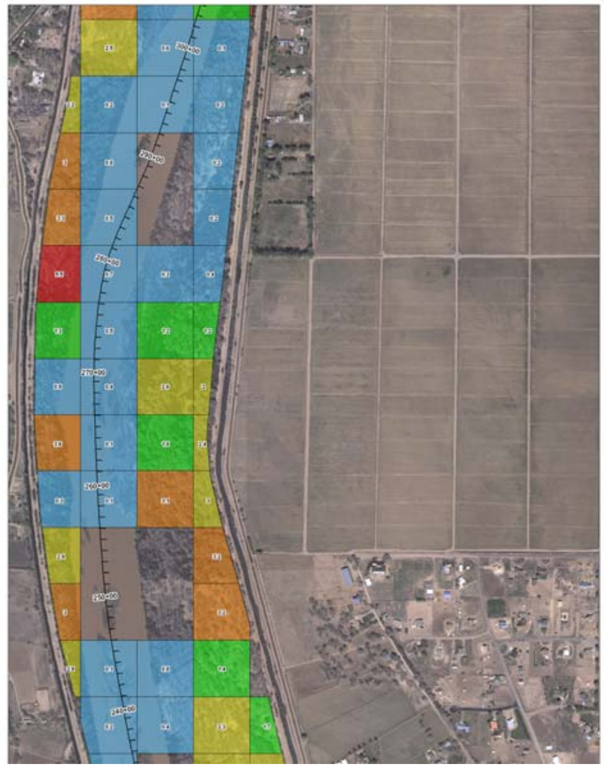
FLO-2D Model Development—Existing Conditions and Restoration Alternatives 1 to 5 Albuquerque Reach, New Mexico



Submitted to: **U.S. Army Corps of Engineers**
Albuquerque District
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MEI Project 05-03

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1. INTRODUCTION

Mussetter Engineering, Inc. (MEI) was retained by the U.S. Army Corps of Engineers (USACE) (Contract DACW47-02-D-005, Delivery Order 0006) to perform FLO-2D modeling to support a planning study of the Albuquerque Reach of the Rio Grande, which extends from Corrales Siphon to the northern boundary of the Pueblo of Isleta (**Figure 1.1**). The objective of the planning study is to increase river channel and bosque overbank connectivity, produce enhanced cover and aquatic habitat diversity, restore healthy riparian function to enhance natural riverine processes, improve terrestrial wildlife habitat, and protect existing structural features such as pipelines, bridges and levees with a preference toward using bank biostabilization techniques when structures are found to be at risk from natural geomorphic processes (USACE, 2004). The FLO-2D modeling provides an assessment of overbank flows, storage, and hydraulic data to facilitate analysis of sediment-transport conditions and geomorphic processes along the reach, results from which will be used to evaluate various restoration alternatives.

A previous report summarized development of the four hydrologic scenarios, development, verification and application of the 500-foot grid FLO-2D model, and the baseline channel stability analysis (MEI, 2006). A subsequent report summarized the development of a new, higher resolution FLO-2D model (250-foot grid), re-analysis of the extent, depth and duration of overbank inundation for existing conditions for each hydrologic scenario, and analysis of the initial restoration alternative developed by the USACE (MEI, 2008a).

This report provides a comprehensive final project report that combines the previous two reports, as well as the analysis of the Restoration Alternatives 2 through 5. In addition, this report includes an evaluation of the effect of sedimentation within the project area under existing conditions and "end-of life" (Year 50) project conditions for the five restoration alternatives.

Mr. Steve Boberg, P.E. was the project manager for the USACE, Dr. Bob Mussetter, P.E., was MEI's Principal Project Manager, and Mr. Dai Thomas, P.E. (CO) was the project engineer.

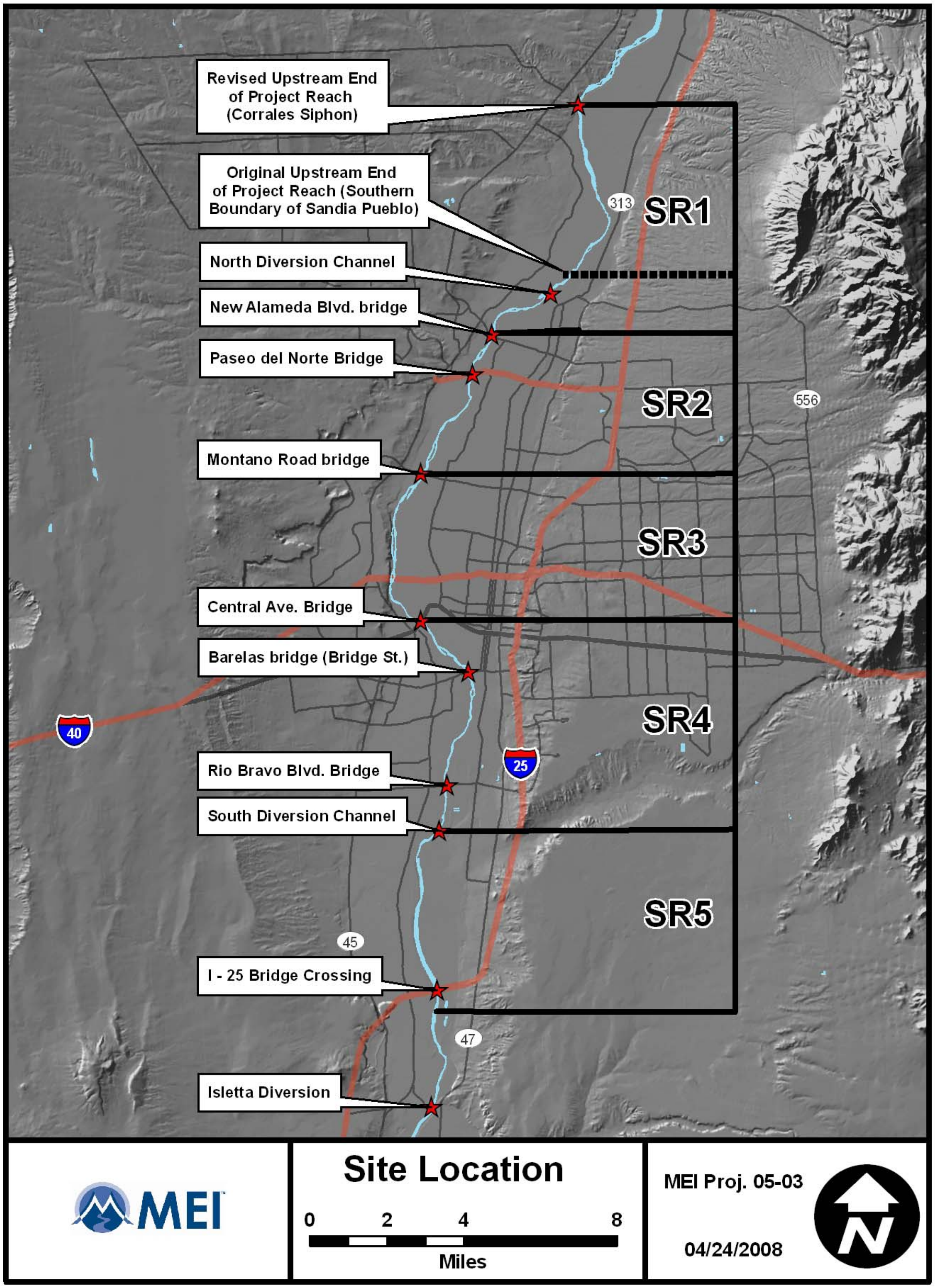

Figure 1.1. Location map showing the project reach and subreach boundaries.

2. REVIEW OF EXISTING STUDIES AND BACKGROUND MATERIAL

In performing this study, MEI reviewed available historic reports and information that were provided by the USACE or that were obtained directly by MEI. Information from previous studies within the reach was also considered. Specific, relevant documents that were considered included the following:

1. The FLO-2D model that extends from Cochiti Dam to Elephant Butte Reservoir that was previously developed by the USACE to support the Upper Rio Grande Water Operations Planning Study (URGWOPS) (Tetra Tech, 2004)
2. A revised FLO-2D model of the reach from Cochiti Dam to Elephant Butte Reservoir that was previously developed by Riada Engineering, Inc. for the USACE (Riada Engineering, Inc, 2008)
3. An existing HEC-RAS model of the project reach that is currently being refined by the USACE.
4. Data from a high-flow monitoring project that was conducted by the USACE and Tetra Tech in May 2005, when peak discharges in the study area reached approximately 6,300cfs. These data included water-surface elevations and field mapping of overbank inundation on May 24 and 25, 2005, near the peak flow.
5. High-water marks surveyed in June and July 2005 by Steve Boberg (USACE) at Old Alameda Boulevard, Central Avenue, Bridge Boulevard, and Rio Bravo Boulevard Bridges.
6. Aerial photography and satellite imagery of the project reach that shows the extent of the June 2003 wildfires.
7. Bosque Wildfires plans that detail the burn restoration and fuel reduction areas (USACE, March 2005).
8. Existing geomorphic, sedimentologic and sediment-continuity reports prepared by MEI (Mussetter and Harvey, 1993; MEI, 2002, 2003, 2004).

3. HYDROLOGY

The scope of work for this project specifies that the following four hydrologic events were to be modeled in evaluating baseline conditions and the five identified restoration alternatives:

1. The active channel-full flow of ±5,000 cfs,
2. A representative post-Cochiti annual spring runoff hydrograph (peak flow ±3,000 cfs),
3. A 10,000-cfs post-Cochiti flow hydrograph, and
4. The 100-year post-Cochiti flood-flow hydrograph.

Hydrology Scenario 1 was modeled as a steady-state flow. Hydrology Scenarios 2 and 3 were developed in consultation with the USACE from an analysis of the flow records at the Rio Grande at Albuquerque gage (USGS Gage No. 08330000) for the post-Cochiti Dam period [Water Year (WY) 1974 to WY2002]. Hydrology Scenario 4 was developed by the Hydrologic Engineering Center (HEC, 2006) based on analysis of the Rio Grande flood hydrology.

The Albuquerque gage (also known as the Central Avenue Gage), is located immediately upstream of the Central Avenue Bridge and 48.6 miles downstream from Cochiti Dam and has a contributing drainage area of 14,500 mi^2 (total drainage area is 17,440 mi^2). The Rio Grande below Cochiti Dam gage (USGS Gage No. 08317400) has a drainage area of 14,900 mi^2; thus, the contributing drainage area between Cochiti Dam and the Albuquerque gage is 400 mi^2.

To place the various flow scenarios into the context of the existing and historic hydrology of the project reach, MEI performed a general analysis of flow records at the Albuquerque gage. Upstream reservoirs and water diversion projects have significantly altered the hydrology of the project reach compared to the pre-Cochiti Dam hydrology. Peak flows at the Albuquerque gage regularly exceeded 10,000 cfs prior to construction of Cochiti Dam in 1974, but they have not exceeded that level since its completion. In spite of the effects on the peak flow regime, the annual runoff increased substantially between the two periods, from an average of about 714,000 ac-ft during the period from 1943 through 1974 to about 1,011,000 ac-ft between 1975 and 2002 (MEI, 2002). The median flow during the post-Cochiti period was about 850 cfs, and flows exceeded 320 cfs about 90 percent of the time and 3,350 cfs about 10 percent of the time (**Figure 3.1**). A Log-Pearson Type III flood-frequency analysis of the post-Cochiti Dam peak flows (1974-2004) at the Albuquerque gage that was performed using the USACE HEC-FFA computer program (USACE, 1992) indicates that the magnitude of the 2-, 5-, and 100-year floods are 5,630, 7,520, and 13,300 cfs, respectively (**Figure 3.2**). The magnitudes of other recurrence interval peak discharges are also summarized in Figure 3.2.

Based on field observations during the 2005 runoff season, the active channel-full flow in this reach is about 6,000 cfs, somewhat higher than the ±5,000 cfs that was originally specified in the scope of work. The discharge for Hydrology Scenario 1 was, therefore, increased to 6,000 cfs. Hydrology Scenario 1 was modeled as a steady-state flow, because the primary purpose is to evaluate the extent and location of overbank flooding that would occur under a sustained discharge at this level. This discharge has a peak flow recurrence interval (RI) of about 2.3 years, and mean daily flow exceedence probability of 1.2 percent (i.e., it occurs 4 to 5 days per year, on average).

A representative post-Cochiti annual spring runoff hydrograph was developed for evaluating the various restoration alternatives. To develop the hydrograph, the mean daily flows for each of the 29 annual hydrographs were initially plotted and compared. Because the individual hydrographs peak at different times each year, the timing was adjusted by centering the hydrographs

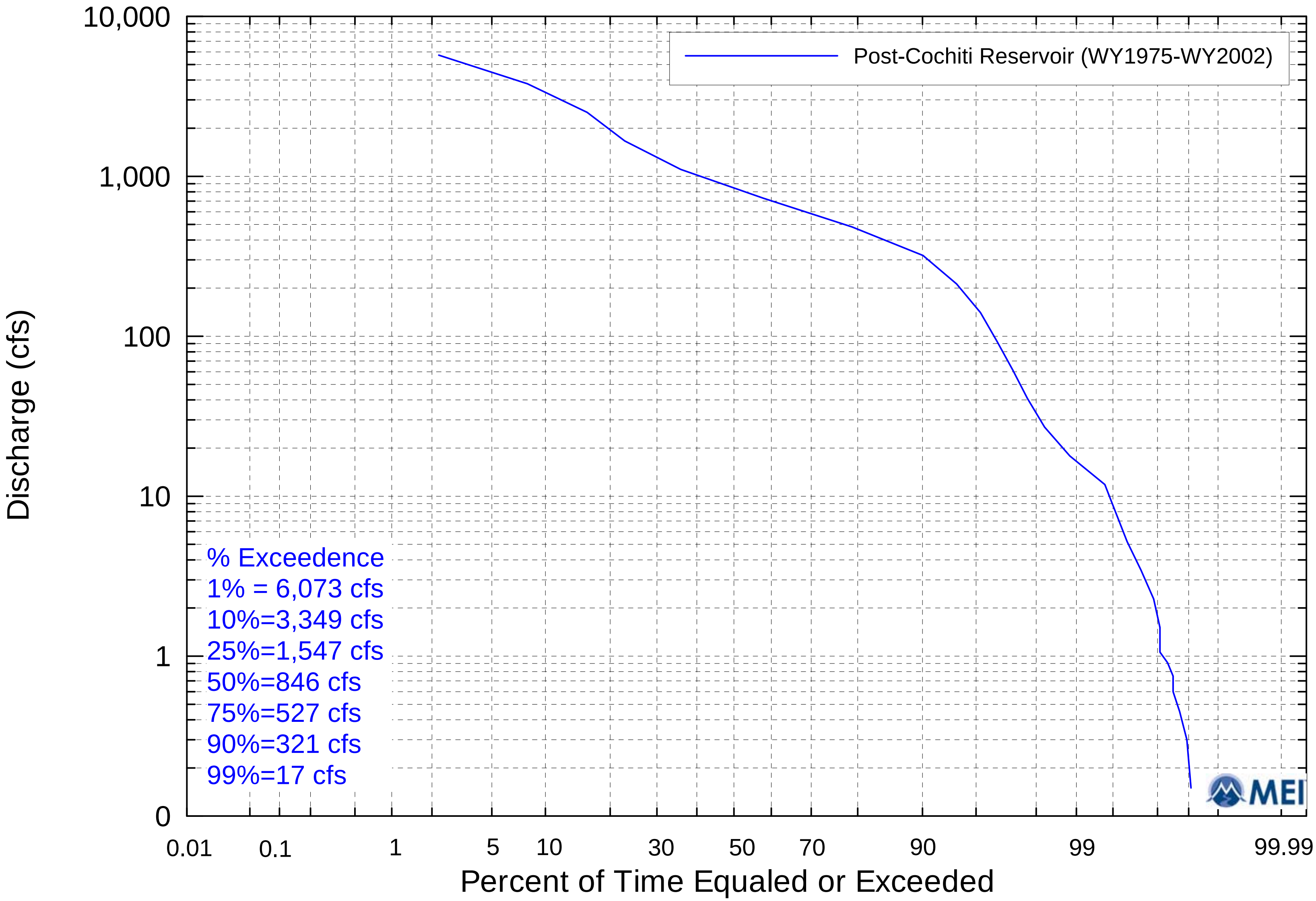

Figure 3.1. Post-Cochiti Reservoir (1974-2002) mean-daily flow-duration curve for the Rio Grande at Albuquerque.

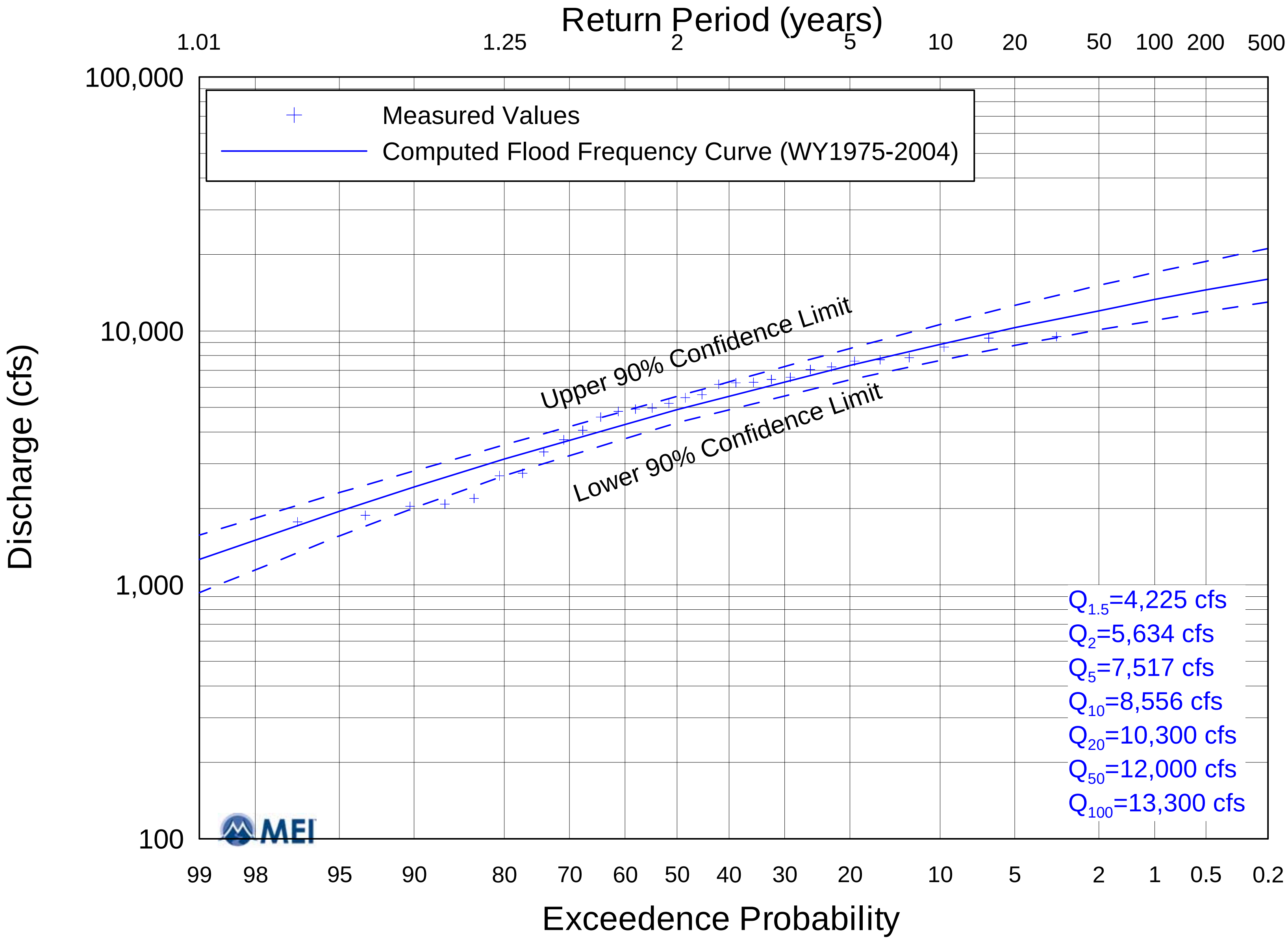

Figure 3.2. Post-Cochiti Reservoir (1974-2004) recorded peak flows and computed flood-frequency curves for Rio Grande at Albuquerque.

so that the rising and falling limbs match as closely as possible to prevent over estimating the hydrograph volume, particularly on the rising and falling limbs. A 50-percent exceedence hydrograph was then estimated based on the shifted hydrographs, yielding a peak discharge of 3,770 cfs. The log-Pearson III frequency analysis of the annual peak flows indicates that the maximum mean daily flow of 3,770 cfs corresponds to a recurrence interval of about 1.4 years, and this flow has an exceedence probability of 8.1 percent on the mean daily flow-duration curve (i.e., it occurs about 30 days per year, on average).

Although hydrographs were translated to match as closely as possible, the resulting hydrograph still appeared to overestimate the volume of flow due to the individual shape and duration characteristics of each hydrograph. To account for this issue, and to obtain a hydrograph with a shape that is representative of the individual yearly hydrographs, a 15-point moving average was applied to smooth out irregularities, and the duration of the hydrograph was scaled to maintain the target run-off volume of 590,190 ac-ft that was determined from a regression relationship between the maximum mean daily flow and the hydrograph volume during the runoff period that typically occurs sometime between the 120th and 340th day of the water year (January 28 through September 5) (**Figure 3.3**). To achieve the target volume, the ordinates of the hydrograph were determined by adjusting the duration and shape until the target volume was achieved while still maintaining a peak discharge of 3,770 cfs. Comparison of the resulting hydrograph with five measured hydrographs that had similar peak discharges indicates that the shape, including the slope of the rising and falling limbs, approximates that of the measured hydrographs reasonably well (**Figure 3.4**).

The 10,000-cfs hydrograph (Hydrology Scenario 3) was developed by scaling the ordinates of the 10-percent exceedence hydrograph to provide a peak discharge of 10,000 cfs, and then adjusting the duration to achieve the target volume of 1,467,000 ac-ft that was determined by extrapolating the best-fit curve in Figure 3.3 to 10,000 cfs (**Figure 3.5**). In developing the 50-percent exceedence hydrograph, the peak discharge was contained within the range of discharges and no scaling of the peak discharge was required. However, since the peak discharge of 10,000 cfs has not occurred during the post-dam period, the hydrograph for this peak scenario was estimated by scaling the 10-percent exceedence hydrograph, rather than the 50-percent hydrograph, because it provides a more realistic shape for the larger hydrographs. The resulting hydrograph is shown in **Figure 3.6**, along with the five largest recorded hydrographs. Although the maximum flows exceed all of the recorded flows, the overall hydrograph duration and slope of the rising and falling limbs are reasonably representative of the recorded high-flow hydrographs.

Analysis of the Rio Grande flood hydrology by the Hydrologic Engineering Center (HEC, 2006) indicated that the 100-year snow melt hydrograph (Scenario 4) has a peak discharge of approximately 7,750 cfs (**Figure 3.7**). This snowmelt hydrograph was developed by routing actual hydrographs from time-series analysis of unregulated flows through the upstream reservoirs using the ResSim model, and then routing the resulting outflow hydrographs from Cochiti Reservoir downstream through the project reach using the FLO-2D model. The snowmelt hydrograph has a duration of approximately 17 weeks, and is regulated by Cochiti Dam at a relatively constant flow of about 7,000 cfs over most of the period. The hydrograph showing the effects of upstream regulation is shown in Figure 3.7.

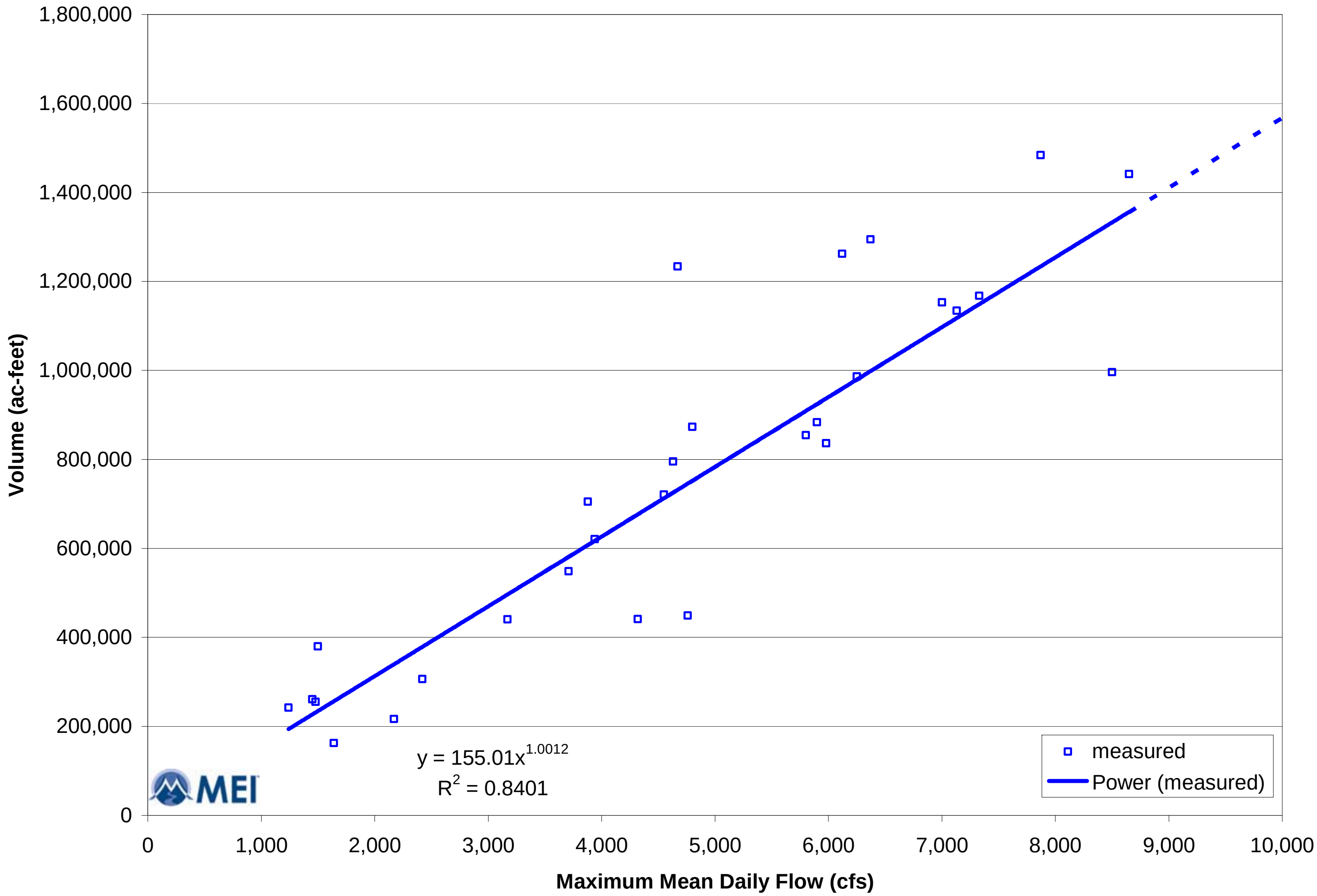

Figure 3.3. Comparison of maximum annual mean daily flow values versus computed volumes during the runoff period for WY1974 to WY2002. The curve is extrapolated to 10,000 cfs using a power function.

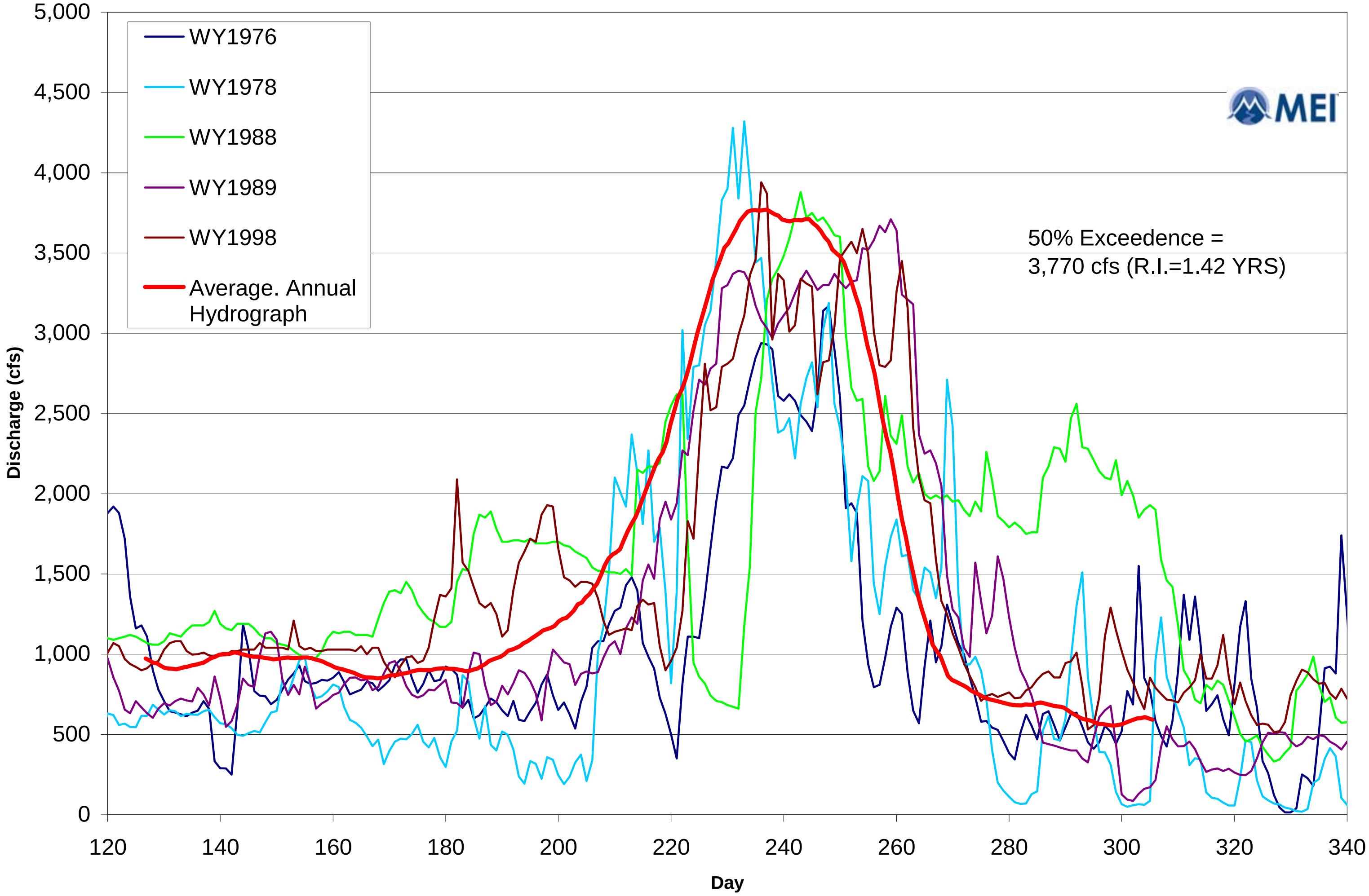

Figure 3.4. The representative 50-percent exceedence hydrograph and a comparison with five natural hydrographs with similar peak discharges.

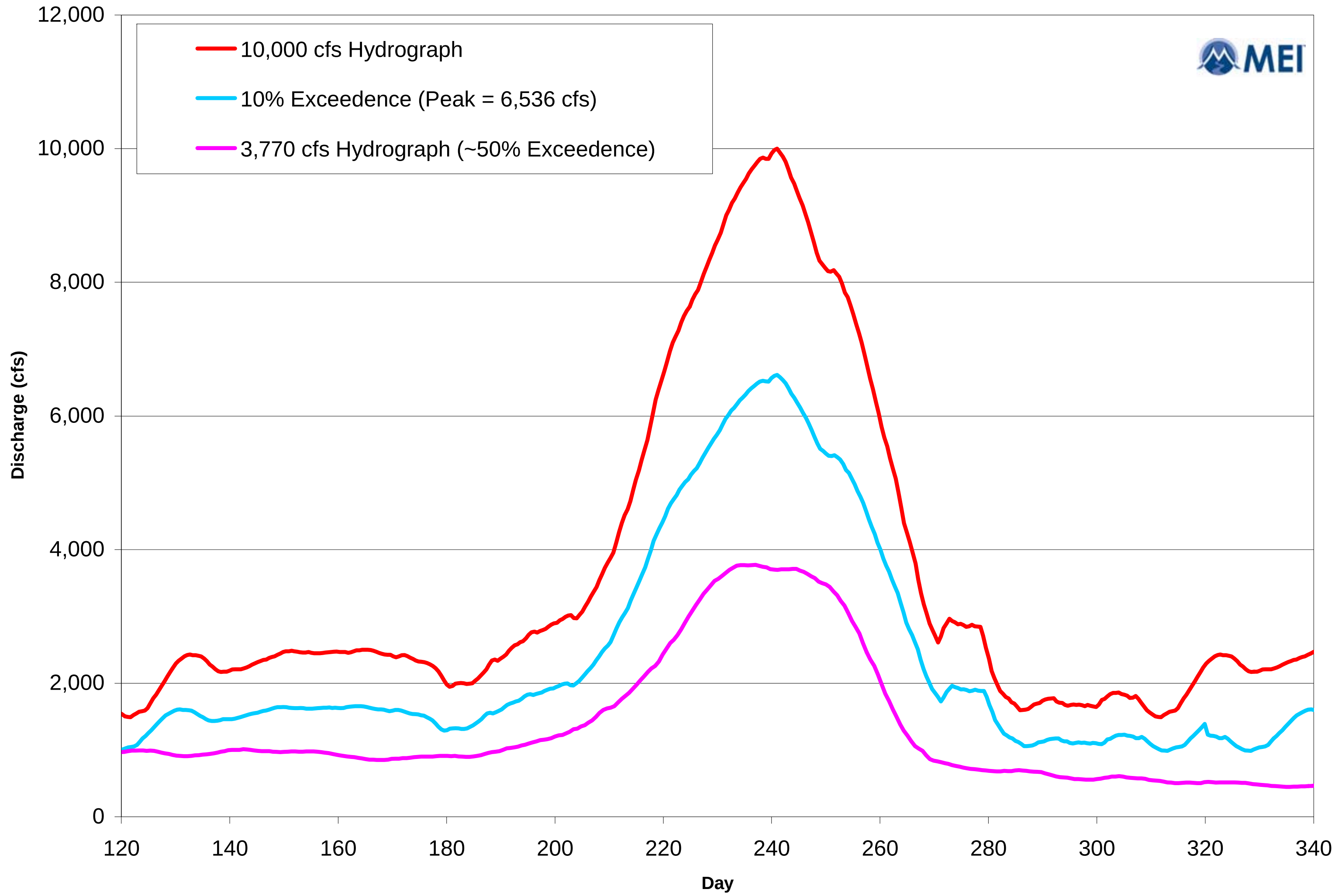

Figure 3.5. Comparison of the 10,000-cfs hydrograph with the 10- and 50-percent exceedence hydrographs.

