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UPPER RIO GRANDE WATER OPERATIONS MODEL PHYSICAL MODEL DOCUMENTATION: TECHNICAL REVIEW COMMITTEE DRAFT

INTRODUCTION

This is the Technical Review Committee draft of the Upper Rio Grande Water Operations Model (URGWOM) physical model documentation. This document describes the data, methods, and assumptions used by RiverWare© to simulate the hydrology and operation of the Rio Grande Basin between Lobatos, Colorado, and Elephant Butte Reservoir. Additional model documentation will be developed as model development continues in the remainder of the Rio Grande Basin between its headwaters and Fort Quitman, Texas. The draft document is intended to accompany the model during the transition between development and operation that will take place during 2000. The URGWOM documentation is considered a working document to be revised when technical review results in additional information or better data.

The documentation describes and analyzes river routing techniques and describes the development of travel time lags used in the variable time lag method, which is the adopted river routing method, and the methods and assumptions used to develop river-channel loss coefficients. The routing procedures and methods for estimating river-channel loss mechanisms are applied in the same manner for many reaches described in the document. Because the application of these techniques is repetitive, the results are summarized in tabular form for each reach. Plots of data used to develop travel times and loss rates are in appendix A.

UPPER RIVER

SUMMARY OF REACH ANALYSIS METHODS – ROUTING AND LOSSES

Comparison of Streamflow Routing Methods

Streamflow (or river) routing is a technique to compute the effect of channel storage on the shape (reduction in peak, or attenuation) and downstream movement (translation, or travel time) of a hydrograph. Standard empirical river routing methods are available in RiverWare, such as the Muskingum-Cunge, kinematic wave, Muskingum, variable time lag, time lag, variable Muskingum, storage (SSARR), variable storage, impulse response, and MacCormack methods. The theory of each of these methods can be found in other documents such as hydrology textbooks or engineering manuals developed by the U.S. Army Corps of Engineers (Corps), Bureau of Reclamation (USBR), or the U.S. Geological Survey (USGS). RiverWare has been evaluated to assure that the program computes the proper results for each routing method.

A river reach (the stretch of river between control points) from Rio Chama below El Vado Dam to Rio Chama above Abiquiu Reservoir was analyzed using several of these routing methods. Not all routing methods available in RiverWare were used in this analysis (SSARR, impulse response, and MacCormack methods). The purpose was to determine an appropriate method to use for this reach and other reaches in the model and to compare the results of the different methods and the sensitivity of the results of one method with another. The period of data analyzed was 1962 to 1996 because 1962 is the beginning of the complete record for the gage above Abiquiu.

The routing coefficients and parameters for each method were estimated or optimized in the Hydrologic Engineering Center Hydrologic Modeling System (HEC-HMS), the Corps' reservoir computer model. The lag and Muskingum K values for the Muskingum, variable time lag, and time lag methods were determined using measured data for the Rio Chama gages below El Vado Dam and above Abiquiu Reservoir, as described in the next section, "Time lags based on wave velocity." The input data required for the Muskingum-Cunge and kinematic wave methods were determined from cross section data in a previous Corps study (1995). The reach length and energy slope input values were derived from information for each gage in USGS Water-Data Reports. The Manning's roughness coefficient, n (n-value), was initially estimated using the "Handbook of hydraulics" (Brater and King, 1976) and "Open channel hydraulics" (Chow, 1959) as references. HEC-HMS was then used to optimize the n-value for the Muskingum-Cunge method. The initial estimated n-value was 0.04, and HEC-HMS optimized the value to 0.05. An n-value of 0.05 was used in RiverWare for the Muskingum-Cunge and kinematic wave routing methods. The Muskingum X value was optimized to a value of 0.3 in HEC-HMS, when the Muskingum K value was locked at 12 hours. A Muskingum X of 0.3 was tried in RiverWare, resulting in negative flows. These were computed because negative coefficients are derived from the Muskingum method when K is much less than the computational time interval. Negative coefficients will result for short travel times, depending on the value of X. Because of the instability of the method for short routing reaches (travel time lags less than 24 hours), the X value was adjusted to 0.1 so that negative values would not be computed. The coefficients and parameters for each routing method are listed in **table 1**.

Table 1. Routing coefficients and parameters for the reach of the Rio Chama from below El Vado Dam to above Abiquiu Reservoir

Routing method	Reach length (miles)	Energy slope	Bottom width (feet)	Side slope (xH:1V)	Man-ning's n-value	Channel shape	Flow rate (cfs x 10^3)	Mus-kingum K or lag (hours)	Mus-Kingum X
Mus-kingum-Cunge	28.8	0.0027	70	2	0.05	Trape-zoid	---	---	---
Kine-matic wave	28.8	0.0027	70	2	0.05	Trape-zoid	---	---	---
Mus-kingum	---	---	---	---	---	---	---	12	0.1
Variable time lag	---	---	---	---	---	---	50	29	---
	---	---	---	---	---	---	200	15	---
	---	---	---	---	---	---	500	9	---
	---	---	---	---	---	---	750	8	---
	---	---	---	---	---	---	1000	7	---
	---	---	---	---	---	---	3000	4	---
	---	---	---	---	---	---	6000	3	---
½ -day time lag	---	---	---	---	---	---	---	12	---
1-day time lag	---	---	---	---	---	---	---	24	---

The model was then set up to run the various routing methods, and the results were plotted for comparison. The results of selected hydrographs compare all the methods. Periods of little or no local inflow were selected so that only water being released could be analyzed for routing. They are shown in **graphs 1-8** (all graphs are in **appendix A** unless it is otherwise stated).

To define the variable time lag method, the time lag method is first defined. The time lag method applies a single time lag to all flows. On a daily time step, a daily flow volume begins arriving downstream at the specified number of hours of lag and continues through a 24-hour period. If this lag is not an integer number of days, the volume of water is apportioned between 2 days. For example, if a volume of 100 cubic-foot-per-second (cfs)-days (100 cfs for a day) has an 8-hour time lag, it begins arriving at the eighth hour of the current day and ends at the eighth hour of the next day. Apportioned, this provides (16/24) x 100 = 67 cfs-days downstream the current day and (8/24) x 100 = 33 cfs-days the next day.

Although the variable time lag method uses the same procedure, it allows for breaking the total flow range into as many as 10 flow ranges, each with associated varying time lags. It also allows for as many as 12 seasons to be defined with different flow range/time lag sets.

The Muskingum-Cunge and variable time lag methods appear to provide the most consistent results for timing and matching peak flows for this reach. All the methods, however, give acceptable results except for the 1-day time lag, which tended to peak later than the other methods. Although the simple time lag routing method is also sometimes effective, the variable time lag method was chosen because of several considerations. First, using the same routing method everywhere has merit, and the variable time lag method provides greater resolution in reaches with tighter correlations. Second, the method is just as easy to develop. Third, the simple time lag routing method takes advantage of the known direct relation between velocity and flow, the benefits of which are realized during extended periods of high or low flows.

Time Lags Based on Wave Velocity

Wave velocity is also known as the Klietz-Seddon or Seddon law (Seddon, 1900). In 1900, J.A. Seddon published a paper regarding the computation of wave velocity. His study concentrated on unsteady flow in rivers. He concluded that the wave velocity, $V_{w,}$ is equal to dQ/dA. By using the power relations developed from historical USGS cross section and measurement data, travel time relations (or time lags) can be computed for their corresponding reaches:

$$A=\alpha Q^{\beta}$$

where:

A = cross section area, in square feet (ft^2);
Q = discharge, in cfs; and
α and β = regression power coefficients and exponents.

The cross section area equation can be rearranged to solve flow as a function of area:

$$Q=\left(\frac{1}{\alpha}\right)^{\frac{1}{\beta}}A^{\frac{1}{\beta}}$$

Solving for dQ/dA:

$$\frac{dQ}{dA}=\frac{1}{\beta}\left(\frac{1}{\alpha}\right)^{\frac{1}{\beta}}\frac{A^{\frac{1}{\beta}}}{A}$$

since:

$$Q=\left(\frac{1}{\alpha}\right)^{\frac{1}{\beta}}A^{\frac{1}{\beta}}$$

substituting Q:

$$\frac{dQ}{dA}=\frac{1}{\beta}\frac{Q}{A}$$

also:

$$\frac{Q}{A} = V_{avg}$$

where:

V_{avg} is the average velocity, in feet per second (ft/s).

Therefore, substituting for Q/A:

$$\frac{dQ}{dA} = \frac{1}{\beta} V_{avg}$$

Because the wave velocity is equal to dQ/dA:

where:

$$V_w = \frac{dQ}{dA} = \frac{1}{\beta} V_{avg}$$

V_w is the wave velocity, in ft/s.

The lag time is then estimated by dividing the reach length by the wave velocity (with the appropriate conversions of time and distance units). Because the wave velocity varies with discharge, the travel time varies with flow. The relation of time lag as a function of discharge can then be incorporated into a table for use in the model. The following is an example calculation of the time lag for a reach length of 28.8 miles (mi) (Rio Chama from below El Vado Dam to above Abiquiu Reservoir).

First, by using data from USGS flow measurements, measured cross section area (sq. ft.) is plotted against stream discharge (cfs), and a power relation is derived (**fig. 1**).

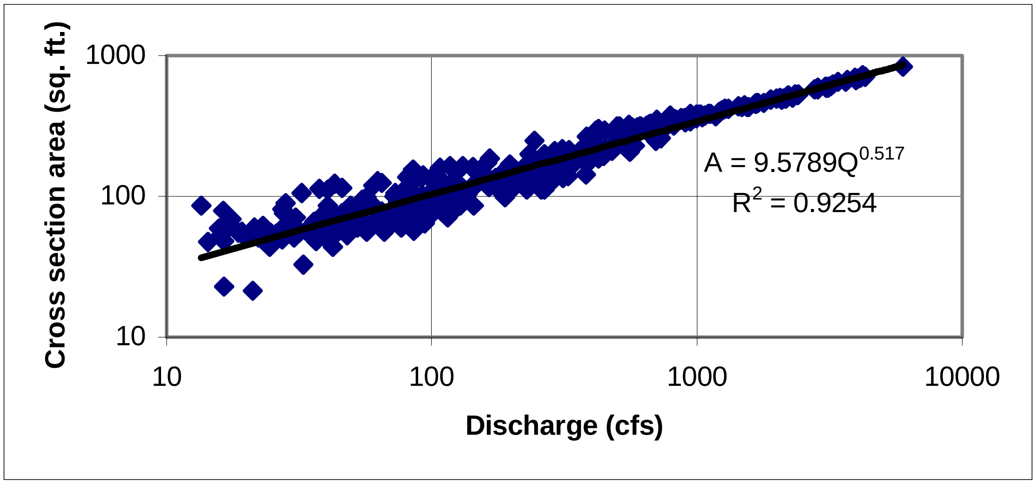

Figure 1. Discharge versus cross section area, Rio Chama below El Vado Dam, 1969-98.

From this power equation, β = 0.517. Each measured average velocity is then divided by β to derive the wave velocity for a given flow measurement:

$$V_w = \frac{V_{avg}}{\beta} = \frac{1.68}{0.517} = 3.25\, {ft}/{s}$$

Once the wave velocity is known, the time lag can be computed for a given flow by dividing reach length by wave velocity and making the appropriate conversions:

$$TL = \frac{L}{V_w}\frac{5280\frac{ft}{mi}}{3600\frac{s}{hr}} = \frac{28.8}{3.25}\frac{5280\frac{ft}{mi}}{3600\frac{s}{hr}} = 13.0hr$$

where:

TL = time lag, in hours; and
L = routing reach length, in mi.

Travel time versus discharge can then be plotted on a scatter graph and a power equation derived, as shown in **figure 2.**

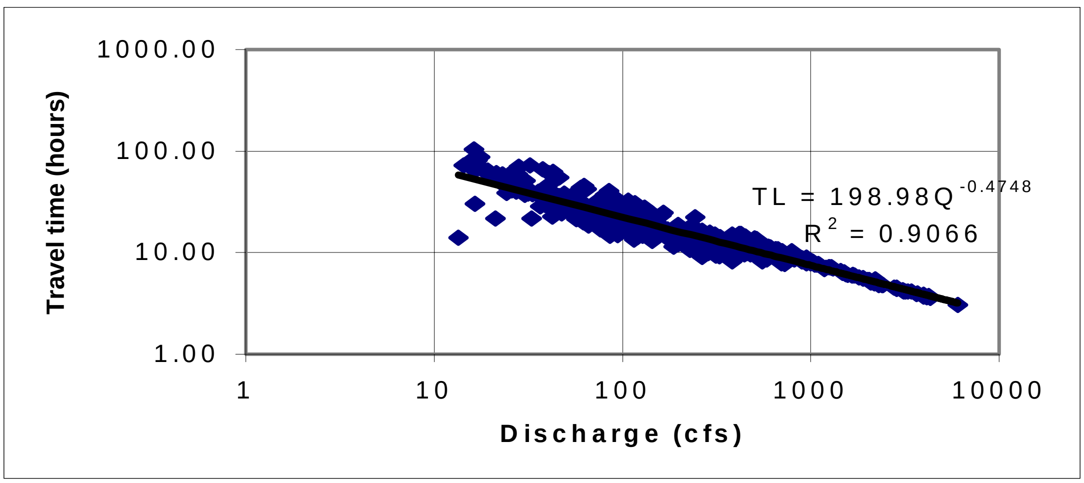

Figure 2. Travel time versus discharge, Rio Chama below El Vado Dam to above Abiquiu Reservoir (based on gage Rio Chama below El Vado Dam, 1968-98).

Because RiverWare does not accept power equations, a table of travel time versus flow rate must be developed for use in the model. Travel time for varying flow ranges can be calculated for use in the model using the power equation derived in **figure 2** (TL = $198.98Q^{-0.4748}$). **Table 2** uses the same example reach.

Table 2. Flow rates and travel times for the reach of the Rio Chama from below El Vado Dam to above Abiquiu Reservoir (based on gage Rio Chama below El Vado Dam)

Flow rate used to determine travel time (cfs)	Travel time (hours)
50	31
200	16
500	10
750	9
1000	7
3000	4
6000	3

When this procedure is used to estimate travel time lags, the gage cross section is assumed to be representative of the entire routing reach. If both upstream and downstream gage measurements are available, the results of the wave velocity analysis are averaged to represent the entire reach. The results are verified by analyzing the upstream and downstream hydrographs at various flow rates.

If no measurement or hydrograph data are available for a given reach, a ratio method can be used to derive a wave velocity. The average velocities can be calculated from Manning's equation with a representative cross section and varying discharges for the routing reach. For various channel shapes, the wave velocity has been found to be a direct ratio of the average velocity (**table 3**). For natural channels, a ratio of 1.5 is suggested.

Table 3. Wave velocity ratios for various channel shapes

Channel width	Ratio V_w/V_{avg}
Wide rectangular	1.67
Wide parabolic	1.44
Triangular	1.33

The procedure just described can also be used for estimating the K value required for the Muskingum routing method or for estimating a single or average time lag for use in the time lag routing method.

Reach Loss Analysis and Gains Determination

The following lists the steps necessary for implementing the variable time lag routing method to develop loss rates and local inflow (gains) in a reach.

1. Select an overall data set.

2. Model all significant human effects in the reach, including diversions.

3. Calibrate the variable time lag routing method for each reach using methods described in the previous section.

4. Create a routed hydrograph by routing the upstream-observed hydrograph using the overall data set.

5. Create a filtered data set to determine only loss relations; keep data for the days when routed flow is greater than downstream-observed flow in groups of 3 or more consecutive days.

6. Plot the (filtered) downstream-observed hydrograph versus the (filtered) routed hydrograph and perform a regression analysis on the data.

7. Create a monthly regression coefficient for each calendar month by using daily data in the regression analyses of the filtered data set. The slope of the linear regression line of best fit represents the loss coefficient. Regression lines of best fit are computed with y-intercepts and with the line forced through the zero y-intercept. On the basis of an analysis of loss rates computed using both regression lines and an analysis of the results, which were not significantly different, it was decided that the use of the y = 0 intercept regression line could be used.

8. Create a “routed with losses” hydrograph using the monthly regression coefficient -1 on the daily numbers (of the corresponding months), for the overall routed hydrograph.

9. Create a local inflow hydrograph that represents gains within the reach by subtracting the routed with losses hydrograph from the downstream-observed flow hydrograph, both for the overall data set.

10. (Optional) Smooth the local inflow hydrograph to minimize large negative daily local inflows using a moving average technique.

RIO CHAMA REACHES

See **figure 3** for the location of reaches used in the Rio Chama Basin.

Willow Creek above Heron Reservoir

Although the reach of Willow Creek between the Azotea Tunnel portal and Heron Reservoir is simulated in the physical model, neither natural flows nor San Juan-Chama Project water was routed through this reach. A fixed loss rate is applied to San Juan-Chama Project water between the Azotea Tunnel portal and Heron Reservoir.

This reach flows down a short reach of Azotea Creek and a portion of Willow Creek for about 12 mi at a slope of about 25 feet per mile (ft/mi). The channel varies from 30 to 65 ft in width and is characterized as a mountainous stream.

Rio Chama from Heron Reservoir to above El Vado Reservoir

This reach of the Rio Chama is not included in the model because of the lack of data that is necessary to develop travel time lags and river-channel losses. In addition, no need has been identified that would require the inclusion of this reach. The Natural Resources Conservation Service/National Weather Service runoff forecast point is at the inflow to El Vado Reservoir, which precludes the need for river routing above this point.

Rio Chama from below El Vado Dam to above Abiquiu Reservoir

The inflow to this reach is water released from El Vado Reservoir, which is measured at a gaging station 1.5 mi downstream from the dam (Rio Chama below El Vado Dam). The downstream end of the reach is the gage above Abiquiu Reservoir (Rio Chama above Abiquiu Reservoir). The reach is 28.8 mi long. The upper portion of this reach is a canyon section with a rocky, narrow river channel and flood plain. The lower 6 mi of this reach flows through a broad alluvial plain that supports a small amount of irrigable land and a riparian bosque.

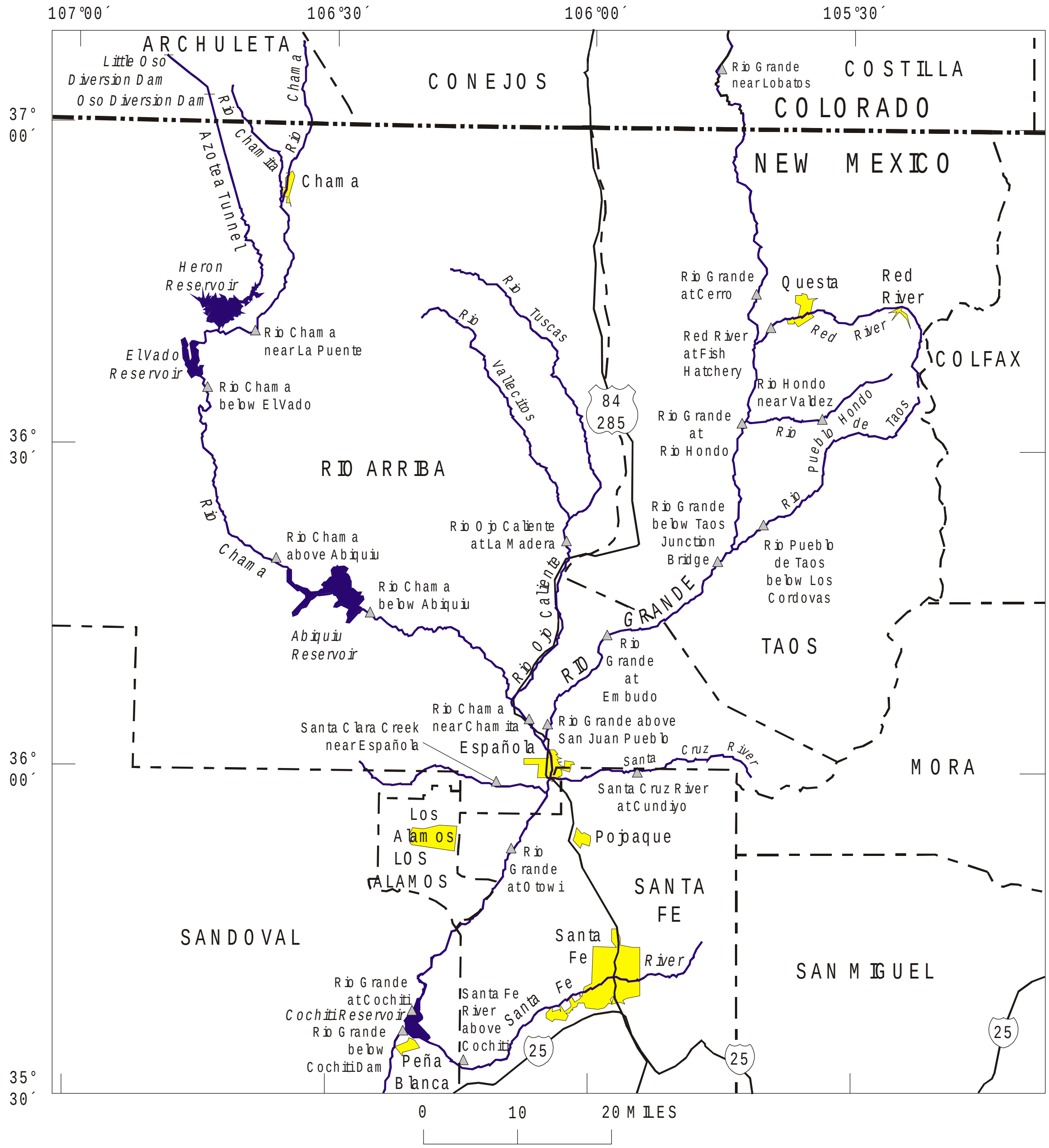

Figure 3. Rio Grande from Lobatos, Colorado, to Cochiti, New Mexico.

