Miscellaneous Paper HL-95-2
May 1995

US Army Corps
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Waterways Experiment
Station

Sedimentation Study, Rio Grande at Espanola, New Mexico

Numerical Model Investigation

by Michael J. Trawle

Prepared for U.S. Army Engineer District, Albuquerque

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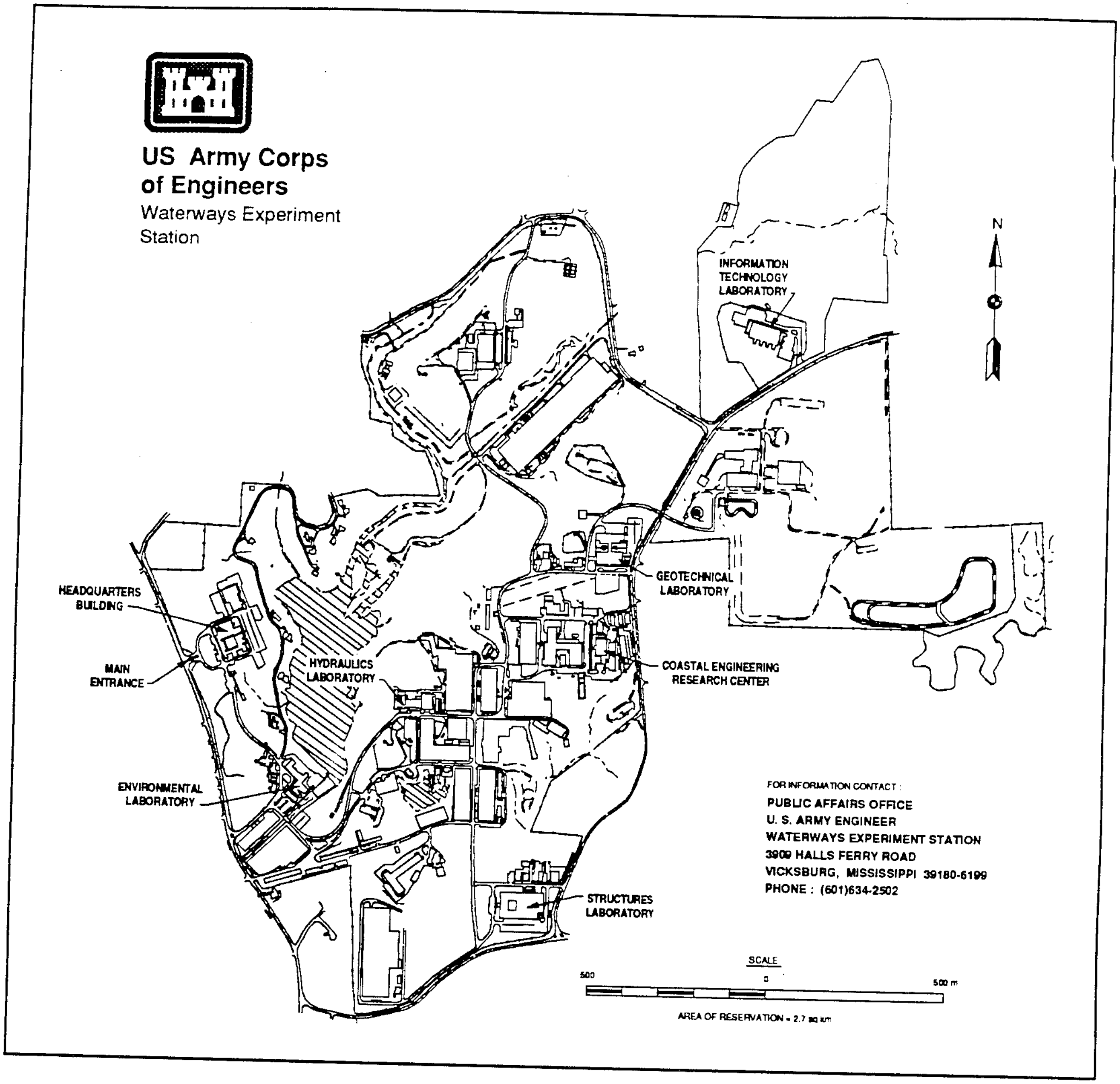

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Contents

Preface

This numerical model investigation of the Rio Grande through Espanola, NM, reported herein was conducted at the U.S. Army Engineer Waterways Experiment Station (WES), at the request of the U.S. Army Engineer District, Albuquerque.

This investigation was conducted during the period October 1993 to September 1994 in the Hydraulics Laboratory (HL), WES, under the direction of Mr. Frank A. Herrmann, Jr., Director of the Hydraulics Laboratory; Mr. Richard A. Sager, Assistant Director of the Hydraulics Laboratory; Dr. Larry L. Daggett, Acting Chief of the Waterways Division (WD), HL; and Mr. Michael J. Trawle, Chief of the Math Modeling Branch, WD. The Project Engineer for this study was Mr. Trawle. Technical assistance was provided by Ms. Peggy H. Hoffman, Math Modeling Branch.

During the course of this study, close working contact was maintained with Mr. Bruce Beach, Albuquerque District, who served as the coordinating engineer, providing required data and technical assistance.

At the time of publication of this report, Director of WES was Dr. Robert W. Whalin. Commander was COL Bruce K. Howard, EN.

Conversion Factors, Non-SI to SI Units of Measurement

Non-SI units of measurement used in this report can be converted to SI units as follows:

Multiply	By	To Obtain
cubic feet	0.02831685	cubic meters
feet	0.3048	meters
miles (U.S. statute)	1.609347	kilometers
square miles	2.589998	square kilometers
tons (2,000 pounds, mass)	907.1847	kilograms

1 Introduction

Description of the Watershed

The Espanola Valley lies in north-central New Mexico and includes the Rio Grande basin from approximately Valarde, New Mexico, to the Otowi Bridge (Figure 1), a distance of about 28 miles[1]. The drainage area of the Rio Grande watershed at Valarde is about 10,400 square miles, at Espanola is about 10,900 square miles, and at Otowi Bridge is about 14,300 square miles. In addition to the Rio Grande, located within the study area are the Rio Chama, the Santa Cruz River, the Arroyo Guachapanque, and several lesser streams. Espanola, the largest community in the Valley, lies 85 miles south of the New Mexico-Colorado border and 25 miles north of Santa Fe (U.S. Army Engineer District, Albuquerque, 1992).

The Espanola Valley lies within the Upper Rio Grande Valley of New Mexico. The area varies in topography form precipitous mountains to broad, relatively featureless plains. Land forms found in the area include plateaus, buttes, mesas, volcanos, lava flows, canyons, and a broad river valley. Elevations of the Rio Grande streambed range from 5,500 ft (msl) at Otowi, to 5580 ft (msl) at Espanola, to 5760 ft (msl) at Valarde. Elevations within the contributing watershed range to the 12,000-ft Jemez Mountains along the western border of the basin, and to the 13,000-ft Truchas Peaks, within the Sangre de Cristo Mountains, along the eastern limits of the basin. The median elevation at Espanola is 5,595 ft (msl) (U.S. Army Engineer District, Albuquerque, 1992).

Description of the Study Area

The city of Espanola is located along both banks of the Rio Grande main stem and includes both major and minor tributaries flowing through its corporate limits (Figure 2). There are three separate sources of flood damages

[1] A table of factors for converting non-SI units of measurement to SI units is found on page vi.

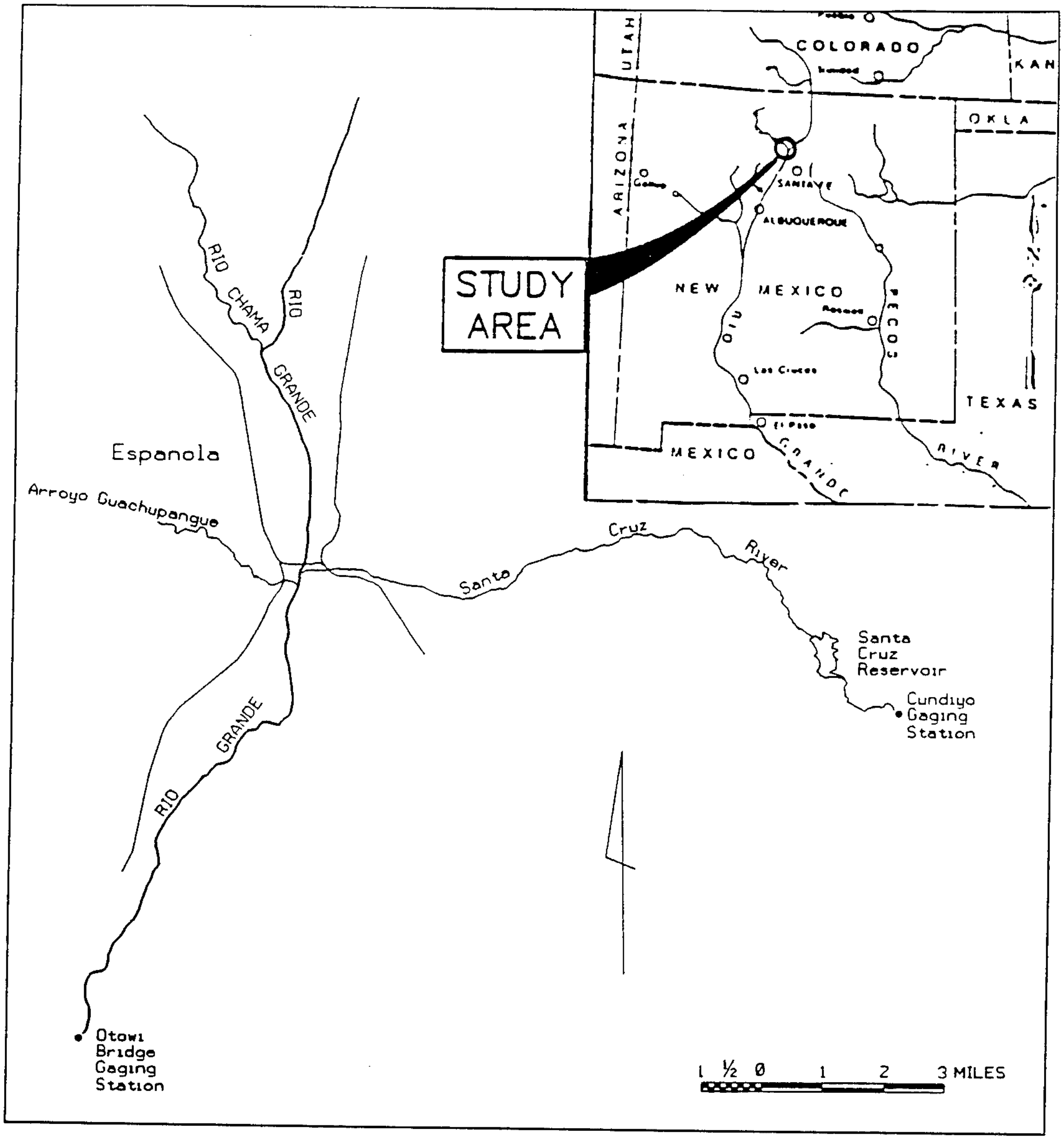

Figure 1. Espanola Valley

at Espanola: the Rio Grande along its east bank, the Rio Grande along its west bank, and the Arroyo Guachupangue. There are existing earthen levees along some reaches of the east and west banks of the Rio Grande through Espanola, constructed by the Bureau of Reclamation and consisting of pushed-up banks. It is estimated that flood damages due to high flows in the Rio Grande will begin with the 10-year event. The Arroyo Guachupangue is a right bank tributary to the Rio Grande. It joins the Rio Grande near the southern corporate limits of Espanola. The Arroyo Guachupangue has no existing flood improvements. Damages begin along its north bank with the 20-year flood. Properties located within these three floodplain areas include public roads and utilities, as well as public, private, and commercial structures (U.S. Army Engineer District, Albuquerque, 1992).

