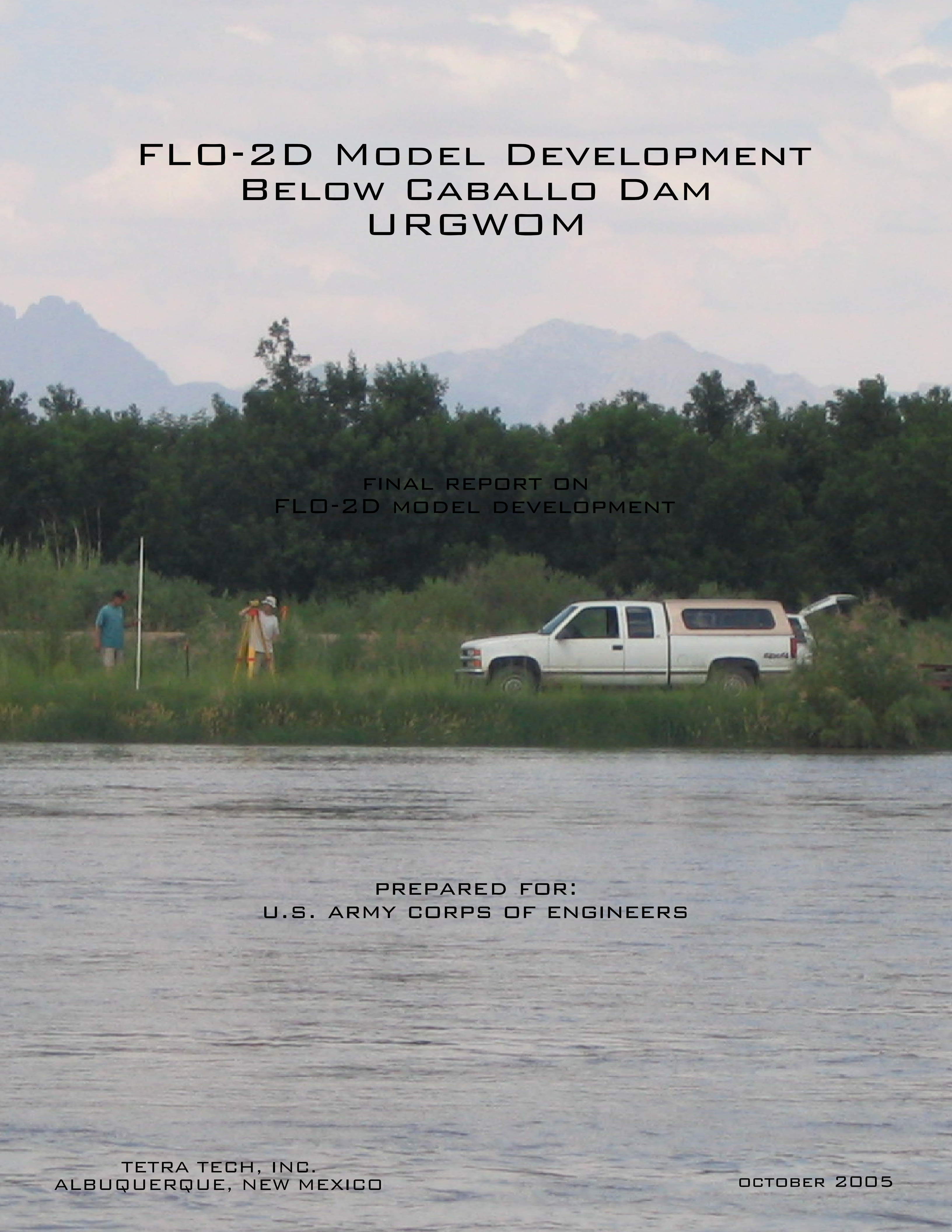

FLO-2D Model Development Below Caballo Dam URGWOM

final report on
FLO-2D model development

prepared for:
U.S. Army Corps of Engineers

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ALBUQUERQUE, NEW MEXICO

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Final Report on
FLO-2D Model Development

Submitted to:
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FLO-2D Model Development Below Caballo Dam

Introduction

This report describes the development of the Rio Grande Canalization Project (RGCP) FLO-2D model from Caballo Dam to American Dam. FLO-2D is a two-dimension channel and floodplain flood routing model for predicting floodwave attenuation, floodplain inundation and spatially variable water surface elevations. This model will be used to support the development of the Upper Rio Grande Water Operations Model (URGWOM) for the reach from Caballo Dam to American Diversion Dam in El Paso, Texas (105 river miles). The model development is a collaborative project between the U.S. Army Corps of Engineers (Corps) and the International Boundary and Water Commission (IBWC).

A flood hydrologic review and sediment supply review were completed as precedents to the development of the RGCP FLO-2D model. The purpose of the hydrologic review was to evaluate various flood events (hydrographs) for simulation in the RGCP reach. The Corps'1996 report, 'Rio Grande Canalization Improvement Project, Hydrologic and Hydraulic Analyses' prepared for IBWC was the primary resource in configuring the flood hydrograph scenarios. The sediment supply review evaluated previous sediment studies and available data in the RGCP reach. The Corps'1996 Rio Grande Canalization Improvement Project, Volume 3, "Sedimentation Analysis from the Rio Grande Tributary Basins" report and the accompanying appendices prepared for IBWC constituted the principle documentation and data for the review. The Corps documentation and analyses were prepared in conjunction with Resource Technology, Inc. (RTI) of Albuquerque and submitted to IBWC in July 1996.

FLO-2D Model Development

This section discusses the compilation of the hydrologic, topographic and channel cross section data bases and the development of the model physical components. The data base represents one of the best data bases ever compiled for a large river reach for the purpose of flood simulation. The topographic resolution and cross section coverage of the river is considered to be excellent.

Data Acquisition and Review – Hydrologic Data

The 1996 Hydrologic and Hydraulic Analyses Report prepared for the IBWC Rio Grande Canalization Improvement Project presented the results of a HEC-1 modeling effort to determine the project design flood peak discharges at selected locations from Caballo Dam to American Dam. The project design flood is represented by the 100-yr, 24-hr design storm centered over the Rio Grande basin below Caballo Dam. The study assessed the Rio Grande channel capacity and potential channel scour to evaluate flood control protection and channel stability in the reach. The total contributing watershed downstream of Caballo Dam (constructed in 1938) is approximately 900 square miles and encompasses numerous tributary arroyos. The contributing basins include steep arroyos, some of which have flood detention storage basins constructed by the Natural Resources Conservation District (NRCS). Several issues related to potential flood inflows to the RGCP were identified.

- Releases from Caballo Reservoir;
- Design storm selection including point rainfall, distribution and depth-area reduction;
- Application of the HEC-1 model including rainfall/runoff method, assumptions and selected parameters;
- Rainfall loss estimates and percent runoff;
- HEC-1 model results.

Each of these issues was discussed in the previously completed Hydrology Technical Review report.

Caballo Reservoir Flood Release. A worst case scenario for RGCP flooding assumed the combined 100-yr, 24-hr general storm below Caballo Dam with occurrence of 100-yr snowmelt conditions in the upper Rio Grande watershed resulting in a 5,000 cfs release from Caballo Dam. Without an increase in water supply, a change in water operations or replacement of the Caballo Dam outlet facilities, it is unlikely that a 5,000 cfs would be released frequently enough to occur during the 100-yr 24-hr flood event. The assumption of a constant outlet release of 5,000 cfs during the 100-yr general storm constitutes 20 to 25% of the downstream 100-yr flood peak discharge. The Corps was directed by IBWC to assume a conservative constant release of 5,000 cfs from Caballo Dam. Discharges over 3,000 cfs have occurred about 11% of the time in post-Caballo gage record. Accordingly, the combined probability of a 5,000 cfs release concurrent with a 100-year, 24-hr storm would exceed a 0.01 chance of occurrence. The assumption of a 5,000 cfs release during the 100-yr 24-hour general storm is no longer valid considering the reservoir operation since the1997 Court Order No. CIV-90-95 HB/WWD.

Appendix F of the IBWC Draft EIS (DEIS, 2004), states that a controlled release of 5,000 cfs (maximum possible outlet discharge) from Caballo Dam can only occur "…when the reservoir reaches maximum water surface elevation (p. F-1, DEIS). The maximum water surface elevation is 4,182, approximately 10 ft above the top of the active conservation pool elevation and "…above typical reservoir operation conditions (p. F-1, DEIS). The Caballo Reservoir water surface elevation has reached 4,182 only once (1942) since dam construction in 1938 (Figure 1). The reservoir has been operated under Court Order No. CIV-90-95 HB/WWD since 1997 and the reservoir level during the summer irrigation has been controlled at about elevation 4,145 (plus or minus about 3 ft), yielding between 50,000 af and 80,000 af of storage. The spillway crest elevation is 4,161 and the spillway is equipped with radial gates that when closed extend the conservation pool up to an elevation of 4,172.44. When the flood pool reaches an elevation of 4,182, the outlet works can discharge 5,000 cfs. The DEIS (Parsons DEIS, p. 19, Appendix G) indicates that "…at present the feasibility of any release is questionable as…Caballo Dam operation regime…would not support peak discharges near the 5,000 cfs theoretical maximum value." The fact that 5,000 cfs dam release or higher has been achieved so infrequently (four times in the past 65 years: 1942, 1987, 1992, 1995 at the USGS gage below Caballo Dam), underscores that it is highly unlikely that 5,000 cfs would be released during a general storm in the valley. In other words, the combined probability of the 100-yr, 24-hr general storm and a coincident 5,000 cfs release from Caballo Dam is less than 1 percent in any given year.

Future opportunities for a 5,000 cfs release appear to be limited. The DEIS (p. F-1, Appendix F) states, "...(w)hile the potential extent of overbank flows was analyzed based on a maximum theoretical value - 5,000 cfs discharge - it is important to emphasize that full discharge conditions would be reached only after several years of planning, gradual implementations and regular monitoring." This statement is echoed on page 4-6 of the DEIS where it is suggested that "…the maximum Caballo Dam discharge value would be reached at the end of a 20-year implementation period by gradually increasing releases of small magnitude." Without an increase in water supply, a change in water operations or replacement of the Caballo Dam outlet works, it is improbable that a 5,000 cfs would be released during the 100-yr flood event.

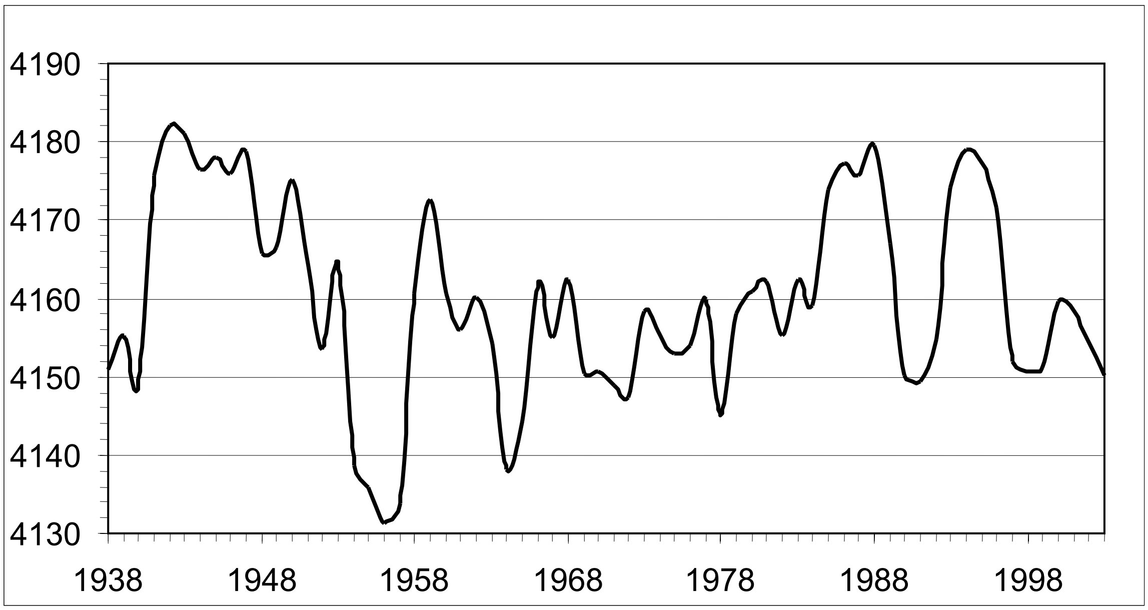

Figure 1. Maximum Annual Water Surface Elevation in Caballo Reservoir 1938-2003 (Bureau of Reclamation, March, 2004, El Paso Office, personal communication)

To assess the appropriate 1% flood, the design storm flood hydrology should be considered to occur with the average release from Caballo Dam. Caballo Dam is typically operated in the summer months at flows less than bankfull discharge. Average monthly flows range from 2,350 cfs in the upper reach to 1,600 cfs in the lower reach. The RGCP channel has a conveyance capacity that ranges from 2,500 cfs to 3,000 cfs in the Upper Rincon Valley to less than 2,000 cfs in the Lower Mesilla and El Paso Valleys. Release scenarios that exceed this amount will be subject to some overbank storage and floodwave attenuation. Following discussions between IBWC and the Corps of Engineers, it was decided that the design flood event would consist of the design storm flood inflow with a constant release of 2,350 cfs from Caballo Dam.

Design Storm Selection. The largest part of the drainage basin contributing to the Rio Grande Canalization Project is in the upper half of the reach extending from Caballo Dam to Leasburg Dam (792 mi^2 out of 894 mi^2 in the basin). This upper basin will generate the flood inflow hydrographs for the lower river reach. The issues to be considered regarding the selection of the design storm are:

- What type of storm should constitute the project design storm?
- Was the 100-yr, 24-hour total rainfall a reasonable estimate for this area?
- Would the river discharge be greater in terms of both peak discharge and volume downstream of any arroyo confluence if a local convective thunderstorm occurred in arroyo watershed instead of the general 24-hour storm over the entire RGCP basin?

In the Hydrology Technical Review it was confirmed that Corps selection of the total rainfall amount for different duration storms was appropriate. The records from two local rain gages within the basin were analyzed for duration and frequency (Table 1). It was also confirmed that the 24-hr general storm over the entire basin will produce the highest peak discharge and largest storm volume from one of the major arroyos in the Rio Grande Canalization Reach (Table 2). The supporting hydrologic analysis and documentation is presented in Appendix A. The runoff simulation was based on the Corps' Synder's unit hydrograph method and uniform loss rates. Other rainfall loss methods such as Green-Ampt infiltration could generate different results. It was concluded that the Corps' selection of the 100-yr 24-hr general storm (3.8 inch point rainfall) as the design storm for tributary arroyo flooding to the Rio Grande Canalization Project was appropriate.

Table 1. 100-year Storm Total Rainfall

Storm Duration	Corps' Point Rainfall (in)	Jornada* Gage 91 yrs	Hillsboro* Gage ('46-'04)
2-hr	2.53	-	2.19
6-hr	3.00	-	2.49
24-hr	3.80	2.82	3.53

* Extreme Value Distribution best fit using a King's Table in the FreqPlot Program (duRoulhac, 1990)

Table 2. Trujillo Arroyo HEC-1 100-yr Storm Rainfall Runoff Simulation Results[1]

Storm Duration	Total Point Rainfall (in)	Depth Area Reduction	Applied Rainfall (in)	Loss (in)	Excess Rainfall (in)	Runoff Volume (af)	Peak Q (cfs)	Time to Peak (hrs)
2-hr	2.53	0.68	1.72	1.07	0.65 (38%)	1,840	3,820	4.83
6-hr	3.00	0.71	2.13	1.33	0.80 (38%)	2,240	4,465	7.75
24-hr	3.80	0.77	2.93	1.92	1.01 (35%)	2,840	5,815	9.67

[1]Based on the Corps Original HEC-1 model using Synder's Unit hydrograph method

HEC-1 Hydrologic Model Application for Inflow Flood Hydrographs. The Snyder Unit hydrograph method was applied in the Corps HEC-1 rainfall/runoff simulations to generate the contributing arroyo flood hydrographs to the Rio Grande. The Snyder unit hydrograph method relates the computed hydrograph characteristics (peak discharge, basin lag time, hydrograph base time, and duration at specified discharges) to the watershed parameters. The primary assumptions associated with applying this method is that the runoff results are not storm sensitive and that the rainfall runoff can be combined linearly. Snyder's unit hydrograph method was based on data from the eastern United States where the watersheds tend to be larger, the basin slopes milder and the time to peak longer. The concern is that the Snyder Unit Hydrograph Method may tend to underpredict the peak discharge because the longer routing times that may not be representative of steep slope, poorly vegetated western semi-arid watersheds with imperious areas. In steep arroyo watersheds, the hydrographs tend to have a fast rising, frontal wave peak discharge. A sensitivity analysis of the peak discharge to variation in the Snyder unit hydrograph parameters was performed. Recommended adjustment of the two primary coefficients results in a 37% increase in the peak discharge of 8,000 cfs for a test arroyo tributary to the Rio Grande. See Appendix A for further analyses and discussion.

Rainfall Loss Estimate and Excess Runoff. The HEC-1 model predicted excess runoff for the subbasin areas ranged from 34 to 39 percent of the total rainfall after depth area reduction. This was based on an initial loss (abstraction) of 0.90 inches and 0.20 inches per hour uniform loss rate. The uniform loss rate of 0.20 inches was noted by the Corps to be widely used in hydrologic studies in the southern New Mexico area. The 0.90 inch initial loss was calibrated to two regional equations; one developed by the USGS (1986) and the other developed by the Albuquerque District (1990). If it is assumed that the USGS equation is underpredicting the 100-yr peaks, then adjustments should be made to the initial loss. When the initial loss rate was reduced from 0.90 to 0.70 inches while maintaining the uniform loss of 0.20 inches, the results from Trujillo Arroyo for the 100-year flood showed an 18% increase in peak discharge. Reducing the initial loss did not appreciably affect the time to peak. Following discussions with the Corps and IBWC, it was decided to apply the lower initial loss rate to determine the return period flood inflows. See Appendix A for further analyses and discussion of the effect of varying the rainfall initial and uniform loss rates on tributary flood peak discharge and volumes.

HEC-1 Model Results. The Rio Grande Canalization Project HEC-1 data files were modified with the following changes:

- Initial Loss was reduced from 0.9 inches to 0.70 inches. *This will increase the subbasin runoff.*
- C_p was increased from 0.61 to 0.70. *This will increase the subbasin runoff.*
- C_t coefficient was decreased from 0.60 to 0.5. *This will decrease the time of concentration.*
- Channel n-values (for flood routing)were increased from 0.02 to 0.032. *This will increase the travel time of the floodwave and increase the overbank flow. This may also steepen the rising limb of the hydrograph.*
- Floodplain n-values were increased from 0.030 or 0.035 to 0.085. *This will increase the travel time of the floodwave and may redistribute some of the flood volume affecting local peak discharge.*
- Wasteway n-values were increased from 0.015 to 0.025. *This should have negligible effects on the results.*

The Corps original 100-year peak discharges for the Rio Grande between Caballo Dam and American Diversion Dam are listed in Table 3 along with the revised values based on the aforementioned rainfall loss parameter changes. The changes to the hydrologic parameters were made following discussions with the Corps and IBWC.

Table 3. 100-year 24-hr Peak Discharge Caballo Dam to American Diversion Dam		
Rio Grande Location Discharge Downstream of the Following Sites	Corps Original Q_p (cfs)	Recomputed Q_p (cfs)
Caballo Dam release	5,000	5,000
Trujillo Canyon	9,100	12,700
Montoya Arroyo	11,300	15,900
Green Canyon	11,700	15,800
Tierra Blanca Arroyo	15,600	23,200
Sibley Arroyo	17,600	24,300
Berrenda Arroyo	18,700	25,200
Arroyo Cuervo	18,900	24,300
Placitas Arroyo	19,100	21,300
Angostura Arroyo	17,800	19,500
Rincon Arroyo	22,400	24,100
Reed Arroyo	22,500	24,300
Broad Canyon	22,400	20,800
Faulkner Canyon	22,200	19,300
Leasburg Diversion Dam	22,200	19,200
Shalem Bridge	20,900	18,100
Dona Ana Dam	21,000	18,200
Picacho Dam	21,300	18,400
Mesilla Diversion Dam	20,000	17,400
Vinton, Texas	16,500	14,600
Nuway, Texas	16,300	14,500
Canutillo, Texas	15,900	14,200
Borderland, Texas	15,000	13,400
Courchesne Bridge	14,400	12,800
American Diversion Dam	14,000	12,500

Table 3 indicates that the HEC-1 data modifications result in a Rio Grande 100-year 24 hour storm peak discharge that occurs further upstream in response to the runoff from the larger watersheds in the upper third of the drainage. Floodwave attenuation was more significant in the downstream reaches. These data revisions produced an increase in the peak discharge with reasonable variation of the HEC-1 parameters.

Data Acquisition – Sediment Supply Analysis and Review

The scope of work for the application of the FLO-2D model to the Rio Grande Canalization Project included a task to review existing sediment studies and recommend sediment loading for the project design event. The goal of the sediment supply review was to evaluate the available data regarding tributary sediment loading completed by the Corps for the RGCP reach. A review of the Corps'1996 Rio Grande Canalization Improvement Project, Volume 3, "Sedimentation Analysis from the Rio Grande Tributary Basins" and the accompanying appendices prepared for IBWC constituted the documentation and data for the review. The Corps documentation and analyses were prepared in conjunction with Resource Technology, Inc. (RTI) of Albuquerque and submitted to IBWC in July 1996.

Since upstream Caballo and Elephant Butte Reservoirs have curtailed the sediment supply from the Middle Rio Grande, the sediment load for the design flood events are limited to that contributed by the arroyo tributaries in the RGCP watershed. One of the important issues is whether the RGCP channel is in approximate sediment transport equilibrium in terms of supporting the existing channel morphology. Channel maintenance activities have initiated channel incision and head cutting for several miles in at least two reaches. This is evidence that there is a sediment deficit in the system and the current tributary sediment load may not sustain the existing channel morphology in response to future channel maintenance activities.

The methodology for predicting the sediment yield to the RGCP presented in the 1996 Corps and RTI report is very good. The selection of the equations, the application of the equations based on size fraction, and the computation of the mean annual sediment load constituted an excellent approach to evaluating the total sediment supply to the river. The report indicates that the variability in the total sediment load results for some basins may be attributed to a combination of factors including hydrology, watershed properties and hydraulics. The final product of this work was a regression equation relating tributary sediment supply Q_s as a function of tributary basin area for each return period storm. The mean annual sediment supply Q_{sm} in acre-feet was then computed for each tributary using the equation:

$$Q_{sm} = 0.015\, Q_{s\,100yr} + 0.015\, Q_{s\,50yr} + 0.04\, Q_{s\,25yr} + 0.08\, Q_{s\,10yr} + 0.2\, Q_{s\,5yr} + 0.4\, Q_{s\,2yr}$$

The results are listed in Table 4 in the column labeled ‘Original Mean Annual Yield’. By dividing the mean annual yield by the drainage area, a sediment yield by unit area is computed. This value is listed in column 5 of Table 4. The average sediment yield per unit area for all the basins is 3.48 af/mi^2/yr as shown at the bottom of the table.

This average sediment yield can be compared with accumulated sediment storage of NRCS detention basins within the RGCP area. The average annual sediment yield by survey or by PSAIC sediment yield estimate of NRCS reservoirs is displayed in Table 5. The average sediment yield ranges between 0.5 and 0.6 af/mi^2/yr. This information is a mixture of actual resurvey data provided by the Corps and RTI report and recent updated sediment yield estimates provided by the NRCS. Some data that was published as resurvey data in the Corps report may have been based on the best available mapping as in the case of Caballo #2 which was later corrected in the more recent NRCS 2005 estimates. It is appropriate to indicate that although there is some variation in the sediment yield estimates, most of the sediment yield is less than 1 af/mi^2/yr. The Corps and RTI average sediment yield per unit area is on the order of 6 to 7 times larger than that estimated from the NRCS detention basin analyses. Some of this difference can be attributed to the limited large storm sediment contribution in period following construction of the detention basin. On the other hand, there is sufficient data to justify stating the average annual sediment yield should be less than 1 af/mi^2/yr. It appears that the sediment yield presented in the Corps and RTI report is over estimated. The reasons for the over predicted sediment yield is discussed following the tables.

