CALABACILLAS ARROYO
GRADE CONTROL STRUCTURES
DESIGN ANALYSIS REPORT

prepared for:

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July 1989

CALABACILLAS ARROYO GRADE CONTROL STRUCTURES

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SECTION 1

INTRODUCTION

This report discusses the investigations and results of the preliminary design of grade control structures for the Calabacillas Arroyo in the northwest part of Albuquerque, New Mexico. Initially these structures will be buried, with their tops at existing grade. On the basis of this and previous studies, it is expected that the arroyo will erode in the coming years and that the structures will become progressively more exposed, to a maximum design depth of about ten feet each. Ultimately, the proposed structures may not be enough to control channel degradation; in that case, additional structures can be added as needed in the future. Additionally, bank stabilization is proposed for several areas.

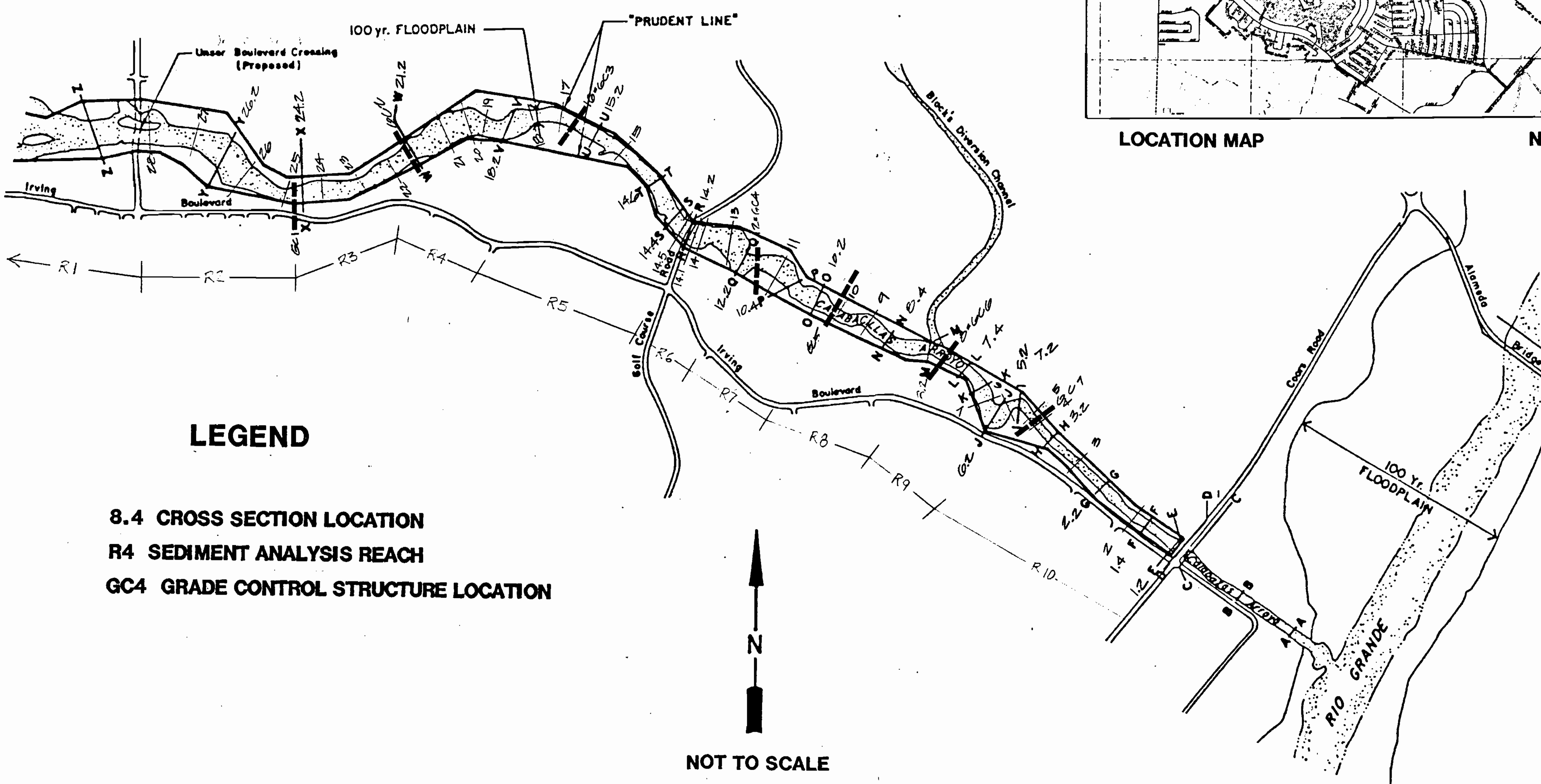
The Calabacillas is a major arroyo, with peak 100-year flows on the order of 13,000 cfs (cubic feet per second) entering the Rio Grande. The arroyo and its tributaries have been studied by various agencies, especially in the last ten years. Some of those studies are described in this report. Erosion is a serious problem in the Calabacillas Arroyo channel and sediment has come dangerously close to plugging the Rio Grande several times, including the most recent incident in 1988.

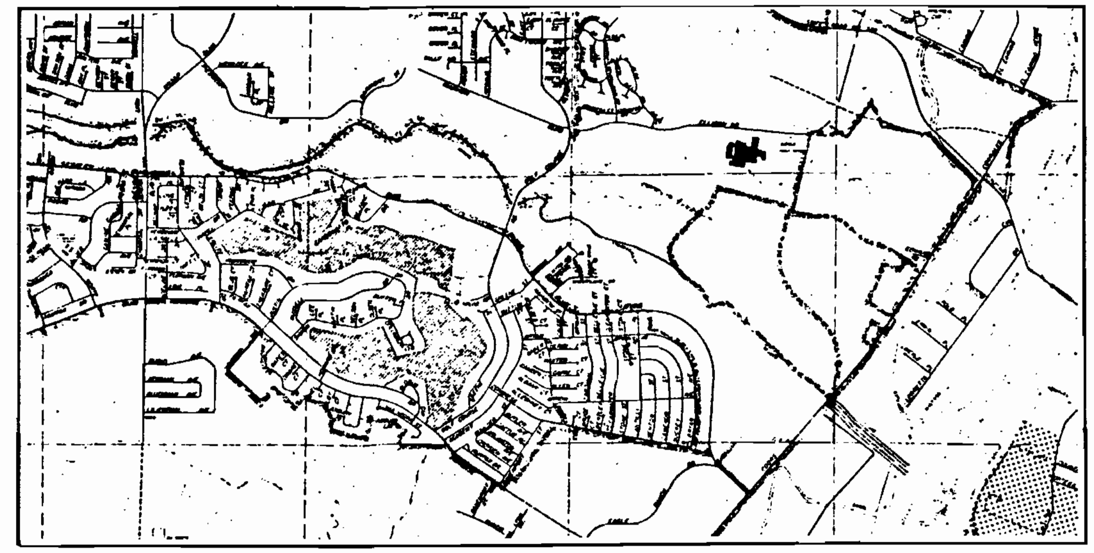
The reach of the arroyo considered in the present study lies just north of Paradise Hills. It begins at Coors Road on the east, about 0.4 miles from the Rio Grande, and continues upstream past Black's Diversion Channel and Golf Course Road to the proposed dam and bridge at Unser Blvd. (currently Lyon Blvd.), a distance of about 3.1 miles. Figure 1 shows the location of the project.

The purpose of the proposed grade control structures is to protect the channel against expected degradation. In this report hydrology, hydraulics, and sediment studies are discussed both for existing conditions and for development expected by the year 2036 as identified in the Calabacillas Arroyo Drainage Management Plan (CADMP, 1987). The structures were sized for the 100-year flows, but the 2-year and 10-year flows were also investigated during both the hydraulic and sediment analyses.

The hydrologic data for this report were developed in previous studies and are adopted here. While this information is considered adequate for present purposes, there are uncertainties involved in using it. Also, the inputs and methods of erosion/sedimentation studies are not very accurate and the unknown progress of development up to the 2036 level may have an important effect on sediment supply and channel stability. Thus

SANDOVAL COUNTY
BERNALILLO COUNTY





LOCATION MAP

NOT TO SCALE

CALABACILLAS ARROYO
PROJECT MAP
FIGURE 1

it will be necessary for the Albuquerque Metropolitan Arroyo Flood Control Authority (AMAFCA) to monitor erosion throughout the channel length, but particularly at the grade control structures.