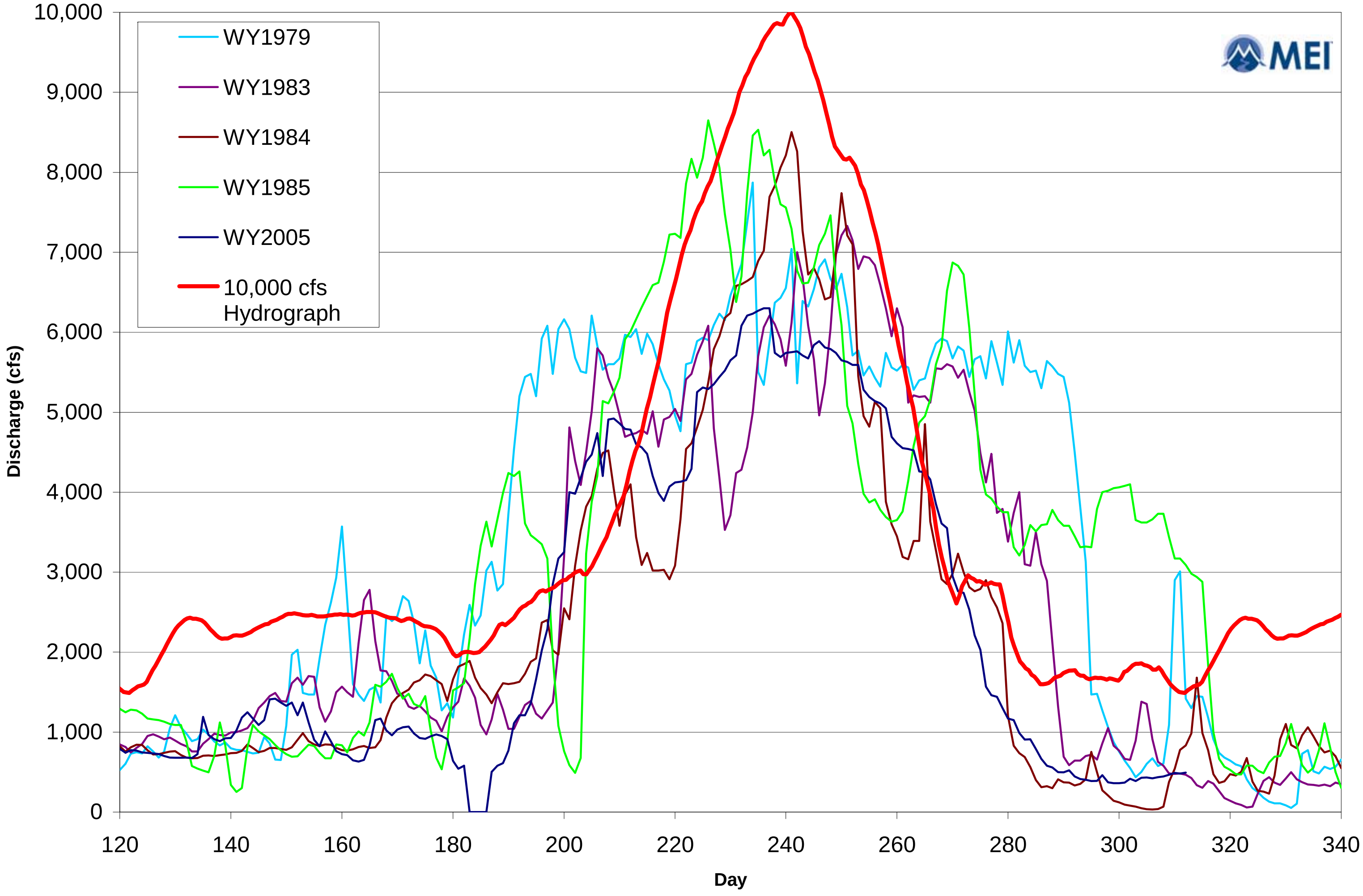

Figure 3.6. Comparison of the 10,000-cfs hydrograph with five largest recorded hydrographs for the post-Cochiti Dam period.

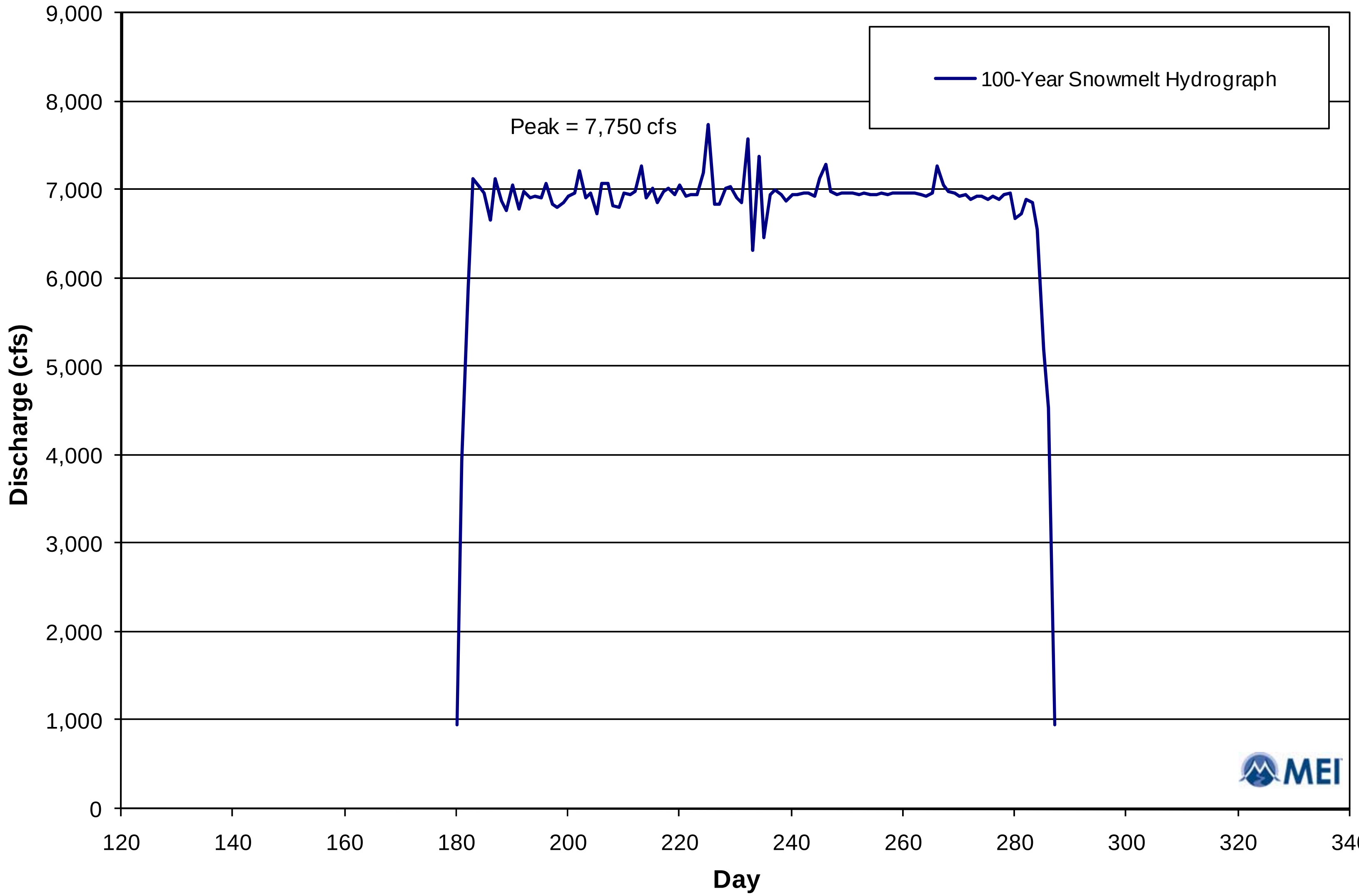

Figure 3.7. The representative 100-year snowmelt hydrograph.

4. MODEL DEVELOPMENT

Two FLO-2D models of the reach from Cochiti Dam to Elephant Butte Reservoir were used in this project. The first FLO-2D model was developed by Tetra Tech Inc, (2004) and has a grid resolution of 500 feet with over 36,000 elements. The hydraulic output from the 500-foot grid model was used to perform the long-term channel stability analysis and to perform the initial existing conditions evaluation under Hydrology Scenarios 1, 2 and 3. Hydrology Scenario 4 was not modeled with the 500-foot grid because the re-evaluation of the 100-year hydrology had not been completed by the USACE at that time.

The second FLO-2D model was subsequently developed by Riada Engineering, Inc. and MEI (2008) for the USACE to update the initial URGWOPS model with a grid resolution of 250 feet. This model contains over 167,000 elements. Results from the existing conditions models were used to provide baseline conditions for comparison with results for the restoration alternatives. As a result, the USACE requested that existing conditions be re-evaluated using the 250-foot grid model over the extended reach for all four hydrology scenarios (**Table 4.1**).

Hydrology Scenario 3, the 10,000 cfs high flow hydrograph was only modeled for the purpose of determining the effect of a high flow release through the project area under existing conditions. Restoration Alternatives 1, 2, and 3 were simulated using Hydrology Scenarios 1, 2, and 4.

Table 4.1. Summary of Hydrologic scenarios.

Hydrologic Scenario	Description	Peak Discharge (cfs)
1	Active channel-full flow	6,000
2	Post-Cochiti annual spring hydrograph	3,770
3	10,000 cfs post-Cochiti hydrograph	10,000
4	100-year post-Cochiti hydrograph	7,750

The 250-foot grid Existing Conditions model was modified to represent each of the restoration alternatives by making appropriate adjustments to the main channel cross-section geometry, overbank grid elevations, and roughness parameters. Alternative 1, which is referred to as the "Maximum Effort" alternative, contains the channel and overbank features that were considered in formulating the restoration plan. Alternatives 2, 3, 4 and 5 are termed the "Minimum Effort", "Moderate Effort", "Moderate Effort-A" and "Moderate Effort-B", respectively, and they were developed using various combinations of the channel and overbank features that were included in Alternative 1. Alternatives 4 and 5 were modeled only for the target restoration flows (Hydrology Scenarios 1 and 2).

The alternative evaluations presented in the remainder of this report are focused on the extent and duration of overbank inundation predicted by the updated 250-foot FLO-2D model. The in-channel hydraulic results from the 250- and 500-foot grid models are very similar throughout the project reach. As a result, the channel stability analysis was not re-evaluated using the 250-foot grid.

The original upstream boundary of the project reach for the 500-foot grid model was located at the southern boundary of the Pueblo of Sandia. After completion of the initial report (MEI, 2005), the USACE requested that the upstream end of the project reach be moved approximately 5.3

miles upstream to the Corrales Siphon (opposite the Rio Rancho Wastewater Treatment Plant) to encompass all of the potential restoration alternatives (Figure 1.1).

To facilitate development of the model, interpretation of the model results, and to assign locations of other GIS data, MEI developed a station line that represents the distance along the approximate centroid of the flow, with the downstream end (Sta 0) located at the Isleta Diversion Dam. Along this station line, the downstream end of the modeled reach for this project is located at Sta 16,050 and the upstream end of the reach is at Sta 159,200. **Table 4.2** lists the MEI stationing and Bureau of Reclamation (BOR, 2003) river miles for key points of interest along the station line.

4.1. 500-foot FLO-2D Grid Model

The existing conditions 500-ft grid model was developed by updating the original URGWOPS FLO-2D model. In developing the original model, Tetra Tech used the Grid Developer System (GDS), which is part of the FLO-2D program, to assign elevations to the FLO-2D grid using a series of digital terrain models (DTMs) that are based on aerial photogrammetry and LIDAR data collected during the 1990s and early 2000s by the Albuquerque District. All horizontal coordinates in the model were specified using the New Mexico Central State Plane (NAD83) coordinate system, and elevations were specified in the North American Vertical Datum (NAVD) of 1988. Cross sections assigned to the main channel grid elements were developed from rangeline cross sections surveyed by the BOR between 1989 and 2002. At channel grids where no surveyed cross sections are available, the cross sections were interpolated between surveyed cross sections using the PROFILES program included in FLO-2D.

A review of the URGWOPS FLO-2D model indicated that modifications to the overbank *n*-values were required due to the 2003 wildfires and non-native vegetation removal program, and more recent surveyed cross sections were available to update the cross-section geometry. Also, two locations in the URGWOPS FLO-2D model were identified and corrected where the channel elements were not continuous.

Overbank *n*-values used in the original URGWOPS FLO-2D model ranged from 0.1 to 0.125. Application of the Arcement and Schneider (1989) method for areas that were not affected by the 2003 wildfires or the fuel reduction and non-native vegetation removal program that is being conducted by the USACE indicates that overbank roughness is in the range of 0.12 (**Table 4.3**). This result agrees well with the original model values; thus, the original overbank *n*-values were retained for all grid elements that were not affected by fire or clearing. The Arcement and Schneider (1989) method also indicates that the *n*-value for the burned and cleared areas should be in the range of 0.065 (Table 4.3). A total of 224 model grid elements that were affected by the wildfires and clearing program were identified from satellite imagery, aerial photography, and available maps (USACE, 2004), and the *n*-value for these elements was changed to 0.065.

Computed water-surface elevations from the URGWOPS model at 6,300 cfs are lower than water-surface elevations measured by the USACE during the 2005 spring runoff at discharges in this range. Comparison of the thalweg profile from the original model with more recent data from surveys that were conducted by MEI in 2004 and 2005 between the South Diversion Channel (SDC) and Rio Bravo Boulevard and in the vicinity of Central Avenue and the North Diversion.

Table 4.2. Stationing of points of interest along the project reach.

River Mile*	Station (ft)	Station (mi)	Description
169.3	0	0	Isleta Diversion
172.5	16,050	3.04	Downstream end of project reach
172.6	17,170	3.25	I-25 Bridge
177.0	41,110	7.79	South Diversion Channel
173.4	47,770	9.05	Rio Bravo Boulevard Bridge
181.6	64,370	12.19	Bridge Street
183.4	74,060	14.03	Central Avenue Bridge (USGS Albuquerque Gage)
185.0	82,510	15.63	I-40 Bridge
188.0	97,910	18.54	Montano Road Bridge
191.0	113,730	21.54	Paseo del Norte Bridge
191.9	118,280	22.40	COA Drinking Water Project, Diversion Dam
192.2	119,810	22.69	New Alameda Boulevard Bridge
192.3	119,960	22.72	Old Alameda Bridge
194.0	129,060	24.44	Original upstream end of Project for the 500-foot grid
194.3	131,100	24.83	North Diversion Channel
199.6	159,200	30.15	Revised upstream end of project for the 250-foot grid (Corrales Siphon)
232.0			Cochiti Dam

*BOR (2003) River Mile stationing

Table 4.3. Overbank Manning's *n*-values (Arcement and Schneider, 1989).

$n=(n_b+n_1+n_2+n_3+n_4)m$

	Bosque	Cleared	Description
n_b	0.04	0.04	Base value of *n* for the floodplains bare surface
n_1	0.01	0.01	Correction factor for the effect of surface irregularities on the floodplain
n_2	0.00	0.00	Value for variations in shape and size of the floodplain cross section, assumed 0.0
n_3	0.02	0.005	Value for obstructions on the floodplain
n_4	0.05	0.011	Value for vegetation on the floodplain
m	1.00	1.00	Correction factor for sinuosity of the floodplain, equal to 1.0
n	0.12	0.065	Final overbank *n*-value

Channel (NDC) outlet, and by Bohannon-Huston in 2003 between I-40 and Montano Avenue, indicates that the model thalweg is, on average, about 1.2 feet low (**Figure 4.1**). A specific gage analysis based on USGS data at the Albuquerque gage shows that the gage rating curve lowered by about 2.5 feet between the mid-1970s and the mid-1980s in the low to intermediate range of flows, which indicates bed lowering during this period (**Figures 4.2 and 4.3**). The water-surface elevations in this range of flows remained relatively stable from 1982 to the late 1990s, and since the late 1990s, the channel has shown a slight aggradational trend, which may explain the difference in thalweg elevations between the URGWOPS FLO-2D sections and the MEI-surveyed sections. This trend is corroborated by data collected at Rangeline CO-36, which

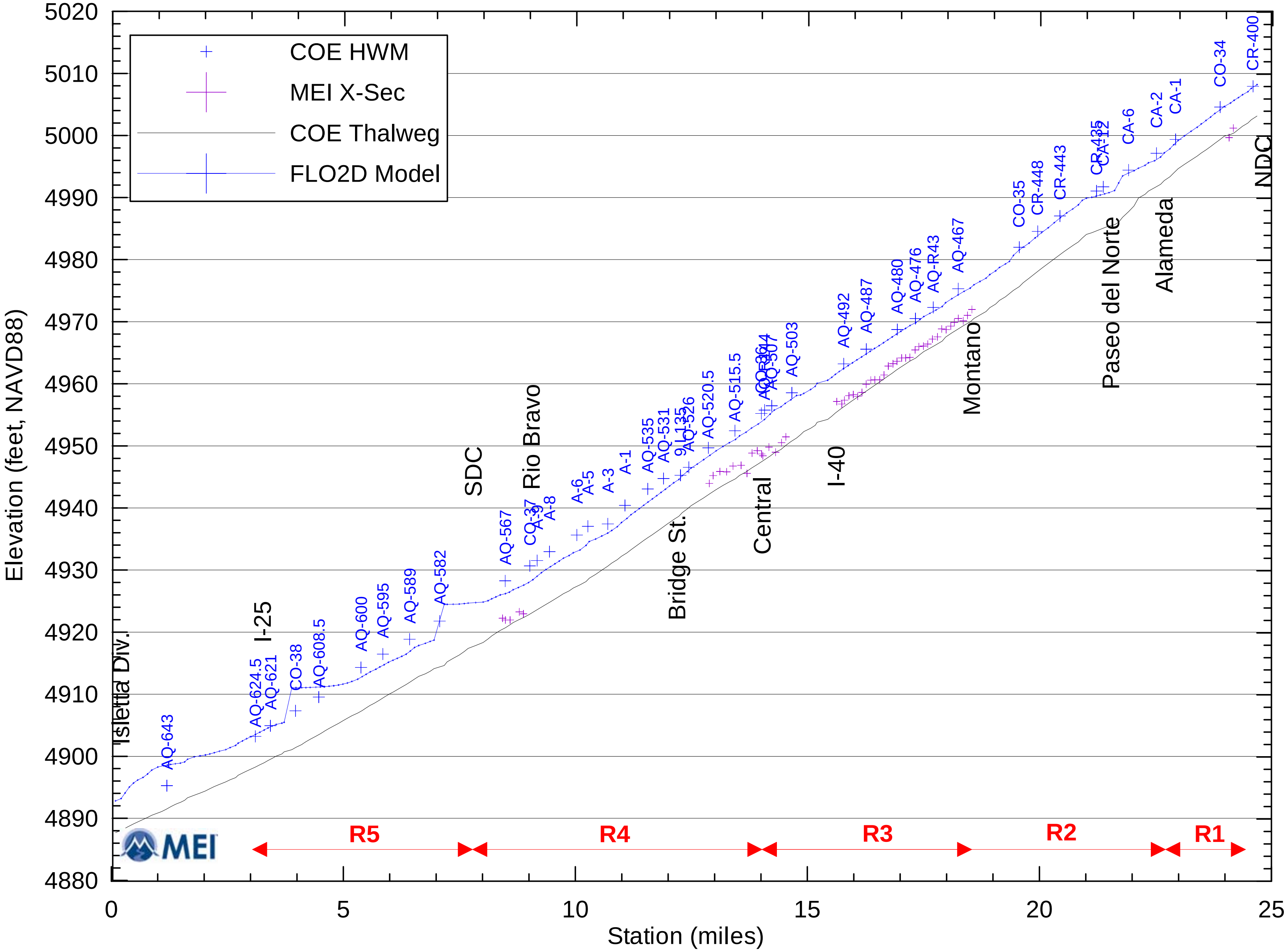

Figure 4.1. Comparison of computed water-surface and thalweg profile from URGWOM FLO-2D model to measured high-water marks at 6,300 cfs and thalweg elevations from surveyed cross sections.

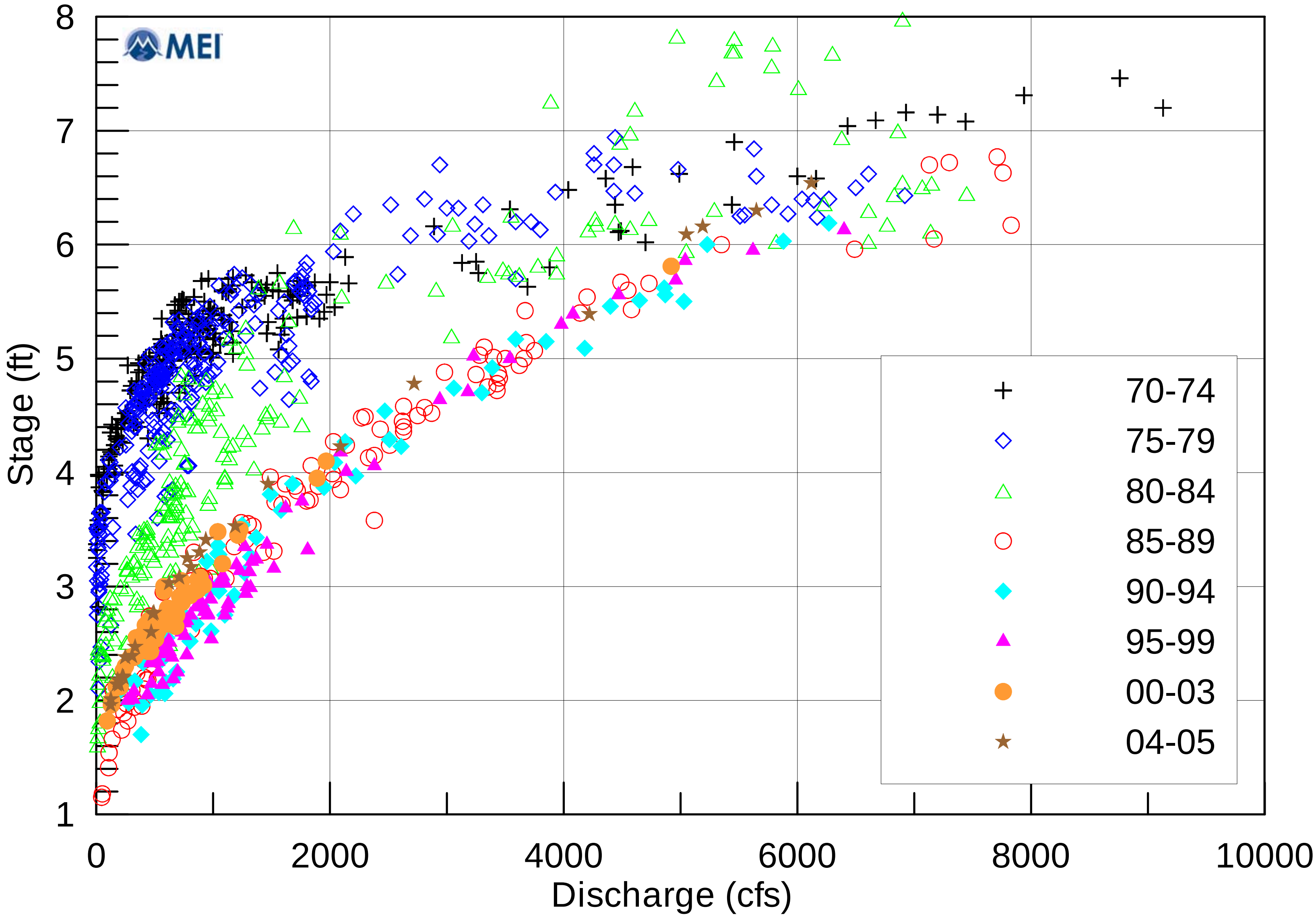

Figure 4.2. Measured stage-discharge data for the Albuquerque gage. Data collected during different time periods are shown with different symbols.

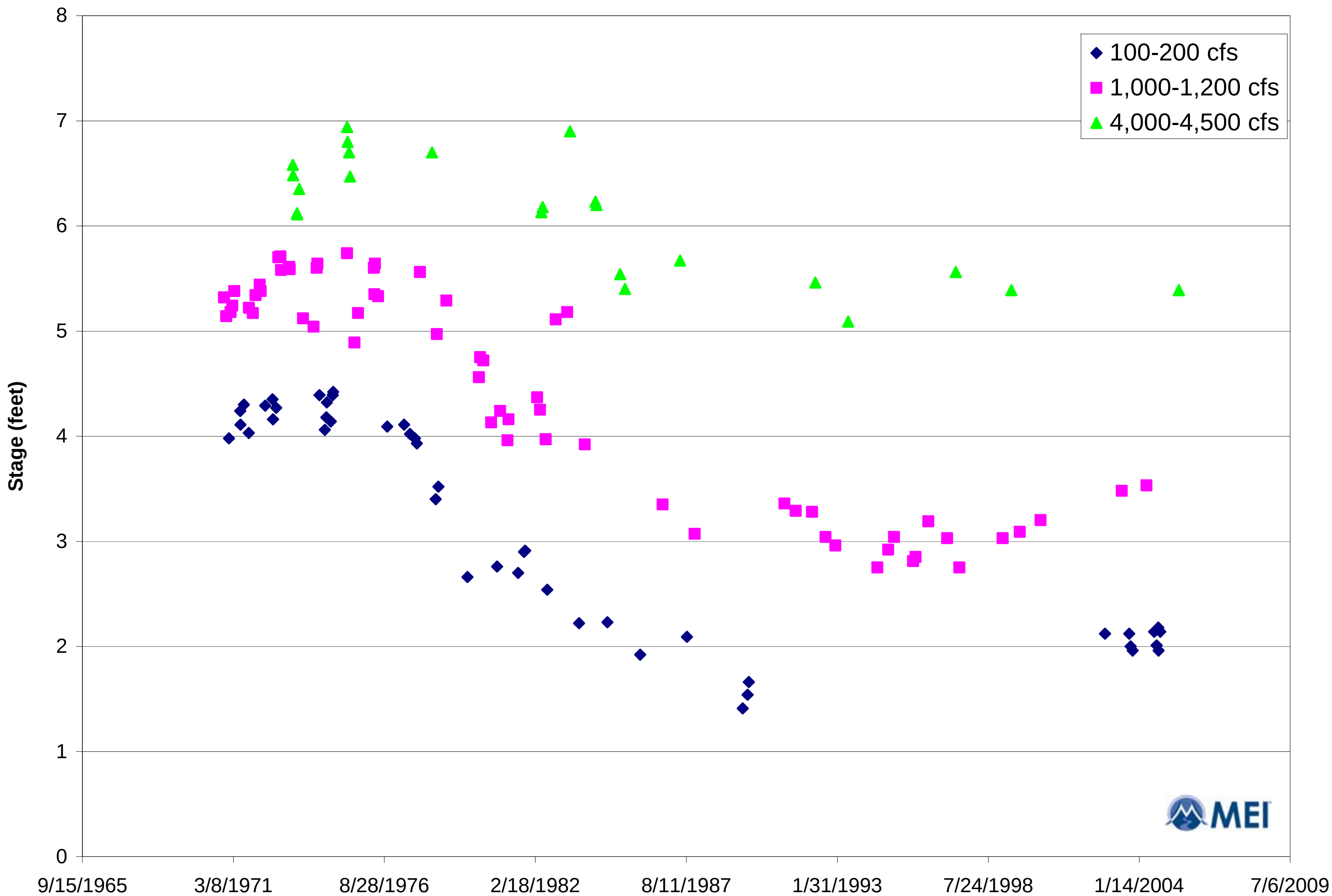

Figure 4.3. Specific gage plot showing changes in stage over time for three ranges of discharges (100 to 200 cfs, 1,000 to 2,000 cfs, and 4,000 to 5,000 cfs).

is located just downstream of the Central Avenue Bridge (**Figure 4.4**). The most recent survey by MEI (2004) shows the thalweg to be 3.1 feet higher than the 2001 BOR survey and 2.1 feet higher than the 2000 BOR survey. It is also interesting to note that the channel width from the BOR photo-interpreted rangeline (Agg/Deg Line 510) and the MEI (2004) survey at this location is about 100 feet less than indicated by the 2001 CO-36 rangeline survey and 2000 cross section that were used in the FLO-2D model, even though all sections appear to have been surveyed in very close proximity to each other. This difference is believed to result from attachment of a small island located under the bridge to the left bank.

To improve calibration of the model to the available high-flow data, cross sections in applicable portions of the project reach were updated with the more recently surveyed sections (**Table 4.4**). In other areas, the model cross sections were updated using data from an HEC-RAS model that is being developed by the USACE using BOR agg/deg lines that were developed from 2002 aerial photography.

Table 4.4. Summary of surveyed cross sections used to update the FLO-2D model.

Number of Cross Sections	Location Description	Station (miles)	Year of Survey
2	Approximately 3,500 feet downstream of North Diversion Channel	24.1 – 24.2	2005
29	Central Avenue to Montano	15.6 – 18.5	2003
14	Up- and downstream of Central Avenue	12.9 – 14.5	2004
3	Rio Bravo to South Diversion Channel	8.4 – 8.5	2005

It is important to note that the cross sections from the agg/deg lines do not include subaqueous data, and therefore, the minimum bed elevations are the same as the water-surface elevation at the time of the aerial photography, when the discharge was about 300 cfs. To account for the subaqueous portion of the channel, the BOR lowered the bed elevations using a prism adjustment method, the details of which are not specifically known. On average, the difference in thalweg elevation between the prism-adjusted and unadjusted cross sections is about 1.0 feet throughout the project reach. As a result, the USACE initially evaluated whether using the unadjusted or adjusted data in their HEC-RAS model would produce better results. USACE efforts to calibrate the models indicated that the unadjusted cross sections produce results that are more consistent with the measured water-surface elevation data. Based on this result, cross sections in the FLO-2D model for locations where more recent surveyed cross sections were not available were updated using the unadjusted agg/deg line data. Of the 234 FLO-2D channel elements within the project reach, 49 were updated with surveyed cross sections and 185 were updated with unadjusted agg/deg line data.

In the original URGWOPS FLO-2D model, main channel Manning's *n*-values varied from 0.025 to 0.047 (**Figure 4.5**). Manning's *n*-values used in the USACE unadjusted HEC-RAS model ranged from 0.025 to 0.034, and there was less variability with distance along the reach. MEI made further adjustments to the Manning's *n*-values in the unadjusted HEC-RAS model to produce results that are more consistent with the 2005 observed high-water profile. The revised Manning's *n*-values used in the HEC-RAS model were applied to the updated FLO-2D model (Figure 4.5).

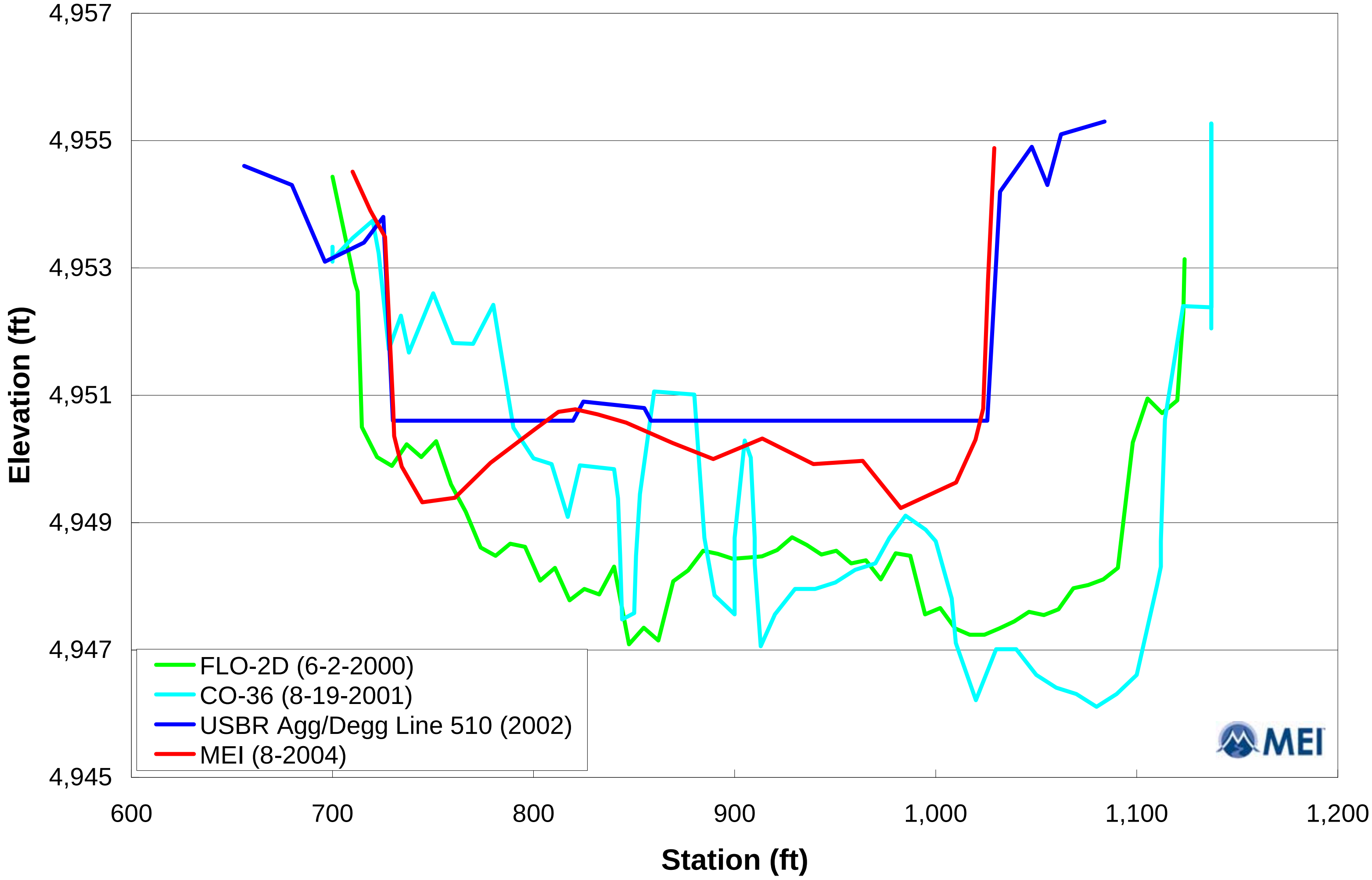

Figure 4.4. Cross-section comparison at Rangeline CO-36, located just below Central Avenue Bridge.

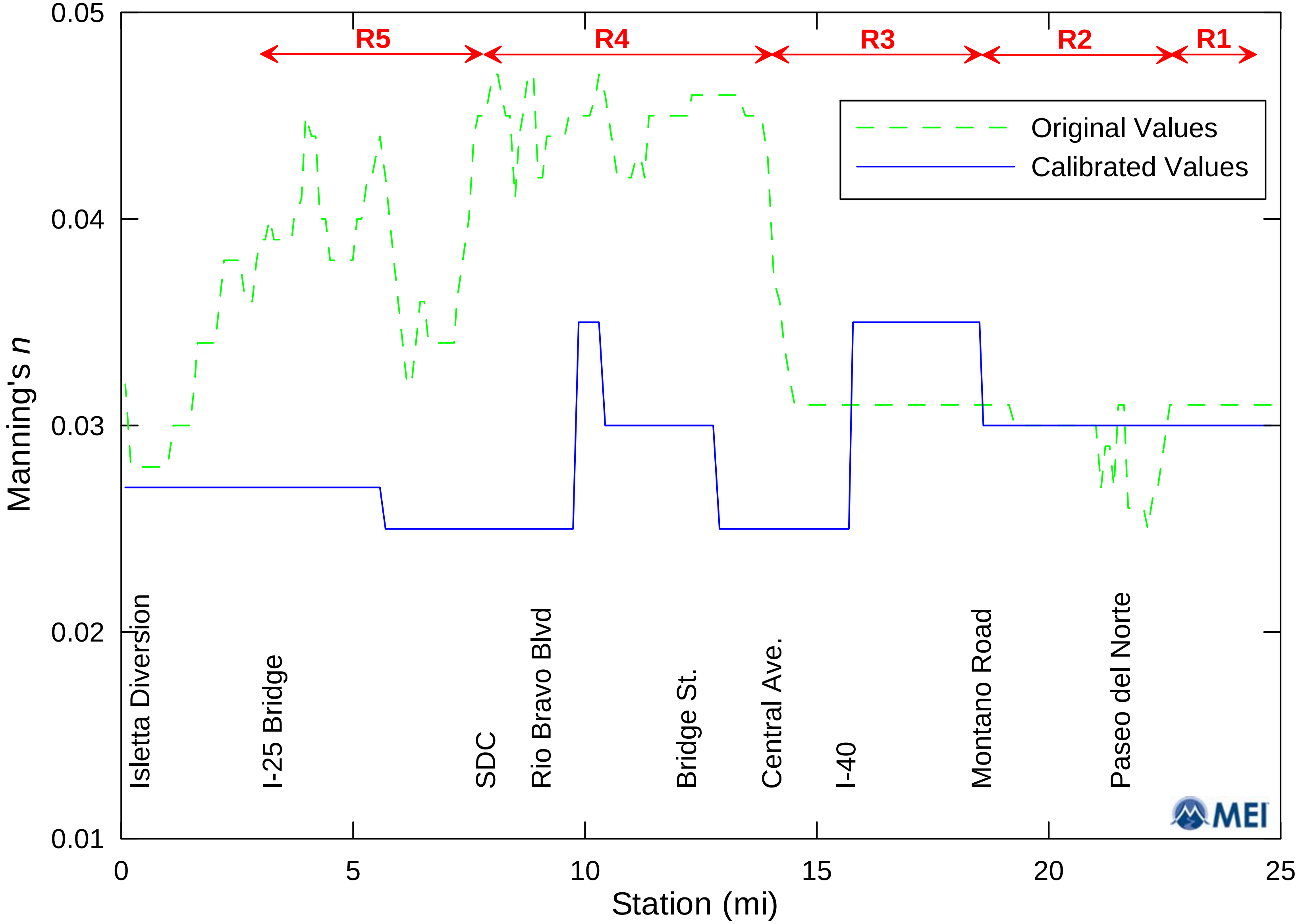

Figure 4.5. Manning's *n*-values used in the updated HEC-RAS and FLO-2D models compared to the original URGWOMS FLO-2D model.

