RIO GRANDE CANALIZATION PROJECT
WATER BUDGET STUDY
Final Report
December 6, 2013

Appendix G1

FLO-2D Modeling and Water Budget Analysis

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1.0 INTRODUCTION

1.1 Study area

The Rio Grande Canalization Project (RGCP) is a 106.8-mile-long[1] river corridor that conveys Rio Grande flows from Caballo Dam in Sierra County, New Mexico, to the American Dam in El Paso County, Texas. Flow releases from the upstream Elephant Butte and Caballo Dams are conveyed by the RGCP for irrigation, water supply and to meet the requirements of equitable distribution of the Rio Grande waters with Mexico based on the Convention of May 21, 1906, entitled "Equitable Distribution of the Waters of the Rio Grande."

The hydraulic modeling discussed in this appendix covers the RGCP from Caballo Dam at the upstream end to a point just above American Dam at the downstream end. As discussed in more detail in the main report (Figure 1) and in Appendix H, the RGCP is subdivided into the following segments:

- Segment 1 - Caballo Dam to Leasburg River Cable metering station
- Segment 2 - Leasburg River Cable metering station to Mesilla Dam
- Segment 3 - Mesilla Dam to the Anthony metering station
- Segment 4 - Anthony metering station to the Below American Dam gage.

As discussed in more detail below, these segments were further divided into 7 subreaches that allowed for spatial variation of the hydraulic conductivity parameters that were used to assess channel seepage.

1.2 Purpose

This appendix documents the development of a two-dimensional (2-D) hydraulic model for the RGCP using the FLO-2D Software. The FLO-2D model was used to estimate water-surface elevations for unsteady flows that were based on reservoir releases, diversions, and returns, and to estimate losses or gains to the channel flows from groundwater interactions.

The following sections summarize the model development, FLO-2D software updates, and model calibration using observed hydrographs during the 2010, 2011 and 2012 irrigation seasons. The calibrated model is then used to evaluate two hypothetical outflow hydrographs: Delayed Single-Pulse Hydrograph (S1) and the Normal Single-pulse Hydrograph (S2) as shown in Figures 6 and 7 of the main report. The hypothetical hydrographs are evaluated during the irrigation season in 2012.

1.3 FLO-2D Model Overview

The initial FLO-2D model calibration simulations indicated that it would be necessary to discretize the global hydraulic conductivity on a reach-wide scale. The model revisions included changes to the code. The accuracy of the calibration effort is limited by several factors especially at the downstream end of the Rio Grande Canalization Project (RGCP) reach:

- Replication of the recorded hydrographs is limited by unmeasured tributary flooding (any storm inflows);

[1] The reported length of the RGCP varies from 105.4 miles for the reach between Percha Dam and American Dam (USIBWC 2012a) to 106.8 miles for the Reach between Caballo Dam and American Dam (RGPAC 2012). Unless otherwise noted, references to the RGCP in this appendix refer to the longer reach.

- Unmeasured irrigation return flows;
- Spatial and temporal variation in the evaporation (shady reaches and cloudy or rainy days);
- Spatial variability in the infiltration rates;
- Inaccuracy in the available river and canal gage data.

The typical inaccuracy in stream gage data is on the order of 5 to 20 percent of the actual flows. Subjectively, the RGCP gages are considered to be accurate to plus or minus about 10 percent. For an irrigation release of about 2,000 cfs, the gages may be incorrect by on the order of 200 cfs. Gages are considered poor if they have a potential error of up to 20 percent. There are many reasons for inaccurate discharge gage reporting. Stage shifts attributed to sediment deposition/scour is one factor for the RGCP reach. Over the course of a dry period, sediment accumulation or scour may cause the primary flows to shift away from the stage recording equipment or the entire local reach may slightly aggrade. Frequent gage calibrations are necessary to maintain RGCP river gage accuracy. It is recognized, however, the RGCP gage record accuracy is highly variable both in space and time.

The resolution of the modeling results may have some impact on the calibration. A daily discharge interval (24 hours) is used for the input data and the reporting output interval is the same. This may distort the hydrograph replication because the timing of the hydrograph spikes and troughs is affected by the daily time-step. Higher resolution results would be obtained with an hourly input and output interval. For the purpose of calibrating the reach wide infiltration hydraulic conductivity, this may not be a significant factor.

Initially, the 2012 irrigation pulse releases were simulated for calibration of the hydraulic conductivity. It was apparent; however, that first release wave had higher seepage rates than the rest of the high flow period. Subsequently, the irrigation release divided into two pulse flow periods for calibration with different hydraulic conductivities. The RGCP reach was also divided into seven subreaches to add spatial variability to the hydraulic conductivity. Following the 2012 calibrations, the 2010 and 2011 irrigation releases were supposed to be simulated with the same infiltration data to determine how well the flows were replicated. Both 2010 and 2011 replications indicated that the replication of the hydrographs was not possible without further varying the seepage estimates. As a result, each year was divided into two irrigation release pulses and the hydraulic conductivity for each pulse was calibrated to better match the recorded hydrographs. The results indicate that 2012 required much higher hydraulic conductivity that the previous years.

2.0 MODEL REVISIONS FOR IRRIGATION PULSE CALIBRATION

2.1 Modifications to Data Files

Several model revisions were undertaken both in the data files and in the model code prior to the model simulations for the calibration to the second irrigation release in 2012. The channel Manning's *n*-values were revised according to flow area at bankfull discharge and slope. During the calibration simulation for the second pulse, some Manning's *n*-values were further revised to reduce numerical surging. The El Paso gage data was made available and this added a new river segment for calibration.

2.2 Modifications to FLO-2D Software Code

FLO-2D code revisions included a decay function for the hydraulic conductivity and a modification to the wetted perimeter calculation. In the original code the wetted perimeter was estimated using the available top width of the channel as an approximation. Since the top-width-based wetted perimeter at low flow is substantially smaller than the actual wetted perimeter, the model code was revised to compute the infiltration for the actual wetted perimeter of the natural cross section.

The hydraulic conductivity for the antecedent moisture conditions in the channel for the first irrigation pulse was made spatially variable by river reach. During the calibration of the first 2012 pulse, it was apparent that a single uniform hydraulic conductivity through the irrigation release period would not be sufficient to calibrate the model. A significant volume of water was required to fill the bank and bed storage of the initial irrigation pulse following the previous eight months of dry channel condition. A modification of the Green-Ampt method was made to enable an exponential decay of the hydraulic conductivity after the infiltated water wetting front reached an assigned limiting flow depth as discussed below. To outline the revision, the following definitions were applied (all variables are unitless unless defined as otherwise):

Porosity: $V_v/V = \phi$ ratio of the volume to the voids (V_v) to the total soil volume (V). V_w is the volume of water in the soil column.

$DTHETA_{np}$: Degree of final saturation S_w - Degree of initial saturation S_i

$$= (V_w/V_v)_{final} - (V_w/V_v)_{initial}$$

Volumetric Soil Moisture Deficit DTHETA: $DTHETA_{np}$ * Porosity

where DTHETA ranges from 0.0 to the Porosity

Soil Depth Storage Limitation F (ft): The available soil column in which water is stored including all the solid, liquid, and air corresponding to the total soil volume V. This is the limiting soil depth for infiltration that is assigned by the user (Figure G1-1).

Available Infiltration Storage Depth F: $= \phi * z_o * DTHETA_{np}$

$= z_o * DTHETA$

Where z_o is the depth (ft) of the piston wetting front (Figure G1-1).

In the FLO-2D model, when the computed wetting front reaches the assigned limiting soil depth, the following occurs:

- For overland flow, the infiltration stops.

- For channel flow, a decay of the hydraulic conductivity H_c (in/hr) from the intitally assigned hydraulic conductivity H_i (in/hr) to a final saturated hydraulic conductivity H_f (in/hr) based on the following equation:

$$H_c = H_f + (H_i - H_f)\, e^{-at}$$

where:

a = decay coefficient hardwired to 0.00002, selected to have the decay from the initial to the final hydraulic conductivity over a 72-hour period with the decay to half the original hydraulic conductivity in 12 hours.

t = time (seconds) from when the wetting front reaches the limiting soild depth.

Additional data to model this temporal variation in the channel seepage are the initial hydraulic conductivity (H_i) at the outset of the simulation, the estimated final saturated conductivity (H_f) at the end of the simulation and the infiltration storage soil depth. The soil storage depth triggers the model to switch to lower final hydraulic conductivity to account for the filling of the bed and bank storage. The bank storage represents a horizontal infiltration that is not accounted for in the Green-Ampt methodology. As a result, it is necessary to use relatively large storage depths to represent the filling of all the channel loss to infiltration (vertical and horizontal).

The initial calibration attempt for the first 2012 pulse justifies this approach. Starting at the Leasburg gage, a significant volume of water that constituted the frontal wave was predicted to exceed the measured hydrograph. This simulated slug of extra water persisted through the remaining downstream gages. It was presumed that an initially high hydraulic conductivity representing the seepage storage that gradually decremented to a saturated value after the frontal wave passed would have to be simulated.

3.0 ACCURACY OF THE AVAILABLE GAGE DATA

In the original 2004 FLO-2D model calibration effort using a uniform and steady hydraulic conductivity, calibrations were made to 2004, 1995 and 1998 data. At that time, some of the gage data was considered to be inaccurate by up to 20 percent. In this current study, IBWC indicated that several gages (Haynor, Picacho and New Anthony) were inaccurate. When reviewing the following FLO-2D-predicted hydrograph replications with the measured gage data, there were several gage record inconsistencies that can be observed. Since the model conserves volume, when a portion of the predicted hydrograph at one gage does not match the measured hydrograph, there are several possibilities:

- If the measured is greater than the predicted discharge, there may be tributary inflows or the predicted seepage rates are too high.
- If the measured hydrograph is less than the predicted hydrograph, there may be unmeasured diversions or the predicted seepage rates may be underestimated.
- If the difference between the measured and predicted gages for a portion of the hydrograph occurs at only a single gage and not at successive downstream gages, the river gage data may be inaccurate.
- If the discrepancy occurs downstream at either Picacho or New Anthony and the difference persists to downstream gages, the diversion canal gage data may be inaccurate.

In the following plotted hydrographs it will be apparent when the gage data is poor.

There are several observations that can be made regarding the gages:

- In general, all of the RGCP gages represent a consistent shape in terms of magnitude and timing with observed spikes and dips occurring at the appropriate times for each gage hydrograph. Using all the gages, including those which are considered to be inaccurate, enables the hydraulic conductivity to be adjusted to match the shape and thereby better match the measured hydrographs for the gages that are considered to be accurate. The following plots will demonstrate this. The value of using all the gages is that it discretizes the hydraulic conductivity between the accurate gages into smaller reaches. Using all the gage data does not affect the results at the accurate gages.
- Some gages may have some responses associated with sediment deposition/scour that results in some minor shifts in stage over time. This could be as much as 0.5 to 1.0 feet, which would result in a 250-cfs error. A shift in the gage rating curve would result in one gage having a period when the discharge was consistently higher or lower than the next upstream or downstream gage. In the case of a gage shift, the predicted discharge may be initially high and then may be lower than the measured discharge for the rest of the hydrograph. In these cases, the hydraulic conductivity was not further adjusted to match the measured (possibly inaccurate) data.
- The gage that appears to have consistently poor replication of the recorded data is the Below American Dam gage with its companion American Canal diversion gage. The combination of these gages (river and diversion canal gage) when compared to the upstream El Paso gage (Figure G1-2) indicates that removing the American Canal diversion water from the FLO-2D model on a daily average basis may affect the calibration of the river gage in both timing and magnitude. Poor replication of the El Paso gage by 50 cfs can result in zero predicted discharge at the Below American Dam gage. This occurs in essentially every pulse simulation. This is because unrecorded return flow or tributary inflow data is diverted as recorded by the American Canal gage, but there

may be very little or no water in the river model. The replication may be poor because of poor diversion records, tributary inflow, unmeasured irrigation or wasteway return flows. The reported canal data has daily variability of 20 cfs or more as shown in Figure G1-3, but the predicted hydrograph at Below American Dam gage in Figure G1-4 is different by 40 to 50 cfs (red and blue ovals). The American Canal diversion record reflects the El Paso gage data in shape, but the difference of 50 cfs in the predicted versus the measured data in Figure G1-2 results in a large spike in Figure G1-4. While a more detailed data base may improve the replication at the Below American Dam gage, the hydraulic conductivity estimates should not be appreciably affected.

While it is understood that some of the river gages are less reliable than others, the cross section flow areas throughout the river system are very consistent. This is due in part to the canalization as well as relatively uniform annual channel forming flows. The cross sections have evolved over the years to the irrigation release as the channel forming flow. Variations of stage with discharge are also limited at high flows because of the limited sediment loading. There is not a lot of deposition/scour for flows within the irrigation flow regime that are less than bankfull discharge. For these reasons, the measured hydrographs in this report appear to be relatively consistent with the predicted flows. At each gage, the predicted versus measured spikes and dips and the shape of the hydrographs can be tracked through the system and justifies using all the gage data.

4.0 IRRIGATION PULSES

The irrigation block releases for a given year were divided into two pulses. This was done because the first pulse is typically a flushing flow pulse and has higher seepage because the channel has been dry throughout the winter. The assigned hydraulic conductivities that were calibrated to replicate the measured hydrographs are consistently different between the two pulses. Some pulses were not separated by a period of zero releases from Caballo Reservoir. In these cases, a relatively low discharge period of several days was used to distinguish between the spring flushing pulse release and the summer irrigation release. Even though the 2012 irrigation releases were calibrated first, the results are presented chronologically starting with the first pulse in 2010.

4.1 First 2010 Irrigation Pulse Calibration

The 2010 first irrigation pulse release extended from March 1 to April 23. The results are presented in Figures G1-5 through G1-11.

Comments on the First 2010 Irrigation Pulse Calibration

The replication of the Haynor gage is excellent, but there is an errant discharge spike in the measured data that probably represents a tributary inflow since there is a small spike in the measured hydrograph at Leasburg. At all the gages downstream of and including the Leasburg gage, it appears that the predicted hydrographs are too low during the initial portion of the hydrograph and too high during the latter portion. This may indicate that the predicted seepage volume is too high for the first half of the hydrograph and too low for the remainder of the hydrograph. It may also mean that the Leasburg diversion gage record is inaccurate, resulting in a possible shift in the canal diversion gage data. This difference is almost 200 cfs at the Leasburg gage, which is too large to be accounted for using the infiltration computation. As a result no further calibration attempt is made to account for the difference. Note that this extra volume predicted by the model results in a significant difference at the Below American Dam gage. As can be observed in Figure G1-8, there is only a partial Mesilla gage record. It also has a discharge spike that does not appear in the downstream measured hydrographs.

4.2 Second 2010 Irrigation Pulse Calibration

The 2010 second irrigation pulse release extended from April 25 to July 10. The results are presented in Figures G1-12 through G1-18. There were several tributary flows that distort the calibration attempt for the remainder of second 2010 irrigation pulse release after July 10.

Comments on the Second 2010 Irrigation Pulse Calibration

Replication of the gage record is relatively good for all the gages except Mesilla and Below American Dam. The Below American Dam has the issues with the river and canal diversion gage records as previously discussed. The Mesilla gage appears to a have a period of missing discharge on the order of about 500 cfs for about 400 hours or more beginning about June 5. This missing portion of the hydrograph re-appears at the New Anthony gage and all subsequent gages. At the remainder of the gages, the predicted results are in general slightly less than the measured values during the first 500 hours of the simulation and are slightly greater during the remainder of the simulations

4.3 First 2011 Irrigation Pulse Calibration

The 2011 first irrigation pulse release extended from March 7 to May 3. The results are presented in Figures G1-19 through G1-25.

Comments on the First 2011 Irrigation Pulse Calibration

There appears to be a small tributary inflow spike near the peak discharge at the Haynor gage that persists through the downstream gages. An additional inflow results in more discharge after hour 1300 that appears in all the gage records. This discharge discrepancy is on the order of 100 cfs and affects the hydrograph timing at the end of the simulation. Through a large portion of the middle of the hydrograph a 200-cfs discrepancy is introduced at the Mesilla gage. This could be tributary inflow flooding or inadequate diversion records since this block appears in all the downstream gage records. Finally, there is a large dip in the recorded discharge at the New Anthony gage near peak flow. This is clearly a gage record error that is inconsistent with upstream and downstream gages.

4.4 Second 2011 Irrigation Pulse Calibration

The 2011 second irrigation pulse release extended from May 4 to September 15. The results are presented in Figures G1-26 through G1-32.

Comments on the Second 2011 Irrigation Pulse Calibration

The second irrigation pulse in 2011 is replicated very well at the Haynor, Leasburg and Picacho gages over the duration of the pulse. At the Mesilla, New Anthony, and El Paso gages, the predicted flows are about 200 cfs higher than the measured flows from about 2200 hours to 2800 hours. A plausible explanation is that the available discharge record at Mesilla diversion canals or return locations is inaccurate during this period. Because the timing and shape of the predicted hydrograph matches the measured hydrographs, this discrepancy cannot be attributed to underestimated seepage volumes. A seepage rate of 200 cfs is too large to account for with increased infiltration. A careful review of the available diversion gage data should be considered, and any additional or revised diversion data should be incorporated into the model.

4.5 First 2012 Irrigation Pulse Calibration

The first irrigation pulse in 2012 was the first block flow to be calibrated and extended from April 1 to May 8 when the discharge release from Caballo Dam dropped to below 100 cfs. Prior to April 1, the last day of release from Caballo Reservoir was September 10, 2011. Any significant flow in the river during this dry period was the result of storm activity and while one storm inflow may have been recorded at the Haynor gage, no flows over 20 cfs were measured during this period at the El Paso gage. The discharge in the river channel was either negligible or a locally small base flow for about 7 months resulting in a relatively dry channel bed and banks. High hydraulic conductivity would be necessary to fill the bank storage during the first irrigation release.

A series of calibration runs were made on this first pulse flow. At the outset, the FLO-2D model was predicting a floodwave that arrived too soon at the lower gages. The hydrograph differences were attributed to the use of the original channel cross section n-values and the need for further adjustment of the channel hydraulic conductivity. A second series of calibrations for the first 2012 irrigation pulse were conducted. Some modifications to the Manning's *n*-values were made on a reach wide basis to speed up or slow the frontal wave movement. In addition, the model was revised to use the new infiltration computation with the decaying hydraulic conductivity. The following conclusions were reported after this calibration attempt:

- The increase in the initial hydraulic conductivity by maintaining a lower final hydraulic conductivity enables the frontal wave to be reduced.

- A reduction in the frontal wave discharge also slows down the arrival of the frontal wave and as a result there may be poor correlation with the frontal timing at the downstream gages while the recessional limb of the hydrograph is still fairly well calibrated.
- After the frontal wave passes the FLO-2D model does a very good job of replicating the shape of the hydrograph (spikes and dips).

There was also a potential issue of missing Leasburg diversion data during this release, and no information about the quality of this data is available on the EBID website. Leasburg first check diversion data appeared to indicate that irrigation diversion flows occurred during the first irrigation pulse in April 2012. After running a number of calibration simulations, it was apparent that there were still some timing issues with the assumed diversion and another investigation of the 2012 Leasburg diversion was launched. The EBID staff was contacted regarding the data. It was learned that the Check 1 gate data and the diversion discharge listed at the EBID web site was based on testing the gate operation during dry canal conditions and that no diversions at Leasburg dam were actually made during the first pulse period. As such, a third and final attempt at calibrating the first pulse was undertaken. The results of that calibration effort are presented in Figures G1-33 through G1-39.

Comments on the First 2012 Irrigation Pulse Calibration

The pulse release hydrograph is accurately replicated through the New Anthony gage. At the El Paso and Below American Dam gages, the peak discharge is over-predicted and the frontal wave lags slightly. Evidence that this is occurring begins at the Mesilla gage. The recessional limb of the predicted hydrograph matches the reported data very well.

Replicating the Below American Dam gage hydrograph requires further discussion. The discharges being replicated at Below American Dam are relatively low (~200 cfs). The differences between the predicted and measured discharges (ignoring differences in the frontal wave) are typically on the order of 50 cfs. At least half of that difference can be attributed to ungaged upstream returned flows after the peak discharge (after 350 hours). Some of the discrepancy is also related to the use of daily diversion estimates. Inputting the diversion data on an hourly basis, instead of a daily average value, would improve the replication of the gage data. Since two gage recordings are required for the calibration at this gage, if a presumed level of accuracy of 10 percent is estimated for both gages, this could also account for some of the difference in the predicted versus measured differences.

4.6 Second 2012 Irrigation Pulse Calibration

The second pulse calibration simulation begins May 9 and extends to June 29, 2012. The actual release extends into September, but the flows remain relatively high and the infiltration rates are expected to be the same through the remainder of the hydrograph. (As discussed below, a model run for the entire 2012 release was made using the calibrated 2012 first pulse model for input to the water budget study.) There were several issues encountered with this block release. Numerical surging and instability at the Tonuco Bridge were attributed to the higher discharge. The bridge rating table was adjusted and the instability was eliminated. In addition, USIBWC requested that the discharge data from several gages be ignored as the data was presumed have poor accuracy (Dr. Al Blair, pers. comm., November 2012). Several days were spent calibrating the runs without the Haynor, Picacho and Anthony gages data. As discussed above, further review of the gage data and modeling results indicated that the overall shape and response of the measured gage hydrographs was consistent. The calibration results of the second pulse of 2012 are presented in Figures G1-40 through G1-46.

Comments on the Second 2012 Irrigation Pulse Calibration

There are three notable issues with the second 2012 pulse calibration: (1) downstream from Leasburg the model once again predicts too much discharge in the pulse frontal wave, (2) at the Mesilla and New Anthony gages, the timing of the pulse frontal wave is off by about 12 hours, and (3) there was an unknown inflow (return flow or tributary inflow) upstream from Haynor that occurred during the first 6 days of the simulation.

Except for the above mentioned items, the calibration is generally very good for the second pulse. It is noteworthy that in the first pulse, the FLO-2D model under-predicts the arrival of the frontal wave (the model predicts a slower arrival) for the Mesilla, New Anthony and El Paso gages whereas for the second pulse, the model over predicts the frontal wave speed (frontal wave arrives too soon) for the same gages. The previous discussion regarding the Below American Dam gage replication again applies to the second pulse replication. The measured discharge is only about 40 cfs, and the river and canal diversion gages are combined to generate the model river flow. To reiterate, some of the difference is attributed to the return flows and some of the difference is the daily diversion values that are used instead of hourly data.

4.7 Hypothetical Release Scenarios

In accordance with the Scope of Work (USIBWC 2012a and 2012b) the calibrated FLO-2D model was used to analyze two hypothetical irrigation release scenarios from the upstream reservoirs to predict the impact on channel seepage and other water budget components.

The analyses were conducted for a delayed single-pulse hydrograph (Release Scenario S1) and a normal single-pulse hydrograph (Release Scenario S2) provided by the RGPAC. Plots of the inflow hydrographs are included in Figures 6 and 7 in the main report, respectively. Both of the hypothetical release scenarios have a release volume that is identical to the actual (baseline) 2012 release of 372,028 acre-feet.

As discussed in more detail in **Appendix H** (HEC-RAS Modeling), the diversion flows under Scenario 1 were adjusted to eliminate the portions of the diversions that occurred prior to the delayed release. This adjustment involved shifting the timing and adjusting the magnitude of the diversions at Percha and Mesilla Diversion Dams to preserve the overall volume of the actual diversions that occurred at these locations in 2012. Likewise, the diversions under Scenario 2 were adjusted to prevent diversion flows from exceeding the inflow hydrograph. This adjustment involved shifting the timing and adjusting the magnitude of the diversion the Mesilla Diversion Dam to match 2012 diversion volume. The adjusted hydrographs that were used in the HEC-RAS modeling were also used for the FLO-2D modeling of Scenarios S1 and S2.

A comparison of the predicted hydrographs under Scenarios S1 and S2 with the predicted baseline 2012 hydrograph at the primary gages along the study reach is shown in Figure G1-47 (Haynor), Figure G1-48 (Leasburg), Figure G1-49 (Picacho), Figure G1-50 (Mesilla), Figure G1-51 (Anthony) and Figure G1-52 (El Paso).

5.0 ESTIMATED SEEPAGE VOLUME AND EVAPORATIVE LOSSES

Table G1-1 presents the FLO-2D predicted seepage results for the irrigation pulse releases. The SUMMARY.OUT file reports volume conservation accurate to millionths of 1 percent (or less than about 0.01 acre-feet). Model volume is conserved and essentially every drop of inflow water in the model is accounted for. The total inflow represents the combined release volume from Caballo Dam plus nominal irrigation return flow.

The disposition of the water at the end of the irrigation pulse simulation indicates the water volume that is lost to infiltration and evaporation as a percent of the Caballo Dam release plus nominal irrigation return flow (last row of numbers in Table G1-1). This estimate does not include potential evaporation variability or potential tributary flooding.

For the 2010 and 2011 irrigation block release, the average percent flux to infiltration and evaporation is about 14 percent of the inflow volume. The fluxes more than double in 2012 with almost 50-percent flux in the first irrigation pulse release. If it is assumed that the El Paso gage is accurate, the flux will be even higher because of the extra volume near the peak of the frontal wave that is not indicated in the predicted model results (Figure G1-38). The average evaporation loss is 2.4 percent and depends on the wetted surface area in the channel and date (summer evaporation loss is higher than spring evaporation loss). The actual channel evaporation is subject to humidity, direct sunlight on the river and wind conditions. It is expected that the variability in the evaporation losses may be on the order of 25 percent. This implies that the channel infiltration volume is the overwhelming factor in the model calibration.

By comparison, the 2004 calibration had only 5.5-percent flux (seepage and evaporation) out of 106,280 acre-feet of inflow volume. The fluxes in 2010 and 2011 are 3 times greater than the 2004 fluxes, and the fluxes in 2012 are over six times greater. The release volume for the 2011 first pulse and the 2012 second pulse were approximately the same. The increase in infiltration volume (seepage) may be attributed to more available bank and bed channel storage (antecedent moisture conditions) and a potential lowering of the groundwater levels in the vicinity of the river.

A comparison of the FLO-2D results to the USIBWC (1993) seepage study was also carried out to evaluate how the model-based seepage estimates compare with historical observations. The gains and losses reported in the UISBWC (1993) are based on measured flows at the river, diversion, and wasteway gages as well as estimated wasteway discharges, so the reported gains and losses include all unknown inflows and outflows (i.e., channel seepage, evaporation, ungaged or unknown inflows and outflows, etc.), so they may not provide a direct comparison with the predicted seepage volumes from the FLO-2D modeling. The values do, however, provide a range of gains and losses that can be used to check the reasonableness of the FLO-2D results.

The USIBWC (1993) study was conducted by the USBR, and includes estimated gains and losses based on 7 years of gage data from 1986 to 1992. The gages (shown in Figure 1 of the main report) included in the study were located between Picacho Bridge and El Paso (Courchesne Bridge), and provide estimates of gains and losses in the overall reaches from Picacho Bridge to Courchesne Bridge (1991 and 1992 only) and Mesilla Dam to Courchesne Bridge (1986 to 1992), as well as for the shorter subreaches from Picacho Bridge to Mesilla Dam, Mesilla Dam to Vado Bridge, Vado Bridge to Canutillo Bridge, and Canutillo Bridge to Courchesne Bridge. Results from the study indicate that the overall study reach from Picacho Bridge to Courchesne Bridge was a losing segment between 1991 and 1992, and that the reach between Mesilla Dam and Courchesne Dam was a losing reach from 1986 to 1992, both of which are consistent with the FLO-2D model results. A comparison of the seepage rates predicted by the FLO-2D model and the losses reported in USIBWC, with normalized rates on a per-unit-mile basis, is presented in Table G1-2.

The comparison of the reported losses and predicted seepage rates on a subreach basis indicates that the maximum predicted seepage rate is somewhat less than the maximum loss reported in USIBWC (1993), but is more consistent than the maximum seepage rates indicated by the HEC-RAS model (see Table 7 in the main report). The average predicted seepage rates, however, appear to be somewhat less than the reported losses. In the 1991 and 1992 period, the maximum losses reported in USIBWC (1993) between Picacho Bridge and Courchesne Bridge was 24.4 acre-feet/day/mile, which is slightly larger than the maximum seepage along the overall RGCP that is predicted by the FLO-2D model (20.5 acre-fee /mile/day). The average seepage rates predicted by the FLO-2D model in the RGCP are about 2.1 acre-feet/mile/day, compared to average loss rates of 1.3 acre-feet/mile/day as indicated in UBIBWC (1993) from Picacho Bridge to Courchesne Bridge.

6.0 HYDRAULIC CONDUCTIVITY VARIABILITY

6.1 Hydraulic Conductivity Spatial Variability

The hydraulic conductivity is assigned by river reach (see Figure 1 in the main report):

Reach 1. Caballo Dam to Haynor Bridge

Reach 2. Haynor Bridge to Leasburg Dam

Reach 3. Leasburg Dam to Picacho Bridge

Reach 4. Picacho Bridge to Mesilla Dam

Reach 5. Mesilla Dam to Anthony Bridge

Reach 6. Anthony Bridge to Courchesne Bridge

Reach 7. Courchesne Bridge to American Dam

Table G1-3 (initial calibration run) and Table G1-4 (final calibration run) indicate the range of variability in attempting to calibrate the 2012 block release hydrographs.

From the calibration effort, the first irrigation pulse indicates very high initial hydraulic conductivity that generally tapers to a small final value as the infiltration bank storage fills. The second irrigation pulse that follows the end of the first release by about one week requires about the same initial hydraulic conductivity. The final hydraulic conductivity for the recessional limb of the second pulse is relatively consistent with the final hydraulic conductivity of the first pulse. A reasonable calibration for the two irrigation pulses is produced by varying the hydraulic conductivity in both time and space. The initial and final hydraulic conductivities for the first and second pulses are similar to values reported by others (TRC, 2010). The soil survey that was conducted for USIBWC (TRC, 2010) indicates that the majority of soils that make up the channel banks are of the Agua variant soils (AJ), Agua variant and Belen variant soils (AK), and Brazito loamy fine sands (Br), which have hydraulic conductivities that generally range from 0 to 2 in./hr, although the Br soils have hydraulic conductivities that range from 6 to 20 in./hr. These values are somewhat lower than the K_{sat} values presented in NMWRRI (2008).

Initial and final hydraulic conductivities for the first and second pulses that were used in the calibrated FLO-2D models for the 2010 through 2012 period are presented in Table G1-5 and Table G1-6, respectively. These values show a generally lower hydraulic conductivity in the second pulse compared to the first pulse, and also indicate that the hydraulic conductivity significantly increases from 2010 to 2012. Limiting storage depths that were used to calibrate the 2010 through 2012 models for the first and second pulses are presented in Table G1-7. The limiting storage depths were then used to develop recommendations for storage depths under wet, average and dry conditions (Table G1-8). A single storage depth is recommended for each reach under wet and average conditions because the calibrated storage depths do not change significantly between 2010 and 2011, while a separate value is recommended for dry conditions. The values used for the 2010 and 2011 pulses are also consistent with values reported by others (TRC, 2010). It should be noted that the limiting storage depths used in the FLO-2D model are somewhat larger than the sediment thickness values that were used in the HEC-RAS modeling. As stated above, it is necessary to use relatively large limiting storage depths to account for bank (horizontal) infiltration that is not accounted for in the Green-Ampt methodology as applied in the FLO-2D modeling. Considering the differences between the Green-Ampt methodology used in the FLO-2D model and the Darcy equation used in the HEC-RAS model, a direct comparison of the limiting storage depth (FLO-2D) and the sediment thickness (HEC-RAS) is not appropriate. However, both models indicate that variation in these parameters have a significant effect on channel seepage (see Section 2.4.3 of Appendix H).