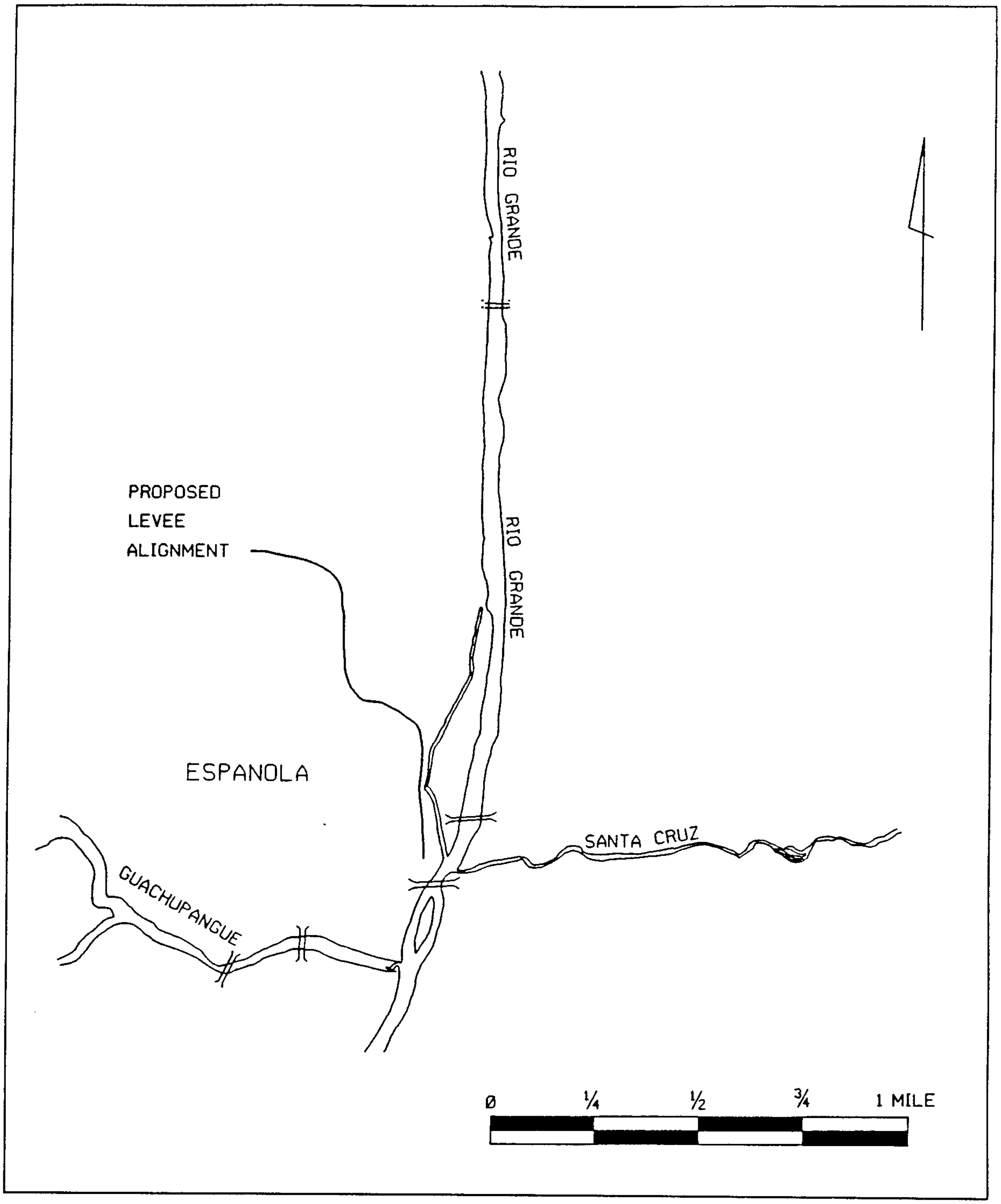

Figure 2. City of Espanola and vicinity

Plan Description

The plan tested in this study consists of a flood-control levee system on the west bank of the Rio Grande as shown in Figure 3. The length of the levee system is approximately 4300 ft.

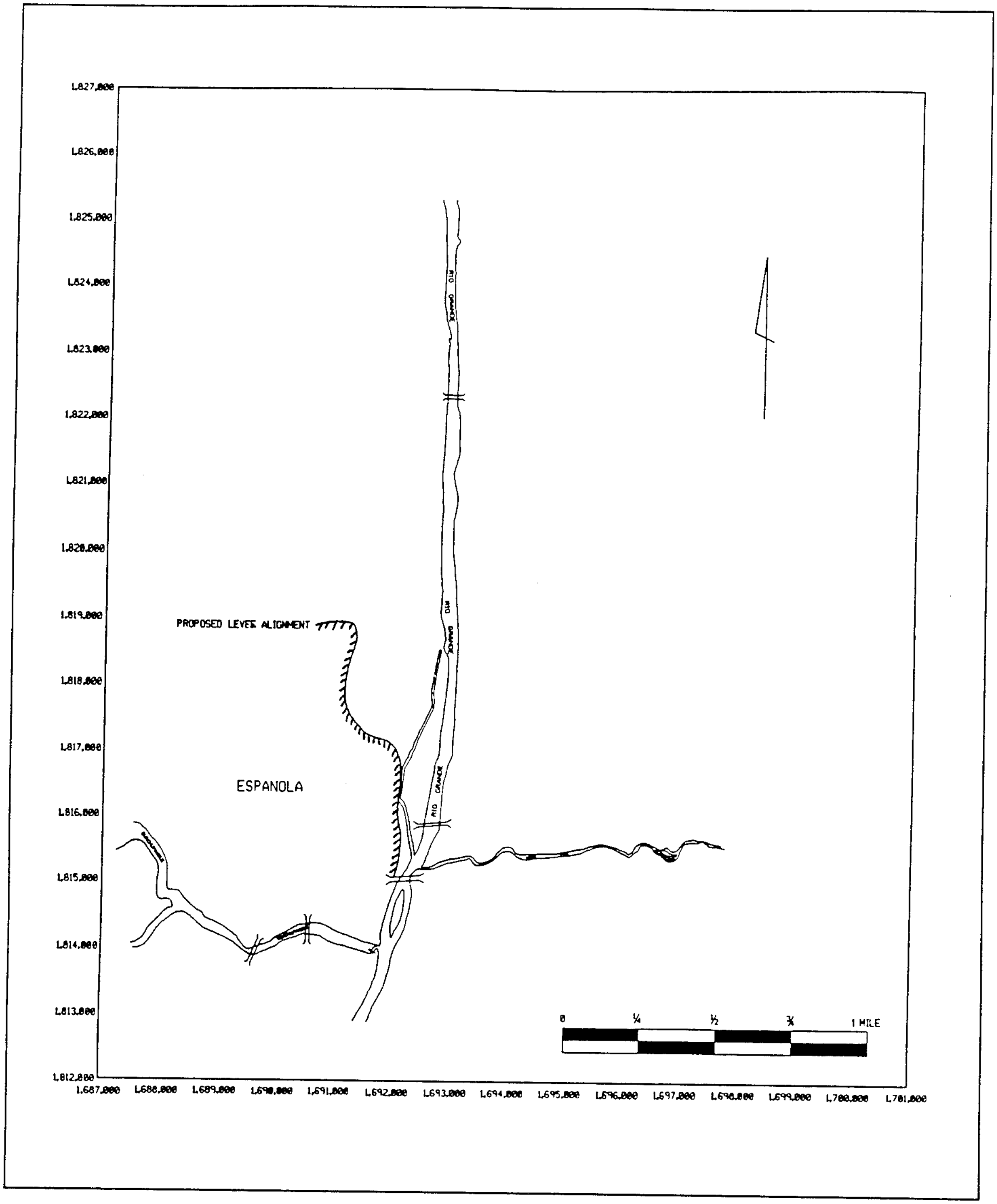

Figure 3. Proposed levee plan

A typical channel and overbank cross section used in this study are shown in Figure 4. The cross-sections used in this study are from the HEC-2 geometry file provided by the Albuquerque District.

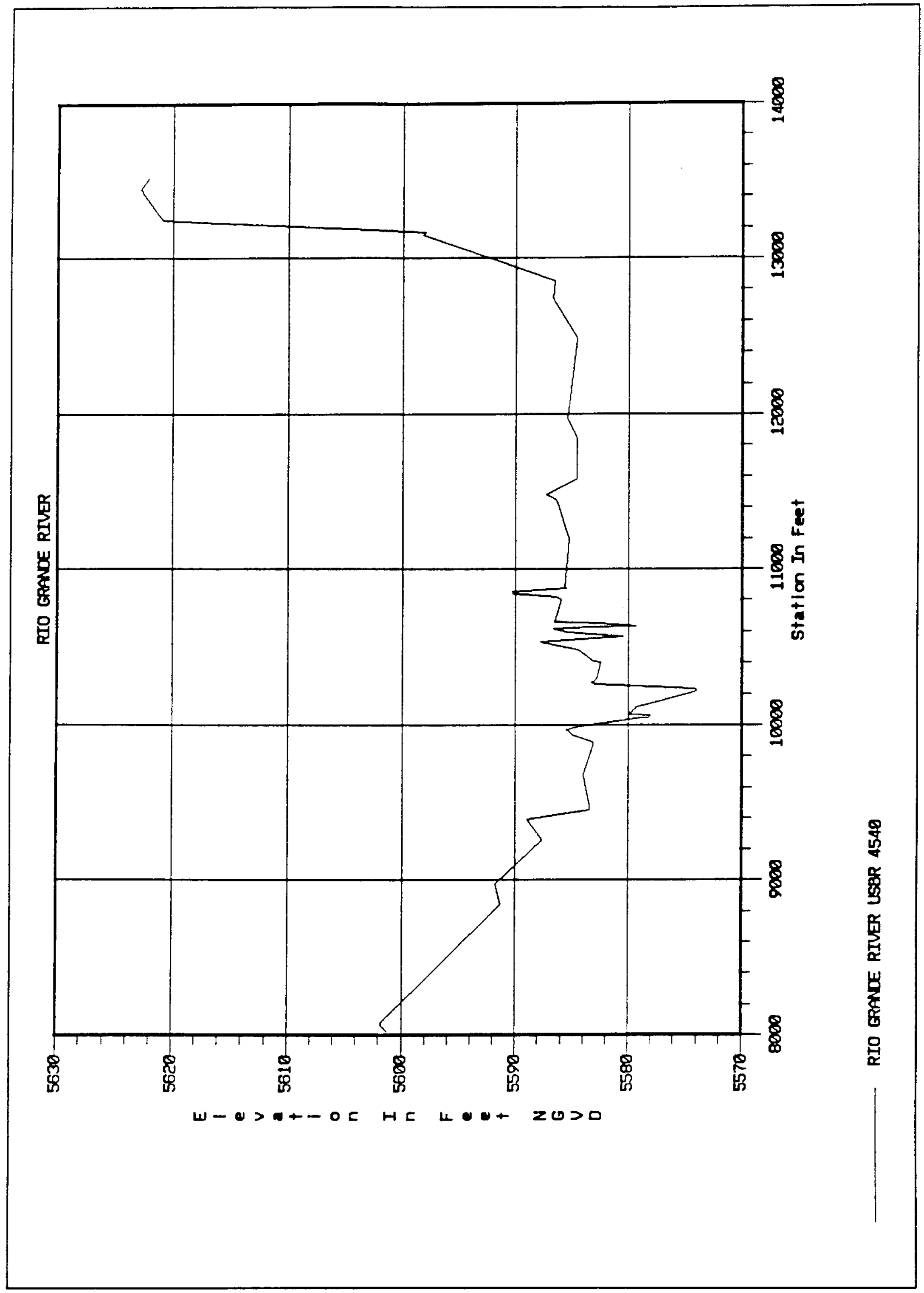

Figure 4. Typical Rio Grande cross section

Purpose and Approach

The purpose of the sedimentation analysis is to evaluate the Rio Grande flood-event bed response to the proposed levee system on the west bank of the Rio Grande. A second objective is to evaluate the flood-event behavior of the Rio Grande streambed at the mouth of the Santa Cruz River. To perform the analysis, the one-dimensional numerical sedimentation model (TABS-1) was applied. The model was used to reproduce historical flow and sedimentation patterns for the Rio Grande through Espanola and, after satisfactory reproduction of the existing flow-sediment regime was accomplished, to determine the impact of the proposed levee system on the bed of the Rio Grande.

2 The Model

Description

The TABS-1 one-dimensional sedimentation code was used to develop the numerical model for this study. Development of this computer program was initiated by Mr. William Thomas at the US Army Engineer District, Little Rock, in 1967. Further development at the US Army Engineer Hydrologic Engineering Center (USAEHEC) by Mr. Thomas produced the widely used HEC-6 generalized computer program for calculating scour and deposition in rivers and reservoirs (USAEHEC 1991). Additional modification and enhancement to the basic program by Mr. Thomas at the US Army Engineer Waterways Experiment Station (WES) led to the TABS-1 program currently in use (Thomas 1980, 1982). TABS-1 is considered to be experimental in that it is not documented to the point that it can be made available for general use, but can be made available by special request.