Table 4. Estimated Adjusted Sediment Yield

Distance from Caballo Dam	Watershed[1] Name	Basin Drainage Area	Original Mean Annual Yield	Original Yield per Unit Area	Adjustment Factor F_a[2]	Adjusted Mean Annual Yield	Adjusted Yield per Unit Area
mi		mi^2	af/yr	$af/mi^2/yr$		af	$af/mi^2/yr$
2.8	**Misc.1**	16.70	10.64	0.64	0.504	5.34	0.320
4.6	Trujillo Canyon	52.90	18.88	0.36	0.775	14.70	0.278
5.9	**Montoya Arroyo**	23.00	12.22	0.53	0.568	6.93	0.301
6.9	Misc. 2	2.20	5.47	2.49	0.236	1.29	0.587
7.6	**Green Canyon**	35.60	15.13	0.43	0.668	10.13	0.284
7.6	**Tierra Blanca Creek**	68.20	22.09	0.32	0.852	18.95	0.278
9.5	**Sibley Arroyo**	27.20	13.22	0.49	0.604	7.99	0.294
11.0	Berrenda Creek	87.40	26.02	0.30	0.935	24.54	0.281
11.0	**Jaralosa Arroyo**	6.80	7.69	1.13	0.360	2.76	0.405
13.4	Misc. 3	9.50	8.6	0.91	0.408	3.49	0.368
14.4	McLeod Arroyo	14.20	9.98	0.70	0.474	4.71	0.332
15.0	**Arroyo Cuervo**	126.20	33.82	0.27	1.073	36.69	0.291
17.1	Misc. 4A	3.00	5.99	2.00	0.265	1.58	0.528
19.7	**Reed-Thurman**	3.25	6.13	1.89	0.273	1.67	0.514
21.5	Misc. 4	14.50	10.06	0.69	0.478	4.79	0.330
22.4	**Placitas Arroyo**	34.60	14.91	0.43	0.661	9.87	0.285
23.0	Spring Canyon	7.40	7.91	1.07	0.371	2.92	0.395
25.0	Misc. 5	11.80	9.3	0.79	0.442	4.10	0.347
27.3	**Ralph Arroyo**	2.45	5.64	2.30	0.246	1.39	0.566
27.0	Angostura Arroyo	8.90	8.41	0.94	0.398	3.33	0.375
28.4	**Rincon Arroyo**	124.70	33.52	0.27	1.068	36.20	0.290
28.9	Reed Arroyo	9.60	8.64	0.90	0.409	3.52	0.367
31.0	**Misc. 6**	43.50	16.88	0.39	0.720	12.19	0.280
37.0	**Lytten Canyon**	0.96	4.24	4.42	0.173	0.74	0.771
39.5	**Buckle Bar Canyon**	2.12	5.41	2.55	0.233	1.26	0.594
39.6	Broad Canyon	68.00	22.05	0.32	0.851	18.89	0.278
41.7	Misc.7	10.38	8.88	0.86	0.422	3.73	0.359
42.5	Foster Canyon	11.00	9.06	0.82	0.431	3.89	0.354
44.0	**Faulkner Canyon**	25.00	12.7	0.51	0.586	7.43	0.297
46.4	Subarea 15	3.40	6.22	1.83	0.278	1.72	0.507
49.0	Subarea 16	3.80	6.43	1.69	0.290	1.86	0.488
51.2	Subarea 17	4.92	6.95	1.41	0.319	2.21	0.449
52.7	Subarea 18	2.80	5.87	2.10	0.258	1.51	0.541
53.5	Subarea 19	2.60	5.74	2.21	0.251	1.44	0.554
55.1	Subarea 20	3.00	5.99	2.00	0.265	1.58	0.528
56.5	**Dona Ana Arroyo**	6.94	7.74	1.12	0.363	2.80	0.403
56.5	**Dona Ana N. Arroyo**	2.16	5.44	2.52	0.234	1.28	0.590
57.0	Apache Canyon	7.80	8.05	1.03	0.379	3.03	0.389
57.8	Box Canyon	8.70	8.35	0.96	0.395	3.28	0.377
65.4	**Subarea 23**	0.87	4.11	4.72	0.167	0.69	0.795
66.3	**Subarea 24**	4.20	6.62	1.58	0.301	1.98	0.472
90.3	Subarea 101	2.90	5.93	2.04	0.262	1.55	0.534
90.6	Subarea 102	6.53	7.59	1.16	0.354	2.68	0.410
92.0	Subarea 103	5.35	7.13	1.33	0.329	2.34	0.437
93.3	Subarea 104	3.54	6.29	1.78	0.282	1.77	0.500
93.7	Subarea 105	0.98	4.27	4.36	0.174	0.75	0.766
95.3	Subarea 106A	1.95	5.28	2.71	0.226	1.19	0.611
95.3	**Subarea 106B**	7.40	7.91	1.07	0.371	2.92	0.395
95.5	Subarea 106C	8.15	8.16	1.00	0.385	3.13	0.384
100.4	Subarea 207	1.50	4.88	3.25	0.205	1.00	0.667
102.4	Subarea 205	0.43	3.18	7.40	0.128	0.42	0.969
102.7	Subarea 206	0.60	3.61	6.02	0.145	0.53	0.887
102.9	Subarea 204	0.42	3.15	7.50	0.127	0.41	0.974
103.5	Subarea 203	0.34	2.88	8.47	0.117	0.35	1.023
103.7	Subarea 202	1.78	5.14	2.89	0.218	1.12	0.630
105.8	Subarea 301	2.58	5.73	2.22	0.250	1.43	0.556
106.1	Subarea 302	2.20	5.47	2.49	0.236	1.29	0.587
			average	3.48		average	0.480

[1]Original study watershed are listed in bold type
[2]Adjustment Factor $F_a = 5.69\ A_B^{(-0.3739)}$ where A_b = basin area

Table 5. NRCS Reservoir Sedimentation			
		Sediment Yield (af/mi^2/yr)	
Watershed	Resurvey (yrs)	Initial Resurvey[1]	NRCS 2005[2]
Caballo #1	7.80	0.64	0.64
Caballo #2	13.10	1.31	0.20
Caballo #3	13.50	0.66	0.66
ABM #1	7.20	0.87	0.87
ABM #2	7.20	0.33	0.33
ABM #3	8.70	0.59	0.59
ABM #4	8.00	0.70	0.70
Fillmore Arroyo #1	10.20	0.34	0.34
Fillmore Arroyo #2	10.20	0.35	0.35
Fillmore Arroyo #3	10.20	0.27	0.27
Tortugas #1	5.66	0.69	0.69
Tortugas #2	9.66	0.38	0.38
Doña Ana #1	14.80	0.66	0.77
Doña Ana #2	13.00	0.15	0.15
Hatch Valley #5 Upstream	9.10	1.55	0.91
Hatch Valley #5 Downstream			0.30
Hatch Valley #2			0.31
Hatch Valley #3			0.51
Hatch Valley #6			0.19
	Average	0.63	0.48
	Std. Dev.	0.38	0.24

[1]Extracted from Table 5-11, Corps Sedimentation Analysis from the Rio Grande Tributary Basins
[2]NRCS sediment yield estimates based on PSAIC and resurveys (provide by Robin White, 2005)

A detailed analysis of the potential sediment loading as estimated in the Corps and RTI report is presented in Appendix B. This analysis indicated that the high tributary sediment loads computed in the report were the result of following factors:

- Overestimate of the tributary flood velocities resulting from low n-values and a supercritical flow assumption.
- Inappropriate computed sediment loads by combining the results of MPM-Woo equation and Colby adjustment procedure for the effects of fine sediment.
- Inappropriate selection of the critical shear stress parameter for both incipient motion and for the MPM bed load equation.
- Possible overestimates of the wash load associated with parameter selection in the MUSLE equation.
- Over estimated total sediment load using the Colby adjustment factors because the MUSLE wash load is overestimated.

Most of the overestimated total load can be attributed to the application of the Colby adjustment factor based on wash load concentration and bed material median diameter. For the Subarea 23 and 24 Arroyos, the Colby adjustment increases the total bed material load by factor of almost 10. It should be noted that the Colby method is based on limited data and a number of uncertainties in the graphical representation of the factors (Simons and Senturk, 1976). Yang (1996) concluded that "(b)ecause of the range of data used in the determination of the rating curves …Colby's approach should not be applied to rivers with median sediment diameter greater than 0.6 mm and depth greater than 3 m." Seventeen of the twenty tributary study basins have a D_{50} size greater than 0.6 mm. Further discussion on the overestimate of the tributary sediment yield is provided in Appendix B along with a list of references.

Several recommendations were made in the Sediment Analysis Technical Report to improve estimated tributary sediment yield for future analyses. These included:

1. Update the NRCS reservoir survey data in the RGCP basin. This was completed and is presented as the column NRCS 2005 in Table 5.
2. The sediment total load computations should be calibrated to the NRCS reservoir survey mean annual load. This was completed and the results are discussed below.
3. A further review of the potential increased runoff from short duration, high intensity storms. This was completed and was discussed in the previous section with the review of the Hillsboro raingage data base.
4. The flood hydraulics should be revised with more representative n-values and a subcritical flow assumption. This task became moot when the recommendation 2 was completed.

Initially a re-analysis of the tributary sediment loading was considered by undertaking a detailed re-evaluation of the wash load and bed material load computations. The end product of this task was to calibrate the sediment yield per unit area to the NRCS 2005 detention basin storage sediment yield estimates. Many of the parameters and assumptions in the analysis would have to be changed several times to complete the calibration. A simpler approach to achieving the same result is to adjust the sediment regression equation as a function of basin area. This equation for total sediment load Q_T was of the form:

$$Q_T = A_1 + A_2\, A_b + A_3 \,\mathrm{Log}\, (A_b)$$

where: A_1, A_2, and A_3 are regression coefficients and A_b is the basin area.

In this equation, individual regression coefficients were derived for each return period. By plotting the sediment yield per unit area as function of the basin area, it was observed that a decreasing power function could be applied to adjust this equation as function of the basin area. The largest sediment yield per unit area resulted from the basins with the smallest drainage area. The derived adjustment equation was:

$$F_a = 5.69 * A_b^{\,(-0.3739)}$$

where: F_a is the adjustment factor in Table 4 that is used to multiply the total sediment load in the above equation.

Table 4 lists the results from the application of the adjustment factor in re-computing the mean annual sediment yield and the sediment yield period unit area (last column). The result is that the average sediment yield (0.48 af/mi^2/yr at the bottom of Table 4) for all the tributary basins matches the average sediment yield per unit area defined by the NRCS analysis (last column of Table 5). This is more realistic of the potential future loading of the basins that are contributing sediment to the RGCP reach.

It is concluded that although the approach in the Corps report to calculate the total sediment supply to the RGCP was excellent, the selection of parameters, application of the sediment transport equations and the supercritical flow assumptions resulted in an over prediction of the mean annual sediment yield. In order to determine an appropriate sediment supply to the RGCP for future modeling efforts, the estimated sediment load as a function of the basin area was reduced using an adjustment factor equation.

Data Acquisition – Diversions and Return Flows

To replicate historic flow events and calibrate the RGCP FLO-2D model, it was necessary to compile diversion flows and return flows for selected periods of record. The channel cross section data was collected in June and July 2004. To calibrate the water surface elevations collected with the cross section data during this period, the flow diverted from the river at the four diversion dams and the associated irrigation return flows are required. July 1995 was a period of relatively high flow releases from Caballo Dam and the diversions and return flow were necessary for these periods. Elephant Butte Irrigation District (EBID) provided the diversion and return flow data for the four diversion dam canals and various wasteways. Only a few selected wasteways for the return flow from the diversion dams are monitored by EBID and some of these had discharges of less than 10 cfs. Those returns that were less than 10 cfs for the calibration period were excluded from the model. The Caballo Dam release, the diversion flows from the four diversion dams and the selected return flows were compiled in the INFLOW.DAT file.

Data Acquisition and Preparation – Topographic Data

In early 2005 a comprehensive digital mapping project was completed for a significant portion of Dona Ana County, New Mexico including the Rio Grande corridor which was fully mapped with color digital orthophotos, digital terrain data and contour graphics files. The project involved a number of stakeholders and was administered by the Dona Ana County Flood Commission (DACFC). Most of the funding was provided by Dona Ana County. Aerial photography and terrain data using a Light Detection and Ranging (LIDAR) sensor was acquired using a fixed-wing aircraft. The Albuquerque based mapping/consulting firm Bohannan Huston, Inc. completed the project in approximately one year for DACFC. The FLO-2D flood simulation project benefited from the enhanced resolution in the new DACFC mapping.

Over 1,200 square miles were topographically mapped by the project. The mapping products were parsed into files that correspond with the Public Land Survey System (PLSS). A digital ortho file, a terrain data file, and a contour file were prepared for each section of land that was mapped. The New Mexico State Plane Coordinate Grid System NAD 83 Central zone was utilized for map geo-reference control. Coordinate files were written in U.S. survey feet and elevation data was referenced to the North American Vertical Datum 1988 (NAVD 88). File names were organized corresponding to township, range, and section. Figure 2 shows the extents of the mapping project and an example set of file names are listed in Table 6.

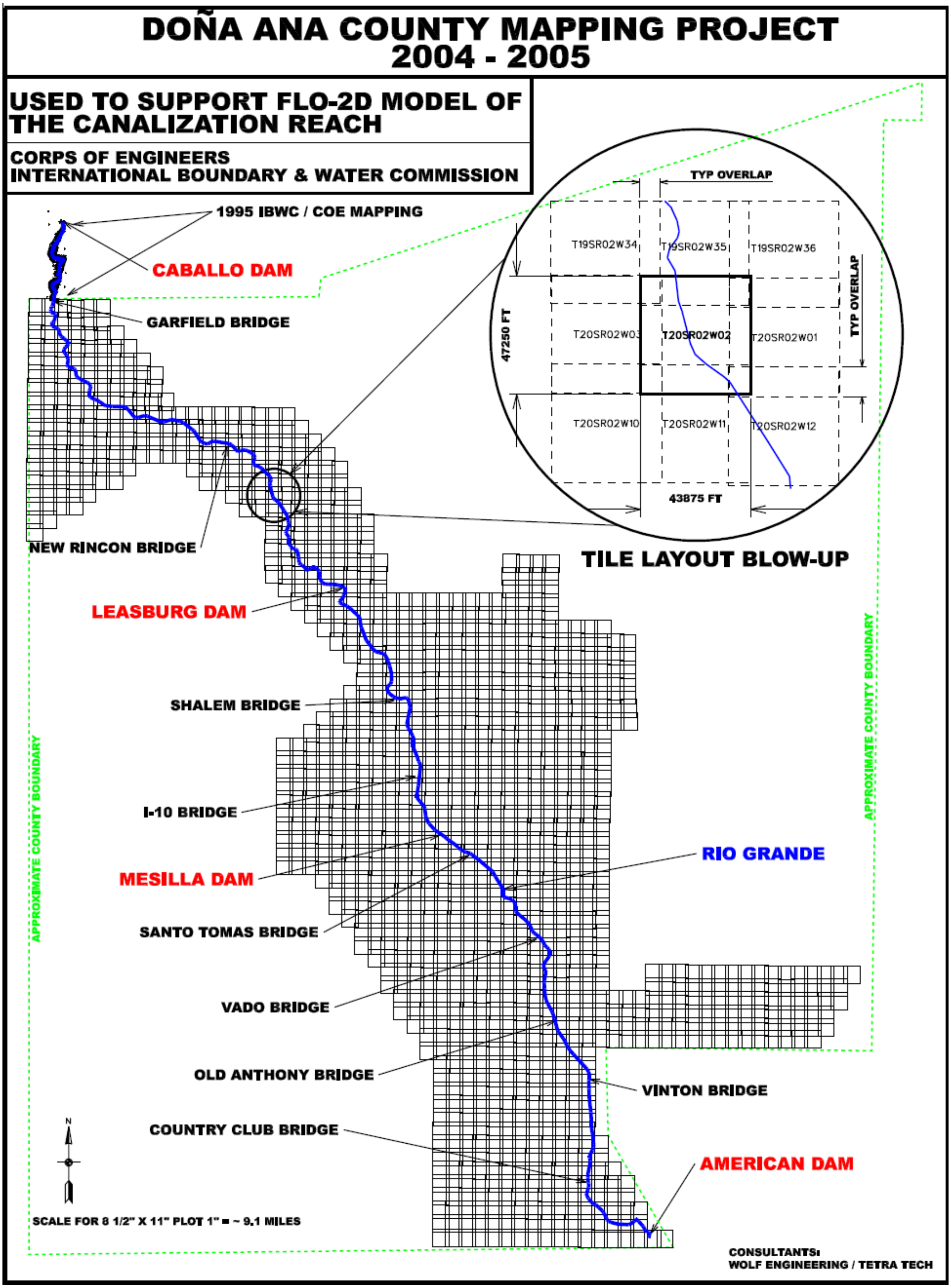

Figure 2. Project Mapping Extent

Table 6. Example Topographic Data File Names and Descriptions	
Typical File name	Description
T18SR04W05pnt.txt	ASCII random point file – northing, easting, elevation – space delimited
T18SR04W05lin.txt	ASCII breakline point file – northing, easting, elevation – space delimited
T18SR04W05pnt.gen	ASCII random point file – northing, easting, elevation – comma delimited
T18SR04W05lin.gen	ASCII breakline point file – northing, easting, elevation – space delimited
T18SR04W05.tif	Natural Color digital orthophoto – uncompressed – 1 foot pixel
T18SR04W05.tfw	World file – works with corresponding tif file to provide reference
T18SR04W05.ecw	Natural Color digital orthophoto - Compressed
T18SR04W05.shp	Contour graphics file – 2 ft CI

The LIDAR data base was used to develop 2 foot contour interval mapping along the Rio Grande corridor. The mass point LIDAR data was edited and filtered to eliminate points that did not reflect "bare earth" ground points. In addition, limited supplemental breakline data was developed and coupled with the LIDAR data. These two types of point files were used to build the digital terrain models (DTM) for the project from which the contour files were generated.

Digital orthophotos were generated from the 1:12000 scale aerial photography. The photography was scanned and rectified at a resolution which produced 1 foot pixels in the final digital image files. These high resolution images were delivered in 120 mega-byte files and show significant detail of the floodplain features. An example 1 foot pixel digital orthophoto is shown in Figure 3.

Figure 3. Example of 1 foot pixel digital orthophoto

The Corps of Engineers and its consultants originally planned to use 1995 digital mapping for the development of the FLO-2D model in the Canalization Project reach. When it became apparent the more comprehensive mapping from DACFC would be available in early 2005, the flood routing project was delayed by several months to be able to utilize the new DTM. The Corps purchased the digital mapping project from DACFC. The entire suite of files required three massive (terabyte +) Lacy hard drives and the Corps retains these drives at their Albuquerque, New Mexico project office.

Tetra Tech acquired a band of the digital mapping tiles from the Corps covering the Rio Grande corridor after the final mapping data was delivered to DACFC. The required files were copied from the Lacy drives to DVDs for use on the FLO-2D flood routing project. The DTM data base files and aerial images were sorted and correlated with the river system by Tetra Tech. A shape file was developed to indicate the image positioning on the FLO-2D grid system. The DTM points were imported to the FLO-2D Grid Developer System (GDS) along with the images in 12 groups. The DTM point data base was edited to reduce the overall number of points that had to be interpolated to assign the grid system elevations. The Rio Grande valley floor was delineated for the potential area of inundation outside the levee system and DTM points outside this area were deleted.

Each of the 12 groups of DTM points were filtered for both high and low DTM elevations and grid element elevations were then interpolated and assigned to the grid system. For the high filter a minimum of 10 DTM points and a difference in the mean elevation of 2 ft was applied to each grid element of 250 ft. All the DTM points within the radius of 250 ft of the grid element center were used to compute a mean grid element elevation with the requirement that there were at least 10 DTM points. If the minimum criteria of 10 DTM points was not met, then the radius is expanded in increments of 250 ft until 10 DTM points were found. The DTM point density was so high however, that it is unlikely that any of the grid element interpolations had to be expanded. When the filter was applied, the mean elevation of all the DTM points within the 250 radius was computed and all those DTM points whose elevation was higher than the mean elevation plus 2 ft were deleted and the grid element mean elevation was recomputed. This filter methodology eliminated those DTM points that might represent trees, buildings or highway on-ramps from being included in the interpolation of the grid element elevation. Similarly a DTM point low filter of 3 ft was applied to eliminate points that might represent the river bed from being incorporated into the interpolation of the floodplain grid elevations.

After the 12 groups of grid elements were interpolated from the DTM points, the groups were combined into one grid system. This was accomplished by starting at the upstream end and appending the subsequent groups of grid elements to the first grid system files (FPLAIN.DAT and CADPTS.DAT). When the grid system for each group was numbered and interpolated, the location of the upper left grid element center of the group was controlled by the lower left row of grid elements of the previous group and the last grid element number in the previous group of grid elements. Once all the files were appended, the final task was to identify the contiguous grid elements along the seam between each group of grid elements. The process consisted of typing in the grid element numbers in the FPLAIN.DAT file that constituted neighbors across the seam of the two groups of grid elements. Accuracy of this manual process was verified by running the FLO-2D CHECKER processor program that checks that each grid element has the correct contiguous grid elements. The FLO-2D grid is shown in Figure 4.

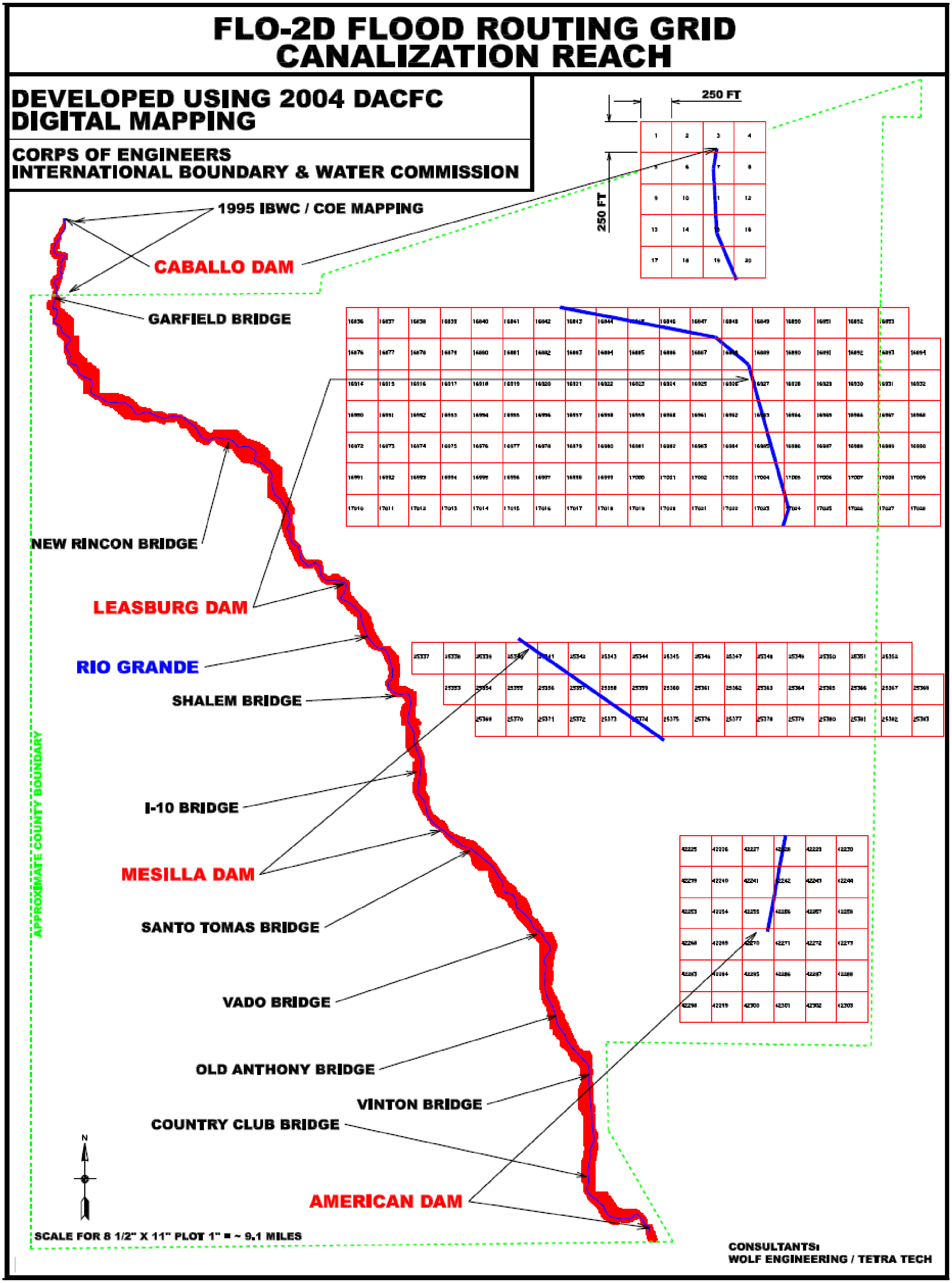

Figure 4. FLO-2D Grid System for the RGCP Reach

The completion of the FPLAIN.DAT and CADPTS.DAT files constituted the topographic representation of the floodplain for the RGCP FLO-2D model. To run a flood simulation without a river channel, a simple artificial hydrograph was prepared in the INFLOW.DAT file, a set of outflow nodes were assigned along the southern boundary of the entire RGCP grid system, and the CONT.DAT and TOLER.DAT files were created for model control and numerical stability. Following the successful simulation of a flood over the Rio Grande floodplain, the channel, infiltration, levees and other physical attribute data files had to be developed to add detail to the RGCP flood model. These details are discussed in the following sections.

