FINAL REPORT

SYSTEM ANALYSIS AND CONCEPTUAL DESIGN OF
CHANNELIZATION IN THE AGUA FRIA RIVER

10/10/83

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SYSTEM ANALYSIS AND CONCEPTUAL DESIGN OF
CHANNELIZATION IN THE AGUA FRIA RIVER

Submitted to:

Flood Control District of Maricopa County
3335 West Durango
Phoenix, Arizona 85009

By:

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P.O. Box 1816
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July 20, 1983

Revised

October 10, 1983

sla
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January 27, 1989

Mr. John Schmeltzer
Flood Control District of
Maricopa County
3335 W. Durango St.
Phoenix, AZ 85009

Re: Riprap Calculation Sheets for Agua Fria River Channelization Design

Dear John:

Enclosed are riprap sizing calculation sheets for the Agua Fria channelization project. There are four sets of calculations. The first two were done in conjunction with the preliminary design for (1) the straight reaches, in general, and (2) the channel bend at Interstate 10. The third set is for the 3:1 bank protection under Buckeye Road and the last applies to the pier protection at Buckeye Road. These were done for the detailed design. I am in the process of determining whether there are additional calculation sheets that apply to specific areas upstream of I-10 that may have been done by our Tucson office.

I trust that these will meet your needs.

Sincerely,

Robert A. Mussetter, Ph.D., P.E.
Associate
Fort Collins Office Manager

RAM/mpp
LDF4/0127RAM2

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EXECUTIVE SUMMARY

The Flood Control District of Maricopa County has contracted Simons, Li & Associates, Inc. (SLA) to conduct a system analysis of the Agua Fria River from its confluence with the New River to its confluence with the Gila River. The system analysis included an assessment of existing conditions, which were documented in the report entitled "Hydraulic and Geomorphic Analysis of the Agua Fria," and an assessment of proposed flood control projects between Camelback and Buckeye Roads, which are documented in this report. Also documented in this report are all conceptual design measures of Agua Fria River channelization between Camelback and Buckeye Roads, along with preliminary cost and quantity estimates. SLA was also contracted to provide design plans and specifications for a flood control project between Camelback Road and Thomas Road.

Included in this report is a system analysis of proposed flood control projects between Camelback and Buckeye Roads, with conceptual design measures. The primary objective of this report is to provide the Flood Control District of Maricopa County with a flood control project that is economically, technically and environmentally acceptable.

Three levels of analysis were conducted to assess channelized conditions in the Agua Fria and included (1) a qualitative geomorphic analysis, (2) an engineering geomorphic analysis, and (3) a mathematical model simulation. The major results for each level of analysis as well as descriptions of recommended conceptual design measures are summarized.

Future channelization measures analyzed for this project were broken down into four channelization reaches. The reaches are described below.

Reach 1. A channel bottom width of 1,630 feet at Camelback Road transitioning to 1,440 feet at Indian School Road Bridge (ISRB). Three options were analyzed in this reach and include (1) one protected dike on the east bank and a partial dike extending 1,650 feet upstream of ISRB terminating in a transverse dike, (2) dikes with protection on both the east and west banks, and (3) a partial dike on the west bank extending 1,650 feet upstream of ISRB terminating in a transverse dike and 2 transverse dikes on the east bank and one spur dike to direct the flow through ISRB.

Reach 2. 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 at Thomas Road. The new alignment in this reach will require significant backfilling of gravel pits on the overbanks.

Reach 3. 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.

Reach 4. 1,410 foot wide channel from I-10 to Van Buren 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. This operation will have to be moved before channelization proceeds downstream of I-10.

Qualitative Geomorphic Analysis

The qualitative geomorphic analysis involves understanding the physical characteristics of the system. The historical changes of the Agua Fria in the study reach were documented in the report "Hydraulic and Geomorphic Analysis of the Agua Fria River." The Agua Fria in the study reach is a braided ephemeral stream and is quite unstable.

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 can be attributed to several factors which include: encroachment of the flood plain by urbanization, gravel mining activities, and the trapping of upstream sediments in Waddell Dam.

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 Road. The reach between Van Buren and Buckeye 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. 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.

Engineering Geomorphic Analysis

The engineering geomorphic analysis quantifies the aggradation/degradation response of the system. Determining the hydraulics is necessary for assessing the sediment transport characteristics of the system.

As explained in the report "Hydraulic and Geomorphic Analysis of the Agua Fria River," the new Waddell Dam will have the largest impact on controlling peak discharges for floods in the channelized reach of the Agua Fria. However, due to the uncertainty of the dam being constructed, the flood peaks for existing conditions at Camelback Road were adopted for conceptual design measures.

Channelization between Camelback and Buckeye Roads will result in flows being carried more efficiently downstream. The peak discharge will not attenuate for channelized conditions as rapidly as in the existing condition. Routing the 100-year flood peak at Camelback Road using HEC-I resulted in a small attenuation of the flood peak at McDowell and Buckeye Roads. Therefore the peak discharge at Camelback Road was adopted as the peak discharge for all downstream reaches.

The hydraulics of the Agua Fria River between Glendale Avenue and the confluence with the Gila River were established using the Army Corps of Engineers HEC-II backwater profile program. Hydraulic characteristics were established for floods with return intervals of 10, 25, 50 and 100 years.

The proposed channelization has levees that contain the 100-year flood plus allow for the freeboard requirements as dictated by the Federal Emergency Management Agency (FEMA). Flow depths for the 100-year peak discharge are generally lower for the channelized condition than for the existing condition because the effective flow width of the main channel has been increased. The channelization will, however, dramatically decrease the 100-year flood plain. The bed response of the Agua Fria, using the dynamic equilibrium slope methodology, indicates degradation potential between Camelback Road and Van Buren, and relative stability between Van Buren and Buckeye Road. The largest degradation potential exists between Indian School and Camelback Roads.

Local scour analysis at existing and proposed bridge crossings in the study reach indicate protection of bridge piers is necessary at I-10, RID flume and Indian School Road crossings. Riprap protection at these crossings is strongly recommended if they are to withstand the 100-year flood.

The low-chord elevations at all crossings are adequate to pass the 100-year discharge. However, the 4-foot freeboard requirement of the Corps of Engineers is not satisfied at Camelback Road and the Southern Pacific Railroad crossings where freeboard heights are 2.8 and 3.7 feet, respectively.

Mathematical Model Analysis

The third level of analysis involves executing the SLA developed sediment routing model to determine the Agua Fria River bed response to the 100-year flood. The SLA model considers sediment routing by size fraction, and therefore can simulate the armoring process of the river bed.

The model was verified by simulating the response of the Agua Fria River to the December 1978, January 1979 and February 1980 floods. Results are documented in the report "Hydraulic and Geomorphic Response of the Agua Fria River." The response of the equilibrium bed profile to the 100-year flood was simulated for channelized conditions. Minimal (less than 1 foot) aggradation and degradation resulted from the 100-year flood occurring in the channelized reach except in the transition from existing to channelized conditions near Camelback Road. Slightly deeper toe down depths will be required near Camelback Road.

Recommendations Regarding Channelization

1. Upstream of Camelback Road a side drainage channel on the west should be provided to direct overbank flow from the west braid into the main channel if continuous levees are constructed between Camelback Road and ISRB. Should a partial levee be constructed between Camelback Road and ISRB on the west bank, the box culvert as designed by PRC Toups will be adequate to eventually drain overbank flow in the west braid. Some rip-rap protection near the box culvert is suggested as well as a small channel on the downstream end of the culvert to carry flow through the gravel pit area.

2. Between Camelback Road and Indian School Road three alternatives are proposed. Alternative 1 has the following components: channelization, levee on east side and 1,650 feet on the west side terminating in an 800 foot transverse dike, one drop structure located 1,000 feet upstream of the Indian School Road Bridge (ISRB), and riprap protection of shallow ISRB piers. Alternative 2 has the following components: channelization, continuous levee on both east and west banks, 3 drop structures located 1,000, 2,000 and 4,000 feet upstream of ISRB, and riprap protection of shallow ISRB piers. Alternative 3 has the following components: channelization, a 1,650 foot levee on the west side terminating in an 800 foot transverse dike, 2 transverse dikes on the east side, a spur dike on

the east bank just north of ISRB, and rip-rap protection of shallow ISRB piers. Alternative 3 is less expensive than the other alternatives; however, the 100-year flood plain is considerably larger for Alternative 3.