The FLO-2D model does not have the capability to model losses through bridges or other in-line hydraulic structures, such as the City of Albuquerque's inflatable raw-water diversion dam that is located at Sta 118,280. The recommended method of accounting for the effects of these structures is to develop rating curves using other techniques that can be applied in the appropriate place in the model. As a result, MEI updated the USACE HEC-RAS model that uses the unadjusted agg/deg data with the newer surveyed cross sections, as appropriate, and applied the updated model to evaluate the effects of the bridges and diversion dam. Results of the analysis indicate that the bridges and the diversion dam (in its deflated position) have very little hydraulic effect and, therefore, it was concluded that rating curves at these locations are not required in the FLO-2D model. The rating curve from the original URGWOPS FLO-2D model at the Isleta Diversion Dam was used as the downstream boundary conditions in the updated model.

4.2. Validation of 500-foot Grid Model

Comparison of the predicted water-surface elevation at 6,300 cfs from the updated FLO-2D model with the 2005 measured profile shows very good agreement (**Figure 4.6**). To evaluate the performance of the model over a broader range of flows, a stepped hydrograph was modeled from 500 to 15,000 cfs with the discharge increasing in 500 cfs increments up to 3,000 cfs, and by 1,000 cfs increments from 3,000 to 15,000 cfs every 72 hours to allow the model to reach steady-state conditions during each discharge period. The results at the end of each period were then used to develop rating curves at four bridges where measured water-surface elevations were available (**Figures 4.7 through 4.10**).

The Old Alameda Bridge is located at the corner of two FLO-2D grid elements. The rating curves predicted by the model for these two grid elements bound the water-surface elevations that were measured at this location at discharges between 1,000 and 4,200 cfs, and the predicted rating curves are also consistent with results from the HEC-RAS model (Figure 4.7).

Similar agreement is obtained at the Central Avenue, Bridge Street, and Rio Bravo bridges (Figures 4.8 through 4.10).

Field mapping indicated that very little overbank inundation occurred during the 2005 peak flows. The validated FLO-2D model predicts that inundation occurs in only two locations: (1) just upstream of the Central Avenue Bridge on the left bank, and (2) midway between Bridge Street and Rio Bravo Boulevard on the right bank, consistent with the field-mapped inundation.

Based on the above-described results, the updated FLO-2D model appears to be reasonably well validated.

4.3. Reach-averaged and Main-channel Hydraulic Results

The one-dimensional hydraulic results for the main channel (e.g., flow velocity, depth, topwidth, and energy slope) were taken from the model output for the stepped hydrograph run that was described in the validation section for use in the sediment-transport and channel stability analysis. These results indicate that main channel velocities vary from approximately 0.9 to 3.4 fps at 1,000 cfs and from 1.7 to 7 fps at 6,000 cfs (**Figure 4.11**). Higher velocities typically occur at contractions created by islands, bank-attached bars, bridges and at tributary confluences; whereas the lower velocity areas occur at locally wide sections. Channel topwidths vary from 160 to 1,060 feet at 1,000 cfs and 200 to 1,060 feet at 6,000 cfs (**Figure 4.12**).

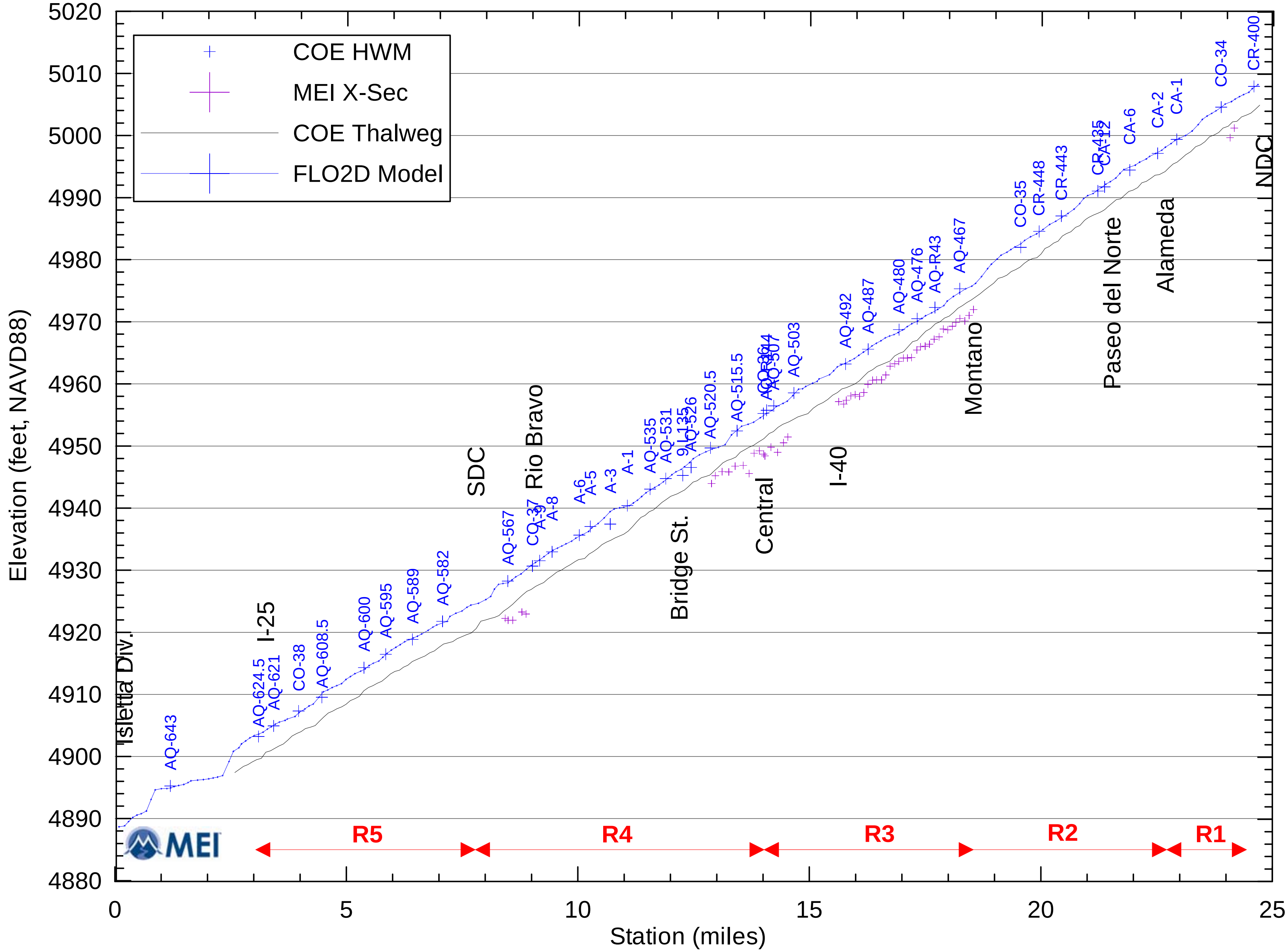

Figure 4.6. Comparison of computed water-surface elevation from the MEI FLO-2D model and the thalweg profile from URGWOM FLO-2D model to measured high-water marks at 6,300 cfs and thalweg from surveyed cross sections.

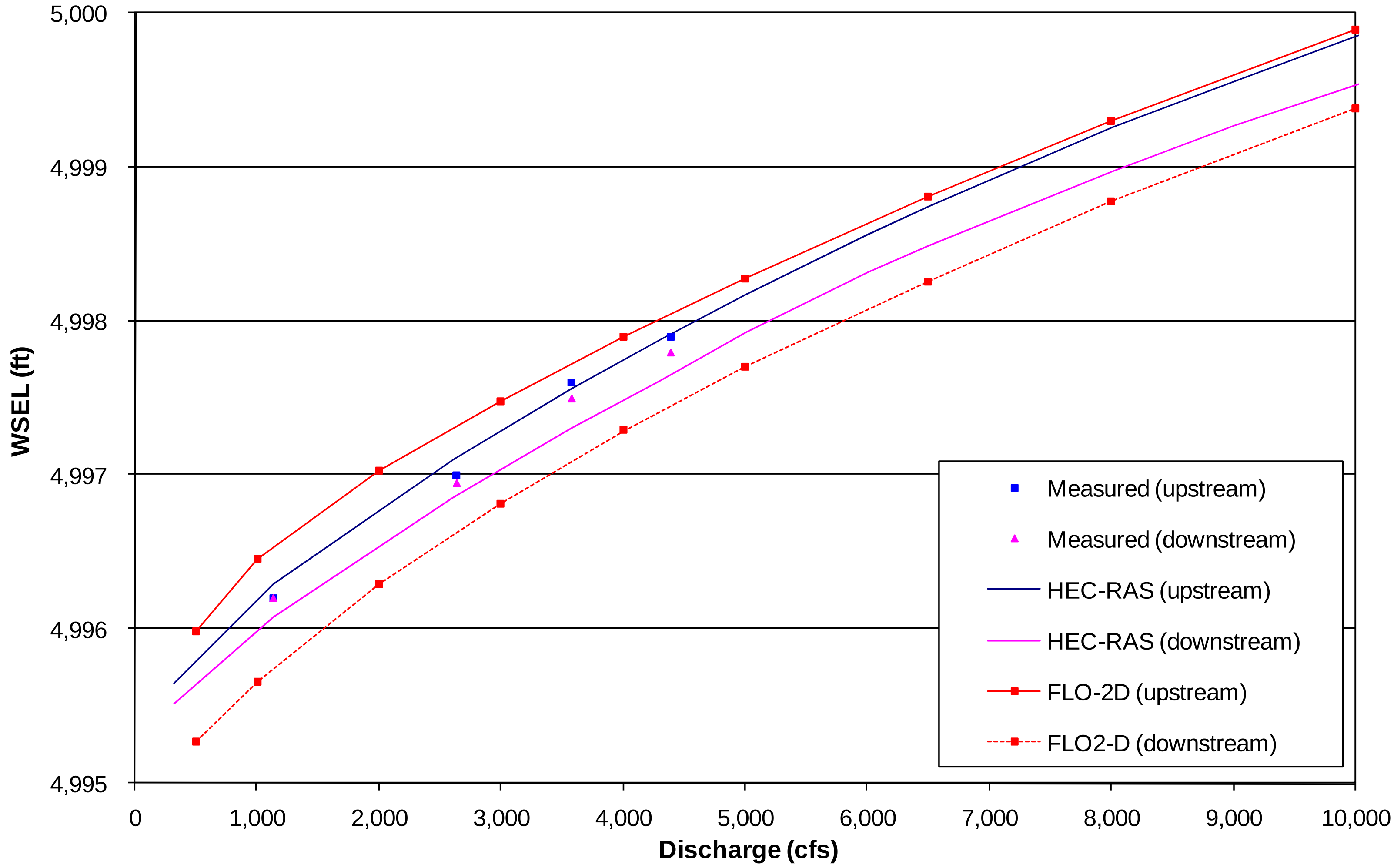

Figure 4.7. Comparison of computed rating curves from the FLO-2D model and HEC-RAS with measured values up- and downstream of the Old Alameda Bridge.

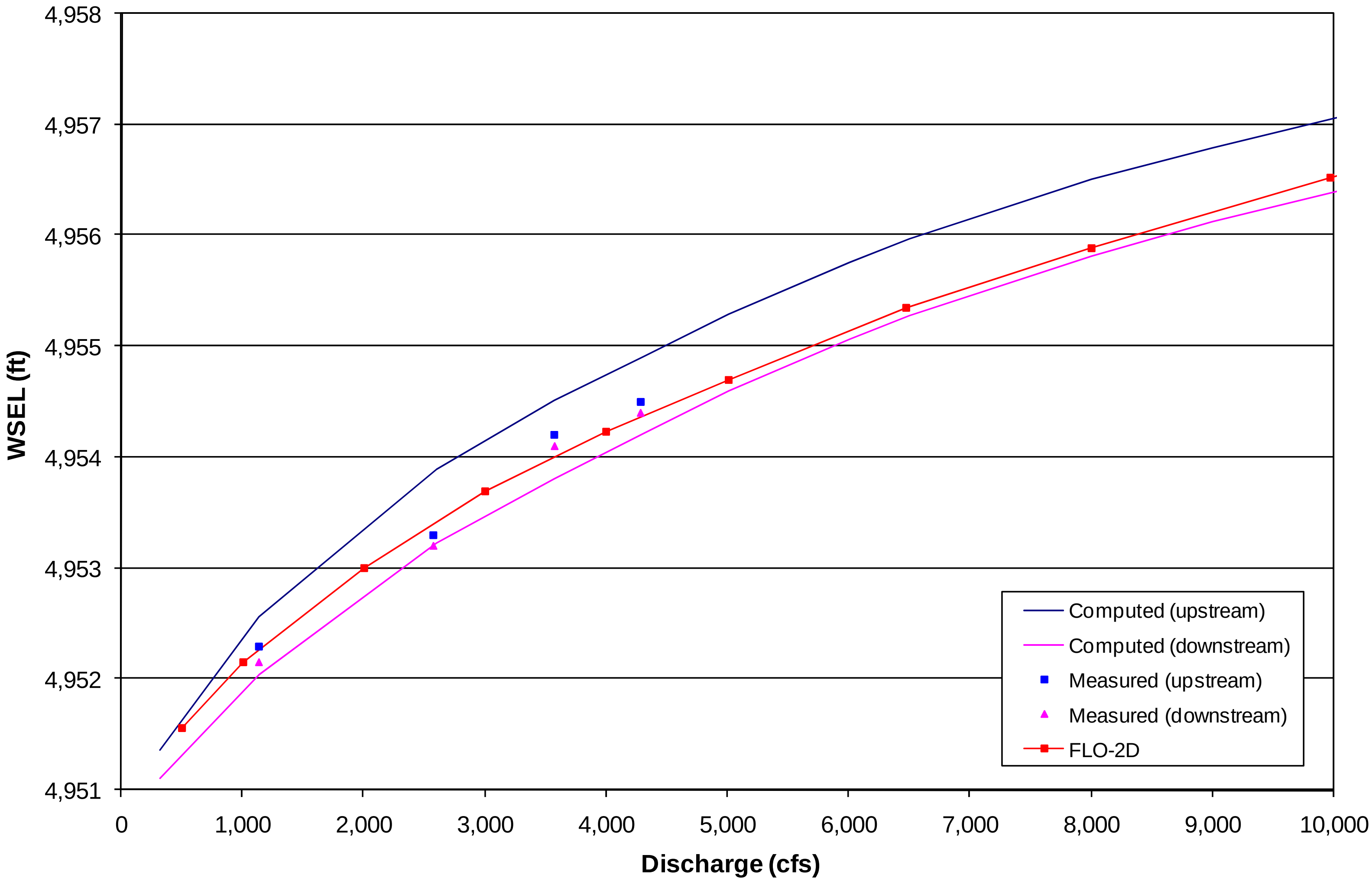

Figure 4.8. Comparison of computed rating curves from the FLO-2D model and HEC-RAS with measured values up- and downstream of the Central Avenue Bridge.

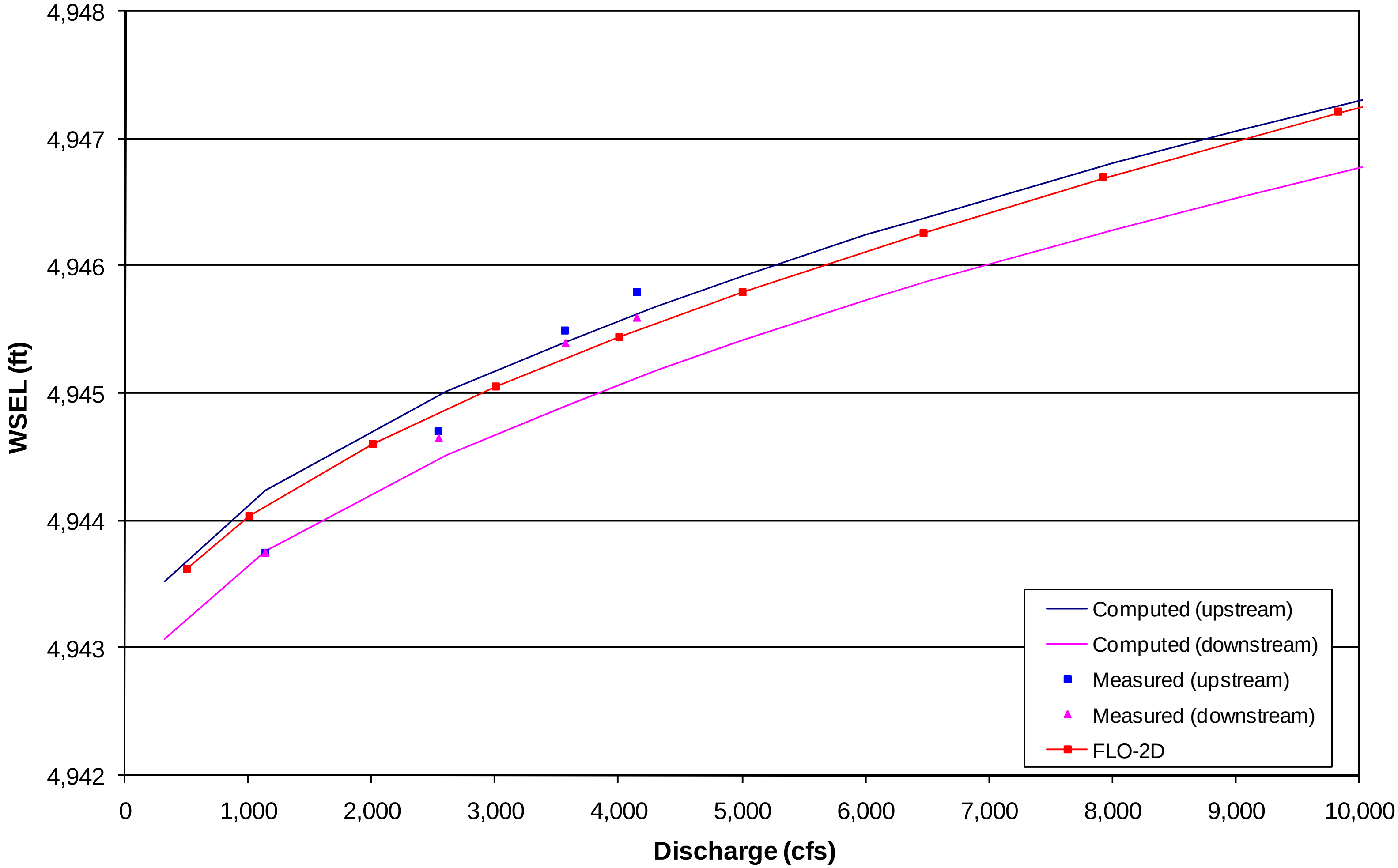

Figure 4.9. Comparison of computed rating curves from the FLO-2D model and HEC-RAS with measured values up- and downstream of Bridge Street.

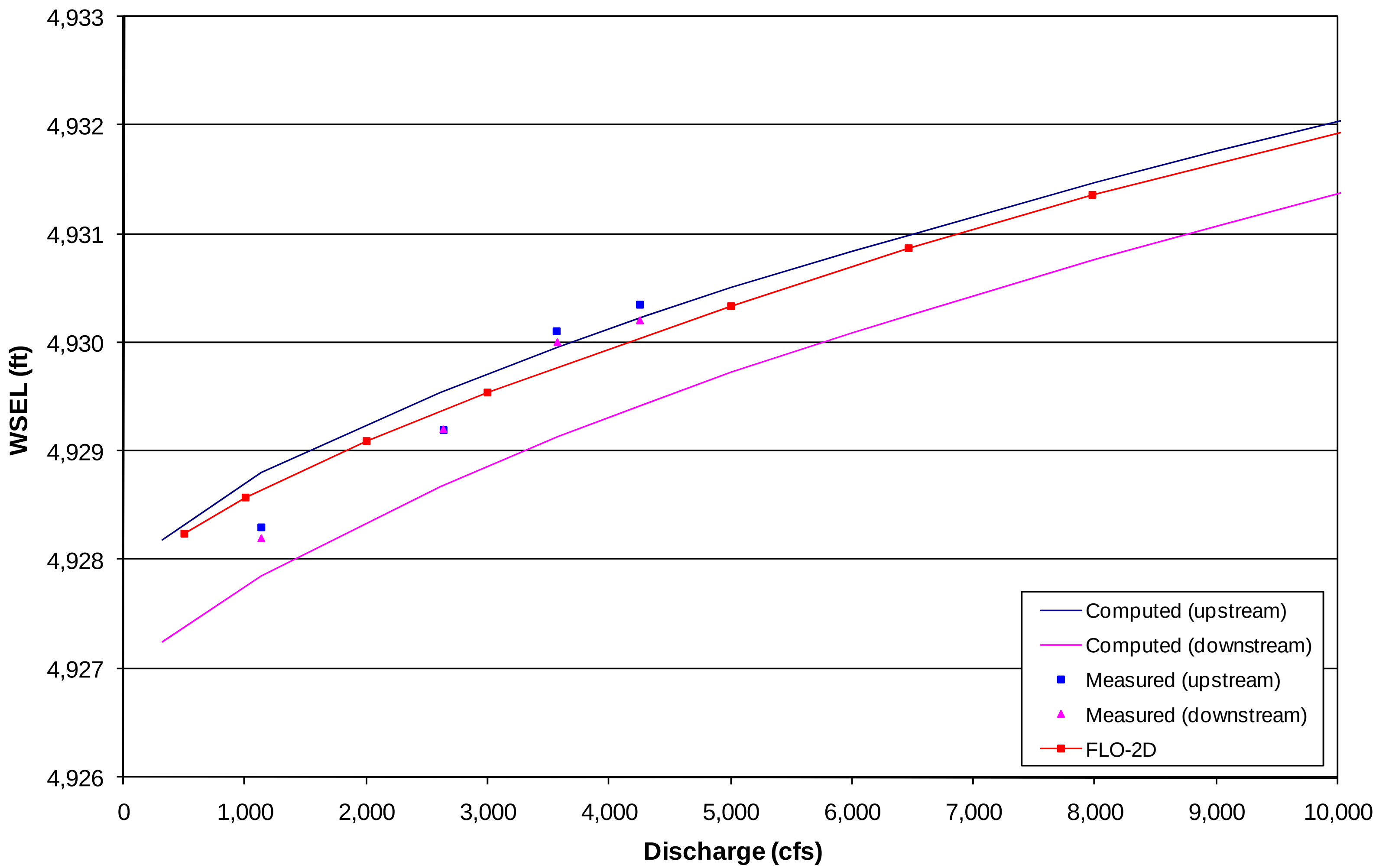

Figure 4.10. Comparison of computed rating curves from the FLO-2D model and HEC-RAS with measured values up- and downstream of the Rio Bravo Bridge.

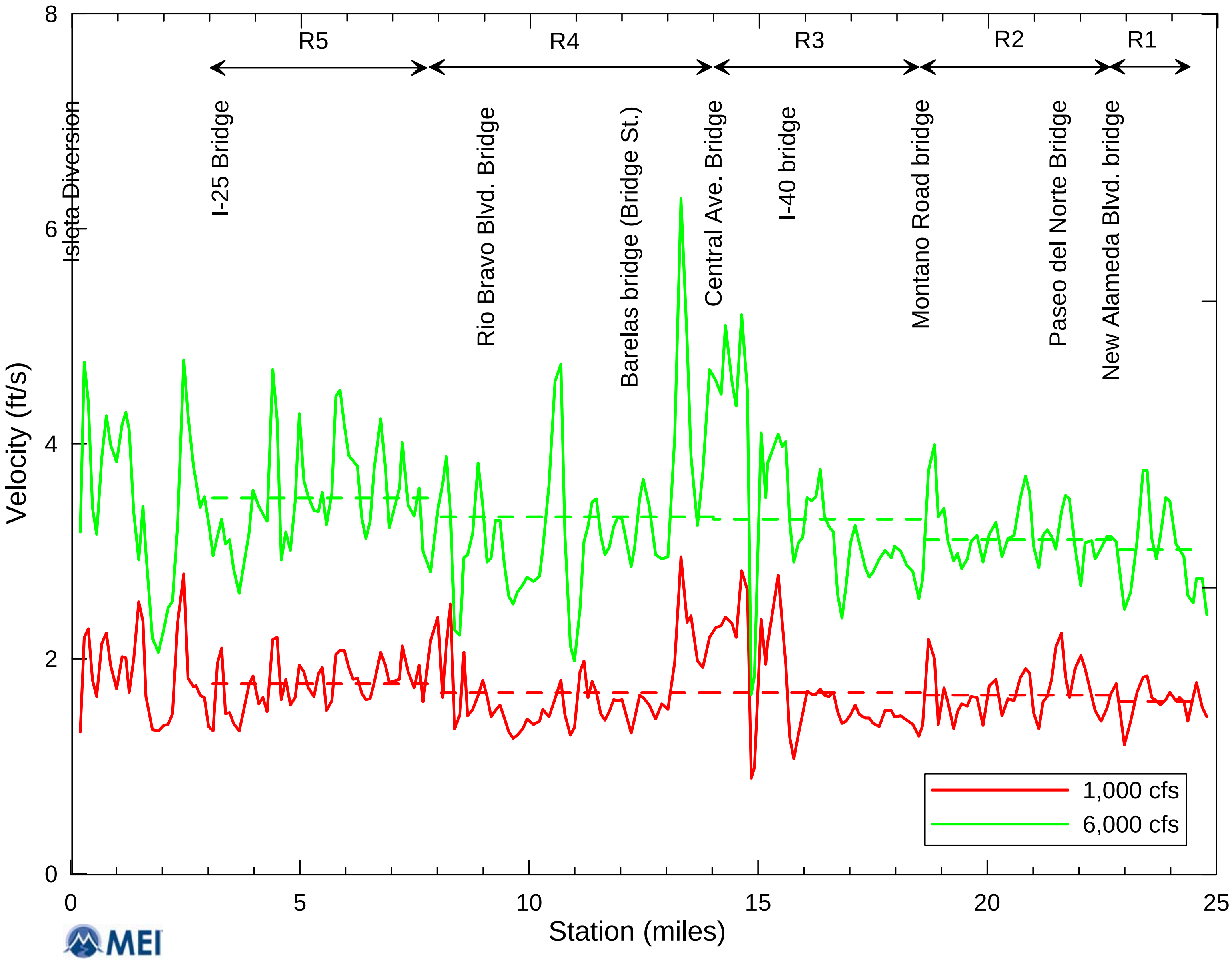

Figure 4.11. Channel velocities and reach-averaged velocities at 1,000 and 6,000 cfs.

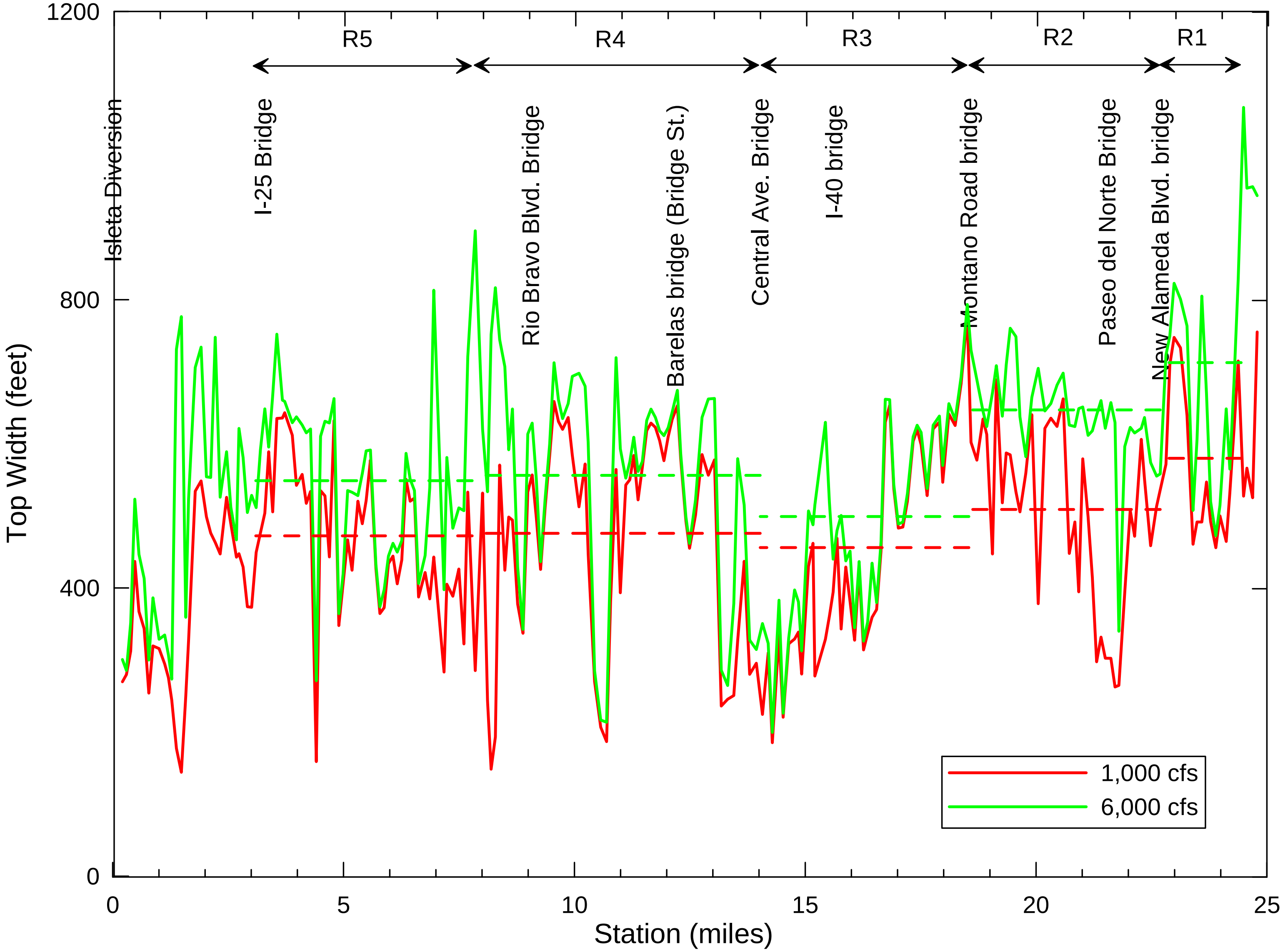

Figure 4.12. Channel topwidth and reach-averaged topwidth at 1,000 and 6,000 cfs.

To facilitate the sediment-transport and channel-stability analysis, the study reach was subdivided into the five subreaches that are being used for the ecological analysis (Figure 1.1, **Table 4.5**). Within these subreaches, the geomorphic and hydraulic characteristics of the channel are generally consistent. Reach-averaged hydraulic conditions were developed from the model output for each subreach (**Table 4.6**).

Table 4.5. Summary of subreaches defined for the channel-stability analyses.

Subreach	Subreach Length (ft)	Main Channel Topwidth (ft)[1]	Limits
1	10,760	710	Southern boundary of the Pueblo of Sandia to Alameda Bridge
2	22,190	650	Alameda Blvd. Bridge to Montano Blvd. Bridge
3	23,430	500	Montano Blvd. Bridge to Central Avenue Bridge
4	32,190	545	Central Avenue Bridge to the South Diversion Channel
5	25,640	550	South Diversion Channel to the northern boundary of the Pueblo of Isleta

[1]at the active channel-full flow of 6,000 cfs

Table 4.6. Reach-averaged hydraulic conditions in the project reach.

Subreach	Discharge (cfs)						
	500	1,000	2,000	4,000	6,000	8,000	10,000
Velocity (ft/s)							
1	1.23	1.60	2.02	2.59	3.01	3.34	3.59
2	1.20	1.66	2.11	2.69	3.11	3.45	3.74
3	1.11	1.69	2.19	2.83	3.30	3.64	3.92
4	1.00	1.69	2.21	2.86	3.32	3.66	3.95
5	0.91	1.77	2.36	3.05	3.50	3.82	4.12
Hydraulic Depth (ft)							
1	0.75	1.07	1.53	2.21	2.79	3.29	3.73
2	0.82	1.16	1.67	2.42	2.97	3.45	3.88
3	0.88	1.22	1.83	2.75	3.52	4.17	4.75
4	0.91	1.19	1.75	2.58	3.22	3.77	4.26
5	0.82	1.12	1.70	2.49	3.08	3.55	4.00
Top Width Channel (ft)							
1	520	580	645	698	713	727	743
2	468	509	562	611	647	671	686
3	415	456	478	494	499	509	515
4	407	476	507	535	556	570	573
5	437	472	488	516	549	576	583
Energy Slope (ft/ft)							
1	0.00090	0.00095	0.00094	0.00094	0.00094	0.00092	0.00090
2	0.00076	0.00092	0.00091	0.00090	0.00092	0.00093	0.00093
3	0.00066	0.00098	0.00096	0.00094	0.00091	0.00088	0.00087
4	0.00040	0.00080	0.00081	0.00082	0.00082	0.00080	0.00080
5	0.00033	0.00083	0.00084	0.00084	0.00084	0.00083	0.00082

4.4. 250-foot FLO-2D Model

In 2007, Riada Engineering (2008) and MEI (2008b) refined the FLO-2D model for the USACE by incorporating new cross-sectional data and increasing the grid resolution to 250 feet. The 250-foot grid FLO-2D model was calibrated, to the extent possible by the following methods:

1. Comparison of the computed water-surface elevations (**Figure 4.13**) with the high-water marks collected during the 2005 high-flow period when the peak discharge was approximately 6,300 cfs (Tetra Tech, 2005),
2. Comparison of the overbank inundation areas with aerial photography and flood mapping collected by the USACE during the 2005 high-flow period, and
3. Comparison of the computed and measured flood-routing characteristics of recorded hydrograph events, including the hydrograph shape (volume), hydrograph timing and water-surface elevations.

MEI subsequently modified the 250-foot FLO-2D grid to incorporate data from a LIDAR survey performed on December 7 and 8, 2006, by LIDAR US LLC when the discharge at the Albuquerque gage was approximately 700 cfs. The LIDAR survey included the area between the levees within the boundaries of the Pueblo of Sandia (**Figure 4.14**). The LIDAR data were used to generate 1-foot contour interval mapping and 30-foot resolution ArcGIS grid files that were used for overbank mapping of the FLO-2D results. The LIDAR data and the Grid Developer System (GDS) that is part of the FLO-2D program were used to assign elevations to the FLO-2D grid within the extent of the LIDAR survey; no changes were made to the channel cross sections as a result of the updated grid elevations.

4.5. Results from the 250-foot FLO-2D Model

The validated existing conditions FLO-2D model was run for the four hydrology scenarios, and the results were used to compare the main channel water-surface elevations with the top-of-bank elevations (**Figures 4.15 through 4.18**), and to map and evaluate the extent, depth and duration of overbank inundation along the reach (Appendix A).