6.2 Sensitivity Analysis of Hydraulic Conductivity

In accordance with the Scope of Work (USIBWC 2012a and 2012b) a sensitivity analysis was performed to evaluate the effects of changes in K_{sat} on the groundwater interflow. Three separate FLO-2D models were run with initial and final K_{sat} increasing by 10, 20, and 30 percent over the base values, and three models were run with initial and final K_{sat} decreasing by 10, 20, and 30 percent. The initial and final K_{sat} values for the baseline model and the six sensitivity models are summarized in Table G1-9. As discussed in more detail in the following sections, FLO-2D model runs for hypothetical release scenarios (Scenarios S1 and S2) were carried out to evaluate the effects of release timing and magnitude on the water budget components. As a result, two separate sets of sensitivity model runs were developed for each of the hypothetical scenarios. Results from the sensitivity analysis of hypothetical release Scenario S1 are presented in Table G1-10 and Table G1-11, while the results from the analysis of hypothetical release Scenario S2 are presented in Table G1-12 and Table G1-13.

7.0 WATER BUDGET ANALYSIS

In general, the water budget analyses that were conducted using the results from the FLO-2D modeling were very similar to the analyses that were based on the HEC-RAS modeling results that are discussed in the main report. Consistent with the HEC-RAS-based analyses, RGCP-scale channel water budget analyses were prepared for the four segments that extend from (1) Caballo Dam to Leasburg River Cable metering station, (2) Leasburg River Cable to Mesilla Dam, (3) Mesilla Dam to the Anthony metering stations, and (4) Anthony metering station to the Below American Dam gage. A separate evaluation of the water budget components was carried out within the local basin that included surface water and groundwater budgets. The analyses were conducted for the entire study period from January 1, 2010, through November 30, 2012, for the 2012 baseline condition (March 31 through November 14, 2012), and for the hypothetical delayed single pulse (Scenario S1) and normal single pulse (Scenario S2) releases that were developed by the Rio Grande Project Allocation Committee for 2012 conditions. The FLO-2D model that was calibrated for the 2012 baseline condition (first pulse) was used to simulate the hypothetical S1 and S2 release scenarios to predict hydrographs at key locations along the study reach, and to estimate channel seepage and evaporation rates that were used as input to the water budget analyses. Because the calibration runs for the 2012 irrigation release included two separate model runs for the first and second release pulses, the model that was calibrated for the first pulse was rerun for the entire 2012 irrigation release period that extended from April 1 to September 14, 2012, to provide a direct comparison to the S1 and S2 hypothetical release scenarios.

7.1 Water Budget Input

Except for the hydrographs, seepage rates and evaporation losses that are predicted by the FLO-2D simulations, input to the water budget analyses was identical to that used for the HEC-RAS-based water budget analyses that is discussed in the main report. As discussed above, the FLO-2D simulations do not extend over the entire study period due to excessive computer run times. Table G1-14 summarizes the periods covered by the FLO-2D simulations. To provide input to the water budget analyses, the FLO-2D results were supplemented with a variety of information, as outlined in the following sections.

7.1.1 Predicted Hydrographs

The predicted hydrographs from the FLO-2D model were used to define the upstream inflow (Q_{cus}) and downstream outflow (Q_{cds}) for each channel segment of the RGCP-scale channel water budget analysis, and to define the surface water inflow (Q_{us}) and outflow (Q_{ds}) components of the local basin scale water budget analysis. The hydrographs were also used to compute the stormwater/ungaged return flow component (Q_{cin}) for the RGCP-scale channel water budget. For the periods when no FLO-2D output was used, the results from the HEC-RAS model were used to complete the hydrograph (Figure G1-53). It should be noted that the measured values could also be used to complete the FLO-2D hydrographs, but this option was not selected because the measured values at the Anthony metering station were presumed not to be accurate by Dr. Al Blair (RGPAC, pers. comm., November 2012) and use of a consistent (gage-based or model-based) dataset is necessary for the comparison of the water budget analyses among the baseline and hypothetical scenarios. It should be noted that similar to all the RGCP gages, the Anthony gage has periods of accurate and inaccurate measurements. As discussed in Appendix H, the HEC-RAS modeling required a 0.5-foot tolerance during periods of low flows to achieve numerical stability, and this tolerance resulted in erroneous baseflows during these periods. The erroneous baseflows were identified through visual observation of the results, and broken down by subreach as presented in Table G1-15. Consistent with the HEC-RAS-based water budget analyses, predicted hydrographs from the HEC-RAS were adjusted to account for the erroneous baseflows by subtracting the observed erroneous baseflows from the predicted flows during periods of low flow

(Table G1-15). For the S1 and S2 release scenarios, the FLO-2D simulations covered the entire duration of the release, so it was not necessary to supplement any hydrograph data.

7.1.2 Predicted Seepage Rates

As discussed in more detail above, the FLO-2D model was used to predict seepage rates for the first and second pulses in 2010, 2011 and 2012, and for the hypothetical release scenarios. To supplement the seepage rate time series that were not included in the FLO-2D simulations, seepage rates were either duplicated from the last day of the FLO-2D simulation or adopted from the HEC-RAS results. For the first and second pulses in 2010 and 2011, the seepage rates do not vary significantly during the latter portions of the pulse, so the by-segment seepage rates that were predicted on the last day of the simulation in 2010 were duplicated for the remaining portion of the pulse that was not covered by the FLO-2D simulation (Figure G1-54). The FLO-2D simulation for the 2011 and 2012 releases covered the entire release period, so it was not necessary to supplement any seepage rate information for these releases. During the non-irrigation season of each year, when very little or no water is released from Caballo Reservoir; the HEC-RAS-based seepage rates were adopted. For the S1 and S2 release scenarios, the FLO-2D simulations covered the entire duration of the release, so it was not necessary to supplement any seepage rate information.

The resulting seepage rates and volumes for the 2010 to 2012 period are somewhat different than the values that are used in the HEC-RAS-based water budget analyses (Table G1-16). The maximum predicted seepage rates are significantly larger for the FLO-2D-based water-budget analysis in each of the segments and along the overall RGCP due to the high hydraulic conductivity values that are used during the initial portions of the annual release. However, the average seepage rates that are used in the FLO-2D water budget are only higher in Segment 1, but are somewhat less in Segments 2 and 4, resulting in a slight decrease to the overall average seepage rate along the RGCP. The same can be said for the overall seepage volume. Interestingly, the average and overall seepage rates in Segment 3 are nearly identical to the seepage rates predicted by the HEC-RAS model in that segment.

7.1.3 Predicted Evaporation Rates

Due to the limited duration of the FLO-2D runs, it was also necessary to supplement the FLO-2D-based evaporation estimates during portions of the study period not covered by the FLO-2D simulations. For periods when no FLO-2D model results were available, the by-segment evaporation rates that were developed for the HEC-RAS-based water budgets were adopted (Figure G1-55).

For the S1 and S2 release scenarios, the FLO-2D simulations covered the entire duration of the release, so it was not necessary to supplement any evaporation rate information. The resulting evaporation rates and volumes are significantly higher than the rates that were used in the HEC-RAS-based water budget study (Table G1-17). Compared to the HEC-RAS-based values, the total volume of evaporation over the 3-year period increases by about 52 percent along the overall RGCP. The total FLO-2D-based evaporation volumes decrease in the downstream direction from about 14,600 acre-feet in Segment 1 to about 5,500 acre-feet in Segment 4. The average FLO-2D-based evaporation rate ranges from 2.6 cfs in Segment 4 to 6.9 cfs in Segment 1, and the maximum evaporation rate ranges from 6.7 to 17.8 cfs in those two segments, respectively.

7.2 RGCP-Scale Channel Water Budget Analysis— Results

7.2.1 Entire Study Period—2010 through 2012

Results from the RGCP-scale channel water budget analysis were evaluated to assess the effects of the individual components on the water budget, and to assess the change in channel storage along each reach. It should be noted that, due to limitations in inflow/outflow information (missing or inaccurate gage data and potential error in estimated or modeled values), a significant portion of the predicted change in storage is likely associated with unknown fluxes into or out of the system. For purposes of this discussion, segments that have a negative change in channel storage will be referred to as losing segments, while segments that have a positive change in channel storage will be referred to as gaining segments. The RGCP-scale channel water budget spreadsheet is included in **Appendix G2**.

As expected, the most significant components of the RGCP-scale channel water budget, in terms of magnitude, are the upstream inflow and downstream outflow (Table G1-18 and Table G1-19). Diversions and channel seepage are the next significant components, followed by irrigation return flow (primarily in Segments 3 and 4) and treated effluent return flow (at least in Segments 2 and 4). The remaining components are much less significant. The total annual volumes indicate that Segments 1, 2 and 4 are slightly gaining segments and that Segment 3 is the only losing segment (Table G1-18). Segment 2 has the most significant gains with the largest gains indicated in 2010. In Segment 3, the most significant losses on a percentage basis occur in 2012. Although the predicted change in volume of channel storage is relatively high in each of the segments, the total change in volume is less than 5 percent of the upstream inflow in each of the segments at the end of the 3-year study period, and the total increase to storage along the RGCP is about 7 percent at the end of the 3-year study period (Table G1-19). Hydrographs of the total inflow, total outflow, and change in storage for each of the segments are included in **Appendix G2**.

Inflows to Segment 1 are greater than the modeled and measured outflows from this segment during the majority of the study period (Figure G1-56), resulting in a net gain of about 42,700 acre-feet. Most of the losses occur during the 2011 irrigation season, while most of the gains occur during the non-irrigation season between 2010 and 2011. The primary gains along this reach are stormwater/ungaged return flow and groundwater return flow, while diversions and channel seepage account for the majority of the losses (Figure G1-57; Table G1-18 and Table G1-19). The total downstream outflow increases from about 75 percent of the upstream inflow in 2010 to about 90 percent in 2011, but decreases back to 75 percent in 2012, and makes up about 79 percent of the upstream inflow at the end of the 3-year period (Table G1-18 and Table G1-19).

Segment 2 is a more significantly gaining segment compared to Segment 1. The total increase to channel storage at the end of the 3-year study period is about 58,200 acre-feet, or about 5 percent of the upstream inflow (Table G1-18 and Table G1-19). The most significant gains result from effluent and irrigation return flows along the segment (Figure G1-58). Downstream outflows are about 63 percent of the upstream inflow, while diversions and seepage make up another 32 and 6 percent, respectively, of the upstream inflow (Table G1-19 and Figure G1-59).

Segment 3 is generally a losing segment in each of the 3 years, with a total decrease to channel storage of about 20,900 acre-feet at the end of the 3-year period. Although irrigation return flows are relatively high along this segment, this component is more than offset by the high downstream outflows and significant seepage that is most significant during the irrigation seasons (Figure G1-60). Irrigation return flows are more significant than in upstream segments, especially in 2010, but diminish with time. Downstream outflows exceed the upstream inflow from Mesilla Dam in 2010 and 2011, but also diminish with time to about 76 percent of the upstream inflow in 2012 (Table G1-19 and Figure G1-61).

In Segment 4, the modeled inflows exceed the modeled outflows by about 25,500 acre-feet by the end of 2012, or by 4 percent of the modeled inflow delivered at the Anthony metering station. Similar to Segment 3, the majority of the gains occur as a result of the large volumes delivered to the channel by irrigation return flows that occur in 2010 and the relatively large effluent inflows from the Northwest Wastewater Treatment Plant (Figure G1-62). Downstream outflows exceed the upstream (modeled) inflow at Anthony in 2010 and 2012, but are only 93 percent of the upstream inflow in 2011 (Table G1-19 and Figure G1-63).

Along the overall RGCP, the most significant inflows are obviously the upstream inflow from Caballo Reservoir, followed by irrigation return flows, treated effluent inflows and stormwater/ungaged return flows, with somewhat less significant effects from groundwater return flows (Figure G1-64 and Figure G1-65). The most significant outflows along the overall RGCP are the downstream channel outflow at American Dam and authorized diversions, followed by channel seepage, evapotranspiration and floodplain recharge, in that order (Figure G1-64 and Figure G1-65).

7.2.2 Delayed Single Pulse (S1) and Normal Single Pulse (S2)

Channel water budget analyses at the RGCP scale of the delayed single pulse scenario (Scenario S1) and the normal single pulse scenario (Scenario S2) are presented in **Appendix G4** and **Appendix G5**, respectively, and were carried out for the 2012 condition and compared to the results from the baseline channel water budget for actual 2012 conditions (**Appendix G3**).

Although the water budget analysis includes an evaluation of the individual water budget components, it is worth noting that the total RGCP FLO-2D-based seepage volume during the 2012 release period (May 31 through September 14) would decrease from about 104,600 acre-feet under baseline conditions to about 84,000 acre-feet under Scenario S1, but is very similar to the predicted 104,000 acre-feet under Scenario S2. This is to be expected, since the period over which seepage occurs is somewhat less under Scenario S1 but is identical under Scenario S2.

Results from the analysis are summarized in Table G1-20 and Table G1-21 and were used to evaluate the change in channel storage over the course of 2012 that would result under the hypothetical single-pulse releases (Scenarios S1 and S2) compared to the actual (baseline) double pulse release that occurred in 2012. This comparison indicates that, by the end of the release, Scenario S1 would result in a moderate increase to the net gain of in-channel storage in Segment 1, and Scenario S2 would result in a less significant gain in storage (Figure G1-66). Compared to baseline conditions, the total volume of seepage in Segment 1 (about 26,300 acre-feet under baseline conditions) would decrease by about 5,000 acre-feet under Scenario S1 (about 21,400 acre-feet) and increase slightly by about 1,700 acre-feet under Scenario S2 (about 29,000 acre-feet). Both the downstream outflow and stormwater/ungaged return flow components are higher than the baseline condition under Scenario S1, with magnitudes increasing by about 4,800 acre-feet and 200 acre-feet, respectively; these two components are lower than the baseline condition under Scenario S2, with magnitudes decreasing by about 4,800 acre-feet and about 1,300 acre-feet, respectively (Table G1-20 and Table G1-21).

In Segment 2, both of the hypothetical release scenarios would result in decreases to the net gain in channel storage that is predicted under baseline conditions at the end of the release, but the decreases are only significant under Scenario S2 (Figure G1-67). The results in Segment 2 indicate that Scenario S1 has a lower cumulative seepage volume compared to baseline conditions, while the seepage is slightly higher under Scenario S2. Compared to baseline conditions, both Scenarios S1 and S2 show reduced in-channel stormwater/ungaged return flows, and that this component is near zero under Scenario S1. The downstream outflow, again compared to baseline conditions, is higher under Scenario S1 and lower under Scenario S2 (Table G1-20 and Table G1-21).

The baseline condition indicates that a decrease to channel storage of about 4,000 acre-feet occurs in Segment 3, but that a net gain in storage of about 3,000 acre-feet occurs under Scenarios S1 and S2 (Figure G1-68). Both Scenarios S1 and S2 indicate lower seepage rates in this segment, but the downstream outflows are significantly higher under Scenario S1 and lower under Scenario S2 (Table G1-20 and Table G1-21).

Increases to channel storage are indicated for all cases in Segment 4, but the increases to channel storage are significantly higher under both of the hypothetical scenarios (Figure G1-69). The hypothetical release scenarios result in slightly decreased channel seepage. Downstream channel outflows are higher under Scenario S1 and lower under Scenario S2, and are primarily controlled by the differences in upstream inflows.

In general, it appears that the differences in in-channel storage among the 2012 release scenarios for each of the segments are primarily caused by differences in the upstream inflow (except in Segment 1), downstream outflow, channel seepage, and to a much lesser degree differences in evapotranspiration and in-channel stormwater/ungaged inflows. The stormwater/ungaged return flow component (Q_{cin}) was estimated by the predicted increase in downstream runoff, and is one of the least substantiated components due to the lack of gage data in the tributary channels and return flow locations. Although the methods used to estimate this component are reasonable, this component may be overestimated during times of hydrograph translation (i.e., on the rising limb of the hydrograph) and could be lumped into the overall change in channel storage component since it is a significant unknown. A sensitivity analysis was therefore conducted by carrying out a separate water budget analysis that removed the Q_{cin} component from the calculations, which obviously results in a reduction to the predicted change in channel storage. A comparison of the results with and without the Q_{cin} component indicates that the effects of this component are most significant in Segments 1 and 4 and relatively insignificant in Segments 2 and 3 (Figure G1-70). Although the Q_{cin} component makes up less than 2 percent of the release volume under each model scenario, the effects of this component are significant on the change in channel storage volumes along the overall RGCP. Under the baseline condition, the change in channel storage along the overall RGCP decreases from about 7,200 acre-feet to about 900 acre feet. Scenario S1 shows a net gain of about 19,100 acre-feet with the Q_{cin} component, reducing to about 14,200 acre-feet when this component is removed. Removal of the Q_{cin} component reduces the overall RGCP net gain in storage from about 16,100 acre-feet to about 11,500 acre-feet under Scenario 2.

7.3 Local Basin Scale Water Budget Analysis— Results

7.3.1 Entire Study Period—2010 through 2012

The local-basin-scale water budget analysis for the 2010 through 2012 study period is presented in **Appendix G2**, along with inflow-outflow hydrographs for the surface-water and groundwater budgets. The annual volumes associated with each component, along with the resulting annual change in surface-water/groundwater storage, are presented in Table G1-22, Figure G1-71 and Figure G1-72. Table G1-22 also shows the percentage of each component and change in storage relative to the total upstream inflow (surface water plus groundwater). Results from the local-basin-scale water budget analysis for the 2010 to 2012 period indicate that, similar to the RGCP-scale channel water budget, the upstream channel inflow and the downstream channel outflow are significant components. However, except for precipitation, upstream groundwater inflow, and downstream groundwater outflow, most of the other components are also very significant. The 2007 groundwater model predicts a zero downstream groundwater outflow because the total depth of alluvium through El Paso Gap is less than 100 feet (SSPA 2007). The volume of the upstream surface water inflow, downstream surface water outflow, and groundwater return flow components decrease from year to year. The increased pumping in 2011 results in a net decrease to the groundwater storage of 137,100 acre-feet over the course of that year. The surface-water budget indicates a net increase in surface-water storage of about 630,300 acre-feet over the 3-year study period, or about 42 percent of the overall upstream inflow, and the magnitude of the surface water gains is most significant in 2010. The groundwater budget indicates a net decrease in groundwater storage of about 218,400 acre-feet (about 15 percent of the overall upstream inflow) by the end of the study period. The resulting total increase to the local-basin scale net storage is about

411,900 acre–feet (630,300 acre-feet minus 218,400 acre-feet), or about 28 percent of the total upstream inflow. Because it is highly unlikely that the net storage at the local-basin scale would increase during periods of drought, there appears to be significant outflows that are not being accounted for in this analysis. Nevertheless, the baseline analysis does provide insight into the relative effects of the hypothetical release patterns through a comparison with the results from the water budget analyses of these releases, as discussed in the following section.

7.3.2 Delayed Single Pulse (S1) and Normal Single Pulse (S2)

Local-basin-scale water budget analyses of the delayed single-pulse release scenario (Scenario S1) and the normal single-pulse release scenario (Scenario S2) are presented in **Appendix G4** and **Appendix G5**, respectively, and were carried out for the 2012 condition and compared to the results from the baseline water budget for actual 2012 conditions (**Appendix G3**). Results from this analysis are generally intuitive in that, by the end of the release, many of the components do not change from baseline conditions (Figure G1-73 and Figure G1-74). This can be said of the upstream discharge, pumping, upstream groundwater inflow, downstream groundwater outflow and groundwater return flow. Surface water flow due to precipitation is slightly lower under Scenario S1 due to the shortened period of release, since this component was assumed to only occur as an addition to the flow in the Rio Grande. Scenario S1 results in a higher absolute downstream surface water discharge, lower absolute groundwater recharge (due to reduced seepage), and slightly lower absolute evapotranspiration (due to a reduced open-water evaporation duration). Scenario S2 results in absolute increases to evapotranspiration and absolute decreases to the downstream surface water outflow. Compared to baseline conditions, the net surface-water storage increases under Scenarios 1 and 2 by about 9 and 6 percent, respectively. The net change in groundwater storage under Scenario S2 is nearly identical to that indicated under baseline conditions, but decreases significantly under Scenario S1. Under Scenario S1, the higher downstream outflow is only partially offset by the reduction to evapotranspiration, resulting in a reduction to overall net storage (surface water plus groundwater) of about 3 percent compared to baseline conditions. The Scenario S2 results indicate that the net storage would increase by about 5 percent compared to baseline conditions, primarily due to the reduction in downstream discharges.

8.0 SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

8.1 Summary and Conclusions

One of the primary goals of this study is to understand the variability of seepage fluxes for a range of antecedent moisture conditions. For the dry antecedent moistures conditions leading up to the first 2012 irrigation block release, the infiltration volume is greater than 40 percent of the volume released from Caballo Dam (Table G1-1), more than twice the seepage volumes predicted in 2010 and 2011. This is very significant and the river discharge should be carefully tracked in 2013 and beyond to see if this trend continues. The evaporation losses range from 2.0 to 3.0 percent of the Caballo Release.

There are several conditions that affect the calibration and the predicted seepage volumes that are basically unknown:

1. While the gages are considered to accurately reflect the overall timing and shape of the irrigation release, the discharge magnitude may be off from 5 to 20 percent based on a number of factors: gage shift, infrequent calibration, changes in channel morphology, sediment scour/deposition, channel roughness or backwater effects. This applies to both the river and the canal diversion gages.
2. Evaporation is based on a mean month average and diurnal variation. Actual individual sunny or cloudy days over the period of a week or more can significantly deviate from the average month evaporation computed by the model. The evaporation loss, however, represents only a small percentage of the net volume of seepage and evaporation.
3. Unmeasured return flows to the river are estimated to range from 15 to 40 cfs. For some periods, this may constitute most the base flow for the Below American Dam gage. Return flows include irrigation drain flows, groundwater return flows, wastewater treatment plant discharges and others.
4. Unmeasured storm inflows are typically identified by an obvious spike in the gage hydrograph that does not appear in the upstream gage record. These spikes generally result from high intensity rainfall events ranging from a few hours to a few days from one or more of the twenty one major tributary watersheds along the entire RGCP reach. Low intensity storms, however, can occur in tributaries that have a confluence located some distance from a downstream river gage. Storm runoff or possibly even snowmelt from these tributary watersheds can result in low or moderate volumes without definitive spikes in the measured gage hydrographs.

The primary variable that is used in the model to offset these unknown conditions in order to replicate the gage hydrographs is the infiltration hydraulic conductivity. The hydraulic conductivity can vary in time (initial hydraulic conductivity decay to a final hydraulic conductivity) and in space (by reach in the RGCP model). Essentially the timing in the hydrographs that is based on the channel roughness is very good when reviewing dips and spikes. Slowing down the frontal wave has the effect of increasing the seepage rates.

Possible improvements to the calibration effort could include the following:

1. **2010 First Pulse**: The calibration of the Leasburg gage hydrograph may be improved by decreasing the infiltration in the rising limb and increasing it at the peak and during the falling limb of the hydrograph. The justification for this is that additional water near the peak flow in the model persists throughout the downstream gages. The difference between the predicted and measured discharge is over 100 cfs and this may be too much discharge to remove by increased infiltration. The diversion record for the Leasburg canal should also be reviewed.

2. **All Pulse Flow Simulations**: The Below American Dam gage and the American Canal diversion gage could be reviewed for every calibration simulation. The use of hourly gage data and a further review of the wasteway return flows would improve the replication of the Below American Dam flows.
3. **2010 Second Pulse**: The Mesilla gage record includes poor or incomplete data during this period. The inability to calibrate to the Mesilla gage with the available Mesilla diversion causes some minor replication discrepancy at the New Anthony and El Paso gages.
4. **2011 First Pulse**: The calibration for this pulse begins to deviate at the Mesilla gage and propagates downstream. The deviation appears in the calibration at the downstream gages, so the logical explanation would be that the available diversion record at Mesilla is not completely accurate. The model predicts too much flow at the peak, but not enough flow on portions of the recessional limb. The New Anthony gage also has a poor record that does not reflect the peak flow at either the upstream or downstream gages.
5. **2011 Second Pulse**: This is a long simulation that extends from May through mid-September. A calibration discrepancy of about 100 cfs for 40 days is introduced at the Leasburg gage that may be attributed to the available Leasburg canal diversion record. This difference is increased at the Mesilla gage and persists downstream. About 70 days of accurate replication on the falling limb occurs for Haynor, Leasburg, and Picacho and then the model over predicts the hydrograph by 200 cfs for that same period at Mesilla. This over-prediction then is reported for the remaining downstream gages. The only possible explanation for this extra water in the model is the under-prediction of seepage or a potentially poor diversion record at Mesilla. Since a portion of the pulse hydrograph recession limb is replicated for flows of less than 900 cfs at the downstream gages and since the hydrograph shape (spikes and dips) are also replicated, this would indicate that the predicted seepage rates are not the issue, but rather the diversion record is suspect.
6. **2012 First Pulse**: The only issue with this pulse release is that the model under-predicts the seepage rates at the peak flow downstream of the Mesilla gage. Since the seepage rates are already significantly higher than previous years, this could be attributed to a slight shift in the gage record due to sediment scour in the channel.
7. **2012 Second Pulse**: Overall this pulse is replicated well throughout the entire simulation. On the rising limb there is a small slice removed from the record at the Mesilla gage that persists downstream, which is an indication that the canal diversion record does not reflect the missing slice.

While the calibration may improve with possible review of the available gaging records, in general the hydraulic conductivity would not be affected. The variation in the hydraulic conductivity was based on hydrograph shape and timing and reflects the movement of the irrigation pulse through the entire river system. Significant changes in the hydraulic conductivity are not expected even if the gaging record is enhanced. If a portion of the hydrograph is matched by the model, then divergence from the gage record is not due to the prediction of the seepage, but rather should be attributed to poor gage data and unmeasured inflows.

8.2 Recommendations

The most significant result of this work effort is the substantial increase in 2012 irrigation pulse infiltration when compared with the 2010 and 2011 irrigation block releases. It has been suggested that this may be due to lower groundwater levels along the river channel in association with increased agricultural groundwater pumping during the previous dry years. This possibility is also supported by the lower estimated infiltration volumes in the 2004 study which included calibration to three other previous years of record. The increased infiltration in 2012 may constitute a trend that may not be reversed in the near future.

To determine if this increased infiltration rates will be a continuing trend, the following tasks are recommended:

1. Closely monitor the future gage data on a daily basis. The deficiencies in the gage record were clearly observed by using the model results as a basis for comparison, and eliminating the gage deficiencies should be a high priority. It is recommended that the nature of the gage record deficiencies be determined and corrected.
2. Observe the river and canal gages in the field during the irrigation block releases several times to make sure that the diversion record appears to be supported by the flow in the canal.
3. Consider making improvements to the gaging stations to enhance the gage record accuracy.

In future years of water scarcity, there may be only one irrigation block release. It is suggested that these releases be carefully tracked to accurately compile gage records for further calibration of the infiltration. It is recommended that the model be applied to the 2013 irrigation release. A well-calibrated model for 2013 would provide a predictive tool to determine if the high infiltration volumes in 2012 will continue.

It is also recommended that an analysis be carried out to determine the baseflow release rate prior to the irrigation release that would be necessary to reduce the very high seepage rates that are indicated at the beginning of each pulse release. This analysis would likely involve modeling of the proposed baseflow to determine the volume that would be required to maintain a nominal flow in the channel over a given duration. This volume could then be compared to the estimated initial seepage volume that would occur without the baseflow to determine if a baseflow option to improve water delivery efficiency is viable.

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9.0 REFERENCES

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10.0 TABLES

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Table G1-1. FLO-2D Predicted Volumes from the Calibrated Model Simulations (acre-feet)[1]

	2010 1st Pulse	2010 2nd Pulse	2011 1st Pulse	2011 2nd Pulse	2012 1st Pulse	2012 2nd Pulse	Average
1. Inflow volume	154,490	272,560	110,500	299,920	71,610	117,940	171,170
2. Channel storage	4,990	7,370	3,510	470	280	5,980	3,770
3. Channel outflow	127,580	244,070	90,310	247,750	36,810	81,080	137,930
4. Channel infiltration	19,550	15,440	14,000	42,310	32,410	28,120	25,310
5. Channel evaporation	2,370	5,680	2,670	9,390	2,110	2,760	4,160
Infiltration/evaporation as a percent of inflow	14.1	7.8	15.1	17.2	48.2	26.2	17.2

[1]Volumes rounded to nearest 10 acre-feet

Table G1-2. Comparison of FLO-2D-Based Seepage Estimates with the Gains and Losses Reported in USIBWC (1993)

Segment	FLO-2D Seepage (ac-ft/day/mile)			USIBWC (1993) Gains/Losses (ac-ft/day/mile)			
	Minimum	Maximum	Average	Period	Maximum Gain	Maximum Loss	Average
Segment 1	0.0	-15.0	-1.3	NA	NA	NA	NA
Segment 2[1]	0.0	-45.7	-2.6	1991-1992	58.1	-164.6	-8.3
Segment 3[2]	0.0	-52.6	-3.5	1986-1992	83.0	-44.3	-0.7
Segment 3[3]				1986-1992	31.0	-64.0	-3.9
Segment 4[4]	0.0	-35.5	-2.4	1986-1992	23.4	-47.8	-8.4
Total[5]	0.0	-20.5	-2.1	1991-1992	7.2	-24.4	-1.3

[1]USIBWC (1993) includes only the reach from Picacho Bridge to Mesilla Dam.
[2]USIBWC (1993) reach from Mesilla Dam to Vado Bridge.
[3]USIBWC (1993) reach from Vado Bridge to Canutillo Bridge.
[4]USIBWC (1993) reach from Canutillo Bridge to Courchesne Bridge.
[5]USIBWC (1993) reach from Picacho Bridge to Courchesne Bridge (1991 and 1992).