Coordination with public agencies and private entities was a part of this project. In particular, the Calabacillas Arroyo is planned as a major open space and as a major open space link by the City of Albuquerque, with pedestrian and bicycle trails along the sides of the arroyo and an equestrian trail in the channel. Between Unser Blvd. and Golf Course Road the property owner, Bellamah Community Development, currently plans a golf course within the channel. New Mexico Utilities has existing and proposed water and sewer lines in the area, including several crossing the arroyo.

SECTION 2

SITE DESCRIPTION

The Calabacillas Arroyo watershed above Coors Road includes Seven Bar Channel, Black's Arroyo, and Black's Diversion Channel and encompasses about 89.5 square miles in Bernalillo and Sandoval Counties northwest of Albuquerque, New Mexico. The watershed is partly within the administrative jurisdiction of AMAFCA as well as under the planning authority of the City of Albuquerque, City of Rio Rancho, Bernalillo County, and Sandoval County.

As shown in Figure 1, the reach of the arroyo investigated during this study lies just north of Paradise Hills. The reach is about 3.1 miles long, begins about 0.4 miles west of the Rio Grande on the west side of Coors Road, and continues upstream past Black's Diversion Channel and Golf Course Road approximately to proposed Unser Blvd., which will extend what presently is Lyon Blvd. in Paradise Hills. Plates 1A, 1B, and 1C show the study area in detail.

At Unser Blvd. a major project consisting of a road crossing (Unser Bridge), a low flow channel, and a dam and desilting basin has been designed but not built. The study reach for the grade control project ends at the dam outlet transition. The design report calls for the outlet transition to extend approximately 300 feet downstream from the dam and for the transition and downstream face of embankment to have soil-cement protection to a depth of 15 feet (Unser Bridge Report, 1989).

The existing road crossings at Coors (six 14-ft. diameter culverts) and at Golf Course Rd. (a two-span bridge), about halfway between Coors Road and Unser Blvd., also provide grade control in the channel. Although the 8-foot thick soil-cement structure just downstream of Golf Course Road Bridge is called "sacrificial" in the design drawings, it is considered a permanent grade control in this report.

Black's Diversion Channel enters the Calabacillas Arroyo downstream of Golf Course Rd. bringing early-peaking flow from Rio Rancho. At the confluence a large soil-cement transition structure protects the concrete diversion channel; however, the Calabacillas Arroyo Channel has eroded in the confluence area and the cutoff at the end of the transition is exposed more than 3 feet.

As described in the report titled "Erosion Study to Determine Boundaries for Adjacent Development - Calabacillas Arroyo, Bernalillo County, New Mexico, 1983" (Usually called the "Prudent Line Report"), throughout the study reach the arroyo lies in soils that are loamy fine sands and fine sands. Nearby areas that contribute water and sediment include some loamy fine gravel and gravelly sand as well as some sandy loam and some sandy clay loam. The material in the channel bed is generally coarser than the surrounding soils, having lost the finer material as suspended sediment in previous flows, and the surface material is loose. The sandy soils have moderate to high infiltration rates (U.S. Soil Conservation Service Hydrologic Soil Groups A and B) and low amounts of clay. They are susceptible to bank caving and to erosion by water. The finer sands are readily eroded by wind.

Streambed elevation ranges from 5020 to 5250 feet above sea level. The average slope of the channel is about 1.4 percent. The main arroyo channel is typically flat across, wide, sandy, considerably lower than surrounding ground, and incised into one or both of the banks. There are several areas at tight bends where the banks are nearly vertical and are roughly twenty feet high. In many places there is a flood plain covered with scrub growth on one or both sides of the channel, especially in the area of old meander bends. Vegetation within the arroyo consists of small brush and grasses. Vegetation density is considered moderate for the area but there is a large amount of exposed soil. Several views of the channel are shown in Figures 2 and 3.

SECTION 3

INFORMATION SOURCES

There have been many studies of the Calabacillas Arroyo and the surrounding area. Those referenced in this report are listed below. The abbreviated titles as used in this report are shown in parentheses at the end of the reference.

Bohannan-Huston, Inc., 1982. Preliminary Flood Insurance Study, City of Albuquerque, Bernalillo County, New Mexico; Federal Emergency Management Agency, Albuquerque, N.M. (FLOOD PLAIN STUDY)

Resource Technology, Inc. and Resource Consultants, Inc., 1987. Calabacillas Arroyo Drainage Management Plan, Albuquerque Metropolitan Arroyo Flood Control Authority (AMAFCA) Albuquerque, N.M. (CADMP)

Simons, Li & Associates, Inc., 1983. Erosion Study to Determine Boundaries for Adjacent Development, Calabacillas Arroyo, Bernalillo County, New Mexico. AMAFCA, Albuquerque, N.M. (PRUDENT LINE STUDY)

Wilson & Co. and Resource Consultants, Inc., December, 1988 Calabacillas Arroyo Hydrology for Unser/Calabacillas Dam (HEC-1 Program Output). AMAFCA, Albuquerque, N.M. (UNSER DAM REPORT)

Resource Consultants, Inc., April 1989 Unser Bridge/Calabacillas Arroyo Detention Basin, AMAFCA, Albuquerque, N.M. (UNSER BRIDGE REPORT)

The Prudent Line Study (1983) was seminal for erosion analysis of Calabacillas Arroyo. A prudent line is a line outside of which land is generally safe from the effects of erosion under certain conditions; new platting along the Calabacillas Arroyo honors the Calabacillas prudent line. Prudent Line erosion/sedimentation analyses were based on soil samples taken from the arroyo and on hydraulic analyses which used scaled cross-sections and photo-topographic maps developed (1980 photography) for the Flood Plain Study (1982). The prudent line was included on the Flood Plain map in order to show its relation to the floodway and floodplain.

The Unser Dam (1988) and Unser Bridge (1989) Studies provide information on the sediment exiting Unser Dam. Those studies also provided most of the hydrologic data for the present study;

they were supplemented by the CADMP (1987). The main purposes of the dam are to control flood flows and to reduce the sediment load that might lead to a sediment plug in the Rio Grande. The 100-year flow at 2036 Development Conditions will be controlled to approximately the flow at existing conditions. Low flows will be passed but there will be a significant reduction of sediment flows for the 10-year and larger storms. The cutoff wall at the downstream edges of the embankment and the channel transition will extend at least 15 feet below grade.

The CADMP (1987) and the Unser Bridge (1989) and Unser Dam (1988) studies also used the Flood Plain Study topography. The present report uses the Prudent Line (1983) maps to show plan features. See Plates 1A, 1B, and 1C. However, new ground control was established and new aerial photography was used to generate 50 digitized cross-sections. All 21 previous cross-sections in the study reach were repeated and two sections downstream of Coors Road were included for reference. The repeated cross-sections are identified on Plates 1A, 1B and 1C by both the old letter identification and the numerical identification used in this report. The new photography will also be used to prepare detailed (1 ft. contour interval) topographic maps of the sites selected for the grade control structures.

Selected panels of the new photography were rectified, enlarged to the Prudent Line scale (1"=250'), and printed on mylar for comparing and identifying watershed development, channel characteristics, and areas of erosion. See Plates 2A through 2F. In general the channel has not changed much. Exceptions were at the S-bend downstream of Black's Diversion, where a berm had been constructed to direct flows in a shorter path across the meander bend, and at Golf Course Road, where the bridge had been constructed about 100 feet downstream of the old road and the main channel had been straightened downstream of the bridge. Also, head cuts are developing upstream from Black's Diversion Channel and Golf Course Road.

In addition to comparing aerial photographs, RTI staff made a field inspection to identify areas of bank migration and other places, including run-downs, that might need bank protection. RTI also obtained soil samples for use in the sediment analysis.

The City of Albuquerque Planning Dept. supplied the most recent draft (June 1989, unapproved) of the Calabacillas Arroyo Recreational Trails Plan and a set of trail standards.

In response to an inquiry from RTI, Bellamah Community Development discussed their plans for the proposed Paradise North Development between Unser and Golf Course Rd. on both sides of the arroyo, especially in regard to a proposed golf course and a gravity sewer crossing approximately 1500 ft. downstream from Unser Dam. They also provided plat drawings which they claim had been approved as to utility locations.

