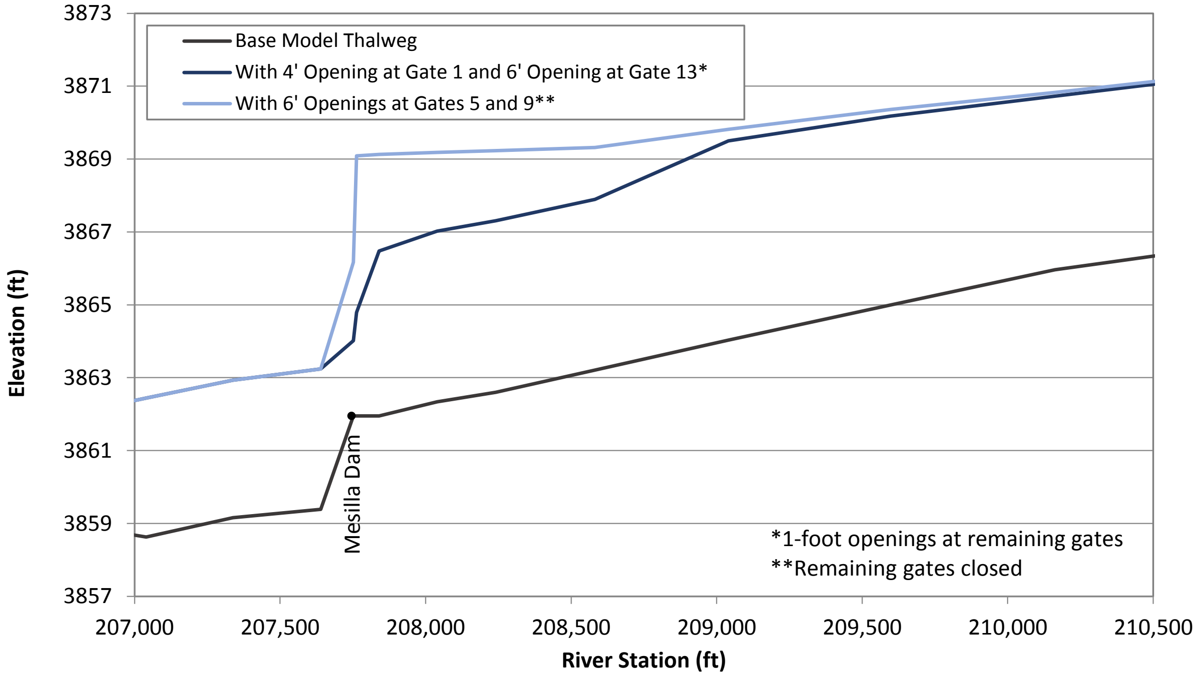

Figure 35. Predicted water surface profiles in the vicinity of Mesilla Dam at a discharge of 2,350 cfs under gate opening scenarios with 1) a 4-foot gate opening at Gate 1, a 6-foot gate opening at Gate 13, and 1-foot openings at the remaining gates, and 2) 6-foot openings at Gates 5 and 9 with the remaining gates closed.

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5 SEDIMENT-TRANSPORT MODELING

5.1 Base Sediment-transport Model Development and Validation

The base sediment-transport modeling was performed with sediment-routing models of the problem location reaches that were developed using the mobile bed sediment-transport feature in HEC-RAS Version 4.1 (USACE 2010). The base sediment-transport models were calibrated and validated using available information, and served as the basis of comparison for the alternatives runs. It should be noted that the base model validation simulations represent the no-action scenario under normal flow conditions (discussed in Section 5.1.3.1); whereas, the base model simulations that were used for purposes of evaluating the alternatives (which use the representative hydrographs discussed in Section 5.1.3.2) represent the no-action scenario under extreme drought conditions. At Problem Location 6, where flow splits upstream from Mesilla Dam deliver flow and sediment to the Eastside and Westside Canals, it was necessary to use the beta-test version of HEC-RAS 5.0 because Version 4.1 is not capable of modeling sediment splits at distributary junctions. The model geometry and basic structure of the models were taken from the localized hydraulic models discussed in the previous sections. In general, the mobile bed sediment-transport feature in HEC-RAS requires input to define the existing bed material, the upstream and lateral sediment supplies, the hydrologic sequence over which sediment transport is evaluated, and a variety of other model inputs that are necessary for the sediment-transport computations. These inputs are described below.

5.1.1 Model Geometry

The cross-sectional geometries developed for the localized steady-state HEC-RAS models for each of the problem locations were incorporated directly into the sediment-transport models, but a number of adjustments were made to facilitate the sediment-transport modeling. Initial test runs of the alternatives indicated that the hydraulic and sediment-transport effects of some of the alternatives could extend beyond the limits of the localized hydraulic models. As a result, the model geometries at a number of locations were extended up- and/or downstream using the geometric information from the updated base model of the overall RGCP. In some cases, where significant amounts of aggradation or degradation was indicated by the 2004 and 2014/2015 surveys at the limits of the problem locations, the updated base model bed geometry that was used to extend the models was adjusted to match the measured change at the survey limits. These adjustments were made along the extended reaches downstream from Problem Locations 5 and 8, and in the extended reaches upstream from Problem Locations 6 and 9. These adjustments resulted in changes to mean bed elevation of 1.1 feet in the reach below Problem Location 5, 2.8 feet in the reach above Problem Location 6, 1.7 feet in the reach below Problem Location 8, and 2.6 feet in the reach above Problem Location 9. The up- and downstream limits of the sediment-transport models, compared to the limits of the steady-state hydraulic models and surveys, are presented in Tables 4 and 5.

Another change to the steady-state model geometry that was required for the sediment-transport modeling included removal of the bridges. The bridge routines in HEC-RAS Version 4.1 are not capable of correctly modeling sediment-transport through the bridge openings, and the HEC-RAS authors recommend that the bridges be removed from the sediment-transport simulations (Dr. Stanford Gibson, HEC, pers. comm., May 2013). The localized hydraulic models were therefore used to evaluate the hydraulic effects of the bridges. Results from this evaluation indicate that the bridges have very little effect on the predicted hydraulic conditions at and upstream from the bridges, and result in no change in water-surface elevation at normal flow conditions (2,350 cfs upstream from Mesilla Dam and 1,400 cfs downstream from Mesilla Dam) and less than 0.3 feet of change at the 100-year peak flow event. As such, removal of the bridges from the sediment-transport models does not appear to be a significant limitation.

Most 1-D sediment-transport model platforms require that the control volumes represented by the cross sections are more or less similar in size because the volume of sediment that is eroded or deposited from a given control volume directly affects the amount of material that is available for transport to the next downstream control volume. For example, model instabilities tend to occur when a large amount of material is either eroded or deposited in a small control volume created by closely spaced cross sections. In some cases, the localized steady-state hydraulic models include cross-sections that are irregularly spaced. The cross sections in the base model at Problem Location 1, for instance, include sections from a variety of sources and have cross-sectional spacings that range from 10 feet to nearly 1,000 feet. It was therefore necessary to first identify the most representative and hydraulically important cross sections among a group of closely spaced sections, and then remove the remainder of the very closely spaced sections that are less hydraulically important.

Other relatively minor changes that were made to the channel geometry were identified as necessary during the model calibration phase. In cases where ineffective flow areas were used to remove conveyance from the cross-sectional area representing the arroyo mouths, the ineffective flow areas were replaced with blocked obstructions to avoid excessive deposition in the ineffective flow areas. The predicted geometry at a few cross sections showed significant lowering of the thalweg point during periods of very low flow (less than 1 cfs) that was not representative of the overall change in channel geometry; the thalweg and adjacent points were smoothed at these cross sections to eliminate this irregularity.

5.1.2 Bed Sediment Reservoir Definition

The existing (resident) bed material that is available for erosion is often referred to in the mobile boundary sediment-transport model as the bed sediment reservoir. Size-gradation data for the bed sediment reservoir were developed from bulk samples of the surface material, and at locations where the bed is armored, pebble counts and bulk samples of the subsurface material, that were collected during the field reconnaissance (see Section 3, above). At locations where tributary sediment loading results in localized coarsening of the bed material, a representative gradation was assigned to the majority of the reach and separate, coarser gradations were used in the vicinity of the fans; at locations where the bed material did not vary significantly, a single representative gradation was used. The geomorphic top of banks were used to define the width of the bed sediment reservoir (i.e., the erosion limits). Except for a few unique locations, the depth of the bed sediment reservoir was set to 30 feet to ensure degradation is not artificially limited. A zero-depth bed sediment reservoir was used at the cross section along the crest of the Tierra Blanca Vortex Weir (Problem Location 1) and at the cross sections at the up- and downstream limits of the grade control structure below the Rincon Siphon (Problem Location 3).

At Problem Location 1, the gradation of Sample S3 was used for the representative gradation of the overall reach since this gradation was collected upstream from the tributaries and therefore does not include the tributary loading influences on bed gradation that are represented in the other samples. At and downstream from the confluence with Tierra Blanca Creek and Green Arroyo, the representative gradation of the fan materials was based on a weighted average of the armor material (Pebble Count PC1) and subsurface material (Sample S1). The weighting was adjusted until the resulting gradation matched the gradation of a sample that is representative of the non-reworked portion of the fan which was collected as part of the USACE (2007) study (MEI Sample S-2). The resulting weighting of 45-percent pebble count and 55-percent bulk sample resulted in a reasonable match between the representative gradation and the gradation of the non-reworked portion of the fan (**Figure 36**). This same weighting was then applied to the armor

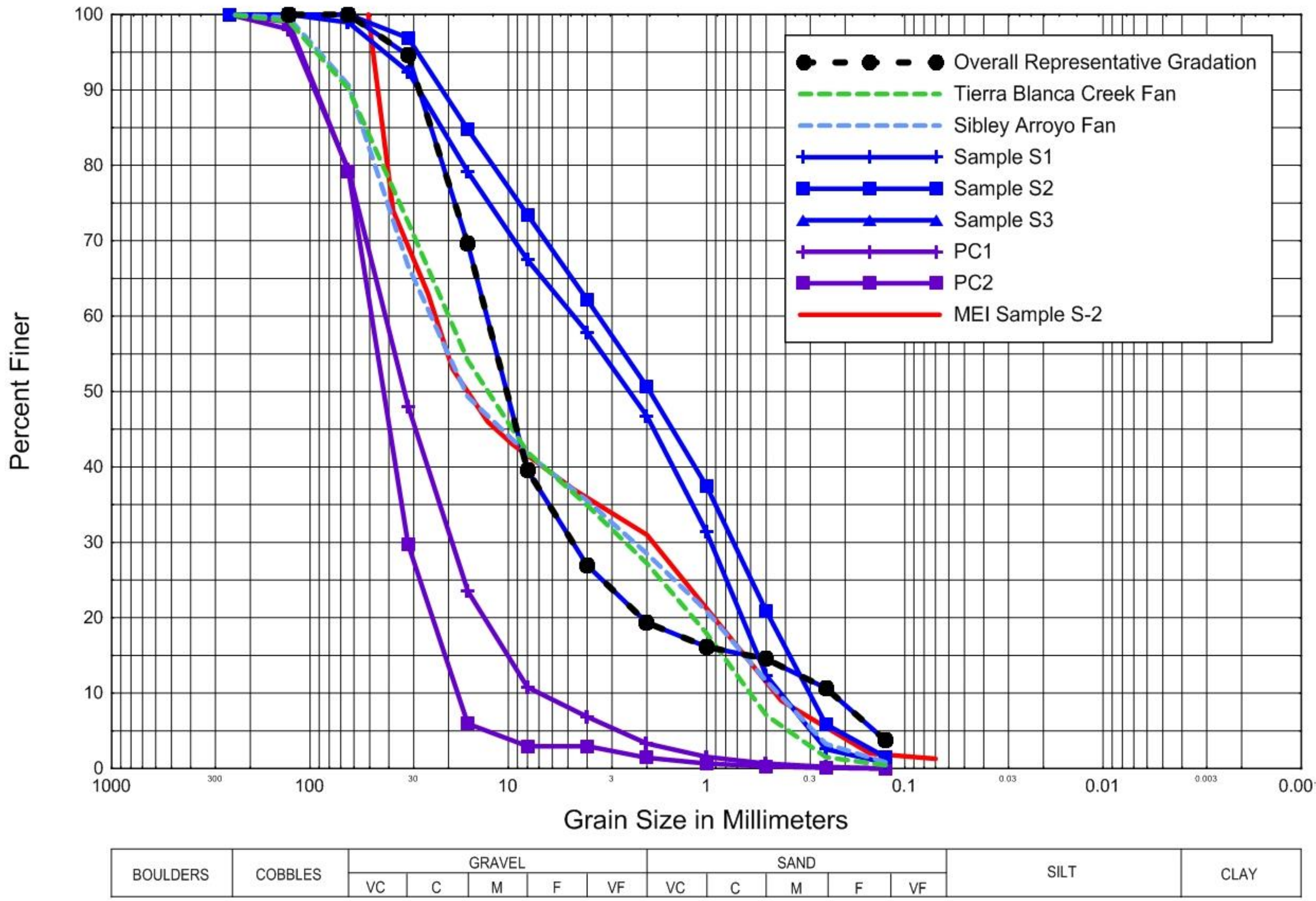

Figure 36. Representative gradation of the bed sediment reservoir for Problem Location 1, and the gradation of the bed sediment reservoir that was used at the Tierra Blanca Creek/Sibley Arroyo fans. Also shown are the gradations of the samples and pebble counts collected at Problem Location 1, and the gradation of MEI Sample S-2 that was used as the basis for developing the weighted average gradations used for the fans.

material (Pebble Count PC2) and subsurface material (Sample S2) at the mouth of Sibley Arroyo to obtain the gradation of sediments making up this fan. The overall representative gradation for Problem Location 1 has a median diameter (D_{50}) of 10.2 mm and contains 20-percent sand (**Table 21**). The gradation for the Tierra Blanca Creek/Green Arroyo fan has a median diameter of about 13 mm and contains 27-percent sand. The gradation for the Sibley Arroyo fan is similar to the upstream fan with a median diameter of about 16 mm and contains 29-percent sand (Table 21).

The gradation of Sample S4, which was taken from the bed of the channel near the upstream limit of Problem Location 2, was used as the representative gradation for the majority of this reach. The bed material appears to be very similar along the fans of Thurman I and Thurman II Arroyos, so the gradation for both fans was developed by taking the weighted average of the armor material (Pebble Count PC3) and Sample S4 materials using the same 45-percent (pebble count) and 55-percent (bulk sample) weighting that was used at the Tierra Blanca Creek fan. This weighting was also applied to the armor material (Pebble Count PC4) and subsurface material (Sample S5) at the Placitas Arroyo fan. The representative gradation for Problem Location 2 has a median diameter of about 5 mm and contains 37-percent sand (Table 21). The gradations for the Thurman I and Thurman II Arroyo fans and the Placitas Arroyo fans have a median diameter of between 25 and 30 mm and include about 20-percent sand.

The representative gradation for Problem Location 3 was developed by averaging the gradations for the two samples collected along this reach (Samples S6 and S7), and has a median diameter of about 1 mm and includes about 60-percent sand. The weighted average of the gradations for Pebble Count PC5 and Sample S7 was used for the Garcia Arroyo fan based on the 45- and 55-percent weightings that were used at the upstream problem locations. The gradation of Sample S7 (D_{50}=2 mm) was also directly used for the gradation of the bed sediment reservoir downstream from the Garcia Arroyo Fan.

Two distinctly different types of bed material are present at Problem Location 4. Upstream from the Rincon Arroyo fan, where the bed material is mostly sand, the gradation of Sample S8 (D_{50} = 0.5 mm) was used as the representative gradation. Downstream from Rincon Arroyo, the bed material coarsens and is predominantly gravel; the gradation of Sample S9 (D_{50} = 3.4 mm) was used as the representative gradation for this portion of the reach. Although Rincon Arroyo drains different geologic formations than those in the Reed Arroyo and Bignell Arroyo watersheds, Pebble Counts PC6, PC7 and PC8 indicate that the armor material along the fans of these three arroyos is very similar. The representative gradation for each of the fans was therefore developed by taking the weighted average of the pebble counts (Pebble Counts PC6, PC7 and PC8) and the bulk sample (Sample S9). During the model validation process, it was determined that use of a 45 percent (pebble count) and 55 percent (bulk sample) resulted in unrealistic amounts of aggradation at the mouth of Rincon Arroyo. As a result, the weightings were adjusted until more realistic amounts of aggradation was predicted, and a 25-percent (pebble count) and 75-percent (bulk sample) weighting was ultimately used.

Two separate representative gradations were also used at Problem Location 5. The gradation of Sample S10 (D_{50} = 1.1 mm) was used for the reach upstream from Rock Canyon and Sample S11 (D_{50} = 2.1 mm) was used for coarser portion of the reach downstream from Rock Canyon. A weighted average of the pebble count (Pebble Count PC9; 45 percent) and bulk sample (Sample S11; 55 percent) was used for the gradation of the Rock Canyon fan, which has a median diameter of about 17 mm and includes 27-percent sand. Although no armoring was evident along the fan of Horse Canyon Creek, this tributary appears to deliver relatively coarse material to the RGCP. The gradation for this fan was therefore assumed to be similar to that along the Rock Canyon fan, and developed by taking the weighted average of Pebble Count PC9 (45-percent

Table 21. Summary of representative gradations used to define the bed sediment reservoir for the sediment-transport models.

Problem Location	Representation	D_{16} (mm)	D_{50} (mm)	D_{84} (mm)	% Sand	% > Sand
1	Overall Representative	0.9	10.2	23.9	19%	81%
	Tierra Blanca Crk/Green Arroyo Fan	0.9	12.7	50.3	27%	73%
	Sibley Fan	0.7	16.5	52.8	29%	71%
2	Overall Representative	0.3	4.9	22.3	37%	63%
	Thurman I & II Arroyo Fans	0.7	27.4	69.3	21%	79%
	Placitas Arroyo Fan	0.8	24.6	68.2	20%	80%
3	Overall Representative	0.2	1.0	15.8	61%	39%
	Garcia Arroyo Fan	0.5	25.3	56.4	28%	72%
	Below Garcia Arroyo Fan	0.3	1.9	23.3	50%	50%
4	Overall Representative	0.3	3.4	14.8	41%	59%
	Supply Reach (Above Arroyos)	0.2	0.5	3.3	80%	20%
	Average Fan Gradation	0.5	8.0	46.0	31%	69%
5	Supply Reach (Above Arroyos)	0.3	1.1	6.8	61%	39%
	Below Rock Canyon	0.3	2.1	12.4	49%	51%
	Rock Canyon Fan	0.5	16.7	88.4	27%	73%
	Horse Crk Canyon Fan	0.3	16.4	88.4	38%	62%
6	Representative (Above Mesilla)	0.26	0.38	0.70	100%	0%
	Representative (Below Mesilla)	0.14	0.25	0.46	100%	0%
7	Overall Representative	0.02	0.10	0.49	100%	0%
	Average Fan Gradation	0.2	0.4	11.3	80%	20%
8	Overall Representative	0.12	0.17	0.28	100%	0%
9	Overall Representative	0.14	0.23	0.42	100%	0%

weighting) and Sample S12 (55-percent weighting). The resulting gradation for Horse Canyon Creek has a median diameter of about 16 mm and contains 38-percent sand.

Although trace gravels are found at Problem Location 6, the bed material along this reach is predominantly sand. The gradation of Sample S15 (D_{50} = 0.4 mm) was used for the representative gradation in the reach upstream from Mesilla Dam, and the gradation of Sample S16 (D_{50} = 0.2 mm) was used for the reach downstream from Mesilla Dam.

The bed material along the majority of the reach at Problem Location 7 is also predominantly sand, and the gradation for Sample S17 was used as the overall representative bed material gradation at this location. Although the fans for the four west side arroyos are not of significant size, the bed material does coarsen somewhat over short distances downstream from the fans. Bulk Sample S18, collected from the bed of the downstream most arroyo, is believed to be representative of the type of materials delivered by these tributaries. However, because the fans are frequently reworked during the irrigation season and are somewhat finer than the tributary loads, the representative fan gradation was developed using a weighted average of Samples S17 (75 percent) and Sample S18 (25 percent).

At Problem Locations 8 and 9, the bed material is entirely sandy material, so the representative gradations were based on the gradations of the bulk sediment samples collected at these two locations. The gradation of Sample S19 (D_{50} = 0.2 mm) was used for the representative gradation at Problem Location 8. The representative gradation at Problem Location 9 was based on the average of the gradations for Samples S13 and S14, and has a median diameter of 0.2 mm.

5.1.3 Hydrology

Two sets of hydrology were used to define the quasi-unsteady flow data for the sediment-transport simulations, including a hydrologic dataset that was used for model validation and a representative hydrologic simulation that was used in the base models to evaluate the alternatives.

5.1.3.1 Hydrology for Model Validation

Survey data that are available for use in the model validation includes data from cross-sectional surveys that were conducted in 2004 as part of the FLO-2D model development project (Tetra Tech, 2005), and the cross-sectional surveys that were conducted in 2014 and 2015 for this study. A few intermediate cross sections were surveyed by EBID in the vicinity of Tierra Blanca Creek at Problem Location 1 in 2007, but these sections were not repeat survey sections and were therefore not used for model validation. The hydrology for the model validation runs was therefore based on measured and estimated flows during the period from Water Year 2005 (WY2005) to WY2014. A number of unmeasured inflows and outflows occur along the RGCP, so it is not possible to exactly duplicate the actual hydrologic conditions that occurred during that period. Thus, to develop the hydrologic sequences that represent the upstream inflows to Problem Locations 1 through 6, the measured Caballo release hydrograph over the 10-year period was adjusted to represent the various inflows and outflows along the RGCP using the flow duration curves presented in USACE (2007) (**Figure 37**). For Problem Locations 7 through 9, the measured flow at the Rio Grande at El Paso gage (USIBWC Gage No. 08364000; referred to as the Courchesne gage) were adjusted using the USACE (2007) flow-duration curves. The resulting hydrographs at each of the problem locations, along with the measured Caballo release hydrograph, are shown in **Figures 38 and 39**. It should be noted that the discharges delivered to the Eastside and Westside Canals at Problem Location 6 were not directly coded into the quasi-unsteady flow files, but were instead based on rating curves representing typical diversion rates as discussed in more detail below.

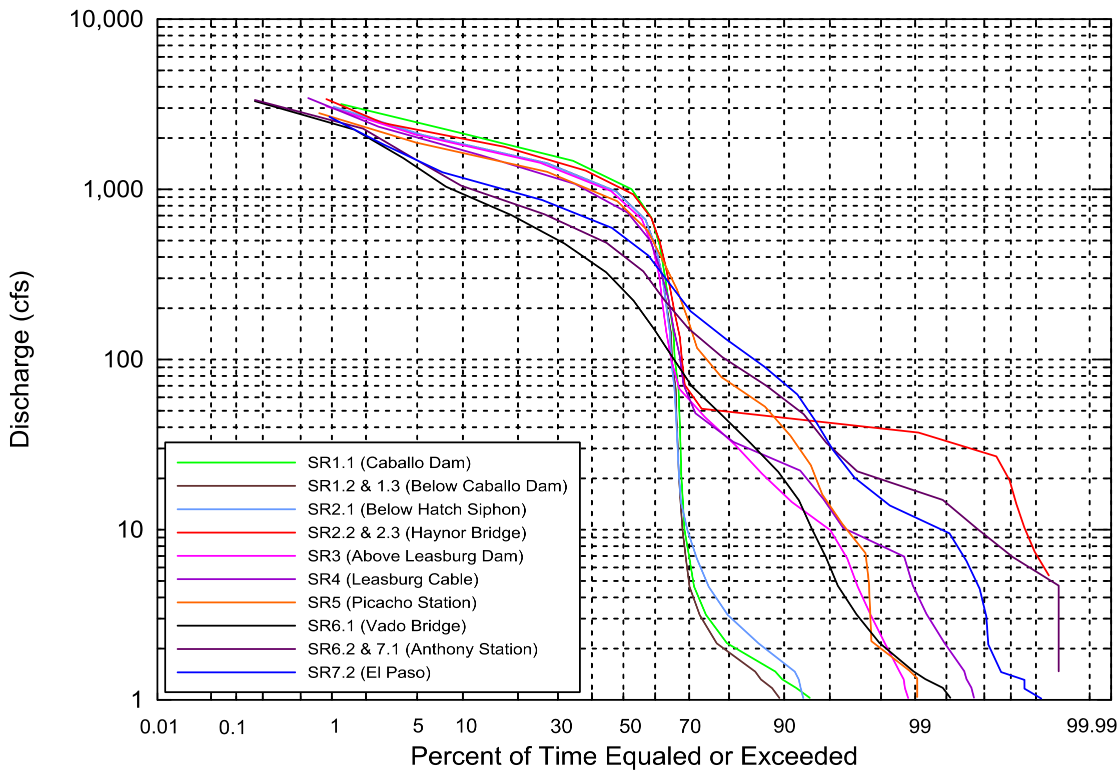

Figure 37. Flow-duration curves for each of the geomorphic subreaches (from USACE, 2007).

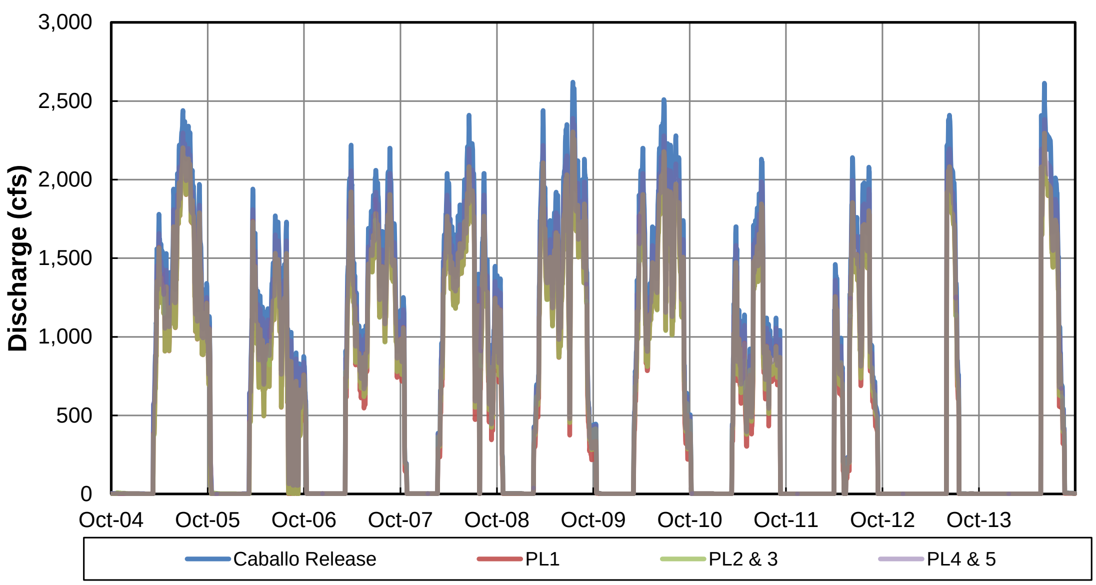

Figure 38. Measured Caballo release hydrograph and the estimated hydrographs at the upstream limit of Problem Locations 1 through 6 for the model validation period from WY2005 to WY2014.

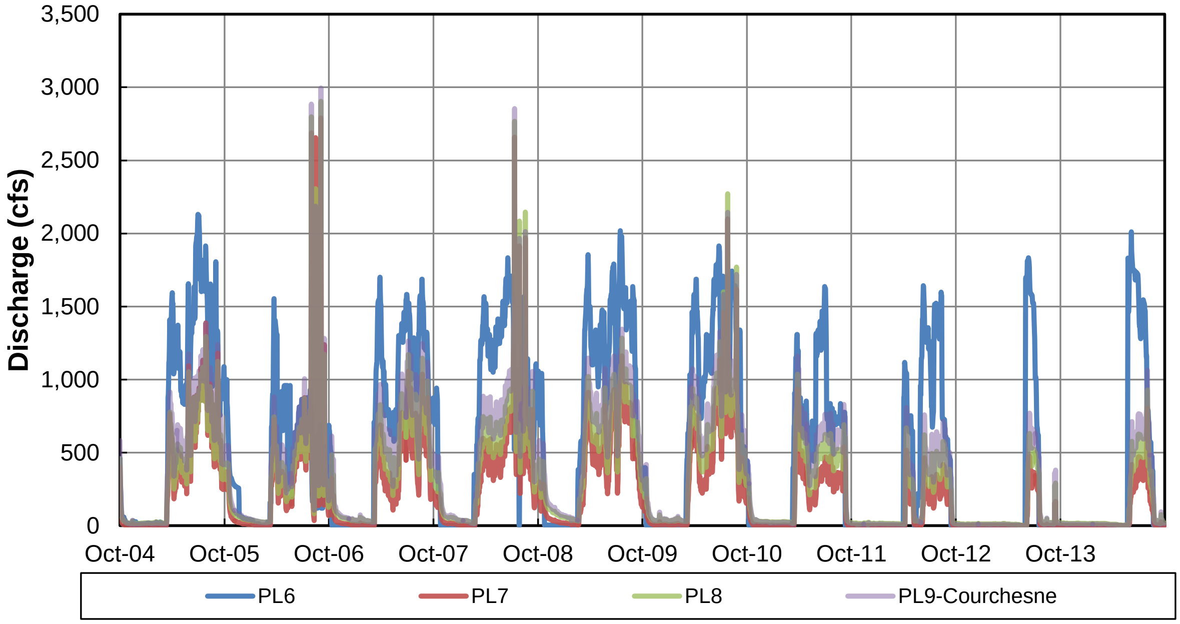

Figure 39. Measured hydrograph at the El Paso (Courchesne) gage that was used to represent the upstream inflow to Problem Location 9 and the estimated hydrographs at the upstream limit of Problem Locations 7 and 8 for the model validation period from WY2005 to WY2014.

5.1.3.2 Hydrology for Alternative Evaluation

Hydrographs were also prepared for the quasi-unsteady flow input to the sediment-transport simulations that were used to evaluate the alternatives. The hydrographs were developed to represent normal operating conditions, which based on the Statement of Work for this study, are represented by a discharge of 2,350 cfs in the reach upper reach and 1,400 cfs in the lower reach. For this study, it was assumed that the “upper” reach represents the reach upstream from Mesilla Dam, and the “lower” reach represents the reach downstream from Mesilla Dam.