Table G1-3. Hydraulic Conductivity Values for the Initial Calibration of the First and Second 2012 Irrigation Pulses

Reach Number	First Pulse Hydraulic Conductivity (in./hr)		Second Pulse Hydraulic Conductivity (in./hr)	
	Initial H_C	Final H_C	Initial H_c	Final H_c
1	0.180	0.080	0.120	0.060
2	0.680	0.033	0.350	0.280
3	0.880	0.055	0.300	0.250
4	0.980	0.090	0.420	0.350
5	1.050	0.250	0.400	0.100
6	0.950	0.200	0.200	0.050
7	0.700	0.080	0.100	0.040

Table G1-4. Hydraulic Conductivity Values for the Final Calibration of the First and Second 2012 Irrigation Pulses

Reach Number	First Pulse Hydraulic Conductivity (in./hr)		Second Pulse Hydraulic Conductivity (in./hr)	
	Initial H_C	Final H_C	Initial H_c	Final H_c
1	0.180	0.090	0.200	0.090
2	0.200	0.025	0.350	0.090
3	0.600	0.090	0.700	0.090
4	0.900	0.250	1.000	0.250
5	1.150	0.300	1.150	0.300
6	1.450	0.055	1.450	0.060
7	1.250	0.015	1.250	0.060

Table G1-5. Hydraulic Conductivity Values for the First Irrigation Pulse in 2010, 2011 and 2012

First Irrigation Pulse Hydraulic Conductivity						
Reach Number	Initial Hydraulic Conductivity (in./hr)			Final Hydraulic Conductivity (in./hr)		
	2010	2011	2012	2010	2011	2012
1	0.030	0.030	0.180	0.030	0.030	0.090
2	0.100	0.100	0.200	0.100	0.030	0.250
3	0.100	0.100	0.600	0.100	0.030	0.090
4	0.200	0.300	0.900	0.050	0.030	0.250
5	0.200	0.300	1.150	0.050	0.030	0.300
6	0.300	0.300	1.450	0.050	0.050	0.055
7	0.300	0.300	1.250	0.050	0.050	0.015

Table G1-6. Hydraulic Conductivity Values for the Second Irrigation Pulses in 2010, 2011 and 2012

Second Irrigation Pulse Hydraulic Conductivity						
Reach Number	Initial Hydraulic Conductivity (in./hr)			Final Hydraulic Conductivity (in./hr)		
	2010	2011	2012	2010	2011	2012
1	0.050	0.050	0.200	0.020	0.030	0.090
2	0.050	0.150	0.350	0.020	0.030	0.090
3	0.050	0.150	0.700	0.010	0.040	0.090
4	0.060	0.150	1.000	0.010	0.060	0.250
5	0.080	0.150	1.150	0.020	0.060	0.300
6	0.150	0.150	1.450	0.060	0.100	0.060
7	0.150	0.150	1.250	0.100	0.100	0.060

Table G1-7. Irrigation Release Calibrated Limiting Storage Depth for 2010 through 2012

Limiting Storage Depth (ft)						
Reach Number	First Irrigation Pulse			Second Irrigation Pulse		
	2010	2011	2012	2010	2011	2012
1	15	15	15	20	20	20
2	18	18	18	15	15	15
3	20	20	20	15	15	15
4	20	20	35	15	15	15
5	20	20	40	15	15	15
6	20	20	30	15	15	15
7	20	20	30	15	15	15

Table G1-8. Recommended Limiting Storage Depth

Limiting Storage Depth (ft)				
Reach Number	First Irrigation Pulse		Second Irrigation Pulse	
	Wet/Ave	Dry	Wet/Ave	Dry
1	15	15	20	20
2	18	18	15	15
3	20	20	15	15
4	20	35	15	15
5	20	40	15	15
6	20	40	15	15
7	20	30	15	15

Table G1-9. Summary of Hydraulic Conductivities Evaluated in the Sensitivity Analysis

Segment	Reach Number	Base	+10%	+20%	+30%	-10%	-20%	-30%
		Initial Hydraulic Conductivity (in./hr)						
1	1	0.18	0.200	0.216	0.234	0.162	0.144	0.126
	2	0.200	0.220	0.240	0.260	0.188	0.160	0.140
2	3	0.600	0.660	0.720	0.780	0.540	0.480	0.422
	4	0.900	0.990	1.080	1.170	0.810	0.720	0.630
3	5	1.150	1.270	1.380	1.495	1.036	0.920	0.805
4	6	1.450	1.600	1.740	1.885	1.305	1.160	1.015
	7	1.250	1.380	1.500	1.663	1.125	1.000	0.875
Segment	Reach Number	Base	+10%	+20%	+30%	-10%	-20%	-30%
		Final Hydraulic Conductivity (in./hr)						
1	1	0.090	0.100	0.108	0.117	0.081	0.072	0.063
	2	0.025	0.030	0.030	0.033	0.023	0.020	0.018
2	3	0.090	0.100	0.108	0.117	0.081	0.072	0.063
	4	0.250	0.280	0.300	0.325	0.225	0.200	0.175
3	5	0.300	0.330	0.360	0.390	0.300	0.240	0.210
4	6	0.055	0.610	0.066	0.072	0.050	0.044	0.039
	7	0.015	0.017	0.018	0.020	0.014	0.012	0.011

Table G1-10. Sensitivity Analyses Results for the Scenario S1 Model, Absolute Change in Seepage vs. Percent Change in K_{sat}

Percent Change in Ks	Seepage Volume (acre-feet)				
	Segment 1	Segment 2	Segment 3	Segment 4	Total
-30%	15,878	19,089	22,845	5,179	62,991
-20%	17,740	21,361	25,536	5,560	70,197
-10%	19,595	23,687	30,610	5,951	79,844
0%	**21,431**	**25,937**	**30,448**	**6,250**	**84,066**
10%	23,665	28,642	32,643	6,576	91,526
20%	25,121	30,478	34,902	6,814	97,316
30%	27,059	32,689	37,045	7,017	103,809

Table G1-11. Sensitivity Analyses Results for the Scenario S1 Model, Percent Change in Seepage vs. Percent Change in K_{sat}

Percent Change in Ks	Percent Change in Seepage Volume				
	Segment 1	Segment 2	Segment 3	Segment 4	Total
-30%	-26%	-26%	-25%	-17%	-25%
-20%	-17%	-18%	-16%	-11%	-16%
-10%	-9%	-9%	1%	-5%	-5%
0%	**0%**	**0%**	**0%**	**0%**	**0%**
10%	10%	10%	7%	5%	9%
20%	17%	18%	15%	9%	16%
30%	26%	26%	22%	12%	23%

Table G1-12. Sensitivity Analyses Results for the Scenario S2 Model, Absolute Change in Seepage vs. Percent Change in K_{sat}

Percent Change in Ks	Seepage Volume (acre-feet)				
	Segment 1	Segment 2	Segment 3	Segment 4	Total
-30%	21,049	25,804	26,777	5,495	79,125
-20%	23,746	29,031	29,530	5,807	88,114
-10%	26,377	32,075	34,656	6,125	99,233
0%	**28,971**	**35,159**	**33,840**	**6,413**	**104,383**
10%	32,173	38,781	35,456	6,665	113,076
20%	34,090	41,126	37,519	6,873	119,608
30%	36,931	44,238	38,983	7,006	127,158

Table G1-13. Sensitivity Analyses Results for the Scenario S2 Model, Percent Change in Seepage vs. Percent Change in K_{sat}

Percent Change in Ks	Percent Change in Seepage Volume				
	Segment 1	Segment 2	Segment 3	Segment 4	Total
-30%	-27%	-27%	-21%	-14%	-24%
-20%	-18%	-17%	-13%	-9%	-16%
-10%	-9%	-9%	2%	-5%	-5%
0%	**0%**	**0%**	**0%**	**0%**	**0%**
10%	11%	10%	5%	4%	8%
20%	18%	17%	11%	7%	15%
30%	27%	26%	15%	9%	22%

Table G1-14. Summary of FLO-2D Simulation Periods

Simulation	Start Date	End Date
2010, First Pulse	3/1/2010	4/23/2010
2010, Second Pulse	4/25/2010	7/10/2010
2011, First Pulse	3/7/2011	5/3/2011
2011, Second Pulse	5/4/2011	9/14/2011
2012, First Pulse	4/1/2012	5/8/2012
2012, Second Pulse	5/9/2012	6/29/2012

Table G1-15. Summary of Observed Erroneous Baseflows in the HEC-RAS Model Results, and the Discharges that were Subtracted from the HEC-RAS Model Results during Low-flow Periods

Gage Location	Observed Erroneous Baseflow (cfs)	Discharge Adjustment (cfs)
Caballo Dam*	25	24
Haynor Bridge	90	89
Leasburg Cable	250	245
Picacho Bridge	225	220
Below Mesilla Dam	215	210
Anthony Bridge	195	194
El Paso (Courchesne)	195	194
American Dam	195	194

*Minimum flow of 25 cfs specified in model input

Table G1-16. Comparison of Seepage Rates used in the HEC-RAS-based and FLO-2D-based Water Budget Analyses for the 2010 through 2012 Study Period

	Segment 1	Segment 2	Segment 3	Segment 4	Total
	HEC-RAS-based Water Budget Analysis				
Min (cfs)	0.0	0.0	0.0	0.0	0.9
Max (cfs)	70.7	142.8	86.7	94.8	362.9
Average (cfs)	25.6	52.6	32.3	36.3	146.8
Total (ac-ft)	54078	111128	68317	76584	310107
	FLO-2D-based Water Budget Analysis				
Min (cfs)	0.0	0.0	0.0	0.0	0.9
Max (cfs)	353.7	526.9	481.6	343.3	1105.3
Average (cfs)	29.8	29.5	32.2	22.8	114.4
Total (ac-ft)	62980	62354	68122	48163	241619
	Percent Difference				
Min (cfs)	0%	0%	0%	0%	0%
Max (cfs)	400%	269%	455%	262%	205%
Average (cfs)	16%	-44%	0%	-37%	-22%
Total (ac-ft)	16%	-44%	0%	-37%	-22%

Table G1-17. Comparison of Evaporation Rates used in the HEC-RAS-based and FLO-2D-based Water Budget Analyses for the 2010 through 2012 Study Period

	Segment 1	Segment 2	Segment 3	Segment 4	Total
	HEC-RAS-based Water Budget Analysis				
Average (cfs)	4.5	2.3	2.0	1.9	10.7
Max (cfs)	10.7	5.4	4.8	4.6	25.5
Total (ac-ft)	9446	4829	4278	4066	22619
	FLO-2D-based Water Budget Analysis				
Average (cfs)	6.9	4.0	2.8	2.6	16.3
Max (cfs)	17.8	10.7	7.1	6.7	42.2
Total (ac-ft)	14640	8411	5832	5502	34385
	Percent Difference				
Average (cfs)	55%	74%	36%	35%	52%
Max (cfs)	67%	96%	47%	45%	66%
Total (ac-ft)	55%	74%	36%	35%	52%

Table G1-18. Annual and Total Water Volumes (acre-feet) for the Various Components of the Channel Water Budget Study

Segment	Year	Qcus[1]	Pc[2]	Qcin[3]	Qirf[4]	Qeff[5]	Qgwrf[6]	Qcds[7]	Qcs[8]	Qfpr[9]	ET[10]	Qda[11]	Qdu[12]	ΔSic[13]
1	2010	652,000	1,100	17,100	0	400	13,300	-489,000	-19,800	-1,600	-14,200	-133,600	-1,300	24,300
	2011	402,500	1,100	17,100	0	400	13,300	-361,300	-16,900	-1,600	-15,000	-26,700	-300	12,700
	2012	372,000	1,000	5,100	0	300	11,700	-278,300	-26,300	-1,600	-14,000	-63,700	-600	5,700
	Total	1,426,500	3,200	39,300	0	1,100	38,300	-1,128,600	-63,000	-4,700	-43,200	-224,000	-2,200	42,700
2	2010	489,000	600	1,700	17,000	16,200	1,500	-272,700	-12,600	-1,500	-4,900	-203,400	-2,000	30,000
	2011	361,300	600	1,100	4,100	16,200	1,500	-292,200	-16,100	-1,500	-5,500	-55,100	-600	14,400
	2012	278,300	600	700	2,000	14,900	1,300	-143,000	-33,600	-1,500	-5,000	-100,500	-1,000	13,800
	Total	1,128,600	1,800	3,500	23,100	47,400	4,300	-708,000	-62,400	-4,600	-15,400	-358,900	-3,600	58,200
3	2010	272,700	400	4,000	35,200	1,100	0	-298,200	-12,900	-2,900	-4,300	0	0	-4,900
	2011	292,200	400	1,700	15,500	1,100	0	-298,400	-14,600	-2,900	-4,500	0	0	-9,600
	2012	143,000	400	200	5,300	1,000	0	-108,600	-40,600	-2,900	-4,100	0	0	-6,400
	Total	708,000	1,200	5,800	56,000	3,100	0	-705,200	-68,100	-8,700	-12,900	0	0	-20,900
4	2010	298,200	500	700	35,000	12,000	100	-311,600	-20,800	-1,600	-3,800	0	0	8,600
	2011	298,400	500	700	4,200	12,000	100	-276,200	-19,200	-1,600	-4,100	0	0	14,800
	2012	108,600	400	1,200	6,400	11,100	100	-112,400	-8,100	-1,600	-3,700	0	0	2,000
	Total	705,200	1,400	2,700	45,600	35,100	300	-700,200	-48,200	-4,700	-11,600	0	0	25,500
Total RGCP	2010	652,000	2,600	23,500	87,200	29,700	14,900	-311,600	-66,100	-7,600	-27,200	-337,000	-3,300	57,100
	2011	402,500	2,600	20,600	23,800	29,700	14,900	-276,200	-66,800	-7,600	-29,100	-81,800	-900	31,700
	2012	372,000	2,400	7,200	13,700	27,300	13,100	-112,400	-108,600	-7,600	-26,800	-164,200	-1,600	14,500
	Total	1,426,500	7,600	51,300	124,700	86,700	42,900	-700,200	-241,700	-22,700	-83,100	-582,900	-5,800	103,300

[1]Qcus Upstream Channel Inflow
[2]Pc Precipitation Flows in River Channel
[3]Qcin In-channel Stormwater/ Ungaged Return Inflow
[5]Qeff Treated Effluent Return Flow
[6]Qgwrf Groundwater Return Flow
[7]Qcds Downstream Channel Outflow
[8]Qcs Channel Seepage
[9]Qfpr Floodplain Recharge
[10]ET Evapotranspiration
[12]Qdu Diversions Unauthorized (1% of Authorized)
[13]ΔSic In-channel Change in Storage

Table G1-19. Annual and Total Water Volumes for the Various Components of the Channel Water Budget Study as Percentage of Upstream Inflow

Segment	Year	Qcus[1]	Pc[2]	Qcin[3]	Qirf[4]	Qeff[5]	Qgwrf[6]	Qcds[7]	Qcs[8]	Qfpr[9]	ET[10]	Qda[11]	Qdu[12]	ΔSic[13]
1	2010	100%	0%	3%	0%	0%	2%	-75%	-3%	0%	-2%	-20%	0%	4%
	2011	100%	0%	4%	0%	0%	3%	-90%	-4%	0%	-4%	-7%	0%	3%
	2012	100%	0%	1%	0%	0%	3%	-75%	-7%	0%	-4%	-17%	0%	2%
	Total	100%	0%	3%	0%	0%	3%	-79%	-4%	0%	-3%	-16%	0%	3%
2	2010	100%	0%	0%	3%	3%	0%	-56%	-3%	0%	-1%	-42%	0%	6%
	2011	100%	0%	0%	1%	4%	0%	-81%	-4%	0%	-2%	-15%	0%	4%
	2012	100%	0%	0%	1%	5%	0%	-51%	-12%	-1%	-2%	-36%	0%	5%
	Total	100%	0%	0%	2%	4%	0%	-63%	-6%	0%	-1%	-32%	0%	5%
3	2010	100%	0%	1%	13%	0%	0%	-109%	-5%	-1%	-2%	0%	0%	-2%
	2011	100%	0%	1%	5%	0%	0%	-102%	-5%	-1%	-2%	0%	0%	-3%
	2012	100%	0%	0%	4%	1%	0%	-76%	-28%	-2%	-3%	0%	0%	-4%
	Total	100%	0%	1%	8%	0%	0%	-100%	-10%	-1%	-2%	0%	0%	-3%
4	2010	100%	0%	0%	12%	4%	0%	-104%	-7%	-1%	-1%	0%	0%	3%
	2011	100%	0%	0%	1%	4%	0%	-93%	-6%	-1%	-1%	0%	0%	5%
	2012	100%	0%	1%	6%	10%	0%	-103%	-7%	-1%	-3%	0%	0%	2%
	Total	100%	0%	0%	6%	5%	0%	-99%	-7%	-1%	-2%	0%	0%	4%
Total RGCP	2010	100%	0%	4%	13%	5%	2%	-48%	-10%	-1%	-4%	-52%	-1%	9%
	2011	100%	1%	5%	6%	7%	4%	-69%	-17%	-2%	-7%	-20%	0%	8%
	2012	100%	1%	2%	4%	7%	4%	-30%	-29%	-2%	-7%	-44%	0%	4%
	Total	100%	1%	4%	9%	6%	3%	-49%	-17%	-2%	-6%	-41%	0%	7%

[1]Qcus Upstream Channel Inflow
[2]Pc Precipitation Flows in River Channel
[3]Qcin In-channel Stormwater/ Ungaged Return Inflow
[5]Qeff Treated Effluent Return Flow
[6]Qgwrf Groundwater Return Flow
[7]Qcds Downstream Channel Outflow
[8]Qcs Channel Seepage
[9]Qfpr Floodplain Recharge
[10]ET Evapotranspiration
[12]Qdu Diversions Unauthorized (1% of Authorized)
[13]ΔSic In-channel Change in Storage

Table G1-20. Comparison of Cumulative Volume (acre-feet) for the RGCP-scale Channel Water Budget Components under Baseline 2012 Conditions and Scenarios S1 and S2

Segment	Scenario	Qcus	Pc	Qcin	Qirf	Qeff	Qgwrf	Total Inflow	Qcds	Qcs	Qfpr	ET	Qda	Qdu	Total Outflow	ΔSic[4]
	Baseline[1]	372,028	642	4,222	0	173	4,844	381,908	-277,221	-26,291	-1,063	-10,019	-63,721	-637	-378,952	2,957
1	Scenario S1[2]	372,028	642	4,410	0	173	4,844	382,096	-282,020	-21,431	-1,063	-8,932	-63,722	-637	-377,805	4,291
	Scenario S2[3]	372,028	642	2,889	0	173	4,844	380,576	-272,388	-29,045	-1,063	-10,581	-63,721	-637	-377,435	3,140
	Baseline[1]	277,221	373	660	1,894	7,476	502	288,127	-141,143	-32,576	-1,056	-3,978	-100,475	-1,005	-280,233	7,894
2	Scenario S1[2]	282,020	373	0	1,894	7,476	502	292,265	-153,042	-25,937	-1,056	-3,430	-100,267	-1,003	-284,735	7,530
	Scenario S2[3]	272,388	373	363	1,894	7,476	502	282,996	-134,484	-35,259	-1,056	-4,414	-100,269	-1,003	-276,485	6,511
	Baseline[1]	141,143	238	176	5,147	485	0	147,189	-107,440	-38,710	-1,976	-3,055	0	0	-151,182	-3,993
3	Scenario S1[2]	153,042	238	0	5,147	485	0	158,912	-120,796	-30,448	-1,976	-2,703	0	0	-155,923	2,990
	Scenario S2[3]	134,484	238	210	5,147	485	0	140,564	-98,374	-33,957	-1,976	-3,178	0	0	-137,486	3,078
	Baseline[1]	107,440	281	1,231	3,072	5,542	0	117,565	-106,377	-6,969	-1,079	-2,777	0	0	-117,203	362
4	Scenario S1[2]	120,796	281	437	3,072	5,542	0	130,126	-116,120	-6,250	-1,079	-2,404	0	0	-125,853	4,273
	Scenario S2[3]	98,374	281	1,136	3,072	5,542	0	108,404	-94,872	-6,423	-1,079	-2,648	0	0	-105,023	3,381
	Baseline[1]	372,028	1,534	6,289	10,113	13,675	5,347	408,985	-106,377	-104,546	-5,174	-19,829	-164,196	-1,642	-401,765	7,220
Total	Scenario S1[2]	372,028	1,534	4,846	10,113	13,675	5,347	407,542	-116,120	-84,066	-5,174	-17,469	-163,989	-1,640	-388,458	19,084
	Scenario S2[3]	372,028	1,534	4,598	10,113	13,675	5,347	407,294	-94,872	-104,684	-5,174	-20,822	-163,991	-1,640	-391,183	16,111

[1]Appendix G3 - 2012 Baseline
[2]Appendix G4 - Scenario S1
[3]Appendix G5 - Scenario S2
[4]ΔSic = Total Inflow + Total Outflow

Table G1-21. Comparison of Cumulative Volume (as Percent of Inflow) of the RGCP-scale Channel Water Budget Components under Baseline 2012 Conditions and Scenarios S1 And S2

Segment	Scenario	Qcus	Pc	Qcin	Qirf	Qeff	Qgwrf	Total Inflow	Qcds	Qcs	Qfpr	ET	Qda	Qdu	Total Outflow	ΔSic[4]
1	Baseline[1]	100.0%	0.2%	1.1%	0.0%	0.0%	1.3%	102.7%	-74.5%	-7.1%	-0.3%	-2.7%	-17.1%	-0.2%	-101.9%	0.8%
	Scenario S1[2]	100.0%	0.2%	1.2%	0.0%	0.0%	1.3%	102.7%	-75.8%	-5.8%	-0.3%	-2.4%	-17.1%	-0.2%	-101.6%	1.2%
	Scenario S2[3]	100.0%	0.2%	0.8%	0.0%	0.0%	1.3%	102.3%	-73.2%	-7.8%	-0.3%	-2.8%	-17.1%	-0.2%	-101.5%	0.8%
2	Baseline[1]	100.0%	0.1%	0.2%	0.7%	2.7%	0.2%	103.9%	-50.9%	-11.8%	-0.4%	-1.4%	-36.2%	-0.4%	-101.1%	2.8%
	Scenario S1[2]	100.0%	0.1%	0.0%	0.7%	2.7%	0.2%	103.6%	-54.3%	-9.2%	-0.4%	-1.2%	-35.6%	-0.4%	-101.0%	2.7%
	Scenario S2[3]	100.0%	0.1%	0.1%	0.7%	2.7%	0.2%	103.9%	-49.4%	-12.9%	-0.4%	-1.6%	-36.8%	-0.4%	-101.5%	2.4%
3	Baseline[1]	100.0%	0.2%	0.1%	3.6%	0.3%	0.0%	104.3%	-76.1%	-27.4%	-1.4%	-2.2%	0.0%	0.0%	-107.1%	-2.8%
	Scenario S1[2]	100.0%	0.2%	0.0%	3.4%	0.3%	0.0%	103.8%	-78.9%	-19.9%	-1.3%	-1.8%	0.0%	0.0%	-101.9%	2.0%
	Scenario S2[3]	100.0%	0.2%	0.2%	3.8%	0.4%	0.0%	104.5%	-73.1%	-25.2%	-1.5%	-2.4%	0.0%	0.0%	-102.2%	2.3%
4	Baseline[1]	100.0%	0.3%	1.1%	2.9%	5.2%	0.0%	109.4%	-99.0%	-6.5%	-1.0%	-2.6%	0.0%	0.0%	-109.1%	0.3%
	Scenario S1[2]	100.0%	0.2%	0.4%	2.5%	4.6%	0.0%	107.7%	-96.1%	-5.2%	-0.9%	-2.0%	0.0%	0.0%	-104.2%	3.5%
	Scenario S2[3]	100.0%	0.3%	1.2%	3.1%	5.6%	0.0%	110.2%	-96.4%	-6.5%	-1.1%	-2.7%	0.0%	0.0%	-106.8%	3.4%
Total	Baseline[1]	100.0%	0.4%	1.7%	2.7%	3.7%	1.4%	109.9%	-28.6%	-28.1%	-1.4%	-5.3%	-44.1%	-0.4%	-108.0%	1.9%
	Scenario S1[2]	100.0%	0.4%	1.3%	2.7%	3.7%	1.4%	109.5%	-31.2%	-22.6%	-1.4%	-4.7%	-44.1%	-0.4%	-104.4%	5.1%
	Scenario S2[3]	100.0%	0.4%	1.2%	2.7%	3.7%	1.4%	109.5%	-25.5%	-28.1%	-1.4%	-5.6%	-44.1%	-0.4%	-105.1%	4.3%

[1]Appendix G3 - 2012 Baseline
[2]Appendix G4 - Scenario S1
[3]Appendix G5 - Scenario S2
[4]ΔSic = Total Inflow + Total Outflow

Table G1-22. Annual and Total Water Volumes (acre-feet or as Percentage of Total Inflow) for the Various Components of the Local Basin Scale Water Budget Study

Budget	Component	Volumes (acre-feet)				As Percentage of Total Inflow			
		2010	2011	2012	Total	2010	2011	2012	Total
Surface-water	Upstream Channel Inflow	652,000	402,500	372,000	1,426,500	97%	95%	95%	96%
	Precipitation Flows in River Channel	2,600	2,600	2,400	7,600	0%	1%	1%	1%
	Pumping	140,100	272,900	199,600	612,600	21%	65%	51%	41%
	Groundwater Return Flow	102,100	38,700	26,900	167,700	15%	9%	7%	11%
	Downstream Channel Outflow	-311,600	-276,200	-112,400	-700,200	-46%	-65%	-29%	-47%
	Groundwater Recharge	-153,300	-154,000	-195,700	-503,000	-23%	-36%	-50%	-34%
	Total ET	-127,200	-130,400	-123,100	-380,700	-19%	-31%	-32%	-26%
	Changes in Surface Water Storage	304,600	156,000	169,700	630,300	45%	37%	44%	42%
Ground-water	Upstream Groundwater Inflow	20,500	20,500	17,900	58,900	3%	5%	5%	4%
	Groundwater Recharge	153,300	154,000	195,700	503,000	23%	36%	50%	34%
	Pumping	-140,100	-272,900	-199,600	-612,600	-21%	-65%	-51%	-41%
	Groundwater Return Flow	-102,100	-38,700	-26,900	-167,700	-15%	-9%	-7%	-11%
	Downstream Groundwater Outflow	0	0	0	0	0%	0%	0%	0%
	Change in Vadose Zone and Groundwater Storage	-68,400	-137,100	-12,900	-218,400	-10%	-32%	-3%	-15%
Net	Net Change in Storage	236,200	18,900	156,800	411,900	35%	4%	40%	28%

11.0 FIGURES

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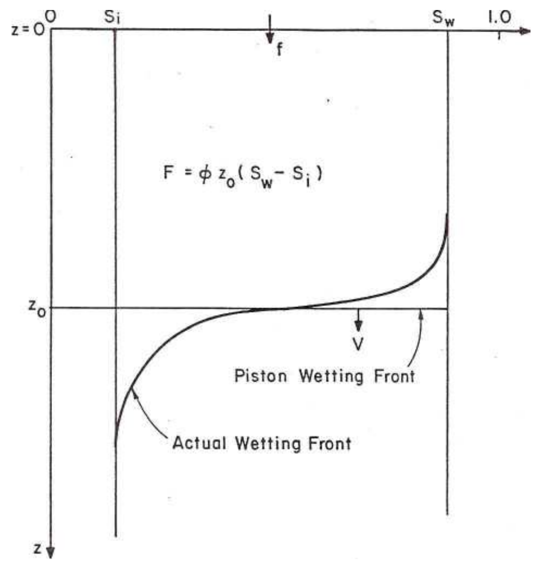

