US Army Corps
Of Engineers
Albuquerque District

AQUATIC HABITAT RESTORATION AT SANTA ANA PUEBLO, NEW MEXICO (Section 1135)

ENGINEERING AND TECHNICAL APPENDIX

CONTENTS

U.S. Army Corps of Engineers
Albuquerque District
4101 Jefferson Plaza NE
Albuquerque, New Mexico 87109

March 2008

TECHNICAL APPENDIX A

MIDDLE RIO GRANDE SANTA ANA REACH

GEOMORPHOLOGY

In Support of Section 1135
Aquatic Habitat Restoration at
Santa Ana Pueblo, New Mexico

Prepared for

Pueblo of Santa Ana
Natural Resources Department
221 Ranchitos Road
Bernalillo, New Mexico 87004

AYRES ASSOCIATES

3665 JFK Parkway, Building 2, Suite 200
Fort Collins, Colorado 80526
970.223.5556, FAX 970.223.5578

Ayres Project No. 32-0615.04
SAOL9AA.DOC

September 2006

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

A. MIDDLE RIO GRANDE SANTA ANA REACH - GEOMORPHOLOGY

This review of Middle Rio Grande geomorphology is provided in support of the aquatic habitat restoration plan for the Pueblo of Santa Ana (Section 1135 directed by the U.S. Army Corps of Engineers, Albuquerque District). The restoration area is located approximately 25 miles downstream of Cochiti Dam on the Rio Grande in what is designated as the Santa Ana Reach. This reach encompasses the length of the Rio Grande beginning immediately downstream from the confluence with the Jemez River to the Highway 550 bridge in Bernalillo, approximately 4.4 river miles (**Figure A.1**).

Historically, the fluvial characteristics of the Middle Rio Grande were those of a wide and shallow river prior to the influence of flood control activities. The channel was described as a sand-bed stream, (Nordin and Beverage 1935), with a braided pattern (Lane and Borland 1953), probably due to sediment overload (Woodson 1961). The referenced studies indicate that the river followed a pattern of scour and fill during floods and was in an aggrading regime. Flood hazards associated with the aggrading riverbed prompted the Middle Rio Grande Conservancy District to build levees along the floodway during the 1930s. However, the levee system confined the sediment and increased the aggradation in the floodway. By 1960 the river channel near Albuquerque was 6 to 8 feet above the elevation of lands outside the levees (Lagasse 1980). Under the Comprehensive Plan of Improvement for the Rio Grande in New Mexico, construction of dams at Cochiti (1973), Abiquiu (1963), Jemez (1953) and Galisteo (1970) were expected to slow aggradation or reverse the trend to degradation in the Middle Rio Grande Valley. Additional channel training included the Kelner jetty system for bank stabilization, which was installed through the 1950s and 1960s. The flood control improvements have reduced the sediment load in the Middle Rio Grande and accomplished flood control objectives for much of the river valley.

As a result of the flood and sediment control measures, the Middle Rio Grande has experienced significant channel degradation. Of greatest significance to the Santa Ana Reach is the Cochiti Dam that was constructed on the main stem of the Rio Grande in 1973. Construction of the dam has cut off the sediment supply, which has resulted in channel degradation and cut off the historical floodplain from the river. This process has resulted in deterioration of the aquatic and terrestrial habitat throughout the river valley. A prior Section 1135 project, involving the construction of multiple grade control structures was completed in 2005 to stabilize the river channel and prevent future degradation.

A.1 Background

Human-induced changes in watersheds often have significant impacts on the receiving waterways. A relationship proposed by Lane (1957) can be used to identify the channel response to modifications in the watershed. The relationship proposed by Lane is:

$$QS \propto Q_s D_{50} \qquad \text{(A.1)}$$

where:

- Q = Water discharge
- S = Channel slope
- Q_s = Sediment discharge
- D_{50} = Median sediment size

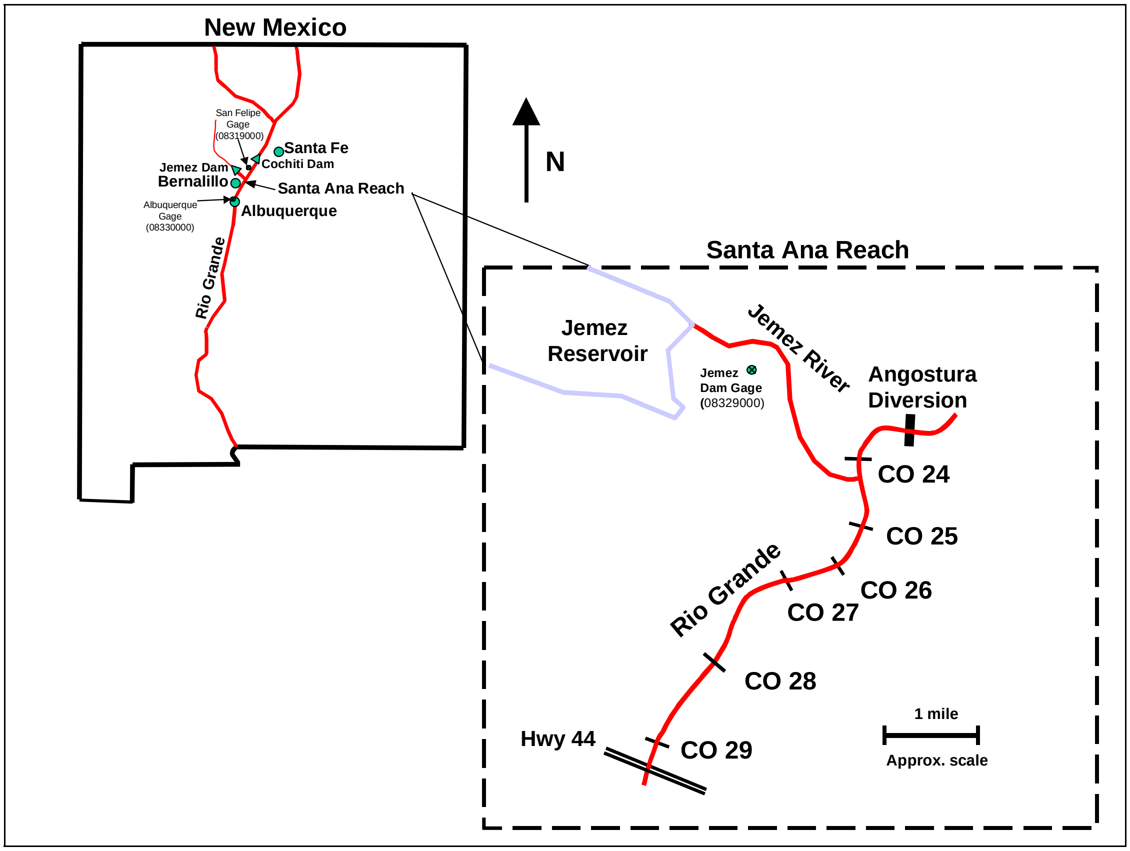

Figure A.1. Location map of Santa Ana reach of the Rio Grande.

Flood control activities on the Middle Rio Grande have mechanically altered both the water (Q) and sediment discharge (Q_s) in the river downstream. The typical river response following dam construction should satisfy the Lane relationship in the form of:

$$Q^{-}S^{+/-} \propto Q_s^{-} D^{+}_{50} \qquad \text{(A.2)}$$

The (-) signs indicate a reduction in sediment and water discharge (Q), the (+) sign for D_{50} indicates coarsening of the bed, and (+/-) indicates the river slope could increase or decrease depending on the relative magnitude of changes in the other parameters. Through the Santa Ana Reach, a flatter slope has been the general trend indicating that reduced sediment supply is the primary control. The slope reduction results in a lowering of the channel bed from upstream to downstream as water entrains sediment from the channel bed and banks. Under the reduced sediment conditions this process continues until the sediment transport capacity equals that supplied from upstream. Alternatively the degradation could stop if the channel becomes armored or structural controls are installed to stabilize the channel slope.

Lagasse (1980) provided an assessment of the initial response of the Middle Rio Grande resulting from dam construction. This study documented channel adjustments from Cochiti Dam to the Isleta Diversion after five years of establishing a pool at Cochiti. A recent study extended the analysis up through 1995 (Salazar 1998), but limited the analysis from Cochiti Dam to the Highway 550 bridge in Bernalillo. Both of these post-dam studies used comparative analyses of river planform, profile, cross section and sediment data to illustrate the degradational channel response to Cochiti Dam. The comparisons show a trend of channel narrowing and lowering of the riverbed. The current analysis extends the study through 1999 and include some quantitative analysis of hydrology, sediment and hydraulic properties through the Santa Ana Reach.

The comparative analysis utilized cross section and sediment data from the "Middle Rio Grande Database" (Julien et al. 1999). The database includes hydraulic geometry, discharge and sediment data for the Middle Rio Grande from pre-dam through 1999. Hydrologic data was obtained from the database and current discharge data was obtained from USGS gaging stations at San Felipe (station # 08319000), Albuquerque (station # 08330000) and below Jemez Dam (station # 08329000) as shown on Figure A.1. The San Felipe gaging station is approximately 7.5 river miles upstream of the Jemez River confluence and the Albuquerque gaging station is approximately 20 river miles downstream of the Highway 550 bridge in Bernalillo.

The comparative analysis for this study was limited to the Santa Ana Reach from Cochiti rangeline CO-24, 0.3 mile upstream of the Jemez River confluence, to rangeline CO-29, 0.3 mile upstream of the Highway 550 bridge in Bernalillo (Figure A.1). The following sections discuss the data utilized and present results from the comparative analysis to illustrate the need for restoration.

A.2 Planform

Historical aerial photography was obtained to review planform changes in the Santa Ana Reach following construction of Cochiti Dam. Aerial photographs from 1972 (pre-dam), 1982, 1991, 1992, 1994, and 1997 were used for the analysis. Flow conditions for the aerial photographs are provided in **Table A.1**.

Table A.1. Aerial Photography Flow Rates.

Year	Approximate Q (cfs)
1972	400
1982	4,500
1991	2,800
1992	300
1994	700
1997	1,800

The 1972 photographs show the channel through the Santa Ana Reach being braided and the active channel existed the full width between vegetated banklines (~500-600 feet). The 1982 photograph exhibits some narrowing of the main channel as compared to 1972 especially near the confluence with the Jemez River. Planform changes are less significant downstream from the confluence. In the 1982 photograph (estimated discharge ~ 4,500 cfs) overbank flooding can be identified in the bosque floodplain west of the river. This indicates that in 1982 the floodplain was hydrologically connected to the Rio Grande through surface flooding. Aerial photography from 1991, 1992, 1994 display significant narrowing of the main channel throughout most of the Santa Ana reach with a braided pattern near the downstream end of the reach. Channel degradation had cut through sand deposits to form split flow channels and mid-channel bars. By the early 1990s the channel appeared entrenched with a planform similar to today's. In the most recent aerial photography (1997) continued narrowing and entrenchment can be observed. Bars and islands observed on the 1992 photo have increased in size and some side channels have been abandoned. All of the planform changes observed from the 1972-1997 aerial photographs are indicative of a degradation trend throughout the Santa Ana Reach. The photographs also indicate a potential for meandering, although at the small scale of the photography, the rate of migration does not appear rapid. However, due to the meandering potential, the restoration plan should consider lateral migration and provide adequate protection to prevent flanking of the structures.

In our effort to relate stream characteristics to the planform of "natural" rivers, relationships such as those developed by Lane (1957) and Leopold and Wolman (1957) can be used to describe river trends. Lane's relationship is based on mean annual discharge. The mean annual discharge on the Middle Rio Grande has remained fairly constant over time. Leopold and Wolman's relationship is based on bankfull discharge that has changed through time due to channel incision and flood flow storage in reservoirs. Application of these relationships to historical data from the Santa Ana Reach indicate a progression from a braided or intermediate stream towards a meandering reach as the channel adjusts and decreases its slope.

A.3 Profiles and Cross Sections

Comparisons of historical profiles and cross sections were used to illustrate trends in channel form in the Santa Ana Reach as a result of flood control dams and other channel rectification activities. Channel geometry, profiles and hydraulic properties were evaluated with the River Analysis System - HEC-RAS (HEC 1998). Hydraulic models were developed using cross section data from 1971, 1975, 1986, 1992, 1995, and 1999. Historical cross sections were obtained from the Middle Rio Grande Database (Julien et al. 1999). The post-dam effective discharge was used for comparative analysis of hydraulic variables.

A comparison of minimum channel elevation (thalweg) profiles is presented in **Figure A.2.** The profiles indicate more than 10 feet of degradation at the upstream end of the reach (CO-24) and approximately 5 feet at the lower end (CO-29) since 1971. The profiles become flatter and slightly longer through time. The lengthening of the profiles results from meandering of the main channel. The observed lengthening supports the need for including lateral migration in the design alternatives. The decrease in channel slope and channel lowering are indicative of the channel adjustment to the reduced sediment supply.

A comparison of the change in channel slope and median bed material size is presented in **Figure A.3**. The slopes were computed as the average of the energy grade and water surface slopes computed from regression analysis of the profiles. The slopes have generally decreased from greater than 0.001 ft/ft to approximately 0.00085 ft/ft since construction of Cochiti, but an increase in the slope was observed from 1995 to 1999. This could be attributed to temporary adjustment to sediment inflows from tributaries and lower flows on the Middle Rio Grande mainstem since 1995. A time history of median channel bed material sizes is also presented in Figure A.3. The bed material has generally become coarser over time as fine sediments are trapped by dams upstream and removed from the channel bed downstream. Prior to dam construction the median bed material was on the order of 0.2 mm which is a fine sand. Recent bed material samples indicate a median size on the order of 7-20 mm which is in the gravel range.