The reach between the gage above Abiquiu Reservoir and Abiquiu Dam is about 15.3 mi long. However, the distance to the headwaters of the reservoir at the top of the existing storage easement (elevation 6220 ft) is only about 4 mi and at the top of the flood control (elevation 6283.5 ft) is less than 2 mi. Because of the short length to the head of the reservoir during normal operations, the reach from the gage above Abiquiu to Abiquiu Reservoir will not include any routing or losses.

Table 4 summarizes stream-gage and calibration data used in determining the power coefficient (wave velocity exponent) that is applied to average velocity measurements to determine river travel time lags in this reach (**fig. 1 and graph 9**).

Table 4. Summary of stream-gage and calibration data for the reach of the Rio Chama from below El Vado Dam to above Abiquiu Reservoir

	Rio Chama below El Vado Dam	Rio Chama above Abiquiu Reservoir	Total Δ
Period of analysis	10/14/1969 – 8/9/1998	8/28/1969 – 8/5/1998	
River mile (above mouth)	76.2	47.4	28.8
Elevation (feet above sea level)	6696	6280	416
Drainage area (square miles)	877	1600	723
Number of measurements	446	373	
Wave velocity exponent (β)	0.517	0.486	
Coefficient of determination (R^2)	0.9254	0.934	

Table 5 summarizes factors used to determine travel time lags for this reach (**fig. 2 and graph 10).**

Table 5. Adopted travel time lags (TL) for the reach of the Rio Chama from below El Vado Dam to above Abiquiu Reservoir

			Time lag (hours) for indicated flow rate (cfs)						
Gage	TL vs. Q equation	R^2	50	200	500	750	1000	3000	6000
El Vado	$TL = 198.98Q^{-0.4748}$	0.907	31	16	10	9	7	4	3
Abiquiu	$TL = 206.53Q^{-0.5137}$	0.9413	28	14	8	7	6	3	2
Adopted travel times for reach→			29	15	9	8	7	4	3

Once the routing parameters were determined and verified for the reaches of the Rio Grande above Cochiti, the loss expected in each reach was analyzed using the filtering procedure described previously. These losses represent losses of flow in the reach from surface-water evaporation, seepage to ground water, and bank storage. The river losses are combined in these reaches because of the lack of readily available data for each of these effects. If data to isolate these losses become available in the future, these loss coefficients can be revised. In RiverWare, the loss coefficient is multiplied by the routed flow, and the resulting value is subtracted from the upstream-routed flow, resulting in a loss at the downstream point.

Sixteen data points were removed from the analysis of wave velocity and travel time using the gage Rio Chama below El Vado Dam because of errors in the measurement data or in the reporting of the measurement data. Some data points were removed from the loss analysis because of timing errors in the routed hydrograph or possible measurement error at one or both of the upstream and downstream gages. Four points were removed from the analysis that use the gage above Abiquiu Reservoir.

Table 6 summarizes the adopted loss coefficients for this reach (**graphs 11-22**).

Table 6. Correlations between routed flow and observed flow and adopted monthly loss coefficients for the reach of the Rio Chama from below El Vado Dam to above Abiquiu Reservoir, 1962-96

Month	n (days)	Slope	y-intercept	R^2	Slope (y=0)	R^2	Adopted monthly loss coefficient
Jan	156	0.988	-7.26	0.996	0.966	0.995	-0.03
Feb	53	0.988	-4.44	0.996	0.966	0.995	-0.03
Mar	93	0.964	-1.75	0.995	0.960	0.995	-0.04
April	84	0.974	-9.67	0.998	0.964	0.998	-0.04
May	162	0.974	-19.61	0.999	0.965	0.998	-0.04
June	347	0.969	-17.74	0.990	0.946	0.990	-0.05
July	363	0.963	-13.97	0.989	0.942	0.988	-0.06
Aug	210	0.975	-19.76	0.991	0.946	0.990	-0.05
Sept	284	0.974	-5.69	0.995	0.962	0.995	-0.04
Oct	227	0.978	-5.78	0.996	0.958	0.995	-0.04
Nov	254	0.980	-4.29	0.999	0.973	0.999	-0.03
Dec	217	0.988	-7.98	0.999	0.980	0.999	-0.02

Rio Chama from below Abiquiu Dam to near Chamita

This reach of the river is 28.5 mi long and includes numerous agricultural diversions. Inflow to this reach is water released from Abiquiu Dam, as recorded by the gage Rio Chama below Abiquiu Dam. Outflow is measured at the gage Rio Chama near Chamita. The only diversion data (monthly) available are for the Rio Chama main-stream acequias from 1971 to 1985**.** These data were uniformly converted to daily values for each measured diversion. No data are available for return flows, so 50 percent of the diversion was assumed to return to the Rio Chama.

The Rio Ojo Caliente, a major tributary to this reach, discharges into the Rio Chama about 6 mi above its mouth. This tributary is not included in the river routing in this reach because of the lack of data needed to reliably estimate losses between the gage Rio Ojo Caliente at La Madera (20 mi above mouth) and the Rio Chama confluence. About 500 acres of land can be irrigated from the Rio Ojo Caliente below the gage at La Madera. Discharges to the Rio Chama during spring runoff can be significant, and the lack of reliable estimates of this discharge to the Rio Chama complicates the reliability of loss estimates for the Abiquiu to Chamita reach. El Rito Creek, which discharges into the Rio Chama about 16 mi above the mouth of Rio Chama, is not represented in the model because of similar circumstances.

Table 7 summarizes stream-gage and calibration data used in determining the power coefficient (wave velocity exponent) that is applied to average velocity measurements to determine river travel time lags in this reach (**graphs 23 and 24**)**.**

Table 7. Summary of stream-gage and calibration data for the reach of the Rio Chama from below Abiquiu Dam to near Chamita

	Rio Chama below Abiquiu Dam	Rio Chama near Chamita	Total Δ
Period of analysis	4/15/70 – 8/5/98	7/2/69 – 8/5/98	
River mile (above mouth)	31.3	2.8	28.5
Elevation (feet above sea level)	6040	5654	386
Drainage area (square miles)	2147	3144	997
Number of measurements	332	410	
Wave velocity exponent (β)	0.587	0.620	
Coefficient of determination (R^2)	0.9362	0.906	

Table 8 summarizes factors used to determine travel time lags for this reach (**graphs 25 and 26**).

Table 8. Adopted travel time lags (TL) for the reach of the Rio Chama from below Abiquiu Dam to near Chamita

			Time lag (hours) for indicated flow rate (cfs)						
Gage	TL vs. Q equation	R^2	50	200	500	750	1000	3000	6000
Abiquiu	$TL = 132.75Q^{-0.414}$	0.877	26	15	10	9	8	5	4
Chamita	$TL = 94.68Q^{-0.380}$	0.783	21	13	9	8	7	5	3
	Adopted travel times for reach→		24	14	10	8	7	5	4

The same procedure for determining the losses for the reach from below El Vado Dam to above Abiquiu Reservoir was applied to determine losses for this reach. Isolating the losses for this reach is less reliable because of the uniform distribution of monthly diversions and substantial unmeasured tributary inflow. The assumed 50-percent return flow is for each of the Rio Chama main-stream section ditches. Therefore, the results of the regression analyses for the irrigation months (April through October) do not result in the expected pattern of seasonal losses, such as the results for the reach from below El Vado Dam to above Abiquiu Reservoir (see computed monthly loss coefficients for the reach from below Abiquiu Dam to near Chamita in **table 9**). The pattern of the results for winter (December through February) is the same as that for the reach from below El Vado Dam to above Abiquiu Reservoir, each month being -0.01 lower for the reach from below Abiquiu Dam to near Chamita, and the other nonirrigation months (November and March) being -0.02 lower (**graphs 27-38**). Because of the assumptions made for the distribution of monthly diversion data and return flow and the limited data available (1973-85), the adopted monthly loss coefficients for the reach from below Abiquiu Dam to near Chamita are based on adding -0.01 to the values for the reach from below El Vado Dam to above Abiquiu Reservoir. **Table 9** summarizes the computed and adopted loss coefficients for this reach.

Table 9. Correlations between routed flow and observed flow and adopted monthly loss coefficients for the reach of the Rio Chama from below Abiquiu Dam to near Chamita, 1973-96

Month	N (days)	Slope	y-inter-cept	R^2	Slope (y=0)	R^2	Computed loss coefficient	Adopted loss coefficient, below El Vado to above Abiquiu	Adopted loss coefficient
Jan	25	0.961	0.325	0.993	0.962	0.993	-0.04	-0.03	-0.04
Feb	34	0.979	-35.29	0.992	0.956	0.992	-0.04	-0.03	-0.04
Mar	67	0.934	10.63	0.996	0.941	0.996	-0.06	-0.04	-0.05
April	9	0.910	11.09	0.991	0.917	0.991	-0.08	-0.04	-0.05
May	52	0.989	-51.34	0.989	0.938	0.986	-0.06	-0.04	-0.05
June	200	0.979	-35.61	0.996	0.958	0.995	-0.04	-0.05	-0.06
July	228	0.968	-31.46	0.994	0.943	0.993	-0.06	-0.06	-0.07
Aug	138	0.980	-36.95	0.990	0.939	0.988	-0.06	-0.05	-0.06
Sept	144	0.930	-2.70	0.969	0.926	0.969	-0.07	-0.04	-0.05
Oct	61	0.968	-10.91	0.992	0.952	0.992	-0.05	-0.04	-0.05
Nov	73	0.943	2.13	0.998	0.946	0.998	-0.05	-0.03	-0.04
Dec	72	1.003	-32.54	0.997	0.969	0.995	-0.03	-0.02	-0.03

Rio Chama from near Chamita to Rio Grande Confluence

This reach of the Rio Chama is not modeled in RiverWare because it is a very short reach (2.8 mi) and no gage is located at the confluence.

UPPER RIO GRANDE REACHES

Rio Grande Basin in Colorado

Streamflow and reservoir operation in the Rio Grande Basin above the gage Rio Grande near Lobatos, Colorado, are not modeled in the model at this time. Discharge from the Rio Grande in Colorado will be represented by the flow measured at the gaging station near Lobatos. In the future, the results from models developed by the State of Colorado's Rio Grande Decision Support System may be linked to URGWOM.

Rio Grande from near Lobatos to near Cerro

The stream gage near Lobatos, Colorado, located 6 mi above the Colorado/New Mexico State line, marks the location where the Rio Grande enters a canyon carved through basalt lava flows and gradually increases in depth to about 1,200 ft at Embudo, about 70 mi south of the State line (**fig. 3**). The river channel in this reach is rocky and has little riparian vegetation. Costilla Creek is a major east-side tributary to the Rio Grande in this reach that contributes very little water to the Rio Grande because its waters are largely regulated and diverted for irrigation before they reach the Rio Grande. Costilla Creek discharges into the Rio Grande during years of very high runoff, but no stream gage is located near its mouth. It is incorporated with local inflows in the river routing of this reach.

Table 10 summarizes stream-gage and calibration data used in determining the power coefficient (wave velocity exponent) that is applied to average velocity measurements to determine river travel time lags in this reach (**graphs 39 and 40**).

Table 10. Summary of stream-gage and calibration data for the reach of the Rio Grande from near Lobatos, Colorado, to near Cerro, New Mexico

	Rio Grande near Lobatos	Rio Grande near Cerro	Total Δ
Period of analysis	5/8/1985 – 8/4/1987 10/2/1990 – 6/1/1999	8/26/1969 – 6/25/1998	
River mile (above mouth)	1719	1693	26
Elevation (feet above sea level)	7428	7110	318
Drainage area (square miles)	7700	8440	740
Number of measurements	323	290	
Wave velocity exponent (β)	0.7273	0.6976	
Coefficient of determination (R^2)	0.9031	0.8572	

Table 11 summarizes factors used to determine travel time lags for this reach (**graphs 41 and 42**).

Table 11. Adopted travel time lags (TL) for the reach of the Rio Grande from near Lobatos, Colorado, to near Cerro, New Mexico

			Time lag (hours) for indicated flow rate (cfs)						
Gage	TL vs. Q equation	R^2	50	200	500	750	1000	3000	6000
Lobatos	$TL = 74.73Q^{-0.2744}$	0.57	25	17	14	12	11	8	7
Cerro	$TL = 112.43Q^{-0.3}$	0.524	28	19	15	13	12	9	7
	Adopted travel times for reach→		27	18	14	13	12	9	7

Table 12 summarizes the adopted loss coefficients for this reach.

Table 12. Adopted monthly loss coefficients for the reach of the Rio Grande from near Lobatos, Colorado, to near Cerro, New Mexico, 1965-94[1]

Month	Adopted loss coefficient
Jan	-0.02
Feb	-0.03
Mar	-0.03
April	-0.05
May	-0.05
June	-0.04
July	-0.04
Aug	-0.04
Sept	-0.03
Oct	-0.03
Nov	-0.02
Dec	-0.03

[1]Based on data developed for the reach from near Arroyo Hondo to below Taos Junction Bridge, near Taos.

The records of discharge measurements at the gage near Lobatos were obtained from the Colorado State Engineer Office in Alamosa, Colorado. Those for the gage near Cerro were obtained from the USGS New Mexico District Office in Albuquerque.

Flow of the Rio Grande from near Lobatos, Colorado, to near Cerro, New Mexico, shows a substantial accretion. This gain in flow is discharge from the ground-water reservoir beneath the lava-capped plateau to the west, Colorado to the north, the Sunshine Valley to the east, and occasional surface water from Costilla Creek. The unmeasured gain in flow in this reach was great enough in most months to mask losses determined by routing the upstream flow down to the gage near Cerro and comparing this routed flow against the observed or recorded flow near Cerro. Except for the months of May, June, and July, applying the filtering criteria to calibrate losses resulted in an average of 28 data points per month for an average of about 4 of the 30 years of flow that were routed. In addition, an analysis of streamflow data shows a substantial change in the rate of gain between these two stations in 1987. Therefore, loss rates developed for the reach of the Rio Grande between the gage Rio Grande near Arroyo Hondo and the gage Rio Grande below Taos Junction Bridge, near Taos were applied to this reach by adjusting the losses proportionally by the length of the two reaches. The flow of the Rio Grande in the reach from near Arroyo Hondo to below Taos Junction Bridge, near Taos is not significantly augmented by unmeasured flow accretions; thus, reasonable monthly loss rates were developed and applied to the reach from near Lobatos to near Cerro. Four outlying data points were removed from the data set used to develop the wave velocity exponent for this reach.

Rio Grande from near Cerro to below Taos Junction Bridge, near Taos

In this reach the Rio Grande continues its descent into the basalt canyon, experiencing very steep gradients of as much as 75 ft/mi between the Cerro gage and the mouth of Red River. The river channel is rocky, with no alluvial material in the bed or banks and a lack of riparian vegetation. Three major tributaries draining the Sangre de Cristo Mountains to the east enter the Rio Grande in this reach: the Red River, the Rio Hondo, and the Rio Pueblo de Taos. Only the gages Red River below Fish Hatchery near Questa and Rio Pueblo de Taos below Los Cordovas are used in the river routing in this reach. The only gage on the Rio Hondo is located 9 mi above its mouth and above all irrigation diversions and is modeled as a local inflow component.

Table 13 summarizes stream-gage and calibration data used in determining the power coefficient (wave velocity exponent) that is applied to average velocity measurements to determine river travel time lags in this reach (**graphs 40 and 43)**.

Table 13. Summary of stream-gage and calibration data for the reach of the Rio Grande from near Cerro to below Taos Junction Bridge, near Taos

	Rio Grande near Cerro	Rio Grande below Taos Junction Bridge, near Taos	Total Δ
Period of analysis	8/26/69 – 6/25/98	6/24/69 – 5/18/99	
River mile (above mouth)	1693	1658	35
Elevation (feet above sea level)	7110	6050	1060
Drainage area (square miles)	8440	9730	1290
Number of measurements	290	331	
Wave velocity exponent (β)	0.6976	0.5412	
Coefficient of determination (R^2)	0.8572	0.7972	

Table 14 summarizes factors used to determine travel time lags for this reach. The stream gage Rio Grande near Arroyo Hondo, which was discontinued in 1996, is included to help define travel time lag for this reach (**graphs 44-46).**

Table 14. Adopted travel time lags (TL) for the reach of the Rio Grande from near Cerro to below Taos Junction Bridge, near Taos

			Time lag (hours) for indicated flow rate (cfs)						
Gage	TL vs. Q equation	R^2	50	200	500	750	1000	3000	6000
Cerro	$TL = 151.35Q^{-0.3}$	0.524	38	26	20	18	17	12	10
Arroyo Hondo	$TL = 266.77Q^{-0.5108}$	0.969	36	18	11	9	8	4	3
Taos	$TL = 186.81Q^{-0.4607}$	0.738	31	16	11	9	8	5	3
	Adopted travel times for reach→		28	15	12	10	8	4	4

The adopted travel times for this reach are weighted more heavily toward the travel times developed using data for the Arroyo Hondo and Taos gages. Velocity data for the Cerro gage are not representative of the entire Cerro to Taos reach because the channel gradient (and hence, velocity) of the Rio Grande increases just downstream from the Cerro gage. The data indicate that a minimum lag time of 4 hours should be used for this reach.

Table 15 summarizes the adopted loss coefficients for this reach.

Table 15. Adopted monthly loss coefficients for the reach of the Rio Grande from near Cerro to below Taos Junction Bridge, near Taos, 1965-94[1]

Month	Adopted loss coefficient
Jan	-0.02
Feb	-0.04
Mar	-0.04
April	-0.07
May	-0.07
June	-0.05
July	-0.05
Aug	-0.05
Sept	-0.04
Oct	-0.04
Nov	-0.03
Dec	-0.04

[1]Based on data developed for the reach from near Arroyo Hondo to below Taos Junction Bridge, near Taos.

Substantial accretions of flow to the Rio Grande continue in this reach, as visibly evidenced by Big Arsenic and Little Arsenic Springs discharging directly into the Rio Grande and springs discharging into the lower Red River. The unmeasured gain in flow in this reach was great enough to mask any losses in the reach determined by routing the lagged upstream flow down to the Taos gage and comparing this flow with observed flow at the Taos gage. As a result, the filtered data generated for this reach were insufficient for developing reliable monthly loss relations. Therefore, as in the upstream reach, loss rates developed for the reach of the Rio Grande between the gages Rio Grande near Arroyo Hondo and Rio Grande below Taos Junction Bridge, near Taos were applied to this reach. The Arroyo Hondo to Taos reach, which is a subreach of the Cerro to Taos reach, does not have significant unmeasured flow accretion; reasonable monthly loss rates were developed and prorated by the difference in length between the two reaches.

Rio Grande from near Arroyo Hondo to below Taos Junction Bridge, near Taos

This reach, located within the Cerro to Taos reach, is not in the model (streamflow measurements at the gage Rio Grande near Arroyo Hondo were discontinued in 1996) and is presented here only because it was used to develop river-channel loss rates for the near Lobatos to near Cerro and the near Cerro to below Taos Junction Bridge, near Taos reaches. The gage Rio Pueblo de Taos below Los Cordovas is used in the river routing in this reach.

Table 16 summarizes stream-gage and calibration data used in determining the power coefficient (wave velocity exponent) that is applied to average velocity measurements to determine river travel time lags in this reach (**graphs 43 and 47**).

Table 16. Summary of stream-gage and calibration data for the reach of the Rio Grande from near Arroyo Hondo to below Taos Junction Bridge, near Taos

	Rio Grande near Arroyo Hondo	Rio Grande below Taos Junction Bridge	Total Δ
Period of analysis	11/12/69 – 10/9/96	6/24/69 – 5/18/99	
River mile (above mouth)	1677	1658	19
Elevation (feet above sea level)	6470	6050	420
Drainage area (square miles)	8760	9730	970
Number of measurements	315	331	
Wave velocity exponent (β)	0.4889	0.5412	
Coefficient of determination (R^2)	0.966	0.7972	

Table 17 summarizes factors used to determine travel time lags for this reach (**graphs 48 and 49**).

