FINAL REPORT

Standard Project Flood Analysis and Conceptual Design of Channelization in the Agua Fria River

Submitted to:

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By

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EXECUTIVE SUMMARY

The Flood Control District of Maricopa County has contracted Simons, Li & Associates, Inc. (SLA) to: (1) conduct a Standard Project Flood (SPF) analysis on the Agua Fria River between the confluence with the New River and the confluence with the Gila River; (2) determine conceptual design measures, preliminary cost and quantity estimates between Camelback and Buckeye Roads for the SPF; (3) provide design plans, specifications, and bidding documents for pier protection of Indian School Road Bridge (ISRB); and 4) provide legal descriptions of property to be acquired upstream of ISRB. This report addresses the SPF analysis, conceptual design measures, and preliminary cost and quantity estimates for a flood control project between Camelback and Buckeye Roads.

Much of the preliminary investigation work for existing and proposed flood control project conditions was conducted by SLA and documented in the reports "Hydraulic and Geomorphic Analysis of the Agua Fria River" and "System Analysis and Conceptual Design of Channelization in the Agua Fria River," respectively. These reports should be consulted for background information.

The Standard Project Flood peak on the Agua Fria River near Camelback Road is 142,000 cubic feet per second (cfs). This represents a return flow of between 250 and 300 years. The flood peak of 142,000 cfs was assumed to attenuate very little through the channelized reach between Camelback and Buckeye Roads. This assumption was based on the fact that the 100-year flood peak of 95,000 cfs at Camelback, when routed through the channelized reach, attenuated less than one percent. Thus, 142,000 cfs was used as the design discharge throughout the study reach.

The hydraulic characteristics of the Agua Fria River for the SPF were established using the Army Corps of Engineers HEC-II backwater profile program. Two alternatives were considered including: (1) without siphon at the Roosevelt Irrigation District (RID) flume; and (2) replacing the RID flume with a siphon. Average velocities, hydraulic depths, and top widths varied from 8.5 to 12.7 ft/sec; 5 to 12 feet; and 1,045 to 1,471 feet, respectively throughout the channelized reach for Alternative 1. The range of velocities, hydraulic depths, and top width are similar for Alternative 2.

The proposed bed profiles are such that a minimum of three feet of freeboard exists at all the river crossings except at the Southern Pacific

Railroad (SPRR) bridge. The freeboard at this crossing is 1.7 feet for existing conditions and 1.3 feet for channelized conditions. The SPRR bridge is located at the downstream limit of proposed channelization, and since 3.7 feet of freeboard exists for channelized conditions for the 100-year flood peak of 95,000 cfs, it is not recommended that the crossing be raised.

After establishing the hydraulics, three levels of analysis were conducted to determine sedimentation impacts for the proposed channelization. The three levels of analysis included a qualitative geomorphic, quantitative geomorphic, and mathematical modeling.

The qualitative analysis indicated that the general trend of the channel bed is degradation. The velocities the channel will experience, necessitate protection for stable banks. To prevent headcuts from progressing through the channel, several grade control structures are recommended.

The quantitative analysis agreed with the qualitative analysis in that degradation occurs throughout the study reach. Local-scour analyses at river crossings indicate protection of bridge piers should be implemented at Indian School Road, I-10, Southern Pacific Railroad, Buckeye Road, and the RID flume (if an inverted siphon does not replace the existing flume). All Tucson Electic Power and Salt River Project transmission towers require local-scour protection.

The dynamic equlibrium slope analysis reveals degradation occurs in all portions of the study reach except between Van Buren Road and Buckeye Road, where the channel bed remains nearly constant. Drop structures are recommended at the following locations:

- 500 feet downstream of I-10
- 2,200 feet upstream of McDowell
- 200 feet downstream of Thomas Road
- 100 feet below the RID flume (not necessary for siphon)
- 100 feet below ISRB

The drop structures are located to protect existing crossings, control headcuts from progressing upstream and minimize toe-down depths of levees.

The SLA-developed water- and sediment-routing mathematical model simulated the channel-bed response to the Standard Project Flood for the dynamic equilibrium bed profile. The bed does not aggrade or degrade more than 1.5 feet throughout the channelized reach. This indicates that the channel bed is reason-ably stable once equilibrium conditions are reached.

Conceptual channelization measures recommended for Alternative 1 include:

- Between Camelback Road and Indian School Road, a dike on the west bank that extends 2,000 feet upstream of ISRB terminating in a 700-foot long transverse dike, two transverse dikes 1,600 feet and 600 feet long, and a spur dike on the east overbank to guide flow through ISRB, partial channelization extending 2,800 feet upstream of ISRB, and floodwall protection along the east approach to ISRB.

- Between Indian School Road and Thomas Road, channelization 1,440 feet wide at ISRB that narrows to 920 feet at the RID flume and then expands to 1,100 feet at Thomas Road, backfilling of overbank gravel pits, a drop structure below ISRB, a drop structure below the RID flume, ISRB and RID flume riprap blanket pier protection, Tucson Electric Power (TEP) and Salt River Project (SRP) transmission tower protection and integration of the RID overflow structure into the east levee.

- Between Thomas Road and I-10, channelization 1,100 feet wide from Thomas Road to McDowell Road, expanding to 1,410 feet at I-10, a drop structure 200 feet below Thomas Road, a drop structure 2,200 feet upstream of McDowell Road, protection of TEP and SRP transmission towers, and a siltation basin at the outlet of the I-10 collector channel.

- Between I-10 and Buckeye Road, channelization 1,410 feet wide from I-10 to Van Buren narrowing to 1,100 feet wide at Buckeye Road, a drop structure 500 feet downstream of I-10 bridge, riprap blanket protection of I-10, SPRR and Buckeye Road bridge piers, backfilling of gravel pits 1,500 feet downstream of I-10 and protection of TEP and SRP transmission towers.

Alternative 2 is similar to Alternative 1 except the RID flume is replaced with an inverted siphon. No grade control structure below the RID crossing is necessary for this alternative. Alternative 1 is approximately $740,000 less expensive than Alternative 2; however, it does have some disadvantages including: (1) it creates a large quantity of excess material necessitated by lowering the bed to provide three feet of freeboard at the flume; (2) the flow near the flume becomes critical, causing unstable wave conditions and high flow velocities which could have deterimental effects on the flume's safety; and (3) a grade-control structure downstream of the flume is needed to stabilize the base level of the channel bed.

I. INTRODUCTION

1.1 General

The existing conditions of the Agua Fria between the confluence of the New River and the confluence with the Gila River were analyzed in the report "Hydraulic and Geomorphic Analysis of the Agua Fria River," by Simons, Li & Associates, Inc. (SLA) in September 1983. Analysis and conceptual design measures for a flood control project between Camelback Road and Buckeye Road (SR-85) for the 100-year flood was documented in the report, "System Analysis and Conceptual Design of Channelization in the Agua Fria River" by SLA in October 1983. This report documents the analysis and proposed conceptual design measures associated with the Standard Project Flood (SPF) between Camelback Road and Buckeye Road.

The analysis of the hydraulic and sediment transport conditions includes a three-level approach involving a qualitative geomorphic, quantitative geomorphic and computer model analysis. Based on the analysis, conceptual design measures for the SPF regarding bed slope profiles, channel shapes, drop structure locations and heights, levee heights, bank protection, hydraulic design of bridges, local scour protection of bridge piers, abutments and utility tower protection are addressed. The river response to the design measures was then determined using the water and sediment routing model.

Two alternative flood control projects were considered for this analysis. Alternative 1 considered lowering the channel gradient such that three feet of freeboard exists for all bridge and flume crossings between Camelback Road and Buckeye Road. Alternative 2 considered lowering the channel bed such that three feet of freeboard exists at all bridge crossings except the RID flume, where an inverted siphon would replace the existing flume.

The project area is broken down into four principal reaches. These reaches are described below.

Reach 1. Between Camelback Road and Indian School Road the following measures are considered: a dike on the west bank extending 2,000 feet upstream of ISRB (Indian School Road Bridge) terminating in a transverse dike 700 feet long, two transverse dikes 1,600 feet and 600 feet long, and a spur dike on the east overbank to guide the flow through ISRB, partial channelization extending 2,800 feet upstream of ISRB and floodwall protection 3,400 feet along the east approach to ISRB.

Reach 2. This channel reach consists of channelization 1,440 feet wide at ISRB to 920 feet at the Roosevelt Irrigation District (RID) flume. The channel then transitions to 1,100 feet wide at Thomas Road. The new alignment in this reach will require significant backfilling of gravel pits on the overbanks.

Reach 3. The third reach contains channelization as proposed by Dibble and Associates from Thomas Road to the proposed McDowell Road Bridge, which is 1,100 feet wide throughout. The channel expands to 1,410 feet wide at the I-10 bridge. The I-10 collector channel empties into the Agua Fria just north of I-10. The I-10 collector channel conveys flood flows from 27th Avenue to the Agua Fria, draining an urbanized area of about 45 square miles.

Reach 4. Reach 4 starts with a 1,410 foot wide channel from I-10 to Van Buren Street transitioning to 1,100 feet wide at the Southern Pacific Railroad Crossing and Buckeye Road. A large gravel mining operation exists approximately 1,500 feet downstream of I-10 and will have to be moved before channelization proceeds downstream of I-10.

Attached to this report are Plates 1 through 4 showing the proposed channelization. Channelization and other channel modifications were not considered for the Agua Fria below Buckeye Road.

1.2 Scope of Work

To assess the response to channelization the following scope of work was performed.

1. Data were collected and assembled. This involved gathering more information on the approaches to ISRB and more information concerning the RID flume.

2. Average hydraulic conditions of the Standard Project Flood were established on the Agua Fria between the confluence with the New River and the confluence with the Gila River by applying the U.S. Army Corps of Engineers HEC-II backwater profile program for both existing and proposed channelization.

3. A qualitative geomorphic analysis of expected responses to the Standard Project Flood with channelization measures between Camelback Road and Buckeye Road was conducted. This involved assessing the short-term and long-term response of the channel bed.

4. A quantitative geomorphic study to determine the bed response to the SPF was conducted. Specifically the following analyses were completed:

   a. Computation of local scour around transmission towers, bridge piers, flume piers, and abutments within the channelized reach between Camelback and Buckeye Roads.

   b. Determination of general regional scour caused by the SPF at constricted areas within the channelized reach.

c. Computation of armor control limits for the SPF and comparison of armor control slopes with dynamic equilibrium slopes to determine optimum locations of grade-control structures.

d. Summation of scour components to determine total scour at all bridge, flume and utility crossings within the proposed channelization.

5. SLA executed the water and sediment routing model for the SPF with proposed channelized conditions to assess bed response and evaluate necessary toe-down depths of levees.

6. A determination of the average annual aggradation/degradation rates along the Agua Fria River was made. This was accomplished by establishing sediment rating curves along the river and using the rating curves to determine sediment transport volumes for reaches along the river for various return flows. The average annual sediment transport rate was computed by incremental weighting of the probability of a flood occurring within a year and then summing the sediment transport volumes for each reach of the river to assess potential aggradation/degradation response.

7. SLA developed hydraulic design measures necessary to pass the Standard Project Flood through Indian School Road, McDowell Road, I-10, Southern Pacific Railroad and Buckeye Road bridge crossings and provide three feet of freeboard as required by the U.S. Army Corps of Engineers.

8. SLA developed conceptual design measures necessary for channelization between Camelback and Buckeye Roads. This included levee heights, toe-down depths, bank protection of levees, bridge pier protection, utility protection measures, grade-control locations, channel gradients, and allowable sand and gravel mining locations.

9. Two conceptual design alternatives to pass the SPF through the RID flume are provided. This involved (1) lowering the channel bed and (2) constructing an inverted siphon.

10. Cost and quantity estimates for all conceptual design measures between Camelback and Buckeye Roads are provided. This included all design measures at the RID flume crossing.

11. Ten copies of the draft final report summarizing all data assumptions, analyses, results and conclusions of the study for review by the FCD of Maricopa County have been delivered.

12. Ten copies of a final report incorporating all comments and suggestions of the FCD of Maricopa County and other reviewing agencies has been provided.

13. Using the results of the systems analysis and conceptual design, SLA has provided the FCD of Maricopa County with legal descriptions of land required for a flood control project between ISRB and Camelback Road.

14. SLA provided the FCD of Maricopa County a map delineating the new 100-year and SPF flood plains with suggested channelization measures.

1.3 Sources of Information

The following is a list of information used for the system analysis of the Agua Fria between the confluence with the Gila River and the confluence with the New River.

Aerial Photos

1936 coverage from Camelback Road to Van Buren. (scale 1"=600').

1/16/63 coverage of the Agua Fria from the confluence with the New River to the confluence with the Gila River. (scale 1"=500').

1/74 coverage of the Agua Fria from the confluence with the New River to the confluence with the Gila River (scale 1"=1000').

3/7/73 coverage of the Agua Fria from Northern Avenue to the confluence with the Gila River (scale 1"=1000').

2/20/80 coverage of the Agua Fria from Northern Avenue to the confluence with the Gila River (scale 1'=600').

Topographic Maps

August 31, 1981, topographic maps of the Agua Fria from Glendale Avenue to McDowell Road (scale 1"=100').

May 15, 1981, topographic maps of the Agua Fria from McDowell Road to the confluence with the Gila River (scale 1'=200').

Survey Information

Land surveys conducted by Samer, Lahlum and Associates, Inc. June 1982 and February 1983.

Bridge Plans

1969 plans for construction of Indian School Road Bridge. Includes boring samples at the bridge site.

1978 as-built plans of east approach to Indian School Road Bridge.

1983 design plans for approaches to Camelback Road Bridge.

1977 plans for addition of the third and fourth lanes on the Indian School Bridge.

3/4/26 as-built plans of the Southern Pacific Railroad Bridge crossing.

1969 design plans for the Buckeye Road Bridge crossing.

1980 as-built bridge plans for I-10.

1983 design plans for the McDowell Road Bridge crossing sheets 1-10.

1983 preliminary bridge plans for Camelback Road Bridge.

Site Visits

2/4/82 site visit of a backhoe pit exposed 800 feet downstream of Indian School Road Bridge by Maricopa County Highway Department.

6/82 site visit of excavation around one of the RID flume piers.

2/83 site visit to gather sediment samples from Waddell Dam to the confluence with the Gila River on the Agua Fria and gather several surface material samples on the New River.

4/83 site observations of backhoe test pits dug for SLA to assess subsurface soil conditions in the Agua Fria and New Rivers.

Soil Reports

Geotechnical Investigation Report "Channelization-Agua Fria River Thomas Road, and I-10, Maricopa County, Arizona," by Sergent, Hauskins and Beckwith, June 9, 1982.

Geotechnical Report for "Camelback Road Bridge Crossing of Agua Fria River, Maricopa County, Arizona," by Engineers Testing Laboratory, April 24, 1981.

Geotechnical Investigation Report "Indian School Road Bridge at Agua Fria River, Maricopa County, Arizona," by Sergent, Hauskins and Beckwith, September 24, 1980.

Geotechnical Investigation Report "Bell Road Bridge at Agua Fria River Maricopa County, Arizona," by Sergent, Hauskins and Beckwith, October 14, 1980.

"Pier Scour Flume Piers in the Agua Fria, Maricopa County, Arizona," by Engineers Testing Laboratories prepared for Roosevelt Irrigation District, Buckeye, Arizona, April 15, 1980.

Hydraulic and Hydrologic Reports

"Hydrology of the Agua Fria River," by the L.A. Corps of Engineers, April, 1981.

"Hydraulic Analysis of Agua Fria Channel McDowell Road to Thomas Road," Maricopa County, Arizona, by Lowry and Associates, October 15, 1982.

"Agua Fria River Study-1982" prepared for Flood Control District of Maricopa County by Willdan Associates.

"New River and Phoenix City Streams, Arizona," Design Memorandum No. 2 Hydrology Part 1, U.S. Army Corps of Engineers Los Angeles District, October 1974.

"Hydraulic and Geomorphic Analysis of the Agua Fria River," prepared for Flood Control District of Maricopa County by Simons, Li & Associates, Inc., September 13, 1983.

"System Analysis and Conceptual Design of Channelization in the Agua Fria River," prepared for the Flood Control District of Maricopa County by Simons, Li & Associates, Inc., October 10, 1983.

Utility Plans

The following agencies were contacted in regard to utility crossings in the channelized reach of the Agua Fria:

1. Tucson Electric Power Company
2. Salt River Project
3. El Paso Gas Company
4. Arizona Public Service
5. Mountain Bell
6. Roosevelt Irrigation District
7. Southern Pacific Pipeline Incorporated
8. City of Avondale
9. City of Phoenix
10. Town of Goodyear
11. Department of Energy, Western Area Power Administration

Hydrographs

100-year flood event downstream of the confluence with the New River on the Agua Fria, extracted from the L.A. Corps of Engineers printout dated March 7, 1981.

10- and 100-year flood hydrographs for the Tenth Street Drain at the Arizona Canal, Arizona Canal Diversion Channel at Skunk Creek, Cudia City Wash at Arizona Canal, Dreamy Draw at Arizona Canal, and Northern Avenue at Arizona Canal, extracted from "Sediment Data Report for Arizona Canal Diversion Channel," final report-draft, Boyle Engineering Corporation, November, 1981.

RID Flume

"The Agua Fria River Flume Crossing, 5959 Feet Long, an Interesting Feature" by M.E. Ready and A.V. Saph, Jr. 1929.

1929 flume as-built plans of Agua Fria crossing.

January 1984 survey by Samer, Lahlum and Associates determining elevations at top of pier footings.

Cross-sectional data of RID canal east of the Agua Fria River crossing.

II. HYDROLOGY

The standard project flood peak is used as the design discharge for channelization measures in the Agua Fria River between the confluence with the New River and the confluence with the Gila River. Table 2.1 presents flood peak information for existing conditions throughout the Agua Fria River from Waddell Dam to the confluence with the New River as reported in the 1981 Corps of Engineers report entitled "Hydrology in the Agua Fria River." For existing conditions, the SPF peak attenuates from 142,000 cfs at Camelback Road to 131,000 cfs at the USGS gage just below Buckeye Road.

The SPF peak associated with the proposed channelization will not attenuate as rapidly as for existing conditions. It was assumed for this study that the SPF peak of 142,000 cfs at Camelback Road does not change throughout the channelized areas. This assumption was based on the fact that the 100-year peak discharge of 95,000 cfs at Camelback Road, when routed through channelized areas between Camelback Road and Buckeye Road, reduced less than one percent. The reasons for the increased efficiency include (1) a more uniform cross section which has a lower flow resistance than natural conditions, (2) a narrower cross section which has lower channel storage and higher velocities, and (3) limiting the in-stream gravel mining to removal of bars that develop in the channelized reach and by not allowing gravel mining below proposed channel grades reduces the channel storage. Thus the peak discharge of 142,000 cfs at Camelback Road was used for analysis and conceptual design measures between Camelback Road and Buckeye Road.

Table 2.1. Design Flood Discharge - Agua Fria River from Waddell Dam to Gila River for Existing Conditions.

Location Along the Agua Fria River	Peak Discharge (cfs)					
	SPF	500-year Flood	100-year Flood	50-year Flood	25-year Flood	10-year Flood
Inflow - Waddell Dam	158,000	190,000	135,000	110,000	90,000	60,000
Outflow - Waddell Dam	158,000	182,000	135,000	110,000	90,000	60,000
Bell Road	151,000	182,000	115,000	87,000	60,000	37,000
U/S New River Confluence	135,000	177,000	90,000	66,000	48,000	30,000
D/S New River Confluence	142,000	184,000	95,000	69,000	50,000	32,000
Camelback Road	142,000	184,000	95,000	69,000	50,000	31,000
Indian School Road	140,000	183,000	94,000	69,000	49,000	30,000
McDowell Road	137,000	182,000	91,000	68,000	48,000	29,000
I-10 Freeway	135,000	181,000	91,000	68,000	48,000	29,000
Avondale	131,000	179,000	90,000	67,000	47,000	28,000
Gila River	130,000	179,000	89,000	67,000	47,000	27,000

Source: U.S. Army Corps of Engineers, Los Angeles, Hydrology of the Agua Fria River, 1981.

III. HYDRAULICS

3.1 General

Backwater profiles were computed for existing conditions and proposed channelized conditions between Camelback Road and Buckeye Road. Existing above-grade channel crossings in this stretch of the Agua Fria include Buckeye Road, Southern Pacific Railroad, I-10, Indian School Road and the RID flume. Proposed bridge crossings include McDowell Road and Camelback Road.

Profiles were computed for the standard project flood for Alternative 1, which will be referred to as the without-siphon option at the RID crossing, and for Alternative 2, which will be referred to as the with-siphon option at the RID crossing. The proposed cross-sectional shape of the channel is trapezoidal with 3:1 side slopes and a bottom width varying from 1,600 feet to 920 feet. The heights of levees were extended to contain the SPF with three feet of freeboard. The proposed bed profile was the profile recommended in the SLA report entitled, "System Analysis and Conceptual Design of Channelization in the Agua Fria River." Figures 3.1 and 3.2 compare the proposed bed profile with the existing bed profile for Alternatives 1 and 2, respectively.

3.2 Flow Resistance

A Manning's roughness coefficient of 0.030 was utilized for the main channel flow resistance for the proposed channelization to determine the 100-year and SPF flood plain, levee heights and low-chord elevation of bridges. For sediment transport analysis the Manning roughness coefficient was lowered to 0.025 in the main channel. The smaller Manning "n" value produces larger flow velocities and more conservative (increased) estimates of sediment transport rates.

Overbank roughness coefficients were not of concern in the channelized reaches as all of the flows were contained within the levees for the SPF flood. For the unchannelized reaches, the Agua Fria upstream of Indian School Road and downstream of Buckeye Road to the confluence with the Gila River, the Manning roughness coefficients adopted were those used in the Corps of Engineers 1981 HEC-II input data. The Manning's resistance coefficient in the main channel was 0.035 and in the overbanks the coefficient ranged 0.04 to 0.07.

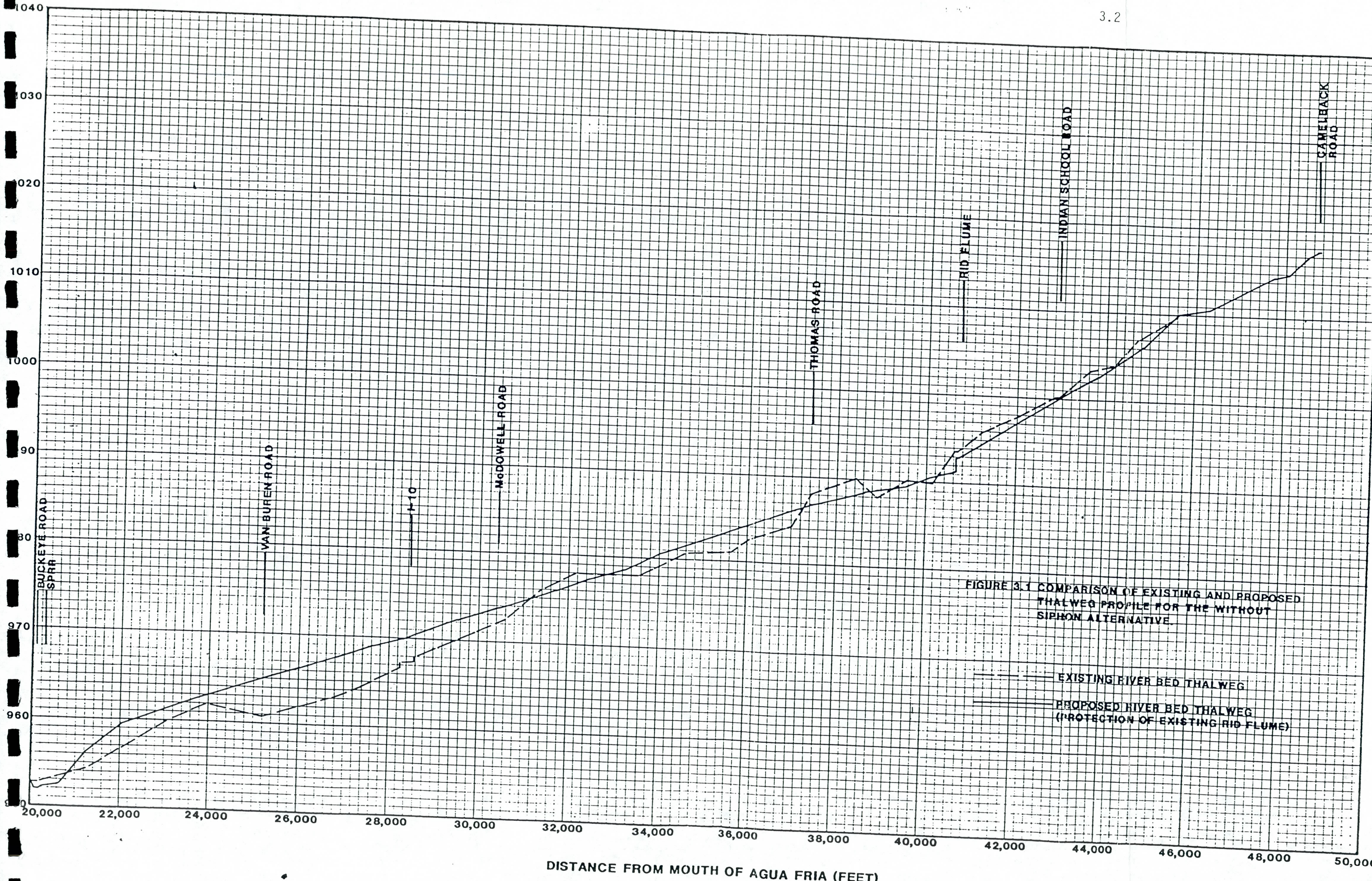


FIGURE 3.1 COMPARISON OF EXISTING AND PROPOSED THALWEG PROFILE FOR THE WITHOUT SIPHON ALTERNATIVE.

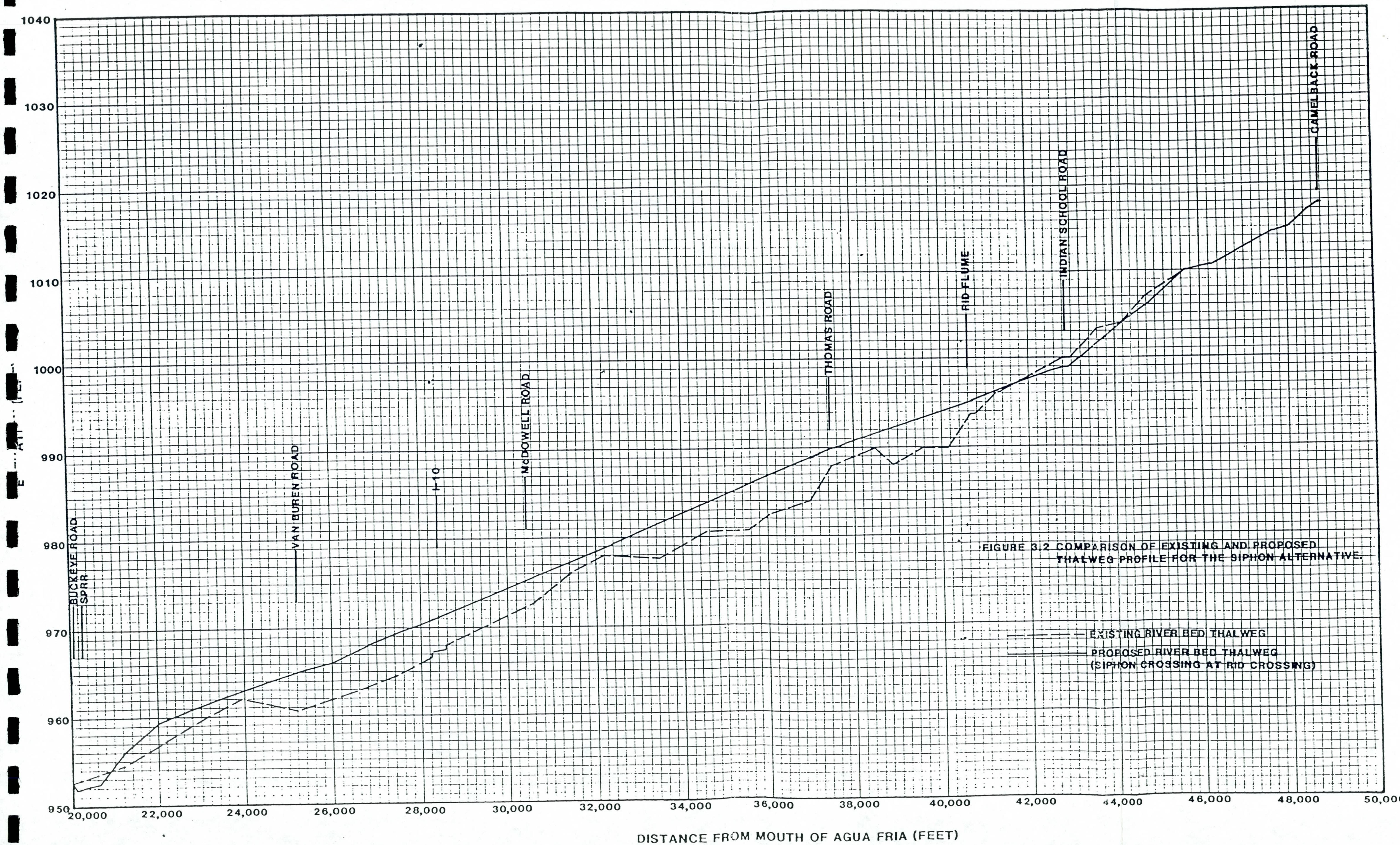


FIGURE 3.2 COMPARISON OF EXISTING AND PROPOSED THALWEG PROFILE FOR THE SIPHON ALTERNATIVE.

3.3 Results of Hydraulic Analysis

The hydraulic characteristics change considerably from existing and proposed channelized conditions between Camelback Road and Buckeye Road. Table 3.1 compares the hydraulic parameters of velocity, flow width, hydraulic depth and discharge in the study reach. It is readily apparent from this comparison that the flow widths are reduced and velocities are substantially increased for channelized conditions. By reducing the effective flow width and increasing the velocities, the sediment transport rates will increase.

Several flow breakout areas occur downstream of Buckeye Road for both existing and proposed channelization conditions for the Standard Project Flood. One of the breakout areas occurs 500 feet upstream of Broadway Road on the east overbank and directly below Broadway Road on the west overbank. Presently, the east and west overbank flows will go through fields and undeveloped land to the Gila River. The other breakout area occurs just downstream of Buckeye Road on the west overbank. This flow goes through the developed area of Avondale about a half a block west of Dysart Road from Buckeye Road to Harrison Drive. The breakout area just downstream of Buckeye Road also exists for the 100-year flood.

Between Buckeye Road and Indian School Road no overbank flow occurs for channelized conditions as levees are extended to contain the SPF. Three feet of freeboard is provided at all existing and proposed river crossings except the Southern Pacific Railroad crossing and Camelback Road. Table 3.2 summarizes the freeboard elevations at all crossings for existing and proposed channelized conditions.

Three feet of freeboard for the SPF does not exist at the Southern Pacific Railroad crossing for both existing and proposed channelization conditions. The freeboard at the crossing is 1.7 feet for existing conditions and 1.3 feet for channelized conditions. The lower freeboard height for channelized conditions results because the peak discharge does not attenuate for proposed channelization as it does for existing conditions as explained in Chapter II.

It is important to note that the approaches to the railroad crossing are high enough to force the entire Standard Project Flood peak underneath the bridge for both existing and channelized conditions; therefore, three feet of freeboard will not exist for either condition. Further, the potential bed response in the general vicinity of the Southern Pacific Railroad (SPRR) crossing is slight aggradation to near equilibrium, as determined in the

Table 3.1. Comparison of Hydraulic Conditions for the SPF for Existing and Channelized Conditions.