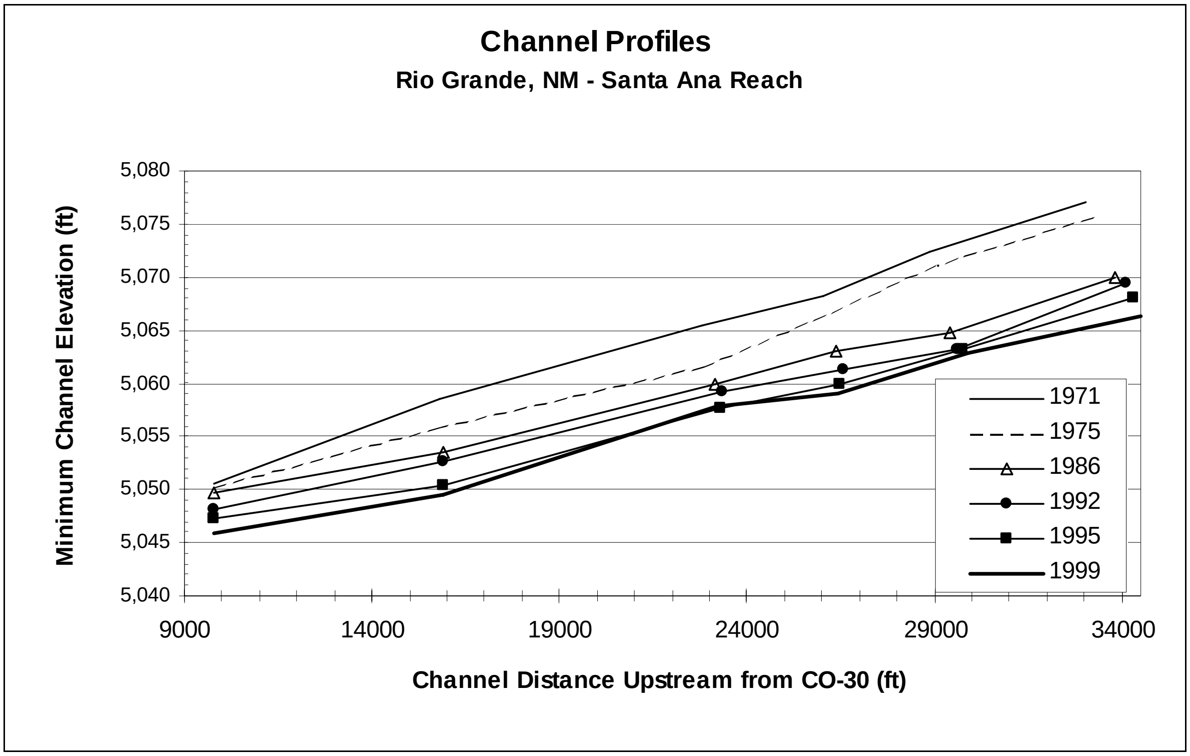

Figure A.2. Historical channel profiles.

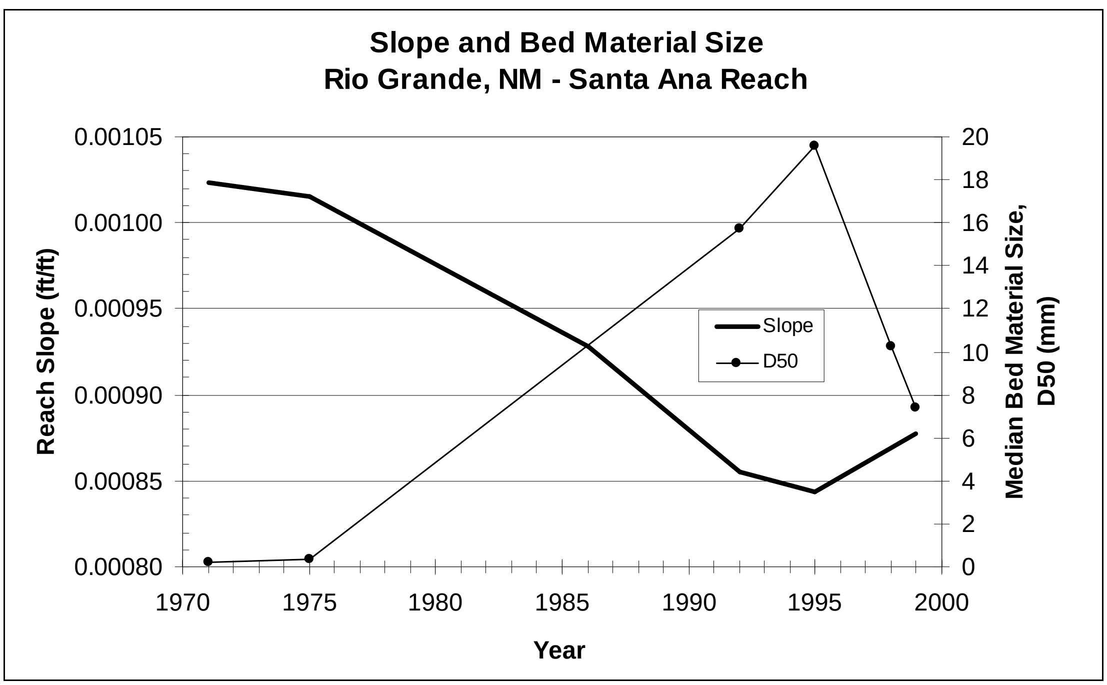

Figure A.3. Historical channel slope and median bed material size.

Comparison of channel cross sections also illustrates the magnitude of degradation in the Santa Ana Reach. Historical surveys of Cochiti Rangeline CO-24 are presented in **Figure A.4**. The transition from a wide shallow channel to the existing entrenched condition is clearly evident in the comparative cross sections.

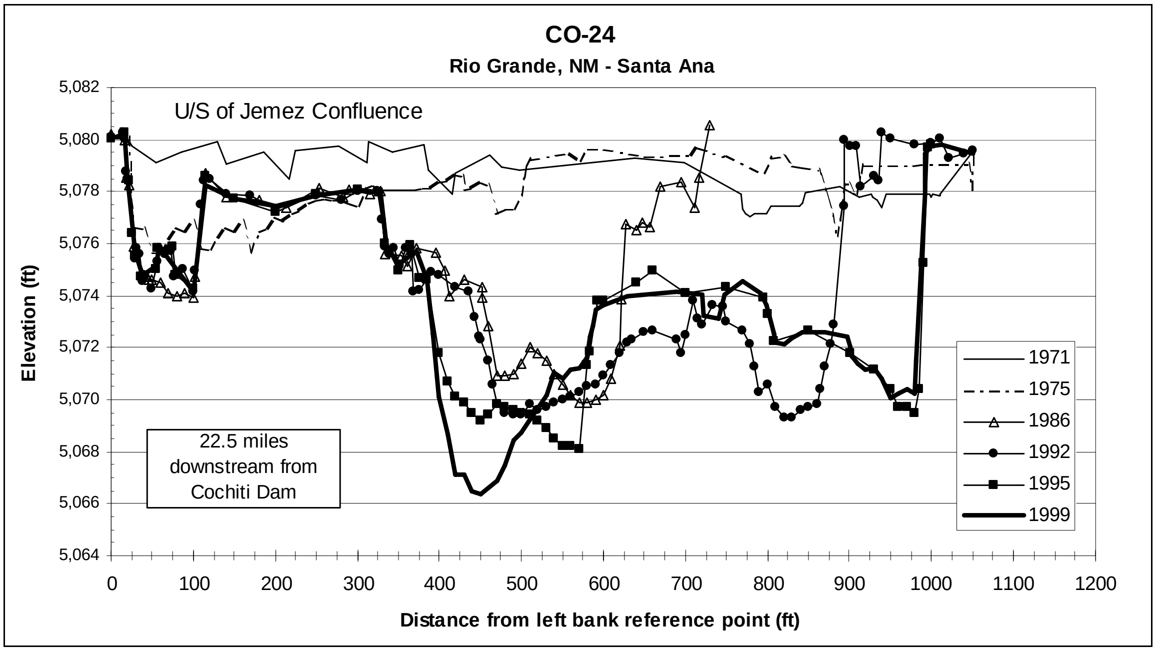

Figure A.4. Historical cross sections at Cochiti Rangeline CO-24.

Hydraulic variables at the post-dam effective discharge were averaged over the Santa Ana Reach from Cochiti Rangelines CO-24 to CO-29. A comparison of reach averaged main channel hydraulic variables is provided in **Figure A.5**. The main channel was identified within the banks of the dominant conveyance section. High flow side channels and bars were not included as part of the main channel. The effects of incision on hydraulics and channel geometry have been decreased channel width and increased depth and velocity. This is significant to aquatic habitat in that fewer shallow, low velocity areas are available for aquatic species. The effective channel width has decreased from approximately 600 feet to less than 300 feet. Simultaneously, the channel depth has increased by a factor of two. This translates into a significant decrease in the width-depth ratio (factor of four) a parameter used to describe the level of entrenchment.

A.4 Hydrology

The Middle Rio Grande follows a pattern of high flows during spring runoff and low flows during the fall and winter months. Additional high flows result from thunderstorms that occur in late summer months. The Middle Rio Grande hydrology has been altered due to the influence of flood control dams. Cochiti Dam primarily acts to decrease peak flows and has a much smaller impact on low flows. Therefore average annual flows have been less affected, while peak flows have been significantly reduced. Average yearly hydrographs from the USGS gaging station at Albuquerque are shown in **Figure A.6**.

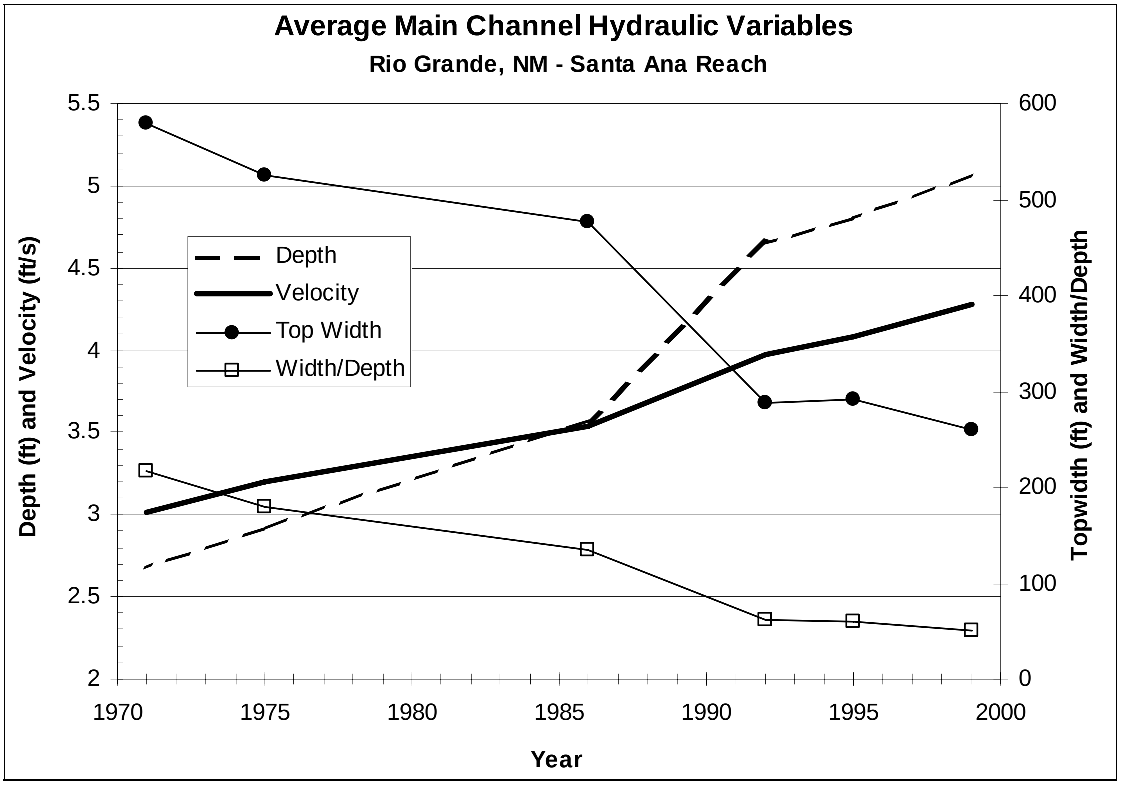

Figure A.5. Reach averaged channel hydraulic variables.