Table 17. Adopted travel time lags (TL) for the reach of the Rio Grande from near Arroyo Hondo to below Taos Junction Bridge, near Taos

Gage	TL vs. Q equation	R^2	Time lag (hours) for indicated flow rate (cfs) 50	200	500	750	1000	3000	6000
Arroyo Hondo	$TL = 150.16Q^{-0.5108}$	0.969	20	10	6	5	4	3	2
Taos	$TL = 101.41Q^{-0.4607}$	0.738	17	9	6	5	4	3	2
	Adopted travel times for reach→		19	9	6	5	4	4	4

Table 18 summarizes the adopted loss coefficients for this reach. See **graphs 50-61** for plots of observed flow versus routed flow, filtered for losses, and regression analysis for this reach.

Table 18. Correlations between routed flow and observed flow and adopted monthly loss coefficients for the reach of the Rio Grande from near Arroyo Hondo to below Taos Junction Bridge, near Taos, 1965-94

Month	n (days)	Slope	y-intercept	R^2	Slope (y=0)	R^2	Adopted loss coefficient
Jan	60	1.0068	-8.29	0.9917	0.9871	0.9913	-0.01
Feb	63	0.9723	3.19	0.9957	0.9779	0.9957	-0.02
Mar	216	0.9648	11.89	0.9961	0.9779	0.9959	-0.02
April	187	0.9592	8.37	0.9950	0.9629	0.9949	-0.04
May	167	0.9657	-11.73	0.9960	0.9621	0.9959	-0.04
June	166	0.9670	15.21	0.9983	0.9709	0.9982	-0.03
July	116	0.9743	-2.86	0.9984	0.9726	0.9984	-0.03
Aug	95	0.9716	-6.40	0.9990	0.9710	0.9990	-0.03
Sept	101	0.9882	-4.42	0.9994	0.9805	0.9993	-0.02
Oct	54	1.0091	-13.55	0.9959	0.9807	0.9951	-0.02
Nov	100	0.9859	-2.07	0.9976	0.9827	0.9976	-0.02
Dec	102	1.0214	-21.02	0.9918	0.9784	0.9900	-0.02

The data indicate that at high flows (greater than 5000 cfs), the plot of travel time versus discharge approaches 4 hours asymptotically. Therefore, a minimum travel time of 4 hours will be used.

Plots of data for the gage Rio Grande below Taos Junction Bridge, near Taos reveal a cluster of data points segregated from the other data and located above the line of best fit (**graph 43**). These data points are based on data collected during a short period of time prior to and immediately following 1982. This segregation of data points likely is due in part to the use of multiple locations of cross sections, depending on the level of streamflow, measured by the USGS. One measurement section used during lows flows is controlled by a riffle section, which can change depending on the movement of sediment from the Rio Pueblo de Taos through the control section. Another section with different control is waded to measure flows less than 1000 cfs, and the cableway at a third location is used to measure flows in excess of 1000 cfs.

Rio Grande from below Taos Junction Bridge, near Taos to Embudo

In this reach, the Rio Grande enters the deepest portion of the gorge, and the river channel begins to widen. Alluvial deposits compose the bed and banks of the river here with the first appearance of any significant riparian vegetation. About 200 acres of irrigable land are served by direct diversion from the Rio Grande in the vicinity of Pilar and Rinconada. Embudo Creek is the major tributary in this reach, entering the Rio Grande about 3 mi above the Rio Grande at Embudo gage. The Embudo Creek at Dixon gage measures the discharge of Embudo Creek into the Rio Grande and is included in the river routing for this reach.

Table 19 summarizes stream-gage and calibration data used in determining the power coefficient (wave velocity exponent) that is applied to average velocity measurements to determine river travel time lags in this reach (**graphs 43 and 62**).

Table 19. Stream-gage and calibration data for the reach of the Rio Grande from below Taos Junction Bridge, near Taos to Embudo

	Rio Grande below Taos Junction Bridge, near Taos	Rio Grande at Embudo	Total Δ
Period of analysis	6/24/69 – 5/18/99	7/25/69 – 6/2/98	
River mile (above mouth)	1658	1643.1	14.9
Elevation (feet above sea level)	6050	5789	261
Drainage area (square miles)	9730	10400	670
Number of measurements	331	342	
Wave velocity exponent (β)	0.5412	0.593	
Coefficient of determination (R^2)	0.7972	0.8747	

Table 20 summarizes factors used to determine travel time lags for this reach (**graphs 63 and 64**).

Table 20. Adopted travel time lags (TL) for the reach of the Rio Grande from below Taos Junction Bridge, near Taos to Embudo

			Time lag (hours) for indicated flow rate (cfs)						
Gage	TL vs. Q equation	R^2	50	200	500	750	1000	3000	6000
Taos	$TL = 80.1Q^{-0.4607}$	0.738	13	7	5	4	3	2	1
Embudo	$TL = 66.1Q^{-0.4078}$	0.750	13	8	5	4	4	3	2
	Adopted travel times for reach→		13	7	5	4	4	2	2

Table 21 summarizes the adopted loss coefficients for this reach (**graphs 65-76**).

Table 21. Correlations between routed flow and observed flow and adopted monthly loss coefficients for the reach of the Rio Grande from below Taos Junction Bridge, near Taos to Embudo

Month	n (days)	Slope	y-intercept	R^2	Slope (y=0)	R^2	Adopted loss coefficient
Jan	470	0.9993	-12.574	0.9942	0.9771	0.9937	-0.02
Feb	401	0.9869	-6.9788	0.996	0.9764	0.9959	-0.02
Mar	437	0.985	-6.7921	0.9973	0.9777	0.9972	-0.02
April	458	0.9713	-1.6403	0.9986	0.9706	0.9986	-0.03
May	606	0.9624	0.2886	0.998	0.9625	0.998	-0.04
June	596	0.9677	-12.416	0.999	0.9646	0.9989	-0.03
July	592	0.9639	-4.4054	0.9979	0.9617	0.9979	-0.04
Aug	508	0.962	1.1265	0.9969	0.9637	0.9969	-0.04
Sept	471	0.9655	1.2802	0.9982	0.9672	0.9982	-0.03
Oct	497	0.9775	-3.3034	0.9983	0.972	0.9983	-0.03
Nov	496	0.9722	1.2038	0.998	0.9735	0.998	-0.03
Dec	443	0.9892	-8.5709	0.9971	0.9701	0.9926	-0.03

Rio Grande from Embudo to Rio Chama Confluence

The 13-mi reach of the Rio Grande between the stream gage at Embudo and the site of the discontinued stream gage above San Juan Pueblo was used to determine time lags and loss relations for the 15-mi reach from Embudo to the Rio Chama confluence. Because the gage above San Juan Pueblo was discontinued in 1987, it is not used in the model to route flow or to compute local inflow. Approximately 5000 acres of irrigable land in this reach are served by direct diversion from the Rio Grande.

Table 22 summarizes stream-gage and calibration data used in determining the power coefficient (wave velocity exponent) that is applied to average velocity measurements to determine river travel time lags in this reach (**graphs 62 and 77**).

Table 22. Stream-gage and calibration data for the reach of the Rio Grande from Embudo to above San Juan Pueblo

	Embudo	Above San Juan Pueblo	Total Δ
Period of analysis	7/25/69 – 6/2/98	6/4/69 – 10/20/87	
River mile (above mouth)	1643.1	1630.1	13
Elevation (feet above sea level)	5789	5630	159
Drainage area (square miles)	10400	10550	150
Number of measurements	344	239	
Wave velocity exponent (β)	0.593	0.5796	
Coefficient of determination (R^2)	0.8747	0.9546	

Table 23 summarizes factors used to determine travel time lags for this reach (**graphs 78 and 79**).

Table 23. Adopted travel time lags (TL) for the reach of the Rio Grande from Embudo to above San Juan Pueblo

Gage	TL vs. Q equation	R^2	Time lag (hours) for indicated flow rate (cfs) 50	200	500	750	1000	3000	6000
Embudo	$TL = 65.97Q^{-0.405}$	0.919	14	8	5	5	4	3	2
San Juan	$TL = 69.75Q^{-0.425}$	0.724	13	7	5	4	4	2	2
	Adopted travel times for reach→		13	8	5	4	4	2	2

Table 24 summarizes the adopted loss coefficients for this reach (**graphs 80-91**).

Table 24. Correlations between routed flow and observed flow and adopted monthly loss coefficients for the reach of the Rio Grande from Embudo to above San Juan Pueblo, 1976-86

Month	n (days)	Slope	y-intercept	R^2	Slope (y=0)	R^2	Adopted loss coefficient[1]
Jan	176	0.987	-12.27	0.984	0.967	0.983	-0.03
Feb	174	0.963	3.21	0.995	0.967	0.995	-0.03
Mar	180	0.914	33.96	0.992	0.951	0.990	-0.05
April	260	0.952	-23.78	0.996	0.942	0.996	-0.06
May	294	0.924	15.25	0.990	0.928	0.990	-0.07
June	279	0.916	18.70	0.994	0.920	0.994	-0.08
July	289	0.874	31.48	0.989	0.891	0.988	-0.11
Aug	328	0.987	-42.43	0.988	0.921	0.982	-0.08
Sept	312	1.006	-38.95	0.980	0.917	0.970	-0.08
Oct	225	1.002	-24.08	0.993	0.945	0.989	-0.06
Nov	185	0.940	1.39	0.997	0.942	0.997	-0.06
Dec	201	0.961	2.49	0.991	0.965	0.990	-0.04

[1]Includes losses from irrigation diversions.

Rio Chama/Rio Grande Confluence to Rio Grande at Otowi Bridge

The Rio Chama enters the Rio Grande 14 mi above Otowi. In this reach, the Rio Grande continues to flow through the alluvium of the Española Valley. Water is diverted to serve a small area of irrigable land on the west side of the river. Santa Clara Creek and the Santa Cruz River discharge to the Rio Grande in this reach, but are not represented in the model, except as a component of local inflow.

Table 25 summarizes stream-gage and calibration data used in determining the power coefficient (wave velocity exponent) that is applied to average velocity measurements to determine river travel time lags in this reach (**graphs 77 and 92**). Data from the reach above San Juan Pueblo to Otowi Bridge can be representative of the Rio Chama confluence to Otowi Bridge reach without any significant loss in reliability.

Table 25. Summary of stream-gage and calibration data for the reach of the Rio Grande from above San Juan Pueblo to Otowi Bridge

	Above San Juan Pueblo	Otowi Bridge	Total Δ
Period of analysis	6/4/1969 – 10/20/1987	7/2/1969 – 7/9/1998	
River mile (above mouth)	1630.1	1614.2	15.9
Elevation (feet above sea level)	5630	5488.9	141.1
Drainage area (square miles)	5630	14300	8670
Number of measurements	239	596	
Wave velocity exponent (β)	0.5796	0.663	
Coefficient of determination R^2)	0.9546	0.907	

Table 26 summarizes factors used to determine travel time lags for this reach (**graphs 93 and 94**).

Table 26. Adopted travel time lags (TL) for the reach of the Rio Grande from Rio Chama confluence to Otowi Bridge

			Time lag (hours) for indicated flow rate (cfs)						
Gage	TL vs. Q equation	R^2	50	200	500	750	1000	3000	6000
San Juan	$TL = 47.24Q^{-0.420}$	0.917	9	5	3	3	3	2	1
Otowi	$TL = 35.03Q^{-0.338}$	0.716	9	6	4	4	3	2	2
	Adopted travel times for reach→		9	5	4	3	3	2	2

Table 27 summarizes the adopted loss coefficients for this reach.

Table 27. Correlations between routed flow and observed flow and adopted monthly loss coefficients for the reach of the Rio Grande from Rio Chama confluence to Otowi Bridge

Month	n (days)	Slope	y-intercept	R^2	Slope (y=0)	R^2	Adopted loss coefficient
Jan	12	0.661	165.84	0.999	0.727	0.982	-0.03
Feb	27	0.982	-2.71	0.994	0.981	0.994	-0.03
Mar	8	0.977	20.67	0.998	0.984	0.998	-0.05
April	28	0.969	-5.28	0.997	0.967	0.997	-0.06
May	48	0.980	-31.87	0.962	0.973	0.962	-0.07
June	79	0.986	-32.47	0.998	0.978	0.998	-0.08
July	42	0.982	-26.05	0.999	0.974	0.999	-0.11
Aug	15	0.980	-12.16	0.995	0.967	0.994	-0.08
Sept	13	1.024	-59.51	0.999	0.978	0.996	-0.08
Oct	16	0.960	11.2	0.998	0.968	0.998	-0.06
Nov	4	1.007	-11.12	0.999	0.997	0.999	-0.06
Dec	5	1.060	-103.79	0.991	0.970	0.984	-0.04

January data show a 27-percent loss of flow, an unrealistic value. Data for the remaining months show loss values that range from 0 to 3 percent. For several months only four or five values could be used in the regression. November data show a zero loss in flow, and data for the remaining months show a consistent loss of 2 to 3 percent. Because the data cannot be used to produce monthly loss rates that demonstrate a reasonable loss pattern, the losses developed for the reach from Embudo to above San Juan Pueblo were applied to this reach. Application of these loss rates is appropriate because of the similarities of the two reaches. These two reaches combined constitute the Española Valley, a broad alluvial valley where land use comprises mainly riparian vegetation and irrigated agriculture.

Within this reach, two tributaries join the Rio Grande in addition to the Rio Chama and the Pojoaque/Nambe. Santa Clara Creek allows inflow to the Rio Grande. At present there is no effective way to estimate inflow from Santa Clara Creek, so the confluence becomes a placeholder. The Santa Cruz River is also represented in the model but has no current source for real-time data. Because the upstream gage at Cundiyo is above the Santa Cruz Dam and the gage at Riverside was discontinued in 1951, there is no meaningful relation between the Cundiyo gage and inflow to the Rio Grande. Flows from Nambe Dam will be modeled in the future. Wastewater-treatment plant inflow from Española to the reach is represented.

Rio Grande from Otowi Bridge to Cochiti

Although this reach is 27 mi long, the reach is considered to be 22 mi for purposes of computing time lag and loss because the reservoir above the dam is about 5 mi long at the permanent pool elevation of 5335.92 ft (U.S. Army Corps of Engineers, 1990, 1996a). The model for this reach includes the gages Rio Grande at Otowi Bridge and Rio Grande at Cochiti, which are used to determine gain-loss relations. Although Santa Fe River data are not available for this reach for calibration within the selected time period (1926-70), this tributary will be represented in the model.

The gage Rio Grande at Cochiti was discontinued in 1970 at the closing of Cochiti Dam. The period of record from 1926 to 1970, for which data are available for the Otowi Bridge gage, was used for routing calibration of this reach. No measurement data since 1970 are available for the downstream end of this reach at Cochiti Reservoir. Time lags between the Otowi Bridge and the old Cochiti gages were established using USGS flow-measurement data for only the Otowi Bridge gage.

Table 28 summarizes stream-gage and calibration data used in determining the power coefficient (wave velocity exponent) that is applied to average velocity measurements to determine river travel time lags in this reach (**graph 92**). Because data for the gage Rio Grande at Cochiti are not available, travel time lags for this reach will be based solely on the gage at Otowi Bridge.

Table 28. Summary of stream-gage and calibration data for the reach of the Rio Grande from Otowi Bridge to Cochiti

	Otowi Bridge	Cochiti	Total Δ
Period of analysis	7/2/1969 – 7/9/1998	n/a	
River mile (above mouth)	1614.2	1587	27.2
Elevation (feet above sea level)	5489	5225	264
Drainage area (square miles)	14300	14600	300
Number of measurements	596	290	
Wave velocity exponent (β)	0.663	n/a	
Coefficient of determination (R^2)	0.907	n/a	

Table 29 summarizes factors used to determine travel time lags for this reach (**graph 95**).

Table 29. Adopted travel time lags (TL) for the reach of the Rio Grande from Otowi Bridge to Cochiti

Gage	TL vs. Q equation	R^2	Time lag (hours) for indicated flow rate (cfs) 50	200	500	750	1000	3000	6000
Otowi	$TL = 99.99Q^{-0.4006}$	0.894	21	12	8	7	6	4	3
	Adopted travel times for reach→		21	12	8	7	6	4	3

Table 30 summarizes the monthly loss coefficients for this reach.

Table 30. Correlations between routed flow and observed flow and adopted monthly loss coefficients for the reach of the Rio Grande from Otowi Bridge to Cochiti

Month	n (days)	Slope	y-intercept	R^2	Slope (y=0)	R^2	Adopted loss coefficient
Jan	374	0.878	39.40	0.944	0.934	0.940	-0.03
Feb	428	0.972	-28.04	0.983	0.945	0.982	-0.03
Mar	931	0.955	-55.94	0.983	0.913	0.981	-0.04
April	1016	0.944	-59.38	0.993	0.926	0.992	-0.04
May	999	0.959	-87.08	0.995	0.943	0.995	-0.04
June	1013	0.968	-103.45	0.996	0.947	0.995	-0.05
July	1096	0.959	-102.00	0.990	0.916	0.985	-0.06
Aug	1082	0.934	-77.76	0.977	0.874	0.971	-0.05
Sept	1157	0.906	-55.48	0.980	0.864	0.976	-0.04
Oct	1203	0.944	-78.75	0.978	0.857	0.965	-0.04
Nov	574	0.990	-65.71	0.983	0.928	0.977	-0.03
Dec	400	0.993	-40.56	0.991	0.955	0.989	-0.02

January data show a flow loss of about 7 percent. Data for the remaining months show loss values that range from 5 to 14 percent. Analysis of flow loss for this reach was complicated because flows to the Sili and East Side Main Canals were diverted above the old Cochiti gage.

Losses computed using the routed flow from Otowi Bridge and the gaged flow at Cochiti include diversions for the canals. Estimated average monthly losses during the irrigation season were as much as 14 percent of routed flow when these data were used. Diversion data for 1926 to 1970 are not available, but data after 1970 are available. Diversions before and after 1970 should be about the same because 6,000 acres were irrigated before and after 1970 (U.S. Geological Survey, 1964; Ortiz and others, 1998). Average daily diversions were determined, by month, using data from 1970 to 1997 and were added to each daily flow at the old Cochiti gage from 1926 to 1970. The gaged flow at the old Cochiti gage and the routed flow from Otowi were filtered, so the data set contained only days of losses in the reach. January, February, November, and December were the only months having numerous days with losses in the reach. Average daily diversions for these months are near zero, so the gaged flow at Cochiti was not adjusted significantly. May, June, and July each had fewer than 7 days of flow losses in the reach for the complete 1926 to 1970 data set. Data analysis for these months showed losses near 0. The small losses in this reach do not seem appropriate and probably are a result of adding the average daily canal diversions to actual daily flows at the old Cochiti gage. In the absence of daily diversion data prior to 1970 and no loss estimates for the reach during the irrigation season, loss rates developed for the Rio Chama reach from below El Vado Dam to above Abiquiu Reservoir were applied to this reach. Application of these loss rates is reasonable because both reaches are in canyon sections that have little or no riparian vegetation. Loss rates for the below El Vado Dam to above Abiquiu Reservoir reach demonstrate a reasonable seasonal loss pattern, which would be expected in the Otowi Bridge to Cochiti reach, where losses are predominantly from evaporation.

RESERVOIRS ON THE RIO CHAMA

DESCRIPTION OF PHYSICAL RESERVOIR ATTRIBUTES

Three reservoirs–Heron, El Vado, and Abiquiu–were constructed on the Rio Chama and its tributaries to store water for flood control and water supply. Hydroelectric power plants are located at El Vado Dam and Abiquiu Dam, which are operated as “run-of-the-river” plants--that is, the demand for release for hydroelectric power at these dams is subservient to other demands. **Table 31** summarizes general information about these dams.

Table 31. General information about Rio Chama Basin reservoirs

	Heron	El Vado	Abiquiu
Type:	Earth fill	Earth fill	Earth fill
Year completed:	1971	1935	1963
Structural height (feet):	269	230	341
Top width (feet):	40	20	30
Width at base (feet):	1500	642	2000
Crest length (feet):	1220	1326	1800
Crest elevation (feet above sea level):	7199	6906	6350
Outlet works discharge capacity (cfs):	510	6890	8200

Heron Reservoir

Heron Reservoir stores and releases water imported from the San Juan River Basin and is the primary storage feature of the San Juan-Chama Project. Owned and operated by the USBR, Heron Reservoir’s entire capacity of about 401,300 acre-feet (acre-ft) is dedicated to storing San Juan-Chama Project water. All native Rio Grande inflow to Heron Reservoir is bypassed. The water imported to the Rio Grande Basin from the San Juan River Basin provides supplemental water supplies for various communities and irrigation districts. The project also provides fish, wildlife, and recreational benefits from the storage and movement of this water. An average of

91210 acre-ft per year of the firm yield is allocated annually by contract or project authorization; the remaining 4990 acre-ft is as yet uncontracted. **Table 32** lists elevation information about Heron Reservoir.