Data Acquisition and Preparation – Channel Cross Section Surveys

An important component of a riverine flood routing model is an accurate assessment of available flow area within the active channel. This defines the relationship between the volume of water in the channel and the volume of water on the floodplain, thereby determining the total storage volume for floodwave attenuation. A total of one hundred forty-five cross sections were established and surveyed in July 2004 by Tetra Tech, Inc. A separate report "Field Data Collection Report Cross Section Surveys and Plots" was submitted to the Corps in November 2004 that transmitted the surveyed cross section data base, plots, photos and other data.

A cross section defines the channel geometry such as top width, depth, slope, and bed material. Simulating channel flood routing is facilitated by the correct selection of cross sections in channel transition reaches,. When channel transition reaches are adequately defined, numerical modeling is more stable. For this reason channel cross sections are more numerous in the vicinity of bridges and diversion dams. The cross sections were surveyed by either wading the channel or by using a small survey boat. The surveys were performed using engineers' level and tag line stretched between cross section end points.

The cross sections were numbered and labeled in three groups. The "Below Caballo" (BC) lines begin below the dam and extend to the Leasburg Diversion Dam. The "Leasburg Dam" (LD) lines begin at Leasburg Dam and extend to the Mesilla Diversion Dam. The "Mesilla Dam" (MD) lines begin at Mesilla Dam and extend to the American Diversion Dam in El Paso. There are sixty-six (66) BC lines, twenty-five (25) LD lines, and fifty-four (54) MD lines. Cross section locations for the three sets of lines are shown in Figures 5, 6, and 7. The endpoint coordinates of the lines are referenced to the New Mexico State Plane Coordinate Grid System (NMSPCGS) central zone NAD83 and the elevations are referenced to NAVD88. The units for the coordinate data are U.S. survey feet. In addition to the cross section surveys, six representative riverbed material sediment samples were obtained and analyzed for size distribution. Coordinates for the cross section data were obtained by surveying end point monuments with an engineering grade Real Time Kinematic (RTK) Global Positioning Satellite (GPS) system. A survey control network established in 1994 to support the topographic mapping associated with the Rio Grande Canalization Improvement Project was used for control of the RTK survey.

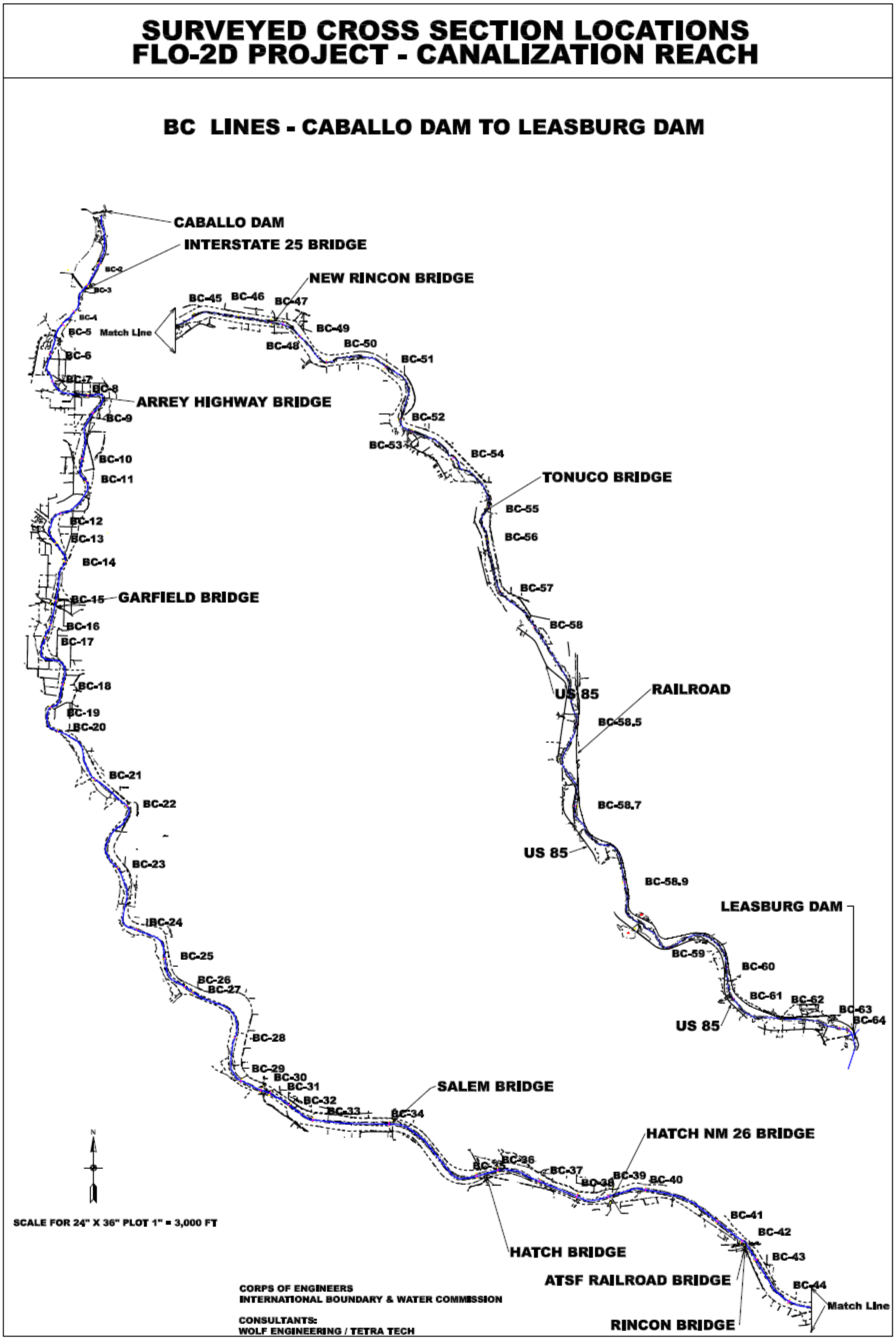

Figure 5. Location of the Survey RGCP Cross Sections - Caballo Dam to Leasburg Dam

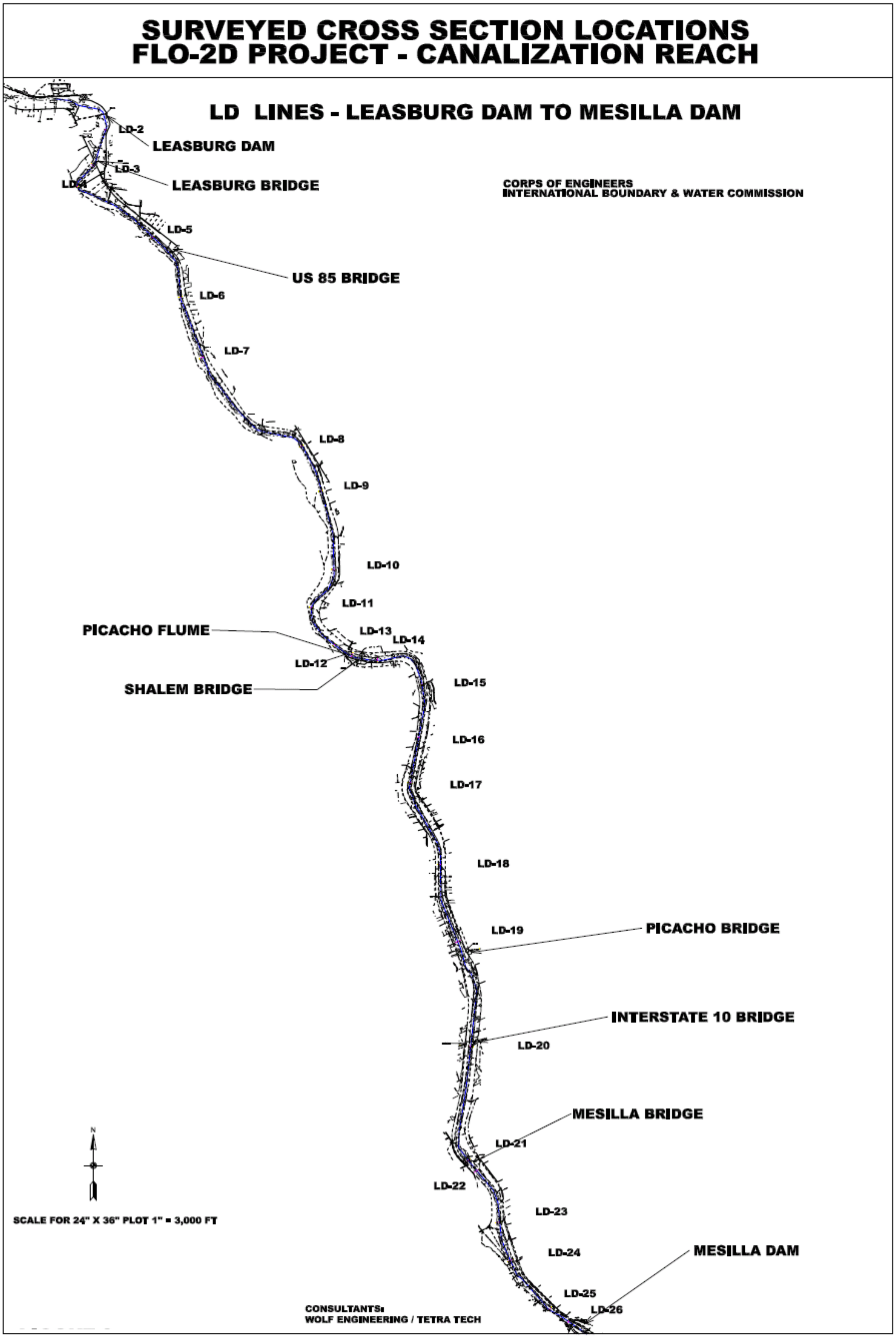

Figure 6. Location of the Survey RGCP Cross Sections - Leasburg Dam to Mesilla Dam

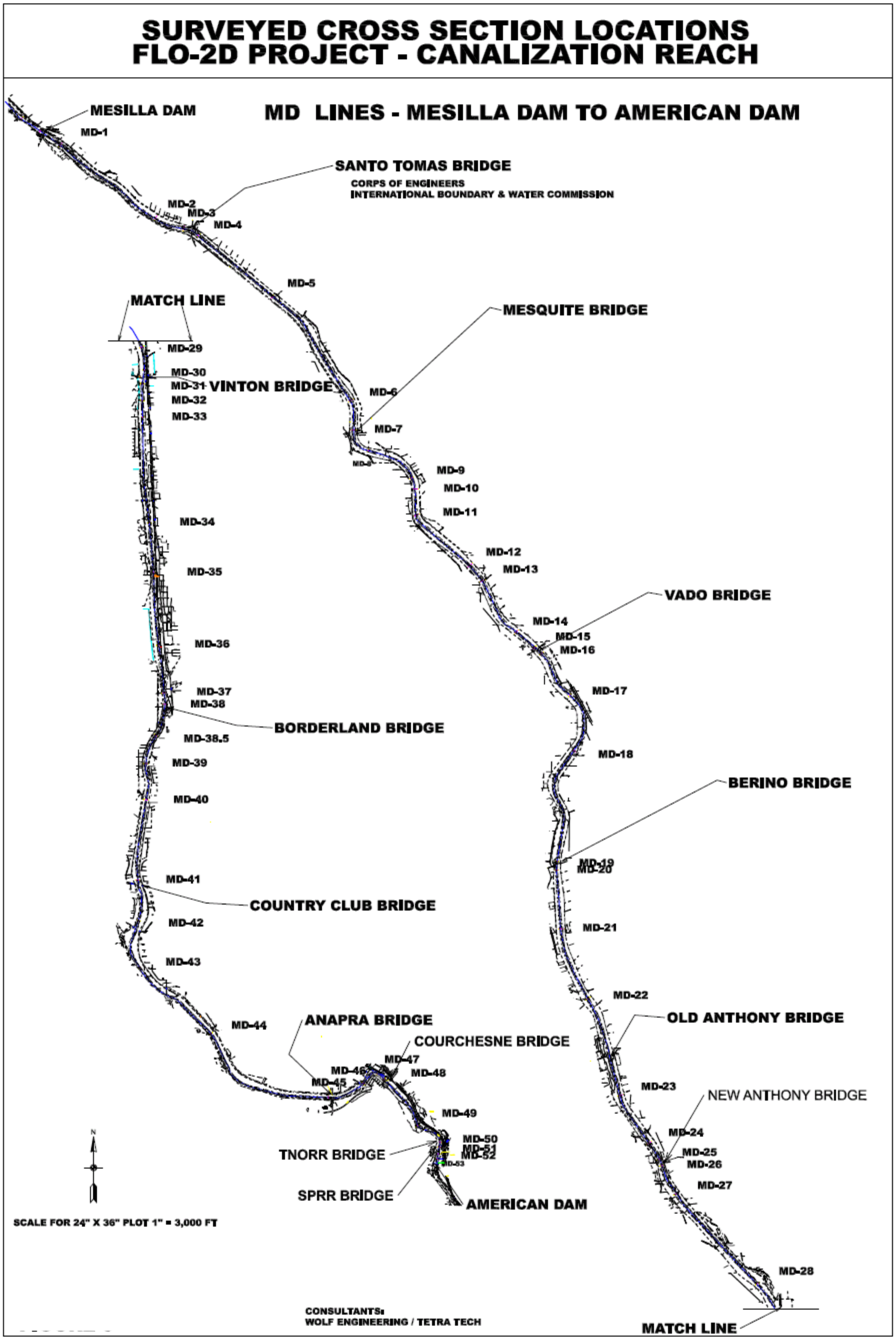

Figure 7. Location of the Survey RGCP Cross Sections - Mesilla Dam to American Dam

The cross section data was reformatted into the XSEC.DAT file for the FLO-2D model. This data file contains cross section name, number, station and elevation. It is referenced by the channel geometry data file (CHAN.DAT). The channel is identified by channel elements assigned in the GDS. With aerial images overlain by the grid system, the GDS highlights those grid elements corresponding to the river channel. Starting at the upstream end of the river system, near Caballo Dam, the channel elements are assigned. The cross sections are then located with respect to the channel elements in the GDS. After providing the appropriate channel parameters with the GDS channel editor, the PROFILES processor program is used to interpolate cross sections and slope between those grid elements assigned the surveyed cross sections. When the process is complete, every channel element has a unique cross section. The river was delineated into 2,046 channel elements approximately 250 ft to 300 ft long. To accurately assess the volume in the river channel, the distance along channel centerline was estimated in ArcGIS® by reach and some of the channel element lengths were adjusted until the total channel length matched the centerline distance.

Data Acquisition and Preparation – Physical Components

To add detail to the RGCP FLO-2D model, several component data files were created including levees, hydraulic structures, infiltration and evaporation. These components are the primary physical features that affect flooding inundation. Each of these components is discussed as it would affect the floodwave movement through the river system.

Levees constitute an important control for limiting overland flooding through most of the RGCP project reach with levees on both sides of the river. Figure 8 shows a schematic of the levees that have been coded into the FLO-2D model for this project. There are approximately 65 miles on the east side and 56 miles on the west side of the river. Generally the levees are set back from the active river channel less than seven hundred and fifty feet. The levees were designed to protect the extensive agricultural lands in the southern New Mexico Rio Grande valley. In addition, the levees provide flood protection for Las Cruces and northern El Paso as well as other smaller communities along the project reach.

Each levee element in the FLO-2D model has a unique crest elevation for one or more of the potential eight flow directions. Microstation/InRoads (CAD environment program) was used to acquire the data needed for the levees. A horizontal alignment representing the approximate centerline of each levee was digitized using two foot CADD contour files. Figure 9 shows this step in the process of assigning the levees. Each alignment is overlaid on the FLO-2D grid and the intersecting grid elements are selected. Establishing crest elevations for the selected levee elements was accomplished using the alignments and digital terrain model data available for the project reach. Tools within the InRoads program allow for the development and display of elevation profiles along horizontal alignments. For each levee alignment a unique profile was created. An elevation was obtained along each station of the profile that corresponded to the to the selected levee grid element. This data was written to Excel spreadsheets. Figures 10 and 11 illustrate the process of creating the levees. Each levee element was reviewed in the GDS program, adjusted if it conflicted with a channel element and assigned directions for the levee flow blockage. Levee discontinuities created by wasteways, tributary arroyos and roadway/railroad embankments are also coded in the model.

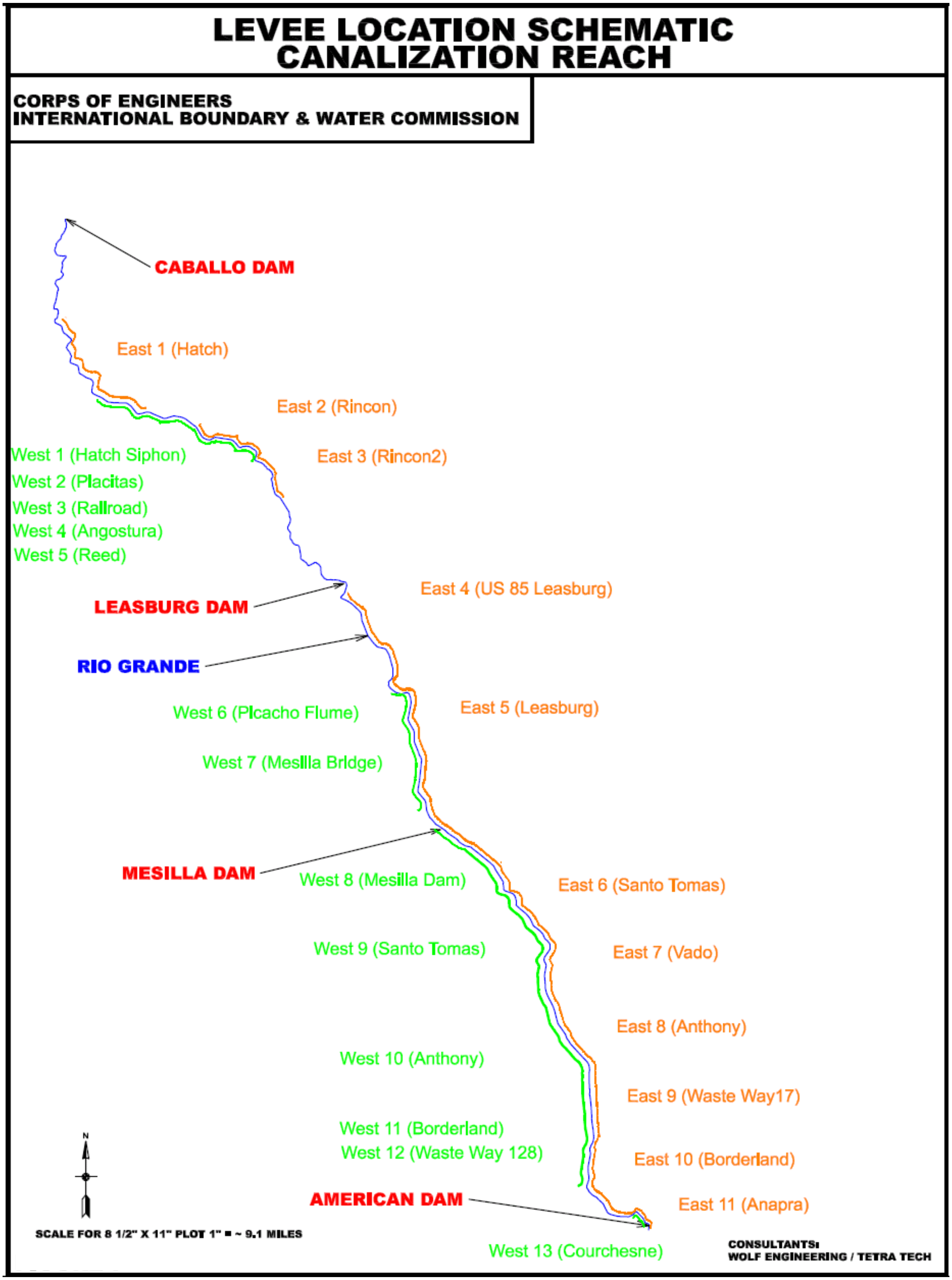

Figure 8. Locations of Levees in the RGCP Reach

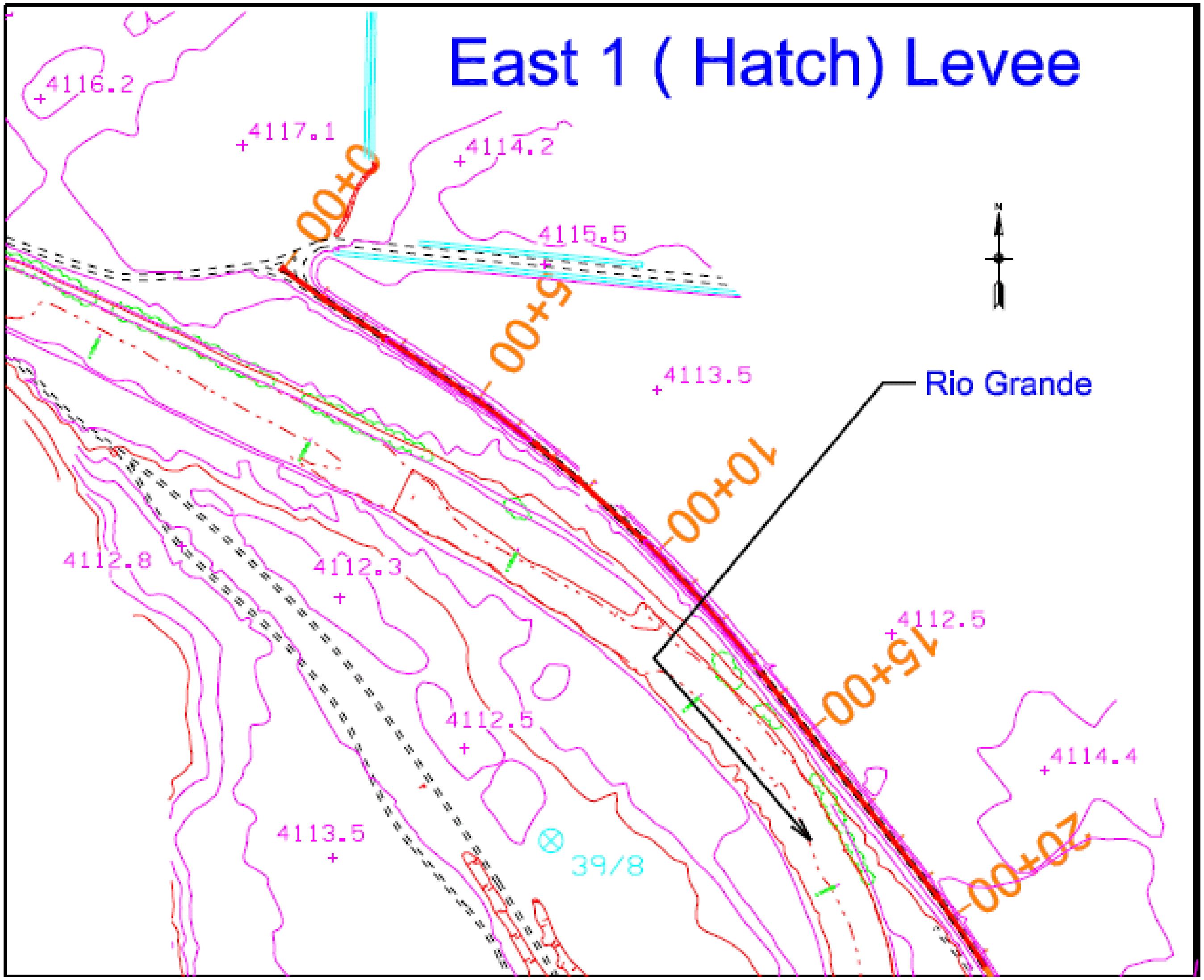

Figure 9. Typical Horizontal Levee Centerline Alignment Using the 2 ft Contour Mapping