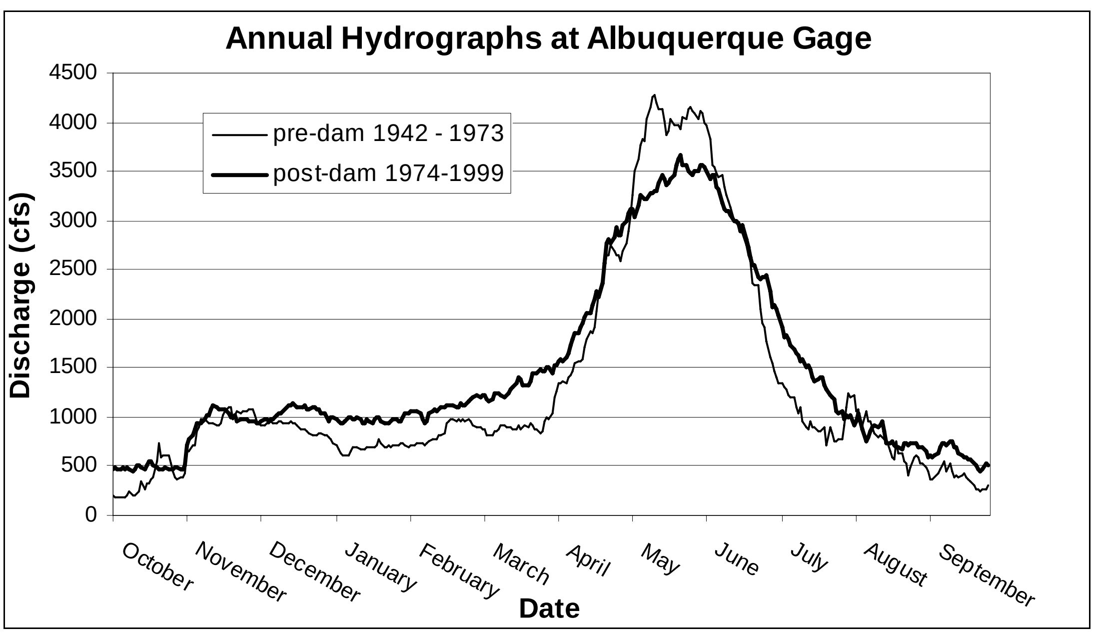

Figure A.6. Average annual hydrograph at Albuquerque gaging station.

Drought years were removed from the pre-dam record. It can be observed from annual hydrographs that the influence of Cochiti Dam has been to reduce the peak flows and extend the duration of the high flow period. Average winter base flows are somewhat larger for post-dam conditions due to storage.

The effects of Cochiti on the Middle Rio Grande hydrology can also be observed using flow duration statistics of mean daily flow records at the San Felipe and Albuquerque gages. The flow duration curve also illustrates that the low flows have been somewhat larger for the post-dam conditions and peak flows are greatly reduced as shown in **Figure A.7**. The breakpoint is the 2-3% exceedence, where flows larger than this have been retained in reservoirs and flows less have been more frequent during the post-dam period.

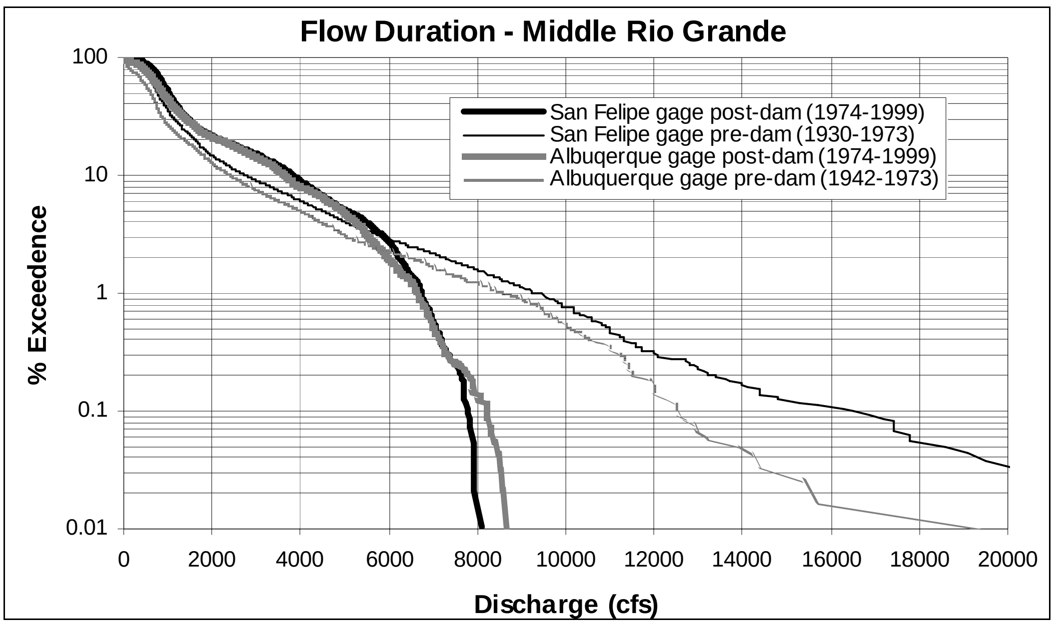

Figure A.7. Flow duration distribution for San Felipe and Albuquerque gages.

Review of annual peak series data also exhibits the influence of flood control. Historical annual peak discharges from the San Felipe gage illustrate the effects of regulation on the Rio Grande (**Figure A.8**). From 1927 to 1945 floods in excess of 20,000 cfs were experienced approximately every five years. From 1945 to construction of Cochiti in 1973 floods in excess of 10,000 cfs were fairly common with the exception of drought years. Following construction of Cochiti regulation has prevented flows from exceeding 10,000 cfs. This has reduced the avereage annual peak dischrge from 9,800 to 5,700 cfs. A study to determine the effects of regulation on the Middle Rio Grande flood hydrology was performed by the U.S. Bureau of Reclamation (USBR) Flood Hydrology Group (Bullard and Lane 1993). This study estimated return period floods at ten USGS gaging stations on the Middle Rio Grande. The study applied a procedure to develop discharge values for regulated (dam) and unregulated (no-dam) conditions. **Table A.2** summarizes the 2-, 5-, and 10-year discharges at the San Felipe and Albuquerque gaging stations as determined from this study.

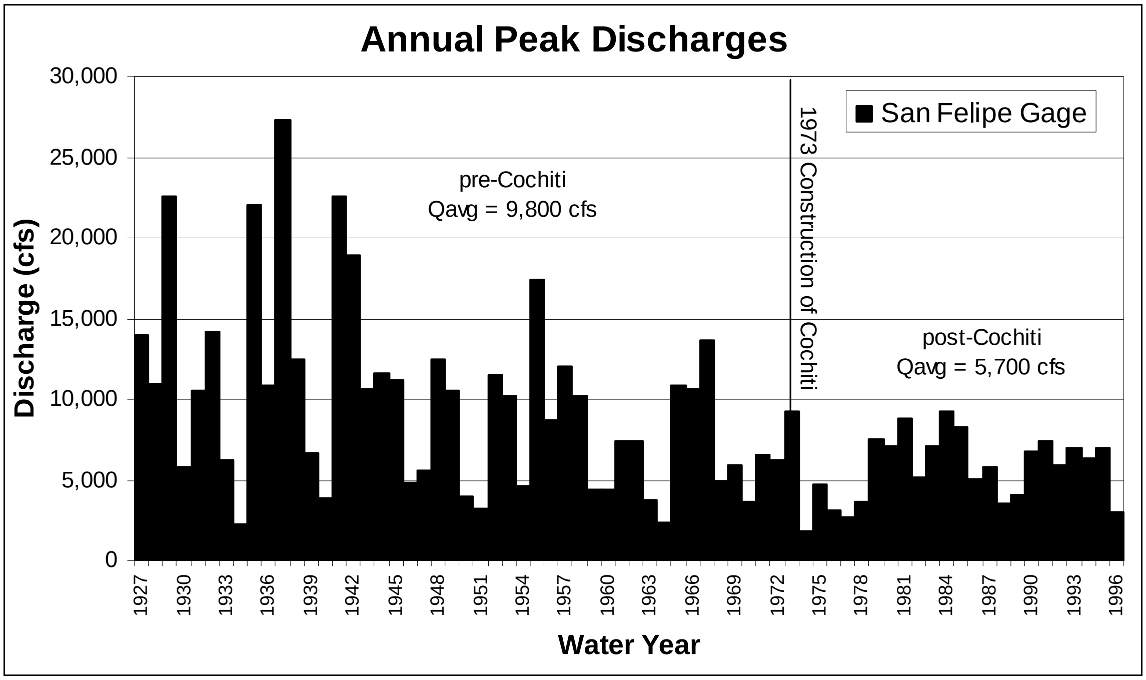

Figure A.8. Annual maximum discharges at the San Felipe Gage.

Table A.2. USBR Flood Flow Discharges (cfs) for Regulated and Unregulated Conditions.

Return Period	San Felipe		Albuquerque	
	Unregulated	Regulated	Unregulated	Regulated
2-year	11,166	5,650	10,647	4,820
5-year	16,965	9,330	15,114	7,450
10-year	20,762	10,000	17,899	9,090

Flood control dams have acted to reduce flood flows by approximately a factor of two. This is significant with respect to geomorphology since channel forming processes are assumed to be dominated by discharges within the range of these recurrence intervals. The study by Bullard and Lane included flood flow data up through 1988. An independent analysis (Salazaar 1998) including peak flows through 1996 verified that the data provided by Bullard and Lane is valid for the current conditions.

A.5 Rio Grande Sediment Load

Observation of historical suspended sediment data indicates significant reductions in sediment load following construction of flood control dams. The USGS maintains suspended sediment data in addition to discharge for many of the stream gages along the Middle Rio Grande. Temporal suspended sediment data was available at the discontinued gage at Bernalillo from water years 1956 through 1969. Suspended sediment data collected at the Albuquerque gage was available for water years 1970 through 1995. From these data, a double mass curve can be developed by plotting the cumulative suspended sediment with the cumulative water discharge as shown in **Figure A.9**. Changes in hydrology and sediment concentration are indicated by breaks in slope in the mass curve. As shown in Figure A.9 there is a noticeable decrease in slope of the mass curve following closure of Cochiti Dam in 1973.

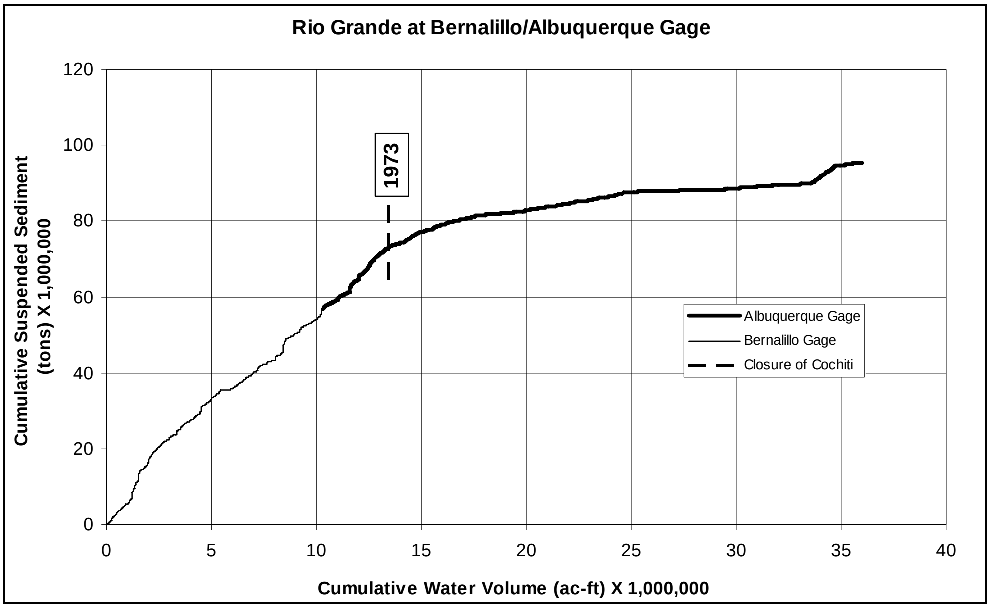

Figure A.9. Cumulative water vs. sediment discharge.

A time series plot of cumulative suspended sediment and yearly loads at the Bernalillo and Albuquerque gages is presented in **Figure A.10**. Prior to construction of Cochtiti, the average annual suspended sediment load was on the order of 4 million tons per year. This has been reduced to an average of approximately 1 million tons per year as shown in Figure A.10. The apparent high sediment loads in 1957, 1958, and 1973 correspond to high runoff volume years preceded by several low runoff volume years. Consequently, the high annual sediment discharges for these years are primarily dependent on hydrology.

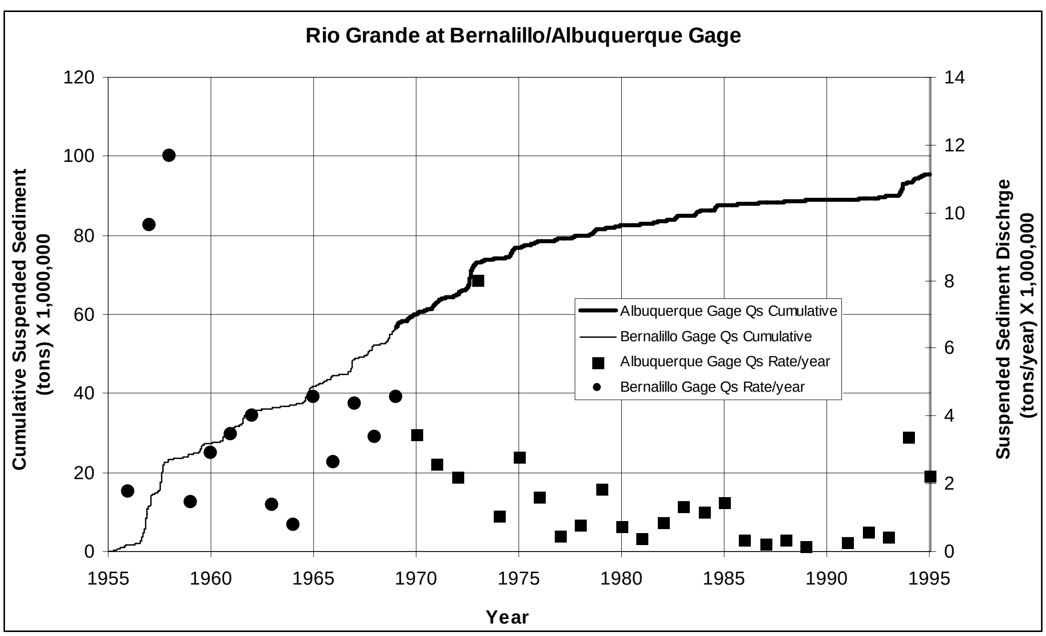

Figure A.10. Cumulative water vs. sediment discharge.