Table 32. Elevation-related information about Heron Reservoir

	Elevation (feet)	Area (acres)	Capacity (acre-feet)
Top of dam:	7199.00	--	--
Maximum pool:	7190.80	6148	429,700
Total storage at spillway crest:	7186.10	5906	401,334
Top of dead pool:	7003.00	106	1218

El Vado Reservoir

El Vado Dam was constructed to provide conservation storage for irrigation purposes on Middle Rio Grande Conservancy District (MRGCD) lands along the Rio Grande between Cochiti and Bosque del Apache National Wildlife Refuge. Operated by the USBR, El Vado Reservoir is used to store San Juan-Chama and native water for use by the MRGCD and associated subcontractors. **Table 33** lists elevation information about El Vado Reservoir.

Table 33. Elevation-related information about El Vado Reservoir

	Elevation (feet)	Area (acres)	Capacity (acre-ft)
Top of dam:	6914.50	--	--
Maximum pool:	6908.00	--	--
Total active conservation storage:	6902.00	3170	186,250
Total storage at spillway crest:	6879.00	2454	120,500
Top of dead pool:	6775.00	84	480

Abiquiu Reservoir

Abiquiu Dam and Reservoir is operated by the Corps for flood and sediment control in accordance with conditions and limitations stipulated in the Flood Control Act of 1960 (Public Law 86-645). Reservoir regulation for flood control is also coordinated with the operation of Jemez Canyon Reservoir, Cochiti Lake, and Galisteo Reservoir **(figs. 3 and 4**). Abiquiu Reservoir is operated to limit flow in the Rio Chama, to the extent possible, to downstream channel capacities of 1800 cfs for the reach below Abiquiu Dam, 3000 cfs for the reach below the confluence with the Rio Ojo Caliente, and 10000 cfs through the Española Valley on the Rio Grande main stem. Irrigation releases from El Vado Reservoir pass through Abiquiu Reservoir. Typically, Rio Grande water is stored in Abiquiu Reservoir during April and May, during the peak of snowmelt runoff, and released during June and early July. Any storage remaining in the reservoir after natural flow at the Otowi Bridge gage drops below 1500 cfs is carried over or stored until after November 1, when it may then be released. In 1981, Congress authorized the use of Abiquiu Reservoir to store up to 200,000 acre-ft for San Juan-Chama Project water. The San Juan-Chama Project water allocated to the City of Albuquerque and other entities is stored in the unused sediment space and a small portion of the flood-control space. **Table 34** lists elevation information about Abiquiu Reservoir.

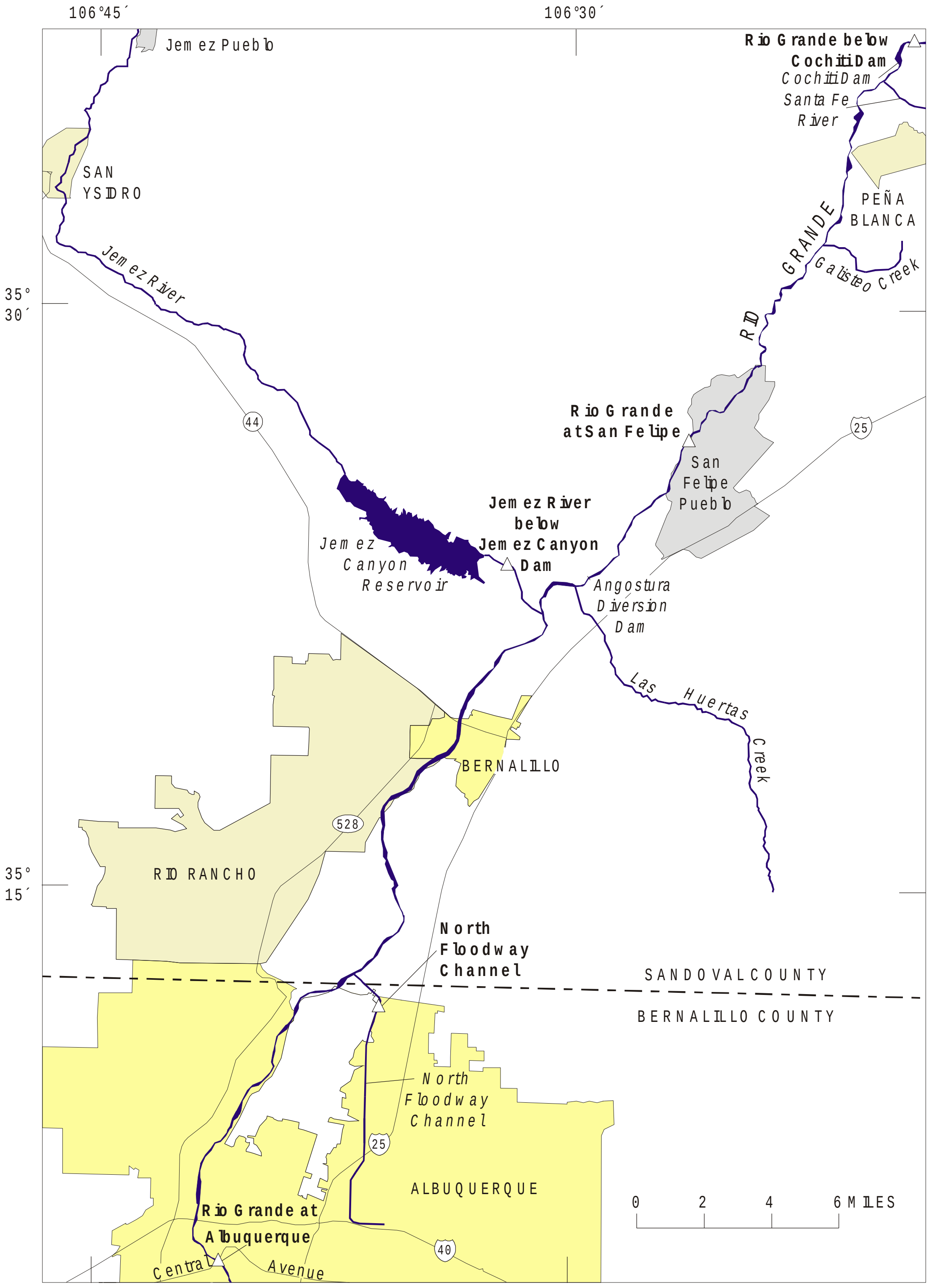

Figure 4. Rio Grande from Cochiti to Albuquerque.

Table 34. Elevation-related information about Abiquiu Reservoir

	Elevation (feet)	Area (acres)	Capacity (acre-feet)
Top of dam:	6381.00	--	--
Maximum pool:	6374.70	15536	1,541,000
Total storage at spillway crest:	6350.00	12430	1,198,500
Top of flood-control pool:	6283.50	7463	551,500
Top of San Juan-Chama storage:	6220.00	4105	189,300
Top of dead pool:	6077.00	--	--

MATHEMATICAL DESCRIPTION OF RESERVOIR PHYSICAL PROPERTIES

All three Rio Chama Basin reservoirs follow the general mass-balance equation for reservoirs:

$$S_t - S_{t-1} - I - P_t + E_t + O = 0$$

where:
S_t = total storage today, in acre-ft;
S_{t-1} = total storage yesterday, in acre-ft;
I = inflow into the reservoir, in acre-ft/day;
P_t = physical model precipitation, in acre-ft/day;
E_t = physical model evaporation, in acre-ft/day; and
O = outflow from the reservoir, in acre-ft/day.

Physical model precipitation is determined by using the equation:

$$P_t = R_t/12(A_{res})$$

where:
R_t = rainfall, in inches/day; and
A_{res} = average reservoir area, in acres.

Physical model evaporation is determined by using one of two equations, depending on the time of year. The summer equation is:

$$E_t = E_p/12\ (coeff)(A_{res})$$

where:
E_p = pan evaporation, in inches per day; and
coeff = pan evaporation coefficient (0.7 for reservoirs in the Rio Grande Basin).

The winter equation is:

$$E_t = [(T_{max} + T_{min})\ /2] * (k/days) * (1\text{-}cov) * A_{res}$$

where:
T_{max} = maximum daily temperature, in degrees Fahrenheit (°F);
T_{min} = minimum daily temperature, in °F;
k = factor for month, in inches per °F;
days = days in the month; and
cov = reservoir ice cover, in percent.

MODEL SIMULATION OF THE RESERVOIR SYSTEM

Each reservoir is simulated with reservoir objects in the model. Heron Reservoir is simulated with a storage reservoir object, and El Vado and Abiquiu Reservoirs are simulated with level power reservoir objects, which give the added capability of simulating a power-generating plant at the reservoirs. Each reservoir object solves a mass-balance equation for the reservoir as well as many user-defined solutions.

Each of the reservoirs in the model can estimate spillway flow using the unregulated spill method. The "pan and ice evaporation" method was used for calculating the amount of evaporation from the surface of each reservoir.

MIDDLE VALLEY

Hydrogeology of the Middle Valley - Albuquerque Basin

The Middle Rio Grande Valley is located in one of several structural basins that are part of the Rio Grande Rift, a region formed by Cenozoic extension that stretches from Colorado through the length of central New Mexico into northern Mexico. The Rio Grande flows through constrictions in the northeastern and southern boundaries to form the Albuquerque Basin, where the eastern and western structural features converge. Basin fill is continuous across these boundaries (Hawley and Haase, 1992, p. II-4).

The predominant basin deposit is the Santa Fe Group. The thickness of the Santa Fe Group ranges from about 3,000 to 4,000 ft along basin margins to greater than 14,000 ft in the center of the basin.

Deposition of post-Santa Fe Group sediments has occurred from 1 million years ago (Ma) to the present. In the early part of this period, the Rio Puerco and Rio Grande deposited channel and flood-plain material during river incision and backfilling episodes. In the last 10,000 to 15,000 years, these rivers have been aggrading. The recent post-Santa Fe Group alluvial deposits average about 80 ft thick. Volcanic rock was emplaced in the central part of the Albuquerque Basin west of the Rio Grande from about 0.2 to 0.1 Ma. Basalt flowed to land surface along presumed fault zones. The exposed part of this rock occupies a small percentage of the basin surface area.

The surface-water hydrology in the inner valley of the Albuquerque Basin includes the flow in the channel of the Rio Grande, storage in Jemez Reservoir and Cochiti Lake, and an extensive, interconnected network of canals and drains. In some areas of the inner valley, the Rio Grande, canals, and drains recharge the ground-water system, whereas in other areas, ground water discharges to surface water. Seepage from Cochiti Lake also recharges the ground-water system.

Historically, high sediment discharges from tributaries of the Rio Grande resulted in a meandering channel that heightened the flood threat due to decreased channel capacity, rising ground water, and establishment of dense stands of phreatophytes. These problems led to the authorization of the Middle Rio Grande Project, which included the construction of flood- and sediment-control reservoirs and the control of the Rio Grande channel and reclamation of seeped lands through the rehabilitation of riverside and interior drains.

The capture of sediment in Jemez Reservoir and Cochiti Lake and the construction of channel control works have changed the channel of the Rio Grande above Bernalillo from a braided channel with sand bars to a straight, incised channel with an armored bed. The effects of the operation of sediment-control pools are moving downstream. However, the reduced peak flows of the Rio Grande below Cochiti are insufficient to move sediment contributed by small tributaries, resulting in continued instability in some reaches.

The recent chronic high reservoir state of Elephant Butte Reservoir has resulted in sediment deposition and aggradation of the Rio Grande Floodway below San Acacia. The aggradation has largely reduced the capacity of the channel of the Rio Grande above Elephant Butte Reservoir and has required concentrated maintenance on the levee to maintain this unstable channel.

Impoundment of water in Cochiti Reservoir began in 1973, and mean annual water levels in the reservoir from 1974 through 1995 ranged from 5,260 to 5,390 ft above sea level. Seepage from the reservoir to the ground-water flow system was estimated to be 84,000 acre-ft/yr in 1978 and 1979 and 21,000 acre-ft/yr in 1980 and 1981 (Blanchard, 1993). Seepage from Cochiti Lake increases with increased reservoir storage, which occurs during flood-control operations.

Physical description of model reaches

The Middle Rio Grande Valley is defined in this model as Cochiti Lake to Elephant Butte Lake. The middle valley was analyzed in reaches that were delineated at points along the river where discharge readings were available for the historical calibration period across the entire river valley. These locations are referred to as full cross section and provide calibration points for each canal and drain as well as the river. The first reach starts at Cochiti Lake and ends at the San Felipe gage. The second reach starts at the San Felipe gage and ends at the Central Avenue Bridge where the Rio Grande at Albuquerque gage is located (**fig. 4**). The third reach starts at the Central Avenue Bridge and continues to the Rio Grande Floodway near Bernardo gage (**fig. 5**). For this third reach, it was necessary to extend the analysis downstream to Bernardo, rather than stopping at Isleta diversion dam, which would be preferred. The reason for bypassing Isleta is that, until recently, recording river discharges in a backwater condition behind the diversion dam resulted in unreliable data. For current and future daily operations, however, Isleta could be an ending point for a reach because the gage site has been moved to just upstream from the diversion dam and is now recording reliable data. The fourth reach starts at the Rio Grande Floodway near Bernardo gage and ends at the Rio Grande Floodway at San Acacia gage, just downstream from the San Acacia diversion structure (**fig. 6**). The fifth reach starts at the gage Rio Grande Floodway near San Acacia and ends at the inflow to Elephant Butte. As seen on the model topology, each reach includes data objects that estimate losses and gains based on available information and data.

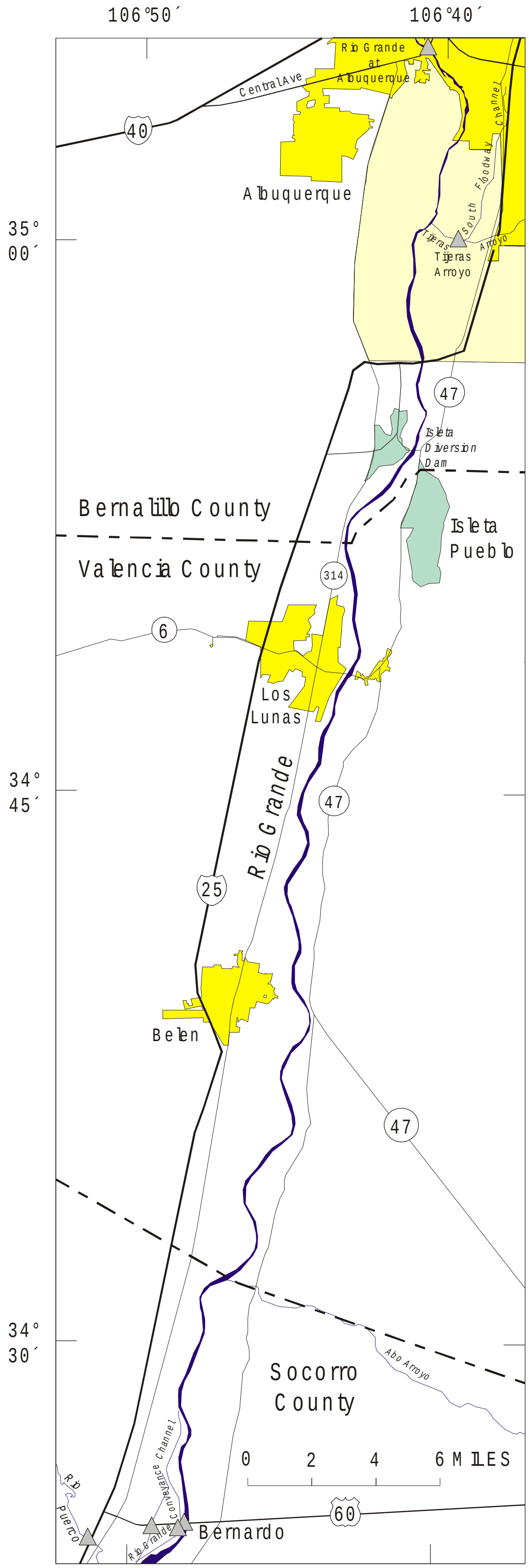

Figure 5. Rio Grande from Albuquerque to Bernardo.

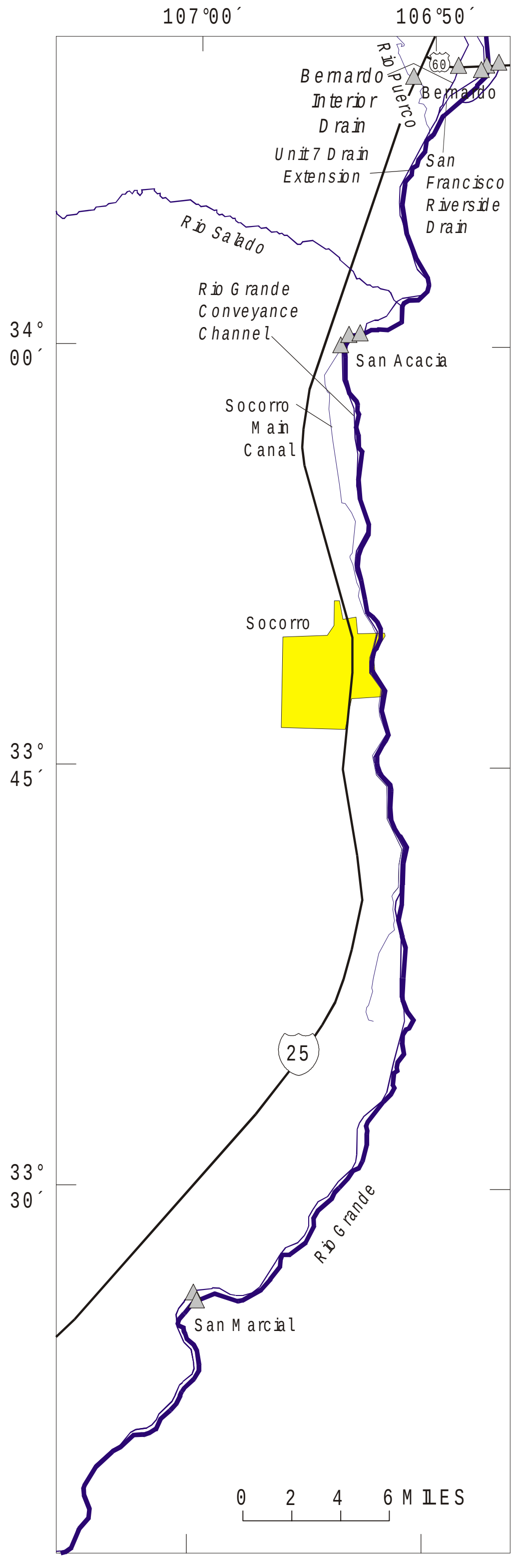

Figure 6. Rio Grande from Bernardo to San Marcial

Description of MODEL methods

The development of river routing and channel-loss methods, calibration of unmeasured return flows, and solving for local inflow are outlined below.

1. Select an overall data set that is post-Cochiti Dam construction and has data on major anthropogenic effects (1985-97).

2. Using available gage data, statistically develop time lags for varying flow ranges for each reach in the middle valley. The methodology used was similar to all other reaches in the model above Cochiti and are discussed in detail in the Rio Chama section of this document.

3. Input channel-loss coefficients, river-channel leakage, and river-channel water surface evaporation.

4. Model all significant and known human effects in the reach such as:
   Agricultural diversions
   Agricultural depletions
   Gaged tributary inflows
   Reservoir effects
   Wastewater returns.

5. Calibrate return flows back into the river within the reach by calibrating against a full downstream cross section. This cross section has discharge data for across the entire river valley, indicating how much water resides in the irrigation distribution and drainage system, including agricultural diversion flows and ground-water interception.

6. Create an overall routed hydrograph from the upstream-observed hydrograph, accounting for anthropogenic effects, channel losses, routing, and return flows.

7. Compute local inflow by subtracting the downstream-observed river hydrograph from the overall routed hydrograph. This local inflow encompasses effects on the river that could not be accounted for such as:
   Precipitation
   Urban runoff
   Springs or "gains" in the channel
   Ungaged inflow
   Errors resulting from invalid engineering assumptions
   Measurement errors.

8. (Optional) Smooth the local inflow data set to minimize negative effects using a running average but still maintaining a constant volume.

Selection of Overall Data Set

The data set selected for calibrating the model through the middle valley extends from Cochiti Lake and Dam to the inflow to Elephant Butte Reservoir. Data for streamflow- or reservoir-gaging stations located on the main stem and on the mouths of tributaries, measurements of diversions and return flows, and climatological data for stations located in the valley or close by are included in the model. The data set period of record used for calibration and validation of the model is 1985-97. The period 1985-89 will be used for calibrating the data, and the later period, 1990-97, will be used for validating the model. The data set chosen most accurately represents the current geomorphic character of the middle valley from Cochiti to Elephant Butte. The construction of levees, drains, and other channel improvement works, as well as the effect from the operation of sediment-control pools at Cochiti and Jemez Canyon Reservoirs, has changed the character of the upper river to the extent that records prior to 1985 cannot represent current conditions.