New Mexico Utilities provided master plans for their water and sewer systems and also some details on existing crossings. PNM provided maps of their facilities in the area. U.S. West responded that there were no crossings in the area except the suspended lines just upstream from the culverts at Coors Road.

SECTION 4

HYDROLOGY

Previously-developed hydrology was adapted to provide input for the hydraulic (HEC-2) and sediment analyses. The present investigation uses the 2-, 10-, and 100-year flows for both analyses. With Unser Dam in place the 100-year flow for 2036 Conditions will be controlled to approximately the existing 100-year flow. Low flow sediment will pass through but there will be a significant sediment reduction for the 10-year and larger flows.

The most recent hydrology for the study area is contained in the Unser Bridge Report (1989) and in the HEC-1 runs (Unser Dam Report, 1988) for that study. The Unser Dam Report used HEC-1 (the U.S. Army Corps of Engineers Flood Hydrograph computer program) to analyze the 10-, 100-, 270-, and 500-year storms and also the PMF (Probable Maximum Flood) and one half the PMF. The methodology and assumptions used and approved therein were somewhat different from those used and approved in the CADMP(1987); however, the CADMP (1987) did analyze the 2-year storm. Both studies were based on the same Existing Conditions and Year 2036 Development Conditions.

The Unser Bridge Report presents a comparison of the two methods for the existing-conditions 100-year storm and found that the difference in total runoff was generally small. Table 1, reproduced from the Unser Bridge Report, summarizes this comparison.

The peak flows used in the present study are shown in Table 2. The most recent (Unser Bridge Report, 1989) values are used where possible; that is, for the 10- and 100-year storms, except as noted in Table 2. For the 2-year storm the CADMP values were

TABLE 1
COMPARISON OF CADMP AND UNSER DAM
RUNOFF VALUES *

	Land Use					
	Existing		Future (2036)		Fully Developed	
Return Frequency	Peak (cfs)	Volume (inches)	Peak (cfs)	Volume (inches)	Peak (cfs)	Volume (inches)
<u>CADMP Values</u>						
10-year	4490	.277	6940	.373	10730	0.472
100-year	12400	.771	15300	.904	22700	1.033
270-year	-	-	18990	1.146	-	-
500-year	-	-	21130	1.288	-	-
<u>Proposed Design Values</u>						
10-year	5390	.329	9000	.476	14500	0.611
100-year	12300	0.74	16700	0.930	25600	1.01
270-year	15300	0.934	20000	1.12	30400	1.29
500-year	17400	1.060	22305	1.25	33900	1.43
PMF	92400	6.60	102000	7.87	146800	9.11
1/2 PMF	46200	3.30	51000	3.93	73400	4.56

* This table reproduced from Table 2.3 of Unser Bridge/ Calabacillas Arroyo Detention Basin, Albuquerque, New Mexico, Resource Consultants Inc., April 1989.

TABLE 2

PEAK FLOWS (C.F.S.) IN CALABACILLAS ARROYO

Location/Condition	Return Period 2-Year(1)	10-Year	100-Year
Existing Conditions, No Dam			
Below Unser Blvd.	290	5390	12300
Below Black's Diversion	3160	5440	12455 (3)
2036 Development With Dam			
Below Unser Blvd.	1430	6300	12930
Below Black's Diversion	3925 (2)	6375	13000

Notes:

1. Based on ratio (Unser Dam flow)/(CADMP flow) for 10-year flows.
2. This is the same as the 2036 value without the dam. The 2-year storm below Black's Diversion is dominated by flow from the diversion. This flow peaks early and is unaffected by the dam.
3. The Unser Dam Report (1988) asumed the dam was in place for downstream flood routing so this value was obtained indirectly. The CADMP (1987) identified an increase in peak flow at Black's Diversion. This increase was modified by the ratio (Unser Dam flow) for 100-year flow and added to 12300.

Framework File: Peak
Date: 7/3/89

adjusted by the ratio of Unser Bridge Report values to CADMP values for the 10-year storm. Downstream of Black's Diversion the 2-year values are dominated by flow from the diversion, which peaks earlier than the Calabacillas and is not affected by the dam. Therefore the no-dam value was still used for the 2036 case. For lower frequency floods, the hydrographs and the peak rates are dominated by the drainage area upstream of Unser Blvd.

The sediment analysis uses hydrographs simplified from calculated hydrographs. Unser Dam Report (1988) hydrographs were not readily available so the CADMP (1987) hydrographs were used to estimate hydrograph shape and duration. The hydrograph peaks were modified to match Unser Bridge Report (1989) peak flows.

SECTION 5

HYDRAULIC ANALYSIS

The hydraulic analyses were made using HEC-2, the U.S. Army Corps of Engineers Water Surface Profiles computer program (1984 revision of the 1974 version, IBM-PC-XT version). This program assumes that Manning's equation for normal flow in uniform channels of small slope can be used to approximate gradually-varied flow. The results of the HEC-2 runs provided design values for the channel and grade control structures and provided input for the sediment analysis.

Three development conditions were analyzed under the 2-, 10-, and 100-year storms:

- o Existing Development, without Unser Dam or grade control
- o 2036 Development, with Unser Dam with grade control structures buried (use existing slope)
- o 2036 Development, with Unser Dam and with grade control structures exposed (use eroded slope)

In the last case, it was assumed that the channel bed had eroded by the design amount --10 feet-- from one grade control structure to the next but that the cross-sections had not otherwise changed. Five feet of erosion was assumed downstream of the existing 8-foot thick soil cement sill at Golf Course Road. All HEC-2 runs assumed supercritical flow.

The design flows used are shown in Table 2; these are peak flows based on HEC-1 hydrology as discussed earlier. Only two flow rates are used in each analysis, the peak flow leaving Unser Bridge and the peak flow just below the confluence with Black's Diversion Channel between Golf Course Road and Coors Road. This choice corresponds to the HEC-1 peak flows, which tend to decrease downstream because of routing effects. Black's Diversion increases the peak flow in the main channel but peaks early and recedes early; it has a major effect on peak flow only for the 2-year storm. The analysis for that case is conservative, in that inflow from the diversion will actually have decreased by the time the peak from the main channel reaches the confluence. Although there are several inflows in the study reach, particularly from development near Golf Course Rd. and near Coors Road, they also peak early and do not noticeably increase peak hydrograph rates. These tributary flow rates were not determined for this report.

Channel geometry was based on digitized cross-sections prepared from aerial photography taken April 28, 1989 by Thomas R. Mann and Associates. The distance between sections varies between 250 and 500 feet, depending on channel conditions. The approximate location of the sections is shown in Figure 1 as well as on Plates 1A, 1B, and 1C. Gross channel slope from beginning to end of the project is 1.39% Both the 1989 and 1980 thalweg profiles are shown on Plate 3.

Typically the main channel is almost flat across, 70 to 150 feet wide, considerably lower than surrounding areas, and many vehicles have been driven along the arroyo bed. The banks and overbank areas (floodplain) have low brush and grasses of moderate density but also many bare soil areas. See Figures 2 and 3. The bed material is mostly loose sand with some gravel. Figure 4 shows the representative size distribution for the bed material.

Manning's n, the roughness parameter, was chosen after the field inspection. The roughness parameter was assumed to be the same throughout the study reach and not to vary with depth of flow. The values used were n = 0.030 for the main channel and n = 0.045 for the overbank areas.

These roughness values are in line with previous studies. The CADMP (1987) used n = 0.030 to 0.035 for the main channel and n = 0.040 to 0.050 for the overbank areas; as reported in the Prudent Line Study (1983), the Flood Plain Study (1982) used n = 0.035 for the main channel and n = 0.040 for the overbanks; and the Prudent Line Study (1983) used n = 0.040 for the overbank and n = 0.030 for the channel, based on potential flow regime and bed form condition. A value of n = 0.030 for the channel produces higher velocities (compared to n = 0.035) and is conservative for erosion considerations. A value of n = 0.045 for the overbank areas produces lower velocities in the overbank and more flow in the main channel (compared to n = 0. 040) and is conservative for depth considerations.

Table 3 summarizes the results of the HEC-2 runs. Copies of the complete HEC-2 summary output are included in Appendix A. The corresponding water surface profiles are shown on Plate 3.