During the course of this study, stakeholders expressed concern that “normal operating conditions” are not representative of the current drought condition. As a result, a variety of Caballo release hydrographs were considered to provide a basis for the hydrographs that were used in the simulations. The release hydrographs that were considered included the 2009 release, which represents a relatively normal year with a release volume of about 693,500 (February 17 to October 13), the 2012 release which represents a dry year with a release volume of about 371,400 ac-ft (April 1 to September 13), and the 2013 release which represents a very dry year with a release volume of about 168,300 ac-ft (June 1 to July 17). The 2013 release hydrograph was ultimately selected as the basis for the hydrographs, at the direction of USIBWC, since this year is representative of severe drought conditions that need to be considered during the planning of adaptive management strategies such as the channel maintenance alternatives considered herein. Consistent with the validation hydrographs, the hydrologic sequences that represent the upstream inflows to Problem Locations 1 through 6, the measured 2013 Caballo release hydrograph was adjusted to represent the unknown inflows and outflows along the RGCP using the flow-duration curves presented in USACE (2007) (Figure 37). The hydrographs were then further adjusted to include slightly reduced peak discharges of 2,350 cfs during the period between June 7 and June 15 (**Figure 40**). For Problem Locations 7 through 9, the measured 2013

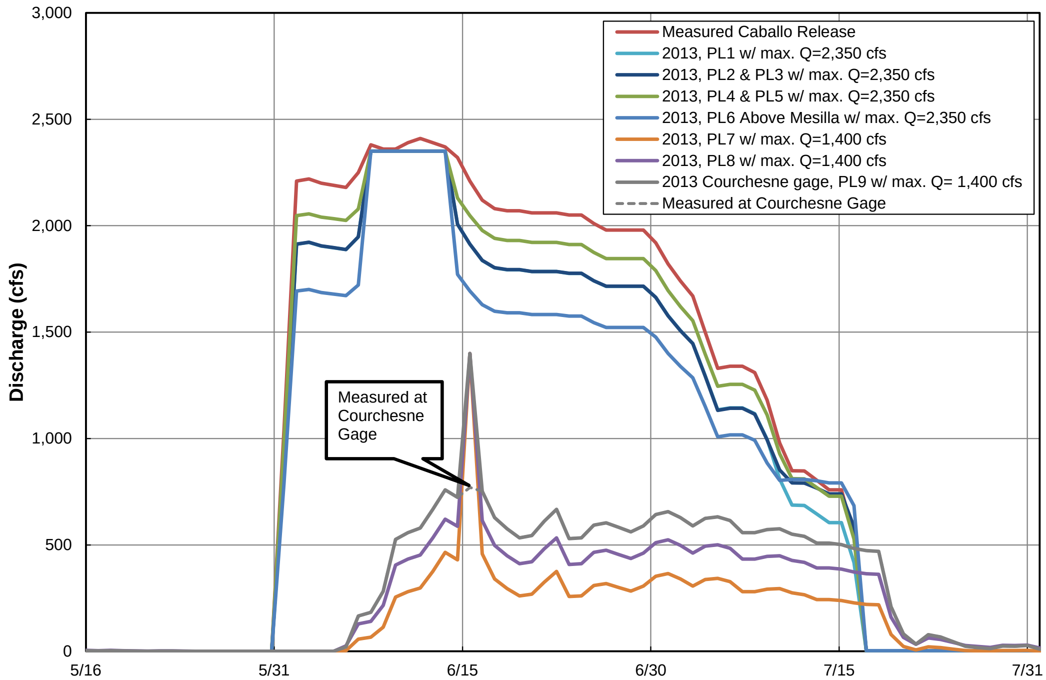

Figure 40. Measured Caballo release hydrograph in WY2013 and the adjusted 2013 hydrographs that were used to define the upstream inflow to each of the problem locations for the alternative evaluation models. Note the adjusted 2013 hydrographs were duplicated over 10 consecutive years in the quasi-unsteady flow model input.

flows at the Courchesne gage were first adjusted to represent the flow at the problem location using the USACE (2007) flow duration curves, and then the peak discharge on June 16 was increased to a discharge of 1,400 cfs (Figure 40). Once the representative hydrographs were prepared for each of the problem locations, the hydrographs were repeated over a 10-year period to assess the long-term effects of the alternatives. During periods when the estimated hydrographs had essentially no flow, nominal base flows of less than 1 cfs were incorporated in the hydrologic datasets to ensure model execution.

In addition to the flow sequences that represent “normal” operating conditions, separate quasi-unsteady flow files were prepared for the routed, 100-year storm event. These hydrographs were prepared using the results from the existing FLO-2D model of the RGCP. To extract the hydrographs, FLO-2D “floodplain cross sections” were added to the input files at points of interest along the problem location reaches that represent the upstream inflow and at locations of flow change (i.e., below the tributaries), and the model was re-run to obtain the desired output. As discussed in more detail below, significant model instabilities were encountered in the sediment-transport modeling of the 100-year event. As a result, the analysis of the 100-year event was revised to assess the long-term effects of the alternatives on the 100-year event water-surface profiles by extracting the predicted model geometry at the end of the simulations and incorporating the geometry into the steady-state hydraulic models. This approach addresses the fundamental question regarding the potential effects of the alternatives on flood conditions.

5.1.4 Sediment-transport Function Selection

Measured bed-material transport data are not available for the study reach, so it was not possible to select an appropriate sediment-transport function based on a comparison of predicted transport rates with measured data. Instead, the sediment-transport formulae were selected based on the range of bed-material sizes, hydraulic characteristics within the study reach, previous experience with similar channels, and the reasonableness of the predicted sediment-transport conditions. Seven bed-material transport functions are available in HEC-RAS (Version 4.1), including:

1. Ackers-White,
2. Engelund-Hansen,
3. Laursen (Copeland),
4. Meyer-Peter Müller,
5. Toffaletti,
6. Yang (sand and gravel), and
7. Wilcock.

For Problem Locations 1 through 5, where the bed material is bimodal (containing both sand and gravel or larger material) due to the influences of coarse-grained tributary sediment supplies, initial test runs were made using the Ackers-White, Meyer-Peter Müller (MPM), Yang (sand and gravel) and Wilcock formulae. A review of the results indicated that although the Wilcock equation (Wilcock, 2001) was developed for hydraulic conditions and bed-material sizes that include substantial quantities of sand and gravel similar to those at these problem locations, the model predicted significant oscillations in sediment loading over series of constant discharges, so use of this formula was ruled out. Results from the models that used the Yang formula indicated that the sediment-transport potential by size class (i.e., the amount of sediment transport that would occur if the bed were entirely made up of each size class) increases in the gravel-size class ranges. This result appears nonsensical, and as such the Yang formula was not further considered at these locations. A comparison of the results from the models at Problem Location 1 that used the Ackers-White and MPM formulae indicated that, at normal operation flows of 2,350 cfs, the Ackers-White based model transported up to the very fine gravel-size class (2 to 4 mm), whereas the MPM-based model transported size classes up to medium gravels (4 to 8 mm). Evidence of recent (2014 irrigation season) transport of gravels up to at least 8 mm was observed during the field reconnaissance, so the MPM formula was selected for use in the models of Problem Locations 1 through 5. The MPM formula computes the bed-load portion of the total sediment load and does not include the suspended load. While this limitation is significant in areas where the bed material is mostly sand and the suspended loads make up a substantial portion of the total sediment load, it is much less significant in areas where materials coarser than sand dominate the sediment-transport conditions such as in the vicinity of the arroyos evaluated at Problem Locations 1 through 5.

In a previous study for the URGWOPS EIS, MEI (2004) evaluated a range of possible transport equations that were developed for conditions similar to those at Problem Locations 6 through 9, and determined that the Yang (sand) equation (Yang, 1973) produced results that were the most consistent with the available measured data at the Rio Grande gages between Cochiti Dam and Elephant Butte Reservoir. Based on this conclusion and the general similarity of conditions between that study reach and those observed at the lower four problem locations, the Yang (sand) sediment-transport equation was also selected for use in the sediment-transport modeling at those problem locations. The Yang (sand) equation relates the concentration of the bed-material discharge to the rate of energy dissipation in the flow using dimensional analysis and the unit stream power concept.

5.1.5 Boundary Conditions

In addition to the upstream inflows discussed above, boundary conditions for the sediment-transport model include the downstream hydraulic boundary condition, the upstream sediment supply, and any tributary flow and sediment supplies. The gates at Mesilla Dam and at the canal headworks for the Eastside and Westside Canals are site-specific boundary conditions discussed for Problem Location 6, below.

5.1.5.1 Downstream Boundary Condition

For sediment-transport models that had a downstream boundary at the same location as the localized steady-state models, the downstream boundary condition for the sediment-transport models was input using a stage-discharge rating curve that was developed from the localized steady-state hydraulic models. For the sediment-transport models that were extended beyond the downstream boundary condition of the localized steady-state hydraulic models, the stage-discharge rating curve was obtained from the results of the updated base model of the overall RGCP. As discussed above, the portions of the reaches that were extended downstream from Problem Locations 5 and 8 included minor adjustments to the channel bed geometry to reflect the aggradation that was indicated at the downstream limit of the surveys. These adjustments were incorporated in a revised version of the updated base model of the overall RGCP, and the results from this model were then used to prepare the downstream stage-discharge rating curves.

5.1.5.2 Upstream Boundary Condition

Upstream sediment loads to each of the problem location reaches were estimated for the validation and base-model simulations using the HEC-RAS "Equilibrium Load" option, which computes the sediment load (and gradation of the sediment load) that is in balance with the sediment-transport capacity, thereby creating an equilibrium condition at the upstream limit of the model. Although the upstream limits of the models were extended with the intent of being outside of the hydraulic effects of the alternatives, initial test runs for the alternatives using the equilibrium load option indicated that in some cases the alternatives did have some effect on the sediment supply. Because a difference in the sediment supply under base and alternative conditions could mask the effects of the alternatives, it was necessary to ensure the upstream sediment supply was consistent among the model runs. Results from the base sediment-transport model simulations were therefore used to prepare sediment load time series that were input to the alternative models. Results from the base sediment-transport model runs were also used to generate incremental sediment load (by size class) versus total load rating curves, which were in turn used to prepare the gradation of the inflowing sediment load over the range of input total loads. For consistency, the final base model simulations were re-run using the inflowing sediment load time series input.

5.1.5.3 Tributary Boundary Conditions

Significant tributaries deliver flow and sediment to Problem Locations 1 through 7. Sediment load time series were developed for these tributaries using the average annual bed-load yields presented in Table 9. To develop the time series, it was assumed the average annual yield is delivered during single annual events that occur during the monsoon season (Table 9). Model instabilities could result where multiple tributaries deliver large amounts of sediment to the RGCP on a single simulation day. Therefore, events were staggered throughout the monsoon season that was assumed to extend from July 1 to September 1 (Table 9). Although staggering of the tributary sediment loadings may not represent the actual timing of the tributary sediment supply, considering the unknown timing of the monsoon events and the unknown volume of sediment delivered by these events, this assumption is not a significant limitation to the modeling. For each sediment loading event, a corresponding water discharge was specified using quasi-unsteady flow time series, the magnitude of which was determined by assuming the bed-load sediment concentration was about 5,000 ppm, which is a reasonably high bed-load concentration for flash-flooding arroyos such as those considered in this study. The assumed corresponding water discharges are also presented in Table 9. Because the 10th tributary event occurs near the end of the simulations (i.e., September 30, 2014, for the validation model runs) and there is insufficient time for the channel to respond to this last event, the 10th tributary event was not included in the simulations.

At some locations where the tributary sediment yields was very large, the sediment supplies overwhelmed the channel and resulted in model runtime errors. The tributary sediment supplies at these locations were distributed over an additional one to two cross sections below the section representing the confluence with the tributary to eliminate the model instabilities. In these circumstances, the total corresponding water discharges were assumed to enter at the upstream cross section that represents the confluence with the tributary.

5.1.6 Computational Time Steps

The overall flow duration associated with the hydrologic (quasi-unsteady flow) input is broken down into smaller computation intervals to insure the effects of changes in bed geometry are appropriately accounted for in the hydraulic computations. As discussed in the HEC-RAS User's Manual (USACE 2010), "…smaller computation increments will increase (model) run time, re-computing geometry and hydraulics too infrequently (e.g., computation increments that are too large) is the most common source of model instability." For this study, the computation interval was determined using procedures outlined in the Corps' Guidelines for the Calibration and Application of Computer Program HEC-6 (USACE 1992). These time-steps were then further reduced for flows that resulted in oscillations in the predicted gradation of the active layer. The resulting time steps range from 1.2 minutes for flows that exceed 10,000 cfs to 12 hours for flows that are less than 100 cfs (**Table 22**).

Table 22. Computation Increments used for the sediment-transport simulations.

Minimum Flow (cfs)	Maximum Flow (cfs)	Computation Increment (hours)	Computation Increment (minutes)
0	100	12.00	720.0
100	200	6.00	360.0
200	500	1.50	90.0
500	1,000	1.00	60.0
1,000	2,000	0.50	30.0
2,000	5,000	0.25	15.0
5,000	10,000	0.05	3.0
10,000	20,000	0.02	1.2

5.1.7 Other Model Input

Numerous computation options and tolerance settings are available for execution of the mobile boundary model. The bed exchange variable (also referred to as the SPI factor as defined in the HEC-6 software) is the number of iterations performed by the sorting and armoring algorithms for each calculation to account for changes in the amount of available bed material within the bed sediment reservoir. A value of 40 was initially selected for this option to enhance model stability, and then reduced until the model results did not change. A final SPI factor of 10 was used at all problem locations except Problem Location 4, where an SPI factor of 30 was used. The default value of 0.02 feet was used as the minimum bed change before the cross-sectional geometry is updated. The default value of 0.02 feet was also used for the minimum change in cross-section before the hydraulic conditions are re-computed. The mobile boundary model of HEC-RAS generates computational and remainder errors when the cross-sectional geometry is adjusted by the computed aggradation or degradation volume. Preliminary tests of the validation runs were carried out with and without the option to carry these errors over to the next time step to determine if use of this option had any effect on the model results. Because the option did not affect the results appreciably, the default option to carry these errors over to the next time step was used to reduce model run times.

The sediment routing is performed by comparing the upstream bed-material sediment supply with the transport capacity at each cross section. Cross-section weighting factors can be used in the model to average the hydraulic and sediment-transport conditions over more than one cross section to dampen the effects of abrupt changes in hydraulics between cross sections. Because the sediment aggradation or degradation within a control volume (i.e., the portion of the model represented by a single cross section) is most significantly affected by the hydraulic conditions at that cross section, no weighting factors were applied to the up- and downstream sections (i.e., a weighting factor of 0 was used for the upstream section, a weighting factor of 1 was used for the cross section representing the control volume, and a weighting factor of 0 was used for the downstream cross section). This weighting scheme was used at all of the problem locations except Problem Location 6, where use of non-weighted hydraulics resulted in model instabilities associated with flow and sediment optimizations at Mesilla Dam and the canal headworks. For this location, a weighting factor of 0.1 was used for the upstream cross section, a weighting factor of 0.8 was used at the cross section representing the control volume, and a weighting factor of 0.1 was used at the downstream section.

Similar weighting factors are applied to dampen the effects at the up- and downstream boundaries of the model. For the upstream boundary condition, a weighting factor of 1 was used for the upstream cross section and a weighting factor of 0 was used for the next downstream cross section since it was assumed that the transport capacity of the upstream cross section was in balance the sediment supply under the equilibrium load option used to define the sediment supply. For the downstream boundary condition, a weighting factor of 0.5 was used for the downstream section and a weighting factor of 0.5 was used for the next upstream cross section.

HEC-RAS includes the option for setting “pass-through nodes” that are defined as cross sections where no aggradation or degradation is allowed. At all problem locations, this option was used at the downstream cross section to ensure the stage-discharge rating cure that was used to define the downstream hydraulic boundary condition was appropriate through the entire simulation period.

Other sediment-transport related specifications that are available in HEC-RAS involve the control of overbank deposition, specification of the sorting method and definition of the fall velocity calculations. Initial test runs of the validation models were carried out to assess the effect of overbank deposition (i.e., deposition outside of the erosion limits that define the lateral extents of the bed sediment reservoir) on main channel sediment-transport characteristics. Most of the problem locations experienced very little but realistic overbank deposition, so deposition was allowed at these locations. Significant overbank deposition that resulted in unreasonable amounts of overbank sediment storage was indicated at Problem Locations 5 and 8, so overbank deposition was not allowed at these locations. The bed sorting method (sometimes called the mixing or armoring method) keeps track of the bed gradation which the model uses to compute grain-class specific transport capacities and can also simulate armoring processes which regulate supply from the bed. The Exner 5 bed-sorting method was used for this modeling, and fall velocities were computed using the Report 12 method, since both of these methods proved successful in the sediment-transport modeling of other similar reaches of the Rio Grande (MEI, 2007c; Tetra Tech, 2013).

5.1.8 Special Considerations at Problem Location 6

Problem Location 6 is a hydraulically complex configuration of diversions and returns controlled by gates. As discussed above, it was not possible to use HEC-RAS (Version 4.1) to conduct the sediment-transport modeling for Problem Location 6 because this version of the software is not capable of modeling sediment splits[5]. It was therefore necessary to use the beta test version of HEC-RAS Version 5.0 for the sediment-transport modeling at this location[6]. In addition to the complications associated with the flow and sediment splits, the operations of the Mesilla Dam gates and the gates for the canal headworks need to be considered. Although two-dimensional (2-D) modeling would better analyze the behavior of sediment movement in the vicinity of the dam, this type of modeling was beyond the scope of this study[7]. Instead, a number of simplifying assumptions were made to conduct the modeling in a 1-D platform. While these assumptions are believed to be reasonable for purposes of this evaluation of channel maintenance strategies, a more detailed evaluation of any alternatives selected for implementation is recommended. The following steps were carried out to conduct the sediment-transport modeling at this location:

1. Measured discharges entering the Eastside Canal and Westside Canal were compared with measured flows at the Picacho gage for the 2010 to 2012 period to develop a relationship between upstream inflows and flows diverted to each of the canals (**Figure 41**). The 2010 to 2012 period was selected because that data was made available to Tetra Tech for the Rio Grande Canalization Water Budget Study (Tetra Tech, 2013).
2. The HEC-RAS gate optimization feature determines the gate openings that are necessary to deliver specified flows to canals (or downstream reaches) for a given upstream water-surface elevation. This option was used to determine the gate openings at Mesilla Dam and the headworks to the canals that would be necessary to deliver the flows to the canals determined in Step 1 under the assumption that during the irrigation season the water surface upstream from Mesilla Dam is held at a constant elevation of 3869.4 feet (NAVD 88), or 3826.1 feet (BOR datum). For the Mesilla Dam gates, it was assumed that Gate #2 would be opened first, and if required Gate #12 would be the second to open. For the gate sets at both of the canal headworks, it was assumed the gates would open sequentially from left to right. At upstream river discharges of less than 10 cfs, it was assumed that the canals are essentially non-operational and that no effort is made to maintain specified water surfaces upstream from the dam. At these low discharges, an opening height of 1 foot was used for Mesilla Dam Gate #2, and nominal openings were used for the gates at the canal headworks because HEC-RAS requires non-zero flow in all reaches. Results from the gate optimization model runs are presented in **Table 23**.

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[5]Although HEC-RAS Version 4.1 is capable of optimizing flow splits at junctions that deliver flow to distributary channels, the sediment-transport simulation generates the following error when a sediment split is incorporated into the model geometry: "Currently, sediment in RAS functionalities only handle dendritic systems. Each junction should only have on reach leaving it. HEC-RAS does not yet handle sediment flow splits."

[6]Over the past 8 years, Tetra Tech has assisted the USACE Hydrologic Engineering Center (HEC) as beta-testers for HEC-RAS software development.

[7]The statement of work for this study required the use of 1-D HEC-RAS modeling for the evaluation of the alternatives.

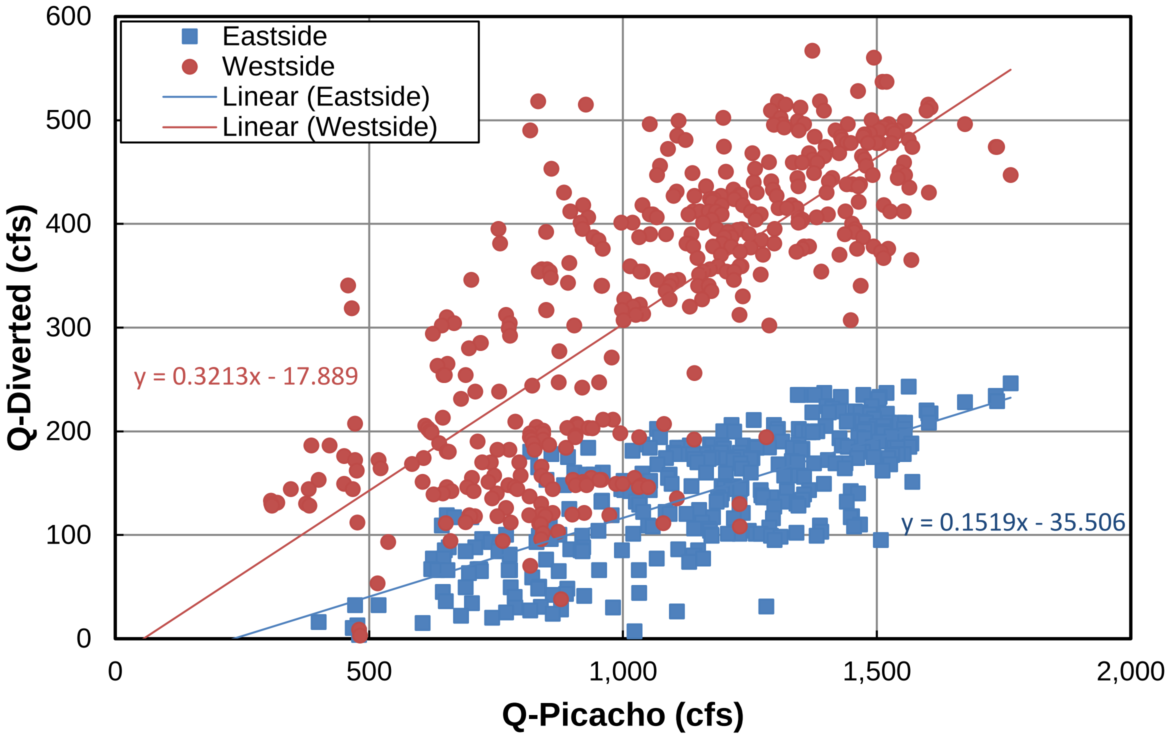

Figure 41. Measured flows at the Eastside Main Canal and Westside Main Canal gages as a function of the flows at the Rio Grande at Picacho gage during the period from 2010 to 2012.

Table 23. Optimized gate openings at Mesilla Dam and the Eastside and Westside Main Canal Headworks required for a range of upstream and diversion discharges and an upstream water-surface elevation of 3869.4 feet (NAVD 88).

Discharge (cfs)				Gate Opening Height (ft)						
Above Mesilla	Eastside Canal	Westside Canal	Below Mesilla	Mesilla #2	Mesilla #12	East #1	East #2	West #1	West #2	West #3
0.3	0.1	0.1	0.1	1.00	0.00	0.10	0.00	0.10	0.00	0.00
10	0.2	0.2	9.6	0.01	0.00	0.02	0.00	0.01	0.00	0.00
20	0.3	0.3	19.4	0.02	0.00	0.02	0.00	0.02	0.00	0.00
50	0.4	0.4	49.2	0.09	0.00	0.03	0.00	0.02	0.00	0.00
100	0.5	0.5	99.0	0.23	0.00	0.04	0.00	0.03	0.00	0.00
200	0.6	0.6	198.8	0.63	0.00	0.05	0.00	0.03	0.00	0.00
299.9	0.7	0.7	298.5	0.95	0.00	0.03	0.00	0.03	0.00	0.00
300	10.1	78.5	211.4	0.20	0.00	0.19	0.00	1.43	0.00	0.00
400	25.3	110.6	264.1	0.28	0.00	0.48	0.00	2.02	0.00	0.00
500	40.4	142.8	316.8	0.36	0.00	0.76	0.00	2.60	0.00	0.00
600	55.6	174.9	369.5	0.45	0.00	1.05	0.00	3.19	0.00	0.00
700	70.8	207.0	422.2	0.54	0.00	1.34	0.00	3.75	0.03	0.00
800	86.0	239.2	474.8	0.64	0.00	1.62	0.00	3.75	0.61	0.00
900	101.2	271.3	527.5	0.75	0.00	1.91	0.00	3.75	1.20	0.00
1000	116.4	303.4	580.2	0.86	0.00	2.20	0.00	3.75	1.78	0.00
1100	131.6	335.5	632.9	0.98	0.00	2.48	0.00	3.75	2.37	0.00
1200	146.8	367.7	685.6	1.10	0.00	2.77	0.00	3.75	2.96	0.00
1300	162.0	399.8	738.2	1.23	0.00	3.05	0.00	3.75	3.54	0.00
1400	177.2	431.9	790.9	1.36	0.00	3.34	0.00	3.75	3.75	0.38
1500	192.3	464.1	843.6	1.50	0.00	3.63	0.00	3.75	3.75	0.97
1600	207.5	496.2	896.3	1.65	0.00	3.75	0.17	3.75	3.75	1.54
1700	222.7	528.3	949.0	1.80	0.00	3.75	0.45	3.75	3.75	2.13
1800	237.9	560.5	1001.6	1.96	0.00	3.75	0.74	3.75	3.75	2.71
1900	253.1	592.6	1054.3	2.12	0.00	3.75	1.03	3.75	3.75	3.29
2000	268.3	600.0	1131.7	2.38	0.00	3.75	1.31	3.75	3.75	3.43
2100	283.5	600.0	1216.5	2.68	0.00	3.75	1.60	3.75	3.75	3.43
2200	298.7	600.0	1301.3	3.00	0.00	3.75	1.88	3.75	3.75	3.43
2300	313.9	600.0	1386.1	3.35	0.00	3.75	2.17	3.75	3.75	3.43
2400	329.1	600.0	1470.9	3.72	0.00	3.75	2.46	3.75	3.75	3.43
2500	344.2	600.0	1555.8	4.14	0.00	3.75	2.74	3.75	3.75	3.43
2600	350.0	600.0	1650.0	4.65	0.00	3.75	2.85	3.75	3.75	3.43
2700	350.0	600.0	1750.0	12.28	0.00	3.75	2.85	3.75	3.75	3.43
2800	350.0	600.0	1850.0	12.28	0.11	3.75	2.85	3.75	3.75	3.43
2900	350.0	600.0	1950.0	12.28	0.27	3.75	2.85	3.75	3.75	3.43
3000	350.0	600.0	2050.0	12.28	0.48	3.75	2.85	3.75	3.75	3.43

3. The gate openings determined in Step 2 (Table 23) were then used to prepare rating curves that relate the gate opening heights to upstream river discharge, which were in turn used to prepare gate opening time series that were input into the sediment-transport model.
4. Based on the results from the version of the localized steady-state hydraulic model without gate optimizations, stage-discharge rating curves were used to specify the downstream boundary condition for the RGCP, Eastside Canal, and Westside Canal reaches.
5. In addition to the pass-through node that was used at the downstream limit of the RGCP reach, pass-through nodes were also specified at the downstream limits of the canals, and at the artificial cross sections that were required upstream from the canal headworks. A zero-depth bed sediment reservoir was used for the portions of the canals that are covered by concrete, and a nominal (0.25 feet) bed sediment reservoir depth was used along the remainder of the canals to represent the thin veneer of sand that overlays the hardened canal beds.
6. The sediment-transport model was initially executed over the full 10-year simulation periods for the validation and base model hydrology using the HEC-RAS flow/sediment optimization feature at the Mesilla Dam junction. The HEC-RAS default option for flow-weighted sediment splits were used in the simulations. While these runs indicated that the gate opening time series resulted in the desired flow splits into the canals, the optimizations at discharges of less than 20 cfs often did not converge. Since these discharges do not convey considerable amounts of sediment, the quasi-unsteady flow time series and corresponding sediment time series were revised to remove dates that had inflowing discharges of less than 20 cfs. The resulting simulation for the validation model spans a period of about 5.4 years. Similar adjustments were also required for the 10-year base-model simulation that included the representative hydrograph which was duplicated over a 10-year period (Figure 40). This hydrograph has flows in excess of 20 cfs over a 47-day period from June 1 to July 18, so the resulting simulation period covers 470 days.

5.2 Sediment-transport Model Validation

The base sediment-transport models were validated by executing the models over the estimated 10-year hydrologic time series from WY2005 to WY2014, and comparing the predicted aggradation/degradation trends at the end of the simulations with surveyed or estimated changes in channel bed elevation. Because changes in thalweg elevation may not be representative of the overall erosion or deposition that occurs along a cross section, the change in mean bed elevation was used instead of the change in thalweg elevation. The primary data that was used to validate the models were obtained from the repeat cross section surveys conducted in 2004 and 2014/2015. This information was used to compute the mean bed elevation at the time of each survey to determine the change in mean bed elevation over that 10-year period. In addition to the change in mean bed elevation data obtained from the surveyed cross sections, similar data was computed using the estimated non-survey sections in the original (2004) and updated (2014/2015) base models. At Problem Locations 5 through 8, no repeat cross-section survey information was available, so the estimated non-survey section-based data were the only data available for comparison with the predicted changes.

It should be noted that a number of limitations associated with the models could affect the validation, including:

- The base models include the existing channel geometry and the existing gradation of the bed material at the start of the simulation, and not the geometry and bed material gradation that was present in 2004.

- The hydrologic sequence used in the validation runs may be somewhat different than the actual hydrology that occurred during the 10-year validation period.
- The timing, duration and magnitude of the actual water and sediment inflows from the tributaries is not known and was therefore assumed.
- The simulations do not reflect sediment removal activities performed by USIBWC during the validation time period.
- The hydraulic roughness values used in the base models represent the existing vegetation condition and do not reflect any changes to the vegetative cover that has occurred since 2004.

As a result, the trends in aggradation/degradation were considered of highest importance in the model validation, while the magnitude of the changes was considered of secondary importance. The comparison of the computed change in mean bed elevation and observed or estimated changes indicates that, in general, the predicted trends in aggradation and degradation match the observed/estimated dataset reasonably well at each of the problem locations.

- At Problem Location 1 (**Figure 42**), the model predicts over 3 feet of aggradation in the vicinity of Tierra Blanca Creek which is about twice that indicated by the estimated change. This difference is to be expected considering the model does not consider USIBWC activities that removed about 4,740 CY of material prior to the 2014 irrigation season. At Sibley Arroyo, where about 6,000 CY of sediment was relocated from the arroyo mouth to the opposite bank in the 2006/2007 non-irrigation season, the predicted aggradation and measured bed elevation change were approximately 1 foot. The overall trend in predicted aggradation and degradation at this location is similar to the measured and estimated changes, so the model results appear reasonable.
- The modeling at Problem Location 2 does not factor in the numerous sediment removal and relocation activities that occurred over the validation period. These activities included:
- Relocation of sediment from the mouth of Placitas Arroyo to the opposite bank in the non-irrigation seasons of 2005/2006 (4,000 CY) and 2006/2007 (6,750 CY).
- Removal of about 7,540 CY of sediment from the mouth of Placitas Arroyo in 2013.
- Clearing of vegetation from the mouth of Placitas Arroyo in the 2008/2009 non-irrigation season.
- Relocation of sediment from the mouth of one of the Thurman Arroyos to the opposite bank in the 2006/2007 non-irrigation season (7,250 CY).

Nevertheless, both the magnitude and trend of the model results match the measured and estimated changes reasonably well (**Figure 43**). This is not surprising since the only activity that removed sediment occurred at the mouth of Placitas Arroyo in 2013, where the model predicts over three feet of aggradation compared to the 1.8 feet of observed aggradation.