The overbank inundation and duration of inundation results from the 250-foot grid model compare favorably to the results from the 500-foot model.

In the FLO-2D model, a representative elevation is assigned to each grid cell; thus, the local depth or duration of inundation at any point within the cell may vary from the representative value predicted by the model due to variations in the ground elevations. To provide a more detailed depiction of the variation in depth than is shown with the 250-foot grid spacing, a new water-surface DTM with 30-foot pixel resolution was developed based on maximum water-surface elevations predicted by the FLO-2D model for each simulation. The local depth within each 30-foot pixel was then determined by overlaying the water-surface DTM onto the detailed ground-surface DTM.

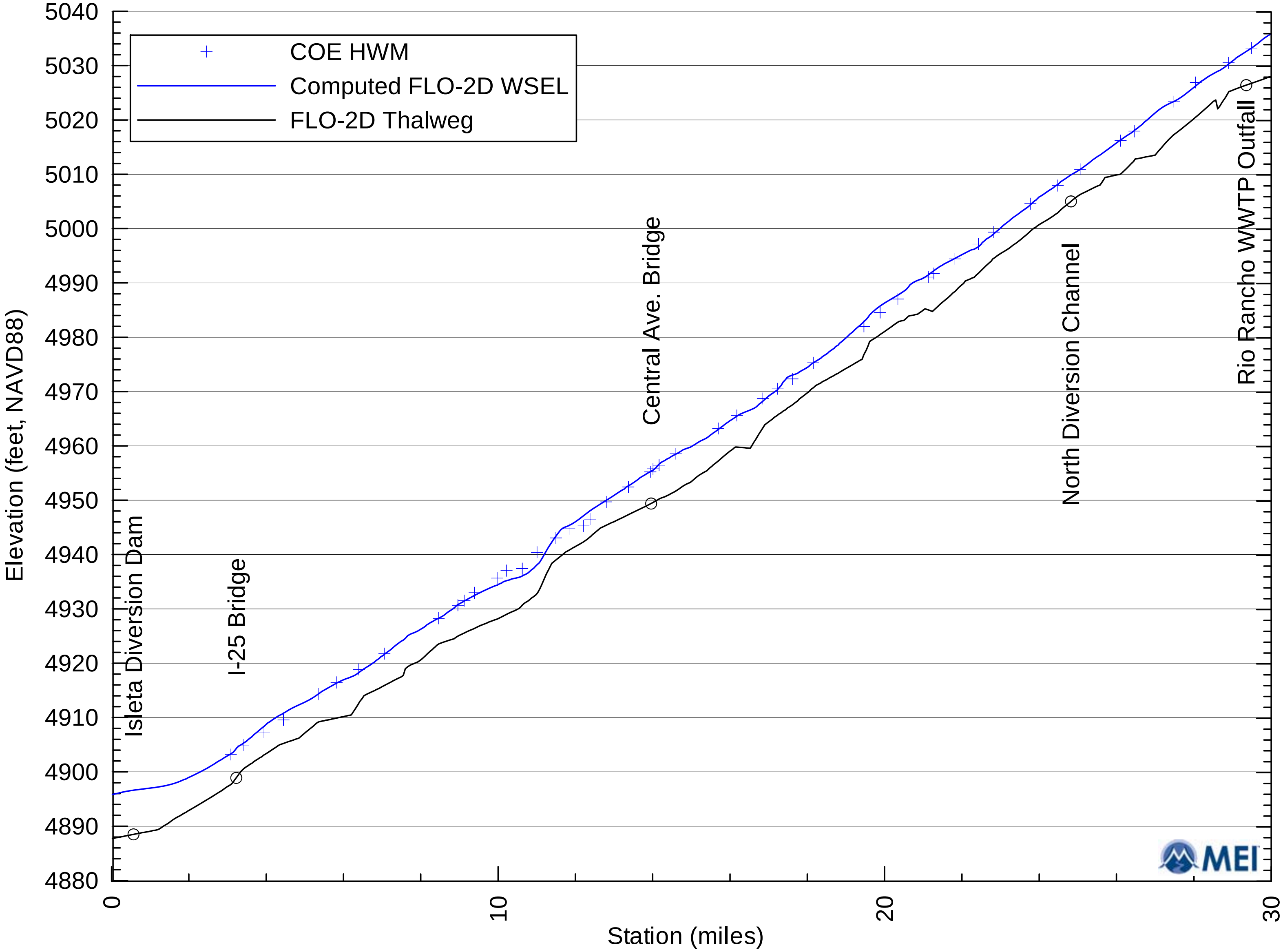

Figure 4.13. Comparison of the predicted and measured water-surface profiles from the 250-foot FLO-2D model at a discharge of 6,300 cfs.

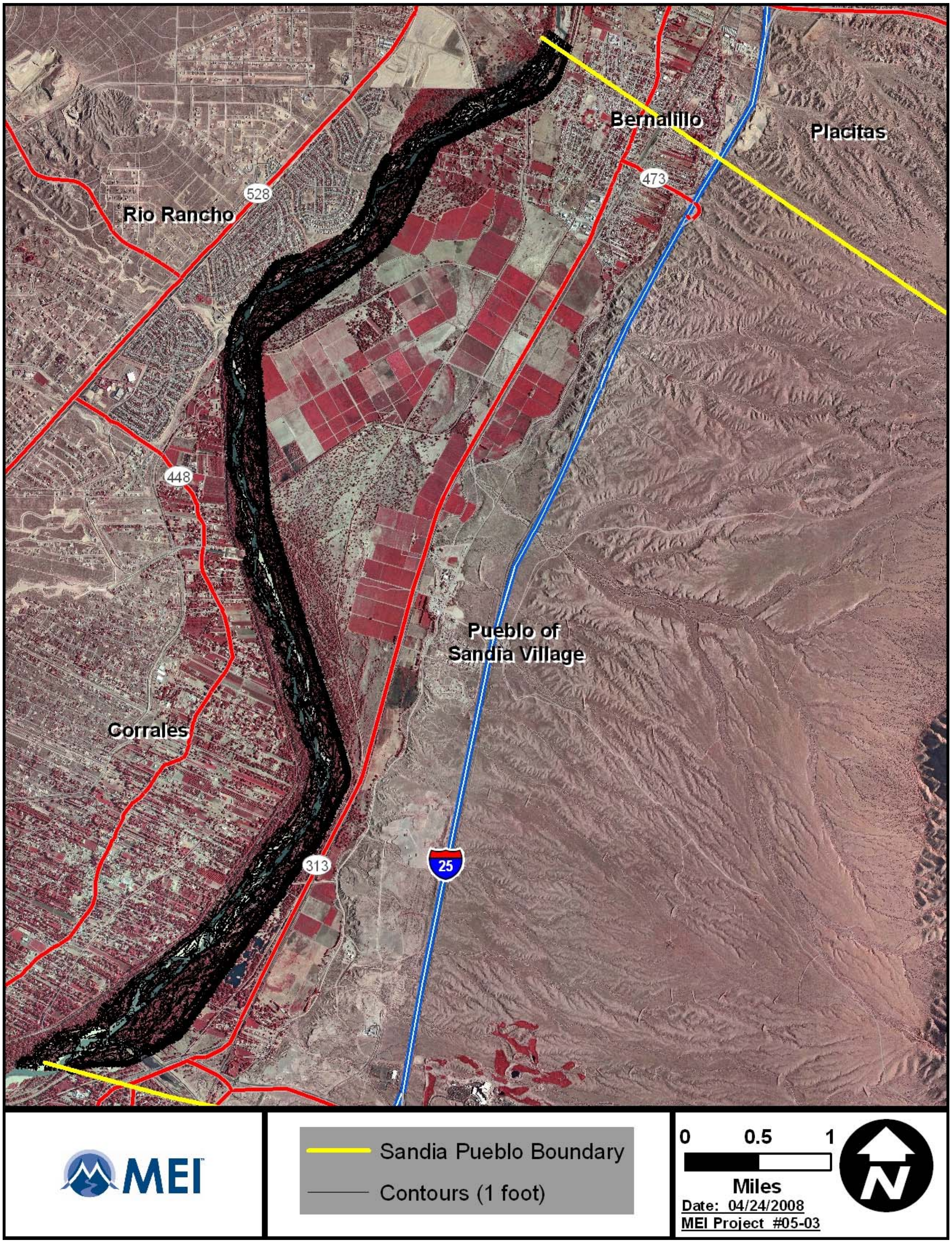

Figure 4.14. 2006 LiDAR-based 1-foot contour mapping overlain on July 2005 aerial photograph.

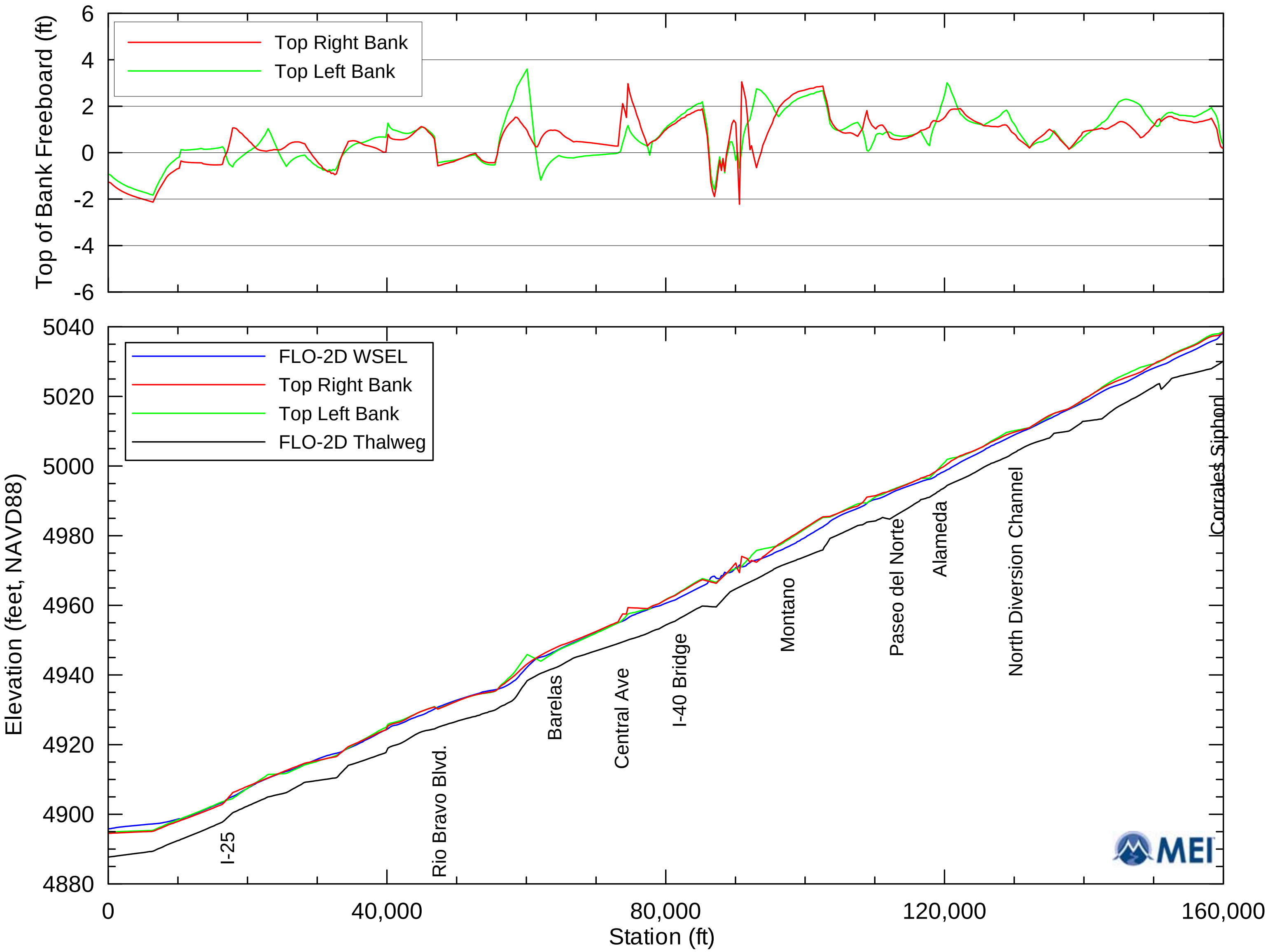

Figure 4.15. Comparison of the computed water-surface profile for channel full flow (Hydrology Scenario 1, Steady State discharge 6,000 cfs) with the thalweg, top of left bank and top of right bank elevations.

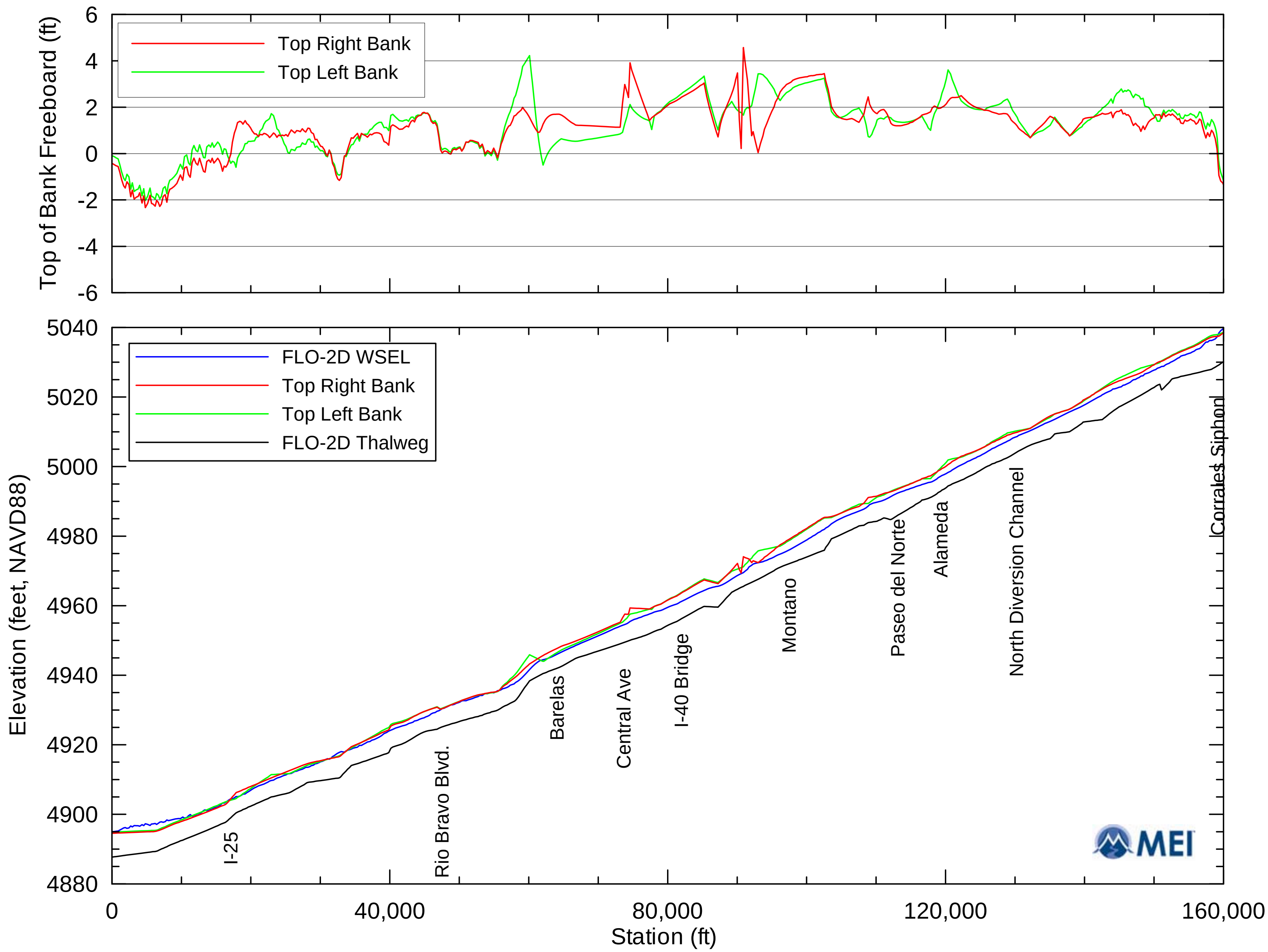

Figure 4.16. Comparison of the computed maximum water-surface profile for Annual Spring Runoff hydrograph (Hydrology Scenario 2, peak flow 3,770 cfs) with the thalweg, top of left bank and top of right bank elevations.

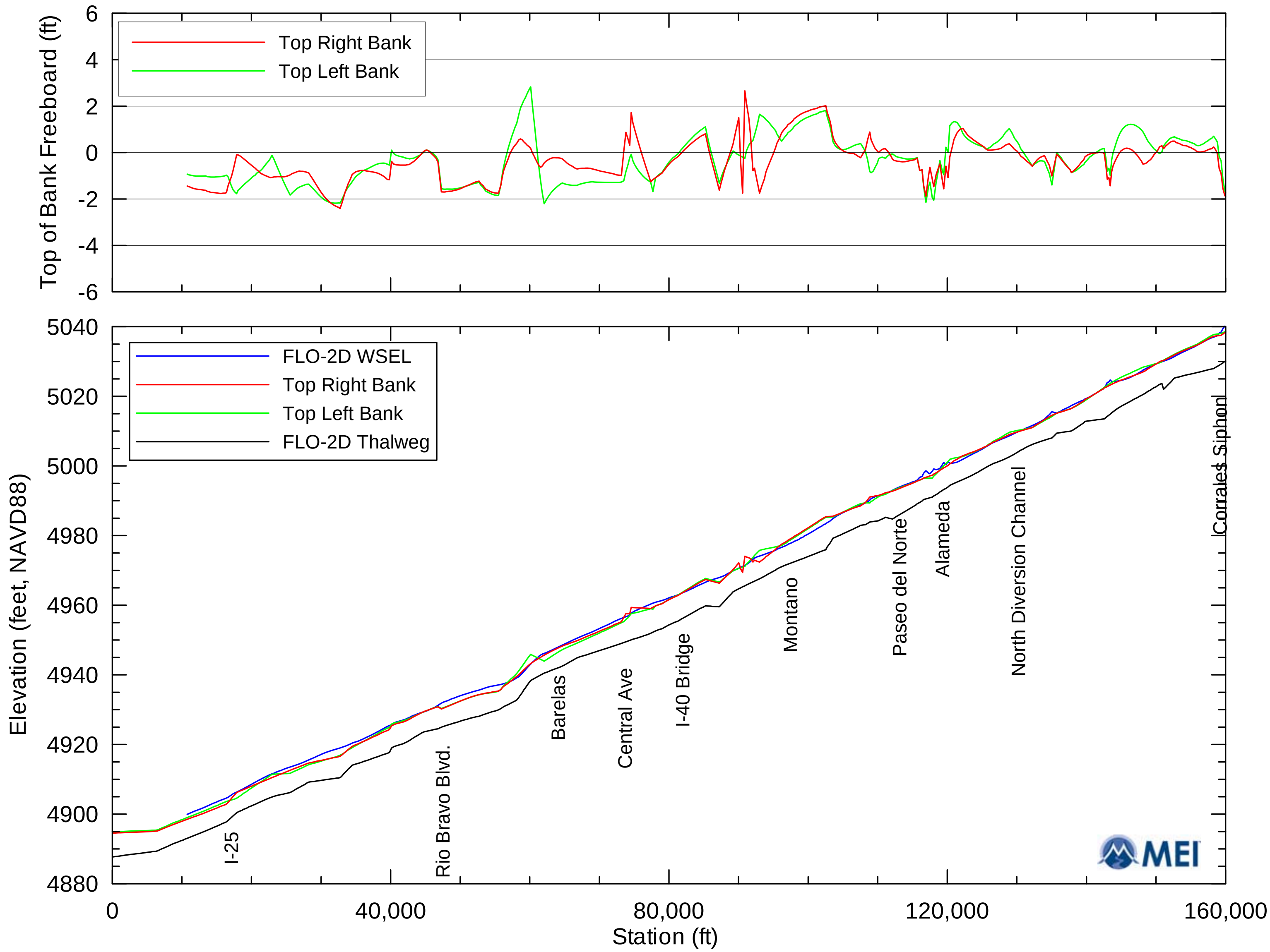

Figure 4.17. Comparison of the computed maximum water-surface profile for 10,000 cfs hydrograph (Hydrology Scenario 3, peak flow 10,000 cfs) with the thalweg, top of left bank and top of right bank elevations.

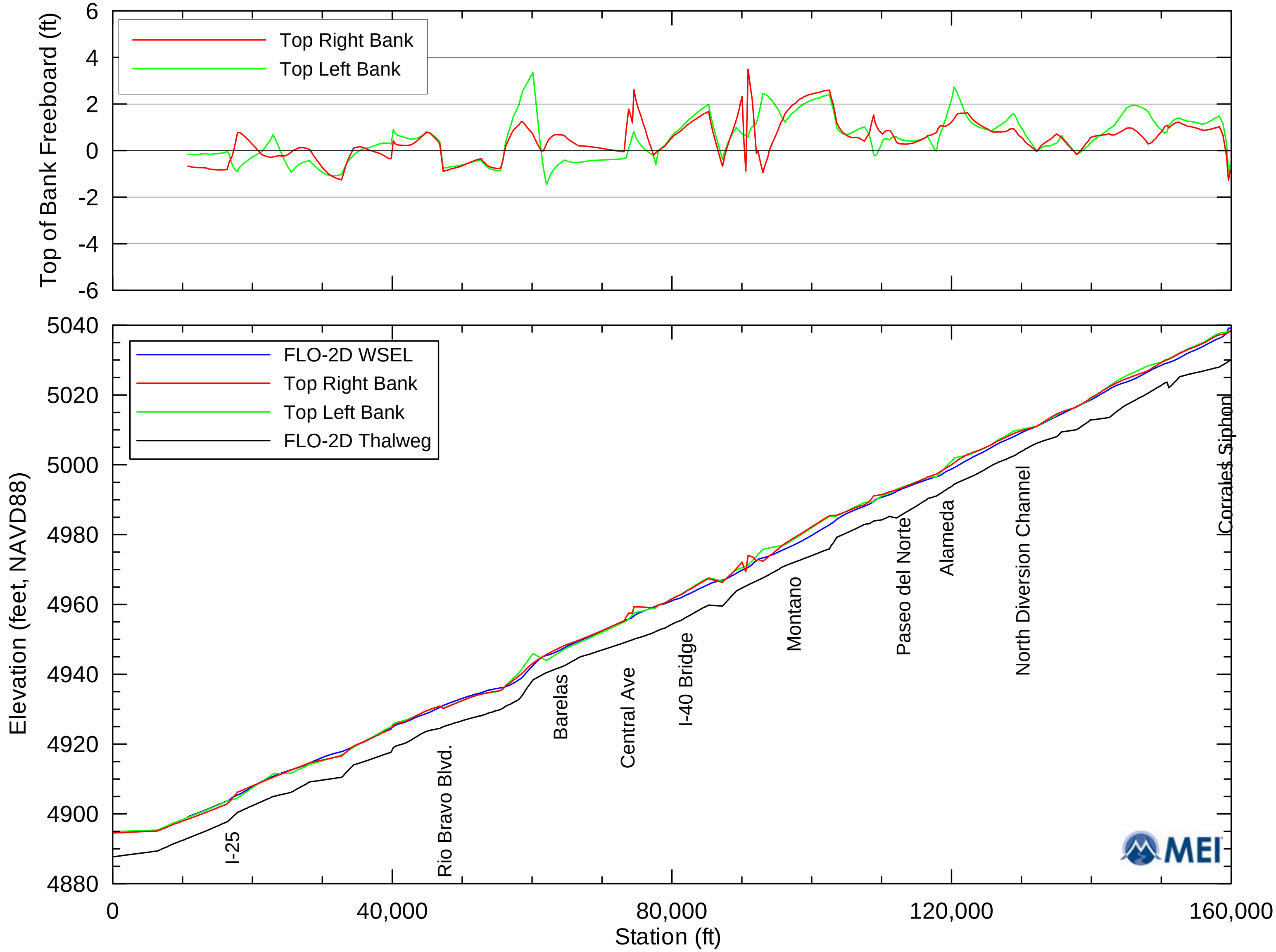

Figure 4.18. Comparison of the computed maximum water-surface profile for 100-year Snowmelt hydrograph (Scenario 4, peak flow 7,750 cfs) with the thalweg, top of left bank and top of right bank elevations.

4.5.1. Future Channel Condition Analysis

To reflect future channel conditions in the project reach under the existing conditions, changes in the channel cross sections associated with aggradation/degradation 5, 20, 30 and 50 years after project implementation were estimated using a HEC-6T model of the reach that was previously developed by MEI for the New Mexico Interstate Stream Commission (NMISC) (MEI, 2007). The calibrated HEC-6T model was completed after the baseline conditions channel stability analysis that is described in Chapter 5 was conducted. The HEC-6T model was used to predict the amount of aggradation/degradation for this study because it is considered a more appropriate model for predicting aggradation/degradation and because of its much shorter computation times compared to FLO-2D. Results from the existing conditions models were used to provide baseline conditions for comparison with the future conditions results for the restoration alternatives, as discussed in Chapter 6.

To facilitate the modeling, a 50-year mean-daily flow record was developed based on flow records at the Central Avenue Gage at Albuquerque for the post-Cochiti Dam period. Since the post-Cochiti Dam period of record is only 30 years in length (WY1974 to WY2004), the additional 20 years of data were developed by repeating the record for WY1985 to WY2004. This period was selected for the extended period because the average mean daily flow was very similar to the longer-term, post-Cochiti average mean daily flow (1,349 cfs for the period from WY1985 to WY2004 versus 1,340 cfs for the entire 30-year period) (**Figure 4.19**).

The HEC-6T model was run over the entire 50-year period, and cross-sectional geometry at 5, 20, 30, and 50 years was evaluated to determine aggradation/degradation changes throughout the reach. Because of the uncertainty in how each specific cross section will change as the aggradation or degradation occurs, the model results were used to estimate a representative change in cross-sectional depth within each segment of the reach that exhibits consistent aggradation/degradation trends in the detailed model results. **Figures 4.20 and 4.21** show the predicted change in cross-sectional area from the model results and the assigned representative changes in channel depths for the 5- and 50-year conditions. The HEC-6T analysis indicates that both aggradational and degradational trends occur along the reach at Year 5. Over time, the aggradational areas shown in Year 5, change to stable or slightly degradational at Years 20 and 30, and there is a slight degradational trend along the entire project reach over the 50-year simulation. The cross sections for the future conditions FLO-2D models were developed by adjusting the existing conditions cross sections to account for the indicated amount of aggradation or degradation by shifting the elevations within the channel banks up or down, as appropriate, to reflect the estimated change in cross-sectional area in each segment of the reach (**Figure 4.22**).

4.5.2. Simulated Hydrology Scenarios

The target restoration flows (Hydrology Scenarios 1 and 2) are simulated for existing and future-year conditions (0, 5, 20, 30 and 50). Hydrology Scenarios 3 and 4 were only modeled for Existing and Year 0 Conditions, and show extensive overbank inundation. It was, therefore, determined that it is not necessary to evaluate the extent of overbank flooding in future years under these scenarios, since these are not the flows targeted for restoration (Table 4.6).

4.5.3. Existing Conditions: Hydrology Scenario 1 (Active Channel Full Flow)

Existing conditions results for the active channel-full flow hydrograph (Hydrology Scenario 1) indicate that the water-surface elevation is at or above the top of bank at several locations along the project reach (Figure 4.15), including:

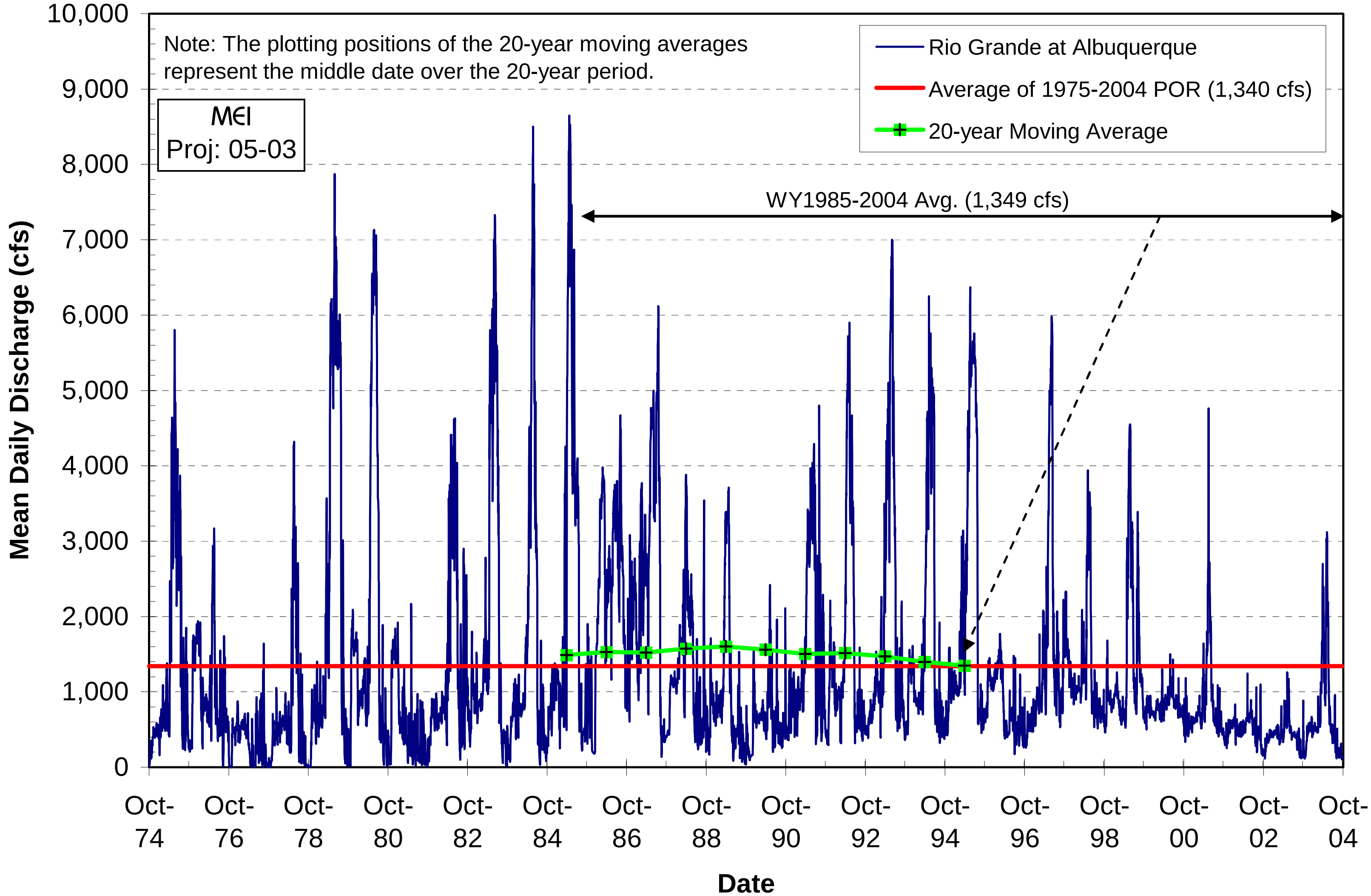

Figure 4.19. Mean daily discharge record at the Rio Grande at Albuquerque Gage showing the overall and 20-year moving averages.

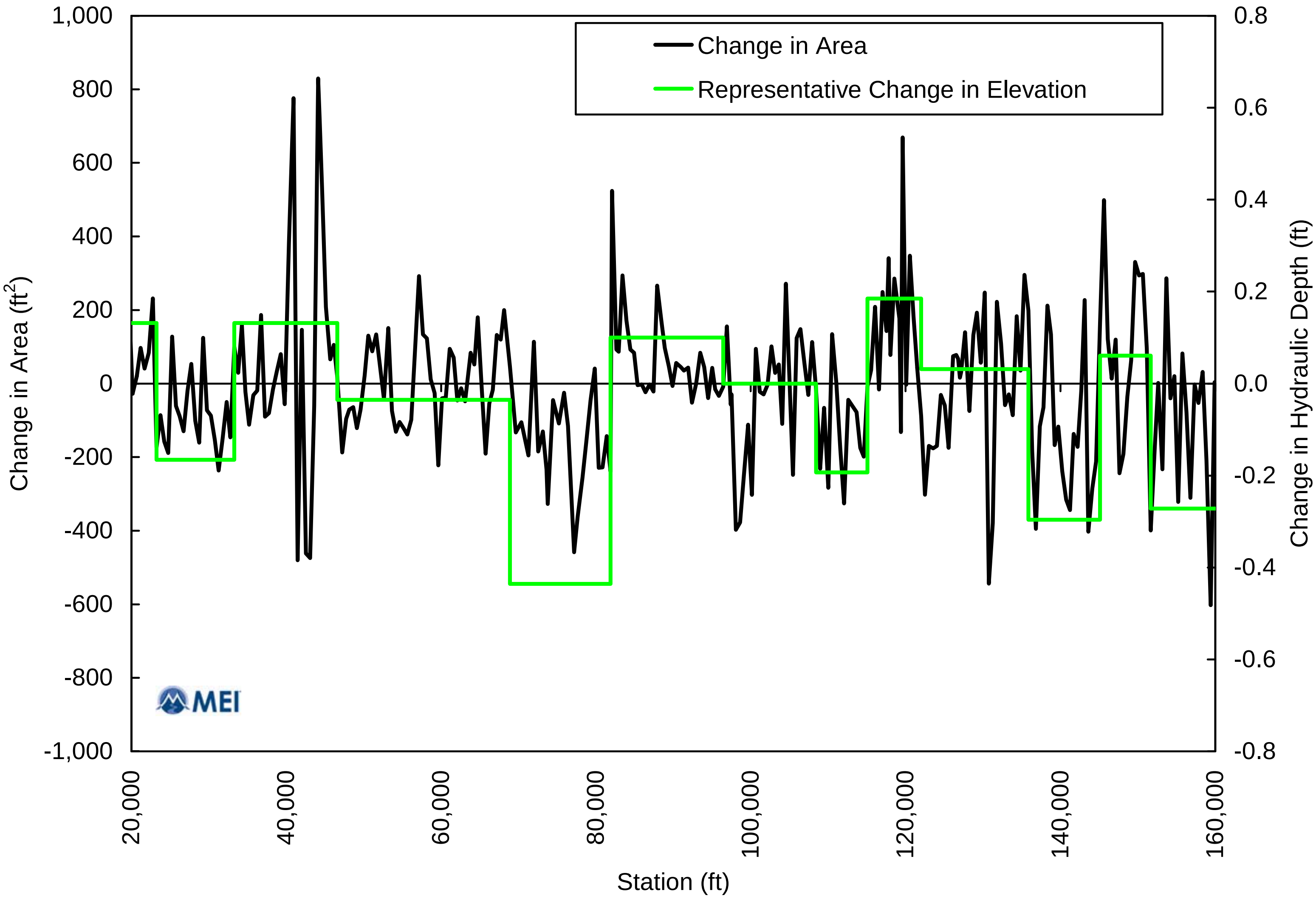

Figure 4.20. Predicted change in channel cross-sectional area at Year 5 and representative change in channel elevation.