All the HEC-2 runs were for supercritical conditions. The results indicate that the flow depth is usually close to critical. In some cases, particularly for the smaller storms, the flow tends toward subcritical in sections of flatter slope only to become supercritical again downstream. The slope of the thalweg, as indicated by the current (1989) cross-sections, changes fairly often and significantly, this may be smoothed during high flows. However, using critical velocities and depths in lieu of subcritical values is conservative for the sediment analysis and the design of the grade control structures.

SECTION 6

EROSION AND SEDIMENTATION ANALYSIS

PURPOSE

The purpose of the erosion and sedimentation analysis was to determine the quantity and general location of erosion or sedimentation (deposition) that may occur in the study reach for 2036 Development Conditions, with the Unser Bridge and the 7 grade control structures. The depth of local scour at the base of a drop structure and areas requiring bank protection were also analyzed. The results of this analysis will help determine interim and long term solutions for grade control and bank erosion protection in the study reach. Previous erosion studies in the study reach which were reviewed for this effort include the Prudent Line Report (1983), the CADMP (1987), and the Unser Bridge Report (1989).

METHODOLOGY

The quantitative approach is based on the sediment transport relationships previously developed for the study reach as listed in the Prudent Line Report. Based on similar sedimentation

TABLE 3

SUMMARY OF HYDRAULIC ANALYSES

CONDITION & RETURN PERIOD	Q		Depth			Top Width			Velocity		
(yr.)	MIN. (cfs)	MAX. (cfs)	MIN. (ft.)	MAX. (ft.)	MID. RANGE (ft.)	MIN. (ft.)	MAX. (ft.)	MID. RANGE (ft.)	MIN. (ft./sec.)	MAX. (ft./sec.)	MID. RANGE (ft./sec.)
EXISTING CONDITIONS											
2	290	3160	.8	5.0	1-3.5	55	259	90-150	4.1	13.2	4.5-11
10	5390	5440	2.9	6.3	4-5.5	74	513	120-250	9.1	18.7	11-15
100	12300	12455	3.9	9.6	5-7.5	82	702	200-350	11.8	22.9	15-20
2036 DEVELOPMENT CONDITIONS DAM <u>ONLY</u>											
2	1430	3925	1.4	5.5	2.5-4	63.1	249	80-150	6.2	13	8-12
10	6300	6375	3.1	6.8	4-6	75	525	130-250	9.6	19.5	12-16
100	12930	13000	4.5	9.9	5.5-8.5	83	565	130-330	12	23	15-19
2036 DEVELOPMENT CONDITIONS DAM AND GRADE CONTROL (ASSUMING 10' OF DROP TYPICAL AT EACH STRUCTURE)											
2	1430	3925	.8	5.4	2-4	60.1	286	80-160	6	29.1*	6.5-12
10	6300	6375	2.2	7	4-6.5	76	525	110-250	9.2	30.4*	10-14
100	12930	13000	3.5	10.8	5-8	86	585	130-380	11.8	33.2*	13-17

characteristics at present (1989) compared to those at the time of the Prudent Line Report (1983) and basically similar arroyo alignment and cross sectional shape, it was assumed that the sediment transport relationships originally developed in the Prudent Line Report were applicable to this study. Also, for the 9-year period between surveys (1980 to 1989) the arroyo bed has not changed considerably except immediately upstream of Black's Diversion Channel and Golf Course Road where active head cutting is in progress and upstream from Coors Road where the channel is aggrading. This comparison is depicted on Plate 3 and changes at six specific cross-sections are shown on Plate 4. However, it appears that the 1980 vintage cross-sections were scaled off the topographic maps and may not be strictly representative of the true cross-section in 1980.

However, the unit sediment transport equation used in the Prudent Line Report was modified for use in the Unser Bridge Study. The sediment bed loads available to the study reach as a result of the Unser Bridge were provided in the Unser Bridge Report and are used as the basis to predict the total sediment load (suspended load plus bed load) in the present study.

ANALYSIS APPROACH

The erosion/sedimentation process is very dynamic in the natural state and complicated by activities of man. The analytical process used to predict erosion/sedimentation and equilibrium slope is an iterative process to balance aggradation/degradation with a computed equilibrium slope. The analysis approach used in this study was to compute the aggradation/degradation for three conditions assuming that the actual process may be within the range of these conditions. The three conditions analyzed for prediction of aggradation/degradation and corresponding equilibrium slope are:

1. Existing Development Conditions - peak discharge, sediment supply and channel slope.

2. 2036 Development Conditions - peak discharge with Unser Bridge sediment supply and existing channel slope.

3. 2036 Development Conditions - peak discharge with Unser Bridge sediment supply and 10 feet of assumed erosion at the base of each grade control structure, and 5 feet of erosion at Golf Course Road.

The significance of the slope difference between conditions two and three is a considerable difference in flow velocities which could significantly affect the sediment transport capacity and consequently the predicted aggradation/degradation volume and equilibrium slope.

AGGRADATION/DEGRADATION AND EQUILIBRIUM SLOPE ANALYSIS

Sediment Particle Size Analysis

During the field observation, RTI staff obtained seven soil samples from the arroyo bottom throughout the study reach. A sieve analysis was performed on all samples and the representative bed material gradation curve was plotted as shown on Figure 4. The gradation curve was compared to those given in the Prudent Line Report to determine if the sediment characteristics were similar; and consequently to justify use of the previously derived sediment transport equations discussed below. The comparison indicates the sediment size distribution is similar to those provided in the Prudent Line Report.

Sediment Continuity

Representative Subreaches

This study reach of the Calabacillas Arroyo was divided into 10 representative subreaches for analysis of sediment continuity. The subreaches are numbered beginning at the upstream end of the Unser Bridge (the supply subreach) and continue to Coors Road, and were delineated according to the locations of the proposed grade control structures and Golf Course Road. The subreaches are shown on Figure 1 and Plates 1A, 1B and 1C in plan view and on Plate 3 in profile.

Sediment Transport Relations

A sediment discharge per unit width equation was developed for this study reach of Calabacillas Arroyo and was given in the Prudent Line Report as

qs = 1.1*10**-4 (Y**0.436)(V**3.852)

qs = unit sediment discharge (cfs./ft. of width)

Y = hydraulic depth (ft.)

V = velocity (ft./sec.)

However, this equation was modified by Resource Consultants Inc. for the Unser Bridge Report as

$qs = 0.5*10**-4 (Y**0.436)(V**3.852)$

and this equation was used in the present study. Using this unit sediment discharge equation, the total sediment transport capacity of each subreach for the 2-, 10- and 100-yr. return period storms was determined as follows:

Step 1 -

The HEC-2 computer program was run for the total reach based on the peak discharges (Qp) discussed previously and listed in Table 2, for the 2036 Development Condition with Unser Bridge and with and without the seven proposed grade control structures (assuming each drop will have 10 feet of fall).

For each subreach and each Qp above, the qs was determined (using the derived equation above) at each cross-section.

Step 2 -

The qs per cross-section within a subreach was then multiplied by the top width of that cross-section to obtain the total sediment discharge (Qs) for that cross-section. The Qs values per cross-section in a subreach were averaged to obtain an average Qs per subreach for each Qp.

Therefore, the range of peak discharges (Qp) and the corresponding average sediment discharges (Qp) were determined for each subreach. A regression equation of the form

$Qs = a\ Qp**b$

was then developed for each subreach. Coefficients a and b were derived for each subreach.

Step 3 -

The hydrographs from the Unser Bridge report were not readily available and therefore the CADMP hydrographs were assumed to be similar. Triangular shaped hydrographs were developed from the runoff hydrographs obtained from the CADMP hydrographs of the 2-yr., 10-yr., and 100-yr. storms as shown on Figures 5A and 5B. The hydrographs shown on Figure 5A were assumed for subreaches 2 through 8 and hydrographs shown on Figure 5B were assumed for subreaches 9 and 10. The procedure for determining the triangular hydrographs is as follows:

1. The volume under the triangular hydrograph and the computed runoff hydrograph must be equal.

2. The rising and falling limbs of the triangular hydrograph should be equal (triangular hydrograph is symmetrical about the peak).