Only one repeat cross section was available over the relatively short reach of Problem Location 3, which shows about 0.2 feet of aggradation (**Figure 44**). The predicted change in mean bed elevation of about 0.5 feet is similar to this data point, but the predicted changes along the remainder of the reach (less than 1 foot of aggradation) are somewhat less than the estimated changes (which exceed 4 feet in some locations). An investigation of the original base model geometry indicated that the cross-sectional geometry at and upstream from the siphon may have been influenced by standing water that extended about 1,000 feet upstream from the siphon, and therefore may not be correct. While this may explain the differences in this portion of the reach, the differences outside of this area warranted further evaluation. To better assess the model

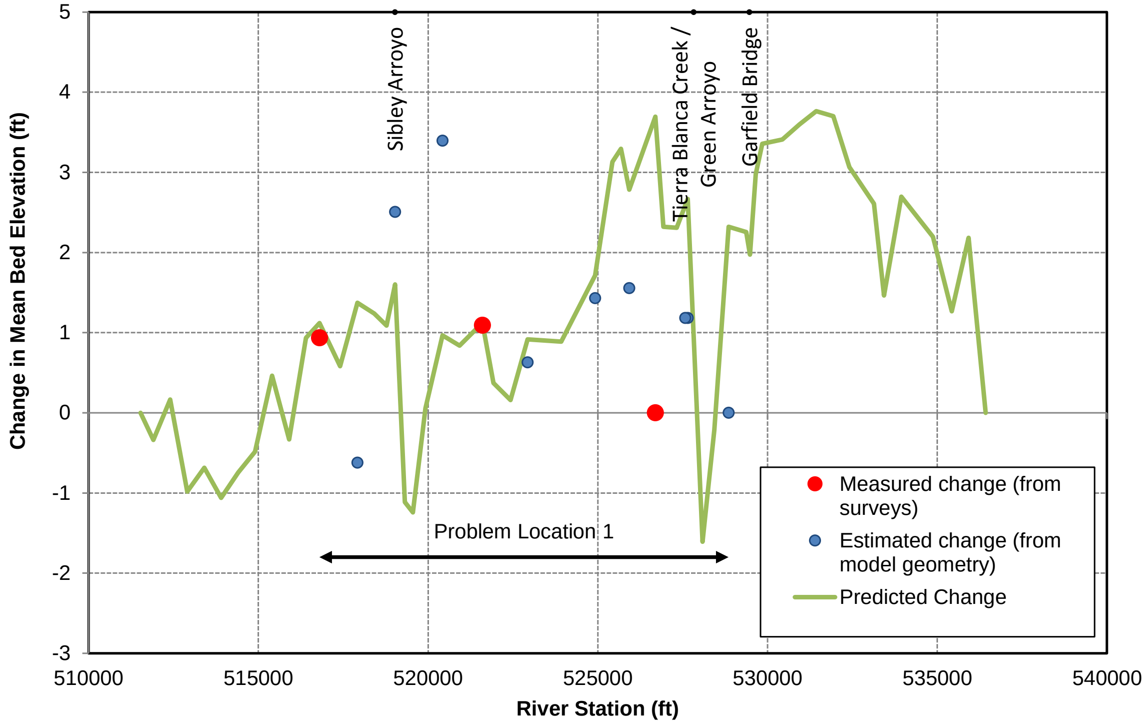

Figure 42. Comparison of observed (survey-based) or estimated (base model cross section-based) change in mean bed elevation and the predicted change in mean bed elevation at Problem Location 1.

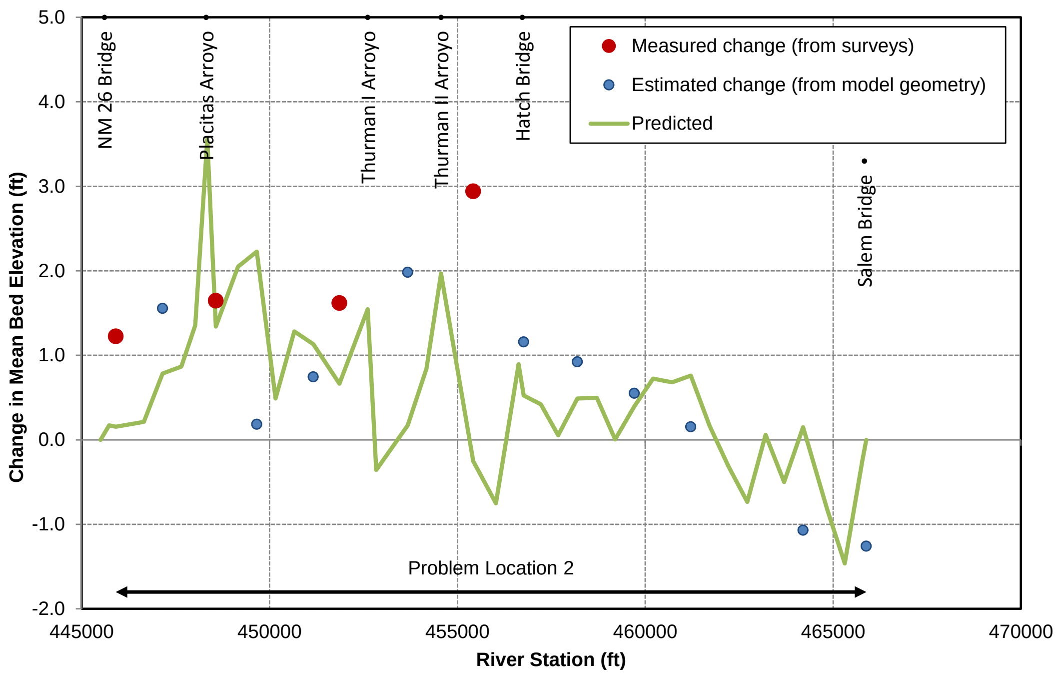

Figure 43. Comparison of observed (survey-based) or estimated (base model cross section-based) change in mean bed elevation and the predicted change in mean bed elevation at Problem Location 2.

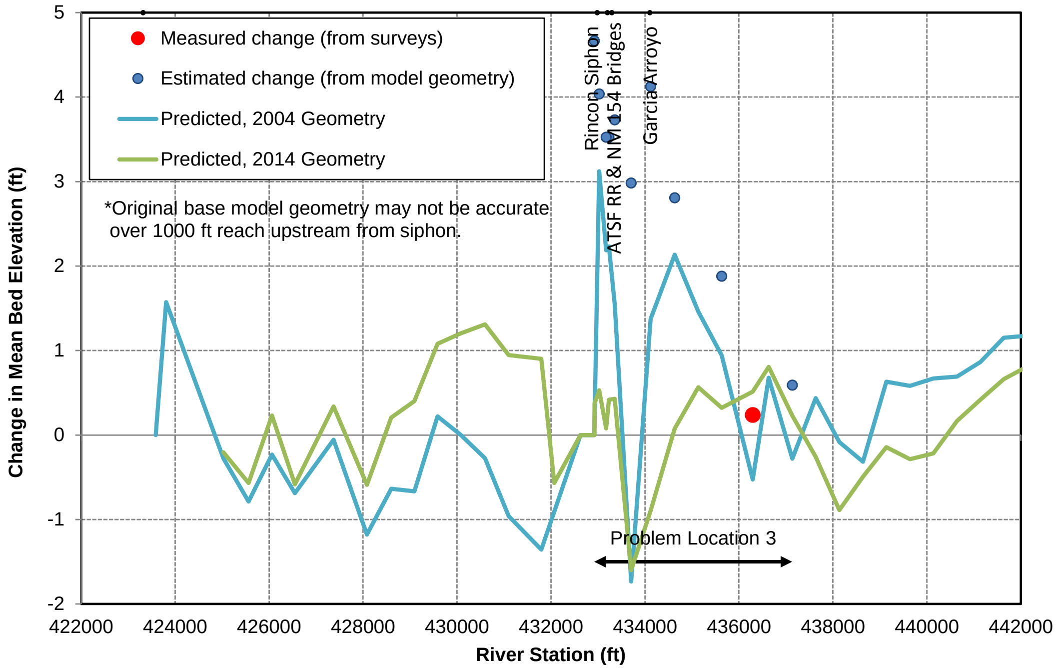

Figure 44. Comparison of observed (survey-based) or estimated (base model cross section-based) change in mean bed elevation and the predicted change in mean bed elevation from the models with 2014 and 2004 cross sections at Problem Location 3.

behavior in this apparent zone of high deposition, the model was revised to include the 2004 base-model geometry and re-run. Results from this model indicate a much better match between the predicted and estimated trends in aggradation; thus, the model with the 2014 geometry is considered to be a reasonable tool for use in this evaluation of channel maintenance strategies.

One of the most significant sediment-removal activities that occurred at any of the problem location arroyos was undertaken at the mouth of the Rincon Arroyo, where about 21,190 CY of material was removed in early 2014. This activity was not incorporated into the modeling of Problem Location 4, and does not explain the difference between the predicted aggradation upstream from Rincon Arroyo of about 1.7 feet and the estimated change of nearly 3 feet (**Figure 45**). Other than the comparison at this data point, the comparison shows good agreement between the predicted and measured/estimated changes along the upstream portion of the reach. However, in the downstream half of the reach, the model results indicate general equilibrium and degradation at some locations, whereas the measured and estimated changes show about 1 foot of aggradation. This difference was investigated further by replacing the 2014 model geometry with the 2004 model geometry, similar to the evaluation conducted at Problem Location 3. Results from the model with the 2004 geometry indicate similar trends between the predicted and measured/observed data along the majority of the reach. Considering the significance of the very high sediment loads delivered by Rincon Arroyo, the bed-load fraction of which is estimated to be 20,440 CY/yr (12.67 ac-ft; Table 9), and the unknown actual timing and magnitude of these sediment supplies, the base model with the 2014 geometry was deemed sufficient for this study.

No repeat cross-section surveys were available at Problem Location 5, but the estimated changes in mean bed elevation indicate aggradation levels of less than 1 foot through most of the reach (**Figure 46**). Exceptions to this general statement are at the confluence with Rock Canyon, where

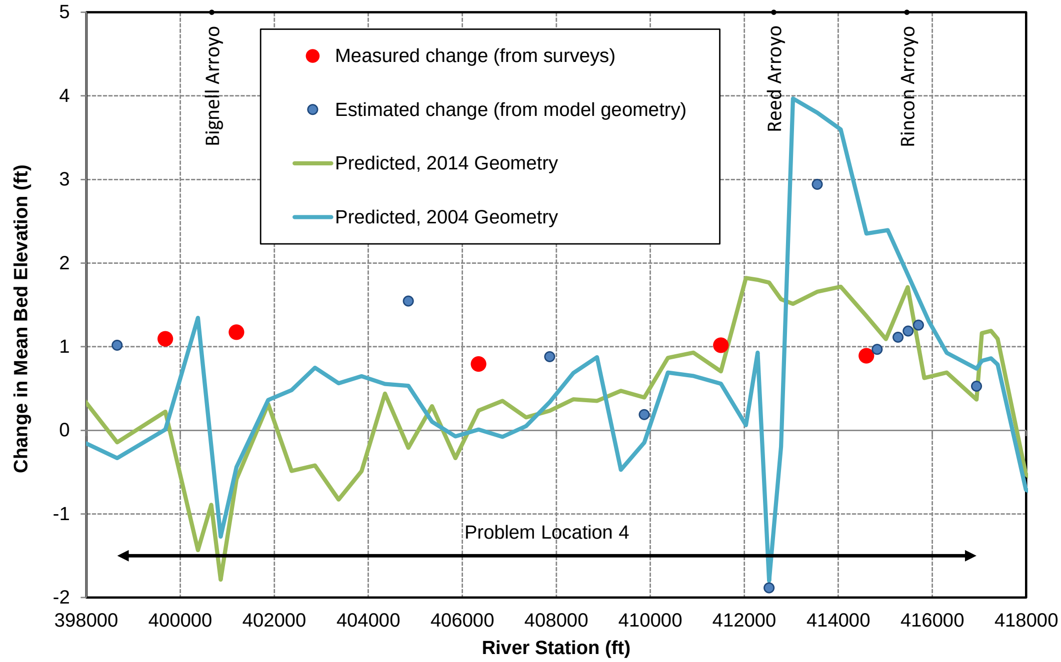

Figure 45. Comparison of observed (survey-based) or estimated (base model cross section-based) change in mean bed elevation and the predicted change in mean bed elevation from the models with 2014 and 2004 cross sections at Problem Location 4.

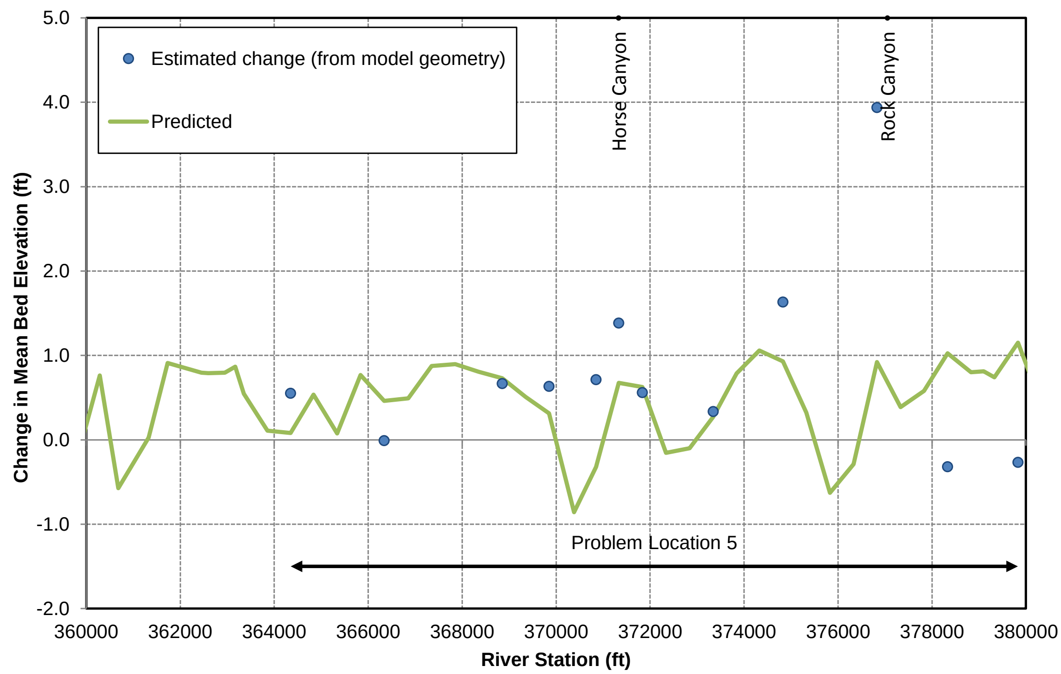

Figure 46. Comparison of estimated (base model cross section-based) change in mean bed elevation and the predicted change in mean bed elevation from the model at Problem Location 5.

about 4 feet of aggradation is indicated, and at the confluence of Horse Canyon Creek that is opposite the Rincon/Tonuco Drain, where about 1.5 feet of aggradation is indicated. Although the model does not predict aggradation levels of these amounts at the mouths of the canyons, the predicted trends are similar to the estimated trends and the aggradation levels are similar through the majority of the reach. This indicates that the adopted mean annual sediment loading, especially from Rock Canyon, may underrepresent the actual loading that occurred during the 10-year validation period. This consideration should not be a significant limitation in using the models for evaluating the relative effects of the alternatives.

As expected, the estimated changes in mean bed elevation at Problem Location 6 show aggradation along the majority of the reach upstream from Mesilla Diversion Dam, with levels reaching almost 2 feet upstream from the dam (**Figure 47**). Results from the model show similar trends of consistent aggradation above the dam, but the magnitudes are slightly higher, especially at the upstream face of the dam, where the model predicts nearly 4 feet of aggradation. These differences are not surprising considering that the model does not include USIBWC activities that removed 60,000 CY of material upstream from Mesilla Dam in the 2005-2006 non-irrigation season. The model results also are generally consistent with the estimated changes downstream from the dam, which show a general trend of degradation along most of this portion of the study reach.

The predicted changes in mean bed elevation are also slightly higher than the estimated changes in the upstream portion of Problem Location 7, where estimated aggradation levels are generally less than 1 foot (**Figure 48**). However, the predicted trend of aggradation along the majority of the reach is consistent with the estimated changes, and the magnitude of the change is very similar at and downstream from Vinton Bridge, so the results from this model appear reasonable.

At Problem Location 8, the predicted changes match the measured changes very well in the reach upstream from Country Club Bridge, but are under-predicted in the downstream portion of the reach (**Figure 49**). Although the cross-sectional geometry in the downstream extended portion of this reach below the extents of the survey was adjusted to reflect estimated aggradation levels (the bed geometry in this reach was adjusted up by 1.7 feet based on the downstream-most estimated change in mean bed elevation point), it is highly possible that more significant aggradation occurred in this reach than what is reflected in the adjusted sections. No information was available to make further adjustments to the extended portion of the reach, and considering the overall trend of predicted aggradation is similar to the estimated trend of aggradation, the model of this location should be sufficient for this study.

USIBWC removes sediment from the reach upstream from American Dam on a relatively frequent basis. During the 10-year validation period, sediment was removed in the non-irrigation seasons of 2005/2006 (20,000 CY), 2006/2007 (10,340 CY) and 2010/2011 (14,060 CY)[8]. Consistent with the models of other problem locations where sediment removal activities occurred but were not incorporated into the models, the predicted levels of aggradation at Problem Location 9 are higher than the observed and estimated changes (**Figure 50**). This is especially true in the reach immediately upstream from the dam, where the model predicts about 3 feet of aggradation compared to the very small change indicated by the survey data. Because the overall trend of consistent aggradation that is predicted through the reach is consistent with the measured/estimated trends, and the predicted magnitude of aggradation without sediment removal should be higher than the actual change, the results from this model appear within reason.

[8]An additional 14,200 CY of material was removed after the validation period in May 2015.

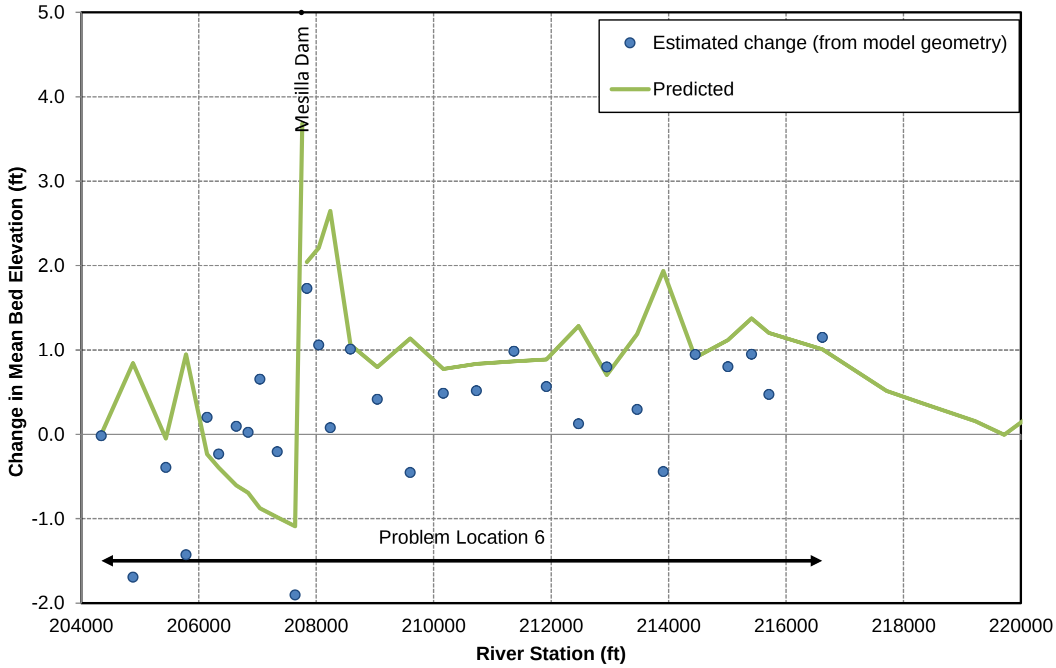

Figure 47. Comparison of estimated (base model cross section-based) change in mean bed elevation and the predicted change in mean bed elevation from the model at Problem Location 6.

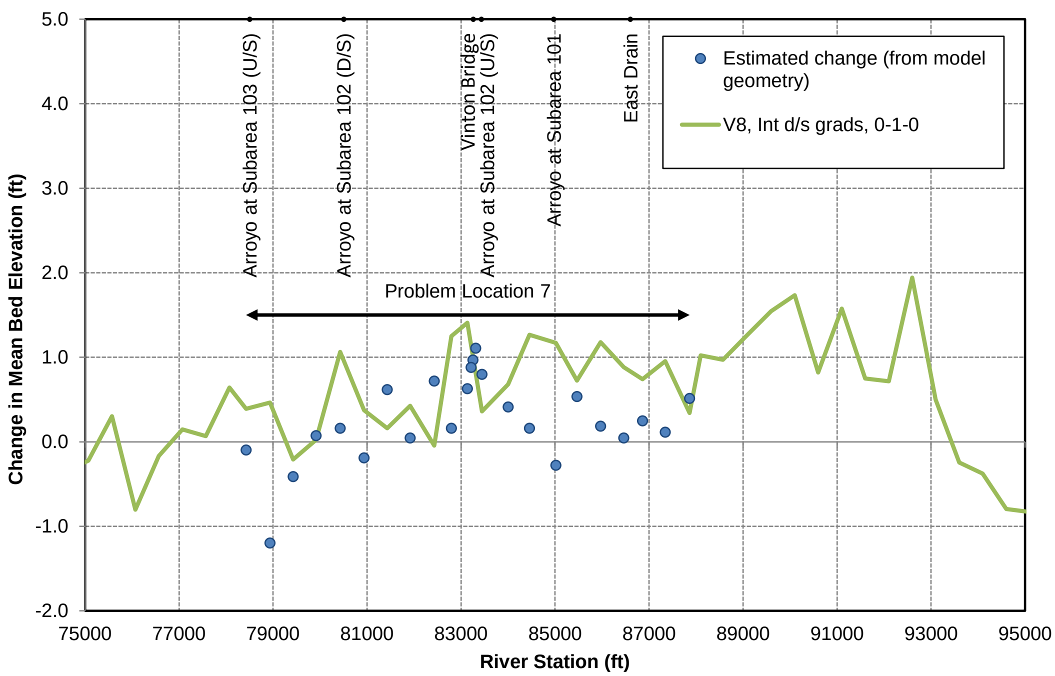

Figure 48. Comparison of estimated (base model cross section-based) change in mean bed elevation and the predicted change in mean bed elevation from the model at Problem Location 7.

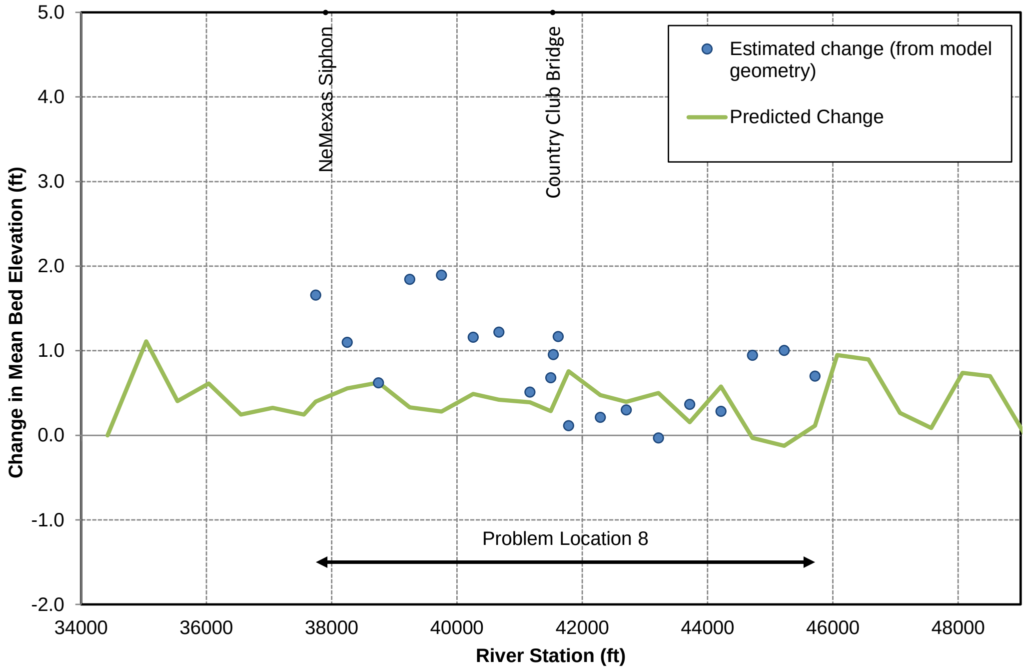

Figure 49. Comparison of estimated (base model cross section-based) change in mean bed elevation and the predicted change in mean bed elevation from the model at Problem Location 8.

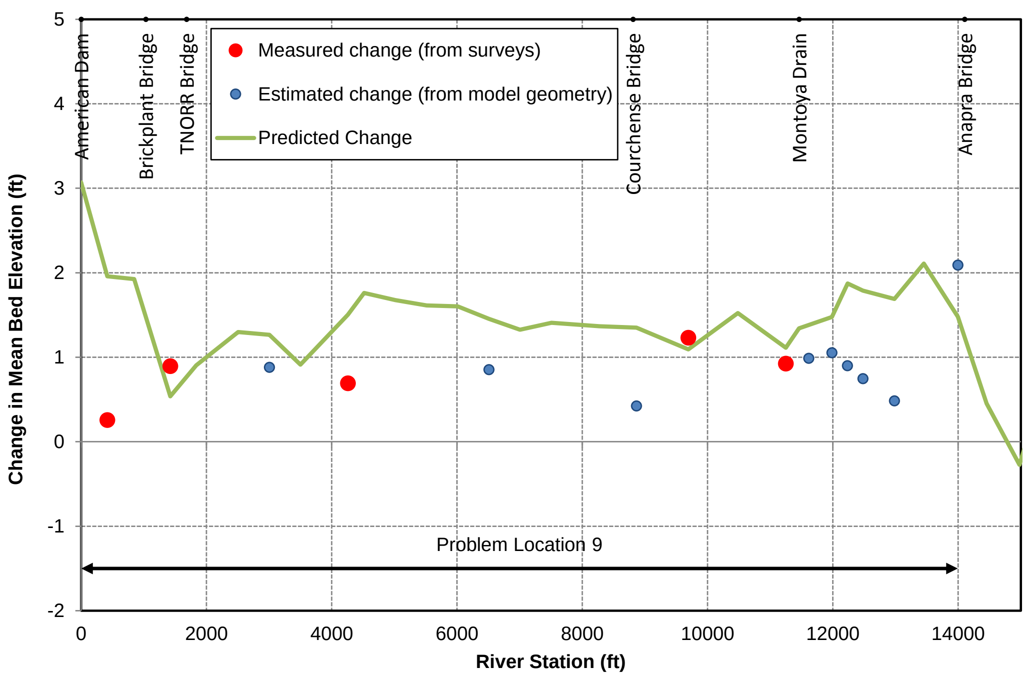

Figure 50. Comparison of observed (survey-based) or estimated (base model cross section-based) change in mean bed elevation and the predicted change in mean bed elevation from the model at Problem Location 9.

5.3 Alternative Sediment-transport Model Development

5.3.1 General Alternative Model Development

In general, the models for the alternatives were developed by inserting the model geometry from the localized steady-state hydraulic model of each alternative. All of the adjustments that were made to the localized base model geometries for purposes of the sediment-transport modeling (i.e., removal of bridges and irregularly spaced cross sections, changes to ineffective flow areas, minor adjustments to the geometry of the channel thalweg, and extension of the models in the up- and downstream directions) were also made to the geometries for the alternative sediment-transport models. The geometry and gradation of the bed sediment reservoir that was used in the base model was typically not changed, except for alternatives that included in-channel structural measures that would not erode. These alternatives included the spur dike alternatives at Problem Locations 4, 5, 7, 8 and 9 and the riprap alternative at Problem Location 8, where the limit of erosion was moved to the toe of the structure to ensure unrealistic erosion into the structure did not occur. The remainder of the model input for the alternative models was identical to that for the base models, except for the modeling of the sediment trap alternatives and the alternative modeling at Problem Location 6, as discussed below.

5.3.2 Modeling of the Sediment Trap Alternatives

Direct incorporation of the individual sediment traps into the models of the sediment trap alternatives was beyond the scope of this study. Rather, a generalized qualitative assessment of the sediment traps was carried out to determine the likely sediment trap efficiencies. This assessment involved the modeling of a representative sediment trap and computation of the sediment trap efficiencies by size fraction.

The proposed sediment trap in Placitas Arroyo was selected for the modeling because of the wide range of sediment sizes that this tributary delivers to the Rio Grande, and because this trap would not require excavation and the geometry of the trap could be inferred from the existing topography. A short HEC-RAS model of the 1,500-foot reach upstream from the confluence with the Rio Grande was prepared using the 2011 LiDAR mapping, with cross sections laid out in a manner such that the trap fences could be inserted. The downstream boundary condition for this model was set by assuming the events occur during periods of low flow in the Rio Grande, a conservative assumption that would reduce the predicted trap efficiencies because the traps are not affected by backwater from the Rio Grande. The gradation of Placitas Arroyo fan that was used in the model for Problem Location 2 was used to define the gradation of the bed sediment reservoir. Simple hydrographs with a duration of 1 day and a triangular shape were prepared for the 2- and 5-year peak flows based USGS Regional Regression relationships (Waltemeyer, 1996). (These flows were selected because they bracket the mean annual flood event.) These hydrographs were then coded into the quasi-unsteady flow editor of HEC-RAS. The models without the fences were then executed using the equilibrium sediment supply option to determine the load and gradation of the upstream sediment supply. The model geometry was then adjusted to reflect the sediment traps by incorporating inline structures (sharp crested weirs) at the trap screens. Although the proposed trap screens are 4 feet tall, the inline structures were coded in to a height of 1 foot under the conservative assumption that the effective blockage height of the screen would not exceed 1 foot. The computed equilibrium sediment supply load and gradation from the without-trap model was then used to define the inflowing sediment load time series, and the models were executed over the same two events.

As expected, results from the models indicate that the sediment traps would have relatively high trap efficiencies. At the 2-year event, the total trap efficiency is about 67 percent, and the trap efficiency by size class ranges from 42 percent of the very fine sand size class (<0.125 mm) to

100 percent of the sizes greater than fine gravel (>8 mm) (**Figure 51**). At the 5-year event, the total trap efficiency increases to about 82 percent, and the trap efficiency by size class ranges from 56 percent of the very fine sand size class to 100 percent of the sizes greater than fine gravel (>8 mm) (**Figure 52**).

This information was then used to adjust the magnitude and gradation of the sediment load delivered by the tributary to the sediment trap alternative models of the problem locations under the assumption that each of the sediment traps would have similar trap efficiencies. Although the sediment trap efficiencies at the mean annual event are probably somewhere between the predicted trap efficiencies at the 2- and 5-year events, the tributary loadings were conservatively adjusted using the results from the 2-year event. The results from the sediment trap alternative model runs are presented along with the results of the other alternatives below.

5.3.3 Alternative Modeling at Problem Location 6

Consistent with the base modeling that was conducted at Problem Location 6, site-specific techniques were required to model the alternatives that were evaluated at this location. For the short and long excavation alternatives, the base model geometry was simply adjusted to include the excavated channel geometries, and it was assumed that no change to the gate opening time series would be required. This is a reasonable assumption because the water-surface elevation upstream from the dam was set to be constant at elevation 3869.4 feet (NAVD88), so the channel bed geometry upstream from the dam would not affect the gate openings necessary to deliver the desired flows to the canals and downstream river. The models were then executed over the same hydrologic simulation that was used for the base model run which had removed days with inflows of less than 20 cfs.

The other alternatives at this location (construction of check/sluiceway structures or vortex weirs in the canals and installation of automated gates on Mesilla Dam) are heavily influenced by the operations associated with each of those alternatives. A 1-year period is sufficient to evaluate the effects of these alternatives, so the base model was re-run over a single irrigation season from June 1 to July 18 for purposes of comparison to the alternative model runs.