Determination of Stream-Channel Losses

Several methods are used to address the loss and movement of water in the channel of the Rio Grande as it flows downstream. These methods are determination of losses from the river channel through water surface evaporation, leakage or gain from the bed and banks of the channel of the Rio Grande, and determination of travel time lags (river routing).

Method for Estimating River-Channel Evaporation Loss

The following equations were used to estimate evaporation losses from the water surface and wetted sands within the river channel in the Middle Rio Grande Valley between Cochiti Dam and the San Acacia diversion dam.

Cochiti to San Felipe (bankfull discharge = 5650 cfs and corresponding surface area = 625 acres):

$$\text{For } Q < 5650 \text{ cfs}; \; L = Pan_e(111\, Q^{.20}) + 0.25\, Pan_e\, (625 - 111\, Q^{.20})$$

$$\text{For } Q \geq 5650 \text{ cfs}; \; L = Pan_e\, (111\, Q^{.20})$$

San Felipe to Albuquerque (bankfull discharge = 4820 cfs and corresponding surface area = 2718 acres):

$$\text{For } Q < 4820 \text{ cfs}; \; L = Pan_e(84\, Q^{.41}) + 0.25\, Pan_e(2718 - 84\, Q^{.41})$$

$$\text{For } Q \geq 4820 \text{ cfs}; \; L = Pan_e(84\, Q^{.41})$$

Albuquerque to Bernardo (bankfull discharge = 4820 cfs and corresponding surface area = 5175 acres):

$$\text{For } Q < 4820 \text{ cfs}; \; L = Pan_e(124\, Q^{.44}) + 0.25\, Pan_e(5175 - 124\, Q^{.44})$$

$$\text{For } Q \geq 4820 \text{ cfs}; \; L = Pan_e(124\, Q^{.44})$$

Bernardo to San Acacia (bankfull discharge = 4000 cfs and corresponding surface area = 1054 acres):

$$\text{For } Q < 4000 \text{ cfs}; \; L = Pan_e(13\, Q^{.53}) + 0.25\, Pan_e(1054 - 13\, Q^{.53})$$

$$\text{For } Q \geq 4000 \text{ cfs}; \; L = Pan_e(13\, Q^{.53})$$

where:

Q = Mean daily discharge at the upstream end of the reach, in cfs;
L = Loss from water surface evaporation and wetted sands in the reach, in acre-ft/day; and
Pan_e = Pan evaporation data for the site nearest to reach under consideration, in ft/day.

Data used to develop the stream discharge/loss equations were obtained from three sources: Fenton (1996); Bureau of Reclamation and U.S. Army Corps of Engineers (1998); and Bureau of Reclamation (1985). The USBR mapped river-channel areas in 1992 during high-flow and low-flow conditions (Fenton, 1996). The water surface areas of the main channel and areas of inundated bosque were tabulated for each reach during high flow and low flow. River-channel bankfull widths at the computed channel maintenance discharge for each reach (Bureau of Reclamation and U.S. Army Corps of Engineers, 1998) also were included in the development of the stream-discharge/water surface area relation. Finally, data developed in the San Juan-Chama incremental loss study (Bureau of Reclamation, 1985) were incorporated into the analysis. These data include river-channel cross section data obtained from an ongoing program to measure the river-channel response to the construction and operation of flood-control works in the Middle Rio Grande Valley and data collected by the USGS during the course of stream-gage calibration.

Leopold and Maddock (1953) described the relation between channel width and stream discharge in the form $w = aQ^b$, where Q is the discharge, w is the channel width, and a and b are constants. This equation was used in developing the loss equations described above. River-channel areas were developed by multiplying the channel width times the length of the reach. Losses are estimated by multiplying the areas by an evaporation rate, such as that measured at a nearby evaporation pan.

For this analysis, at the bankfull discharge, all sand bars in the river channel are assumed to be covered with water and the wetted sands not subject to evaporation loss. This bankfull discharge generally has a 1- to 2-year return interval (Leopold and others, 1964). A peak annual discharge with a 2-year return interval for each reach was used to estimate the bankfull discharge (Bureau of Reclamation and U.S. Army Corps of Engineers, 1998). As stream discharge drops below the bankfull discharge, sand bars become exposed and subject to evaporative losses. A factor of 0.25 is used to correlate the evaporation of water by capillary action through sand bars in the river channel to an adjacent evaporation pan measurement (Sorey and Matlock, 1969).

Some of the data used in this analysis were collected prior to the time that the influence of the construction and operation of Cochiti and Jemez Canyon Dams were manifest on the river channel. The construction and operation of these flood- and sediment-control facilities have resulted in deeper and narrower river-channel cross sections in the upper reaches. Data are inadequate to evaluate the change in river-channel cross sections or to predict future changes in river cross sections.

The constant used to correlate evaporation from wetted sand surfaces with pan evaporation data from a nearby station varies between 0.25 for wetted sands within 1 foot above the water surface to 0.05 for wetted sands between 1 and 4 ft above the water surface (Sorey and Matlock, 1969). A constant of 0.25 is used here for all water levels.

Graphs 96-99 show plots of the relation between streamflow discharge and water surface area for each reach of the middle valley.

Method for Modeling Ground Water (Channel Leakage)

A direct way to estimate leakage to or from the river is to use the difference between surface-water measurements at upstream and downstream sites. Seepage investigations (Jack Veenhuis, U.S. Geological Survey, oral commun.) have shown that this method is generally applicable only to selected stream reaches and only during winter when inflows and outflows are nonexistent or very small within the selected reach. In addition to the seasonal constraints, this approach is usually not applicable to a daily time-step model because routine streamflow measurements are generally conducted every 4 to 6 weeks as part of the normal operation of stream gages. Another problem with using surface-water measurements or average daily flow at stream gages is that information available about inflow and outflow between stream-gaging sites is generally very limited. Without inflow and outflow data, the flow differences are invalid.

Another direct method of calculating leakage is to use the hydraulic gradient from the river to the shallow ground-water system, an estimate of the hydraulic conductivity of the aquifer sediments, and the area of the riverbed and bank through which water can flow. Daily river stage or ground-water head data are not available on a spatial scale that are representative of the total hydraulic system. Information on stream stage and shallow ground-water heads can be used for calibration of a model or analytical solution of the ground-water flow equation. Limited information of this type probably is available for the Los Alamos-Santa Fe area, the Albuquerque Basin, the Mesilla Basin, and the El Paso, Texas, area.

The Albuquerque Basin is an important part of the RiverWare model. The complexity of the surface-water system, the effects of storage and delivery requirements of water users in the basin on the upstream river system, the large population that depends on surface and ground water, the importance of ground water as a public water supply, and the interaction of the surface- and ground-water system require definition of the complete hydrologic system. Surface-water/ground-water exchange can be estimated in the Albuquerque Basin because data are available that are needed to calibrate the analytical solution of ground-water flow equations. In addition, leakage has been measured in specific reaches of the Rio Grande that can be compared with other methods.

Methods considered for estimating leakage to or from the Rio Grande and the shallow ground-water system include the MODFLOW Albuquerque Basin model (Kernodle and others, 1995). This model simulated the ground-water system from Cochiti Dam on the north to San Acacia on the south **(figs. 4-6)** and from the Sandia Mountains on the east to the escarpment on the west with emphasis on the Albuquerque area. The model grid was variable; the smallest cells were in the Albuquerque area. Dimensions of cells were 656 ft on a side in the Albuquerque area and as great as 3,281 ft on a side at the margins of the model. The model had 11 layers, and each of the top 4 model layers was 20 ft thick. The Rio Grande was considered a head-dependent boundary with a constant depth of 3 ft. Canals, laterals, and drains were also considered in the model. Stress periods used in the simulation were 1 year from 1961 through 1979 and 28 summer and winter periods from 1980 through 1994. The use of this model to estimate leakage to or from the river was ruled out, however, because (1) daily time steps are needed in the RiverWare model, (2) river stage was constant throughout the MODFLOW model simulation period, (3) the model was not calibrated to shallow piezometers immediately adjacent to the river, and (4) the area immediately adjacent to the Rio Grande was a very small part of the ground-water flow model. Hydrologic characteristics were delineated on a spatial scale representative of the river/shallow aquifer system for the basin as a whole but were not representative of just the river and shallow aquifer system. A MODFLOW three-dimensional model of just the river and the immediate aquifer system might be developed specifically for an URGWOM application. A model of this system would require daily data that are not currently available on a daily time step and at an appropriate spatial scale.

A preliminary computation of leakage to or from the river and the shallow ground-water system was made for comparison to actual seepage measurements in the reach from just below the Highway 44 Bridge to the Rio Bravo Bridge. This estimate was made using a Fortran program developed for the Rio Grande reach from Cochiti Dam to San Acacia. The leakage estimate is based on hydraulic gradients between the river and riverside drains. Reported hydraulic-conductivity values of the shallow aquifer sediments from the Bureau of Reclamation were used as initial values in the calculations. Daily ground-water head data were not available for the shallow aquifer adjacent to this reach of the river. Drain water surface elevations of 5 ft below land surface at each Albuquerque Basin model drain cell (Kernodle and others, 1995) were used to establish a constant ground-water elevation in the shallow aquifer. Daily river surface elevations at each model river cell were estimated using daily river surface gradients from the Cochiti to San Felipe gages, the San Felipe to Central Avenue gages, and the Central Avenue to San Acacia gages and the length of each model cell. River cell surface areas were taken from the model. The estimated channel bank area, through which water flows, was estimated by assuming a 3-ft-deep flow in the river times the length of the cell, which was taken from the model. The channel bank area was multiplied by two to account for both sides of the river. Vertical or horizontal hydraulic conductivity times an estimated vertical gradient (Jim Bartolino, U.S. Geological Survey, oral commun.) and the calculated horizontal gradient, respectively, were used to calculate flow to or from the river. Seepage was estimated for October 1, 1994, to September 30, 1998. Estimates of leakage to or from the river for December, January, and February were compared to actual seepage measurements made by Jack Veenhuis (U.S. Geological Survey, oral commun.). Close agreement was obtained between the measured seepage rate of 85 to 95 cfs and the estimated (discussed above) leakage rates derived using horizontal and vertical hydraulic-conductivity values of 160 and 1.6 ft/day, respectively.

In February 1999, computer programs became available for analytical solutions of the ground-water flow equation using a surface-water stage hydrograph as input and the convolution method of solution (Barlow and Moench, 1998). The solutions are based on the partial differential equation of transient ground-water flow in a saturated, homogeneous, slightly compressible, and anisotropic aquifer in which the principal directions of hydraulic conductivity are oriented parallel to the coordinate axes. The most general case of the equation written in two dimensions is:

$$K_x \frac{\partial^2 h}{\partial x^2} + K_z \frac{\partial^2 h}{\partial z^2} = S_s \frac{\partial h}{\partial t} + q'$$

where:

K_x, K_z	= horizontal and vertical hydraulic conductivity of the aquifer, respectively (units of length per time);
x,z	= horizontal and vertical coordinate directions, respectively (units of length);
h	= ground-water head (units of length);
S_s	= specific storage of the aquifer (units of inverse length);
t	= time (units of time); and
q'	= a volumetric flow rate to or from the aquifer per unit volume of aquifer, representing sources or sinks of water to the aquifer (units of inverse time).

The solutions are for confined aquifers (program STLK1) and for unconfined aquifers (program STWT1). One of the hydrologic requirements for proper use of the programs is that ground-water flows are primarily perpendicular to the river channel and, for unconfined aquifers, vertical below the channel.

Next attempted was calibration of the analytical solution using measured leakage from the river in the reach from just below the Highway 44 Bridge to the Rio Bravo Bridge to actual seepage measurements made by Jack Veenhuis (U.S. Geological Survey, oral commun.). The actual seepage measurements determined that 85 to 95 cfs leaks from the river in this reach during December, January, and February. The depth of the aquifer was set at 250 ft, a depth at which changes in river stage are not noticed in nearby piezometers. The width of the aquifer, from the midpoint of the river to one side of the aquifer, was set at 5,600 ft, again a distance at which stage changes are not noticed in the aquifer. With the aquifer at these dimensions, boundary conditions, including the drains and canals within the specified area, were thought to be insignificant. Analytical solution results could not match this leakage rate. Horizontal hydraulic conductivity of 500 ft/day, a vertical to horizontal hydraulic-conductivity ratio of 0.5, specific yield of 1×10^{-3}, and a specific yield of 0.25 were tried in the solutions to obtain the measured leakage rate. Although these input values were greater than any reported values in the area, the maximum leakage rate was only about 30 cfs. The method and calibrations were reviewed with staff of the USGS. Even with the aquifer dimensions set beyond apparent influences of the drain, the methods and calibrations created a boundary condition that was significant. The constant head of the drain and the drainage of water out of the system by the drain created a condition that could not be duplicated by the STWT program.

The drain elevations that are in the MIDRIODRN file represent an elevation 5 ft below the minimum river elevation in each river cell from an Arc/Info coverage. The model by Tiedeman and others (1998) used this elevation as the bottom of the drain, as is assumed here, and also as the water surface in the drain. The depth of flow in the drains is varied monthly on the basis of data in **table 35.**

Table 35. Average depth of water in drains

	Average depth (feet)					
Month	Corrales Riverside Drain	Bernardo Interior Drain	Atrisco Riverside Drain	Albuquerque Riverside Drain	Monthly depth (feet)	Rounded depth (feet)
Jan	1.18	0.99			1.09	1.1
Feb	1.66	1.00	1.18	1.13	1.24	1.2
Mar	1.42	1.31	1.29	2.21	1.56	1.6
Apr	1.97	1.50	1.24	2.19	1.73	1.7
May	1.91	1.69	2.43	2.18	2.05	2
June	1.97	1.62	1.57	2.57	1.93	1.9
July	1.79	1.69	1.36	2.55	1.85	1.8
Aug	1.95	1.77	1.13	2.43	1.82	1.8
Sept	1.82	2.19	1.18	2.60	1.95	1.9
Oct	2.09	2.20	1.35	2.43	2.02	2
Nov	1.29	1.04	1.23	1.54	1.28	1.3
Dec	1.25	1.05	1.20	1.28	1.19	1.2

River depth is varied according to the relations established between gage height and flow depth for each gage in the middle valley. The initial bank cross section area through which water will flow from the river will be the depth of flow plus 14 ft. This 14-ft value comes from the USBR study (Rio Grande Water Assessment, Riparian Corridor Alluvium Steady State Ground Water Investigation Concerning Rio Grande Channel Loss Contributions to Recharge conducted by USBR Albuquerque Projects Office, by Steve Hansen). The cross section area of 14 ft for each foot of river channel was required to match the flow rate from the river to the shallow ground-water system that was obtained using a mass balance. In this study the USBR used a hydraulic conductivity of 350 ft^2/day.

The Fortran program was modified to change the stage of drains monthly on each side of the river on the basis of the average depths presented in **table 35**. Flow depth in the river was changed daily on the basis of gage height/flow depth relations presented in **graphs 100-107**. Flow depths from the Cochiti gage to one-half the distance to the San Felipe gage were calculated using the gage height/flow depth relation at the Cochiti gage. For average riverflow depths from one-half the distance from the Cochiti gage to the San Felipe gage plus one-half the distance from the San Felipe gage to the Albuquerque (Central Avenue Bridge) gage, the gage height/flow depth relation at the San Felipe gage was used. From one-half the distance from the San Felipe gage to the Albuquerque gage plus one-half the distance from the Albuquerque gage to the Rio Grande Floodway near Bernardo gage, the gage height/flow depth relation at the Albuquerque gage was used. From one-half the distance from the Albuquerque gage to the Rio Grande Floodway near Bernardo gage to the end of the reach below the Rio Grande Floodway near Bernardo gage, the gage height/flow depth relation at the Rio Grande Floodway near Bernardo gage was used.

The Fortran program was then calibrated using the measured winter leakage for the reach from about 3 mi below the Highway 44 Bridge to the Rio Bravo Bridge. Jack Veenhuis (U.S. Geological Survey, written commun.) completed a study consisting of multiple discharge measurements at a cross section from about 3 mi below the Highway 44 Bridge near Bernalillo to the Rio Bravo Bridge in Albuquerque. During two measurement cycles, February 5-6, 1997, and February 27-28, 1998, all inflows between the top and bottom of the measurement reach were measured. Gross leakage from the river during these two measurement cycles was 293 and 310 cfs per day, respectively. The program was calibrated using a horizontal hydraulic conductivity of 165 ft^2/day, a vertical hydraulic conductivity of 1.65 ft^2/day, and a riverbank cross section area of the calculated depth plus 11.5 ft. This program is used to calculate leakage for the entire Middle

Valley. The Middle Rio Grande Valley is modeled using subreaches from Cochiti to the San Felipe gage, San Felipe gage to the Central Avenue gage, and Central Avenue gage to Bernardo. Total leakage for these subreaches is calculated by the program. Average leakage from each of these subreaches, by month, is shown in **table 36**.

Table 36. Average monthly leakage from the river, January 1, 1985, to December 31, 1997

	Average daily leakage (cfs)		
	Cochiti to San Felipe	San Felipe to Albuquerque	Albuquerque to Rio Grande Floodway near Bernardo
Jan	14.4	406	326
Feb	15.3	412	331
Mar	14.4	413	332
Apr	17.1	432	353
May	17.6	438	358
June	16.9	434	348
July	12.1	410	320
Aug	9.2	392	300
Sept	7.9	386	293
Oct	7.3	381	286
Nov	13.1	400	323
Dec	13.5	401	324

Because of the lack of data, the model has been configured to estimate a return flow to the river from drains. The estimate is based on diversions, reduced by the estimated water used by acreage of specific crops served by the diversion. This method of computing return flow does not allow for return flow during periods of no diversions during the winter. The riverside drains flow year-round because the river is leaking water and some part of river leakage is picked up by riverside drains and contributes return flow to the river. A method of estimating flow in drains resulting from river leakage through the shallow ground-water system was needed. Selected winter flow measurements by Jack Veenhuis (written commun., 1999) were used to determine flow in riverside drains that results from river leakage. During winter all flow in riverside drains is assumed to result from river leakage. Drain flow resulting from river leakage was estimated by summing all drain inflows to the river between measurement section ends and the outflow from the measurement section. From this sum the drain flow coming into the section was subtracted. By using data for two measurement cycles and the estimated leakage from the river for specific days, about 54 percent of river leakage is intercepted by the riverside drains. A minimum of 54 percent of this estimated river leakage was return flow to the river. This very preliminary value, which needs to be refined, is used to make the model simulate the river system more realistically.

The method of estimating leakage from the Rio Grande to the shallow ground-water system in the reach from Bernardo to below the gage at San Marcial (the lower reach) is basically the same as that described for the reach from Cochiti to Bernardo. Differences for the lower reach are (1) drain stage elevations are known at each of the gage sites (Bernardo Conveyance Channel, San Acacia Conveyance Channel, and San Marcial Conveyance Channel) and (2) drains or conveyance channels are only on the west side of river for most of the lower reach. The lower reach was divided into 68 cells using selected USBR aggradation-degradation cross sections as north and south cell boundaries. The cells were extended east and west a sufficient distance to include the river and drains or conveyance channels. The cells are about 1 river mile in length except at the San Acacia diversion structure and in the vicinity of the Tiffany Channel. The lengths of the river and drain or conveyance channels in each cell were determined from 7½-minute digital raster graphic maps. The maps were displayed in Arc/Info and overlain by an

Arc/Info coverage of the cells. River and drain water surface elevations for each cell were computed using the method described for the upper reach.

Distances from the river channel to the drain or conveyance channel that are needed to compute the gradient from the river to the shallow ground-water system were computed using aggradation-degradation cross section data from February 1992. Zero distance was on the left side of each section and distance increased to the right. Each distance was accompanied by an elevation of the land or water surface. Data were entered into a spreadsheet and plotted. After the river and the drain or conveyance channel locations were visually verified, the distance between the channels was determined. The river-to-drain distance for a cell is the average of the distance at the upstream and downstream ends of the cell. Ground-water-level data for 1985 to 1997 for shallow wells near the river were retrieved from the USGS Ground-Water Site-Inventory (GWSI) data base. These data were used to compute gradients from the river to the shallow ground-water system to the east. For those days for which there were shallow ground-water-level gradients to the east, gradients to the east were compared with those to the west. Because the averages of the east and west gradients were nearly identical, the gradients to the east of the river were assumed to be equal to the computed gradients to the west.

Vertical flow from the river channel is computed using the area of the river channel in each cell and the vertical gradient. Vertical gradient data were taken from work done by Bartolino and Niswonger (1999). The average vertical gradient at sites upstream from the Highway 44 Bridge, above the Rio Rancho sewage plant outfall, near the Paseo del Norte Bridge, and near the Rio Bravo Bridge is 0.0058. Data were collected sporadically from 1996 through 1998. Vertical hydraulic conductivity was assumed to be 1/100 of the horizontal hydraulic conductivity.