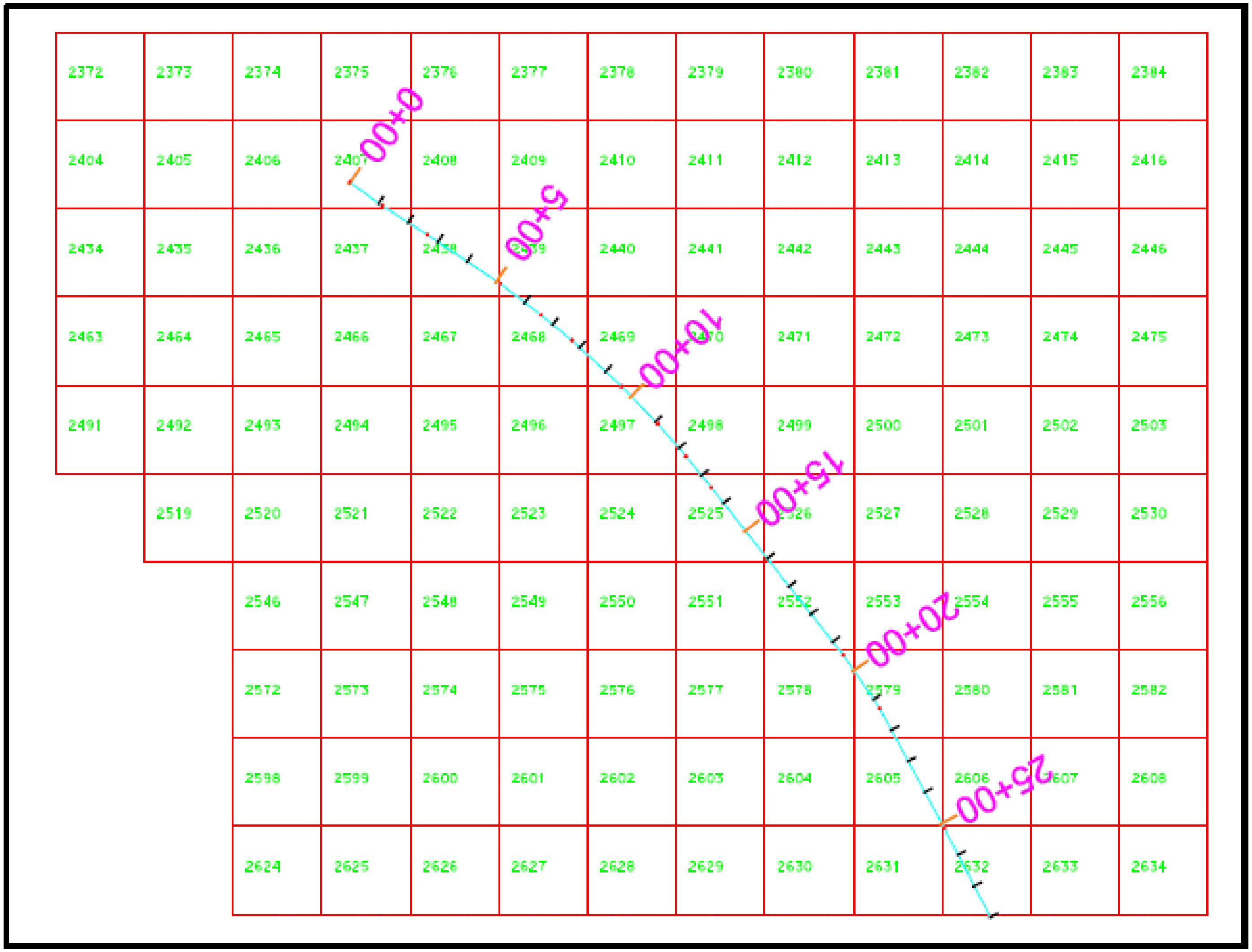

Figure 10. Typical Levee Alignment with Respect to the FLO-2D Grid System

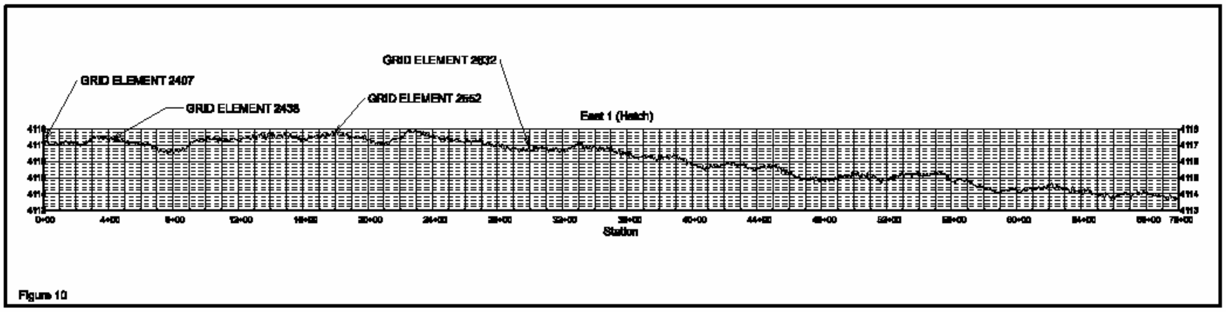

Figure 11. Levee Crest Elevation Profile

The bridges in the project include varying pier designs and some of the bridges encroach on the active channel or create constrictions in the floodway. The Corps HEC-2 model developed for the 1996, 'Rio Grande Canalization Improvement Project, Hydrologic and Hydraulic Analyses' had the bridges coded into the data base. To generate rating table for the FLO-2D model, the HEC-2 data was upgraded to a HEC-RAS model and a series of discharges ranging from 100 cfs to 30,000 cfs were run to compile a table of discharge as function of flow depth. Only the flow through the bridge needs to be represented by the rating table. The bridge coding was revised to force all the flow between the bridge abutments so that the bridge rating table reflected only the flow through or over the bridge. The floodplain flow around the bridge would be simulated by the two-dimensional overland flow component in the FLO-2D model. The HEC-RAS summary output tables were reformatted to generate a HYSTRUC.DAT file for the bridges. A summary of the hydraulic structures in the FLO-2D model is shown in Table 7. Bridge locations are shown in Figures 4, 5 and 6.

There are four diversion dams in the RGCP reach and from upstream to downstream these are: Percha Dam, Leasburg Dam, Mesilla Dam and American Dam. A rating table is used to represent the flow past the four diversion dams in the FLO-2D model. To construct the rating table, a HEC-RAS model was applied with critical flow for the range of discharges up to the diversion canal capacity. A broad crested weir equation with a coefficient of 2.85 was applied to compute the discharges over the diversion dam. The discharges over the weir were compiled in a rating table in the HYSTRUC.DAT file using the channel bed upstream of the weir as the reference for the headwater depth.

The FLO-2D model computes water losses due to infiltration and evaporation. Channel and overland flow infiltration is calculated using the Green-Ampt infiltration model. The important Green-Ampt parameters including hydraulic conductivity, soil suction and moisture deficiency can be spatially assigned on a grid element basis. At the present time, in the absence of spatially variable infiltration and soil data data, uniform values were assigned for the entire RGCP floodplain. In addition, a uniform channel hydraulic conductivity was assigned which computes a uniform seepage loss for the entire channel. In the future, spatially variable infiltration values can be considered.

Table 7. Hydraulic Structures – Caballo Dam to American Dam			
Structure Name	FLO-2D Grid Outflow Node #	FLO-2D Grid Inflow Node #	1994 IBWC Mapping (Sheet # 1 through 55)
I-25	156	150	54
Percha Diversion Dam	301	290	54
Arrey Highway Bridge	565	564	53
US 85 Bridge	1416	1404	52
Hatch Siphon	5835	5834	48
Salem Bridge	6416	6415	46
Hatch Bridge (US 85)	7294	7293	46
Hatch Bridge (NM 26)	7920	7919	45
Rincon Siphon	8919	8892	43
New Ricon Bridge	10194	10111	42
Tonuco Bridge	13181	13160	40
Leasburg Diversion Dam	16964	16927	34
Leasburg Bridge	17261	17240	33
Shalem Bridge	20964	20963	28
Picacho Bridge	23136	23116	25
I-10 Bridge	23645	23628	24
Mesilla Bridge	24394	24373	23
Mesilla Diversion Dam	25342	25325	21
Santo Tomas Bridge	26192	26157	20
Mesquite Bridge	28211	28176	18
Vado Bridge	30295	30274	16
Berino Bridge	31982	31962	14
Old Anthony Bridge	33322	33296	13
New Anthony Bridge	34306	34280	12
Vinton Bridge	35841	35822	10
Canutillo Bridge	37186	37167	9
Borderland Bridge	38021	38000	8
Country Club Bridge	39235	39214	6
Anapra Bridge	41761	41670	3
Courchesne Bridge	41420	41351	2
Brick Plant Bridge	42216	42203	1
American Diversion Dam	42256	42242	1

An open water surface evaporation routine in the FLO-2D model accounts for evaporation losses associated with long duration flood flows. Evaporation is computed based on a mean monthly total evaporation that is prorated for the number of flood days in the given month. The mean monthly evaporation is input along with the presumed diurnal hourly percentage of the daily evaporation and the clock time at the start of the flood simulation. The total evaporation is then computed for each computational timestep based on the combined wetted surface area for the floodplain and channel. The evaporation loss does not include evapotranspiration from vegetation. The evaporation and infiltration losses are reported in the SUMMARY.OUT file.

FLO-2D Model Calibration

Flood routing model calibration encompasses three routing characteristics: hydrograph shape (volume), timing and water surface elevation. The flood hydrograph shape is primarily defined by inflow discharge, irrigation diversions and return flows, and system losses (infiltration and evaporation). Timing of the hydrograph arrival at various locations in the system is primarily a function of volume, but is also dependent on resistance to flow. To calibrate the FLO-2D model, historical hydrographs are replicated and the water surface elevation surveys are matched.

The first calibration effort was focused on the water surface elevations surveyed during the channel cross section data collection in June and July 2004. To perform this initial calibration, the cross section survey water surface elevation data was obtained and written to the WSTIME.DAT file along with associated channel element and time of the survey. Since mean daily flows were used in this effort, the survey time was reported in the output file based on a 24 hour period. There was only a limited variation in the river discharge during the cross section survey, so the application of mean daily flows was a justified assumption. The FLO-2D model reports the measured and predicted water surface and the difference between them for each channel element on the specific survey date in WSTIME.OUT. This file can then be reviewed while adjusting the n-values for each channel element. The calibration procedure applied in the FLO-2D model was as follows:

- Reach n-value adjustment (if the water surface is low or high, the n-value is adjusted accordingly). Several channel element n-values upstream and downstream of a cross section may be adjusted noting the location of the diversion dams.
- Calibration is assumed to be reasonable if the water surface is plus or minus 0.5 ft of the measured water surface. This range of calibration allows for mobile bed, variation in discharge, change in bed form, etc.

The water surface elevation calibration results are presented in the Table 8 (WSTIME.OUT file). The difference between the surveyed and predicted water surface elevations ranges from -0.45 ft to 0.49 ft with an average of 0.034 ft difference per cross section.

Recorded gage discharge hydrographs are compared with the FLO-2D predicted hydrographs for the period of June-July 2004 in Figures 12-15. The difference between the measured and predicted hydrographs are attributed to gage calibration errors and return irrigation drain flow that is not considered in the model. The return irrigation drainage increases in the downstream direction resulting in more discrepancy between the measured and predicted. It is apparent that a rainstorm or some other inflow between the Haydon and Leasburg gages that was only partially represented by irrigation return flows in the model inflow data. Note the change in axis scale of the graphs as the flows decrease in the downstream direction. There were no overbank flows for the 2004 simulation. It should also be noted that the channel in the model is initially dry at the outset of all of the simulations so the first portion of the rising limb of the hydrograph should be ignored.

Table 8. 2004 Water Surface Elevation Calibration Results			
Cross Section Grid Element	Surveyed Cross Section Water Surface Elevation	FLO-2D Predicted Water Surface Elevation	Difference Between Surveyed and Predicted Water Surface Elevations (ft)
42256	3724.70	3724.21	0.49
2837	4100.90	4100.44	0.46
28389	3831.80	3831.35	0.45
16657	3969.80	3969.35	0.45
37586	3758.50	3758.06	0.44
17836	3946.50	3946.07	0.43
699	4127.30	4126.88	0.42
8014	4047.80	4047.38	0.42
13160	4005.10	4004.69	0.41
14747	3993.40	3992.99	0.41
1293	4117.20	4116.80	0.40
28865	3827.90	3827.50	0.40
26155	3849.80	3849.41	0.39
35803	3771.70	3771.31	0.39
15601	3983.40	3983.01	0.39
461	4130.30	4129.92	0.38
1948	4109.30	4108.92	0.38
20969	3911.80	3911.44	0.36
16109	3978.70	3978.35	0.35
10197	4023.10	4022.76	0.34
24737	3874.60	3874.26	0.34
16275	3974.40	3974.06	0.34
1089	4121.00	4120.67	0.33
7738	4045.20	4044.87	0.33
10527	4022.00	4021.67	0.33
18324	3942.10	3941.77	0.33
19481	3928.20	3927.87	0.33
1510	4114.20	4113.88	0.32
7229	4054.40	4054.08	0.32
21902	3901.80	3901.48	0.32
35655	3772.80	3772.48	0.32
38000	3755.50	3755.18	0.32
252	4149.30	4148.98	0.32
348	4132.50	4132.19	0.31
8600	4043.40	4043.09	0.31
18770	3938.10	3937.80	0.30
20804	3914.70	3914.40	0.30
37913	3755.80	3755.50	0.30
13358	4003.00	4002.71	0.29
16803	3965.80	3965.52	0.28
8827	4042.00	4041.73	0.27
34075	3784.10	3783.83	0.27
19796	3925.00	3924.76	0.24
30328	3814.40	3814.16	0.24
21235	3907.70	3907.47	0.23
17043	3957.00	3956.77	0.23
13755	4000.60	4000.38	0.22
7908	4050.80	4050.60	0.20
24914	3872.30	3872.10	0.20
29123	3826.30	3826.10	0.20
12140	4013.70	4013.51	0.19
26040	3851.80	3851.62	0.18
35393	3774.50	3774.32	0.18
21582	3904.50	3904.34	0.16
35964	3770.20	3770.04	0.16
38396	3752.30	3752.15	0.15
41477	3730.00	3729.85	0.15

Table 8. 2004 Water Surface Elevation Calibration Results (cont.)			
23645	3885.10	3884.96	0.14
29612	3821.70	3821.57	0.13
37167	3762.20	3762.07	0.13
2404	4103.80	4103.68	0.12
12334	4013.70	4013.58	0.12
14109	3998.20	3998.09	0.11
24352	3879.00	3878.89	0.11
6067	4064.70	4064.60	0.10
20962	3913.50	3913.40	0.10
1393	4115.10	4115.01	0.09
11229	4016.40	4016.31	0.09
34252	3782.90	3782.81	0.09
7359	4055.20	4055.12	0.08
27826	3835.40	3835.32	0.08
30274	3815.00	3814.94	0.06
35879	3771.10	3771.05	0.05
38231	3753.90	3753.85	0.05
5780	4074.90	4074.86	0.04
17410	3952.60	3952.56	0.04
214	4149.30	4149.26	0.04
10018	4024.60	4024.57	0.03
25376	3860.00	3859.97	0.03
16416	3972.40	3972.37	0.03
4844	4081.00	4080.98	0.02
4772	4081.50	4081.49	0.01
36044	3769.50	3769.50	0.00
17301	3954.40	3954.40	0.00
12694	4010.70	4010.71	-0.01
39740	3742.80	3742.82	-0.02
15210	3988.50	3988.53	-0.03
34537	3780.70	3780.74	-0.04
9002	4031.60	4031.65	-0.05
41351	3728.80	3728.87	-0.07
142	4149.70	4149.77	-0.07
6414	4061.40	4061.49	-0.09
28176	3833.70	3833.80	-0.10
24416	3877.70	3877.81	-0.11
33632	3787.20	3787.31	-0.11
42167	3725.50	3725.62	-0.12
90	4149.90	4150.02	-0.12
39172	3746.80	3746.93	-0.13
5625	4075.10	4075.24	-0.14
32365	3797.20	3797.34	-0.14
882	4124.90	4125.06	-0.16
9922	4027.10	4027.26	-0.16
36794	3764.60	3764.76	-0.16
26228	3848.80	3848.97	-0.17
20473	3917.40	3917.58	-0.18
20251	3920.00	3920.19	-0.19
31962	3801.10	3801.32	-0.22
38649	3750.60	3750.82	-0.22
42228	3724.90	3725.12	-0.22
955	4122.60	4122.83	-0.23
5836	4065.50	4065.73	-0.23
11086	4019.00	4019.23	-0.23
16848	3965.40	3965.64	-0.24
7452	4051.40	4051.67	-0.27
1225	4118.00	4118.30	-0.30
3505	4090.80	4091.10	-0.30
9991	4028.70	4029.00	-0.30
34306	3782.40	3782.70	-0.30
42203	3724.90	3725.20	-0.30
6337	4063.40	4063.71	-0.31
32853	3792.80	3793.12	-0.32

Table 8. 2004 Water Surface Elevation Calibration Results (cont.)			
3050	4095.70	4096.03	-0.33
5273	4076.30	4076.64	-0.34
10110	4023.50	4023.84	-0.34
25225	3869.30	3869.64	-0.34
28730	3828.60	3828.95	-0.35
23033	3891.50	3891.87	-0.37
566	4128.00	4128.38	-0.38
22521	3896.50	3896.88	-0.38
2172	4106.40	4106.79	-0.39
42080	3726.60	3726.99	-0.39
30725	3811.20	3811.60	-0.40
32002	3800.60	3801.00	-0.40
39479	3745.20	3745.60	-0.40
41668	3731.90	3732.30	-0.40
4424	4083.90	4084.32	-0.42
26868	3843.30	3843.72	-0.42
31251	3807.60	3808.02	-0.42
1596	4111.70	4112.13	-0.43
25307	3869.00	3869.43	-0.43
29716	3820.40	3820.83	-0.43
30157	3816.40	3816.84	-0.44
40511	3738.50	3738.95	-0.45
		Total Cumulative Difference	4.85
		Average Difference	0.034