5.3.3.1 Sluiceway and Check Structure Alternative Modeling

For the check/sluiceway structure alternative, the model geometry was adjusted to include the structures in the canal reaches. The structures were coded into the models as inline weirs based on the information presented in EBID (2014) (Appendix K). The 30-foot wide sluiceway channels identified in EBID (2014) were coded into the model as separate reaches, with a slope of 0.002 per the recommendation that the channels have a slope that exceeds 0.001. The flap gates for the check structure in the Eastside Canal were assumed to be laid down such that the effective gate height was 3 feet during the entire simulation (i.e., the 8.5-foot high gates are 5.5 feet open measured from the fully upright position). For the sluiceway structure in the Eastside Canal, it was assumed that a 6-foot wide, 3-foot tall radial gate would be used because EBID (2014) identified this type of gate as “ideal”. The flap gates for the check structure in the Westside Canal were assumed to be laid down such that the effective gate height was 4 feet during the entire simulation (i.e., the 7-foot high gates are 3 feet open measured from the fully upright position). For the sluiceway structure in the Westside Canal, it was assumed that an 8-foot wide, 3-foot tall radial gate would be used consistent with the type of gate used in the Eastside Canal sluiceway. The sluiceway gates for both structures were assumed to be fully open during the simulation.

Because the Mesilla Dam and canal headworks gate openings would need to be adjusted to deliver increased canal flows to operate the check structures, the localized steady-state hydraulic model that was used to optimize the Mesilla Dam and canal headworks gate openings was re-run with increased target canal discharges. For this analysis, it was assumed that the sluiceways

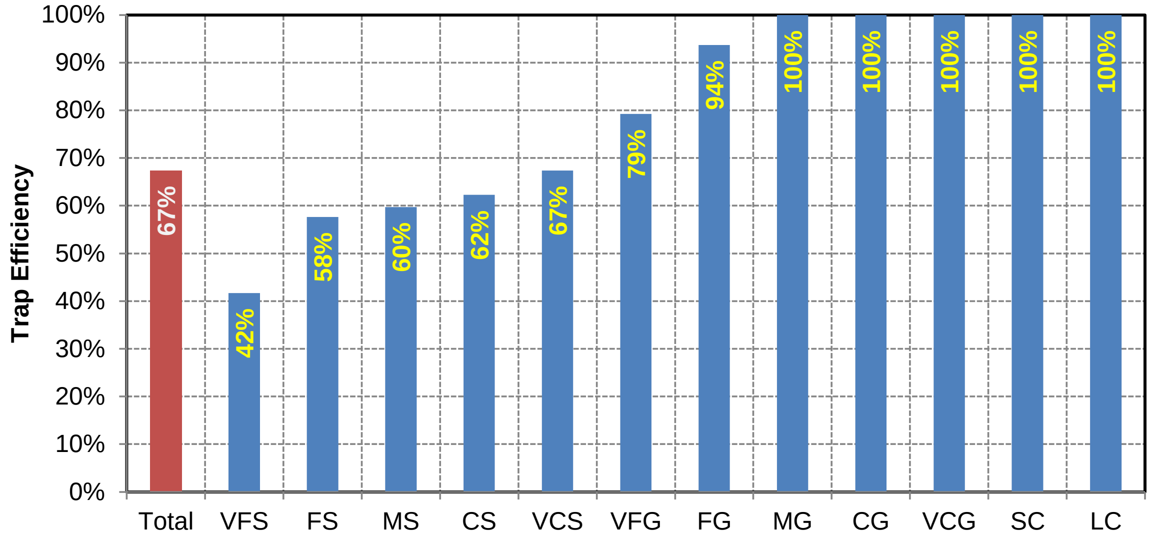

Figure 51. Total and by-size class sediment trap efficiencies predicted by the model of the sediment trap on Placitas Arroyo at the 2-year event.

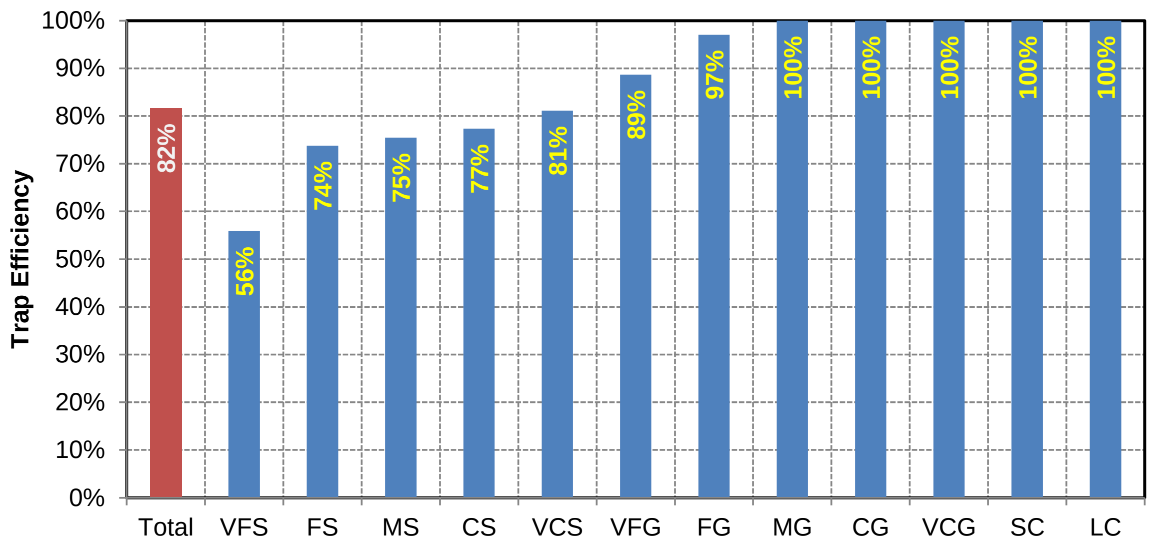

Figure 52. Total and by-size class sediment trap efficiencies predicted by the model of the sediment trap on Placitas Arroyo at the 5-year event.

would become fully operational at a total inflowing discharge of 2,000 cfs, and that the Eastside and Westside sluiceways would have a fully operational discharges of 300 and 400 cfs, respectively (**Table 24**)[9]. The resulting gate-opening versus inflowing discharge rating curves were then used to adjust the gate opening time series input for the model. The simulation was executed over the representative single irrigation season from June 1 to July 18 using the HEC-RAS flow/sediment optimization feature at the Mesilla Dam, Eastside Canal Sluiceway, and Westside Canal sluiceway junctions.

Initial model runs that included junctions representing return sluiceway flows and sediment loads to the Rio Grande resulted in model instabilities. As a result, these junctions were removed from the model, and the model was executed assuming the sluiceway flows and loads did not return to the Rio Grande. The results from this model were then used to prepare time series of the flow and sediment load delivered by the sluiceway channels, and these time series were input as point sources to the downstream river in the final check/sluiceway alternative model run.

5.3.3.2 Vortex Tube Alternative Modeling

Because the flow and sediment extraction of vortex tubes is controlled by three-dimensional flow patterns that involve very complex hydraulic conditions, 1-D modeling of the sediment-removal characteristics of the vortex tubes is not possible. The modeling for this alternative was therefore performed by assuming a reasonable geometric layout for the tubes and computing the “extraction ratio” (R) defined by Robinson (1960), which is the ratio of the water discharge extracted by the tube to the total discharge in the canal. This extraction ratio is a function of the hydraulic parameters at the tube inlet and the geometric layout of the tube:

$$R = c' \frac{\sqrt{\left(1 + \frac{dB}{2}\right)}}{\frac{Fr * d}{D}} \tag{5.1}$$

Where:	*c’*	=	Coefficient
	d	=	Average flow depth in the canal (ft)
	F_r	=	Froude number in the canal
	D	=	Slite opening of vortex tube (ft), and
	B	=	Depth of tube measured from bottom of tube to slit opening (ft)

The coefficient c’ is defined by the geometry of the tube as:

$$c' = 100 \left[\frac{A_t}{DLsin\theta}\right] \tag{5.2}$$

Where:	A_t	=	Area of tube (ft^2)
	L	=	Total length of tube (ft)
	Θ	=	Angle of tube relative to the direction of flow (degrees)

[9] The diversion discharges presented in Table 24 are based on the assumed target sluiceway flows. Higher diversion discharges would result in more sediment delivered to the canals and would require larger sluiceway flows to maintain the assumed downstream canal flows; lower diversion discharges would result in less sediment delivered to the canals and would require smaller sluiceway flows to maintain the assumed downstream canal flows. If different diversion and target sluiceway discharges are desirable, the steady-state hydraulic model with the gate optimization feature can be used to revise the gate opening rating curves and input time series.

Table 24. Optimized gate openings at Mesilla Dam and the Eastside and Westside Main Canal Headworks required for a range of upstream and diversion discharges under the canal check/sluiceway structure alternative and an upstream water-surface elevation of 3869.4 feet (NAVD 88).

Discharge (cfs)							Gate Opening Height (ft)															
Above Mesilla	Eastside Canal (Total)	Eastside Canal Below Check	Eastside Sluiceway Returns	Westside Canal (Total)	Westside Canal Below Check	Westside Sluiceway Returns	Mesilla #2	Mesilla #12	East #1	East #2	East #3	East #4	East #5	East# 6	West #1	West #2	West #3	West #4	West #5	West #6	West #7	West #8
300	20.0	0.0	20.0	110.1	33.3	76.9	0.46	0.00	0.35	0.00	0.00	0.00	0.00	0.00	1.87	0.00	0.00	0.00	0.00	0.00	0.00	0.00
400	50.6	13.5	37.1	154.7	70.2	84.5	0.53	0.00	0.89	0.00	0.00	0.00	0.00	0.00	2.64	0.00	0.00	0.00	0.00	0.00	0.00	0.00
500	80.8	31.5	49.3	198.7	105.5	93.3	0.60	0.00	1.42	0.00	0.00	0.00	0.00	0.00	3.41	0.00	0.00	0.00	0.00	0.00	0.00	0.00
600	111.3	53.1	58.2	245.6	140.1	105.5	0.66	0.00	1.95	0.00	0.00	0.00	0.00	0.00	3.75	0.43	0.00	0.00	0.00	0.00	0.00	0.00
700	141.8	73.3	68.6	289.6	167.0	122.6	0.73	0.00	2.49	0.00	0.00	0.00	0.00	0.00	3.75	1.19	0.00	0.00	0.00	0.00	0.00	0.00
800	171.8	96.3	75.5	334.6	190.0	144.6	0.80	0.00	3.02	0.00	0.00	0.00	0.00	0.00	3.75	1.96	0.00	0.00	0.00	0.00	0.00	0.00
900	203.1	112.5	90.6	380.2	220.2	160.0	0.86	0.00	3.56	0.00	0.00	0.00	0.00	0.00	3.75	2.73	0.00	0.00	0.00	0.00	0.00	0.00
1000	233.0	124.7	108.2	424.8	247.2	177.6	0.93	0.00	3.75	0.34	0.00	0.00	0.00	0.00	3.75	3.49	0.00	0.00	0.00	0.00	0.00	0.00
1100	263.0	141.9	121.1	469.3	283.3	185.9	1.00	0.00	3.75	0.87	0.00	0.00	0.00	0.00	3.75	3.75	0.51	0.00	0.00	0.00	0.00	0.00
1200	292.8	157.6	135.2	514.0	316.7	197.2	1.07	0.00	3.75	1.40	0.00	0.00	0.00	0.00	3.75	3.75	1.27	0.00	0.00	0.00	0.00	0.00
1300	324.5	175.5	149.0	559.5	346.8	212.7	1.13	0.00	3.75	1.94	0.00	0.00	0.00	0.00	3.75	3.75	2.03	0.00	0.00	0.00	0.00	0.00
1400	354.2	193.6	160.7	604.2	379.0	225.3	1.20	0.00	3.75	2.47	0.00	0.00	0.00	0.00	3.75	3.75	2.80	0.00	0.00	0.00	0.00	0.00
1500	384.3	211.6	172.6	648.8	415.7	233.0	1.27	0.00	3.75	3.00	0.00	0.00	0.00	0.00	3.75	3.75	3.57	0.00	0.00	0.00	0.00	0.00
1600	415.3	232.4	182.9	694.8	450.0	244.9	1.33	0.00	3.75	3.53	0.00	0.00	0.00	0.00	3.75	3.75	3.75	0.58	0.00	0.00	0.00	0.00
1700	445.3	250.9	194.4	739.1	483.8	255.2	1.40	0.00	3.75	3.75	0.31	0.00	0.00	0.00	3.75	3.75	3.75	1.33	0.00	0.00	0.00	0.00
1800	476.0	269.9	206.1	783.4	522.1	261.2	1.47	0.00	3.75	3.75	0.85	0.00	0.00	0.00	3.75	3.75	3.75	2.10	0.00	0.00	0.00	0.00
1900	506.8	289.4	217.4	829.3	556.9	272.4	1.53	0.00	3.75	3.75	1.38	0.00	0.00	0.00	3.75	3.75	3.75	2.86	0.00	0.00	0.00	0.00
1999.9	536.8	310.1	226.8	839.7	564.4	275.3	1.69	0.00	3.75	3.75	1.90	0.00	0.00	0.00	3.75	3.75	3.75	3.03	0.00	0.00	0.00	0.00
2000	650.0	350.0	300.0	1000.0	600.0	400.0	0.95	0.00	3.75	3.75	3.75	3.75	1.65	0.00	3.75	3.75	3.75	3.75	3.75	3.75	2.25	0.00
2100	650.0	350.0	300.0	1000.0	600.0	400.0	1.19	0.00	3.75	3.75	3.75	3.75	1.65	0.00	3.75	3.75	3.75	3.75	3.75	3.75	2.25	0.00
2200	650.0	350.0	300.0	1000.0	600.0	400.0	1.42	0.00	3.75	3.75	3.75	3.75	1.65	0.00	3.75	3.75	3.75	3.75	3.75	3.75	2.25	0.00
2300	650.0	350.0	300.0	1000.0	600.0	400.0	1.66	0.00	3.75	3.75	3.75	3.75	1.65	0.00	3.75	3.75	3.75	3.75	3.75	3.75	2.25	0.00
2350	650.0	350.0	300.0	1000.0	600.0	400.0	1.89	0.00	3.75	3.75	3.75	3.75	1.65	0.00	3.75	3.75	3.75	3.75	3.75	3.75	2.25	0.00

For this evaluation, it was assumed that each of the tubes (two tubes in both the Eastside and Westside Canals) had a “full” diameter of 1 foot and a slit opening of 6 inches resulting in a tube depth (B) of 0.93 feet and area of about 0.76 ft^2. The tubes were assumed to be oriented 45 degrees with the direction of flow, and a canal bottom width of 30 feet was used in the Eastside Canal and 60 feet was used in the Westside Canal, resulting in effective tube lengths of about 42 feet and 85 feet, respectively.

The above calculations were performed using the hydraulic conditions (depth and Froude number) predicted by the base sediment-transport model at the two cross sections most representative of the Eastside and Westside vortex tubes (Figure 32) for each ordinate of the simulation. Initial estimates of the extraction ratios indicated the tube discharges would be less than 5 percent of the canal discharges, so the Mesilla Dam and canal headworks gate configurations that were used in the baseline model were used in the modeling of this alternative, which, therefore, results in the same flow deliveries to the canals but slightly lower canal discharges downstream from the vortex tubes. Equation 5.1 was then used to determine the discharge delivered to each vortex tube based on the total canal discharge for that ordinate. The resulting discharges were then multiplied by 2 to represent the two vortex tubes and entered as lateral outflow time series from each of the canals.

Atkinson (1994a) reported field measured trap efficiencies for vortex tubes that ranged from 30 to 45 percent. Initially, the corresponding lateral sediment outflows were computed using the HEC-RAS flow-weighted sediment split option, but this method resulted in under prediction of the sediment outflows due to the low extraction ratios. Thus, the corresponding lateral sediment outflows were computed by assuming the tubes extract 30 percent of the total bed material load in the canals. Outflowing sediment load time series were prepared by taking 30 percent of the sediment load predicted by the baseline model at the representative vortex tube locations. The water and sediment outflows from each of the vortex tube configurations were then added back into the RGCP as lateral inflow time series at the appropriate location below Mesilla Dam.

5.3.3.3 Automated Gate Operator Modeling

Similar to the vortex tube alternative, the effects of automated gate operators on Mesilla Dam would be controlled by three-dimensional (3-D) flow patterns so 1-D modeling of the specific sluicing conditions is not possible. The modeling for this alternative was therefore performed to demonstrate the likely changes in erosion and depositional patterns that would likely occur as a result of using different gates for sluicing operations. To accomplish this, the base model of the single irrigation season was adjusted to reflect a first scenario wherein Gate #2 was used for the sluicing and a second scenario where Gate #5 was used for the sluicing. Under both scenarios, the erosion limits of the bed sediment reservoir was set at the geometric data points that are just outside of the left and right sides of the gate. Ineffective flow areas were also added to remove flow conveyance from areas that would not convey flow during the Gate #2 or Gate #5 sluicing operations. The erosion limits and ineffective flow areas used in each of the models are shown in **Figures 53 and 54**. Lastly, the gate opening time series that defines the gate opening configuration of Mesilla Dam that was used in the base condition model was revised for the sluicing operations to include a fully open gate during the sluicing period, which was assumed to occur over the first two weeks of the simulation (June 1 to June 14).

5.4 Base and Alternative Sediment-transport Model Results

A very large number of variables are output by the HEC-RAS sediment-transport models. The model output is available spatially for a specific point in simulation time as well as temporally for a specific cross section. Although many of the variables may be of importance in the evaluation of a specific alternative at a single problem location, the variables that are of most importance to

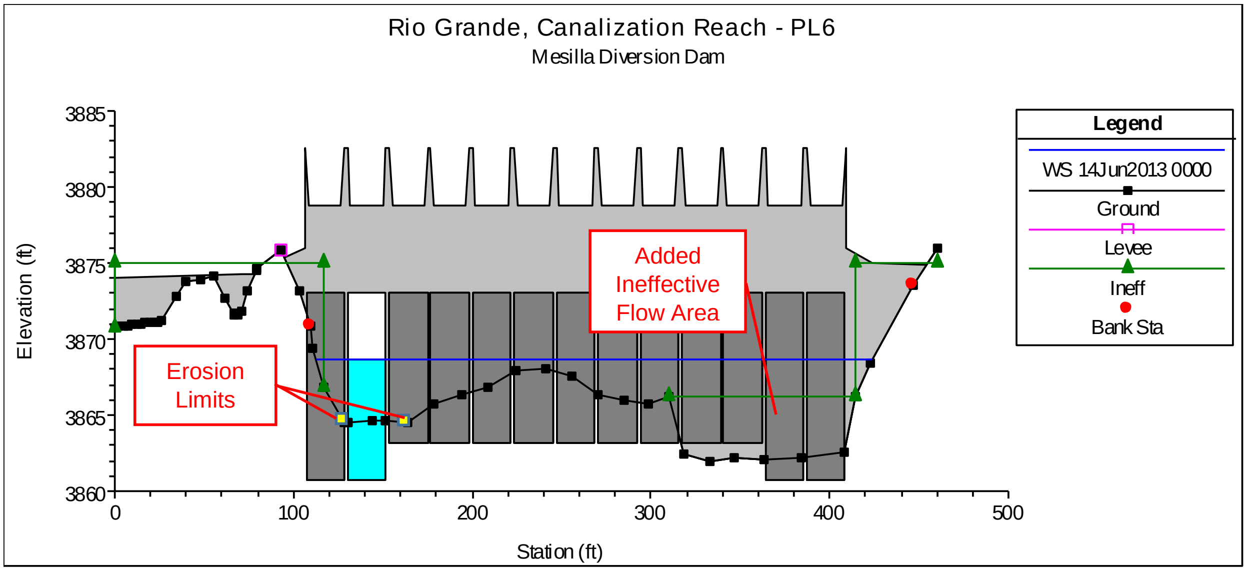

Figure 53. Cross section plot showing the erosion limits and added ineffective flow areas that were used to model the Gate #2 sluicing operation.

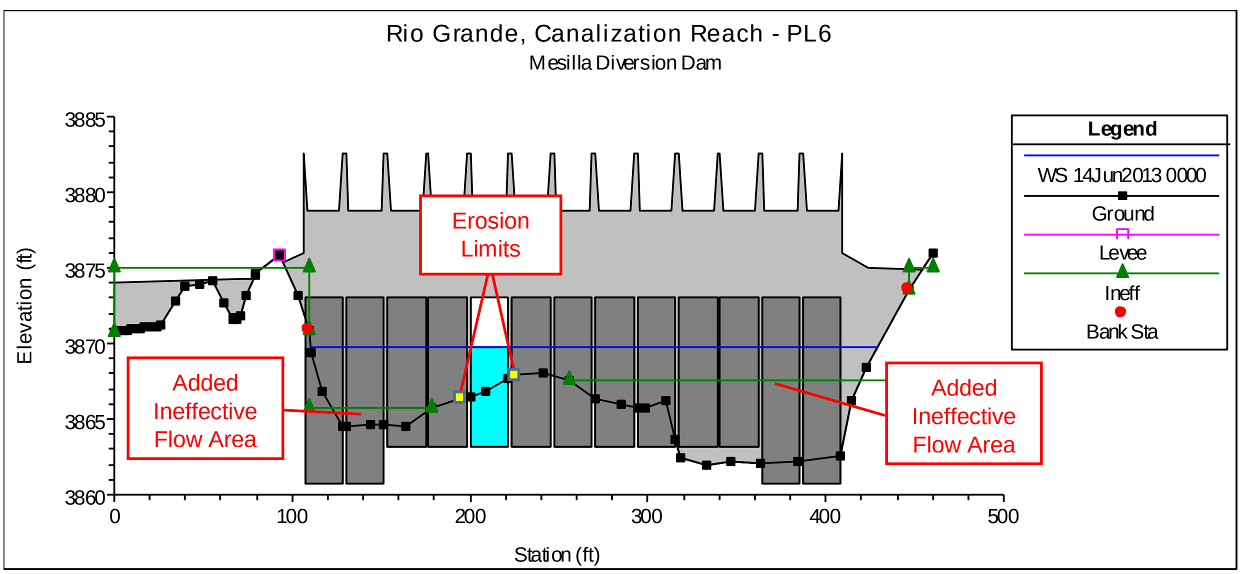

Figure 54. Cross-section plot showing the erosion limits and added ineffective flow areas that were used to model the Gate #5 sluicing operation.

the relative benefits of the alternatives include the amount of aggradation or degradation within a problem location reach (i.e., the mass of aggradation or degradation, or the amount of vertical bed elevation change), and the amount of sediment leaving the problem location reach. In addition, the predicted model geometry at the end of the simulation is also important because it provides a basis for evaluating the long-term (10-year) effects of the alternatives on water-surface elevation.

The amount of aggradation or degradation along the project reach, including both the total mass and corresponding vertical mean bed elevation change, was considered spatially to assess the effects of the alternatives at specific locations along the project reach. The mass of aggradation and degradation was also considered temporally along with the cumulative mass delivered to downstream reaches to provide a means of quantifying the benefits and consequences of the alternatives. Plots showing a comparison of these variables relative to the baseline condition are presented in **Appendix M**. The long-term effect of the alternatives on water-surface elevation, which include the effects at the 100-year water-surface elevation and an assessment of levee freeboard encroachments, also served as a basis for evaluating the benefits and consequences of the alternatives. Comparative long-term water-surface elevation profiles are presented in **Appendix N**. Site-specific results were also considered for certain alternatives in the vicinity of Mesilla Dam at Problem Location 6.

Each of the alternatives required a modification or combination of modifications to the baseline model (geometric, roughness, sediment input, etc.). The 1-D sediment-transport module of HEC-RAS is better suited for simulating some of the conditions than others, and the results are considered more reliable. Based on model reliability, the alternatives can be ranked from most to least as:

1. Sediment Removal and Sediment Trap Alternatives
2. Removal of the Rincon Siphon Grade Control Structure
3. Island/Bar Destabilization and Vegetation Removal Alternative
4. Spur Dikes
5. Modifications to the Tierra Blanca Vortex Weir
6. Riprap at Problem Location 8
7. Non-sediment Removal Alternatives at Mesilla Dam

5.4.1 Spatial Comparisons of Aggradation and Degradation

The aggradation/degradation patterns along each of the problem location reaches for the alternative conditions was compared to the base condition to assess the likely spatial changes to erosion or deposition that would result from the channel maintenance strategies. The spatial profiles are also beneficial in that these results can be used to evaluate the aggradation/degradation patterns along a specific portion of the model reach, such as within the limits of the problem location or along a treatment reach. Spatial profiles comparing the predicted total mass of aggradation or degradation and the predicted change in mean bed elevation under the base and alternative conditions are presented in Appendix M.1. (Spatial plots for the non-sediment removal alternatives at Problem Location 6 were not prepared since these model simulations were only executed over a single irrigation season; the results from these simulations are presented in a separate section below). The average and total predicted mass of aggradation or degradation along the model reach and within the extents of the problem locations under the base and alternative conditions are presented in **Table 25**, along with the percent change over the base condition.

Table 25. Summary of average predicted mass of aggradation or degradation at the end of the simulations for the base and alternative conditions.

Problem Location	Base or Alternative Condition	Full Model Extents: Average Mass of Agg/Deg (tons)	Full Model Extents: Total Mass of Agg/Deg (tons)	Full Model Extents: Percent Change from Base	Within Problem Location: Average Mass of Agg/Deg (tons)	Within Problem Location: Total Mass of Agg/Deg (tons)	Within Problem Location: Percent Change from Base
1	Base Model	5,277	279,698	-	5,826	157,301	-
	Localized Excavation	4,939	261,752	94%	5,152	139,091	-12%
	Short Channel Excavation	4,919	260,713	93%	5,270	142,286	-10%
	Long Channel Excavation	4,486	237,743	85%	5,040	136,090	-13%
	Modified Vortex Weir	5,256	278,592	100%	5,802	156,655	0%
	Sediment Traps	2,312	122,529	44%	1,186	32,028	-80%
2	Base Model	2,383	109,615	-	3,170	129,963	-
	Localized Excavation	1,996	91,801	84%	2,541	104,170	-20%
	Short Channel Excavation	1,907	87,729	80%	2,197	90,087	-31%
	Long Channel Excavation	1,947	89,555	82%	2,647	108,520	-16%
	Island Destabilization	2,492	114,632	105%	3,053	125,187	-4%
	Sediment Traps	-69	-3,166	-3%	0	-4	-100%
3	Base Model	1,526	70,179	-	1,144	16,013	-
	Localized Excavation	1,442	66,348	95%	345	4,833	-70%
	Short Channel Excavation	1,587	73,014	104%	1,448	20,275	27%
	Long Channel Excavation	1,668	76,729	109%	3,252	45,521	184%
	Remove Siphon	1,268	57,066	81%	-8,036	-104,471	-752%
	Sediment Traps	1,371	63,051	90%	262	3,662	-77%
4	Base Model	4,843	242,144	-	5,587	217,899	-
	Localized Excavation	4,776	238,795	99%	5,774	225,178	3%
	Short Channel Excavation	4,872	243,608	101%	6,295	245,519	13%
	Long Channel Excavation	5,124	256,189	106%	7,441	290,208	33%
	Island Destabilization	4,879	243,937	101%	5,751	224,304	3%
	Spur Dikes	4,810	240,517	99%	5,040	196,552	-10%
5	Base Model	636	29,239	-	986	32,544	-
	Localized Excavation	719	33,067	113%	1,503	49,599	52%
	Short Channel Excavation	631	29,018	99%	1,562	51,546	58%
	Long Channel Excavation	1,106	50,889	174%	2,733	90,201	177%
	Spur Dikes	635	29,197	100%	866	28,576	-12%
	Sediment Traps	390	17,925	61%	757	24,984	-23%
6	Base Model	4,078	154,983	-	5,552	133,240	-
	Short Channel Excavation	5,244	199,264	129%	7,453	178,864	34%
	Long Channel Excavation	6,109	232,131	150%	8,907	213,780	60%
7	Base Model	215	11,852	-	1,934	40,605	-
	Localized Excavation	205	11,278	95%	2,166	45,483	12%
	Short Channel Excavation	210	11,530	97%	3,091	64,902	60%
	Long Channel Excavation	205	11,249	95%	3,263	68,527	69%
	Spur Dikes	-280	-15,395	-130%	-233	-4,884	-112%
	Sediment Traps	-547	-30,111	-254%	-228	-4,788	-112%
8	Base Model	1,570	50,244	-	1,161	20,902	-
	Localized Excavation	1,791	59,087	118%	1,992	35,851	72%
	Short Channel Excavation	1,981	63,385	126%	2,680	48,246	131%
	Long Channel Excavation	2,351	75,223	150%	3,589	64,594	209%
	Riprap	1,553	49,698	99%	1,111	20,006	-4%
	Spur Dikes	2,048	65,531	130%	950	17,100	-18%
9	Base Model	6,030	223,126	-	5,509	154,242	-
	Localized Excavation	6,137	227,087	102%	6,136	171,803	11%
	Short Channel Excavation	6,230	230,502	103%	6,566	183,855	19%
	Long Channel Excavation	7,307	270,363	121%	8,240	230,714	50%
	Island Destabilization	6,211	229,823	103%	5,881	164,666	7%
	Spur Dikes	5,987	221,501	99%	5,512	154,338	0%

These results generally indicate that each of the alternatives would have some effect on erosion and sedimentation along the model reach. As expected, the majority of the sediment removal scenarios result in increased aggradation levels compared to the base condition. The predicted increase in aggradation under these alternatives is a result of filling in of the excavated areas. At locations where the sediment removal alternatives remove a significant hydraulic control, the level of aggradation reduces compared to the base condition because the excavated channel more efficiently transports sediment. This response occurs under each of the sediment removal scenarios at Problem Locations 1 and 2 and under the localized excavation scenario at Problem Location 3. It should be noted that, although the total mass of aggradation decreases at these locations, these decreased levels tend to occur outside of the excavated areas, while increased levels of aggradation are indicated along the excavated areas. An example of this response occurs at Problem Location 3, where increased levels of aggradation occur in the vicinity of the localized excavation at Garcia Arroyo, and decreased aggradation levels are indicated upstream from the excavation [**Figure 55** (same as Figure M.1.3 in Appendix M)].

Of the general non-sediment removal alternatives, the sediment-trap alternatives resulted in the largest reduction to aggradation levels, followed by the spur dike alternatives and then the island/bar destabilization alternatives (Table 25). The sediment trap alternatives substantially reduced the volume of aggradation along the extents of the problem locations where this treatment was considered. The sediment traps resulted in net degradation at Problem Location 7 and net equilibrium at Problem Location 2. The traps reduced the levels of aggradation along the extents of the other problem locations by as little as 23 percent (Problem Location 5) to as much as 80 percent (Problem Location 1). The spur dike alternatives reduced the volume of aggradation along the extents of most of the problem locations where this treatment was considered (10, 12, 23 and 18 percent at Problem Locations 4, 5, 7 and 8, respectively), but had very little effect on overall aggradation levels at Problem Location 9. Island/bar destabilization resulted in slight reductions to aggradation levels within the extents of Problem Location 2 that appears to result from increased hydraulic efficiencies over the destabilized surfaces, but resulted in slight increases to aggradation levels at Problem Locations 4 and 9, suggesting the bars and islands would likely rebuild.

Most of the site-specific alternatives also affected the levels of aggradation, although the modified Tierra Blanca Vortex Weir had very little effect. The removal of the grade control structure at the Rincon Siphon resulted in the largest reduction to aggradation levels, reducing aggradation by a factor of 7.5 within the extents of the problem location and by a factor of about 18 percent along the model reach of Problem Location 3, although significant aggradation is indicated along the reach below the grade-control structure (Table 25). Installation of riprap at Problem Location 8 results in slightly lower levels of aggradation along the revetment reach due to the slight constriction caused by the riprap, which also results in small backwater effects and increased levels of aggradation in the reach above Country Club Bridge. The modifications to the Tierra Blanca Vortex Weir at Problem Location 1 has very little impact on aggradation along the reach, although the levels of aggradation are slightly less in the vicinity of the weir.