Reach	Discharge			Average Hydraulic Depth			Average Flow Velocity			Average Top Width		
	Exist.	Channel Alternate 2	Channel Alternate 1	Exist.	Channel Alternate 2	Channel Alternate 1	Exist.	Channel Alternate 2	Channel Alternate 1	Exist.	Channel Alternate 2	Channel Alternate 1
Camelback Road to Ind. Sch. Rd.	142,000 (95,360)	142,000 (135,150)	142,000 (135,150)	4.6 (7.3)	5.0 (8.4)	5.0 (8.2)	3.6 (5.2)	8.5 (9.0)	8.6 (9.1)	8,512 (2,512)	3,328 (1,811)	3,327 (1,814)
Ind. Sch. Road to RID Flume	137,000 (96,650)	142,000	142,000	5.1 (12.9)	12.4	10.6	3.8 (7.7)	8.7	10.3	7,060 (973)	1,319	1,306
RID Flume to Thomas Road	137,000 (85,700)	142,000	142,000	5.3 (9.1)	11.7	11.7	4.9 (8.8)	11.6	11.7	5,265 (1,070)	1,045	1,038
Thomas Rd to I-10	137,000 (70,600)	142,000	142,000	4.35 (6.1)	10.2	11.0	5.5 (8.9)	11.8	10.9	5,725 (1,300)	1,180	1,185
I-10 to Van Buren Steet	135,000 (99,500)	142,000	142,000	4.9 (9.1)	9.8	10.3	5.9 (9.9)	9.8	9.4	4,698 (1,104)	1,471	1,472
Van Buren St. to Buckeye Rd.	131,000 (109,300)	142,000	142,000	5.0 (9.3)	8.8	10.7	8.0 (9.5)	12.7	10.6	3,292 (1,237)	1,255	1,255

Values in parentheses are the average hydraulic conditions that occur in the main channel. When two values are not given, all the flow is contained within the main channel.

Alternative 1 considers no siphon at the RID flume crossing
Alternative 2 considers a siphon at the RID crossing

Table 3.2. Summary of Freeboard Heights at all Agua Fria River Crossings for SPF.

Crossing	Freeboard: Existing Condition (ft)	Freeboard: Channelization Alternative 1 (ft)	Freeboard: Channelization Alternative 2 (ft)
Buckeye Road	4.9	4.2	4.2
Southern Pacific Railroad	1.7	1.3	1.3
I-10	7.7	7.7	7.2
McDowell Road	---	7.5	6.4
RID Flume	0[1]	3.7	---
ISRB	0[2]	4.7	3.3
Camelback Road	---	0	0

[1] pressure and weir flow

[2] pressure flow

report "System Analysis and Conceptual Design of Channelization in the Agua Fria River." Thus Excavation of the channel bed in the area will only result in temporary lowering of the water surface. Finally, since the SPRR crossing is at the end of proposed channelization, and flowage easements will be purchased for the existing 100-year flood plain downstream of the SPRR crossing and 3.7 feet of freeboard exist for the 100-year flood for channelized conditions, it is not recommended that the SPRR crossing be raised.

Three feet of freeboard do not exist at Camelback Road for the SPF; however, channelization does not extend to Camelback Road. The bridge does have three feet of freeboard for the 100-year design discharge.

IV. QUALITATIVE GEOMORPHIC ANALYSIS

4.1 General

The historical changes of the Agua Fria River in the study reach were documented in the report, "Hydraulic and Geomorphic Analysis of the Agua Fria River" by SLA, September 13, 1983. The Agua Fria River in the study reach is a braided ephemeral stream, and is quite unstable. The river flows in a canyon reach for several miles below Waddell Dam before it enters the valley and exhibits its braided characteristics.

The thalweg of the river has dropped between 0.5 and 3 feet between Camelback Road and the confluence with the Gila River from 1973 to 1981. Not only has the thalweg dropped, but the entire cross section has lowered.

The degradation trend can be attributed to several factors which include: encroachment of the flood plain by urbanization, gravel mining activities, and the trapping of upstream sediments by Waddell Dam. A complete summary of the qualitative analysis can be found in the above referenced report.

From the report "System Analysis and Conceptual Design of Channelization in the Agua Fria River" by SLA, October 10, 1983, the following conclusions were made regarding river response to channelization measures for the 100-year flood. Channelization will further encroach the flood plain. The expected long-term channel bed response is degradation for all the channelized reaches, except between Van Buren and Buckeye Roads. The reach between Van Buren and Buckeye Roads is in approximate equilibrium.

Armor layer material is in evidence on the bed surface from Bethany Home Road upstream to Waddell Dam on the Agua Fria. Should the armor layer develop downstream to Camelback Road, the sediment supply from the channel bed will be drastically reduced. Consequently, the supply of sediment being transported into the channelized reaches will be greatly reduced, further increasing the degradation potential in the channelized reaches.

With the large degradation potential and no apparent natural grade controls in the subsurface stratum, man-made grade controls will be necessary to stabilize the channel reaches. Details of the drop structures proposed to serve as grade controls are discussed in Chapters V and VII.

4.2 Qualitative Evaluation of Proposed Channelization

For future channelization conditions for the SPF the channel will be narrowed appreciably from the existing condition. Accompanying the narrowing of the channel will be increased flow velocities. Aggradation and degradation response within a channel is related to sediment transport capacity which in turn is directly proportional to top width and proportional to the velocity to approximately the fourth power. Changes in flow depth, except those directly related to velocity, have a smaller influence on sediment transport. The potential for aggradation or degradation can be qualitatively evaluated by comparing the top width and velocity from reach to reach. A reach is defined as a lumping together of cross sections with similar hydraulic properties. Figure 4.1 gives the reach definitions in terms of cross sections and river distance for the study area.

Short- and long-term responses can be evaluated using velocity and top width comparison. By comparing these parameters with the reach immediately upstream, short-term responses can be estimated. Long-term responses are determined using a single upstream reach, assumed to be in equilibrium and not expected to experience changes in sediment transport rates in the future, as a sediment supply reach. For the long-term response, sediment transport capacities of all downstream reaches are compared with the supply reach, rather than the reach immediately upstream. The reasoning behind the two types of comparison are in the short term only the closest reach immediately upstream will significantly impact the downstream reach; however, over a longer period the system adjusts to meet the supply of the upstream reach that is in equilibrium.

The short-term channel bed responses of Alternatives 1 and 2 are summarized in Table 4.1. The channelized areas between ISRB and the RID flume, and I-10 and Van Buren Road, show slight tendencies to aggrade in the short term. The reason for slight aggradation between ISRB and the RID flume is the velocities are greater in the reach upstream of ISRB because of the relatively steep gradient. The short-term aggradation response between I-10 and Van Buren is the result of the channel velocities being slower in this wider reach than the velocities in the narrower reach from I-10 to Thomas Road.

The short-term responses are not indicative of the long-term responses. Using the existing cross sections upstream of Camelback Road as the long-term sediment supply reach to compare with the channelized reaches downstream of

Section Number	River Distance From Confluence With Gila River (ft)	Reach Number	River Distance	Features
7.2	715			
13.7	1,370			
20.1	2,000			
26.9	2,690			Southern Avenue
35.2	3,520			
44.6	4,450			
53.6	5,350			
61.9	6,180			
70.4	7,020			
75.0	7,490			Broadway
82.6	8,250			
93.8	9,370			
103.9	10,380			
117.3	11,725			
121.4	12,135			
130.6	13,055			
135.4	13,530			Lower Buckeye Road
151.4	15,125			
171.4	17,125			
181.6	18,145			
190.2	19,010			
200.2	20,010		20,285	
201.4	20,285			
201.8	20,385			Buckeye Road
202.0	20,470			So. Pacific RR Bridge
202.5	20,500			
206.3	20,880			
211.6	21,405	7		
219.6	22,205			
227.8	23,030			
236.0	23,850			
244.1	24,660			
254.3	25,450		25,850	Van Buren
262.3	26,250			
270.3	27,050	6		
278.3	27,850			
281.5	28,540		28,665	I-10
283.5	28,790			
293.5	29,790			
298.0	30,620			McDowell Road
300.0	30,710			
312.5	31,960	5		
319.6	32,670			
327.6	33,470			
335.5	34,265			
347.5	35,460			
354.8	36,190		36,631	

Figure 4.1. Schematic diagram of reaches in channelized area.

Section Number	River Distance From Confluence With Gila River (ft)	Reach Number	River Distance	Features
363.6	37,072			
369.6	37,672			Thomas Road
375.6	38,272			
381.6	38,872	4		
390.6	39,772			
395.1	40,222			
399.5	40,667			
403.7	40,860		40,868	RID flume
403.9	40,876			
405.5	41,031			
410.5	41,531	3		
415.5	42,031			
422.0	42,676			
427.0	43,046		43,086	ISR Bridge
427.4	43,126			
433.5	43,976	2		
439.5	44,526			
444.8	45,056		45,341	
452.6	45,841			
459.5	46,531	1		
466.6	47,241			
473.3	47,911			
476.9	48,271			
480.7	48,681			
483.0	48,911			
483.3	48,981		49,121	Camelback Road
486.0	49,261			
490.9	47,741			
496.7	50,321			
501.5	50,976			
510.3	51,681			Confluence, New River
520.2	52,671			
531.2	53,771			
544.7	55,121			
558.6	56,511			
568.7	57,521			
580.2	58,666			
589.3	59,576			

Figure 4.1 (continued)

Table 4.1. Expected Short-Term Qualitative Response of Reaches Based on HEC-II Analysis.

Reach	Change in Top Width Alternate 1	Change in Top Width Alternate 2	Change in Velocity Alternate 1	Change in Velocity Alternate 2	Overall Response Alternate 1	Overall Response Alternate 2
1	Same	Decrease	Increase	Increase	Degrade	Degrade
2	Decrease	Decrease	Increase	Increase	Degrade	Degrade
3	Decrease	Decrease	Increase	Same	Degrade	Aggrade
4	Decrease	Decrease	Increase	Increase	Degrade	Degrade
5	Increase	Increase	Decrease	Decrease	Slight Aggrade	Equilibrium
6	Increase	Increase	Decrease	Decrease	Aggrade	Aggrade
7	Decrease	Decrease	Increase	Increase	Degrade	Degrade

Reach 1 Camelback Road to 2,200 ft upstream of Indian School Road
Reach 2 2,200 ft upstream of Indian School Road to Indian School Road
Reach 3 Indian School Road to RID flume
Reach 4 RID flume to Thomas Road
Reach 5 Thomas Road to I-10
Reach 6 I-10 to Van Buren
Reach 7 Van Buren to Buckeye Road

Camelback, the expected bed responses for the SPF are summarized in Table 4.2. The long-term response for all the reaches is to degrade for the Standard Project Flood. With this degrading tendency and no apparent natural grade controls in the subsurface stratum, man-made grade controls will be necessary to control degradation.

Table 4.2. Expected Long-Term Qualitative Response of Reaches Based on HEC-II Analysis.

Reach	Change in Top Width Alternate 1	Change in Top Width Alternate 2	Change in Velocity Alternate 1	Change in Velocity Alternate 2	Overall Response Alternate 1	Overall Response Alternate 2
1	Same	Decrease	Increase	Increase	Degrades	Degrades
2	Decrease	Decrease	Increase	Increase	Degrades	Degrades
3	Decrease	Decrease	Increase	Increase	Degrades	Degrades
4	Decrease	Decrease	Increase	Increase	Degrades	Degrades
5	Decrease	Decrease	Increase	Increase	Degrades	Degrades
6	Decrease	Decrease	Increase	Increase	Degrades	Degrades
7	Decrease	Decrease	Increase	Increase	Degrades	Degrades

Reach 1 Camelback Road to 2,200 feet upstream of Indian School Road
Reach 2 2,200 feet upstream of Indian School Road to Indian School Road
Reach 3 Indian School Road to RID Flume
Reach 4 RID Flume to Thomas Road
Reach 5 Thomas Road to I-10
Reach 6 I-10 to Van Buren
Reach 7 Van Buren to Buckeye Road

V. QUANTITATIVE ANALYSIS

The quantitative analysis consists of (1) computing local scour depths at obstructions in the flow, such as bridge piers, bridge abutments, transmission towers, etc., (2) computing the general regional scour depths at all contractions in the flow, and (3) determining the aggradation/degradation response of the channel bed. The following sections discuss the results of the quantitative analysis.

5.1 Local Scour at Bridge Crossings and Transmission Towers

As explained in the previous reports, local scour was computed at the bridge sites using Shen and Neil's methods and compared to determine which of the two methods yielded the most reasonable local scour depth for bridge piers. Shen and Neil's equations were empirically developed from extensive test data on sand-bed channels and will provide reasonable approximations of local scour depths on the Agua Fria River. Since the suggested channelization involves levees on both sides of the river, the bridge abutments will not be protruding into the flow, therefore any scour that occurs near the bridge abutments will be from the general degradation response of the bed. Consequently, only general scour at abutments was considered in the analysis.

For all local scour computations two feet of width was added to either side of the piers to account for accumulation of debris. Also considered in the analysis was any flow skew potential that might result from channelization at bridge crossings. Where possible, flow skew was avoided in the design; however, because of the alignment of existing bridge piers, this was not always possible. Hydraulic conditions at each of the bridge and flume crossings were determined using HEC-II.

For Alternative 1 at each of the seven crossings, Table 5.1 summarizes the proposed bed elevation, the depth bridge piers extend below the proposed bed elevation, the dimensions of bridge piers, spacing between piers, span length of the bridge, skew angle expected, scour depths for the SPF discharge of 142,000 cfs computed using Shen and Neil's methodologies, and the adopted local scour expected at the bridge. Table 5.2 summarizes the same information for Alternative 2.

Table 5.1. Summary of Local Scour Depths Expected at Bridge Crossings for the SPF with Proposed Channelization for Alternative 1, Without Siphon.

Bridge Crossing	Proposed Bed Elevation	Approximate Depth of Supports Below Proposed Bed (ft)	Dimensions of Bridge Piers	Spacing Between Piers (ft)	Bridge Span (ft)	Skew Angle Considered (degrees)	Local Scour Shen (ft)	Local Scour Neil (ft)	Adopted Local Scour Value (ft)
Camelback Road	1,017.4	70	4' diameter	115	1,725	0	13.6	13.1	13.4
Indian School Road	1,000.0	Piers 1-12 25' Piers 13-17 70'	1'8" wide Piers 1-12	90	1,620	10	25.8	22.1	24.0
			4' diameter Piers 13-17						
Roosevelt Irrigation District flume	992.9	21-29	4' wide	72	1,008	0	19.0	16.4	17.7
McDowell Road	974.0	70	5' diameter	125	1,250	0	17.5	16.2	17.0
I-10	970.0	25	3.3' diameter	75	1,500	0	16.5	14.2	15.4
Southern Pacific Railroad	952.0	30	6'8" pier deck support section	153	1,200	0	19.3	18.8	19.0
			2' ballast support section	15					
Buckeye Road	951.8	28	3' wide	80	1,200	0	14.5	13.7	14.1

Table 5.2. Summary of Local Scour Depths Expected at Bridge Crossings for the SPF with Proposed Channelization for Alternative 2 With Siphon.

Bridge Crossing	Proposed Bed Elevation	Approximate Depth of Supports Below Proposed Bed (ft)	Dimensions of Bridge Piers	Spacing Between Piers (ft)	Bridge Span (ft)	Skew Angle Considered (degrees)	Local Scour Shen (ft)	Local Scour Neil (ft)	Adopted Local Scour Value (ft)
Camelback Road	1,017.4	70	4' diameter	115	1,725	0	13.7	13.2	13.5
Indian School Road	999.0	Piers 1-12 25' Piers 13-17 70'	1'8" wide Piers 1-12 4' diameter Piers 13-17	90	1,620	10	25.8	22.1	24.0
McDowell Road	975.5	70	5' diameter	125	1,250	0	18.0	16.4	17.2
I-10	971.0	26	3.3' diameter	75	1,500	0	16.5	14.2	15.4
Southern Pacific Railroad	952.0	30	6'8" pier deck support section 2' ballast support section	153 15	1,200	0	19.3	18.7	19.0
Buckeye Road	951.8	28	3' wide	80	1,200	0	14.5	13.7	14.1

With the suggested channelization several transmission towers will be inside the levees. Both the Salt River Project and the Tucson Electric Power Company have towers within the levees that will be subjected to local scour. Plates 1 through 4 attached with this report show locations of towers within the channelized reach.

The Salt River Project has 4 towers within the channelization reach near Thomas Road. Table 5.3 summarizes for each tower for the with-siphon alternative the obstruction width of each footing, the SPF flow velocity and depth, the elevation of the bottom of the footing, the SPF local scour depth as computed using Shen and Neil's equations, the adopted local scour, the approximate ground elevation after channelization in the vicinity of the tower and the expected elevation after scour. All towers will require some type of protection as the scour depths combined with the proposed channelization would otherwise undermine the towers. The local scour depths for the without-siphon alternative are similar.

Tucson Electric Power Company has 13 towers within the channelized reach. Table 5.4 summarizes local scour depths for the 13 towers for the SPF. All of these towers will require protection.

5.2 General Regional Scour

General regional scour at contractions occurs because the effective flow area is reduced, thus increasing the local velocity and bed shear stress. Hence, there is an increase in stream power at the contraction and more bed material is transported through the contracted section than is transported into the section. As the bed level is lowered, velocity decreases, shear stress decreases and equilibrium is restored when the sediment transport rate from the contracted section is equal to the incoming rate.

Two areas where the contraction scour is the most severe within the study reach are near the proposed Camelback Road bridge and Indian School Road bridge. At these locations the effective flow width reduces appreciably. The general regional scour at Camelback Road and Indian School Road was computed to be 1.5 ft. and 2.5 ft., respectively, for the SPF. Thus toe-down protection in these areas must be increased to reflect this additional scour potential. For the remaining channelization, general regional scour becomes negligible due to gradual expansion and contraction of the proposed alignment.

Table 5.3. Local Scour Around Towers - Salt River Project for With-Siphon Channelization Alternative.

					Local Scour				
Tower Number	Obstruction Width	Flow Velocity (ft/sec)	Flow Depth (ft)	Elevation at Bottom of Footing (ft)	Shen (ft)	Neil (ft)	Adopted Scour (ft)	Approximate Channelized Ground Elevation	Elevation After Scour
58	3'	11.2	10.2	987	8.3	10.0	9.2	990.5	981.3
59	3'	11.1	11.0	987	8.2	9.9	9.1	985.5	976.4
60	3'	11.1	11.0	979	8.3	9.9	9.1	984.0	974.9
61	3'	11.1	11.0	981	8.3	9.9	9.1	982.0	972.9

Table 5.4. Local Scour Around Towers - Tucson Gas & Electric Co. for With-Siphon Channelization Alternative.

Tower Number	Obstruction Width	Flow Velocity (ft/sec)	Elevation at Bottom of Footing (ft)	Flow Depth (ft)	Local Scour: Shen (ft)	Local Scour: Neil (ft)	Local Scour: Adopted Scour (ft)	Approximate Channelized Ground Elevation	Elevation After Scour
87	5'	11.1	977	11.2	11.3	11.6	11.5	990.0	978.5
88 (R)	10'	11.1	*	10.9	17.4	21.7	19.6	984.8	965.2
89 (R)	10'	11.1	971	10.9	17.4	21.7	19.6	983.5	963.9
94	5'	11.3	961.8	10.5	11.5	13.8	12.7	974.1	961.4
95 (R)	5'	11.1	953.5	10.5	11.3	13.7	12.5	973.6	961.1
96	10'	12.3	958.3	9.4	18.5	22.2	20.4	972.1	951.7
97	5'	9.8	*	9.9	10.5	12.9	11.7	967.0	955.3
98	5'	9.8	*	9.9	10.5	12.9	11.7	964.6	952.9
99 (R)	10'	9.0	*	10.7	15.3	19.7	17.5	962.6	945.1
100 (R)	10'	9.6	947.3	10.5	15.9	20.2	18.0	960.4	942.4
101 (R)	10'	10.0	944.5	10.4	16.4	20.7	18.6	958.3	939.7
102 (R)	10'	12.0	938.8	9.2	18.7	22.2	20.5	956.1	935.6
103 (R)	10'	9.8	*	12.5	16.1	20.9	18.5	953.7	935.2

R = Reinforced

* = Elevation unknown

5.3 Aggradation/Degradation Response

The aggradation/degradation response of a river can be quantified through several different methodologies, including an equilibrium slope and armor control process. Proposed channelization reaches from Camelback Road to Buckeye were evaluated considering present upstream conditions and future upstream developments.

The equilibrium slope analysis is usually determined for the dominant discharge in the river, defined as the discharge that has the most influence in shaping the channel. As explained in the SLA report "System Analysis and Conceptual Design of Channelization in the Agua Fria River," the bankfull discharge is considered the dominant discharge in the Agua Fria River. Since the river is braided in the study reach, the bankfull discharge is difficult to define. The 10-year discharge of 31,000 cfs was selected as having the most influence in shaping the channel within the study reach. The 10-year discharge is within the range of discharges that can be considered bankfull along the Agua Fria River. Table 5.5 summarizes the equilibrium slopes for each of the reaches defined in Section 4.1.

5.4 Armor Control

The armoring process begins as the nonmoving coarser particles segregate from the finer material in transport. The coarser particles are gradually worked down in the bed, where they accumulate in a sublayer. Fine bed material is removed through this coarse sublayer to augment the material in transport. As movement continues and degradation progresses, an increasing number of nonmoving particles accumulate in the sublayer. This accumulation interferes with the removal of fine material so that the rate of transport over the sublayer is not maintained at its former capacity. Eventually, enough coarse particles accumulate to shield, or "armor," the entire bed surface. When fines can no longer be removed from the underlying bed, degradation is arrested.

The armor layer will form over a long period of time, or during a large event, such as the 100-year flood or a Standard Project Flood. With the gradual depletion of upstream sediment supply into the channelized reach, between Camelback Road and Buckeye Road, the armor control process could dictate the future downstream gradient. The question that must be answered is whether the degradation that would occur before armoring would be too large to be compatible with the channelization.

Table 5.5. Summary of Equilibrium Slope Analysis, 10-Year Return Flood.

Reach	Description of Reach	TW (ft)	Q (cfs)	V (fps)	HY (ft)	Q_s (cfs)	Conc. (ppm by wgt)	S*	S_{eq}
Supply	Upstream of Camelback Road[1]	1,673	31,000	5.36	3.26	105.5	9,020	0.0017	0.0017
1	Camelback Road to 4000' below Camelback	1,626	31,000	6.04	3.16	157.7	13,480	0.0021	0.0014
2	4000' below Camelback to ISRB	1,567	31,000	7.55	2.62	326.6	27,920	0.0048	0.0016
3	ISRB to RID flume	1,115	31,000	6.80	4.10	182.4	15,190	0.0024	0.0014
4	RID flume to Thomas Road	1,045	31,000	6.80	4.36	174.4	14,910	0.0023	0.0014
5	Thomas Road to I-10	1,181	31,000	6.71	3.90	181.2	15,490	0.0021	0.0012
6	I-10 to Van Buren	1,435	31,000	6.07	3.56	148.0	12,650	0.0023	0.0017
7	Van Buren to Buckeye Road	1,227	31,000	5.37	4.70	88.0	7,520	0.0016	0.0017

[1] Average hydraulics of main channel braid
*Profile determined from thalweg of August 31, 1981, topographic map.

TW = top width
Q = water discharge
V = flow velocity
HY = hydraulic depth
Q_s = sediment transport rate
Conc = sediment concentration in parts per million by weight
S = existing thalweg slope
S_{eq} = equilibrium slope

Two methods were used to compute the armor control depths for the SPF. The first method utilized the Shields relationship for incipient motion and is sometimes referred to as the static equilibrium slope. The second method is referred to as the particle size armoring method (for a discussion on the derivation of the methods, please see "System Analysis and Conceptual Design of Channelization in the Agua Fria River").

Table 5.6 summarizes the static equilibrium slopes computed for the SPF discharge of 142,000 cfs for each reach assuming a two-inch armor material will form on the surface. The static equilibrium slopes are considerably flatter than the dynamic equilibrium slopes. The static equilibrium slopes give a reasonable approximation of the long-term response of the channel bed and the dynamic equilibrium response is an indication of expected short-term responses. This statement is based on the fact that Waddell Dam will continue to trap sediment and the channel bed upstream of the channelized reach is armoring, thus the supply from which the dynamic equilibrium slopes were computed will eventually be reduced.

The particle size armoring method assumes an armor layer will develop when a layer twice the thickness of the largest nonmoving sediment particle forms on the channel bed. Using two inches as the armoring size, and a reasonable estimate that 95 percent of the subsurface material is finer than two inches, an armor layer will develop at a depth of 11.6 feet for reaches 1 through 7.

Table 5.7 compares the dynamic equilibrium, static equilibrium and particle size armoring bed responses for each of the seven reaches. The dynamic equilibrium slope methodology was the controlling bed response for all reaches.

5.5 Grade Control Locations

The qualitative and quantitative analyses indicate the river response in the study reach is degradation. Since the sediment transport capacity is greater than the upstream sediment supply, and since the banks will be protected, the difference between transport capacity and sediment supply will come from the channel bed. With no apparent natural grade controls in the subsurface stratum, man-made grade controls will be necessary to check the degradation potential.

Table 5.6. Static Equilibrium Slopes for SPF Discharge of 142,000 cfs.

Reach	Description of Reach	Existing Thalweg Slope	Static Equilibrium Slope
1	Camelback Rd. to 2200' upstream of ISRB	0.0021	0.0014
2	2200' upstream of ISRB to ISRB	0.0048	0.0011
3	ISRB to RID flume	0.0024	0.0007
4	RID flume to Thomas Rd.	0.0023	0.0006
5	Thomas Rd. to I-10	0.0021	0.0008
6	I-10 to Van Buren	0.0023	0.0010
7	Van Buren to Buckeye Rd.	0.0016	0.0008

Table 5.7. Summary of Degradation Depths Using Different Methodologies for Predicting Bed Response.

Reach No.	Dynamic Equilibrium Slope (ft)	Static Equilibrium Slope (ft)	Flat Slope[1] (ft)	Particle Armor Control (ft)
1	2.6	6.0	7.9	11.6
2	7.2	9.5	10.8	11.6
3	2.3	5.4	5.6	11.6
4	3.8	8.5	9.7	11.6
5	7.6	12.9	17.7	11.6
6	1.4	5.2	5.4	11.6
7	0.6*	7.0	9.1	11.6

*Aggradation in this reach.

[1]Flat slope assumes the bed will be horizontal in the reach.

One design philosophy would be to consider the transient nature of the governing physical processes. However, due to the uncertainty of long-term developments and projects that will affect flows and sediment supply rates into the channelized reaches, such as New Waddell Dam, New River Dam and possible future diversions into or out of the basin, the short-term response (dynamic equilibrium slopes) was used for conceptual design. Should sediment supply rates be significantly reduced in the future from the watershed and channel, additional drop structures can be incorporated into the system.

Grade-control structures were located and sized to account for the potential degradation between the existing slope and the dynamic equilibrium slope. Further, the locations of drop structures provided protection of existing structures whenever possible.

Figures 5.1 and 5.2 show the proposed profile and location of drop structures for Alternative 1 (without siphon at the RID crossing) and Alternative 2 (with siphon at the RID crossing), respectively. An extra drop structure located just downstream of the RID flume is necessary for Alternative 1 to provide three feet of freeboard underneath the flume and protect the foundation.

For Alternative 1 the suggested drop structure locations are as follows: a three-foot drop 500 feet downstream of I-10, a three-foot drop 2,200 feet upstream of the proposed McDowell Road Bridge, a three-foot drop 200 feet downstream of Thomas Road, a three-foot drop directly below the RID flume, and a four-foot drop directly below ISRB. For Alternative 2 the suggested drop structure locations are as follows: a four-foot drop structure 500 feet downstream of I-10, a four-foot drop structure 2,200 feet upstream of the proposed McDowell Road Bridge, a four-foot drop 200 feet below Thomas Road, and a three-foot drop just downstream of ISRB.

5.6 Total Scour at Major Bridge, Flume and Utility Crossings

The total scour at major bridge, flume and utility crossings can be broken down into the following four components: local scour, general regional scour, general aggradation/degradation response, and bed form heights. Antidunes form on sand-bed channels when the flow enters the upper regime. For discharges approaching and exceeding those of the 10-year peak, upper regime flow conditions exist.

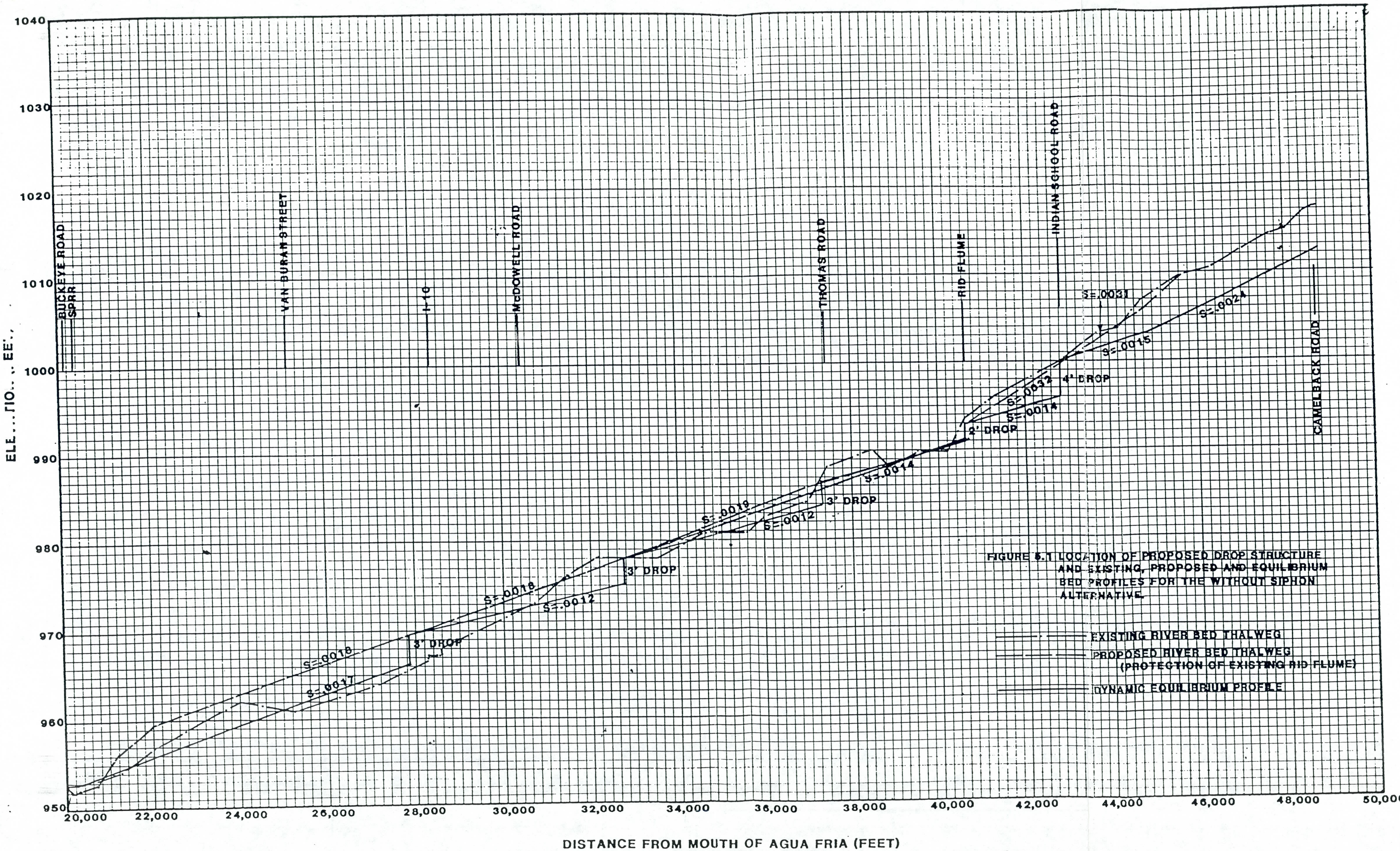


FIGURE 5.1 LOCATION OF PROPOSED DROP STRUCTURE AND EXISTING, PROPOSED AND EQUILIBRIUM BED PROFILES FOR THE WITHOUT SIPHON ALTERNATIVE.