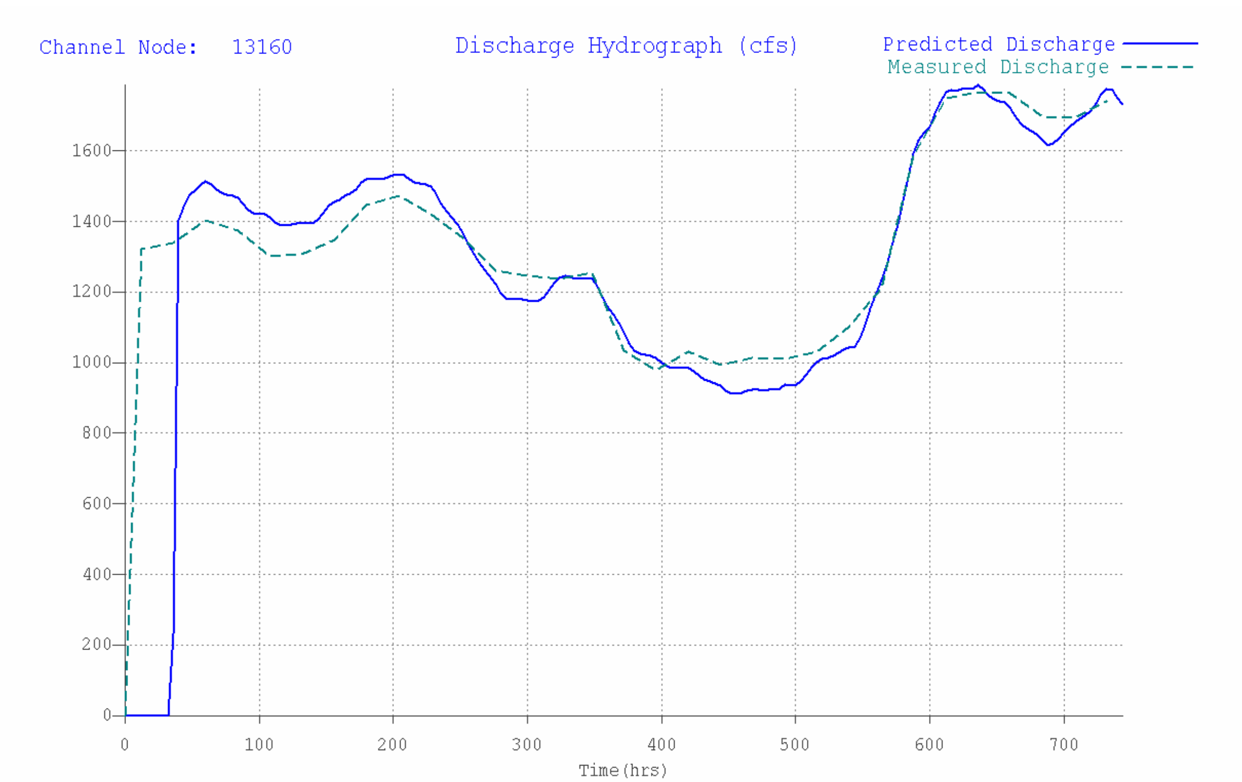

Figure 12. 2004 Haydon Gage Data vs. FLO-2D Predicted Hydrograph

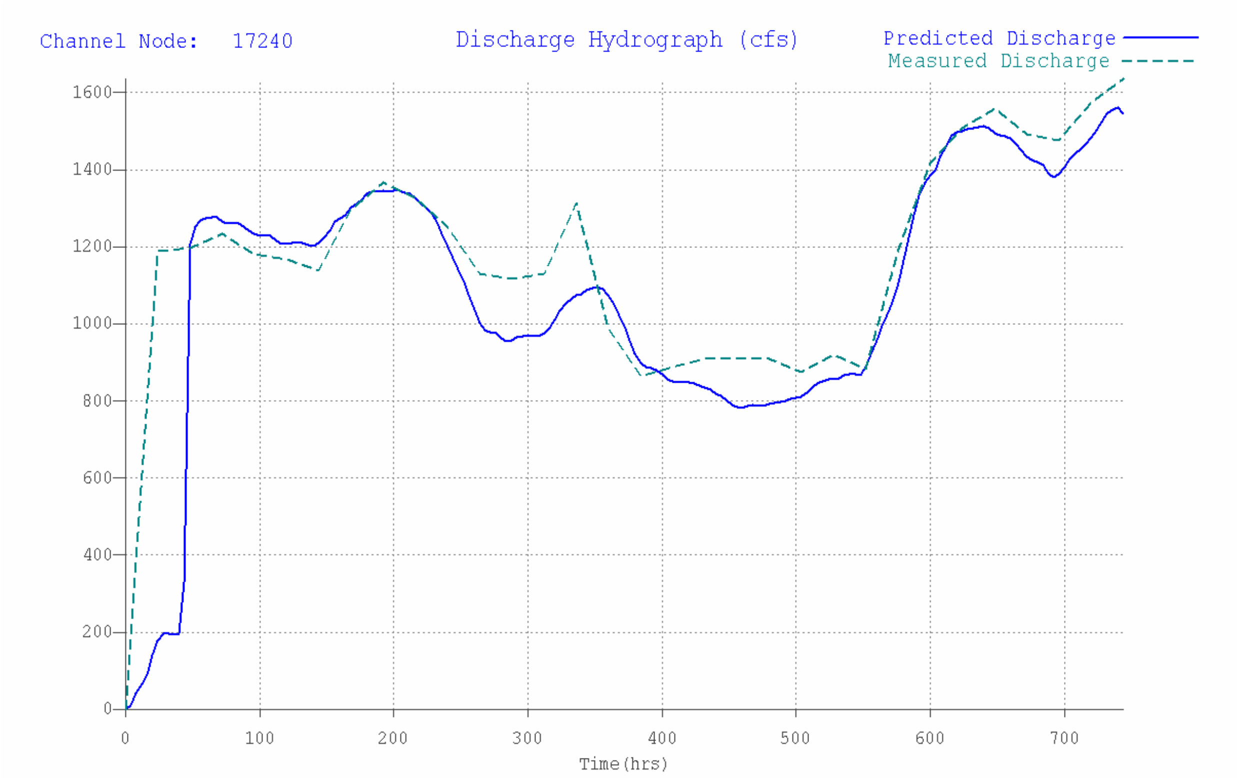

Figure 13. 2004 Leasburg Gage vs. FLO-2D Predicted Hydrograph

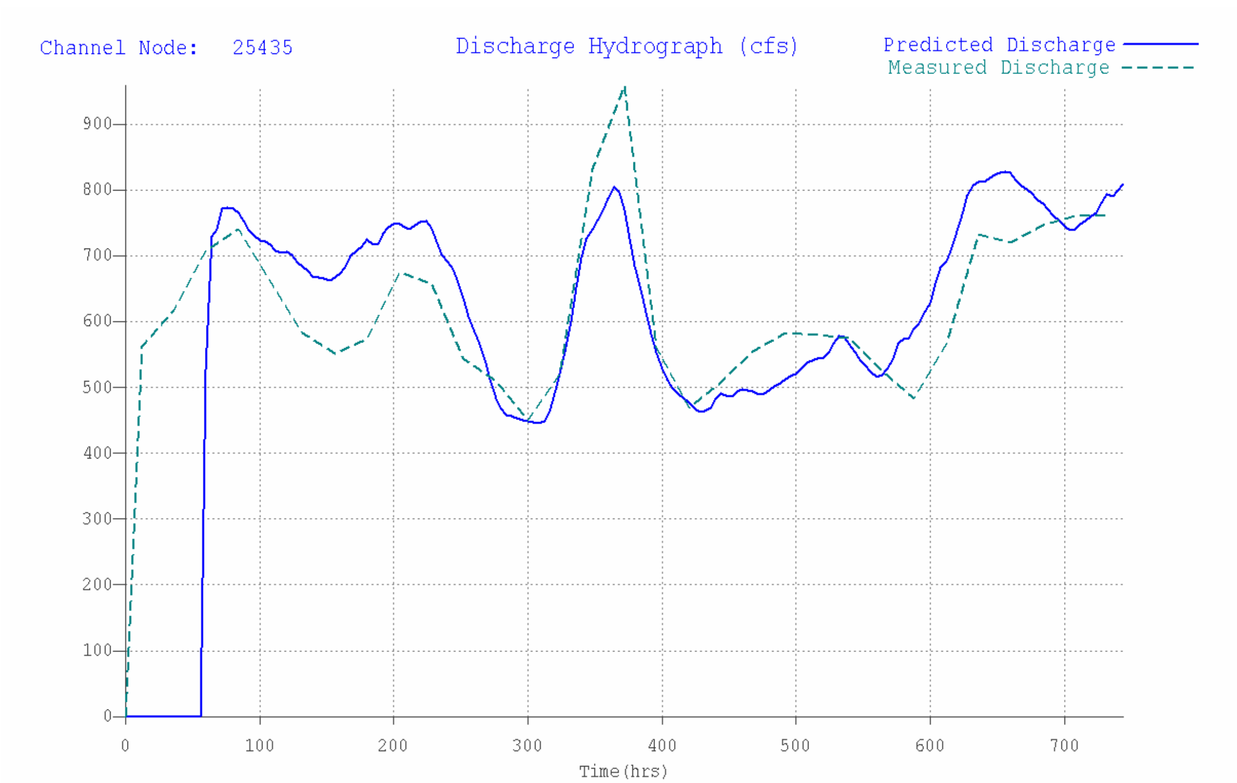

Figure 14. 2004 Mesilla Gage vs. FLO-2D Predicted Hydrograph

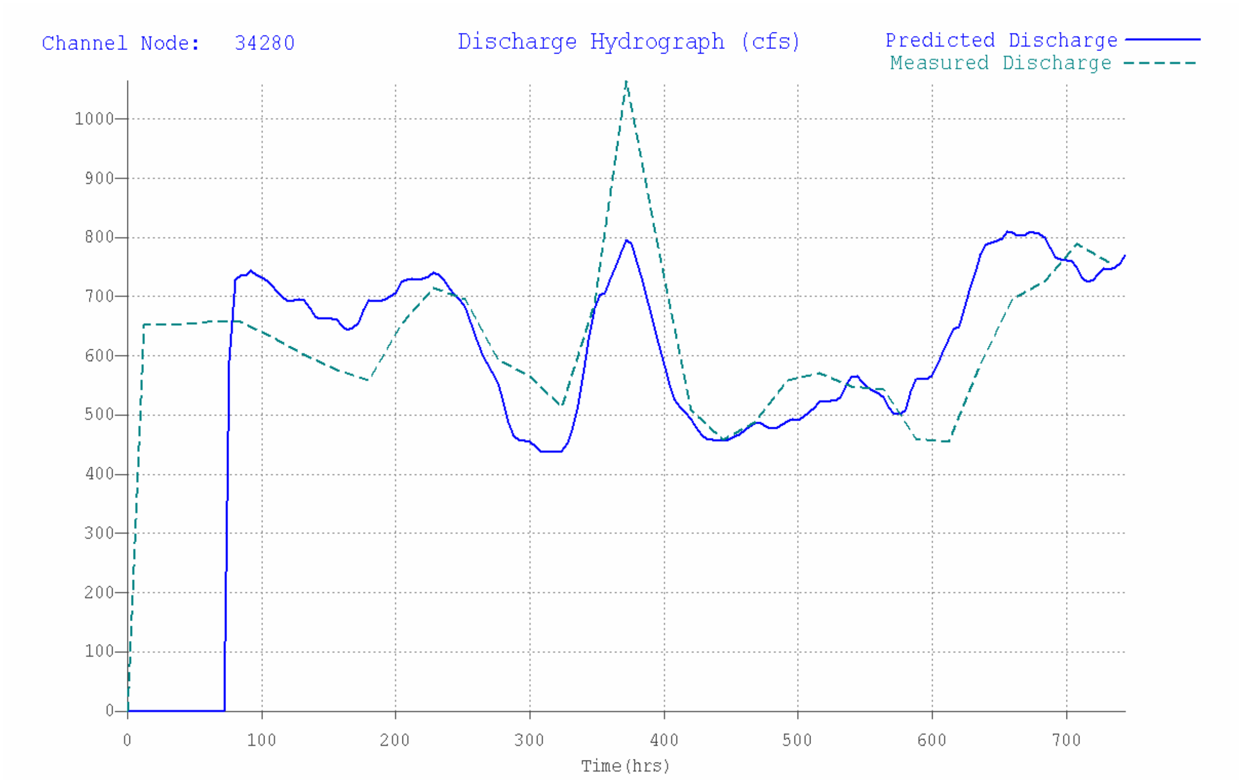

Figure 15. 2004 Anthony Gage vs. FLO-2D Predicted Hydrograph

The second calibration effort focused on the shape of the hydrograph for the month of July 1995 where the Caballo Dam release ranged from 3,600 to 4,540 cfs. This 31 day period of record had a relatively complete record of irrigation diversions and return flows throughout river system that was made available by EBID. Since the n-values were calibrated for the 2004 surfaced water surface elevations, only minor n-value adjustments were made to the 1995 data set to account for potential variation in roughness with flow depth. The evaporation was not adjusted and should be assumed to be conservatively low. To match the hydrographs at the various river gage locations, the channel hydraulic conductivity was adjusted by reach. This improved both the replication of the measured hydrographs and the timing of hydrograph spikes and troughs slightly. It should be noted that the gaging data may be inaccurate based on several factors including gage rating curve shifts, estimated gage data or gaging error. In addition, there are a numerous irrigation return flows to the river that are either not gaged or not included in the available data base. Those measured wasteway flows that were less than 10 cfs were not included in the data file. The cumulative return flow that is not accounted for is probably less than 200 to 300 cfs. Gage versus predicted hydrograph results are shown in Figures 16-18.

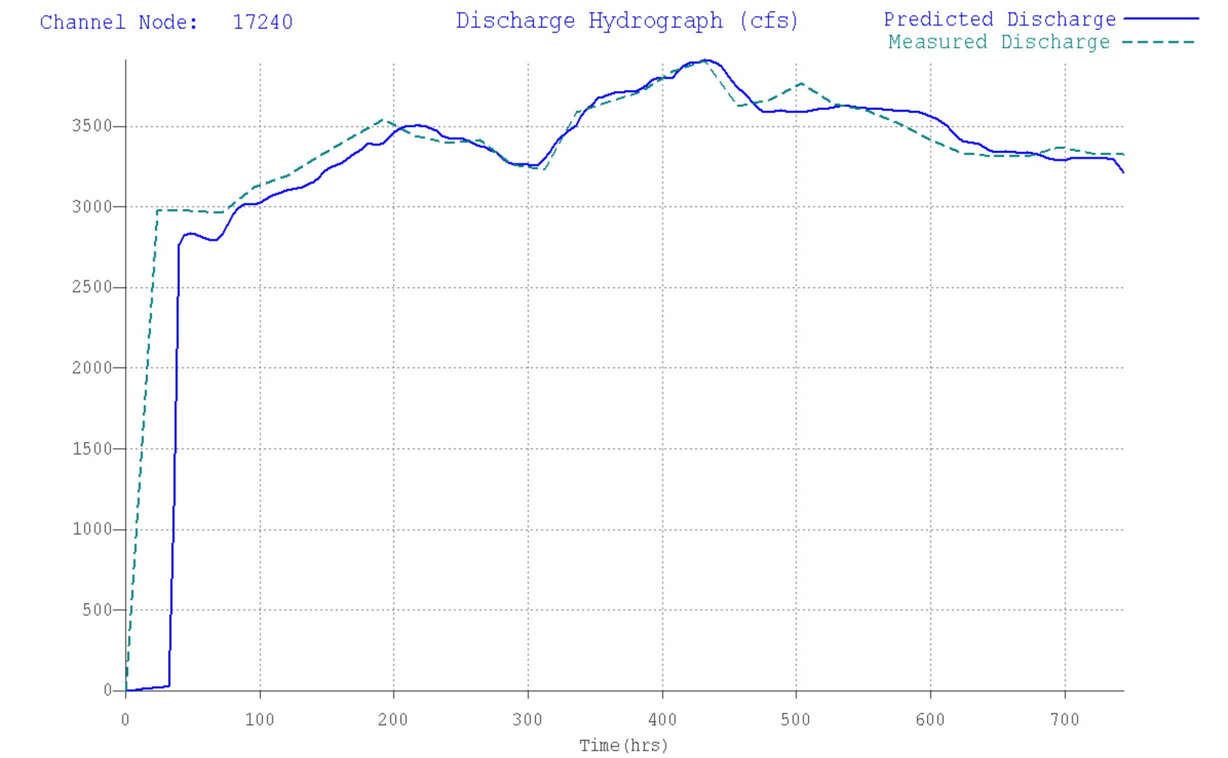

Figure 16. 1995 Leasburg Gage vs. FLO-2D Predicted Hydrograph

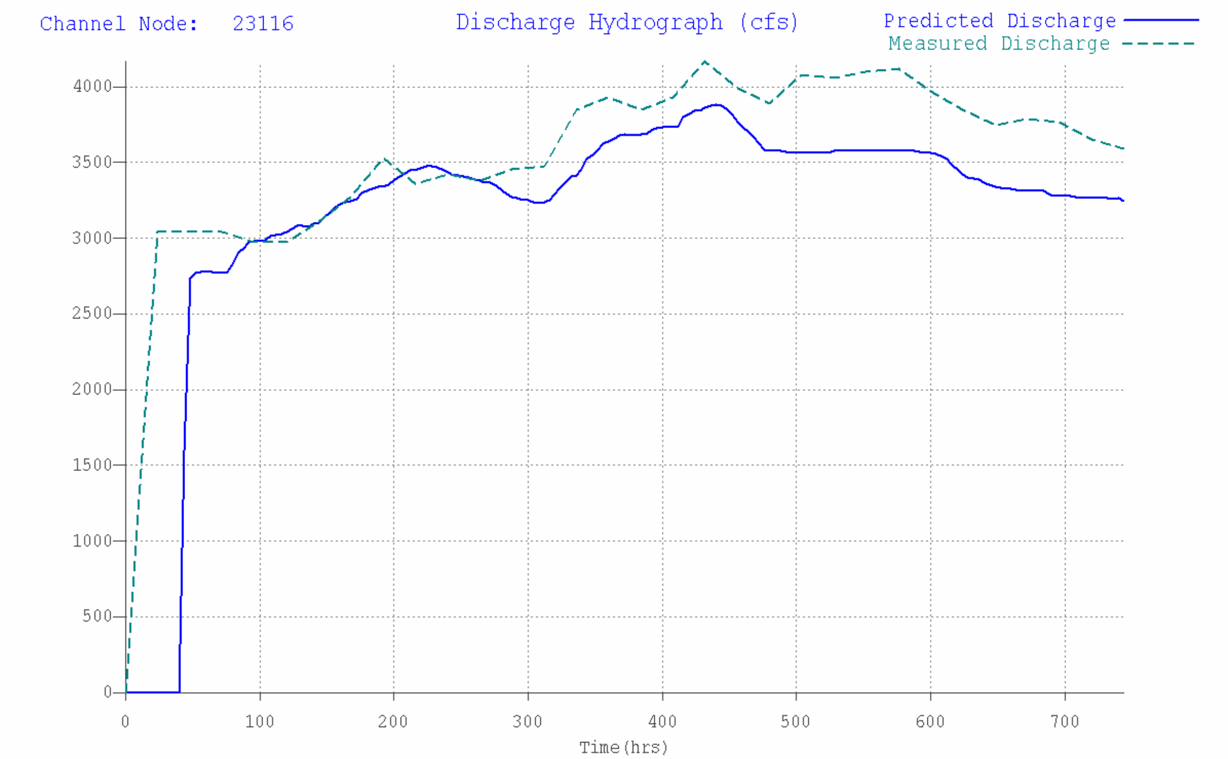

Figure 17. 1995 Picacho Gage vs. FLO-2D Predicted Hydrograph

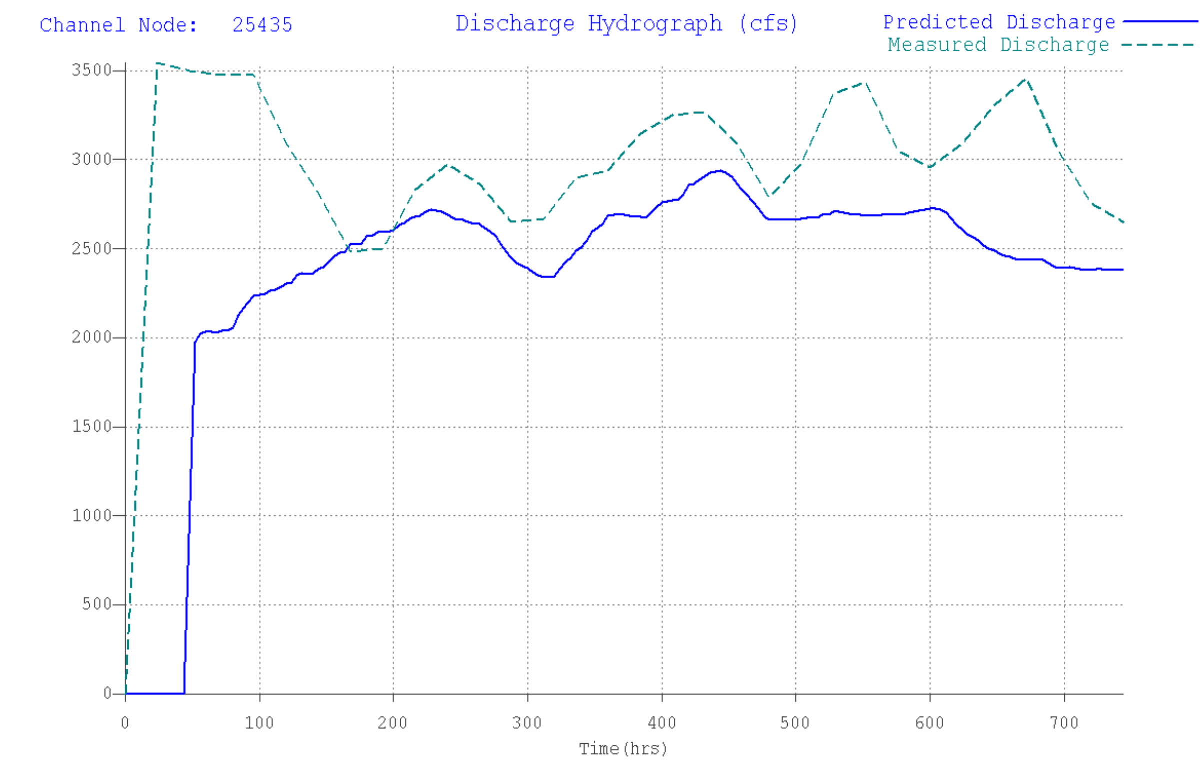

Figure 18. 1995 Mesilla Gage vs. FLO-2D Predicted Hydrograph

A final calibration simulation was undertaken for July 1998 where the discharge was entirely contained with the channel. This calibration effort was to verify the previously calibrated channel n-values and geometry. The original calibrated Manning's n-values were used in this simulation. The n-values varied spatially throughout the channel and ranged from 0.02 in the downstream reach to 0.05 below diversion dams. The average n-value in the RGCP reach was 0.029. The previous 1995 calibration did have overbank flow. The results are shown in Figures 19-22 and indicate a similar pattern as the previous calibration simulations where the difference between measured and predicted discharge increased in the downstream direction probably in response to unmeasured or poorly gaged irrigation return flows. Both the Picacho and Mesilla gages show increased divergence between measured and predicted flows with time possibly revealing increased return flows with time in both 1995 and 1998.

These calibration results confirm that the model can replicate both water surface elevation and hydrographs shape and timing as noted by the exact timing of the many spikes and troughs in the discharge hydrographs. It is concluded from the calibration effort that simulation of the proposed RGCP flood scenarios is appropriate with the calibrated FLO-2D model.

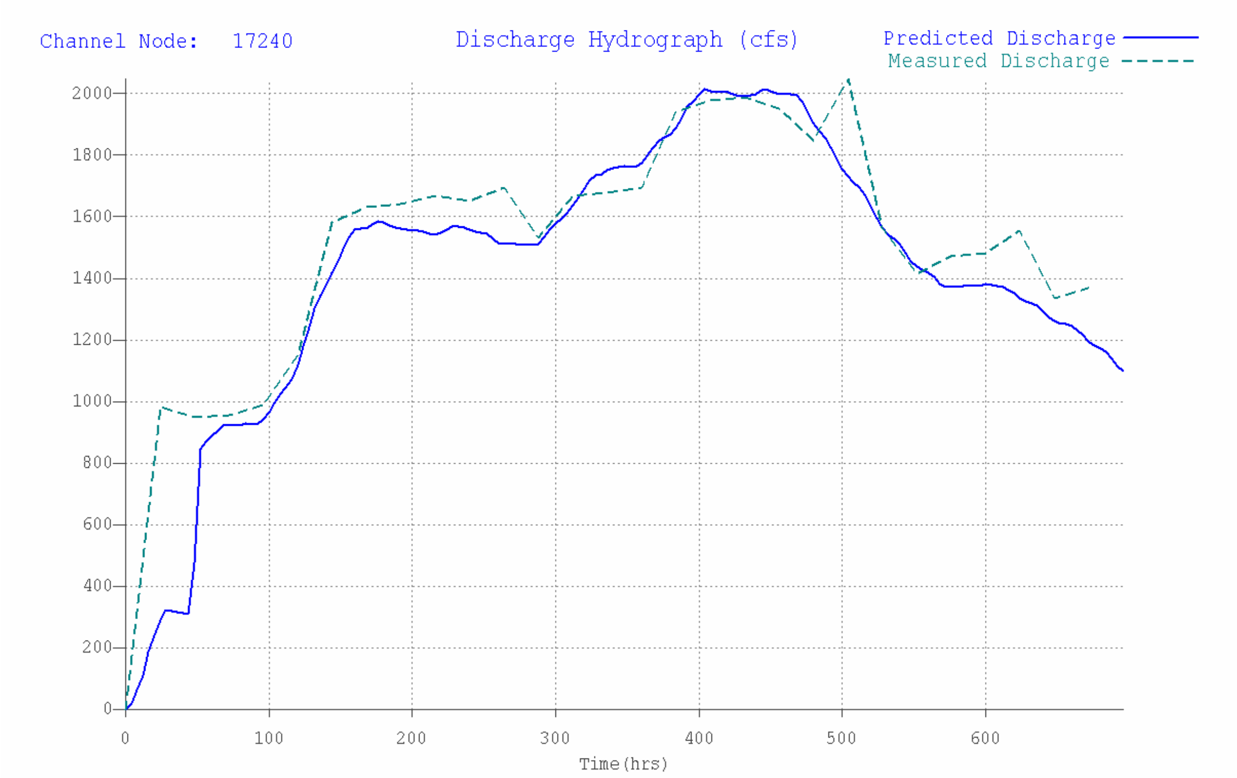

Figure 19. 1998 Leasburg Gage vs. FLO-2D Predicted Hydrograph

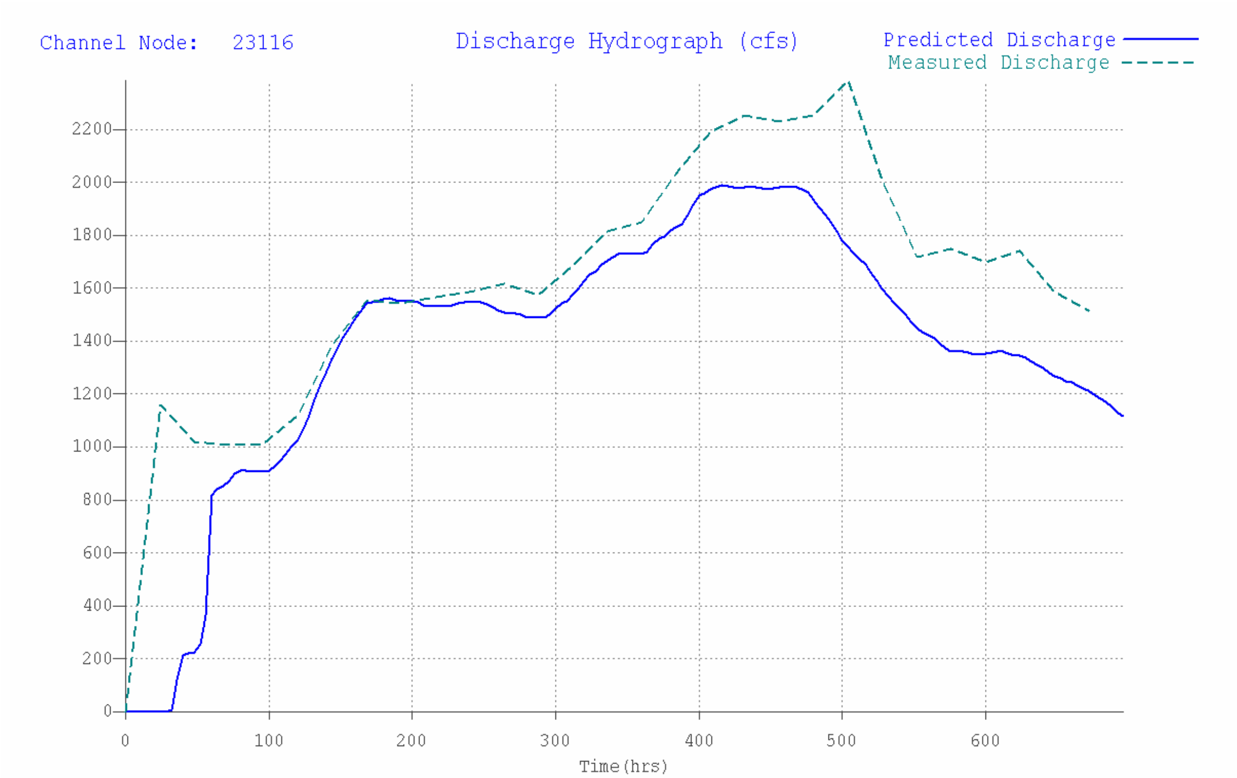

Figure 20. 1998 Picacho Gage vs. FLO-2D Predicted Hydrograph

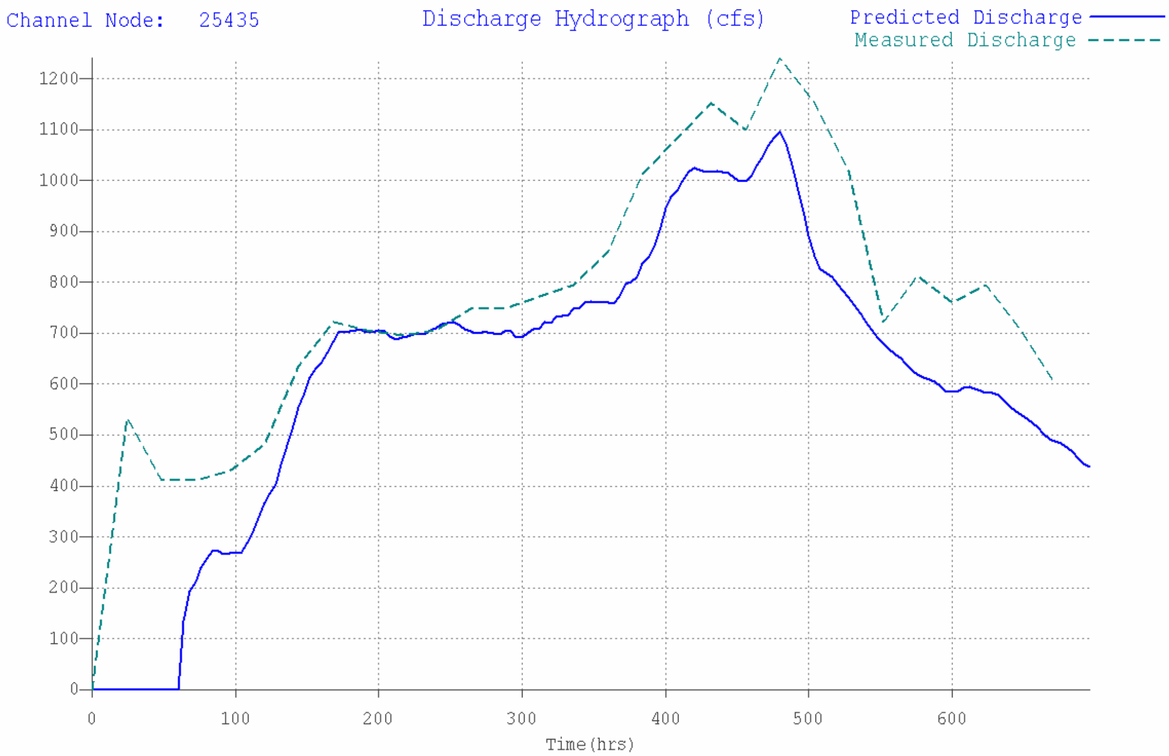

Figure 21. 1998 Mesilla Gage vs. FLO-2D Predicted Hydrograph

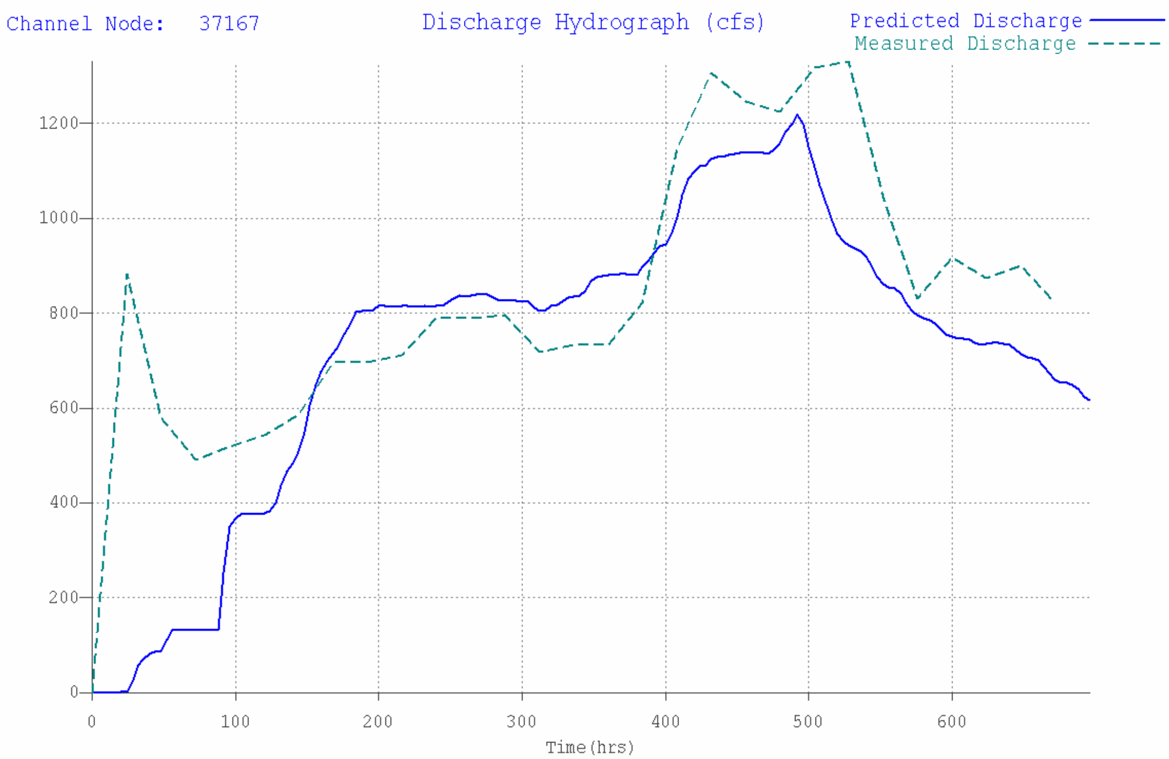

Figure 22. 1998 Canutillo Gage vs. FLO-2D Predicted Hydrograph

FLO-2D Results - Flood Scenarios

The Tetra Tech project scope of work stipulates that 5 flood scenarios should be selected for FLO-2D model simulation. Using the modified HEC-1 model, return period flood hydrographs were simulated so that a levee damage assessment and economic analysis can be performed for the levee improvements. The return period floods include:

- 2-yr
- 5-yr
- 10-yr
- 25-yr
- 50-yr
- 100-yr

The simulated 100-year flood results are discussed in the following section. All data files and output results for the various return period floods are presented in the project CDs submitted to the Corps.