River depth is needed to determine the area through which water can flow from the river channel. In the upper reach, there were poorly defined relations between river gage height and river depth. The lower reach, however, has no such relations, except at Bernardo. In the interest of computation consistency in the Fortran program, the relation at Bernardo was not used. Monthly average river depth was determined at the Bernardo Floodway, San Acacia Floodway, and San Marcial Floodway gages using USGS flow measurements from 1970 to 1998. **Table 37** shows these average monthly depths.

Table 37. Average depth of Rio Grande at three floodway gages

	Average river depth, in feet		
Month	Bernardo	San Acacia	San Marcial
Jan	1.55	0.44	1.73
Feb	1.69	0.79	3.49
Mar	1.38	1.41	2.89
Apr	1.48	1.15	1.95
May	1.64	0.96	1.62
June	1.64	1.31	1.22
July	2.20	2.01	2.27
Aug	2.75	2.99	1.80
Sept	1.72	2.21	1.90
Oct	2.16	3.30	1.70
Nov	2.26	2.45	1.82
Dec	1.71	2.47	1.61

To compute vertical leakage from the river, the area of the channel through which water can flow from the river to the shallow ground-water system is needed. The length of the river channel in each cell had been determined; the width of the channel is estimated using average monthly river widths taken from USGS flow measurements from 1970 to 1998. **Table 38** shows the monthly average river widths at the three floodway gages in this reach.

Table 38. Average width of Rio Grande at three floodway gages

Month	Average river width, in feet		
	Bernardo	San Acacial	San Marcial
Jan	262	85	204
Feb	232	101	232
Mar	266	97	237
Apr	312	99	219
May	406	163	220
June	370	122	178
July	243	106	169
Aug	189	79	157
Sept	172	82	148
Oct	165	56	155
Nov	256	76	209
Dec	244	71	223

The Fortran program was modified to compute the elevation of the water surface in the drains and conveyance channel on the west side of the Rio Grande. This computation is done in addition to computing the river water surface elevation as described previously. The gradient between the river and the drain or conveyance channel was computed using the estimated water surface elevation in the river and the drain or conveyance channel in each cell.

A spreadsheet was developed that duplicated the logic in the Fortran program to ensure that the Fortran program was computing leakage using the correct stage in the river and the drain or conveyance channel at each cell; the correct gradient from the river to the drain; and the correct flow depth of the river. An input data set used in the Fortran program was developed that should give the same answers as in the spreadsheet. The Fortran program and the spreadsheet gave identical calculations of leakage.

The Fortran program was calibrated using data from 10 seepage measurements from San Acacia to San Marcial from January 1995 to October 1997. The seepage runs were completed by the USBR. Rio Grande flow was measured by the USGS at the San Acacia gage and on the next day at the San Marcial gage. The difference between these two floodway measurements is considered to be leakage to or from the Rio Grande and was used to calibrate the Fortran program. The input parameters of horizontal and vertical hydraulic conductivity and the channel section distance that is added to the depth of flow through which water can flow from the river to the ground-water system were varied. For each day there was a seepage run the computed leakage was compared. The input parameters were varied until the sum of the differences was a minimum. The input values for the calibrated run are a horizontal conductivity of 200 ft/day, vertical hydraulic conductivity of 2 ft/day, and a cross section distance of 30 ft. **Table 39** shows the seepage run losses, calculated leakage, and the difference between the measured and calculated values.

Table 39. Seepage run losses, calculated leakage, and difference between measured and calculated values

Date	San Acacia minus San Marcial (cfs)	Calculated river leakage (cfs)	Riverflow minus calculated leakage (cfs)
Jan 1, 1995	285.151	391	-105.849
Mar 7, 1995	310.0398	190	120.0398
May 16, 1995	379.4757	300	79.4757
Oct 15, 1996	138.3117	88	50.3117
Nov 13, 1996	266.1438	409	-142.8562
Jan 28, 1997	302.7278	246	56.7278
Feb 11, 1997	301.3411	280	21.3411
July 22, 1997	110.4181	186.2	-75.7819
Sept 16, 1997	226.5351	380	-153.4649
Oct 22, 1997	299.6453	150	149.6453

Leakage from the river to the shallow ground-water system was estimated for 1985 through 1997. **Figure 7** shows the average daily estimated leakage by month for the Bernardo to San Acacia, San Acacia to San Marcial, and below San Marcial reaches.

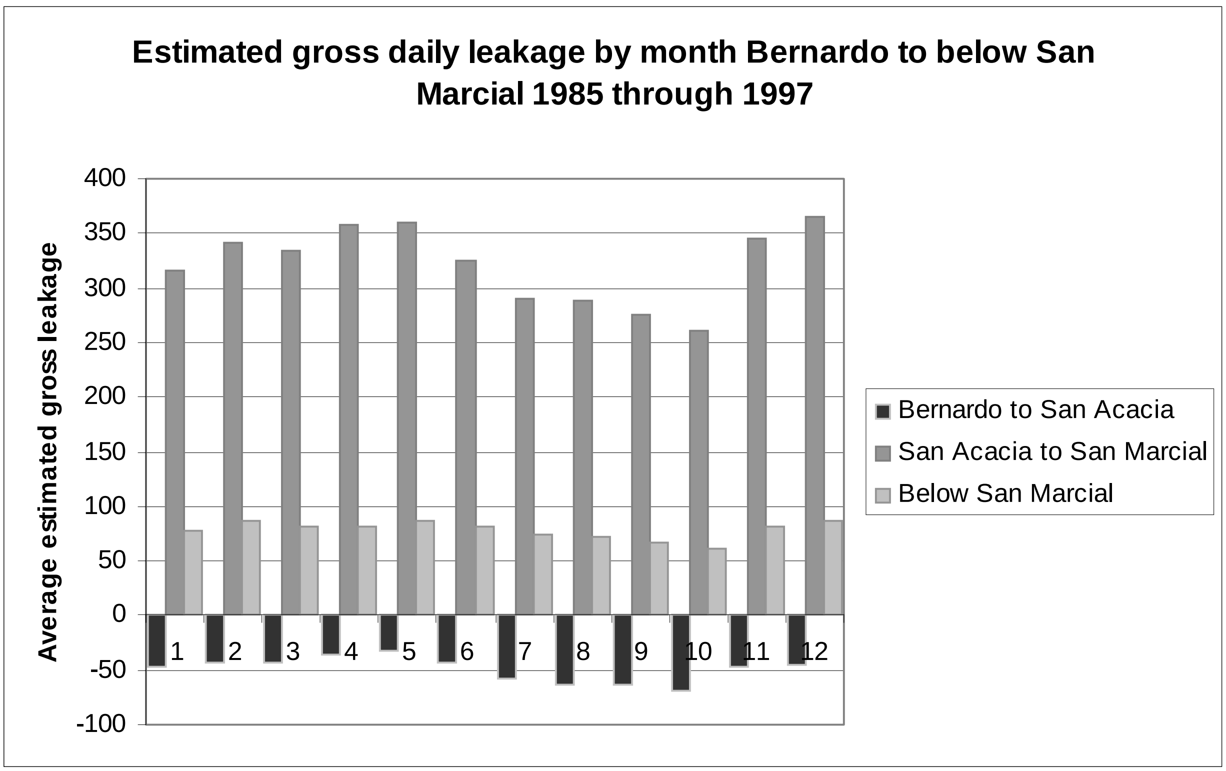

Figure 7. Estimated gross leakage by month, Bernardo to below San Marcial, 1985-97.

The floodway reach from the Bernardo gage to the San Acacia gage gains water most days. As in the upper reach river/shallow ground-water system, not all water that leaks from Unit 7 Drain stays in the shallow ground-water system. Some water is used by riparian vegetation, some flows to the deep ground-water system, and some enters the river channel. An estimate was needed of the amount of Unit 7 Drain water that leaks to the shallow ground-water system and actually enters the river channel. There have been no seepage measurements in the Bernardo to San Acacia reach that could be used to estimate that part of total Unit 7 Drain leakage that enters the river channel by way of the shallow ground-water system. This volume of water was estimated using flow data at the Bernardo and San Acacia floodway gages and the calculated leakage to the river channel. Only data for the winter months of December through February were considered. Inflow from the Rio Puerco was subtracted from the gaged flow at San Acacia. Only those days when calculated leakage to the river was greater than the gain shown by the gage data were used in the analysis. Only those days met the assumption that water lost from the Unit 7 Drain goes to riparian vegetation and the deeper ground-water system and enters the river channel. This analysis showed that 47 percent of the water that leaves Unit 7 Drain enters the river channel. For this reach, the RiverWare model adjusts the leakage from Unit 7 Drain to the river channel to be 47 percent of the calculated leakage. This percentage is very preliminary and needs to be refined for later versions of the model.

The reach from San Acacia to below San Marcial behaves similarly to the upper reach in that water leaks from the river to the conveyance channel. As in the upper reach and the Bernardo to San Acacia reach, some water that leaks to the shallow ground-water system is used by riparian vegetation, some goes to the deep ground-water system, and some enters the conveyance channel. Seepage measurements in this reach were used to estimate the amount of river-channel leakage that is intercepted by the conveyance channel and enters Elephant Butte. By using USBR winter seepage data for 1995 and 1997, the average calculated river leakage intercepted by the conveyance channel is 68 percent. Again this is a preliminary value and needs to be refined.

Method for Determining Travel Time Lag (River Routing)

River routing describes the techniques used to account for the attenuation and downstream travel time of streamflow (hydrograph). The variable time lag method, which was described in the Rio Chama section of this documentation, is also used for the reach from Cochiti to Elephant Butte Reservoir. The following summarizes travel time determinations for reaches in the middle valley.

Rio Grande from Cochiti to San Felipe

The stream gage Rio Grande below Cochiti Dam is located 700 ft downstream from Cochiti Dam. For approximately 15 mi, from Cochiti Dam to San Felipe, the Rio Grande consists of a sand and gravel riverbed and has highly erodible, sandy banks. A reduction of sediment load from the dam has resulted in the degradation and armoring of the riverbed and made relatively erodible banks increasingly more vulnerable. The storage and release of floodwaters have diminished the magnitude of peak flows, increased the duration of lower flows, and narrowed the river channel. Throughout this reach the river channel averages approximately 320 ft in width and the river’s longitudinal slope is approximately 7 ft/mi.

Cochiti Dam is the beginning of the MRGCD Cochiti Division. Diversions of water into the Sili Canal and the Cochiti East Side Main Canal bypass the gage Rio Grande below Cochiti Dam. These diversions are measured and are used in agricultural-depletion computations for the middle valley. Galisteo Creek is a major east-side tributary to the Rio Grande in this reach. It contributes little water but is included in this reach routing.

Table 40 summarizes stream-gage and calibration data used in determining the power coefficient (wave velocity exponent) that is applied to average velocity measurements to determine river travel time lags in this reach (**graphs 108 and 109)**.

Table 40. Summary of stream-gage and calibration data for the reach of the Rio Grande from Cochiti to San Felipe

	Below Cochiti Dam	San Felipe	Total Δ
Period of analysis	11/70 – 6/99	9/21/70 – 6/2/99	
River mile (above mouth)	1588	1573	15
Elevation (feet above sea level)	5226	5116	110
Drainage area (square miles)	14900	16100	1200
Number of measurements	237	566	
Wave velocity exponent (β)	0.398	0.591	
Coefficient of determination (R^2)	0.862	0.907	

Table 41 summarizes factors used to determine travel time lags for this reach (**graphs 110 and 111).**

Table 41. Adopted travel time lags (TL) for the reach of the Rio Grande from Cochiti to San Felipe

			Time lag (hours) for indicated flow rate (cfs)						
Gage	TL vs. Q equation	R^2	50	200	500	750	1000	3000	6000
Cochiti	$TL = 217.95Q^{-0.5979}$	0.9233	21	9	5	4	4	2	1
San Felipe	$TL = 71.114Q^{-0.4087}$	0.8230	14	8	6	5	4	3	2
	Adopted travel times for reach→		18	9	5	4	4	2	2

When the wave velocity was computed, the gage cross section was assumed to be representative of the entire routing reach. The Rio Grande below Cochiti Dam cross section is fixed because it has riprapped banks, and the area remains relatively the same for most measurements. To get a power regression curve to represent time lag versus flow, measurements were deleted at the lower end discharges when a power regression curve was developed for the Rio Grande below Cochiti Dam gage. The measurements were deleted so that the curve would have a representative number of discharges at the low and high ends. A large part of the record had discharge measurements at the lower end. Six data points from San Felipe gage measurement data were not used in the determination of the wave velocity exponent and travel time.

Rio Grande from San Felipe to Albuquerque

The stream gage Rio Grande at San Felipe is located 0.8 mi upstream from San Felipe Pueblo. The Rio Grande consists of a sand and gravel riverbed from San Felipe to Bernalillo and a predominantly sand riverbed from Bernalillo to Albuquerque. The river has low, sandy, erodible banks in this area, and the wide sandy river channel has many bars. Levees line both sides of the river to protect heavily developed valley areas. A strip of riparian cottonwood bosque lies between the levees and riverbanks. Throughout this reach the river channel averages approximately 420 ft in width and the river's longitudinal slope is approximately 5 ft/mi. The Corps' Water Control Manual for Cochiti Dam (1996b) designates 7000 cfs as the channel capacity at Albuquerque.

The effect of cutting off the sediment supply at Cochiti Dam has manifested itself in the upper portion of this reach. The river channel is presently degrading throughout the reach, and gravel is appearing in the riverbed material.

Major tributaries in this reach include the Arroyo de las Barrancas, Calabacillas Arroyo, and Las Huertas Creek. In addition, the Albuquerque Metropolitan Arroyo Flood Control Authority's (AMAFCA's) North Floodway Channel discharges into the Rio Grande. The Jemez River enters the Rio Grande just below Angostura diversion dam. Formerly, this tributary contributed a heavy sediment load to the Rio Grande, but Jemez Dam now controls the Jemez River's sediment load near its mouth. Tributary inflow from the North Floodway Channel, Jemez River, and Angostura diversions are modeled. The remaining tributaries are modeled as unmeasured local inflows. Wastewater inflow from the Bernalillo and Rio Rancho wastewater treatment plants are also modeled.

Table 42 summarizes stream-gage and calibration data used in determining the power coefficient (wave velocity exponent) that is applied to average velocity measurements to determine river travel time lags in this reach (**graphs 109 and 112**).

Table 42. Summary of stream-gage and calibration data for the reach of the Rio Grande from San Felipe to Albuquerque

	Rio Grande at San Felipe	Rio Grande at Albuquerque	Total Δ
Period of analysis	9/21/70 – 6/2/99	9/28/70 – 9/1/99	
River mile (above mouth)	1573	1540	33
Elevation (feet above sea level)	5116	4946	170
Drainage area (square miles)	16100	17440	1340
Number of measurements	566	807	
Wave velocity exponent (β)	0.591	0.7310	
Coefficient of determination (R^2)	0.907	0.974	

Table 43 summarizes factors used to determine travel time lags for this reach (**graphs 113 and 114**).

Table 43. Adopted travel time lags (TL) for the reach of the Rio Grande from San Felipe to Albuquerque

			Time lag (hours) for indicated flow rate (cfs)						
Gage	TL vs. Q equation	R^2	50	200	500	750	1000	3000	6000
San Felipe	$TL = 157.5Q^{-0.4087}$	0.823	32	18	12	11	9	6	5
Albu-querque	$TL = 94.62Q^{-0.2695}$	0.831	33	23	18	16	15	11	9
	Adopted travel times for reach→		32	20	15	13	12	8	7

Forty-two data points from the stream-gage data for Rio Grande at Albuquerque were not used in the development of the wave velocity exponent or in the travel time determination.

Rio Grande from Albuquerque to Rio Grande Floodway near Bernardo

The stream gage for the Rio Grande at Albuquerque is located on the downstream side of the Central Avenue Bridge. For approximately 53 mi from Albuquerque to the Rio Grande Floodway near Bernardo, the Rio Grande consists of a sand riverbed. The presence of numerous alternating bars and middle islands is a strong characteristic of this reach. In the lower portion of the reach the flood plain widens, and salt cedar stands start to become denser. Throughout this reach the width of the river channel averages between 420 and 510 ft when flowing bankfull.

AMAFCA's South Floodway Channel discharges into Tijeras Arroyo, which discharges into the Rio Grande in this reach. In the Albuquerque area over-bank flows are limited. Over-bank flooding starts below the Isleta diversion. In this reach riverbanks are well vegetated and more stable.

Discharge from the Albuquerque, Los Lunas, and Belen wastewater treatment plants; Tijeras Arroyo; and the diversions at Isleta Dam are modeled.

Table 44 summarizes stream-gage and calibration data used in determining the power coefficient (wave velocity exponent) that is applied to average velocity measurements to determine river travel time lags in this reach (**graphs 112 and 115)**.

Table 44. Summary of stream-gage and calibration data for the reach of the Rio Grande from Albuquerque to Rio Grande Floodway near Bernardo

	Rio Grande at Albuquerque	Rio Grande Floodway near Bernardo	Total Δ
Period of analysis	9/28/70 – 9/1/98	6/10/70 – 8/11/98	
River mile (above mouth)	1540	1487	53
Elevation (feet above sea level)	4946	4723	223
Drainage area (square miles)	17440	19230	1790
Number of measurements	807	511	
Wave velocity exponent (β)	0.7310	0.748	
Coefficient of determination (R^2)	0.974	0.932	

Table 45 summarizes factors used to determine travel time lags for this reach (**graphs 116 and 117**).

Table 45. Adopted travel time lags (TL) for the reach of the Rio Grande from Albuquerque to Rio Grande Floodway near Bernardo

Gage	TL vs. Q equation	R^2	Time lag (hours) for indicated flow rate (cfs) 50	200	500	750	1000	3000	6000
Albu-querque	$TL = 151.96Q^{-0.2695}$	0.831	53	36	28	26	24	18	15
Bernardo	$TL = 135.05Q^{-0.2471}$	0.725	51	36	29	26	25	19	16
	Adopted travel times for reach→		52	36	29	26	24	18	15

Fifteen data points for the gage Rio Grande Floodway near Bernardo were removed and not used in the determination of the wave velocity exponent and travel times.

Rio Grande Floodway near Bernardo to Rio Grande Floodway at San Acacia

The gage Rio Grande Floodway near Bernardo is located on the downstream side of the U.S. Highway 60 Bridge, 2 mi east of Bernardo, and 5 mi above the mouth of the Rio Puerco. The character of the Rio Grande begins a transition below the confluence of the Rio Puerco between aggrading and degrading conditions. The system is generally degrading but periodically is subject to heavy sediment loads from the Rio Puerco and the Rio Salado. Both tributaries are ephemeral and contribute heavy, sediment-laden flows during the summer. Between Bernardo and the Rio

Puerco are dense salt cedar stands. The river channel averages approximately 560 ft in width when bankfull throughout this reach.

The RiverWare model for this reach includes flow routing, local inflow, tributary inflow, diversions, return flow from drains, evaporation, and ground-water gain or loss. The model includes reach objects for routing flow and computing local inflow. Tributary inflow from the Rio Puerco and diversions at the San Acacia Diversion Dam to the Socorro Main Canal and the Rio Grande Conveyance Channel are modeled. Also modeled in this reach are return flow from the lower San Juan Riverside Drain and the Unit 7 Drain. Four streamflow gages near Bernardo measure the combined flow in the river and drains. Downstream near San Acacia, three streamflow gages measure the combined flow in the river and drains.

Table 46 summarizes stream-gage and calibration data used in determining the power coefficient (wave velocity exponent) that is applied to average velocity measurements to determine river travel time lags in this reach (**graphs 115 and 118**).

Table 46. Summary of stream-gage and calibration data for the reach of the Rio Grande from Rio Grande Floodway near Bernardo to Rio Grande Floodway at San Acacia

	Rio Grande Floodway near Bernardo	Rio Grande Floodway at San Acacia	Total Δ
Period of analysis	6/10/70 – 8/11/98	9/29/70 – 10/20/99	
River mile (above mouth)	1487	1473	14
Elevation (feet above sea level)	4723	4655	68
Drainage area (square miles)	19230	26770	7540
Number of measurements	511	797	
Wave velocity exponent (β)	0.7479	0.7535	
Coefficient of determination (R^2)	0.932	0.984	

Table 47 summarizes factors used to determine travel time lags for this reach (**graphs 119 and 120**).

Table 47. Adopted travel time lags (TL) for the reach of the Rio Grande from Rio Grande Floodway near Bernardo to Rio Grande Floodway at San Acacia

			Time lag (hours) for indicated flow rate (cfs)						
Gage	TL vs. Q equation	R^2	50	200	500	750	1000	3000	6000
Bernardo	$TL = 37.202Q^{-0.2471}$	0.725	14	10	8	7	7	5	4
San Acacia	$TL = 26.739Q^{-0.2157}$	0.857	11	9	7	6	6	5	4
	Adopted travel times for reach→		13	9	8	7	7	5	4

Seven data points for the stream gage Rio Grande Floodway at San Acacia were removed and not used in the computation of the wave velocity exponent and time lag.