5.4.2 Temporal Comparisons of Aggradation and Degradation

The temporal results are of benefit in evaluating the channel response to the input hydrologic sequence and tributary loading events as well as the relative time-based effects of the individual alternatives. Plots showing the total cumulative mass of aggradation or degradation along the modeled reach are presented in Appendix M.2. Although the cumulative volumes at the end of the simulation are identical to those presented in the spatial comparisons discussed above (Table 25 and Appendix M.1), the results are somewhat different at various points in the simulation (**Table 26**). In general, the most significant response of the channel to the sediment removal

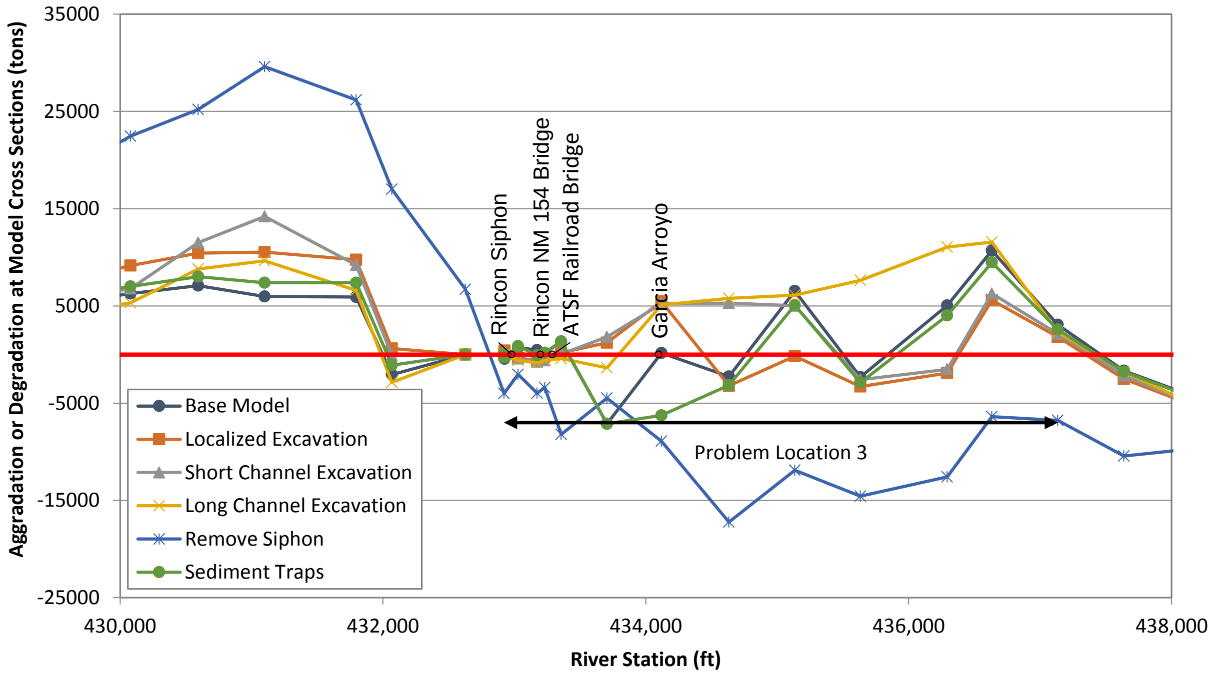

Figure 55. Mass of aggradation or degradation predicted by the sediment-transport simulations of the base and alternative conditions at Problem Location 3 (same as Figure M.1.3 in Appendix M.1).

Table 26. Summary of predicted cumulative mass of aggradation or degradation along the modeled reach at various points during the simulations for the base and alternative conditions.

Problem Location	Base or Alternative Condition	Total Agg/Deg Mass, 2-yrs (tons)	Percent Change from Base	Total Agg/Deg Mass, 5-yrs (tons)	Percent Change from Base	Total Agg/Deg Mass, 8-yrs (tons)	Percent Change from Base
1	Base Model	69,185	-	152,366	-	228,878	-
	Localized Excavation	68,745	-1%	144,596	-5%	214,977	-6%
	Short Channel Excavatio	67,848	-2%	143,468	-6%	214,047	-6%
	Long Channel Excavatior	67,827	-2%	133,954	-12%	196,341	-14%
	Modified Vortex Weir	69,005	0%	151,966	0%	228,468	0%
	Sediment Traps	43,720	-37%	82,930	-46%	108,323	-53%
2	Base Model	14,185	-	42,935	-	82,130	-
	Localized Excavation	13,229	-7%	41,155	-4%	74,507	-9%
	Short Channel Excavatio	12,261	-14%	31,855	-26%	64,077	-22%
	Long Channel Excavatior	12,898	-9%	37,204	-13%	67,050	-18%
	Island Destabilization	14,186	0%	44,761	4%	86,443	5%
	Sediment Traps	-1,437	-110%	1,523	-96%	3,051	-96%
3	Base Model	41,747	-	65,175	-	80,476	-
	Localized Excavation	41,760	0%	64,318	-1%	78,559	-2%
	Short Channel Excavatio	41,760	0%	65,311	0%	81,840	2%
	Long Channel Excavatior	41,735	0%	64,838	-1%	82,979	3%
	Remove Siphon	41,736	0%	62,164	-5%	71,386	-11%
	Sediment Traps	40,591	-3%	62,265	-4%	75,359	-6%
4	Base Model	56,688	-	138,349	-	207,473	-
	Localized Excavation	52,680	-7%	130,420	-6%	200,792	-3%
	Short Channel Excavatio	57,356	1%	140,476	2%	215,425	4%
	Long Channel Excavatior	55,938	-1%	141,629	2%	219,835	6%
	Island Destabilization	57,906	2%	139,538	1%	209,658	1%
	Spur Dikes	48,621	-14%	124,505	-10%	193,479	-7%
5	Base Model	9,610	-	16,648	-	25,271	-
	Localized Excavation	9,329	-3%	17,601	6%	26,994	7%
	Short Channel Excavatio	9,397	-2%	16,508	-1%	24,992	-1%
	Long Channel Excavatior	10,600	10%	25,264	52%	41,677	65%
	Spur Dikes	9,432	-2%	16,589	0%	25,167	0%
	Sediment Traps	7,848	-18%	12,065	-28%	17,055	-33%
6	Base Model	90,410	-	138,930	-	157,899	-
	Short Channel Excavation	121,284	34%	179,687	29%	200,397	27%
	Long Channel Excavation	134,736	49%	207,561	49%	231,868	47%
7	Base Model	-13,059	-	-9,361	-	2,390	-
	Localized Excavation	-13,336	2%	-10,353	11%	1,192	-50%
	Short Channel Excavation	-13,065	0%	-10,498	12%	2,034	-15%
	Long Channel Excavation	-13,150	1%	-10,919	17%	1,430	-40%
	Spur Dikes	-12,557	-4%	-9,247	-1%	1,675	-30%
	Sediment Traps	-18,846	44%	-27,218	191%	-28,523	-1293%
8	Base Model	29,682	-	43,115	-	53,378	-
	Localized Excavation	32,736	10%	48,014	11%	60,088	13%
	Short Channel Excavation	32,996	11%	50,140	16%	62,255	17%
	Long Channel Excavation	35,578	20%	56,908	32%	73,664	38%
	Riprap	29,136	-2%	42,638	-1%	52,208	-2%
	Spur Dikes	34,972	18%	50,105	16%	65,931	24%
9	Base Model	65,690	-	138,962	-	194,294	-
	Localized Excavation	65,463	0%	139,777	1%	196,334	1%
	Short Channel Excavation	65,376	0%	142,622	3%	197,375	2%
	Long Channel Excavation	63,662	-3%	146,048	5%	217,915	12%
	Island Destabilization	65,502	0%	144,467	4%	200,456	3%
	Spur Dikes	65,621	0%	139,043	0%	192,672	-1%

alternatives occurs during the first 5 to 6 years of the simulation, and the response tends to dampen during the later portions of the simulation [**Figure 56** (Same as Figure M.2.2 in Appendix M)]. The exception to this general statement occurs when the excavation volumes are very large relative to the inflowing sediment load, in which case the increased aggradation persists into the later portions of the simulation (or in some instances clearly beyond the end of the simulation). In contrast to the sediment-removal alternatives, the non-sediment removal alternatives typically result in a channel response that is pervasive through the simulation. For example, the sediment-trap alternatives usually show percent reductions to the predicted levels of aggradation relative to the base condition that increase through time (Figure 56).

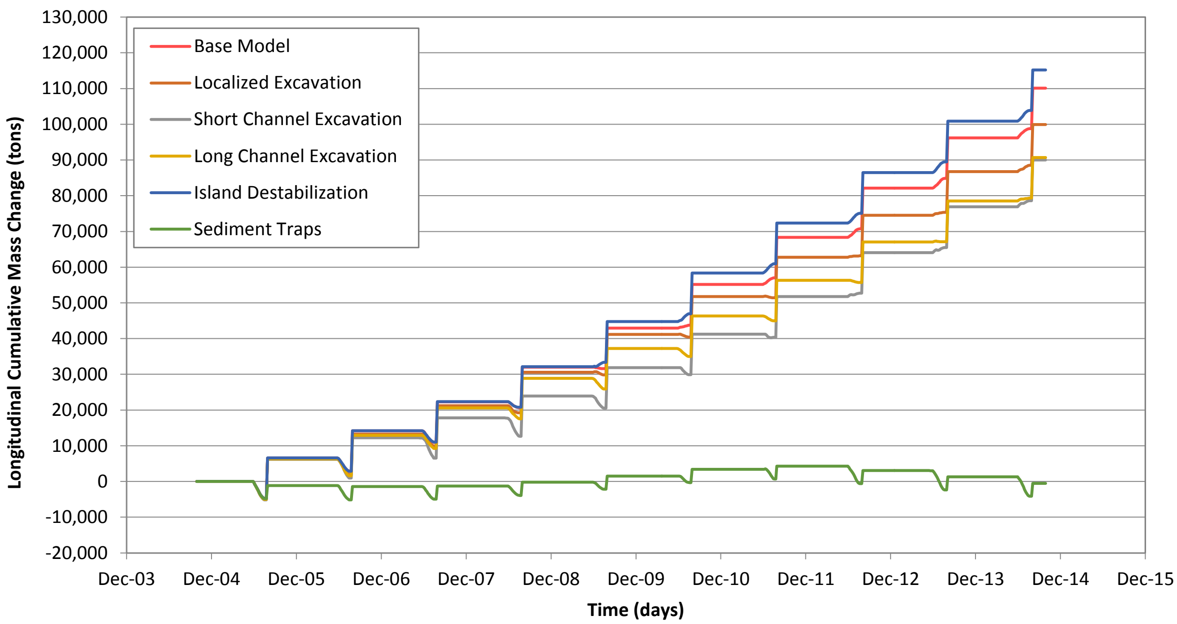

Figure 56. Cumulative mass of aggradation or degradation over time along the modeled reach at Problem Location 2 predicted by the sediment-transport simulations of the base and alternative conditions (same as Figure M.2.2 in Appendix M.2).

Temporal plots showing the cumulative mass delivered past the downstream model boundary condition were also prepared to assess the effects of the alternatives on downstream sediment loading (Appendix M.2). Initial inspection of these results are somewhat surprising in that many of the alternatives result in increases to downstream sediment deliveries relative to the base condition (**Table 27**). Upon further investigation, the different responses to the same type of channel maintenance strategy are explicable. The sediment removal alternatives at some of the problem locations involve removal of the arroyo fans that are a significant hydraulic control under the base condition, so the excavations result in more efficient sediment transport and thus increased downstream loadings. All of the sediment trap alternatives show some increase to the downstream sediment loads because (1) the sediment traps reduce buildup on the fans and associated substantial reductions to storage upstream, and (2) the sediment traps reduce the supply of coarse

Table 27. Summary of predicted cumulative mass passing the downstream boundary of the modeled reach at the end of the simulations for the base and alternative conditions.

Problem Location	Base or Alternative Condition	Total Mass Out, 2-yrs (tons)	Percent Change from Base	Total Mass Out, 5-yrs (tons)	Percent Change from Base	Total Mass Out, 10-yrs (tons)	Percent Change from Base
1	Base Model	15,450	-	31,094	-	49,327	-
	Localized Excavation	15,957	3%	39,653	28%	67,272	36%
	Short Channel Excavatio	16,872	9%	40,769	31%	68,312	38%
	Long Channel Excavatior	17,142	11%	50,542	63%	91,282	85%
	Modified Vortex Weir	15,658	1%	31,673	2%	50,433	2%
	Sediment Traps	15,347	-1%	37,321	20%	78,520	59%
2	Base Model	15,292	-	31,061	-	40,950	-
	Localized Excavation	16,371	7%	33,291	7%	51,579	26%
	Short Channel Excavatio	16,824	10%	38,009	22%	61,011	49%
	Long Channel Excavatior	16,824	10%	38,009	22%	61,011	49%
	Island Destabilization	15,315	0%	29,223	-6%	35,934	-12%
	Sediment Traps	15,731	3%	34,721	12%	77,397	89%
3	Base Model	7,723	-	28,919	-	72,698	-
	Localized Excavation	7,713	0%	30,031	4%	76,529	5%
	Short Channel Excavatio	7,752	0%	28,948	0%	69,863	-4%
	Long Channel Excavatior	7,771	1%	28,835	0%	66,149	-9%
	Remove Siphon	7,850	2%	32,840	14%	85,811	18%
	Sediment Traps	7,723	0%	28,997	0%	74,045	2%
4	Base Model	12,480	-	30,546	-	72,421	-
	Localized Excavation	12,388	-1%	33,453	10%	75,266	4%
	Short Channel Excavatio	12,489	0%	32,295	6%	70,694	-2%
	Long Channel Excavatior	11,603	-7%	27,455	-10%	61,576	-15%
	Island Destabilization	12,301	-1%	30,628	0%	70,406	-3%
	Spur Dikes	13,050	5%	33,146	9%	73,727	2%
5	Base Model	22,549	-	55,084	-	104,232	-
	Localized Excavation	22,830	1%	54,131	-2%	100,404	-4%
	Short Channel Excavatio	22,769	1%	55,206	0%	104,453	0%
	Long Channel Excavatior	21,394	-5%	46,310	-16%	82,582	-21%
	Spur Dikes	22,690	1%	55,121	0%	104,275	0%
	Sediment Traps	22,322	-1%	54,736	-1%	105,720	1%

Problem Location	Base or Alternative Condition	Total Agg/Deg Mass, 2-yrs (tons)	Percent Change from Base	Total Agg/Deg Mass, 5-yrs (tons)	Percent Change from Base	Total Agg/Deg Mass, 8-yrs (tons)	Percent Change from Base
6	Base Model	51,699	-	110,669	-	234,423	-
	Short Channel Excavation	50,519	-2%	96,460	-13%	218,754	-7%
	Long Channel Excavation	50,319	-3%	93,907	-15%	217,357	-7%
7	Base Model	25,209	-	49,735	-	79,448	-
	Localized Excavation	25,520	1%	50,733	2%	80,022	1%
	Short Channel Excavation	25,203	0%	50,506	2%	79,770	0%
	Long Channel Excavation	25,260	0%	50,998	3%	80,051	1%
	Spur Dikes	24,655	-2%	49,642	0%	80,758	2%
	Sediment Traps	25,741	2%	54,517	10%	94,847	19%
8	Base Model	14,661	-	46,589	-	102,708	-
	Localized Excavation	11,653	-21%	41,216	-12%	93,864	-9%
	Short Channel Excavation	11,242	-23%	39,168	-16%	89,567	-13%
	Long Channel Excavation	8,229	-44%	31,513	-32%	77,729	-24%
	Riprap	15,256	4%	47,105	1%	103,254	1%
	Spur Dikes	11,250	-23%	40,817	-12%	87,421	-15%
9	Base Model	1,745	-	14,479	-	50,739	-
	Localized Excavation	1,310	-25%	13,695	-5%	46,777	-8%
	Short Channel Excavation	1,262	-28%	10,897	-25%	43,363	-15%
	Long Channel Excavation	631	-64%	1,637	-89%	3,502	-93%
	Island Destabilization	798	-54%	9,551	-34%	44,042	-13%
	Spur Dikes	1,705	-2%	14,759	2%	52,364	3%

material that, under the base condition, result in continued armoring of the bed that reduces the amount of material that is available for transport. The spur dikes typically cause relatively small increases to downstream sediment loads due to the increased erosion caused by the spurs, but a reduction is indicated at Problem Location 8, where the model predicts more significant aggradation on top of the spurs that represents the formation of an inset floodplain (**Figure 57**). The formation of the inset floodplain occurs to various degrees under each of the spur dike alternatives and may be of benefit to improved conveyance efficiencies during periods of relatively low flow, but may also affect levee freeboard at some locations, as discussed below. The island and bar destabilization and vegetation removal alternatives result in moderate reductions to downstream sediment loading due to increased storage along the cleared surfaces. Among the site-specific alternatives, removal of the grade-control structure at the Rincon Siphon results in the largest increase to downstream sediment loads (18-percent increase over the base condition), while the modified vortex weir at Problem Location 1 and the riprap at Problem Location 8 have very little effect.

The temporal results were also used to evaluate the duration over which the sediment-removal scenarios would re-fill with sediment. This assessment is important for purposes of evaluating the durability and long-term maintenance of the sediment-removal alternatives. The analysis involved computation of the cumulative volume of aggradation that occurs between the up- and downstream limits of the excavations by comparing the total inflowing (upstream and tributary) sediment load with the outflowing sediment load. This cumulative aggradation volume series was then used to determine the point in the simulation at which the predicted aggradation volume surpasses the excavation volume. The results from the analysis are presented in **Table 28**, and plots showing the cumulative aggradation volume relative to the excavation volume are included in Appendix M.2 [see example plot shown in **Figure 58** (Figure M.2.20 in Appendix M)]. In some cases, the predicted storage volume did not reach the excavated volume during the 10-year simulation, so the filling time was estimated based on the percentage of the excavated volume that was stored at the end of the simulation. For cases where there were multiple excavations for a single alternative (e.g. the localized excavations at the mouths of Rincon, Reed and Bignell Arroyos at Problem Location 4), average time-to-fill values were computed for purposes of preparing costs for operation and maintenance (O&M; **Table 29**).

5.4.3 Long-term Effects on Water-surface Elevation

The steady-state hydraulic models of the base and alternative conditions were used to compare the short-term effects of the alternatives on water-surface elevation (see Section 4.7.2). These comparisons represent the changes to water-surface profiles that would occur immediately after implementation of the alternative, prior to any channel adjustments that would occur in response to the alternative. Since accounting for these channel adjustments is important, results from the sediment-transport simulations were used to evaluate the long-term effects of the alternatives on water-surface elevation. This evaluation involved extraction of the predicted model geometry at the end of the simulation for incorporation into the localized steady-state hydraulic models. At locations

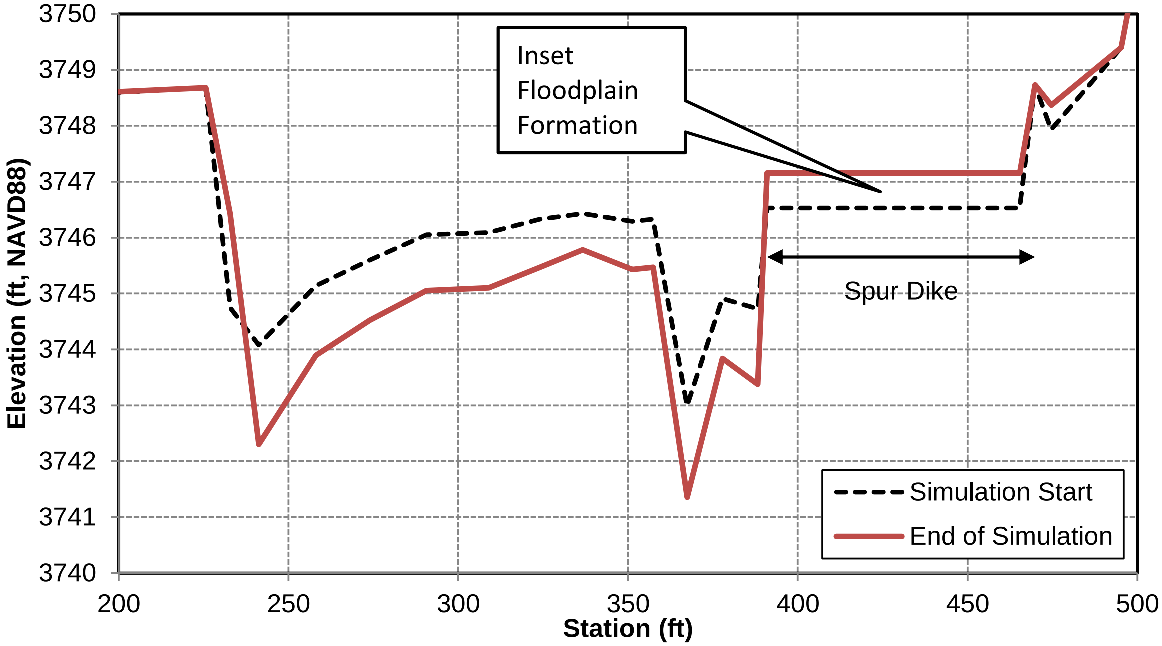

Figure 57. Comparison of the cross-sectional geometry for Cross Section 81428.7 at Problem Location 7 at the beginning and end of the simulation of the spur dike alternative showing example of inset floodplain formation.

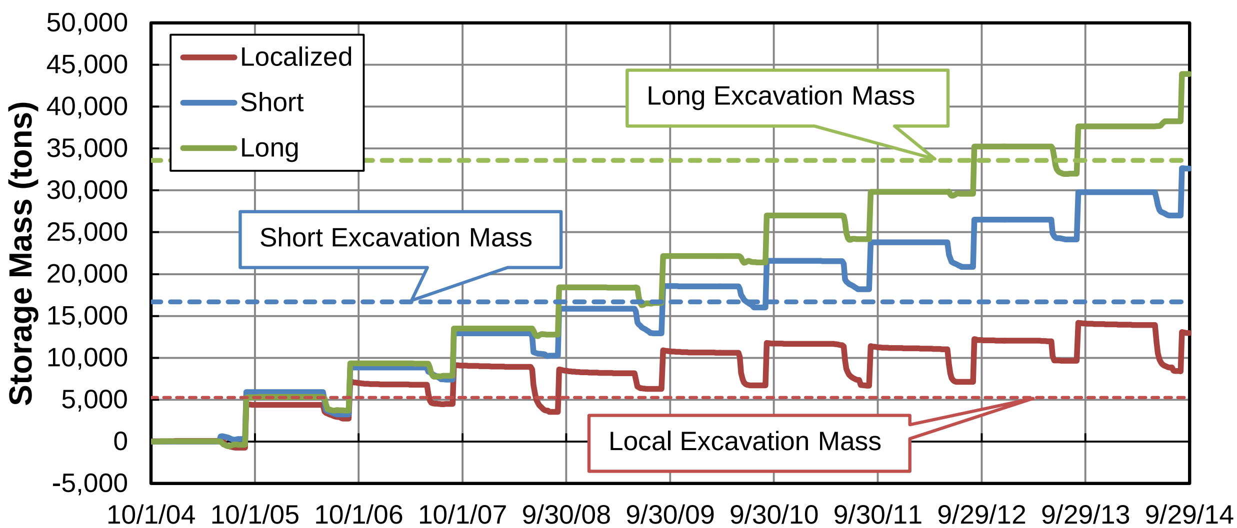

Figure 58. Predicted cumulative mass of aggradation or degradation over time along the extents of the excavated reaches at Sibley Arroyo (Problem Location 1) from the sediment-transport simulations of the sediment-removal alternatives. The dashed lines represent the excavated mass for each type of excavation.

Table 28. Summary of time required for the volume of sediment storage along the excavated reaches to surpass the excavated volume under each of the sediment removal alternatives.

Problem Location	Location	Time Required for Storage Volume to Exceed Excavated Volume (yrs)		
		Localized Excavation	Short Excavation	Long Excavation
1	Tierra Blanca Creek	0.8	0.8	2.8
	Sibley Arroyo	1.9	4.9	7.9
2	Thurman II Arroyo	8.8	9.8	10.3
	Thurman I Arroyo	6.8		
	Placitas Arroyo	2.8	4.8	
3	Garcia Arroyo	8.8	9.7	7.8
4	Rincon Arroyo	1.7	1.7	9.7
	Reed Arroyo	0.8	1.7	
	Bignell Arroyo	13.0	11.0	10.0
5	Rock Canyon	11.8	13.5	18.1
	Rincon/Tonuco Drain	10.4	13.8	34.4
6	Mesilla Dam	-	0.7	6.7
7	East Drain	2.7	7.7	7.7
	Vinton Bridge	-	5.8	
	Unnamed Lower Trib.	-	11.8	15.8
8	Country Club Bridge	1.7	2.7	4.7
9	Montoyo Drain to Dam	0.7	3.8	10.4

Table 29. Summary of average time required for the volume of sediment storage along the excavated reaches to surpass the excavated volume under each of the sediment removal alternatives.

Problem Location	Time Required for Storage Volume to Exceed Excavated Volume (yrs)		
	Localized Excavation	Short Excavation	Long Excavation
1	1.6	3.5	5.6
2	7.5	9.1	10.3
3	7.8	8.8	9.7
4	2.7	2.7	9.7
5	10.9	13.7	27.7
6	-	0.7	6.7
7	2.7	7.6	8.7
8	1.7	2.7	4.7
9	9.7	9.7	9.7

where it was necessary to remove closely spaced cross sections from the sediment-transport models, the removed cross sections were adjusted to reflect the post-simulation geometry using the HEC-RAS cross-section interpolation routine. Because the downstream boundary condition of the sediment-transport model was defined as a pass-through node, it was appropriate to use the same downstream boundary condition that was used in the original localized hydraulic models. The end-of-simulation (EOS) models were then executed over the same steady-state discharges that were evaluated in the original localized hydraulic models. To provide a basis for comparison, the EOS model suite included a version of the base condition model with end-of-simulation geometry. Comparative water-surface profiles and profiles showing the change in water-surface elevation for normal operating flows of 2,350 and 1,400 cfs (above and below Mesilla Dam, respectively) and at the 100-year peak flow are presented in **Appendix N**. The EOS-model based predicted water-surface profiles at the 100-year peak flow were also used to assess levee freeboard in areas where freeboard encroachments could be of concern in the long term.

5.4.3.1 Comparative Long-term Water-surface Elevations

The comparative water-surface elevation profiles indicate that most of the alternatives would result in reduced water surfaces in the long-term compared to the base condition at normal operating flows and at the 100-year peak flow (**Tables 30 and 31**). The sediment removal alternatives show larger reductions to water-surface elevation with increasing excavation volume. The long excavation alternative typically results in the largest average reduction to normal flow water levels, followed by either the sediment trap alternatives or the short excavation alternatives. Island/bar destabilization and vegetation removal results in varied changes to long-term water surface profiles, reducing the average normal flow profile by 1.2 feet at Problem Location 2 but slight increases at Problem Location 4. The spur dike alternative is the only general alternative that would result in increases to long-term water-surface levels, with a maximum increase of about 0.5 feet (Problem Location 7) at normal operating flows and about 0.3 feet (Problem Location 4) at the 100-year peak discharge. Of the site-specific alternatives, the modifications to the Rincon Siphon resulted in the largest reduction to water-surface elevation, with average reduction levels along the problem location reach of nearly 3 feet at normal flows and over 1 foot at the 100-year event. However, this alternative would result in increased aggradation and an associated increase to water-surface elevations downstream from the present siphon. The modified Tierra Blanca Vortex Weir alternative reduced the average normal flow water surface by 0.3 feet but has no effect on the average 100-year water-surface elevation. As expected, the riprap at Problem Location 8 does not affect water-surface elevations appreciably.

Flood conditions at Problem Location 8 is of special concern due to known conveyance issues in the vicinity of Country Club Bridge. Results from the EOS modeling indicates that the base and alternative conditions would result in water-surface elevations that come into contact with the low chord of the bridge deck, but due to the bridge camber, no pressure flow is indicated under any of the model scenarios (**Figure 59**). This is somewhat different than the short-term effects of the alternatives on water-surface levels, which indicated that the water surface under the long excavation alternatives would not come into contact with the lowest point of the bridge deck at elevation 3753.49 feet NAVD88 (**Table 32**).

5.4.3.2 Long-term Levee Freeboard Impacts

No levees are present at Problem Location 1 and along the majority of the Problem 5 reach, so an evaluation of levee freeboard was not conducted at these two locations. At the remainder of the locations, the comparative long-term levee freeboard analysis indicates that most alternatives would result in increased levee freeboard at the 100-year peak flow (**Table 33**; Appendix O). The sediment-removal alternatives show either no change, or an average increase in freeboard at all problem locations except for Problem Location 6 where the both the long and short excavation alternatives result in a 0.1-foot decrease in freeboard. This decrease can be attributed to increased levels of

Table 30. Summary of predicted water-surface elevations at 2,350 cfs (above Mesilla Dam) and 1,400 cfs (below Mesilla Dam) based on the steady-state hydraulic model that uses the predicted geometry at the end of the sediment-transport model simulations for the base and alternative conditions.

Problem Location	Alternative	Change in Water-surface Elevation (ft)*		
		Average	Max. Increase	Max. Decrease
1	Localized Excavation	-0.7	0.1	-1.6
	Short Channel Excavation	-0.8	0.0	-1.6
	Long Channel Excavation	-1.6	0.0	-3.3
	Modified Vortex Weir	-0.3	0.5	-1.1
	Sediment Traps	-1.8	0.0	-3.4
2	Localized Excavation	-0.4	0.0	-0.9
	Short Channel Excavation	-0.9	0.0	-1.8
	Long Channel Excavation	-1.2	0.0	-2.5
	Island Destabilization	-1.2	0.0	-2.5
	Sediment Traps	-1.2	0.0	-3.4
3	Localized Excavation	-0.1	0.5	-0.4
	Short Channel Excavation	-0.3	0.2	-0.6
	Long Channel Excavation	-0.2	0.0	-0.6
	Remove Siphon	-0.2	0.1	-2.1
	Sediment Traps	0.0	0.2	-0.3
4	Localized Excavation	-0.2	0.0	-0.6
	Short Channel Excavation	-0.3	0.0	-0.5
	Long Channel Excavation	-1.2	0.0	-1.6
	Island Destabilization	0.1	0.4	-0.2
	Spur Dikes	0.0	0.3	-0.2
5	Localized Excavation	-0.4	0.0	-0.8
	Short Channel Excavation	-0.5	0.0	-1.1
	Long Channel Excavation	-1.2	0.0	-1.9
	Spur Dikes	0.0	0.4	-0.2
	Sediment Traps	-0.2	0.0	-0.6
6	Short Channel Excavation			
	Long Channel Excavation			
7	Localized Excavation	0.1	0.4	0.0
	Short Channel Excavation	-0.2	0.0	-0.3
	Long Channel Excavation	-0.4	0.0	-0.6
	Spur Dikes	0.2	0.5	0.0
	Sediment Traps	-0.6	0.0	-1.2
8	Localized Excavation	-0.2	0.0	-0.3
	Short Channel Excavation	-0.4	0.0	-0.4
	Long Channel Excavation	-0.3	0.0	-0.4
	Riprap	0.0	0.1	0.0
	Spur Dikes	-0.1	0.0	-0.2
9	Localized Excavation	-0.1	0.0	-0.2
	Short Channel Excavation	-0.2	0.0	-0.3
	Long Channel Excavation	-1.4	0.0	-2.3
	Island Destabilization	-0.2	0.1	-0.3
	Spur Dikes	0.0	0.1	-0.1

*Change in water-surface elevation relative to the base condition model with end of simulation model geometry.