The program produces a one-dimensional model that simulates a series of steady-state discharge events and their effect on the sediment transport capacity at cross-sections and the resulting degradation or aggradation. The program calculates hydraulic parameters using a standard-step backwater method assuming subcritical flow. The program assigns critical depth for water-surface elevation if the backwater calculations indicate transitions to supercritical flow. However, for supercritical flow, hydraulic parameters for sediment transport are calculated assuming normal depth in the channel. A more detailed description of the program capabilities is included in Appendix A.

Channel Geometry

The study reach extended from Rio Grande sta 8+00 at the downstream end to Rio Grande sta 120+00 at the upstream end and up the Santa Cruz River from its confluence with the Rio Grande for 5500 ft. The channel geometry for the model was based on cross sections from a HEC-2 backwater model provided by the USAED, Albuquerque. Cross-section locations used in the model are shown in Figure 5.

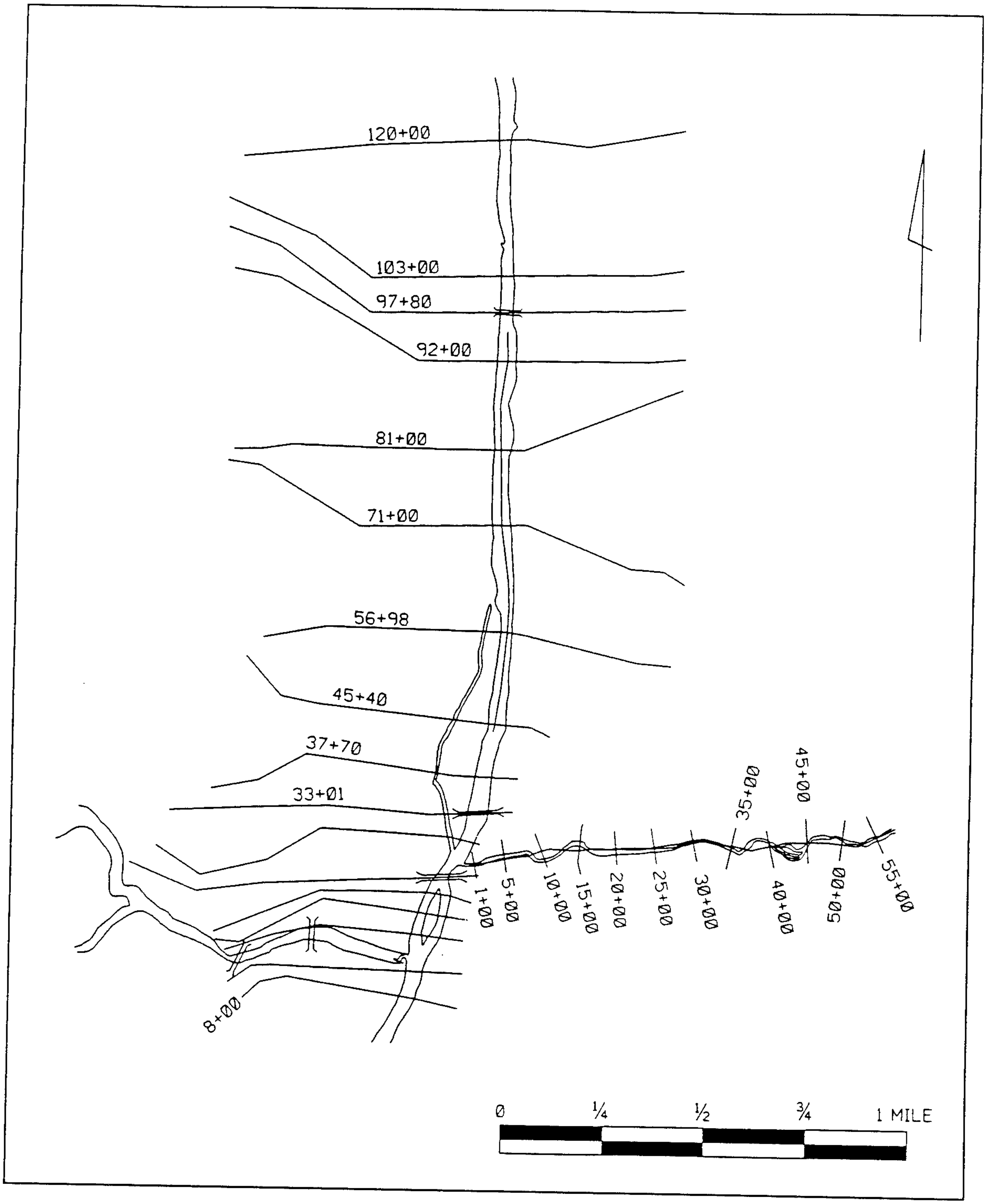

Figure 5. TABS-1 model cross-section locations

Hydrographs

Discharge hydrographs are simulated in the numerical model by a series of steady-state events. The duration of each event is chosen such that changes in bed elevation due to deposition or scour do not significantly change the hydraulic parameters during the event. At relatively high discharges, durations need to be short; time intervals as low as 15 minutes were used for the flood peaks on the Rio Grande. At low discharges, the time interval may be extended Time intervals up to 14 days were used during low flow periods for the historical simulations.

A hydrograph simulated by a series of steady-state events for varying durations is called a histograph. The historical histographs used in the numerical model were based on data from the USGS gages on the Rio Grande and Santa Cruz River. The USGS gage used for the Rio Grande was located at Otowi Bridge, which is about 9 miles below the downstream end (sta 8+00) of the model. The USGS gage used for the Santa Cruz was at Cundiyo, located about 12 miles above the upstream end of the Santa Cruz River portion of the model.

Downstream Water Surface Elevation

Downstream water surface elevations on the Rio Grande were established using a stage-discharge rating curve developed using the SAM program (Thomas et al., in preparation). The SAM program was used on the cross-section at sta 8+00 to develop a stage-discharge rating curve using normal depth calculations.

Bed Material

The bed material of the Rio Grande and Santa Cruz River were characterized from bed material samples collected and Analyzed by Flo Engineering (1993a and b). These samples were collected within the upper 6 to 12 inches of the bed surface. Water depths were typically less than 2 ft and the samples were scooped and bagged. Sample volume typically ranged between 3 and 6 liters for the gravelly samples to less than a liter for the sand samples. In the Rio Grande through the study area, the bed material consists primarily of gravels and coarse sands, as shown in Figure 6. In the lower Santa Cruz River, the bed material consists primarily of coarse sands and fine gravels (Figure 6). It should be noted that seasonal variability was observed in the samples collected due to such influences as the sediment contribution from the Rio Chama and other local sources.

Channel Roughness

Hydraulic roughness is influenced by grain size, bed form, water depth, bank roughness, changes in channel shape, and changes in flow direction or

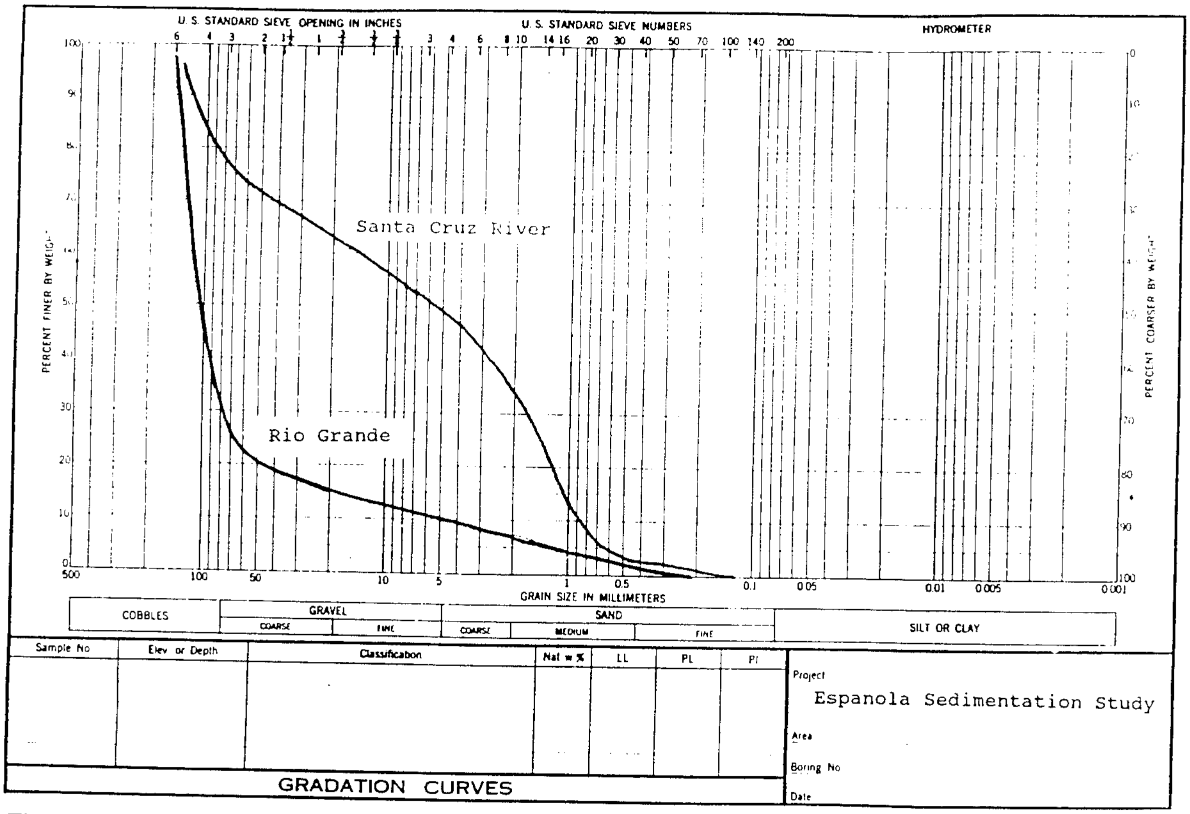

Figure 6. Bed gradation curves used in TABS-1 model

concentration of flow due to bends and confluences. In the one-dimensional numerical model, these effects are accounted for by the Manning's roughness coefficient. The roughness coefficient may vary significantly with discharge and time. The influence of grain roughness is known to decrease with increases in depth. Resistance due to bed forms can decline dramatically when dunes are washed out and replaced by a plane bed or antidunes. Greater momentum at high flows increases resistance due to channel bends and confluences. Local scour at high flow also tends to make channel cross sections more irregular, increasing roughness. High-water marks from events of known magnitude and hydraulic geometry are frequently used to estimate roughness coefficients.

The roughness coefficients used in the TABS-1 model were the same used in the HEC-2 model previously verified by the USAED, Albuquerque and provided to WES. Only some minor adjustments were made in Manning's n values during the course of TABS-1 model development.

Sediment Inflow

Sediment inflow at the upstream end of the model on the Rio Grande was estimated from the suspended sediment data, representing a mix of wash load and bed material sediment from the USGS gaging station at Otowi Bridge and modified as necessary during model adjustment. Sediment inflow at the upstream end of the Santa Cruz River reach was estimated from the observed deposition rates in the SCS detention dams on tributaries to the Santa Cruz River and modified as necessary during model adjustment. In this analysis the detention dams were considered as sediment traps, this providing a good estimate of sediment yield from the upstream watershed. The resulting sediment discharge rating curves are shown in Figures 7 and 8. The suspended sediment gradations used in model testing for the Rio Grande and Santa Cruz River are presented in Figure 9. In the absence of observed data the curves were developed using a "trial and error," technique in which sediment inflow is adjusted until "channel equilibrium" is achieved within the study reach.