This procedure along with the fact that a runoff hydrograph has more volume to the right of the peak than to the left (not symmetrical about the peak) accounts for the lagged triangular hydrograph peaks as shown in Figures 5A and 5B. The total duration of each triangular hydrograph (d) was then determined from Figures 5A and 5B.

Step 4 -

The total sediment transport capacity per storm for each subreach was then determined using an equation of the form

```
Qs=((a Qp**b)*d*3600)/(b+1))*(62.4*2.65/2000)
```

where:

a and b were previously derived

d = total duration of the triangular hydrograph (hours)

Qp = peak discharge (cfs.)

Qs = sediment transport capacity (tons)

Storm Related Aggradation/Degradation

The sediment bed material loads available to the reach below the Unser Bridge for existing conditions through 2036 Development Conditions with the Unser Bridge in place were provided in the Unser Bridge Report (Table 5.6) and is repeated here on Table 4. The bed material load values presented are assumed to be the total load available to the subreach below the bridge -the wash load is not of consequence. The 2-year values were computed based on linear regression of the 10-, 25- and 100-year sediment data provided in the Unser Bridge Report.

As presented in the CADMP and also in the Unser Bridge Report, sediment yield from urbanized pervious areas was evaluated assuming that 80 percent of remaining pervious area was insignificant as a source area (Case 1) and alternately, 40 percent (Case 2).

This study analyzed the erosion/deposition within the study reach assuming the sediment yield from the detention pond based on 2036 Development Conditions and Case 2. The total sediment load (bed load + suspended load) that will pass from the dam is necessary to the subreach below the dam.

TABLE 4

TOTAL SEDIMENT LOAD DELIVERED
TO THE
REACH BELOW THE UNSER BRIDGE

Sediment Load Type	Return Period	Conditions*					
		Existing	After Bridge Only	2036 No	Development Bridge	2036 With	Development Bridge
				Case 1	Case 2	Case 1	Case 2
	(yr.)	(tons)					
Bed Load	* 100	312,000	62,000	178,000	210,000	36,000	42,000
Bed Load	* 25	172,000	34,000	98,000	116,000	20,000	23,000
Bed Load	* 10	84,000	17,000	48,000	56,000	10,000	11,000
Bed Load	2(a)	36,925	8,000		25,000		5,000

* The Bed Loads for all Conditions and 10-, 25- and 100-year return periods were determined in the "Unser Bridge/Calabacillas Arroyo Detention Basin, Albuquerque, New Mexico" Report. The 2-yr. values were not provided. The Bed Loads given are actually the total load supplied to the subreach below the bridge.

(a) The 2-yr. values were determined by linear regression based on the 10-, 25- and 100-yr. data provided from the Unser Bridge Report.

The aggradation/degradation volumes for each subreach were determined using the sediment transport capacity of each subreach and sediment supply to each subreach by application of sediment continuity principles. In short, if the transport capacity of an upstream subreach is greater than that of a downstream subreach, then deposition will occur in the downstream subreach. If the transport capacity of the upstream subreach is less than the downstream subreach, then erosion will occur in the downstream subreach.

The procedure used to determine the aggradation/degradation per return period per subreach is based on comparison of the computed transport capacity to the available aggradation/degradation volume a subreach may provide. The latter was determined as the difference in volume between the existing slope and computed equilibrium slope. The process of aggradation/degradation and equilibrium slope are directly related and computed conjunctively by subreach.

Equilibrium Slope

Equilibrium slope is determined by assuming that, if over the long term there are no changes in channel geometry or sediment supply, an equilibrium slope would be achieved. Equilibrium slope implies that Qs(in) = Qs(out), therefore no aggradation or degradation would occur.

The unit sediment transport equation derived previously was based on factors for the entire reach and therefore is applicable for equilibrium slope determination. The following equation (from the Prudent Line Report) was used to determine the equilibrium slope for each reach.

Seq. = Sex. (Qs supply/Qs capacity)**0.976

Seq. = subreach equilibrium slope (feet/foot)

Sex. = subreach existing slope (feet/foot)

Qs supply = supply of sediment to the subreach

Qs capacity = sediment transport capacity of the subreach

The computational steps used to determine the aggradation/ degradation and equilibrium slope per subreach are as follows:

Step A -

Compute the equilibrium slope in the subreach based on the sediment supply to the subreach and the transport capacity of the subreach.

Step B -

Compute the aggradation/degradation required to satisfy the difference of sediment supply to the downstream subreach to the transport capacity of the subreach. Based on a sediment continuity approach, this numerical value would be the amount used to determine the aggradation/degradation in the subreach; however, that approach does not account for the length and width differences between subreaches. For example, if Reach 1 (upstream) is 100 feet long and 1 foot wide, and Reach 2 (downstream) is 10 feet long and 1 foot wide, the subtraction of the transport capacity from Reach 2 from the supply from Reach 1 would force the entire aggradation or degradation to occur in the 10 feet of Reach 2 and will lead to erroneous results. Therefore to account for the differences in subreach length and widths and equilibrium slopes Step C was developed.

Step C -

Compute the available aggradation/degradation as the difference between the existing and equilibrium slopes. Assume a triangular volume due to control at the top of proposed downstream grade control with maximum erosion occurring at the base of the upstream grade control structure.

Step D -

Compute the sediment obtained from each subreach based on comparison of 1). Aggradation/degradation volume to satisfy transport capacity; to 2). Available aggradation/degradation volume.

Step E -

Compute the sediment supply from current subreach to downstream subreach based on comparison of sediment supply into current subreach, transport capacity of current subreach, aggradation/degradation volume to satisfy transport capacity and available aggradation/degradation volume. The logical steps of the comparison are listed in Appendix B.

Step F -

Compute the maximum potential aggradation/degradation depth as the difference between the existing and equilibrium slopes, occurring at the base of the upstream grade control structure assuming that the top of the downstream grade control structure does not change.

Step G -

Compute the actual aggradation/degradation volume based on comparison of sediment obtained from a subreach to the aggradation/degradation volume required to satisfy transport capacity.

Step H -

Compute the average aggradation/degradation depth based on the actual volume (Step 7) assuming a triangular shaped deposition or erosion pattern (between grade control structures). Assume the average depth occurs at the centroid of the triangle.

For aggrading reaches, a bulking factor of 0.654 was applied to account for void spaces in deposited sediment. Therefore, the volume of deposited sediment was assumed to the 65.4 percent greater than the volume of sediment in the water flow. However, in degrading reaches the volume of sediment eroded and the volume in the water flow were assumed to be equal.

AGGRADATION/DEGRADATION AND EQUILIBRIUM SLOPE RESULTS

This procedure was applied to the three conditions analyzed which are:

1. Existing Development Conditions - peak discharge, sediment supply and channel slope.

2. 2036 Development Conditions - peak discharge with Unser Bridge sediment supply and existing channel slope.

3. 2036 Development Conditions - peak discharge with Unser Bridge sediment supply and 10 feet of assumed erosion at the base of each grade control structure and 5 feet at Golf Course Road.

The velocity is considerably higher between Condition 2 compared to Condition 3 (as expected) and consequently produces a larger sediment transport capacity.

Tables 5, 6 and 7 summarize the results of the equilibrium slope analysis for the three conditions for the 2-, 10- and 100-yr. return periods, respectively. Table 8 is a rough comparison of the equilibrium slopes determined in this study and those determined in the Prudent Line Report (1983). The slopes between the two are generally in the same range. In order to facilitate

TABLE 5
SUMMARY OF EQUILIBRIUM SLOPE COMPUTATIONS
(2-Year Return Period)

Subreach	Reach Length (ft)	Average** Assumed Slope (ft/ft)	Average Existing Slope (ft/ft)	Equilibrium Slopes Condition* 1 (ft/ft)	Condition* 2 (ft/ft)	Condition* 3 (ft/ft)
1	NA	NA	NA	NA	NA	NA
2	2065	.00825	.00825	.04442	.00631	.00611
3	1670	.00882	.01485	.00842	.00842	.01788
4	2485	.00971	.01378	.01645	.01645	.00996
5	2265	.00904	.01351	.01186	.01186	.01162
6	825	.01371	.02000	.01712	.01712	.01936
7	1455	.00862	.01553	.01388	.01388	.01643
8	1635	.00913	.01530	.01220	.01220	.00765
9	1655	.00697	.01304	.00539	.00539	.00763
10	2385	.00718	.01140	.01267	.01267	.01105

* Condition 1 - Existing development condition peak discharge, sediment supply and channel slope.
* Condition 2 - 2036 development condition peak discharge with Unser Bridge sediment supply and existing channel slope.
* Condition 3 - 2036 development condition peak discharge with Unser Bridge sediment supply and 10 feet of assumed erosion at base of each grade control structure.
** Average slope assuming 10 feet of drop at each grade control structure.