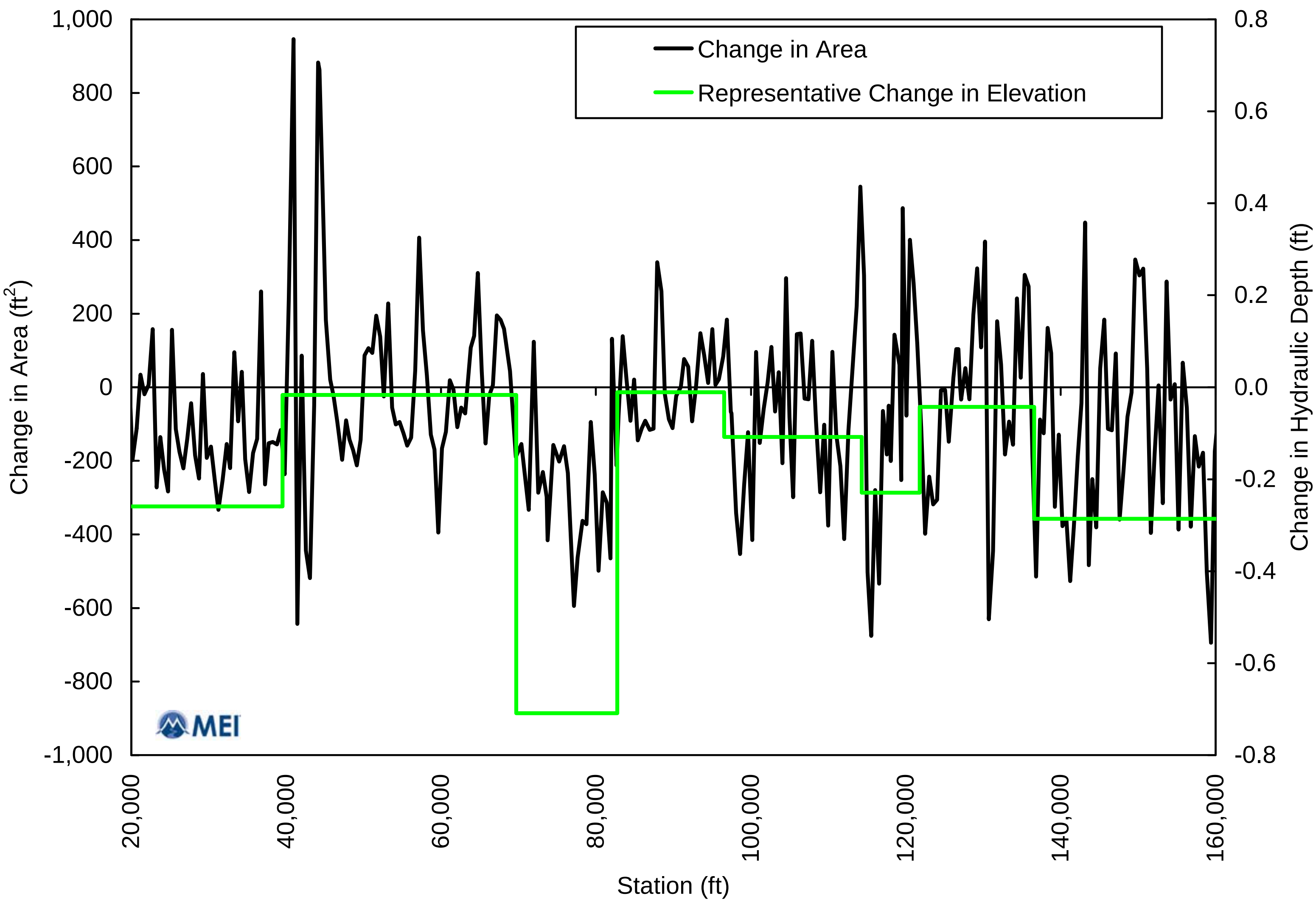

Figure 4.21. Predicted change in channel cross-sectional area at Year 50 and representative change in channel elevation.

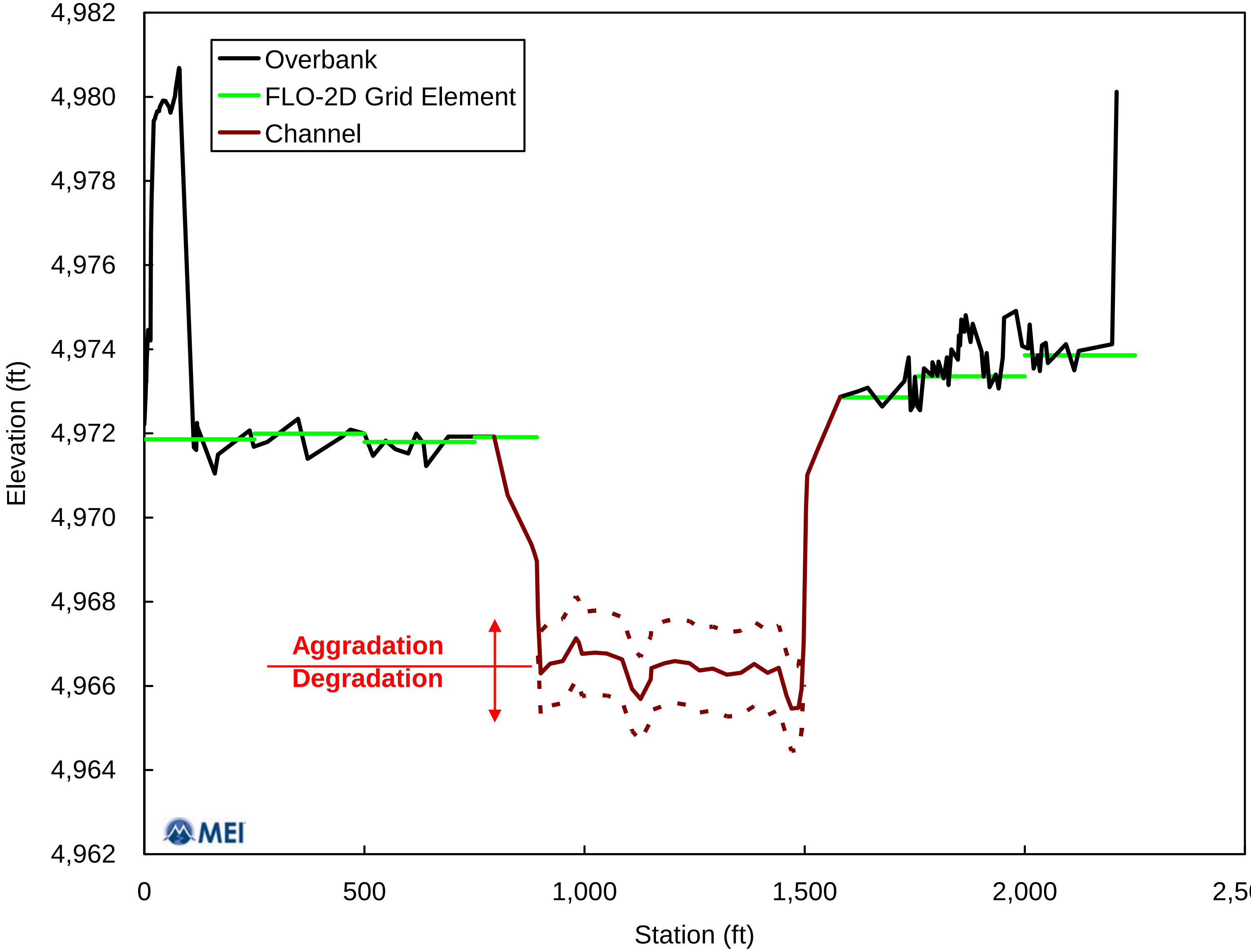

Figure 4.22. Schematic representation of development of the FLO-2D channel cross-sectional geometry for the 5-, 20-, 30-, and 50-year scenarios by applying the representative elevation change.

1. left bank, approximately 4,000 feet upstream from the Central Avenue Bridge (Sta 78,000) (approximately midway between Central Avenue and I-40 Bridges)
2. extensively along the left and right banks from approximately 8,000 feet upstream from the Rio Bravo Bridge to just downstream from the Rio Bravo Bridge,
3. extensively along the left and right banks from approximately 7,000 feet downstream from the South Diversion Channel to just downstream from the I-25 Bridge.

Maps showing the extent of inundation for the channel-full conditions are provided in Appendix A. Inundation areas are color-coded with different shading in 1-foot increments to distinguish depths. The overbank inundation for channel-full flow conditions were mapped using the 30-foot grid resolution and the amount of overbank inundation was summarized for each subreach for Years 0, 5, 20, 30 and 50 (**Table 4.7**).

Table 4.7. Summary of areas of inundation for existing conditions (Years 0, 5, 20, 30 and 50) (acres).

Hydrology Scenario	Description	Future Channel Condition (yr)	Reach					
			1	2	3	4	5	Total
1	Channel Full Conditions	0	77.2	41.3	25.2	34.4	75.6	253.7
		5	78.0	41.1	23.9	34.0	74.0	251.0
		20	76.7	40.9	23.5	32.0	73.5	246.6
		30	76.7	40.7	23.3	32.0	74.6	247.3
		50	75.9	40.7	23.7	30.0	73.6	243.9
2	Annual Spring Runoff	0	45.2	23.1	7.7	4.0	7.9	87.9
		5	45.2	23.0	7.9	4.0	8.0	88.1
		20	43.6	22.1	8.3	6.7	5.7	86.4
		30	43.9	22.8	7.9	7.0	6.3	87.9
		50	43.2	22.3	7.9	6.8	6.1	86.3
3	10,000-cfs Snowmelt Hydrograph	0	181.9	125.6	82.2	233.7	412.9	1,036.3
4	100-Year Peak Snowmelt	0	84.4	59.9	14.6	133.4	364.9	657.2

The overbank areas of inundation were computed based on the area outside of the water-surface margin, defined based on aerial photography that at a flow of 635 cfs at the Albuquerque Gage. Table 4.7 indicates that approximately 77, 41, 25, 34 and 76 acres are inundated in Subreaches 1, 2, 3, 4, and 5, respectively, at Year 0. The extent and maximum depth of inundation for this scenario is shown in Appendix A.1.

The maximum amount of inundation occurs during Year 5, when localized areas of aggradation are predicted along the reach. The total amount of overbank inundation decreases between Years 5 and 50 as the channel becomes slightly degradational along the reach; however, some areas of aggradation occur along the reach between Years 5 and 50. These slightly aggradational areas maintain hydraulic controls throughout the reach that, in turn, maintain relatively consistent water-surface elevations and similar amounts of overbank inundation compared to existing conditions, even though the channel is slightly degradational in some locations.

4.5.4. Existing Conditions: Hydrology Scenario 2 (Annual Spring Runoff Hydrograph)

The maximum computed water-surface elevations during the average annual hydrograph (Hydrology Scenario 2) indicate that the top-of-bank elevation is exceeded at two locations along the project reach (Figure 4.16): (1) along the left bank 1,500 feet downstream from Bridge Street (Sta 62,000), and (2) at a channel contraction located approximately 8,000 feet downstream from the South Diversion Channel (Sta 32,400). Very little overbank inundation occurs under Hydrology Scenario 2, because the peak discharge of 3,770 cfs is substantially less than the channel capacity along the majority of the reach. Table 4.6 indicates that approximately 45, 23, 8, 4 and 8 acres are inundated in Subreaches 1, 2, 3, 4, and 5, respectively, at Year 0. The inundation occurs around the margins of the banks and reflects the inundation that occurs in small embayments adjacent to the channel, not overtopping of the channel banks. The amount of inundation remains relatively consistent for the Years 5, 20, 30 and 50 conditions, with the maximum total inundation occurring at Year 5 and the minimum inundation occurring at Year 50. The extent and maximum depth of inundation for the Year 0 scenario is shown in Appendix A.2 and the duration of inundation is shown in Appendix A.3.

4.5.5. Existing Conditions: Hydrology Scenario 3 (10,000-cfs Hydrograph)

The maximum computed water-surface elevations during the 10,000 cfs snowmelt hydrograph (Hydrology Scenario 3) indicates that overbank inundation occurs at similar locations to the channel full hydrograph, but with larger areas of inundation (Figure 4.17). Additional overbank inundation areas occur downstream from the Corrales Siphon. Significant inundation areas include the following:

1. extensive inundation along the left bank from Corrales Siphon to just downstream from the North Diversion Channel.
2. left bank, approximately 4,000 feet upstream from the Central Avenue Bridge (Sta 78,000) (approximately midway between Central Avenue and I-40 Bridges) to midway between Central Avenue and Bridge Street Bridges (Sta 66,000).
3. extensively along the left and right banks from approximately 8,000 feet upstream from the Rio Bravo Bridge (Sta 56,000) to just downstream from the Rio Bravo Bridge (Sta 46,000).
4. extensive inundation along the left and right banks from the South Diversion Channel (Sta 41,000) to the downstream end of the project reach (Sta 16,000).

Under Hydrology Scenario 3, approximately 1,036 of the 5,840 acres of available floodplain (about 18 percent) are inundated during the hydrograph. The extent, maximum depth and duration of inundation for this scenario are shown in Appendices A.4 and A.5.

4.5.6. Existing Conditions: Hydrology Scenario 4 (100-year Snowmelt Hydrograph)

Based on the maximum computed water-surface elevations during the 100-year snowmelt hydrograph (Hydrology Scenario 4), overbank inundation occurs at similar locations to the 10,000 cfs hydrograph (Figure 4.18), but with less total area of inundation. Under this scenario, in which the peak discharge is about 7,750 cfs, approximately 660 of the 5,840 acres of available floodplain (about 12 percent) is inundated during the hydrograph. The majority of the overbank inundation occurs for approximately 14 to 16 days during the 3-month hydrograph. The extent, maximum depth and duration of inundation for this scenario are shown in Appendices A.6 and A.7.

5. SEDIMENT-CONTINUITY ANALYSIS

A baseline sediment-continuity analysis was performed to evaluate the potential for aggradation or degradation in response to both individual short-term hydrographs and longer-term flows (50-year project life) with the present channel configuration and reservoir operations. In general, the analysis was conducted by estimating the bed-material transport capacity of the supply reach and each subreach within the study area for each hydrology scenario and comparing the resulting capacity with the supply from the upstream river and tributaries within the reach. For this analysis, Hydrology Scenarios 2, 3 and 4 (mean annual runoff, 10,000-cfs, and 100-year snowmelt hydrographs, respectively) were used for the individual hydrographs, and the mean daily flow-duration curve from the Albuquerque gage for the post-Cochiti Dam period was used for the long-term analysis.

To facilitate the analysis, bed-material transport capacity rating curves were developed for each subreach using hydraulic output from the 500-foot grid FLO-2D model, representative bed-material gradations and the Yang (Sand) sediment-transport equation (Yang, 1973). 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 in the project reach, and determined that this equation produced results that were the most consistent with the available measured data at the Rio Grande gages downstream from Cochiti Dam among the available equations. The sediment-transport rating curves were then integrated over the individual hydrographs or the flow-duration curve to obtain a transport capacity volume for each hydrology scenario. In comparing the volumes, when the transport capacity of a particular subreach exceeds the supply, the channel will respond by either degrading (i.e., channel downcutting) or coarsening its bed material, and when the supply exceeds the capacity, the channel will respond by aggrading or fining its bed material. It should be noted, however, that significant amounts of downcutting or aggradation can also lead to lateral instability. The upstream supply reach used for this study extends from the upstream limit of the project reach to Arroyo de la Baranca (located approximately 2 miles downstream of Bernalillo), a distance of approximately 29,000 feet.

The representative bed-material gradations used in the analysis were taken from MEI (2004), with the gradation for URGWOPS Subreach 12a (Bernalillo to Rio Rancho Wastewater Treatment Plant) representing the supply reach and Subreach 12b (Rio Rancho to Isleta Diversion Dam) representing the primary study reach for this project (**Figure 5.1**). These gradations were developed using data collected by the BOR and USGS after 1990 and by MEI for various studies in 2002 and 2003. Observations by the BOR indicate that fine material that is not characteristic of the typical bed material that controls the form of the channel tends to accumulate as a veneer over the primary bed material during the non-runoff season but is removed during the runoff season. To avoid biasing the results to this finer material, the data sets were restricted to samples that were collected between May 1 and August 31 because this is the period of highest flows when the fine material is not likely present.

The bed-material gradations for the supply reach were based on a previous analysis of bed-material data collected at BOR Rangelines BB340 and BB345 in May 2001 (MEI, 2004). These data were used to develop a representative bed material gradation for Subreach 12a that is located between Bernalillo and Rio Rancho (**Figure 5.2**). The data set for the primary project reach consisted of 17 bed-material samples collected by the USGS at the Albuquerque gage between 1990 and 1996, and 16 samples collected by the BOR at Rangelines CA-1 to CA-13, A-1, A-4, A-6, and CR355, CR378 and CR443 between 1998 and 2001. The BOR data typically included several surface bed-material measurements along each range line. As a result, the

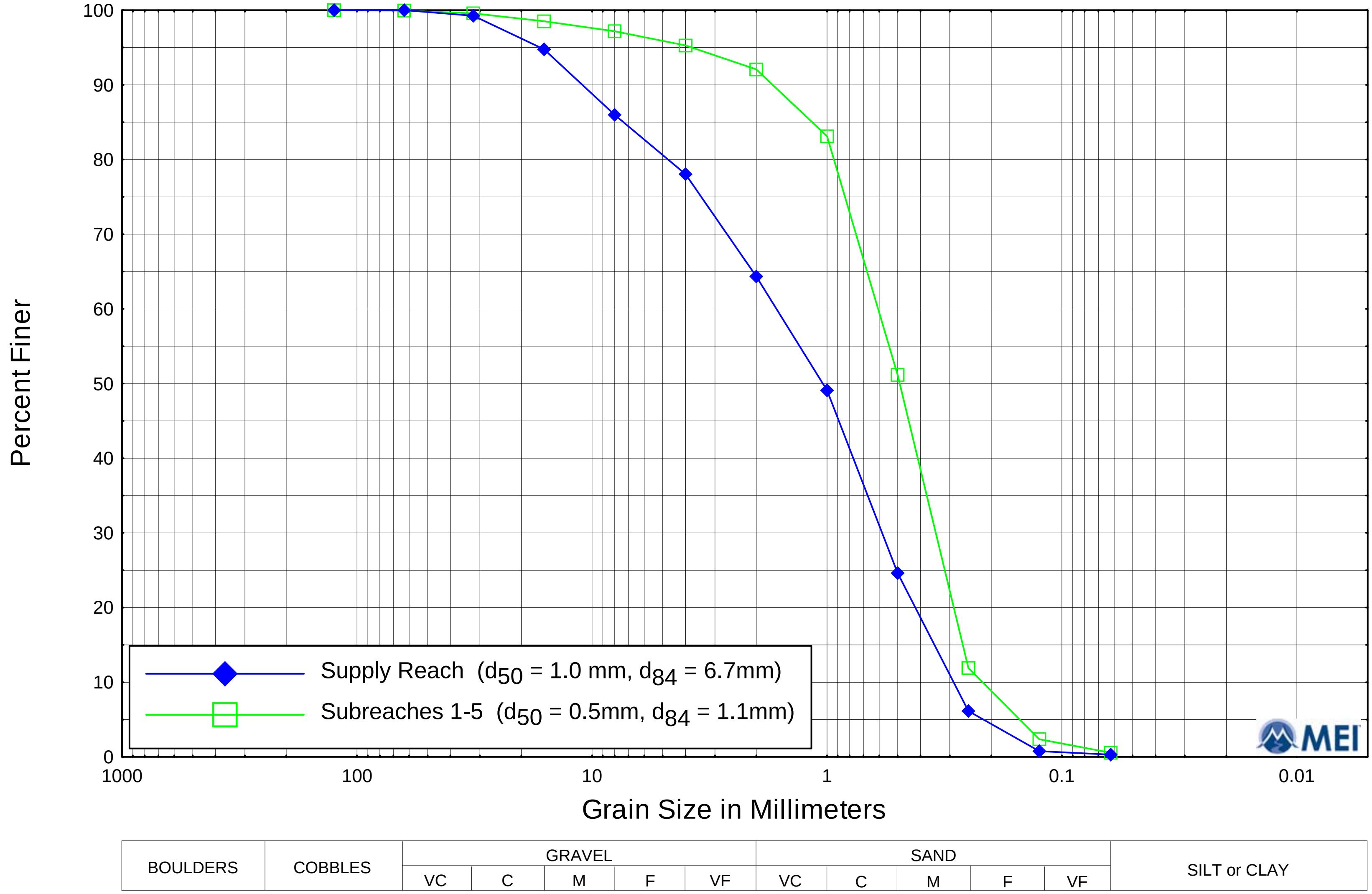

Figure 5.1. Representative bed-material gradation curve for the project reach that was used in the sediment-continuity analysis.

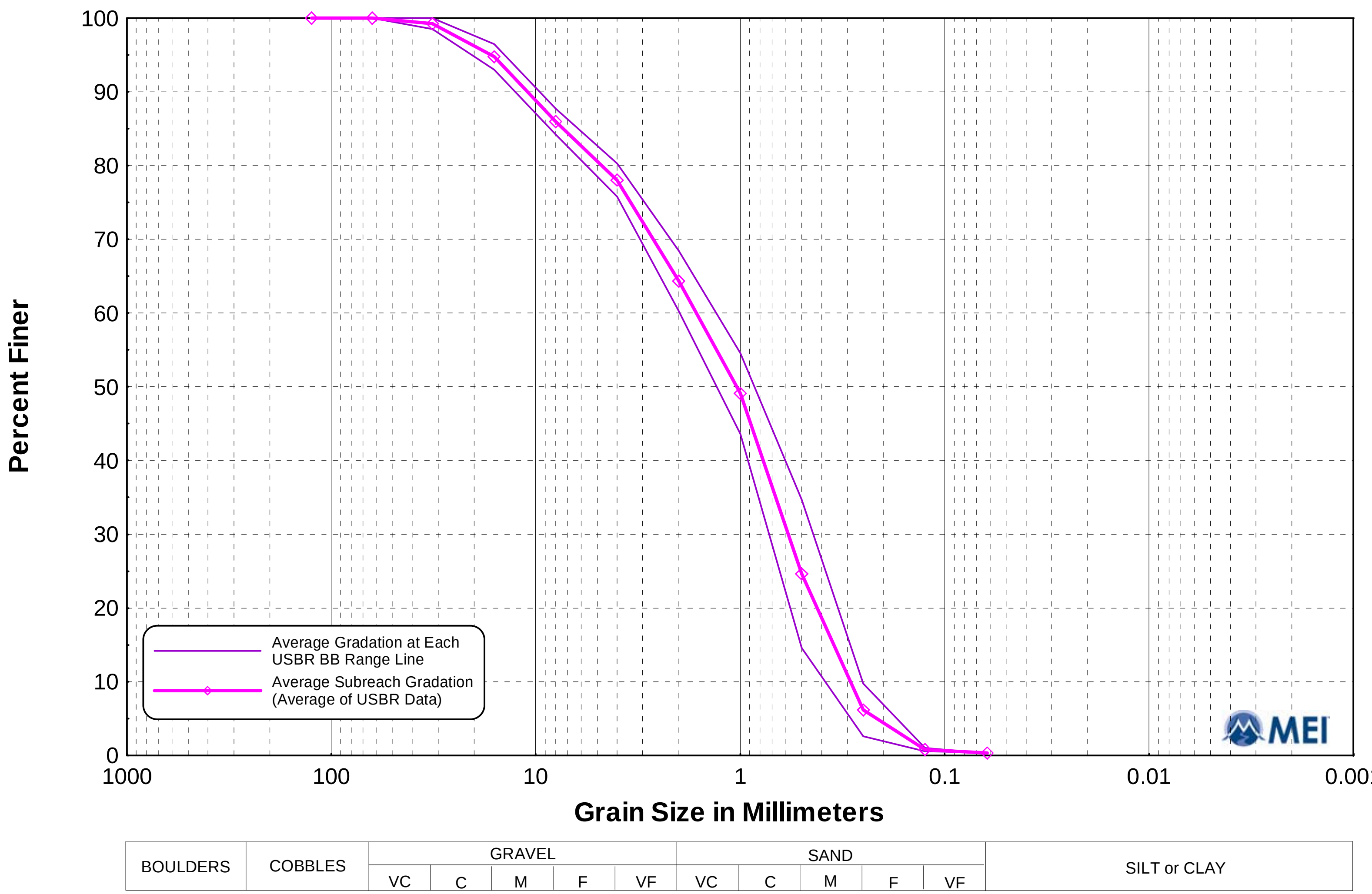

Figure 5.2. Representative bed-material gradation curve for the supply reach that was used in the sediment-continuity analysis.

samples collected at each range line were averaged to represent a single measurement location. The USGS samples also include several surface bed-material measurements collected along the cross section where their discharge measurements were collected. Similar to the BOR data, the samples collected along the cross section were averaged to represent a single measurement location. The project reach data set also included three bulk samples collected by MEI in July 2003 from exposed channel bars between Interstate 40 and Montano Boulevard that are representative of the surface bed material in this reach (MEI, 2003).

The supply reach gradation has a median size of about 1 mm (coarse sand), contains material up to about 128 mm, and about 42 percent of the material is in the gravel- and cobble-size range (Figure 5.1). The gradation for the primary project reach has a median size of 0.5 mm (medium and coarse sand), contains material up to about 32 mm, and about 92 percent of the material is sand.

To validate the general approach for estimating the transport capacity rating curves, a bed-material rating curve was developed using hydraulic results from the FLO-2D model for the main channel at Albuquerque gage and compared to measured values at the gage (**Figure 5.3**). The resulting rating curve is consistent with the measured data, indicating that the approach is appropriate. Rating curves based on the reach-averaged hydraulics for each of the subreaches are shown in **Figure 5.4**.

5.1. Tributary Bed-material Contributions

Three tributaries (Calabacillas Arroyo, North Diversion Channel, and South Diversion Channel) were identified along the study reach that have the capability to deliver significant quantities of sediment to the Rio Grande (**Table 5.1**). Sediment loads from the North Diversion Channel (NDC) were obtained from a study performed by the USACE Waterways Experiment Station (WES) to evaluate sedimentation conditions in the NDC (Copeland, 1995). The basic sediment supply information used by Copeland (1995) was developed from a study of the arroyos draining to the NDC that was performed by Mussetter and Harvey (1993). Due to the lack of available data for Calabacillas Arroyo and the South Diversion Channel (SDC), annual bed-material loads were estimated by assuming a unit bed-material supply of 0.1 ac-ft/mi^2, which is generally consistent with the range of unit yields from the tributaries for which information is available. Calabacillas Arroyo, the NDC and the SDC are ephemeral channels that flow in response to rainfall events. Historically, significant floods from Calabacillas Arroyo have formed a large fan at the confluence with the Rio Grande that have fully or partially blocked the river at various times. Large magnitude events in the arroyo, such as the 1941 and 1988 floods, caused the Calabacillas Arroyo fan to prograde into the Rio Grande. Development of the watershed, channelization of Calabacillas Arroyo and construction of Swinburne Dam (completed in 1991) has likely reduced the sediment load to the Rio Grande.

5.2. Sediment-continuity Analysis Results

Integration of the transport capacity rating curves over the mean annual hydrograph results in a transported volume through the study reach of about 100 ac-ft of sediment (Figures 5.5). The transported volume increases to about 450 ac-ft and 630 ac-ft for the 10,000-cfs and 100-year snowmelt hydrographs, respectively (**Figures 5.6 and 5.7**). Based on integration of the annual flow-duration curve, the long-term, average annual bed-material load through the study reach is about 240 ac-ft (**Figure 5.8**). (This value is higher than obtained for the mean annual hydrograph because the flow-duration curve includes flows that significantly exceed the mean annual flood peak.)

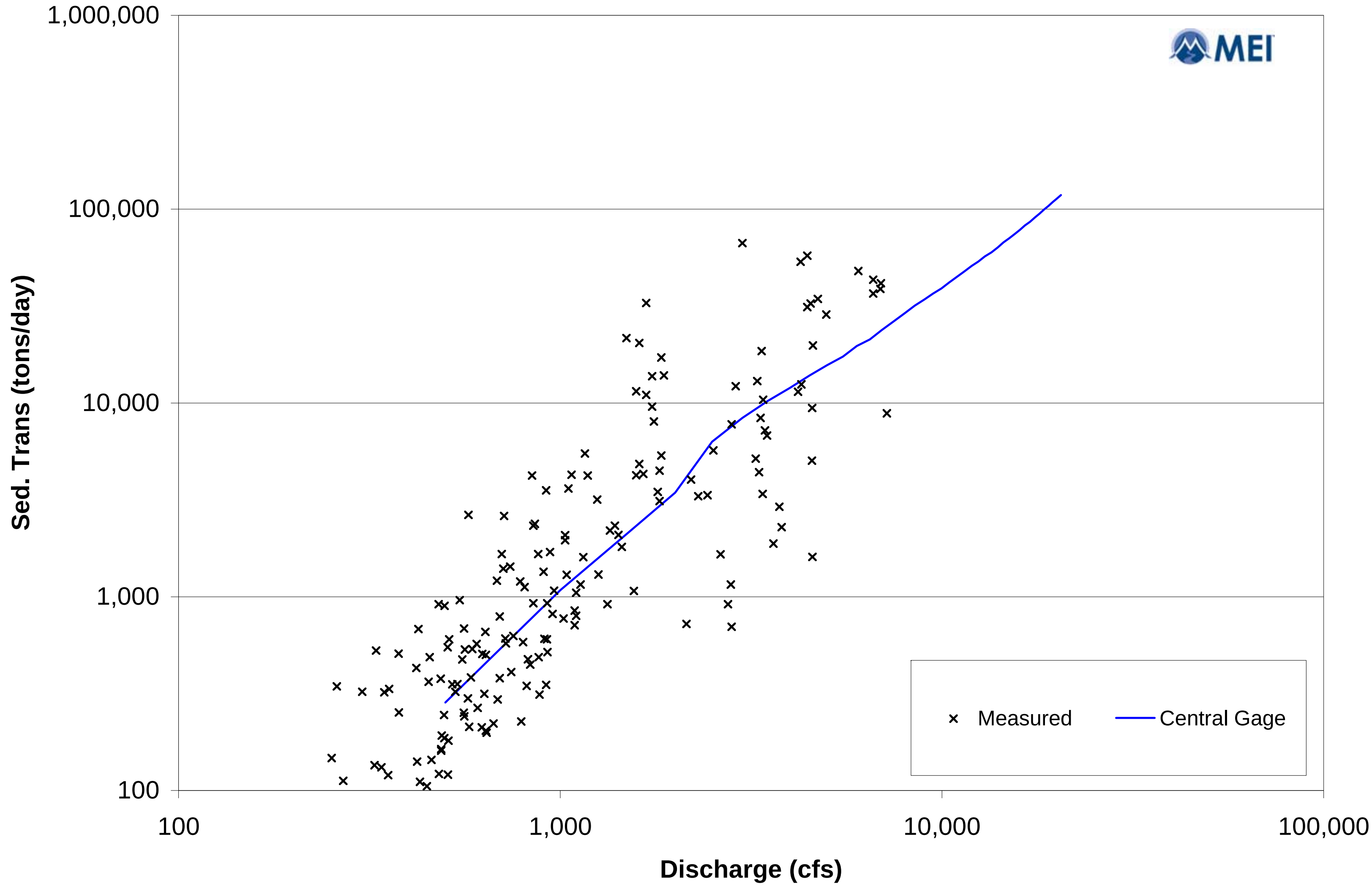

Figure 5.3. Bed-material rating curve at the Albuquerque gage developed using the Yang (Sand) (1973) relationship and measured bed-material loads at the Albuquerque gage.

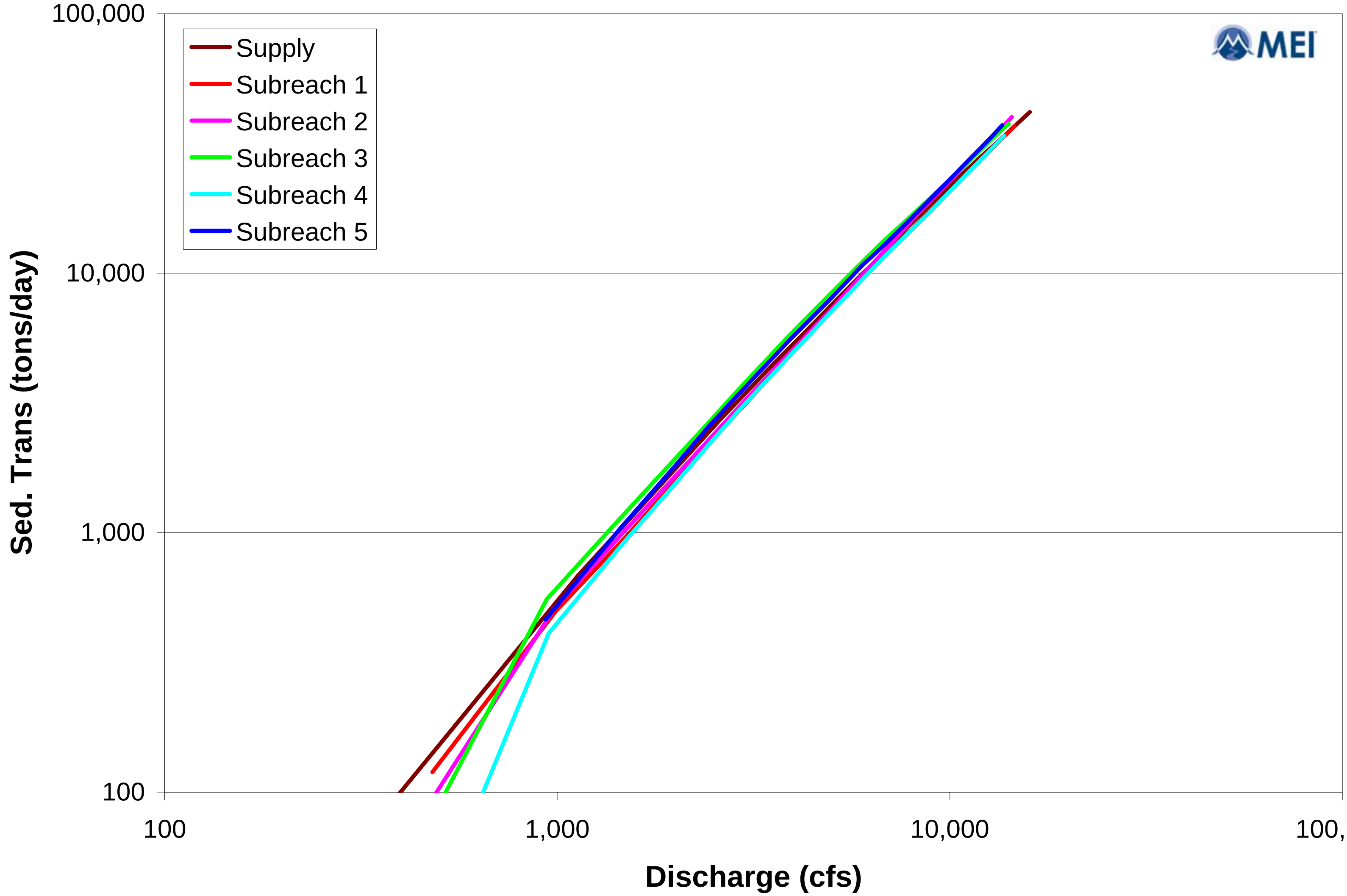

Figure 5.4. Bed-material rating curves for each of the subreaches in the sediment-continuity analysis.

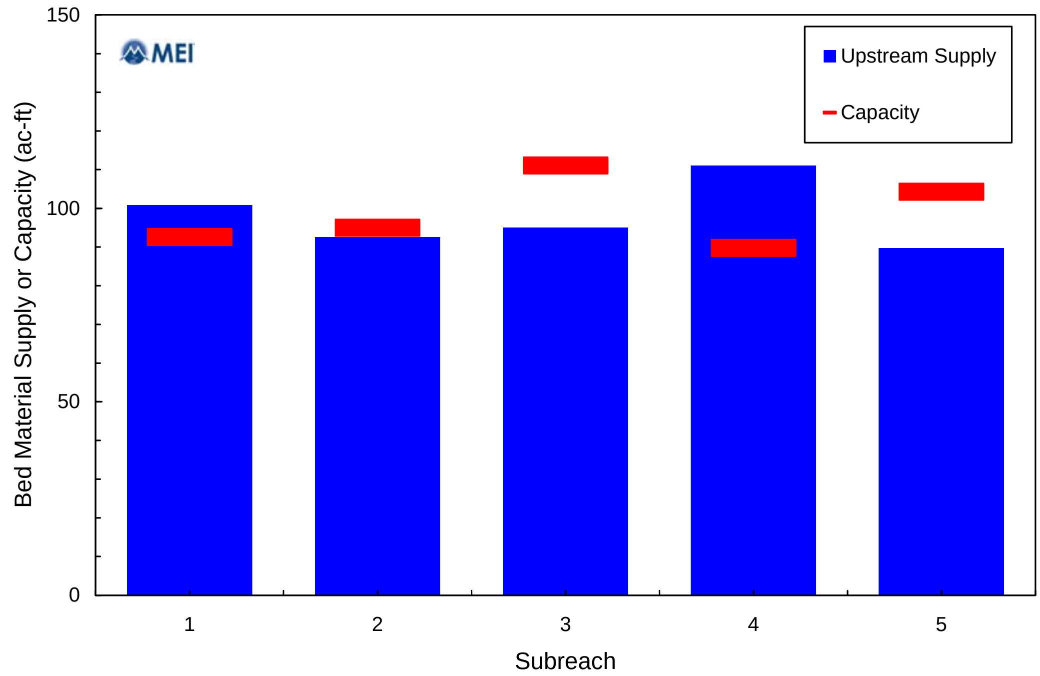

Figure 5.5. Comparison of average annual supply and bed-material transport capacity for each subreach.