A.6 Channel Forming Discharge

The adjustment of a channel's geometry is dependent on many factors including hydrology, channel bed and bank material and sediment transport. Discharge is the dominant variable that impacts channel adjustment. Channel geometry probably results from a range of discharges over time, but it is convenient to select a single value for the basis of stable channel design.

The channel forming discharge used for river analysis and design has been termed as the bankfull, dominant or effective discharge. Bankfull discharge has been equated with dominant discharge on the supposition that rivers adjust to the flow that just fills the available cross-section (from Knighton 1998). Dominant or effective discharge has been defined as the discharge that cumulatively performs the most sediment transport over time. In an incised stream the bankfull condition may only occur at low frequency events and therefore may not correspond to the dominant or effective discharge. The terms dominant discharge or effective discharge may be used interchangeably, but not necessarily with bankfull discharge. Leopold, Wolman and Miller (1964) observed a correspondence between the frequency of the bankfull discharge and the discharge that cumulatively transports the most sediment. The supposed recurrence interval was in the range of 1 to 2 years. For this reason the 2-year discharge is frequently adopted as the effective discharge for river restoration projects.

It should be noted that channel forming discharge is significantly different from the design discharge used for stability analysis of revetments and armoring materials. The design discharge is used to design materials (riprap) such that they will not be displaced during high flow events. The design discharge frequently corresponds to higher magnitude events such as the 50- or 100-year flood.

An effective discharge calculation was completed for post-dam conditions to provide a basis for geomorphic comparisons and sediment transport calculations. Because the Santa Ana Reach is incised and the term bankfull is more problematic, the dominant/effective discharge was adopted for the analyses. The effective discharge was calculated as the discharge corresponding to the maximum collective sediment discharge as described in "Channel Rehabilitation: Processes, Design and Implementation" (Watson et al. 1999). The collective sediment discharge is the product of the sediment transport rate and the probability of a given discharge.

Sediment transport through the Santa Ana Reach was quantified using data collected upstream from the Highway 550 bridge near Bernalillo. This location is at the downstream end of the Santa Ana Reach near Cochiti Rangeline CO-29 (Figure A.1). Suspended sediment and bed material samples were collected with discharge measurements from 1992 to 1996 (FLO Engineering 1998). The data included a total of 33 measurements covering discharges from approximately 600 to 6,000 cfs. The Modified Einstein Procedure (MEP) was used to estimate the unmeasured bed load for each sample. The MEP provided the bed material sediment load for use in channel adjustment calculations. A sediment discharge rating curve that relates bed material sediment load to water discharge was developed from the data. The sediment discharge rating curve has the form of:

$$Q_s = \alpha Q_w^{\beta} \qquad \text{(A.3)}$$

where:

Q_s = Bed material sediment load (tons/day)
Q_w = Water discharge (cfs)

The sediment rating curve is presented in **Figure A.11**.

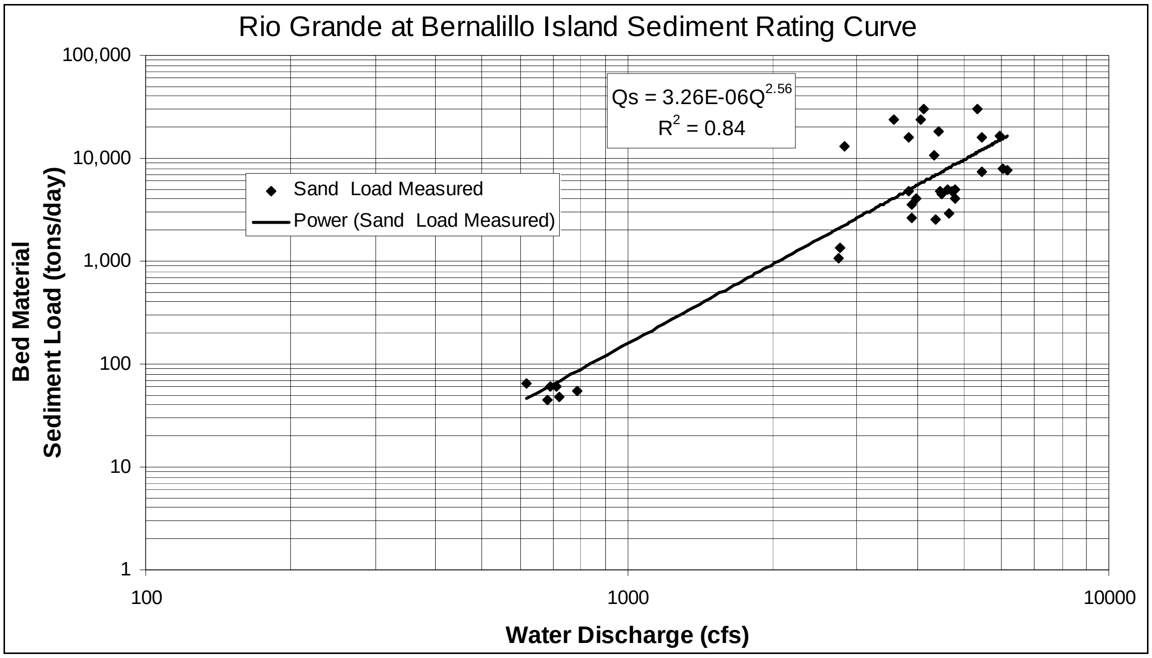

Figure A.11. Santa Ana reach sediment rating curve.

A probability distribution of post-dam discharges was developed using mean daily discharges from the USGS gaging stations at San Felipe and Albuquerque as shown in Figure A.7.

The flow duration data was used to develop occurrence probabilities for specific discharge ranges. The collective sediment discharge was then computed from the probability and corresponding sediment discharge for numerous intervals of discharge. The effective discharge was that corresponding to the maximum collective sediment discharge as shown in **Figures A.12 and A.13**.

The effective discharge calculated from the flow record at the San Felipe gage was approximately 6,000 and 5,500 cfs for the Albuquerque gage. Figures A.12 and A.13 illustrate calculations using 25 equal intervals of discharge. Using both equal and logarithmic intervals and numbers of intervals ranging from 25 to 100, a sensitivity analysis of the calculation was performed. The analysis revealed that the effective discharge was not highly sensitive to the number of intervals and was not sensitive to the use of linear or logarithmic intervals. The effective discharge using the average of the San Felipe and Albuquerque flow data was selected as 5,800 cfs. This flowrate is slightly greater than the 2-year flood and was used for comparative analyses and stable channel analyses.

As mentioned previously channel form probably results from a range of discharges. Therefore effective discharge could be assumed for a range of flows which perform the most work. Figures A.12 and A.13 illustrate that discharges in the range of 3,000 to 7,000 cfs would perform the most work and would have the greatest effect on the channel form.

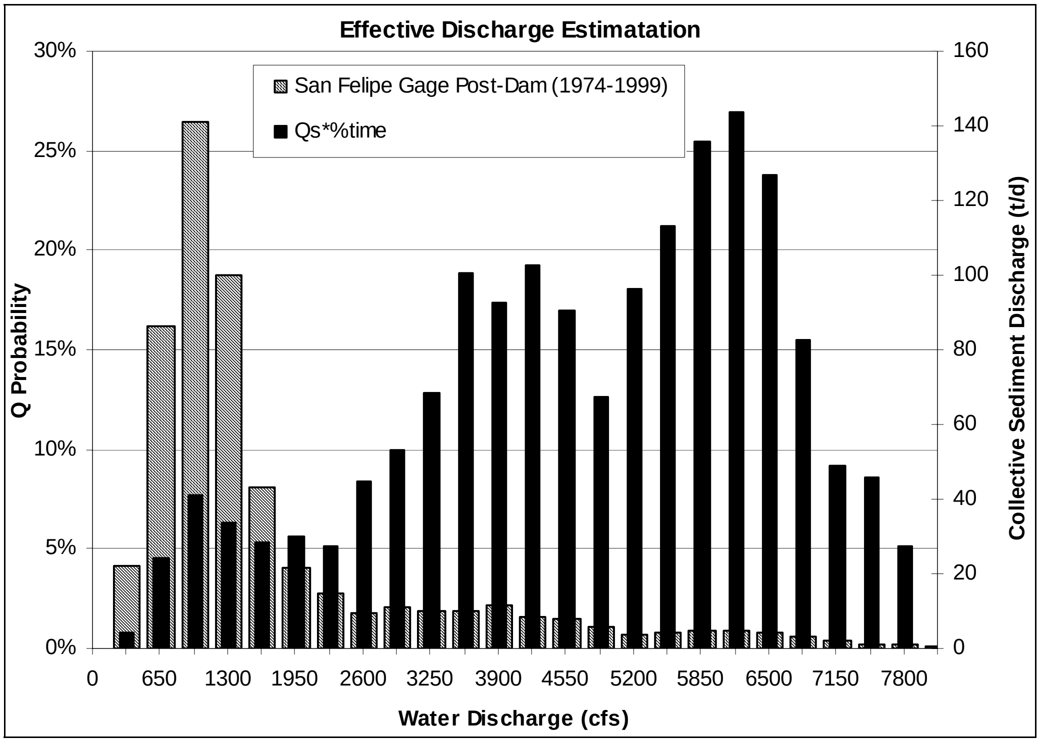

Figure A.12. Effective discharge estimation using San Felipe gage.

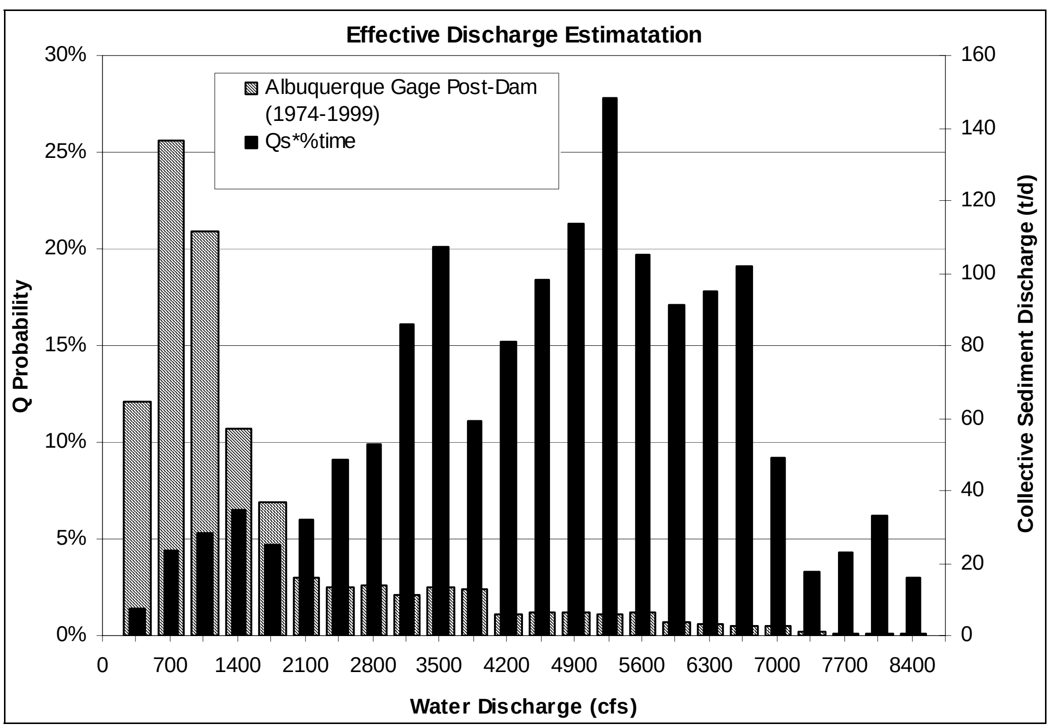

Figure A.13. Effective discharge estimation using the Albuquerque gage.

A.7 Santa Ana Sediment Budget

To determine the magnitude of sediment deficiency in the Santa Ana Reach, a sediment budget was developed. The budget was estimated using measured sediment data and historical cross sections. Channel adjustment results from the removal or accumulation of sediment in the channel. With respect to vertical adjustment, this material is most likely scoured from or deposited in the channel bed. The magnitude of this adjustment is dependent on the amount of sediment supplied from upstream sources and the amount that is carried out of the reach. This is the principle of conservation of mass or more specifically sediment continuity. Sediment continuity can be expressed as:

$$\Delta S = S_{s(supply)} - S_{s(outflow)} \qquad (A.4)$$

where:

ΔS	=	Amount of sediment stored (+) or lost (-) in the reach
$S_{s(supply)}$	=	Sediment transport into the reach from upstream sources
$S_{s(outflow)}$	=	Sediment transport out of the reach

Therefore, the sediment supply to the reach can be estimated as the difference in the sediment being transported out of the reach and the amount of sediment removed from the channel as represented with the following equation:

$$S_{s(supply)} = S_{s(outflow)} - \Delta S_{(degradation)} \qquad (A.5)$$

The amount of degradation in the Santa Ana Reach was computed from the summation of change in area of surveyed cross sections multiplied by the weighted length of each cross section for Cochiti Rangelines CO-24 through CO-29. The sediment rating curve shown in Figure A.9 is representative of the sediment outflow. Because the sediment rating curve was developed from data observed from 1992 to 1996, it was appropriate to compute degradation from this time period in development of the sediment budget.