3. Between Indian School Road and Thomas Road the following components are being recommended: channelization, levees on east and west banks, back-filling of flood plain gravel pits, 1 drop structure located just downstream of the RID flume, riprap protection of RID flume piers, and protection of 2 transmission towers.

4. Between Thomas Road and I-10 the following components of channelization are recommended: channelization, levee construction on the east and west banks, 2 drop structures located 2,200 feet upstream of the proposed McDowell Road Bridge and 200 feet downstream of Thomas Road, protection of 8 transmission towers, integration of the I-10 collector channel drainage into the Agua Fria, and protection of 2 pipeline crossings near Thomas Road.

5. Between I-10 and Buckeye Road the following components are recommended: channelization, levee construction on the east and west banks, back-filling of instream gravel pits, one drop structure located downstream of the I-10 bridge, riprap protection of I-10 bridge piers and protection of 7 transmission towers.

I. INTRODUCTION

The existing conditions of the Agua Fria between the confluence with the Gila River to the confluence with the New River were assessed in the report "Hydraulic and Geomorphic Analysis of the Agua Fria River." Future channelization measures between the confluence with the New River and Buckeye Road (SR-85) are analyzed in this report. The future channelization analysis includes a three-level approach involving a qualitative geomorphic, quantitative geomorphic, and computer model analysis. After the analysis, preliminary recommendations regarding bed slope profiles, channel shapes, slope protection requirements, drop structure locations and heights, dike heights, hydraulic design of bridges, lateral migration of channelization, local scour protection at bridges and utility crossings are addressed. The project was broken into four principal channelization reaches. These reaches are described below:

Reach 1. A channel bottom width of 1,630 feet at Camelback Road transitioning to 1,440 feet at Indian School Road Bridge (ISRB). Three options were analyzed in this reach and include (1) one protected dike on the east bank and a partial dike extending 1,650 feet upstream of ISRB terminating in a transverse dike, (2) dikes with protection on both the east and west banks, and (3) a partial dike on the west bank extending 1,650 feet upstream of ISRB terminating in a transverse dike and 2 transverse dikes on the east bank as well as a spur dike just north of ISRB.

Reach 2. 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 at Thomas Road. The new alignment in this reach will require significant backfilling of gravel pits on the overbanks.

Reach 3. 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. 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. This operation will have to be moved before channelization proceeds downstream of I-10.

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

1.1 Scope of Work

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

1. Site visit to familiarize ourselves with the area. Simons, Li & Associates, Inc. (SLA) has extensive knowledge of the Agua Fria system based on our previous study of the failure of Indian School Road Bridge. Additional information regarding the characteristics of the Agua Fria system from Waddell Dam to the Gila River was gathered on a site visit in February 1983 by John Lynch and Dick Bumgardener of SLA with the assistance of Richard Perreault of the Flood Control District of Maricopa County. Several sediment samples were obtained to supplement the existing date base. A subsequent site visit was made by John Lynch and Dick Bumgardener in April 1983 to observe backhoe test pits between Buckeye Road and Bethany Home Road, on the Agua Fria and New Rivers, to assess subsurface soil conditions.

2. SLA collected all the available data pertinent to the nine-mile reach of the Agua Fria from its confluence with the New River to its confluence with the Gila River. Information regarding all proposed and existing channelization and hydraulic structures upstream of the New River was gathered. The data base included all hydrologic, hydraulic, channel geometry, sediment, hydraulic structures, gravel mining operations, aerial photographs, topographical and geologic information.

3. SLA commented on the peak discharges expected in channelized reaches after upstream developments such as the New River Dam, Arizona Canal Diversion Channel and Waddell Dam are constructed. SLA performed a hydrologic analysis to determine the effects of channelization on peak discharges in the downstream reaches of the study area.

4. Established the hydraulics of channelization using the Army Corps of Engineers HEC-II backwater profile program. Hydraulics for discharges with magnitudes of 10-, 25-, 50- and 100-year return intervals were assessed.

5. A qualitative assessment of the impacts resulting from proposed channelization from the confluence of the Gila River to the confluence with the New River was performed. Sediment supply and hydrologic impacts on the channelized reach of future developments such as the new Waddell Dam, New River Dam, and the Arizona Canal Diversion Channel are discussed.

6. SLA conducted a quantitative engineering geomorphic analysis on the Agua Fria between its confluence with the Gila River and its confluence with the New River. The tasks performed in the quantitative analysis included:

a. Perform equilibrium slope computations to determine potential bed gradient changes for the dominant discharge. Grade controls, whether man-made or natural, were identified and considered in the analysis.

b. Utilizing the available sediment size distributions for surface and subsurface bed material, and the hydraulics of the Agua Fria, the armoring potential of the bed for the 100-year discharge was determined.

c. The equilibrium slope considering the results of the bed armor method was calculated to determine which one or combination of the two processes controlled the gradient of the Agua Fria. The gradient near the I-10 Bridge, the proposed McDowell Road Bridge, RID flume, Indian School Road Bridge, Camelback Road Bridge and locations of utility crossings are of special concern.

d. Determined general regional scour at the bridge sites or other areas along the Agua Fria that are constricted due to urban encroachments, levees, etc.

e. Estimated expected ranges of lateral migration at bridge sites during the design life of the bridges and recommend measures to control migration tendencies if the banks of the proposed levees are not protected.

f. Determined the potential local scour depth around bridge piers and abutments.

7. Utilizing the results of the first two levels of analysis (qualitative and quantitative geomorphic analysis), the SLA sediment routing model was executed to determine the dynamic response of the bed of the Agua Fria for the 100-year flood for channelized conditions.

8. The hydraulic design constraints of the proposed McDowell Road Bridge and Camelback Road Bridge were determined. The adequacy of existing bridges to pass the 100-year discharge was also addressed. Hydraulic design included establishing the low-chord elevation and bridge opening necessary to pass the design flood, which is the 100-year event. The low-chord elevation was estimated considering aggradation (if any), wave height, depth of flow, superelevation and potential blockage due to debris. The hydraulic analysis for estimation of the low-chord elevation was separate from that for the sedimentation analysis. More conservative estimates regarding the bed roughness were necessary to determine flow depths for determining the low-chord elevation. Channelization and guide structures are considered in this analysis. Conceptual designs of channelization, flow guides, bridge opening spans, and low-chord elevations at McDowell Road and Camelback Road are recommended.

9. Utilizing the results of the qualitative and quantitative geomorphic analysis and the sediment routing model, SLA has recommended the following concerning the proposed McDowell Road and Camelback Road Bridge:

a. total scour depth expected at bridge piers and abutments

b. skew angle of flow expected at bridge

c. range of lateral migration expected during the lifetime of the bridge considering channelization

d. recommendations regarding the degree of encroachment due to urbanization and gravel mining that is acceptable in the near vicinity of the proposed bridges.

10. Utilizing the results of the qualitative and quantitative geomorphic and sediment routing model, SLA has recommended the following concerning the I-10 Bridge crossing:

    a. total scour depth expected at bridge piers and abutments

    b. skew angle of flow expected at bridge

    c. range of lateral migration expected during the lifetime of the bridge

    d. recommendations regarding the degree of encroachment due to urbanization and gravel mining that is acceptable in the near vicinity of the bridge

11. SLA has recommended preliminary design, alignment, grade, etc. for a flood control project between I-10 and Buckeye Road. An analysis for Buckeye Road Bridge and the Southern Pacific Railroad Bridge was conducted similar to that done for the I-10, McDowell Road and Camelback Road Bridges.

12. SLA has documented all findings of the study in this final report.

1.2 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

Geodetic 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.

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 sheets 25, 29, 34-36.

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.

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.

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

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.

II. HYDROLOGY

2.1 Hydrology for Future Channelization

For the proposed channelization in the Agua Fria establishing existing and future hydrologic conditions becomes important for analysis and design work. As discussed in the report "Hydraulic and Geomorphic Analysis of the Agua Fria", the proposed new Waddell Dam will have the most influence, if constructed, in controlling peak discharges on the Agua Fria in the study reach. The recommended new Waddell Dam alternative would release a maximum discharge of 25,000 cfs for the standard project flood, which is presently 158,000 cfs. By increasing the storage capacity of Waddell Dam to 891,400 acre feet (presently the capacity is 157,600 acre feet) significant reductions in the standard project flood, 100-, 50-, 25- and 10-year flood peaks will occur.