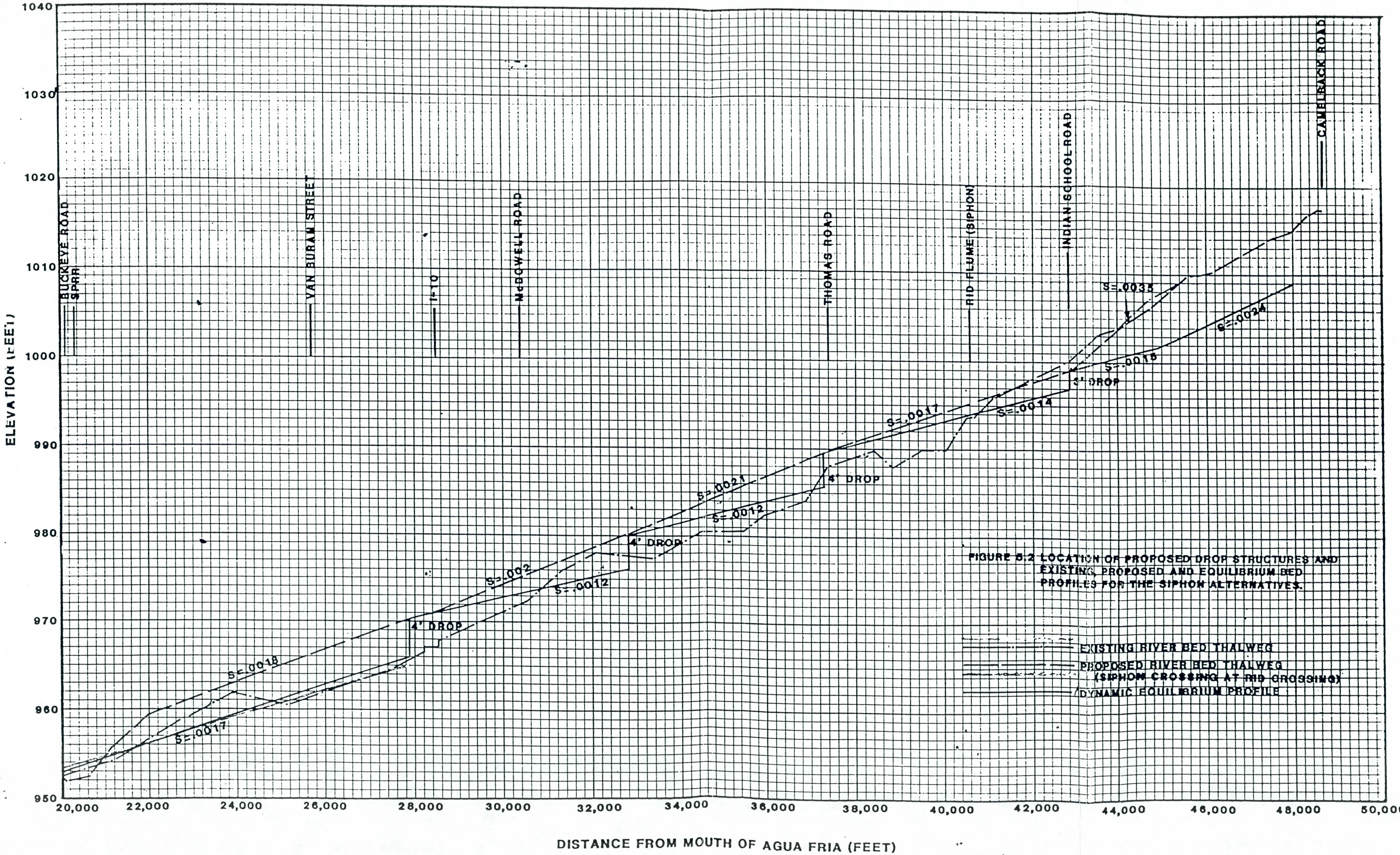


FIGURE 5.2 LOCATION OF PROPOSED DROP STRUCTURES AND EXISTING, PROPOSED AND EQUILIBRIUM BED PROFILES FOR THE SIPHON ALTERNATIVES.

Summaries of expected total scour depths at all major bridge and flume crossings are listed in Tables 5.8 and 5.9 for Alternatives 1 and 2, respectively. The general aggradation/degradation response was estimated using the dynamic equilibrium slope and pivoting about proposed grade-control structures.

Based on the summation of the four scour components at the bridge and flume crossings in the Agua Fria, it is recommended that Indian School Road Bridge, RID flume, I-10, SPRR and Buckeye Road Bridge piers be protected with riprap to prevent potential damage during the SPF. It should be noted here that the local scour potential at the SPRR and Buckeye Road crossings for existing conditions is 19.5 feet and 14.2 feet, respectively. These potential local scour depths are slightly greater than for proposed channelization conditions. If the channel is properly maintained near the bridges, the flow will be properly guided through the piers, minimizing the angle of attack and lessening the local scour potential.

The local scour at the Salt River Project and Tucson Electric Power Company transmission towers is excessive enough to require protection at all locations. Tables 5.3 and 5.4 summarize scour depths at all towers within the channelization reach. Due to the fact that channelization plus degradation will lower the channel bed significantly near the towers, it is suggested that a streamlined island, with soil cement or gabions, be provided around the towers to provide proper foundation protection.

Several pipeline crossings exist within the proposed channelization. El Paso Gas Company has a high-pressure 10-inch gas line located 150 feet upstream of Buckeye Road and a 20-inch high-pressure line adjacent to the east bank of the Agua Fria between ISRB and Camelback Road. The proposed channel invert upstream of Buckeye Road crossing is 952 ft. Figure 5.3 shows the soil cover over the existing crossing. The toe down of the levees is 8.5 feet below the proposed channel invert. It is recommended that the pipeline be lowered so 8.5 feet of cover exists over the pipe. This will involve lowering 80 feet of pipe near the west bank.

Along the east bank of the Agua Fria River between ISRB and Camelback Roads the El Paso gas pipeline is buried from 10.5 feet to 15 feet below the existing ground. The suggested channelization measures will not affect the pipeline in this reach. However, the local scour depth near Transverse Dike

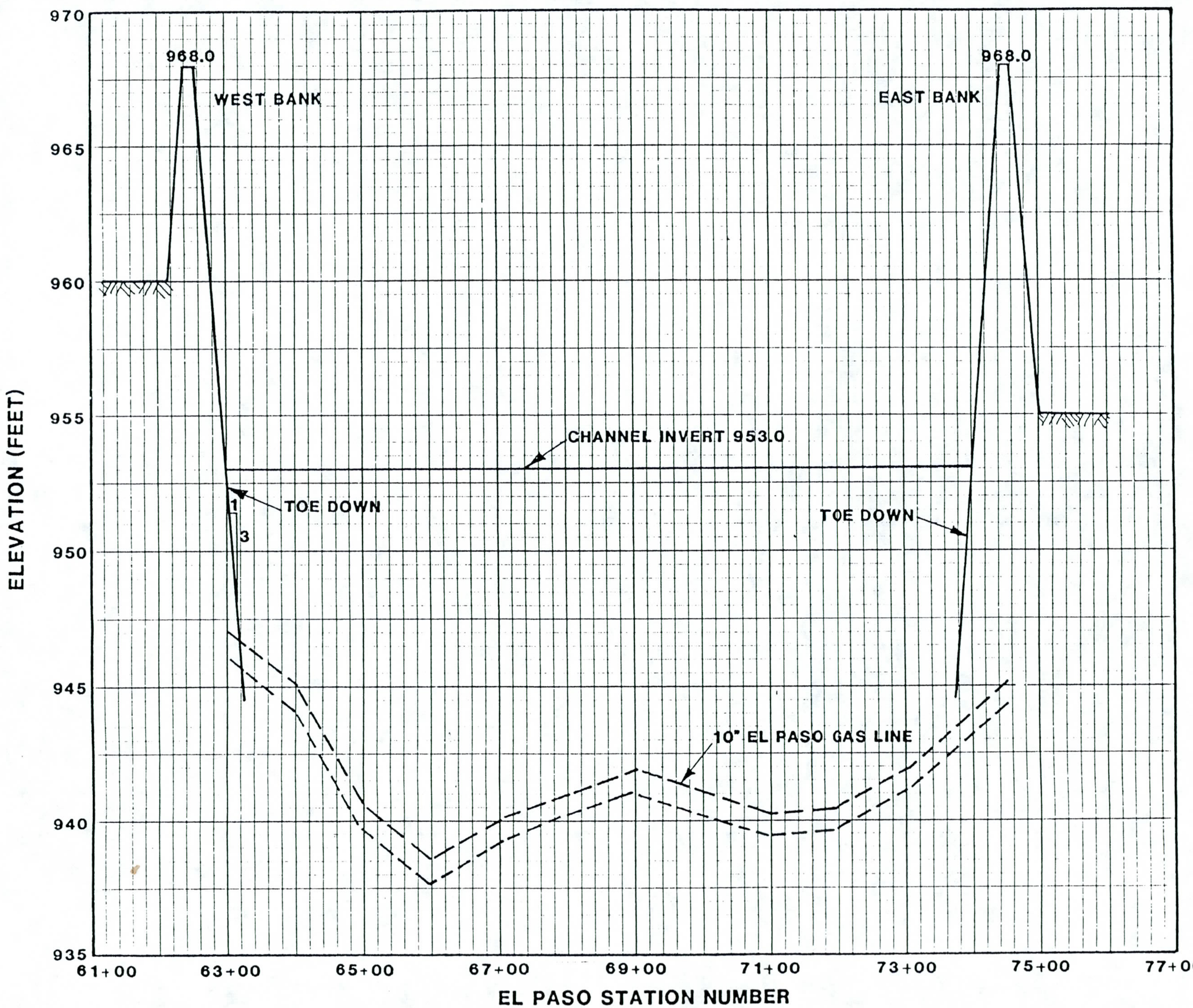


Figure 5.3. Sketch of 10" El Paso gas pipeline crossing of the Agua Fria River near Buckeye Road.

Table 5.8. Summary of Total Scour Depths for the SPF Expected at Major Bridge and Flume Crossings in the Agua Fria for Alternative 1.

Channel Feature	Depth of Burial of Piers	Local Scour (SPF Discharge) (ft)	General Regional Scour (ft)	General Aggradation/ Degradation/ (Dynamic Equilibrium) (ft)	One-Half Antidune Height (SPF Discharge) (ft)	Expected Total Scour (ft)
Camelback Rd. Bridge	70	13.4	1.5	5	1.5	21.4
Indian School Road Bridge	25	24.0	2.5	0	1.5	28.0
Roosevelt Irrigation Dist. flume	21-29	17.7	---	0	4.0	21.7
McDowell Road Bridge	70	17.0	---	1.9	2.4	21.3
I-10 Bridge	25	15.4	---	0.3	2.0	17.7
Southern Pacific Railroad Bridge	30	19.0	---	---	2.0	21.0
Buckeye Road Bridge	28	14.1	---	---	2.2	16.3

Table 5.9. Summary of Total Scour Depths for the SPF Expected at Major Bridge and Flume Crossings in the Agua Fria for Alternative 2.

Channel Feature	Depth of Burial of Piers	Local Scour (SPF Discharge) (ft)	General Regional Scour (ft)	General Aggradation/ Degradation/ (Dynamic Equilibrium) (ft)	One-Half Antidune Height (SPF Discharge) (ft)	Expected Total Scour (ft)
Camelback Rd. Bridge	70	13.5	1.5	7.0	1.5	23.5
Indian School Road Bridge	25	24.0	2.5	0	0.9	27.4
McDowell Road Bridge	70	17.2	---	1.9	2.6	21.7
I-10 Bridge	23	15.4	---	0.4	2.4	18.2
Southern Pacific Railroad Bridge	30	19.0	---	---	2.0	21.0
Buckeye Road Bridge	28	14.1	---	---	2.2	16.3

#2 will approach this depth and therefore the pipeline should be lowered approximately five feet for a distance of approximately 50 feet upstream of the dike.

The city of Avondale has a 16-inch water line crossing the Agua Fria at Thomas Road. Approximately 600 feet of this line will have to be lowered near the west bank. The Southern Pacific Pipeline, Inc. has a six-inch high-pressure gas line crossing the Agua Fria at Thomas Road. The depth of burial of this pipeline will have to be field verified before recommendations regarding relocation are made. Some channelization and degradation will result in a lowering of the channel bed at this pipeline crossing.

VI. APPLICATION OF MATHEMATICAL MODEL TO DETERMINE AGGRADATION/DEGRADATION RESPONSE OF CHANNELIZATION

6.1 General

To determine the general response of the Agua Fria bed to the SPF, water and sediment routing was performed using QUASED, a sediment routing procedure developed by Simons, Li & Associates, Inc. (SLA).

In using the QUASED model, the main river is subdivided into a series of computational reaches. Each of these subreaches is selected as a portion of the main river where hydraulic and geomorphic characteristics are similar. For this study, each subreach had sediment discharge input from the upstream portion of the main river. Hydraulic conditions for each subreach were calculated using the U.S. Army Corps of Engineers HEC-II water-surface profile program.

The general model concept was discussed in the previous report entitled "Hydraulic and Geomorphic Analysis of Agua Fria River" (please refer to this report for descriptions of the model). The QUASED model simulated the 1978, 1979, and 1980 floods as well as the 100-year flood for existing channel conditions. Results of the simulations were discussed in the above-referenced report. Sediment routing results for the channelized reaches are discussed in the following section.

6.2 Sediment Routing Results

The bed response of the Agua Fria from the confluence of the Gila River to Glendale Avenue was simulated for the SPF. Channelization measures considered included the channelization near Camelback Road Bridge as designed by PRC Toups, Inc., and channelization from approximately 2,000 feet upstream of ISRB to Buckeye Road. The thalweg profile is the dynamic equilibrium slope as shown in Figure 6.1. Also shown on Figure 6.1 is the simulated bed response to the SPF.

The bed never aggrades or degrades more than 1.5 feet in the channelized reach. Slight aggradation occurs near Camelback Road Bridge; however, the aggradation is occurring during the recession limb of the flood. Near the peak discharge the bed is slightly degrading at Camelback Road.

Some degradation occurs just upstream of ISRB where the channelization transitions to natural conditions. The proposed channelization is narrower than natural conditions and the velocities and sediment transport rates increase, which results in a degradation response.

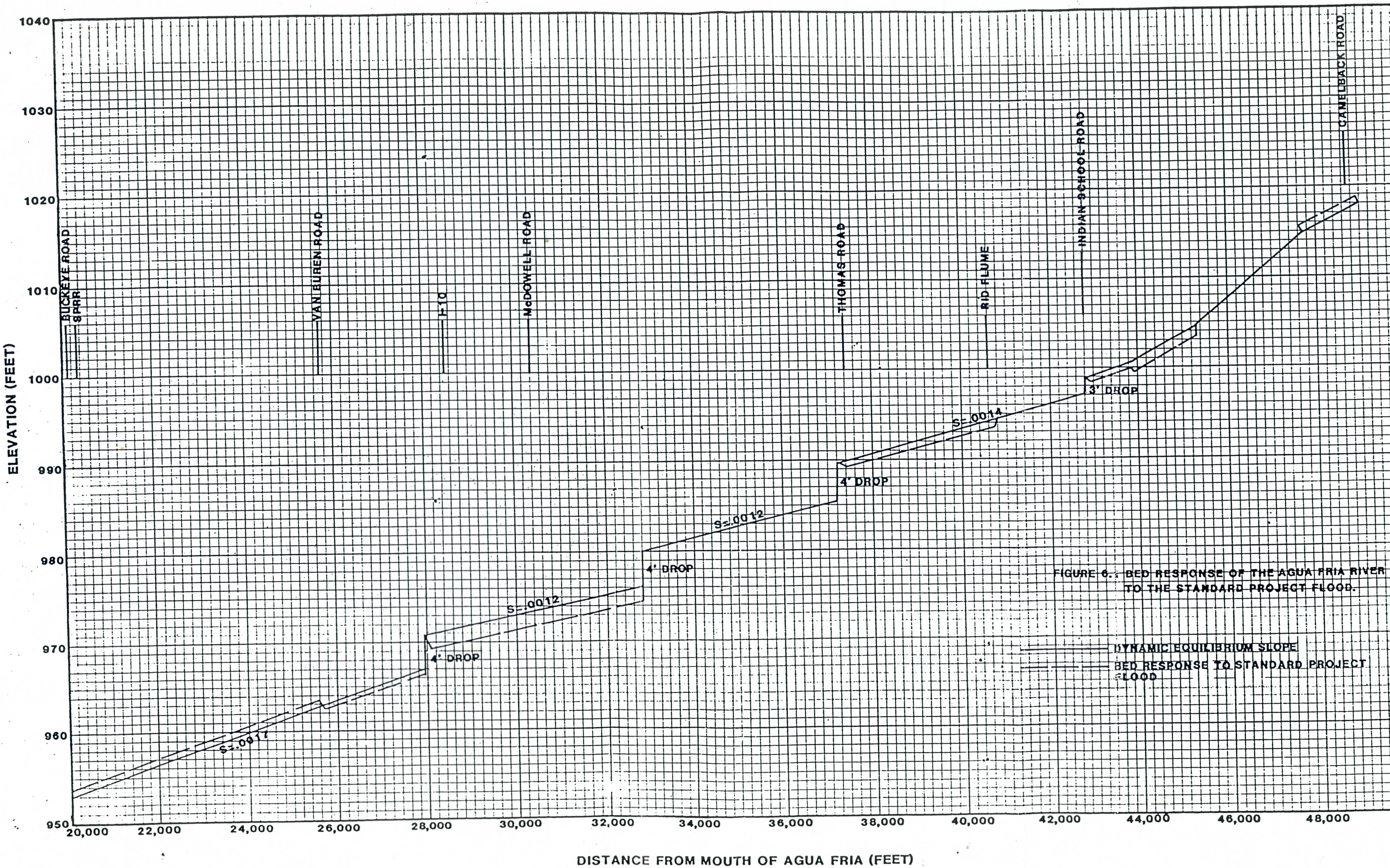


FIGURE 6.: BED RESPONSE OF THE AGUA FRIA RIVER TO THE STANDARD PROJECT FLOOD.

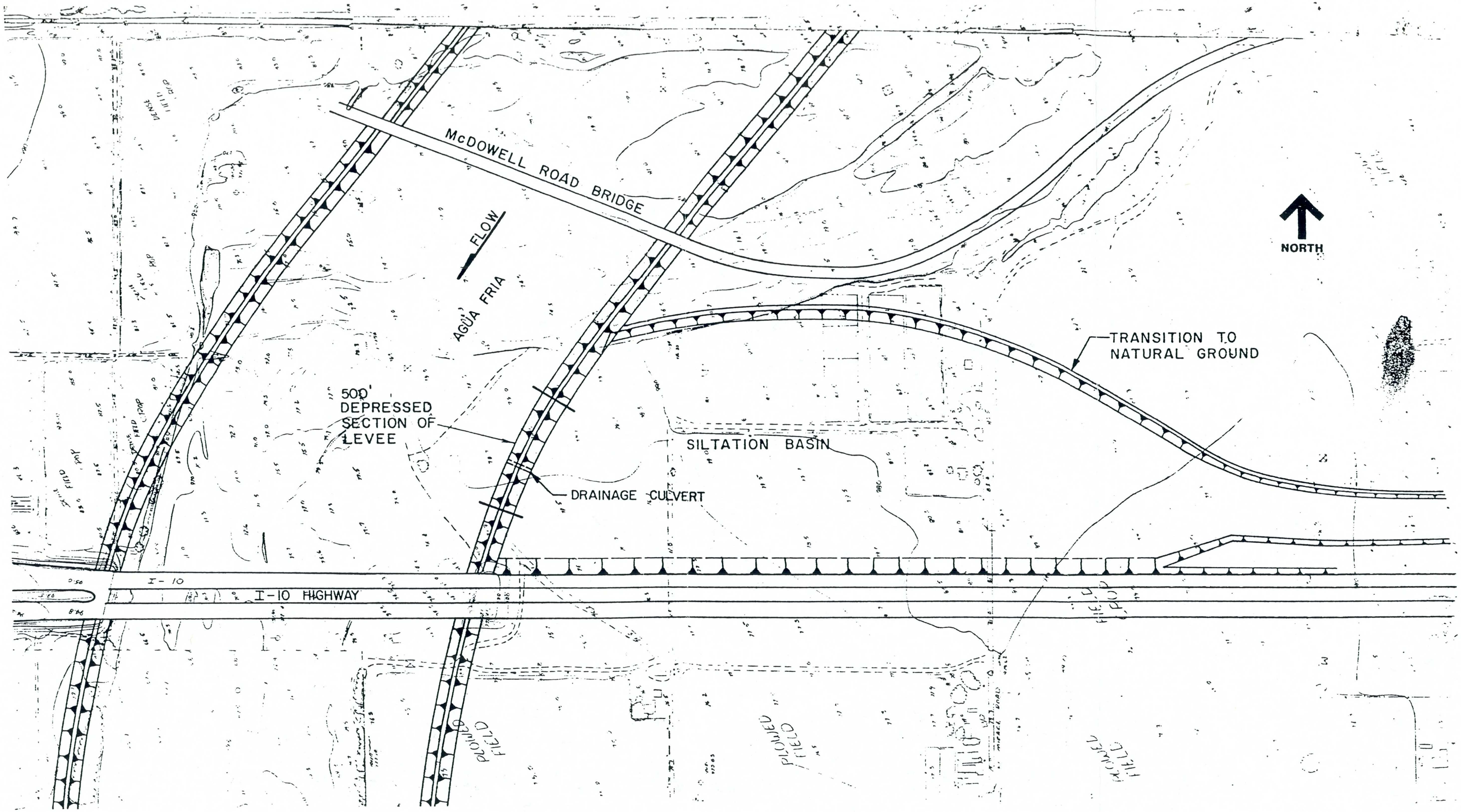


FIGURE 7 8 ALTERNATIVE 3 FOR I-10 COLLECTOR CHANNEL

Between the RID crossing and Thomas Road the channel degrades slightly due to channelization narrowing the river. The other degradation response occurs between I-10 and 2,200 feet upstream of the proposed McDowell Road Bridge.

A slight aggradation response occurs between Buckeye Road and Van Buren for the SPF. The bed response near the SPRR for the SPF, coupled with the slight aggradation bed response for events of smaller magnitude, indicates that lowering the bed near the bridge would result in future deposition and constant maintenance to keep the channel invert low enough to pass the SPF and provide three feet of freeboard.

6.3 Annual Aggradation/Degradation Analysis

An analysis to evaluate the average annual aggradation/degradation response of the channel bed considering bank protection was performed on all channelized reaches to determine if sediment deposition would reduce the flood carrying capacity of the channel. The procedure involved:

1. Divide the Agua Fria into reaches with similar hydraulic and sediment transport characteristics.

2. Compute the average hydraulic properties for each reach, such as flow velocity, top width and depth for a range of water discharges.

3. Establish sediment rating curves for each reach.

4. Use sediment rating curves to establish sediment transport rates for floods with return intervals of 10, 25, 50, and 100 years.

5. Using a weighted incremental probability method, determine annual sediment transport rates for each reach.

6. Compare the average annual sediment transport rate of each reach with that of the upstream supply reach to determine net deposition or degradation rates per year.

Average hydraulic properties for the previously defined seven reaches were established by using the Army Corps of Engineers HEC-II backwater profile program. Average flow velocity, depth, and top width were determined for a range of water discharges in each reach and used in deriving sediment rating curves.

The Meyer-Peter Muller (MPM) bed-load equation in combination with Einstein's integration of the suspended bed material was used to determine the sediment transport capacity of each reach. The sediment transport capacity was correlated with the water discharge to establish rating curves of the following form:

$$Q_s = aQ^b \tag{1}$$

where Q_s is the sediment transport capacity in cfs, Q is the water discharge in cfs, and a and b are the best fit coefficient and exponent. Table 6.1 lists the coefficients and exponents a and b for each of the seven reaches.

Sediment transport rates for the 10-, 25-, 50-, and 100-year floods were determined by applying Equation 1 to the discretized flood hydrographs. The average annual sediment yield for each reach was then computed using the weighted incremental probability of occurence of floods.

$$Q_{s_{annual}} = (.01)\,(Q_s)_{100} + (.01)(Q_s)_{50} + (.02)(Q_s)_{25} + (.06)(Q_s)_{10} \tag{2}$$

where Q_s is the average annual sediment yield (ft^3) and the 100-, 50-, 25-, and 10-year subscripts are for floods with these respective return intervals.

Table 6.2 summarizes average annual sediment yields for each reach and compares the yields with the supply reach to determine net aggradation/degradation response. All reaches display net degradation except reach 7, which shows a very slight aggradation response. The degradation response should continue until an armor control or equilibrium condition develops. Therefore, no major channel excavations are expected after channelization. The channel, however, should be monitored for any development of bars or islands, and any bars or islands that develop should be removed.

Table 6.1. Coefficients and exponents of sediment rating curves for each reach of the Agua Fria River.

Reach No.	a	b
Upstream Supply	1.223×10^{-5}	1.545
1	1.102×10^{-9}	2.535
2	1.170×10^{-7}	2.155
3	1.751×10^{-7}	2.046
4	4.643×10^{-7}	1.928
5	2.900×10^{-6}	1.741
6	1.762 X 10-5	1.558
7	5.611×10^{-7}	1.827

Supply	Camelback to the confluence with New River
Reach 1	2,200 ft. upstream of ISRB to Camelback Road.
Reach 2	ISRB to 12,200 ft. upstream of ISRB
Reach 3	RID flume to ISRB
Reach 4	Thomas Road to RID flume
Reach 5	I-10 to Thomas Road
Reach 6	Van Buren to I-10
Reach 7	Buckeye to Van Buren

Table 6.2. Average Annual Sediment Transport Yields for Channelization in the Agua Fria River.

Reach No.	Sediment Transport Rate (ft^3)	Degradation/ Aggradation (ft^3)	Average Depth of* Degradation/Aggradation (ft)
Supply	1,202,000		
1	3,597,000	-2,395,000	-0.4
2	6,672,000	-5,470,000	-1.6
3	3,161,000	-1,959,000	-0.7
4	2,247,000	-1,225,000	-0.3
5	2,157,000	- 955,000	-0.1
6	1,979,000	- 777,000	-0.2
7	1,021,000	181,000	<0.1

Supply Camelback Road to confluence with New River.
Reach 1 2,200 ft. upstream of ISRB to Camelback Road.
Reach 2 ISRB to 2,200 ft. upstream of ISRB.
Reach 3 RID flume to ISRB.
Reach 4 Thomas Road to RID flume.
Reach 5 I-10 to Thomas Road.
Reach 6 Van Buren to I-10.
Reach 7 Buckeye to Van Buren.

* The degradation/aggradation responses are computed for initial conditions, and as the bed responds toward equilibrium conditions the net degradation/aggradation tends toward zero. Therefore, this is just a measure of the direction in which each channel reach will respond.

VII. CONCEPTUAL DESIGN OF AGUA FRIA RIVER CHANNELIZATION

7.1 General

The conceptual channelization design of the Agua Fria River from Camelback Road to Buckeye Road is presented in this chapter. Two alternatives are discussed and include (1) channelization that protects the existing RID flume crossing and (2) channelization measures that replace the RID flume crossing with a siphon. Preliminary cost and quantity estimates for both alternatives are presented in Appendix A.

7.2 Description of Alternative Without Siphon at RID Flume

The Standard Project Flood channelization design components between Camelback Road and Buckeye Road are discussed in this section. The study reach is broken down into the following four reaches for the without-siphon alternative.

- Camelback Road to Indian School Road
- Indian School Road to Thomas Road
- Thomas Road to I-10
- I-10 to Buckeye Road

7.2.1 Camelback Road to Indian School Road

Design components between Camelback Road and Indian School Road include channelization, a 1,600-foot and 600-foot transverse dike on the east overbank north of ISRB, a spur dike just north of ISRB on the east bank, a partial levee extending approximately 2,000 feet along the west bank just north of ISRB termininating in a 700-foot-long transverse dike, and 3,400 feet of floodwall protection along the east approach of ISRB.

7.2.1.1 Channelization

Plates 1 and 2 attached to this report show the proposed alignment for levees and transverse dikes between Camelback Road and ISRB. The channelization extends approximately 2,800 feet upstream of ISRB. Approximately 157.5 acres of right of way (ROW) need to be acquired for construction of dikes and 43 acres associated with a construction easements for excavation upstream of the proposed ROW. Approximately 600 acres of land will be inundated outside the 157.5 acres of ROW between ISRB and Camelback Road for the 100-year flood. Plates 1 and 2 show the pre-project 100-year floodway and flood plain and the post-project 100-year floodway, flood plain and SPF plain. Figure 7.1 shows the proposed bed profile for this reach.

7.2.1.2 Levees

All levees will have 3:1 riverside slopes if protected with riprap, and 1:1 side slopes if protected with soil cement. The landward slope of the levees is 3:1 to the natural ground. The crown width is 15 feet. Figure 7.1 shows the levee height and toe-down depths for the partial levee on the west dike of ISRB. The dikes were extended a minimum of three feet above the SPF water surface, or the sum of one-half the bed-form height, one-half the water surface wave height, and the aggradation height (if any). Toe-down depths were extended below the proposed channel bed to a depth equal to the sum of (1) the general degradation plus (2) one-half the bed-form height plus (3) the historical low-flow thalweg depth plus (4) the local scour depth near levees.

An analysis of slope stability for the proposed levees was conducted. The results are reported in the following sections for riprap and soil cement bank protection.