Comparison of CO rangelines surveyed on July 21,1992 and August 4,1995 indicate that approximately 300,000 yd^3 of sediment was removed from the Santa Ana reach during this time period. This translates into approximately 400,000 tons of sediment removed from the channel.

The sediment outflow rating curve and mean daily discharge values from the Albuquerque gage were used to compute a value of approximately 2.6×10^6 tons of sediment transported out of the reach for the time period between the 1992 and 1995 cross section surveys. Using sediment continuity, the supply to the reach would have been 2.2×10^6 tons $[(2.6 – 0.4) \times 10^6)]$ of sediment which is 85% of the outflow. **Figure A.14** illustrates the cumulative sediment supply and outflow from the Santa Ana Reach for the 1992 - 1995 period.

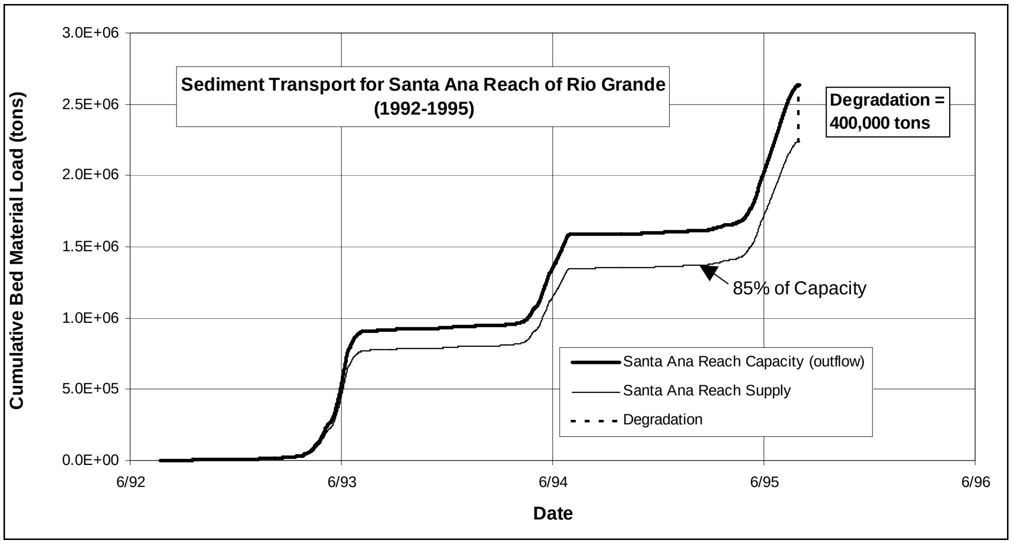

Figure A.14. Cumulative sediment supply and outflow from the Santa Ana reach.

Cross sections from 1975, 1986, 1992, 1995, and 1999 were compared to compute sediment losses since construction of Cochiti Dam. Comparison of the cross sections indicates that the Santa Ana Reach has been losing approximately 140,000 tons of sediment per year from 1975 to 1995. Somewhat less degradation was experienced from 1995 to 1999 due to lower annual flow volumes, as shown in **Figure A.15** in this period. To verify the use of the supply and outflow sediment rating curves for the restoration plan, the amount of degradation between 1995 and 1999 was computed using the sediment rating curves and sediment continuity. The amount of degradation computed from sediment continuity matched reasonably well with that observed from cross section data as shown in **Figure A.16**. The measured and predicted degradation between 1992 and 1995 are equal because this period was used to determine the sediment supply.

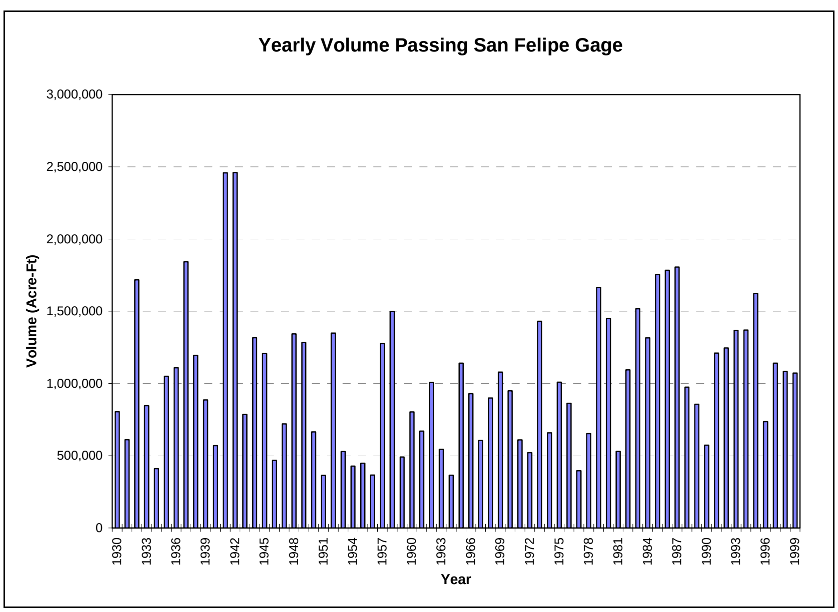

Figure A.15. Historic flow volumes passing through the project reach.

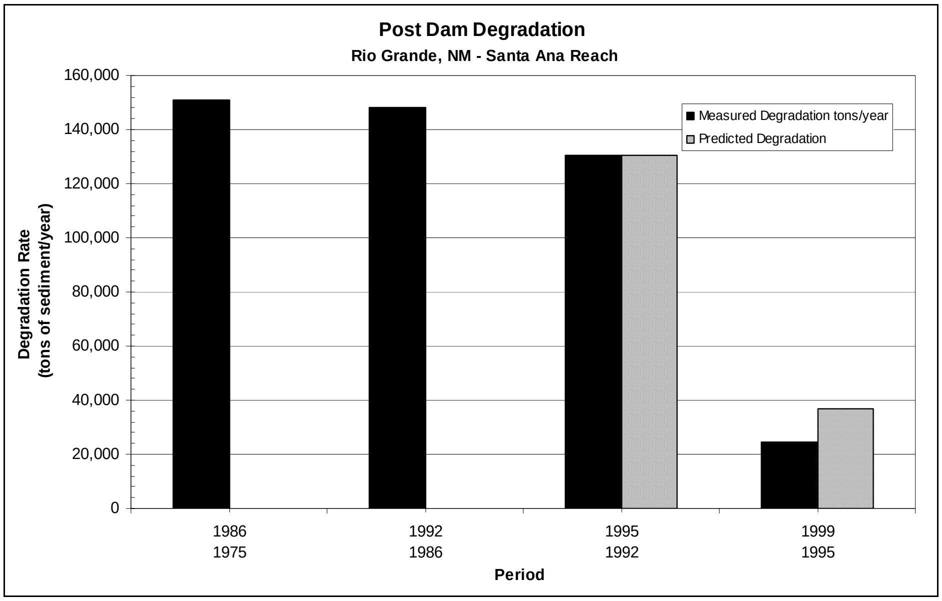

Figure A.16. Measured and predicted degradation rates.

A.8 Stable Channel Design And Equilibrium Slope

For a channel to be relatively stable three physical relations must be satisfied: flow continuity, flow resistance and sediment continuity. In design, these relations are used to predict channel response to imposed conditions. Water and sediment inflows to a reach are considered dependent variables and the resulting channel geometric parameters are the independent variables. The channel geometric variables are planform, cross section, slope and bed configuration. Because there are many degrees of freedom in the channel geometric variables, numerous solutions could provide continuity of water and sediment. Therefore, there is some indeterminacy involved in predicting the channel geometry of streams with mobile beds (from Knighton 1998).

In stable channel design the width, depth and slope of the channel can be solved to provide combinations that satisfy the continuity of sediment and water. The equilibrium slope is that which can transport the incoming sediment load given an assumed channel width. The depth of flow corresponding to the equilibrium slope and width can be solved using a resistance equation (i.e., Manning's). To simplify stable channel calculations, the bed material transport capacity for a range of flow conditions can be computed using a power function that relates the sediment transport to hydraulic conditions. The general form of the power function is:

$$Q_s = W^a V^b Y^c \qquad \text{(A.6)}$$

where:

Q_s	=	Bed material load (cfs)
W	=	Channel topwidth (ft)
V	=	Average channel velocity (fps)
Y	=	Average channel hydraulic depth (ft)
a, b, c	=	Coefficient and exponents determined from regression

For the Santa Ana restoration project the bed material sediment load was determined from measured data (FLO 1999). Reach averaged main channel velocities, depths and topwidths were determined from detailed hydraulic modeling using HEC-RAS. The detailed hydraulic modeling included approximately 60 cross sections through the reach as compared to the six cross sections used for comparative analysis of historical hydraulic variables described. Hydraulic modeling will be described in Appendix C. Similar to the historical comparative analysis, the main channel was identified within the banks of the dominant conveyance section. High flow side channels and overbank bars were not included as part of the main channel. It was assumed that the bulk of sediment transport occurs in the main channel and transport in overbank areas and side channels is negligible. Regression analysis provided values of a, b and c in the sediment transport power function. Results from the regression analysis are shown in **Figure A.17**.

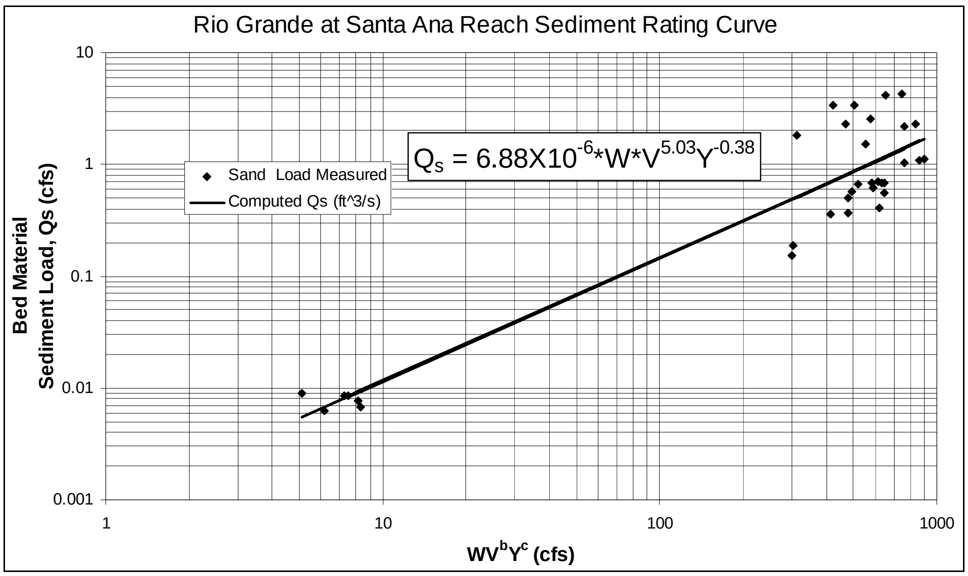

Figure A.17. Sediment transport power function.

Equating the sediment transport capacity in Equation A.5 with the sediment supply at the effective discharge provides channel dimensions that satisfy sediment continuity. For channels where the width-depth ratio exceeds 10, a wide rectangular channel assumption is reasonable for hydraulic calculations (Chow 1959). Using the wide rectangular assumption and a uniform distribution of roughness, the equilibrium slope can be estimated with the following closed form solution (adapted from Mussetter et al. 1994):

$$S_{eq} = \left(\frac{a}{Q_s}\right)^{\frac{10}{3(c-b)}} Q_w^{\frac{2(2b+3c)}{3(c-b)}} W^{\frac{2(5-2b-3c)}{3(c-b)}} \left(\frac{n}{1.486}\right)^2 \qquad \text{(A.7)}$$

where:

S_{eq} = Equilibrium slope
Q_w = Water discharge (cfs)
n = Manning's roughness coefficient
All other terms previously defined

There are multiple combinations of width, depth velocity and slope that could satisfy the above relation. Additionally, the inclusion of bank roughness results in a quadratic solution of depth in Manning's Equation. An alternative method for computing stable channel dimensions partitions channel roughness between the channel bed and banks. This method which requires an iterative solution is incorporated in the computer program SAM developed at U.S. Army Corps of Engineers Waterways Experiment Station (WES 1992).

Hydraulic modeling indicates that at the effective discharge of 5,800 cfs, approximately 5,300 cfs is conveyed in the main channel and 500 cfs flows in the overbank. Therefore the 5,300 cfs of main channel flow is transporting the bulk of sediment at the effective discharge. As computed from the sediment budget, the sediment supply was estimated as 85% of the transport capacity. Therefore stable channel dimensions are those which would transport 85% of the existing transport capacity. Using reach averaged hydraulic variables and the sediment rating curve shown in Figure A.17, the existing bed material sediment load is approximately 10,800 tons/day and the supply is 9,200 tons/day. Stable channel dimensions computed using rectangular channel geometry, uniform roughness assumptions and the sediment supply for the Santa Ana Reach of the Middle Rio Grande are shown in **Figure A.18**.