With the exception of the proposed new Waddell Dam, other proposed water resource projects will not significantly alter peak discharges in the Agua Fria study reach. Proposed projects or those currently under construction include the Arizona Canal Diversion Channel (ACDC), the New River Dam and the I-10 collector channel. During typical floods, runoff from the New River and I-10 precede peak Agua Fria flows and are, therefore, not additive. As a result, these projects will not change the peak discharges on the Agua Fria.

Due to the uncertainty of new Waddell Dam being constructed as well as the minimal effect of the other proposed projects, analyses in this report are based on the hydrology presented in the Corps of Engineers 1981 report "Hydrology in the Agua Fria River." Table 2.1 presents the flood peak information.

The channelization will improve the conveyance of the channel and as a result increase the downstream discharges. This is a result of a reduction in attenuation of the flood peak. The analysis to determine the downstream peak discharges that result from channelization are explained in the following section.

2.2 Flood Peak Changes as a Result of Channelization

With the Agua Fria channelized between Camelback Road and Buckeye Road, the 100-year peak discharge will be conveyed more efficiently down the channel. The reasons for the increased efficiency include: (1) a more uniform cross section which has a lower flow resistance than natural conditions, (2) a

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 proposed channelized conditions between Camelback Road and Buckeye Road. Existing bridges in this stretch of the Agua Fria include Buckeye Road, Southern Pacific Railroad, Indian School Road, I-10 and the RID flume crossing. Proposed bridges include Camelback Road and McDowell Road.

Water-surface profiles were run for the 10-, 25-, 50- and 100-year discharges, which at Camelback Road, respectively, are 31,000, 50,000, 69,000 and 95,000 cfs. The cross-sectional shape of the proposed channelization is trapezoidal with 3:1 side slopes. Bottom widths vary from 900 feet to 1,630 feet depending on location in the study reach. The heights of the levees were extended to contain the 100-year discharge. Figure 3.1 shows the typical cross sections considered in hydraulic analysis. Thalweg channel elevations were used to determine existing bed slopes in the study reach. Bed slopes varied between 0.0015 and 0.0048, averaging 0.0023 for the entire study reach.

3.2 Flow Resistance

A Manning's roughness coefficient of 0.030 was utilized for the main channel flow resistance to determine the 100-year flood plain, levee height 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 estimates of sediment transport rates.

Overbank roughness coefficients were not of concern in the channelized reach as all of the flows were contained within the levees for the 100-year flood. For the Agua Fria upstream of Camelback 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.

Figures 3.2 through 3.4 show the velocity, top width and depth vs. river distance from the confluence with the Gila for the 10- and 100-year flood peaks. Flow velocities for the 100-year discharge in the channelized reach between Buckeye Road and Camelback Road range from 6.3 fps to 14.0 fps. Top widths range from 975 feet to 1,665 feet, and depths range from 5.2 feet to 10.6 feet.

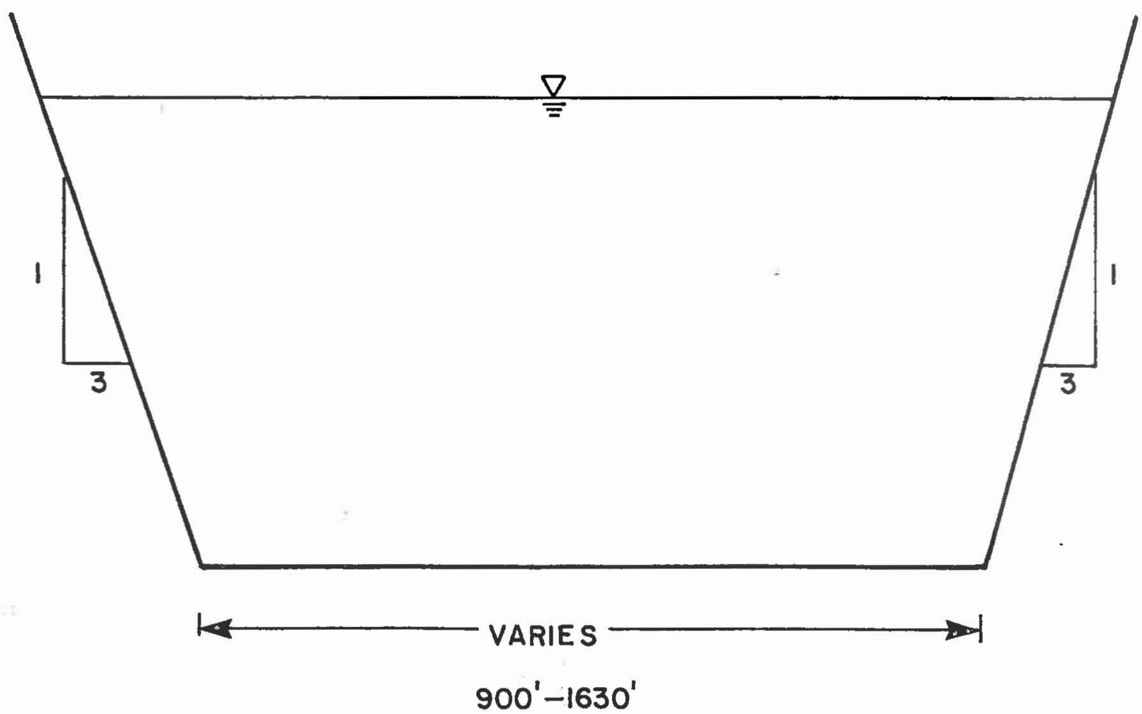


Figure 3.1. Typical cross section of proposed channelization between Camelback and Buckeye Roads.