Framwork File: Sum2
Date: 7-3-89

TABLE 6
SUMMARY OF EQUILIBRIUM SLOPE COMPUTATIONS
(10-Year Return Period)

Subreach	Reach Length (ft)	Average** Assumed Slope (ft/ft)	Average Existing Slope (ft/ft)	Equilibrium Slopes Condition* 1 (ft/ft)	Condition* 2 (ft/ft)	Condition* 3 (ft/ft)
1	NA	NA	NA	NA	NA	NA
2	2065	.00825	.00825	.00638	.00088	.00102
3	1670	.00882	.01485	.01014	.01014	.01325
4	2485	.00971	.01378	.01622	.01622	.01339
5	2265	.00904	.01351	.01244	.01244	.01266
6	825	.01371	.02000	.01440	.01440	.01444
7	1455	.00862	.01553	.01275	.01275	.01835
8	1635	.00913	.01530	.01373	.01373	.01075
9	1655	.00697	.01304	.01582	.01582	.01781
10	2385	.00718	.01140	.01227	.01227	.01097

* Condition 1 - Existing development condition peak discharge, sediment supply and channel slope.
* Condition 2 - 2036 development condition peak discharge with Unser Bridge sediment supply and existing channel slope.
* Condition 3 - 2036 development condition peak discharge with Unser Bridge sediment supply and 10 feet of assumed erosion at base of each grade control structure.
** Average slope assuming 10 feet of drop at each grade control structure.

Framwork File: Sum10
Date: 7-3-89

TABLE 7
SUMMARY OF EQUILIBRIUM SLOPE COMPUTATIONS
(100-Year Return Period)

Subreach	Reach Length (ft)	Average** Assumed Slope (ft/ft)	Average Existing Slope (ft/ft)	Equilibrium Slopes Condition* 1 (ft/ft)	Equilibrium Slopes Condition* 2 (ft/ft)	Equilibrium Slopes Condition* 3 (ft/ft)
1	NA	NA	NA	NA	NA	NA
2	2065	.00825	.00825	.00593	.00084	.00107
3	1670	.00882	.01485	.01110	.01110	.01146
4	2485	.00971	.01378	.01611	.01611	.01545
5	2265	.00904	.01351	.01273	.01273	.01319
6	825	.01371	.02000	.01318	.01318	.01253
7	1455	.00862	.01553	.01151	.01151	.01683
8	1635	.00913	.01530	.01221	.01221	.01325
9	1655	.00697	.01304	.01255	.01255	.01373
10	2385	.00718	.01140	.01044	.01044	.01120

* Condition 1 - Existing development condition peak discharge, sediment supply and channel slope.
* Condition 2 - 2036 development condition peak discharge with Unser Bridge sediment supply and existing channel slope.
* Condition 3 - 2036 development condition peak discharge with Unser Bridge sediment supply and 10 feet of assumed erosion at base of each grade control structure.

** Average slope assuming 10 feet of drop at each grade control structure.

Framwork File: Sum100
Date: 7-3-89

TABLE 8
COMPARISON OF EQUILIBRIUM SLOPES

Subreach (a)	Subreach (b)	Average Existing Slope (ft/ft) (a)	Average Existing Slope (ft/ft) (b)	2-Year Equilibrium Slope (Long Term) (ft/ft) (a)	2-Year Equilibrium Slope (Long Term) (ft/ft) (b)	100-Year Equilibrium Slope (Short Term) (ft/ft) (a)	100-Year Equilibrium Slope (Short Term) (ft/ft) (b)
10	NA			Supply Reach			
9	NA	.01394		.01443		.01542	
8	1	.01450	NA	.02936	NA	.03830	NA
7	2	.01333	.00825	.00442	.04442	.00473	.00593
6	3	.01337	.01485	.01015	.00842	.01085	.01110
5	4	.01315	.01378	.01536	.01645	.01081	.01611
4	5	.01540	.01351	.01132	.01186	.01478	.01273
	6		.02000		.01712		.01318
3	7	.01400	.01553	.01170	.01388	.01058	.01151
	8		.01530		.01220		.01221
2		.01325		.01608		.02171	
	9		.01304		.00539		.01255
1	10	.01053	.01140	.00794	.01267	.00482	.01044

(a) Information from Prudent Line Report (1983).
Based on qs = (1.1*10**-4)(V**3.852)(Y**0.436)

(b) Computed in this study.
Based on qs = (0.5*10**-4)(V**3.852)(Y**0.436)

Existing Development Condition peak discharge, sediment supply and existing channel slope.

Framework File: Compare
Date: 7/3/89

the understanding of data in Tables 5, 6 and 7, a simplified and alternate representation of the equilibrium slope results is presented in Tables 9, 10 and 11 which summarize the maximum potential aggradation/degradation and average aggradation/degradation depths for the three conditions for the 2-, 10- and 100-yr. return periods, respectively. The maximum potential aggradation/degradation depth is computed as the difference between existing slope and equilibrium slope; and equilibrium slope is computed based on sediment supply and transport capacity.

In general, for the three conditions analyzed, the greatest maximum potential change occurs in Subreach 2 (immediately below Unser Bridge) or Subreach 3 which is expected because they are the nearest subreaches to the change in sediment supply. However, as a direct result of the computational procedure and the sediment supply assumed for the 2-yr. return period, the value of 75 feet of aggradation shown on Table 9 is unrealistic. The sediment supply for Subreach 2 existing conditions was estimated based on a linear regression of the 10-, 25- and 100-yr. bed material load values, and appears to be high. Consequently the numerical method using the equilibrium slope produced a result of 75 feet. The average aggradation depth computed based on the computed sediment volume is 1 foot which is reasonable.

Subreach 9 for the 2-year return period also shows a large maximum potential erosion depth (13 feet). This a direct result of larger discharges (and increased transport capacity) in subreaches 9 and 10 due increases in peak discharge from 1430 cfs to 3925 cfs as a result of Black's Arroyo inflow into the Calabaçillas Arroyo. Sediment load information from Black's Arroyo was not available and was not considered in this analysis. The actual average degradation computed is 2 feet and 1 foot for subreaches 9 and 10, respectively, based on the computed sediment volume.

Comparison of the average aggradation/degradation values of the three conditions for the 2-year return period (Table 9) indicates that the arroyo will remain stable with minor erosion of 2 feet in subreaches 8, 9 and 10. The same comparison for the 10-year return period (Table 10) indicates a few subreaches may aggrade up to 4 feet and most will degrade up to 5 feet with the average about 3 feet. The comparison of the 100-year return period (Table 11) indicates that nearly all subreaches will degrade with the average of about 4 feet and maximum degradation in Subreach 2 of 8 feet. However, Subreach 4 may aggrade up to 4 feet.

In summary, the 2-year storm for the three conditions analyzed will cause minor degradation of 2 feet. The 10-year storm will

TABLE 9
SUMMARY OF AGGRADATION/DEGRADATION COMPUTATIONS
(2-Year Return Period)

Subreach	Maximum Potential** Aggradation(+)/Degradation(-)			Average Aggradation(+)/Degradation(-)		
	Condition* 1 (ft)	Condition* 2 (ft)	Condition* 3 (ft)	Condition* 1 (ft)	Condition* 2 (ft)	Condition* 3 (ft)
1	NA	NA	NA	NA	NA	NA
2	75	-4	-4	1	0	0
3	-11	-11	5	0	0	0
4	7	7	-9	0	0	0
5	-4	-4	-4	0	0	0
6	-2	-2	-1	0	0	0
7	-2	-2	1	0	0	0
8	-5	-5	-13	-1	-1	-1
9	-13	-13	-9	-2	-2	-1
10	3	3	-1	1	1	0

* Condition 1 - Existing development condition peak discharge, sediment supply and channel slope.
* Condition 2 - 2036 development condition peak discharge with Unser Bridge sediment supply and existing channel slope.
* Condition 3 - 2036 development condition peak discharge with Unser Bridge sediment supply and 10 feet of assumed erosion at base of each grade control structure.