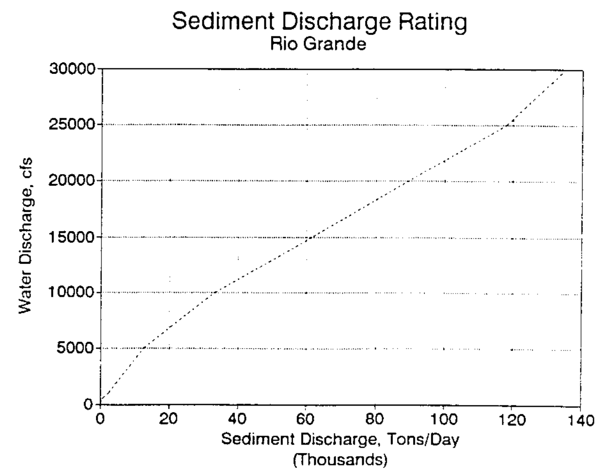

Figure 7. Sediment discharge rating curve for the Rio Grande

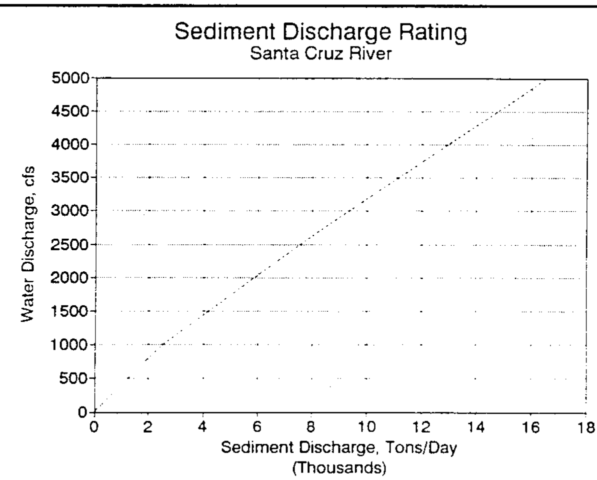

Figure 8. Sediment discharge rating curve - Santa Cruz River

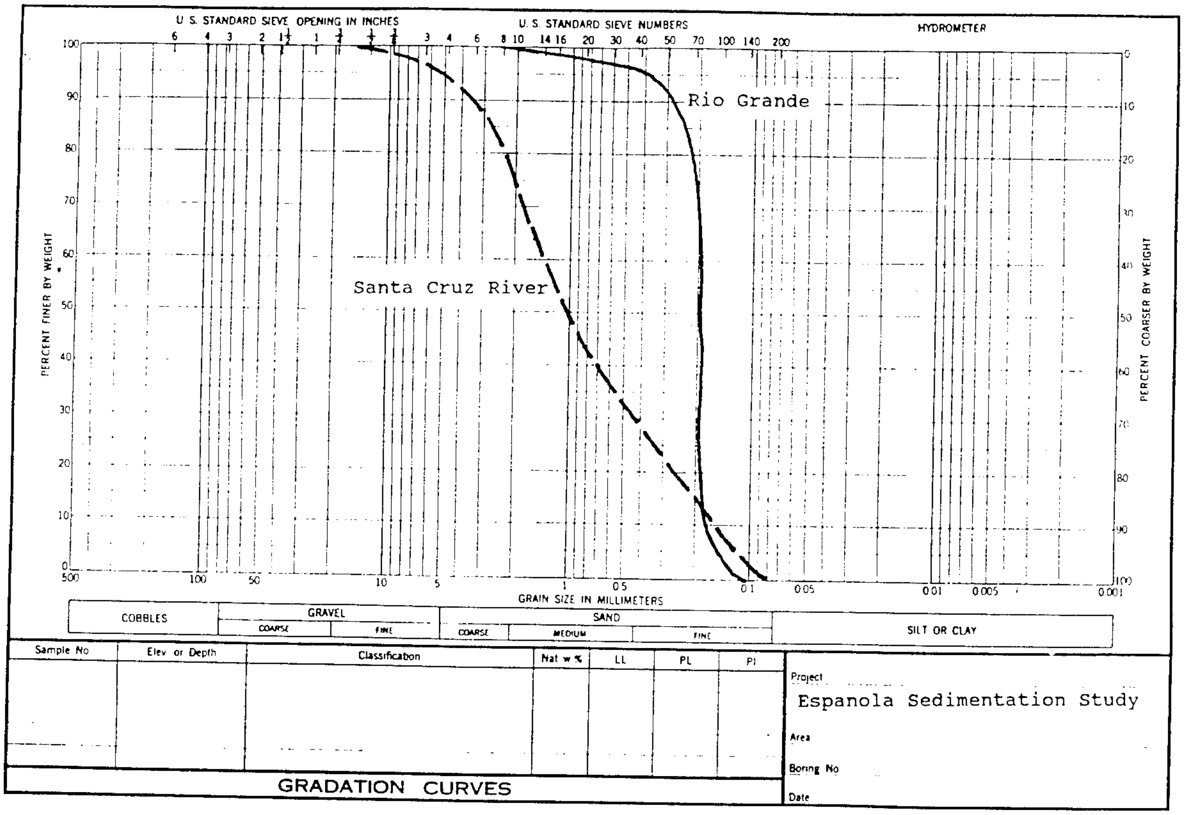

Figure 9. Suspended sediment gradation curves

Transport Function

Several transport functions were investigated for use in this study. After sensitivity testing of several functions, the Toffaleti-Schoklitsch transport function was selected as appropriate because of the model's behavior over a range of conditions.

The Toffaleti function calculates both bed load and suspended load. The calculated load from the Schoklitsch equation is assumed to represent only bed load. The combined function uses the Toffaleti function to calculate suspended load. Bed loads are calculated using both equations and the largest is combined with the suspended load to determine total bed material load.

3 Model Adjustment

Steady State Tests

The initial model adjustment was to compare existing-condition water surface elevations from the TABS-1 model to results from the HEC-2 model previously developed by USAED, Albuquerque. Steady-state discharges of 2,000 cfs, 5,000 cfs, 10,000 cfs, 15,000 cfs, 20,000 cfs and 25,000 cfs were compared to the HEC-2 model results. Water surface slope comparisons between the HEC-2 and the TABS-1 model showed good agreement in all cases.

Historical 30-year Hydrograph

Using the daily discharge data from the Otowi Bridge and Cunyido gages, histographs from 1962 to 1992 were developed. The Otowi Bridge and Cundiyo gage daily-discharge hydrographs for the period from 1960 to 1992 are shown in Figures 10 and 11.

Model performance for the existing condition was evaluated using several different transport functions. The objective was to use the transport function which exhibited reasonable bed behavior, i.e., no major aggradational or degradational shifts during the 30-year period. The Toffaleti-Schoklitsch function was selected as the function which behaved most realistically in the study area. Additionally, some minor adjustments were made to various model coefficients at selected cross sections to further improve reasonableness of bed response to the 30-yr histograph. Typical results of the existing-condition bed response to the 30-year histograph along the Rio Grande are shown in Figure 12. The four cross-sections presented in Figure 12 (stations 3301, 3770, 4540, and 5690) are within the reach adjacent to the proposed west-bank levee system. As can be seen the bed elevations at stations 3770, 4540, and 5690 are relatively stable during the 30-year simulation, with no long term aggradational or degradational trends. The bed elevation at cross section station 3301 degraded about 3 to 4 ft before reaching equilibrium at about 8000 days (21 years). From that point on, that station also appears relatively stable.

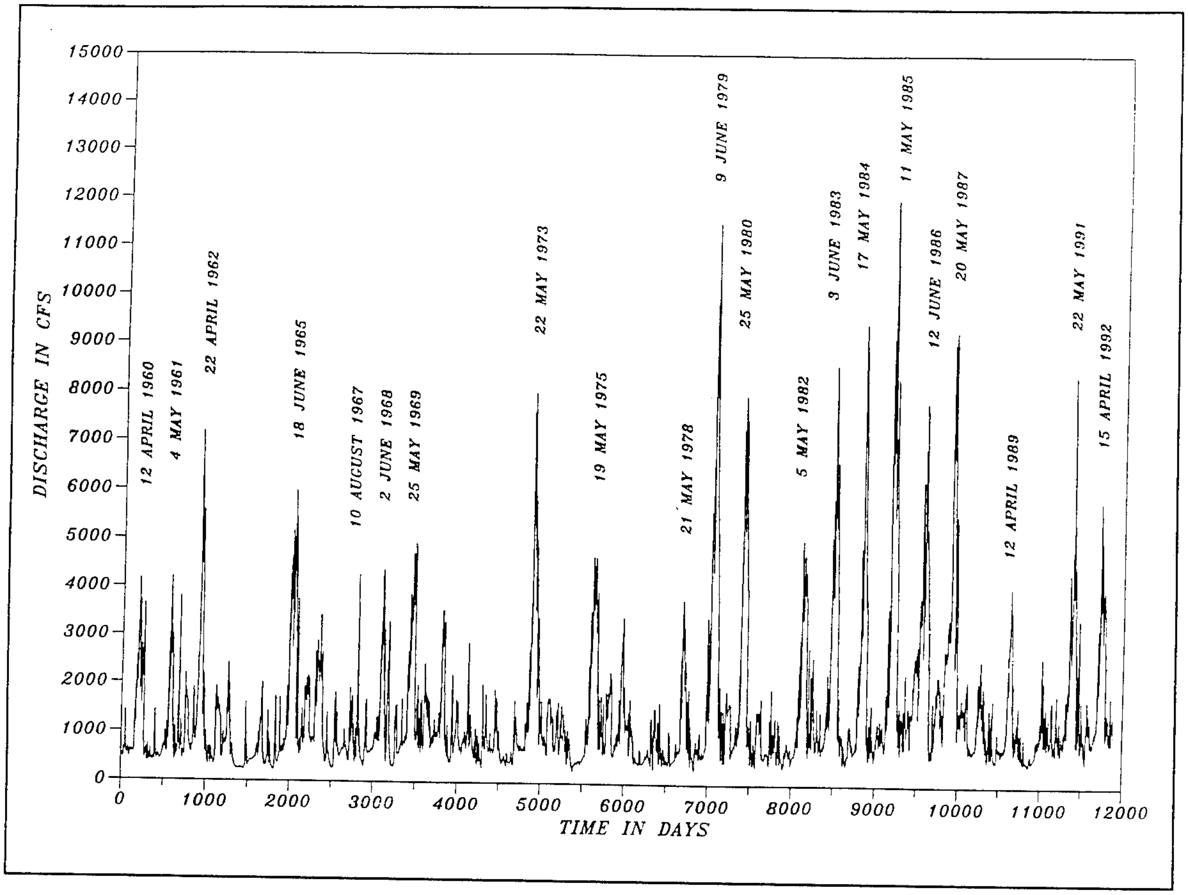

Figure 10. Discharge hydrograph from Otowi Bridge gaging station - 1960 to 1992

Bed Response to 1979 Hydrograph

A detailed evaluation of bed response from the 1979 hydrograph was conducted. The model test actually had a four-year duration, i.e., from 1976 through 1979, meaning that the historical hydrograph at Otawi Bridge was actually used from 1976 through 1979. As can be seen in Figure 13, the bed response (thalweg elevation) to the 1979 event at cross-sections 3301, 3770, 4540, and 5640 behaved in a reasonable fashion. Maximum degradation of about 0.6 ft occurred at cross-section 3301 with recovery of about 0.2 ft by the end of the year.

Bed Response to Balanced Hydrograph (100-Year Event)

A detailed evaluation of bed response from the balanced 100-year hydrograph (Figure 14), provided by the Albuquerque District, was conducted. As can be seen from Figure 15, the bed response to the balanced

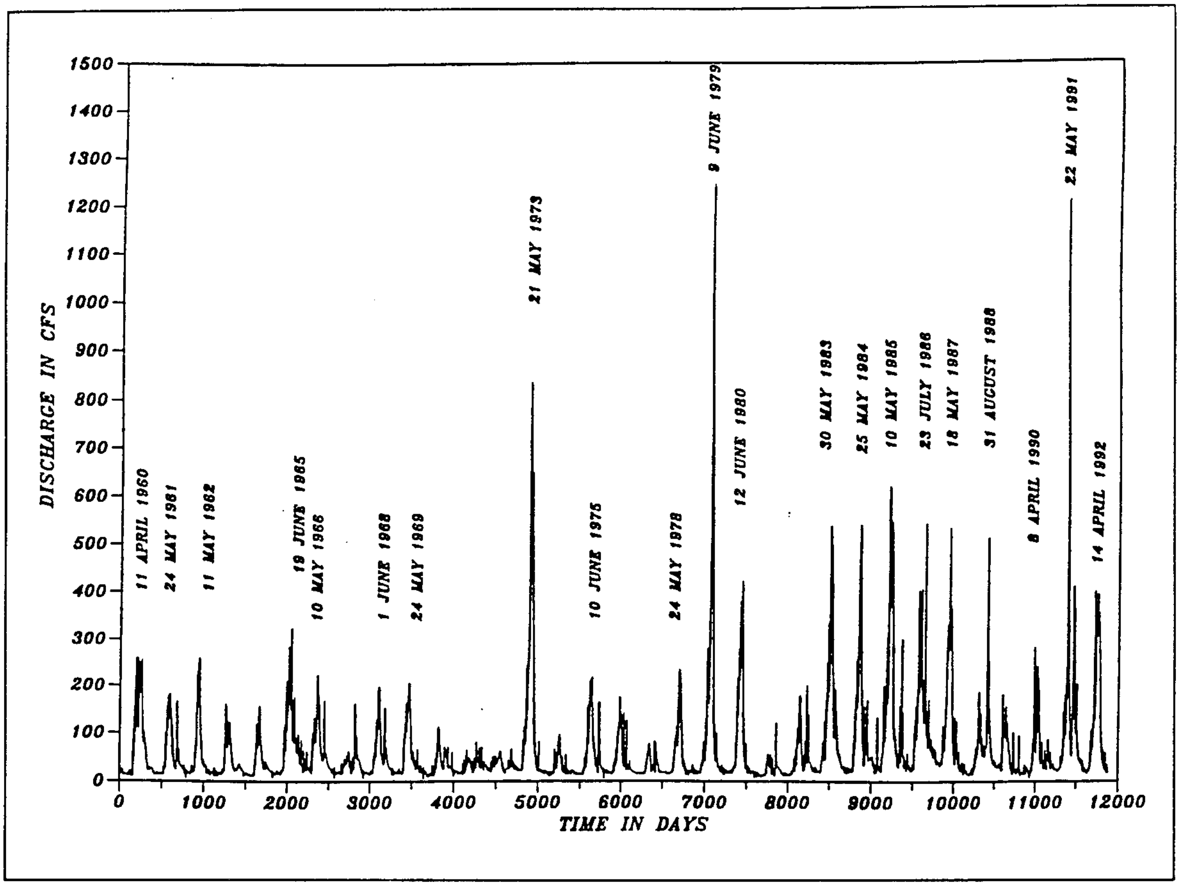

Figure 11. Discharge hydrograph from Cundiyo gaging station - 1960 to 1992

hydrograph at cross-sections 3301, 3770, 4540, and 5690 appeared reasonable. Maximum degradation of about 1.5 ft at cross-section 3301 was observed during the model test.