The 100-yr flood was simulated for the 5,000 cfs constant release from Caballo Dam and the flood hydrology provided in the 1996 RGCP Corps report. The 100-year flood was also simulated with the revised hydrology applying a 2,350 cfs constant release from Caballo Dam representing a typical irrigation season hydrograph. The results are displayed in Figure 23 and are plotted against the Corps 1996 estimated HEC-2 model peak discharge as a function of the river mile. The HEC-2 peak discharge was computed using the HEC-1 model and Muskingum-Cunge routing. The FLO-2D results for the constant 5,000 cfs Caballo Dam release and the original flood hydrology indicates a significantly smaller peak discharge throughout the entire river reach. The HEC-2 peak discharge ranges from 5,000 cfs at Caballo Dam to 22,500 cfs at river mile 27.4 and to 14,300 cfs at American Dam. The FLO-2D predicted peak discharge reaches 17,700 cfs at river 28.8 and is only 10,800 cfs at American Dam. Throughout most of the RGCP reach the FLO-2D peak discharge is less than about 60% of the HEC-2 peak discharge. The smaller FLO-2D peak discharge can be attributed to two factors:

- Inaccurate floodwave attenuation due in the 1996 Corps HEC-1 model.
- No estimate of infiltration or evaporation losses in the 1996 Corps HEC-1 routing model.

Since the FLO-2D model conserves volume, the floodwave attenuation shown in Figure 23 will be much more realistic than the HEC-1 routing model which has limited overbank flood routing capabilities. The channel discharge for the constant 5,000 cfs release from Caballo Dam and the original 100-yr 24-hour flood hydrology is also displayed in Figure 23 to assess how much of the peak discharge is being computed as overbank floodplain flow.

The FLO-2D simulation of RGCP revised tributary flood hydrology and 2,350 cfs release resulted in higher river peak discharges (red line in Figure 23) than the original hydrology with the 5,000 cfs release. The revised tributary flood hydrology includes a reduced initial loss, adjusted time of concentration coefficients and higher n-values. Except for one location of tributary inflow at river mile 27.4, the revised hydrology results in river peak discharges that are consistently lower than the Corps 1996 HEC-2 peak discharges due to floodwave attenuation and infiltration and evaporation losses.

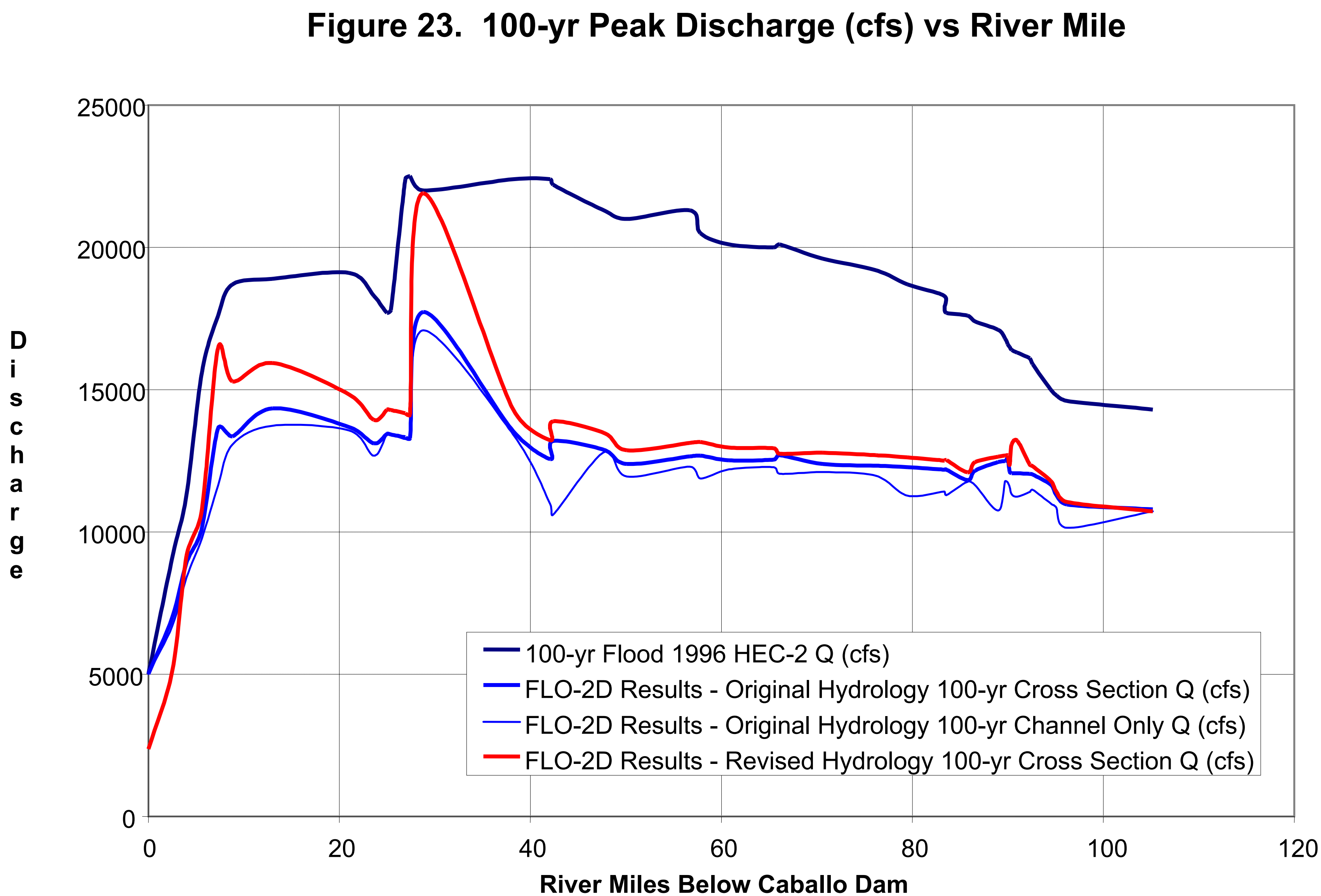

Figure 23. 100-yr Peak Discharge (cfs) vs River Mile
Discharge
25000
20000
15000
10000
5000
0
0
20
40
60
80
100
120
River Miles Below Caballo Dam
100-yr Flood 1996 HEC-2 Q (cfs)
FLO-2D Results - Original Hydrology 100-yr Cross Section Q (cfs)
FLO-2D Results - Original Hydrology 100-yr Channel Only Q (cfs)
FLO-2D Results - Revised Hydrology 100-yr Cross Section Q (cfs)

The model results include the area of inundation, discharge hydrographs, peak discharges, maximum flow depths and velocities and levee overtopping discharges. Figures 24 and 25 are examples of the maximum area of inundation using the aerial photography. Hard copy maps of the 100-yr flood (2,350 cfs release, revised tributary flood hydrology) are provided in a supplement map document. Mapping for the entire set of return period flood simulations contained on the submittal CD can be generated by the Mapper post-processor program. Table 9 illustrates the difference in the maximum areas of inundation for the 5,000 cfs constant release from Caballo Dam with the original flood hydrology and the 2,350 cfs constant release from Caballo Dam with the revised hydrology.

Table 9. Predicted Areas of Inundation

	5,000 cfs Release – Original Flood Hydrology (acres)	2,350 cfs Release – Revised Flood Hydrology (acres)
Inflow Flood Volume (af-ft)	163,937	100,207
Maximum Wetted Floodplain	6100	6089
Maximum Wetted Channel Surface Area	3835	3761
Total Maximum Area	9935	9850

The 2,350 cfs release with the revised flood hydrology has an inflow volume 64,000 af-ft less than the 5,000 cfs release with the original flood hydrology because of the reduced dam release but the predicted maximum areas of inundation are approximately the same for the two simulations. This is an indication that area of inundation is approaching a maximum for this range of discharge. Levee confinement limits the area of inundation.

For the 5,000 cfs release with the original flood hydrology, the simulated levees are overtopped by flood flows in a total of 63 grid elements. It is assumed that none of the levees fail when overtopped. The 2,350 cfs release with the revised flood hydrology flood simulation results in levees being overtopped in 77 grid elements. Parsons (DEIS, 2003) identified locations where levee height deficiency are a concern using the HEC-2 model updated to HEC-RAS and the Corps 1996 RGCP flood hydrology. A significant levee deficiency was defined by Parsons (DEIS, 2003) as freeboard less than 1 ft. The estimated areas of significant deficiency based on the HEC-RAS model were in El Paso reach (RM 3.5-13), near Mesilla dam (RM 40) and in Lower Rincon valley (RM 72-76). For comparison, levee overtopping was predicted by the FLO-2D model from RM 69.0 to 72.7 and RM 3.0 to 10.7. Except for Mesilla Dam the locations of FLO-2D predicted levee overtopping correlated well with the Parsons (2003) delineation of areas of significant levee deficiency. The levee deficiency for freeboard less than 3 ft, less than 2 ft and less than 1 ft can reviewed for all the return period floods using the Maxplot post-processor program.

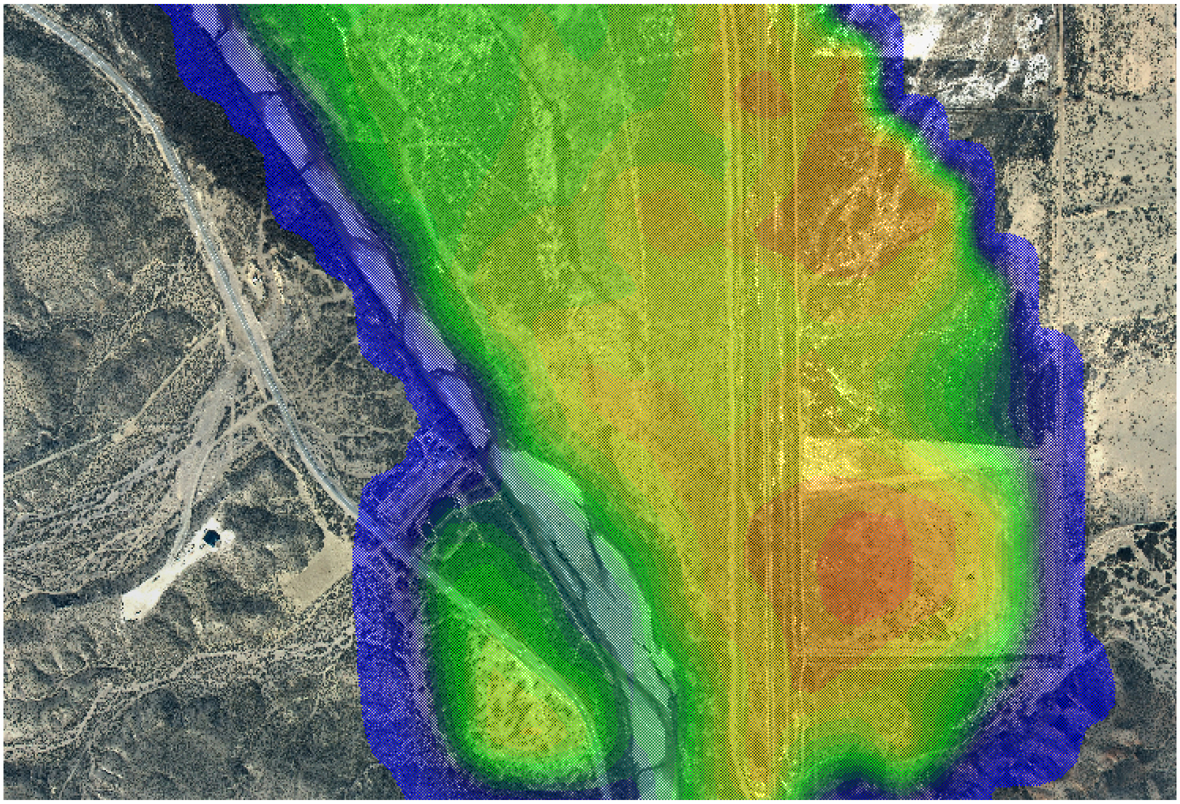

Figure 24. Example of the Area of Inundation for the 100-yr 24-hr Storm
2,350 cfs Release, Revised Hydrology Using Shaded Contours

Figure 25. Example of the Area of Inundation for the 100-yr 24-hr Storm,
2,350 cfs Release, Revised Hydrology Using Line Contours

Summary

The application of the FLO-2D model to the Rio Grande Canalization Project represents an opportunity to reliably predict floodplain inundation, maximum water surface elevation and levee inundation or overtopping associated with floodwave attenuation of design project storms and return period flood events. The results of the FLO-2D model application can now be used to support the development of the URGWOM model for Canalization reach. The most important component of the FLO-2D model is the channel-floodplain exchange which distributes the overbank flows. Flood volume estimates are more important than the peak discharges, and for that reason the previous analyses in Hydrologic Review report focused on the HEC-1 model parameters that controlled the inflow flood volume.

The development of the FLO-2D RGCP model involved detailed channel geometry, levees coding, spatially variable channel and floodplain roughness, and representation of some roadway embankments. Calibration of the FLO-2D model was performed using the 2004 river cross section surveyed water surfaces and gaging station records for flows in 1995 and 1998. The model calibration required adjustment of the channel roughness n-values and channel infiltration hydraulic conductivity to match water surface, hydrograph shape and timing. Calibration of the model was shown to be excellent and justified the application of the model for simulating design flood events.

Two design flood events were simulated: 1) A constant release of 5,000 cfs from Caballo Dam combined with the 100-yr 24-hr storm flood hydrology in the Corps 1996 report; 2) A constant release of 2,350 cfs from Caballo Dam combined with the revised 100-yr 24-hr storm hydrology. The 2,350 cfs represents a typical irrigation release during the summer months whereas it was shown in the hydrology report that combined probabilities of the 100-yr, 24-hr general storm and a coincident 5,000 cfs release from Caballo Dam is less than 1% in any given year. For that reason, the 2,350 cfs release with the revised flood hydrology was selected as the design storm.

The FLO-2D results indicate that floodwave attenuation for the 100-yr 24-hr storm is more significant that that presumed in the Corps 1996 HEC-2 analysis and the more recent Parsons (2003) HEC-RAS analysis. Since the FLO-2D conserves volume, the FLO-2D flood routing results are more accurate in terms of flood volume distribution from tributary flooding in the RGCP reach. The peak discharges are substantially less throughout the reach in the FLO-2D model results than those estimated using the HEC-1 model in the 1996 Corps study. The FLO-2D model simulated overbank flooding and levee overtopping. Levee overtopping was concentrated in two reaches, the Lower Rincon Valley and the El Paso area.

The RGCP model represents a excellent addition to the overall Rio Grande system that is now simulated by the FLO-2D model. This continuous flood routing model now extends from Abiquiu Reservoir on the Chama River to American Dam in El Paso, Texas interrupted only by Cochiti, Elephant Butte and Caballo Reservoirs. The sediment supply from the tributaries has been adjusted to the NRCS annual sediment yield for basins in the RGCP reach the purpose of future FLO-2D mobile bed simulations.

Recommendations

Local Flood Details. The FLO-2D model is essentially complete except for localized flood detail in some areas. Accurate flood hazard delineation in local reaches depends on roadway/railroad embankment, wasteways and irrigation system ditches and spoil pile embankments. This detail is not anticipated to significantly affect the movement of the floodwave or alter the maximum water surfaces or discharges. It may impact the area of inundation in an local overbank area. Carefully inspection and an understanding of local flooding conditions and topographic features would be required to accurately delineate the local flood hazard. This level of flood hazard detail is now possible because of the high resolution DTM and aerial orthophotos but is beyond the original scope of the work.

Hydraulic Structure Operation During Flooding. Hydraulic structures are important to local flooding but are not critical to the passage of the floodwave through the system. The bridges, diversion dams and siphons have very limited (almost negligible) upstream storage and therefore accuracy of the rating tables is not critical to the floodwave movement. Improvements to the rating tables will enhance the accuracy of local upstream water surface elevations and areas of inundation. Careful inspection of the field conditions of the various hydraulic structures and an understanding of the flood operation of the diversion dams may improve the flood hazard delineation upstream of these structures.

Spatially Variable Infiltration. Spatial variability of infiltration could slightly improve the movement of the floodwave through the system if it presumed that the floodplain soil conditions and channel seepage vary significantly throughout the Canalization Reach. A detailed review of soil maps and low flows conditions in the river would be required to add this component detail.

Low Flow Calibration. If the RGCP FLO-2D model is used in the future for low flow applications, it is necessary to further calibrate the model. Low flow simulations encompass flows less than 500 cfs because of the increased n-value with flow depth. The depth variable roughness adjustment factor in the CHAN.DAT file will assist in this calibration. Different reaches may respond uniquely to low flows based on bed forms.

References

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U.S. Army Corps of Engineers and Resource Technology, Inc. 1993-1996. "Rio Grande Canalization Improvement Project, Percha Dam to American Diversion Dam, Texas, Study Documentation, Volume 3," Prepared for the IBWC.

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Appendix A

Design Storm Hydrologic Analyses
and Documentation

Design Storm Selection

The Albuquerque District of the Corps of Engineers selected a general 24-storm centered over the entire 900 square mile Rio Grange Canalization Project reach as the design storm. The 100-year point rainfall was estimated from the NOAA Rainfall Atlas II, Volume IV (1973) for New Mexico. The various duration point rainfalls for the 100-yr return period storm were listed as follows:

24-hr duration	3.80 inches
6-hr duration	3.00 inches
1-hr duration	2.28 inches

Following the application of depth area reduction values for the 900 square mile watershed, the uniform rainfall over the basin was estimated to be:

24-hr duration	2.39 inches
6-hr duration	1.53 inches
1-hr duration	1.09 inches

The largest part of the drainage basin is in the upper half of the reach extending from Caballo Dam to Leasburg Dam (792 mi^2 out of 894 mi^2 in the basin) and will generally produce flooding in the lower portion of the reach. The issues to be considered regarding the selection of the design storm are:

- What type of storm should constitute the project design storm?
- Is the 100-yr, 24-hour total rainfall a reasonable estimate for this area?
- Would the river discharge be greater in terms of peak discharge and volume downstream of any arroyo confluence with the Rio Grande if a local convective thunderstorm were simulated in arroyo watershed instead of the general 24-hour storm over the entire RGCP basin?

To address these issues additional research was necessary.

What rainfall event should constitute the project design storm?

The moisture systems that produce severe flooding vary seasonally. The Rio Grande valley lies in a subtropical (semi-arid) region and during the summer the primary source of moisture is the Gulf of Mexico. Precipitation frequently occurs with scattered thunderstorms that can increase with tropical disturbances. Usually July and August are the wettest months. During the late spring and early summer, slow moving, isolated thunderstorms can cause local flooding. In the fall, precipitation occurs when southward-moving frontal systems interact with residual moisture from the Gulf of Mexico. The remnants of a Pacific tropical cyclone can also bring local intense rainfall and occasionally, a tropical disturbance can settle into the region and cause widespread rainfall lasting several days (Waltemeyer, et al., 1989).

Of the two types of storms, the frontal storms (general storms) produce long duration, low intensity rainfall that create slow rising floods of high volume. The summer convective thunderstorm is usually a localized, high intensity storm centered over a small watershed that lasts 1 to 2 hours but is always less than 6 hrs in duration. The resulting flood event has a fast rising, high peak discharge with a small volume that quickly attenuates when the peak arrives at the flatter slope river channel and floodplain in the valley bottom. The general storm will result in long duration flooding throughout the Rio Grande Canalization reach. The localized storm may generate high peak discharges, but only localized flooding because of the limited storm volume. In the southwest, the local storm usually constitutes the critical design event except in the case of large watershed (Sabol and Stevens, 1990).

General storm: Is the 100-yr, 24-hour total rainfall a reasonable estimate for this area?

The largest storm and river flood on record occurred on September 11-12, 1958 with an average rainfall of 2.36 inches over the Caballo Dam to Leasburg Dam portion of the watershed. This storm had a local rainfall in a northern subbasin of at least 3.27 inches in 6 hours. To determine if the NOAA Atlas provides a reasonable estimate of the 24 hour general storm, an investigation of rain gages in the Las Cruces area was conducted. This was accomplished through a review of co-op weather station data the on-line NOAA National Climatic Data Center (NCDC). Several rain gages in Dona Ana County were identified with long term historic records including:

- ✓ Afton (1942-1951)
- ✓ Hatch (1894-1946)
- ✓ Jornada Range Experiment Station (1914-2004)
- ✓ New Mexico State (1892-1946)
- ✓ Hillsboro (1946-2004)
- ✓ Las Cruces White Sands Missile Range (1949-1962

There were also a number of rains gages with short or intermittent records including Santa Teresa Airport, Chamberino and others. The NCDC data base does not contain the numerous gages in the RGCP drainage that have been operated by various agencies and entities for short periods. An extensive research effort would be necessary to locate these data bases and conduct a more thorough frequency analysis.

The Jornada Range Experiment Station record of 91 years was almost complete with only 13 years missing one or more months of record. This station is located on the lower portion of the peneplains draining the San Andres Mountains about 10 miles east of the river at an elevation of 4266 ft, about 200 ft above the river. The record is representative of general storms over the valley but may not be representative of local convective summer thunderstorms for the steep arroyos on the western side of the valley. The daily rainfall data base for the 91 years of record was acquired for use in this investigation. From a statistical standpoint this gage record has approximately a 60% percent chance of containing the 100-yr 24-hour storm. The record was re-organized by months and then sorted for the maximum daily rainfall for each year. It is probable that some of the annual maximum daily storms may have extended past midnight and thereby the 24-hour storm could have higher than recorded on a daily basis, but the record is long enough to be able to determine if the NOAA 100-year, 24-hr point rainfall estimate is reasonable. The maximum daily record rainfall was 3.48 inches in 1959. This is approaching the 100-yr, 24-hr storm point rainfall reported by the Corps of 3.80 inches.