Figure G1-1. Soil Saturation as Function of Soil Depth

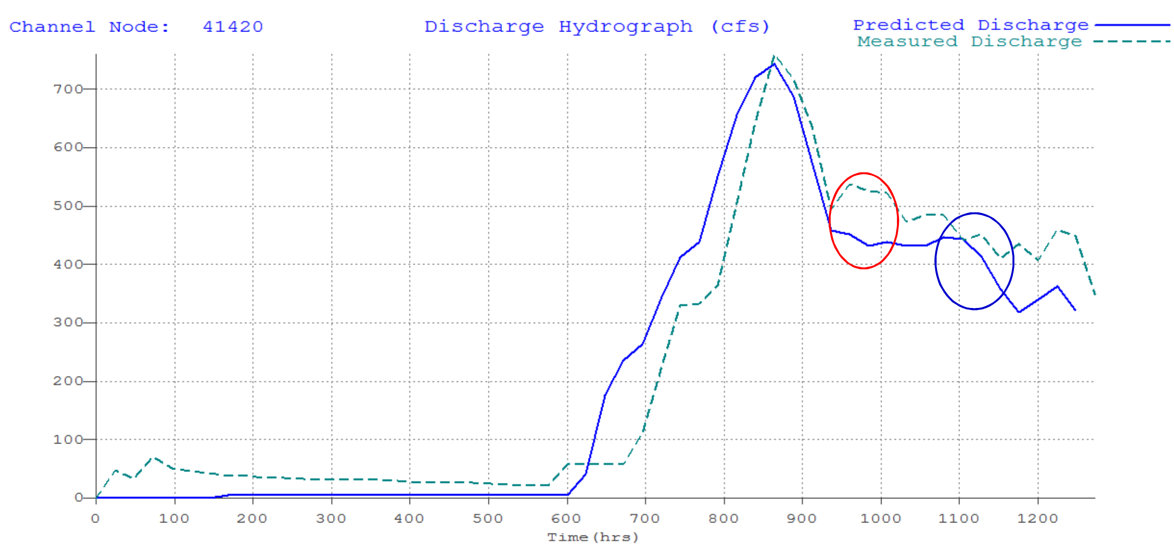

Figure G1-2. El Paso Gage Second Pulse 2012

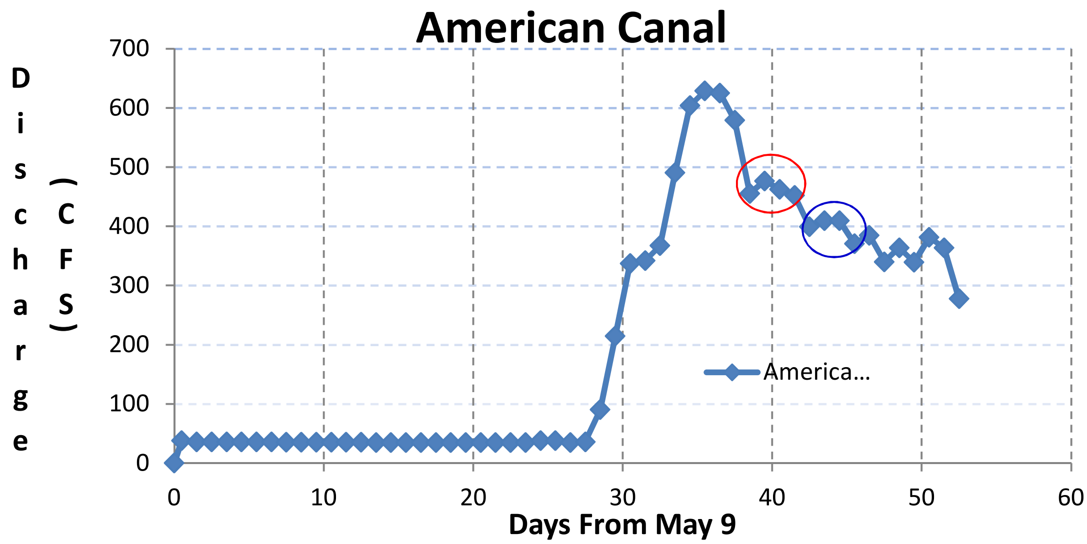

Figure G1-3. American Canal Diversion Discharge for the Second 2012 Pulse

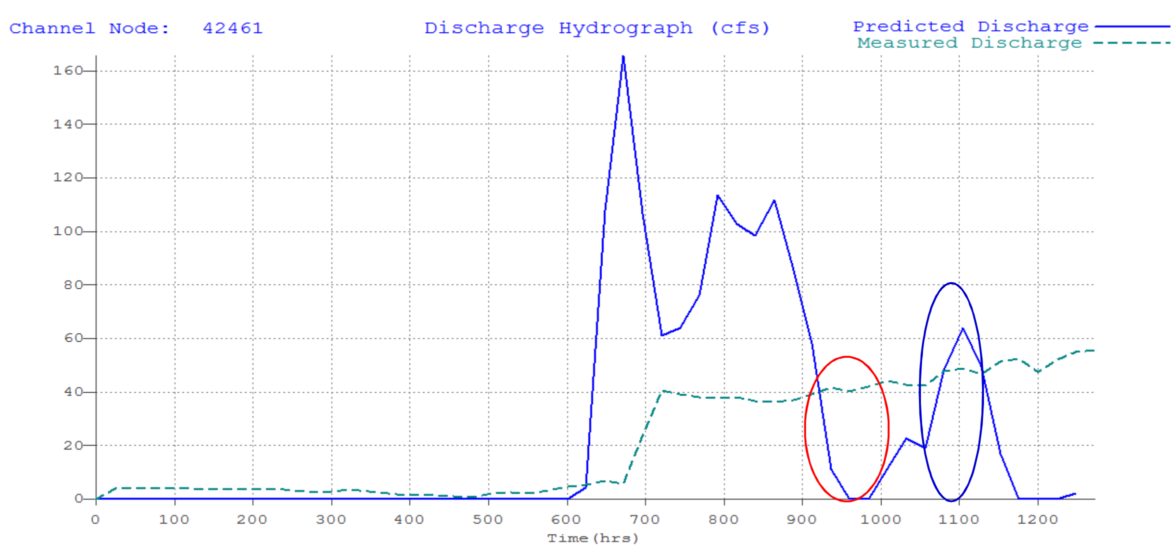

Figure G1-4. Below American Dam Gage Second Pulse 2012

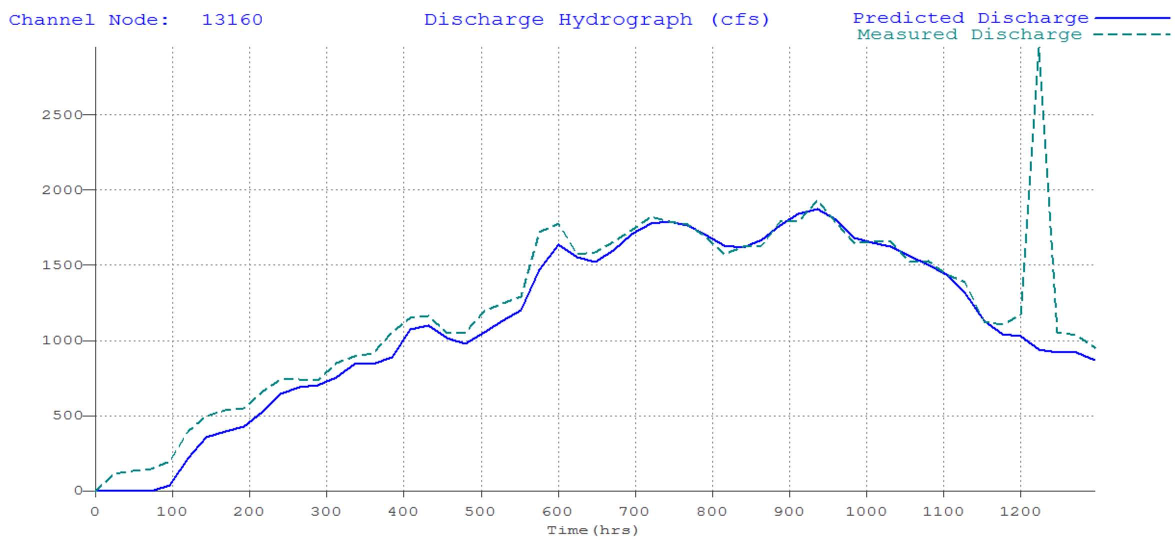

Figure G1-5. Haynor Gage, March 1 - April 23, 2010

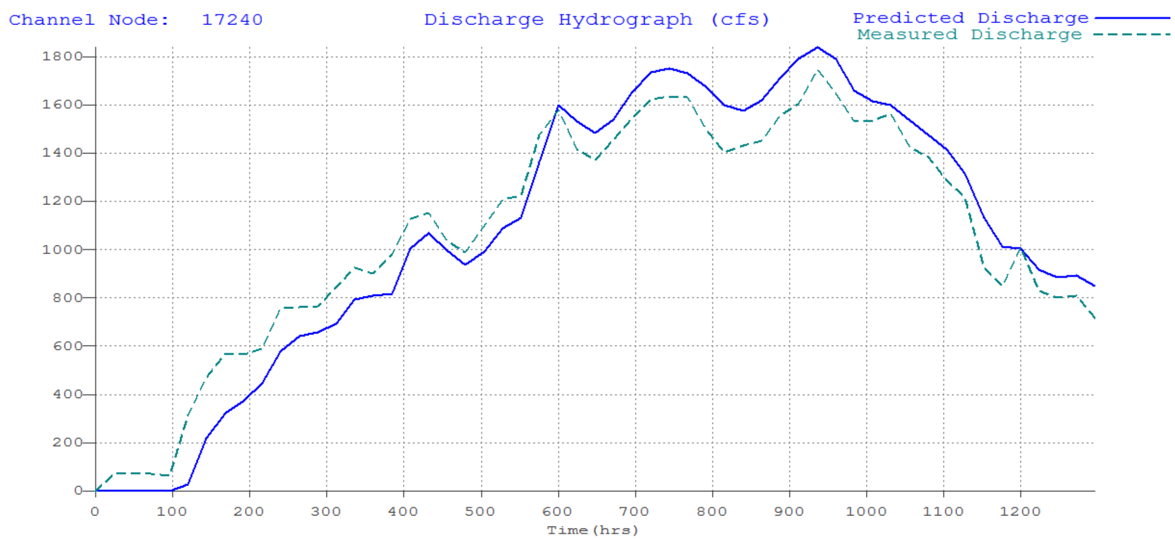

Figure G1-6. Leasburg Gage, March 1 - April 23, 2010

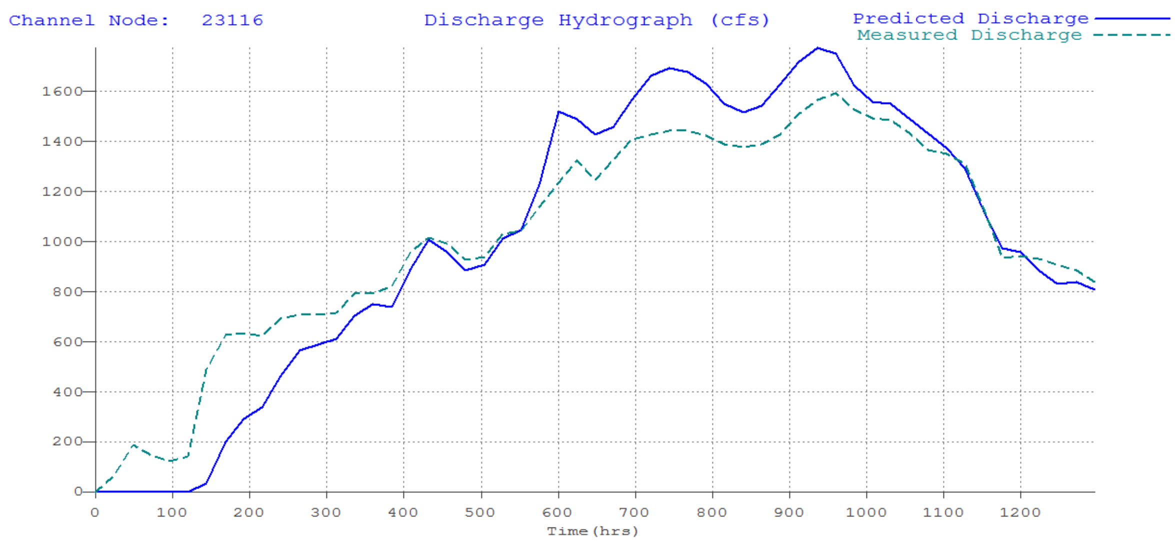

Figure G1-7. Picacho Gage, March 1 - April 23, 2010

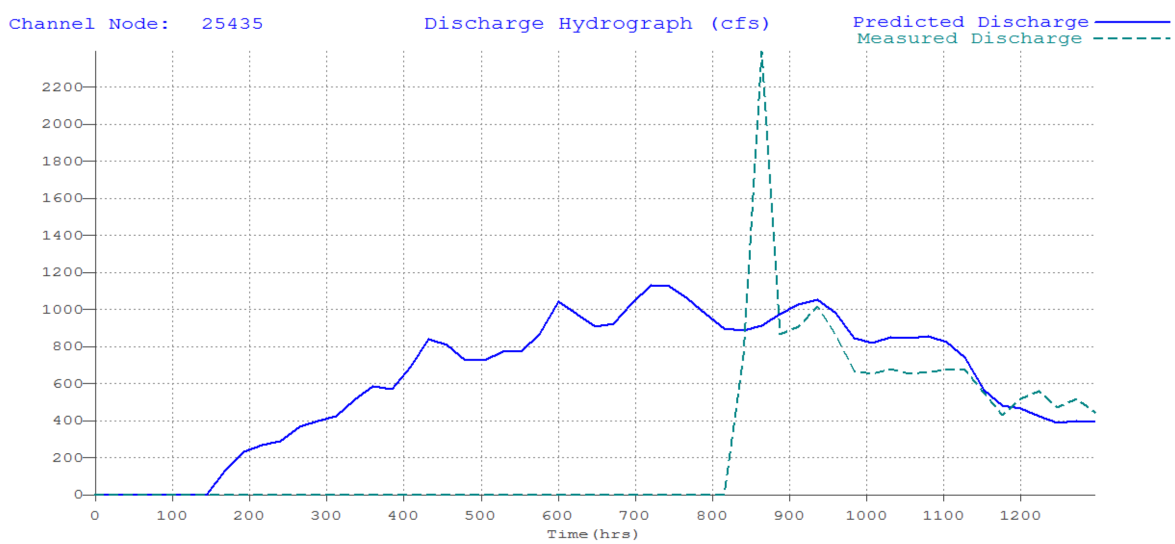

Figure G1-8. Mesilla Gage, March 1 - April 23, 2010

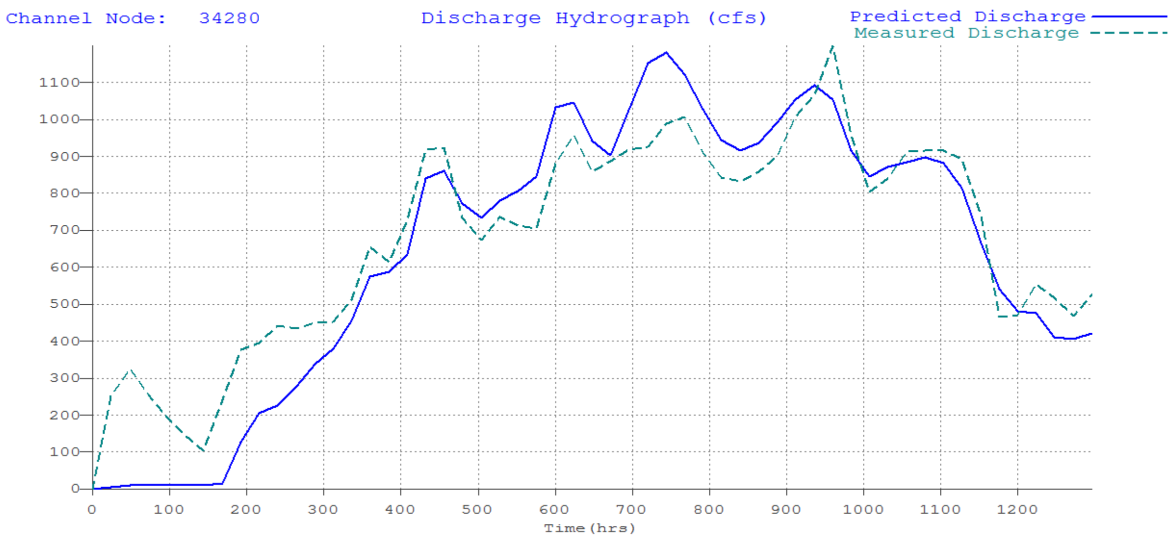

Figure G1-9. New Anthony Gage, March 1 - April 23, 2010

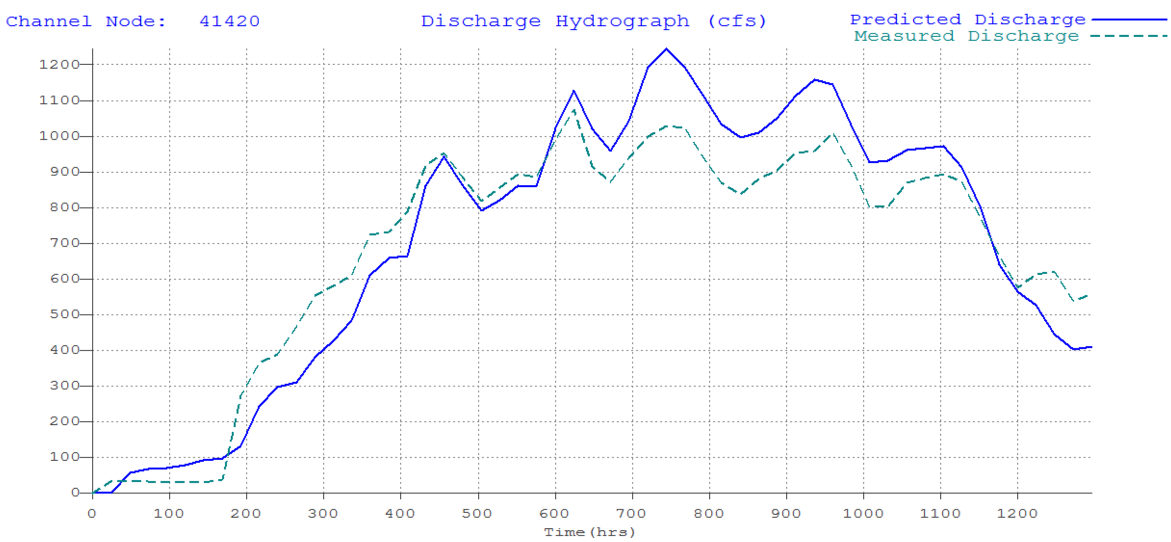

Figure G1-10. El Paso Gage, March 1 - April 23, 2010

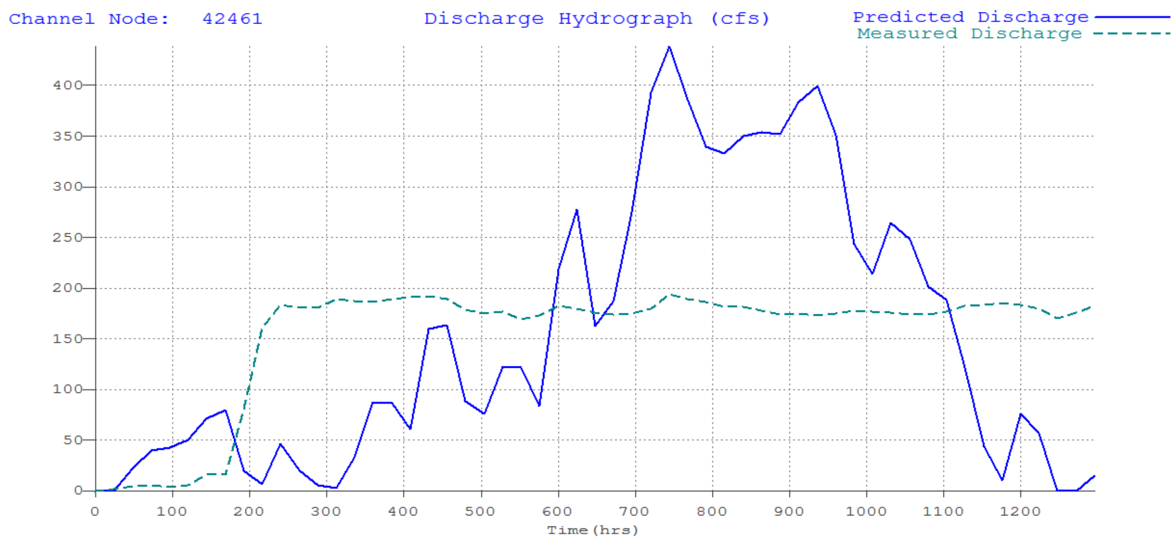

Figure G1-11. Below American Dam Gage, March 1 - April 23, 2010

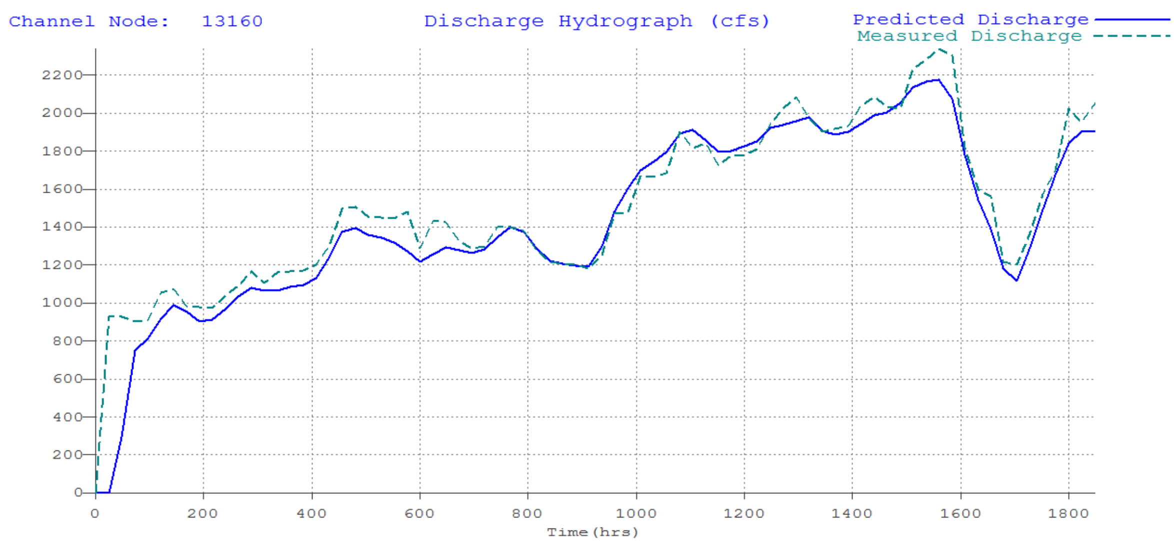

Figure G1-12. Haynor Gage, April 25 - July 10, 2010

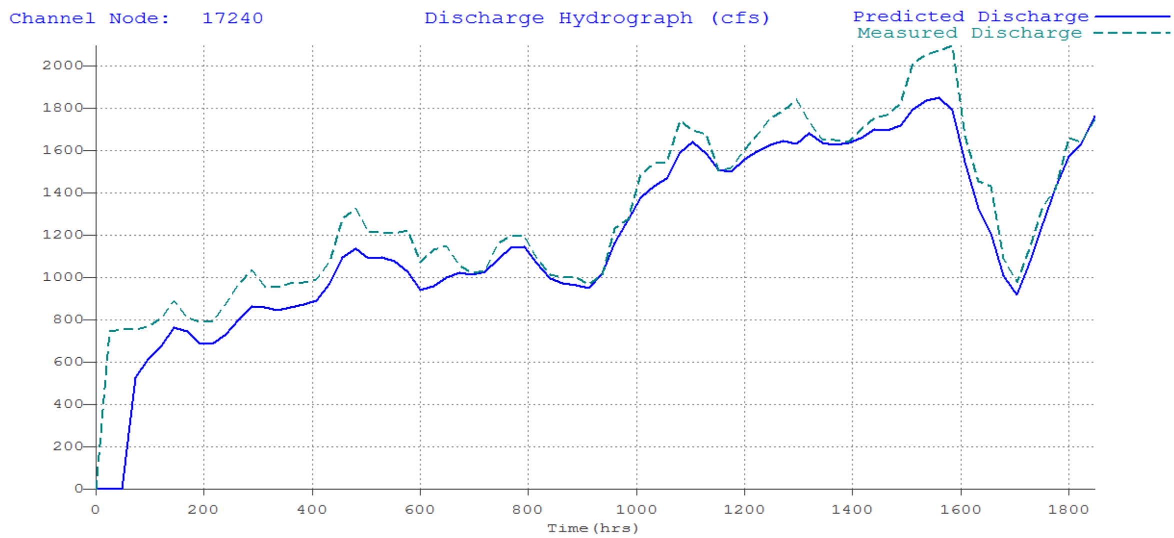

Figure G1-13. Leasburg Gage, April 25 - July 10, 2010

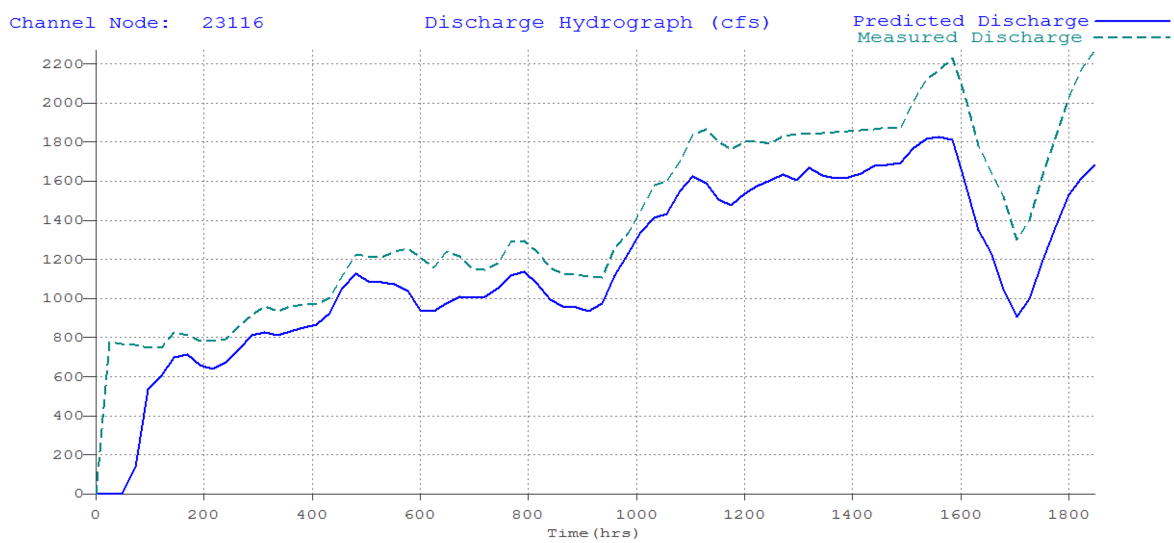

Figure G1-14. Picacho Gage, April 25 - July 10, 2010

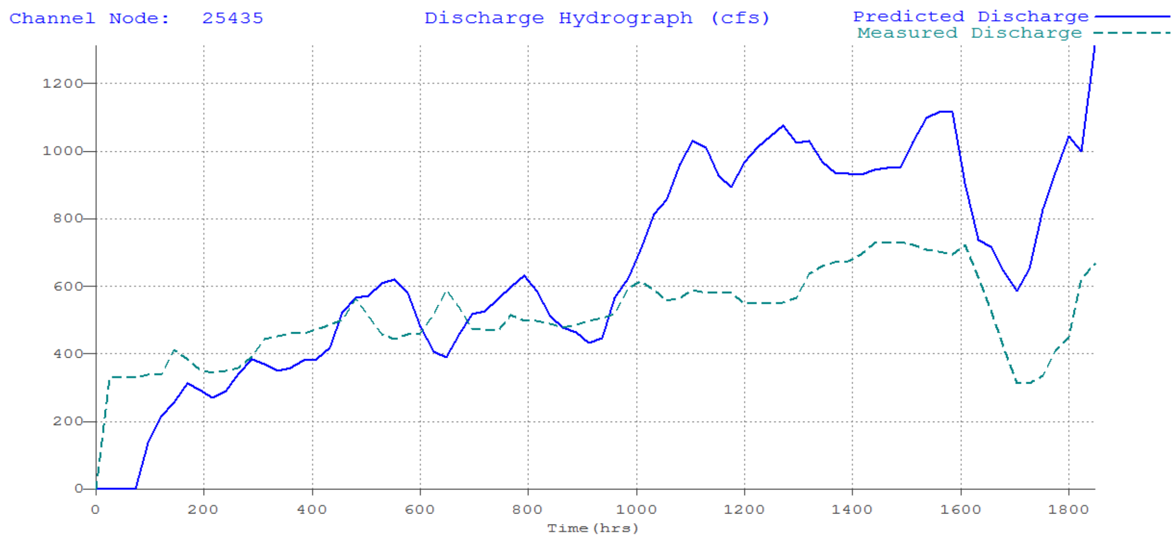

Figure G1-15. Mesilla Gage, April 25 - July 10, 2010

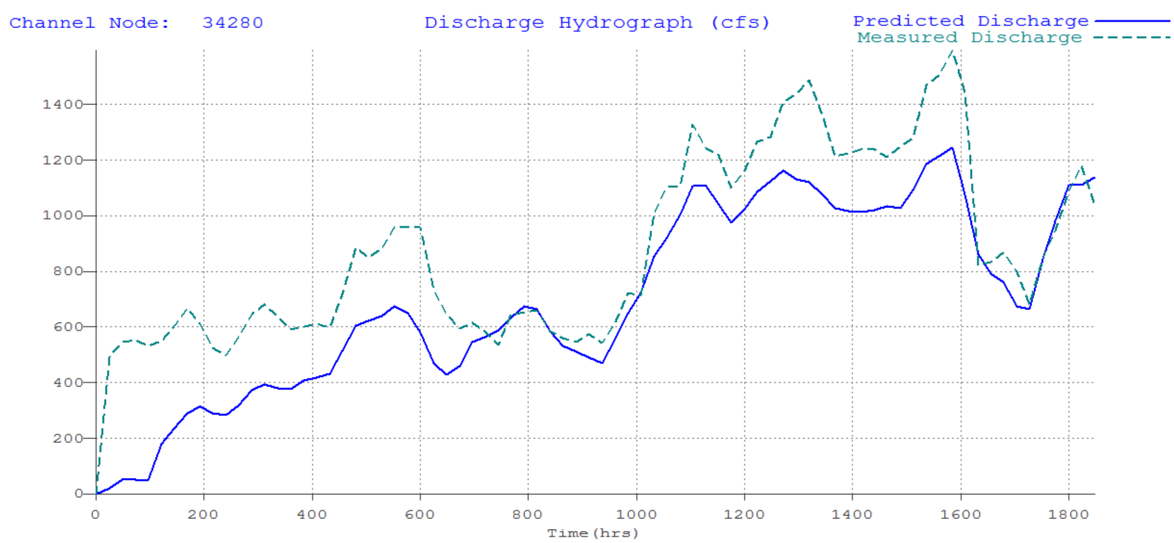

Figure G1-16. New Anthony Gage, April 25 - July 10, 2010

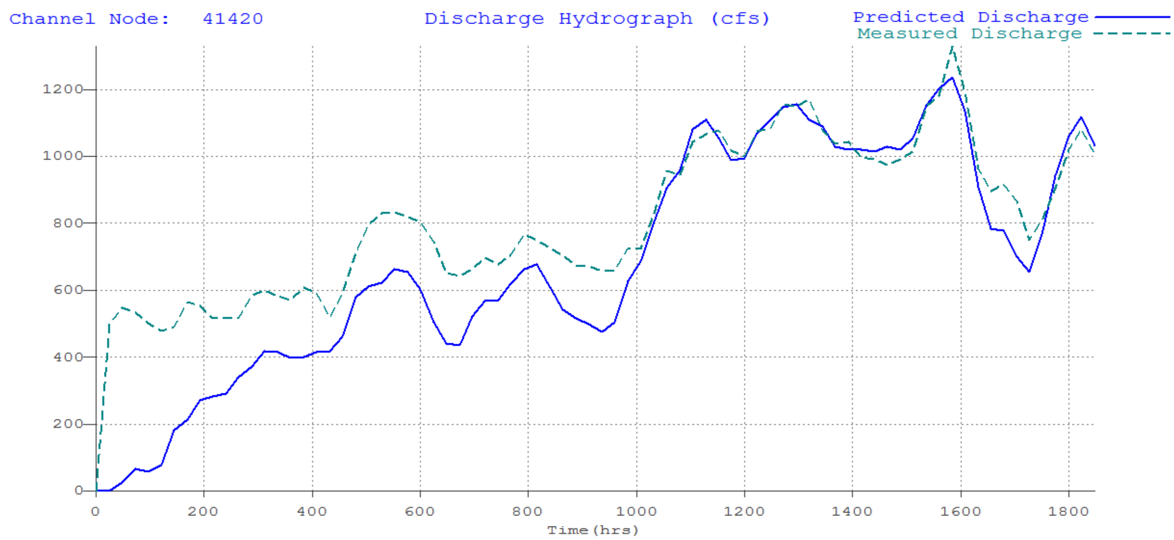

Figure G1-17. El Paso Gage, April 25 - July 10, 2010

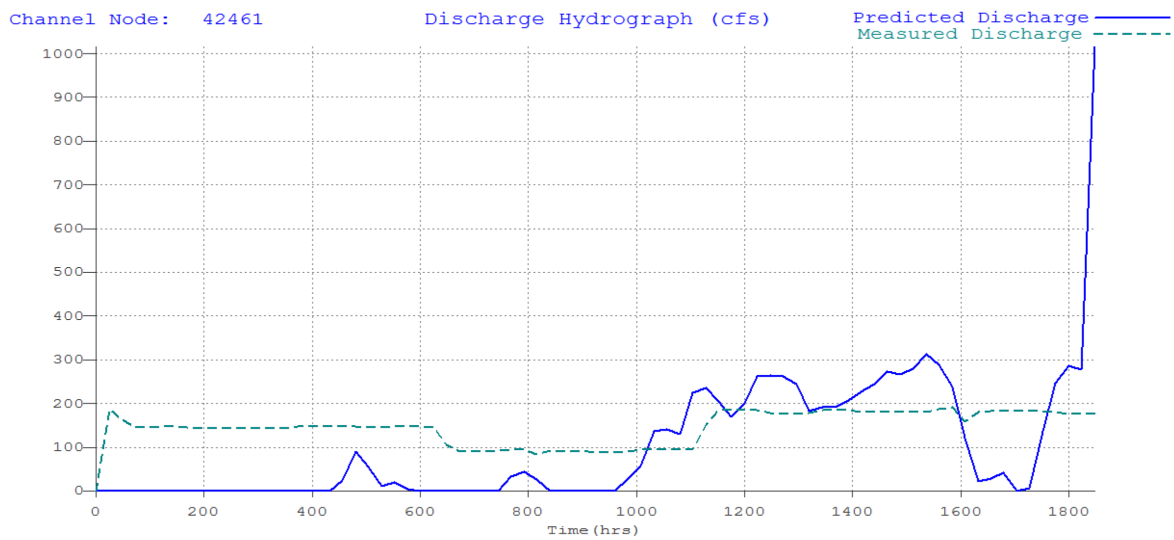

Figure G1-18. Below American Dam Gage, April 25 - July 10, 2010