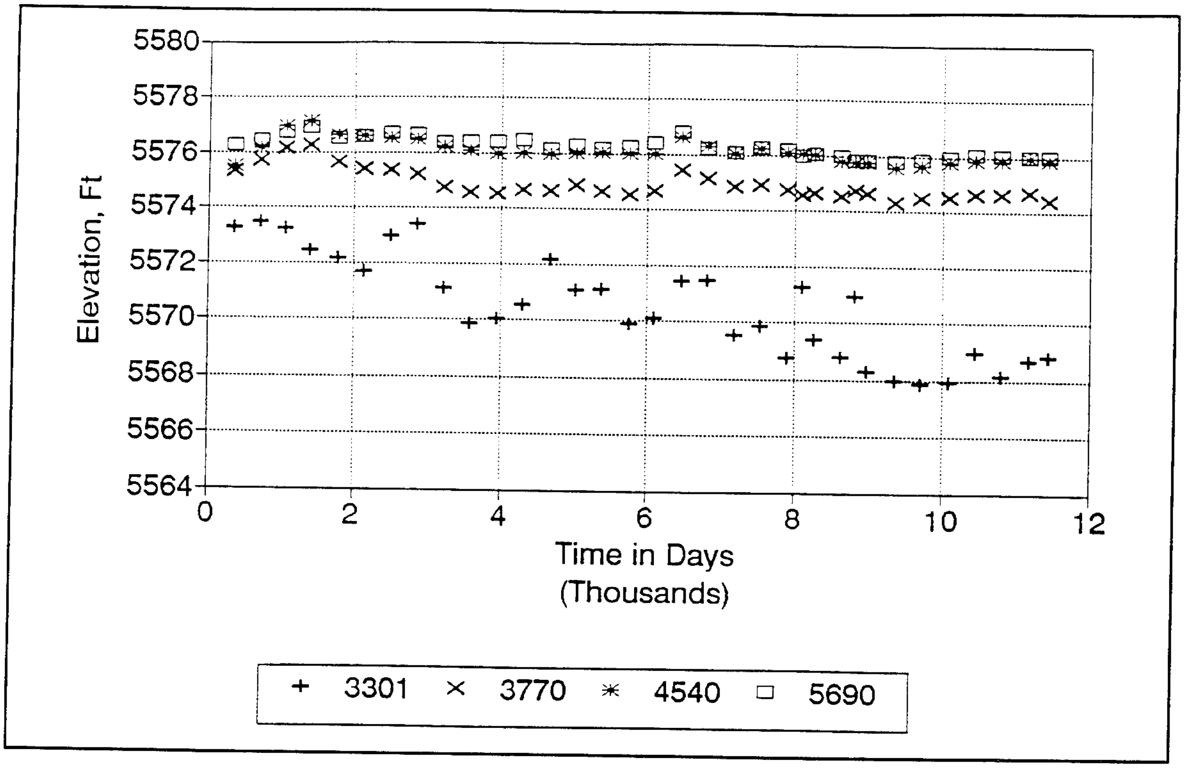

Figure 12. Bed response (thalweg) to 30-year hydrograph for existing condition at cross sections 3301, 3770, 4540, and 5690

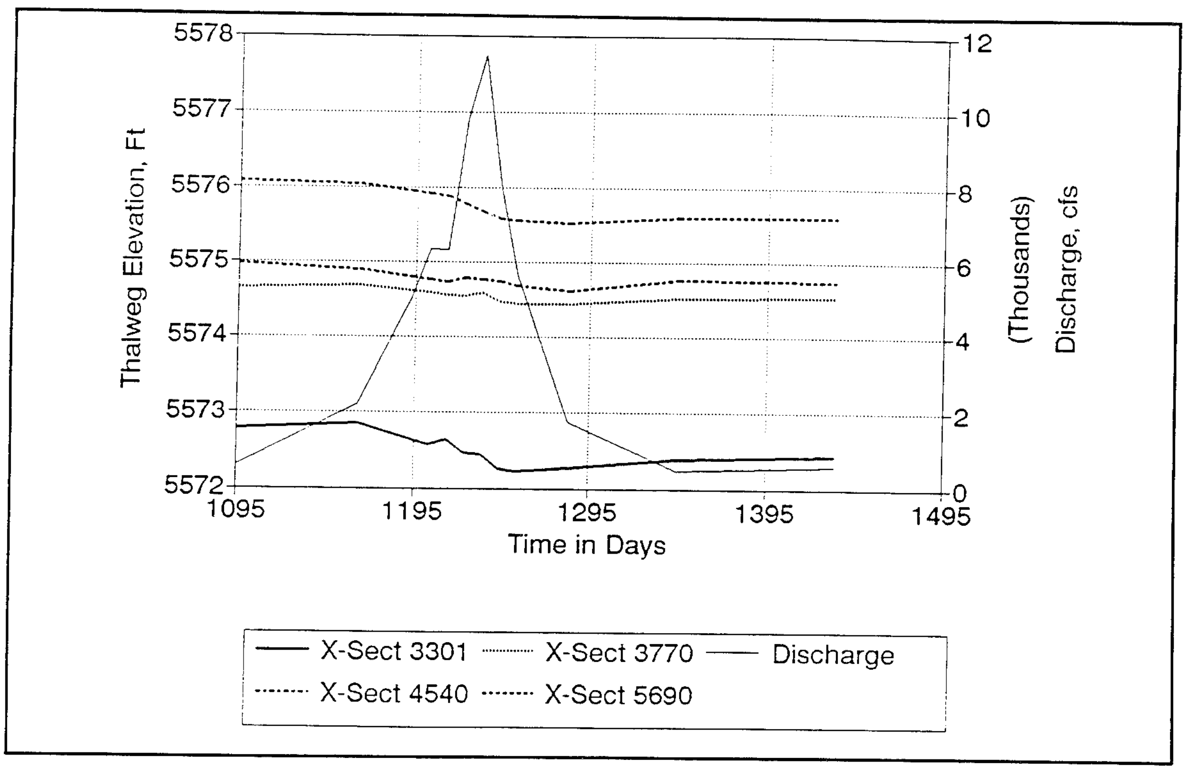

Figure 13. Bed response (thalweg) to 1979 hydrograph for existing condition at cross sections 3301, 3770, 4540, and 5690

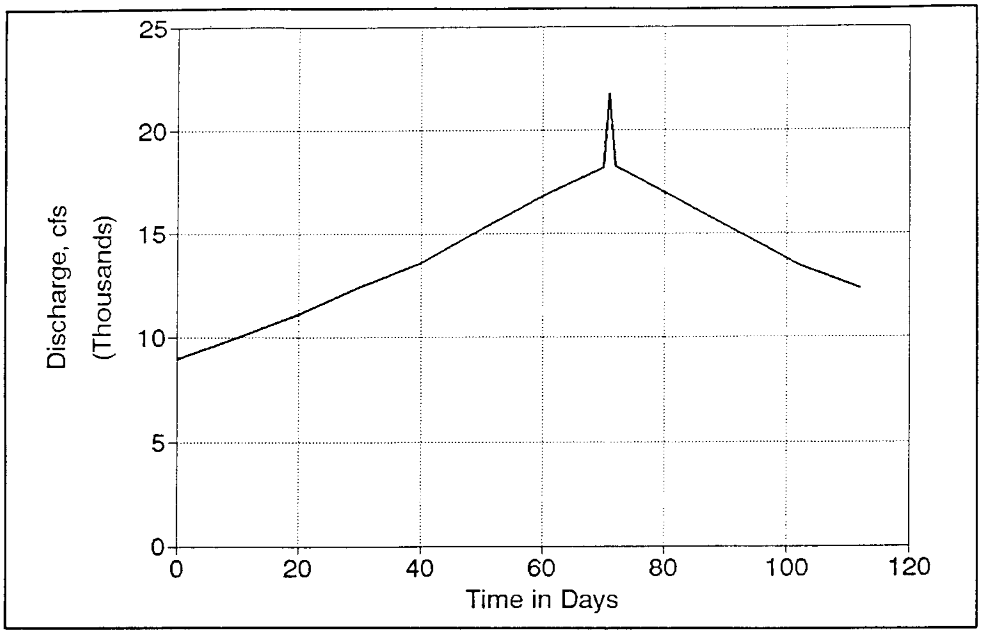

Figure 14. Balanced (1 percent chance flood) hydrograph developed from Otowi Bridge 100-year flood gaging station data

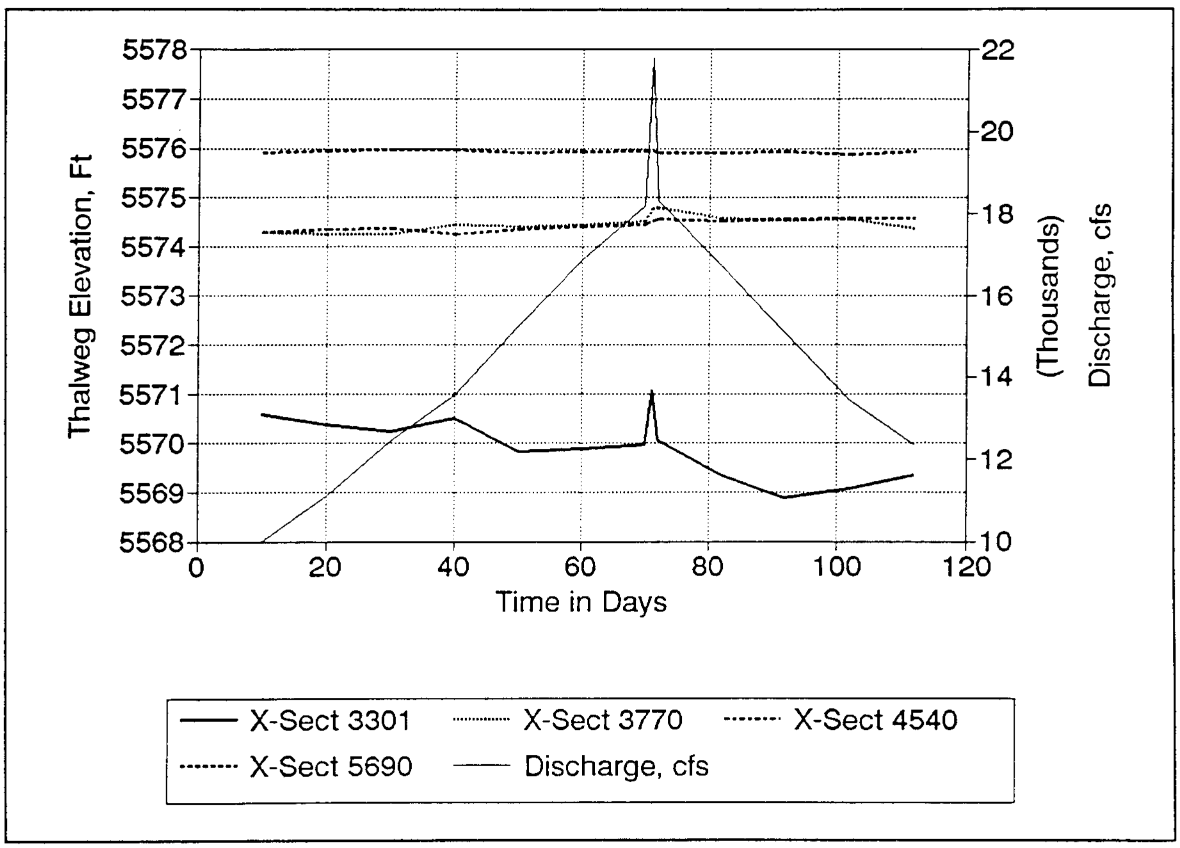

Figure 15. Bed response (thalweg) to balanced hydrograph for existing condition at cross sections 3301, 3770, 4540, and 5690

4 Model Results

Levee Plan testing included the application of three different hydrographs to the study area; (1), the 30-year historical hydrograph (1962-1992); (2), the 1979 hydrograph; and (3), the balanced (100-year event) hydrograph. Additionally, sediment behavior at the confluence of the Rio Grande and Santa Cruz River was evaluated using the 1979 hydrograph.