Using the FREQPLOT program developed by DeRoulhac (1990), a King's Table of four probability distributions were reviewed including the extreme value, log extreme value, log normal and log Pearson type III. The King's Table provides method to review the best probability distribution (linear distribution) of the four choices. Table A1 displays the frequency results for the four distributions. A calculated skew coefficient of -0.3 was used in the analysis. Visually, the extreme value and the log Pearson Type III distribute provide the best 'fit' to the data. The extreme value is the distribution normally applied by the National Weather Service (NWS) for rainfall analyses.

Table A1. Point Rainfall Frequency for the Jornada Gage

Return Period (yrs)	Extreme Value (inches)	Log Extreme Value (inches)	Log Normal (inches)	Log Pearson III (inches)
2	1.12	1.03	1.12	1.10
5	1.57	1.51	1.59	1.58
10	1.88	1.95	1.88	1.41
25	2.26	2.70	2.23	2.33
50	2.54	3.43	2.48	2.66
100	2.82	4.35	2.72	2.99

The 100-yr rainfall is estimated at approximately 3.0 inches for the Jornada gage 91 year record. This is reasonably close to the NOAA atlas value of 3.8 inches conceding that some of the daily records may have underestimated the 24 hour storm. In addition, the maximum daily value of 3.4 inches in 1959 for the 91 year record also confirms the reasonableness of the NOAA estimate. The NOAA Atlas II, Volume IV shows that the 100-yr, 24 hour precipitation contour for the Jornada gage area north of Las Cruces is 3.4 inches. It was concluded that the 3.8 inch point rainfall for the 100-yr, 24 hour storm is appropriate.

A similar analysis was performed for the Hillsboro Raingage with a partial record extending from 1946 to 2004. This data base was punctuated by large gaps of missing data and it is unclear if these were periods of no rainfall or just lost data. It was concluded that there was sufficient data representing potential convective storms to proceed with the frequency analysis. The results are shown in Table 1 page 4 of the report. The Hillsboro gage further justifies the use of 3.8 inches total rainfall for the 100-yr, 24 hour storm.

Local Convective Thunderstorms Generate Severe Arroyo Flooding

The Las Cruces, New Mexico storm of August 29-30, 1935 had a maximum rainfall of 8 inches in 12 hours, with an average rainfall of 6.7 inches over an area of 38 mi^2. Generally the NOAA atlas tends to be a poor indicator of local convective thunderstorms and under predicts the point rainfall for thunderstorms. This is partially due to the fact that rain gages did not capture the intense rainfall that might occur in a small arroyo watershed. Historically, the distribution of rain gages was so sparse that convective thunderstorms in remote locations were missed entirely. Short duration thunderstorms of 1 or 2 hours, may produce high peak discharges that could exceed 100-yr, 24-hr peak discharge in the Rio Grande, but generally these local storms have limited volumes and may quickly attenuate in the mild slope of the Rio Grande channel.

Design rainfall criteria that are based on the analysis of rain gage data will yield more reliable flood estimates than either the NOAA Atlas or regional criteria such as the USGS 100-yr flood regression relationships (Sabol and Stevens, 1990). An assessment of hourly data rain gage data was undertaken for the Hillsboro Gage (previously discussed) to determine the 100-yr, 1-hr, 2-hr and 6-hr storms. The results are shown in Table 1 in the report. To check whether a shorter duration storms would produce a higher peak discharge or higher volume runoff using the same loss rates and unit hydrograph methods applied in the 1996 study, the Trujillo Arroyo HEC-1 data file was used. A 2-hr and 6-hr storm distribution provided in the Flood Control District of Maricopa County Drainage Manual (1991) as representing semi-arid southwestern watersheds was used to check runoff characteristics. Five 6-hour storm patterns are presented in the Maricopa County Drainage Manual. Storm pattern number 4 was applied to Trujillo Arroyo based on the watershed area. The depth area reduction values for Trujillo Arroyo watershed area (52.9 sq. mi.) shown in Table 2 (report page 4) were estimated from Figure 12 of the NOAA Technical Memorandum NWS Hydro 40 (1984). The Corps used the same document to derive the depth area reduction values used for the 100-yr, 24-hr storm.

Table 2 confirms that the 24-hr general storm over the basin will produce the highest peak discharge and largest storm volume from one of the major arroyos in the Rio Grande Canalization Reach. Table 2 is based on a short duration, high intensity, convective thunderstorms using the Corps total point rainfall estimates, the Snyder Unit Hydrograph method and the Corps rainfall loss estimates. The Maricopa County 100-yr 2-hr and 6-hr distributions represent intense storms with high peak discharge, but they did not exceed the 24-hr storm peak discharge. It is possible that a different loss rate function such as Green-Ampt infiltration instead of the Snyder Unit Hydrograph Method would generate a higher peak discharge, but examining other rainfall loss methods is beyond the scope of work for this project. It was concluded that the selection of the 100-yr 24-hr general storm as the design storm for the arroyo flooding is appropriate in comparison with the Corps selected 2-hr and 6-hr storm data.

Application of the HEC-1 model unit hydrograph method

The Snyder Unit hydrograph method was applied in the Corps HEC-1 rainfall/runoff simulations to generate the contributing arroyo flood hydrographs to the Rio Grande. The Snyder unit hydrograph method relates the computed hydrograph characteristics (peak discharge, basin lag time, hydrograph base time, and duration at specified discharges) to the watershed parameters. The unit hydrograph is the basin surface discharge resulting from a unit (1 inch) rainfall excess applied uniformly over the drainage in one hour. The primary assumptions associated with applying this method is that the results are not storm sensitive and that the rainfall runoff can be combined linearly. Snyder's unit hydrograph method computes the peak discharge, time to peak and durations at 50% and 75% of the peak discharge. HEC-1 then uses the Clark unit hydrograph method in a trial and error procedure to complete the rest of the flood hydrograph. Snyder's unit hydrograph method was based on data from the Eastern United States where the watersheds tend to be larger, the basin slopes milder and the time to peak slower. The concern is that the Snyder Unit Hydrograph Method may tend to underpredict the peak discharge because the longer routing times that may not be representative of steep slope, poorly vegetated western semi-arid watersheds with imperious areas. In steep arroyo watersheds, the hydrographs tend to have a fast rising, frontal wave peak discharge.

The application of the Snyder's unit hydrograph method depends on the selection of two coefficients C_t (a time lag coefficient representing basin slopes and storage) and C_p (a peak discharge coefficient accounting for storage and various runoff conditions). Values of C_t can range from 0.2 for steep slopes to 8.0 for very flat areas. C_p can range from 0.4 to 0.8 depending on storage capacity of the basin (Hoggan, 1989). C_t ranged from 0.2 to 0.72 and C_p ranged from 0.61 to 0.67 in the Corps (1993-6) Study Documentation. The selection of these parameters was not discussed in the Corps Study Documentation. The values of C_t were extracted from a relationship between C_t and slope (Plate 3 in the Study Documentation). An additional C_t versus slope figure (Plate B-3 in the Study Documentation) provided a range of C_t values for watersheds throughout New Mexico and Arizona. To test the sensitivity of these parameters, a reasonable range of values was selected from Plate 3B in the Study Documentation as shown in Table A2. Trujillo Arroyo in the RGCP reach was selected as a test watershed.

Table A2. Snyder's Unit Hydrograph Parameters

Basin	Area (mi^2)	Slope	C_t	C_p
Trujillo Arroyo, RGCP	52.9	0.0160	0.60	0.61
Skunk Creek near Phoenix, AZ	64.6	0.0193	0.44	-
Alamogordo Creek, Pecos River, NM	67.0	0.0120	0.54	0.79
New River near Rock Springs, AZ	67.3	0.0267	0.52	-

The basins in Table A2 all have about the same area, but the two of the basin slopes are steeper and one is milder. All three example basins have C_t values that are lower and will reduce the time to peak. For all eleven basins listed on Plate 3 in the Study Documentation with drainage areas ranging from less than one mile to over 6,000 miles, all the storage coefficients are higher than those used in the RGCP study, ranging from 0.79 to 0.84 (> 0.61 as selected for the Trujillo basin). A higher C_p values would have the effect of increasing the discharge.

A sensitivity test was conducted on the Trujillo Arroyo basin 100-yr, 24-hr design storm. The selected test range of the C_t coefficient was from 0.44 (the lowest value in Table 3) to 0.72 (20% higher than the original C_t = 0.60). For the C_p coefficient, the test range was 20% higher and lower than the Corps value of 0.61. The results are shown in Table A3.

Table A3. Sensitivity Test of Snyder's Unit Hydrograph Parameters for the 100-yr, 24-hr Storm

C_t	t_p	C_p	Q_p (cfs)	Volume (af)	Time to Peak (hrs)	Unit Peak Discharge (cfs/mi^2)
0.60[1]	3.75	0.61	5,820	2,840	9.67	110
0.44	2.75	0.48	6,170	2,840	8.67	116
0.72	4.49	0.72	5,730	2,840	10.33	108
0.44	2.75	0.72	9,290	2,840	8.67	176
0.72	4.49	0.48	3,840	2,840	10.33	73
0.60	3.75	0.48	4,560	2,840	9.50	86
0.60	3.75	0.72	6,850	2,840	9.50	129
0.50[2]	3.12	0.70	8,000	2,840	9.00	151

[1]Original parameters and results used by the Corps
[2]Suggested values

The t_p and C_p parameters in the sensitivity test simulations were varied as follows: low-low, high-low, low-high, high-low, original-low and original-high. The time to peak in table A3 is solely a function of the C_t variable. It is also noted that since the rainfall loss function parameters did not change, there was no variation in the flood hydrograph volume at the arroyo mouth. The most important result of this sensitivity analysis is the variation in the peak discharge. A 20% increase in the C_p coefficient results in an 18% change in the peak discharge, a difference of 1,030 cfs. Based on the data presented in Table A2 (Plate B-3 in the Study Documentation), it appears that the value of C_t should be about 0.50. All the C_p values in Plate B-3 exceed 0.79, so a selected value of $C_p = 0.72$ also appears to be reasonable. Applying these changes in these two coefficients produces the results shown in the last line in Table A3 (shaded line). The revised peak discharge of 8,000 cfs represents an increase of 37% in the peak discharge.

Rainfall Loss Estimate and Excess Runoff

Excess runoff was predicted by the HEC-1 model for the subbasin areas on the order of 34 to 39 percent of the total rainfall after depth area reduction. This was based on an initial loss (abstraction) of 0.90 inches and 0.20 inches per hour uniform loss rate. The uniform loss rate of 0.20 inches was noted by the Corps to be widely used in hydrologic studies in the southern New Mexico area. The 0.90 inch initial loss was calibrated to two regional equations; one developed by the USGS (1986) and the other developed by the Albuquerque District (1990). The RGCP watershed is located with the study region for these two regional analyses of stream gage records. The equations are presented in the Corps RGCP 1996 Hydrology and Hydraulics Report. The drainage areas of the original stream gage records used in developing the Corps' Report were widely varying and in the case of the USGS regression equation, the 22 stations had watershed areas ranging from 0.2 to 2,830 square miles. The USGS equation is a power function of drainage area whereas the Corps' equation is based on drainage area, slope and total rainfall for the 50-yr, 6-hr storm. The shorter duration storms may have high unit peak discharge.

In the RGCP Hydrology and Hydraulics Report, the Corps noted that its regression equation computes increasing peak discharge with decreasing slope for a given watershed, thus overpredicting peak discharges for large basins with flatter slopes. In general, log regressed power equations, tend to be distorted by large data groups near the plot origin as well as by the bias generated by least squared fit regressions that occur when variables extend over the several orders of magnitude. If most of the station basin areas are small and clustered near the origin, small variations in the regressed line slope can result in significant over- or under-prediction of peak discharge for large basin areas. The predicted peak discharge using the two methodologies as shown by Table 1-9 page 1-17 of the RGCP Hydrology and Hydraulics Report indicates that the USGS regression equation more than doubles the peak discharge of the Corps regression equation for small basins but the USGS equation predicts less than 50% of the peak discharge computed by the Corps regression for large basins. The Corps indicated in the RGCP 1996 Hydrology and Hydraulics Report that a reasonable correlation was observed between the HEC-1 predicted peak discharges and the USGS regression equation and that the correlation was consistent over the entire range of basin areas.

Upon further review, it is noted that the USGS regression equation for 100-yr peak discharges (cfs):

$$Q_p = 932.0 * A^{0.48}$$ where A is the drainage area in square miles;

predicts about 1,000 cfs/mi^2 for basins less than two square miles. This is typical for small, steep watersheds with impervious areas. Conversely, the Corps' regression equation predicts on the order of 500 cfs/mi^2. For larger watersheds, the USGS regression, is predicting less than 100 cfs/mi^2. Of the 22 gaged stations used in the USGS regression (Waltemeyer, 1986);

3 gages had drainage areas > 2,000 mi^2

4 gages: 1,000 mi^2 < drainage area < 2,000 mi^2

7 gages: 100 mi^2 < drainage area < 1,000 mi^2

3 gages: 1 mi^2 < drainage area < 100 mi^2

5 gages had drainage areas < 1 mi^2

It is unclear if the peak discharge-drainage area relationship is controlled by either the small or large drainages but at closer examination, six of large drainage gages represent the same two rivers (San Francisco and Gila in Western New Mexico and Eastern Arizona) whose runoff and potential infrequent flooding may occur in the winter months. The same is true for the Blue River near Clifton, Arizona (506 mi^2), the Mimbres near Faywood, New Mexico (440 mi^2) and Whitewater Draw near Douglas, Arizona (1,023 mi^2). There appears to be a mixed population of large drainages with flood events that occur with winter storms and small drainages with summer thunderstorm floods. In addition, there may duplicity in the data for the large basins. The USGS relationship may predict reasonable 100-yr peak discharges for small drainages and underpredict the peak discharges for the large drainage areas. Conversely, the Corps regression may underpredict the 100-yr peak discharge for the small drainages and slightly overpredict for the large drainages. Figure 5 of the Corps RGCP Hydrology and Hydraulics report is reproduced below (Figure 2) with a hand drawn curve that might represent a reasonable compromise for the calibration of the HEC-1 model to subbasins in the RGCP reach since the majority of the basins are less than 100 square miles. Based on the line in this figure, the Trujillo Arroyo basin of 52.9 square miles should have a 100-yr peak discharge of 7,940 cfs, approximately the computed peak discharge shown on the last line of Table A3 using the best estimate of the selected parameters.

In the RGCP Study Documentation, a table is presented (Table 4, page 13; as identified as the Caballo Dam Gaging Station, February 1965) that lists a series of historical storms and estimated storm runoffs for the Rio Grande Basin between Caballo Dam and Leasburg. The percent runoff for seven storms ranged from 7 to 41 percent with six of the storms having less than 25 percent runoff. Most of these storms were listed as two day duration with 2.5 inches or less total rainfall. The rainfall intensity was likely very low and resulted in a low percent runoff. One storm was listed as a 12 hour rainfall event with a 41% runoff which is comparable to that reported in Table 2 (main report) for Trujillo Canyon.

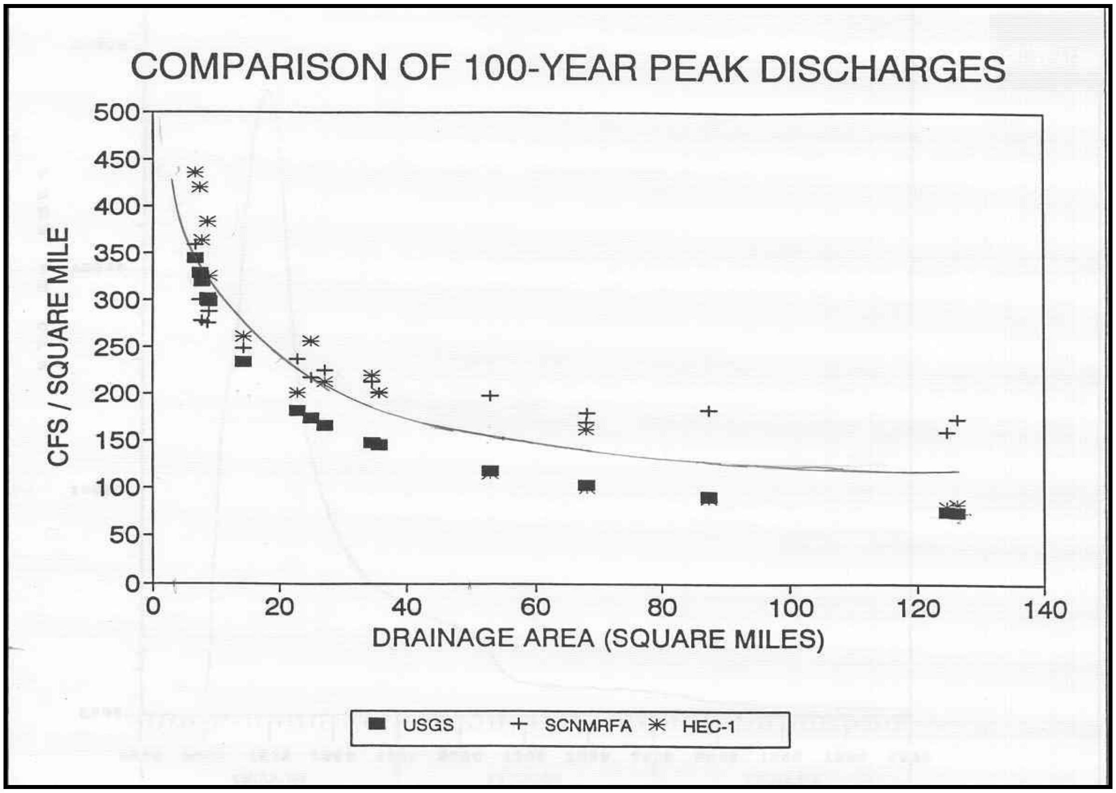

Figure A1. 100-yr unit Peak Discharge for the USGS and the Corps' Regional Relationships

Infiltration rates and initial losses were published in the Las Cruces Local Protection Project Design Memorandum No. 1 by the Albuquerque District in 1964 and referenced in the Corps RGCP 1996 Study Documentation. Initial losses were established for the Las Cruces Local Protection Project by calibrating a Los Cruces Arroyo model to its frequency curve. The hourly loss was listed as 0.20 inches per hour and that value was used in the 1996 RGCP project. Initial loses were 0.70 inches for the 10-yr flood, 0.40 inches for the 50-yr flood and 0.0 for both the 100-yr and 500-yr floods. It the El Paso Local Protection Project, Design Memorandum No. 9 by the Albuquerque District in 1972 (also referenced in the RGCP Study Documentation), it was reported that losses varied with topography and ranged from 0.50 inches initial loss and 0.25 inches uniform loss for mountainous areas to 0.80 inches initial loss and 0.40 inch uniform loss for flat areas. It was noted in the Corps 1996 Hydrology and Hydraulics Analyses Report that a constant loss of 0.20 inches per hour has been used extensively for hydrologic studies in the southern New Mexico. It also stated that the 0.90 initial loss was calibrated for the peak discharges with the USGS regional regression equation. If it is assumed that the USGS equation is under predicting the 100-yr peaks, then adjustments should be made to the initial loss. When the initial loss rate is reduced from 0.90 to 0.70 inches while maintaining the uniform loss of 0.20 inches, the results from Trujillo Arroyo for the 100-year flood show an increase in peak discharge (Table A4).

Table A4. Trujillo Arroyo HEC-1 100-yr, 24-hr Storm Rainfall Runoff Simulation Results with Modified Initial Loss = 0.70								
Initial Loss (in)	Total Point Rainfall (in)	Depth Area Reduction	Applied Rainfall (in)	Loss (in)	Excess Rainfall (in)	Runoff Volume (af)	Peak Q (cfs)	Time to Peak (hrs)
0.90[1]	3.80	0.77	2.93	1.92	1.01 (35%)	2,840	5,820	9.67
0.70	3.80	0.77	2.93	1.74	1.19 (41 %)	3,350	6,840	9.50
0.40	3.80	0.77	2.93	1.62	1.31 (45%)	3,680	7,520	9.50
0.70[2]	3.80	0.77	2.93	1.74	1.19 (41%)	3,350	9,420	9.00

[1]Original parameters and results used by the Corps
[2]Suggested values including C_p = 0.70 and t_p = 3.12 as applied in Table 4.

Table A4 indicates that lowering the initial loss to 0.7 inches increases the runoff volume and peak discharge by 18%. It did not appreciably affect the time to peak. When combined with the previously suggested revisions to the Snyder Unit Hydrograph coefficients, the peak discharge increases from 8,000 cfs to 9,420 cfs with no change to the time to peak.

Volume Analysis

A volume frequency analysis was conducted for the Corps 1996 Hydrologic and Hydraulic study based on normal volume per square mile as a function of drainage area using large subbasins in the Rio Puerco and Rio Salado drainages. These tributary areas to the upstream Rio Grande ranged from 400 to 6,000 square miles and the resulting relationship did not correlate very well with the smaller area RGCP subbasin runoff volume. The Corps' 1996 Report then states that additional smaller watersheds were considered and the HEC-1 model generated volumes compared more favorably. There is no supporting data in the Corps' report and limited information in the Corps' 1996 Study Documentation. It appears that no modifications to the HEC-1 data files were made as a result of this analysis.

The runoff volume is a function of the assumed loss rate. The proposed adjustments in this study (reduced the initial loss rate from 0.90 inches to 0.70 inches) were made to the HEC-1 files based on discussions with the Corps and IBWC. This increased the runoff volume as shown in Table A4. For the various return period events, the initial loss rates presented in the Corps Sedimentation Analysis From the Rio Grande Tributary Basins Volume 3 (1996) were reduced as percentage of the reduction from 0.9 to 0.7 inches as shown in Table A5.

Table A5. RGCP Tributary Return Period Storm Initial Loss Rates				
Return Period (yrs)	> 40 sq. mi. drainage		< 40 sq. mi. drainage	
	Corps' Values	Revised	Corps' Values	Revised
2	0.5	0.39	0.6	0.47
5	0.7	0.54	0.8	0.62
10	0.8	0.62	0.9	0.7
25	0.9	0.7	0.9	0.7
50	0.9	0.7	0.9	0.7
100	0.9	0.7	0.9	0.7

Appendix B

Sediment Supply Analyses
and Documentation

Introduction

The sediment supply to the RGCP is limited to arroyo tributary inflow because of the upstream mainstem Caballo Reservoir on the river. In addition, a number of arroyo tributaries have flood NRCS retention structures that also store sediment. This has created a long term sediment deficit in the RGCP system. It is unclear whether the canalization reach is in sediment transport equilibrium with respect to its average annual sediment load. Past dredging activities in the channel in the last ten years have initiated channel incision that has extended over miles. In the absence of the historical sand load, the channel bed is armored by the coarse sediment in short reaches where there are substantial cobble and gravel sources from local hillslopes or arroyos. The long term channel morphology response to IBWC maintenance activities is largely a function of the limited sediment supply. The success or failure of channel maintenance and river restoration activities will depend on the available sand-size sediment load.

The purpose of the Corps 'Sediment Analysis' study was to quantify the sediment yield from the twenty major arroyo basins located between Percha Diversion Dam downstream of Caballo and American Diversion Dam in El Paso, Texas. The results from the sediment yield analyses of the twenty arroyo basins was then used to develop a predictive equation to estimate the sediment loading from the remaining tributary basins in the 922 square mile watershed. The challenge was to estimate sediment loading from steep arroyos with high concentrations of fine sediment using existing sediment transport technology that has been basically developed for mild sloped river systems.

The computation of the total sediment load for the twenty study basins presented in the Volume 3 report was used to generate an instantaneous sediment load rate for each peak discharge for the various return period hydrographs for each of the study basins. This constituted the local sediment inflow to a HEC-6 sediment transport model. The "Scour and Deposition Analysis of the Rio Grande" which constitutes Volume 4 of the Corps July 1996 Report, discusses the application of the HEC-6 sediment transport model to simulate the scour and deposition analysis for a low flow and high flow period and a 100-year storm for existing and future conditions. For future applications of the FLO-2D model to the RGCP with a mobile bed, the sediment supply to the RGCP reported in Volume 3 was evaluated

Tributary Hydrology and Hydraulics

The flood hydrology for the sediment yield analysis was based on the Corps' 1996 HEC-1 model prepared for the IBWC Rio Grande Canalization Improvement Project. The Corps' HEC-1 modeling results were used to determine the project design flood peak discharges at selected locations in the reach from Caballo Dam to American Dam. For the sediment analysis, the point rainfall depths, depth area reduction and the rainfall distribution were provided by the Corps. Some of the Corps basins delineations were adjusted to identify some of the small arroyo basins that were lumped together in the HEC-1 model. Essentially the HEC-1 model parameters used in the Corps's 1996 hydrology study were adopted for the sediment analysis including the Snyder's Unit Hydrograph runoff method parameters and the initial and uniform losses. The initial loss was varied according to return period and basin and area. The same hydrology issues raised in the Tetra Tech (2004) Hydrology Review are also valid for the hydrology used in the sediment analysis including:

- Rainfall duration and intensity
- Snyder unit hydrograph parameter sensitivity and initial loss estimate
- Channel routing

While the 6-hr duration storm was mentioned in the report, only the 24-hr duration results are presented. The concern was that the 2-hr or 6-hr storm may produce a higher peak discharge in response to higher rainfall intensity. A comparison of the 100-yr 2-hr and 6-hr storms was completed in the hydrology analysis and it was determined that the 24-hr storm generated the highest peak and volumes.