7.2.1.2.1 Levee Stability. Slope stability analyses were conducted throughout the study reach. The levee soils consist largely of sands and gravels with a total unit weight of approximately 132 pcf. A minimum 50-foot buffer zone behind the toe on the land side of the levee and future excavations is required for slope stability of the levees. The analysis was conducted utilizing the Modified Bishop Method of Slices for the following cases from the Army Corps of Engineers, Engineering Manual EM 1110-2-1913:

Case	Description
I	End of Construction
II	Sudden Drawdown
III	Critical Flood Stage
IV	Steady Seepage from Full Flood Stage (Fully Developed Phreatic Surface)
V	Steady Seepage from Full Flood Stage (Partially Developed Phreatic Surface)
VI	Earthquake Loading

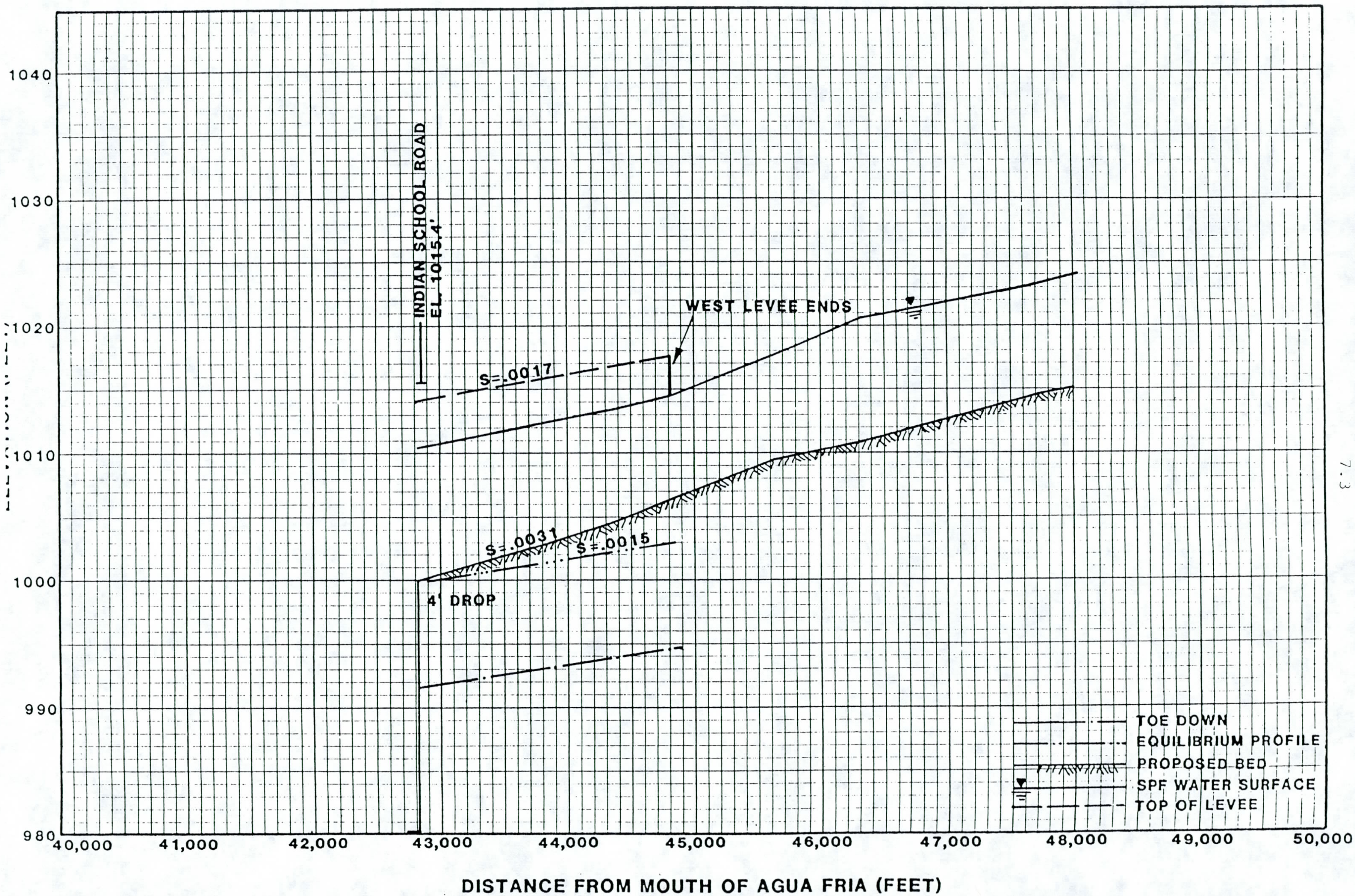


Figure 7.1. Proposed bed profile, equilibrium bed profile, SPF water-surface elevation, levee height and toe-down depth between ISRB and Camelback Roads.

Case I, end of construction, was not considered due to the free-draining nature of the levee soils and the expectation that the soils will be properly compacted during construction. Any excessive pore pressures created by construction will be of very short duration.

The phreatic surface for Cases II, III and IV was developed utilizing Casagrande's method for flow through an embankment. This method assumes an impervious surface below the levee base as a boundary condition. In actuality, the in-site foundation soils are also free draining, resulting in a much lower phreatic surface on the land side of the levee. The resulting surface was determined for a maximum SPF depth of 13.5 feet with the embankment geometry mentioned previously.

Case II, the rapid drawdown, represents the situation where the stage falls faster than the saturated embankment can drain. Unless the pore pressures adjust to the stage change, slope instabilities result on the river side of the levee. The minimum factor of safety found for this case was 1.0. This value is equal to the minimum value recommended by the Corps of Engineers for this particular loading. Furthermore, the analysis assumed that the levee remained saturated to the SPF level without any water in the channel. In light of the nature of the soils discussed previously, the actual expected factor of safety is significantly higher.

For this study, Cases III and IV, critical flood stage and steady seepage from full flood stage, were deemed to represent the same condition. Moreover, Case IV exemplifies the worst-case condition for slope stability on the land side of the levee. Using the phreatic surface and levee geometries and the SPF water depth, the minimum factor of safety was found to be 1.4. Again, this is equal to the recommended Corps value. Case IV was analyzed in lieu of Case V since it represents the worst condition. The earthquake loading, Case VI, was deemed inappropriate due to the extremely small probability of an earthquake occurring during the SPF or even any time the channel had a significant head.

The factors of safety determined for the stability of the proposed levee configuration were found to be equal or greater than the Corps of Engineers values listed in the Engineering Manual EM 1110-2-1913 for various loading conditions. In addition, the actual soil properties and lack of impervious foundation soils further increase the expected factors of safety for the extreme events analyzed.

7.2.1.2.2 <u>Levee With Soil Cement Protection</u>. A stability analysis was performed for a levee with a land side slope of 3:1, a crest width of 15 feet, and a river side slope of 1:1 protected by an eight-foot-wide soil cement layer.

The critical land side condition for the configuration is similar to the fully developed phreatic surface discussed in the previous section. The critical scenario for the soil cement configuration would be similar to the rapid drawdown, Case II, with an elevated seepage surface and without water in the channel. However, since the soil cement will act as a monolithic block or retaining wall, the overturning moment and sliding tendency was deemed more representative as a potential mode of failure. The calculated factor of safety for overturning was found to be 3.37. The factor of safety against sliding was found to be 1.04.

The computations indicate that the bank protection structure is safe from overturning failure, though only marginally safe from sliding. Wall friction was neglected in the analysis, which is conservative, as the soil-cement/sand interface probably has a wall friction angle close to the angle of internal friction for the sand. By considering wall friction, both the normal force and the force resisting sliding could increase by a substantial amount. In actuality, therefore, the factor of safety against sliding is well above one. Furthermore, the rapid-drawdown case considered in the analysis is probably extreme. Since soil-cement bank protection is generally used only where sandy material is present, the material behind the bank protection is likely to be well drained. It is quite unlikely that a rapid drawdown could occur which could simulate the analysis considered here. Since the hydrostatic loading is the largest force acting on the structure, the rapid-drawdown case produces over-conservative results. Hydrostatic pressures against the back of the wall could be relieved by the installation of drains and weepholes, although this will not be required where well drained materials are present. The factor of safety could be recalculated assuming some value for the wall friction, ϕ_w. However, even with such extreme conditions, the worst factor of safety was still close to 1.0. This indicates that the bank protection, over the range of expected operating conditions, will be stable.

7.2.1.3 Transverse Dikes

The transverse dike that terminates on the upstream end of the west levee is extended to intercept the SPF and realign the overbank flow through ISRB. The two transverse dikes and spur dike on the east overbank are provided to realign the flow through ISRB.

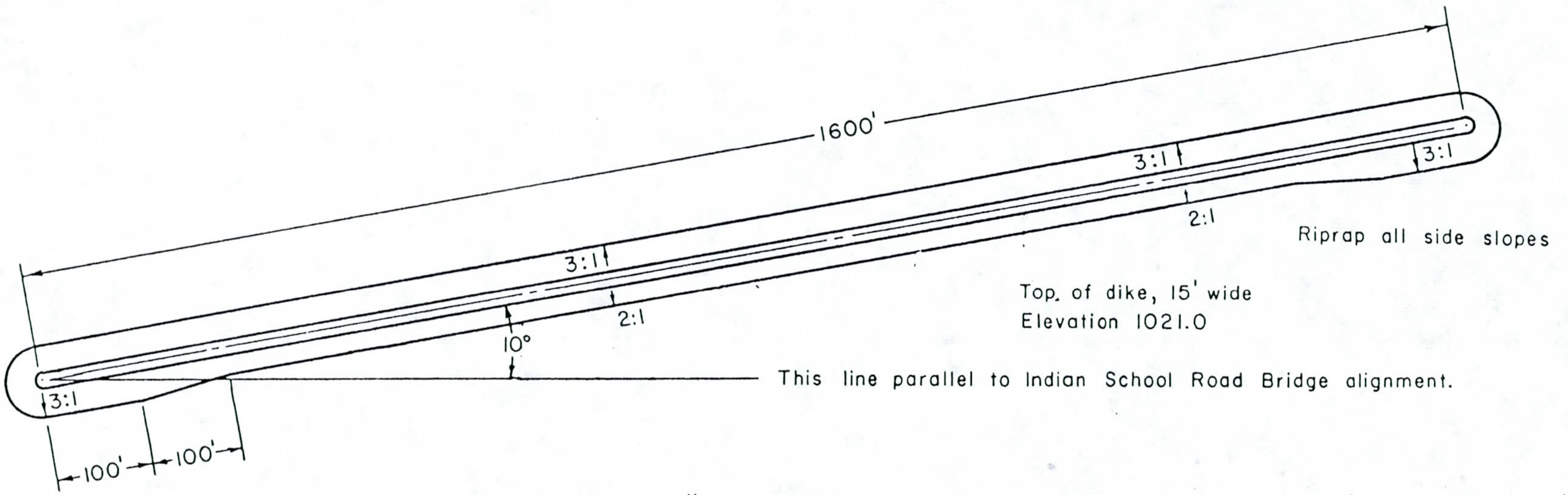
The east bank transverse dikes are tilted at an angle of 10 degrees to the ISRB alignment in order to prevent flows from striking them perpendicular and to reduce local scour along the upstream face. The spacing between the transverse dikes and spur dike was designed to prevent low-flow channel meandering from circumventing the dike system. The spacing of 600 feet and 350 feet is less than the historical low flow channel wave length of 1,000 to 2,000 feet.

Figure 7.2 provides the conceptual design of transverse Dike 2 with riprap protection. Transverse Dike 1 is identical, except the length is 600 feet instead of 1,600 feet. The transverse dike heights were extended three feet above the standard project flood water surface.

Figure 7.3 provides the conceptual design of the spur dike just north of ISRB on the east bank. The spur dike was designed utilizing the concepts presented in "Hydraulics of Bridge Waterways" (U.S. Department of Transportation, 1970). It is an elliptical spur dike with a shank length of 300 feet. The ratio of the minor to major axis is 0.5.

7.2.1.4 Floodwall Protection of ISRB

Presently 3,400 feet of the east approach to Indian School Road Bridge does not provide three feet of freeboard for the SPF. From approximately 900 feet east of the ISRB east abutment to approximately 1,300 feet east of 113th Avenue a floodwall is necessary to provide three feet of freeboard. A precast concrete median barrier approximately 3.5 feet in height, anchored in the shoulder of Indian School Road (ISR) is proposed to provide the required freeboard and ensure the SPF flow will funnel through the bridge. Additional work required is a partial raising of 113th Avenue at its intersection north of ISR and protection of the ISR embankment. Velocities approach seven feet per second along the embankment, and 20,000 cfs flows east around Transverse Dike #2 during the SPF. Riprap protection extending two-thirds of the height of the embankment and one foot below the existing ground is recommended to prevent erosion of the embankment. The work associated with the floodwall



Note: Transverse Dike #1 is identical except for length dimension, which is 600' rather than 1600'.
The top of Transverse Dike 1 is at elevation 1017.0'

Scale: 1" = 200'

Figure 7.2. Conceptual design of transverse dikes.

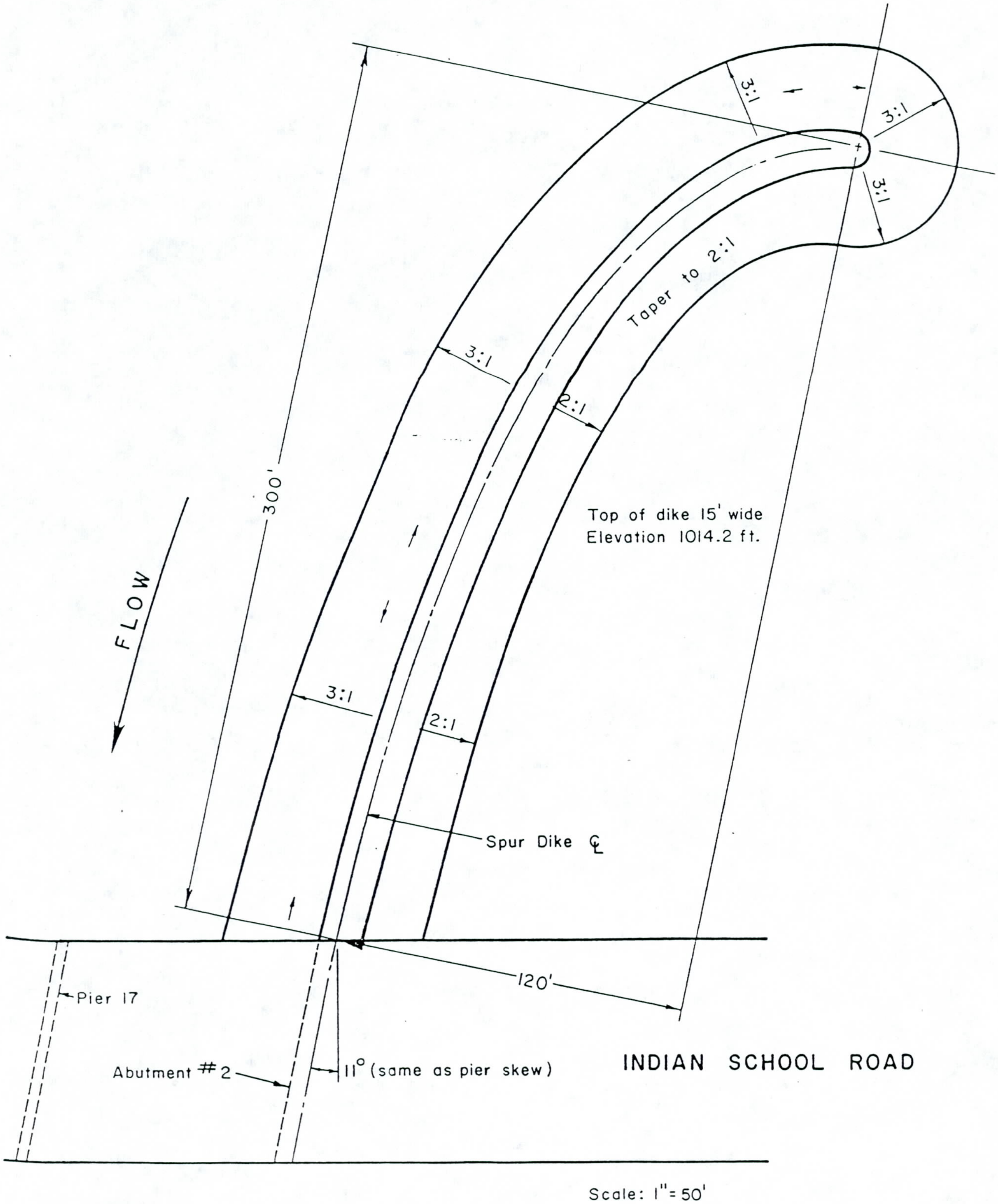


Figure 7.3. Spur dike design.

will be coordinated with the Maricopa County Highway Department, Indian School Road Project #12800, which will widen the road from 115th Avenue to 91st Avenue.

7.2.1.5 Protection of Dikes and Levees

The levee, transverse dikes, spur dike and Indian School Road embankment need some form of protection to resist erosive forces during the SPF. Two types of protection are being considered and include riprap and soil cement.

Sizing of riprap for the levees, transverse and spur dikes and ISR embankment were based on the factor of safety method presented in Sediment Transport Technology (Simons and Senturk, 1977). The riprap was sized to have a factor of safety of 1.4 or more. Table 7.1 summarizes the gradations of riprap. The thickness of the riprap should be twice the D_{50} size at all locations.

A filter fabric placed between the riprap and native soil is recommended to prevent piping of fine materials through the riprap. A three- to six-inch blanket of native material between the riprap and filter fabric is suggested to prevent the ripping or tearing of the filter fabric when the riprap is placed. If soil cement protection is used, the width should be eight feet and can be placed at 1:1 side slopes.

7.2.2 Indian School Road to Thomas Road

Design components considered between Indian School Road and Thomas Road include channelization, levees, ISRB pier protection, RID flume pier protection, a drop structure located just downstream of the RID flume and a drop just downstream of ISRB, backfilling of flood plain gravel pits, protection of utility transmission towers, and integration of the RID flume overflow structure into the Agua Fria.

7.2.2.1 Channelization

Plate 2, attached to the back of this report, shows the alignment between ISRB and Thomas Road. The channel bottom narrows from 1,440 feet at ISRB to 920 feet at the RID flume, transitioning to 1,100 feet at Thomas Road. The alignment is partially the result of an agreement reached between Allied Concrete Company on the west bank, Phoenix Sand and Rock Company on the east bank, and the Maricopa County Attorney. Approximately 154 acres of right of way have been acquired for the alignment between ISRB and Thomas Road.

Table 7.1. Riprap Gradations Between ISRB and Camelback Road.

Percent Passing Sieve	Levee, Transverse Dike, Spur Dike Rock Size (inches)	ISR Embankment Rock Size (inches)
100	36	8
50-70	18	4
15-30	9	2
0-5	4	1

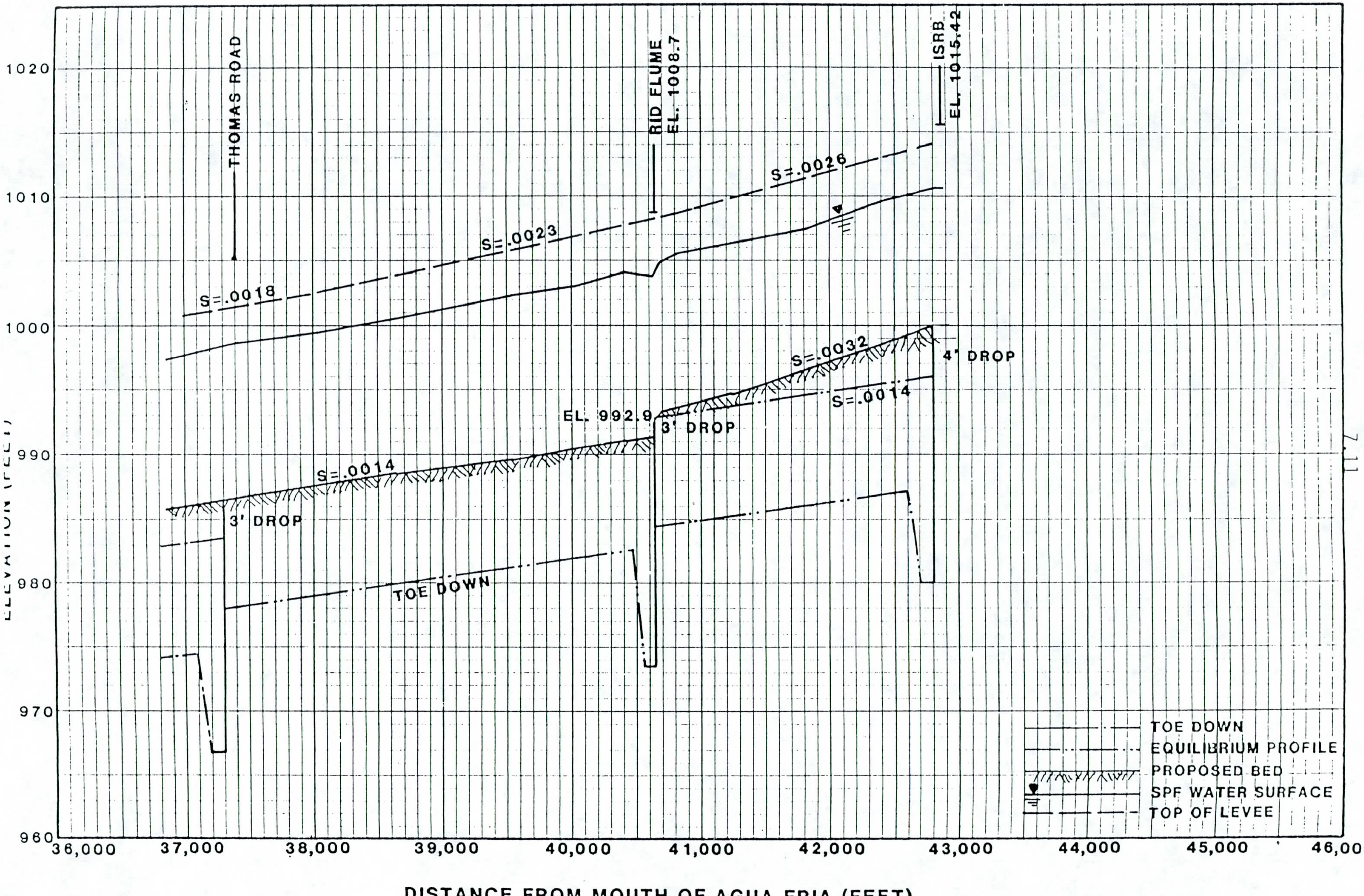


Figure 7.4. Proposed bed profile, equilibrium bed profile, SPF water-surface elevation, levee height and toe-down depth between Thomas Road and ISRB.

Figure 7.4 shows the proposed grade, levee heights and toe-down depths between ISRB and Thomas Road. A good portion of the levees will be constructed upon backfilled gravel pits, therefore riprap protection of levees is recommended because it is more flexible than soil cement in case of settlement of the gravel pit fill.

7.2.2.2 Drop Structures

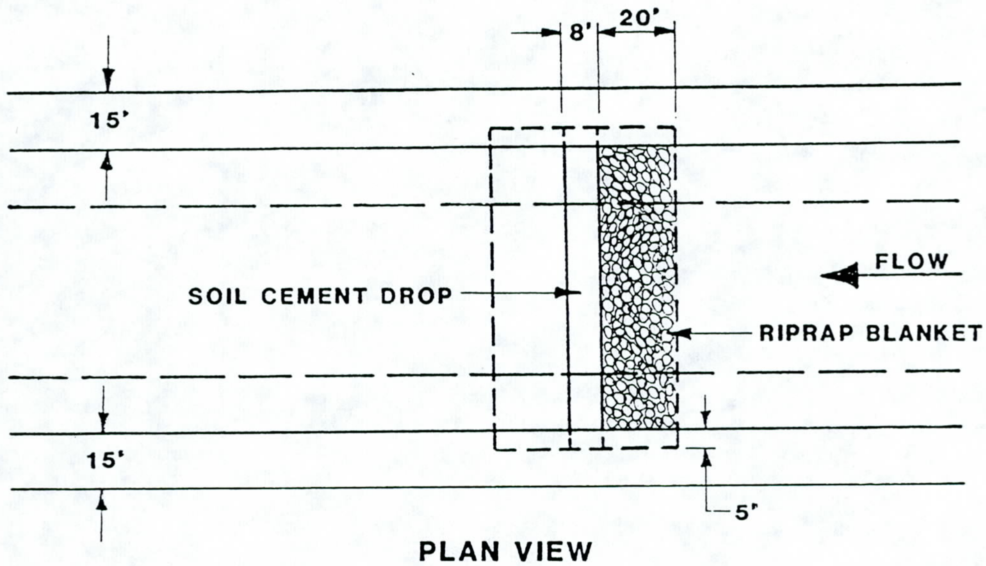
A three-foot-deep drop structure located directly downstream of the RID flume at elevation 992.9 feet and a four-foot drop structure located directly downstream of ISRB at elevation 1,000.0 feet is necessary to (1) protect existing bridge foundations from headcuts, and (2) reduce toe-down depths of levees upstream of structures. Soil cement grade controls are recommended.

The foundation of the soil cement grade control structure will extend below the local-scour hole on the downstream side of the structure. A riprap blanket will be placed on the upstream side of the grade control to prevent local scour. The drop structure is trapazoidal in shape. A plan and elevation view of the proposed grade control is shown in Figure 7.5.

The soil-cement grade-control structure was analyzed to determine its adequacy to resist overturning, sliding, bearing, piping, and undermining due to local scour. The proposed grade-control structure below ISRB had a factor of safety well above 1.5 for overturning, sliding, and bearing capacity. Piping and local scour were analyzed and found not to be a problem.

7.2.2.3 Protection of the RID Flume

A recent survey at the RID flume crossing indicates the elevation of the top of footings is approximately 992.9 feet and the low-chord elevation is 1,008.7 feet. To pass the SPF underneath the flume and provide three feet of freeboard, the bed elevation was lowered to 992.9 feet. Thus, protection of the footing is required. A riprap blanket three feet thick extending from the drop structure downstream of the flume, to 20 feet upstream of the flume is necessary to protect the existing structure.



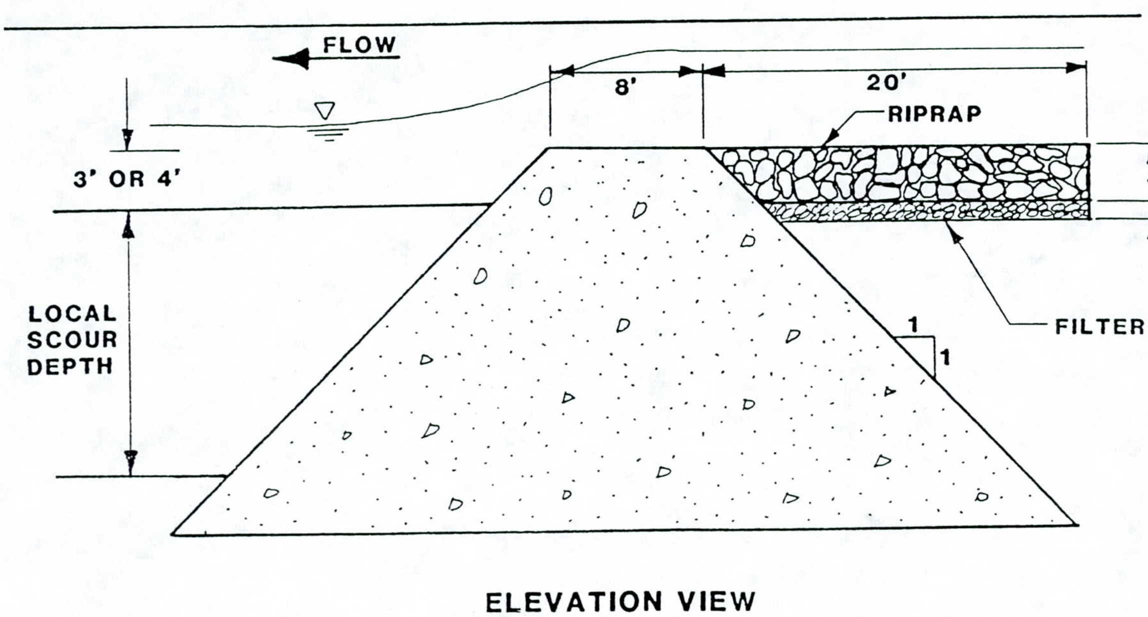


Figure 7.5 Plan and elevation view of proposed grade control.

7.2.2.4 Indian School Road Bridge Pier Protection

Bridge piers 1 through 12 on the west side of the river at ISRB are buried approximately 25 feet below the present thalweg elevation. The total unprotected scour potential at the bridge for the SPF is 27.6 feet, which exceeds the depth of burial of the piers. Therefore, protection measures for the shallow bridge piers are recommended.

A riprap blanket extending 20 feet around the piers is suggested. The blanket will be three feet thick and have a gravel filter of approximately 1 foot. The top of the riprap protection should start at elevation 999.0.

Bridge piers 13 through 17 have been replaced with drilled shaft caissons buried 50 feet below the local scour hole of the February 20, 1980 flood. No riprap protection is required for these piers. Thus, the riprap protection can be terminated at pier 12. The proposed levee alignment will go inside of pier 1 so only piers 2 through 12 require protection.

7.2.2.5 Backfilling of Gravel Pits

Several gravel pits on the east and west overbanks between ISRB and Thomas Road will have to be backfilled before levees can be constructed. On the west overbank Allied Concrete, Inc. has a sludge pond that will have to be drained and backfilled. The volume of fill required for the pit is 40,000 cubic yards.

Directly downstream of the sludge pit and just north of the RID flume is a large gravel pit that has been used by Allied Concrete as a land disposal site. Approximately 170,000 cubic yards of trash material will have to be removed from the proposed levee location. This pit will require approximately 380,000 cubic yards of fill material.

Downstream of the RID flume, on the west overbank, Allied Concrete has a gravel pit that will require 450,000 cubic yards of fill material.

7.2.2.6 Riprap Protection

Riprap protection of levees, RID flume piers, ISRB piers, and upstream of drop structures is required. Adequate toe-down depths downstream of drop structures will be provided for levees, and drop structure foundations will be extended below the local scour potential; therefore no riprap is provided on the downstream side of drop structures. Riprap with a D_{50} of 18 inches and a gradation shown in Table 7.1 will be adequate. Filter fabric can be used underneath the riprap for levees; however, a gravel filter is recommended for blanket protection around bridge piers and upstream of drop structures.

The filter should meet the following Arizona Department of Transportation Standard Specifications:

$$\frac{D_{50}\ (\text{riprap})}{D_{50}\ (\text{native soil})} < 40 \qquad (1)$$

$$5 < \frac{D_{15}\ (\text{riprap})}{D_{15}\ (\text{native soil})} < 40 \qquad (2)$$

$$\frac{D_{15}\ (\text{riprap})}{D_{85}\ (\text{native soil})} < 5 \qquad (3)$$

7.2.3 I-10 to Thomas Road

The following components are considered between Thomas Road and I-10: channelization, levees, two drop structures located 200 feet downstream of Thomas Road and 2,200 feet upstream of the proposed McDowell Road bridge, protection of eight transmission towers, protection of pipeline crossings near Thomas Road and integration of the I-10 collector channel into the Agua Fria.

7.2.3.1 Channelization

Plates 2 and 3 show the channel alignment between Thomas Road and I-10. The channel width is 1,100 feet between Thomas Road and McDowell Road and expands to 1,410 feet at I-10. The proposed bed slope from I-10 to the drop structure located 2,200 feet upstream of the proposed McDowell Road Bridge is 0.0018, and from this drop structure to the drop structure located 200 feet downstream of Thomas Road the proposed bed slope is 0.0019. Approximately 280 acres of right of way are required for this alignment, of which approximately 240 acres have been acquired. Figure 7.6 shows the proposed bed profile from I-10 to Thomas Road.