Existing versus Plan Testing using the 30-Year Historical Hydrograph

The existing versus plan comparisons of the channel bed elevations for cross-sections 3301, 3770, 4540, and 5690 after 30 years are shown in Figures 16 to 19. At cross section 3301, the levee plan resulted in a bed lowering of about 0.7 ft compared to the existing condition (Figure 16). At cross-section 3770, the bed was only about 0.3 ft lower than the existing condition (Figure 17). At cross-sections 4540 and 5690, the bed was unchanged from the existing condition (Figures 18 and 19).

Existing versus Plan Testing using the Balanced Hydrograph

The existing versus plan comparisons showed no significant differences in bed response to the balanced hydrograph. The bed profiles through the study area at day 71 of the test are shown in Figure 20. Day 71 is when the peak discharge of 21,800 cfs occurred (Figure 14). Additionally, no significant differences were noted on the downside of the hydrograph.

Existing versus Plan High-Discharge Steady-State Tests

Steady-state tests for flows of 20,000 cfs, 25,000 cfs, and 30,000 cfs, each for 10 days in duration, were conducted for both existing and plan conditions.

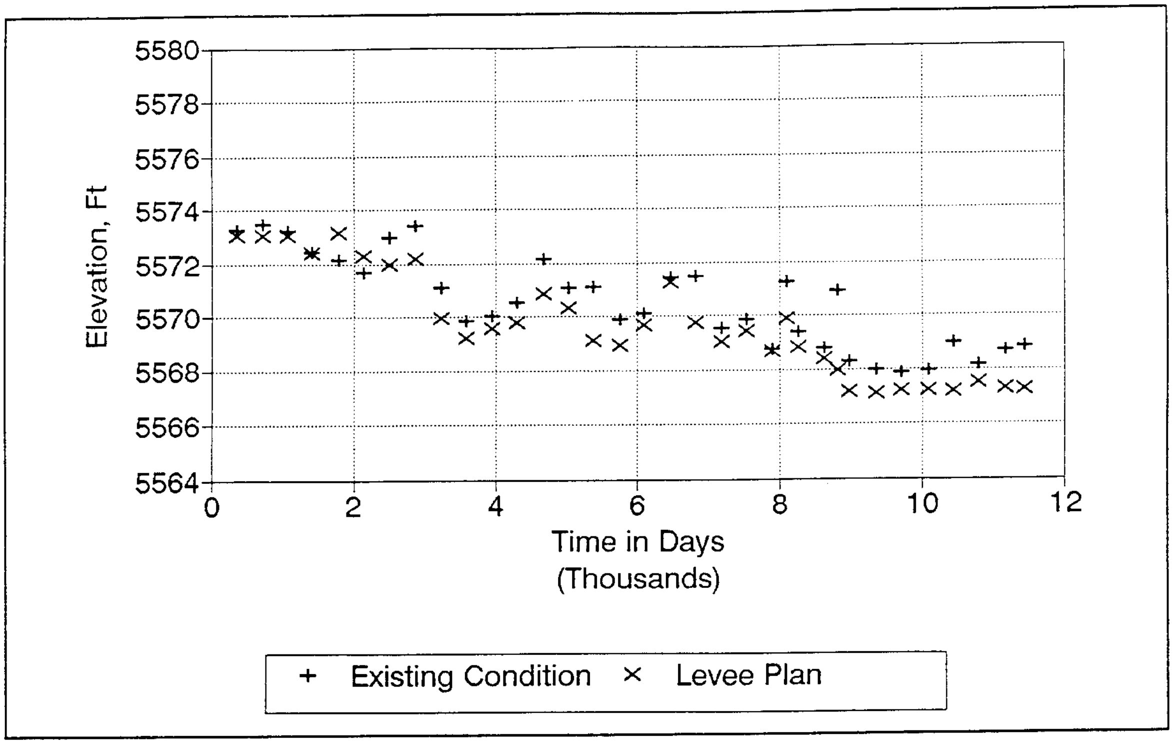

Figure 16. Existing-versus-plan bed response to 30-year hydrograph at cross section 3301

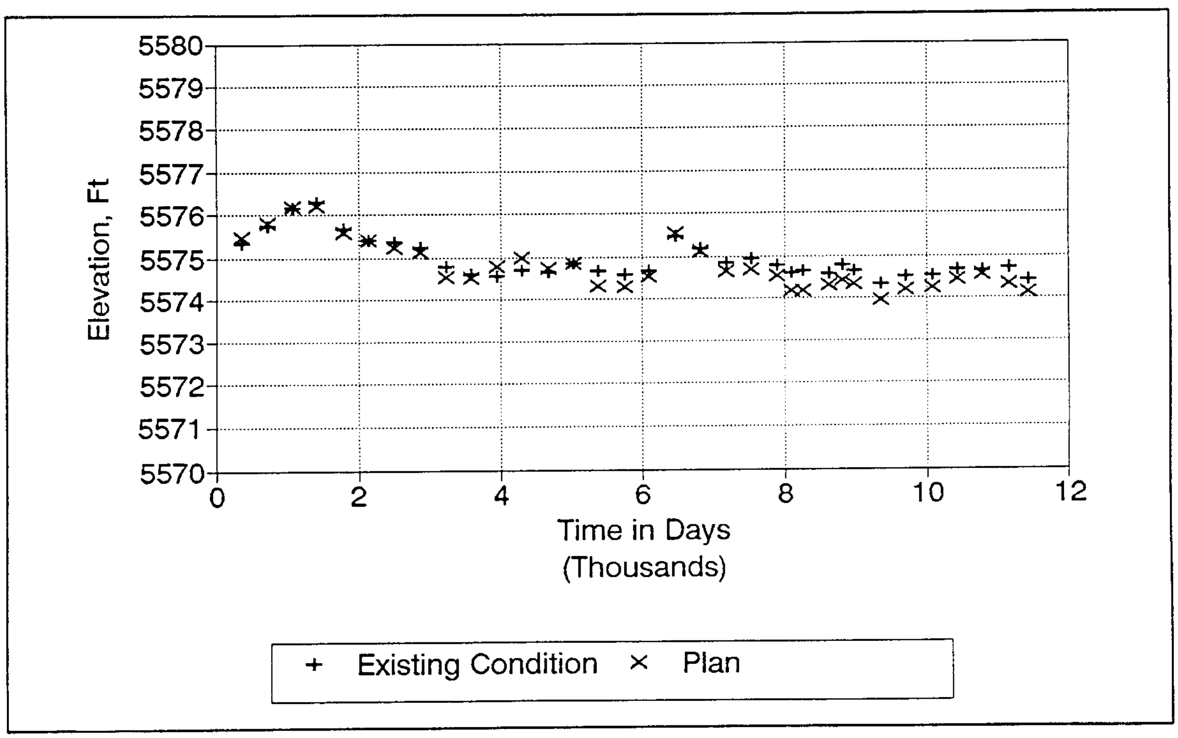

Figure 17. Existing-versus-plan bed response to 30-year hydrograph at cross section 3770

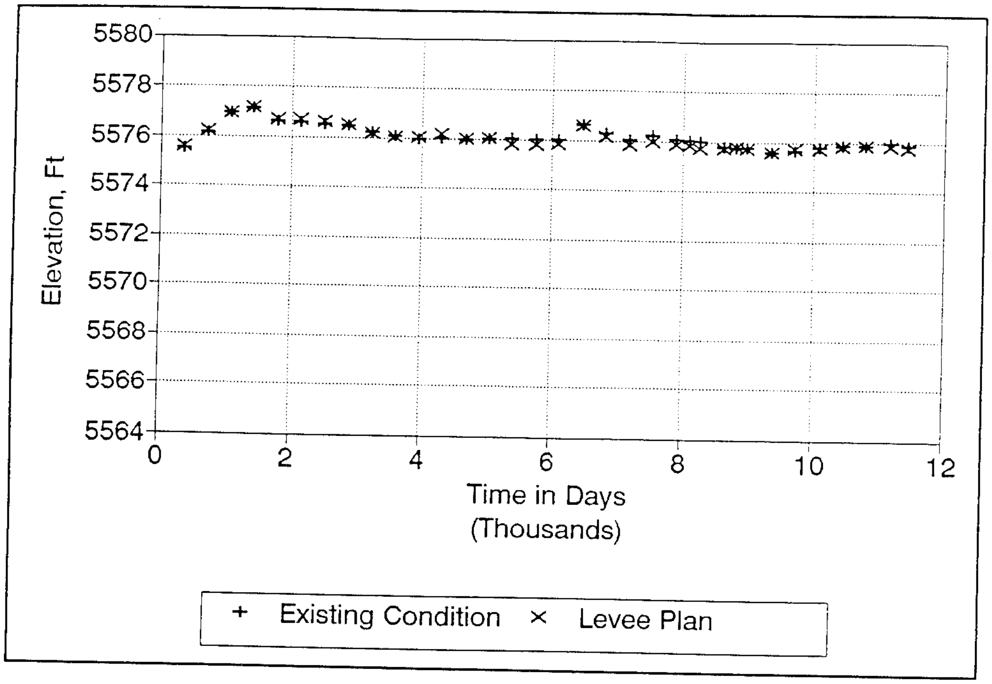

Figure 18. Existing-versus-plan bed response to 30-year hydrograph at cross section 4540

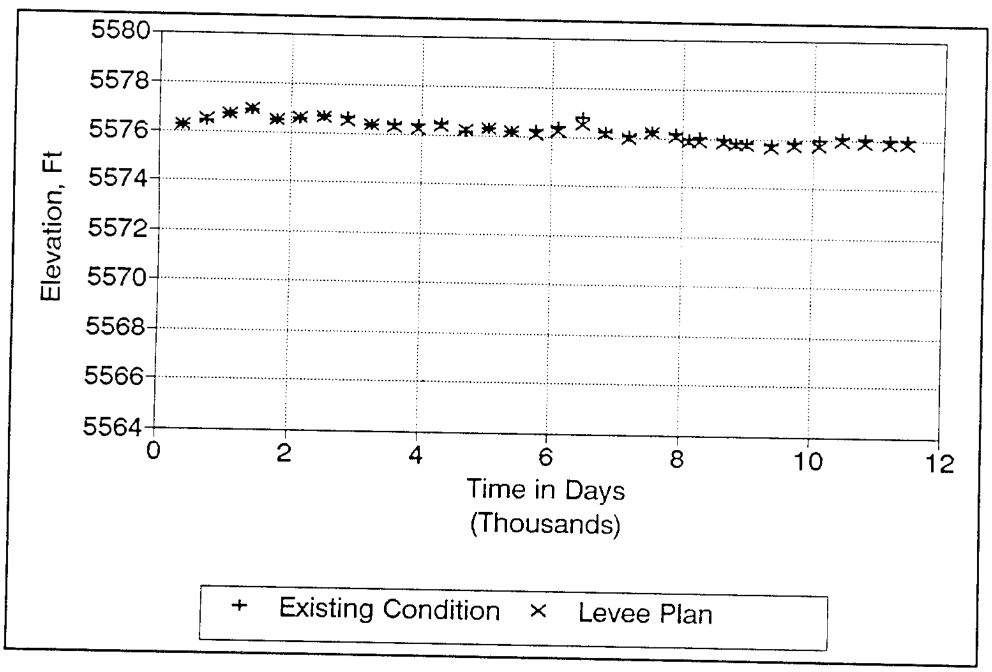

Figure 19. Existing-versus-plan bed response to 30-year hydrograph at cross section 5690

Results of these tests showed no significant differences in bed response between existing and plan conditions after 10 days, with only one exception.