Several parameter adjustments were made to the HEC-1 model including a decrease in the Snyder's unit hydrograph time to peak coefficient from 0.6 to 0.5, an increase in the storage coefficient from 0.61 to 0.7 and a decrease in the initial loss from 0.9 inches to 0.7 inches. Trujillo Canyon watershed was used as a test basin to determine the potential effect of the three parameter changes on the HEC-1 runoff simulation. The parameter modifications increased the peak discharge from 5,820 cfs to 9,420 cfs, increased the runoff volume from 2,840 af to 3,350 af (18% increase) and reduced the time to peak from 9.67 to 9.00 hours. These parameter changes would be applied to all the test basins and consequently increased runoff would be computed for each basin.

With respect to channel routing, the application of the HEC-2 model to compute flow hydraulics in the tributary channels was limited to study reaches with surveyed cross sections. The average reach for the HEC-2 application in the 20 study basins was 731 ft, with a range from 371 ft to 1,366 ft. These relatively short reaches of channel would probably not significantly effect the hydrograph shape and timing. The assigned arroyo channel n-values were exceedingly low ranging from 0.025 to 0.040. For steep arroyo channels with vegetation, exposed bedrock, and numerous channel constrictions and expansions, the selected n-values are considered to be approximately half of the actual hydraulic resistance. The overbank roughness values ranged from 0.045 to 0.100. While the assigned overland n-values are more realistic, the preferred range of n-values should be 0.065 to 0.125. The effect of low n-values is to predict high velocities that may be supercritical and thus may overestimate the sediment delivery to the Rio Grande.

Sediment Yield Estimates

The arroyo sediment supply was divided into bed material load and wash load. The wash load or overland sediment yield from the upper watersheds was estimated by the Modified Soil Loss Equation (MUSLE). There was no sensitivity analysis performed for the MUSLE sediment yield (wash load) analysis. There are only four parameters that were considered to be variable in the analysis because the support practice factor was assumed to be 1.0. The LS topographic factor representing the watershed slope and flow path length to the point of concentration were estimated from available mapping and should be reasonably accurate. The remaining factors, the storm runoff energy factors, soil erodibility factor and cover and management factor are variable and will impact the potential sediment load.

The storm runoff energy factor R_w is computed from a power regression equation that is a function of runoff volume V and peak discharge q_p:

$$R_w = a(Vq_p)^b$$

where a is a coefficient (95) and b is an exponent (0.56) as selected for the study. The selection of the exponent and coefficient was based on experimental watersheds and arroyos near Albuquerque. The potential range of these values would require further research. In the Tetra Tech review of the Volume 2 Hydrology and Hydraulics Report, a sensitivity analysis was performed in which a range runoff volume and peak discharge was determined by varying the rainfall runoff Snyder unit hydrograph parameters and loss coefficients. For Trujillo Canyon, the runoff volume ranged from 2,840 af to 3,350 af and the peak discharge ranged from 5,820 cfs to 9,420 cfs. The R_w range for Trujillo Canyon using the selected coefficient (95) and exponent (0.56) is 1,046,980 to 1,504,890. This is a relatively wide variation of about 50% indicating the need for a sensitivity analysis. Base on the original hydrology, the sediment yield could be underpredicted. This same approach related to the hydrology could be applied to each of the twenty test basins.

In Appendix C the watershed soil erodibility factors K were weighted adjusted based soil unit mapping and ranged from 0.09 to 0.22. One basin was assigned a K factor of 0.01 (Buckle Bar watershed). The average for the 22 basins was 0.15 and the standard deviation was 0.033. The assigned value for Placitas Canyon was 0.18 and the K range that could be applied is 0.147 to 0.213. This range results in a about a 44% variation in the K parameter. The cover and management factor C as determined by RTI varied from 0.071 to 0.313. A significant amount of effort was expended to provide watershed detail to the C factor.

The sediment load results of the MUSLE computation were assumed to constitute the wash load from upper watershed to the short reach of the arroyo channel where the bed material load was calculated. The sediment transport capacity from this short channel reach constitutes the bed material load to the Rio Grande. The bed material in this reach is assumed to be unlimited. This is fair assumption given that the bed load supply from the upstream steep watershed probably exceeds the sediment transport capacity.

Bed Material Load Prediction

The bed material load was evaluated using a combination of three equations, Zeller-Fullerton equation, the Meyer-Peter, Muller and Woo (MPM-W) method and Meyer-Peter, Muller and Einstein (MPM-E) equation. A short discussion of each equation follows with emphasis on it's applicability to the range of conditions found in the arroyo watersheds.

The Zeller-Fullerton equation is a multiple regression sediment transport equation for a range of channel bed and alluvial floodplain conditions. The data base and variable range used in the development of the equation was from arid regions in Pima County Arizona. This empirical equation is a computer generated solution of the Meyer-Peter, Muller bed-load equation combined with Einstein's suspended load to generate a bed material load (Zeller and Fullerton, 1983). The bed material discharge q_s is calculated in cfs per unit width as follows:

$$q_s = 0.0064 \ n^{1.77} V^{4.32} G^{0.45} d^{-0.30} D_{50}^{-0.61}$$

where n is Manning's roughness coefficient, V is the mean velocity, G is the gradation coefficient, d is the hydraulic depth and D_{50} is the median sediment diameter. All units in this equation are in the ft-lb-sec system except D_{50}, which is in millimeters. For a range of bed material from 0.1 mm to 5.0 mm and a gradation coefficient from 1.0 to 4.0, Julien (1995) reported that this equation should be accurate with 10% of the combined Meyer-Peter Muller and Einstein equations. The Zeller-Fullerton equation assumes that all sediment sizes are available for transport (no armoring). The original Einstein method is assumed to work best when the bedload constitutes a significant portion of the total load (Yang, 1996). This equation should be relatively accurate for the most of conditions found in the arroyo basins.

For computing the bed material load in steep sloped sand bed channels such as arroyos, washes and alluvial fans, Mussetter, et al. (1994) linked Woo's relationship for computing the suspended sediment concentration with the Meyer-Peter-Mueller bed-load equation. Woo et al. (1988) developed an equation to account for the variation in fluid properties associated with high sediment concentration. By estimating the bed material transport capacity for a range of hydraulic and bed conditions typical of the Albuquerque, New Mexico area, Mussetter et al. (1994) derived a multiple regression relationship to compute the bed material load as a function of velocity, depth, slope, sediment size and concentration of fine sediment. The equation requires estimates of exponents and a coefficient and is applicable for velocities up to 20 fps (6 mps), a bed slope < 0.04, a D_{50} < 4.0 mm, and a sediment concentration of less than 60,000 ppm. As shown in Table 1 below, a number of the watersheds have a sediment wash load that exceeds this value. This equation provides a method for estimating high bed material load in steep, sand bed channels that are beyond the conditions for which the other sediment transport equations are applicable. This equation should be applicable to most of the arroyo basins of the RGCP watershed.

The Meyer-Peter Muller and Einstein equation as presented and used in the report is simply the Meyer-Peter Muller bedload equation. It was based on experiments with sand and gravel in 1948. It is noted that the equation provides satisfactory results when the slope is less than 0.001 and the bed material is fine to medium sand. It can give significant discrepancy for coarse bed material (Simons and Senturk, 1976). While typically the MPM bedload equation is used for coarse sediment, the coefficient can be adjusted to reflect the bed material size found in most of the arroyo watersheds. The equation was not modified for the bed material size in this analysis.

The equations were applied in the 1996 Corps study following manner:

- The Zeller-Fullerton equation was applied to arroyo channel where the data was appropriate.
- The MPM-W equation was used for arroyos where the sediment gradation was out of the range of the Zeller-Fullerton equation and D_{50} was greater than 1.5 mm.
- The MPM-E equation was applied to the arroyos with coarse bed material.

The bed material load results were adjusted for the sediment size fraction that the above equations were applicable and for high sediment concentration using the Colby's procedure. To apply Colby's modification for high sediment concentrations, the sediment concentration of the wash load computed by MUSLE was used. The total sediment load to the Rio Grande was the sum of the bed material load and the wash load (computed by MUSLE).

Overall this is an excellent approach to estimate the potential sediment loading to the Rio Grande. The following observations were made with respect to the application of this multiple equation method:

- The hydraulic variables in the sediment transport equation included average depth and average velocity, bed shear stress, and top width. These were computed with the steady flow, one-dimensional HEC-2 model using six to eight cross sections. To account for unsteady nature of the flood hydrograph, the rising limb of the hydrograph was discretized into six increments. The hydraulic results from these increments were then used in a regression analysis to determine the hydraulic variables as power function of discharge. The regression statistics were not presented in the Volume 2 report. The power regression relationships were then used to calculate the hydraulic variables for the entire discretized hydrograph using a 10 minute increment of steady flow. The model results were analyzed and the supercritical hydraulic results from fifteen arroyos and the subcritical hydraulics results from five arroyos were used in the sediment computations. The supercritical flow results were in part the consequence of selecting low n-values.

- It is widely recognized that supercritical flow in alluvial channels is rare and limited to localized reaches. This is because flow acceleration to critical depth entrains more sediment and reduces the available flow energy. Often critical flow is used a limiting hydraulic condition on alluvial fans or steep arroyos. Supercritical flow in southwestern arroyos is possible in locations where bedrock is encountered. The additional energy loss associated with sediment entrainment can be accounted for by increasing the n-value. The ramification of predicting the high flow velocities associated with an assumption of supercritical flow is to overpredict the sediment transport.

- The Woo (1988) equation was developed to account for the changes in the fluid properties associated with increasing sediment concentrations. It is supposed to converge to the same solution as the 1937 Rouse equation for low sediment concentrations. Linking the Woo equation with the Meyer-Peter, Muller bedload equation provides a method to compute bed material load in channels with high concentrations of suspended sediment. This method was reported to provide better results in steep arroyos in the Albuquerque area. The Corps 1996 report presents the range of conditions for which the MPF-Woo equation is valid. This includes concentrations of fine sediment up to 60,000 ppm. The application of Colby's method for computing total bed material load based on the MPM-W equation is essentially double accounting for the effects of the high concentrations of fine sediment. Colby's factors for increasing the bed material load for the effects of fine sediment concentration should not be applied to the basins where the MPM-W was used to compute the bed material load. MPM-W equation was applied to the Green Arroyo (D_{50} = 6.50 mm) and the Sub Area 106B Arroyo (D_{50} = 7.4 mm). In both these cases, the median sediment size exceeds the suggested range for the application of the MPM-W equation (D_{50} = 4.0 mm). The Volume 3 report indicates that the MPM-Woo equation should be used if the gradation data is significantly out of range for the Zeller-Fullerton equation and the D_{50} is greater 1.5 mm. In fact, the suggested sediment size range for the Zeller-Fullerton equation is 0.2 mm < D_{50} < 4.0 mm and MPM-W should be applied in steep arroyos where the suspended fine sediment load is a significant portion of the total bed material load.

- The gradation adjustments for applying the Zeller-Fullerton equation to the representative size fraction of the bed material were appropriate. The MPM bedload equation can then be applied to the remaining coarse size fraction. The Shields incipient motion criteria was applied to the coarse size fraction to determine what portion of the bed material would be in motion for the range of hydraulic conditions. This is also a good approach when using the steady flow, one-dimensional model. The Shields' parameter has a general range of 0.02 to 0.07 with a typical value of 0.047 for sand. Julien (1995) reported a range of 0.029 for coarse sand to 0.054 for large cobbles and larger sediment particles. Field data on gravel and cobble bed channels (Parker and Klingeman, 1982; and Andrews, 1983; Andrews and Parker, 1987) demonstrated that the critical shear stress parameter varies with particle size. The critical shear stress parameter should be adjusted for particle sediment size in both the incipient motion analysis and in the MPM bedload equation. While the Corps 1996 bed material analysis to separate the coarse size fraction that is outside the range of applicability of the Zeller-Fullerton equation is a good approach, the Shields' parameter for incipient motion of the coarse sediment should also be appropriately selected.

Total Sediment Supply to the Rio Grande Canalization Project

The Corps Volume 3 report recognized that the average annual sediment supply estimates to the Rio Grande Canalization Project reach were high. In the report summary and conclusions, it is stated that "(t)he comparison of the RTI average annual sediment yield estimates to the SCS average annual reservoir resurvey data indicates that the RTI estimates may be high for drainage areas less than 25 square miles. Consequently the total sediment yield prediction equations may produce larger than expected sediment yield values for drainage areas less than 25 square miles." In addition, the report states that, "(e)valuation of the average annual total sediment load results…indicate some extremely large values for several of the smaller basins…". It was noted that "(s)ome of the other smaller basins also have relatively large values greater than 2 ac-ft/sq mi." A close inspection of the study data and results presented in Table B1 reveals the following:

- Sediment concentration by volumes for the 100-year total sediment load exceeded mudflow criteria for some of the analyzed watersheds.
- Exceedingly high sediment concentrations by volume for the bed material load for some of the arroyo watersheds.
- Seven out of the twenty-two analyzed watershed that have very high annual sediment yield per square mile (> 1.50 af/mi^2/yr).
- Average sediment yield for 19 local basins of 0.48 af/mi^2/yr with a standard deviation of 0.25 af/mi^2/yr based analyses of NRCS reservoirs (see Table 5 in the report).

Table B2 indicates that the predicted sediment yield exceeds 1.5 af/mi^2/yr for seven of the arroyo study watersheds. Five of these watersheds have predicted sediment yields that exceed 2 af/mi^2/yr. In the Pacific Southwest Inter-Agency Committee Report (1968) on “Factors Affecting Sediment Yield in the Pacific Southwest Area,” sediment yields were divided into five classes of average annual yield. Based on this classification, sediment yields less than 0.2 af/mi^2/yr were considered to be low, sediment yields from 0.2 to 0.5 af/mi^2/yr were moderate, and sediment yields from 0.5 to 1.0 af/mi^2/yr were moderately high. Those sediment yields ranging from 1.0 to 3.0 af/mi^2/yr were classified as high and sediment yields greater 3.0 af/mi^2/yr were ranked as very high. Based on this classification nine of the twenty basins were predicted to have a high or very high sediment yield.

Typically bed material sediment concentration in river systems is on the order of 1 to 3 percent concentration by volume. For steep arroyo watershed, bed material sediment concentrations can be as be as high as 7 to 10 percent by volume. The Rio Puerco is recognized as one of the world’s highest sediment transport streams with sediment concentrations approaching 200,000 ppm (~ 8.6% by volume). Most of the sediment load is related to the tremendous quantities of fine sediment comprising the wash load. Colby’s method for adjusting the bed material load on the basis of high concentrations of fine sediment accounts for concentrations by weight up to 200,000 ppm (8.6 percent concentration by volume). In Table 1, estimated wash load concentrations were in excess of 200,000 ppm for three of the small study watersheds for the 100-year flood. While these wash low concentrations are very high, they are not impossible to attain. Conversely, bed material concentrations in excess of 5 per cent by volume are predicted in Table B1. Two of the basins in Table 1 have bed material concentrations in excess of 30 percent by volume (see red highlight). Three of the total loads have sediment concentrations in excess of 20 percent by volume.

Table B1. Total Sediment Load for the 100-year Flood[1]

Arroyo	Drainage Area (sq mi)	Runoff Volume (af)	Wash Load Ave Conc (ppm)	Wash Load Ave Conc by Vol[2]	Bed Material Load Volume (af)	Bed Material Load Conc. by Volume[2]	Total Load Volume (af)	Total Load Conc. by Volume[2]
Sub Area 23	0.51	51	278,420	0.127	36.13	0.41	44.98	0.469
Lytten	0.96	95	160,686	0.067	4.77	0.05	14.29	0.131
Buckle Bar	2.12	184	10,103	0.004	5.55	0.03	6.71	0.035
Dona Ana N	2.16	187	216,159	0.094	5.29	0.03	30.51	0.140
Sub Area 24	2.20	192	192,865	0.083	103.59	0.35	126.69	0.398
Ralph	2.45	212	274,278	0.125	18.37	0.08	54.65	0.205
Reed Thurman	3.25	281	121,680	0.050	1.73	0.01	23.07	0.076
Sub Area 106 B	3.63	311	150,013	0.062	5.49	0.02	34.99	0.101
Nordstrom	3.72	322	61,820	0.024	4.79	0.01	17.21	0.051
Bignell	6.12	485	114,074	0.046	18.41	0.04	52.93	0.098
Jaralosa	6.80	539	39,679	0.015	7.05	0.01	20.40	0.036
Dona Ana	6.94	550	159,768	0.067	38.17	0.06	93.00	0.145
Montoya	23.00	1465	47,881	0.019	20.46	0.01	64.22	0.042
Faulkner	25.00	1593	125,159	0.051	26.27	0.02	150.67	0.086
Sibley	27.20	1697	35,370	0.014	21.84	0.01	59.29	0.034
Placitas	34.60	2020	124,842	0.051	36.33	0.02	193.68	0.087
Green	35.60	2078	62,424	0.025	15.36	0.01	96.29	0.044
Tierra Blanca	68.20	3380	143,607	0.060	19.67	0.01	322.54	0.087
Cuervo	90.40	4284	32,226	0.012	35.83	0.01	121.97	0.028
Rincon	124.70	5332	52,307	0.020	184.52	0.03	358.53	0.063

[1]Based on Table 5-7, "Sedimentation Analysis From the Rio Grande Tributary Basins," July, 1996, Corps of Engineers and RTI.
[2]Calculations prepared for this study based on Table 5-7.

Table B2. Average Annual Sediment Load[1]		
Arroyo Watershed	Drainage Area (sq. mi.)	Ave. Annual Load (af/sq mi/yr)
Sub Area 23	0.51	11.05
Sub Area 24	2.20	5.76
Ralph	2.45	3.34
Lytten	0.96	2.63
Dona Ana N	2.16	2.31
Dona Ana	6.94	1.77
Sub Area 106 B	3.63	1.55
Bignell	6.12	1.28
Reed Thurman	3.25	1.24
Nordstrom	3.72	0.81
Faulkner	25.00	0.73
Tierra Blanca	68.20	0.59
Placitas	34.60	0.58
Buckle Bar	2.12	0.44
Jaralosa	6.80	0.44
Montoya	23.00	0.33
Green	35.60	0.28
Rincon	124.70	0.27
Sibley	27.20	0.25
Cuervo	90.40	0.16

[1] From Table 5-7, "Sedimentation Analysis From the Rio Grande Tributary Basins," July, 1996, Corps of Engineers and RTI.

Table B3 was reproduced from the FLO-2D Manual. It describes the properties of hyperconcentrated sediment flows as function of sediment concentration by volume. For sediment concentrations by volume exceeding 20 percent by volume, mud floods and mudflows can be expected. RTI predicted that two of the arroyo basins would produce design floods that were mud floods (approaching mudflow) events. This is a clear indication that the sediment loading for the 20 basins is being over estimated. In general, flows with concentrations greater than 10 percent by volume are considered to be hyperconcentrated sediment flows and exceed the applicability of most sediment transport capacity equations that were developed for river flows (Julien, 1995).

Table B3. Flow Behavior as a Function of Sediment Concentration			
Flow Type	Sediment Concentration by Volume	Sediment Concentration by Weight	Flow Characteristics
Landslide	0.65 - 0.80	0.83 - 0.91	Will not flow; failure by block sliding
	0.55 - 0.65	0.76 - 0.83	Block sliding failure with internal deformation during the slide; slow creep prior to failure
Mudflow	0.48 - 0.55	0.72 - 0.76	Flow evident; slow creep sustained mudflow; plastic deformation under its own weight; cohesive; will not spread on level surface
	0.45 - 0.48	0.69 - 0.72	Flow spreading on level surface; cohesive flow; some mixing
Mud Flood	0.40 - 0.45	0.65 - 0.69	Flow mixes easily; shows fluid properties in deformation; spreads on horizontal surface but maintains an inclined fluid surface; large particle (boulder) setting; waves appear but dissipate rapidly
	0.35 - 0.40	0.59 - 0.65	Marked settling of gravels and cobbles; spreading nearly complete on horizontal surface; liquid surface with two fluid phases appears; waves travel on surface
	0.30 - 0.35	0.54 - 0.59	Separation of water on surface; waves travel easily; most sand and gravel has settled out and moves as bedload
	0.20 - 0.30	0.41 - 0.54	Distinct wave action; fluid surface; all particles resting on bed in quiescent fluid condition
Water Flood	< 0.20	< 0.41	Water flood with conventional suspended load and bedload

Table 5 in the report indicates a sediment yield range of 0.19 to 0.91 af/mi^2/yr for 19 NRCS sediment retention dams based on NRCS reservoir resurveys or sediment yield rates that encompassed 5 or more years of sediment storage. The average sediment yield was 0.48 af/mi^2/yr with a standard deviation of 0.25 af/mi^2/yr. It is acknowledged that an infrequent flood event such the 100-year flood may increase the assumed historical sediment yield in specific basins. It not known how many of the basins have experienced large flood events after the NRCS retention structures were completed. The sediment yield computed by RTI for nine of the basins in Table 2 exceed the maximum NRCS sediment yield presented in Table 5 for basins in the RGCP watershed.

The predicted total sediment load results of the twenty study basins were used to develop a regression relationship for application to the remaining basins in the RGCP watershed. Based on the overpredicted total sediment load for at least seven of the twenty basins, the regression equation will overpredict the sediment load for all the basins. This is demonstrated by Figure 5-

21 in the Corps report for basins with drainage areas less than 25 mi^2. For basins greater 25 mi^2 there was no reservoir survey data for comparison and RTI infers therefore that the predictions equations were "…assumed to provide reasonable approximations of the total sediment yields based on a range of drainage areas".

Summary

The scope of work for the application of the FLO-2D model to the Rio Grande Canalization Project included a task to review existing sediment studies and recommend sediment loading for the project design event. Since Caballo and Elephant Butte Reservoirs have curtailed the sediment supply from the upstream Middle Rio Grande, the total sediment load for the design flood events would be contributed by the arroyo tributaries in the RGCP watershed. The 1996 study conducted by the Corps and RTI represents the best available data and methods for estimating the tributary sediment. One of the important issues for channel morphological stability is the estimate of the long term mean annual sediment supply.

The methodology for predicting the sediment yield to the RGCP presented in the 1996 Corps and RTI report on "Sediment Analysis from the Rio Grande Tributary Basins" is very good. The selection of the equations, the application of the equations based on size fraction, and the computation of the mean annual sediment load constituted an excellent approach to evaluating the total sediment supply to the river. The report indicates that the variability in the total sediment load results for some basins may be attributed to a combination of factors including hydrology, watershed properties and hydraulics. The overestimate of the average annual sediment yield can be attributed to the following factors:

- Overestimate of the velocity associated with low n-values and a supercritical flow assumption.
- Inappropriate application of the combined MPM-Woo equation and Colby adjustment.
- Inappropriate selection of the critical shear stress parameter for both incipient motion and for the MPM bed load equation.
- Possible overestimates of the wash load associate with parameter selection in the MUSLE equation.
- Over estimated total sediment load using the Colby adjustment factors because the MUSLE wash load is overestimated.

Most of the overestimated total load can be attributed to the application of the Colby adjustment factor based on wash load concentration and bed material median diameter. For the Subarea 23 and 24 Arroyos the Colby adjustment increases the total bed material load by factor of almost 10. It should be noted that the Colby method is based on limited data and a number of uncertainties in the graphical representation of the factors (Simons and Senturk, 1976). Yang (1996) concluded that "(b)ecause of the range of data used in the determination of the rating curves …Colby's approach should not be applied to rivers with median sediment diameter greater than 0.6 mm and depth greater than 3 m." Seventeen of the twenty study basins have a D_{50} size greater than 0.6 mm.

It is concluded that although the approach to calculate the total sediment supply to the RGCP was an excellent one, the selection of parameters and application of the sediment transport equations and the supercritical flow assumptions have resulted in over prediction of the mean annual sediment yield.

Recommendations

Based on the sediment supply analysis, it was recommended that the sediment load be adjusted to reflect the mean annual sediment yield based on the analysis of the NRCS retention basins. This adjustment was discussed in the main report and included the following tasks:

1. A further review of the storm hydrology to determine if the 2-hr and 6-hr return period storm hydrographs would exceed the peak discharge and volumes in comparison to the 24-hr storm peak discharge volume.
2. An update of the NRCS reservoir sedimentation data.
3. Calibration of the tributary total sediment loads to the available NRCS reservoir data.

These tasks were completed and reported on in the sediment load analysis section of the main report.

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