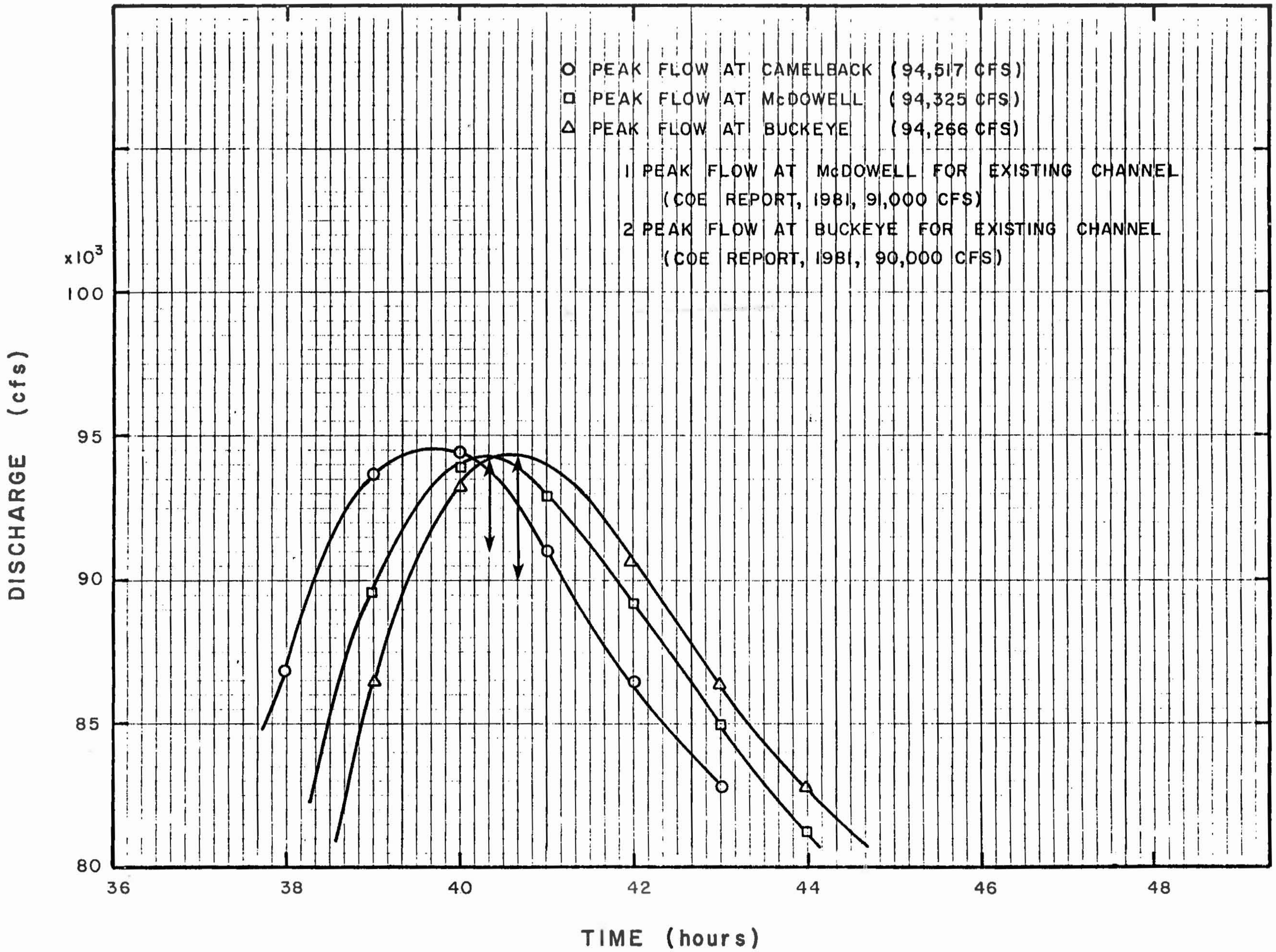


Figure 2.1. Comparison of 100-year hydrographs near peak discharge for the Agua Fria at Camelback, McDowell and Buckeye Roads.

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 suggested grades, channel storage will be reduced. To assess the channelization effects on downstream peak discharges the following analysis was conducted.

A multiple water-surface profile of the Agua Fria was computed utilizing the Army Corps of Engineers HEC-II program. The range of discharges varied between 5,000 cfs and 95,000 cfs. The Agua Fria was delineated into two reaches: the upper reach extending from Camelback Road to McDowell Road, and the bottom reach extending from McDowell Road to Buckeye Road.

Output from HEC-II that was used to evaluate flood peaks in downstream sections included discharge, storage volume and time of travel between reaches. A storage volume versus water discharge relationship was determined at the downstream section of each reach and used in the hydrologic routing procedure of the U.S. Army Corps of Engineers' HEC-I model. HEC-I has the ability to route water in channels using a Modified Puls Method, a channel storage technique. This option was used in the Agua Fria analysis.

The peak discharge presented in the Corps of Engineers 1981 hydrology report attenuated from 94,517 cfs at Camelback Road to 91,000 cfs at McDowell Road to 90,000 cfs at Buckeye Road. With channelization below Camelback Road the peak discharges at McDowell Road and Buckeye as analyzed using HEC-I increased to 94,325 cfs and 94,266 cfs, respectively. These discharges represent increases of 3.7 percent and 4.7 percent at McDowell and Buckeye Roads, respectively.

Figure 2.1 illustrates the attenuation of the 100-year peak at Camelback, McDowell and Buckeye Roads. The travel time between Camelback and Buckeye was 0.57 hours, which represents an average velocity of 14 feet per second. The average velocity of the 100-year flood for the existing channel is 5.8 feet per second, as reported in the 1981 COE report. Therefore, the peak discharge at Camelback Road was adopted as the design discharge for subsequent downstream channelized reaches.