Table 31. Summary of predicted water-surface elevations at the 100-year peak discharge based on the steady-state hydraulic model that uses the predicted geometry at the end of the sediment-transport model simulations for the base and alternative conditions.*Change in water-surface elevation relative to the base condition model with end of simulation model geometry.

Problem Location	Alternative	Change in Water-surface Elevation (ft)*		
		Average	Max. Increase	Max. Decrease
1	Localized Excavation	-0.2	0.1	-0.7
	Short Channel Excavation	-0.1	0.1	-0.3
	Long Channel Excavation	-0.4	0.1	-1.0
	Modified Vortex Weir	0.0	0.2	-0.1
	Sediment Traps	-0.3	0.0	-0.7
2	Localized Excavation	-0.2	0.4	-1.1
	Short Channel Excavation	-0.5	0.1	-1.7
	Long Channel Excavation	-0.6	0.0	-1.8
	Island Destabilization	-0.2	0.1	-0.9
	Sediment Traps	-0.3	0.0	-1.2
3	Localized Excavation	-0.2	0.3	-0.9
	Short Channel Excavation	-0.3	0.0	-0.7
	Long Channel Excavation	-0.3	0.0	-0.6
	Remove Siphon	212.2	4044.7	-1.9
	Sediment Traps	0.0	0.1	-0.2
4	Localized Excavation	-0.1	0.0	-0.3
	Short Channel Excavation	-0.1	0.0	-0.3
	Long Channel Excavation	-0.6	0.0	-0.9
	Island Destabilization	-0.2	0.0	-0.4
	Spur Dikes	0.1	0.3	-0.1
5	Localized Excavation	-0.1	0.0	-0.2
	Short Channel Excavation	-0.1	0.0	-0.3
	Long Channel Excavation	-0.2	0.0	-0.5
	Spur Dikes	0.1	0.1	0.0
	Sediment Traps	-0.1	0.0	-0.1
6	Short Channel Excavation	0.1	0.0	0.0
	Long Channel Excavation	0.1	0.0	0.0
7	Localized Excavation	0.0	0.1	0.0
	Short Channel Excavation	-0.1	0.0	-0.1
	Long Channel Excavation	-0.1	0.0	-0.2
	Spur Dikes	0.2	0.3	0.0
	Sediment Traps	-0.2	0.0	-0.2
8	Localized Excavation	-0.1	0.0	-0.2
	Short Channel Excavation	-0.1	0.0	-0.2
	Long Channel Excavation	-0.1	0.0	-0.2
	Riprap	0.0	0.1	0.0
	Spur Dikes	0.1	0.1	0.0
9	Localized Excavation	-0.1	0.0	-0.1
	Short Channel Excavation	-0.1	0.0	-0.2
	Long Channel Excavation	-1.1	0.0	-1.6
	Island Destabilization	-0.4	0.0	-0.6
	Spur Dikes	0.0	0.0	-0.1

*Change in water-surface elevation relative to the base condition model with end of simulation model geometry.

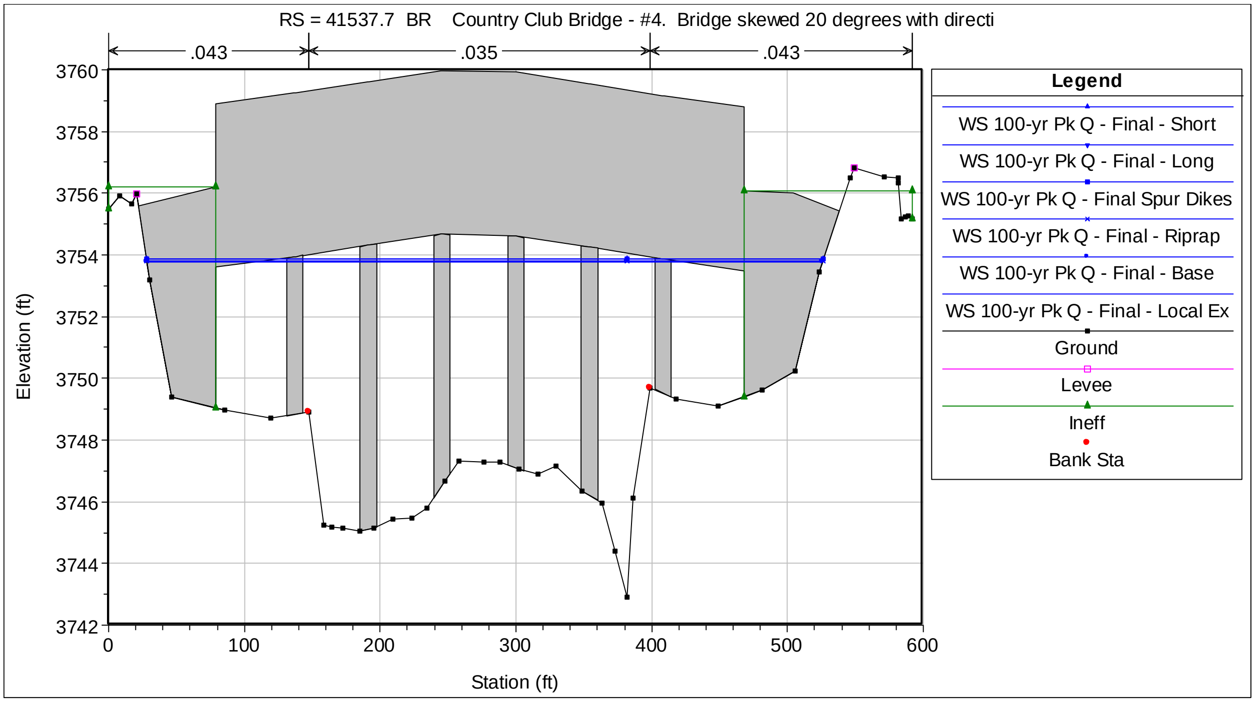

Figure 59. Predicted water-surface elevation at the upstream face of the Country Club Bridge based on the localized steady-state hydraulic models that have the geometry from the end of the sediment-transport simulations for base and alternative conditions

Table 32. Summary of predicted water-surface elevation at the upstream face of the Country Club Bridge.

Condition	U/S Cross Section		U/S Face of Bridge	
	Localized Models[1]	EOS Models[2]	Localized Models[1]	EOS Models[2]
	Predicted Water-surface Elevation (ft, NAVD88)			
Base	3753.98	3754.1	3753.68	3753.78
Localized Excavation	3753.84	3754.01	3753.67	3753.78
Short Channel Excavation	3753.67	3754.08	3753.52	3753.88
Long Channel Excavation	3753.54	3754.07	3753.38	3753.87
Riprap	3754.04	3754.14	3753.73	3753.81
Spur Dikes	3754.19	3754.19	3753.86	3753.86
	Distance Above Lowest Point on Low Chord (ft)			
Base	0.49	0.61	0.19	0.29
Localized Excavation	0.35	0.52	0.18	0.29
Short Channel Excavation	0.18	0.59	0.03	0.39
Long Channel Excavation	0.05	0.58	-0.11	0.38
Riprap	0.55	0.65	0.24	0.32
Spur Dikes	0.7	0.7	0.37	0.37

[1]Localized model of the alternative immediately after implementation.
[2]Localized model with predicted geometry at end of 10-year sediment-transport simulation.

Table 33. Summary of predicted change in levee freeboard at the 100-year event based on the steady-state hydraulic model that uses the predicted geometry at the end of the sediment-transport model simulations for the base and alternative conditions.

Problem Location	Alternative	Change in Freeboard (ft)*					
		Average Left	Average Right	Max. Increase Left	Max. Increase Right	Max. Decrease Left	Max. Decrease Right
1	Localized Excavation	No levees					
	Short Channel Excavation						
	Long Channel Excavation						
	Modified Vortex Weir						
	Sediment Traps						
2	Localized Excavation	0.2	0.6	0.7	1.1	-0.4	-0.1
	Short Channel Excavation	0.5	1.1	1.4	1.7	-0.1	0.0
	Long Channel Excavation	0.6	1.2	1.6	1.8	0.0	0.0
	Island Destabilization	0.0	0.0	0.1	0.0	-0.1	0.0
	Sediment Traps	0.4	0.9	0.9	1.2	0.0	0.0
3	Localized Excavation	0.0	0.2	0.0	0.2	0.0	0.0
	Short Channel Excavation	0.0	0.3	0.0	0.3	0.0	0.0
	Long Channel Excavation	0.0	0.3	0.0	0.5	0.0	0.0
	Remove Siphon	0.0	1.3	0.0	1.4	0.0	0.0
	Sediment Traps	0.0	0.1	0.0	0.2	0.0	0.0
4	Localized Excavation	0.1	0.2	0.3	0.2	0.0	0.0
	Short Channel Excavation	0.2	0.2	0.3	0.2	0.0	0.0
	Long Channel Excavation	0.6	0.5	0.9	0.5	0.0	0.0
	Island Destabilization	0.2	0.1	0.4	0.1	0.0	0.0
	Spur Dikes	-0.1	0.0	0.1	0.0	-0.3	0.0
5	Localized Excavation	No levees					
	Short Channel Excavation						
	Long Channel Excavation						
	Spur Dikes						
	Sediment Traps						

Problem Location	Alternative	Freeboard (ft)*					
		Average Left	Average Right	Max. Increase Left	Max. Increase Right	Max. Decrease Left	Max. Decrease Right
6	Short Channel Excavation	-0.1	0.0	0.0	0.0	-0.1	0.0
	Long Channel Excavation	-0.1	0.0	0.0	0.0	-0.2	0.0
7	Localized Excavation	0.0	0.0	0.0	0.0	0.0	0.0
	Short Channel Excavation	0.1	0.1	0.1	0.1	0.0	0.0
	Long Channel Excavation	0.2	0.2	0.2	0.2	0.0	0.0
	Spur Dikes	-0.2	-0.2	0.0	0.0	-0.3	-0.3
	Sediment Traps	0.2	0.2	0.2	0.2	0.0	0.0
8	Localized Excavation	0.1	0.1	0.2	0.2	0.0	0.0
	Short Channel Excavation	0.1	0.1	0.2	0.2	0.0	0.0
	Long Channel Excavation	0.1	0.1	0.2	0.2	0.0	0.0
	Riprap	0.0	0.0	0.0	0.0	-0.1	-0.1
	Spur Dikes	-0.1	-0.1	0.0	0.0	-0.1	-0.1
9	Localized Excavation	0.1	0.0	0.1	0.1	0.0	0.0
	Short Channel Excavation	0.2	0.1	0.2	0.2	0.0	0.0
	Long Channel Excavation	0.6	0.6	0.7	1.0	0.0	0.0
	Island Destabilization	0.2	0.2	0.3	0.3	0.0	0.0
	Spur Dikes	0.0	0.0	0.1	0.0	0.0	0.0

aggradation at Problem Location 6 when compared to the base conditions. The long excavation alternative typically results in the greatest increase in levee freeboard, followed by the short excavation, and then either the localized excavation or sediment trap alternatives. Island destabilization alternatives also produced favorable results with increases in freeboard at all selected problem locations. On average, the spur dike alternative either had no effect on the freeboard or raised the 100-year water surface slightly and reduced the average levee freeboard in the reach when compared to the base conditions 100-year flow. The most significant spur dike reductions of freeboard (0.3 feet) occurred in Problem Location 7. For the site-specific alternatives, the Rincon Siphon resulted in the largest increase levee freeboard with an average gain of 1.3 feet, and the riprap at Problem Location 8 has almost no effect on levee freeboard.

5.4.4 Site-specific Results at Problem Location 6

As discussed above, the non-sediment removal alternatives that were evaluated in the vicinity of Mesilla Dam at Problem Location 6 are heavily influenced by the operations associated with those alternatives, so the sediment-transport simulations were limited to a single irrigation season. Unlike the sediment removal alternatives that would have immediate and long-term effects on water-surface elevation, the non-sediment removal alternatives would not affect water levels appreciably over significant distances. The check/sluice structure alternative would deliver more flow to the canals upstream from the check structures, but this additional flow would be returned via the sluiceway channels, so reduced water-surface elevations would only be expected in the short reach between the dam and the sluiceway channel outflows. The vortex tubes would probably be operated in a similar manner, and because the flows extracted by the tubes represent a small portion of the total canal flows, the reduction to water-surface elevations between the dam and the return channels would probably be unsubstantial. The effect of the additional automated gate operators at Mesilla Dam depends on the actual sluicing operations that would be undertaken, but altered sluicing operations would likely have similar effects on water-surface elevation compared to current practices.

The check/sluiceway structure and vortex tube alternatives will therefore primarily affect sediment loading to the canals and in the RGCP downstream from Mesilla Dam. To assess the effects on sediment loading in the canals, the cumulative mass passing the downstream boundary condition of the canal reaches was compared between the base condition, check/sluiceway alternative, and vortex tube alternatives (**Figures 60 and 61**). The results indicate that the check/sluiceway alternative would significantly reduce the sediment delivery to the canals, with reductions of between 89 percent (Eastside Canal) to 95 percent (Westside Canal). The results of the simulations of these alternatives were compared to the results of the baseline models to assess these effects. The predicted reduction of sediment delivery to the canals that would result from the vortex tubes is less substantial, with about a 26 percent reduction in the Eastside Canal and a 24 percent reduction in the Westside Canal (Figures 60 and 61). These reductions are slightly less than the 30 percent trap efficiency assumption that was used to prepare the model input because of the small differences in sediment loading along the modeled canal reaches. The response of the downstream river to these alternatives is more complicated because the alternatives affect the flow and sediment loading between the dam and the return channels. The check and sluiceway alternative results in reduced sediment loads passing the downstream river boundary condition during and after the period of “full” operation (when upstream inflows are greater than 2,000 cfs) due to reduced degradation levels between the dam and the sluiceway returns (**Figures 62 and 63**). Later in the simulation, downstream river deliveries are slightly higher than under the base condition due to the increased sediment loads delivered by the sluiceway channels. The results from the simulation of the vortex tube alternative indicate that the aggradation/degradation patterns in the downstream river are very similar to the base condition, so the resulting increase to downstream river loads are primarily a result of the increased sediment loads delivered by the returns (Figures 62 and 63).

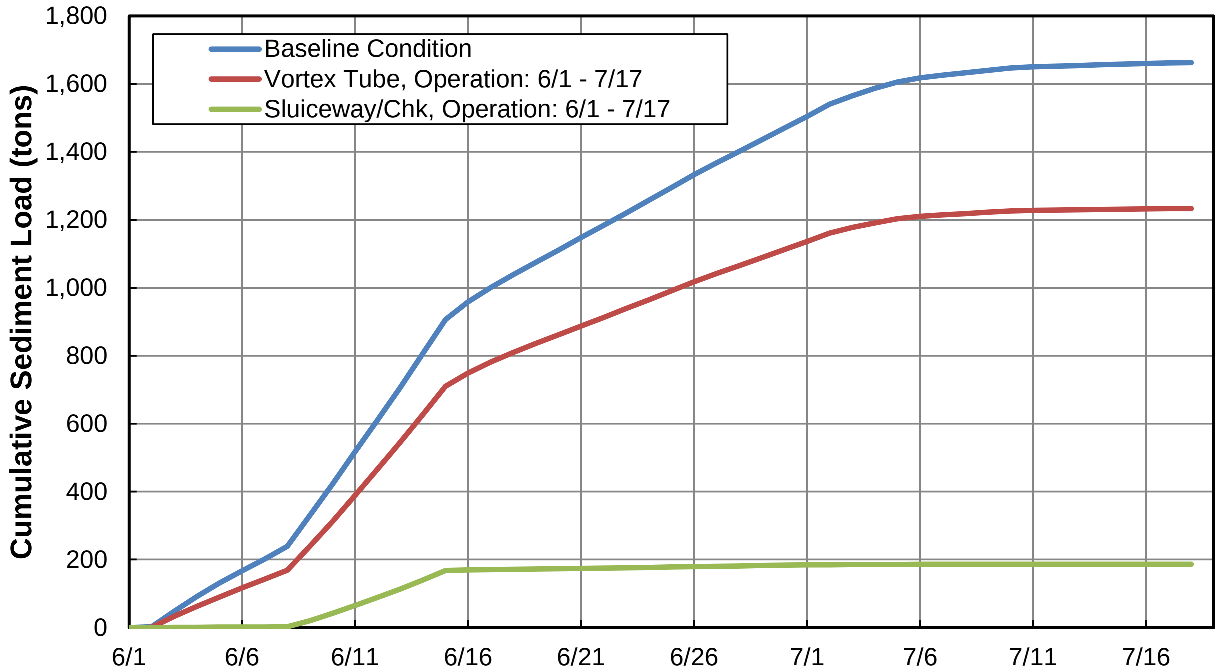

Figure 60. Predicted cumulative mass passing the downstream boundary condition of the Eastside Canal model reach for the base condition and alternatives for the check/sluiceway and vortex tubes.

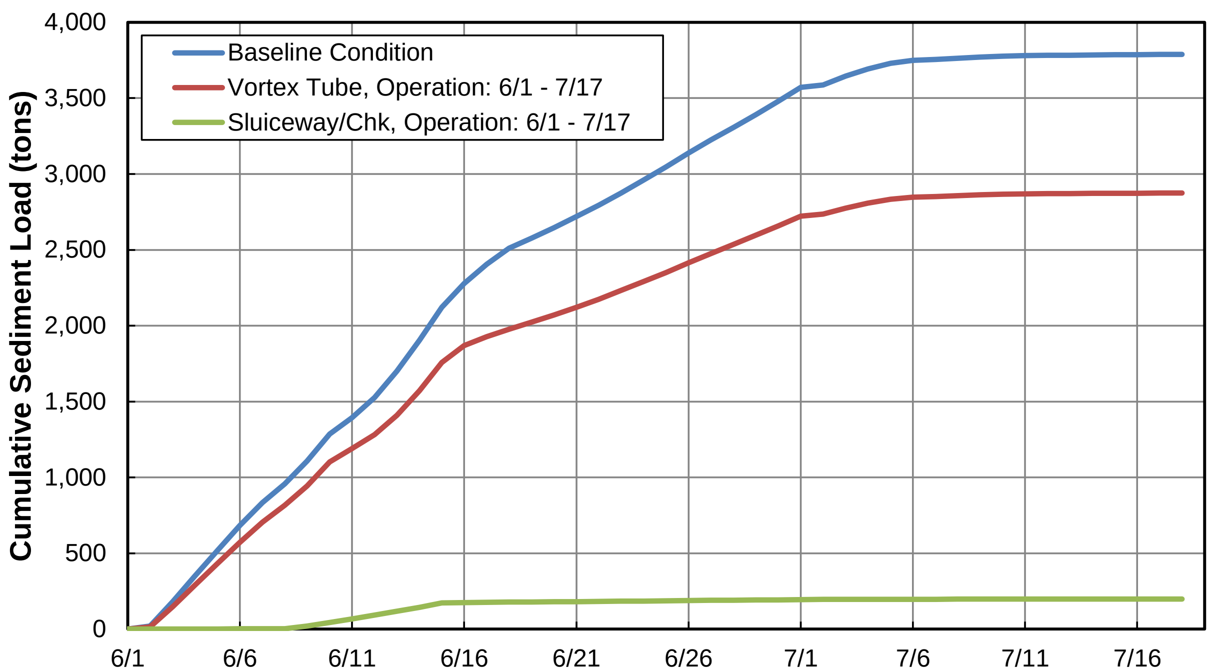

Figure 61. Predicted cumulative mass passing the downstream boundary condition of the Westside Canal model reach for the base condition and alternatives for the check/sluiceway and vortex tubes.

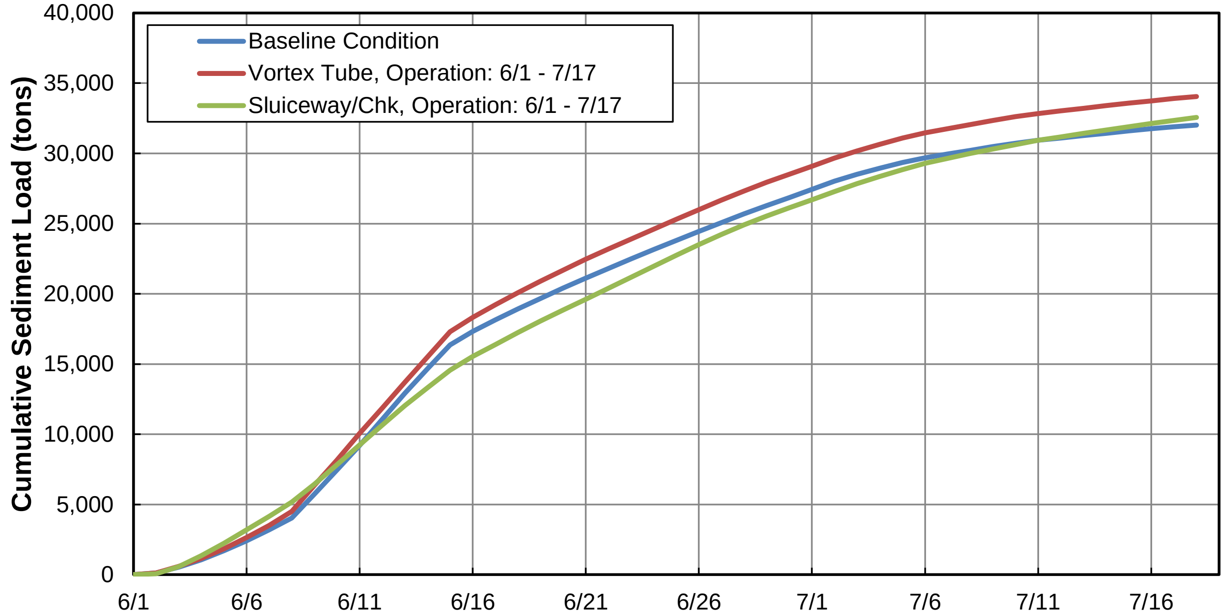

Figure 62. Predicted cumulative mass passing the downstream boundary condition of the RGCP model reach for the base condition and alternatives for the check/sluiceway and vortex tubes.

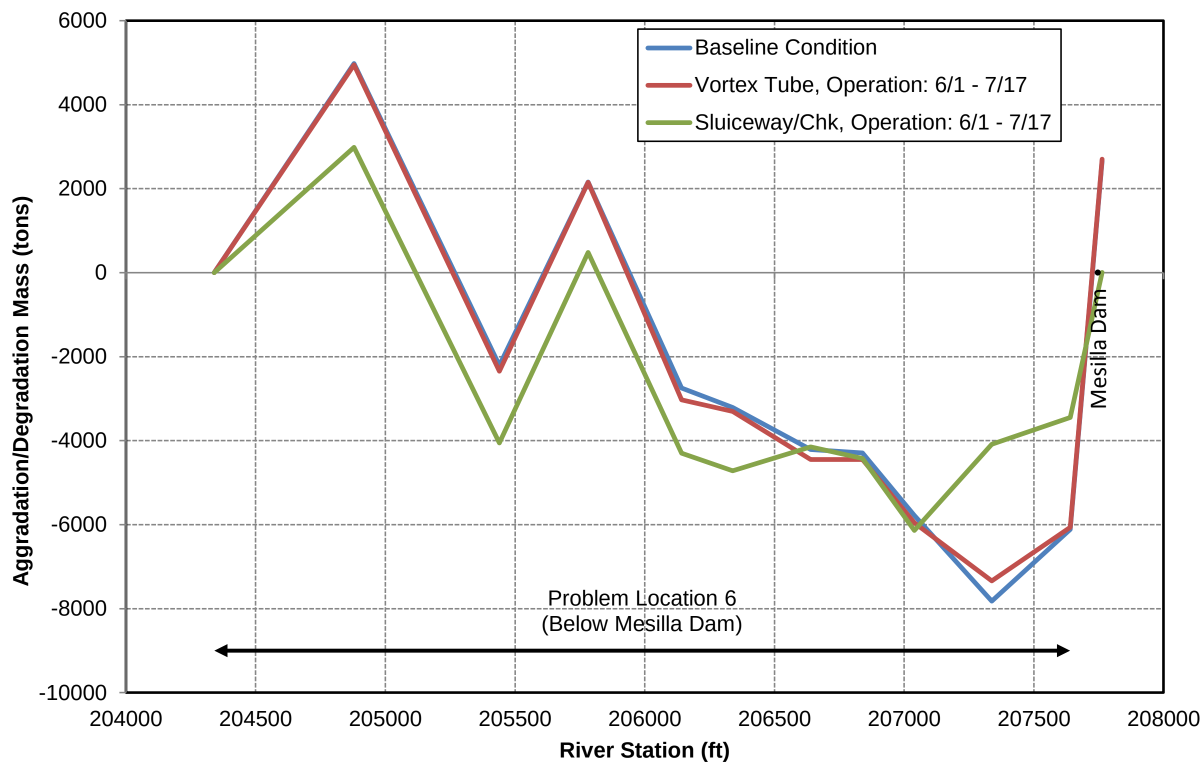

Figure 63. Predicted mass of aggradation or degradation along the reach of the RGCP downstream from Mesilla Dam at the end of the simulations for the base condition and alternatives for the check/sluiceway and vortex tubes.

Results from the sediment-transport simulations that were used to assess sluicing operations at Mesilla Dam Gates 2 and 5 were reviewed to determine the potential benefits associated with the alternative that involves installation of additional automated gate operators at Mesilla Dam. Because the 1-D sediment-transport routines in HEC-RAS are not capable of directly evaluating the 2-D effects of this alternative on water-surface elevation and sediment load, the predicted channel geometry at the end of the sluice operation simulations were compared to determine the lateral erosion and deposition patterns upstream from the dam (**Figure 64**). As expected, this comparison indicates that use of different gates would concentrate the erosion in the immediate area upstream from the gate that is being used for sluicing, while deposition would occur along the portion of the bed that is outside of the influence of the sluice gate. This conclusion indicates that use of a variety of gates for sluicing operations would be beneficial for sediment management purposes.

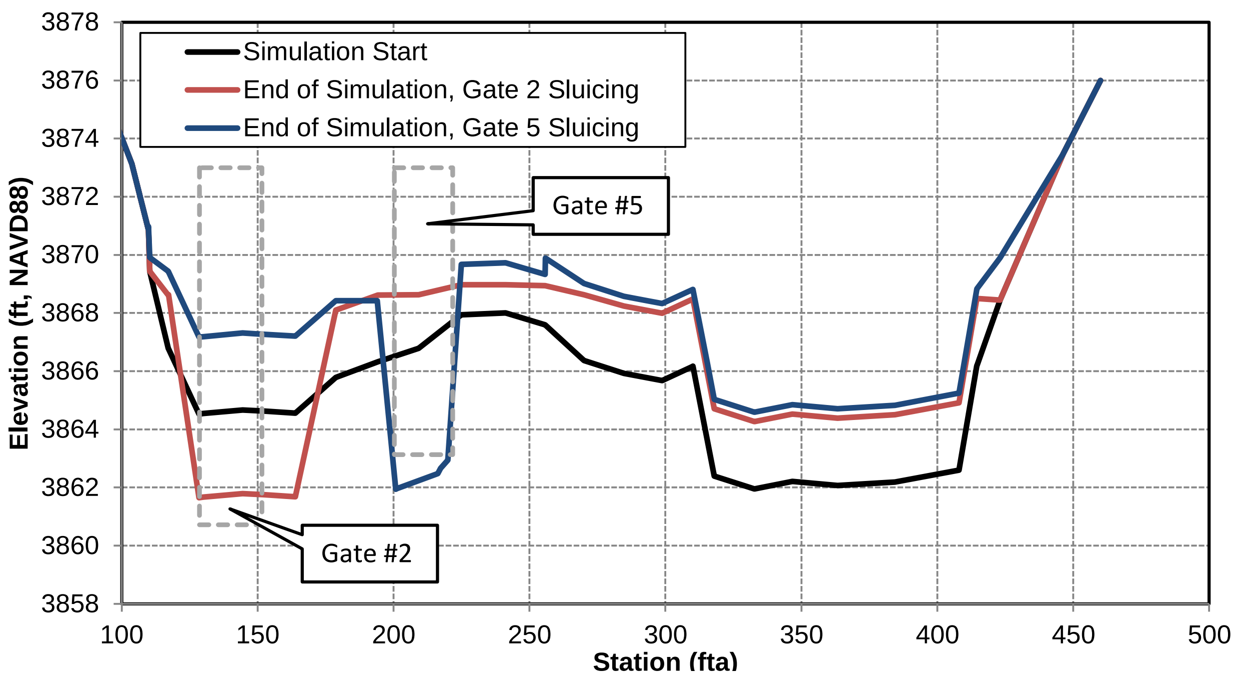

Figure 64. Cross-sectional geometry at the start of the simulations and predicted geometry at the end of the simulations for the models with sluicing operations for Mesilla Dam Gates 2 and 5

6 ALTERNATIVE EVALUATION

The expected benefits, costs and consequences associated with the alternatives were assessed in concert to identify the two alternatives that had the highest benefit relative to cost/consequence at each problem location. The benefits considered in this assessment included:

1. Reduction in water-surface elevation along the modeled reach.
2. Reduced levee freeboard encroachments.
3. Groundwater benefits, which include the benefit of increased groundwater levels in the vicinity of restoration sites as well as reduced groundwater levels elsewhere.
4. Reduction in aggradation and downstream sediment loading.
5. Improved irrigation drain return flows.
6. Durability of the alternative.
7. Restoration benefits, in addition to those benefits associated with increased groundwater levels.
8. Additional site-specific benefits.

The costs and consequences considered in this assessment included:

1. Annualized total cost of the alternative based on the up-front construction cost and projected O&M costs.
2. Increased water-surface elevations along the model reach.
3. Levee freeboard encroachments.
4. Groundwater consequences, which include the consequence of decreased groundwater levels in the vicinity of restoration sites as well as increased groundwater levels elsewhere.
5. Increases to aggradation and downstream sediment loading.
6. Increased bank erosion potential.
7. Restoration consequences, in addition to those consequences associated with increased groundwater levels.
8. Additional site-specific consequences.

A scoring system was prepared for each of these parameters using the results from the hydraulic and sediment-transport modeling discussed in the previous sections. These systems were used to score each of the alternatives under the various scoring parameters and prepare overall benefit and cost/consequence scores. The two alternatives at each of the problem locations with the highest difference of benefit to cost were then identified and recommended for further consideration. The methods used in the alternative scoring and ranking are outlined below, along with the details associated with the determination of annualized project costs of the alternatives.

6.1 Alternative Construction Cost Estimates

Construction cost estimates have been developed for all alternatives within each problem location as discussed in previous sections of this report. The purpose of this section is to present the capital costs, along with annual operations and maintenance (O&M) costs for each alternative. The estimates have been prepared with the best information available at this time and are considered pre-feasibility level estimates that are for comparison purposes and not for budgeting purposes. It is anticipated that many of the assumptions used within the estimates will be modified as more detailed information becomes available (see **Appendix P** for the detailed cost estimates for each alternative and problem location).

6.1.1 Basis of Alternative Estimates

A detailed discussion of each of the alternatives, for each of the nine problem locations, can be seen in Section 4. Table 21 associates the problem locations and the proposed alternatives at each location.