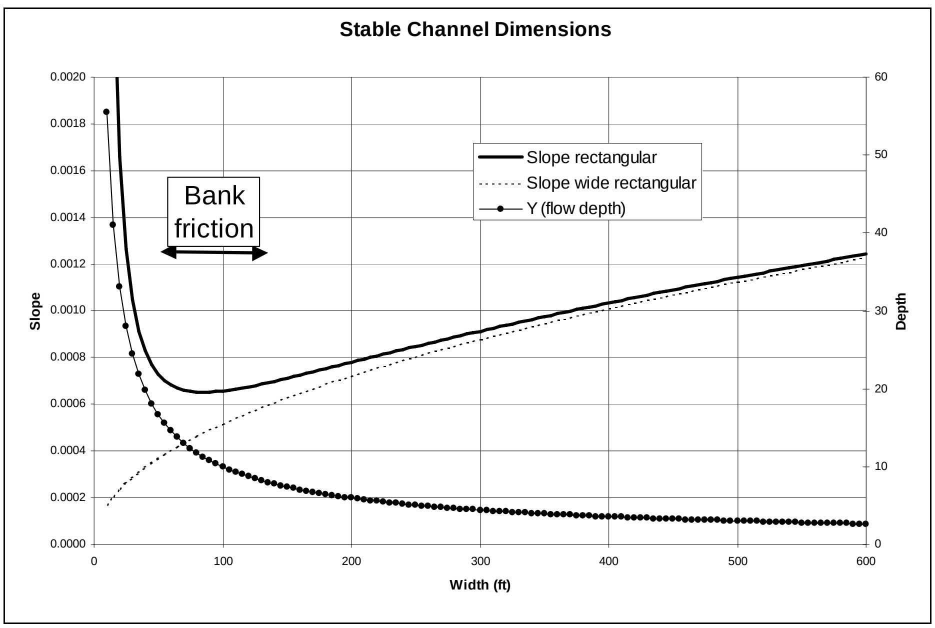

Figure A.18. Stable channel dimensions for the Santa Ana reach of the Rio Grande.

It can be observed from Figure A.18 that friction from the banks becomes significant for channel widths less than 200 feet. Therefore the wide rectangular assumption is valid for the Santa Ana Reach of the Rio Grande. The existing channel slope is approximately 0.00085 ft/ft with a width of 228 feet. The equilibrium slope calculations indicate that the slope required for dynamic equilibrium at this width would be 0.00077 ft/ft.

Extremal hypotheses suggest that the equilibrium channel dimensions would occur at some minimum level of energy dissipation such as minimum energy slope. However current research recommends the use of a hydraulic geometry predictor to estimate the equilibrium width (Copeland 1994) and sediment continuity for the resulting depth and slope. For this analysis hydraulic geometry equations developed by Julien and Wargadalam (1995) were used to estimate the equilibrium channel width. These equations were developed from analysis of 835 stable rivers with a correlation coefficient greater than 90%. The regime equation for channel width is:

$$W = 0.512 Q_w^{\frac{2m+1}{3m+2}} d_s^{\frac{-4m-1}{6m+4}} \tau_{\theta^*}^{\frac{-2m-1}{6m+4}} \tag{A.8}$$

where:

Q_w = Water discharge (cms)
d_s = Median grain size = d_{50} (m)
τ_{θ^*} = Shields Parameter
m = Resistance exponent = 1/ln (12.2 Y/d_{50})

Application of this relation at the effective channel discharge results in a channel width of approximately 200 feet. Using Equation A.6 for a channel width of 200 feet the corresponding equilibrium slope would be approximately 0.00072. However hydraulic modeling indicates that as the channel degrades and narrows overbank flows would be diminished and the effective discharge of 5,800 cfs would be contained within the main channel. Therefore the required slope to reduce the sediment transport to 85% of the existing would be approximately 0.0006 ft/ft. In absence of armoring and grade control this could result in significant additional degradation through the Santa Ana Reach. Results of the stable channel analysis are presented in **Table A.3**.

Table A.3. Results of Equilibrium Slope Calculations.

Total Effective Q (cfs)	Reach Average Channel Hydraulic Variables					
	Q (cfs)	Topwidth (ft)	Velocity (fps)	Depth (ft)	Slope	Sediment Transport (tons/day)
5,800 (existing)	5,300	228	4.4	5.2	0.00085	10,800
5,800	5,300	228	4.3	5.4	0.00077	9,200
5,800	5,300	200	4.5	6.0	0.00072	9,200
5,800	5,800	200	4.5	6.9	0.00060	9,200

Results indicate that the equilibrium channel slope could range from 0.00077 to 0.00060 depending on the adjustment of the channel width. These values provide quantitative estimates of the amount of degradation the Santa Ana Reach could experience in the future.

A.9 Future Conditions Analysis

A future condition of the river was to be estimated in support of the restoration incremental cost analyses. The life of the restoration project was designated to be 50 years, and therefore, a future condition was estimated at 50 years from the present. The future condition was used for comparative analyses of the hydraulic conditions for restoration alternatives.

The 50-year future condition was estimated using equilibrium slope analysis and sediment continuity. The equilibrium slope analysis provides an estimate of ultimate channel slope neglecting the time required to transport sediment from the reach. Therefore sediment continuity was utilized to determine the channel adjustment that could be reasonably achieved within 50 years. The equilibrium slope analysis resulted in a channel slope of 0.0006 for a channel topwidth of 200 feet as listed in Table A.3.

To apply the equilibrium slope to a channel reach, a location downstream from which to project the slope must be designated. Usually a stable control location such as near a dam, diversion structure or bedrock outcrop is used for the projection point. The farther downstream this projection point is located from the reach of interest, the larger the resulting bed elevation adjustment. The nearest known control of this type would be the Isleta diversion which is approximately 30 miles downstream from the Santa Ana Reach.

To use this location as the downstream projection point would result in a volume of degradation that is unlikely to occur within 50 years. Therefore a downstream location that would result in an amount of degradation that could be realistically be transported within 50 years was selected. Recently a possible bedrock outcrop was discovered just upstream of the Bernalillo bridge. As insufficient information about the outcrop is available at this time, further analysis considering the outcrop's impact on the project reach will be addressed during the final design phase.

Sediment continuity was applied to estimate a reasonable amount of degradation that could be expected within the next 50 years. The degradation volume was computed using an average annual hydrograph and the sediment transport rating curves developed for the supply and outflow of the Santa Ana Reach. The annual hydrograph was developed using the average of mean daily flows from the Albuquerque gage for water years of the post-Cochiti area (Figure A.6). Sediment continuity was then applied to estimate an amount of degradation that would occur for this average annual hydrograph. The analysis resulted in approximately 42,000 tons of degradation for the Santa Ana reach in a single year. This degradation rate is somewhat less than that observed between 1975 and 1992 and is more consistent with observations between 1995 and 1999 (Figure A.16). The computed degradation value applied equally over 50 years would result in 2.1×10^6 tons of degradation. Therefore the equilibrium slope analysis was applied to be consistent with this amount of degradation. The location of Cochiti Rangeline CO-30, which is approximately 1.5 miles downstream of the Highway 550 Bridge in Bernalillo, was selected as the downstream projection point for the equilibrium slope. For the Santa Ana reach to attain the equilibrium slope of 0.0006 and channel topwidth of 200 feet, 1.7×10^6 tons of degradation would be required. The amount of degradation for the 50-year future condition was computed from the change in cross sectional area of Cochiti Rangelines CO-24 through CO-29 as performed for the sediment budget. These cross sections were modified and hydraulic models representing the future channel geometry and profile were developed for the analysis. Although the amount of degradation computed from applying the equilibrium slope at CO-30 (1.7×10^6 tons) is slightly less than the amount computed from sediment continuity (2.1×10^6 tons), it is reasonable to expect that the degradation rate will decrease over time as the channel approaches dynamic equilibrium. The historical and assumed future minimum channel elevation profiles are presented in **Figure A.19** and historical and assumed future channel cross section at Cochiti Rangeline CO-26 are shown in **Figure A.20**.

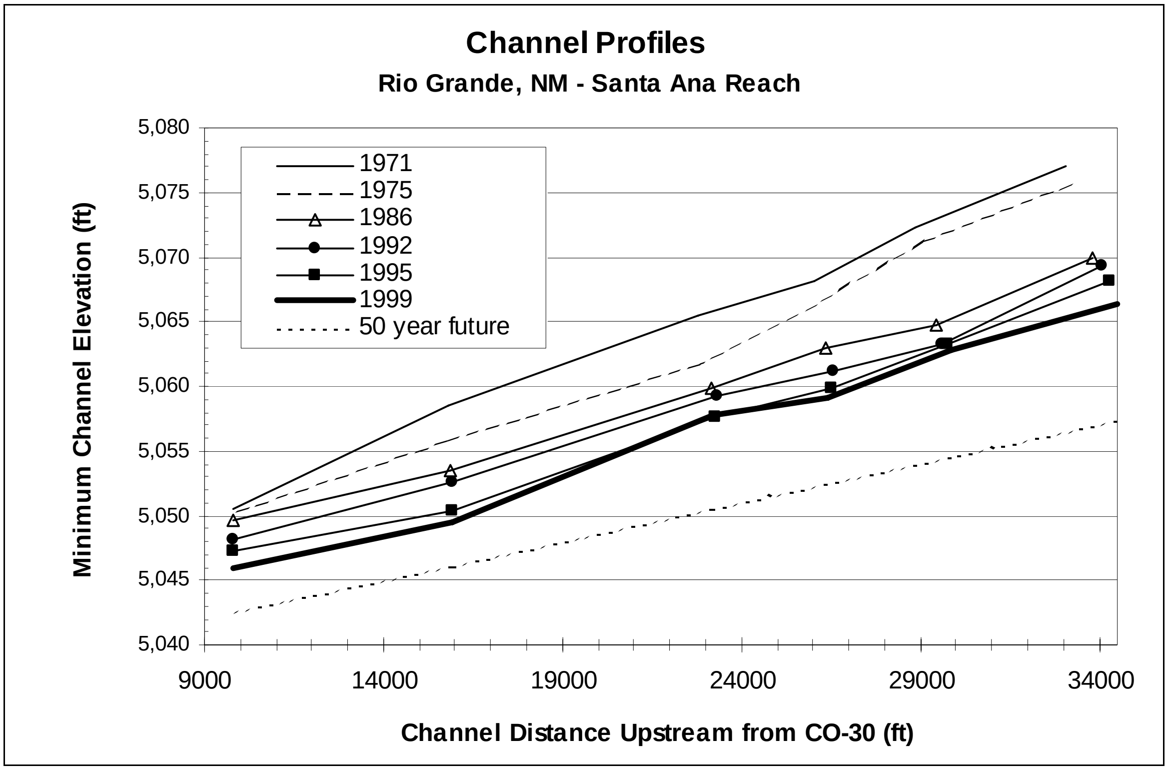

Figure A.19. Historical and predicted future profiles.

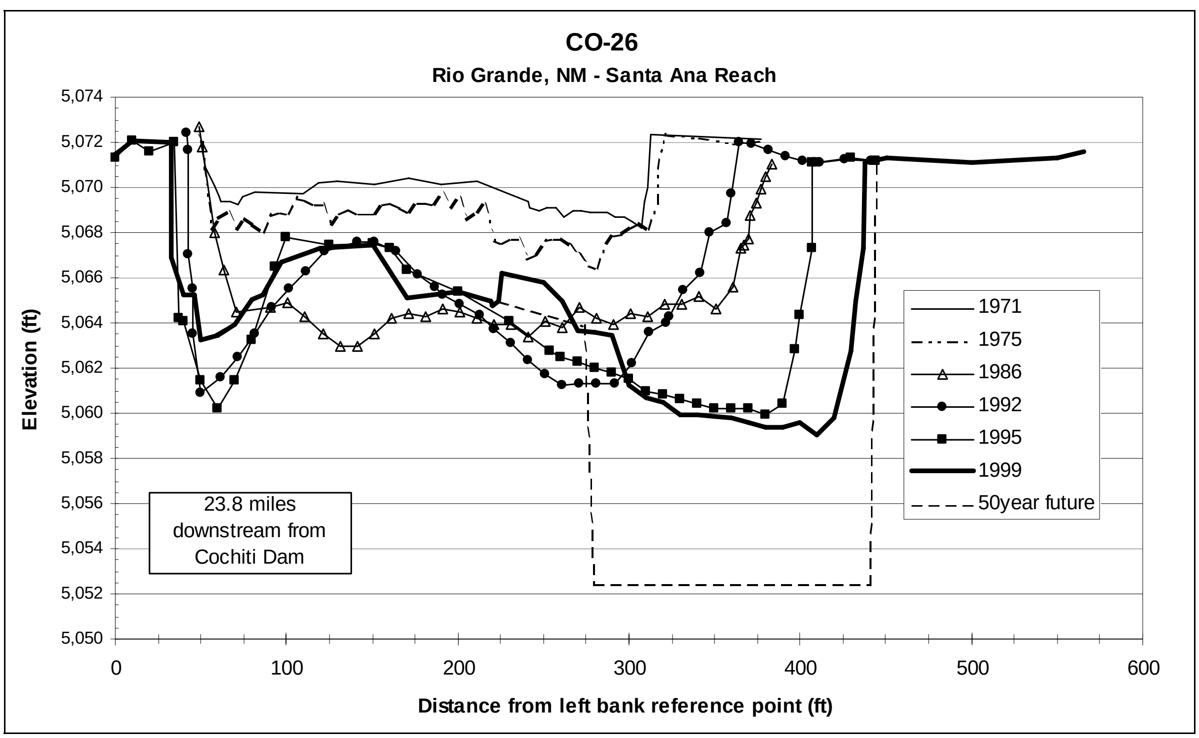

Figure A.20. Historical and assumed future cross section at CO-26.