Figures 7.6 and 7.7 show the levee heights and toe-down depths for the east and west bank, respectively. On the west bank, approximately 1,200 feet upstream of the I-10 bridge, a rather severe bend begins and extends through the I-10 bridge ending about 2,000 feet downstream of the bridge. Velocities will increase on the outside of this bend, causing increased degradation to occur near the toe of the levee. Also superelevation around the bend will

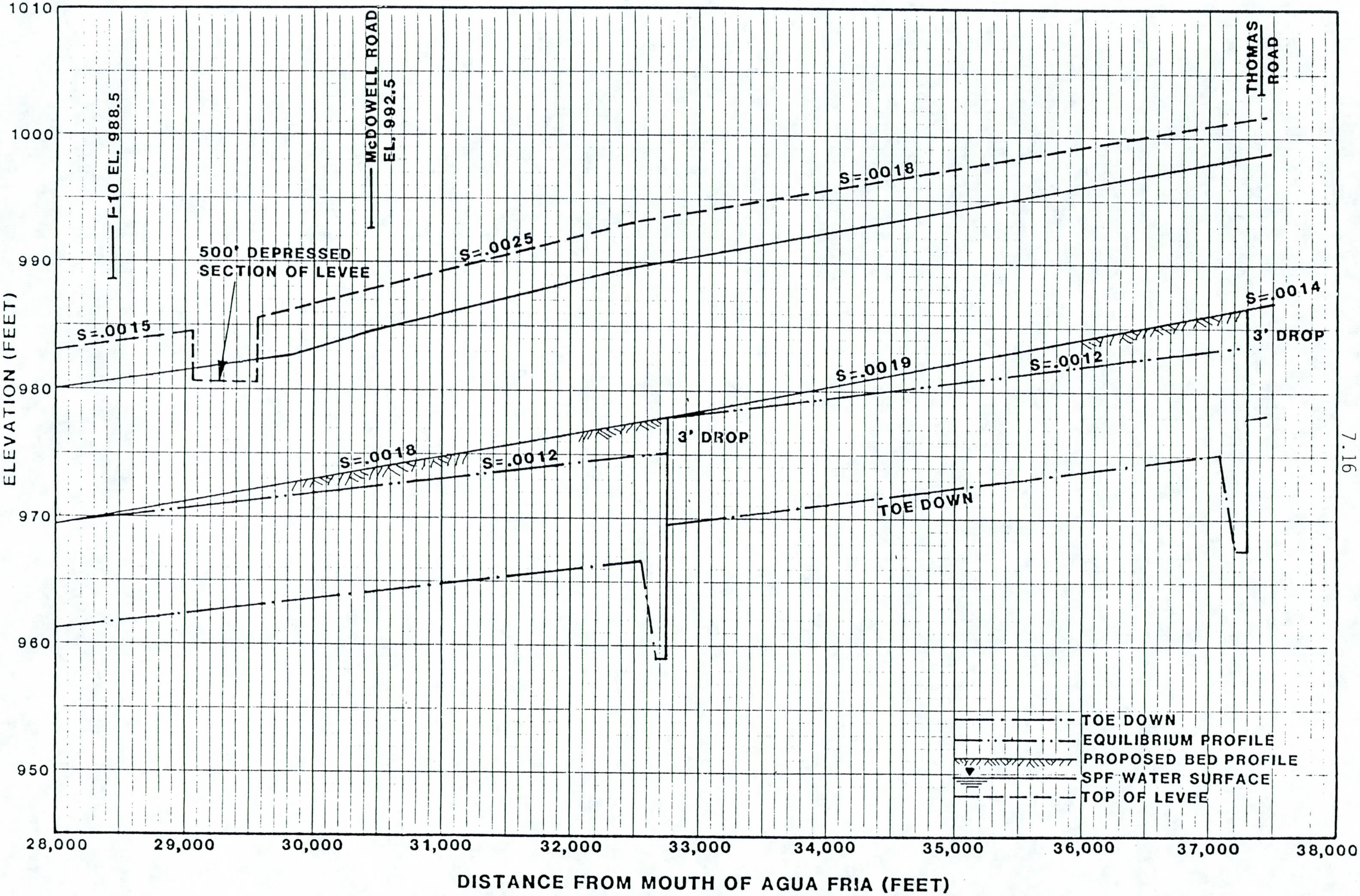


Figure 7.6. Proposed bed profile, equilibrium bed profile, SPF water-surface elevation, levee height and toe-down depth between I-10 and Thomas Road for the east bank.

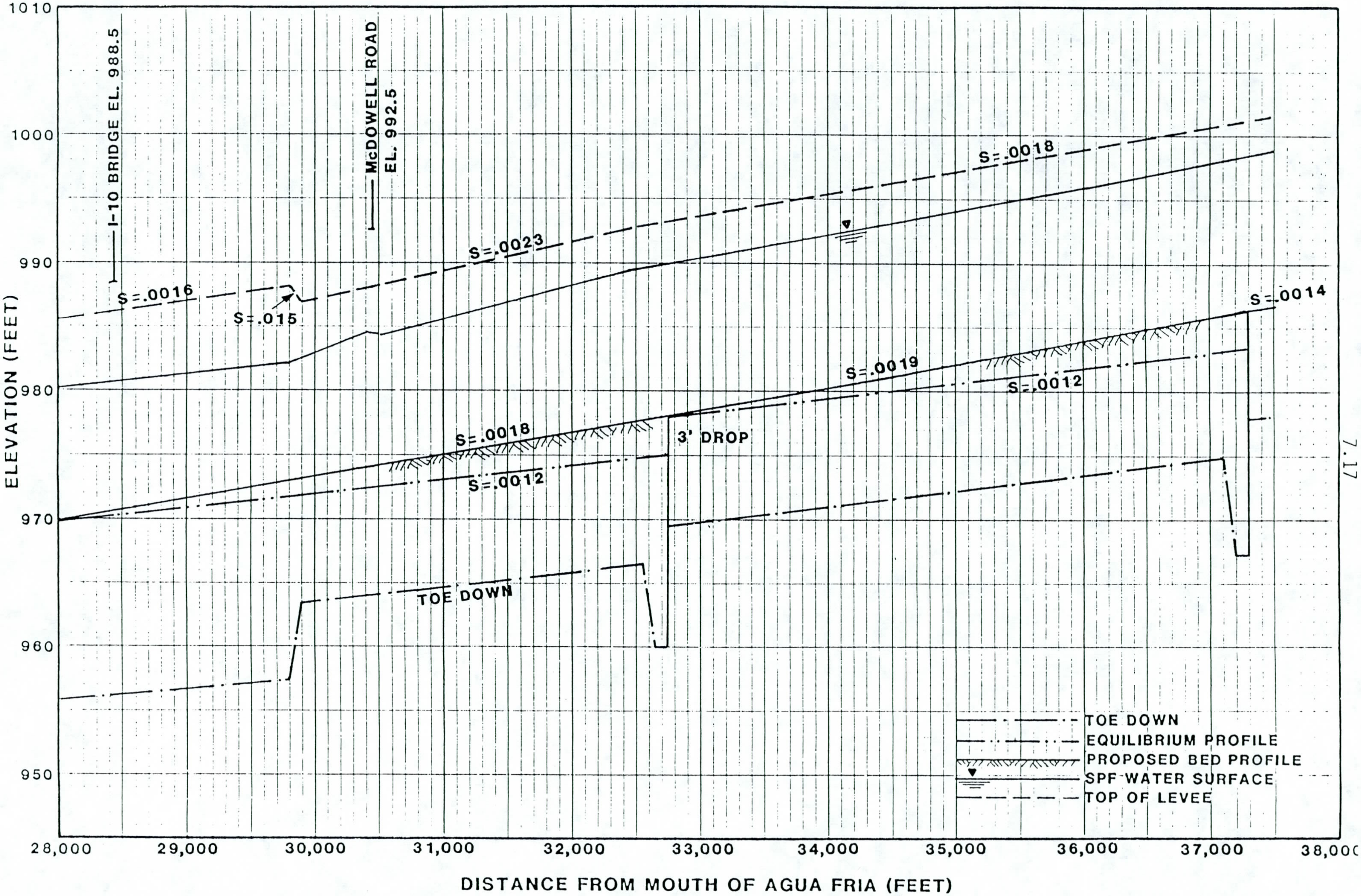


Figure 7.7. Proposed bed profile, equilibrium bed profile, SPF water-surface elevation, levee height and toe-down depth between I-10 and Thomas Road for the west bank.

increase flow depths, necessitating an increase in freeboard height above the SPF water surface. Required levee heights increase approximately 5.5 feet above the SPF surface around the bend because one-half the antidune height and the superelevation around the bend sum to 5.5 feet. Toe-down depths are extended 14 feet below the equilibrium bed profile due to the increased degradation potential.

Due to the acceleration of flow around the outside of the bend near I-10 the riprap size will increase appreciably. The D_{50} size increases to 2.5 feet in diameter and the gradation is as follows:

Rock Size	Percent Finer
60"	100
30"	50-70
15"	15-30
7"	0-5

Selecting a filter that will prevent piping of embankment material through the riprap with such a large difference in gradations, becomes almost impossible. Therefore soil cement is recommended for protection of the bend.

7.2.3.2 Drop Structures

Two three-foot drop structures are necessary between I-10 and Thomas Road. The locations of drops are 2,200 feet upstream of McDowell Road at elevation 978.0 and 200 feet downstream of Thomas Road at elevation 986.5. Both drops will be constructed of soil cement.

The drop structure located 200 feet downstream of Thomas Road will help stabilize the grade near Thomas Road. This will provide protection for the 16-inch water pipeline and six-inch high-pressure gas pipeline. However, portions of these lines will have to be lowered due to the channel bed being lowered in this vicinity.

7.2.3.3 I-10 Collector Channel

The I-10 collector channel ends approximately 2,900 feet from the proposed levees between McDowell Road and I-10. Presently, the collector channel empties into a 40-foot wide pilot channel, which carries water into the Agua Fria. A siltation basin between the collector channel and Agua Fria River with a controlled spill section over the levees is proposed to handle drainage from the I-10 collector channel. A park area around the siltation basin is also proposed.

Figure 7.8 shows an overview of the proposed alternative. The east levee between McDowell and I-10 will have a depressed section 500 feet wide. The depressed section in the levee will contain the 50-year return flow in the Agua Fria. The siltation basin between the collector channel outlet and the Agua Fria will store approximately 300 acre-feet, which is considerably less than the 100-year volume of 1,710 acre-feet; however, it should be large enough to handle a majority of nuisance flows.

Approximately 13.5 acre-feet of sediment per year will be generated from the I-10 collector channel. Periodic maintenance of the siltation basin will be required to maintain its full storage capacity for large floods.

It is suggested that a culvert be placed at the bottom of the depressed section of the levee to allow for evacuation of water from the siltation basin after rains. It is also suggested that the fill material of I-10 be tested for its suitability as an embankment for the siltation basin.

7.2.4 Buckeye Road to I-10

The following components are being considered between I-10 and Buckeye Road: channelization, levees, one three-foot drop structure located approximately 500 feet downstream of I-10, I-10 bridge pier protection, Southern Pacific Railroad pier protection, Buckeye Road bridge pier protection, protection of seven transmission towers, and backfilling of abandoned gravel pits just north of Van Buren Road.

7.2.4.1 Channelization

Plates 3 and 4 show the channel alignment between I-10 and Buckeye Road. The channel width is 1,410 feet between I-10 and Van Buren Street and gradually decreases to 1,100 feet wide at the SPRR crossing. Three hundred acres of channel right of way are required for the proposed alignment, of which 150 acres is owned by the State of Arizona and the Maricopa County Highway Department. Figure 7.9 shows the proposed bed profile.

Figures 7.9 and 7.10 show required levee heights and toe-down depths for the east and west banks, respectively. Levee heights extend 5.5 feet above the SPF water surface and toe-down depths extend 14 feet below the equilibrium bed slope on the west bank from I-10 to 2,000 feet downstream of I-10.

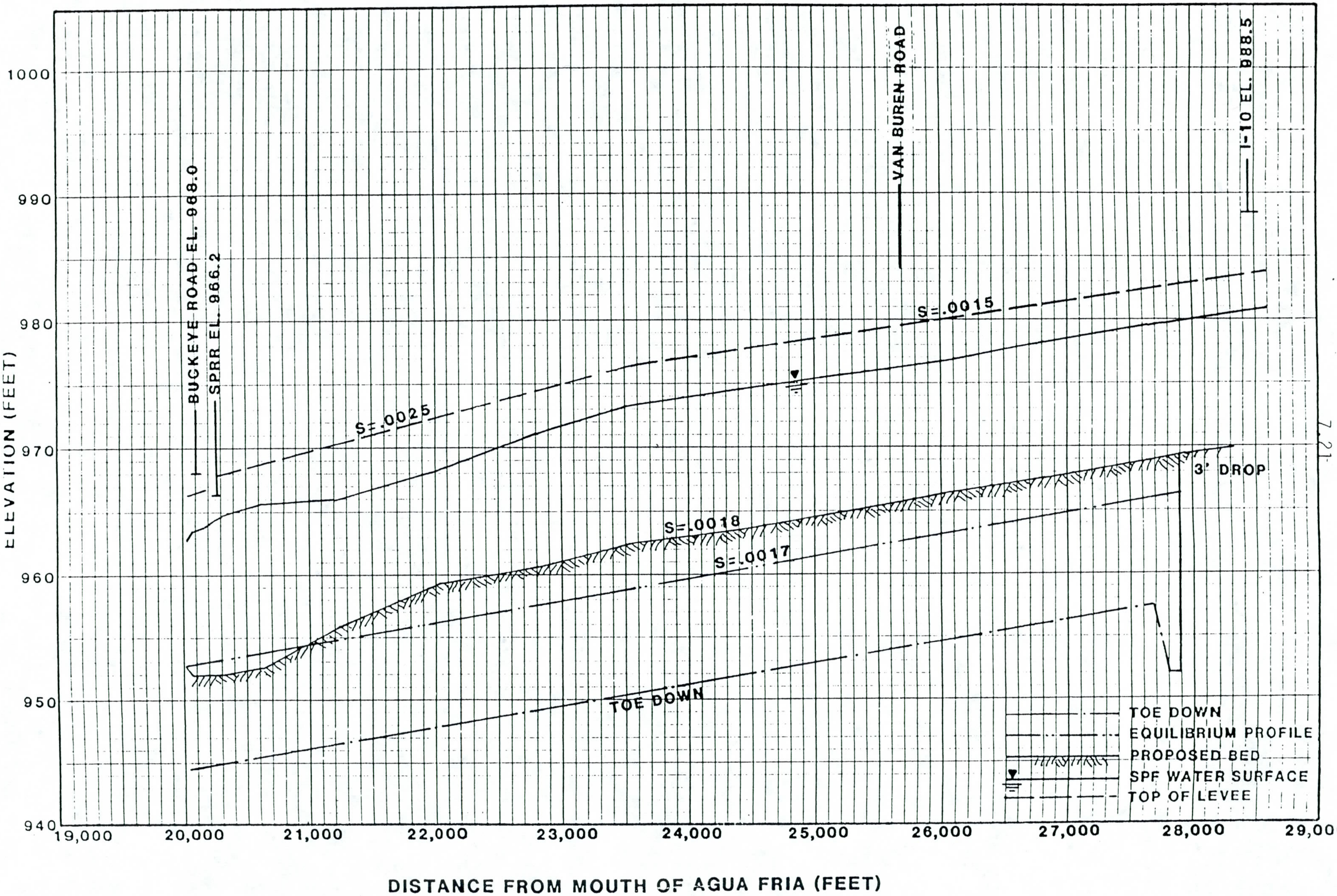


Figure 7.9. Proposed bed profile, equilibrium bed profile, SPF water-surface elevation, levee height and toe-down depth between Buckeye Road and I-10 for the east bank.

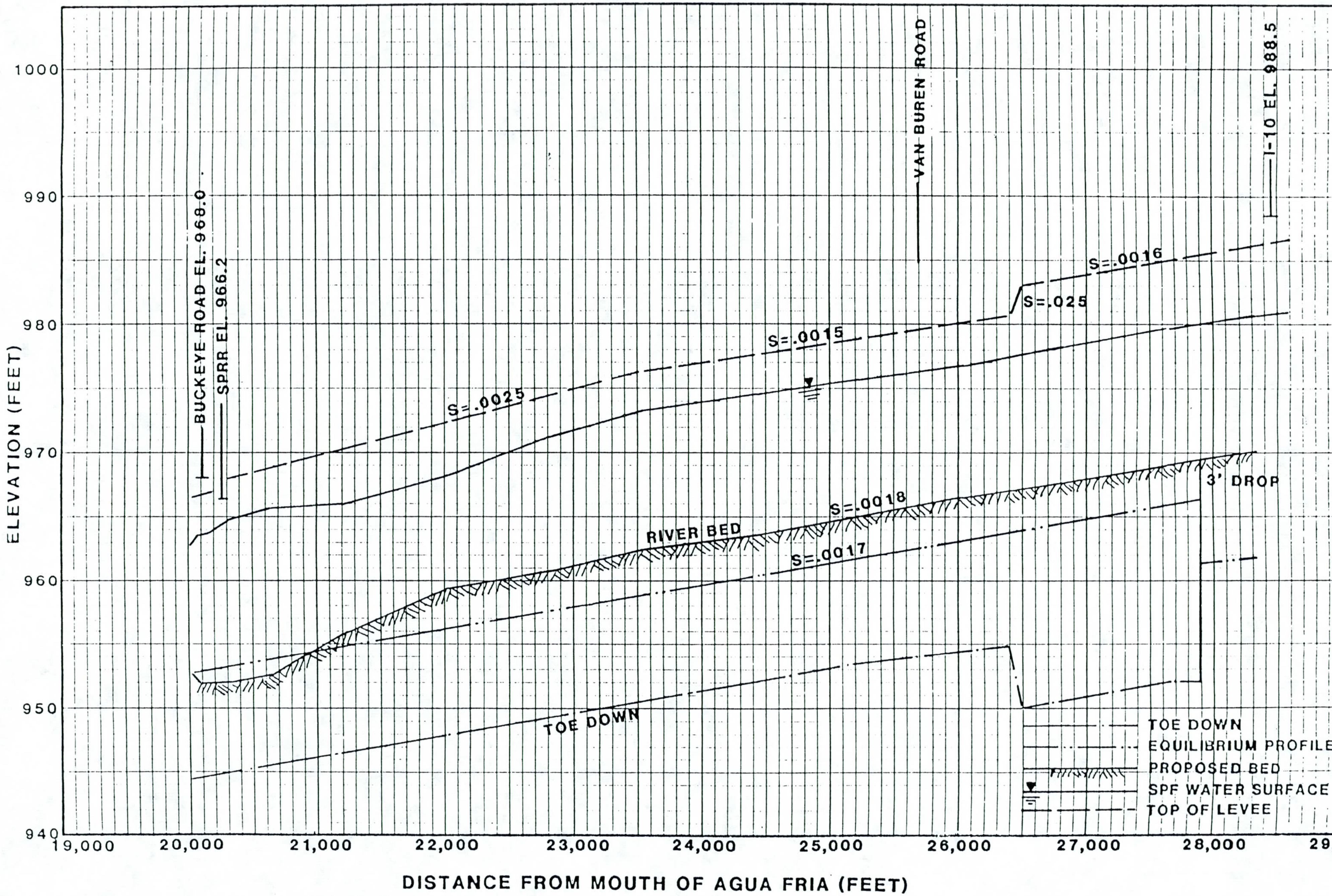


Figure 7.10. Proposed bed profile, equilibrium bed profile, SPF water-surface elevation, levee height and toe-down depth between Buckeye Road and I-10 for the west bank.

7.2.4.2 Drop Structure

One three-foot drop structure located 500 feet downstream of I-10 at elevation 969.5 is proposed. The approximate width of the drop structure is 1,410 feet. The drop structure is located 500 feet downstream of I-10 because of the large transmission tower located 150 feet downstream of the bridge.

The grade-control structure should not be constructed until the instream gravel pits located 1,500 feet downstream of the bridge are backfilled. A headcut progressing upstream from the gravel pits could cause the channel bed to lower more than the equilibrium bed slope would indicate, thereby possibly undermining the drop structure.

7.2.4.3 Backfilling of Gravel Pits

Several abandoned instream gravel pits will have to be backfilled before channelization occurs in this reach. Two large pits located approximately 1,500 feet downstream of I-10 have volumes of 96,000 cubic yards and 74,000 cubic yards, respectively. These pits extend the full width of the channel.

7.2.4.4 I-10 Bridge Pier Protection

The I-10 bridge piers extend approximately 23 feet below the present thalweg elevation. The piers are circular in shape and have a diameter of 3.33 feet. The computed local scour depth for the piers was 15.4 feet at the SPF peak discharge of 142,000 cfs and the general bed response near the bridge is slight degradation. Thus, the scour potential is severe near the piers.

A rather sharp bend exists in the proposed channel between McDowell Road and I-10 due to the locations of the two bridge crossings. The bend may result in several hundred feet of the eastern portion of the bridge, becoming an ineffective flow area. The unit width discharge near the west section of the bridge will increase, resulting in larger velocities than predicted in HEC-2 and a larger local scour potential. Further, the centrifugal force acting on the outside of the bend will tend to increase velocities and sediment transport rates.

Therefore a riprap blanket protection is recommended at the crossing. The blanket will extend 20 feet in all directions around the piers and the thickness of the blanket should be three feet. A one foot thick gravel filter beneath the riprap is recommended. The top elevation of the blanket should be at elevation 970.0 feet.

7.2.4.5 Protection of Southern Pacific Railroad Bridge Piers

The local scour potential for the SPF peak discharge for channelized conditions at the SPRR crossing is 19.2 feet, which is actually less than the local scour potential for existing conditions, which is 19.5 feet. The SPRR crossing has wall piers which have an effective obstruction width of six feet eight inches. With such a large obstruction width, debris will probably accumulate along the sides of the walls, further accelerating the local scour potential. The piers are buried approximately 30 feet below the bed, and therefore riprap blanket protection around the piers is recommended to retard the local scour process.

The blanket will extend 20 feet around the concrete piers in all directions, and a continuous riprap blanket is recommended for protecting the wooden trestle portion of the bridge. The thickness of the riprap should be three feet with a D_{50} size of 18 inches. A gravel filter beneath the riprap should be one foot thick. The top of the blanket should be at elevation 951.8 feet.

7.2.4.6 Protection of Buckeye Road Bridge Piers

The local scour potential at Buckeye Road for existing and channelized conditions is approximately 14.1 feet for the SPF. Presently the supports are buried approximately 28 feet below the channel bed; however, the piers are only buried five feet. The piers rest on pile caps and piles are driven another 23 feet below the piers. With local scour depths approaching 14 feet, some of the friction support will be lost in the piles, thereby decreasing the bearing support of the bridge. Therefore, it is recommended that riprap blanket protection be provided around the piers.

The riprap blanket should extend 20 feet in all directions around the pier and the top of the blanket should be at elevation 951.5 feet. The thickness of the blanket should be three feet and the D_{50} size of riprap 18 inches. A one-foot gravel filter meeting the Arizona Department of Transportation criteria should be provided beneath the riprap.

7.2.5 Cost of Without-Siphon Alternative

Table 7.2 compares the cost of providing riprap protection of levees, dikes and embankments with the cost of providing soil cement protection for the without-siphon alternative for the following reaches:

- Camelback Road to Indian School Road
- Indian School Road to Thomas Road
- Thomas Road to I-10
- I-10 to Buckeye Road

Only riprap protection was considered for levees between Indian School Road and 1,300 feet downstream of the RID flume, because backfilling of deep gravel pits may lead to settling of levees, which could result in cracking of soil cement. The estimated total cost of channelization between Buckeye Road and Camelback Road for riprap protection of levees, dikes, etc. is $30,776,885, and for soil cement protection, levees, dikes, etc., is $27,253,620.

Appendix A includes tables which summarize the cost of design components per reach and itemizes quantities, unit costs and total costs for each design component in each reach. A large imbalance of excavation material results from this alternative due to the lowering of the bed to provide three feet of freeboard at bridge crossings and in particular at the RID crossing. Allowances will have to be made to dispose of the excess material in back of the levees.

7.3 Description of Alternative With Siphon at RID Crossing

This alternative is similar to the without-siphon alternative except the RID flume is replaced with a siphon underneath the river. By replacing the flume with a siphon the freeboard requirement and foundation protection at the existing flume do not dictate downstream and upstream bed profiles.

Specific changes that result from the siphon being installed are:

1. Elimination of a grade-control structure immediately downstream of the RID flume.

2. Elimination of RID flume pier protection.

3. Less channel excavation to lower the water-surface profile downstream of the RID crossing.

Table 7.2 Summary of Costs per Reach for Without-Siphon Alternative.

Reach	Cost with Riprap, Dike and Levee Protection	Cost with Soil Cement Dike and Levee Protection
ISRB to Camelback Road	$ 2,981,195	$ 2,718,610
Thomas Road to ISRB	8,033,015	8,033,015*
I-10 to Thomas Road	10,172,760	8,544,535
Buckeye Road to I-10	9,589,915	7,957,460
Total	$30,776,885	$27,253,620
Cost per Mile	$ 6,155,400	$ 5,450,700

*Reach has riprap protection of levees.

4. Increase in drop structure heights from three to four feet at the drops located 200 feet downstream of Thomas Road, 2,200 feet upstream of the proposed McDowell Road bridge, and 500 feet downstream of I-10.

5. Slight decrease in levee heights due to steeper gradients; however, this accompanies a slight increase in toe-down depths.

Figures B.1 through B.6 in Appendix B show the resultant changes in levee heights, toe-down depths, and proposed channel bed and drop structure heights between Camelback Road and Buckeye Road. The channelization quantities balance closer for this alternative as less excavation is required.

7.3.1 Description of Siphon

SLA investigated the feasibility of replacing the existing RID elevated flume with a combination canal-inverted siphon system designed to convey the water under the Agua Fria River. The existing elevated flume has the following characteristics:

1. Length is 5,959 feet at a slope of 0.0006.

2. Semicircular cross section, with a radius of about 6.5 feet.

3. Design discharge is 390 cfs with no reserve freeboard.

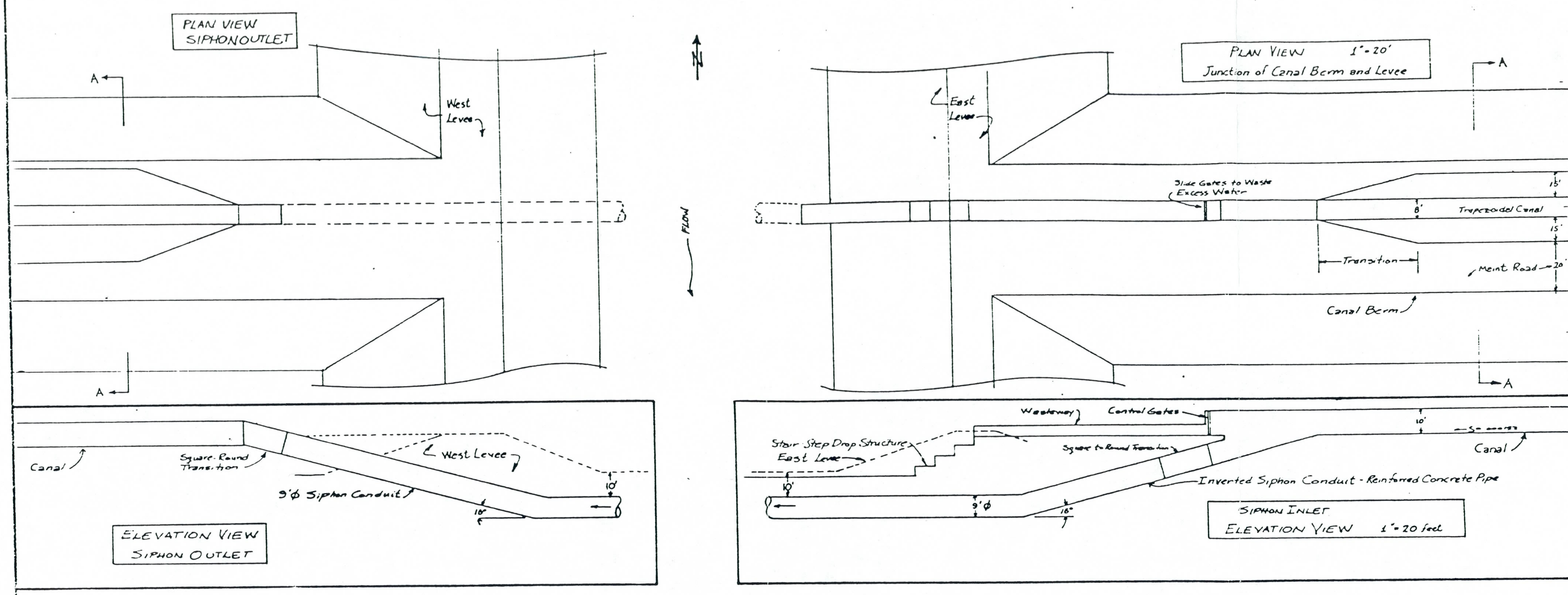
The upstream end of the flume begins approximately 2,200 feet east of the present RID overflow structure. The flume conveys water in a generally westerly direction to the extreme west edge of the historical river overbank, the downstream limit of the elevated flume.

The proposed canal-siphon alternative consists of the following (see Figures 7.11 and 7.12):

1. Construction of an inverted siphon 10 feet below the design grade of the Agua Fria River. The upstream transition to the siphon will begin a short distance east of the east levee. The downstream transition will terminate a short distance west of the west levee.

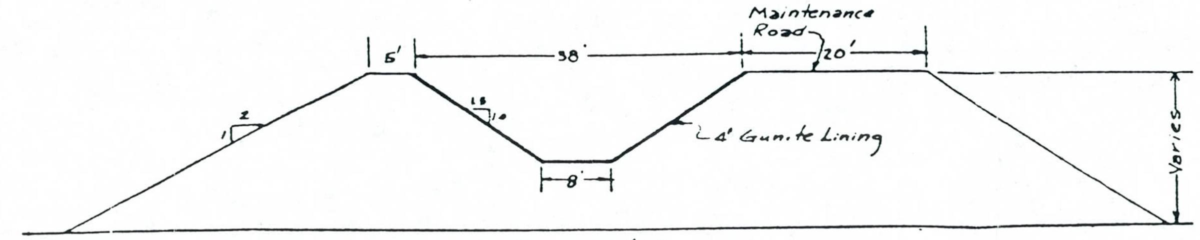
2. Construction of an open channel canal, having the same cross-sectional shape and slope as the existing canal, between the ends of the siphon and the ends of the existing canal.

3. Dismantling and removing the existing steel flume with its concrete piers and footings.



NOTES

1. Top of Ground Line corresponds to top of canal.
2. New canal cross section is same as existing canal cross section.
3. Canal lining constructed of gunite. Transitions constructed of reinforced concrete.
4. Conduit is 108-in. R.C.P.