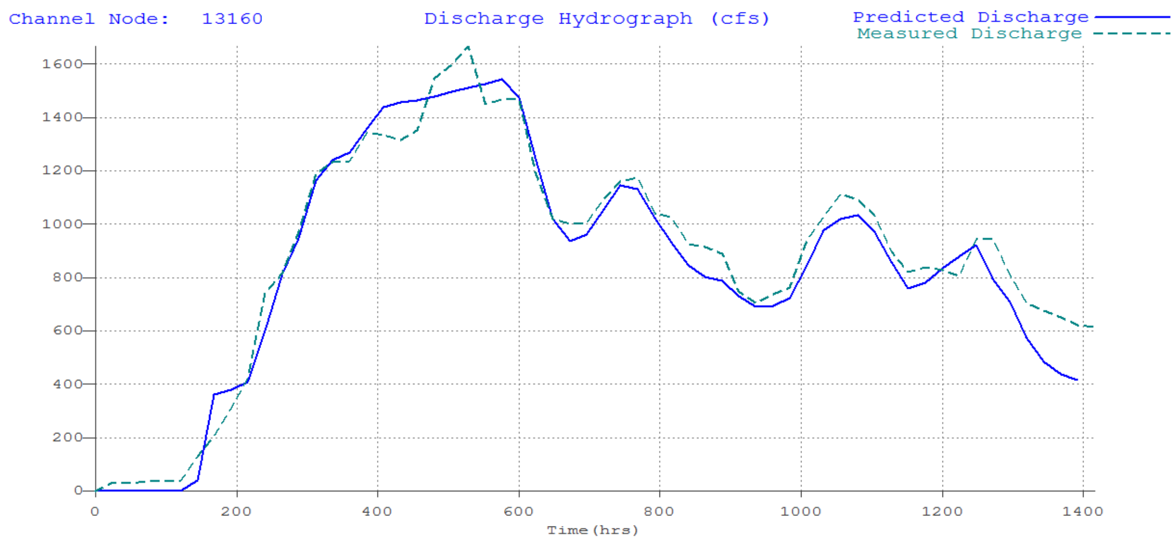

Figure G1-19. Haynor Gage, March 7 - May 3, 2011

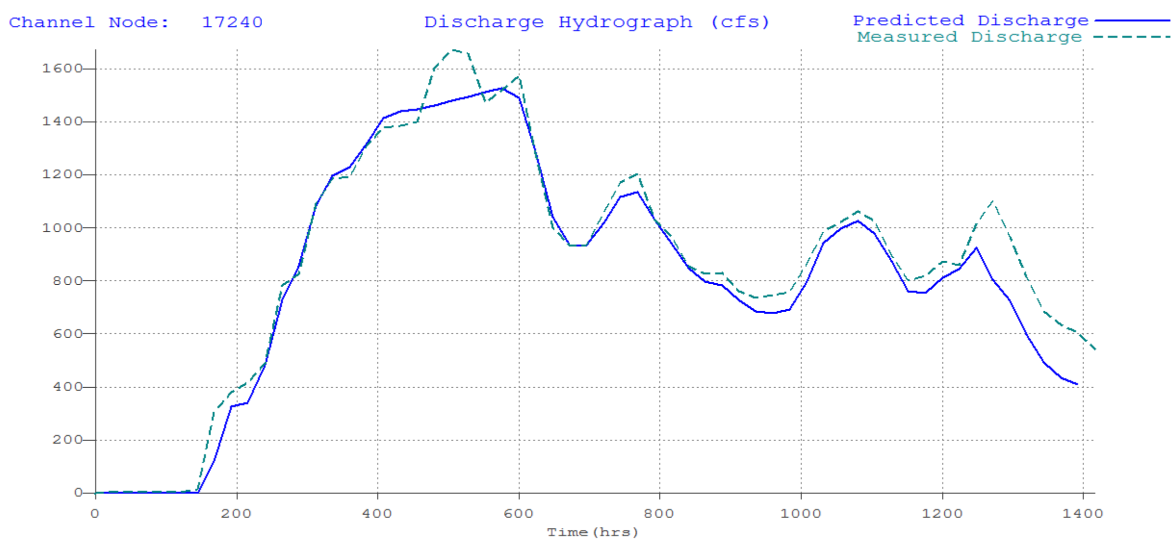

Figure G1-20. Leasburg Gage, March 7 - May 3, 2011

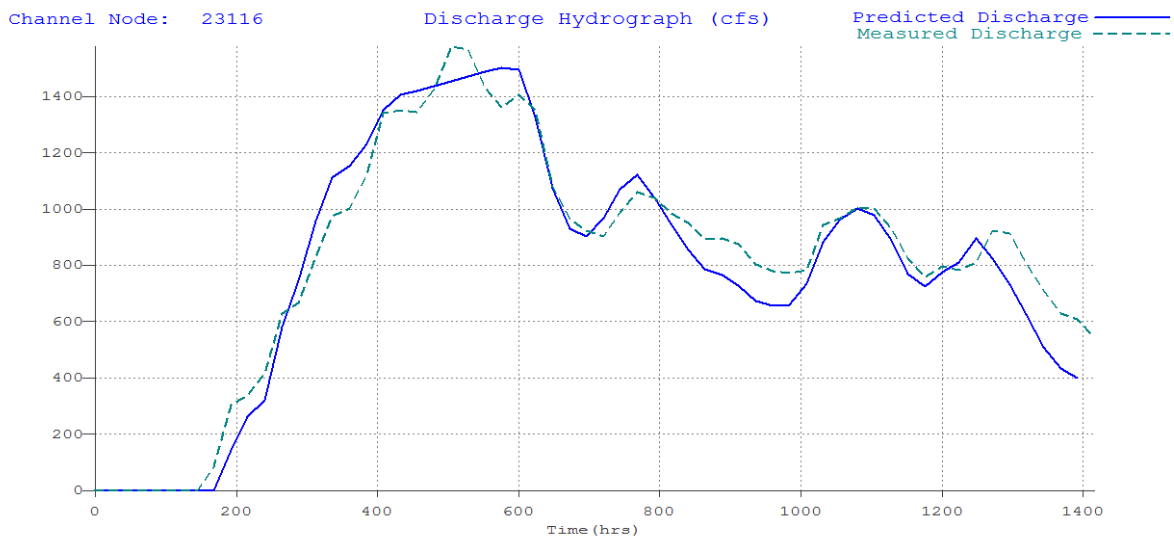

Figure G1-21. Picacho Gage, March 7 - May 3, 2011

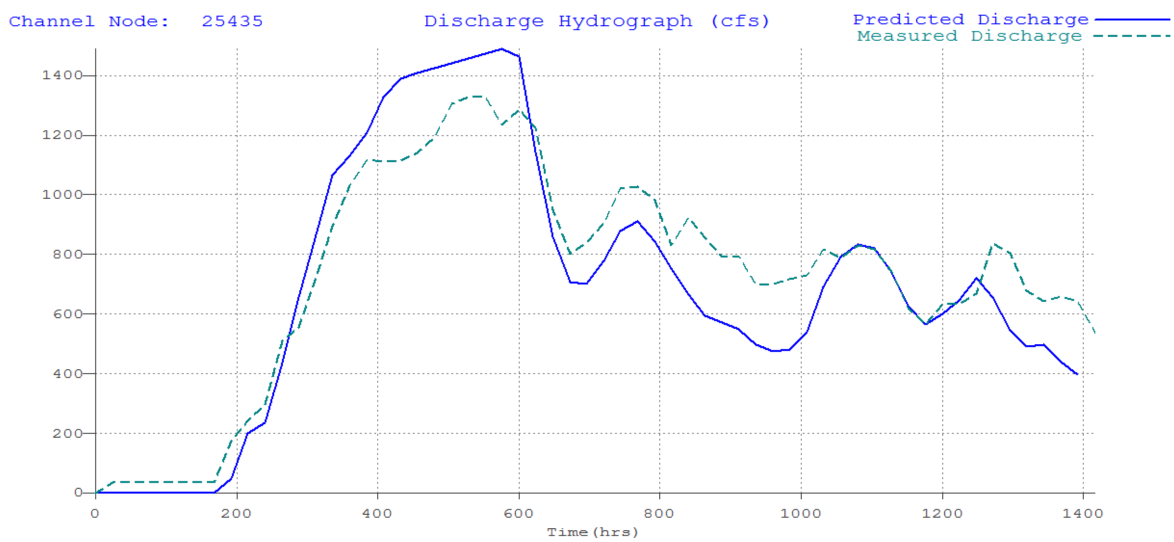

Figure G1-22. Mesilla Gage, March 7 - May 3, 2011

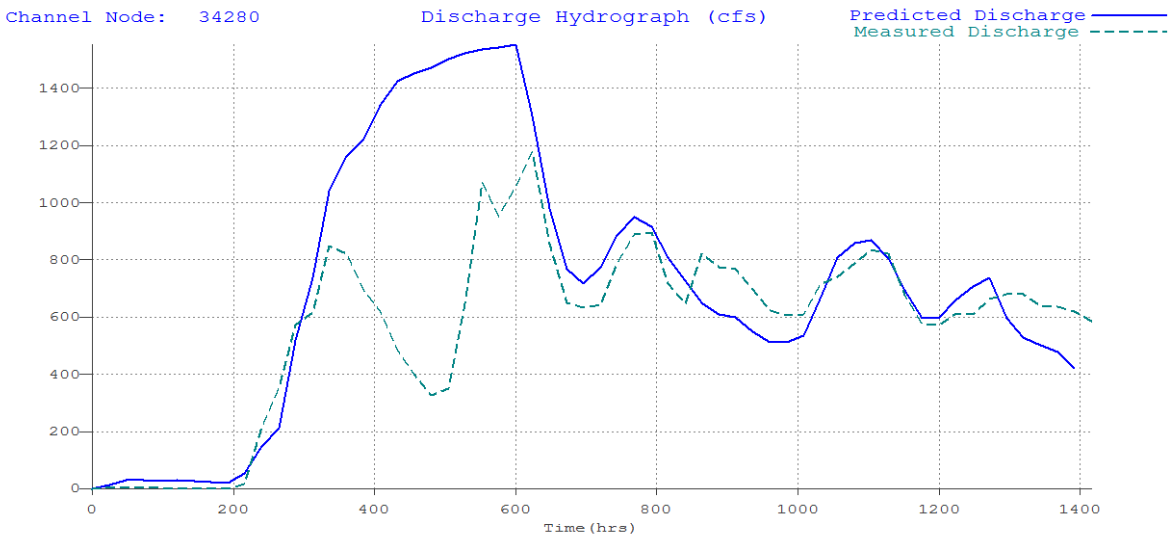

Figure G1-23. New Anthony Gage, March 7 - May 3, 2011

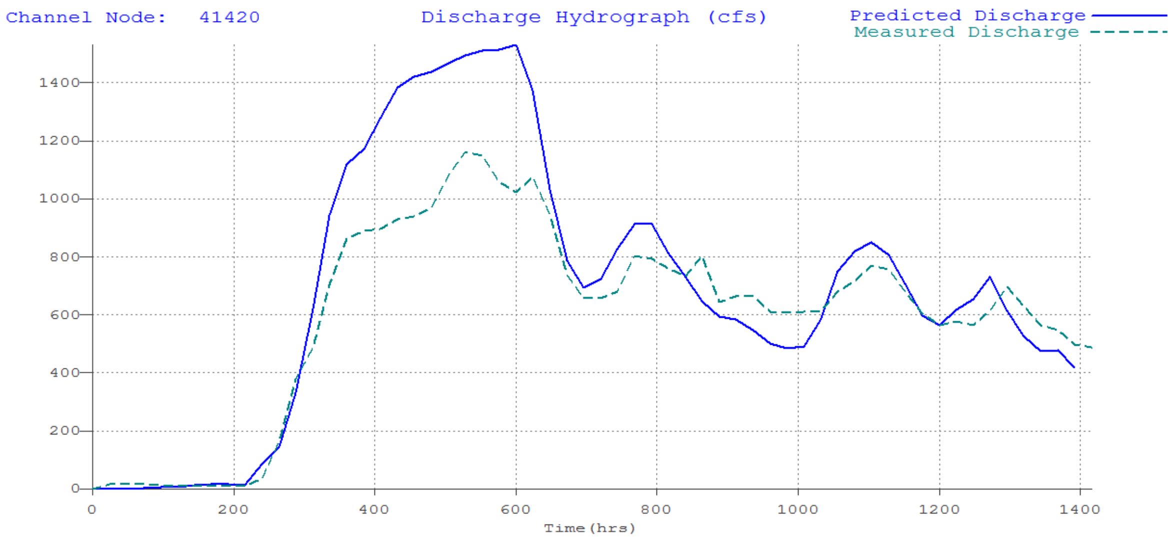

Figure G1-24. El Paso Gage, March 7 - May 3, 2011

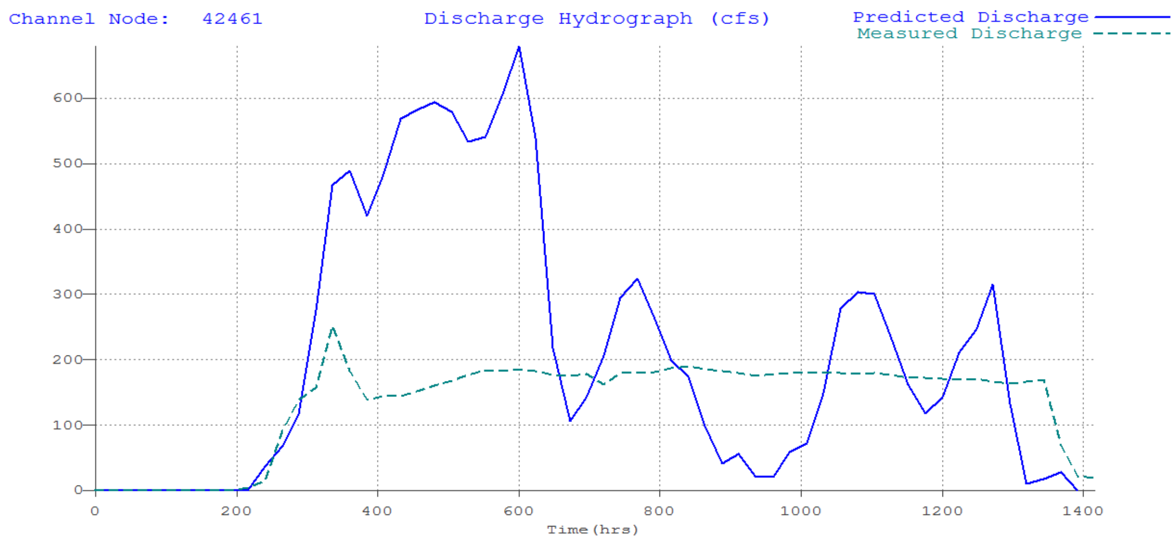

Figure G1-25. Below American Dam Gage, March 7 - May 3, 2011

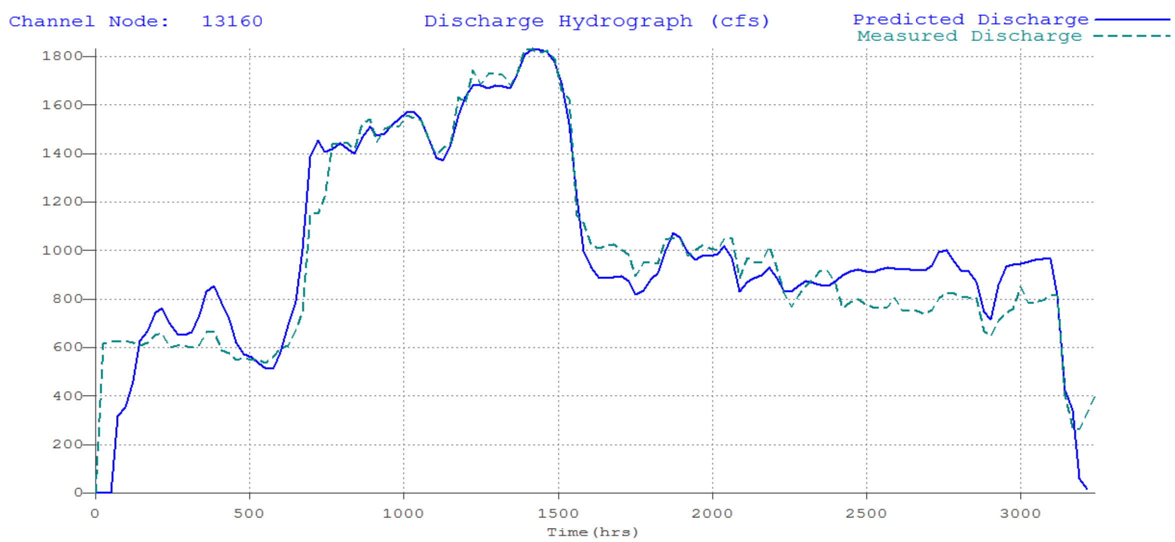

Figure G1-26. Haynor Gage, May 4 - September 15, 2011

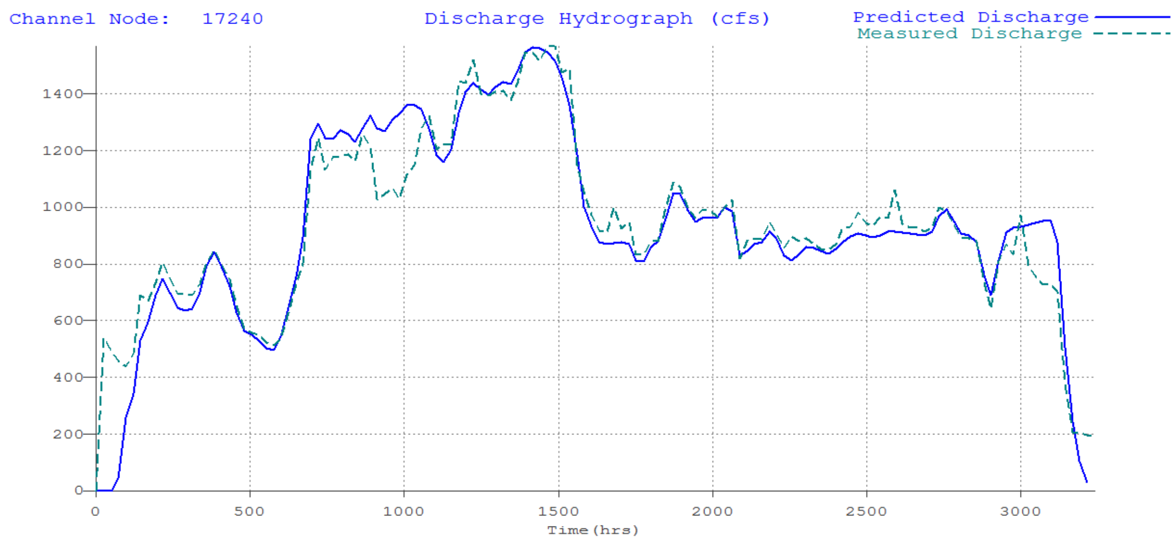

Figure G1-27. Leasburg Gage, May 4 - September 15, 2011

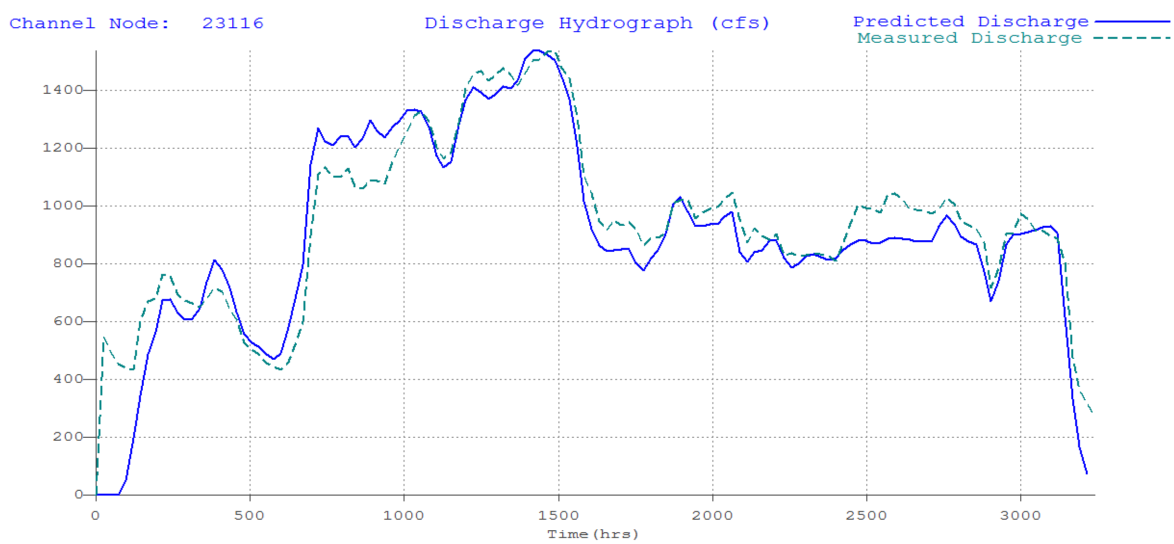

Figure G1-28. Picacho Gage, May 4 - September 15, 2011

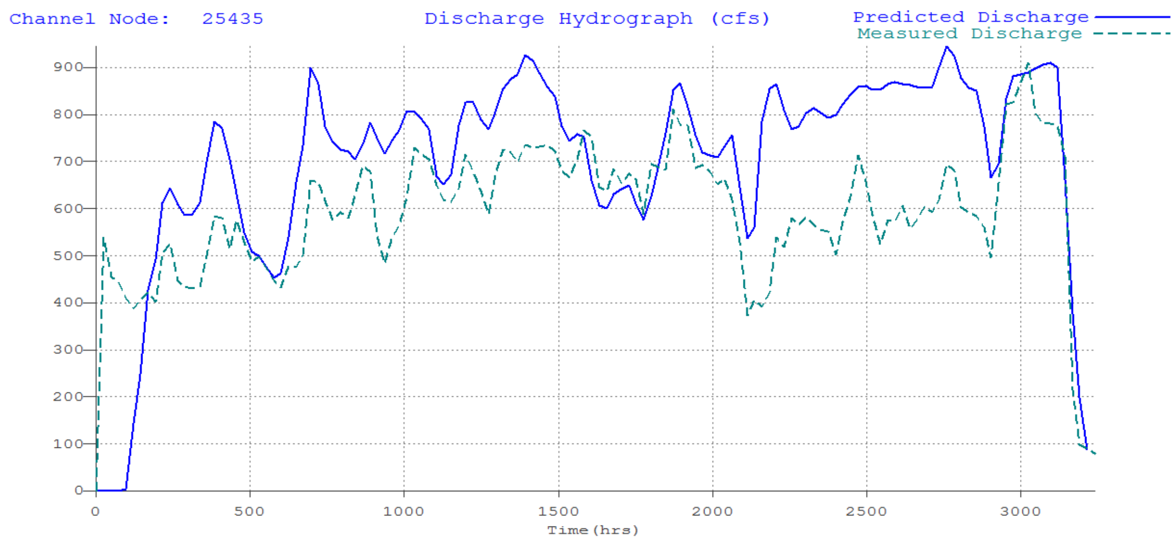

Figure G1-29. Mesilla Gage, May 4 - September 15, 2011

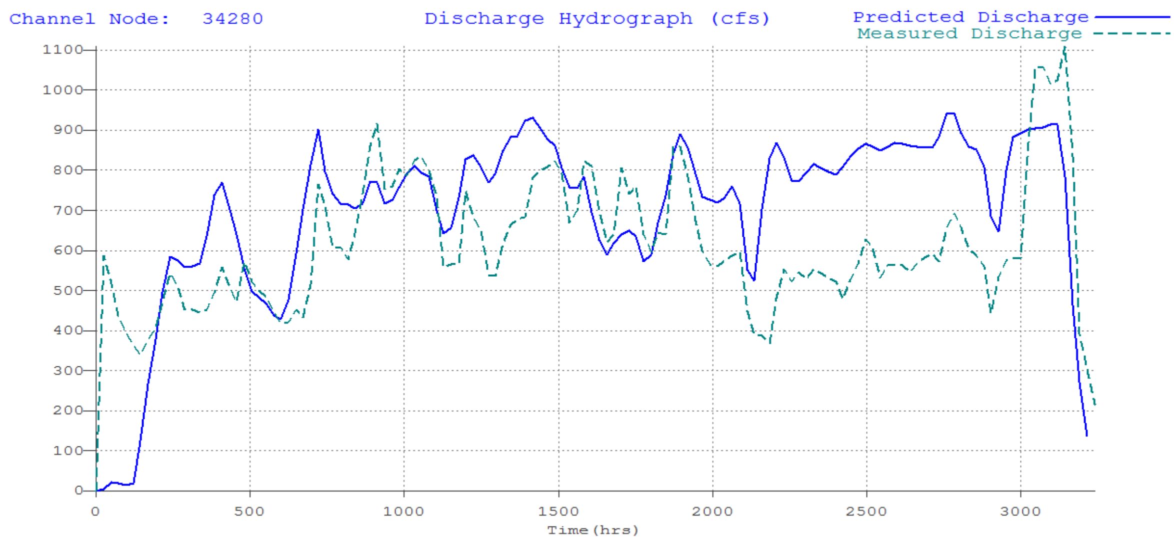

Figure G1-30. New Anthony Gage, May 4 - September 15, 2011

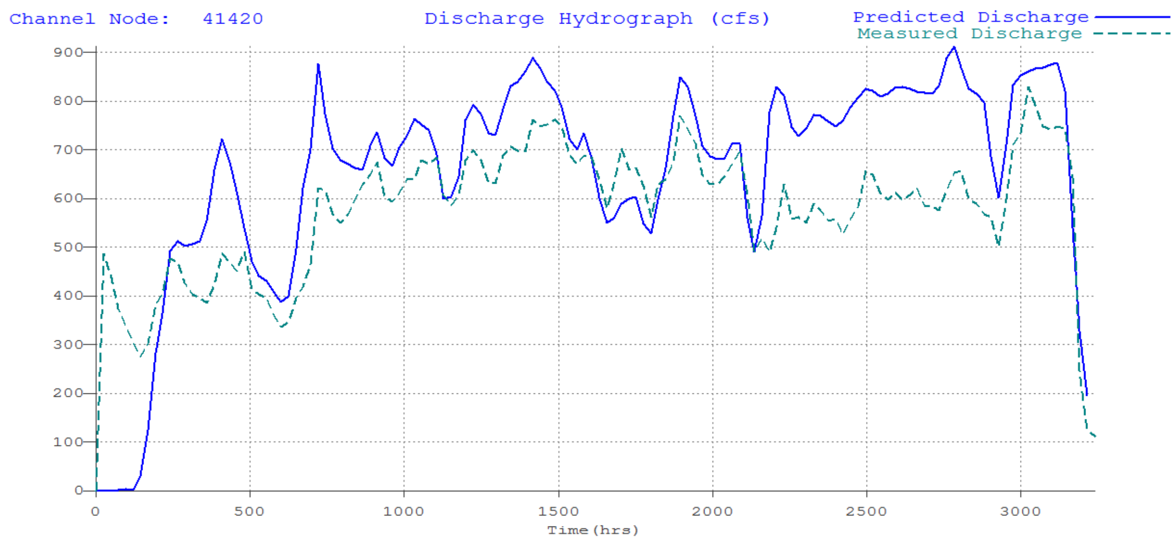

Figure G1-31. El Paso Gage, May 4 - September 15, 2011

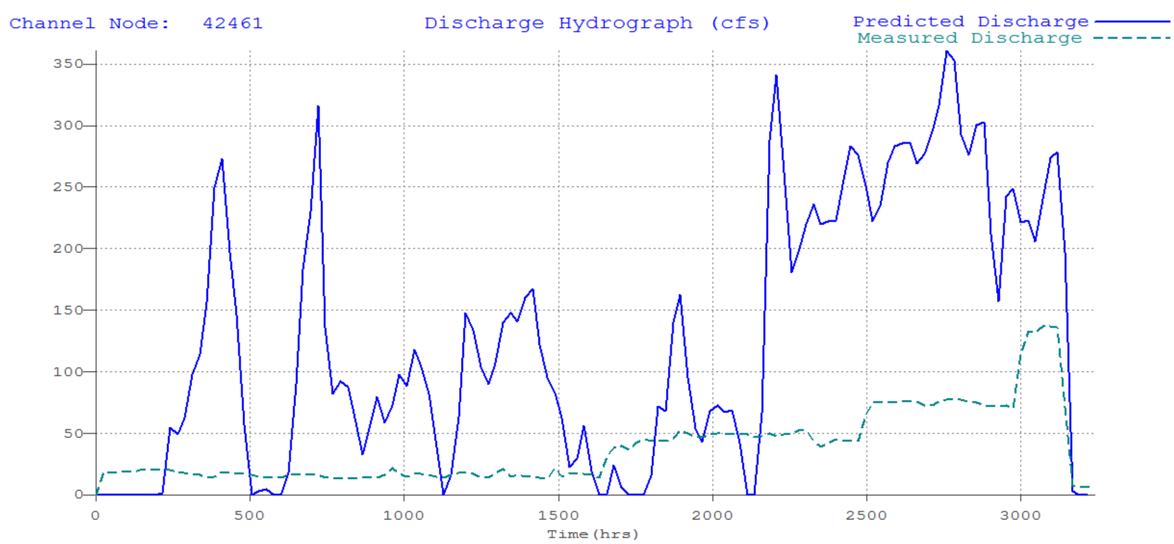

Figure G1-32. Below American Dam Gage, May 4 - September 15, 2011

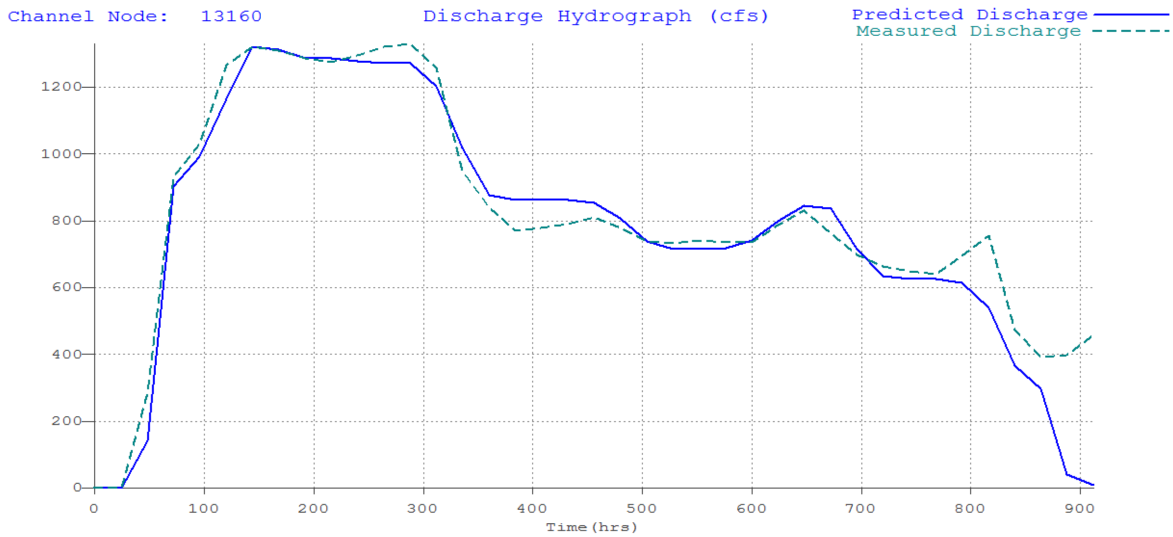

Figure G1-33. Haynor Gage, April 1 - May 8, 2012

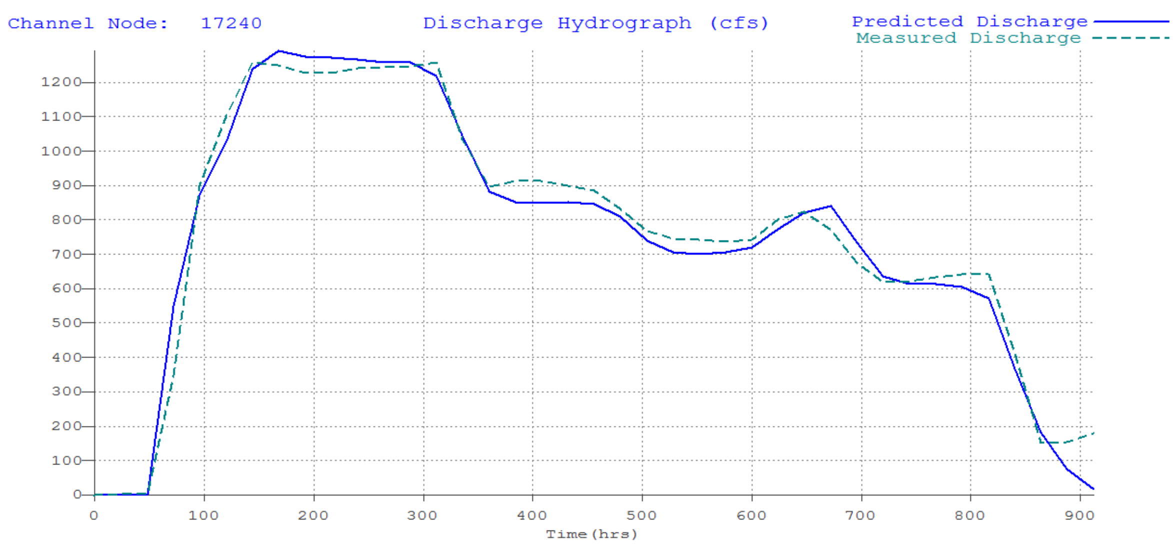

Figure G1-34. Leasburg Gage, April 1 - May 8, 2012

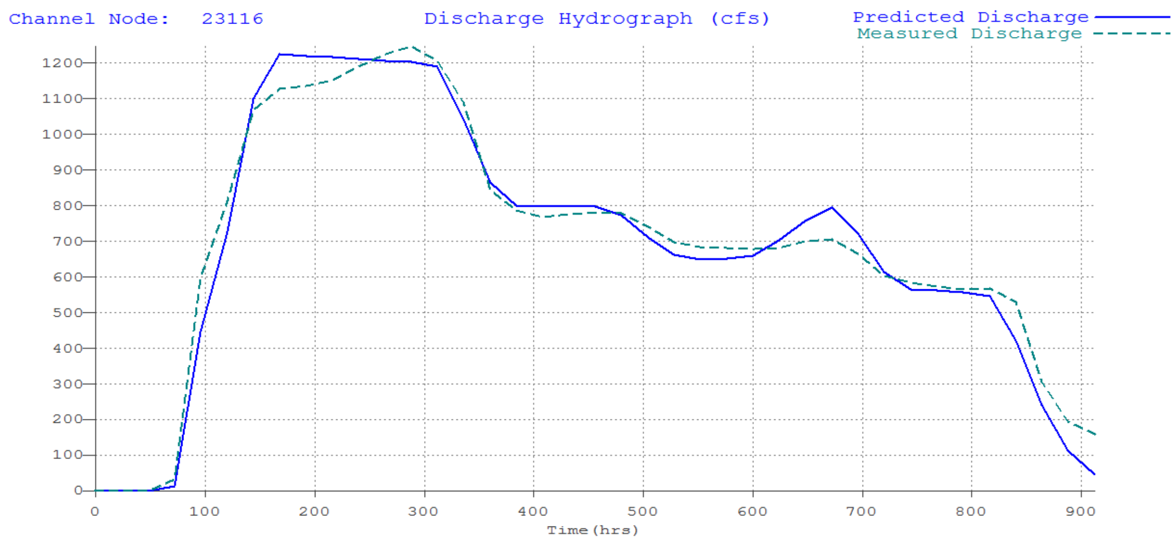

Figure G1-35. Picacho Gage, April 1 - May 8, 2012

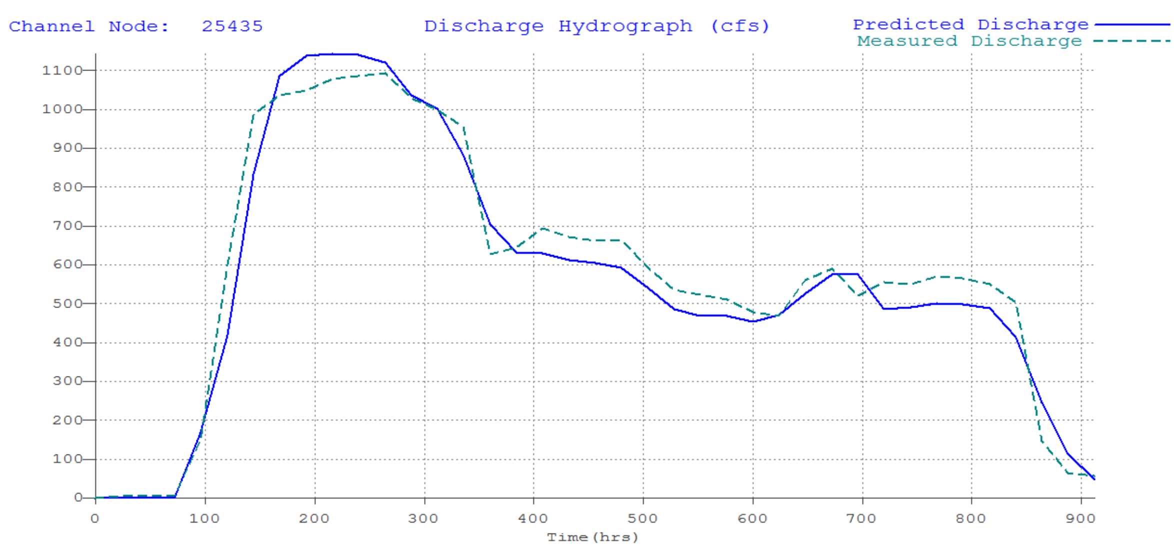