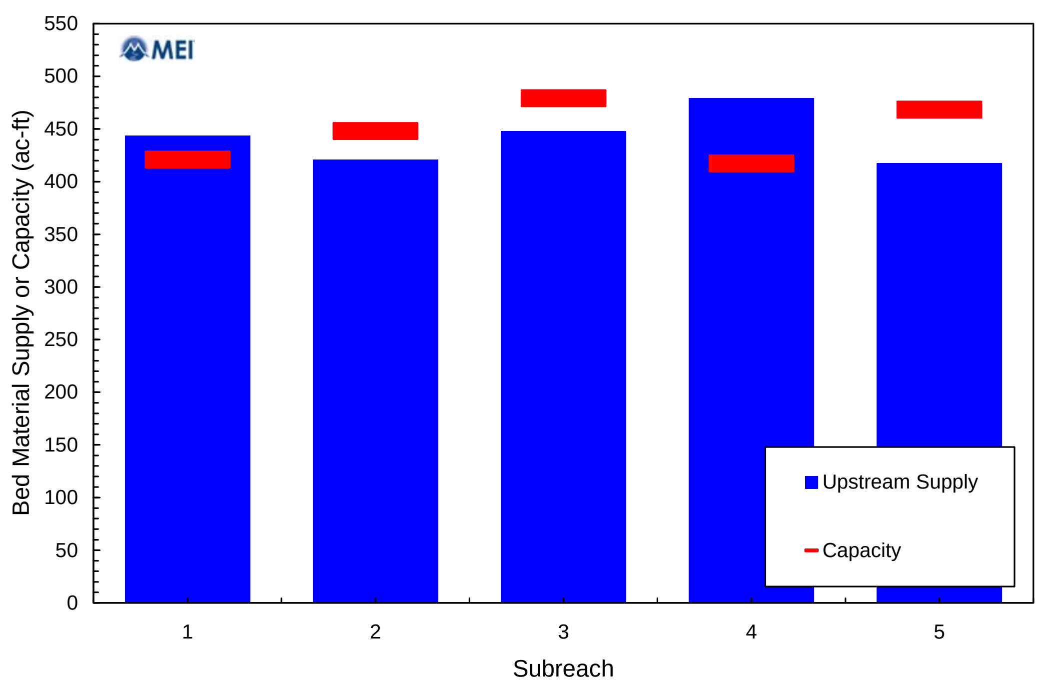

Figure 5.6. Comparison of supply and bed-material transport capacity for each subreach for the 10,000-cfs hydrograph.

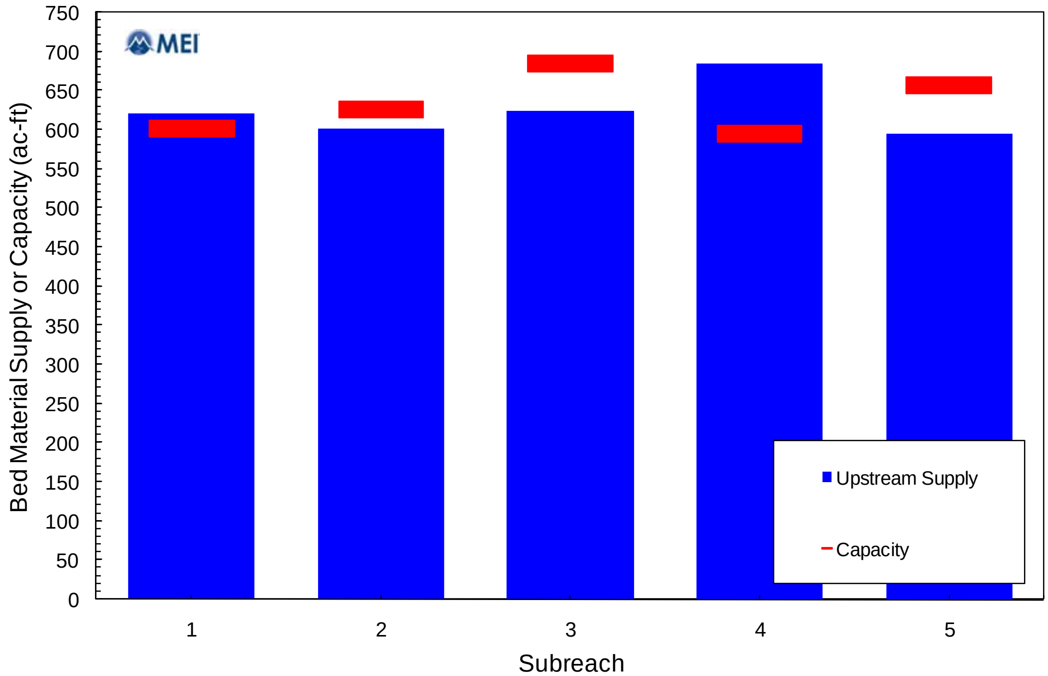

Figure 5.7. Comparison of supply and bed-material transport capacity for each subreach for the 100-Year snowmelt hydrograph.

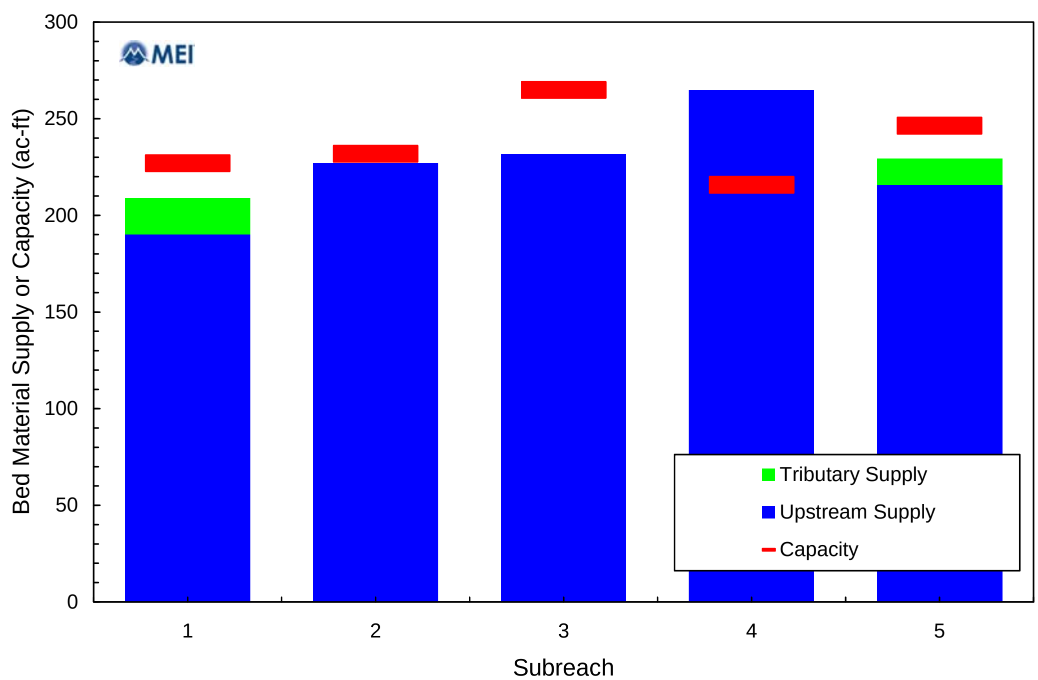

Figure 5.8. Comparison of supply and bed-material transport capacity for each subreach for the flow-duration curve.

Table 5.1. Summary of tributaries included in the sediment-continuity analysis, and the average annual bed-material contribution from each of the tributaries (modified from MEI (2004).

Tributary Name	Drainage Area (mi^2)	Average Annual Sediment Volume (ac-ft)	Unit Volume (ac/mi^2)	Source
Calabacillas Arroyo	100.8	10.1	0.10	Assumed 0.1 ac-ft/mi^2
North Diversion Channel	102	8.3	0.08	Copeland (1995)
South Diversion Channel	133	13.3	0.10	Assumed 0.1 ac-ft/mi^2

The results shown in Figures 5.5 through 5.8 indicate that the bed-material transport capacity is relatively consistent from subreach to subreach, although there is a slight net degradational tendency, in the absence of tributary sediment inputs, for the overall study reach for all three of the individual storm hydrographs that were analyzed. For the average annual hydrograph, the transport capacity at the downstream end of the reach is about 104 ac-ft compared to the upstream supply of about 101 ac-ft (Figure 5.5). For the 10,000-cfs hydrograph, the transport capacity at the downstream end is about 468 ac-ft capacity versus 444 ac-ft of supply (Figure 5.6), and the 100-year snowmelt hydrograph, the downstream capacity is about 657 ac-ft capacity at the downstream end versus 622 ac-ft of supply (Figure 5.7). (Note that tributary inputs were not considered for the mean annual, 10,000-cfs and 100-year snowmelt hydrographs because storms in the tributaries will most likely occur during the monsoon season in late-summer and early-fall, while the large runoff hydrographs in the river typically occur during the spring snowmelt runoff period.) On a long-term average annual basis, the transport capacity at the downstream end of the reach is about 246 ac-ft compared to the supply of 209 ac-ft (Figure 5.8).

In spite of the overall degradational tendency, Subreach 4 tends to be aggradational for all of the hydrology scenarios. Over time, the upstream Subreaches 1, 2 and 3 will probably respond to the deficit by coarsening of the bed material as these subreaches approach a balance between the supply and capacity. The coarsening will decrease the supply to Subreach 4 which will bring this reach into closer balance between the supply and capacity, reducing the aggradation potential.

The approximate change in bed elevation (i.e., aggradation/degradation potential) associated with these differences in volume were estimated by dividing the difference between the bed material supply and capacity of the subreach by the surface area of the channel, based on the product of the subreach length and channel topwidth (Table 4.4). In evaluating this information, it is important to note that the actual changes will not occur uniformly throughout the reach or across the channel at any given location, nor will they continue progressively for a long period of time because the bed material, channel geometry and gradient will adjust to compensate for imbalances between the sediment supply and transport capacity. In spite of this limitation, the analysis provides a reasonable basis for comparing results from the sediment-continuity analysis.

For the average annual hydrograph, Subreaches 1 and 4 are net aggradational (average of 0.04 and 0.05 feet, respectively) with no tributary inputs (**Figure 5.9**). Subreach 2 is approximately in balance with the upstream supply (-0.01 feet) and Subreaches 3 and 5 are net degradational (average depth of -0.06 and -0.04 feet, respectively). For the 10,000-cfs hydrograph,

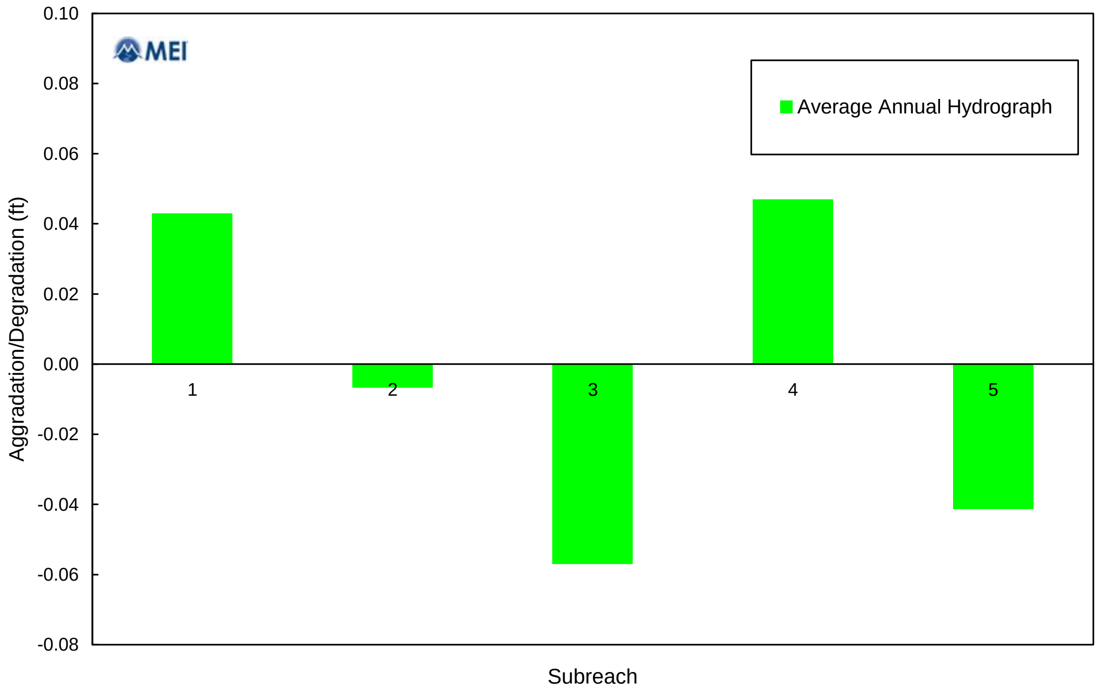

Figure 5.9. Computed average annual aggradation/degradation depths for each subreach.

Subreaches 1 and 4 are net aggradational (both have an average of 0.13 feet) with no tributary inputs (**Figure 5.10**). Subreaches 2, 3 and 5 are net degradational (average of -0.07, -0.11, and -0.15 feet, respectively) in the absence of tributary inputs. For the 100-year snowmelt hydrograph, Subreaches 1 and 4 are net aggradational (average of 0.12 and 0.19 feet, respectively t) with no tributary inputs (**Figure 5.11**). Subreaches 2, 3 and 5 are net degradational (average of -0.07, -0.21, and -0.18 feet, respectively) in the absence of tributary inputs. On a long-term, average annual basis, Subreaches 1, 3 and 5 are net degradational (average of -0.11, -0.11, and -0.05 feet, respectively). Subreach 2 is approximately in balance with the upstream supply (-0.01 feet, on average) and Subreach 4 is net aggradational (average of about 0.13 feet) with tributary inputs (**Figure 5.12**).

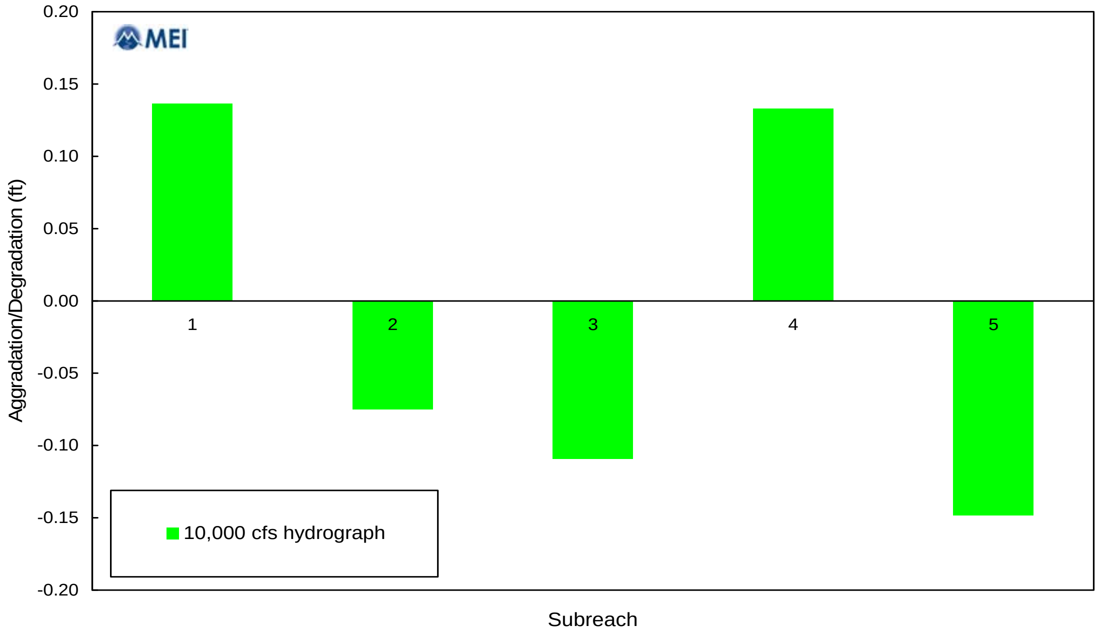

Figure 5.10. Computed aggradation/degradation depths for each subreach for the 10,000-cfs hydrograph.

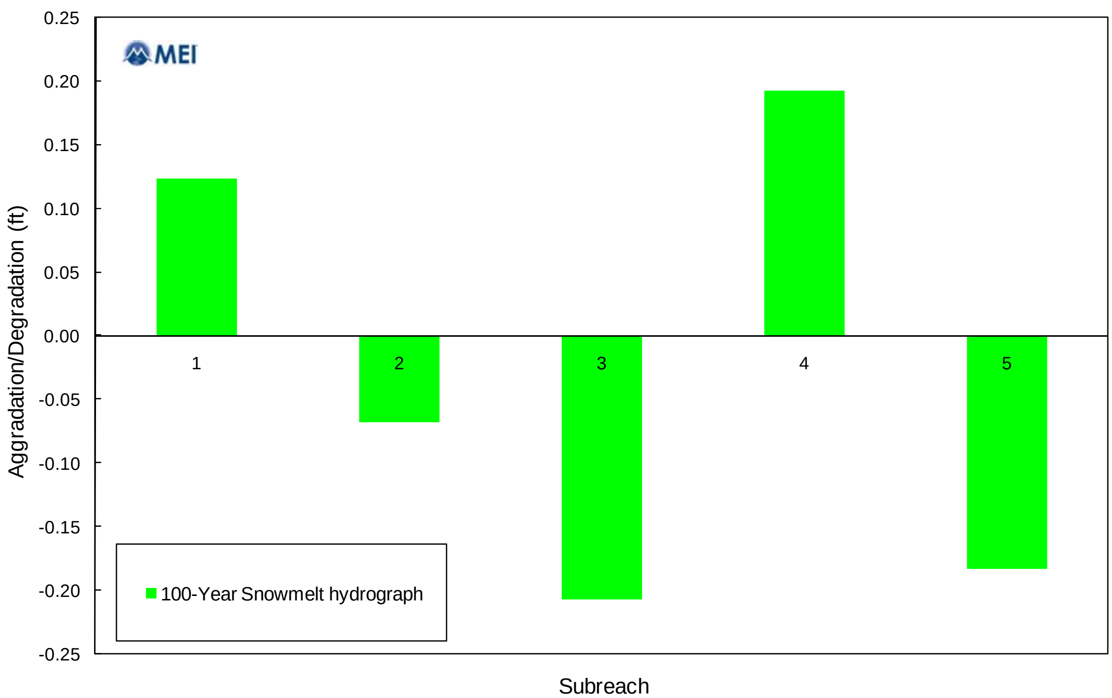

Figure 5.11. Computed aggradation/degradation depths for each subreach for the 100-year snowmelt hydrograph.

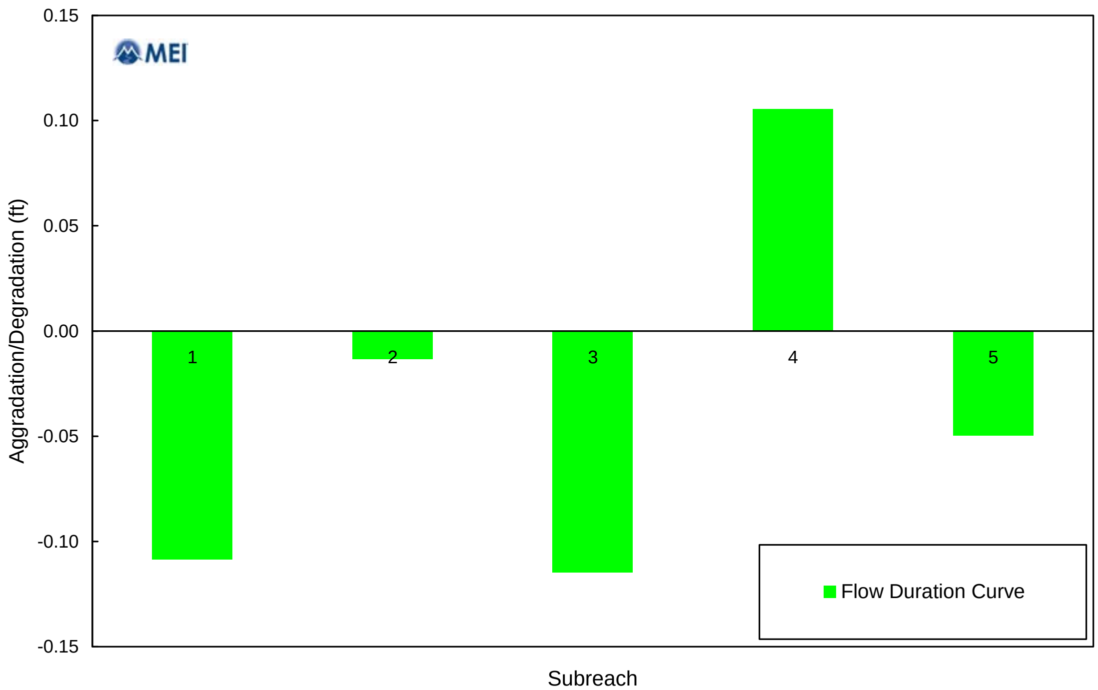

Figure 5.12. Computed aggradation/degradation depths for each subreach for the flow-duration curve.

6. HYDRAULIC MODELING OF RESTORATION ALTERNATIVES

FLO-2D modeling was conducted using the 250-foot grid resolution model to evaluate depth, extent and duration of overbank inundation for five restoration alternatives provided by the USACE (Maximum Effort, Minimum Effort, Moderate Effort, Moderate Effort-A and Moderate Effort-B). The analysis was conducted for initial channel conditions immediately after construction of the project (Year 0) and four (4) future channel conditions (5-, 20-, 30- and 50-years after construction of the restoration features) (**Table 6.1**). Results from these simulations were then compared with the existing conditions model results to assess the effects of the alternatives. As indicated in Table 6.1, the inundation mapping was prepared only for Year 0 conditions. The evaluations for this and the other future-year conditions were made based on the numerical results.

The target restoration flows (Hydrology Scenarios 1 and 2) are simulated in Restoration Alternatives 1 through 5 for the future year conditions (0, 5, 20, 30 and 50). Hydrology Scenario 3 was not simulated for the restoration alternatives since the existing conditions results show extensive overbank inundation and the hydrology analysis of the 100-year flood performed by the Corps (HEC, 2006) indicated that the peak of the 100-year flood (7,750 cfs) would be regulated by the current operating releases from Cochiti Reservoir. Hydrology Scenario 4 was simulated for Restoration Alternatives 1, 2 and 3 for Year 0 conditions.

6.1. Model Development—Restoration Alternatives

The 250-foot grid Existing Conditions model was modified to represent each of the restoration alternatives by making appropriate adjustments to the main channel cross-sectional geometry, overbank grid elevations, and roughness parameters. Alternative 1, which is referred to as the "Maximum Effort" alternative, contains all of the channel and overbank features that were considered in formulating the alternative. Alternatives 2, 3, 4 and 5 are termed the "Minimum Effort", "Moderate Effort", and "Moderate Effort-A" and "Moderate Effort-B", respectively, and they were developed using various combinations of the channel and overbank features that were included in Alternative 1.

In developing the restoration alternatives, the USACE identified the following five categories of features:

1. Water Features (300 cfs)
2. Water Features (3,500 cfs)
3. Bank Destabilization
4. Swale Trench Excavation
5. Overbank Treat-Retreat-Revegetation

These features were delineated in their proposed spatial locations on the project mapping and provided to MEI in ArcGIS shape file format. MEI overlaid the features onto the FLO-2D grid in ArcGIS to determine the grid elements to be modified. An example overlay map showing the delineated features is shown in **Figure 6.1**.

Table 6.1. FLO-2D model runs for the five Restoration Alternatives.					
Alternative	Description	Future Channel Condition	Hydrology Scenario 1 (Channel Full Flow)	Hydrology Scenario 2 (Annual Snowmelt Hydrograph)	Hydrology Scenario 4 (100-year Hydrograph)
1	Maximum Effort	Year 0	Model and Map	Model and Map	Model and Map
		Year 5	Model only	Model only	
		Year 20	Model only	Model only	
		Year 30	Model only	Model only	
		Year 50	Model only	Model only	
2	Minimal Effort	Year 0	Model and Map	Model and Map	Model and Map
		Year 5	Model only	Model only	
		Year 20	Model only	Model only	
		Year 30	Model only	Model only	
		Year 50	Model only	Model only	
3	Preferred Effort	Year 0	Model and Map	Model and Map	Model and Map
		Year 5	Model only	Model only	
		Year 20	Model only	Model only	
		Year 30	Model only	Model only	
		Year 50	Model only	Model only	
4	Moderate Effort-A	Year 0	Model and Map	Model and Map	
		Year 5	Model only	Model only	
		Year 20	Model only	Model only	
		Year 30	Model only	Model only	
		Year 50	Model only	Model only	
5	Moderate Effort-B	Year 0	Model and Map	Model and Map	
		Year 5	Model only	Model only	
		Year 20	Model only	Model only	
		Year 30	Model only	Model only	
		Year 50	Model only	Model only	
Key					
Model and Map		Modify FLO-2D model, run hydrograph, prepare 30-foot water-surface DTM, overlay on 30-foot ground DTM			
Model only		Modify FLO-2D model, run hydrograph, analyze inundation areas based on 250-foot model grid			

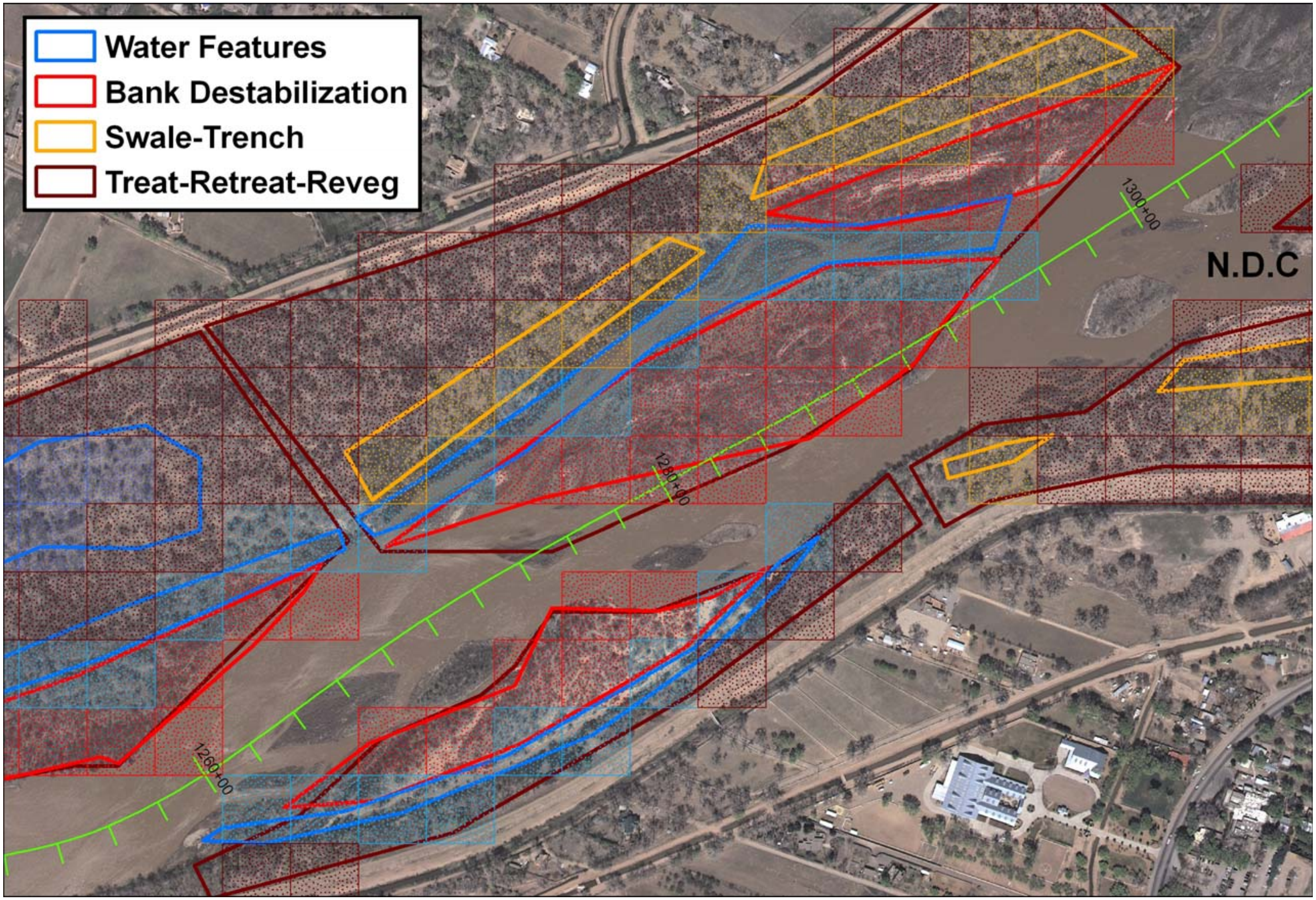

Figure 6.1. Example of delineated FLO-2D grid elements used to represent the restoration alternatives in vicinity of the North Diversion Channel.

The Water Features (300 cfs) represent ponds that are disconnected from the main channel and embayments that are directly connected to the main channel. The 300-cfs designation represents the lowest elevation of the feature that corresponds to the channel water-surface elevation adjacent to the feature at a discharge of 300 cfs. **Figure 6.2** contains a schematic cross section showing the modifications to the existing conditions FLO-2D grid elevations to represent the delineated channel and overbank restoration features

In cases where the restoration features encompass more than one grid element, the grid elevations representing the features are sloped in the downstream direction to match the water-surface slope. The pond features are designed to the 300-cfs water-surface elevation and are intended to be sufficiently low to be hydraulically connected to the groundwater. The embayment features are typically located at existing drain returns, and were designed to connect the river to the drains. In addition to changing the grid elevation to represent the 300-cfs water features, the banks of the channel cross sections were lowered to the grid elevation to ensure that flows would be conveyed to the embayment features.

The Water Features (3,500 cfs) are typically high-flow channels that follow historic high-flow paths in the overbanks. Based on guidance from the USACE, the grid elevations identified for these features were lowered 1-foot below the corresponding 3,500-cfs water-surface elevation. The channel cross sections at the up- and downstream ends of the features were also lowered to ensure that water would be conveyed from the channel into the features at the upstream end and from the overbank features back to the channel at the downstream end.

The bank destabilization features are connected directly to the river and were designed to provide habitat along the channel margins. The bank destabilization features were incorporated into the FLO-2D model by lowering the FLO-2D grid and bank elevations at the corresponding channel cross sections to the 3,500-cfs water-surface elevation.

In some areas, channel widening associated with the embayments and bank destabilization features causes the channel water-surface elevations along the reach to decrease compared to the existing conditions. As a result, an iterative procedure was used to ensure that the designed restoration features are inundated at the desired 3,500 cfs discharge. The iteration procedure was conducted by running the Year 0 restoration alternatives at a discharge of 3,500 cfs, and comparing the resulting water-surface elevation to the elevation of the design feature. If the difference between the design elevation and the predicted water-surface elevation was greater than approximately 0.05 feet, then the elevation of the design feature was adjusted to the new predicted water-surface elevation, and the simulation was re-run. Typically, only one iteration was required for the design and water-surface elevations to converge within the specified tolerance.

The swale-trench features are low-elevation features in the overbanks, designed to be connected to the groundwater. They are not hydraulically connected to the main channel when flows are sufficiently low to be contained within the main channel; therefore, no cross-section changes were made for these features.

The overbank-treat-retreat-revegetation features represent the ongoing fuel reduction and non-native vegetation removal program that is being conducted by the USACE. These programs involve clearing and re-vegetation of the overbanks. These features are represented in the FLO-2D model by adjusting the overbank roughness of the grid elements (**Table 6.2**). No elevation or cross-section adjustments were made for these features.

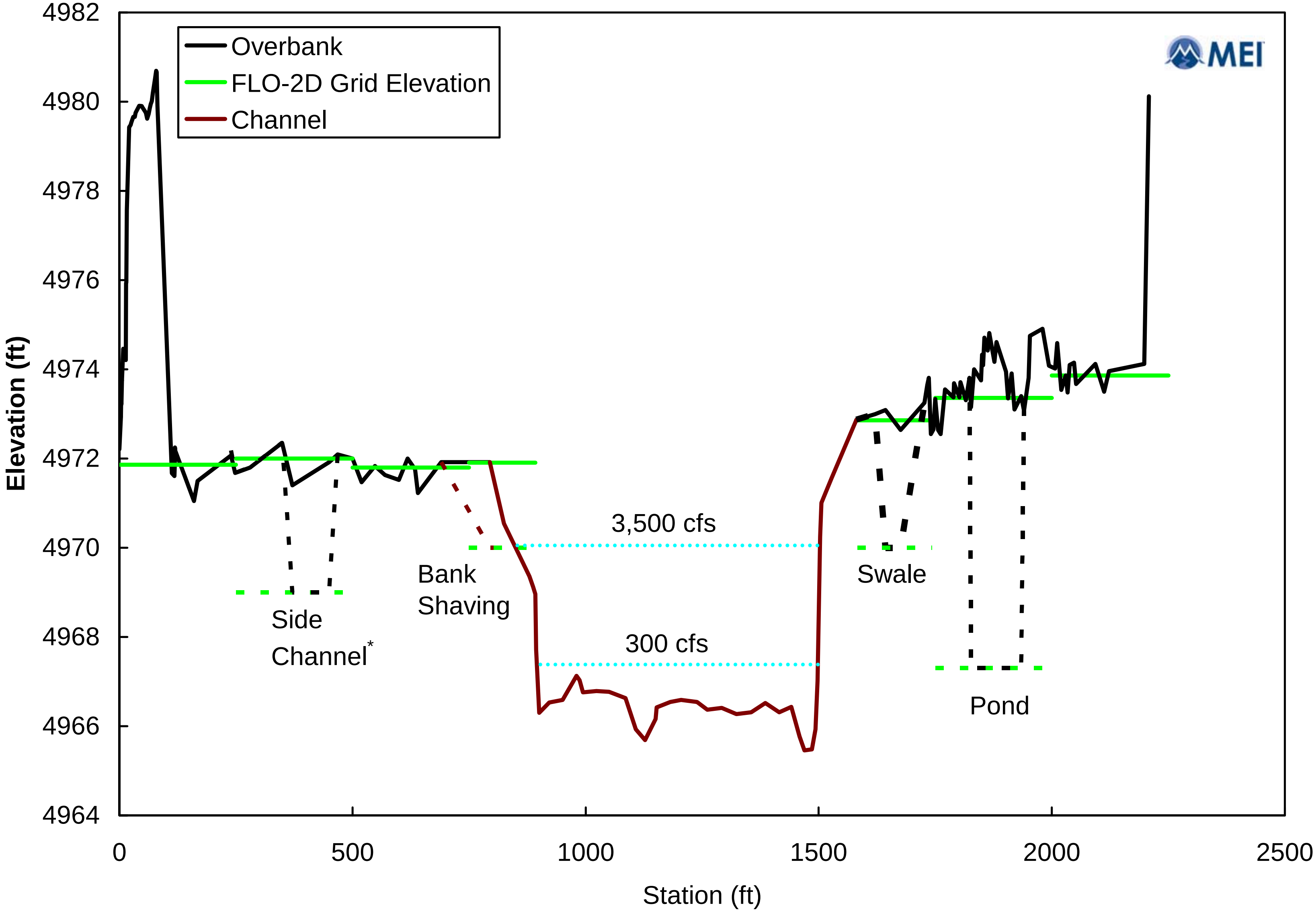

Figure 6.2. Schematic representation of FLO-2D grid modification to represent proposed alternatives.