SECTION A-A
CANAL BERM X-SECTION
1"=10 feet

REDUCED DRAWINGS

SLA
SIMONS, LI & ASSOCIATES, INC.
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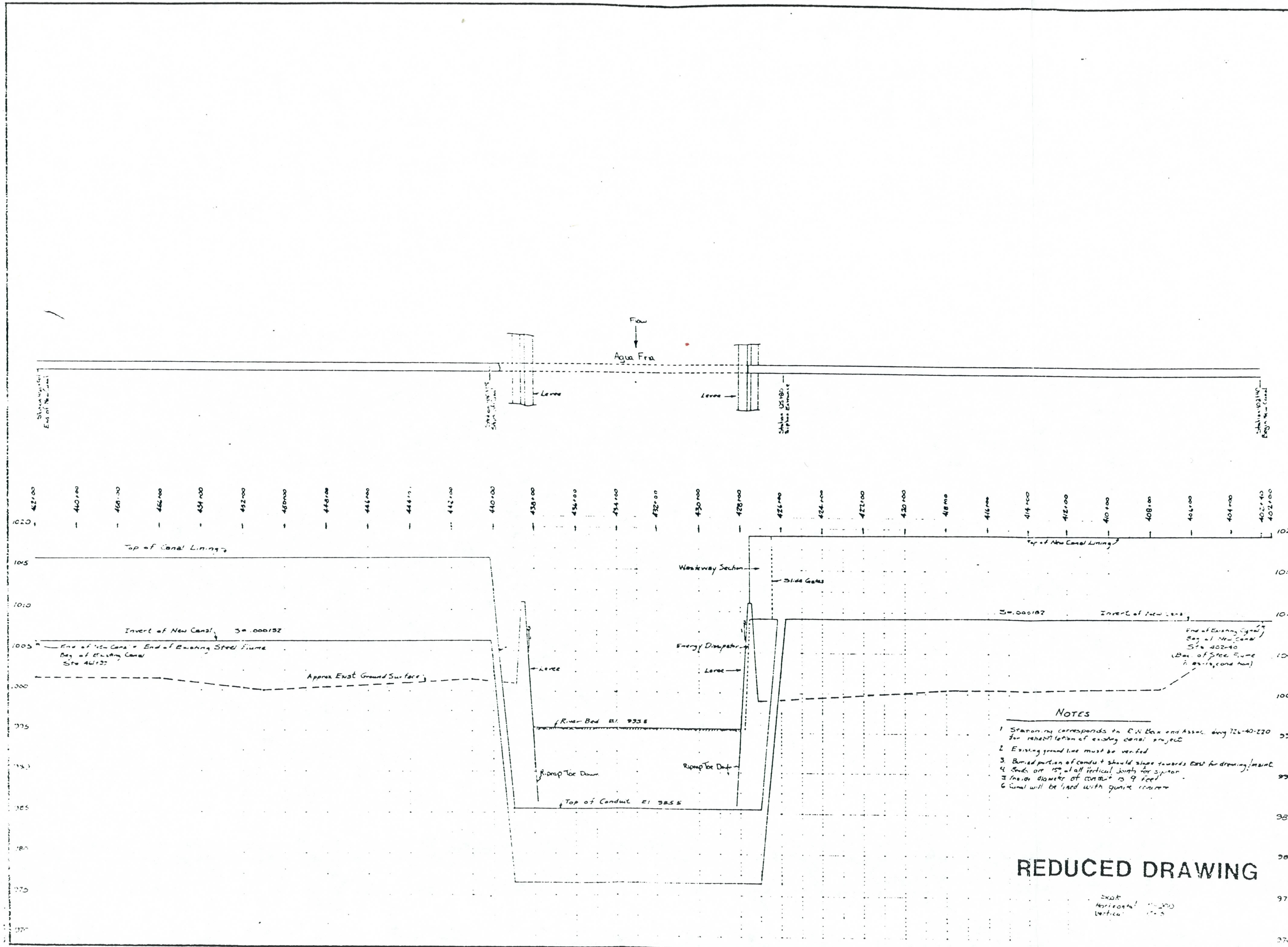
INITIAL CONCEPT PLAN
AGUA FRIA CROSSING
— INVERTED SIPHON —
— STRUCTURES —

Project No.	AZ-W...
Date	2-22-84
Design	LJS
Drawn	LJS
Check	
Revisions	

Figure 7.12



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INITIAL CONCEPT PLAN
AGUA FRIA RIVER CROSSING
– INVERTED SIPHON –
– PROFILES –

Date	2-22-84
Design	LJS
Drawn	LJS
Check	
Revisions	

Figure 7.11

Hydraulic analyses were conducted to determine the geometric properties of both the canal extensions and the siphon conduit and transitions. SLA was provided profile and cross section data describing the existing canal upstream of the steel flume. No data were received that covered the canal downstream of the flume. The canal upstream of the flume has an average slope of about 0.000152. It was assumed that this slope is also valid for the canal downstream of the flume. According to eyewitness accounts, the water surface is about one foot below the top of the canal banks, at the design discharge of about 390 cfs. The canal banks are about 10 feet above the canal invert. The side slopes are 1.5:1 and the bottom width averages about eight feet. The canal is lined with gunite.

The primary canal-siphon design constraints were:

1. The entire existing steel flume shall be removed.

2. The freeboard in the canal shall not be less than one foot.

The hydraulic analysis proceeded in the following sequence:

1. The distance between the upstream end of the siphon and the downstream end of the existing canal on the east side of the river is approximately 2,400 feet. The distance between the downstream end of the siphon and the upstream end of the existing canal on the west side of the river is approximately 2,100 feet, for a total length of new canal equal to about 4,500 feet. The existing flume is installed at a slope of 0.0006. The new canal extensions will be constructed to a grade of 0.000152. Therefore, the total change in energy grade line that may be caused by the siphon and transitions is computed as follows:

   $$4500 \text{ feet} \times (0.0006 - 0.000152) = 2.02 \text{ feet}$$

   A siphon conduit having a diameter of 8.5 to 9.0 feet will be required, depending upon transition design.

2. A series of backwater curve calculations were made in order to determine the influence of the siphon upon the magnitude of the freeboard in both the new and existing canals. The results indicate that the one foot of freeboard can be maintained.

3. An overflow/canal drainage structure has been included in the conceptual design. The data indicating the required rate of wasteway discharge has not yet been received by SLA.

A preliminary cost estimate to construct the elevated canal extensions and siphon and transitions has been made. A cost breakdown is summarized in Table 7.3. Note that the cost of the canal sections and removing the flume is

Table 7.3. Cost Estimate for Inverted Siphon with Approach Channels.

Item	Quantity	Unit Cost	Total Cost
Canal berm upstream of siphon	174,463 yd^3	$2.50/$yd^3$	$ 436,160
Canal berm downstream of siphon	120,762 yd^3	$2.50/$yd^3$	$ 301,905
Excavate trapezoidal canal	40,000 yd^3	$3.50/$yd^3$	$ 140,000
Gunite lining (4" thick)	22,500 yd^2	$20/$yd^2$	$ 450,000
Dismantle existing flume	5,960 ft	$50/ft	$ 300,000
Siphon pipe transitions	2	50,000 ea	$ 100,000
108" RCP	1,500 ft	$400/ft	$ 600,000
Canal drainage overflow structure	1	40,000/ea	$ 40,000
		Subtotal	$2,368,065
10% contingencies and construction supervision			$ 236,805
		Total	$2,604,870

about twice that of the siphon itself. During the final design phase it may prove advisable to investigate the feasibility of increasing the length of the conduit and decreasing the length of the canal extensions in order to minimize total cost. More detailed information will be required concerning the topography of the area to be covered by the canal extensions. The entrance and exit transitions on the siphon should be carefully analyzed in order to ensure that minimum energy will be lost. The design of the wasteway section could impact the transition design at its associated coefficients of energy loss.

7.3.2 Cost of With-Siphon Alternative

Table 7.4 compares the cost of providing riprap protection of levees, dikes and embankments with the cost of providing soil cement protection for the with-siphon alternative for the following reaches:

- Camelback Road to Indian School Road
- Indian School Road to Thomas Road
- Thomas Road to I-10
- I-10 to Buckeye Road

Only riprap protection was considered for levee protection between Indian School Road and Thomas Road.

Appendix A includes tables which summarize the cost of design components for each reach and itemizes quantities, unit costs and total costs for each item.

The without-siphon alternative considering soil cement protection of levees and dikes is approximately $740,000 cheaper ($27,253,620 vs. $27,946,225) than the with-siphon alternative, or about three percent cheaper. However, if funding from the Soil Conservation Service were available to subsidize the cost of the siphon, that alternative may become more economically feasible than protecting the existing flume.

Table 7.4 Summary of Costs per Reach for Siphon Alternative.

Reach	Cost with Riprap, Dike and Levee Protection	Cost with Soil Cement Dike and Levee Protection
ISRB to Camelback Road	$ 2,959,755	$ 2,712,790
Thomas Road to ISRB	8,947,730	8,947,730*
I-10 to Thomas Road	9,847,590	8,222,650
Buckeye Road to I-10	9,734,775	8,113,055
Total	$31,489,850	$27,996,225
Cost per Mile	$ 6,298,000	$ 5,599,200

*Reach has riprap protection of levees.

VIII. CONCLUSIONS

The following are conclusions regarding the channelization response to the Standard Project Flood in the Agua Fria.

1. Channelization between Camelback and Buckeye Roads will improve the discharge-carrying capacity of the Agua Fria River. Peak discharges will increase in the lower reaches of channelization due to more efficient flow conveyance.

2. Channelization will contain the SPF and provide three feet of freeboard. Thus the Standard Project Flood plain width and subsequently the 100-year flood plain width reduce significantly compared to existing conditions.

3. Several flow breakout areas occur downstream of Buckeye Road during the SPF. This includes the east overbank 500 feet upstream of Broadway Road and just downstream of Buckeye Road on the west overbank.

4. Three feet of freeboard is provided at all major bridge and flume crossings for the SPF, with the exception of the SPRR crossing. The freeboard at this crossing is 1.7 feet for existing conditions and 1.3 feet for channelized conditions. The SPRR bridge is located at the downstream end of the proposed channelization, and flowage easements will be purchased for the existing 100-year flood plain downstream of the crossing, and since 3.7 feet of freeboard exists for the 100-year peak discharge for channelized conditions, it is not recommended that the SPRR crossing be raised.

5. The long-term bed response of the Agua Fria River considering channelization between Camelback and Buckeye Roads is degradation. Grade controls are suggested to stabilize the channelized areas between Buckeye and Camelback Roads.

6. Local scour analyses at existing and proposed bridge crossings in the study reach of the Agua Fria indicate protection of bridge piers should be implemented at Indian School Road, I-10, the SPRR, Buckeye Road and the RID flume (if an inverted siphon does not replace the existing flume).

7. The engineering geomorphic analysis agrees with the qualitative geomorphic analysis that channelization will result in degradation of the bed. The aggradation/degradation analysis indicated the need for several drop structures located throughout the study reach.

8. Computer modeling of the channel bed response to the SPF resulted in minimal aggradation/degradation throughout the study reach.

Two channelization alternatives were examined between Camelback Road and Buckeye Road. These alternatives included:

1. Providing channelization to pass the SPF through all bridge and flume crossings while providing three feet of freeboard. This involved lowering the bed substantially downstream of the RID flume to achieve the required freeboard.

2. Providing channelization to pass the SPF through all bridge and flume crossings while providing three feet of freeboard except at the RID flume where an inverted siphon will replace the existing flume.

Alternative 1 was approximately $740,000 less than Alternative 2 ($27,253,620 vs. $27,996,225). However, should the Soil Conservation Service (SCS) subsidize part of the siphon costs, Alternative 2 could become less expensive than Alternative 1. Alternative 1 has several disadvantages, including (1) it creates a massive quantity of excess material due to the large excavations associated with lowering the channel bed, (2) the flow is near critical at the RID flume crossing, causing unstable wave conditions and high velocities which could have detrimental effects on the flume's safety, and (3) a grade-control structure downstream of the flume will be required to prevent headcuts from progressing through the flume.

Alternative 2 also has some disadvantages in that (1) the gradient of the existing flume is so flat (0.06 percent) that sizing an inverted siphon with a discharge capacity of 390 cfs (the current maximum discharge capacity of the flume) and not creating adverse backwater effects in the upstream channel becomes difficult, (2) the right of way for constructing an approach and outlet channel to the inverted siphon is narrow, which results in steep (2:1) back sides of the channel, and (3) the possibility of the SCS not subsidizing part of the costs of the siphon make this alternative more expensive.

APPENDIX A
Summary of Costs and
Quantities for Proposed Standard
Project Flood Channelization

Table A.1. Cost Summary of Items Between Indian School Road and Camelback Road for Siphon Alternative.

Item		Riprap Protection of Dikes and Levees	Soil Cement Protection of Dikes and Levees
Channelization		$ 430,065	$ 430,065
Levee		$ 438,250	$ 317,275
Transverse Dike #2		$ 534,420	$ 485,770
Transverse Dike #1		$ 209,295	$ 190,440
West Bank Transverse Dike		$ 80,290	$ 68,250
Spur Dike		$ 124,445	$ 100,450
Floodwall and Embankment Protection of Indian School Road		$ 158,010	$ 158,010
	Subtotal	$1,974,775	$1,750,260
10% Contingencies and Construction Supervision		$ 197,480	$ 175,030
Land Acquisition		$ 787,500	$ 787,500
	Total	$2,959,755	$2,712,790

Table A.2. Cost Summary of Items Between Thomas Road and Indian School Road for Siphon Alternative.

Item		Riprap Protection of Levees
Channelization		$ 486,900
Drop Structure (soil cement)		$ 812,885
Levees		$ 2,757,560
Transmission Tower Protection		$ 200,000
Gravel Pit Restoration		$ 1,212,500
ISRB Pier Protection		$ 296,390
Siphon and Canal		$ 2,368,065
	Subtotal	$ 8,134,300
10% Contingencies and Construction Supervision		$ 813,430
	Total	$ 8,947,730

Table A.3. Cost Summary of Items Between I-10 and Thomas Road for Siphon Alternative.

Item		Riprap Protection of Dikes and Levees	Soil Cement Protection of Dikes and Levees
Channelization		$ 915,300	$ 915,300
2 Drop Structures (soil cement)		$1,747,490	$1,747,490
Levees		$4,737,660	$3,260,440
Transmission Tower Protection		$ 800,000	$ 800,000
I-10 Siltation Basin		$ 544,950	$ 544,950
	Subtotal	$8,745,400	$7,268,180
10% Contingencies and Construction Supervision		$ 874,540	$ 726,820
Land Acquisition		$ 227,650	$ 227,650
	Total	$9,847,590	$8,222,650

Table A.4. Cost Summary of Items Between Buckeye Road and I-10 for Siphon Alternative.

Item		Riprap Protection of Dikes and Levees	Soil Cement Protection of Dikes and Levees
Channelization		$ 667,765	$ 667,765
Drop Structure (soil cement)		$ 960,500	$ 960,500
Transmission Tower Protection		$ 700,000	$ 700,000
Levees		$4,603,495	$3,129,205
I-10 Bridge Pier Protection		$ 553,060	$ 553,060
Southern Pacific Railroad Pier Protection		$ 230,255	$ 230,255
Buckeye Road Bridge Pier Protection		$ 240,400	$ 240,400
Gravel Pit Restoration		$ 212,500	$ 212,500
	Subtotal	$8,167,975	$6,693,685
10% Contigencies and Construction Supervision		$ 816,800	$ 669,370
Land Acquisition		$ 750,000	$ 750,000
	Total	$9,734,775	$8,113,055

Table A.5. Cost Summary of Items Between Indian School Road and Camelback Road for Without-Siphon Alternative.

Item		Riprap Protection of Dikes and Levees	Soil Cement Protection of Dikes and Levees
Channelization		$ 430,065	$ 430,065
Levee		$ 457,740	$ 322,570
Transverse Dike #2		$ 534,420	$ 485,770
Transverse Dike #1		$ 209,295	$ 190,440
West Bank Transverse Dike		$ 80,290	$ 68,250
Spur Dike		$ 124,445	$ 100,450
Floodwall and Embankment Protection of Indian School Road		$ 158,010	$ 158,010
	Subtotal	$1,994,265	$1,755,555
10% Contingencies and Construction Supervision		$ 199,430	$ 175,555
Land Acquisition		$ 787,500	$ 787,550
	Total	$2,981,195	$2,718,610

Table A.6. Cost Summary of Items Between Thomas Road and Indian School Road for Without-Siphon Alternative.

Item		Riprap Dike and Levee Protection
Channelization		$ 892,620
ISRB Drop Structure (soil cement)		$ 812,885
RID Flume Drop Structure and Riprap Blanket (soil cement)		$1,116,510
Levees		$2,771,835
Gravel Pit Restoration		$1,212,500
Indian School Road Bridge Pier Protection		$ 296,390
Transmission Tower Protection		$ 200,000
	Subtotal	$7,302,740
10% Contingencies and Construction Supervision		$ 730,275
		$8,033,015

Table A.7. Cost Summary of Items Between I-10 and Thomas Road for Without-Siphon Alternative.

Item		Riprap Protection of Dikes and Levees	Soil Cement Protection of Dikes and Levees
Channelization		$ 1,572,525	$1,572,525
2 Drop Structures (soil cement)		$ 1,415,805	$1,415,805
Levees		$ 4,707,730	$3,227,525
Transmission Tower Protection		$ 800,000	$ 800,000
I-10 Siltation Basin		$ 544,950	$ 544,950
	Subtotal	$ 9,041,010	$7,560,805
10% Contingencies and Construction Supervision		$ 904,100	$ 756,080
Land Acquisition		$ 227,650	$ 227,650
	Total	$10,172,760	$8,544,535

Table A.8. Cost Summary of Items Between Buckeye Road and I-10 for Without-Siphon Alternative.

Item		Riprap Protection of Dikes and Levees	Soil Cement Protection of Dikes and Levees
Channelization		$ 787,980	$ 787,980
2 Drop Structures (soil cement)		$ 786,180	$ 786,180
Levees		$4,525,910	$3,041,860
Transmission Tower Protection		$ 700,000	$ 700,000
Gravel Pit Restoration		$ 212,500	$ 212,500
I-10 Bridge Pier Protection		$ 553,060	$ 553,060
Southern Pacific Railroad Pier Protection		$ 230,255	$ 230,255
Buckeye Road Bridge Pier Protection		$ 240,400	$ 240,400
	Subtotal	$8,036,285	$6,552,235
10% Contingencies and Construction Supervision		$ 803,630	$ 655,225
Land Acquisition		$ 750,000	$ 750,000
	Total	$9,589,915	$7,957,460

Table A.9. Preliminary Cost and Quantity Estimates Between ISRB and Camelback Road for the Siphon Alternative and Riprap Protection of Dikes and Levees.

Item	Channelization	Levee	Transverse Dikes	Spur Dike	ISR Floodwall & Embankment Protection	Total	Unit Cost	Total Cost
Common Fill		16,050 yd^3	50,345 yd^3	8,000 yd^3		74,395 yd^3	\$ 1.25/yd^3	\$ 92,995
Drainage Excavation	477,850 yd^3					477,850 yd^3	0.90/yd^3	\$ 430,065
Structural Excavation						--	2.00/yd^3	--
Special Backfill						--	2.00/yd^3	--
Riprap		18,660 yd^3	32,120 yd^3	4,870 yd^3	1,325 yd^{3*}	56,975 yd^3	22.00/yd^3	\$1,253,450
Filter Fabric (includes 6" soil cover)		5,110 yd^2	36,290 yd^2	4,870 yd^2	5,950 yd^2	52,220 yd^2	1.50/yd^3	\$ 78,330
Soil Cement						--	22.50/yd^2	--
Jersey Barrier					3,610 ft.	3,610	30/ft	\$ 108,300
Raise 113th Avenue					--	--	--	\$ 11,635
							Subtotal	\$1,974,775
							10% Contingencies and Construction Supervision	\$ 197,480
							Land Acquisition (157.5 acres)	\$ 787,500
							Total	\$2,959,755

*D_{50} of Riprap along Indian School Road is four inches.

Table A.10. Preliminary Cost and Quantity Estimate of Items Between ISRB and Camelback Road for the Siphon Alternative and Soil Cement Protection of Dikes and Levees.

Item	Channelization	Levee	Transverse Dikes	Spur Dike	ISR Floodwall & Embankment Protection	Total	Unit Cost	Total Cost
Common Fill		8,120 yd^3	13,355 yd^3	2,240 yd^3		23,715 yd^3	\$ 1.25/yd^3	\$ 29,645
Drainage Excavation	477,850 yd^3					477,850 yd^3	0.90/yd^3	430,065
Structural Excavation						--	2.00/yd^3	--
Special Backfill						--	2.00/yd^3	--
Riprap					1,325 yd^3*	1,325 yd^3	22.00/yd^3	29,150
Filter Fabric (includes 6" soil cover)					5,950 yd^2	5,950 yd^2	1.50/yd^2	8,925
Soil Cement		13,650 yd^3	32,345 yd^3	4,340 yd^3		50,335 yd^3	22.50/yd^3	1,132,540
Jersey Barrier					3,610 ft	3,610 ft	30/ft	108,300
Raise 113th Avenue						--	--	11,635
							Subtotal	\$1,750,260
							10% Contingencies and Construction Supervision	\$ 175,030
							Land Acquisition (157.5 acres)	\$ 787,500
							Total	\$2,712,790

*D_{50} of riprap along Indian School Road is four inches.

Table A.11. Preliminary Cost and Quantity Estimate of Items Between Thomas Road and ISRB for the Siphon Alternative with Riprap Protection of Levees.

Item	Channelization	Levee	Drop Structure	ISRB Pier Protection	Gravel Pit Restoration	Total	Unit Cost	Total Cost
Common Fill		177,670 yd^3	16,920 yd^3		870,000 yd^3	1,064,590 yd^3	\$ 1.25/yd^3	\$ 1,330,740
Drainage Excavation	541,000 yd^3			14,410 yd^3		555,410 yd^3	0.90/yd^3	499,870
Structural Excavation			58,280 yd^3			58,280 yd^3	2.00/yd^3	116,560
Special Backfill			16,920 yd^3			16,920 yd^3	2.00/yd^3	33,840
Riprap		109,040 yd^3	3,130 yd^3	9,610 yd^3		121,780 yd^3	22.00/yd^3	2,679,160
Filter Fabric (includes 6" soil cover)		91,060 yd^3				91,060 yd^3	1.50/yd^2	136,590
Gravel Filter			1,565 yd^3	4,800 yd^3		6,365 yd^3	15.00/yd^3	95,475
Soil Cement			24,400 yd^3			24,400 yd^3	22.50/yd^3	549,000
Trash Removal					170,000 yd^3	170,000 yd^3	0.50/yd^3	85,000
Sludge Removal					20,000 yd^3	20,000 yd^3	2.00/yd^3	40,000
Transmission Tower Protection						2 towers	100,000/tower	200,000
Siphon & Canal						1 unit	---	2,368,065
							Subtotal	\$ 8,134,300
							10% Contingencies and Construction Supervision	\$ 813,430
							Total	\$ 8,947,730

Table A.12. Preliminary Cost and Quantity Estimate of Items Between I-10 and Thomas Road for the Siphon Alternative with Riprap Protection of Levees.

Item	Channelization	Levees	2 Drop Structures	I-10 Siltation Basin	Total	Unit Cost	Total Cost
Common Fill		254,320 yd^3	39,440 yd^3		293,760 yd^3	$1.25/$yd^3$	$ 367,200
Drainage Excavation	1,017,000 yd^3			440,000 yd^3	1,457,000 yd^3	0.90/yd^3	$ 1,311,300
Structural Excavation			132,650 yd^3		132,650 yd^3	2.00/yd^3	$ 265,300
Special Backfill			39,440 yd^3		39,440 yd^3	2.00/yd^3	$ 78,880
Riprap		190,600 yd^3	4,880 yd^3		195,480 yd^3	22.00/yd^3	$ 4,300,560
Filter Fabric (includes 6" soil cover)		151,040 yd^2			151,040 yd^2	1.50/yd^2	$ 226,560
Gravel Filter			2,440 yd^3		2,440 yd^3	15/yd^3	$ 36,600
Soil Cement			53,780 yd^3	6,420 yd^3	60,200 yd^3	22.50/yd^3	$ 1,354,500
3' Reinforced Concrete Pipe				50 ft	50 ft	50 ft	$ 2,500
1 Flap Gate				1 unit	1	2,000/unit	$ 2,000
Transmission Tower Protection					8 towers	100,000/tower	$ 800,000
						Subtotal	$ 8,745,400
						10% Contingencies and Construction Supervision	$ 874,540
						Land Acquisition (45.53 acres)	$ 227,650
						Total	$ 9,847,590

Table A.13. Preliminary Cost and Quantity Estimate Between I-10 and Thomas Road for the Siphon Alternative with Soil Cement Protection of Levees.

Item	Channelization	Levees	2 Drop Structures	I-10 Siltation Basin	Total	Unit Cost	Total Cost
Common Fill		184,560 yd^3	39,440 yd^3		224,760 yd^3	\$ 1.25/yd^3	\$ 280,000
Drainage Excavation	1,017,000 yd^3			440,000 yd^3	1,457,000 yd^3	0.90/yd^3	\$ 1,311,300
Structural Excavation			132,650 yd^3		132,650 yd^3	2.00/yd^3	\$ 265,300
Special Backfill			39,440 yd^3		39,440 yd^3	2.00/yd^3	\$ 78,880
Riprap			4,880 yd^3		4,880 yd^3	22/yd^3	\$ 107,360
Gravel Filter			2,440 yd^3		2,440 yd^3	15/yd^3	\$ 36,600
Soil Cement		134,655 yd^3	53,780 yd^3	6,420 yd^3	194,855 yd^3	22.50/yd^3	\$ 4,384,240
3' Reinforced Concrete Pipe				50 ft	50 ft	50 ft	\$ 2,500
1 Flap Gate				1 unit	1 unit	2,000/unit	\$ 2,000
Transmission Tower Protection					8 towers	100,000/tower	\$ 800,000
						Subtotal	\$ 7,268,180
						10% Contingencies and Construction Supervision	\$ 726,820
						Land Acquisition (45.53 acres)	\$ 227,650
						Total	\$ 8,222,650

Table A.14. Preliminary Cost and Quantity Estimate Between Buckeye Road and I-10 for the Siphon Alternative with Riprap Protection of Levees.

Item	Channelization	Levees	Drop Structure	I-10 Bridge Pier Protection	SPRR Bridge Pier Protection	Buckeye Rd Bridge Pier Protection	Gravel Pit Restoration	Total	Unit Cost	Total Cost
Common Fill		183,540 yd^3	20,900 yd^3				170,000 yd^3	374,440 yd^3	$ 1.25/yd^3	$ 468,050
Drainage Excavation	741,960 yd^3			26,890 yd^3	11,195 yd^3	11,690 yd^3		791,735 yd^3	0.90/yd^3	712,560
Structural Excavation			71,020 yd^3					71,020 yd^3	2.00/yd^3	142,040
Special Backfill			20,900 yd^3					20,900 yd^3	2.00/yd^3	41,800
Riprap		189,065 yd^3	3,130 yd^3	17,930 yd^3	7,465 yd^3	7,790 yd^3		225,380 yd^3	22.00/yd^3	4,958,360
Gravel Filter Material			1,570 yd^3	8,960 yd^3	3,730 yd^3	3,900 yd^3		18,160 yd^3	15/yd^3	272,400
Filter Fabric (includes 6" soil cover)		143,095 yd^2						143,095 yd^2	1.50/yd^2	214,640
Soil Cement			29,250 yd^3					29,250 yd^3	22.50/yd^3	658,125
Transmission Tower Protection								7 towers	100,000 tower	700,000
									Subtotal	$8,167,975
									10% Contingencies and Construction Supervision	$ 816,800
									Land Acquisition (150 acres)	$ 750,000
									Total	$9,734,775

Table A.15. Preliminary Cost and Quantity Estimate Between Buckeye Road and I-10 for the Siphon Alternative and Soil Cement Protection of Levees.

Item	Channelization	Levees	Drop Structure	I-10 Bridge Pier Protection	SPRR Bridge Pier Protection	Buckeye Rd Bridge Pier Protection	Gravel Pit Restoration	Total	Unit Cost	Total Cost
Common Fill		152,475 yd^3	20,900 yd^3				170,000 yd^3	343,375 yd^3	\$ 1.25/yd^3	\$ 429,220
Drainage Excavation	741,960 yd^3			26,890 yd^3	11,195 yd^3	11,690 yd^3		791,735 yd^3	0.90/yd^3	712,560
Structural Excavation			71,020 yd^3					71,020 yd^3	2.00/yd^3	142,040
Special Backfill			20,900 yd^3					20,900 yd^3	2.00/yd^3	41,800
Riprap			3,130 yd^3	17,930 yd^3	7,465 yd^3	7,790 yd^3		36,315 yd^3	22/yd^3	798,930
Gravel Filter Material			1,570 yd^3	8,960 yd^3	3,730 yd^3	3,900 yd^3		18,160 yd^3	15/yd^3	272,400
Filter Fabric (includes 6" soil cover)								---	1.50/yd^2	---
Soil Cement		130,605 yd^3	29,250 yd^3					159,855 yd^3	22.50/yd^3	3,596,735
Transmission Tower Protection								7 towers	100,000/tower	700,000
									Subtotal	\$6,693,685
									10% Contingencies and Construction Supervision	\$ 669,370
									Land Acquisition (150 acres)	\$ 750,000
									Total	\$8,113,055

Table A.16. Preliminary Cost and Quantity Estimate Between ISRB and Camelback Road for the Siphon Alternative and Riprap Protection of Levees.

Item	Channelization	Levees	Transverse Dike	Spur Dike	ISR Flood-wall & Embankment Protection	Total	Unit Price	Total Cost
Common Fill		14,410 yd^3	50,345 yd^3	8,000 yd^3		72,755 yd^3	\$1.25 yd^3	\$ 90,945
Drainage Excavation	474,850 yd^3					477,850 yd^3	0.90 yd^3	430,065
Structural Excavation						--	2.00/yd^3	--
Special Backfill						--	2.00/yd^3	--
Riprap		18,930 yd^3	32,120 yd^3	4,870 yd^3	1,325 yd^{3*}	57,245 yd^3	22.00/yd^3	1,259,390
Gravel Filter						--	15/yd^3	--
Filter Fabric (includes 6" soil cover)		15,510 yd^2	36,290 yd^2	4,870 yd^2	5,950 yd^2	62,620 yd^2	1.50/yd^2	93,930
Soil Cement						--	22.50/yd^3	--
Jersey Barrier					3,610 ft	3,610 ft	30/ft	108,300
Raise 113th Ave						--	11,635/int	11,635
							Subtotal	\$1,994,265
						10% Contingencies and Construction Supervision		\$ 199,430
						Land Acquisition (157.51 acres)		\$ 787,500
							Total	\$2,981,195

*D_{50} of Riprap along Indian School Road is Four Inches.

Table A.17. Preliminary Cost and Quantity Estimate Between ISRB and Camelback Road for the Without-Siphon Alternative and Soil Cement Protection of Dikes and Levees.

Item	Channelization	Levees	Transverse Dike	Spur Dike	ISR Floodwall & Embankment Protection	Total	Unit Price	Total Cost
Common Fill		8,575 yd^3	13,355 yd^3	2,240 yd^3		24,170 yd^3	$1.25 yd^3	$ 30,215
Drainage Excavation	477,850 yd^3					477,850 yd^3	0.90 yd^3	430,065
Structural Excavation						--	2.00/yd^3	--
Special Backfill						--	2.00/yd^3	--
Riprap					1,325 yd^{3*}	1,325 yd^3	22.00/yd^3	29,150
Gravel Filter							15.00/yd^3	--
Filter Fabric (includes 6" soil cover)					5,950 yd^2	5,950 yd^2	1.50/yd^2	8,925
Soil Cement		13,860 yd^3	32,345 yd^3	4,340 yd^3		50,545 yd^3	22.50/yd^3	1,137,265
Jersey Barrier					3,610 ft	3,610 ft	30/ft	108,300
Raise 113th Ave.						1	11,635/int	11,635
							Subtotal	$1,755,555
							10% Contingencies and Construction Supervision	$ 175,555
							Land Acquisition (157.51 acres)	$ 787,500
							Total	$2,718,610

*D_{50} of riprap along Indian School Road is four inches.

Table A.18. Preliminary Cost and Quantity Estimate Between Thomas Road and Indian School Road for the Without-Siphon Alternative and Riprap Bank Protection.