** The maximum potential aggradation/degradation depth is assumed to occur at the downstream base of a grade control structure based on the difference of equilibrium slope and existing slope.

Framwork File: ADSum2
Date: 7-3-89

TABLE 10
SUMMARY OF AGGRADATION/DEGRADATION COMPUTATIONS
(10-Year Return Period)

Subreach	Maximum Potential** Aggradation(+)/Degradation(-)			Average Aggradation(+)/Degradation(-)		
	Condition* 1 (ft)	Condition* 2 (ft)	Condition* 3 (ft)	Condition* 1 (ft)	Condition* 2 (ft)	Condition* 3 (ft)
1	NA	NA	NA	NA	NA	NA
2	-4	-15	-15	-1	-3	-2
3	-8	-8	-3	-2	-2	-1
4	6	6	-1	1	1	0
5	-2	-2	-2	-1	-1	0
6	-5	-5	-5	-3	-3	-3
7	-4	-4	4	-3	-3	2
8	-3	-3	-7	-2	-2	-5
9	5	5	8	3	3	4
10	2	2	-1	1	1	0

* Condition 1 - Existing development condition peak discharge, sediment supply and channel slope.

* Condition 2 - 2036 development condition peak discharge with Unser Bridge sediment supply and existing channel slope.

* Condition 3 - 2036 development condition peak discharge with Unser Bridge sediment supply and 10 feet of assumed erosion at base of each grade control structure.

** The maximum potential aggradation/degradation depth is assumed to occur at the downstream base of a grade control structure based on the difference of equilibrium slope and existing slope.

Framwork File: ADSum10
Date: 7-3-89

TABLE 11
SUMMARY OF AGGRADATION/DEGRADATION COMPUTATIONS
(100-Year Return Period)

Subreach	Maximum Potential** Aggradation(+)/Degradation(-) Condition* 1 (ft)	Condition* 2 (ft)	Condition* 3 (ft)	Average Aggradation(+)/Degradation(-) Condition* 1 (ft)	Condition* 2 (ft)	Condition* 3 (ft)
1	NA	NA	NA	NA	NA	NA
2	-5	-15	-15	-3	-8	-5
3	-6	-6	-6	-4	-4	-3
4	6	6	4	4	4	2
5	-2	-2	-1	-1	-1	0
6	-6	-6	-6	-4	-4	-4
7	-6	-6	2	-4	-4	1
8	-5	-5	-3	-3	-3	-2
9	-1	-1	1	-1	-1	1
10	-2	-2	0	-2	-2	0

* Condition 1 - Existing development condition peak discharge, sediment supply and channel slope.
* Condition 2 - 2036 development condition peak discharge with Unser Bridge sediment supply and existing channel slope.
* Condition 3 - 2036 development condition peak discharge with Unser Bridge sediment supply and 10 feet of assumed erosion at base of each grade control structure.

** The maximum potential aggradation/degradation depth is assumed to occur at the downstream base of a grade control structure based on the difference of equilibrium slope and existing slope.

Framwork File: ADSum100
Date: 7-3-89

cause an average of about 3 feet of degradation and the 100-year an average of about 4 feet of degradation.

To account for analytical errors resulting from the dynamic nature of the sediment transport process and the computational limits of this analysis, the computed degradation values may be off by a factor of 2. Therefore the 2-, 10- and 100-yr. average degradation depths could be 4, 6, and 8 feet, respectively. Consequently, a 10 foot design depth is appropriate for the proposed grade control structures. Based on Tables 9, 10, and 11, grade control structures No. 2, 3 and 6 should be built first and others as needed.

The step by step results of the computational procedure for the 2-, 10- and 100-yr. return periods for the three conditions are presented in Appendix B.

LOCAL SCOUR ANALYSIS

The local scour depth at the base of a grade control structure was analyzed using the Schoklitsch Formula which was developed to analyze scour due to water discharge over a hydraulic structure with a drop (Simons and Li, 1981). The formula is:

S= (3.75 * h**0.2 * q**0.5 / D90**0.32) - hd

where :

S = depth of scour hole (feet)
hd = downstream water depth (feet)
qs = water discharge (cfs)
D90 = sediment size of which 90% is finer (mm)
H = vertical distance between the energy grade line and the downstream water surface (feet)

Two grade control structures were analyzed using the HEC-2 results for the 100-yr. return period assuming 10 feet of drop at the base of each grade control structure. Structures 1 and 3 were analyzed to compare the results of a wide structure and flow width (Structure 1) to a narrow structure and flow width (Structure 3). Based on the assumption of 10 feet of drop, the results from the Schoklitsch Formula are 10 feet of local scour at Structure 1 and 27 feet at Structure 3. Therefore a stilling basin is recommended at the base of each structure to provide toe protection and to prevent undermining and failure of the structure. The length of the stilling basin required was determined to be 50 feet based on the trajectory of the 100-year flow over the top of the structure and a fall of 10 feet.

ARROYO BANK EROSION ANALYSIS

Due to the tendency of the arroyo to meander and cause bank erosion, the study reach was analyzed to determine areas which may require bank stabilization. The analysis consisted of a field inspection noting potential problem areas and comparison of previous (1980) and present (1989) aerial photographs. The aerial photograph analysis compared the lateral migration of the arroyo between these two years.

Based on the field inspection and aerial photograph comparison, the arroyo banks in some areas have been eroded. The degree of erosion and potential for further erosion was estimated to determine which areas will require bank protection. Bank protection has been recommended in areas where the bank has approached existing development and/or the Prudent Line which was determined in the Prudent Line Report (1983).

The locations of arroyo migration (bank erosion) based on the field inspection and the aerial photograph comparison, are shown on Plates 1A, 1B and 1C along with areas where bank protection has been recommended. After considering the various types of bank protection available, soil cement lining was selected as the best available treatment. The bank protection design is based on the following:

1. Soil cement is buried 3 feet below the lowest point in the closest HEC-2 cross-section where bank protection is recommended.

2. Soil cement will be used along the existing arroyo bank and will rise three feet above the 100-yr. water surface elevation for existing conditions. (Based on the calculated freeboard requirement per City of Albuquerque criteria).

Figure 6 shows a typical section with bank protection. The material quantities and associated costs are discussed in Section 8 of this report.

SECTION 7

GRADE CONTROL STRUCTURE EVALUATION

Because of the relatively wide channel section and deep valley, this reach of the Calabacillas Arroyo is well suited to installation of grade control structures to control head cuts from progressing upstream. The channel bottom width varies from 110 to 400 feet and 100-year flow velocities range from 12 to 18 fps. Also, the sediment transport analysis (Section 6) indicates that the channel will be relatively stable with the seven proposed grade control structures. Therefore, it was initially assumed that only minimal scour protection would be required along the channel and local scour at each structure would be significant.

The seven grade control structures were roughly located in the Unser Bridge Report. These locations were refined during the present study and are also shown on Plates 1A, 1B and 1C. Final location will be based on adequacy of the foundation material for support strength, resistance to sliding, and relative homogeneity to avoid differential settlement of the structure.

The sediment transport analysis indicated that the allowable scour depth of 10 feet at the toe of each structure will be adequate in most cases. Depth of scour up to 15 feet were computed; but, given the large number of assumptions and regressed coefficients, computed depths cannot be considered sufficiently accurate. Therefore, only a 10-foot drop is allowed and should the scour depth increase beyond 10 feet a second structure may be necessary at a later date. The accuracy of the available analytical methods cannot be relied upon to justify increasing the total drop height. Conversely, it may be desirable to reduce the total drop height at some locations eg. Structure Number 4 approximately 800 feet downstream from Golf Course Road. However, for the same reasons mentioned above, it was decided that a 10-foot drop would be adopted for all structures.

The draw-down water surface profile at the top of each structure will result in higher velocities and increased erosion potential in the approach channel. Simons Li and Associates (1981) recommend that the inlet channel be two times as long as the sum of the approach velocity head and depth, which ranges from 8 to 14 feet for the seven structures. Therefore the inlet channel will have to be 16 to 28 feet long.