Figure G1-36. Mesilla Gage, April 1 - May 8, 2012

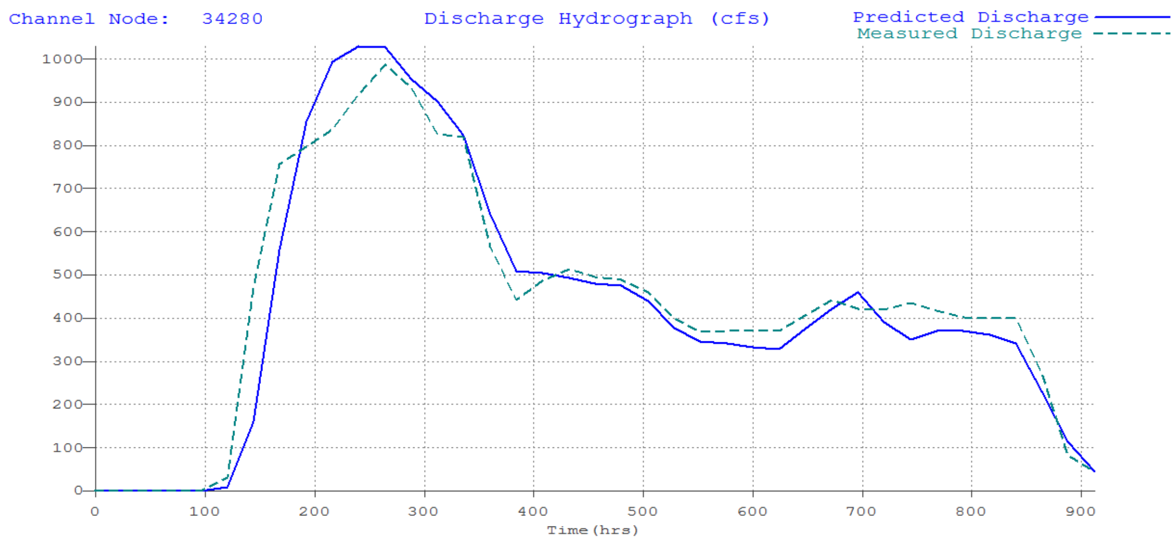

Figure G1-37. New Anthony Gage, April 1 - May 8, 2012

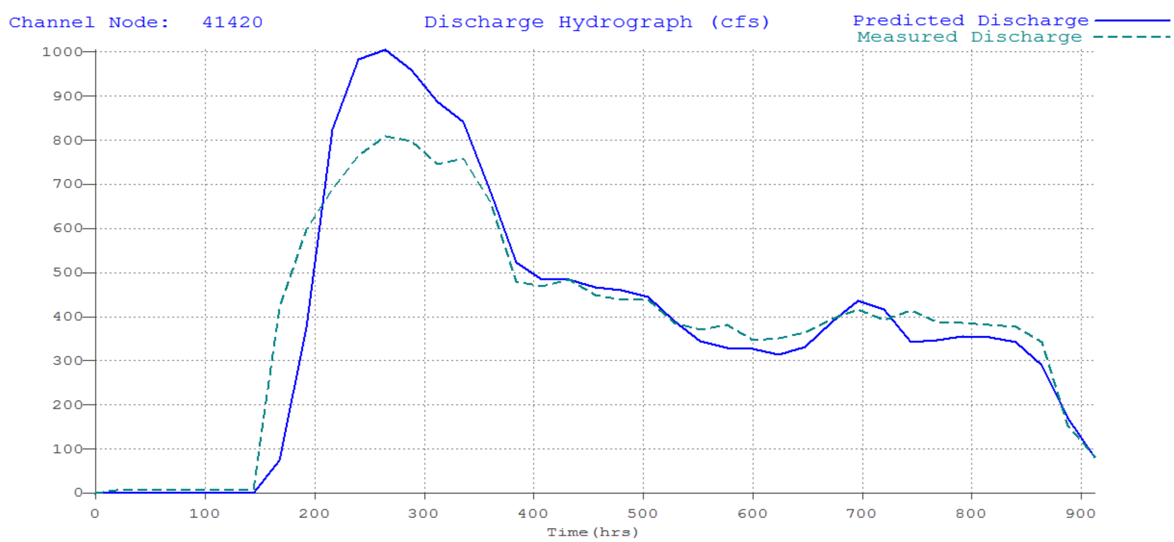

Figure G1-38. El Paso Gage, April 1 - May 8, 2012

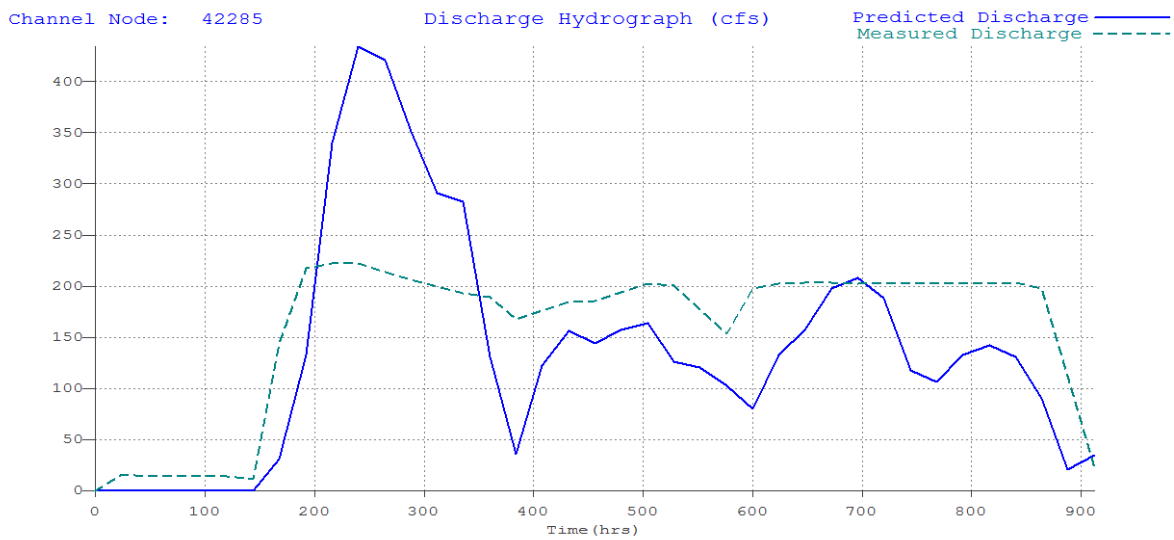

Figure G1-39. Below American Dam Gage, April 1 - May 8, 2012

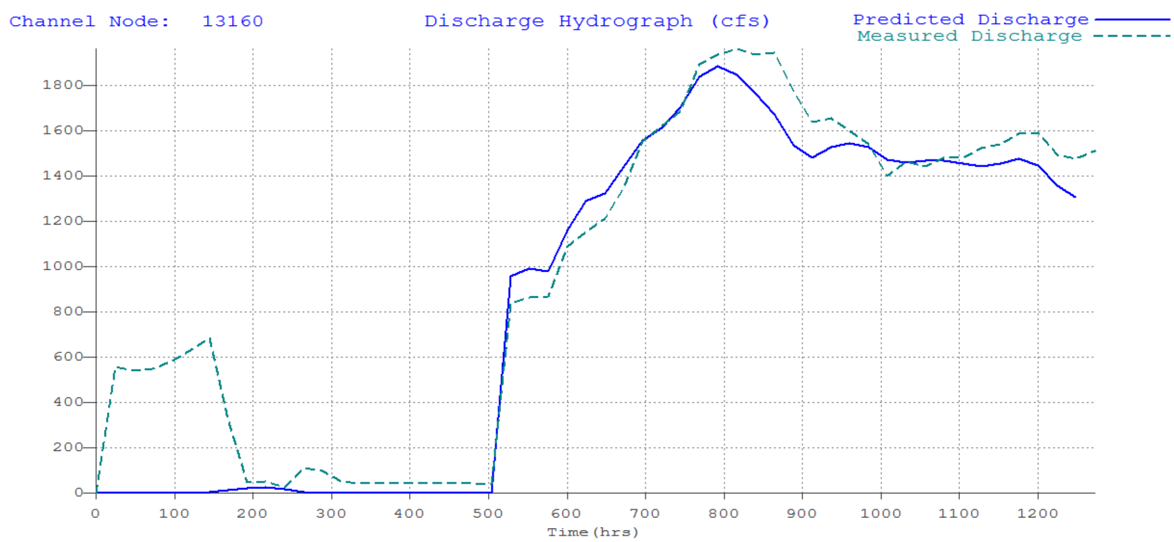

Figure G1-40. Haynor Gage, May 9 - June 29, 2012

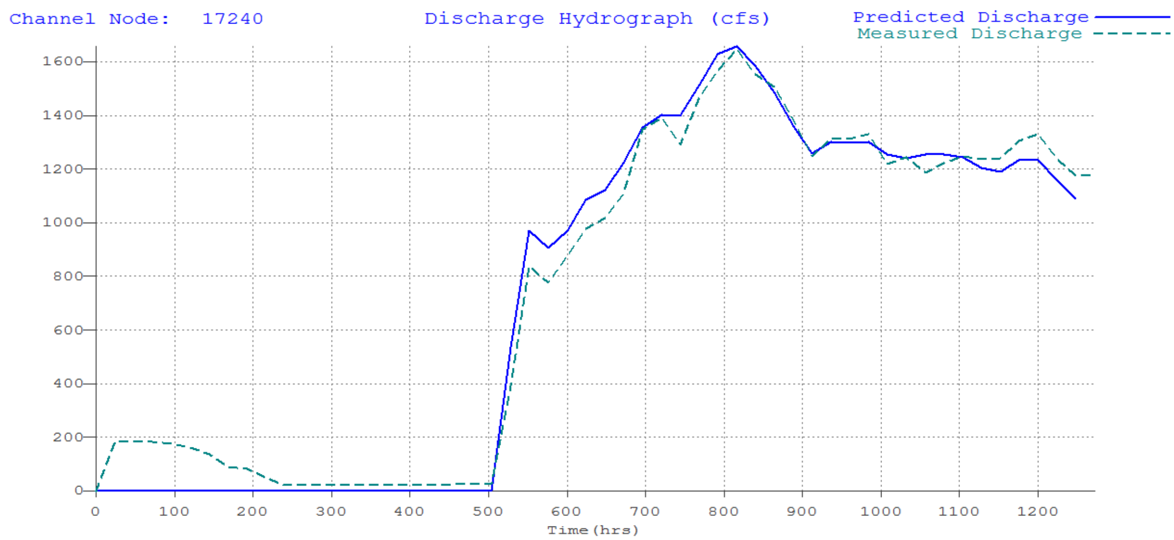

Figure G1-41. Leasburg Gage, May 9 - June 29, 2012

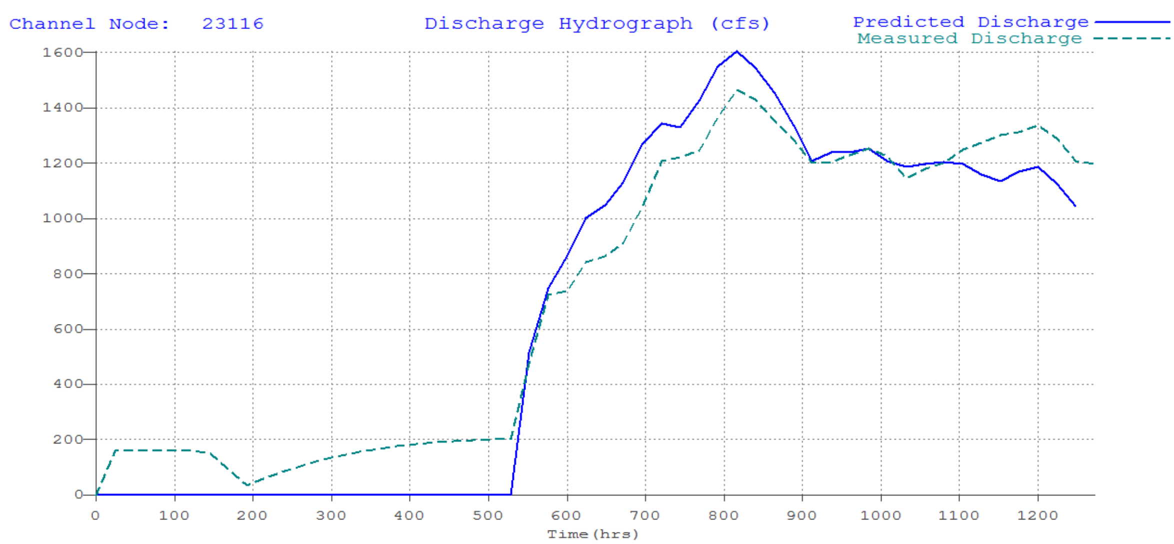

Figure G1-42. Picacho Gage, May 9 - June 29, 2012

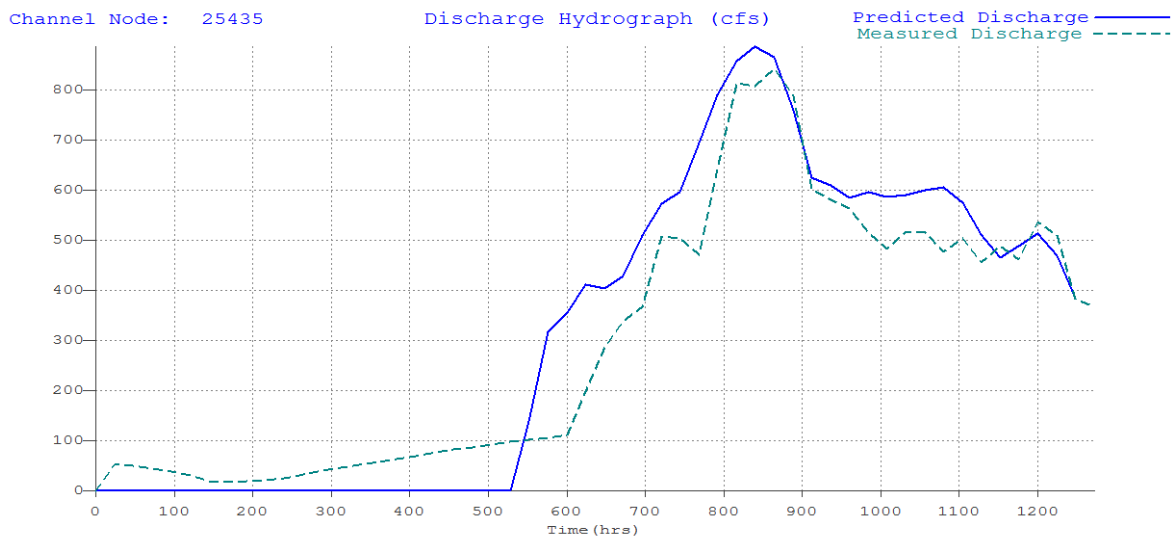

Figure G1-43. Mesilla Gage, May 9 - June 29, 2012

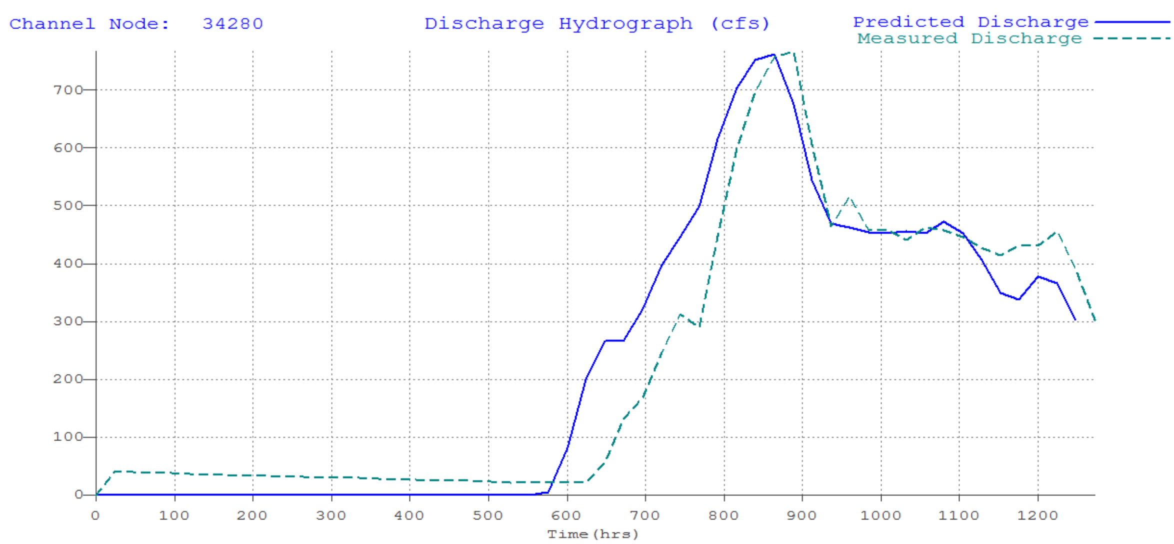

Figure G1-44. Anthony Gage, May 9 - June 29, 2012

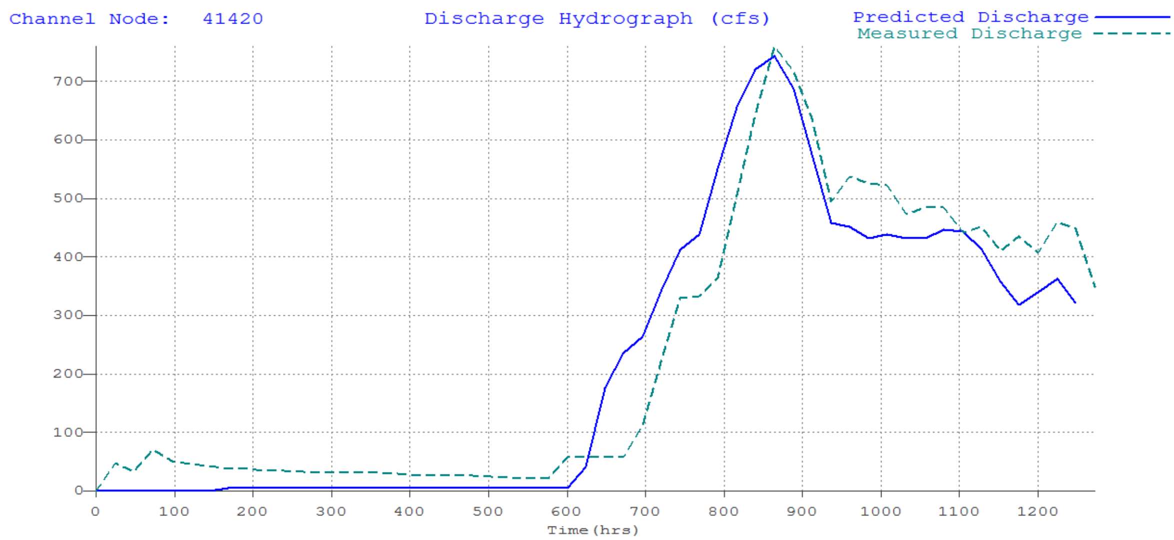

Figure G1-45. El Paso Gage, May 9 - June 29, 2012

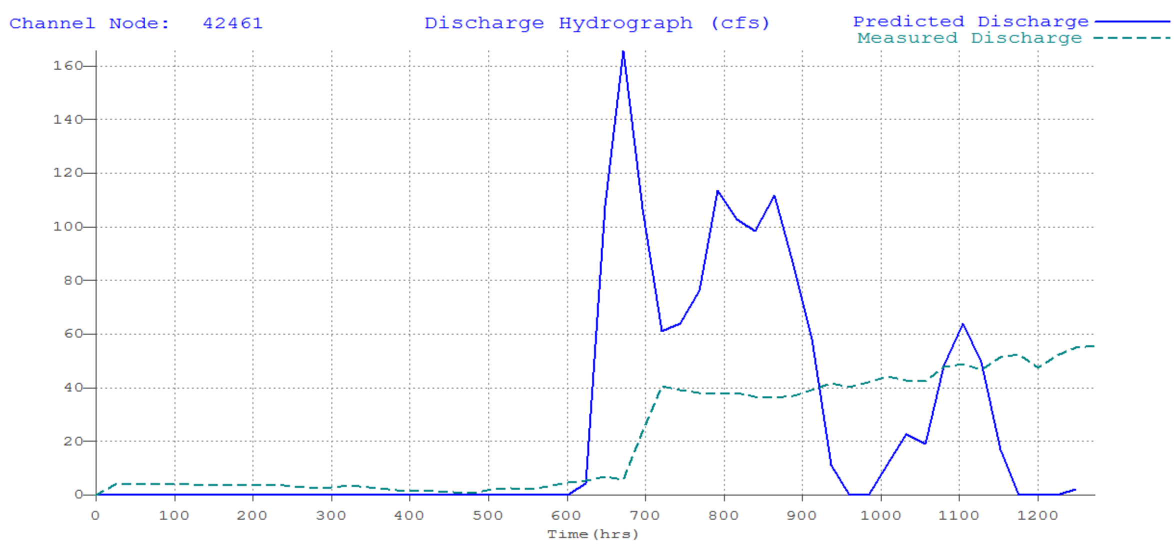

Figure G1-46. Below American Dam Gage, May - June 29, 2012

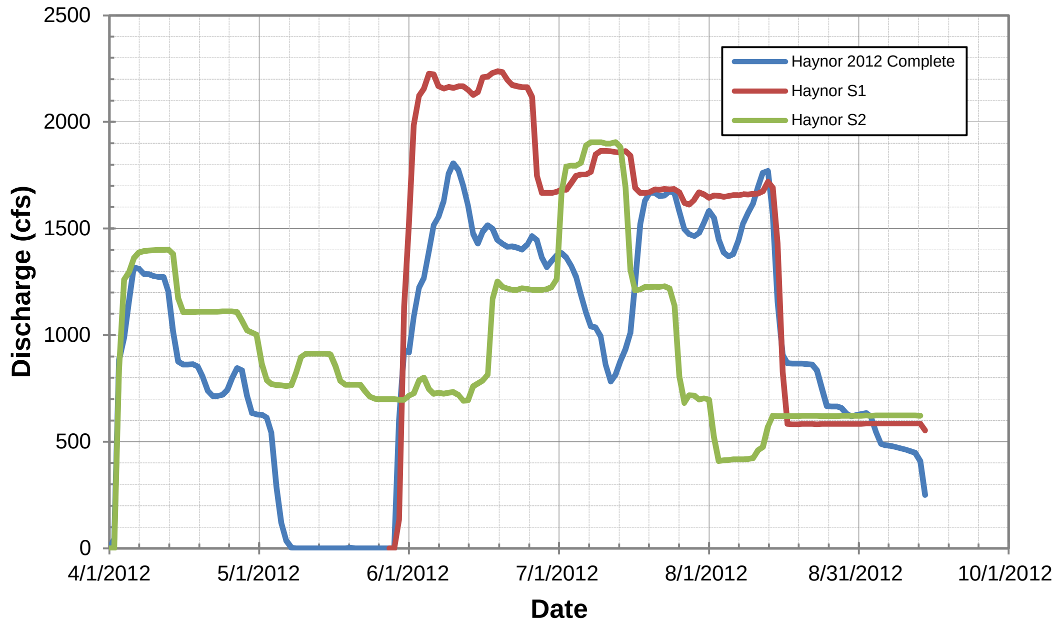

Figure G1-47. Comparison of Predicted Hydrographs under the 2012 Baseline Condition and Scenarios S1 and S2 at the Haynor Bridge Gage

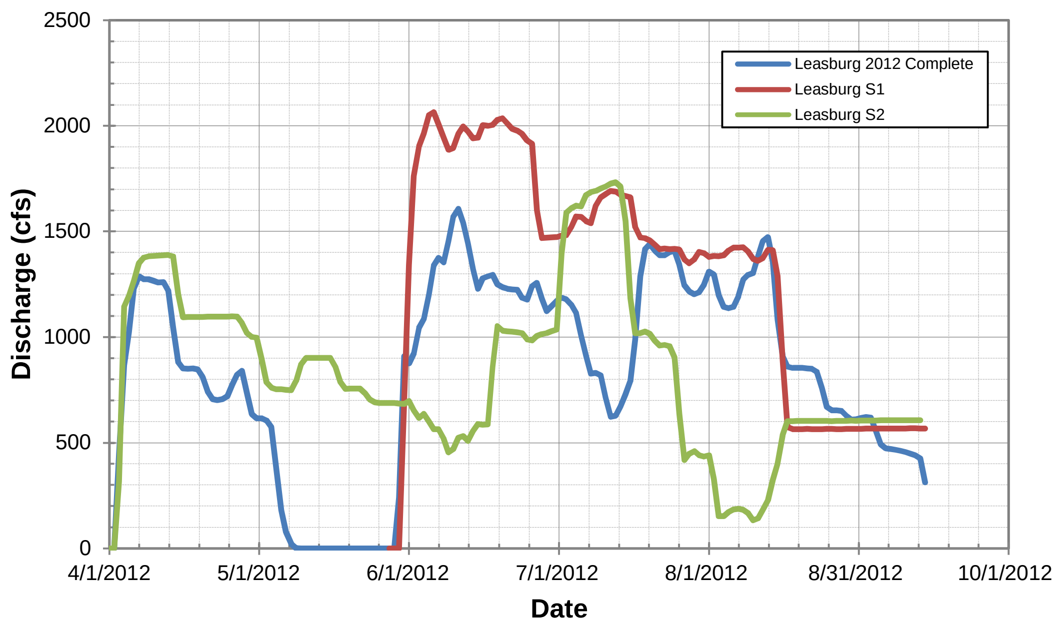

Figure G1-48. Comparison of Predicted Hydrographs under the 2012 Baseline Condition and Scenarios S1 and S2 at the Leasburg River Cable Gage

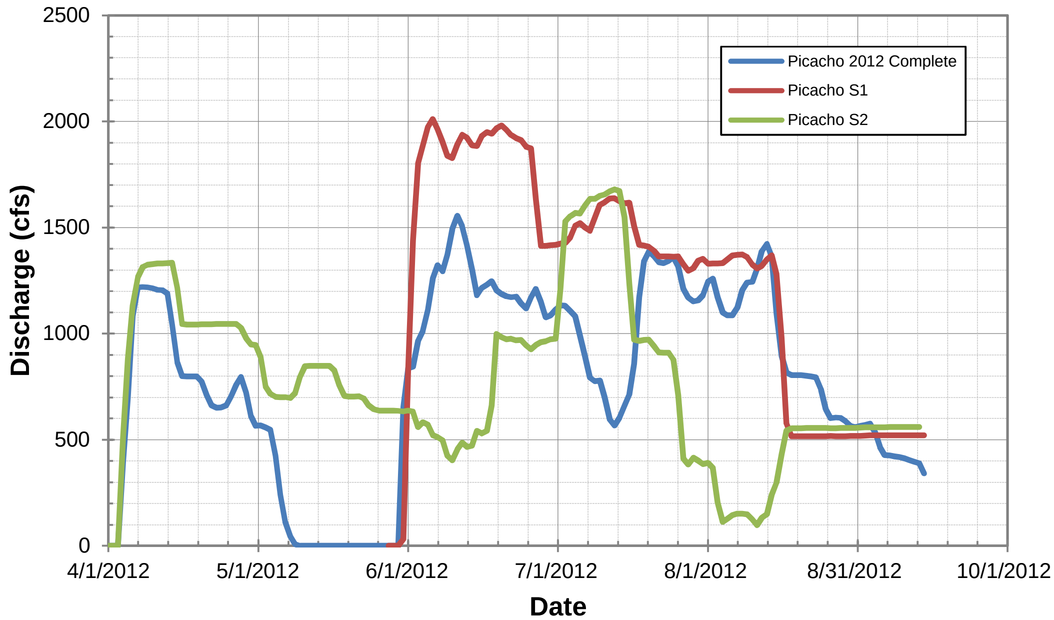

Figure G1-49. Comparison of Predicted Hydrographs under the 2012 Baseline Condition and Scenarios S1 and S2 at the Picacho Bridge Gage

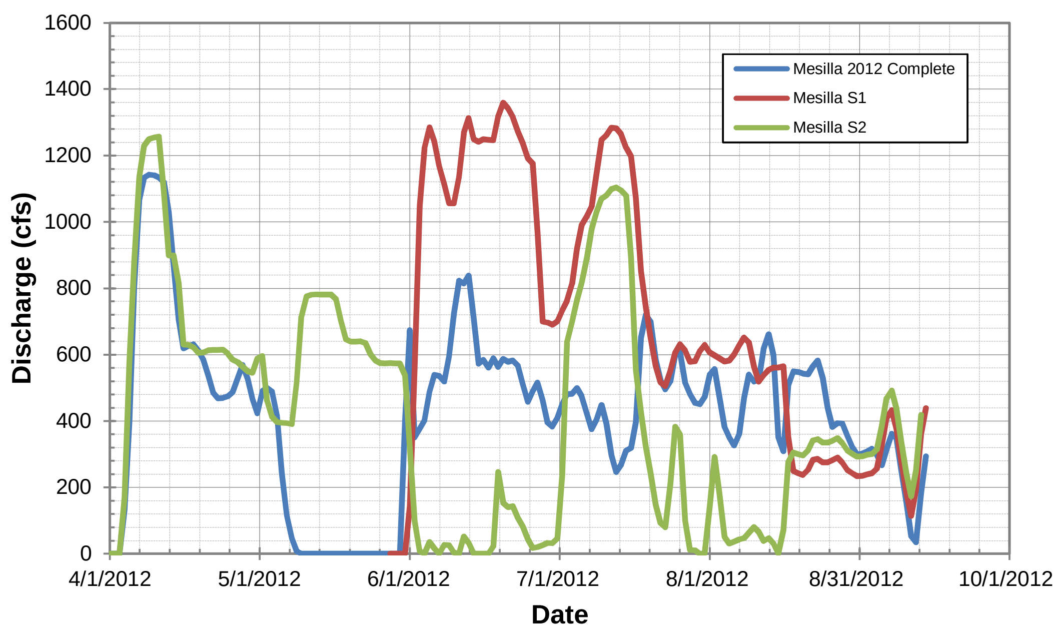

Figure G1-50. Comparison of Predicted Hydrographs under the 2012 Baseline Condition and Scenarios S1 and S2 at the Below Mesilla Dam Gage