6.1.2 Basis of O&M Estimates

O&M costs have been estimated for each alternative at each problem location. The O&M costs have been estimated based on an assumed 50-year project life cycle. To calculate the annual O&M costs within the 50-year cycle, assumed percentages of the original construction costs have been utilized. The percentages and frequencies of O&M have all been estimated individually for each alternative and problem location. Therefore the O&M estimates are site and alternative specific.

The percentages used for the O&M costs associated with the sediment-removal alternatives are primarily derived from the sediment-transport models developed for this work (see Section 5.4.2). If, for example, an area is anticipated to fill in after five and a half (5.5) years, then the O&M estimate would assume excavating a proportion of the full fill-in volume every fifth year such that the area does not fill in one hundred percent. For the sediment-trap alternatives, the O&M schedule was based on the ratio of the trap volume to the average annual bed-load yield. Use of this ratio is conservative in that the trap screens were designed to pass the finer portions of the tributary loadings, so it would take a longer sequence of mean annual loading events to fill the traps. For the other non-sediment removal alternatives, engineering judgment was used to determine how often, and the level of effort required for the O&M schedule.

6.1.3 Key Cost Estimate Assumptions

Below is a discussion of the key cost estimating assumptions. These items are attributable to most of the alternatives and provide a basis for the unit cost development.

6.1.3.1 Mobilization and Demobilization

The costs for mobilization and demobilization have been estimated at 12.5 percent of the total construction cost for each alternative. This cost is assumed to cover the contractor's cost to move labor personnel and equipment to and from the project site, as well as set up any contractor facilities.

6.1.3.2 Site Access and Staging

The costs for this item have been estimated at 2.5 percent of the total construction cost for each alternative. This cost is assumed to account for any miscellaneous site access requirements and the set-up of any staging areas.

6.1.3.3 Excavation

All excavation is assumed to be completed with either dozers and/or hydraulic excavators. It has been assumed that for the sediment removal alternatives, the excavated material would be hauled by truck a short distance (assumed to be 1 mile) to be disposed. No tipping fees would be included. For the other alternatives all excavated material is assumed to be disposed of outside the channel but would not require any truck hauling.

6.1.3.4 Sediment Traps Steel

The sediment traps consist of various size mesh rows that would collect sediment within the arroyos. These traps range in mesh size and material, and include 1-foot rebar, 8-inch rebar, 6-inch rebar, 4-inch wire, 2-inch wire, and 1-wire meshes. Each of the lines of steel mesh would have steel posts placed every 12 feet. It is assumed that the traps would be fabricated in the field.

6.1.3.5 Compacted Fill

Compacted fill is required in various construction alternatives. No borrow material is assumed to be required for any backfill as sufficient excavated material would be available for reuse.

6.1.3.6 Stone Placement

Various alternatives require stone materials for spur dikes, bank protection and/or bedding materials. All stone is assumed to be purchased from a quarry within 200 miles and delivered to the project locations for placement.

6.1.3.7 Maintenance Roads

All maintenance roads noted on the alternative maps are assumed to be 15-foot wide graded and compacted roadways, with some stabilizing material included to prevent degradation of the roadway.

6.1.4 Project Mark-ups

6.1.4.1 Planning, Engineering, & Design

Costs for this item were estimated at 15 percent of the total construction costs. This percentage is assumed to cover the preparation of the plans and specifications.

6.1.4.2 Construction Management

Costs for this item were estimated at 10 percent of the total construction costs. This percentage is assumed cover construction management costs during the construction phase.

6.1.4.3 Contingency

Contingencies represent allowances to cover unknowns, uncertainties and/or unanticipated conditions that are not possible to adequately evaluate from the data on hand at the time the cost estimate is prepared. Due to the low level of design and the number of general assumptions used in the estimate, a 30-percent contingency has been included for each alternative and site.

6.1.4.4 Mitigation Costs

For this assessment, no mitigation costs have been included in the alternative cost estimates. Considering that this study is at a pre-feasibility level, this assumption is appropriate for purposes of this evaluation of channel maintenance strategies. However, mitigation may be required for certain USACE Section 404 permitted projects such as the arroyo sediment traps, spur dikes, island/bar destabilization and spur dike alternatives.

6.1.5 Summary of Annualized Costs

In order to adequately compare costs between each of the alternatives, an annualized cost was calculated (**Table 34**). This value takes into account all costs, construction and annual O&M, over the project life span. The estimates have assumed that the O&M cycle would occur over 50 years, and that the current discount rate is 3.375 percent (USACE, 2014) to be used in the annualized costs calculations. The total annualized costs indicate that the sediment removal alternatives are generally more expensive than the non-sediment removal alternatives due to the large O&M costs associated with the required excavation frequency. In some cases, the annualized cost for the

Table 34. Annualized first costs, O&M costs, and total costs for each of the alternatives evaluated in this study.

Problem Location	Alternative	Total First Costs	Total O&M Costs	Total Annualized Costs	Problem Location	Alternative	Total First Costs	Total O&M Costs	Total Annualized Costs
1	Channel Excavation (Short)	$304,001	$3,467,572	$157,300	6	Channel Excavation (Short)	$514,781	$30,654,358	$1,299,100
	Channel Excavation (Long)	$717,610	$5,356,608	$253,300		Channel Excavation (Long)	$843,744	$5,095,123	$247,600
	Channel Excavation (Localized)	$91,028	$2,385,810	$103,300		New Check/Sluice Structures in Canals	$3,298,338	$414,952	$154,800
	Sediment Traps in Arroyos	$1,094,920	$5,739,501	$285,000		Mesilla Dam Gate Automation	$3,565,000	$373,750	$164,200
	Modification of the TB Vortex Weir	$31,092	$65,193	$4,100		Installation of Vortex Tubes	$422,453	$177,158	$25,100
2	Channel Excavation (Short)	$1,229,157	$5,102,983	$264,000	7	Channel Excavation (Short)	$554,219	$2,993,500	$147,900
	Channel Excavation (Long)	$1,842,150	$7,493,393	$389,200		Channel Excavation (Long)	$701,998	$3,250,026	$164,800
	Channel Excavation (Localized)	$667,455	$3,683,492	$181,500		Channel Excavation (Localized)	$63,986	$1,006,224	$44,700
	Sediment Traps in Arroyos	$721,108	$1,451,520	$90,600		Sediment Traps in Arroyos	$600,031	$1,256,116	$77,500
	Island Destabilization / Vegetation Removal	$525,357	$1,321,865	$77,000		Low-Elevation Spur Dikes	$634,828	$1,064,874	$70,900
3	Channel Excavation (Short)	$251,299	$1,150,786	$58,500	8	Channel Excavation (Short)	$311,110	$4,892,462	$217,000
	Channel Excavation (Long)	$530,398	$2,068,552	$108,500		Channel Excavation (Long)	$624,356	$5,341,267	$248,800
	Channel Excavation (Localized)	$163,823	$904,093	$44,600		Channel Excavation (Localized)	$128,123	$3,223,751	$139,800
	Sediment Traps in Arroyos	$154,634	$182,089	$14,100		Riprap in Narrow Floodplain Areas	$415,412	$261,307	$28,300
	Replace Rincon Siphon with Flume	$1,989,426	$417,138	$100,400		Low-Elevation Spur Dikes	$305,367	$512,229	$34,200
4	Channel Excavation (Short)	$942,131	$14,815,763	$656,800	9	Channel Excavation (Short)	$562,034	$5,958,284	$271,900
	Channel Excavation (Long)	$3,199,444	$12,477,832	$653,500		Channel Excavation (Long)	$2,552,634	$10,276,409	$534,700
	Channel Excavation (Localized)	$548,800	$8,630,317	$382,600		Channel Excavation (Localized)	$225,419	$13,517,893	$572,800
	Island Destabilization / Spur Dikes	$664,644	$1,672,331	$97,500		Island Destabilization / Vegetation Removal	$219,529	$552,364	$32,300
	Low-Elevation Spur Dikes	$579,401	$971,898	$64,800		Low-Elevation Spur Dikes	$232,840	$390,570	$26,100
5	Channel Excavation (Short)	$3,830,434	$3,472,452	$304,500					
	Channel Excavation (Long)	$3,830,434	$2,632,747	$269,500					
	Channel Excavation (Localized)	$3,830,434	$3,968,730	$325,200					
	Sediment Traps in Arroyos	$3,830,434	$386,636	$175,900					
	Low-Elevation Spur Dikes	$3,830,434	$1,500,055	$222,300					

localized excavation was the most expensive of the sediment removal alternatives due to the high frequency of O&M events, while the long excavation alternative had the lowest annualized cost. The lowest total annualized cost was associated with the modified Tierra Blanca Vortex Weir alternative ($4,100/year) while the most expensive alternative was the short excavation scenario above Mesilla Dam at Problem Location 6 (about $1.3 million annually).

6.2 Scoring System

Scoring systems were developed for each of the benefit and cost/consequence parameters, as discussed below. These systems were then used to assign scores to the individual alternatives for the various parameters. Tables showing the actual scoring for each parameter are presented in **Appendix Q**.

6.2.1 Changes in Water-surface Elevation

Any reduction in water-surface elevation that would result from alternative implementation would be a benefit, while increased water-surface elevations would be a consequence. These benefits and consequences were assessed using the results from the long-term water-surface elevation analysis presented in Section 5.4.3. For each alternative, the average reduction or increase to water-surface elevation and the maximum decrease or increase in water-surface elevation, relative to the base condition, were used to determine the benefit scores as presented in **Table 35**. Because average reductions or increases along the model reach are more representative of the reach-wide condition than the two maximum decrease categories (normal operating flows and 100-year peak discharge) each of the maximum categories were assigned a 10-percent weighting, while the two average categories were each assigned a 40-percent weighting. The scoring breakdown for reduction in water-surface elevation under each of the alternatives is presented in Table Q.1 of Appendix Q.

6.2.2 Groundwater Effects

The effects of the alternatives on groundwater levels could result in both benefits and consequences. Lowered groundwater levels along the riparian corridor would, in general, reduce the salinity of surface soils and thus be beneficial. Lowered groundwater in the vicinity of drains would also be beneficial due to increased drain return efficiencies. However, an increase in groundwater level is desirable and would be of benefit at the restoration sites. A detailed groundwater analysis was not included as part of this study, so the average long-term water-surface profiles at normal operating flows were used to estimate the effects of the alternatives on groundwater levels. The scoring breakdown uses the same water-surface elevation reduction and increase classes used for the water-surface elevation scoring, but the parameters include the three groundwater considerations (**Table 36**). It should be noted that a number of problem locations have both drains and restoration sites, so the scorings associated with these two considerations cancel each other out, leaving only the general reduction in groundwater (salinity-based) parameter to drive the scores; this occurs at Problem Locations 2, 3, 5, 6 and 7. The scoring breakdown for groundwater effects under each of the alternatives is presented in Table Q.2 of Appendix Q.

6.2.3 Aggradation/Degradation and Sediment Loading

Results from the sediment-transport modeling were used to prepare the scorings for assessing the effects of the alternatives on aggradation/degradation and on downstream sediment loading. This scoring parameter covers the benefits of reduced aggradation along the reach and reduced sediment deliveries to downstream reaches and the consequences associated with increased aggradation along the reach and increased sediment deliveries to downstream reaches. Both of

Table 35. Water-surface elevation scoring system.

Score Type	Parameter	Reduction of Water-surface Elevation Relative to Base Condition (ft)									
Benefits	Average Reduction, Normal Operating Flows	0 - 0.05	0.05 - 0.1	0.1 - 0.2	0.2 - 0.4	0.4 - 0.6	0.6-0.8	0.8-1	1-1.5	1.5-2	> 2
	Score	1	2	3	4	5	6	7	8	9	10
	Average Reduction, 100-yr Flood	0 - 0.05	0.05 - 0.1	0.1 - 0.15	0.15 - 0.2	0.2 - 0.3	0.3 - 0.4	0.4 - 0.5	0.5 - 0.75	0.75 - 1	> 1
	Score	1	2	3	4	5	6	7	8	9	10
	Maximum Decrease, Normal Operating Flows	0 - 0.1	0.1 - 0.25	0.25 - 0.5	0.5 - 0.75	0.75 - 1	1 -1.5	1.5 - 2	2 - 2.5	2.5 - 3	>3
	Score	0.25	0.5	0.75	1	1.25	1.5	1.75	2	2.25	2.5
	Maximum Decrease, 100-yr Flood	0 - 0.1	0.1 - 0.25	0.25 - 0.5	0.5 - 0.75	0.75 - 1	1 -1.5	1.5 - 2	2 - 2.5	2.5 - 3	>3
	Score	0.25	0.5	0.75	1	1.25	1.5	1.75	2	2.25	2.5
	Net Score = Sum/2.5	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**
Consequences	Average Increase, Normal Operating Flows	0 - 0.05	0.05 - 0.1	0.1 - 0.15	0.15 - 0.2	0.2 - 0.25	0.25 - 0.3	0.3 - 0.35	0.35 - 0.4	0.4 - 0.5	> 0.5
	Score	1	2	3	4	5	6	7	8	9	10
	Average Increase, 100-yr Flood	0 - 0.05	0.05 - 0.1	0.1 - 0.15	0.15 - 0.2	0.2 - 0.25	0.25 - 0.3	0.3 - 0.35	0.35 - 0.4	0.4 - 0.5	> 0.5
	Score	1	2	3	4	5	6	7	8	9	10
	Maximum Increase, Normal Operating Flows	0 - 0.05	0.05 - 0.1	0.1 - 0.15	0.15 - 0.2	0.2 - 0.25	0.25 - 0.3	0.3 - 0.35	0.35 - 0.4	0.4 - 0.5	> 0.5
	Score	0.25	0.5	0.75	1	1.25	1.5	1.75	2	2.25	2.5
	Maximum Increase, 100-yr Flood	0 - 0.05	0.05 - 0.1	0.1 - 0.15	0.15 - 0.2	0.2 - 0.25	0.25 - 0.3	0.3 - 0.35	0.35 - 0.4	0.4 - 0.5	> 0.5
	Score	0.25	0.5	0.75	1	1.25	1.5	1.75	2	2.25	2.5
	Net Score = Sum/2.5	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**

Table 36. Groundwater level scoring system.

Score Type	Parameter	Reduction or Increase of Water-surface Elevation Relative to Base Condition (ft)									
Benefits	General Reduction (Improves Salinity)	0 - 0.05	0.05 - 0.1	0.1 - 0.2	0.2 - 0.4	0.4 - 0.6	0.6-0.8	0.8-1	1-1.5	1.5-2	> 2
	Score	1	2	3	4	5	6	7	8	9	10
	Average Reduction at Drains	0 - 0.05	0.05 - 0.1	0.1 - 0.2	0.2 - 0.4	0.4 - 0.6	0.6-0.8	0.8-1	1-1.5	1.5-2	> 2
	Score	1	2	3	4	5	6	7	8	9	10
	Average Increase at Restoration Sites	0 - 0.05	0.05 - 0.1	0.1 - 0.15	0.15 - 0.2	0.2 - 0.25	0.25 - 0.3	0.3 - 0.35	0.35 - 0.4	0.4 - 0.5	> 0.5
	Score	1	2	3	4	5	6	7	8	9	10
	Net Score = Sum/3	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**
Consequences	General Increase (Worsens Salinity)	0 - 0.05	0.05 - 0.1	0.1 - 0.15	0.15 - 0.2	0.2 - 0.25	0.25 - 0.3	0.3 - 0.35	0.35 - 0.4	0.4 - 0.5	> 0.5
	Score	1	2	3	4	5	6	7	8	9	10
	Average Increase at Drains	0 - 0.05	0.05 - 0.1	0.1 - 0.15	0.15 - 0.2	0.2 - 0.25	0.25 - 0.3	0.3 - 0.35	0.35 - 0.4	0.4 - 0.5	> 0.5
	Score	1	2	3	4	5	6	7	8	9	10
	Average Reduction at Restoration Sites	0 - 0.05	0.05 - 0.1	0.1 - 0.2	0.2 - 0.4	0.4 - 0.6	0.6-0.8	0.8-1	1-1.5	1.5-2	> 2
	Score	1	2	3	4	5	6	7	8	9	10
	Net Score = Sum/3	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**

these parameters were scored based on the percent difference with the base condition (**Table 37**). The percent difference range used for each score was based on the relative frequency of results in each range. The aggradation along the extents of the problem location was used together with the aggradation along the overall model reach because the predicted trends of aggradation and degradation along the overall model reach should represent the response of the channel downstream from the modeled reach, and therefore should bare more weight than the downstream sediment loading category in the scoring. In addition, the increased downstream sediment loads are primarily sand (similar to the base condition), which is less problematic because the RGCP is more capable of transporting sands than gravels. As such, the downstream sediment loading parameter received a 20-percent weight while the aggradation along the model reach and within the extents of the problem location each received a weight of 40 percent. The scoring breakdown for this parameter under each of the alternatives is presented in Table Q.3 of Appendix Q.

At Problem Location 6, the modeling of the check/sluiceway structure and vortex tube alternatives was simulated over a single irrigation season. Since the benefits associated with reductions to sedimentation in the canals are of upmost importance, the predicted reduction to sediment deliveries to the downstream canals, relative to the base condition, were used in lieu of the composite scoring system presented in Table 37. The results indicate that the sluiceway/check structure alternative would reduce the downstream sediment loading in the canals by 92 percent and the vortex tube alternative would result in a 25-percent reduction (see Section 5.4.4); benefit scores of 9.2 and 2.5 were therefore assigned to these two alternatives, respectively. Both of these alternatives would result in relatively small increases to sediment deliveries to the downstream river, so the scoring breakdown for this category presented in Table 37 was used to prepare the consequence scores for increased downstream sediment loading. For the automated gate alternative, it was assumed that the benefits associated with reduced aggradation upstream from the dam would be offset by increases to downstream sediment loading, so zero scores were used for the benefits and consequences of this alternative. Additional site-specific benefits and consequences associated with the non-sediment removal alternatives at Problem Location 6 are factored into the scoring as discussed below.

6.2.4 Improved Irrigation Return Flows

The relative water-surface profile comparisons were also used to score the benefits associated with enhanced irrigation return flow efficiencies since lowered water-surface elevations in the river would improve performance of the drains. The initial water-surface comparison prior to any channel response was used along with the long-term water-surface profile comparison to assign these scores, since some of the localized excavation alternatives are currently being carried out at the mouths of the drains with the purpose of obtaining short-term benefits. The system used for scoring the benefits to irrigation return flows is presented in **Table 38**. This scoring parameter was used at all sites where a drain of any size returns flow to the RGCP (including, for example, the small drains at Problem Location 2). A benefit score of zero was assigned at problem locations where no drains enter the RGCP (Problem Locations 1 through 8). At Problem Location 6, neither the check/sluiceway structure alternative nor the vortex tube alternative would have an appreciable effect on drain return flows. However, it was assumed that the gate automation alternative would reduce the level of aggradation upstream from the dam, and therefore improve irrigation drain return flows, so a benefit score of 6 was assigned to this alternative. The scoring breakdown for improved irrigation return flows under each of the alternatives is presented in Table Q.4 of Appendix Q.

Table 37. Sediment loading scoring system.

Rank Class	Parameter	Percent Reduction or Increase Relative to Base Condition									
Benefits	Aggradation Within Extent of Problem Location[1]	0% - 5%	5% - 10%	10% - 20%	20% - 30%	30% - 40%	40% - 50%	50% - 75%	75% - 100%	100% - 150%	> 150%
	Score	1	2	3	4	5	6	7	8	9	10
	Aggradation Along Model Reach[1]	0% - 5%	5% - 10%	10% - 20%	20% - 30%	30% - 40%	40% - 50%	50% - 75%	75% - 100%	100% - 150%	> 150%
	Score	1	2	3	4	5	6	7	8	9	10
	Cumulative Downstream Sediment Load[1]	0% - 2.5%	2.5% - 5%	5% - 10%	10% - 15%	15% - 20%	20% - 30%	30% - 40%	40% - 50%	50% - 75%	75% - 100%
	Score	1	2	3	4	5	6	7	8	9	10
	Net Score = Sum/3	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**
Consequences	Aggradation Within Extent of Problem Location[2]	0% - 5%	5% - 10%	10% - 20%	20% - 30%	30% - 40%	40% - 50%	50% - 75%	75% - 100%	100% - 150%	> 150%
	Score	1	2	3	4	5	6	7	8	9	10
	Aggradation Along Model Reach[2]	0% - 5%	5% - 10%	10% - 20%	20% - 30%	30% - 40%	40% - 50%	50% - 75%	75% - 100%	100% - 150%	> 150%
	Score	1	2	3	4	5	6	7	8	9	10
	Cumulative Downstream Sediment Load[2]	0% - 2.5%	2.5% - 5%	5% - 10%	10% - 15%	15% - 20%	20% - 30%	30% - 40%	40% - 50%	50% - 75%	75% - 100%
	Score	1	2	3	4	5	6	7	8	9	10
	Net Score = Sum/3	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**

[1]Percent Reduction Relative to Base
[2]Percent Increase Relative to Base

Table 38. Improved drain efficiency scoring system.

Score Type	Parameter	Reduction of Water-surface Elevation Relative to Base Condition (ft)									
Benefits	Initial Average Reduction, Normal Operating Flows	0 - 0.05	0.05 - 0.1	0.1 - 0.2	0.2 - 0.4	0.4 - 0.6	0.6-0.8	0.8-1	1-1.5	1.5-2	> 2
	Score	1	2	3	4	5	6	7	8	9	10
	Long-term Average Reduction, Normal Operating Flows	0 - 0.05	0.05 - 0.1	0.1 - 0.2	0.2 - 0.4	0.4 - 0.6	0.6-0.8	0.8-1	1-1.5	1.5-2	> 2
	Score	1	2	3	4	5	6	7	8	9	10
	Net Score = Sum/2	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**

6.2.5 Durability

The durability of a particular alternative refers to the time period over which maintenance would be required to ensure the strategy remains effective. The maintenance periods for the sediment removal alternatives were determined using the results from the sediment-transport modeling presented in Section 5.4.2, consistent with the O&M maintenance schedule that was prepared for the cost estimates. Similarly, the maintenance periods for the sediment traps were based on the ratio of the trap volume to the average annual bed-load yield, also consistent with the cost-based O&M maintenance schedule. For the remainder of the non-sediment removal alternatives, the maintenance periods were estimated based on engineering judgement. It was assumed that the modified Tierra Blanca Vortex Weir would require very little maintenance (once every ten years). Maintenance of the Rincon Flume, which was proposed to replace the Rincon Siphon, was assumed to be bi-annual, primarily requiring basic structural and sediment management activities. A 3-year year maintenance cycle was assumed for the spur dikes to remove material that accumulates between the spurs under the assumption that this would be necessary for purposes of flood conveyance. Each of the non-sediment removal alternatives at Mesilla Dam were assumed to require annual maintenance due to the mechanical nature of these proposals. A 10-year maintenance period for the riprap alternative at Problem Location 8 was used, since the riprap would probably not require more frequent enhancements. The scoring system for the durability benefits is presented in **Table 39**, and the scoring breakdown benefits under each of the alternatives along with the assumed maintenance schedule is presented in Table Q.5 of Appendix Q.

6.2.6 Costs

The cost rating score was normalized to provide a zero to the lowest cost alternative and a nine for the second most expensive alternative based on the total annualized costs. Each of the other alternative ratings fall proportionally within the zero to nine range, and the most expensive option was assigned a score of ten. This was used in lieu of a set rating range (i.e., 1 = <$30k, 2 = $30k – 50k, etc.) due to the large spread of the data and due to outliers on the expensive end of the cost range. The cost scoring breakdown is shown graphically in **Figure 65**, and the scoring for each alternative is presented in Table Q.6 of Appendix Q.

6.2.7 Levee Freeboard Encroachments

Any reduction in the 3 feet of levee freeboard required to protect against the 100-year flood would be a consequence for an individual alternative, while an increase in levee freeboard at any location where the 100-year flood encroaches into the required 3 feet of freeboard would be a benefit. These benefits and consequences were assessed using the long-term 100-year water-surface elevation presented in Section 5.4.3 and the top of levee elevations from the model. For each alternative, the average increase or decrease in levee freeboard, when compared to the long-term base condition, were used to calculate the alternative score based on the breakdown shown in **Table 40**. The average reductions or increases along the model reach were assigned a 40-percent weighting for both the left and right levees, while the maximum increase and maximum decrease in freeboard were each assigned a 10-percent weighting. The scorings for levee freeboard encroachment among each of the alternatives is presented in Table Q.7 of Appendix Q.

6.2.8 Increased Bank Erosion Potential

The potential for increased bank erosion is another consequence that could result from the alternatives. Because the 1-D sediment-transport modeling is not capable of directly predicting bank erosion, this consequence was evaluated using the results from the localized steady-state hydraulic models with the end of simulation geometry. The total shear stress in the channel should

Table 39. Durability scoring system.

Score Type	Parameter	Maintenance Interval (years)									
Benefit	Estimated Maintenance Interval	< 1	1 - 2	2 - 3	3 - 4	4 - 5	5 - 6	6 - 8	8 - 10	10 - 20	> 20
	Net Score	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**

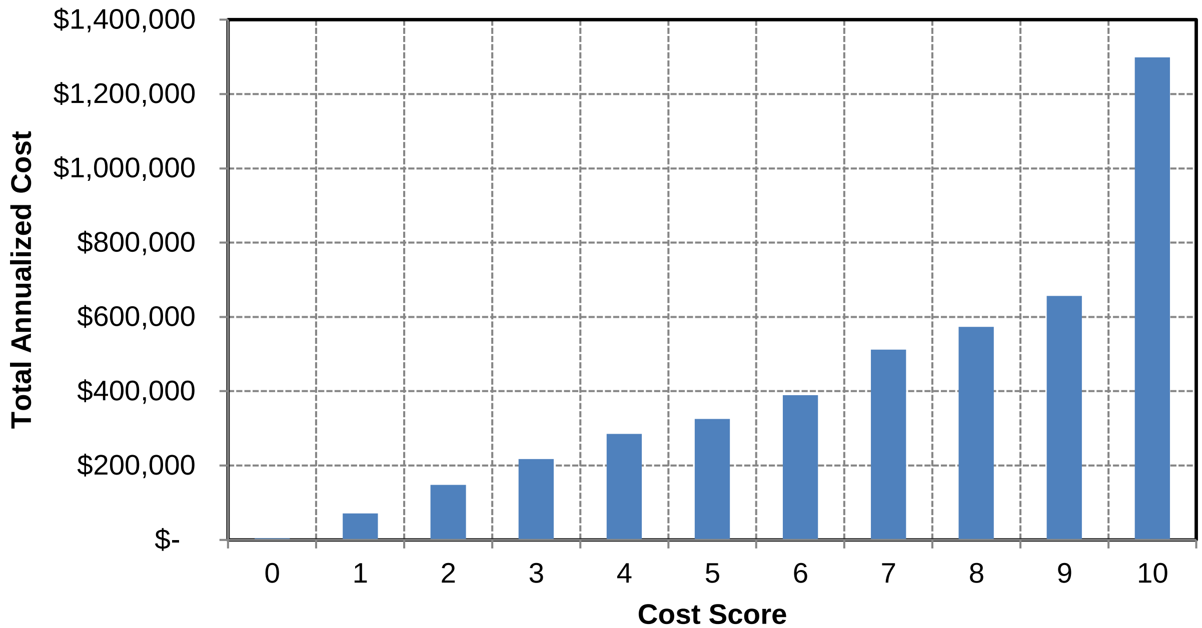

Figure 65. Cost scoring system based on the total annualized costs.

Table 40. Freeboard scoring system.

Score Type	Parameter	Change in Levee Freeboard Relative to Base Condition (ft)									
Benefits	Average Increase in Left Levee Freeboard	0 - 0.001	0.001 - 0.01	0.01 - 0.125	0.125 - 0.15	0.15 - 0.2	0.2 - 0.25	0.25 - 0.5	0.5 - 0.75	0.75 - 1	> 1
	Score	1	2	3	4	5	6	7	8	9	10
	Average Increase in Right Levee Freeboard	0 - 0.001	0.001 - 0.01	0.01 - 0.125	0.125 - 0.15	0.15 - 0.2	0.2 - 0.25	0.25 - 0.5	0.5 - 0.75	0.75 - 1	> 1
	Score	1	2	3	4	5	6	7	8	9	10
	Maximum Increase in Left Levee Freeboard	0 - 0.01	0.01 - 0.1	0.1 - 0.15	0.15 - 0.2	0.2 - 0.25	0.25 - 0.5	0.5 - 0.75	0.75 - 1	1 - 1.5	> 1.5
	Score	0.25	0.5	0.75	1	1.25	1.5	1.75	2	2.25	2.5
	Maximum Increase in Left Levee Freeboard	0 - 0.01	0.01 - 0.1	0.1 - 0.15	0.15 - 0.2	0.2 - 0.25	0.25 - 0.5	0.5 - 0.75	0.75 - 1	1 - 1.5	> 1.5
	Score	0.25	0.5	0.75	1	1.25	1.5	1.75	2	2.25	2.5
	Net Score = Sum/2.5	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**
Consequences	Average Decrease in Left Levee Freeboard	0 - 0.001	0.001 - 0.01	0.01 - 0.125	0.125 - 0.15	0.15 - 0.2	0.2 - 0.25	0.25 - 0.5	0.5 - 0.75	0.75 - 1	> 1
	Score	1	2	3	4	5	6	7	8	9	10
	Average Decrease in Right Levee Freeboard	0 - 0.001	0.001 - 0.01	0.01 - 0.125	0.125 - 0.15	0.15 - 0.2	0.2 - 0.25	0.25 - 0.5	0.5 - 0.75	0.75 - 1	> 1
	Score	1	2	3	4	5	6	7	8	9	10
	Maximum Decrease in Left Levee Freeboard	0 - 0.01	0.01 - 0.1	0.1 - 0.15	0.15 - 0.2	0.2 - 0.25	0.25 - 0.5	0.5 - 0.75	0.75 - 1	1 - 1.5	> 1.5
	Score	0.25	0.5	0.75	1	1.25	1.5	1.75	2	2.25	2.5
	Maximum Decrease in Right Levee Freeboard	0 - 0.01	0.01 - 0.1	0.1 - 0.15	0.15 - 0.2	0.2 - 0.25	0.25 - 0.5	0.5 - 0.75	0.75 - 1	1 - 1.5	> 1.5
	Score	0.25	0.5	0.75	1	1.25	1.5	1.75	2	2.25	2.5
	Net Score = Sum/2.5	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**

be a reasonable indicator of increased bank erosion potential, so this hydraulic parameter was used as the basis for the scoring (**Table 41**). The scorings for increased bank erosion potential among each of the alternatives is presented in Table Q.8 of Appendix Q.

6.2.9 Additional Restoration and Site-specific Benefits and Consequences

Many of the alternatives could result in a number of additional benefits and consequences that could affect habitat restoration or be site-specific. These benefits and consequences are qualitative in nature, so a qualitative class breakdown was used for the scoring (**Table 42**). The classes were assigned based on engineering judgement, and take into account the level of confidence associated with the model results.

The sediment removal alternatives would result in no additional restoration or site specific benefits, but because the excavations in many cases pass through vegetated islands and bars, a minor restoration consequence was assigned to these alternatives. Because an embayment is proposed for the arroyo sediment traps, this alternative was classified as having a moderate restoration benefit. While most of the sediment traps would not result in any site specific or restoration consequences, the sediment trap on Placitas Arroyo would probably reduce aggradation on the fan surface, which would in turn reduce the rate of erosion on the east bank opposite the fan. Because erosion of the east bank is desirable for purposes of habitat restoration at the Placitas Arroyo Restoration Site, the sediment-trap alternative received a minor consequence rating at Problem Location 2. Island/bar destabilization and vegetation removal was assigned a major site specific benefit class because this treatment would likely decrease evapotranspiration and improve conveyance efficiencies. Although these treatments are not recommended at any restoration sites, the clearing and grubbing could have adverse impacts on habitat, so a minor restoration consequence was assigned to this alternative. The spur dikes received a moderate restoration benefit since the spurs would result in increased hydraulic diversity that would probably improve aquatic habitat, and aggradation between the spurs would result in an inset floodplain that would improve riparian habitat.