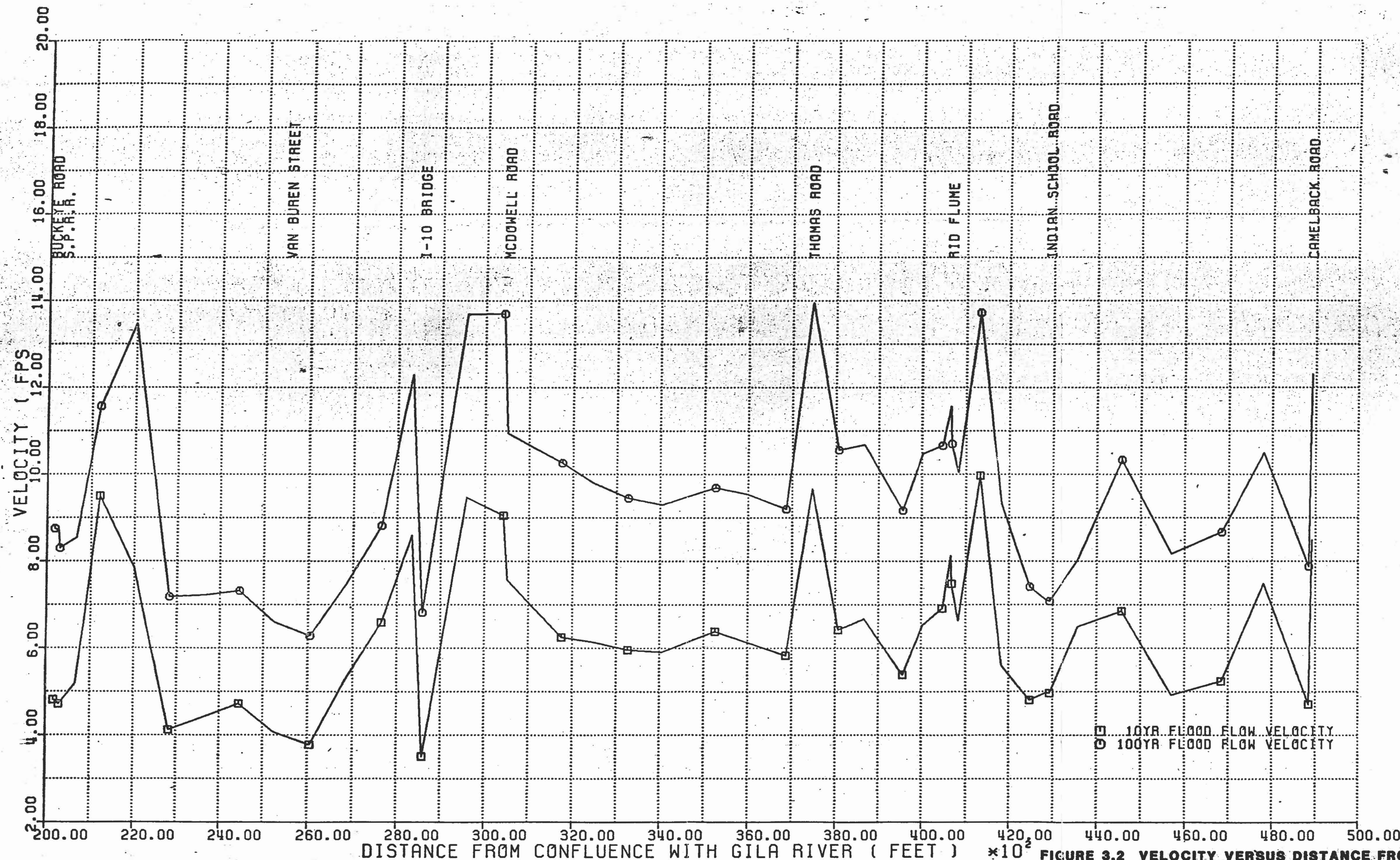


FIGURE 3.2 VELOCITY VERSUS DISTANCE FROM CAMELBACK TO BUCKEYE ROADS FOR THE 10 YEAR AND 100 YEAR PEAK DISCHARGES

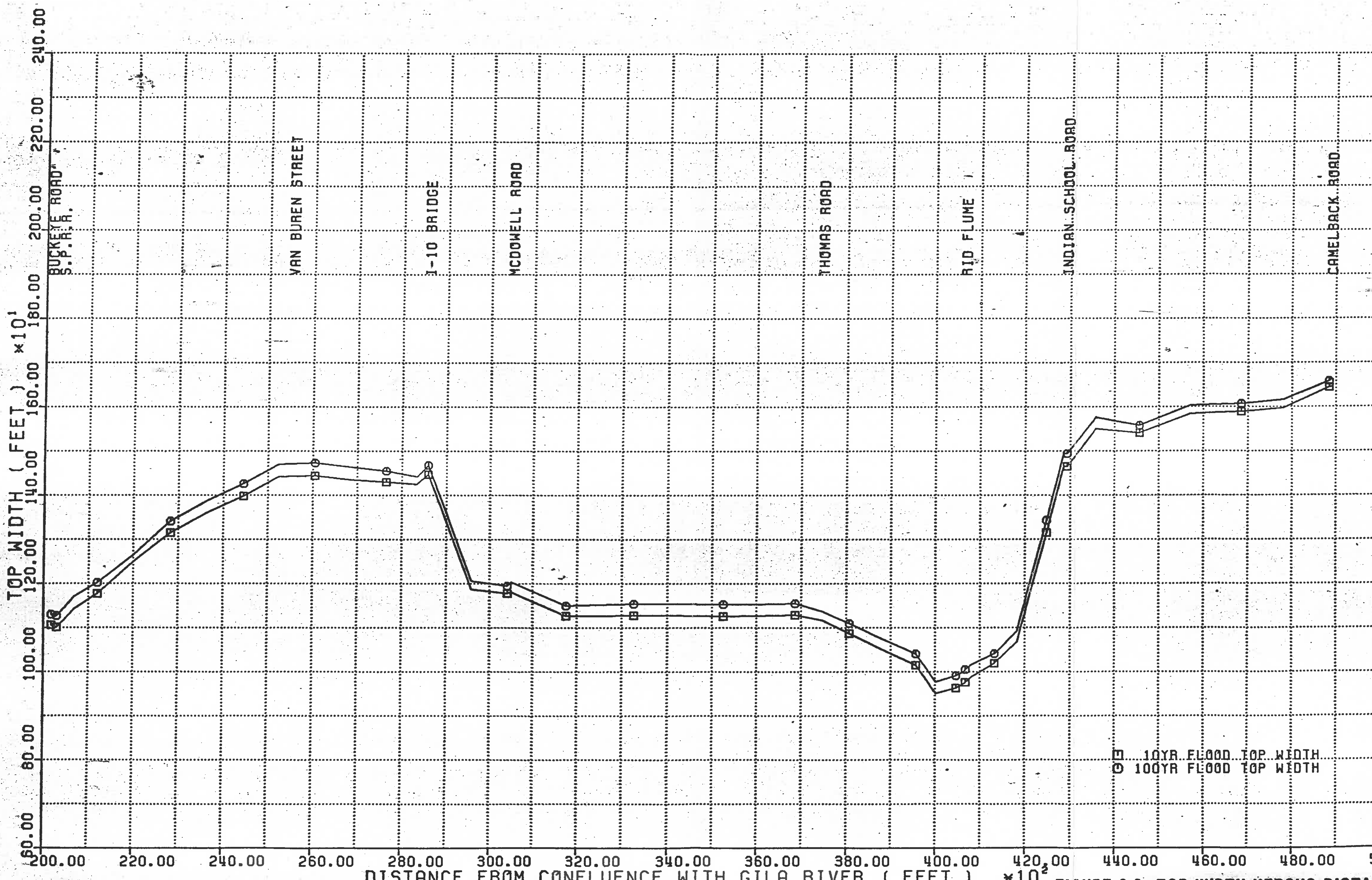


FIGURE 3.3 TOP WIDTH VERSUS DISTANCE FROM CAMELBACK TO BUCKEYE ROADS FOR THE 10 YEAR AND 100 YEAR PEAK

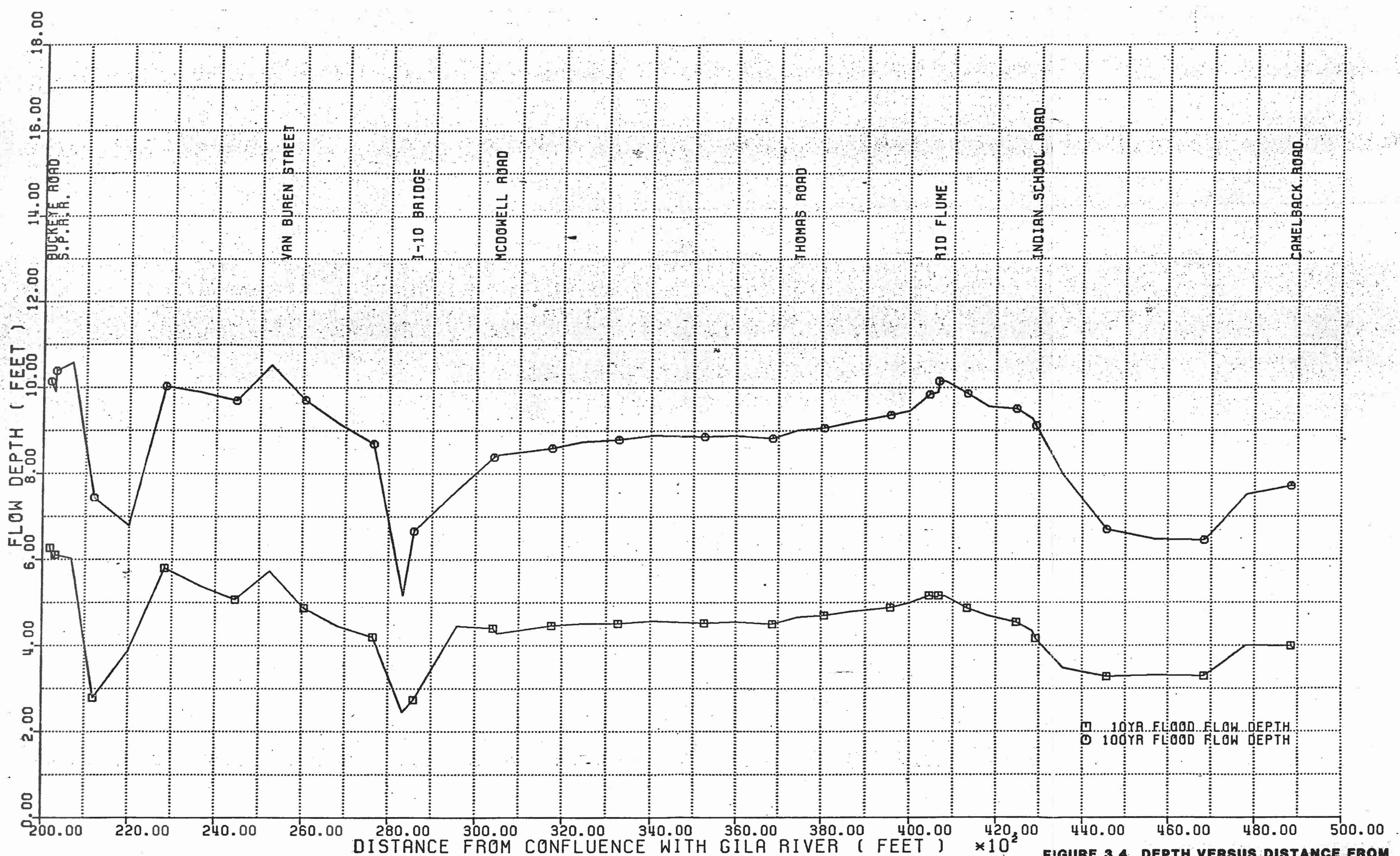


FIGURE 3.4 DEPTH VERSUS DISTANCE FROM CAMELBACK TO BUCKEYE ROADS FOR THE 10 YEAR AND 100 YEAR PEAK DISCHARGES

Tables 3.1 and 3.2 compare the average hydraulic depths, velocities and top widths for existing conditions and proposed channelization conditions for the 10-year and 100-year discharge, respectively. The hydraulic depth is defined as the flow area divided by the channel top width.

Figure 3.5 compares the 100-year backwater elevations for existing and channelized conditions from Buckeye Road to Camelback Road. As Figure 3.5 shows, the channelization will lower the 100-year water-surface elevations in most places because the effective flow width of the main channel has been increased with channelization. The total flood plain widths are drastically reduced for channelized conditions, as shown in Table 3.2.