Accounting of Measured Diversions, Return Flows, and Inflows

The middle valley model reaches isolate many losses or gains occurring on the river. Diversions from the MRGCD are represented in the model starting below Cochiti Lake. Historical diversion information from 1985 to 1997 was obtained from the MRGCD. Some of the water that is diverted for agricultural uses is known to eventually return to the river; however, the return-flow locations have never been measured. The return-flow points are only now being equipped with measurement devices, which will greatly increase the ability to predict the amount of water in the river. Because of a lack of return-flow data for the past, the model was built and calibrated to estimate total return flows based on other known or empirically derived values.

All known measured diversions and returns to the Rio Grande are accounted for as described below. These data represent the effects of human activities within the basin, such as diversions and return flows for agricultural, municipal, and industrial uses.

Middle Rio Grande Conservancy District Diversion Data

Diversion data for the four MRGCD diversions are available and used in the agricultural depletions and return-flow calculations for the Middle Rio Grande Valley. The USGS collects and publishes data on diversions at Cochiti Dam into the Cochiti Eastside Main Canal and the Sili Canal and on the flow of the Socorro Main Canal at San Acacia.

The records of daily diversion by the MRGCD at the Angostura diversion dam (Angostura diversion dam is a combination of Atrisco feeder, Albuquerque Main Canal, and Algodones Riverside Drain, when available) and the Isleta diversion dam (combination of Belen High Line Canal and Peralta Main Canal) were reviewed by plotting monthly hydrographs of the average monthly discharge for each year to determine any changes or trends in monthly diversion data over each year of available data. Monthly hydrographs of daily data were plotted to compare each year's operation, view any abrupt or unusual changes in daily discharge, and evaluate the data relative to the capacity of the canals. The review did not include evaluating the reliability of the stage-discharge relation or the record of stage.

Data on the capacity of the irrigation canals were taken from USBR's table F-8 of the Plan for Development (1947) and were listed as follows:

- Albuquerque Main Canal (intake capacity below heading): 570 cfs;
- Peralta Main Canal (intake capacity): 350 cfs; and
- Belen High Line Canal (intake capacity): 410 cfs.

Records of the relatively small discharge of the Cacique Acequia, Chical Acequia, and Chical Lateral, all diverting water from the Isleta diversion dam, were not reviewed. The period of record of the data used in the review was 1978-95 for the Angostura diversions and 1974-95 for the Isleta diversions.

Monthly hydrographs of discharge for each year show greater amounts of water diverted during dry years and smaller amounts during wet years and no major, long-term anomalies. The plots of daily data are difficult to interpret and reflect the daily fluctuations from changing irrigation demands because of rainfall or operating practices. Anomalies or discrepancies will be evaluated, and estimates of daily values can be substituted.

The measured flow of the Socorro Main Canal may include flow from the Unit 7 Drain, flow diverted from the San Acacia diversion dam, or a combination of flow from both of these sources. The flow of Unit 7 Drain has no historical record, and for purposes of including its contribution into the model, the sum of the flow of the conveyance channel near Bernardo and the Bernardo Interior Drain is assumed to be equal to the contribution of Unit 7 Drain into the Socorro Main Canal. When flow in the Socorro Main Canal is less than flow in the Unit 7 Drain, then the excess flow in the Unit 7 Drain is assumed to return through a gate to the Rio Grande above the San Acacia diversion.

Estimate of Middle Rio Grande Conservancy District Agricultural Depletions

One part of estimating return-flow values is predicting how much water irrigated crops might use on a daily basis. Currently, no technology exists in the MRGCD fields that accurately measures how much water each farmer is applying. The best way to estimate crop usage is to empirically derive an evapotranspiration (ET) rate for varying crops in the valley. Although ET rates are still being developed, some rates are available and are being successfully implemented by the USBR. A web site has been established that reports precipitation and other weather data such as temperature, solar radiation, and wind speed at pertinent weather stations along the Rio Grande. This information is then compiled and used in conjunction with the latest geographic information system (GIS) land coverages for the river. This information allows the USBR to then compute ET rates based on the methodology for estimating consumptive use of water by crops. The empirical algorithm used is the modified Penman method. The URGWOM crop acreage was estimated using MRGCD annual crop reports rather than the GIS coverage provided by the ET toolbox because the GIS coverage did not have historical land coverages and was fixed with the limited coverages available. Isolating the crop acreages allows the user to change this variable in the model for different years in a forecast scenario.

Each reach in the URGWOM model middle valley represents historical irrigation diversions. This diversion request at each diversion dam location is subtracted from the amount of water in the river. Estimates of crop ET are subtracted from the diverted water as well as canal seepage and deep percolation. Crop types predominantly grown and represented in the model are the following:

Alfalfa	Pasture grasses	Grapes
Sorghum	Wheat	Barley
Corn/silage	Chile peppers	Cotton
Nursery	Orchards	Silage
Oats	Misc. fruits	Melons
Misc. vegetables	Apples	

Measured Tributary Inflows and Return Flows

Measured inflows to the reach of the Rio Grande between Cochiti and Elephant Butte Reservoir that are modeled are compiled in **table 48**.

Table 48. Measured inflows that are modeled

Tributary inflow	Irrigation- and wastewater-return flow
Galisteo Creek	Bernalillo wastewater treatment plant
Jemez River	Rio Rancho wastewater treatment plant
AMAFCA North Floodway Channel	Albuquerque wastewater treatment plant
Tijeras Arroyo	Los Lunas wastewater treatment plant
Rio Puerco	Belen wastewater treatment plant

Jemez River from the Gage near Jemez to Jemez Canyon Reservoir

The discharge from the Jemez River is accounted for as a measured inflow to the Rio Grande. This reach is 23.5 miles long. Inflow to this reach is from the Jemez Mountains and from snowpack runoff March through early June. The Rio Salado is the major tributary to the Jemez and enters the Jemez River near San Ysidro. About 2,700 acres are irrigable with water diverted from the Jemez River; however, no major diversion structures are in this reach. Inflow to this reach is recorded by the gage Jemez River near Jemez. Outflow from the reach and inflow to the Jemez Reservoir are estimated by the USBR on a daily basis. From 1953 to 1958 there was a gage above Jemez Canyon Dam. Outflow from Jemez Canyon Dam is recorded by the gage Jemez River below Jemez Canyon Dam and is the flow that the model uses as inflow to the Rio Grande at the confluence. No lag time or losses are considered between Jemez Canyon Dam and the confluence.

Table 49 summarizes stream-gage and calibration data used in determining the power coefficient (wave velocity exponent) that is applied to average velocity measurements to determine river travel time lags in this reach (**graph 121**).

Table 49. Summary of stream-gage and calibration data for the reach of the Jemez River near Jemez

	Jemez River near Jemez
Period of analysis	10/5/1970 – 10/25/1999
River mile (above mouth)	29.5
Elevation (feet above sea level)	5622
Drainage area (square miles)	470
Number of measurements	619
Wave velocity component (β)	0.7004
Coefficient of determination (R^2)	0.9591

Table 50 summarizes factors used to determine travel time lags for this reach (**graph 122**).

Table 50. Adopted travel time lags (TL) for the reach of the Jemez River near Jemez

			Time lag (hours) for indicated flow rate (cfs)					
Gage	TL vs. Q equation	R^2	25	37	75	150	300	400+
Near Jemez	$TL=19.903Q^{-0.3015}$	0.808	9	7	5	4	3	2
	Adopted travel times for reach→		9	7	5	4	3	2

Channel losses were estimated using daily flows at the gage near Jemez and daily inflows to the Jemez Canyon Dam estimated by the USBR. For some months the coefficient of determination was very low. As a check, the 1953-58 data for the gages near Jemez and above Jemez Canyon Dam were used to determine channel losses. The remaining data set, after filtering and removal of the data when flows had to be estimated at one or both gages, was sparse and probably not representative of the reach. In addition, the 1953-58 period was dry and not representative of the model period of 1985-96. Regressions using data for the gage above Jemez Canyon Dam and estimated daily inflows are presented in **table 51**.

Table 51. Correlations between routed flow and estimated inflow and adopted monthly loss coefficients for the reach of the Jemez River from near Jemez to above Jemez Canyon Reservoir, 1985-96

Month	n (days)	Slope	y-intercept	R^2	Slope (y=0)	R^2	Adopted loss coefficient	Adopted loss constant
Jan	142	0.748	-3.51	0.454	0.6579	0.4465	-0.1	-3.5
Feb	157	0.8998	-11.49	0.829	0.7223	0.7863	-0.1	-3.5
Mar	152	0.9577	-19.24	0.961	0.8653	0.9458	-0.14	0
April	184	0.8644	-3.26	0.952	0.8589	0.9515	-0.14	0
May	211	0.9087	-24.52	0.968	0.8364	0.9577	-0.09	-24.5
June	203	0.8536	-18.95	0.874	0.6802	0.8203	-0.15	-19
July	181	0.7228	-10.98	0.754	0.5529	0.6965	-0.28	-11
Aug	154	0.7503	-10.62	0.675	0.5624	0.6223	-0.25	-10.6
Sept	140	0.8464	-14.99	0.554	0.4652	0.4283	-0.15	-15
Oct	120	0.8588	-13.8	0.716	0.5498	0.6047	-0.15	-13.8
Nov	199	0.9253	-11.37	0.754	0.7011	0.7008	-0.07	-11.3
Dec	163	0.9509	-11.17	0.454	0.6287	0.3981	-0.07	-11.3

Regression equations with a 0-intercept were used only for March and April. For the other months the 0-intercept regression equation did not fit the data and a non-0-intercept was used. For June through September, when flows are low, the model produced negative outflow from the reach from near Jemez to the Jemez Canyon Reservoir. A model rule was written that replaced a negative flow with 0. During the investigation using the 1953-58 flow data from near Jemez to above Jemez Canyon Dam, it was observed that there were many days during June through September when flow at the gage above Jemez Canyon Dam would go to zero although there was flow at the gage near Jemez. Although much more representative of channel losses, the regression equations that use a non-0-intercept could not account for all the flow being lost without applying the constant, therefore creating a negative flow.

Estimates of Canal Seepage

For the purpose of this analysis, seepage from canals and laterals is assumed lost to the surface-water system and to instead recharge the deep aquifer because of the “disconnect” between the canals and ground-water table.

The USBR has undertaken an analysis of seepage from canals and laterals in the Middle Rio Grande Valley (Middle Rio Grande Water Assessment, 1997, supporting documents 6, 12, and 15). The study was able to identify seepage losses for 1935, 1955, 1975, and 1993 using soil surveys and other field data. The seepage rates for 1975 and 1993 were used in the RiverWare model because these data are consistent with the selected model data set. The seepage data are summarized in **table 52**.

Table 52. Canal seepage rates for the Middle Rio Grande Valley

Reach	Canal length (miles)	1975 Seepage rate (feet/day)	1975 Daily seepage loss (cfs)	1993 Seepage rate (feet/day)	1993 Daily seepage loss (cfs)
Cochiti to Angostura	100	0.4	36	0.40	37
Angostura to Albuquerque	209	0.01	2	0.15	29
Albuquerque to Bernardo	491	0.10	45	0.10	45
Bernardo to San Acacia	174	0.20	32	0.20	32

A canal average wetted perimeter of 15 ft was assumed. Canal losses occurred only during the irrigation season (March 1 - October 31) when the canals are operating. The increase in the seepage rate between 1975 and 1993 for the Angostura to Cochiti reach is a result of the effects of the sediment-control function of Cochiti Lake, which is assumed to have been entirely manifest on the reach after 1975. Seepage rates since 1993 are extrapolated to 1997 rates.

Historical Crop Acreage Data

Data obtained from the MRGCD include 1997 and 1998 crop census (irrigated acreage) reports by MRGCD division and annual irrigated-crop acreage for 1985-97. **Table 53** is a tabulation of MRGCD annual irrigated-crop acreage for 1985-97. Individual crop acreage for 1985-96 was estimated by proportioning the 1985-96 annual totals by the percentage of each individual crop to the total amount in 1997-98. These data were then disaggregated into division data by the percentage of irrigated acreage of each division to the total District-wide irrigated acreage. The data then were adjusted to estimate irrigated-crop acreage by URGWOM river reach.

Table 53. Middle Rio Grande Conservancy District total irrigated-crop acreage, 1985-97

Year	Alfalfa	Pasture grass	Sorghum	Wheat	Corn	Chile peppers	Grapes	Melons	Apples	Nursery	Oats
1985	32225	12448	115	525	2241	505	0	55	290	150	146
1986	28625	13273	360	721	2755	560	0	105	290	175	255
1987	28455	12445	375	740	2645	555	0	81	275	185	275
1988	28725	12565	380	945	2843	575	0	90	275	185	325
1989	31526	14274	28	369	2673	687	0	104	547	516	1007
1990	28697	13979	168	564	576	719	56	29	407	625	0
1991	28725	12565	380	945	2843	575	140	90	296	185	355
1992	28835	13125	400	915	2867	595	140	60	184	190	330
1993	20921	15677	48	2024	2000	1376	9	57	357	2343	2433
1994	30837	15848	0	0	0	896	7	0	294	22	0
1995	23336	17517	0	0	0	881	64	0	202	0	0
1996	25451	17569	0	0	0	631	19	0	198	0	0
1997	25706	16200	0	0	0	606	40	101	203	0	0

Year	Misc. fruit	Misc. vege-tables	Garden/ orchard	Not har-vested	Fallow land	Barley	Hay	Silage	Cotton	Total acres
1985	161	580	759	275	11648	380	4025	2945	82	58433
1986	201	828	885	285	11821	395	4435	3331	0	58081
1987	199	768	795	290	14246	390	4115	2995	0	55561
1988	161	834	825	283	12343	395	3918	3433	0	57341
1989	73	924	825	400	12549	0	2651	0	12	56331
1990	0	611	599	400	14481	56	4640	2306	38	68951
1991	316	628	825	283	12343	395	4512	3433	0	69834
1992	107	836	975	295	12241	395	3818	3592	0	69900
1993	652	592	552	2329	14511	7	362	2886	0	69136
1994	0	0	168	300	15000	0	2664	3621	0	69657
1995	0	0	159	0	5087	0	3601	3998	0	54845
1996	0	0	835	0	5335	0	3580	4104	0	57722
1997	0	0	743	0	9742	0	3184	4400	0	60925

Table 54 shows irrigated acreage in each MRGCD division as a percentage of total irrigated acreage based on the 1997 and 1998 crop census reports. The annual crop census data were then adjusted, as described below, to develop data that could be used in URGWOM river reaches.

Table 54. Middle Rio Grande Conservancy District (MRGCD) irrigated acreage, by division, as percentage of total irrigated acreage

MRGCD division	Percentage of total
Cochiti	7
Albuquerque	14
Belen	56
Socorro	23
Total	100

The URGWOM river reach from Cochiti to San Felipe extends to the gage Rio Grande at San Felipe; the MGRCD's Cochiti Division extends an additional 6 mi downstream to the Angostura diversion dam. Copies of the 1992 USBR aerial mosaic of the Middle Rio Grande and copies of the 1936 Rio Grande Joint Investigation surveys were used to estimate the amount of irrigable lands along the Rio Grande between San Felipe and Angostura. On the basis of interpretations and scaling from these surveys, an estimated 16 percent of the irrigable lands in the Cochiti Division are in the reach between San Felipe and Angostura. This percentage was applied to the annual irrigated acreage in the Cochiti Division, and the crop acreage for the Cochiti Division was reduced by 16 percent to calculate crop data for the URGWOM reach from Cochiti to San Felipe. **Table 55** is a tabulation of annual irrigated-crop acreage used in the Cochiti to San Felipe reach.

Table 55. Annual irrigated-crop acreage, Cochiti to San Felipe, 1985-97

Year	Alfalfa	Pas- ture grass	Sor- ghum	Wheat	Corn	Chile pep- pers	Grapes	Melons	Apples	Nur- sery	Oats
1985	1886	728	7	31	131	30	0	3	17	9	9
1986	1675	777	21	42	161	33	0	6	17	10	15
1987	1665	728	22	43	155	32	0	5	16	11	16
1988	1681	735	22	55	166	34	0	5	16	11	19
1989	1845	835	2	22	156	40	0	6	32	30	59
1990	1679	818	10	33	34	42	3	2	24	37	0
1991	1681	735	22	55	166	34	8	5	17	11	21
1992	1687	768	23	54	168	35	8	4	11	11	19
1993	1224	917	3	118	117	81	1	3	21	137	142
1994	1805	927	0	0	0	52	0	0	17	1	0
1995	1366	1025	0	0	0	52	4	0	12	0	0
1996	1489	1028	0	0	0	37	1	0	12	0	0
1997	1504	948	0	0	0	35	2	6	12	0	0

Year	Misc. fruit	Misc. vege- tables	Garden/ orchard	Not har- vested	Fallow land	Barley	Hay	Silage	Cotton	Total acres
1985	9	34	44	16	682	22	236	172	5	3419
1986	12	48	52	17	692	23	260	195	0	3399
1987	12	45	47	17	834	23	241	175	0	3251
1988	9	49	48	17	722	23	229	201	0	3356
1989	4	54	48	23	734	0	155	0	1	3296
1990	0	36	35	23	847	3	272	135	2	4035
1991	18	37	48	17	722	23	264	201	0	4087
1992	6	49	57	17	716	23	223	210	0	4091
1993	38	35	32	136	849	0	21	169	0	4046
1994	0	0	10	18	878	0	156	212	0	4076
1995	0	0	9	0	298	0	211	234	0	3210
1996	0	0	49	0	312	0	210	240	0	3378
1997	0	0	43	0	570	0	186	257	0	3565

The MRGCD Albuquerque Division extends from Angostura to Isleta; the URGWOM San Felipe to Albuquerque reach extends to the gage Rio Grande at Albuquerque, 14 mi above Isleta Dam. On the basis of documents accompanying applications made by the MRGCD to the New Mexico State Engineer for permits to change the point of diversion and point of use (1930), about 52 percent of the irrigated lands in the Albuquerque District are located between the Albuquerque gage and Isleta Dam. Therefore, the irrigated-crop acreage for the San Felipe to Albuquerque reach was estimated by adding 16 percent of the Cochiti Division crop acreage to the Albuquerque Division acreage and reducing the initial Albuquerque Division acreage by 52 percent. **Table 56** is a tabulation of annual irrigated-crop acreage for the reach from San Felipe to Albuquerque.

Table 56. Annual irrigated-crop acreage, San Felipe to Albuquerque, 1985-97

Year	Alfalfa	Pas-ture grass	Sor-ghum	Wheat	Corn	Chile pep-pers	Grapes	Melons	Apples	Nur-sery	Oats
1985	2535	979	9	41	176	40	0	4	23	12	11
1986	2252	1044	28	57	217	44	0	8	23	14	20
1987	2239	979	30	58	208	44	0	6	22	15	22
1988	2260	989	30	74	224	45	0	7	22	15	26
1989	2480	1123	2	29	210	54	0	8	43	41	79
1990	2258	1100	13	44	45	57	4	2	32	49	0
1991	2260	989	30	74	224	45	11	7	23	15	28
1992	2269	1033	31	72	226	47	11	5	14	15	26
1993	1646	1233	4	159	157	108	1	4	28	184	191
1994	2426	1247	0	0	0	70	1	0	23	2	0
1995	1836	1378	0	0	0	69	5	0	16	0	0
1996	2002	1382	0	0	0	50	1	0	16	0	0
1997	2023	1275	0	0	0	48	3	8	16	0	0

Year	Misc. fruit	Misc. vege-tables	Garden/ orchard	Not har-vested	Fallow land	Barley	Hay	Silage	Cotton	Total acres
1985	13	46	60	22	916	30	317	232	6	4598
1986	16	65	70	22	930	31	349	262	0	4570
1987	16	60	63	23	1121	31	324	236	0	4372
1988	13	66	65	22	971	31	308	270	0	4512
1989	6	73	65	31	987	0	209	0	1	4432
1990	0	48	47	31	1139	4	365	181	3	5425
1991	25	49	65	22	971	31	355	270	0	5495
1992	8	66	77	23	963	31	300	283	0	5500
1993	51	47	43	183	1142	1	28	227	0	5440
1994	0	0	13	24	1180	0	210	285	0	5481
1995	0	0	13	0	400	0	283	315	0	4315
1996	0	0	66	0	420	0	282	323	0	4542
1997	0	0	58	0	767	0	251	346	0	4794

The MRGCD Belen Division extends from Isleta Dam to just above San Acacia Dam. The URGWOM reach from Albuquerque to Bernardo extends to the gage Rio Grande Floodway at Bernardo, about 14 mi above the San Acacia dam. On the basis of a review of the 1992 USBR aerial mosaic, all irrigated acres in the Belen Division were determined to be located above the Bernardo gage. Therefore, the irrigated-crop acreage for the Albuquerque to Bernardo reach was estimated by adding 52 percent of the Albuquerque Division irrigated-crop acreage to the Belen Division irrigated-crop acreage. **Table 57** is a tabulation of annual irrigated-crop acreage for the reach from Albuquerque to Bernardo.