For the equilibrium slope of 0.0006 an average of 6 feet of additional degradation could result throughout the Santa Ana Reach. As a result of this predicted incision, two gradient restoration facilities (GRFs) and a bedsill were constructed under a previous Section 1135 restoration project by the Albuquerque District COE, and one GRF was constructed by the U.S. Bureau of Reclamation, Albuquerque area. These grade control structures were designed to maintain an equilibrium channel slope through the Santa Ana reach for the current sediment inflows, and therefore prevent future channel degradation.

A.10 References

Ayres Associates, 2003. "Design Letter Report, Supporting Final Design and Preparation of Construction Plans and Specifications for the GRFs, Bed Sill, and Associated Vegetated Planting, Rio Grande Riparian and Wetland Restoration Section 1135 Project," prepared for the Pueblo of Santa Ana, February 7, 2003.

Baird, D. and Sanchez, V. 1996. "River Channel Changes Downstream of Cochiti Dam. Middle Rio Grande, New Mexico," Proceedings of the Conference of Management of Landscapes Disturbed by Channel Incision, University of Mississippi, Oxford, MS.

Bullard, K.L. and Lane, W.L., 1993. "Middle Rio Grande Peak Flow Frequency Study," U.S. Department of the Interior, Bureau of Reclamation, Technical Service Center, Flood Hydrology Group, Denver, CO.

Copeland, R.R., 1994. "Application of Channel Stability Methods – Case Studies," TR HL-94-11, USACE Waterways Experiment Station, Vicksburg, MS.

Chow, V.T., 1959. "Open Channel Hydraulics," McGraw-Hill Book Company, New York, NY.

Julien, P.Y. and Wargadalam, J., 1995. "Alluvial Channel Geometry: Theory and Applications," ASCE Journal of Hydraulic Engineering, Vol. 121(4), pp. 312-325.

Julien, P., Bauer, T., Leon, C., and Richard, G., 1999. "Middle Rio Grande Cochiti to Bernalillo Bridge Hydraulic Geometry, Discharge and Sediment Data Base and Report," prepared for U.S. Bureau of Reclamation, Albuquerque, NM, prepared by Colorado State University Engineering Research Center, Fort Collins, CO, January.

Nordin, C.F. and Beverage, J.P., 1965. "Sediment Transport in the Rio Grande, New Mexico," USGS Professional Paper 462-F.

HEC, 1998. "HEC-RAS - River Analysis System, Version 2.2, User's Manual" U.S. Army Corps of Engineers, Hydrologic Engineering Center, Davis, CA.

FLO Engineering, 1998. " San Felipe and Albuquerque Gage Analysis Sediment Data Analysis," prepared for the U.S. Bureau of Reclamation under contract No. 1425-6-CA-40-1730A Delivery Order 8, Breckenridge, CO.

Leopold, L.B., Wolman, M.G., and Miller, J.P., 1964, "Fluvial Processes in Geomorphology," Dover Publications, NY.

Lane, E.W. and Borland, W.M., 1953. "River Bed Scour During Floods," Transactions, ASCE, Paper No. 2712

Mussetter, R.A., Lagasse, P.F., and Harvey, M.D., 1994. "Sediment and Erosion Guide," prepared for the Albuquerque Metropolitan Flood Control Authority (AMAFCA), prepared by Resource Consultants & Engineers, Inc., Fort Collins, CO.

Waterways Experiment Station, 1992. "Hydraulic Design Package for Channels (SAM)," Hydraulics Laboratory Department of the Army Waterways Experiment Station Vicksburg, MS.

Woodson, R.C., 1961. "Stabilization of the Middle Rio Grande in New Mexico," ASCE Journal of the Waterways and Harbors Division, WW4, paper 2980, November.

Lagasse, P.F., 1980. "An Assessment of the Response of the Rio Grande to Dam Construction – Cochiti to Isleta Reach," A Technical Report for the U.S. Army Corps of Engineers, Albuquerque District, Albuquerque, NM, December.

Knighton, D., 1998. Fluvial Forms and Processes, A New Perspective," John Wiley & Sons, Inc., New York, NY.

Salazar, C.L., 1998. "Morphology of the Middle Rio Grande from Cochiti to Bernalillo Bridge, New Mexico," Master's Thesis Colorado State University.

Watson, C.W., Biedenharn, D.S., and Scott, S.H., "Channel Rehabilitation: Processes, Design and Implementation," presented by U.S. Army Engineer, Engineer Research and Development Center, Vicksburg, MS, Draft, July.

TECHNICAL APPENDIX B

HYDROLOGY

In Support of Section 1135
Aquatic Habitat Restoration at
Santa Ana Pueblo, New Mexico

Prepared for

Pueblo of Santa Ana
Natural Resources Department
221 Ranchitos Road
Bernalillo, New Mexico 87004

AYRES ASSOCIATES
3665 JFK Parkway, Building 2, Suite 200
Fort Collins, Colorado 80526
970.223.5556, FAX 970.223.5578

Ayres Project No. 32-0615.04
SAOL9AB.DOC

September 2006

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

B. HYDROLOGY

The hydrology of the Middle Rio Grande was reviewed to estimate the magnitude and frequency of flowrates expected in the Santa Ana Reach. This review focused on post-dam (1974 to present) hydrology. A discussion of historical (pre-dam) hydrology is provided in Appendix A. The flow information in this section was used for hydraulic modeling of the restoration alternatives. A range of flows were modeled to evaluate the aquatic and riparian habitat developed by each alternative.

B.1 Watershed Description

The reach of interest is designated as the Santa Ana Reach of the Rio Grande and includes the river in the vicinity of the Angostura Diversion to the Highway 550 Bridge in Bernalillo as shown in **Figure B.1**. The Santa Ana Reach is located in the Middle Rio Grande Basin approximately 25 river miles downstream of Cochiti Reservoir. The total drainage area of the Santa Ana Reach is approximately 16,400 square miles with a contributing drainage area of approximately 13,500 square miles. Approximately 2,900 square miles of the total drainage area exists in a closed basin in San Luis Valley, Colorado. The contributing watershed originates in southern Colorado and portions of northern New Mexico. The upstream watershed includes flows originating from snowmelt in southern Colorado and tributary flows in New Mexico.

Several flood control reservoirs exist in the watershed upstream of the Santa Ana Reach including: El Vado, Abiquiu, Galisteo, Cochiti and Jemez Reservoirs. Of greatest significance to the Santa Ana Reach is the Cochiti Reservoir that was constructed on the main stem of the Rio Grande in 1973. Cochiti Reservoir has 492,000 acre-feet of flood control storage and 110,000 acre-feet of sediment storage. Second in significance is Jemez Reservoir, which has had an adverse impact by significantly reducing the sediment supply. The reservoirs act to reduce sediment and peak discharges and therefore the Santa Ana Reach hydrology cannot be characterized without considering the effects of regulation.

Downstream of the Cochiti Reservoir there are numerous tributaries that can contribute discharges to the Santa Ana Reach. The tributaries are mostly uncontrolled and ephemeral, but some include perennial characteristics in higher elevations. Tributary dams downstream of Cochiti that also may control flood flows are on Galisteo Creek and the Jemez River. Water and sediment delivery from the uncontrolled tributaries primarily results from rainfall events. Some of the major unregulated tributaries and their drainage areas downstream of Cochiti Reservoir include the Peralta Canyon (56 mi^2), Borrego Canyon (117 mi^2), Arroyo De La Vega Los Tanos (21 mi^2), Las Huertas Creek (30 mi^2), and Tonque Arroyo (192 mi^2) as illustrated in Figure B.1. The drainage areas were obtained from "Analysis of Possible Channel Improvements to the Rio Grande from Albuquerque to Elephant Butte Lake" (USACE 1994).

Additional features of significance in the upstream watershed include the Angostura diversion, located immediately upstream of the Santa Ana Reach, and the Algodones riverside drain located approximately 2 miles downstream of Tonque Arroyo. The Angostura Diversion currently draws approximately 250 - 300 cfs from the Rio Grande (personal correspondence with David Gensler at Middle Rio Grande Conservancy District, 09/1999). Discussions on future operations of the Angostura Diversion indicate that withdrawal rates may increase, but the magnitude of this increase should have negligible impacts on the restoration plan for the Santa Ana Reach.

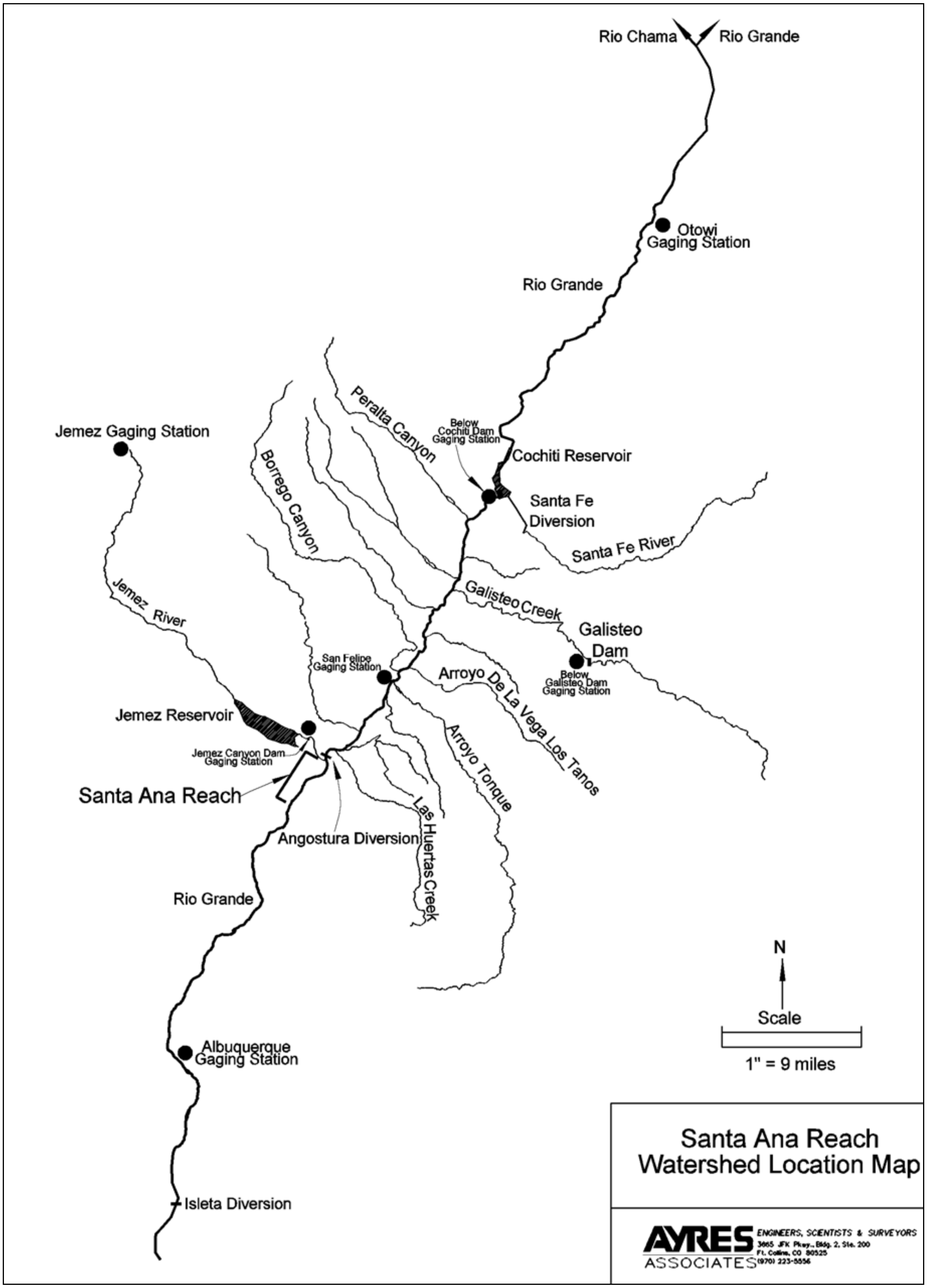

Figure B.1. Watershed Location Map.

B.2 Hydrologic Data Sources

The hydrologic investigation utilized information from regional stream flow gages and results from previous hydrologic studies performed on the Rio Grande. There are three USGS gaging stations in the vicinity of the Santa Ana Reach of the Rio Grande. The gages are identified as stations at San Felipe (station # 08319000), Albuquerque (station # 08330000) and below Jemez Dam (station # 08329000) as shown on Figure B.1. The San Felipe and Albuquerque gages record flow rates on the Rio Grande main stem and the Jemez Dam gage records flows on the Jemez River out of Jemez Reservoir. The San Felipe gaging station is approximately 7.5 river miles upstream of the Jemez River confluence on the Rio Grande. Mean daily flow values from the San Felipe gages were available for water years 1930 through 1999. The water year begins on October 1 of the previous calendar year and ends on September 30. The Albuquerque gaging station is approximately 20 river miles downstream of the Highway 550 bridge in Bernalillo and included mean daily flow values for water years 1942 through 1999. The Jemez Dam gage is less than 1 river mile downstream from the outlet of Jemez Dam and included mean daily flow values from 1943 through 1999. Records from the three gages also included annual peak flow values for each water year. Flow distributions and peak flows recorded at these gages are presented and discussed in Sections B.5 and B.6.