Table 6.2. Manning's *n*-values for delineated features for Years 0, 5, 20, 30 and 50.

Feature	Year 0	Year 5	Year 20	Year 30	Year 50
Water features (300 cfs)	0.040	0.050	0.060	0.060	0.060
Water feature (3,500 cfs)	0.040	0.050	0.060	0.060	0.060
Bank destabilization	0.055	0.100	0.100	0.100	0.100
Swale trench	0.050	0.065	0.100	0.100	0.100
Overbank treat-retreat-revegetation	0.040	0.075	0.085	0.085	0.085

For the future conditions analysis, the overbank Manning's *n*-values were adjusted to reflect changes in roughness due to the establishment and growth of vegetation within the features (Table 6.2). Estimates of overbank roughness were developed in consultation with the USACE based on evaluation of the observed vegetation growth in other restoration projects within the project reach. In general, the roughness values in the overbank treat-retreat-revegetation features will be low after the initial vegetation clearing (Year 0). The roughness will increase after replanting, and will continue to increase as the vegetation becomes more established through Year 20. It was assumed the plants are fully established by Year 20, and the roughness values will remain constant for Years 20 through 50.

6.2. Restoration Alternative 1 Results

The amount of overbank inundation predicted by the FLO-2D model for each simulation under the "Maximum Effort" alternative (Alternative 1) was estimated for each subreach based on the number of pixels inundated on the 30-foot grid (**Table 6.3**). The following sections summarize the results of these simulations.

6.2.1. Channel Full Conditions

The channel-full flow simulations (6,000 cfs) indicate that the area of overbank inundation would increase significantly in all subreaches under the "Maximum Effort" (Table 6.3) alternative compared to existing conditions (Table 4.6). The maximum amount of inundation occurs during Year 0, when the overbank roughness values are lowest, because the vegetation has not had sufficient time to fully establish, and prior to channel degradation that is predicted to occur during Years 5 to 50. In Subreach 1, approximately 205 acres are inundated under this alternative, compared to 77 acres under existing conditions (Table 4.6). In Subreaches 2, 3, 4 and 5, the amount of overbank inundation increases from 41 to 182 acres, from 25 to 123, from 34 to 125 and 75 to 175 acres, respectively.

From Year 0 to 5, the amount of overbank inundation at the channel-full flow remains relatively constant in each of the subreaches. From Years 5 to 50, the amount of overbank inundation decreases slightly in all subreaches, except Subreach 3, as the overbank roughness increases and the channel degrades. At Year 50, the amount of overbank inundation in Subreaches 1 and 2 decrease by 3 and 5 acres respectively, compared to the inundation levels during Year 5. In Subreach 3, the overbank inundation remains the same as in Year 5, and in Subreaches 4 and 5, the inundation decreases by 10 and 7 acres, respectively, compared to the inundation levels during Year 5. The extent and maximum depth of inundation for this scenario are shown in Appendix B.1).

Table 6.3. Summary of areas of inundation for Restoration Alternative 1 (Years 0, 5, 20, 30 and 50) (acres).								
Hydrology Scenario	Description	Future Channel Condition (yr)	Reach					
			1	2	3	4	5	Total
1	Channel Full Conditions	Existing Conditions (Year 0)	77.2	41.3	25.2	34.4	75.6	253.7
		0	204.6	182.2	122.6	125.5	175.1	810.0
		5	199.6	179.5	122.1	132.0	179.4	812.6
		20	196.1	176.1	122.6	124.1	174.2	793.1
		30	201.8	174.7	122.5	121.5	173.6	794.1
		50	196.3	174.8	122.4	121.2	172.7	787.4
2	Annual Spring Runoff	Existing Conditions (Year 0)	45.2	23.1	7.7	4.0	7.9	87.9
		0	175.8	109.3	88.0	62.3	77.7	513.1
		5	180.0	109.0	89.0	65.4	93.8	537.2
		20	178.8	110.0	86.7	53.1	70.1	498.7
		30	178.8	110.4	88.8	54.3	70.8	503.1
		50	175.5	108.7	89.5	54.0	73.0	500.7
4	100-Year Peak Snowmelt	Existing Conditions (Year 0)	84.4	59.9	14.6	133.4	364.9	657.2
		0	277.5	186.1	162.7	298.3	392.9	1,317.5

These simulations also indicate that the predicted water-surface elevations would decrease by a maximum of 0.8 feet in the vicinity of Central Avenue and upstream from the Montano Bridge under Year 0 conditions, and by up to 0.9 feet under Year 50 conditions (**Figure 6.3**). This lowering is caused by the increased conveyance capacity associated with the restoration features, particularly the bank destabilization features that create a wider channel and the connected water features that allow more flow in the overbanks. The maximum decrease in the water-surface elevation occurs in the vicinity of Central Avenue and upstream from Montano Bridge. On average, the water-surface elevations throughout the entire reach will decrease by an average of 0.14 and 0.27 feet for the Year 0 and Year 50 conditions, respectively compared to existing conditions.

6.2.2. Average Annual Flow Hydrograph

The average annual flow simulations for Alternative 1 indicate that the amount of overbank inundation increases significantly in all subreaches and for all five future channel conditions compared to existing conditions.

The maximum amount of inundation generally occurs during Year 5 when there is a modest amount of channel aggradation in some locations along the project reach. In Subreaches 1 through 5, the inundation increases by 131, 86, 80, 58 and 70 acres, respectively, compared to the existing conditions (Table 6.3).

The future channel conditions simulations indicate that the amount of inundation in the subreaches will decrease by approximately 12 acres along the entire reach between Years 0 and 50 with a maximum decrease of 8 acres in Subreach 4. The extent, maximum depth and duration of inundation for this scenario are shown in Appendices B.2 and B.3.

The simulations also indicate that the predicted maximum water-surface elevations over the duration of the hydrograph will decrease by a maximum of 1.7 feet in the vicinity of the I-40 and Central Avenue Bridge for both the Years 0 and 50 conditions (Figure 6.3). On average, the maximum water-surface elevations throughout the entire study reach decrease by 0.34 and 0.48 feet for the Year 0 and Year 50 conditions, respectively.

6.2.3. 100-Year Snowmelt Hydrograph

The 100-year snowmelt hydrograph was simulated for the Year 0 conditions only. For this simulation, the amount of overbank inundation increases significantly (between 126 and 193 acres) in Subreaches 1 through 4, and increases slightly (28 acres) in Subreach 5 (Table 6.3). compared to existing conditions (Table 4.6). The extent, maximum depth and duration of inundation for this scenario are shown in Appendices B.4 and B.5. These simulations also indicate that the predicted maximum water-surface elevations over the duration of the hydrograph will decrease by a maximum of 0.8 feet between the Montano and Paseo del Norte Bridges under Year 0 conditions (Figure 6.3). On average, the maximum water-surface elevations throughout the entire reach will decrease by 0.13 feet for the Year 0 conditions.

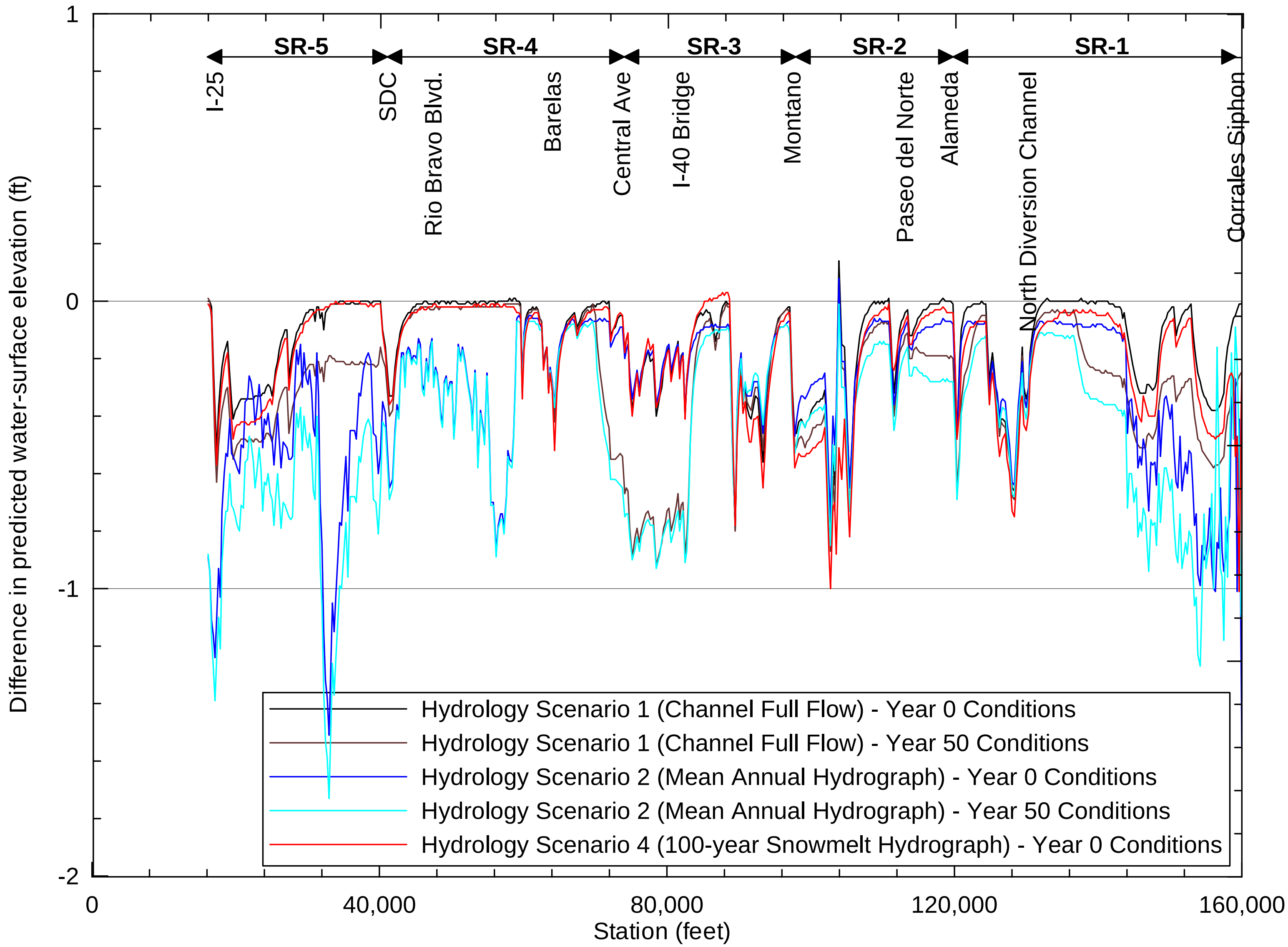

Figure 6.3. Predicted difference in maximum predicted water-surface elevation between the existing conditions and Restoration Alternative 1 (Maximum Effort), for Hydrology Scenarios 1, 2, and 4 at Year 0 and Year 50 Conditions.

6.3. Restoration Alternative 2 Results

The amount of overbank inundation for each simulation under the Minimal Effort alternative (Alternative 2) is summarized in **Table 6.4**.

6.3.1. Channel Full Conditions

The amount of overbank inundation for channel-full flow increases by a total of 8 acres in all subreaches with a maximum increase of 5 acres in Subreach 3 (Table 6.4). In Subreach 1, approximately 78 acres are inundated with the Restoration Alternative 2 features, compared to 77 acres under existing conditions. In Subreaches 2 and 3, the amount of overbank inundation increases from 41 to 43 acres and 25 to 30 acres, respectively. The overbank inundation increases from 34 to 35 acres in Subreach 4, and remains constant at 76 acres in Subreach 5.

Table 6.4. Summary of areas of inundation for Restoration Alternative 2 (Years 0, 5, 20, 30 and 50) (acres).

Hydrology Scenario	Description	Future Channel Condition (yr)	Reach					
			1	2	3	4	5	Total
1	Channel Full Conditions	Existing Conditions (Year 0)	77.2	41.3	25.2	34.4	75.6	253.7
		0	77.5	42.9	30.0	35.2	76.0	261.6
		5	80.9	40.7	40.9	35.3	74.2	272.0
		20	79.4	39.7	40.3	32.6	73.7	265.7
		30	79.8	39.7	40.3	32.7	74.2	266.7
		50	70.1	36.8	34.6	30.6	73.2	245.3
2	Annual Spring Runoff	Existing Conditions (Year 0)	45.2	23.1	7.7	4.0	7.9	87.9
		0	45.1	24.6	21.0	11.2	18.3	120.2
		5	50.1	24.0	25.5	9.8	12.4	121.8
		20	43.4	22.3	24.7	9.1	11.3	110.8
		30	47.2	24.1	25.3	9.6	11.9	118.1
		50	47.0	24.5	25.4	9.7	11.9	118.5
4	100-Year Peak Snowmelt	Existing Conditions (Year 0)	84.4	59.9	14.6	133.4	364.9	657.2
		0	93.8	65.8	51.6	225.2	290.3	726.7

From Year 0 to Year 5, the amount of overbank inundation at the channel-full flow remains relatively constant in Subreaches 1 and 2. In Subreach 3, the inundation increases from 30 to 41 acres, and in Subreaches 4 and 5, the inundation remains relatively constant at approximately 35 and 76 acres, respectively. From Years 5 to 50, the amount of overbank inundation at 6,000 cfs decreases by a total of 27 acres in all subreaches with a maximum decrease of 11 acres in Subreach 1. Subreaches 2, 3, 4 and 5, decrease by 4, 6, 5, and 1 acres respectively, compared to Year 5. The extent and maximum depth of inundation for this scenario are shown in Appendix C.1.

The channel-full flow simulations also indicate that the predicted water-surface elevations will decrease by a maximum of 0.76 feet under Year 0 conditions, and up to 0.78 feet under the Year 50 conditions between I-40 and Central Avenue (**Figure 6.4**). On average, the water-surface elevations throughout the entire reach decrease by 0.02 and 0.17 feet for the Year 0 and Year 50 conditions, respectively.

6.3.2. Average Annual Flow Hydrograph

The amount of overbank inundation remains constant in Subreach 1, an increases slightly in Subreaches 2 through 5, compared to existing conditions, during the average annual hydrograph for Year 0. In Subreach 2 through 5, the amount of overbank inundation for Year 0 increases from 23, 8, 4, and 8 acres, respectively under existing conditions to approximately 25, 21, 11, and 18 acres, respectively, with the Alternative 2 features (Table 6.4). Under future channel conditions, the amount of inundation in Subreach 1 increases from 45 to 47 acres between Years 0 and 50, the amount of inundation remains constant in Subreach 2, increases by 4 acres in Subreach 3, and decreases by 2 and 6 acres in Subreaches 4 and 5, respectively. The extent, maximum depth and duration of inundation for this scenario are shown in Appendices C.2 and C.3).

The predicted maximum water-surface elevations for these simulations over the duration of the hydrograph decrease by a maximum of 1.3 feet and 1.7 feet, respectively, between I-25 and the South Diversion Channel for Year 0 and Year 50 conditions (Figure 6.4). On average, the maximum water-surface elevations throughout the entire reach decrease by 0.22 and 0.39 feet for Year 0 and Year 50 conditions, respectively.

6.3.3. 100-Year Snowmelt Hydrograph

The 100-year snowmelt hydrograph was simulated for the Year 0 conditions only. The simulation indicates that the amount of overbank inundation will increase by 9, 6, 37 and 92 acres in Subreaches 1 to 4 compared to Existing Conditions. In Subreaches 1 to 4, the inundation will increase from 84 to 94, 60 to 65, 14 to 51, and 133 to 225 acres, respectively. In Subreach 5, the inundation will decrease from 365 to 290 acres compared to existing conditions (Table 6.4). The extent, maximum depth and duration of inundation for this scenario are shown in Appendices D.4 and D.5.

The simulations also indicate that the predicted maximum water-surface elevations over the duration of the hydrograph will decrease by a maximum of 0.8 feet between I-40 and Montano Bridge during the Year 0 conditions (Figure 6.4). On average, the maximum water-surface elevations throughout the entire reach decrease by about 0.06 feet for the Year 0 conditions.

6.4. Restoration Alternative 3 Results

The amount of overbank inundation for each simulation under the Moderate Effort Alternative (Alternative 3) is summarized in **Table 6.5**.

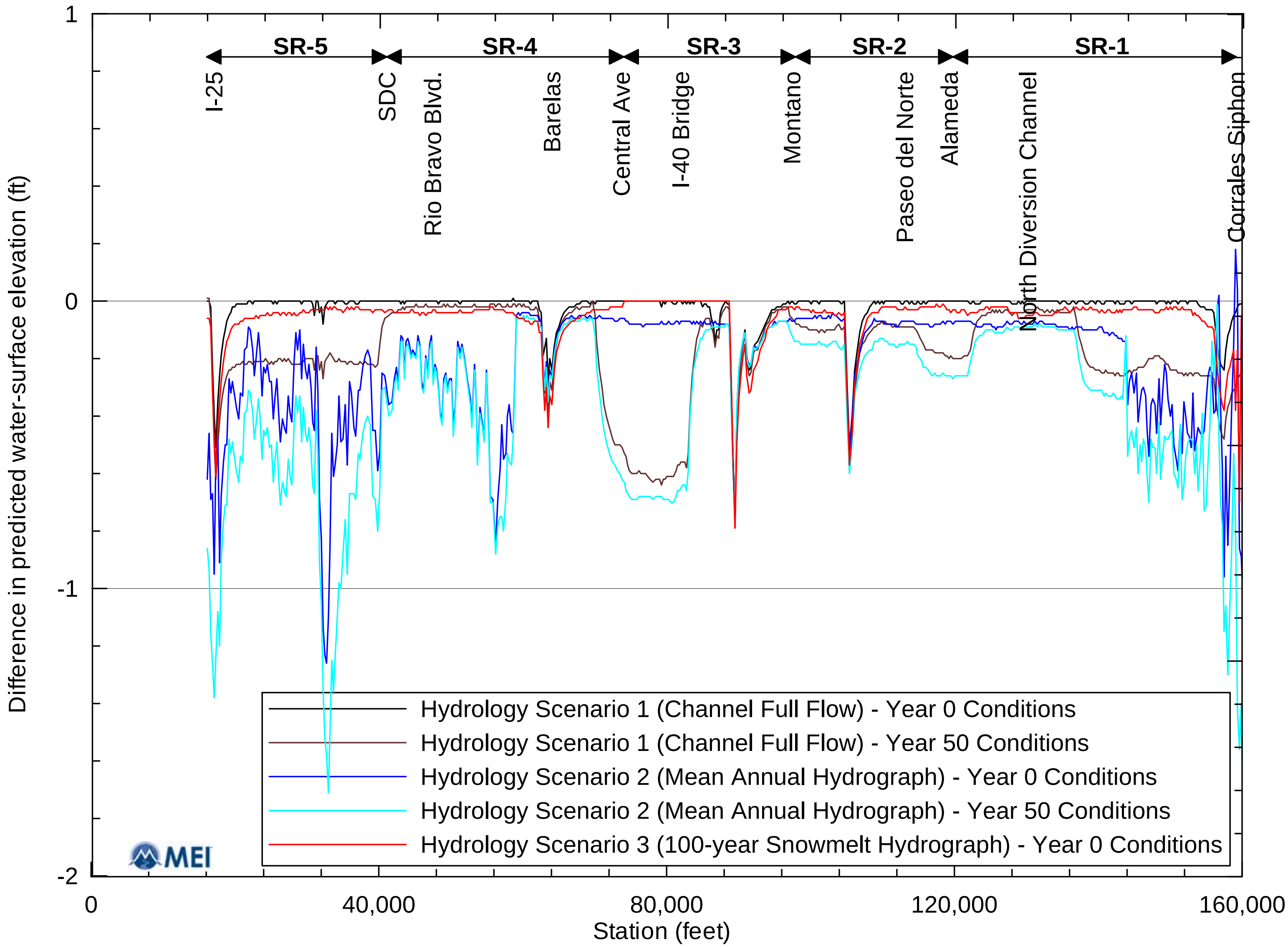

Figure 6.4. Predicted difference in maximum predicted water-surface elevation between the existing conditions and Restoration Alternative 2 (Minimum Effort), for Hydrology Scenarios 1, 2, and 4 at Year 0 and Year 50 conditions.

Table 6.5. Summary of areas of inundation for Restoration Alternative 3 (Years 0, 5, 20, 30 and 50) (acres).								
Hydrology Scenario	Description	Future Channel Condition (yr)	Reach					
			1	2	3	4	5	Total
1	Channel Full Conditions	Existing Conditions (Year 0)	77.2	41.3	25.2	34.4	75.6	253.7
		0	152.3	133.2	40.8	66.1	85.7	478.0
		5	159.7	126.8	44.8	66.8	83.3	481.5
		20	157.2	123.5	45.1	66.1	80.7	472.7
		30	158.0	123.8	44.9	66.9	80.2	473.8
		50	159.5	123.6	45.1	67.0	80.2	475.4
2	Annual Spring Runoff	Existing Conditions (Year 0)	45.2	23.1	7.7	4.0	7.9	87.9
		0	116.7	86.5	32.5	41.8	42.2	319.7
		5	116.7	86.7	35.8	49.2	36.3	324.8
		20	115.2	87.3	38.5	41.7	35.2	317.9
		30	116.2	87.2	38.5	41.3	35.8	319.0
		50	115.1	87.2	34.5	40.7	35.8	313.3
4	100-Year Peak Snowmelt	Existing Conditions (Year 0)	77.2	41.3	25.2	34.4	75.6	253.7
		0	84.4	59.9	14.6	133.4	364.9	657.2

6.4.1. Channel Full Conditions

The channel full flow simulations for Alternative 3 indicate that the amount of overbank inundation would increase in all Subreaches compared to the Existing Conditions (Table 6.5). In Subreaches 1 through 5, the amount of inundation would increase from 77 to 152, 41 to 133, 25 to 41, 34 to 66 and from 71 to 86 acres, respectively, with the Alternative 3 features.

From Year 0 to 5, the amount of overbank inundation in Subreach 1 increases from 152 to 160 acres. The overbank inundation in decreases from 133 to 127 in Subreach 2, increases from 41 to 45 in Subreach 3, remains constant in subreach 4, and decreases from 86 to 83 acres in Subreach 5. At Year 50, the inundated area remains constant in subreaches 1, 2 and 3 and decreases by 3 acres in both Subreaches 3 and 5, compared to Year 5. The extent and maximum depth of inundation for this scenario are shown in Appendix D.1.

These simulations also indicate that the predicted water-surface elevations will decrease by a maximum of 0.76 feet under the Year 0 conditions and by up to 0.88 feet under the Year 50 conditions between the Barelas and I-40 Bridges (**Figure 6.5**). On average, the water-surface elevations throughout the entire reach decrease by 0.08 and 0.22 feet for the Year 0 and Year 50 conditions, respectively.

6.4.2. Average Annual Hydrograph

For the average-annual hydrograph, the amount of overbank inundation increases moderately in all subreaches and for all five future channel conditions compared to existing conditions. In Subreaches 1 through 5, the amount of overbank inundation for Year 0 increases from 45 to 117, 23 to 87, 8 to 32, 4 to 42 and 8 to 42 acres, respectively, with the Alternative 3 features (Table 6.5).

The future channel conditions simulations indicate that the amount of inundation does not change between Year 0 and Year 5 Subreaches 1 and 2. The amount of inundation increases by 3 and 7 acres in Subreaches 3 and 4, respectively, and decreases by 6 acres in Subreach 5, in Year 5, compared to Year 0.

At Year 50, the area of inundation decreases by 2 acres in Subreach 1 remains constant in Subreach 2, and decreases by 1 and 8 in Subreaches 3 and 4, respectively, and remains constant in Subreach 5 compared to Year 5. The extent, maximum depth and duration of inundation for this scenario are shown in Appendices D.2 and D.3.

The predicted maximum water-surface elevations over the duration of the hydrograph for these simulations will decrease by a maximum of 1.3 feet and 1.7 feet for the Years 0 and 50 conditions, respectively, between I-25 and the South Diversion Channel (Figure 6.5). On average, the maximum water-surface elevations throughout the entire reach decrease by 0.29 and 0.44 feet for the Years 0 and 50 conditions, respectively.

6.4.3. 100-Year Snowmelt Hydrograph

For the 100-year snowmelt hydrograph, the amount of overbank inundation at Year 0 increases in Subreaches 1 to 4 from 84 to 253, 60 to 151, 15 to 127 and from 133 to 277 acres, respectively, and decreases in Subreach 5 from 365 to 304 acres compared to existing

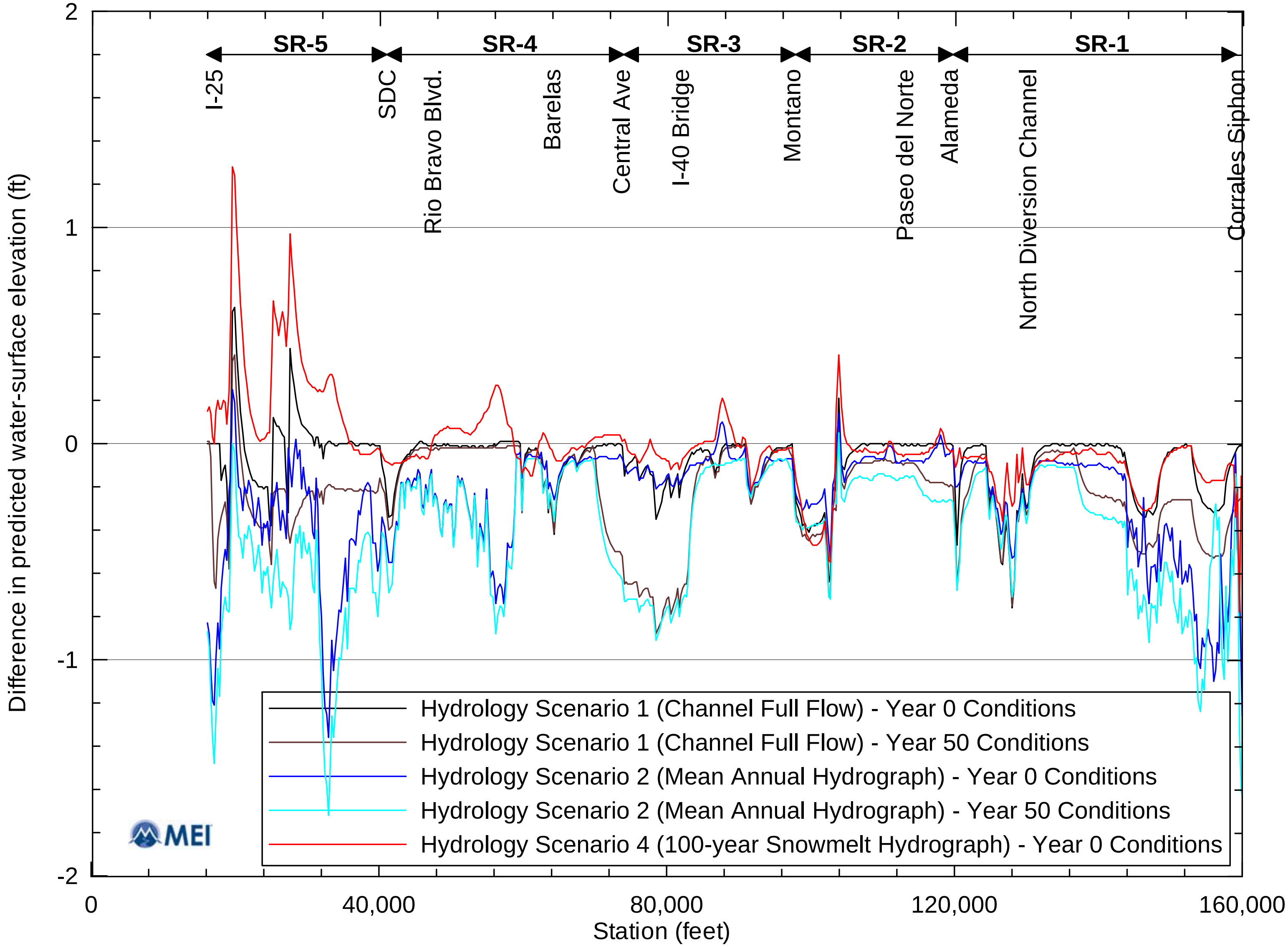

Figure 6.5. Predicted difference in maximum predicted water-surface elevation between the existing conditions and Restoration Alternative 3 (Moderate Effort), for Hydrology Scenarios 1, 2, and 4 at Year 0 and Year 50 conditions.

conditions (Table 6.5). The extent, maximum depth and duration of inundation for this scenario are shown in Appendices D.4 and D.5.

The predicted maximum water-surface elevations for these simulations over the duration of the hydrograph will decrease by a maximum of 0.4 feet between I-25 and Barelas Bridge during the Year 0 conditions (Figure 6.3). On average, the maximum water-surface elevations throughout the entire reach will increase by 0.01 feet for the Year 0 conditions.

6.5. Restoration Alternative 4 Results

The amount of overbank inundation for each simulation under the Moderate Effort-A Alternative (Alternative 4) is summarized in **Table 6.6**.

Table 6.6. Summary of areas of inundation for Restoration Alternative 4 (Years 0, 5, 20, 30 and 50) (acres).

Hydrology Scenario	Description	Future Channel Condition (yr)	Reach					
			1	2	3	4	5	Total
1	Channel Full Conditions	Existing Conditions (Year 0)	77.2	41.3	25.2	34.4	75.6	253.7
		0	145.2	46.1	31.3	55.5	85.5	363.6
		5	147.7	45.5	28.8	53.8	84.1	360.0
		20	146.7	45.4	28.6	53.9	83.3	357.8
		30	147.5	45.7	28.4	54.7	82.7	358.9
		50	148.9	45.5	28.6	54.8	82.8	360.5
2	Annual Spring Runoff	Existing Conditions (Year 0)	45.2	23.1	7.7	4.0	7.9	87.9
		0	113.3	25.2	16.6	30.7	42.9	228.7
		5	113.3	25.4	20.0	38.1	48.8	245.6
		20	111.8	26.0	16.7	30.6	28.7	213.8
		30	112.8	25.8	16.7	30.2	28.5	214.2
		50	111.6	25.8	16.9	29.6	28.1	212.1

6.5.1. Channel Full Conditions

The channel full flow simulations for Alternative 4 indicate that the amount of overbank inundation would increase in all subreaches compared to the Existing Conditions (Table 6.6). In Subreaches 1 through 5, the amount of inundation would increase from 77 to 145, 41 to 46, 25 to 31, 34 to 56, and from 76 to 86 acres, respectively. The maximum amount of inundation occurs during Year 0. From Year 0 to 50, the amount of overbank inundation in Subreach 1 increases slightly from 145 to 149 acres. The overbank inundation in deceases slightly in Subreaches 2 through 5 from 46 to 45, 31 to 29, 56 to 55 and 86 to 83 respectively, compared to Year 0. The extent and maximum depth of inundation for this scenario are shown in Appendix E.1.

These simulations also indicate that the predicted water-surface elevations will decrease by a maximum of 0.25 feet under the Year 0 conditions and by up to 0.58 feet under the Year 50 conditions (**Figure 6.6**). On average, the water-surface elevations throughout the entire reach decrease by 0.04 and 0.10 feet for the Year 0 and Year 50 conditions, respectively.

6.5.2. Average Annual Hydrograph

For the average-annual hydrograph, the amount of overbank inundation increases in all subreaches and for all five future channel conditions compared to existing conditions. In Subreaches 1 through 5, the amount of overbank inundation for Year 0 increases from 45 to 113, 23 to 25, 8 to 17, 4 to 31 and 8 to 43 acres, respectively, with the Alternative 4 features (Table 6.6).

The future channel conditions simulations indicate that the amount of inundation at Year 5 remains constant compared to Year 0 in Subreaches 1 and 2. The amount of inundation increases by 3, 7 and 6 acres in Subreaches 3, 4, and 5, respectively, compared to Year 0.

At Year 50, the area of inundation decreases by 2 acres in Subreach 1, remains constant in Subreach 2, and decreases by 3, 9 and 21 acres in Subreaches 3 through 5, respectively, compared to Year 0. The extent, maximum depth and duration of inundation for this scenario are shown in Appendices E.2 and E.3).

The predicted maximum water-surface elevations over the duration of the hydrograph for these simulations will decrease by a maximum of 1.3 feet and 1.7 feet for the Years 0 and 50 conditions, respectively, between I-25 and the South Diversion Channel (Figure 6.6). On average, the maximum water-surface elevations throughout the entire reach decrease by 0.24 and 0.39 feet for the Years 0 and 50 conditions, respectively.

6.6. Restoration Alternative 5 Results

The amount of overbank inundation for each simulation under the Moderate Effort-B Alternative (Alternative 5) is summarized in **Table 6.7**.

6.6.1. Channel Full Conditions

The channel full flow simulations for Alternative 5 indicate that the amount of overbank inundation would increase in all subreaches compared to the Existing Conditions (Table 6.7). In Subreaches 1 through 5, the amount of inundation would increase from 77 to 191, 41 to 45, 25 to 49, 34 to 97 and from 76 to 85 acres, respectively with the Alternative 5 features.

The maximum amount of inundation occurs during Year 0. From Year 0 to 50, the amount of overbank inundation in Subreach 1 increases slightly from 191 to 192 acres. The overbank inundation in Subreach 2 remains constant in Year 5, and deceases slightly in Subreaches 3 through 5 from 49 to 44, 97 to 91 and 85 to 82 acres respectively, compared to Year 0. The extent and maximum depth of inundation for this scenario are shown in Appendix F.1.