A soil cement cut-off wall on the downstream side of the Golf Course Road crossing is shown to be 8 feet deep on the construction plans which also refer to this cut-off wall as "sacrificial". However, this wall can be retained as a grade control structure but only 5 feet of scour is allowed in this analysis in order to protect the cut-off wall from being undermined. If additional scour should occur at this location, a riprap mat may have to be installed to protect the toe of this wall. In any case it should no longer be considered sacrificial but a permanent feature of the road crossing improvements.

Proposed grade control Structure No. 6, immediately downstream from Black's Diversion Channel, will control further erosion at the confluence where a 5-foot cut-off wall at the mouth of Black's Diversion Channel is shown on the construction plans. The exposed portion of this feature suggests that the wall may actually be a sloped (not vertical) structure. It is presently partially exposed and additional scour may not be acceptable. Therefore, the location of structure No.6 is quite appropriate and will provide additional protection to the terminal structure for Black's Diversion Channel.

It is intended that the entire structure will be buried after construction. Consequently, the existing hydraulic and sediment transport characteristics will not be significantly affected. Then, as the arroyo bed degrades, the structures will gradually become exposed. This approach will allow for minimal construction impacts on the overall channel reach because only the area immediately around the structure will be disturbed. Furthermore, the natural (modified by the Unser Bridge) channel processes will be used to allow the channel slope to fluctuate according to varying flow conditions - aggrading or degrading as flows vary and runoff seasons range from wet to dry. If the drop at the structure is left exposed, extensive downstream cut or upstream fill would be required, and the final graded slope will still aggrade or degrade in response to flows and runoff seasons.

Also, because the channel bed is so wide, the possibility of using a narrower grade control structure was investigated. If the channel width is reduced from 250 feet to 100 feet, the 100-year flow depth increases by more than 4 feet which would cause significant backwater effects including deposition of sediment upstream from the structure. A reduction to 150 feet wide would cause a 1.5-foot rise in the water surface which is more acceptable because of the reduced backwater effects. However, any reduction in channel width will require training dikes upstream and downstream of the structure and more concentrated flows at the toe of the structure.

Furthermore, Simons Li and Associates (1981) recommend that concrete structures be designed for 100 cfs per foot of width and 35 cfs per foot for riprap structures. Based on this criterion, the structure width would be 130 feet for concrete and over 300 feet for riprap.

A cost comparison of the reduced width structure as compared to the full width structure indicates a construction cost reduction of approximately 15 percent. Increased maintenance costs primarily for upstream sediment removal may offset any cost savings. Also, adjacent land owners will have to approve the change in water surface elevation, or be compensated for the change. Therefore, full channel width structures, initially buried to existing channel grade were selected for the final cost comparison.

According to the preliminary plans for the proposed recreational trails along Calabacillas Arroyo, it appears that within the study reach a paved trail will be located on top of the arroyo bank (but within the Prudent Line) and only an equestrian trail will be located in the channel bottom. Discussions with City of Albuquerque Planning Department staff confirmed this evaluation. Therefore, only an equestrian trail will have to be accommodated within the area affected by the grade control structure.

Because the trail crossing must be accommodated within this structure, an 8-foot width reduction will still be required. Also, because the maximum slope on the trail will be 10 percent, one side of the structure will require additional fill to sustain this grade.

After the allowable 10 feet of erosion has occurred, the expected 100-year flow trajectory is 32 feet from the top of the drop. Therefore the downstream apron will be 50 feet long to allow for a safety margin, and the remaining downstream channel length modified for inclusion of the equestrian trail will be unlined.

A final comment regarding the trail is that although the structure will be buried and the trail could initially proceed up the center of the channel, subsequent erosion will cause a continually increasing step at the structure location; therefore, the trail will be permanently located along the north side of the structure (for solar access), even when the channel centerline profile will be continuous immediately after construction.

Although the original channel profile and contours will be maintained and/or restored the ultimate profile (after 10 feet of erosion) will be adapted to fit the expected channel shape. Therefore, the approach channel lining as well as the toe apron will have to be transitioned to fit the future channel shape.

In selecting the type of grade control structure, the following factors were considered:

1) Expected flow depths, velocities, and discharges.

2) Soil size and other characteristics.

3) Erosion/deposition processes.

4) Seepage and uplift forces.

5) Local scour potential.

Based on our initial screening of candidate grade control structure designs, four alternatives were selected. These are depicted on Plates 4 and 5 and described as follows:

1. Reinforced concrete - this type of structure was assumed to be concave in the upstream direction with a small notch to confine trickle flows to a single location. A riprap mat would be provided for toe and downstream bank protection.

2. Gabion - the entire structure would consist of three almost vertical sides constructed out of stacked gabion baskets and gabion mats on the upstream and downstream sides of the drop.

3. Soil Cement - this type of structure would be constructed on 8-foot wide, 8-inch lifts with three IH:IV sloped sides and a riprap bottom. If riprap is not readily available gabions or other erosion resistant materials may have to be substituted.

4. Steel sheet piling - use of COR-TEN steel (surface rust only) will provide a low maintenance steel structure which will function similar to the concrete structure. A riprap mat will also be required. However, the sheet piling can be driven into the ground without the need for major excavation.

In all cases, having the riprap or gabions initially covered will preclude the availability of these areas for rodent or snake infestation. However, given the open space nature of the Calabaçillas Arroyo, this type of fauna presently exist and will continue to exist. Therefore, after the riprap or gabion lined areas are exposed, if natural siltation fails to adequately fill the voids in these materials the presence of rodents and/or snakes will have to be accommodated, or a surface application of stiff grout may be required.

Also construction of the structures and after the first few runoff events, when erosional tendencies around the structures become evident, it may be necessary to provide additional bed and bank protection.

SECTION 8

QUANTITIES AND COST ESTIMATES

Unit prices for construction have been taken from City of Albuquerque (COA) Unit Prices whenever possible. Materials not covered by the COA Unit Prices have been estimated based on information from previous jobs and/or local suppliers.

GRADE CONTROL STRUCTURES

Four typical preliminary drop structure designs (gabions, concrete, soil cement and sheet piling structures) were developed to estimate material quantities and associated costs. Plates 4 and 5 show a section and plan views of each structure design. These drawings were used to determine structure sizes, quantity estimates and construction costs. Quantities and associated costs for each drop structure are given in Table 12.

BANK PROTECTION

The quantities and associated costs of the required bank protection is based on Figure 6 and are listed in Table 13.

SECTION 9

RECOMMENDATIONS

MONITORING PROGRAM

RTI strongly recommends that AMAFCA implement an erosion/deposition (channel morphology) monitoring program consisting of periodic aerial photography and comparison of representative cross-sections - similar to sediment range comparisons. The cross-sections shown on Plate 4 could be adopted for this purpose.

GRADE CONTROL STRUCTURES

The four types of grade control structures evaluated in this study appear to be approximately equal in cost and function. However, the sheet piling grade control structure does allow an alternative delayed construction program because the piling can be driven without any major excavation or backfill and the riprap protection at the toe can be added at a later time when erosion has progressed sufficiently to necessitate that action. the sediment analysis indicates that sudden failure of a structure without toe protection is unlikely, except immediately downstream from the Unser Bridge, and in narrow channel sections.

To reduce both the risk of failure as well as construction and maintenance costs, the soil cement structure appears to be the most appropriate selection. This type of structure is relatively easy to construct and the final appearance is quite amenable to the Calabacillas Arroyo landscape. Also, the top 8-foot wide soil cement lift on the north side of the arroyo can be paved with asphalt or other appropriate material for equestrian trails which will reduce the future cost of trail construction.

RTI further recommends that structures No. 1, 2 and 4 be constructed first as mentioned in Section 6 and the remainder can be constructed as and if the need arises. This recommendation is based upon the sediment analysis results which, as previously stated, cannot be considered exact and only the proposed monitoring program can be relied upon.

BANK PROTECTION

Although five locations for bank protection are identified, we recommend that only the following areas be fully protected with the proposed soil cement lining.

1. Near Sta. 27+00 - the S-bend below Black's Diversion Channel.

2. Near Sta. 61+00 - the south bank near Silvergrade Street.

At the remaining locations, we recommend the use of low cost protection such as Kellner Jacks on a test basis. Although Kellner Jacks have been very successful for bank erosion control on the Rio Grande mainstem in wetland "bosque" environments, their rate of success in arroyos remains to be determined. The cost for such treatment will be on the order of $60 per linear foot of jacks required.