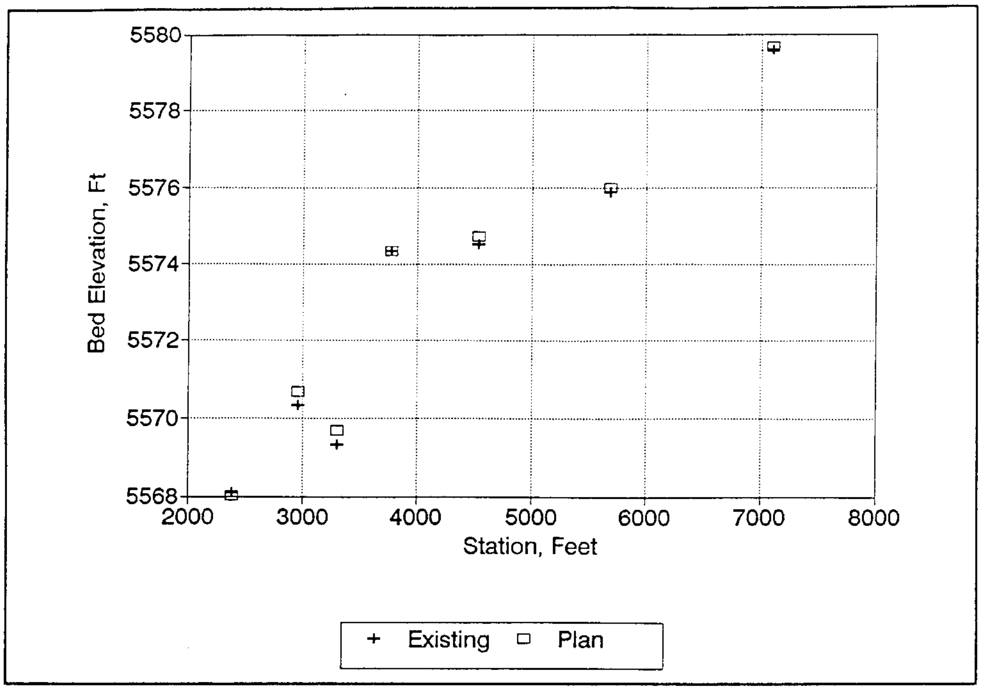

Figure 20. Existing-versus-plan bed profiles from the balanced hydrograph - day 71

The bed profiles for the three discharges tested are shown in Figures 21, 22, and 23. The only significant difference in bed elevation occurred in the 30,000 cfs test (Figure 23), in which the existing-condition overbank flooding resulted in significant channel deposition at station 3770. Because the levee confined the flow to the channel area, such deposition did not occur in the plan test. In none of the three tests was there any evidence to suggest that the plan caused any additional channel degradation when compared to the existing condition.

Existing versus Plan Testing Using the 1979 Hydrograph

The existing versus plan comparisons showed no significant differences in bed profiles within the study area resulting from the application of the 1979 hydrograph. The bed profiles through the study area at day 1233 of the test are shown in Figure 24. Day 1233 represents the period of peak discharge (11,500 cfs). Furthermore, no significant differences were noted on the downside of the hydrograph.

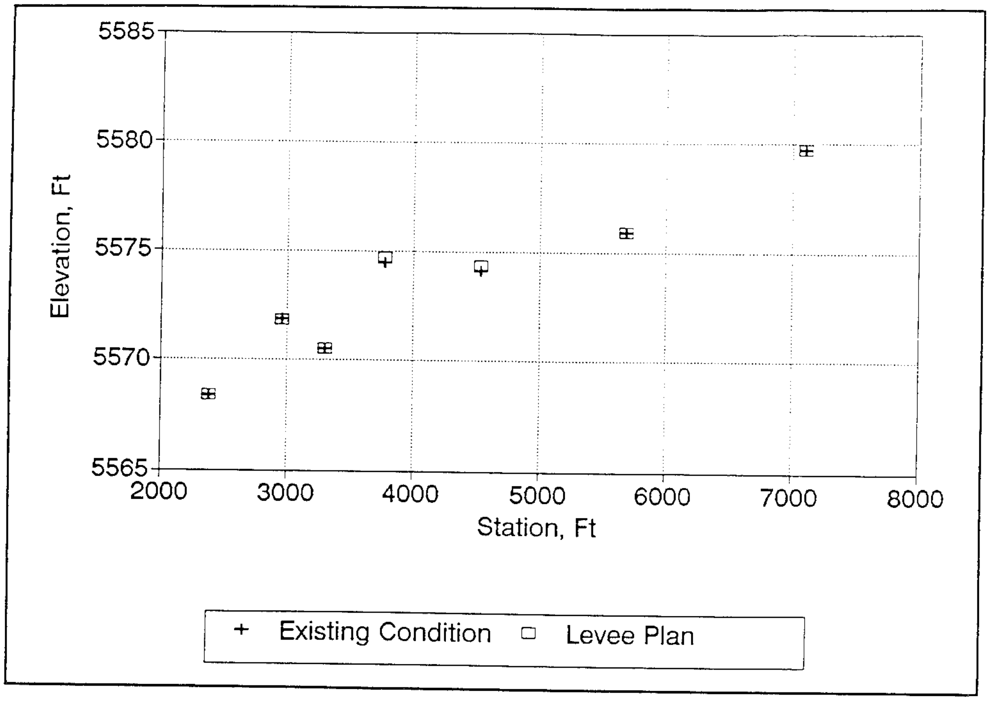

Figure 21. Existing-versus-plan bed profiles for steady-state discharge of 20,000 cfs - day 10

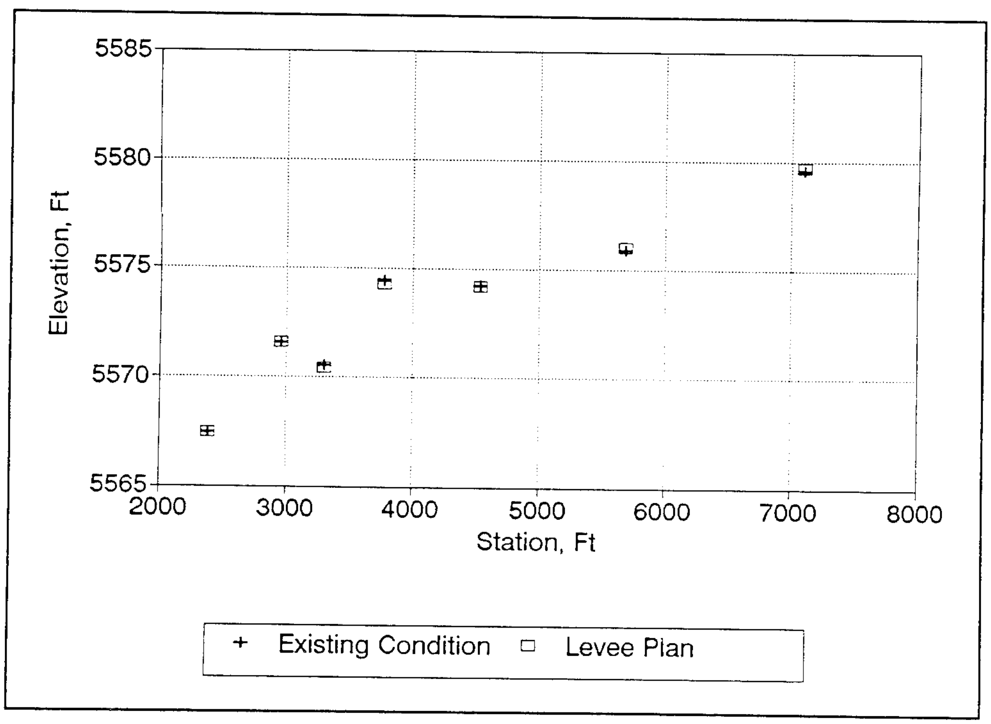

Figure 22. Existing-versus-plan bed profiles for steady-state discharge of 25,000 cfs - day 10

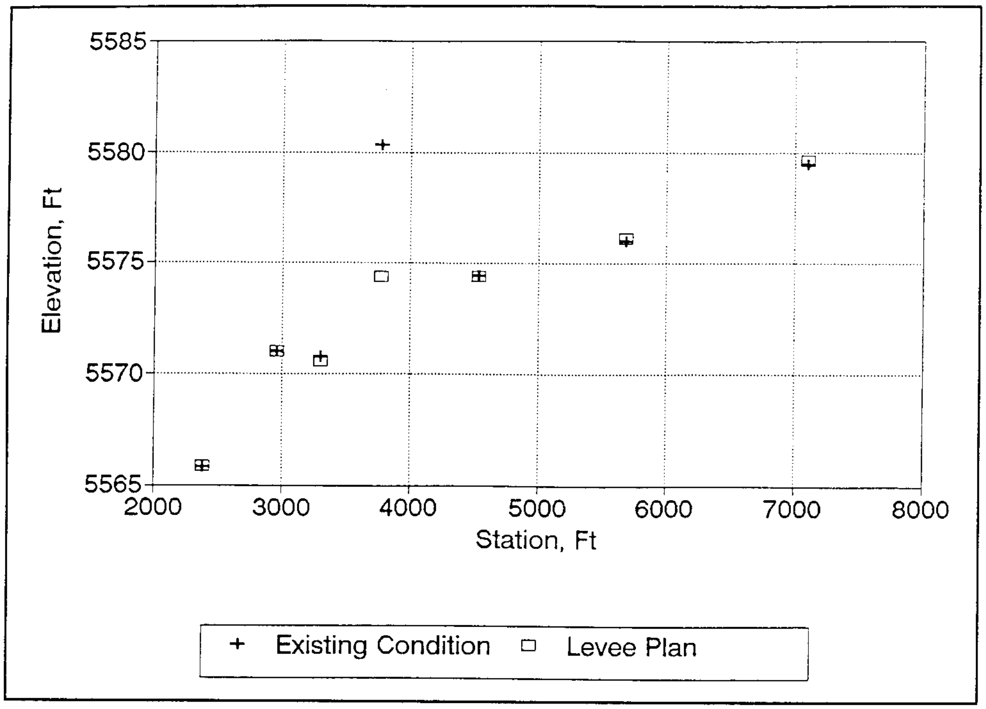

Figure 23. Existing-versus-plan bed profiles for steady-state discharge of 30,000 cfs - day 10

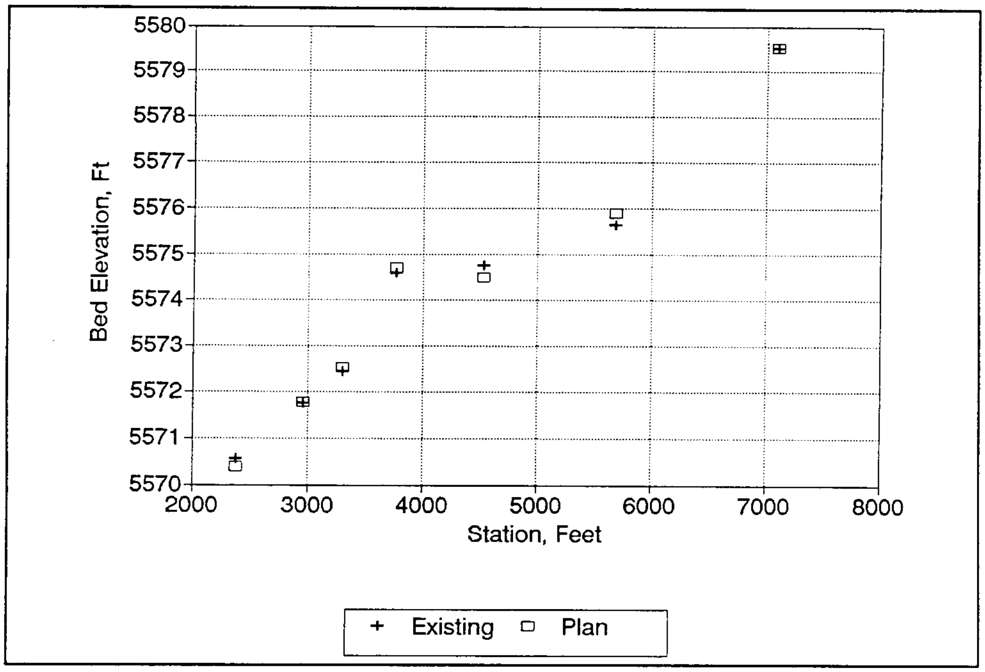

Figure 24. Existing-versus-plan bed profiles from the 1979 hydrograph - day 1233 (peak discharge 11,500 cfs)

Sediment Behavior at the Confluence of the Rio Grande and Santa Cruz River

The year 1979 was used to evaluate the impact of sediment delivery to the Rio Grande from the Santa Cruz River. The year 1979 included a flood event on the Santa Cruz River with a daily discharge that peaked around 1300 cfs, which represented a 20% change flood (U.S. Army Engineer District, Albuquerque, 1992).