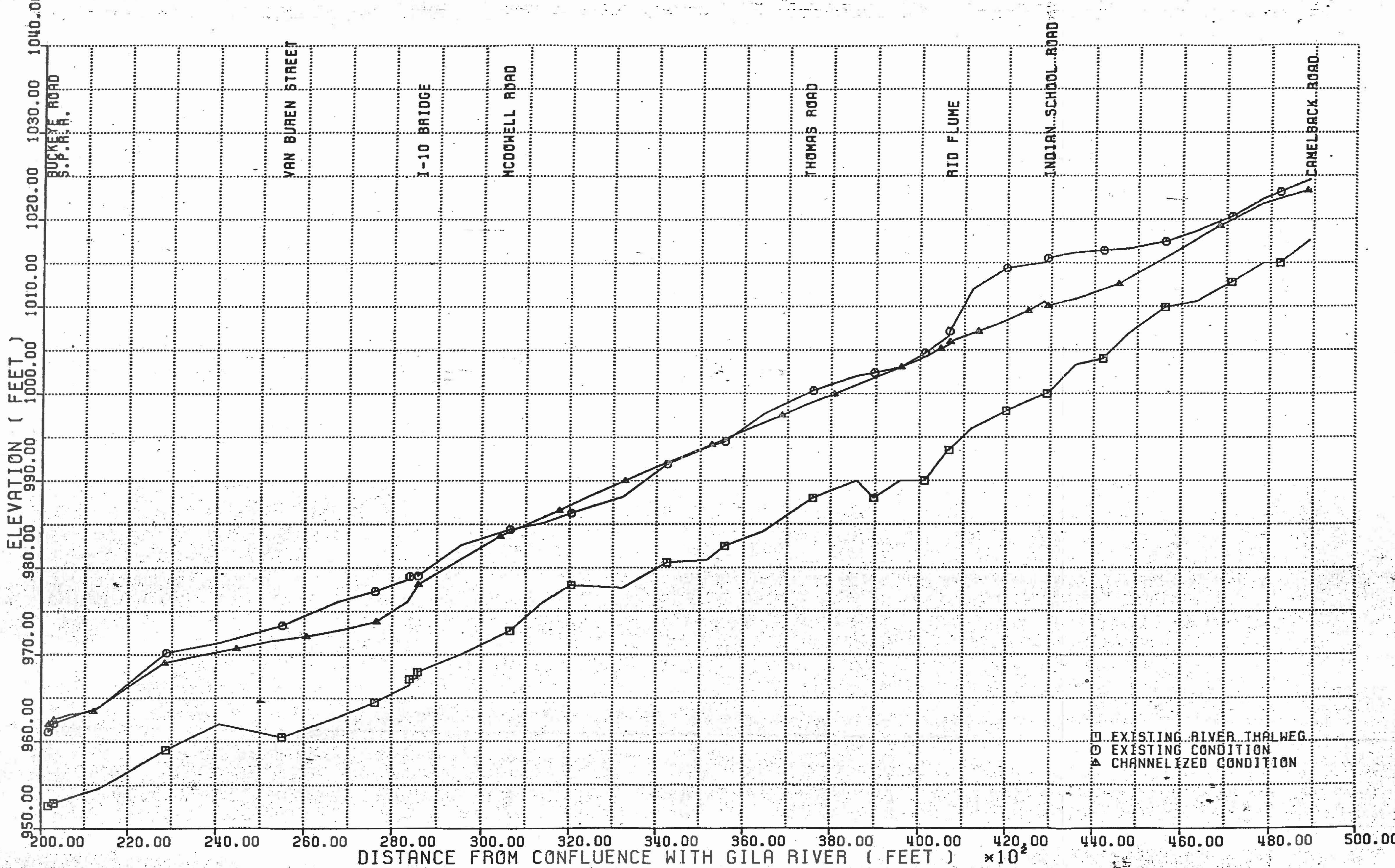
Table 3.1. Comparison of Hydraulics for Existing and Channelized Conditions for Ten-Year Flood.

Reach	Discharge		Average Hydraulic Depth		Average Flow Velocity		Average Top Width	
	Existing (cfs)	Channelized (cfs)	Existing (ft)	Channelized (ft)	Existing (fps)	Channelized (fps)	Existing (ft)	Channelized (ft)
Camelback Rd. to Indian School Rd.	31,000	31,000	2.36	2.97	4.86	6.63	2,703	1,575
Indian School Rd. to RID flume	30,000	31,000	6.0	4.1	7.94	6.8	631	1,115
RID flume to Thomas Rd.	30,000	31,000	3.9	4.36	6.14	6.8	1,252	1,045
Thomas Rd. to I-10	30,000	31,000	2.23	3.9	5.44	6.71	2,474	1,181
I-10 to Van Buren St.	29,000	31,000	3.96	3.56	6.81	6.07	1,075	1,435
Van Buren St. to Buckeye Rd.	28,000	31,000	4.34	4.7	6.0	5.37	1,075	1,227

Table 3.2. Comparison of Hydraulics for Existing and Channelized Conditions for 100-Year Flood.

Reach	Discharge Existing (cfs)	Discharge Channelized (cfs)	Average Hydraulic Depth Existing (ft)	Average Hydraulic Depth Channelized (ft)	Average Flow Velocity Existing (fps)	Average Flow Velocity Channelized (fps)	Average Top Width Existing (ft)	Average Top Width Channelized (ft)
Camelback Rd. to Indian School Rd.	95,000 (77,000)	95,000	3.3 (6.8)	6.1	4.29 (6.25)	9.8	6,715 (1,812)	1,595
Indian School Rd. to RID flume	91,000 (76,200)	95,000	3.4 (11.9)	8.3	5.0 (9.6)	10.0	5,292 (667)	1,142
RID flume to Thomas Rd.	91,000 (66,600)	95,000	3.7 (9.1)	8.3	5.3 (9.45)	10.7	4,665 (775)	1,070
Thomas Rd. to I-10	91,000 (61,900)	95,000	3.2 (7.1)	7.6	5.7 (9.1)	10.4	5,035 (958)	1,204
I-10 to Van Buren St.	91,000 (75,230)	95,000	3.3 (10.3)	7.5	6.4 (8.8)	8.7	4,337 (830)	1,460
Van Buren St. to Buckeye Rd.	90,000 (83,300)	95,000	4.5 (8.0)	8.6	8.5 (9.2)	8.8	2,360 (1,132)	1,252

Values in parentheses are the average hydraulic conditions that occur in the main channel.



COMPARISON OF 100YR FLOOD BACKWATER PROFILE FOR EXISTING AND CHANNELIZED CONDITIONS

FIGURE 3.5

IV. QUALITATIVE GEOMORPHIC ANALYSIS

The historical changes of the Agua Fria in the study reach were documented in the report, "Hydraulic and Geomorphic Analysis of the Agua Fria River." 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.

4.1 Qualitative Evaluation of Proposed Channelization

For future channelization conditions 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 the forth 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 will not experience changes in sediment transport rates in the future, as a supply reach. 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

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			
430.8	43,766	2		
438.8	44,771		45,341	
450.2	45,911			
460.5	47,046			
468.5	48,011	1		
475.5	49,051			Camelback Road
483.3	49,121		49,121	
490.9	49,881			
496.7	50,461			
501.5	50,936			Confluence, New River
510.3	51,821			
520.2	52,811			
531.2	53,911			
544.7	55,261			
558.6	56,651			
568.7	57,661			
580.2	58,806			
589.3	59,716			

Figure 4.1 (continued)

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 for the channelized condition are summarized in Table 4.1. Using the existing cross sections upstream of Camelback Road as the long-term supply reach to compare with the channelized reaches downstream of Camelback, the expected bed responses for the 10-year and 100-year floods are summarized in Table 4.2. All the reaches exhibit a tendency to degrade for the 10-year flood, except Reach 7. For the 100-year flood, all reaches degrade.

With so many reaches in the degrading mode, a general lowering of the channel bed will occur throughout the system unless natural or man-made grade controls exist. After examination of the boring logs at several bridge sites and reviewing the geology reports of the valley, no natural grade controls were detected, therefore man-made grade controls are necessary to control the degradation that will be caused by the channelization.

Historically, low-flow channel meander lengths have ranged from 1,000 to 2,000 feet long, and low-flow braids have moved 700 feet laterally (i.e., the low-flow braid just upstream of ISRB moved 700 feet from 1970 to the present). Combined with the high velocities during flood events, there exists a high potential for damage to the levees if not protected. Therefore, bank protection will be necessary to insure stability.

4.2 Armoring Potential of Bed

As a result of channelization the Agua Fria will exhibit a high potential for degradation, the extent to which this degradation may be limited by channel bed armoring must be investigated. From the analysis of the existing condition presented in the report "Hydraulic and Geomorphic Analysis of the Agua Fria River," it was shown that an armor layer is in existence for a large portion of the channel between Glendale Avenue and Waddell Dam. The extent of this armor layer is summarized in the following paragraphs.