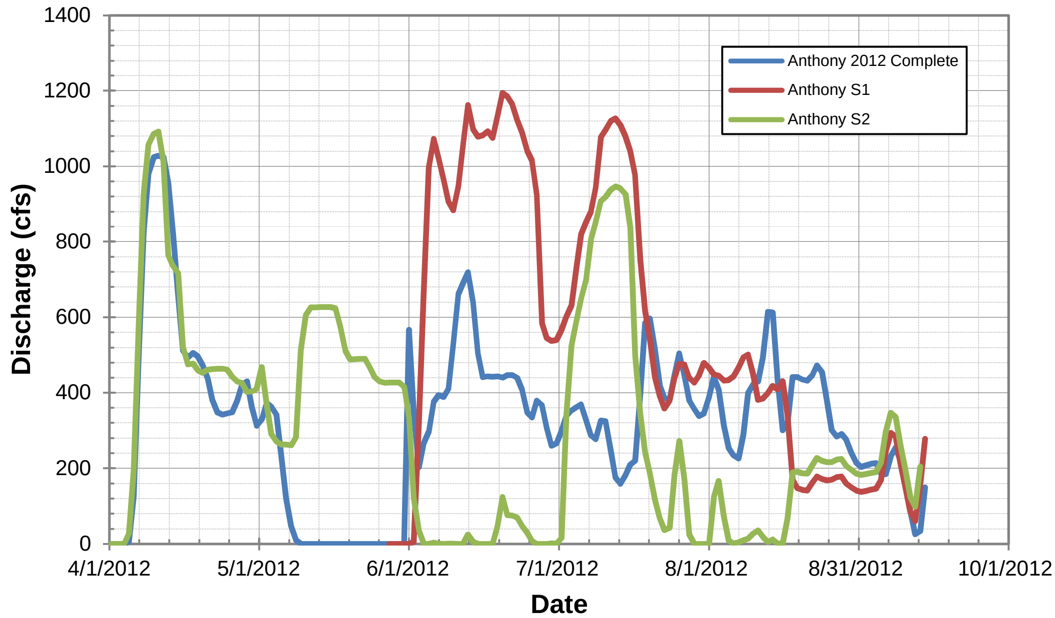

Figure G1-51. Comparison of Predicted Hydrographs under the 2012 Baseline Condition and Scenarios S1 and S2 at the Anthony Bridge Gage

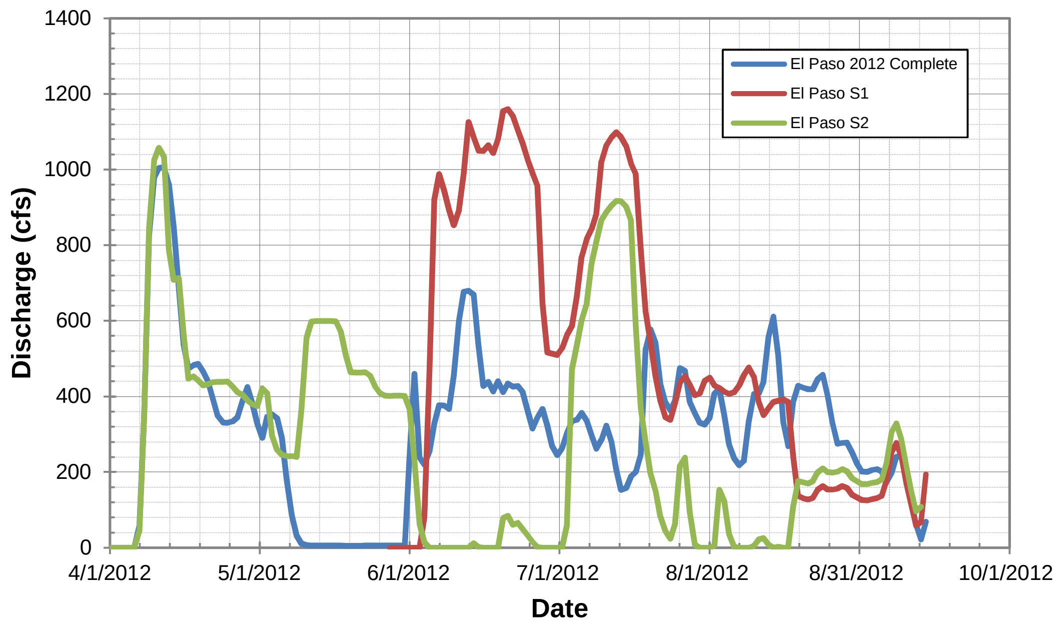

Figure G1-52. Comparison of Predicted Hydrographs under the 2012 Baseline Condition and Scenarios S1 and S2 at the El Paso (Courchesne Bridge) Gage

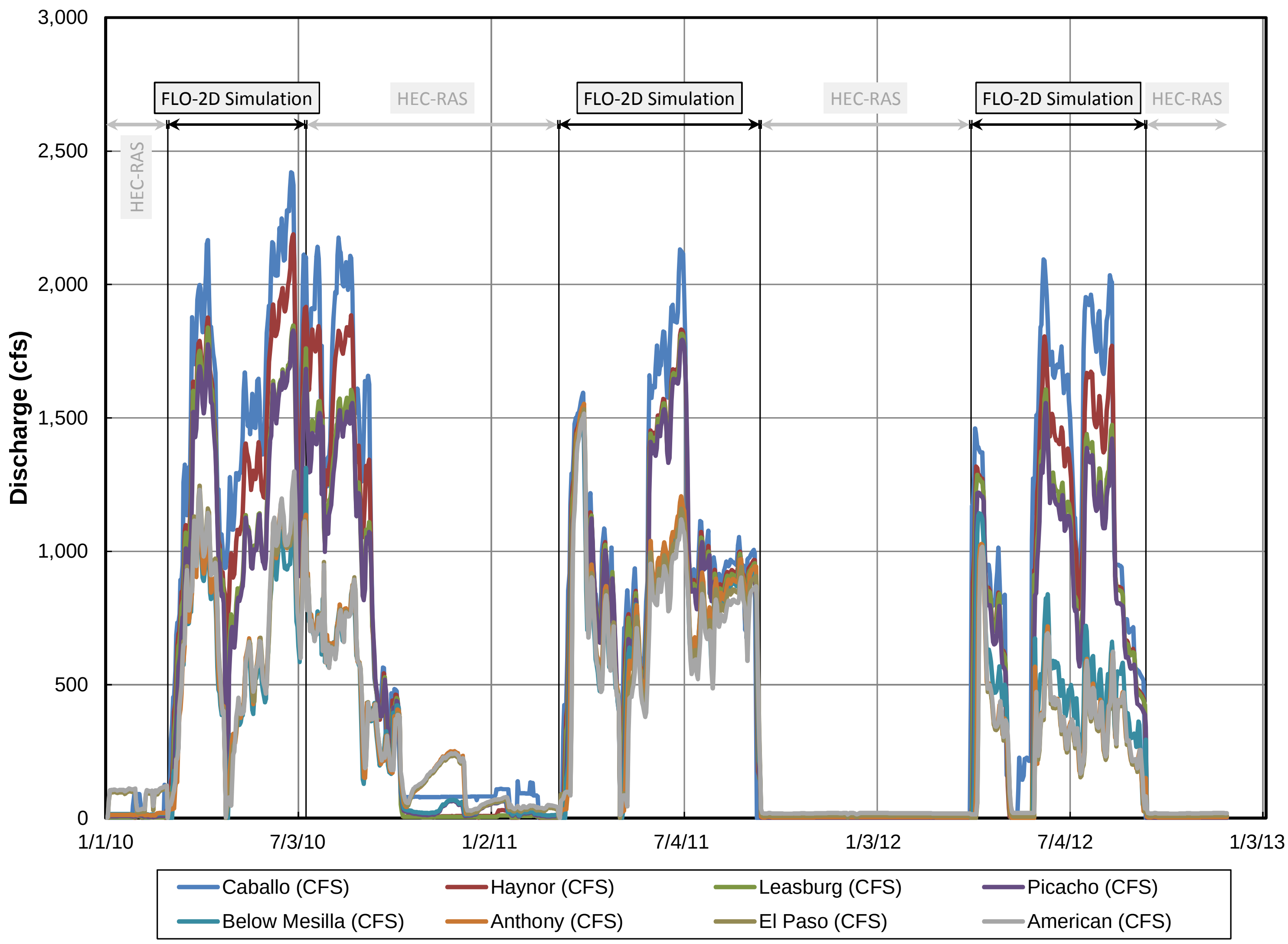

Figure G1-53. Hydrographs Predicted by the FLO-2D Model Runs, and the Adopted HEC-RAS Results for Periods Outside of the FLO-2D Model Simulations

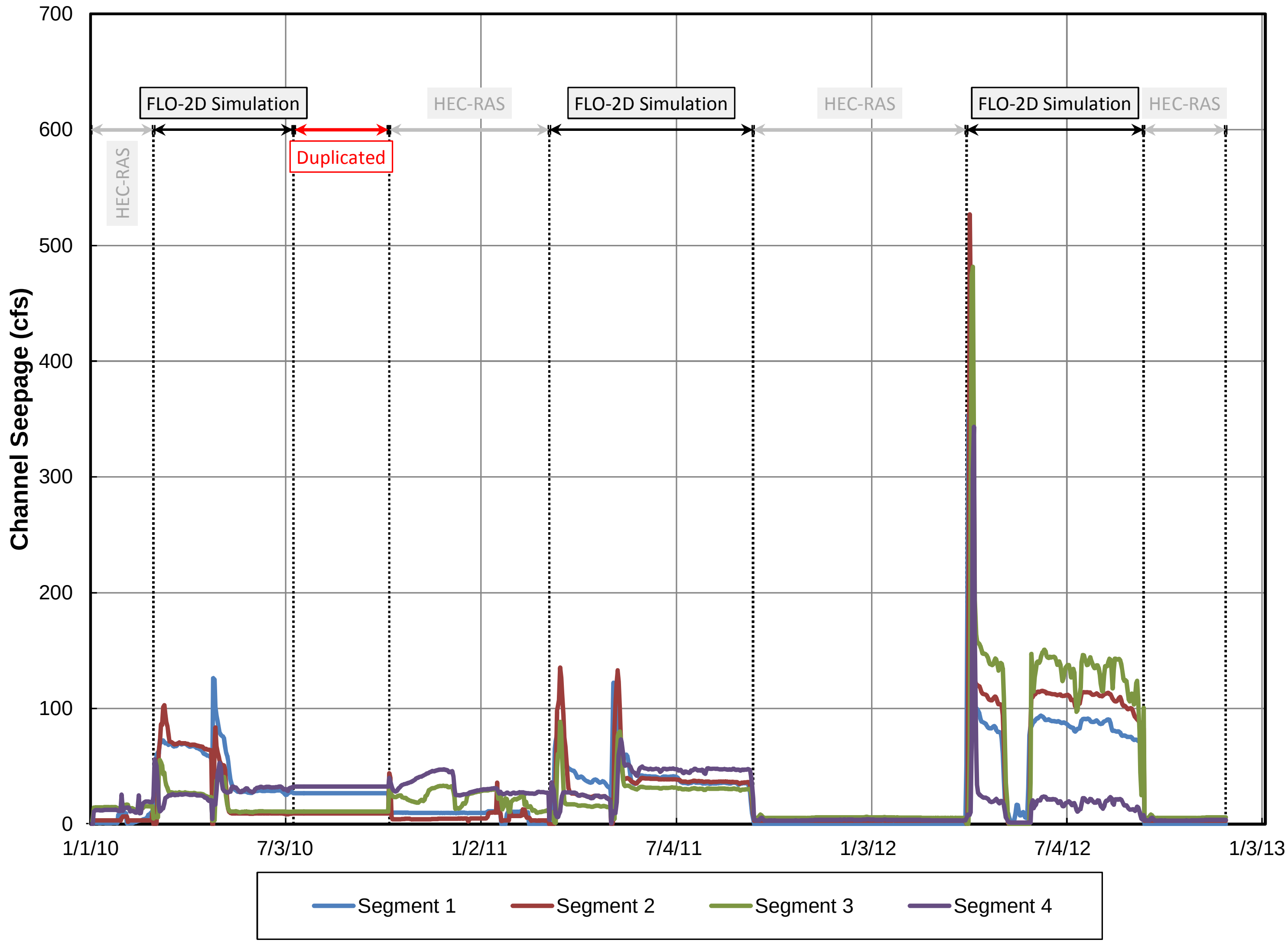

Figure G1-54. Seepage Rates from the FLO-2D Simulations and the Duplicated or Adopted (HEC-RAS) Seepage Rates that were used in the Water Budget Analyses

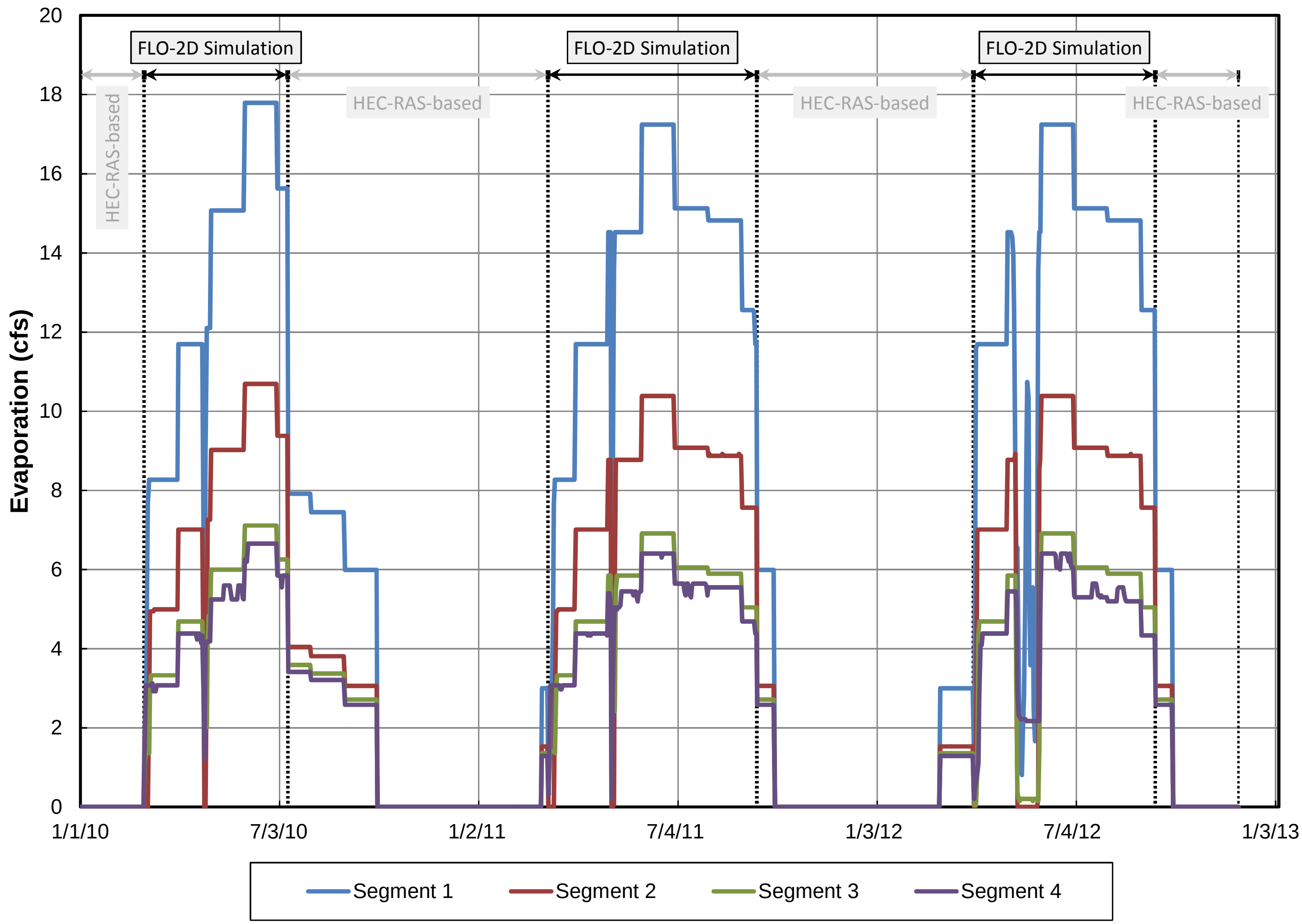

Figure G1-55. Evaporation Rates from the FLO-2D Simulations and the Adopted (HEC-RAS-Based) Seepage Rates that were used in the Water Budget Analyses

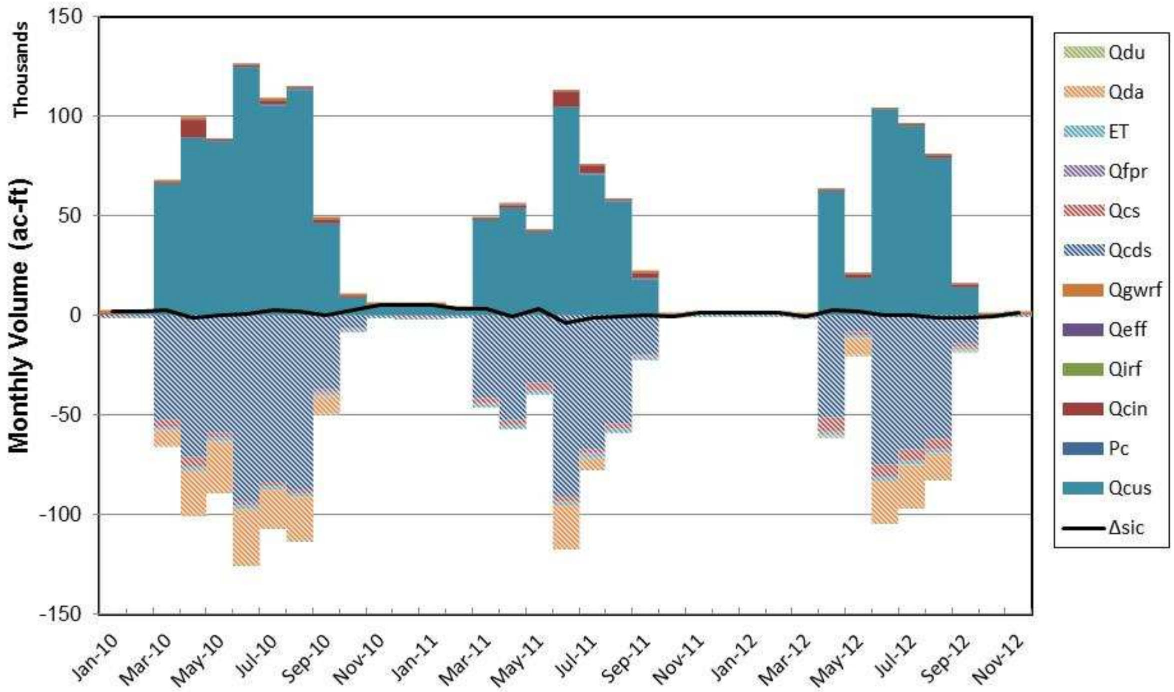

Figure G1-56. Stacked Bar Chart Showing Monthly Volumes of the RGCP-Scale Channel Water Budget Analysis and the Resulting Monthly Change in Channel Storage in Segment 1

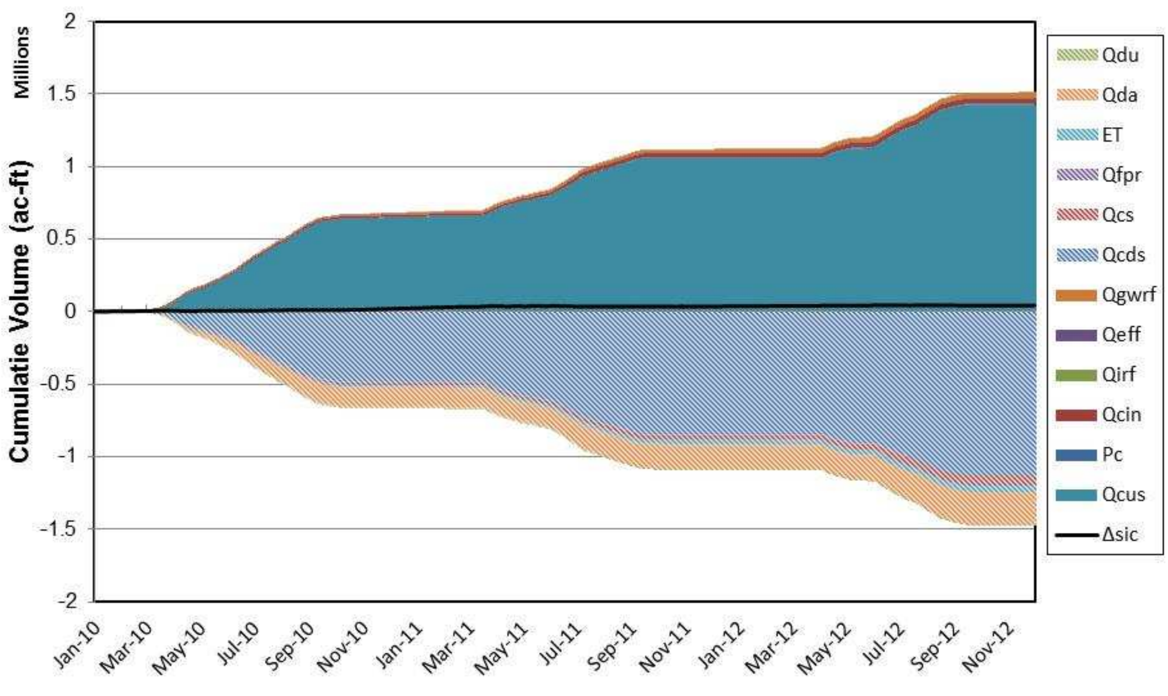

Figure G1-57. Stacked Bar Chart Showing Cumulative Volumes of the RGCP-Scale Channel Water Budget Analysis and the Resulting Cumulative Change in Channel Storage in Segment 1

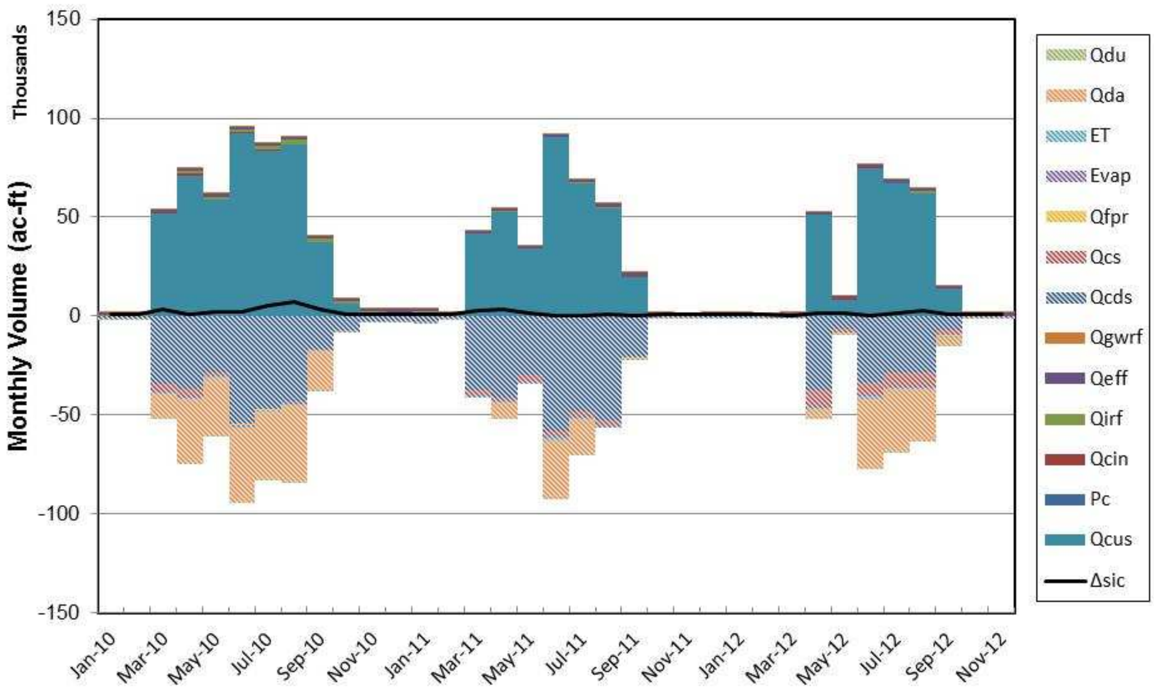

Figure G1-58. Stacked Bar Chart Showing Monthly Volumes of the RGCP-Scale Channel Water Budget Analysis and the Resulting Monthly Change in Channel Storage in Segment 2

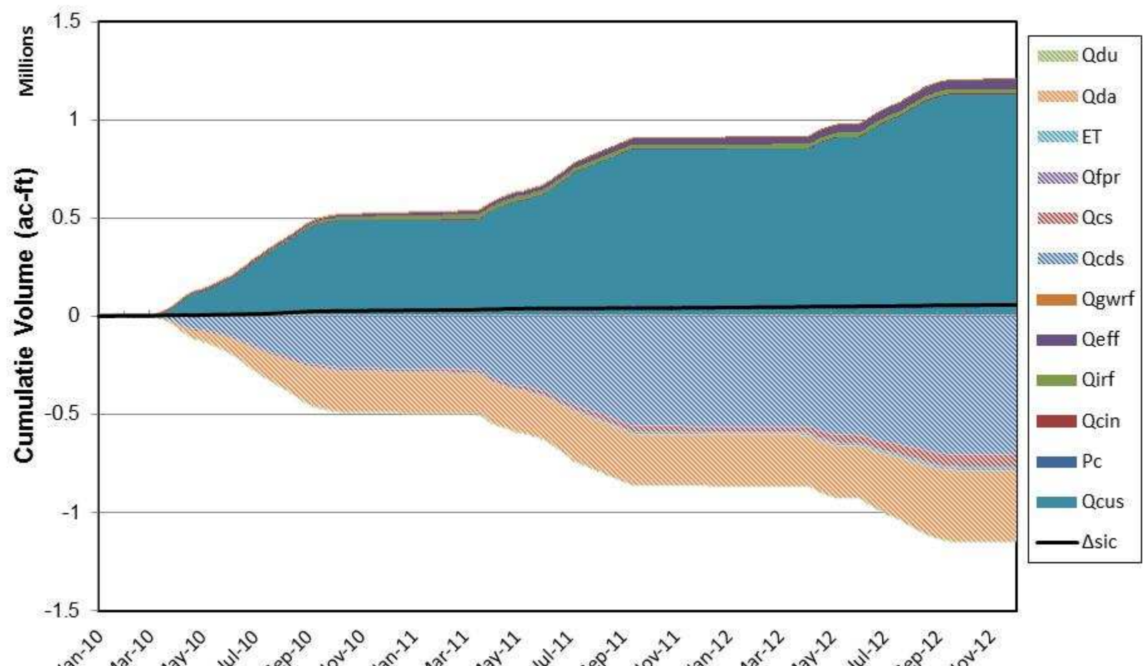

Figure G1-59. Stacked Bar Chart Showing Cumulative Volumes of the RGCP-Scale Channel Water Budget Analysis and the Resulting Cumulative Change in Channel Storage in Segment 2

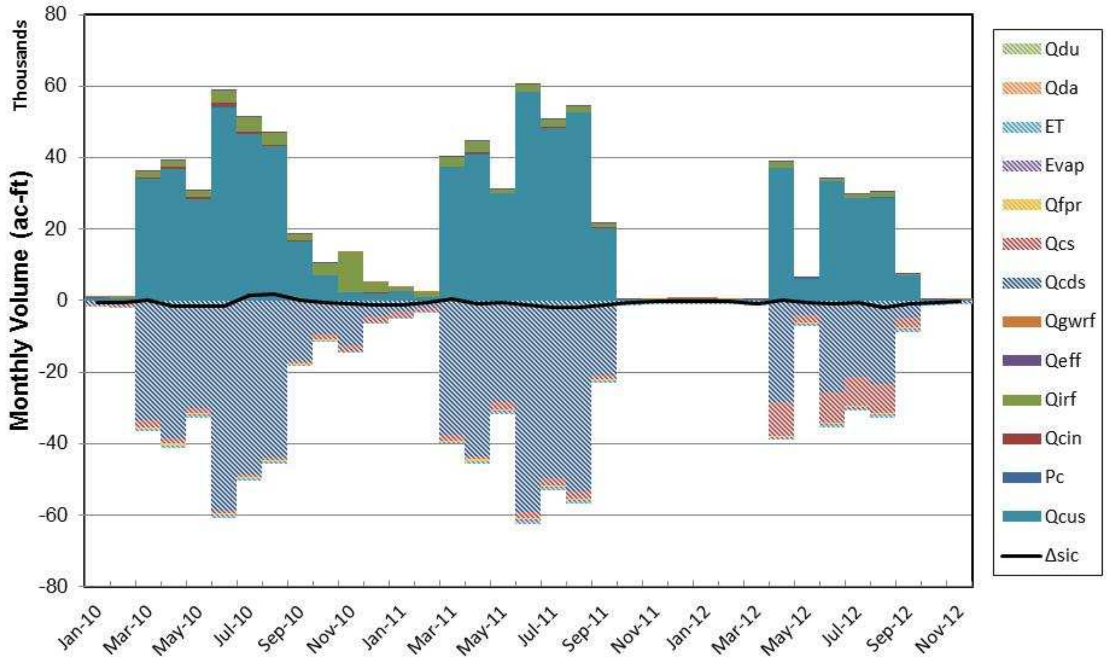

Figure G1-60. Stacked Bar Chart Showing Monthly Volumes of the RGCP-Scale Channel Water Budget Analysis and the Resulting Monthly Change in Channel Storage in Segment 3

Figure G1-61. Stacked Bar Chart Showing Cumulative Volumes of the RGCP-Scale Channel Water Budget Analysis and the Resulting Cumulative Change in Channel Storage in Segment 3

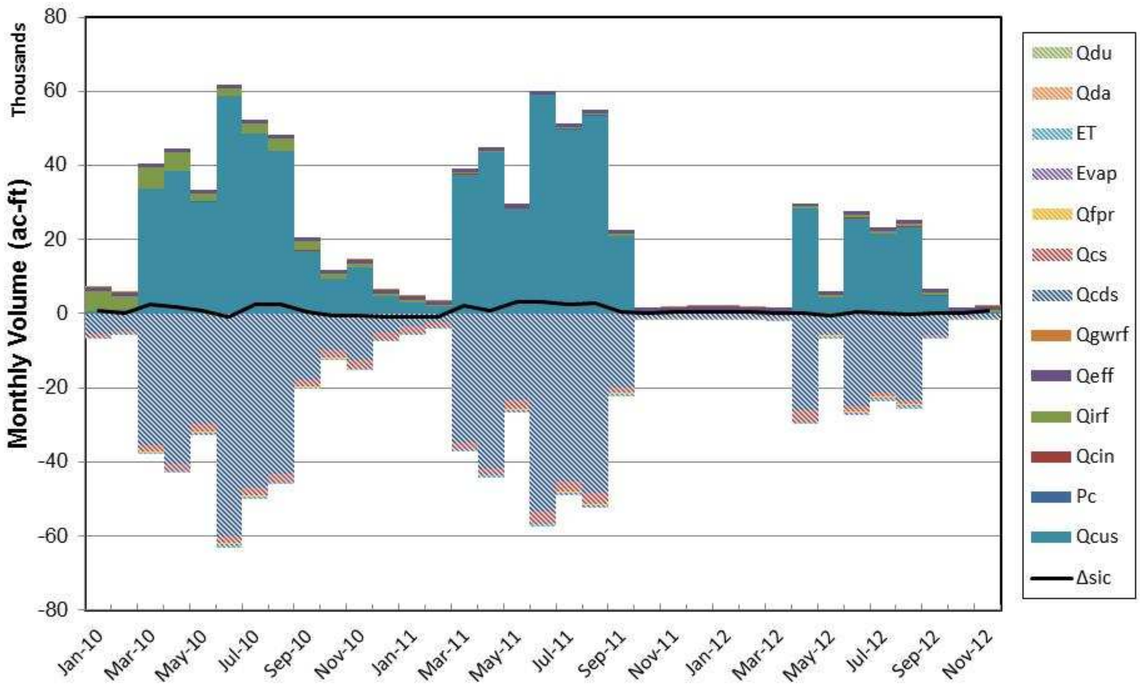

Figure G1-62. Stacked Bar Chart Showing Monthly Volumes of the RGCP-Scale Channel Water Budget Analysis and the Resulting Monthly Change in Channel Storage in Segment 4