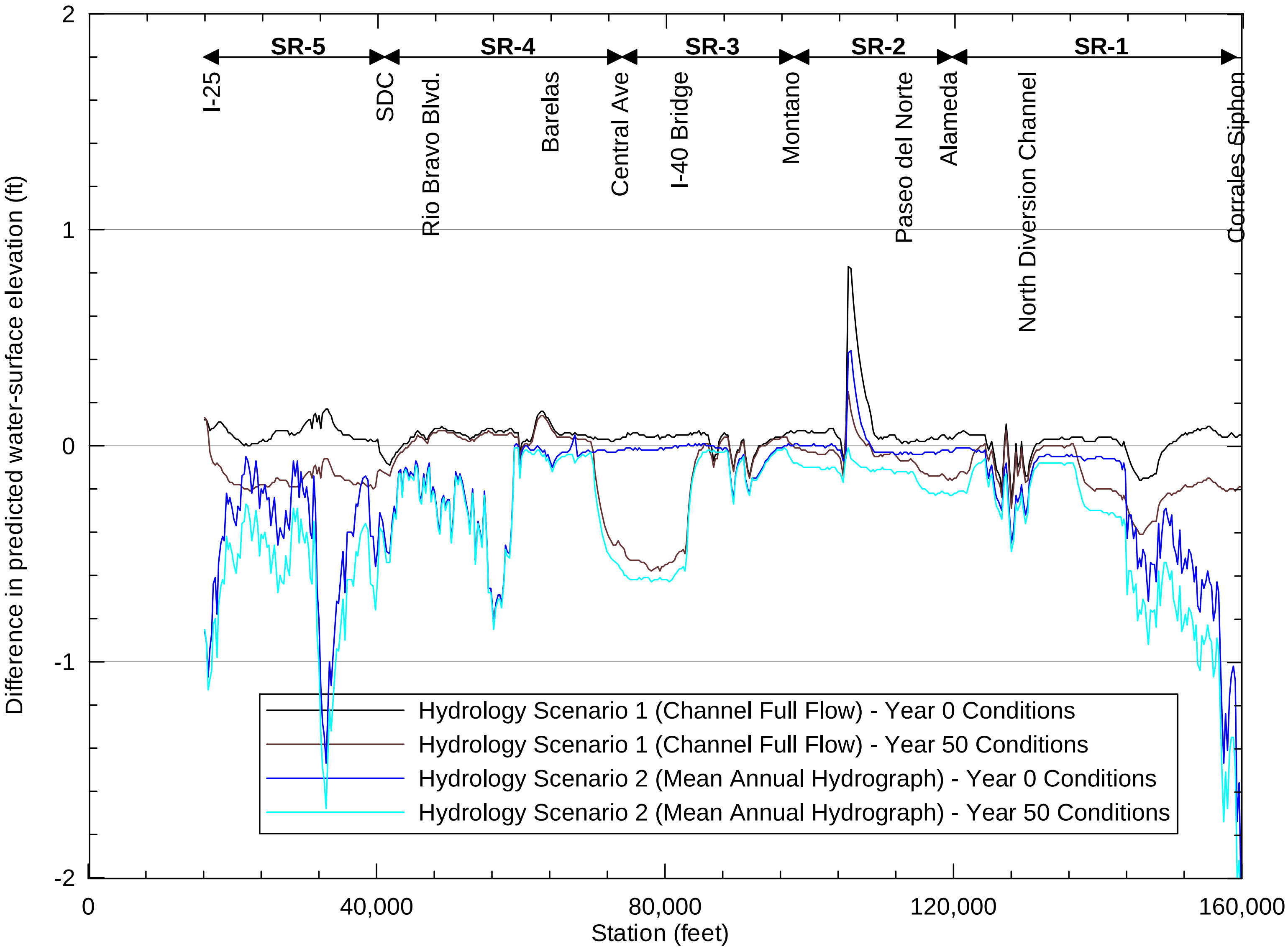

Figure 6.6. Predicted difference in maximum predicted water-surface elevation between the existing conditions and Restoration Alternative 4 (Moderate Effort-A), for Hydrology Scenarios 1 and 2 at Year 0 and Year 50 Conditions.

Table 6.7. Summary of areas of inundation for Restoration Alternative 5 (Years 0, 5, 20, 30 and 50) (acres).

Hydrology Scenario	Description	Future Channel Condition (yr)	Reach					
			1	2	3	4	5	Total
.	Channel Full Conditions	Existing Conditions (Year 0)	77.2	41.3	25.2	34.4	75.6	253.7
		0	190.6	45.4	49.0	96.6	85.3	466.8
		5	193.4	44.6	47.9	94.4	84.4	464.8
		20	192.5	45.3	44.4	95.1	88.1	465.4
		30	194.0	45.0	48.2	95.6	83.6	466.4
		50	192.4	45.7	43.9	91.3	82.3	455.6
2	Annual Spring Runoff	Existing Conditions (Year 0)	45.2	23.1	7.7	4.0	7.9	87.9
		0	165.8	40.3	39.5	73.6	41.0	360.2
		5	158.1	37.5	45.6	74.4	37.3	352.9
		20	145.3	40.0	45.5	67.7	21.4	319.9
		30	147.7	38.1	45.0	79.6	22.6	333.0
		50	144.1	37.6	41.5	65.2	19.1	307.6

These simulations also indicate that the predicted water-surface elevations will decrease by a maximum of 0.26 feet under the Year 0 conditions and by up to 0.64 feet under the Year 50. (**Figure 6.7**). On average, the water-surface elevations throughout the entire reach decrease by 0.02 and 0.12 feet for the Year 0 and Year 50 conditions, respectively.

6.6.2. Average Annual Hydrograph

For the average-annual hydrograph, the amount of overbank inundation increases in all subreaches and for all five future channel conditions compared to existing conditions. In Subreaches 1 through 5, the amount of overbank inundation for Year 0 increases from 45 to 166, 23 to 40, 8 to 40, 4 to 74 and 8 to 41 acres, respectively, with the Alternative 5 features (Table 6.7).

The maximum amount of inundation occurs during Year 0. From Year 0 to 50, the amount of overbank inundation in Subreaches 1 and 2 decreases from 166 to 144 and from 40 to 38 acres, respectively. The inundation increases from 40 to 42 acres in Subreach 3, and decreases in Subreaches 4 and 5 from 74 to 65 and 41 to 19 acres, respectively. The extent and maximum depth of inundation for this scenario are shown in Appendices F.2 and F.3.

The predicted maximum water-surface elevations over the duration of the hydrograph for these simulations will decrease by a maximum of 1.4 feet and 1.7 feet for the Years 0 and 50 conditions, respectively, between I-25 and the South Diversion Channel (Figure 6.7). On average, the maximum water-surface elevations throughout the entire reach decrease by 0.26 and 0.41 feet for the Years 0 and 50 conditions, respectively.

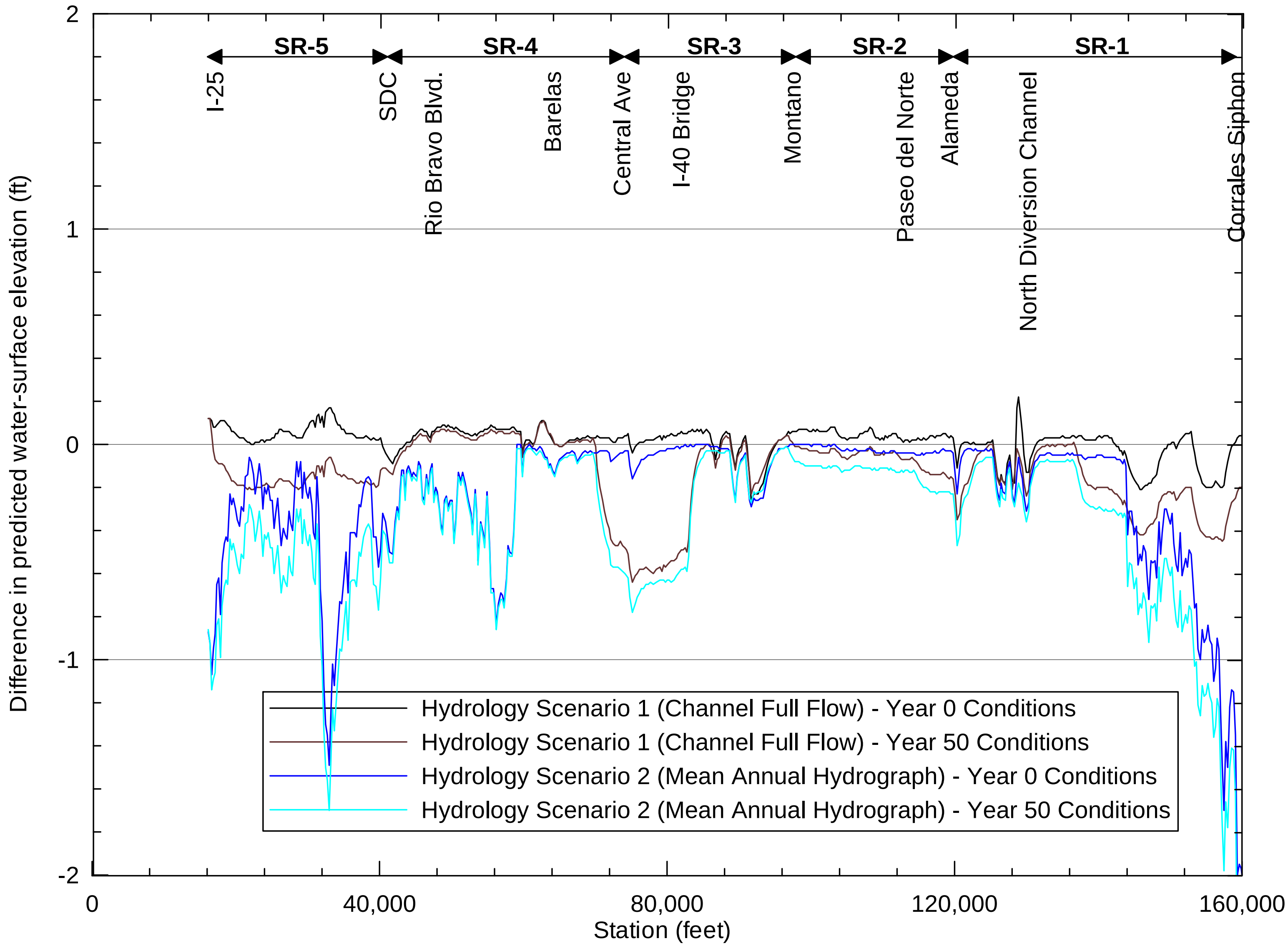

Figure 6.7. Predicted difference in maximum predicted water-surface elevation between the existing conditions and Restoration Alternative 5 (Moderate Effort-B), for Hydrology Scenarios 1 and 2 at Year 0 and Year 50 conditions.

6.7. Sustainability of Restoration Features

An analysis of the overbank sediment-transport characteristics was conducted to evaluate the long term sustainability of restoration features. Overbank flows will cause sediment deposition on the floodplain and sediment deposition will also occur in the proposed channel restoration features, particularly after the vegetation has established. An estimate of the amount and rate of sediment deposition within the features was made for Restoration Alternative 1 (Maximum Effort alternative) under the Hydrology Scenario 4 (100-year post-Cochiti flood-flow snowmelt hydrograph) in order to evaluate the long-term sustainability of the proposed features.

Table 6.8 summarizes the total amount of predicted overbank inundation and the design area of each type of restoration feature in each subreach. In Restoration Alternative 1, there are approximately 232 acres of swale features, 95 acres of water-channel feature and 174 acres of water-pond features. The bank features are not included in the analysis, because they are designed to be eroded by the river in order to increase channel sinuosity, and are, therefore, not considered to be permanent features. The predicted area of overbank inundation under Hydrology Scenario 4 is 278, 186, 163, 298, and 393 acres for Subreaches 1 through 5 respectively. The swale features, water-channel features, and water-pond features account for 18, 7, and 13 percent of the total inundation area.

Table 6.8. Summary of total predicted area of inundation for the Maximum Effort Conditions (Hydrology Scenario 4) and the area for each feature class.

Total Inundation Area	SR1	SR2	SR3	SR4	SR5	Total
	277.5	186.1	162.7	298.3	392.9	1317.5
Inundation Area for each Restoration Feature Class						
Swale	92.8	28.8	37.2	34.3	39.1	232.2
Water-Channel	45.5	13.2	8.4	6.9	20.7	94.6
Water-Pond	16.6	33.3	86.3	23.8	14.2	174.2
Percentage of Inundation Area						
Swale	33%	15%	23%	11%	10%	18%
Water-Channel	16%	7%	5%	2%	5%	7%
Water-Pond	6%	18%	53%	8%	4%	13%
Total	56%	40%	81%	22%	19%	38%

The amount of overbank sedimentation that would occur during Hydrology Scenario 4 was estimated from the amount of sediment in the main channel water column that would be conveyed onto the overbank. This estimate represents an upper limit of sediment transport in the overbanks, as sediment transport rates would be higher near the channel margins and would drop rapidly further away from the channel. The estimates were made based on one representative restoration site that was selected in each subreach (**Table 6.9**). The Rouse suspended sediment concentration profile equation (Vanoni, 1977) was applied with the main channel hydraulic results from the FLO-2D model and a representative particle size of 0.5 mm to assess the characteristics of the sediment concentration profile at the five representative subreach sites at a discharge of 7,000 cfs (the Cochiti Reservoir release and dominant discharge in Hydrology Scenario 4).

Table 6.9. Location of representative restoration locations.		
Subreach	Station	Description
SR-1	126,858	Just below the North Diversion Channel
SR-2	81,531	Just below I-40 Bridge
SR-3	76,092	Just upstream of Central Ave. Bridge
SR-4	66,432	Just upstream of Barelas Bridge
SR-5	9,183	Just upstream of I-25 Bridge

An example of the predicted cumulative sediment-transport profiles in the main channel for Subreach 3 at 7,000 cfs is shown in **Figure 6.8**. The square symbols represent the elevation in the water column at which flows would be conveyed into the channel features (designed to the 3,500-cfs water-surface elevation), and the circular symbols represent the top of bank elevations. For Subreach 3, approximately 34 percent of the bed material load is carried in the portion of the water column above the elevation of the channel feature design elevation, and 7 percent of the bed-material load is carried in the portion of the water column above the bank elevation. **Figure 6.9** shows the predicted cumulative sediment-transport profiles in the main channel for each of the representative sites at 7,000 cfs. Based on this analysis, 30 to 38 percent of the bed-material load (average is 35 percent) is carried in the portion of the water column above the elevation of the channel feature design elevation, and between 5 and 22 percent (average is 12 percent) of the bed-material load is carried in the portion of the water column above the bank elevation.

The depth of sediment deposition on the overbank during the 100-year snowmelt hydrograph was estimated by integrating the subreach sediment-rating curves over the period of the hydrograph (approximately 102 days at 7,000 cfs) to obtain the total volume of sediment and then dividing by the subreach inundation area to obtain the inundation depth (**Table 6.10**). Assuming that 12 percent of the suspended bed-material load of the main-channel is transported onto the overbank, the predicted average depth of sedimentation on the overbanks is 0.19, 0.25, 0.29, 0.14 and 0.12 feet for Subreaches 1 through 5, respectively. Since the restoration features are designed to be lower than the surrounding overbank elevation, they would likely receive more sediment deposition than the higher surrounding overbanks due to the higher roughness values created by the vegetation and the associated decreased velocities. Assuming that the sediment deposition rate is 5 times higher in the restoration features than on the overbank features, the predicted average depth of sedimentation would increase to 0.9, 1.2, 1.4, 0.7 and 0.6 feet for Subreaches 1 through 5, respectively.

For the channel restoration features, it was assumed that 35-percent of the suspended bed-load would be conveyed into the features. The estimated amount of sedimentation in the channel restoration features is 0.6, 0.7, 0.9, 0.4, and 0.4 feet for Subreaches 1 through 5 respectively (**Table 6.11**). Given that the 100-year hydrograph has a duration of approximately 102 days (3.4 months) above 7,000 cfs, the predicted amount of overbank deposition appears reasonable and relatively low during the 100-year event. Furthermore, given that the predicted depth of overbank is an upper limit and the depth of deposition is significantly less than the depth of the features, the overbank features should not be unreasonably affected by sediment deposition over the 50-year life of the project.

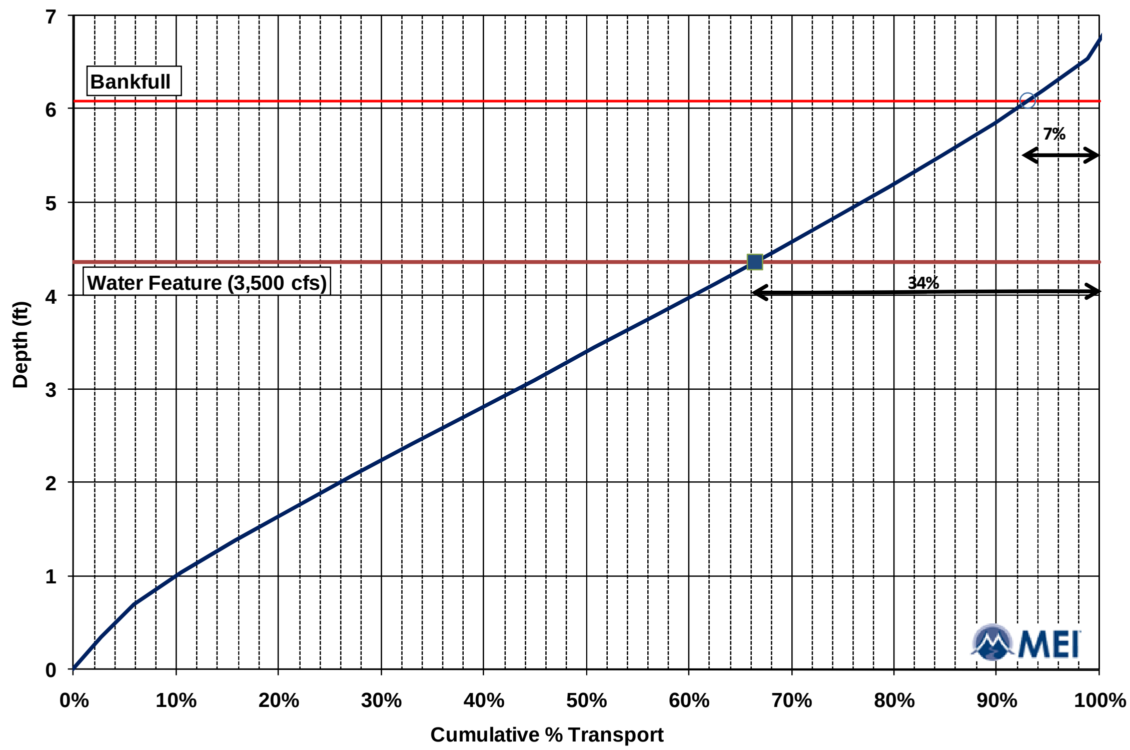

Figure 6.8. Cumulative percent of bed-material load as a function of height above the channel bed at 7,000 cfs for the representative Subreach 3 site.

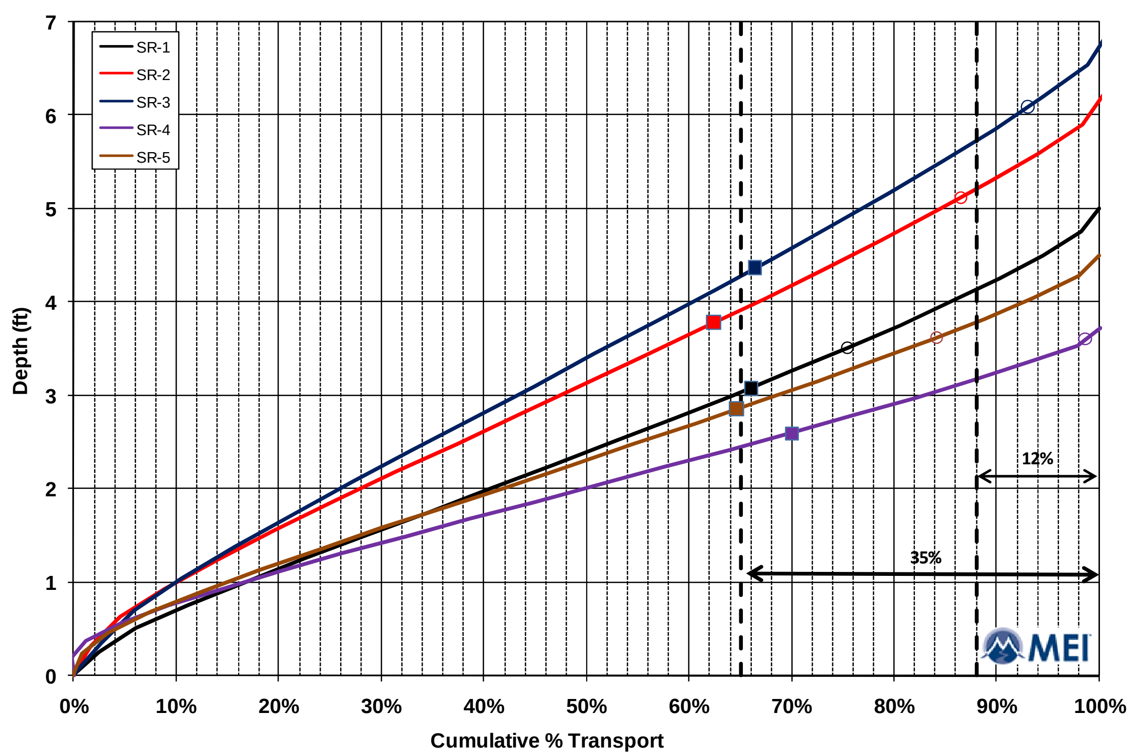

Figure 6.9. Cumulative percent of bed-material load as a function of height above the channel bed at 7,000 cfs for the representative sites at Subreaches 1 to 5.

Table 6.10. Predicted overbank sedimentation depths for the Maximum Effort, 100-year snowmelt scenario.				
Subreach	Sediment Transport- Main Channel(tons/day)	Sediment Transport Channel Features (tons/day)	Average Overbank Sedimentation Depth (ft)	Five Times Average Sedimentation Depth (ft)
SR-1	12,181.07	1,461.73	0.2	0.9
SR-2	12,644.97	1,517.40	0.2	1.2
SR-3	13,239.62	1,588.75	0.3	1.4
SR-4	11,885.03	1,426.20	0.1	0.7
SR-5	13,048.20	1,565.78	0.1	0.6

Table 6.11. Predicted sedimentation depths in the channel restoration features (3,500-cfs features) for the Maximum Effort, 100-year snowmelt scenario.			
Subreach	Sediment Transport- Main Channel (tons/day)	Sediment Transport Channel Features (tons/day)	Average Channel Restoration Sedimentation Depth (ft)
SR-1	12,181.07	4,263.37	0.6
SR-2	12,644.97	4,425.74	0.7
SR-3	13,239.62	4,633.87	0.9
SR-4	11,885.03	4,159.76	0.4
SR-5	13,048.20	4,566.87	0.4

7. SUMMARY

7.1. Summary

Mussetter Engineering, Inc. (MEI) was retained by the U.S. Army Corps of Engineers (USACE) (Contract DACW47-02-D-005, Delivery Order 0006) to perform FLO-2D modeling to support a planning study of the Albuquerque Reach of the Rio Grande, which extends from Corrales Siphon to the northern boundary of the Pueblo of Isleta (Figure 1.1). The objective of the planning study is to increase river channel and bosque overbank connectivity, produce enhanced cover and aquatic habitat diversity, restore healthy riparian function to enhance natural riverine processes, improve terrestrial wildlife habitat, and protect existing structural features such as pipelines, bridges and levees with a preference toward using bank biostabilization techniques when structures are found to be at risk from natural geomorphic processes (USACE, 2004). The FLO-2D modeling provides an assessment of overbank flows, storage, and hydraulic data to facilitate analysis of sediment-transport conditions and geomorphic processes along the reach, results from which will be used to evaluate various restoration alternatives.

A previous report summarized development of the four hydrologic scenarios, development, verification and application of the 500-foot grid FLO-2D model, and the baseline channel stability analysis (MEI, 2006). A subsequent report summarized the development of a new, higher resolution FLO-2D model (250-foot grid), re-analysis of the extent, depth and duration of overbank inundation for existing conditions for each hydrologic scenario, and analysis of the initial restoration alternative developed by the USACE (MEI, 2008a).

This report provides a comprehensive final project report that combines the previous two reports, as well as the analysis of the Restoration Alternatives 2 through 5. In addition, this report includes an evaluation of the effect of sedimentation within the project area under existing conditions and "end-of life" (Year 50) project conditions for the five restoration alternatives.

7.1.1. Hydrology

A hydrology analysis was performed to develop the following four hydrologic scenarios (**Table 7.1**) that were used to evaluate the baseline conditions and the five identified restoration alternatives:

Table 7.1. Summary of hydrologic scenarios.

Hydrologic Scenario	Description	Peak Discharge (cfs)
1	Active channel-full flow	6,000
2	Post-Cochiti annual spring hydrograph	3,770
3	10,000 cfs post-Cochiti hydrograph	10,000
4	100-year post-Cochiti hydrograph	7,750

Mean-daily flow-duration and flood-frequency curves were developed at the study site using the USGS flow records for the Albuquerque Gage. Results for the mean-daily flow analysis indicate the 1-, 10-, 50- and 99-percent exceedence values are 6,073, 3,349, 846 and 321 cfs, respectively. Results for the flood-frequency analysis indicate the 2-, 5-, 10-, 50- and 100-year peak flows are 5,630, 7,520, 8,560, 12,000 and 13,300 cfs, respectively.

7.1.2. FLO-2D Modeling

Two FLO-2D models of the reach from Cochiti Dam to Elephant Butte Reservoir were used in this project. The first FLO-2D model was developed by Tetra Tech Inc, (2004) and has a grid resolution of 500 feet with over 36,000 elements. The hydraulic output from the 500-foot grid model was used to perform the long-term channel stability analysis and to perform the initial existing conditions evaluation under Hydrology Scenarios 1, 2 and 3. Hydrology Scenario 4 was not modeled with the 500-foot grid because the re-evaluation of the 100-year hydrology had not been completed by the USACE at that time.

The second FLO-2D model was subsequently developed by Riada Engineering, Inc. and MEI (2008) for the USACE to update the initial URGWOPS model with a grid resolution of 250 feet. MEI subsequently modified the 250-foot FLO-2D grid to incorporate data from a LiDAR survey performed on December 7 and 8, 2006, by LiDAR US, LLC when the discharge at the Albuquerque gage was approximately 700 cfs. The LiDAR survey included the area between the levees within the boundaries of the Pueblo of Sandia.

The existing conditions model was re-evaluated using the validated 250-foot grid model for each of the four the hydrology scenarios. The analysis was conducted for initial channel conditions (Year 0) and four future channel conditions to evaluate the effects of aggradation or degradation on overbank inundation after 5-, 20-, 30- and 50-years. The results of the existing conditions models were used to provide baseline conditions for comparison with results of the restoration alternatives, and the results were used to compare the main channel water-surface elevations with the top-of-bank elevations and to map and evaluate the extent, depth and duration of overbank inundation along the reach. Results of the existing conditions models predicts that 253, 88 and 657 acres will are inundated during Hydrology Scenarios 1, 2 and 4, respectively, for Year 0 conditions.

The target restoration flows (Hydrology Scenarios 1 and 2) are simulated in Restoration Alternatives 1 through 5 for the future year conditions (0, 5, 20, 30 and 50). Hydrology Scenario 3 was not simulated for the restoration alternatives since the existing conditions analysis shows extensive overbank inundation and the hydrology analysis of the 100-year flood performed by the Corps (HEC, 2006) indicated the peak of the 100-year flood (7,750 cfs) was regulated by the current maximum operating release from Cochiti Reservoir. Hydrology Scenario 4 was simulated for Restoration Alternatives 1, 2 and 3 for the Year 0 conditions

7.1.3. Sediment-continuity Analysis

A baseline sediment-continuity analysis was performed to evaluate the potential for aggradation or degradation in response to both individual short-term hydrographs and longer-term flows (50-year project life) with the present channel configuration and reservoir operations. In general, the analysis was conducted by estimating the bed-material transport capacity of the supply reach and each subreach within the study area for each hydrology scenario and comparing the resulting capacity with the supply from the upstream river and tributaries within the reach. For this analysis, Hydrology Scenarios 2, 3 and 4 (mean annual runoff, 10,000-cfs, and 100-year snowmelt hydrographs, respectively) were used for the individual hydrographs, and the mean daily flow-duration curve from the Albuquerque gage for the post-Cochiti Dam period was used for the long-term analysis.

To facilitate the analysis, bed-material transport capacity rating curves were developed for each subreach using hydraulic output from the 500-foot grid FLO-2D model, representative bed-material gradations and the Yang (Sand) sediment-transport equation (Yang, 1973). The representative bed-material gradations used in the analysis were taken from MEI (2004), with the gradation for URGWOPS Subreach 12a (Bernalillo to Rio Rancho Wastewater Treatment Plant) representing the supply reach and Subreach 12b (Rio Rancho to Isleta Diversion Dam) representing the primary study reach for this project. The supply reach gradation has a median size of about 1 mm (coarse sand), contains material up to about 128 mm, and about 42 percent of the material is in the gravel- and cobble-size range (Figure 5.1). The gradation for the primary project reach has a median size of 0.5 mm (medium and coarse sand), contains material up to about 32 mm, and about 92 percent of the material is sand.

Results of the sediment-continuity analysis indicate that the bed-material transport capacity is relatively consistent from subreach to subreach, although there is a slight net degradational tendency, in the absence of tributary sediment inputs, for the overall study reach for all three of the individual storm hydrographs that were analyzed.

7.1.4. Hydraulic Modeling of Restoration Alternatives

FLO-2D modeling was conducted using the 250-foot grid resolution model to evaluate depth, extent and duration of overbank inundation for five restoration alternatives provided by the USACE (Maximum Effort, Minimum Effort, Moderate Effort, Moderate Effort-A and Moderate Effort-B). The analysis was conducted for initial channel conditions immediately after construction of the project (Year 0) and four future channel (5, 20, 30 and 50 years after construction of the restoration features)

The 250-foot grid Existing Conditions model was modified to represent each of the restoration alternatives by making appropriate adjustments to the main channel cross-sectional geometry, overbank grid elevations, and roughness parameters. Alternative 1, which is referred to as the “Maximum Effort” alternative, contains all of the channel and overbank features that were considered in formulating the alternative. Alternatives 2, 3, 4 and 5 are termed the “Minimum Effort”, “Moderate Effort”, and “Moderate Effort-A” and “Moderate Effort-B”, respectively, and they were developed using various combinations of the channel and overbank features that were included in Alternative 1.

For the future conditions analysis, the overbank Manning’s *n*-values were adjusted to reflect changes in roughness due to the establishment and growth of vegetation within the features (Table 6.2). Estimates of overbank roughness were developed in consultation with the USACE based on evaluation of the observed vegetation growth in other restoration projects within the project reach. In general, the roughness values in the overbank treat-retreat-revegetation features will be low after the initial vegetation clearing (Year 0). The roughness will increase after replanting, and will continue to increase as the vegetation becomes more established through Year 20. It was assumed the plants are fully established by Year 20, and the roughness values will remain constant for Years 20 through 50.

To reflect future channel conditions in the project reach under the existing conditions and alternatives, changes in the channel cross sections associated with aggradation/degradation 5, 20, 30 and 50 years after project implementation were estimated using a HEC-6T model of the reach that was previously developed by MEI for the New Mexico Interstate Stream Commission (NMISC) (MEI, 2007). The HEC-6T analysis indicates that both aggradational and degradational trends occur along the reach in Year 5. Over time, the aggradational areas shown in Year 5, change to a stable or slightly degradational at Years 20 and 30, and there is a slight degradational trend along the entire project reach over the 50-year simulation.

Results of the Restoration Alternative 1, Year 0 scenarios predicts that the total inundation will increase from 254 to 796, 88 to 513 and 657 to 1,318 acres, compared to existing conditions for Hydrology Scenarios 1, 2, and 4, respectively. From Year 0 to Year 50, the amount of overbank inundation typically decreases as the overbank roughness increases and the channel degrades slightly.

The channel full simulation (Hydrology Scenario 1) indicates that the predicted water-surface elevations would decrease by a maximum of 0.8 feet reaching the vicinity of Central Avenue and upstream from the Montano Bridge under Year 0 conditions, and by up to 0.9 feet under Year 50 conditions. This lowering is caused by the increased conveyance capacity associated with the restoration features, particularly the bank destabilization features that create a wider channel and the connected water features that allow more flow in the overbanks. On average, the water-surface elevations throughout the entire reach will decrease by 0.14 and 0.27 feet for the Year 0 and Year 50 conditions, respectively compared to existing conditions.

Results of the Restoration Alternative 2, Year 0 scenarios predicts that the total inundation will increase from 254 to 262, 88 to 120 and 657 to 726 acres, compared to existing conditions for Hydrology Scenarios 1, 2, and 4, respectively. The channel full simulation (Hydrology Scenario 1) indicates that the predicted water-surface elevations would decrease by a maximum of 0.8 feet between I-40 and Central Avenue Bridge for both Year 0 and Year 50 conditions. On average, the water-surface elevations throughout the entire reach will decrease by 0.02 and 0.17 feet for the Year 0 and Year 50 conditions, respectively compared to existing conditions.

Results of the Restoration Alternative 3, Year 0 scenarios predicts that the total inundation will increase from 254 to 478, 88 to 320 and 657 to 1,112 acres, compared to existing conditions for Hydrology Scenarios 1, 2, and 4, respectively. The channel full simulation (Hydrology Scenario 1) indicates that the predicted water-surface elevations would decrease by a maximum of 0.8 for Year 0 conditions, and by up to 0.9 feet under Year 50 conditions. On average, the water-surface elevations throughout the entire reach will decrease by 0.08 and 0.22 feet for the Year 0 and Year 50 conditions, respectively compared to existing conditions.

Results of the Restoration Alternative 4, Year 0 scenarios predicts that the total inundation will increase from 254 to 364 and 88 to 228 acres, compared to existing conditions for Hydrology Scenarios 1 and 2, respectively. The channel full simulation (Hydrology Scenario 1) indicates that the predicted water-surface elevations would decrease by a maximum of 0.25 feet under Year 0 conditions and by up to 0.58 feet under Year 50 conditions. On average, the water-surface elevations throughout the entire reach will decrease by 0.04 and 0.10 feet for the Year 0 and Year 50 conditions, respectively compared to existing conditions.

Results of the Restoration Alternative 5, Year 0 scenarios predicts that the total inundation will increase from 254 to 467 and 88 to 360 acres, compared to existing conditions for Hydrology Scenarios 1 and 2, respectively. The channel full simulation (Hydrology Scenario 1) indicates that the predicted water-surface elevations would decrease by a maximum of 0.26 feet under Year 0 conditions and by up to 0.64 feet under Year 50 conditions. On average, the water-surface elevations throughout the entire reach will decrease by 0.02 and 0.12 feet for the Year 0 and Year 50 conditions, respectively compared to existing conditions.

An analysis of the overbank sediment-transport characteristics was conducted to evaluate the long term sustainability of restoration features. Overbank flows will cause sediment deposition on the floodplain and sediment deposition will also occur in the proposed channel restoration features particularly after the vegetation has established. An estimate of the amount and rate of sediment deposition within the features was made for Restoration Alternative 1 (Maximum Effort alternative) under the Hydrology Scenario 4 (100-year post-Cochiti flood-flow snowmelt hydrograph) in order to evaluate the long-term sustainability of the proposed features. The amount of overbank sedimentation that would occur during Hydrology Scenario 4 was estimated from the amount of sediment in the main channel water column that would be conveyed on to the overbank over the duration of the hydrograph. Results of the analysis indicate that approximately 12 percent of the suspended bed-material load is transported onto the overbank. The predicted average depth of sedimentation is 0, 0.2, 0.3, 0.1 and 0.1 feet for Subreaches 1 through 5 respectively. For the channel restoration features, it was assumed that 35 percent of the suspended bed load would be conveyed into the features. The estimated amount of sedimentation in the channel features is 0.6, 0.7, 0.9, 0.4, and 0.4 feet for Subreaches 1 through 5, respectively. The amount of sediment deposition on the overbanks appears to be relatively low during the 100-year event. Given the relatively low amount of deposition during large events, the overbank features should not be unreasonably affected by sediment deposition over the 50-year life of the project.

Table 7.2. Summary of total inundation area for Existing Conditions and for the five restoration alternatives.

Alternative	Description	Hydrologic Event / Future Channel Condition	Channel Full Flow (Steady-state)	Annual Snowmelt Hydrograph	100-year Hydrograph
	Existing Conditions	Year 0	253.7	87.9	657.2
		Year 5	251.0	88.1	
		Year 20	246.6	86.4	
		Year 30	247.3	87.9	
		Year 50	243.9	86.3	
1	Maximum Effort	Year 0	796.0	513.1	1,317.5
		Year 5	806.4	537.2	
		Year 20	789.5	498.7	
		Year 30	786.3	503.1	
		Year 50	783.5	500.7	
2	Minimal Effort	Year 0	261.6	120.2	726.7
		Year 5	272.0	121.8	
		Year 20	265.7	110.8	
		Year 30	266.7	118.1	
		Year 50	245.3	118.5	
3	Moderate Effort	Year 0	478.0	319.7	1,111.7
		Year 5	481.5	324.8	
		Year 20	472.7	317.9	
		Year 30	473.8	319.0	
		Year 50	475.4	313.3	
4	Moderate Effort A	Year 0	363.7	228.7	
		Year 5	360.0	245.6	
		Year 20	357.8	213.8	
		Year 30	358.9	214.2	
		Year 50	360.5	212.1	
5	Moderate Effort B	Year 0	466.8	360.2	
		Year 5	464.8	352.9	
		Year 20	465.7	319.9	
		Year 30	466.7	333.0	
		Year 50	455.6	307.6	

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