Analyses for peak flow recurrence intervals were available from previous studies conducted by the U.S. Bureau of Reclamation (USBR) and U.S. Army Corps of Engineers (USACE). The peak flow studies include "Middle Rio Grande Peak Flow Frequency Study" (Bullard and Lane 1993) and "Middle Rio Grande Flood Protection, Bernalillo to Belen, New Mexico, General Design Memorandum No. 1 Main Report," (USACE 1986). The peak flow values for the USACE report have been updated and were included in a memorandum distributed by Bruce Beach of the USACE, Albuquerque District (Beach 1997).

B.3 Methodology

Seasonal and average annual hydrologic occurrences were evaluated using statistical analysis of mean daily flow values from the gages at San Felipe and Albuquerque. This analysis focuses on the post-dam period following construction of Cochiti Dam. Historic flows and the hydrologic effects of regulation are described in Appendix A, Middle Rio Grande Santa Ana Reach - Geomorphology. Observation of flow values from the Jemez Dam Gage revealed that the average contribution from the Jemez River watershed was negligible compared to flows on the Rio Grande. Therefore the Jemez Dam gage data was not included in the current study. However contributions from the Jemez River watershed are implicitly included in data recorded at the Albuquerque gage. Because the San Felipe gage is upstream of the Santa Ana Reach and the Albuquerque gage is downstream, these two gages are considered representative of the approximate discharge in the Santa Ana Reach. Mean daily discharges from the San Felipe and Albuquerque gages provided data for developing average annual hydrographs, flow duration curves and probability distributions of observed flows on the Rio Grande. These data provided information to the day-to day and seasonal hydrologic fluctuations on the Rio Grande.

Flood frequencies for peak flows were obtained from previous studies conducted by the USBR and USACE. These studies include flood frequency analyses of gage data and hydrologic modeling of ungaged tributaries to the Rio Grande.

B.4 Average Annual Hydrographs

The Middle Rio Grande follows a pattern of high flows during spring runoff and low flows during the fall and winter months. Additional high flows result from thunderstorms that occur in late summer months. This can be observed from annual average hydrographs from the San Felipe and Albuquerque gages. Average annual hydrographs were developed by averaging mean daily flows from the gages of the post-dam era (water years 1974 – 1999). The average annual hydrographs for the two gages are presented in **Figure B.2.**

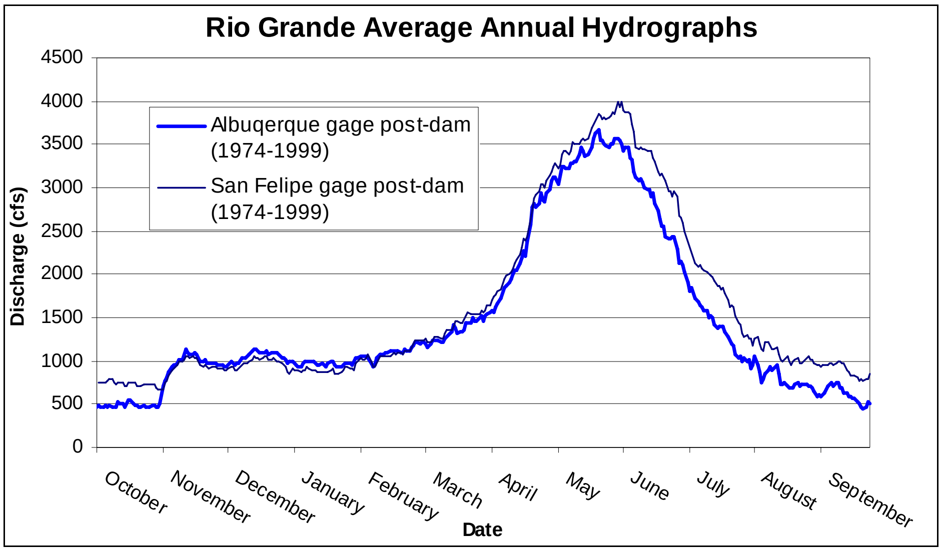

Figure B.2. Average annual hydrographs at San Felipe and Albuquerque gages.

The hydrographs indicate that the seasonal peak discharge usually occurs in late-May to early- June from snowmelt originating in Colorado. Average maximum daily discharges range from approximately 3,500 to 4,000 cfs. The average winter base flow of approximately 1,000 cfs usually persists from November to March and the lowest average flows have been observed in October. The average annual hydrographs represent approximately 1 million acre-ft of water per year.

B.5 Mean Daily Flow Statistics

Stream flow variability can also be evaluated with a flow-duration curve and probability distribution. Development of post-dam flow-duration curves was performed using mean daily stream flow data from the USGS gages at San Felipe and Albuquerque. Mean daily flow values from the post-dam era (water years 1974 – 1999) were ranked and each value was divided by the number of observations to compute the percent time exceeded as in the following equation:

$$\%\,\text{Exceedence} = \frac{m}{n} \qquad \text{(B.1)}$$

where:

m = Daily flow rank
n = Number of observations

Post-dam flow duration curves for the San Felipe and Albuquerque gages are presented in **Figure B.3** and **Table B.1.**

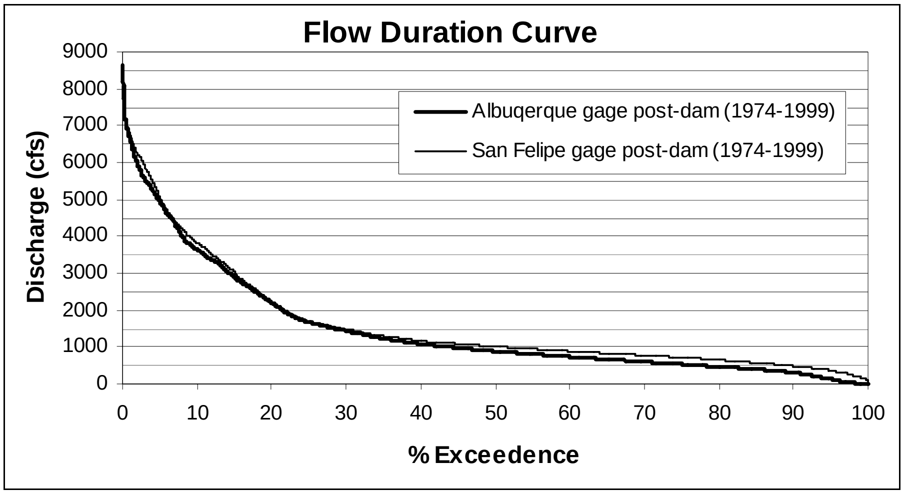

Figure B.3. Post-Dam flow-duration curve for San Felipe and Albuquerque gages.

Table B.1. Post-Dam Flow Duration Curve Values for San Felipe and Albuquerque Gages.		
Q (cfs)	Albuquerque Gage	San Felipe Gage
	% exceedence	% exceedence
9,000	0.0	0.00
8,000	0.1	0.02
7,000	0.5	0.6
6,000	1.9	2.7
5,000	4.7	5.1
4,000	8.0	8.8
2,000	21.4	21.7
1,000	43.7	50.5
750	58.9	72.0
500	77.1	89.1
200	93.0	98.6
100	95.7	100.0

Additionally a discrete probability distribution was developed from the post-dam gage record. This was accomplished by determining the number of occurrences in a particular flow range and dividing by the number of observations. The probability distribution can illustrate the percent of time a particular flow range has occurred. The probability distributions developed from the San Felipe and Albuquerque gages are presented in **Figure B.4** and **Table B.2**.

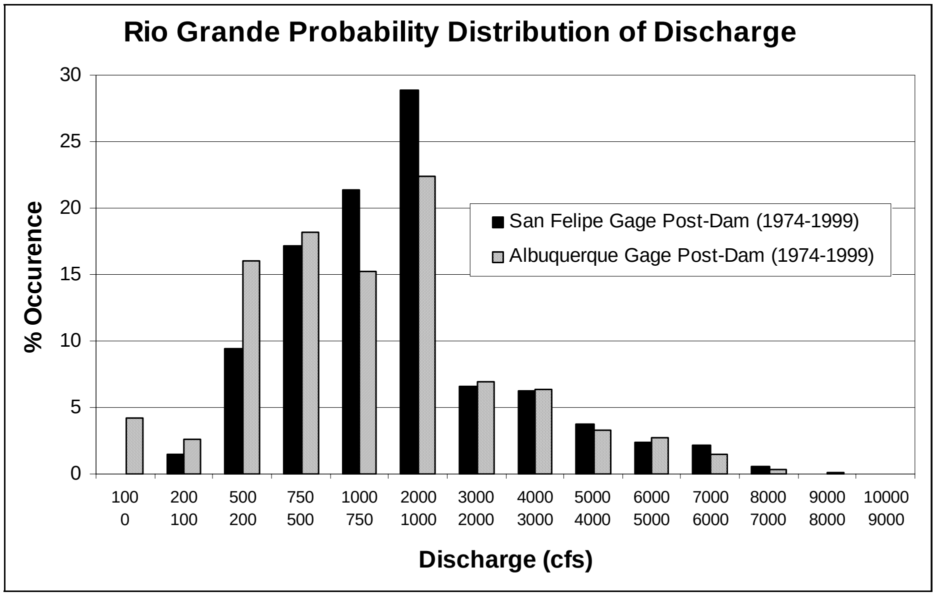

Figure B.4. Probability Distribution for Mean Daily Flows at San Felipe and Albuquerque Gages.

Table B.2. Probability Distributions of Mean Daily Flows at San Felipe and Albuquerque Gages.

Discharge Range (cfs)		Percent Occurrence	
Qmin	Qmax	Albuquerque Gage	San Felipe Gage
0	100	4.22	0.01
100	200	2.66	1.44
200	500	15.98	9.45
500	750	18.16	17.15
750	1,000	15.24	21.42
1,000	2,000	22.37	28.88
2,000	3,000	6.98	6.55
3,000	4,000	6.37	6.26
4,000	5,000	3.29	3.75
5,000	6,000	2.77	2.39
6,000	7,000	1.49	2.15
7,000	8,000	0.34	0.54
8,000	9,000	0.14	0.01
9,000	10,000	0.00	0.00

The probability distribution and flow-duration curve illustrate that flows less than 1,000 cfs have been observed approximately 50% of the time following construction of Cochiti Dam. This range corresponds to the low flow fall-winter-spring months, which represent approximately half of the water year. Discharge values in the 1,000 to 2,000 cfs have been observed approximately 20 to 30% of the time and flows in excess of 2,000 cfs have been observed approximately 20% of the time on average. The higher flows (>2,000 cfs) represent the runoff season, which typically includes a duration of approximately 2.5 months of the water year.

B.6 Peak Flow Analysis

Flood frequency statistics were obtained from previous studies conducted by the USBR (Bullard and Lane) and USACE (Beach 1997). These studies included flood frequency analyses and hydrologic modeling to provide estimates of flood flows for specific recurrence intervals.

The USBR study applied a procedure to develop discharge values for regulated (dam) and unregulated (no-dam) conditions using historic gage data and typical dam operations. Discharge values were developed for gage locations on the Rio Grande. Peak flows from the USBR study at the San Felipe and Albuquerque gages are listed in **Table B.3**.

Table B.3. USBR Flood Flow Discharges (cfs) for Regulated Conditions.

Return Period	San Felipe	Albuquerque
2-year	5,650	4,820
5-year	9,330	7,450
10-year	10,000	9,090
25-year	10,000	10,000
50-year	10,000	10,000
100-year	10,000	10,000

The USBR study assumed that Cochiti Dam would be operated such that the maximum release would not exceed 10,000 cfs. Therefore low frequency discharges at the San Felipe and Albuquerque gages were limited to this regulation value. The approach is somewhat controversial in that flows resulting from rainfall events in some of the tributaries downstream of Cochiti were not included.

The peak flow analyses provided by the USACE included regulation and inputs from the tributaries downstream of Cochiti Reservoir. The USACE analysis provided values specific to Bernalillo and the Santa Ana Reach. The discharges correspond to the Middle Rio Grande from the confluence with the Jemez River to approximately 4 miles downstream of the Highway 550 bridge. These values will be used the for design hydrology and for analyses of restoration alternatives. The USACE discharge frequency values are listed in **Table B.4**.

Table B.4. USACE Discharge Frequency Values for Santa Ana Reach.

Return Period	Discharge (cfs)
2-year	5,400
5-year	7,000
10-year	7,100
25-year	10,800
50-year	15,300
100-year	22,300
500-year	44,000

B.7 References

Beach, B.C., 1997. "Belen LRR, Discharge Frequency Values, Without Expected Probability," Memorandum for the Record dated 11/14/97 received 10/18/99, U.S. Army Corps of Engineers, Albuquerque District.

Bullard, K.L. and Lane, W.L., 1993. "Middle Rio Grande Peak Flow Frequency Study," U.S. Department of the Interior, Bureau of Reclamation, Technical Service Center, Flood Hydrology Group, Denver, CO.

USACE, 1986. "Middle Rio Grande Flood Protection, Bernalillo to Belen, New Mexico, General Design Memorandum No. 1, Main Report," U.S. Army Corps of Engineers, Albuquerque District, April.

USACE, 1994. "Analysis of Possible Channel Improvements to the Rio Grande from Albuquerque to Elephant Lake Phase 1A Sediment Yield Analysis from the Rio Grande Tributary Basins," U.S. Army Corps of Engineers, Albuquerque District, January.