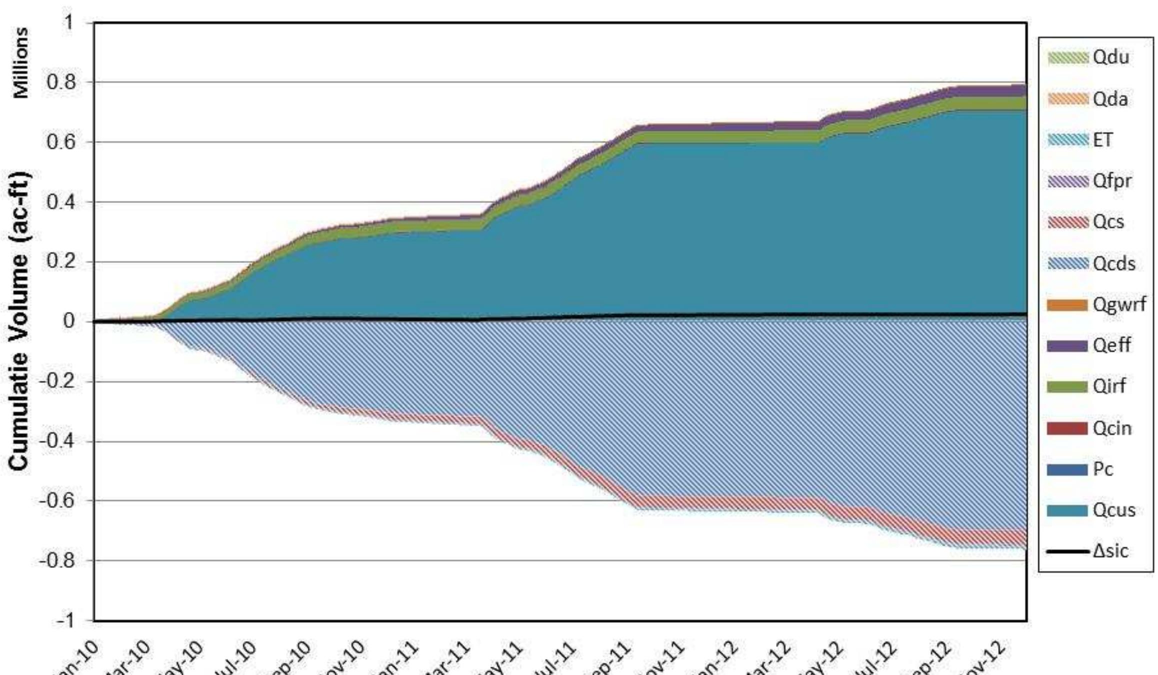

Figure G1-63. Stacked Bar Chart Showing Cumulative Volumes of the RGCP-Scale Channel Water Budget Analysis and the Resulting Cumulative Change in Channel Storage in Segment 4

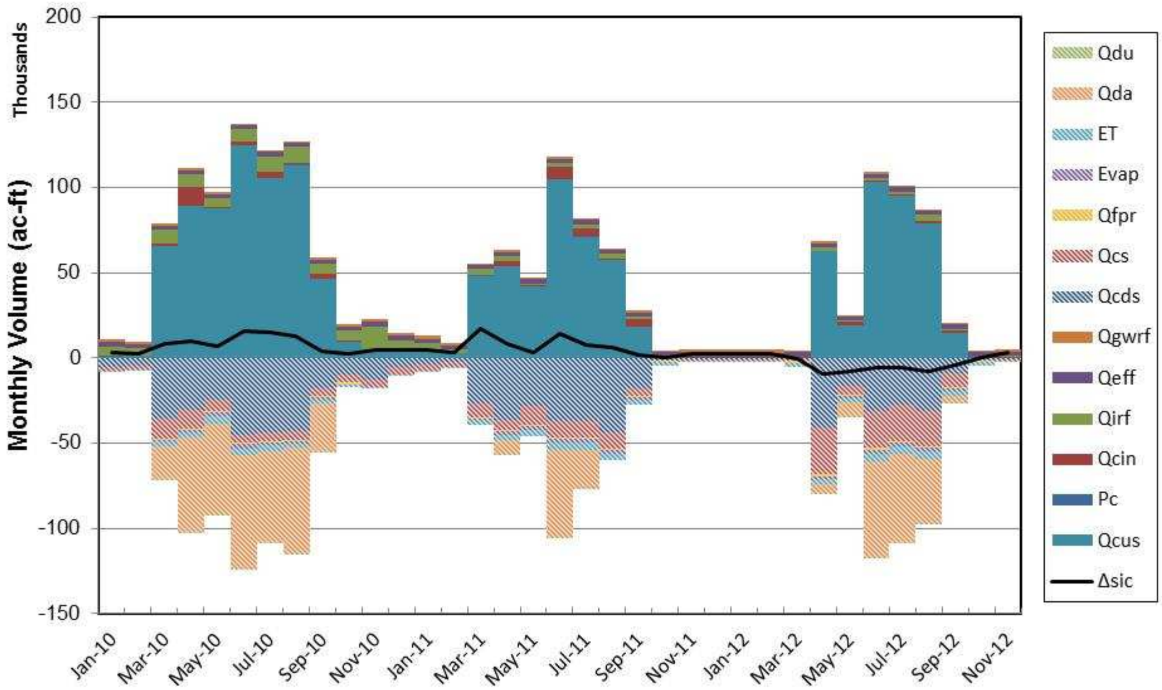

Figure G1-64. Stacked Bar Chart Showing Monthly Volumes of the RGCP-Scale Channel Water Budget Analysis and the Resulting Monthly Change in Channel Storage along the Overall RGCP

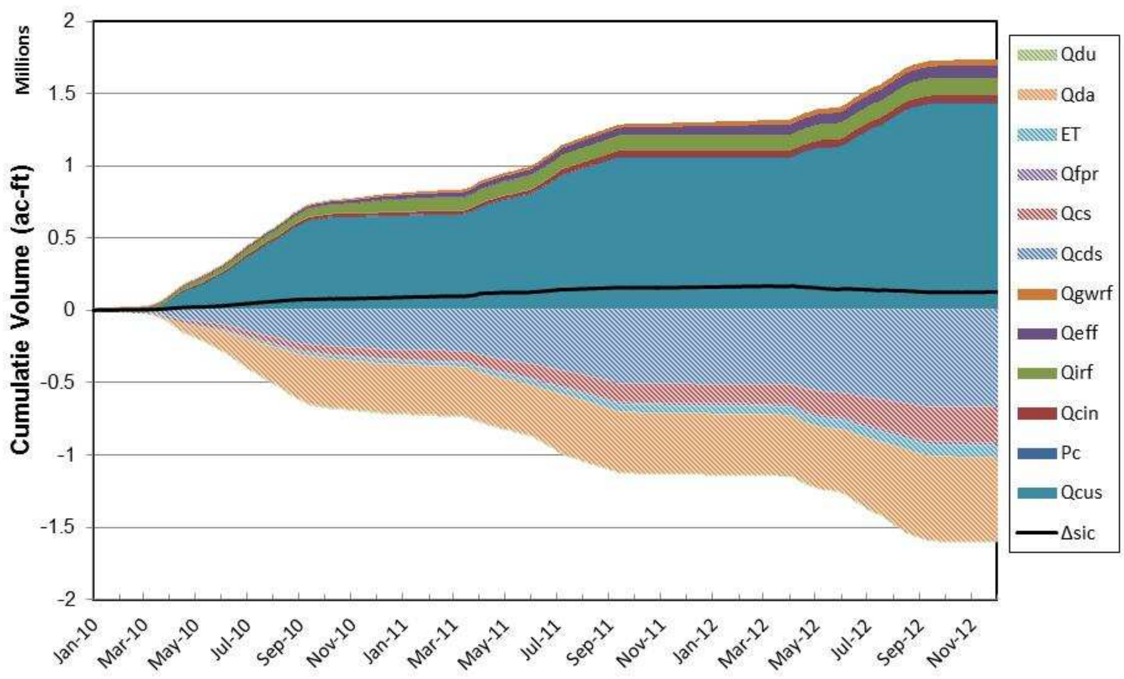

Figure G1-65. Stacked Bar Chart Showing Cumulative Volumes of the RGCP-Scale Channel Water Budget Analysis and the Resulting Cumulative Change in Channel Storage along the Overall RGCP

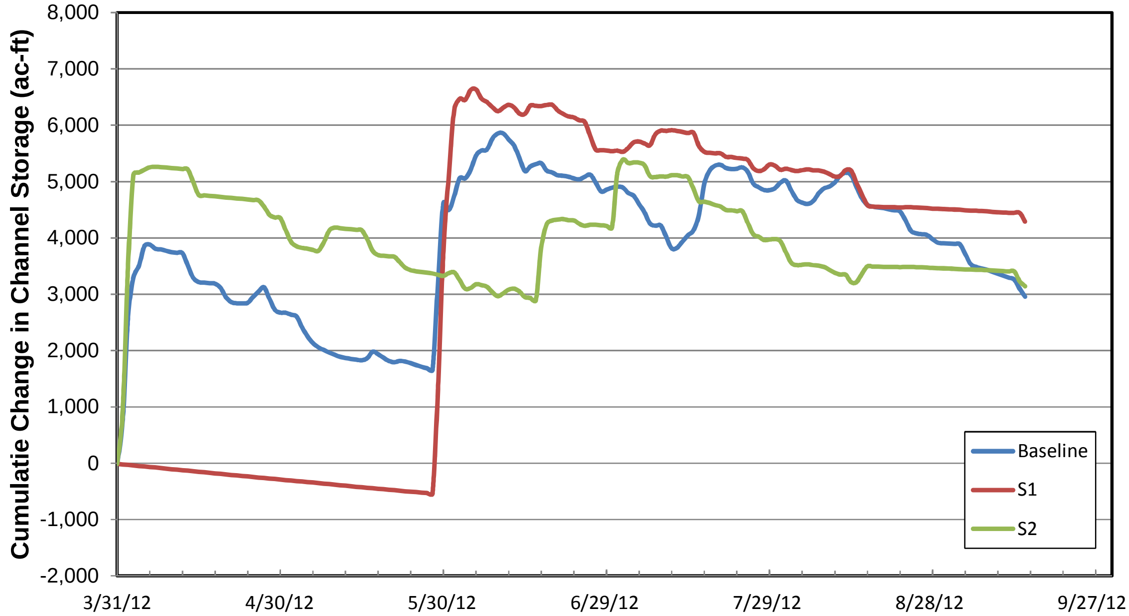

Figure G1-66. Cumulative Change in Channel Storage during 2012 under Baseline (Actual) Conditions and under the Hypothetical Release Scenarios (Scenarios S1 and S2) – Segment 1

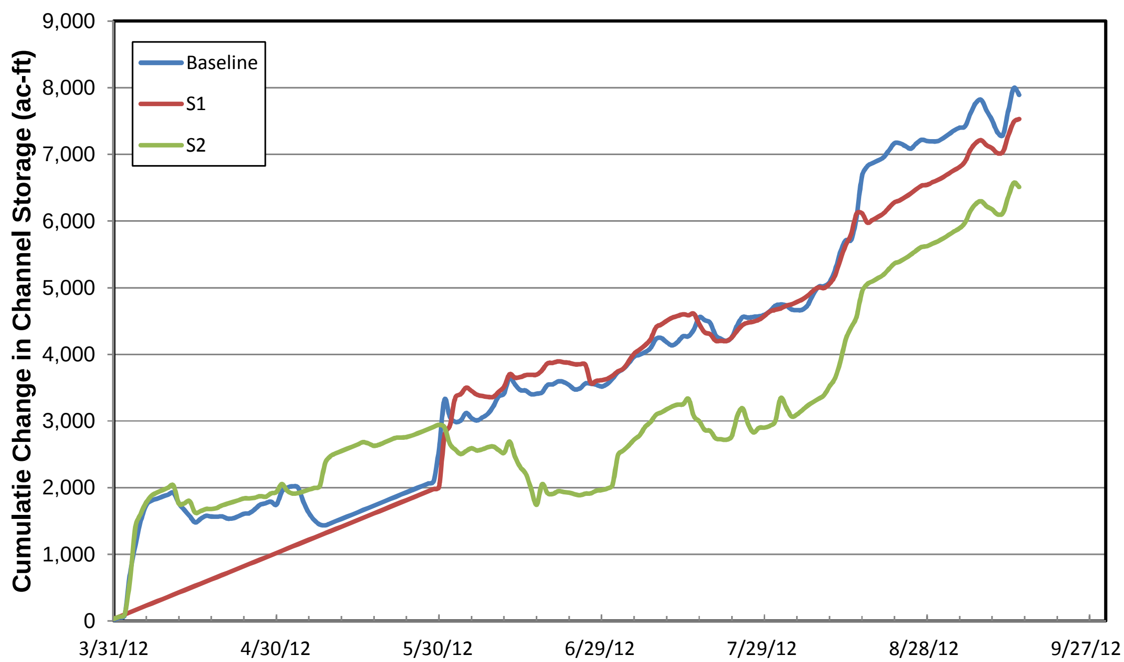

Figure G1-67. Cumulative Change in Channel Storage During 2012 under Baseline (Actual) Conditions and under the Hypothetical Release Scenarios (Scenarios S1 and S2) – Segment 2

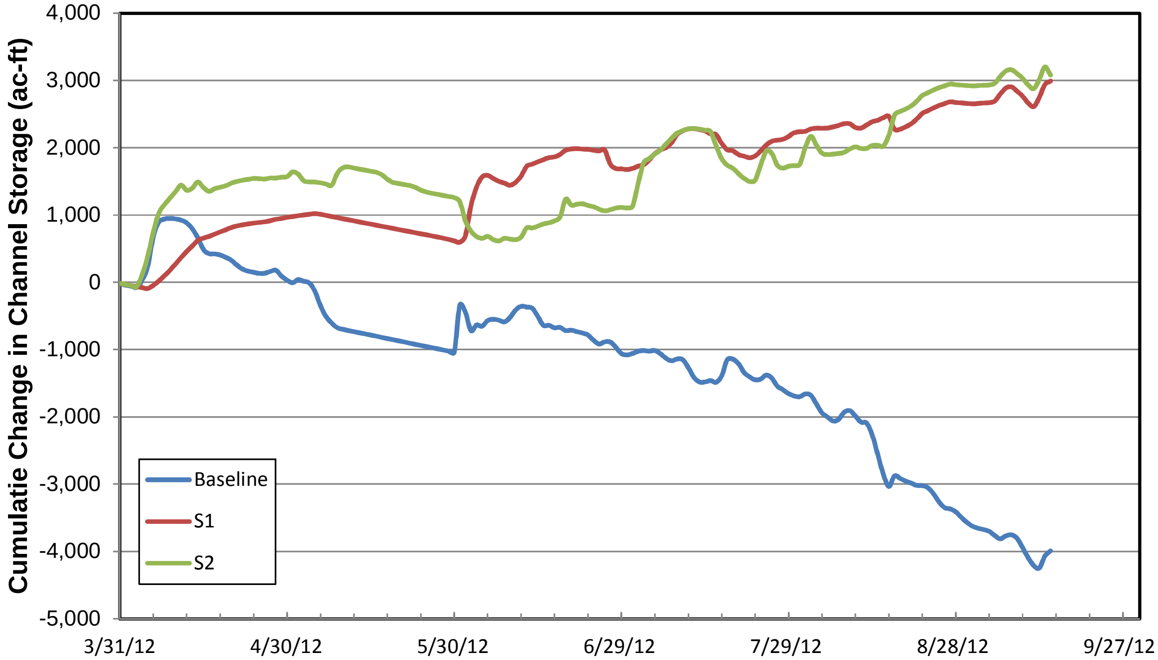

Figure G1-68. Cumulative Change in Channel Storage during 2012 under Baseline (Actual) Conditions and under the Hypothetical Release Scenarios (Scenarios S1 and S2) – Segment 3

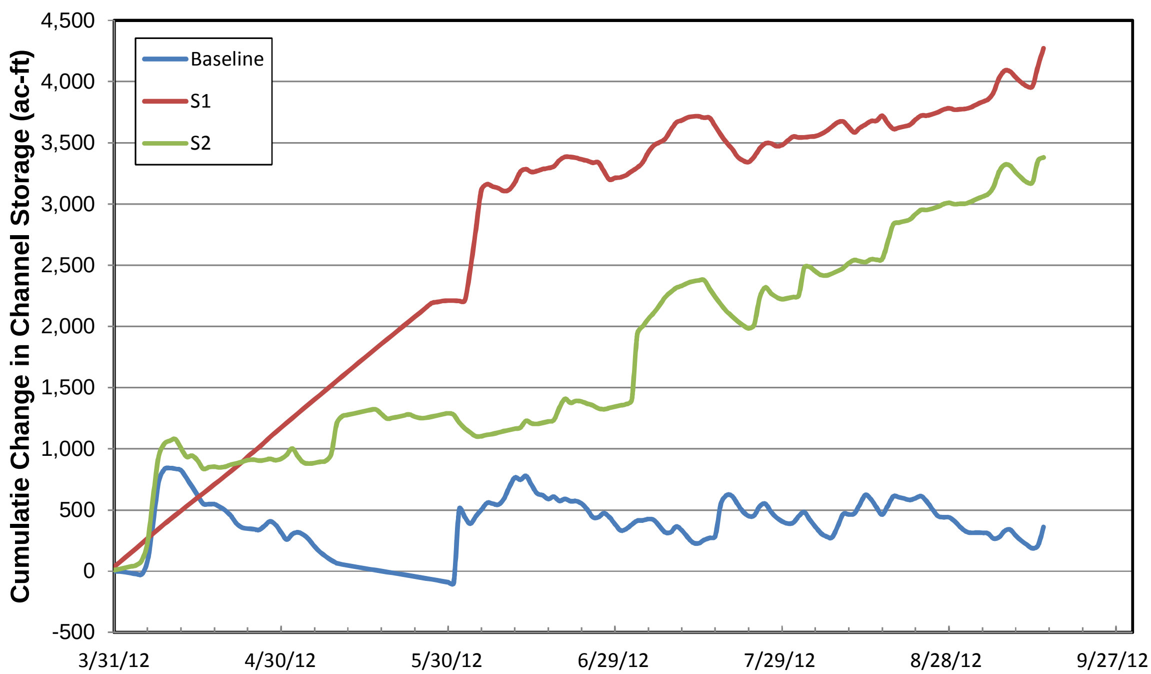

Figure G1-69. Cumulative Change in Channel Storage during 2012 under Baseline (Actual) Conditions and under the Hypothetical Release Scenarios (Scenarios S1 and S2) – Segment 4

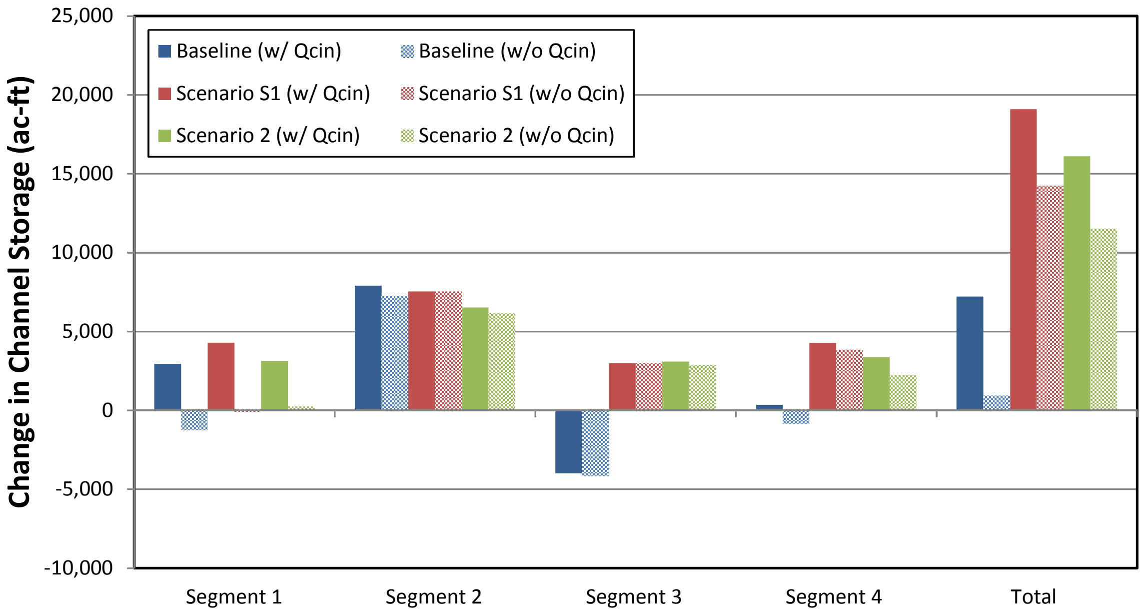

Figure G1-70. Total Change in Channel Storage with and without the Q_{cin} Component at the end of the 2012 Release under Baseline (Actual) Conditions and under the Hypothetical Release Scenarios (Scenarios S1 and S2)

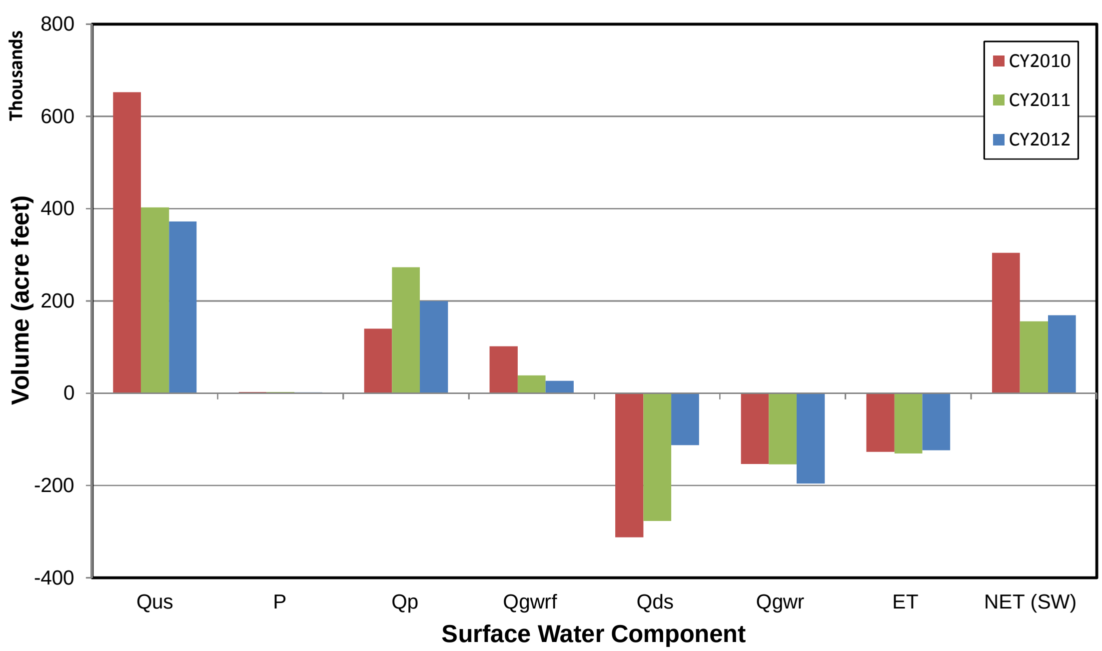

Figure G1-71. Annual and Total Volume for each Component of the Local-Basin-Scale Surface-water Budget

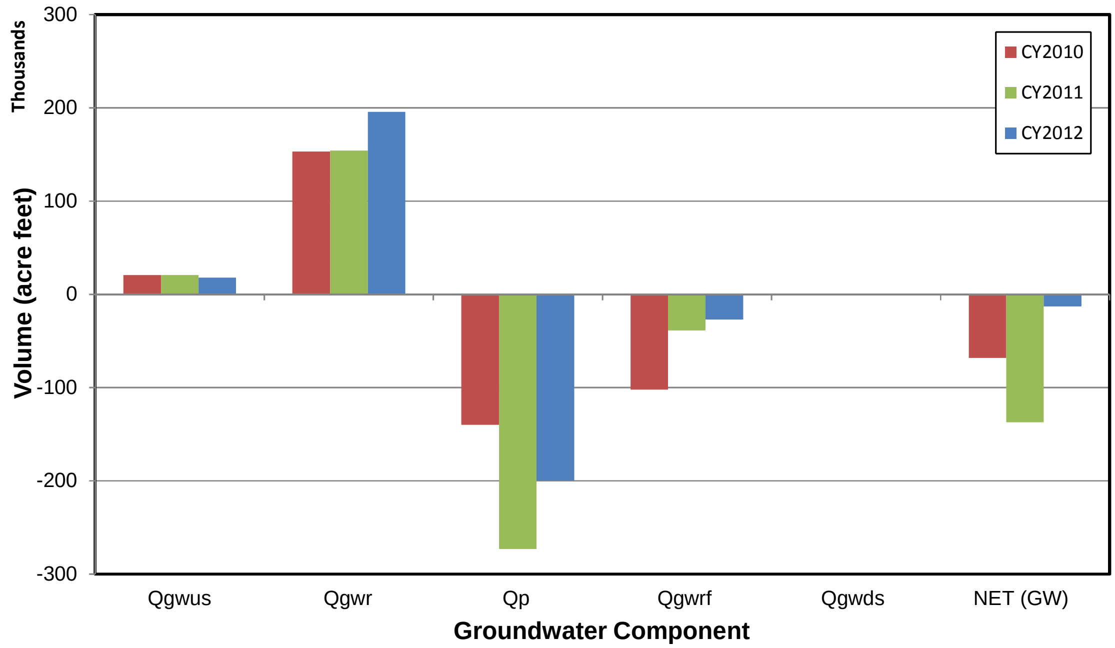

Figure G1-72. Annual and Total Volume for each Component of the Local-Basin-Scale Groundwater Budget

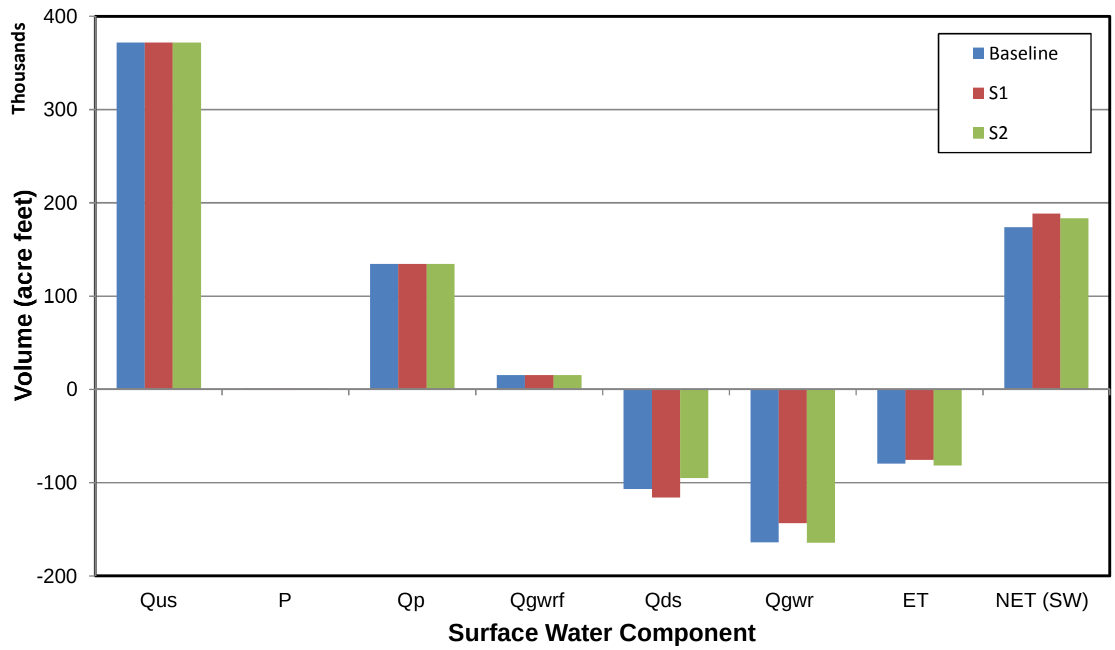

Figure G1-73. Comparison of the Local-Basin-Scale Surface-water Components Under Baseline (Actual) Conditions and under Scenarios S1 and S2

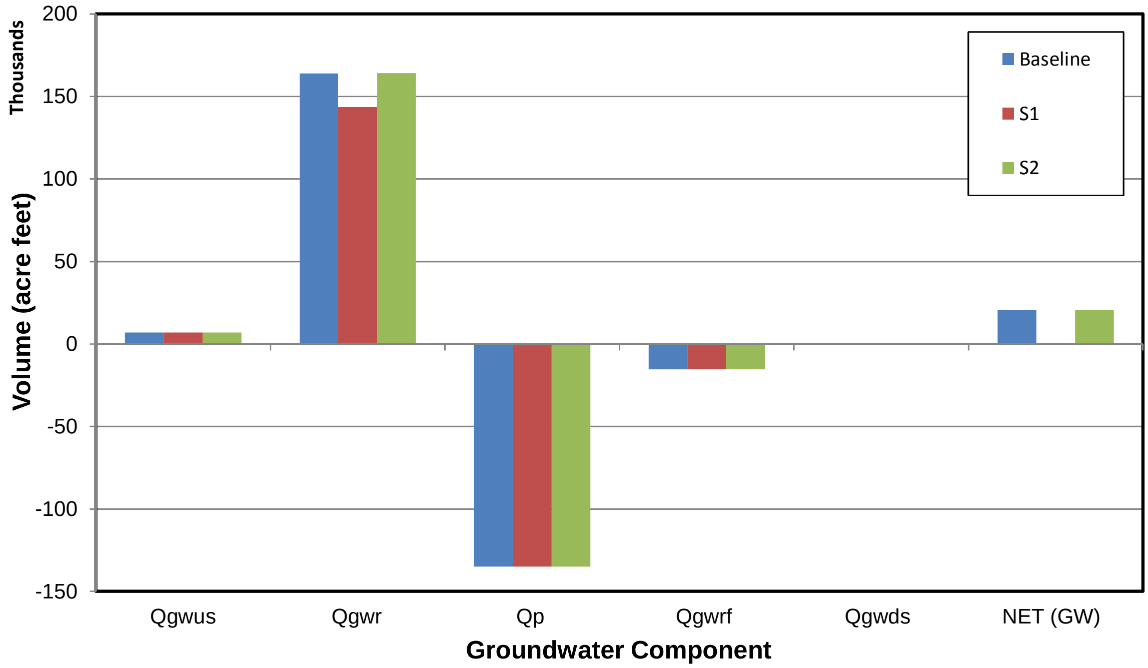

Figure G1-74. Comparison of the Local-Basin-Scale Groundwater Components under Baseline (Actual) Conditions and under Scenarios S1 and S2