Item	Channelization	Levees	ISRB Drop Structure	RID Flume Drop Structure	ISRB Pier Protection	Gravel Pit Restoration	Total	Unit Cost	Total Cost
Common Fill		155,380 yd^3	16,920 yd^3	19,010 yd^3		870,000 yd^3	1,061,310 yd^3	\$ 1.25/yd^3	\$1,326,635
Drainage Excavation	991,800 yd^3				14,410 yd^3		1,006,210 yd^3	0.90/yd^3	905,590
Structural Excavation			58,280 yd^3	63,690 yd^3			121,970 yd^3	2.00/yd^3	243,940
Special Backfill			16,920 yd^3	19,010 yd^3			35,930 yd^3	2.00/yd^3	71,860
Riprap		110,860 yd^3	3,130 yd^3	11,825 yd^3	9,610 yd^3		135,425 yd^3	22/yd^3	2,979,350
Gravel Filter			1,565 yd^3	5,915 yd^3	4,800 yd^3		12,280 yd^3	15/yd^3	184,200
Filter Fabric (includes 6" soil cover)		92,460 yd^2					92,460 yd^2	1.50/yd^2	138,690
Soil Cement			24,400 yd^3	25,710 yd^3			50,110 yd^3	22.50/yd^3	1,127,475
Transmission Tower Protection							2 towers	100,000/tower	200,000
Sludge Removal					20,000 yd^3		20,000 yd^3	2.00/yd^3	40,000
Trash Removal					170,000 yd^3		170,000 yd^3	0.50/yd^3	85,000
								Subtotal	\$7,302,740
							10% Contingencies and Construction Supervision		\$ 730,275
								Total	\$8,033,015

Table A.19. Preliminary Cost and Quantity Estimate Between I-10 and Thomas Road for the Without Siphon Alternative and Riprap Protection of Levees.

Item	Channelization	Levees	Drop Structures	I-10 Siltation Basin	Total	Unit Cost	Total Cost
Common Fill		206,540 yd^3	30,350 yd^3		236,890 yd^3	\$ 1.25/yd^3	\$ 296,115
Drainage Excavation	1,747,250 yd^3			440,000 yd^3	2,187,250 yd^3	0.90/yd^3	1,968,525
Structural Excavation			103,640 yd^3		103,640 yd^3	2.00/yd^3	207,280
Special Backfill			30,350 yd^3		30,350 yd^3	2.00/yd^3	60,700
Riprap		191,970 yd^3	4,880 yd^3		196,850 yd^3	22/yd^3	4,330,700
Gravel Filter Material			2,440 yd^3		2,440 yd^3	15/yd^3	36,600
Filter Fabric (includes 6" soil cover)		150,810 yd^2			150,810 yd^2	1.50/yd^2	226,215
Soil Cement			42,930 yd^3	6,420 yd^3	49,350 yd^3	22.50/yd^3	1,110,375
Transmission					8 towers	100,000/tower	800,000
Reinforced Concrete Pipe				50 ft	50 ft	50.00/ft	2,500
Flap Gate				1	1	2,000/gate	2,000
						Subtotal	\$ 9,041,010
						10% Contingencies and Construction Supervision	\$ 904,100
						Land Acquisition (45.52 acres)	\$ 227,650
						Total	\$10,172,760

Table A.20. Preliminary Cost and Quantity Estimate Between I-10 and Thomas Road for the Without-Siphon Alternative and Soil Cement Levee Protection.

Item	Channelization	Levees	Drop Structures	I-10 Siltation Basin	Total	Unit Cost	Total Cost
Common Fill		162,460 yd^3	30,350 yd^3		192,810 yd^3	\$ 1.25/yd^3	\$ 241,015
Drainage Excavation	1,747,250 yd^3			440,000 yd^3	2,187,250 yd^3	0.90/yd^3	1,968,525
Structural Excavation			103,640 yd^3		103,640 yd^3	2.00/yd^3	207,280
Special Backfill			30,350 yd^3		30,350 yd^3	2.00/yd^3	60,700
Riprap			4,880 yd^3		4,880 yd^3	22/yd^3	107,360
Gravel Filter Material			2,440 yd^3		2,440 yd^3	15/yd^3	36,600
Filter Fabric (includes 6" soil cover)					--	1.50/yd^2	--
Soil Cement		134,420	42,930 yd^3	6,420 yd^3	183,770 yd^3	22.50/yd^3	4,134,825
Transmission Tower Protection					8 towers	100,000/tower	800,000
Reinforced Concrete Pipe				50 ft	50 ft	50/ft	2,500
Flap Gate				1	1	2,000/gate	2,000
						Subtotal	\$ 7,560,805
						10% Contingencies and Construction Supervision	\$ 756,080
						Land Acquisition (45.52 acres)	\$ 227,650
						Total	\$ 8,544,535

Table A.21. Preliminary Cost and Quantity Estimate Between Buckeye Road and I-10 for the Without-Siphon Alternative and Riprap Protection of Levees.

Item	Channelization	Levees	Drop Structure	I-10 Pier Protection	Buckeye Road Pier Protection	Gravel Pit Restoration	SPRR Bridge Pier Protection	Total	Unit Cost	Total Cost
Common Fill		155,530 yd^3	16,180 yd^3			170,000 yd^3		341,710 yd^3	1.25/yd^3	427,135
Drainage Excavation	875,535 yd^3			26,890 yd^3	11,690 yd^3		11,195 yd^3	925,310 yd^3	0.90/yd^3	832,780
Structural Excavation			55,880 yd^3					55,880 yd^3	2.00/yd^3	111,760
Special Backfill			16,180 yd^3					16,180 yd^3	2.00/yd^3	32,360
Riprap		187,100 yd^3	3,130 yd^3	17,930 yd^3	7,790 yd^3		7,465 yd^3	223,415 yd^3	22/yd^3	4,915,130
Gravel Filter Material			1,570 yd^3	8,960 yd^3	3,900 yd^3		3,730 yd^3	18,160 yd^3	15/yd^3	272,400
Filer Fabric (includes 6" Soil Cover)		143,530 yd^2						143,530 yd^2	1.50/yd^2	215,295
Soil Cement			23,530 yd^3					23,530 yd^3	22.50/yd^3	529,425
Transmission Tower Protection								7 towers	100,000/tower	700,000
									Subtotal	$8,036,285
									10% Contingencies and Construction Supervision	$ 803,630
									Land Acquisition (150 acres)	$ 750,000
									Total	$9,589,915

Table A.22. Preliminary Cost and Quantity Estimate Between Buckeye Road and I-10 for the Without-Siphon Alternative and Soil Cement Protection of Levees.

Item	Channelization	Levees	Drop Structure	I-10 Pier Protection	Buckeye Road Pier Protection	Gravel Pit Restoration	SPRR Bridge Pier Protection	Total	Unit Cost	Total Cost
Common Fill		136,865 yd^3	16,180 yd^3			170,000 yd^3		323,045 yd^3	1.25/yd^3	403,805
Drainage Excavation	875,535 yd^3			26,890 yd^3	11,690 yd^3		11,195 yd^3	925,310 yd^3	0.90/yd^3	832,780
Structural Excavation			55,880 yd^3					55,880 yd^3	2.00/yd^3	111,760
Special Backfill			16,180 yd^3					16,180 yd^3	2.00/yd^3	32,360
Riprap			3,130 yd^3	17,930 yd^3	7,790 yd^3		7,465 yd^3	36,315 yd^3	22/yd^3	798,930
Gravel Filter Material			1,570 yd^3	8,960 yd^3	3,900 yd^3		3,730 yd^3	18,160 yd^3	15/yd^3	272,400
Filer Fabric (includes 6" Soil Cover)								--	1.50/yd^2	--
Soil Cement		127,590 yd^3	23,530 yd^3					150,120 yd^3	22.50/yd^3	3,400,200
Transmission Tower Protection								7 towers	100,000/tower	700,000
									Subtotal	$6,552,235
									10% Contingencies and Construction Supervision	$ 655,225
									Land Acquisition (150 acres)	$ 750,000
									Total	$7,957,460

APPENDIX B
Figures Showing Proposed Channelization Profile
Measures Between Camelback Road
and Buckeye Road for the Siphon Alternative

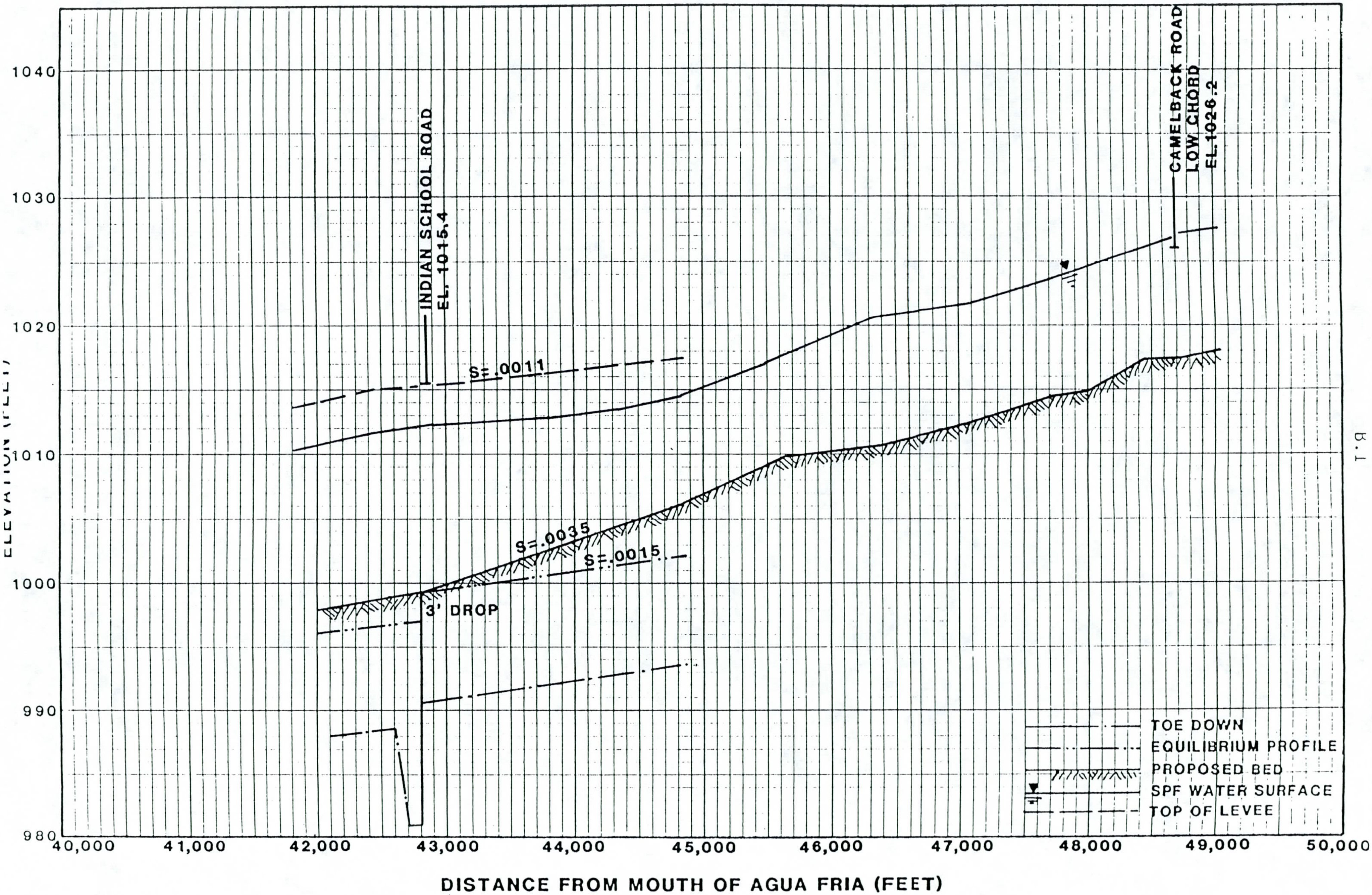

Figure B.1. Proposed bed profile, equilibrium bed profile, SPF water-surface elevation, levee height and toe-down depth between ISRB and Camelback Road.

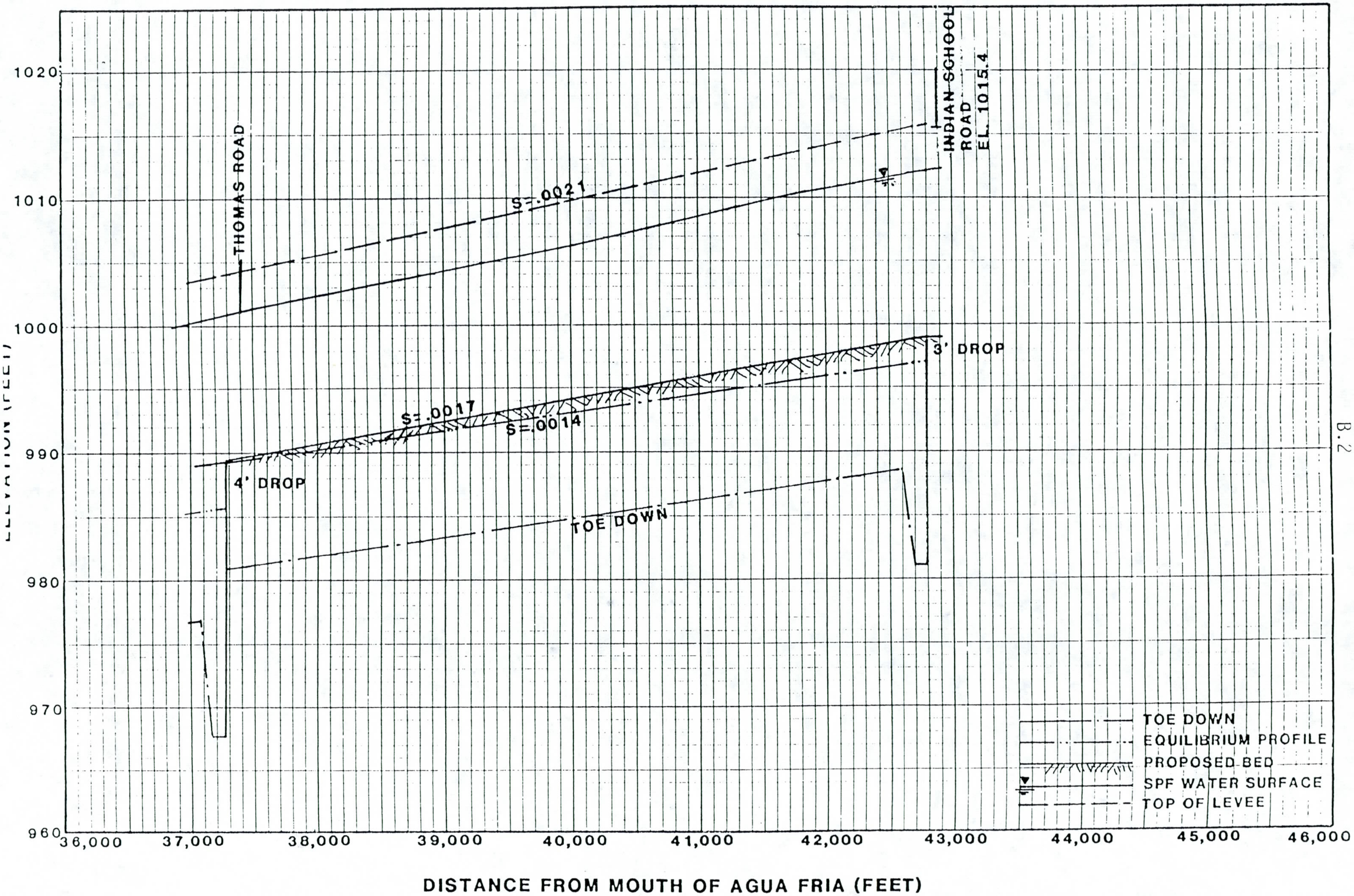

Figure B.2. Proposed bed profile, equilibrium bed profile, SPF water-surface elevation, levee height and toe-down depth between Thomas Road and ISRB.

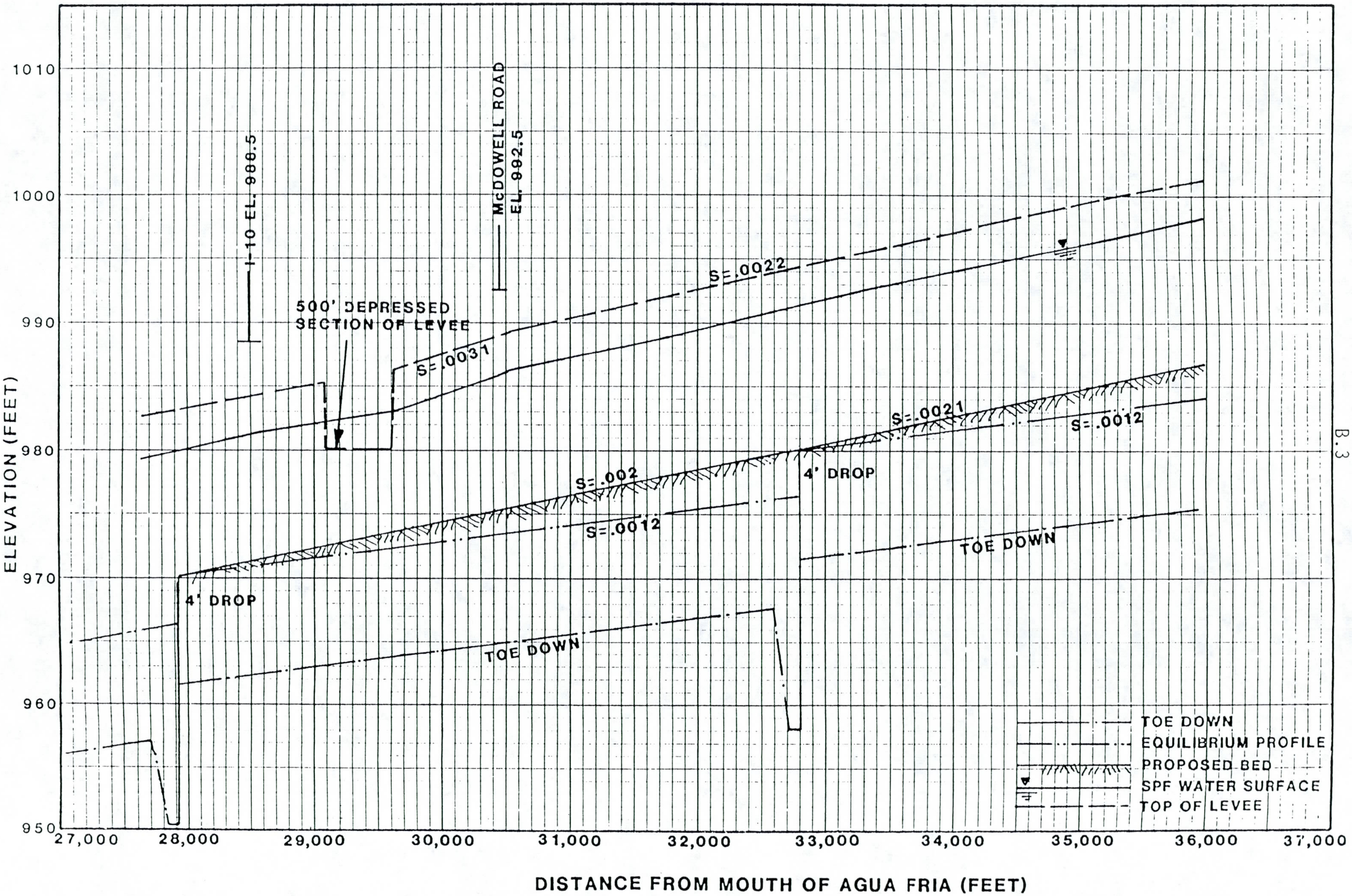

Figure B.3. Proposed bed profile, equilibrium bed profile, SPF water-surface elevation, levee height and toe-down depth between I-10 and Thomas Road for the east bank.

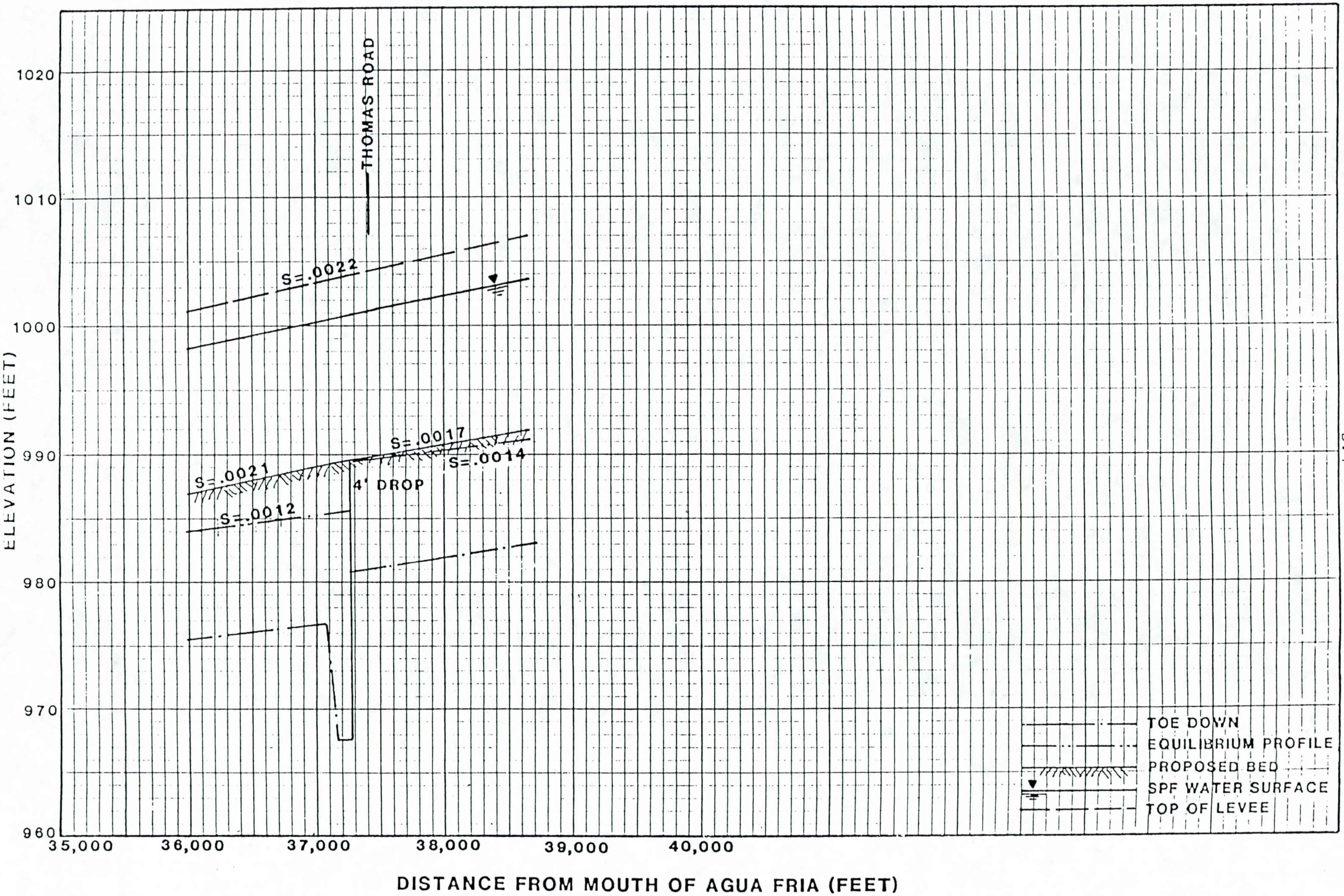

Figure B.3 (continued)

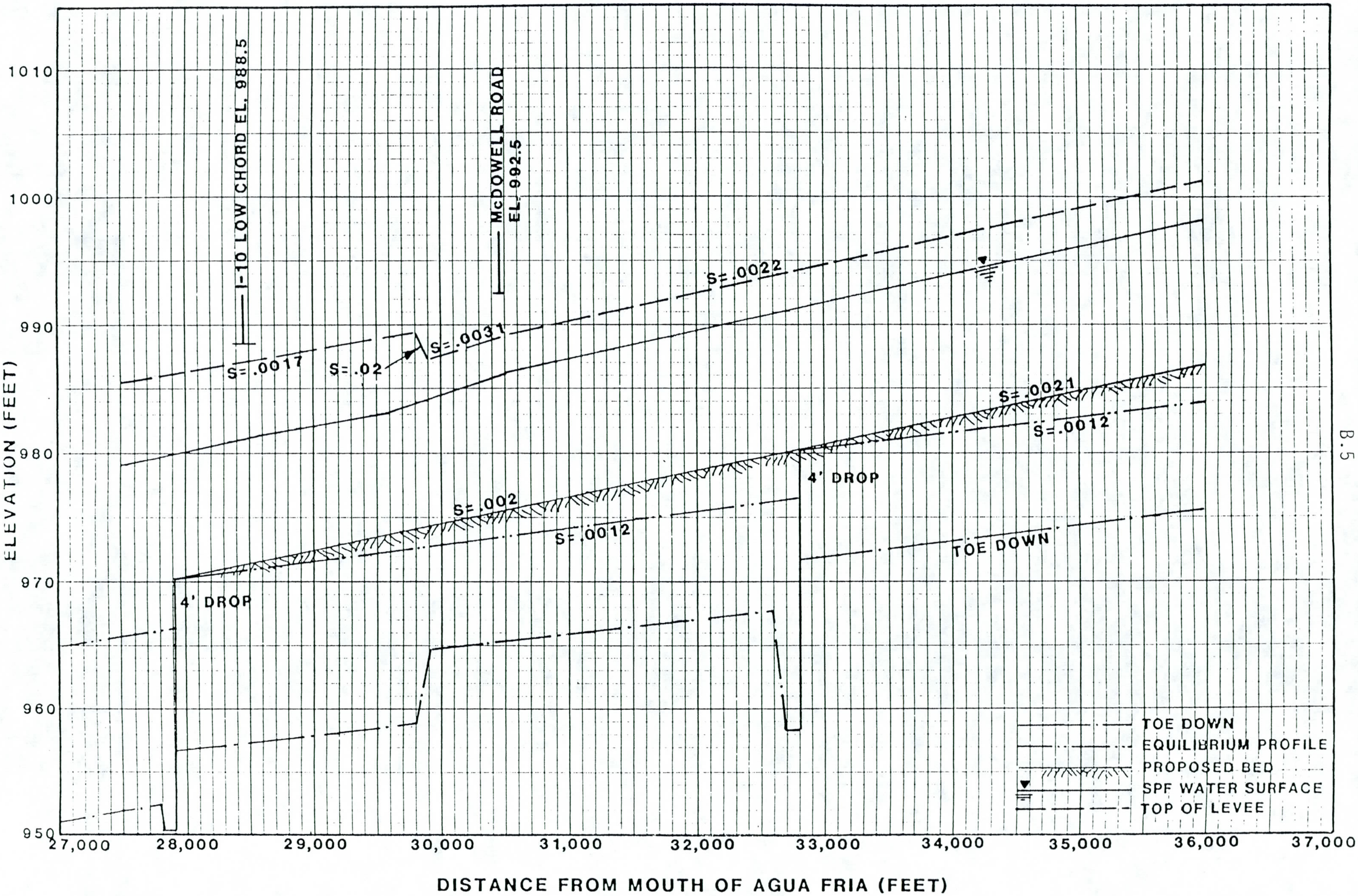

Figure B.4. Proposed bed profile, equilibrium bed profile, SPF water-surface elevation, levee height and toe-down depth between I-10 and Thomas Road for the west bank.

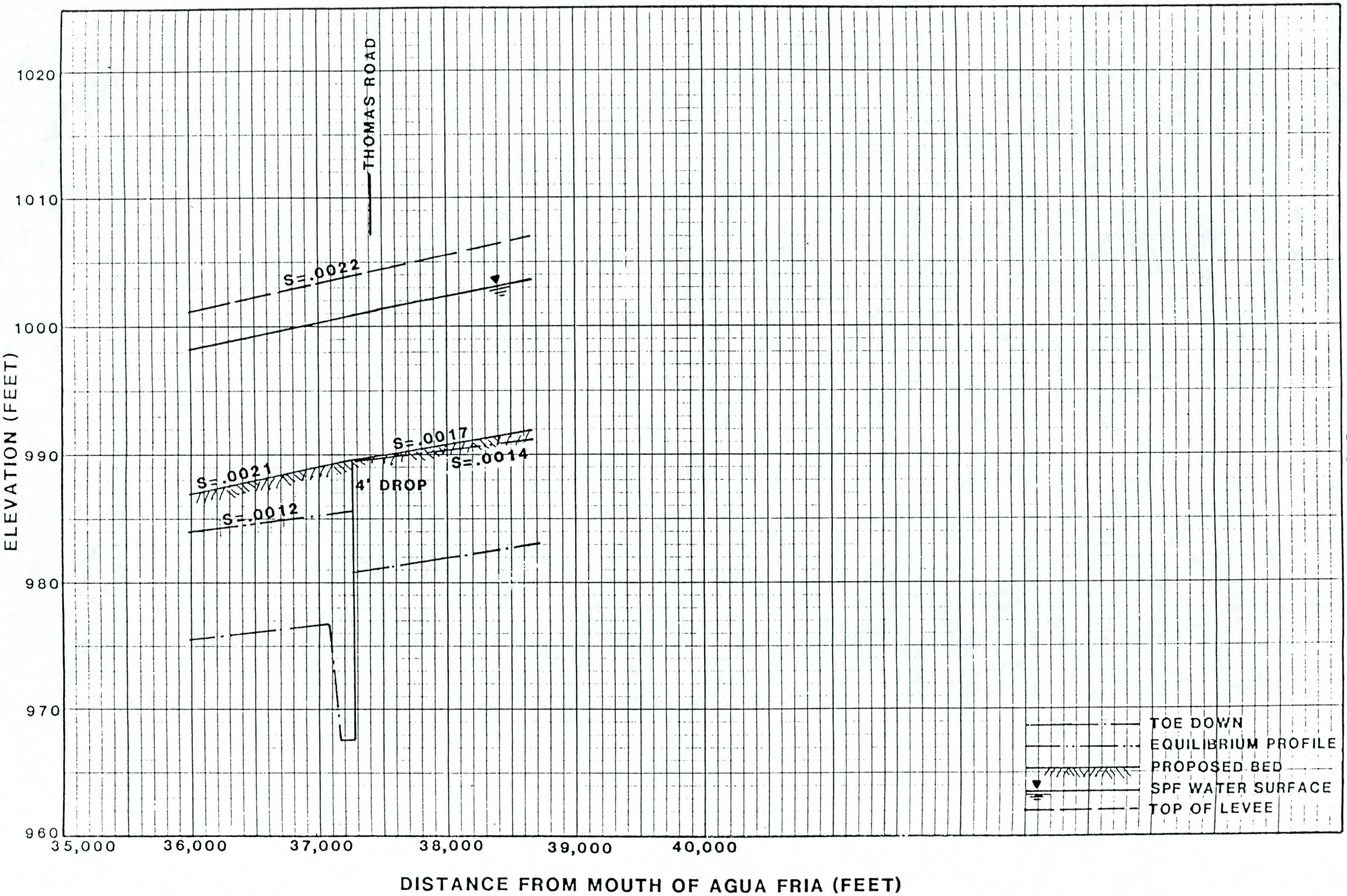

Figure B.4 (continued)

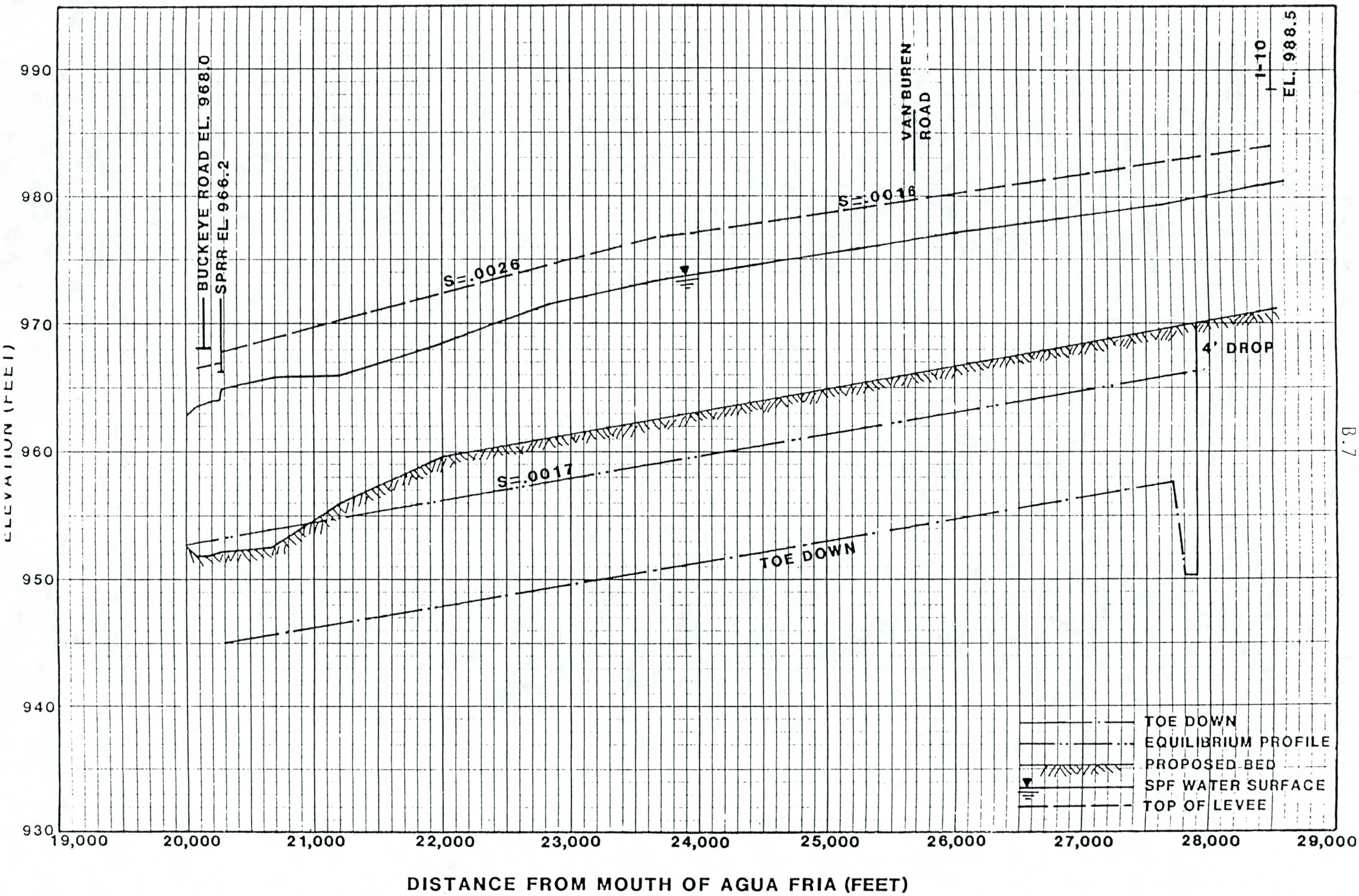

Figure B.5. Proposed bed profile, equilibrium bed profile, SPF water-surface elevation, levee height and toe-down depth between Buckeye Road and I-10 for the east bank.