An armor layer is in evidence at Grand Avenue on the Agua Fria, which is located approximately 7 miles upstream of the proposed channelized reach at Camelback Road. Surface material sizes upwards of 3 inches in diameter are present on the bed at Grand Avenue (see Figure 4.2). Because of the armoring,

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

Reach	Change in Top Width		Change in Velocity		Overall Response	
	10-year	100-year	10-year	100-year	10-year	100-year
1	Decrease	Decrease	Increase	Increase	Degrades	Degrades
2	Decrease	Decrease	Increase	Increase	Degrades	Degrades
3	Decrease	Decrease	Decrease	Decrease	Aggrades	Aggrades
4	Decrease	Decrease	Same	Same	Aggrades	Aggrades
5	Increase	Increase	Same	Decrease	Degrades	Aggrades
6	Increase	Increase	Decrease	Decrease	Aggrades	Aggrades
7	Decrease	Decrease	Decrease	Same	Aggrades	Aggrades

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 the 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.

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

Reach	Change in Top Width		Change in Velocity		Overall Response	
	10-year	100-year	10-year	100-year	10-year	100-year
1	Decrease	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	Slight Increase	Increase	Degrades Slightly	Degrades
7	Decrease	Decrease	Approximately Same	Increase	Aggrades	Degrades

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 the 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.

Figure 4.2. Overview of the armored river bed of the Agua Fria River at Grand Avenue, looking downstream.

sediment supply from the bed and banks in this area will be negligible for future flood flows.

Progressing downstream near Glendale Avenue, an armor layer of 1- to 2-inch material is evident near the surface as shown in Figure 4.3. This is a picture of an instream gravel pit located approximately 2,000 feet upstream of the Glendale Avenue bridge. There is more sand available for sediment transport at Glendale Avenue than at Grand Avenue; however, the average surface material is much coarser than the surface material found near Camelback Road.

Near Camelback Road several test pits were dug by Engineering Testing Laboratories in April 1981. The log descriptions indicate a trace of cobbles in the upper 5-foot layer with more occasional occurrences of cobbles, upwards to 8 inches in diameter, at depths greater than 5 feet. Although not visible at the surface, an armor layer is ultimately expected to form in the Camelback Road area if the larger material is not extracted by gravel mining operations.

Due to the armor layer forming in the area above Camelback Road on upstream to Waddell Dam limiting the supply of sediment from the channel, and Waddell Dam cutting off the supply of sediment from further upstream, eventually the amount of sediment being transported into the proposed channelized reaches will be greatly reduced. This will further increase the potential for degradation in the channelized reaches.

Inspection of the logs from borings and test pits in the proposed channelized reaches revealed that the potential for armoring in these areas is either nonexistent or will only become significant after degradation becomes quite deep (probably in excess of 5 to 10 feet). This is based on a lack of larger sediments, such as cobbles, in the layers of sediment near the surface of the channel. Many test pits did not uncover any large material until depths on the order of 10 feet were encountered. Oftentimes, the amount of large material encountered is too small to form an effective armor layer.

Overall, the potential for armoring along the Agua Fria is mainly above the channelization reaches. As a consequence, over time, the supply of sediment to the channelization reaches will lessen. Unfortunately, the potential for armoring in the channelization reaches is small and cannot be counted on to limit degradation. Therefore, the need for man-made grade controls to control degradation of the channelization reaches is not diminished after armoring is considered.

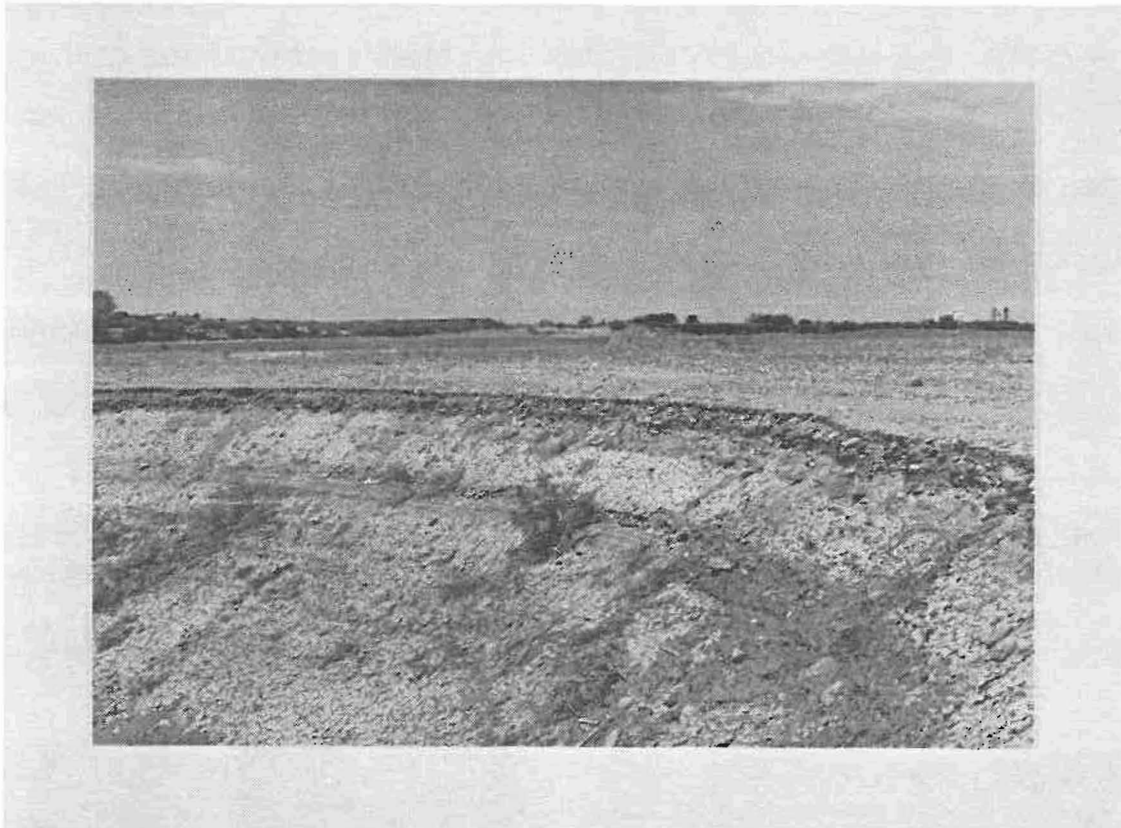


Figure 4.3. Armor layer in evidence at abandoned gravel pit upstream of Glendale Avenue.

V. QUANTITATIVE GEOMORPHIC ANALYSIS

5.1 Quantitative Geomorphic Analysis of Channelization

Total scour is broken down into three components which include (1) local scour due to acceleration of flow around obstructions in the flow, (2) contractual scour or general regional scour due to a constriction in the flow, and (3) the general aggradation/degradation response of the river bed. The total scour analysis was performed at all the existing bridge sites, flume crossings and proposed bridges in the study reach.

5.2 Local Scour Analysis

As explained in the previous report for existing conditions, 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, abutment scour 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.

Table 5.1 summarizes for each of the seven crossings the present bed elevation, the depth bridge piers extend below the present bed elevation, the dimensions of bridge piers, spacing between piers, span length of the bridge, skew angle expected, scour depths for the 100-year discharge of 95,000 cfs computed using Shen and Neil's methodologies, and the adopted local scour expected at the bridge.

Table 5.1. Summary of Local Scour Depths Expected at Bridge Crossings with Proposed Channelization.

Bridge Crossing	Present Bed Elevation	Approximate Depth of Supports Below Present 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]	1,017.1	70	4' diameter	115	1,725	0	10.4	13.3	11.9
Camelback Road[2]	1,017.1	70	4' diameter	115	1,725	0	10.3	13.4	11.9
Indian School Road	1,000.0	Piers 1-12 25' Piers 13-17 70'	1'8" wide Piers 1-12	90	1,620	10	16.9	17.0	17.0
			4' diameter Piers 13-17						
Roosevelt Irrigation District flume	995.0	23-31	4' wide	72	1,008	0	14.1	13.6	13.9
McDowell Road	977.0	70	5' diameter	125	1,250	0	18.0	15.8	16.9
I-10	968.0	23	3.3' diameter	75	1,500	0	15.3	12.9	14.1
Southern Pacific Railroad	951.2	30	6'8" pier deck support section	153	1,200	0	16.6	16.6	16.6
			2' ballast support section	15					
Buckeye Road	951.0	28	3' wide	80	1,200	0	12.1	12.0	12.0

[1] This case considers a levee on the east bank and partial levee construction on the west bank downstream of Camelback Road.