Each of the site-specific alternatives were also scored for additional restoration and site-specific benefits and consequences. Although the modeling of the modified Tierra Blanca Vortex Weir indicated this alternative would have very little effect, the 1-D modeling is not capable of simulating the 2-D hydraulic conditions that would drive erosion and sedimentation at and downstream from the weir. In addition, removal of the grade control provided by the weir would increase the potential for headcut development that might occur during sustained periods of high flow, so this alternative was assigned a major site-specific benefit class. Removal of the grade control structure at the Rincon Siphon would improve fish passage, so a minor restoration benefit was assigned to this alternative. However, because this alternative would significantly increase sediment loading to downstream reaches and would result in downcutting that could affect the NM 154 and ATSF Railroad Bridges, as well as the restoration sites, this alternative received a major restoration consequence class and an extreme site-specific consequence class. An extreme site-specific benefit level was assigned to the riprap alternative at Problem Location 8, since the revetment would probably be very effective at protecting the west levee. The spur dike alternative at Problem Location 9 also includes island/bar destabilization and vegetation removal, so this alternative received restoration and site-specific scores for both types of alternative.

The 1-D modeling of the non-sediment removal alternatives at Problem Location 6 does not account for the 2-D and 3-D flow and sediment conditions at the dam and in the vicinity of the proposed alternatives, so the level of confidence of the model is somewhat less than the vast majority of the other alternative models. Both the check/sluice structure and vortex tube alternatives would reduce maintenance in the canals, so a moderate site-specific benefit class was assigned to these alternatives. An extreme benefit classification was use for the gate

Table 41. Increased bank erosion potential scoring system.

Score Type	Parameter	Increase of Total Shear Stress Relative to Base Condition (psf)									
Conse-quence	Maximum Increase in Total Shear Stress	0 - 0.1	0.1 - 0.2	0.2 - 0.3	0.3 - 0.4	0.4 - 0.5	0.5 - 0.6	0.6 - 0.7	0.7 - 0.8	0.8 - 1	>1
	Net Score	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**

Table 42. Scoring system for additional restoration and site-specific benefits and consequences.

Score Type	Parameter	Qualitative Benefit or Consequence Class				
Benefits	Qualitative Description	No Benefit	Minor Benefit	Moderate Benefit	Major Benefit	Extreme Benefit
	Net Score	**0**	**2.5**	**5**	**7.5**	**10**
Conse-quences	Qualitative Description	No Conseq.	Minor Conseq.	Moderate Conseq.	Major Conseq.	Extreme Conseq.
	Net Score	**0**	**2.5**	**5**	**7.5**	**10**

automation alternative because installation of automated gate operators for Mesilla Dam Gates 5 and 9 would improve the lateral effectiveness of sluicing operations, and therefore reduce the amount of sediment maintenance required upstream from the dam.

Flood conveyance in the vicinity of Country Club Bridge (Problem Location 8) is of specific concern at Problem Location 8, so site-specific scores for the benefits and consequences were prepared to reflect the effect of the alternatives on stage at the upstream face of the bridge. The results presented in Table 32 were used to prepare these scores. Because the model results indicate that aggradation would occur to various degrees under the base and alternative conditions, the scores were based on the results from the localized models representing conditions immediately after implementation of the alternative and the models with the end-of-simulation geometry (i.e., the EOS models). These results indicate the stage would reduce under the sediment removal alternatives shortly after implementation, but would be higher than the base condition in the long-term (Table 32). To prepare the scores, the average change in stage indicated by the localized and EOS models were used, which indicate an average benefit would result from the sediment-removal alternatives, while the non-sediment-removal alternatives would have a net consequence. Because the change in average stage relative to the base condition is relatively small, low scores in the zero to two range were used. The benefit scores ranged from 0 for the localized excavation scenario (average stage reduction of less than 0.01 feet) to 2 for the long excavation scenario (average stage reduction of 0.1 feet), and the consequences ranged from 1 for the riprap alternative (average stage increase of less than 0.05 feet) to 2 for the spur dike alternative (average stage increase of 0.13 feet).

6.3 Alternative Rankings

The benefit and cost/consequence scores were used to identify the two alternatives that had the largest benefit relative to the costs and consequences. Initially, normalized net benefit and net cost/consequence scores were computed by summing the individual benefit and cost/ consequence scores and dividing by the number of rating parameter. The resulting benefit to cost/consequence ratio does not take into account alternatives that received very high scores for both the benefit and cost/consequence rating parameters. As such, the net scoring was revised to reflect the overall sum of the benefits and costs/consequences, and the ranking was assigned based on the difference between the benefits and costs/consequences (**Table 43**). At Problem Location 2, where the difference between the benefits and costs/consequences for the island/bar destabilization with vegetation removal alternative was the same as that for the long excavation alternative, the benefit to cost/consequence ratio was used to select the alternative with the higher rank. In general, the benefits of the sediment trap alternatives far outweigh the costs and consequences. This can also be said of the vast majority of the site specific alternatives. In contrast, the localized and short excavation scenarios typically have small differences between benefit and cost/ consequence, and in many cases the costs and consequences associated with the excavations exceed the expected benefit.

Results from the rankings were then used to identify the two alternatives that have the highest benefit relative to cost and consequence at each site. At all five locations where the sediment trap alternative was considered (Problem Locations 1 – 3, 5 and 7), this strategy had the number one ranking. At Problem Location 1 the modified Tierra Blanca Vortex Weir alternative had the second highest ranking. The long excavation and the island/bar destabilization and vegetation removal alternatives had identical benefit to cost/consequence differences, both of which had the second highest ranking at Problem Location 2. The benefit to cost ratios were computed and indicate the island/bar destabilization and vegetation removal would have the higher relative benefits, so this alternative was selected as the second rank at Problem Location 2. At Problem Location 3, the modifications to the Rincon Siphon was ranked second behind the sediment-trap

Table 43. Alternative scoring matrix table.

Problem Location	Alternative	Scoring Parameter: Benefits									Scoring Parameter: Costs and Consequences									Difference b/w Benefits and Costs/ Consequences	Rank
		Reduction in WSE	Ground-water Level Benefits	Levee Freeboard Benefits	Reduction in Aggradation/ Downstream Sediment Loading	Improved Irrigation Drain Return Flows	Durability	Additional Restoration Benefits	Additional Site Specific Benefits	Cumulative Benefit Score	Annualized Total Cost	Increase to WSE	Ground-water Level Consequences	Levee Freeboard Encroachments	Increased Aggradation/ Downstream Sediment Loading	Increased Bank Erosion Potential	Additional Restoration Consequences	Additional Site Specific Consequences	Cumulative Cost/ Consequences		
	Relative Weight of Parameter:	1	1	1	1	1	1	1	1		1	1	1	1	1	1	1	1			
	Range of Scores:	0-10	0-10	0-10	0-10	0-10	0-10	0-10	0-10	0-80	0-10	0-10	0-10	0-10	0-10	0-10	0-10	0-10	0-80		
1	Localized Excavation	5.5	2.0	0.0	2.0	0.0	10.0	0.0	0.0	19.5	1.4	0.4	0.0	0.0	1.4	10.0	2.5	0.0	15.7	3.8	4
	Short Excavation	5.0	2.3	0.0	1.6	0.0	6.0	0.0	0.0	14.9	2.1	0.3	0.0	0.0	1.4	6.0	2.5	0.0	12.3	2.6	5
	Long Excavation	6.8	3.0	0.0	2.4	0.0	3.0	0.0	0.0	15.2	3.4	0.2	0.0	0.0	2.0	3.0	2.5	0.0	11.1	4.1	3
	Vortex Weir	2.3	1.3	0.0	0.8	0.0	3.0	0.0	7.5	14.9	0.0	1.8	0.0	0.0	0.2	3.0	0.0	0.0	5.0	9.9	2
	Arroyo Sediment Traps	6.6	3.0	0.0	6.0	0.0	8.0	5.0	0.0	28.6	3.9	0.1	0.0	0.0	1.8	8.0	0.0	0.0	13.8	14.8	1
2	Localized Excavation	4.3	2.7	6.8	2.4	4.0	4.0	0.0	0.0	24.2	2.4	0.8	1.3	0.8	1.2	4.0	2.5	0.0	13.1	11.1	5
	Short Excavation	6.2	4.7	8.7	3.2	6.0	2.0	0.0	0.0	30.8	3.6	0.3	2.3	0.2	1.8	2.0	2.5	0.0	12.7	18.1	4
	Long Excavation	7.1	5.3	9.2	2.4	7.5	5.0	0.0	0.0	36.5	5.3	0.1	2.7	0.2	1.6	5.0	2.5	0.0	17.4	19.2	2
	Island Destabilization/Vegetation Removal	5.7	5.3	1.7	1.2	5.5	5.0	0.0	7.5	31.9	1.0	0.3	2.7	0.9	0.4	5.0	2.5	0.0	12.8	19.2	2
	Arroyo Sediment Traps	6.4	5.3	8.1	7.2	4.0	0.0	5.0	0.0	36.0	1.2	0.2	2.7	0.0	2.0	0.0	2.5	0.0	8.6	27.5	1
3	Localized Excavation	3.2	2.0	2.9	3.6	4.5	2.0	0.0	0.0	18.2	0.6	1.7	1.0	0.0	0.6	2.0	2.5	0.0	8.4	9.8	3
	Short Excavation	4.0	2.7	3.4	0.4	5.5	2.0	0.0	0.0	18.0	0.8	0.4	1.3	0.0	2.0	2.0	2.5	0.0	9.0	9.0	4
	Long Excavation	4.0	2.7	3.4	0.6	6.0	1.0	0.0	0.0	17.7	1.4	0.2	1.3	0.0	4.8	1.0	2.5	0.0	11.3	6.4	5
	Rincon Siphon Modifcations	8.9	6.7	4.9	4.8	9.0	10.0	2.5	0.0	46.8	1.3	1.0	3.3	0.0	1.4	10.0	7.5	10.0	34.6	12.2	2
	Arroyo Sediment Traps	1.3	0.7	1.6	4.8	0.5	0.0	5.0	0.0	13.9	0.1	0.8	0.3	0.0	0.2	0.0	0.0	0.0	1.5	12.4	1
4	Localized Excavation	3.5	2.7	4.6	0.4	4.0	0.0	0.0	0.0	15.2	5.2	0.0	0.0	0.0	0.8	0.0	2.5	0.0	8.5	6.6	3
	Short Excavation	3.5	2.7	5.5	0.2	4.5	5.0	0.0	0.0	21.4	9.0	0.1	0.0	0.2	1.6	5.0	2.5	0.0	18.4	3.0	5
	Long Excavation	6.4	5.3	7.4	0.8	8.5	2.0	0.0	0.0	30.4	9.0	0.0	0.0	0.0	2.8	2.0	2.5	0.0	16.3	14.2	1
	Island Destabilization/Vegetation Removal	2.1	0.0	4.0	0.4	0.0	1.0	0.0	7.5	15.0	1.3	0.9	1.3	0.0	0.8	1.0	2.5	0.0	7.8	7.2	2
	Spur Dikes	0.4	0.0	1.7	1.2	0.0	4.0	2.5	5.0	14.8	0.8	2.2	0.7	1.8	0.2	4.0	0.0	0.0	9.7	5.1	4
5	Localized Excavation	3.1	2.7	0.0	0.4	4.0	2.0	0.0	0.0	12.2	4.4	0.0	1.3	0.0	4.0	2.0	2.5	0.0	14.3	-2.1	5
	Short Excavation	4.1	3.3	0.0	0.4	5.0	1.0	0.0	0.0	13.8	4.1	0.0	1.7	0.0	3.0	1.0	2.5	0.0	12.3	1.5	4
	Long Excavation	5.8	5.3	0.0	1.2	8.0	2.0	0.0	0.0	22.3	3.7	0.0	2.7	0.0	6.8	2.0	2.5	0.0	17.6	4.7	2
	Spur Dikes	0.3	0.3	0.0	1.6	0.0	0.0	2.5	5.0	9.7	3.0	2.0	0.7	0.0	0.2	0.0	0.0	0.0	5.9	3.9	3
	Arroyo Sediment Traps	2.9	2.7	0.0	3.6	2.0	2.0	5.0	0.0	18.2	2.4	0.2	1.3	0.0	0.2	2.0	0.0	0.0	6.1	12.1	1
6	Short Excavation	0.5	0.7	0.0	0.6	4.0	0.0	0.0	0.0	5.8	10.0	2.0	0.3	1.5	3.6	0.0	2.5	0.0	19.9	-14.2	5
	Long Excavation	0.2	0.3	0.0	0.6	4.0	0.0	0.0	0.0	5.1	3.4	2.2	0.7	1.6	5.2	0.0	2.5	0.0	15.5	-10.4	4
	Sluiceway and Check Structures	0.0	0.0	0.0	9.2	0.0	0.0	0.0	5.0	14.2	2.1	0.0	0.0	0.0	1.0	0.0	0.0	0.0	3.1	11.1	2
	Gate Automation	0.0	0.0	0.0	0.0	6.0	0.0	0.0	10.0	16.0	2.2	0.0	0.0	0.0	0.0	0.0	0.0	0.0	2.2	13.8	1
	Vortex Tubes	0.0	0.0	0.0	2.5	0.0	0.0	0.0	5.0	7.5	0.3	0.0	0.0	0.0	3.0	0.0	0.0	0.0	3.3	4.2	3
7	Localized Excavation	0.1	0.7	0.0	0.4	1.0	7.0	0.0	0.0	9.2	2.0	1.5	1.3	2.8	1.4	7.0	2.5	0.0	18.5	-9.3	5
	Short Excavation	2.5	2.0	2.9	0.4	4.0	0.0	0.0	0.0	11.8	2.2	0.0	1.0	0.0	3.0	0.0	2.5	0.0	8.7	3.1	3
	Long Excavation	3.4	2.7	4.8	0.8	5.0	2.0	0.0	0.0	18.7	0.6	0.0	1.3	0.0	3.0	2.0	2.5	0.0	9.4	9.3	2
	Spur Dikes	0.0	1.7	0.0	2.8	0.0	1.0	2.5	5.0	13.0	0.9	3.1	3.3	6.0	0.2	1.0	0.0	0.0	14.6	-1.6	4
	Arroyo Sediment Traps	5.2	4.0	5.8	7.6	3.0	3.0	5.0	0.0	33.6	1.0	0.0	2.0	0.0	1.0	3.0	0.0	0.0	7.0	26.6	1
8	Localized Excavation	3.3	1.3	3.6	0.6	0.0	0.0	0.0	0.0	8.8	2.9	0.0	1.3	0.0	4.0	0.0	2.5	0.0	10.8	-1.9	5
	Short Excavation	3.3	1.3	4.2	0.8	0.0	0.0	0.0	1.0	10.6	3.4	0.0	1.3	0.0	5.2	0.0	2.5	0.0	12.4	-1.8	4
	Long Excavation	2.9	1.3	3.6	1.2	0.0	0.0	0.0	2.0	11.0	1.9	0.0	1.3	0.0	6.4	0.0	2.5	0.0	12.1	-1.1	3
	Riprap	0.2	0.3	0.2	0.8	0.0	0.0	0.0	10.0	11.5	0.3	0.8	0.3	2.8	0.2	0.0	0.0	1.0	5.5	6.1	1
	Spur Dikes	1.1	0.7	0.2	2.0	0.0	0.0	2.5	5.0	11.5	0.4	1.1	0.7	2.8	2.0	0.0	0.0	2.0	9.0	2.5	2
9	Localized Excavation	2.0	1.3	3.0	0.6	3.0	0.0	0.0	0.0	9.9	3.7	0.0	0.0	0.0	1.6	0.0	2.5	0.0	7.8	2.1	5
	Short Excavation	2.9	2.0	4.2	0.8	4.0	1.0	0.0	0.0	14.9	7.3	0.0	0.0	0.0	1.6	1.0	2.5	0.0	12.4	2.5	4
	Long Excavation	8.7	5.3	7.9	2.0	8.5	2.0	0.0	0.0	34.4	7.8	0.0	0.0	0.0	4.0	2.0	2.5	0.0	16.3	18.1	2
	Island Destabilization/Vegetation Removal	4.7	2.0	5.6	0.8	3.5	0.0	0.0	7.5	24.1	0.4	0.2	0.0	0.0	1.2	0.0	2.5	0.0	4.3	19.8	1
	Spur Dikes with Island Destabilization	1.1	0.7	1.7	0.4	0.5	0.0	2.5	5.0	11.9	0.3	0.3	0.0	1.6	0.8	0.0	2.5	0.0	5.5	6.4	3

*Scores for groundwater level parameters are not solely based on site-specific benefits and consequences. See text presented in Section 6.2.2 for explanation.

alternative. The long excavation alternative had the highest rank at Problem Location 4, followed by island/bar destabilization and vegetation removal. The long excavation alternative also ranked relatively high, but behind the sediment trap alternative, at Problem Locations 5 and 7. At Problem Location 6, installation of automated gate operators was ranked best, followed by the sluiceway and check structure alternative. The riprap alternative at Problem Location 8 had the highest ranking, followed by the spur dike alternative. The long excavation alternative received the best ranking at Problem Location 9, followed by the combined spur dike and island/bar destabilization and vegetation removal alternative.

7 SUMMARY AND RECOMMENDATIONS

This channel maintenance alternatives and sediment-transport study builds upon previously developed conceptual restoration plans (USACE, 2009) and river management plans (Parsons, 2004) to specifically address issues associated with sedimentation along the RGCP (sediment plugs, island formation, raising of the river bed, reduced irrigation drain efficiency, and increased threats to levee freeboard and flooding). These problems occur at the nine representative problem locations that are evaluated in this study. Results from the study provide a suite of alternatives to reduce or minimize the sediment issues at the 9 problem locations, and identify the most efficient, sustainable and environmentally beneficial methods. Once identified, the preferred alternatives can then be applied to other locations along the RGCP that have similar issues to the problem locations evaluated in this study.

In general, this study evaluated five channel maintenance alternatives (CMAs) at each of the nine problem locations. At each of the problem locations except for the Mesilla Dam site, three of the CMAs are classified as "sediment-removal alternatives" and include "short excavation", "long excavation" and "localized excavation" scenarios that involve excavation of sediments over varying distances and widths. At the Mesilla Dam problem location, the sediment removal alternatives only included the "short" and "long" scenarios. The remaining alternatives are classified as "Non-sediment Removal Alternatives" and vary by problem location.

A number of tasks were carried out as part of this assessment of CMAs for the RGCP that included a field assessment, targeted cross-section surveys, steady-state modeling of the overall RGCP, localized steady-state hydraulic modeling of the problem locations, sediment-transport modeling of the problem locations under existing (base) and with-CMA conditions. The results from the hydraulic and sediment-transport modeling were used to assess the expected benefits and consequences associated with the alternatives. Cost estimates, including construction costs and O&M costs, were also developed for each of the alternatives. This information was used to and prepare a benefit-cost/consequence analysis that served as a basis for ranking the alternatives to identify the two best CMAs at each location. The two alternatives that received the highest ranking and the net difference between the expected benefit scores and the costs/consequence scores are summarized in **Table 44**. These two highest ranked alternatives are based on the annualized costs presented in Table 34, the scoring of the assessment parameters outlined in Tables 35 through 42, and the net scores presented in Table 43, and are summarized as follows:

Problem Location 1: First Rank – Arroyo sediment traps.
Second Rank – Modified Tierra Blanca Vortex Weir.

Problem Location 2: First Rank – Arroyo sediment traps.
Second Rank – Island/bar destabilization and vegetation removal.

Problem Location 3: First Rank – Arroyo sediment traps.
Second Rank – Rincon Siphon Modifications

Problem Location 4: First Rank – Long excavation.
Second Rank – Island/bar destabilization and vegetation removal.

Problem Location 5: First Rank – Arroyo sediment traps.
Second Rank – Long excavation.

Problem Location 6: First Rank – Automated gate operators at Mesilla Dam.

Second Rank – Check/Sluiceway structures in Eastside and Westside Main Canals.

Problem Location 7: First Rank – Arroyo sediment traps.

Second Rank – Long excavation.

Problem Location 8: First Rank – Riprap bank protection for west levee.

Second Rank – Spur dikes.

Problem Location 9: First Rank – Island/bar destabilization and vegetation removal.

Second Rank – Long excavation.

Table 44. Summary of the two alternatives that received the highest ranking at each of the problem locations evaluated in this study, and the difference between the net benefit and net cost/consequence scores. Also shown are the estimated total annualized costs.

Problem Location	Alternative	Difference b/w Benefits and Costs/ Consequences	Total Annualized Cost	Rank
1	Arroyo Sediment Traps	14.8	$285,000	1
	Vortex Weir	9.9	$4,100	2
2	Arroyo Sediment Traps	27.5	$90,600	1
	Island Destabilization/Vegetation Removal	19.2	$77,000	2
3	Arroyo Sediment Traps	12.4	$14,100	1
	Rincon Siphon Modifications	12.2	$100,400	2
4	Long Excavation	14.2	$653,500	1
	Island Destabilization/Vegetation Removal	7.2	$97,500	2
5	Arroyo Sediment Traps	12.1	$175,900	1
	Long Excavation	4.7	$269,500	2
6	Gate Automation	13.8	$164,200	1
	Sluiceway and Check Structures	11.1	$154,800	2
7	Arroyo Sediment Traps	26.6	$77,500	1
	Long Excavation	9.3	$164,800	2
8	Riprap	6.1	$28,300	1
	Spur Dikes	2.5	$34,200	2
9	Island Destabilization/Vegetation Removal	19.8	$32,300	1
	Long Excavation	18.1	$534,700	2

All of the top two alternatives would not be expected to result in channel plugging or perched channel conditions. Only the spur dike alternative could result in vegetation encroachment or channel narrowing if the sedimentation that occurs between the spurs is not maintained. The long excavation alternatives could result in increased bank heights if the excavated channel is located along the banks, so properly designed excavations would be required if increased bank heights are not desirable. All of the top two alternatives that result in a reduction to sediment loading could result in bed degradation, but the model results indicate this degradation would be localized and not substantial.

Based on the findings of this study, a number of recommendations were identified for further evaluation, for incorporation into adaptive management practices, or for improving channel maintenance along the RGCP. These recommendations include:

1. Any of the recommended alternatives that are ultimately selected for implementation should be monitored locally as part of the adaptive management approach. It is recommended that this monitoring involves the establishment of monumented cross sections that are surveyed prior to and immediately after implementation to establish a base condition. Repeat surveys through time would be beneficial for evaluating channel response. These monitoring sections should also be included in the arroyos when applicable.
2. Because the sediment trap alternatives provide the highest benefit relative to costs and consequences, this alternative should be considered at all problem locations, and elsewhere along the RGCP, where tributaries deliver coarse sediment loads to the river. It is recommended that, as part of the adaptive management approach, at least one sediment trap be constructed as laid out conceptually in this report for purposes of testing the trap efficiencies. During the testing of the sediment traps, if it is determined that it is desirable to also eliminate the finer fractions of the tributary bed material sediment supply, the trap screens could be re-designed to also trap the sand, silt, and clay classes. However, because the fine (sand, silt and clay) sediment loads delivered by the tributaries represents a relatively small portion of the overall fine sediment loads supplied and transported by the RGCP, this may not be worthwhile.
3. The tributary loading was based on a relatively simple approach of targeting a high, but realistic sediment concentration. Because tributary loads are highly variable it is recommended that arroyo sediment traps be monitored annually and enlarged if sediment removal is required too frequently.
4. Any opportunity to construct sediment traps that are larger than those evaluated in this study, which are limited in size due to the ROW constraint, should be considered and evaluated in detail. These opportunities should include the potential for construction of sedimentation basins in the upstream portions of the arroyo watershed.
5. Localized and short excavation scenarios do not appear to provide good value due to the high frequency of the excavations that would be necessary for maintenance purposes, therefore, it is recommended that any excavations be conducted over reaches that are as long as possible. Based on the long excavation alternatives evaluated in this study, a minimum length of 3,900 feet should be targeted for the excavations.
6. During the field reconnaissance of Problem Location 5, a beaver dam was identified at the mouth of the Rincon/Tonuco Drain. The dam appears to have an effect on drain efficiency and probably results in increased groundwater levels, at least during the non-irrigation season, so it is recommended that the dam be removed and the beavers be relocated. It is also recommended that beaver activity be monitored at other drains along the RGCP to ensure beaver dams are not affecting drain performance or local groundwater levels.

7. Because the sedimentation issues result in varying degrees of problems among the problem locations, it is recommended that the problem locations be prioritized during development of the implementation plan. The island/bar destabilization and vegetation removal alternative was evaluated for this study under the assumption that the full set of islands and bars that are selected for treatment be cleared and grubbed in concert. In practice, this work can be prioritized such that the largest islands and bars that have the most significant hydraulic effect receive the highest priority. Similar prioritization of the other alternatives (e.g. identification of the most problematic tributary sediment loadings for installation of the arroyo sediment traps), is also recommended prior to implementation.
8. The non-sediment removal alternatives in the vicinity of Mesilla Dam at Problem Location 6 would be affected by 2-D and 3-D flow and sediment patterns that are not taken into account in the 1-D HEC-RAS modeling conducted for this study. If it is desirable to provide more certainty to the scoring and ranking of the alternatives, the automated gate operator and check/sluiceway structure alternatives that were identified as having the highest benefit to cost/consequence difference should be evaluated further using a 2-D model platform or a physical model. The cost for conducting 2-D modeling of these alternatives is estimated to be less than 5 percent of the estimated cost of construction for each of the alternatives, while the cost for a physical model would probably be about 10 percent of the estimated cost of construction.
9. The proposed check structure and sluiceway alternative at Problem Location 6 will require relocation of the heading of the Del Rio Lateral Canal. Locating the new heading upstream from the Eastside Main Canal first check structure could result in undesirable sediment loading to the lateral if the heading were located upstream from the sluiceway. As such, the lateral canal heading should be located downstream from the sluiceway and upstream from the first check structure.
10. Many of the individual alternatives could be combined to enhance the expected relative benefits. For example, implementation of the modified Tierra Blanca Vortex Weir in addition to the sediment trap alternative may result in significant benefits for very little additional cost. Similarly, a sediment trap combined with localized excavation could produce immediate and long-term benefits. Combining of alternatives should be evaluated using similar methods to those used in this study, at a minimum.
11. The proposed modifications at the Rincon Siphon would include removal of the grade control structure below the current siphon. If this alternative were implemented, downcutting would probably occur along a significant portion of the upstream reach that could affect the foundations of the NM 154 and ATSF Railroad Bridges. It is therefore recommended that the as-built information for both bridges be reviewed to determine the depth to which the piers are buried. If it is determined that the piers are not sufficiently embedded below the channel invert elevation plus scour relative to the downstream limit of the grade control structure, the bridge foundations could be at risk to undermining. As such, it would be necessary to reconstruct the pier footings, which could result in a significant increase to the cost for this alternative that would reduce the difference between benefit and cost/consequence and potentially change the alternative ranking. All of this should be heavily considered in more detailed evaluations of this alternative.
12. Although the model validation simulations represent the no-action scenario under normal flow conditions (WY2005 to WY2014), the alternative evaluation presented in this study is based

on extreme drought condition hydrology (WY2013)[10]. It is recommended that the alternative evaluation also be conducted for normal hydrology to determine whether or not the scoring and ranking of the alternatives would change under a different flow regime. This exercise would be relatively simple because the tools that were developed for this study, primarily the sediment-transport models and alternative scoring systems, are in place.

13. The preliminary cost estimates prepared for this study may not reflect the actual construction and O&M costs, so a detailed cost analysis of the top-ranking alternatives should be carried out to better compare the two alternatives recommended at each problem location.
14. The levee freeboard analysis presented in this study uses the elevations of the levees as indicated by the 2011 LiDAR topography. It is recommended that in areas where activities are planned to improve levee freeboard conditions, the top of the levees be surveyed to ensure that the LiDAR based levee elevations and the associated analysis presented herein are accurate. The surveys of the levees need not be extensive, but should rather include “spot” elevations at selected model cross sections for purposes of validating the 2011 LiDAR-based levee top elevations that are reflected in the localized hydraulic modeling prepared for this study.
15. Although this study was not intended to include a detailed evaluation of the habitat benefits and consequences, the Tetra Tech team of engineers and geomorphologists understands the importance of these considerations, so the parameters were included in the scoring matrix. The scoring of the habitat benefit and consequence parameters presented in this report is somewhat subjective, so it is recommended that these parameters be re-evaluated in a separate study by an entity with appropriate expertise in riparian and aquatic ecology.
16. Each of the above recommendations should be considered as a task under a 5-year adaptive management plan, except recommendation number 8 if deemed unnecessary. In addition, it is recommended that each type of the generalized top-ranked alternatives be implemented at one of the problem locations, at a minimum, for purposes of testing under this 5-year plan. These generalized top-ranked alternatives include arroyo sediment traps, island/bar destabilization with vegetation removal and long excavation. It is also recommended that the two site specific alternatives that received the highest rank, including the installation of additional automated gate operators at Mesilla Dam (Problem Location 6, after the recommended further evaluation) and installation of riprap revetment below Country Club Bridge (Problem Location 8), be implemented as part of this 5-year plan because these alternatives appear to achieve the desired benefits with relatively low cost and consequence. As discussed in recommendation number 7, the 5-year plan should also prioritize the problem locations, and the details associated with the final design of the alternatives should be prioritized as part of the implementation plan.

[10]Although the models were validated using the estimated actual hydrological conditions over the period from WY2005 to WY2014, the analysis and scoring of the alternatives was based on a 10-year simulation of the WY2013 irrigation release (repeated 10 times). As such, the analysis presented herein includes two separate no-action model runs, including: (1) a scenario under which the actual hydrologic conditions that occurred from WY2005 to WY2014, which was represented by the model validation simulations, and (2) a scenario under which extreme drought conditions occur over an extended (10-year) period, which was represented by the base model simulations that were used as the basis for the alternative evaluations.

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APPENDIX A

Statement of Work and Statement of Work Addendum for Channel Maintenance Alternatives and Sediment-transport Studies for the Rio Grande Canalization Project

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Appendix B

Field Assessment Report

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Appendix C

Del Sur Surveying Surveyor’s Report

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Appendix D

Mapping Showing the Survey Point Data

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Stationed Cross-section Data and Plots

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Water-surface Profile Comparisons from the Base Model and Updated Base Model of the RGCP

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Comparative Thalweg Profile and Cross-section Plots for the Sediment Removal Alternatives

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Appendix G.2

Mapping Showing Extents of the Sediment Removal Alternatives

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Conceptual Layouts for Arroyo Sediment Traps

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Appendix I

Conceptual Layouts for Low-elevation Spur Dikes

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Mapping Showing Extents of Island Destabilization and Vegetation Removal Treatments

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EBID River Sediment Management Alternatives Report Excerpts

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Appendix L

Comparative Water-surface Elevation Profile Plots for Modeled Alternatives and Predicted Change from Baseline Conditions

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Appendix M

Sediment-transport Model Results

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Long-term Water-surface Elevation Profiles

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Levee Freeboard Encroachment Profiles

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Cost Estimates

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Appendix Q

Parameter Scoring Development for the Alternatives

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Digital Data Disc

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