Table 57. Annual irrigated-crop acreage, Albuquerque to Rio Grande Floodway near Bernardo, 1985-97

Year	Alfalfa	Pas-ture grass	Sor-ghum	Wheat	Corn	Chile pep-pers	Grapes	Melons	Apples	Nur-sery	Oats
1985	20392	7877	73	332	1418	320	0	35	184	95	92
1986	18114	8399	228	456	1743	354	0	66	184	111	161
1987	18006	7875	237	468	1674	351	0	51	174	117	174
1988	18177	7951	240	598	1799	364	0	57	174	117	206
1989	19950	9033	18	234	1691	435	0	66	346	327	637
1990	18159	8846	106	357	364	455	35	18	258	396	0
1991	18177	7951	240	598	1799	364	89	57	187	117	225
1992	18247	8306	253	579	1814	377	89	38	116	120	209
1993	13239	9920	30	1281	1266	871	6	36	226	1483	1540
1994	19514	10029	0	0	0	567	4	0	186	14	0
1995	14767	11085	0	0	0	557	40	0	128	0	0
1996	16105	11118	0	0	0	399	12	0	125	0	0
1997	16267	10251	1	1	1	383	25	64	128	1	1

Year	Misc. fruit	Misc. vege-tables	Garden/ orchard	Not har-vested	Fallow land	Barley	Hay	Silage	Cotton	Total acres
1985	102	367	480	174	7371	240	2547	1864	52	36976
1986	127	524	560	180	7480	250	2806	2108	0	36754
1987	126	486	503	184	9015	247	2604	1895	0	35159
1988	102	528	522	179	7811	250	2479	2172	0	36285
1989	46	585	522	253	7941	0	1678	0	8	35646
1990	0	387	379	253	9164	35	2936	1459	24	43632
1991	200	397	522	179	7811	250	2855	2172	0	44191
1992	68	529	617	187	7746	250	2416	2273	0	44233
1993	413	375	349	1474	9183	4	229	1826	0	43749
1994	0	0	106	190	9492	0	1686	2291	0	44079
1995	0	0	101	0	3219	0	2279	2530	0	34706
1996	0	0	528	0	3376	0	2265	2597	0	36526
1997	1	1	470	1	6165	1	2015	2784	1	38553

The only lands irrigated in the Bernardo to San Acacia reach are those served by the La Joya Community Ditch. These lands are not part of the MRGCD, so the acreage is not reported by the MRGCD. Irrigated lands in this reach probably do not exceed 100 acres.

Estimates of Crop Consumptive Use

Crop consumptive-use calculations are based on the application of the annual crop census data to consumptive-use coefficients developed by King and Bawazir (1998). Crop coefficients are developed to estimate ET for use in the USBR's automated data-collection network, the ET toolbox. ET crop loss is related to irrigation diversions.

Estimate of Deep Percolation from Irrigation-Return Flow

Deep percolation is the amount of infiltrated water per irrigation event that is not used by crops and moves through the soil profile to the water table. Deep percolation from rainfall on crops is negligible. The USBR (1997, supporting document 7) analyzed soil textures and permeabilities in the Middle Rio Grande Valley. Deep percolation rates for the soil series and crop types in the Middle Rio Grande Valley range from 0.10 to 1.22 ft/year. A deep percolation rate of 1.0 acre-ft/acre/year was applied to MRGCD census data. Deep percolation is modeled as a loss to the amount diverted.

Computation of Unmeasured Return Flows

The amount of water diverted into the MRGCD canals and water collecting in the drains have been depleted by crop consumptive use, canal seepage, and deep percolation. A gain is also recognized in the canals and especially the drains from shallow ground-water interaction with the river. This model accounts for ground-water leakage from the river with an estimated 54 percent of the leakage intercepted by the drains. This gain is added to the remaining diverted flows, after depletions, and is factored into the gross return-flow value. (See channel leakage section for more information.) The balance was compared to the flow at the river cross section at the downstream end of each URGWOM reach. Comparison of these two values indicates the amount of return flow arriving in the river before the drain discharge.

Hydrograph Routing

Flow at the upstream station, adjusted for known human effects, channel losses, and return flows that occur throughout the reach, is routed to the downstream gage.

Computation of Local Inflow

Local inflow is computed by subtracting the downstream-observed hydrograph from the routed hydrograph. The result encompasses the effects on the river that cannot be accounted for or measured. An optional procedure would involve the smoothing of the local inflow data set to minimize negative effects.

Rio Grande Floodway at San Acacia to Rio Grande Floodway at San Marcial

For all intents and purposes the reach begins at the San Acacia diversion dam. Flow arriving at the dam is divided into three components: the Rio Grande Floodway, which is the actual river channel; the Rio Grande Conveyance Channel, also referred to as the low-flow conveyance channel (LFCC), which is a manmade channel constructed in the late 1950's to efficiently carry Rio Grande flows to Elephant Butte Reservoir; and the Socorro Main Canal, which is the source of irrigation water for farms to the south. The other inflow into the reach is the Unit 7 Drain, which flows into the Socorro Main Canal just below the dam. The gage on the Rio Grande Floodway is located approximately 0.2 mi below the dam, the gage on the LFCC is located 1.2 mi south of the dam, and the gage on the Socorro Main Canal is located 0.5 mi below the dam **(fig. 6**).

During the 1960's and 1970's most of the non-flood flow in the Rio Grande was diverted into the LFCC. The LFCC has a capacity of approximately 2000 cfs. During the 1980's and 1990's water was diverted infrequently, then eventually not at all.

The floodway has seen significant aggradation. Toward the lower section of the reach, the river is above the level of the surrounding valley. There is significant leakage from the river channel into the LFCC. The channel profile varies greatly through the reach: some sections are broad and shallow, whereas others are narrow and deep.

Currently, there is only one location where water from the LFCC can be returned to the floodway. Prior to the aggradation of the floodway channel, discharge from the LFCC was diverted to the floodway at other locations as operation and maintenance dictated.

The LFCC also serves as both a drain collecting irrigation-return flows at certain locations and as a source of supply for irrigation. Check structures provide adequate head to deliver water into adjoining laterals.

Table 58 summarizes stream-gage and calibration data used in determining the power coefficient (wave velocity exponent) that is applied to average velocity measurements to determine river travel time lags in this reach (**graphs 118 and 123).**

Table 58. Summary of stream-gage and calibration data for the reach of the Rio Grande from Rio Grande Floodway at San Acacia to Rio Grande Floodway at San Marcial

	Rio Grande Floodway at San Acacia	Rio Grande Floodway at San Marcial	Total Δ
Period of analysis	1970-84	1970-84	
River mile (above mouth)	1473	1425	48
Elevation (feet above sea level)	4654	4455	199
Drainage area (square miles)	26770	27700	930
Number of measurements	483	340	
Wave velocity exponent (β)	0.7469	0.7058	
Coefficient of determination (R^2)	0.98	0.97	

Table 59 summarizes factors used to determine travel time lags for this reach (**graphs 124 and 125**).

Table 59. Adopted travel time lags (TL) for the reach of the Rio Grande from Rio Grande Floodway at San Acacia to Rio Grande Floodway at San Marcial

			Time lag (hours) for indicated flow rate (cfs)						
Gage	TL vs. Q equation	R^2	50	200	500	750	1000	3000	6000
San Acacia	$TL = 133.6Q^{-0.255}$	0.87	49	35	27	25	23	17	15
San Marcial	$TL = 154.24Q^{-0.2942}$	0.85	49	32	25	22	20	15	12
	Adopted travel times for reach→		49	34	26	23	22	16	13

Table 60 summarizes the adopted loss coefficients for this reach. See **graphs 126-137** for plots of observed flow versus routed flow, filtered for losses, and regression analyses for this reach.

Table 60. Correlations between routed flow and observed flow and adopted monthly loss coefficients for the reach of the Rio Grande from Rio Grande Floodway at San Acacia to Rio Grande Floodway at San Marcial, 1987-96

Month	n (days)	Slope	y-intercept	R^2	Slope (y=0)	R^2	Adopted loss coefficient
Jan	217	0.674	93.9	0.64	0.767	0.63	0.767
Feb	213	0.926	-113.3	0.97	0.856	0.96	0.856
Mar	216	0.799	-4.4	0.93	0.797	0.93	0.797
April	210	0.951	-168.4	0.97	0.883	0.96	0.883
May	209	0.888	-111.6	0.95	0.859	0.95	0.859
June	146	0.931	-168.5	0.95	0.889	0.94	0.889
July	101	0.933	-252.9	0.95	0.857	0.94	0.857
Aug	120	0.705	-83.2	0.80	0.660	0.79	0.660
Sept	138	0.669	-83.7	0.72	0.617	0.71	0.617
Oct	146	0.755	-68.2	0.77	0.590	0.72	0.59
Nov	218	0.871	-117.6	0.91	0.779	0.90	0.779
Dec	238	0.928	-171.8	0.83	0.775	0.81	0.775

The two gages used for this reach are Rio Grande Floodway at San Acacia and Rio Grande Floodway at San Marcial. The Rio Grande has been gaged at San Acacia since 1936, but the site has been moved several times since its installation. It was moved to its present location in 1965. From 1958 to 1964, flow into the conveyance channel was included in total flow; since 1964, data for the conveyance channel and the floodway have been reported separately. The Rio Grande has been gaged at San Marcial since 1895. Flows in the floodway were separated from those in the conveyance channel in 1964.

The velocities obtained from the wave velocity formula appeared to be too high based on observed conditions. Because this is a highly variable reach, the cross sections at the gaging stations may not be representative of cross sections found along the length of the reach. After the model was run using several different lag times both greater and less than that obtained from the formula, it was determined that increasing the lag time by 60 percent increased the R^2 value obtained for the regression equation of computed versus observed flow. The lag times also were very close to those widely used as estimates of travel time lags from San Acacia to San Marcial.

Rio Grande Floodway and Low Flow Conveyance Channel at San Marcial to Elephant Butte Reservoir

Flow into Elephant Butte Reservoir from San Marcial is through both the Rio Grande Floodway and the LFCC. Flow from the two waterways used to merge above the reservoir. Flooding and sediment deposition have now caused the flows to enter separately.

There are gaging stations on the river and LFCC at San Marcial. Inflow into the reservoir is calculated using a reservoir budget computation, which takes into account outflows and evaporation losses. The top of the conservation pool is at an elevation of 4407 ft. At this elevation the beginning of the reservoir is essentially at the San Marcial gage. Elephant Butte Dam is approximately 42 mi downstream from the gage. The reservoir stage has been as low as 4220 ft in 1953 and as high as 4409 ft in 1942. This wide range of pool elevations also means a wide range of reservoir lengths that would affect routing. The model currently does not have a method to vary the lag time of a reach based on a changing reach length caused by a rising or falling reservoir stage downstream. The distance between the San Marcial gage and the head of the reservoir was varied to determine the best relation (highest R^2) between observed flow and routed flow.

No diversions or drains (except for the LFCC) are in this reach. There is a large expanse of riparian vegetation, mainly west of the river, from below San Marcial to the head of the reservoir.

Table 61 summarizes stream-gage and calibration data used in determining the power coefficient (wave velocity exponent) that is applied to average velocity measurements to determine river travel time lags in this reach (**graphs X and Y).**

Table 61. Summary of stream-gage and calibration data for the reach of the Rio Grande from Rio Grande Floodway at San Marcial to Rio Grande at Elephant Butte Reservoir

	Rio Grande Floodway at San Marcial	Rio Grande at Elephant Butte Reservoir	Total Δ
Period of analysis	1970-84	1970-84	
River mile (above mouth)	1425	1383	42
Elevation (feet above sea level)	4455	4209*	246
Drainage area (square miles)	27700	29400	2600
Number of measurements	340	n/a	
Wave velocity exponent (β)	0.7058	n/a	
Coefficient of determination (R^2)	0.97	n/a	

* Power-house tail-water elevation.

Table 62 summarizes factors used to determine travel time lags and travel time lags for this reach.

Table 62. Adopted travel time lags (TL) for the reach of the Rio Grande from Rio Grande Floodway at San Marcial to Elephant Butte Reservoir

			Time lag (hours) for indicated flow rate (cfs)						
Gage	TL vs. Q equation	R^2	50	200	500	750	1000	3000	6000
San Marcial	$TL = 84.521Q^{-0.2942}$	0.85	27	18	14	12	11	8	7
	Adopted travel times for reach→		27	18	14	12	11	8	7

Table 63 summarizes the adopted monthly loss coefficients for this reach. See **graphs 137-148** for plots of observed flow versus routed flow, filtered for losses, and regression analyses for this reach.

Table 63. Correlations between routed flow and observed flow and adopted monthly loss coefficients for the reach of the Rio Grande from Rio Grande Floodway at San Marcial to Elephant Butte Reservoir, 1987-96

Month	n (days)	Slope	y-intercept	R^2	Slope (y=0)	R^2	Adopted loss coefficient
Jan	122	0.8544	-138.52	0.73	0.7608	0.72	-0.24
Feb	77	0.8417	-132.19	0.89	0.7844	0.89	-0.22
Mar	82	0.8503	-95.30	0.86	0.8110	0.85	-0.19
April	125	0.9384	-624.9	0.78	0.7807	0.76	-0.22
May	88	0.9198	-394.81	0.89	0.8408	0.88	-0.16
June	52	0.9203	-278.9	0.90	0.8499	0.89	-0.15
July	63	0.8501	-223.49	0.89	0.7862	0.88	-0.21
Aug	62	0.6540	-114.99	0.55	0.5917	0.55	-0.41
Sept	55	0.6018	-120.19	0.82	0.5479	0.81	-0.45
Oct	96	0.8199	-132.12	0.79	0.6873	0.76	-0.31
Nov	169	0.7115	-80.27	0.68	0.6621	0.68	-0.34
Dec	132	0.9358	-208.21	0.85	0.7940	0.83	-0.21

Flow into Elephant Butte Reservoir is through both the Rio Grande and the LFCC. Because only one inflow to the reservoir is calculated, all flow was assumed to be routed in the same manner. Because Elephant Butte is such a large reservoir, calculating the proper lag times was difficult. Values were obtained using regression analysis that yielded the highest R^2 values. Actual inflow values were sometimes very suspect, which made calibration difficult. This is a highly variable reach as evident from the low R^2 values obtained for the loss analysis.

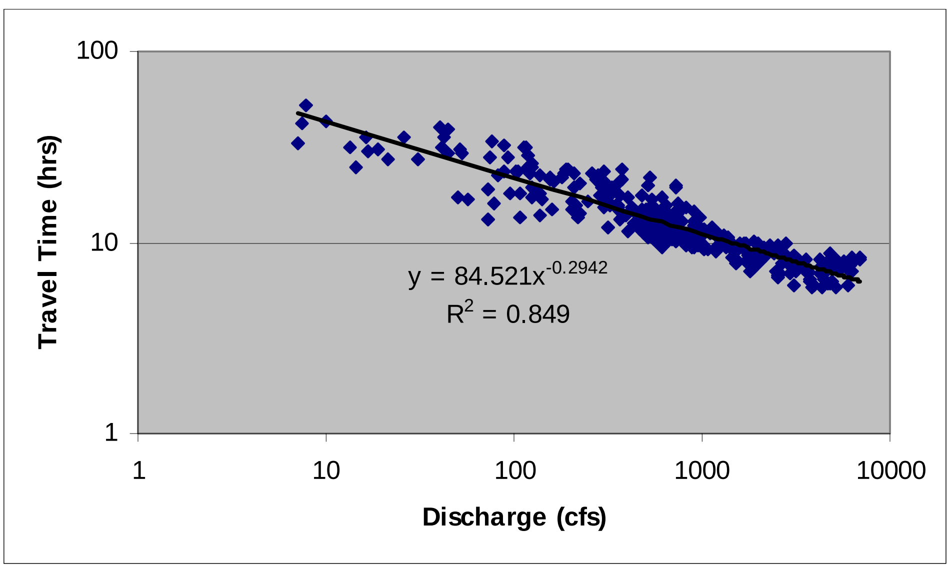

Figure 8. Travel time versus discharge, Rio Grande from Rio Grande Floodway at San Marcial to Elephant Butte Reservoir.

RESERVOIRS IN THE MIDDLE VALLEY

DESCRIPTION OF PHYSICAL RIVER ATTRIBUTES

Three reservoirs were constructed on the Rio Grande in the middle valley and its tributaries. Only two, Jemez Canyon Reservoir and Cochiti Lake, are modeled. Jemez Canyon and Cochiti Dams are operated for flood- and sediment-control purposes only. **Table 64** summarizes general information about these facilities.

Table 64. General information about Middle Rio Grande Valley reservoirs

	Cochiti	Jemez Canyon
Type:	Earth fill	Earth fill
Year completed:	1973	1953
Structural height (feet):	251	149
Top width (feet):	30	23
Width at base (feet):	1760	835
Crest length (feet):	28815	861
Crest elevation (feet above sea level):	5479	5271.6
Outlet works discharge capacity (cfs):	14790	9700

Cochiti Lake

Cochiti Lake is owned and operated by the Corps in coordination with other Corps projects in the basin. Cochiti Lake has maintained a permanent recreation pool of approximately 50000 acre-ft since the dam was completed. The permanent pool, which includes an intermittent pond in the arm of the Santa Fe River, provides sediment-control benefits, trapping about 1000 acre-ft of sediment per year. The permanent pool was established by and is maintained by San Juan-Chama Project water. The remaining capacity of the reservoir, totaling about 545,000 acre-ft, is reserved for flood and sediment control.

Cochiti Dam is operated to bypass all inflow to the lake to the extent that downstream channel conditions are capable of safely bypassing the flow. When inflow to the lake is in excess of the downstream channel capacity, flood-control operations are initiated. Stored floodwaters are released when downstream channel conditions permit, all in accordance with the provisions of Public Law 86-645 and the Rio Grande Compact. **Table 65** contains elevation information about Cochiti Lake.

Table 65. Elevation-related information about Cochiti Lake

	Elevation (feet)	Area (acres)	Total capacity (acre-feet)
Top of dam:	5479.00	11176	778,859
Maximum pool:	5474.10	10636	725,159
Total storage at spillway crest:	5460.50	9307	589,159
Permanent pool (varies):	5335.92	1200	50843
Conduit invert:	5255.00	95	153

Jemez Reservoir

Jemez Canyon Dam is owned and operated by the Corps for flood- and sediment-control purposes. The sediment-control function of Jemez Canyon Reservoir was significantly enhanced by the establishment and maintenance of a permanent pool. A sediment retention pool was about 2000 acre-feet in 1979, which has grown to its present size of about 27,000 acre-feet. Jemez Canyon Dam is operated in conjunction with Cochiti Dam to limit downstream flow to existing channel capacity. **Table 66** contains elevation information about Jemez Canyon Dam.

Table 66. Elevation-related information about Jemez Canyon Dam

	Elevation (feet)	Area (acres)	Total capacity (acre-feet)
Top of embankment:	5271.6	5320	263,773
Maximum pool:	5271.2	5300	262,473
Total storage at spillway crest:	5232.0	2954	100,485
Sediment retention pool:	5196.7	1379	27485
Zero storage:	5154.0	0	0

description of RESERVOIR PHYSICAL PROPERTIES

Cochiti Lake and Jemez Canyon Reservoirs follow the general mass-balance equation for reservoirs:

$$S_t – S_{t-1} – I - P_t + E_t + O = 0$$

where:

S_t = total storage today, in acre-ft;
S_{t-1} = total storage yesterday, in acre-ft;
I = inflow into the reservoir, in acre-ft/day;
P_t = physical model precipitation, in acre-ft/day;
E_t = physical model evaporation, in acre-ft/day; and
O = outflow from the reservoir, in acre-ft/day.

Physical model precipitation is determined by using the equation:

$$P_t = R_t/(A_{res})$$

where:

R_t = rainfall, in inches/day; and

A_{res} = average reservoir area, in acres.

Physical model evaporation is determined by using one of two equations, depending on the time of year. The summer equation is:

$$Et = Ep/12(coeff)(A_{res})$$

where:

E_p = pan evaporation, in inches/day; and

coeff = pan evaporation coefficient (0.7 for reservoirs in the Rio Grande Basin).

The winter equation is:

$$E_t = [(T_{max} + T_{min})/2] * (k/days) * (1\text{-}cov) * A_{res}$$

where:

T_{max} = maximum daily temperature, in °F;

T_{min} = minimum daily temperature, in °F;

k = factor for month, in inches per °F;

days = days in the month; and

cov = reservoir ice cover, in percent.

SIMULATION OF RESERVOIR SYSTEM

Jemez Reservoir and Cochiti Lake are simulated as storage reservoirs in the model. Each reservoir solves a mass-balance equation as well as many user-defined solutions. The start of construction of the reservoir object is setting up the object and selecting user-defined methods or solutions. Data are then put into the object's slots (variable or primary data storage container on any object), which are defined by user-defined methods. With all reservoir objects, the default slots are inflow, storage, pool elevation, release, and an elevation-volume table.

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