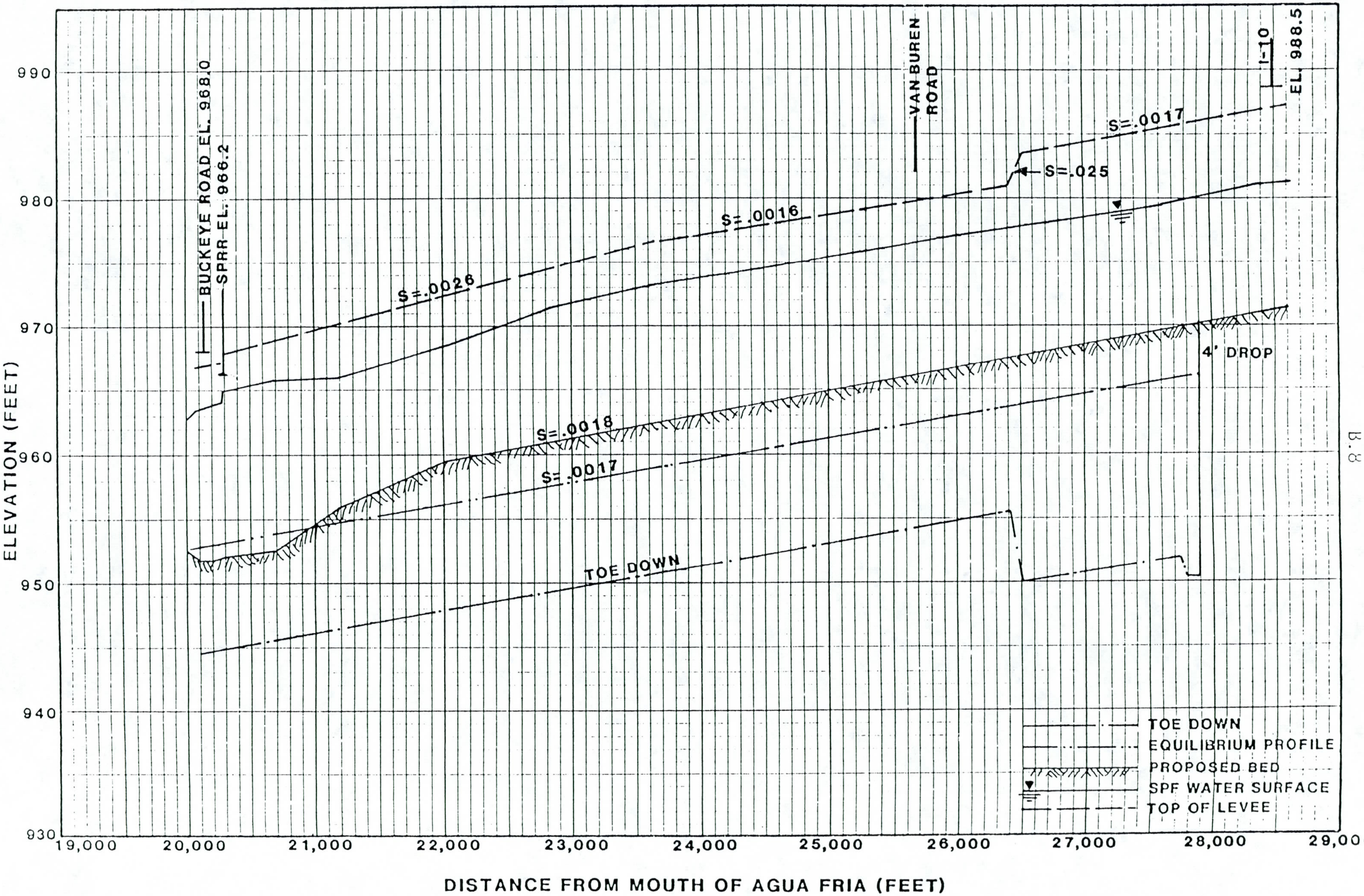

Figure B.6. Proposed bed profile, equilibrium bed profile, SPF water-surface elevation, levee height and toe-down depth between Buckeye Road and I-10 for the west bank.

APPENDIX C
Comments and Responses to Report from
Various Reviewing Agencies

March 27, 1984

FLOOD CONTROL DISTRICT COMMENTS ON DRAFT REPORT,
"SPF ANALYSIS AND CONCEPTUAL DESIGN OF CHANNELIZATION
IN THE AGUA FRIA RIVER"; FEBRUARY 29, 1984

No.	Comment
1.	Title page: delete extra word "in".
2.	Page 1.1: Change Reach 1 levee lengths and limits of channelization to agree with distances shown in paragraph 7.2.1 and 7.2.1.1.
3.	Page 1.2: Correct typo on line 14.
4.	Page 1.3: Correct typo on line 7; and last line of paragraph 11.
5.	Table 3.1 on Page 3.4: in table heading change "option" to "alternative" so that the table agrees with the text. Should the existing discharge at Camelback Road = 142,000 instead of 140,000?
6.	Table 3.2 on Page 3.6: Change "option" to "alternate".
7.	Table 4.1 on Page 4.5: See comment #6
8.	Table 4.2 on Page 4.7: See comment #6
9.	Page 5.1, Paragraph 5.1: last sentence of the paragraph conflicts with the previous sentence. Last sentence should be changed.
10.	Figure 5.2, Page 5.14: Legend is reversed for existing and proposed thalweg.
11.	Page 5.15: Reference is made to Figure 5.3 which was not included in the report. Include Figure 5.3 in the Final Report.
12.	Page 7.6, paragraph 7.2.1.4: Note that MCHD has a road project #12800 to widen ISR between 115th Avenue and 91st Avenue. The 3400' flood wall should be coordinated with this project.
13.	Page 7.12, paragraph 7.2.2.4: Bridge piers 13 through 17 have been replaced.

14. Figure 7.5, page 7.15: on the profile, the top of the depressed section is missing. If possible, put all of Figures 7.5 and 7.6 on one page.

15. Page 7.22, Paragraph 7.2.4.5: Since the SPRR piers are buried 30 feet below invert and scour is 21 feet, is it necessary to protect them? We need to clarify our recommendations for the final report.

16. Page 8.2: Correct typo on the 13th line.

17. Tables A.9 and A.10: Delete the word "cost" from the total quantity heading.

18. Table A.11: Correct heading in the total quantity column. The common fill quantity (870,000 cy) for the gravel pit restoration is on the wrong line.

19. Table A.18: Does the column for the ISRB Pier Protection include quantities to protect the RID Flume? The RID Flume should have a separate column.

20. Plate #1: Add north arrow and scale; add the Camelback Bridge; identify the 100 year pre- and post- project floodway limits.

21. Plate #2: Identify the 100 year pre- and post-development project floodway limits. Show the 3400' floodwall, if possible.

22. Plate #3: Show depressed 500' section upstream of I-10; show existing jetties along I-10 embankment west of diversion channel outlet.

23. General Comment: If known, locations for drainage, culverts, etc. should be shown on all plates.

24. Figure B.2: Elevation scale on left of figure is incorrect. Elevation 980 was deleted.

SLA RESPONSE TO FLOOD CONTROL DISTRICT COMMENTS

Comments 1 through 14, 16 through 22, and 24 are all addressed in the report.

Comment 15: Page 7.22, Paragraph 7.2.4.5: Since the SPRR piers are buried 30 feet below invert, and scour is 21 feet; is it necessary to protect them? We need to clarify our recommendations in the report.

Response: The Southern Pacific Transportation Company responded in a letter dated April 17, 1984 (see enclosed letter), that a minimum soil cover of 12.6 feet is adequate for the Standard Project Flood at the Southern Pacific Railroad Crossing only if a riprap blanket is provided for the full width of the channel. Due to the width of piers being rather large (6'8"), and the uncertainty of the amount of debris accumulating on either side of the piers, which increases local scour, it is SLA's recommendation that a riprap blanket be provided. However, for existing conditions, the local scour potential may be greater than for channelized conditions. This is due to the fact that flow may attack bridge piers at a severe angle for existing conditions, thereby increasing the unit discharge near the piers and causing severe local scour. For channelized conditions, the flow will be guided through the bridge at a proper angle and the flow will be distributed more uniformly. Thus, channelization will improve the hydraulic efficiency of water through the bridge. However, it is still recommended that a riprap blanket be provided for channelized conditions. The financial obligation for providing riprap protection of the railroad piers is uncertain due to the fact that channelization is reducing the local scour potential at the crossing.

Comment 23: If known, locations for drainage, culverts, etc. should be shown on all plates.

Response: For the conceptual design phase of work side drainage was not addressed except for the I-10 drainage collector. The final design will account for all existing side drainages that enter the Agua Fria. Between Indian School Road and Thomas Road this will include the waste canal of the RID flume on the east overbank and local drainage paths that will be intercepted north of Thomas Road on the east and west overbanks. Between McDowell Road and I-10 this will include the I-10 collector channel and local drainages that enter from the west.

C.5

C.K.M.L

Southern Pacific
Transportation Company
400 EAST TOOLE AVENUE, TUCSON, ARIZONA 85701

IN REPLY PLEASE REFER TO

. L. WAMMEL, P.E.
REGIONAL ENGINEER

W. C. DUNN, P.E.
ASST. REGIONAL ENGINEER

T-19349

APR 23 1984

April 17, 1984

Mr. Michael J. Ballantine
Civil Engineer
Simons, Li & Associates, Inc.
P.O. Box 1816
Fort Collins, CO 80522

Dear Mr. Ballantine:

Refer to your letter of February 14, 1984 transmitting plans and calculations regarding proposed channelization measures of the Agua Fria River near our Str. 890.95 at Avondale. Your project No. AZ-MC-05.

We are agreeable to the channelization design with the 12.6 and 14.2 ft. minimum soil covers indicated for the SPF and 100-yr. floods respectively, provided that rip rap blanket protection is installed for the full width of the channel at the bridge. Details including thickness, rock size and width of blanket shall be included in the final design and shall be submitted to the SPTCo. for approval.

Please keep us advised of the progress of this project.

Sincerely,

W C Dunn

W. C. Dunn

APR 25 1984

MARICOPA COUNTY HIGHWAY DEPARTMENT

3325 West Durango Street
Phoenix, Arizona 85009

MARICOPA COUNTY ARIZONA

(602) 262-3611

DATE March 21, 1984

MEMO TO Mr. Dan Sagramoso, Chief Engineer and General Manager, Flood Control District

SUBJECT DRAFT REPORT-STANDARD PROJECT FLOOD ANALYSIS AND CONCEPTUAL DESIGN OF CHANNELIZATION IN THE AGUA FRIA RIVER

We have reviewed the subject report and offer the following comments:

1. The report appears to be complete and thorough except for details of the drop structures.

2. Representatives of the Highway Department attended the presentation of the report on March 15, 1984, by Simons, Li and Associates, Inc., and noted that some persons in attendance had various other comments on the report.

3. We recommend acceptance of the report upon resolution of all comments received.

R C Esterbrook

R. C. Esterbrooks
Director of Public Works
and County Engineer

HRK:cf

FLOOD CONTROL DISTRICT
RECEIVED

MAR 21 '84

4	CH ENG	HYDRO
3	ASST	
	ADMIN	
	C & O	5
	FINANCE	2
	REMARKS	

LH1.4

SLA RESPONSE TO MARICOPA COUNTY HIGHWAY DEPARTMENT COMMENTS

Comment 1: The report appears to be complete and thorough except for details of the drop structures.

Response: More extensive detailing of drop structures will be included in the final design. Figure 7.5 was added to the report and it shows elevation and plan views of the proposed soil cement drop structures. A discussion of factors considered for soil cement drop structures was added in Section 7.2.2.2.

Comment 2 and 3: Representatives of the Highway Department attended the presentation of the report on March 15, 1984, by Simons, Li & Associates, Inc, and noted that some persons in attendance had various other comments on the report. We recommend acceptance of the report upon resolution of all comments received.

Response: SLA has addressed all comments from the various agencies that were in attendance in the March 15, 1984, meeting either in this Appendix or in the main body of the text.

U.S. DEPARTMENT OF TRANSPORTATION
FEDERAL HIGHWAY ADMINISTRATION
REGION NINE

ARIZONA
CALIFORNIA
NEVADA
HAWAII
GUAM
AMERICAN SAMOA

ARIZONA DIVISION
3500 N. Central Ave., Suite 201
Phoenix, Arizona 85012

March 27, 1984

FLOOD CONTROL DISTRICT
RECEIVED
APR 02 '84

CH ENG	HYDRO
ASST	Mgt
ADMIN	SUSP
C & O	FILE
ENGR	DESTROY
FINANCE	2 RGP

IN REPLY REFER TO
HBR-AZ
(409.3)
Flood Control
Maricopa County

Mr. D. E. Sagramoso
Chief Engineer and General Manager
Flood Control District of Maricopa Co.
3335 West Durango Street
Phoenix, Arizona 85009

Dear Mr. Sagramoso:

In all but one respect we find the February 29, 1984 SLA draft report on the Agua Fria River to be acceptable. Further cost beyond our agreement should be borne by your project.

Having the level of the Agua Fria River left bank sill for the I-10 channel the same as the Agua Fria, Q_{50} flow will increase the tributary siltation. In turn, such siltation will greatly increase ADOT's I-10 channel maintenance. If your design can tolerate a lower elevation for the sill, we recommend the outlet still be only two feet above the Agua Fria bed.

Sincerely yours,

Dale E Bender

for Thomas O. Willett
Division Administrator

SLA RESPONSE TO FEDERAL HIGHWAY ADMINISTRATION COMMENTS

Comment: Having the level of the Agua Fria River left bank sill for the I-10 channel the same as the Agua Fria, Q_{50} flow will increase ADOT's I-10 siltation. In turn, such siltation will increase ADOT's I-10 channel maintenance. If your design can tolerate a lower elevation for the sill, we recommend the outlet still be only two feet above the Agua Fria River.

Response: The higher elevation sill was recommended for the following reasons:

1. The higher sill elevation will prevent sediment deposition from the I-10 collection channel into the Agua Fria upstream of the I-10 bridge. An alluvial-fan deposition in the Agua Fria from the collector channel would reduce the effective flow capacity through the I-10 bridge. The deposition would more than likely occur near the east bank forcing Agua Fria flow through the west side of the channel, resulting in higher unit discharges, larger water velocities and increased scour potential on the west side of the bridge.

2. The lower sill elevation will cause more spillover from the Agua Fria into the I-10 basin. The spillover of water will result in loss of sediment-transporting capacity near the east bank, causing deposition of sediment. Possible bars or islands could form near the east bank reducing the effective discharge capacity through the I-10 bridge.

3. Nuissance flows from the I-10 collector channel cannot be stored with the lower sill. Therefore, the dip crossings below I-10; Van Buren, and Lower Buckeye Road, will have to be closed during runoff from the drainage channel.

4. Pre-evacuation of the siltation basin water cannot effectively be accomplished before a large runoff event with the low sill.

Thus, the sill was placed at the Q_{50} level in the Agua Fria to: (1) maximize available storage in the siltation basin; (2) prevent sediment-deposition problems in the Agua Fria that may affect the effective discharge capacity of the channel and the I-10 bridge; (3) prevent nuissance flows from closing dip crossings below I-10; (4) allow for pre-evacuation of the siltation basin; (5) not cause backwater problems in collector channel upstream of the siltation basin; and (6) create an esthetically-pleasing basin for a possible future park area. Therefore, SLA still recommends using the higher sill elevation.

ARIZONA DEPARTMENT OF TRANSPORTATION

HIGHWAYS DIVISION

206 South Seventeenth Avenue Phoenix, Arizona 85007

BRUCE BABBITT
Governor

WILLIAM A. ORDWAY
Director

THOMAS R. LAMMERS
Assistant Director
and State Engineer

FLOOD CONTROL DISTRICT
RECEIVED
APR 04 '84

4	CH ENG		HYDRO
3	ASST		LMgt
	ADMIN		SUSP
	C & O	5	FILE
	ENGR		DESTROY
	FINANCE	2	

LH1.4

April 2, 1984

Mr. Richard Perreault
Flood Control District of Maricopa County
3335 West Durango Street
Phoenix, Arizona 85009

Re: Project I-10-2(112)
Agua Fria River Channelization Study,
Standard Project Flood

Dear Mr. Perreault:

Attached is a copy of an Office Memo from Mr. R. C. Brechler which contains the Arizona Department of Transportation's comments to your draft report on the above referenced study.

If you have any questions regarding the comments, please contact either me or Mr. Marvin Sheldon.

Very truly yours,

Dean Lindsey

DEAN LINDSEY
Principal Engineer I-10
Highway Development Group

DL:ej

Attachment
cc: R. C. Brechler

C.11

ARIZONA DEPARTMENT OF TRANSPORTATION

OFFICE MEMO

March 30, 1984

TO: DEAN LINDSEY, Principal Engineer I-10
Highway Development Group

FROM: R. C. BRECHLER, Assistant State Engineer
Structures Section

SUBJECT: PROJECT I-10-2(112) FUTURE
EHRENBERG - PHOENIX HIGHWAY
AGUA FRIA RIVER CHANNELIZATION BY MCFCD

We have reviewed the draft report "Standard Project Flood Analysis and Conceptual Design of Channelization in the Agua Fria River", February 29, 1984, prepared by Simons, Li and Associates for the Maricopa County Flood Control District.

The report proposes a siltation basin between the end of the I-10 collector channel and the Agua Fria River channel. Drainage from the basin would enter the river through a pipe culvert under the east river levee and over a controlled spill section in the levee at an elevation equal to a 50 year return flow in the river. Before this concept can be endorsed by ADOT, the following items need to be resolved:

1. The report states that approximately 22,000 c.y. of sediment per year will be generated from the I-10 collector channel. In order for the siltation basin to function effectively, this sediment will have to be removed periodically. The division of maintenance responsibilities between ADOT and the MCFCD needs to be clearly defined.

2. The stability of the I-10 highway embankment and the protection of fill slopes from erosion in the siltation basin must be thoroughly analyzed. Any protective work required for the highway embankment must be incorporated with the flood control project.

3. The size of pipe culvert under the levee between the siltation basin and the river channel is not shown in the report. This information should be furnished for our review when it is available.

Please let me know if you have any questions concerning this project.

R C Brechler

R. C. BRECHLER

8-9590 2/75
(FORMERLY 8-954)

RCB:lmh
cc: M. C. Sheldon

SLA RESPONSE TO ARIZONA DEPARTMENT OF TRANSPORTATION'S COMMENTS

Comment 1: The report states that approximately 22,000 cubic yards of sediment per year will be generated from the I-10 collector channel. In order for the siltation basin to function effectively, this sediment will have to be removed periodically. The division of maintenance responsibilities between ADOT and the MCFCD needs to be clearly defined.

Response: The sediment yield of 22,000 cubic yards is an upper range estimate based on yields from nearby watersheds. With a yield of 22,000 cubic yards or 13.6 acre-feet per year deposited material from the basin will have to be removed, on the average, from every three to five years. Periodic inspection after large storms will be required to monitor the sediment deposition volume. The siltation-basin alternative is not maintenance-free and arrangements do need to be agreed upon, as the design will not function as intended without removal of sediment.

Comment 2: The stability of the I-10 highway embankment and the protection of fill slopes from erosion in the siltation basin must be thoroughly analyzed. Any protective work required for the highway embankment must be incorporated with the flood control project.

Response: Analyzing the stability of the I-10 embankment was not included as part of the scope of work for the conceptual design phase. However, this will be addressed as part of the work for the final design. Recommendations and design plans for embankment protection of I-10 (if necessary) will be included in the final design plans.

Comment 3: The size of the pipe under the levee between the siltation basin and river channel is not shown in the report. This information should be furnished for our review when it is available.

Response: The culvert pipe size, location, alignment, etc. will be computed in the final design phase and should be completed at the end of November 1984.

DEPARTMENT OF THE ARMY
LOS ANGELES DISTRICT, CORPS OF ENGINEERS
P. O. BOX 2711
LOS ANGELES, CALIFORNIA 90053

C

REPLY TO
ATTENTION OF:

SPLED-DB

April 17, 1984

FLOOD CONTROL DISTRICT
RECEIVED
APR 2 '84

9	CH ENG		HYDRO
3	ASST		LMGT
	ADMIN		
	C & O	5	
1			
	FIN & C	2	RG

LH 1.4
4/25

Mr. Daniel E. Sagramoso
Chief Engineer and General Manager
Flood Control District of Maricopa County
3335 W. Durango Street
Phoenix, Arizona 85009

Dear Mr. Sagramoso:

The draft report titled: "Standard Project Flood Analysis and Conceptual Design of Channelization in the Agua Fria River," by Simons, Li & Associates, Inc., was reviewed and the comments listed below were made by the various reviewing disciplines. The comments provided address only the design aspects of the report and should not be construed as acceptance by the Corps of the proposed channelization in lieu of the flowage easements required for the Phoenix, Arizona and Vicinity (including New River) project. As you are aware, acceptance of standard project flood channelization is dependent upon the satisfaction of open space and other project environmental requirements as well as upon an adequate technical design for the channelization.

a. Hydraulics Section.

(1) p. 3.1: section 3.1, 1st para., 4th line add "I-10".

(2) p. 5.1: section 5.1, 1st para., add "not" before "considered".

(3) Comment on section 5.3, Aggradation/Degradation Response, p. 5.7: The bankfull discharge of 31,000 cfs, about a 10-year flood, may be applicable upstream of Indian School Road and downstream of Buckeye Road (non-channelized reaches) but table 5.5 has limited significance within the channelized reach. That is, in the channelized reach the dominant bankfull discharge is the SPF flood flow of 142,000 cfs and the 9 to 13 fps velocity would create a very different equilibrium slope analysis. This type of analysis may be inappropriate to river systems with extensive channelization for high flood protection levels.

(4) p. 6.6: Table 6.2, title and heading should reflect the fact that it covers average annual sediment yields.

(5) Chapter VII, Conceptual Design of Agua Fria River Channelization, appears to meet the Corps design standards. The backup data package was not submitted for verification; therefore it has been assumed that all computer modeling, hydraulic coefficients and mathematical calculations are correct.

b. Geotechnical Branch.

(1) The conceptual channel design calls for soil-cement drop structures. Review of the soil logs indicate material in the proposed project area of having 2 to 6 per cent fine (-200 sieve) material. This small amount of fines may be insufficient for the production of soil-cement with suitable quality for use in drop structures even at the cement factors proposed (10 - 12%). As a result, it may be questionable whether soil-cement is a viable option because the contractor will be faced with either (1) setting up an on-site soil-cement batch plant, or (2) transporting materials to an existing plant, batching and then returning the mixture to the job site for placement which may make the soil-cement option economically unfeasible unless soil-cement is also substituted for facing stone on the levees. A source of fine materials suitable for blending should be designated.

(2) Riprap with a minimum D_{50} of 18 inches will probably not be available in sufficient quantity for levee slope protection for this project. It may be available only in sufficient quantities for bridge abutments, piers and smaller spur dikes.

(3) Consideration should be given to the use of gabions or soil-cement for protection of bridge abutments, piers and utility towers. Substantial quantities of suitable cobble size stones are available in the streambed for use in gabions.

(4) Where large differences in gradation occur between riprap and embankment material, multiple layers of graded filter material should ~~may~~ be used to prevent piping.

(5) Filter materials should be well graded. Gravel should not be used as a filter material.

c. Design. Local side drain requirements, such as peak flows, concentration points, methods of collection and inlet structures were not addressed.

Thank you for the opportunity to review the draft report. I hope these comments will be helpful in preparing the final design.

Sincerely,

NORMAN ARNO
Chief, Engineering Division

SLA RESPONSE TO LOS ANGELES CORPS OF ENGINEERS' COMMENTS

GEOTECHNICAL DIVISION

Comment 1: Regards Soil Cement Drop Structures. Sieve analyses show the amount of fine material less than the #200 sieve is two to six percent. For a better mix design, more fines are required.

Response: Sargent, Hauskins, and Beckwith, Inc. (SHB) has tested various mixes of cement, soil and water for the natural material encountered along the Agua Fria channel bottom for Simons, Li & Associates, Inc and has come up with a design that has a compressive strength of over 1,000 psi. The compressive strength is more than adequate for the proposed grade control configuration.

There may be some misunderstanding as to how the contractor will mix the soil cement. An on-site batch plant will be required to separate the coarse material from the mixture. All material coarser than two inches will be removed from the aggregate. The units costs estimated reflect the on-site batch plant, and thus, soil cement is an economically feasible alternative.

Comment 2: Riprap with a minimum D_{50} of 18 inches will probably not be available in sufficient quantity for levee slope protection for this project.

Response: For previous jobs in the area, the source of riprap has been Buckeye Mountain, which is near the confluence with the Gila River. Riprap will more than likely be quarried from the mountain for this project.

Comment 3: Consideration should be given to the use of gabions or soil cement for protection of bridge abutments, piers, and utility towers.

Response: Should larger-sized material transport down the river (greater than two inches in diameter) the wires that tie the gabions together will be subjected to severe abrasion and could possibly fail. This is a distinct possibility if sediment supply continues to be cutoff and armor material develops on the surface.

Soil cement is being considered for utility tower protection. However, for the blanket protection around bridge piers, the uplift forces tend to discourage the use of soil cement.

Comment 4 and 5: Where large differences in gradation occur between riprap and embankment material, multiple layers of graded filter material should be used to prevent piping. Filter materials should be well graded. Gravel should not be used.

Response: The nomenclature for the report may have been a little misleading, as indeed a graded filter material was used satisfying the Arizona Department of Transportation standards requirements for filters (see requirements on page 7.14, Equations 1, 2, and 3). The filter material did not require multiple layers for the riprap-blanket protection near bridge piers and upstream of drop structures as the selected size met the requirements of Equations 1, 2, and 3. Filter fabric was recommended as lining between the riprap and compacted soil on the levees.

ENGINEERING DIVISION

Comment 1: Side drainage needs to be addressed.

Please see response to Comment #23 of the Flood Control District of Maricopa County regarding side-channel drainage.

HYDRAULICS DIVISION

Comments 1, 2, and 4: are addressed in the text of the report.

Comment 3: Page 5.3, Aggradation/Degradation Response. The bankfull discharge of 31,000 cfs, about a 10-year discharge may be applicable upstream of Indian School Road and downstream of Buckeye Road (non-channelized reaches) but Table 5.5 had limited significance within the channelized reach. That is, in the channelized reach, the bankfull discharge is the SPF flood flow of 142,00 cfs and the 9 to 13 fps velocity would create a very different equilibrium slope analysis. This type of analysis may be inappropriate to river systems with extensive channelization for high flood protection levels.

Response: The whole concept of equilibrium slope analysis, whether it be the static or dynamic equilibrium slope, is based on the dominant discharge that most influences the shape of a river. Alluvial geomorphologists commonly define this discharge as: (1) bankfull; or (2) the 2-year or other small frequency event. The bankfull discharge implies the discharge that has occurred as a result of previous flows, not the bankfull channel resulting from man's channelization activities.

Further, the duration of a SPF is limited and it is unreasonable to think that equilibrium conditions can be achieved in an event when the peak lasts only a couple of hours. The SPF on the Agua Fria has a return interval of 250 to 300 years, thus, with such a small probability of occurrence and with a short duration, the equilibrium slope analysis for the SPF would yield unreasonably flat equilibrium slopes.

SLA executed its water- and sediment-routing model for the SPF with the dynamic equilibrium slope throughout the channelized reaches (see Chapter VI) and found a minimum aggradation/degradation response, which indicates the channel has become reasonably stable. Therefore, it is SLA's professional opinion that the values in Table 5.5 are reasonable and approximate future slopes more accurately than an analysis based on the SPF.

Comment 5: Section VII, Conceptual Design of Agua Fria Channelization appears to meet the Corps of Engineers' design standards. The backup data package was not submitted for verification, therefore, it has been assumed that all computer modeling, hydraulic coefficients, and mathematical calculations are accurate.

Response: SLA submitted a package of all computations, model results, etc. to the Flood Control District of Maricopa County on March 15, 1984. A copy of this package can be made available to the Corps of Engineers upon request for their review.

PLATE 2A

PROPOSED CHANNELIZATION
OF THE AGUA FRIA RIVER

sla
SIMONS, LI & ASSOCIATES, INC.

No.	Revision	Date	By

Designed by: JEG, RLM	Scale: 1" = 400'
Designed by: JEG	Date: AUG. 1983
Checked by: MJB	Project No: AZ-MC-05

Ⓢ PROVIDE PROTECTION FOR EXISTING STRUCTURES

NORTH
0' 200' 400

PLATE 4

PROPOSED CHANNELIZATION OF THE AGUA FRIA RIVER

sla
SIMONS, LI & ASSOCIATES, INC.
FORT COLLINS, CHEYENNE, DENVER, TUCSON

No.	Revision	Date	By

Designed by:	YHC	Scale:	1"=400'
Drawn by:	KAS	Date:	JULY 1983
Checked by:	MJB	Project No:	AZ-MC-05