The 1979 hydrographs used in the TABS-1 model for the Rio Grande and the Santa Cruz River are shown in Figure 25. The Santa Cruz River hydrograph was estimated using a multiplying factor of 2.15 times the observed hydrograph at the Cundiyo gaging station, since the drainage area at the Cundiyo gaging station is 85 square miles and at the mouth 185 square miles.

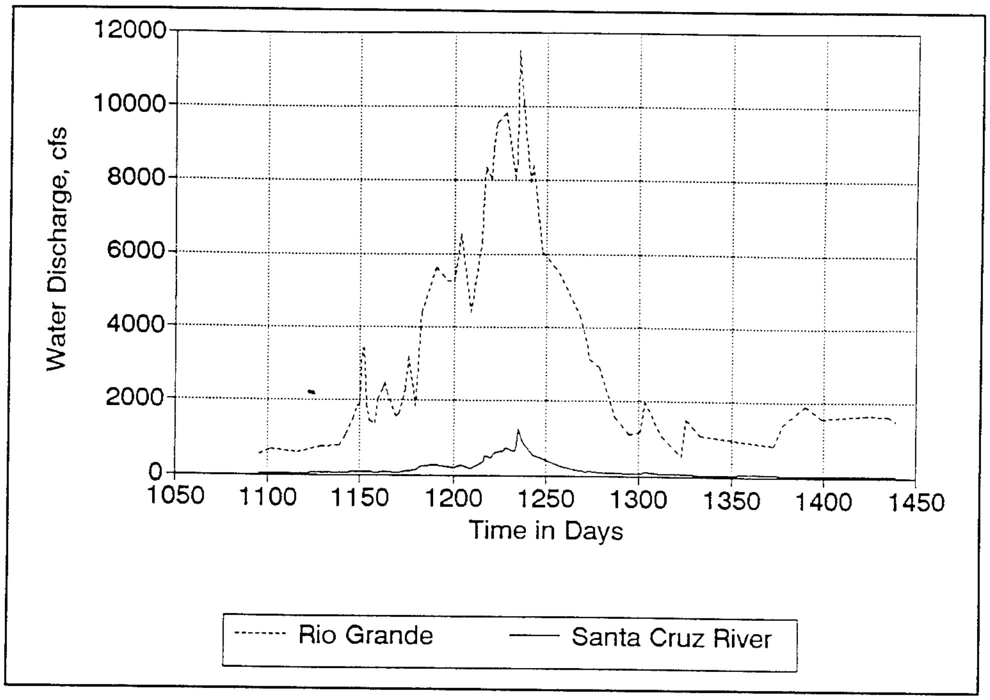

Figure 25. 1979 hydrographs used in TABS-1 for Rio Grande and Santa Cruz River

The Santa Cruz River peak discharge occurred on day 1233 of the TABS-1 model test (9 June 1979), as shown in Figure 25. During the period from day 1203 to day 1243, the model estimated that approximately 38 ac-ft of sand and gravel were delivered to the Rio Grande from the Santa Cruz River. During that same 40-day period, the Rio Grande discharge ranged from 6,000 to 11,000 cfs (Figure 25). Based on the model results, the Rio Grande had no difficulty in transporting the sediment delivered from the Santa Cruz River. Bed responses at station 2960, just upstream of the confluence, and stations 2381 and 1960, downstream of the confluence, are shown in Figure 26.

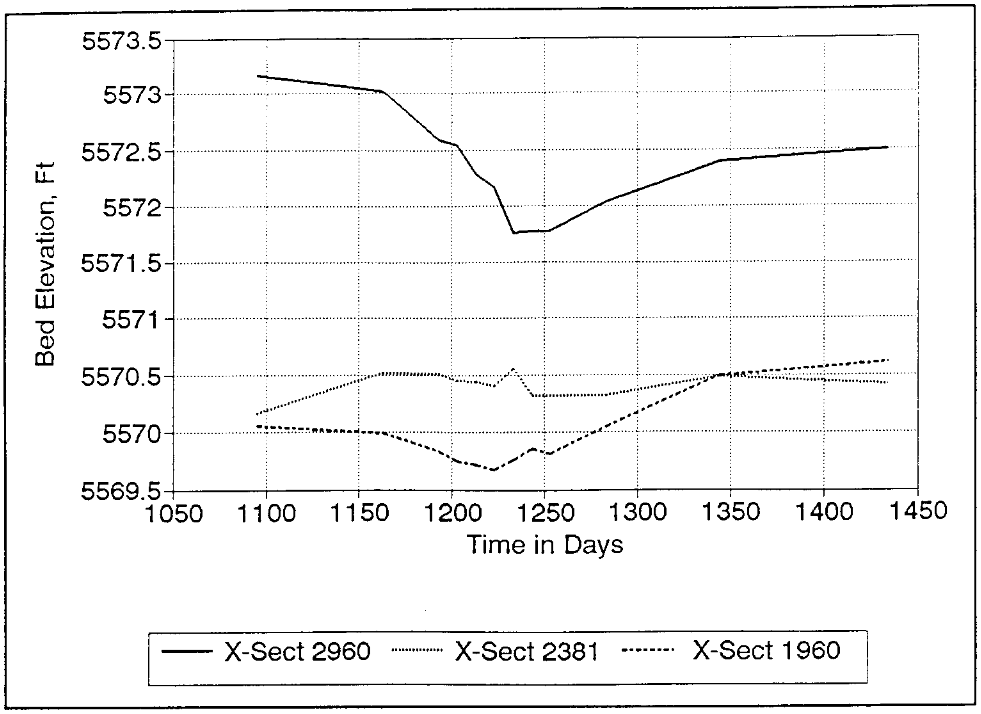

Figure 26. Bed response to 1979 hydrographs at cross sections 2960, 2381, and 1960 above and below Santa Cruz River

5 Conclusions

Based on the results from the model tests described in this report, the following conclusions are made regarding the proposed levee plan shown in Figure 3:

a. The proposed levee should not cause any significant change in bed behavior from the existing condition.

b. No degradational or aggradational trends resulting from the proposed levee plan were observed within the study area.

c. The reason that there is no significant sediment impact is that the proposed levee plan is very limited in scope. If a levee were added to the east bank and levee setbacks were reduced, at some point the channel would begin to respond.

d. Based on the model results from the 1979 hydrograph, there does not appear to be a sediment problem from the Santa Cruz River. The Rio Grande is capable of effectively removing sediments delivered from the Santa Cruz River.

In summary, the proposed levee plan exhibited no adverse impact to the Rio Grande from a sedimentation standpoint.

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Appendix A Description of TABS-1 Computer Program

The computer program TABS-1 calculates water-surface profiles and changes in the streambed profile. Water velocity, water depth, energy slope, sediment load, gradation of the sediment load, and gradation of the bed surface are also computed. Water-surface profile and sediment movement calculations are fully coupled using an explicit computation scheme. First, the conservation of energy equation is solved to determine the water-surface profile and pertinent hydraulic parameters (velocity, depth, width, and slope) at each cross section along the study reach:

$$\frac{\partial H}{\partial X} + \frac{\partial \left(\alpha \, \frac{V^2}{2g} \right)}{\partial X} = S \tag{A1}$$

where

H = water-surface elevation
X = direction of flow
α = coefficient for the horizontal distribution of velocity
V = average flow velocity
g = acceleration due to gravity
S = slope of energy line

In addition, the continuity of sediment material is expressed by

$$\frac{\partial G}{\partial X} + B \cdot \frac{\partial y_s}{\partial t} = q_s \tag{A2}$$

where

G = rate of sediment movement, cu ft/day
X = distance in direction of flow, ft
B = width of movable bed, ft
y_s = change in bed surface elevation, ft
t = time, days
q_s = lateral inflow of sediment, cu ft/ft/day

The third equation relates the rate of sediment movement to hydraulic parameters as follows:

$$G = f(V,y,B,S,T,d_{eff},d_{si},P_i) \tag{A3}$$

where

y = effective depth of flow
T = water temperature
d_{eff} = effective grain size of sediment mixture
d_{si} = geometric mean of class interval
P_i = percentage of i^{th} size class in the bed

The numerical technique used to solve Equation A1 is commonly called the Standard Step Method. Equation A2 has both time and space domains. An explicit form of a six-point finite difference scheme is utilized. Several equations of the form of Equation A3 are available. These transport capacity equations are empirical and G is determined analytically.

Equation A2 is the only explicit equation, but it controls the entire analysis by imposing stability constraints. Several different computation schemes were tested, and the six-point scheme proved the most stable. No stability criteria have been developed for this scheme. The rule of thumb is to observe the amount of bed change during a single computation interval and reduce the computation time until that bed change is tolerable.

Oscillation in the bed elevation is a key factor in selecting a suitable computation interval. The computation time interval must be made short enough to eliminate oscillation. On the other hand, computer time increases as the computation interval decreases. The proper value to use is determined by successive approximations, running test cases, and observing the amount of bed change.

Several supporting equations are required in transforming the field data for the computer analysis. The Manning equation is used to evaluate friction loss. Average geometric properties are combined, using an average end area approach, into an average conveyance for the reach. Manning's roughness coefficients are entered for the channel and both overbanks and may be changed with distance along the channel, discharge, or stage. Construction and expansion losses are calculated as "other" losses by multiplying a coefficient times the change in velocity head. All geometric properties are calculated from cross-section coordinates.

Only subcritical flow may be analyzed in the computer program; however, zones of critical or supercritical flow may occur within the study reach. The program treats supercritical zones as "critical" for determination of water-surface elevation, but calculates hydraulic parameters for sediment transport based on normal depth. Critical depth in a section with both channel and overbank is defined as the minimum specific energy for that section assuming a level water surface. Starting water-surface elevations can be input as a rating curve with stage and discharge, or stage can be set for each specific time interval. Steady-state conditions are assumed for each time interval, although the discharge may be changed to account for tributary inflow. A hydrograph is simulated by creating a histograph of steady-state discharges, using small time intervals when discharge variations are great and longer time intervals when changes in water and sediment discharges are small.

In some cases the temperature of water can be an important parameter in sediment transport and, consequently, may be prescribed with each water discharge in the hydrograph. Flexibility of input permits a value to be entered as needed to change from a previous entry.

Geometry is input into the numerical model as a series of cross sections similar to the widely used HEC-2 backwater program (US Army Engineer Hydrologic Engineering Center 1990[1]). A portion of the cross section is designated as movable and a dredging template may also be specified. Spacing of cross sections is somewhat more critical for TABS-1 than it is for HEC-2 because of numerical stability problems. Long reach lengths are desirable because reach length and computation interval are related. Very short time intervals may be required if excessive bed changes occur within a specific reach. No special provisions are available to calculate head losses at bridges. The contracted opening may be modeled such that scour and deposition are simulated during the passing of a flood event, but calculated results must be interpreted with the aid of a great deal of engineering judgment and sensitivity analysis.

Four different sediment properties are required: (a) the total concentration of suspended and bed loads, (b) grain-size distribution for the total concentration, (c) grain-size distribution for sediment in the streambed, and (d) unit weight of deposits. A wide range of sediment material may be accommodated in the transport calculations (0.004 mm to 64 mm).

The usefulness of a calculation technique depends a great deal upon the coefficients which must be supplied. As in HEC-2, Manning's n values, contraction coefficients, and expansion coefficients must be provided to

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1 References cited in this appendix are included in the References at the end of the main text.

accomplish the water-surface profile calculations. Several other coefficients are required for sediment calculations as follows:

a. The specific gravity and shape of sediment particles must be specified.

b. The bed shear stress at which silt or clay particles begin to move and deposit are required coefficients.

c. The unit weight of silt, clay, and sand deposits is somewhat like a coefficient because of the difficulty in measuring. Also, the density changes with time.

All of the sediment-related coefficients have default values because sediment data seem to be much more scarce than hydraulic data. There are fewer sources for generalized coefficients. All of the default values should be replaced by field data where possible, and the input data are structured for such a process.

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13. ABSTRACT *(Maximum 200 words)*

A numerical model study of the Rio Grande through Espanola, NM, was conducted to evaluate flood-event channel response to the proposed levee system on the west bank of the Rio Grande. A second objective was to evaluate the flood-event behavior of the Rio Grande streambed at the mouth of the Santa Cruz River. To perform the analysis, the one-dimensional sedimentation model (TABS-1) was applied. The model was used to reproduce historical flow and sedimentation patterns through Espanola and to determine the impact of the proposed levee system on the bed of the Rio Grande. The model indicated no degradational or aggradational trends resulting from the proposed levee plan within the study area. Based on the 1979 hydrograph results, there did not appear to be a sediment problem from the Santa Cruz River. The Rio Grande was capable of effectively removing sediments delivered from the Santa Cruz River. In summary, the proposed levee plan exhibited no adverse impact to the Rio Grande from a sedimentation standpoint.

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