[2] This case considers complete levee construction on the east and west banks downstream of Camelback Road.

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 5.2 summarizes for each tower the obstruction width of each footing, the 100-year flow velocity and depth, the elevation of the bottom of the footing, the 100-year 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 the towers will require some type of protection as the scour depths combined with the proposed channelization will undermine all towers.

Tucson Electric Power Company has 13 towers within the channelized reach. Table 5.3 summarizes local scour depths for the 13 towers for the 100-year flood. All of these towers will require protection as all the local scour depths are quite large for the 100-year discharge.

5.3 General Regional Scour

General regional scour at contractions occurs because the effective flow area is reduced by dikes. These contractions increase the local average 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.

The channelized portion of the Agua Fria having the most severe contractual scour is at Camelback Road. The effective flow width upstream of Camelback Road is approximately 2,425 feet. The flow width necks down to approximately 1,630 feet downstream of the proposed bridge. For the remaining channelization, general regional scour becomes negligible due to the gradual expansions and contractions of the proposed alignment.

Table 5.2. Local Scour Around Towers - Salt River Project for Channelized Conditions

					Local Scour				
Tower Number	Obstruction Width	Flow Velocity (ft/sec)	Flow Depth (ft)	Elevation at Bottom of Footing	Shen (ft)	Nell (ft)	Adopted Scour (ft)	Approximate Channelized Ground Elevation	Elevation After Scour
58	3'	10.62	9.15	987'	8.04	9.44	8.7	990.4	981.7
59	3'	9.44	9.45	987'	7.48	9.02	8.3	985.1	976.8
60	3'	9.13	9.34	979'	7.33	8.87	8.1	983.8	975.7
61	3'	9.32	9.25	981'	7.42	8.94	8.2	982.9	974.7

Table 5.3. Local Scour Around Towers - Tucson Gas & Electric Co. for Channelized Conditions

Tower Number	Obstruction Width	Flow Velocity (ft/sec)	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'	10.77	8.94	11.13	13.20	12.2	989.7	977.8
88 (R)	10'	9.30	9.38	15.61	19.57	13.9	984.3	970.4
89 (R)	10'	9.22	9.30	15.53	19.48	17.5	983.4	965.9
90	5'	9.46	9.08	10.27	12.51	11.4	982.2	970.8
94	5'	9.62	8.65	10.38	12.52	11.5	974.0	962.5
96 (R)	10'	10.42	7.38	16.75	19.90	18.3	972.1	953.8
97	5'	8.87	6.94	9.87	11.73	10.8	966.3	955.5
98	5'	8.65	7.16	9.72	11.66	10.7	963.7	953.0
99 (R)	10'	8.47	7.30	14.73	18.18	16.5	961.7	945.2
100 (R)	10'	8.50	7.59	14.77	18.30	16.5	959.0	942.5
101 (R)	10'	8.46	7.89	14.72	18.36	16.5	957.6	941.1
102 (R)	10'	8.41	8.64	14.67	18.54	16.6	954.8	938.2
103 (R)	10'	8.70	9.14	14.98	18.95	17.0	952.9	935.9

R = Reinforced

To determine the amount of general regional scour at Camelback Road the principles of water and sediment continuity are utilized. The hydraulics for the 100-year peak discharge of 95,000 cfs were utilized from the HEC-II water-surface profile program. The sediment transport rates were theoretically determined using a combination of the Meyer-Peter, Muller bed-load transport equation and Einstein's integration of the suspended bed-material load. The sediment transport relations have been applied successfully to numerous sand and gravel bed channels and are considered applicable in the Agua Fria. The general regional scour computed for the constriction at Camelback was 3.2 feet for the 100-year flood.

5.4 Aggradation/Degradation Analysis

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

5.4.1 Estimate of Time Required for Armor Layer to Progress to Camelback Road

Presently there exists a supply of sand from bed and banks upstream of the proposed channelized reach. However, an armor layer of gravel- and cobble-size material is forming on the channel bed of the Agua Fria from Waddell Dam to approximately Bethany Home Road, which is one mile upstream of Camelback Road. Should this armor layer progress to Camelback Road, in combination with the construction of the New River Dam, the sediment supply into the channelized reach will be significantly reduced, thereby resulting in serious degradation problems.

Assuming the bed of the Agua Fria is armored to Bethany Homes Road, and an armor layer develops approximately 5 feet below the present bed elevation near Camelback Road, a volume of 40×10^6 ft^3 of material is available from the bed and banks before armoring occurs from Bethany Home Road to Camelback Road. Using the Meyer-Peter Muller bed-load transport equation in combination with the Einstein suspended sediment procedure, sediment transport rates were determined for the 10-, 25-, 50- and 100-year floods in the reach between Camelback and Bethany Home Road. The number of floods necessary to remove the volume of material available in the bed and banks for the 10-, 25-, 50- and 100-year floods was 6, 3, 2 and 2, respectively. Thus it will take approximately 25 to 50 years to develop an armor layer depending on the sequence and number of floods that occur. This analysis considers no supply from the upstream reach which is very conservative. In actuality, it will take more floods than these to armor the channel bottom.

Not knowing what the future flows will be on the Agua Fria, the gradient of the channelized reach will be based on present sediment supply rates. The Agua Fria should, however, be monitored upstream of Camelback Road to determine when the armor layer progresses to Camelback Road. Additional grade-control structures may be necessary once this armor layer reaches Camelback Road.

5.4.2 Equilibrium Slope Analysis

The equilibrium channel slope is defined as the slope at which the channel's sediment transport capacity is equal to the incoming sediment supply. Under this condition, the channel neither aggrades nor degrades. The equilibrium slope method is sometimes referred to as the dynamic equilibrium slope because the gradient of the channel continually changes with upstream sediment supply.

The equilibrium slope analysis is usually determined for the dominant discharge in the river, or the discharge that most influences the cross-sectional shape. The dominant discharge is often considered to be the bankfull discharge. However, bankfull discharge is hard to determine and highly variable along the Agua Fria River because of the multiple braids. The 10-year discharge of 31,000 cfs was selected as having the most influence in shaping the channel upstream of Camelback Road, as it has occurred several times in the past, most recently in 1979. The 10-year discharge is within the range of discharges that can be considered bankfull along the Agua Fria.

When establishing the equilibrium slope for downstream channelized reaches, it is important to accurately assess the incoming sediment supply. The reach of the Agua Fria upstream of Camelback Road near the New River was used as the sediment supply reach. This section of the river is very wide and should serve as an excellent control for sediment supply if armoring does not occur.

The existing channel bed above Camelback Road is, however, starting to show evidence of an armor layer developing. Patches of larger sized materials on the bed (1 inch in diameter) are in evidence at the New River confluence. The armor layer gets progressively thicker upstream to Grand Avenue where the river bed is completely filled with gravel and cobbles. The armor layer developed as a result of sediment being trapped by Waddell Dam.

Table 5.4 summarizes for the 10-year discharge of 31,000 cfs the existing thalweg slopes, equilibrium slopes, average hydraulics and sediment transport rates for each reach. The equilibrium slope analysis is based on both levees having adequate bank protection, such that bank erosion will be minimal and any degradation that occurs will originate from the channel bed. Should the banks not be protected and the channel not be maintained in its trapezoidal shape, the channel will not reach the equilibrium slope. Another consideration in determining whether the bed will reach the equilibrium slope is to determine if an armor layer will develop before reaching equilibrium.

5.4.3 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. 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 is whether the degradation that would occur before armoring caused it to cease would be too large to be compatible with the channelization.

Table 5.6. 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	5.2	7.9	11.6
2	7.2	8.5	10.8	11.6
3	2.3	5.3	5.6	11.6
4	3.8	7.7	9.7	11.6
5	7.6	10.4	17.7	11.6
6	1.4	5.1	5.4	11.6
7	0.6*	5.2	9.1	11.6

*Aggradation in this reach.

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

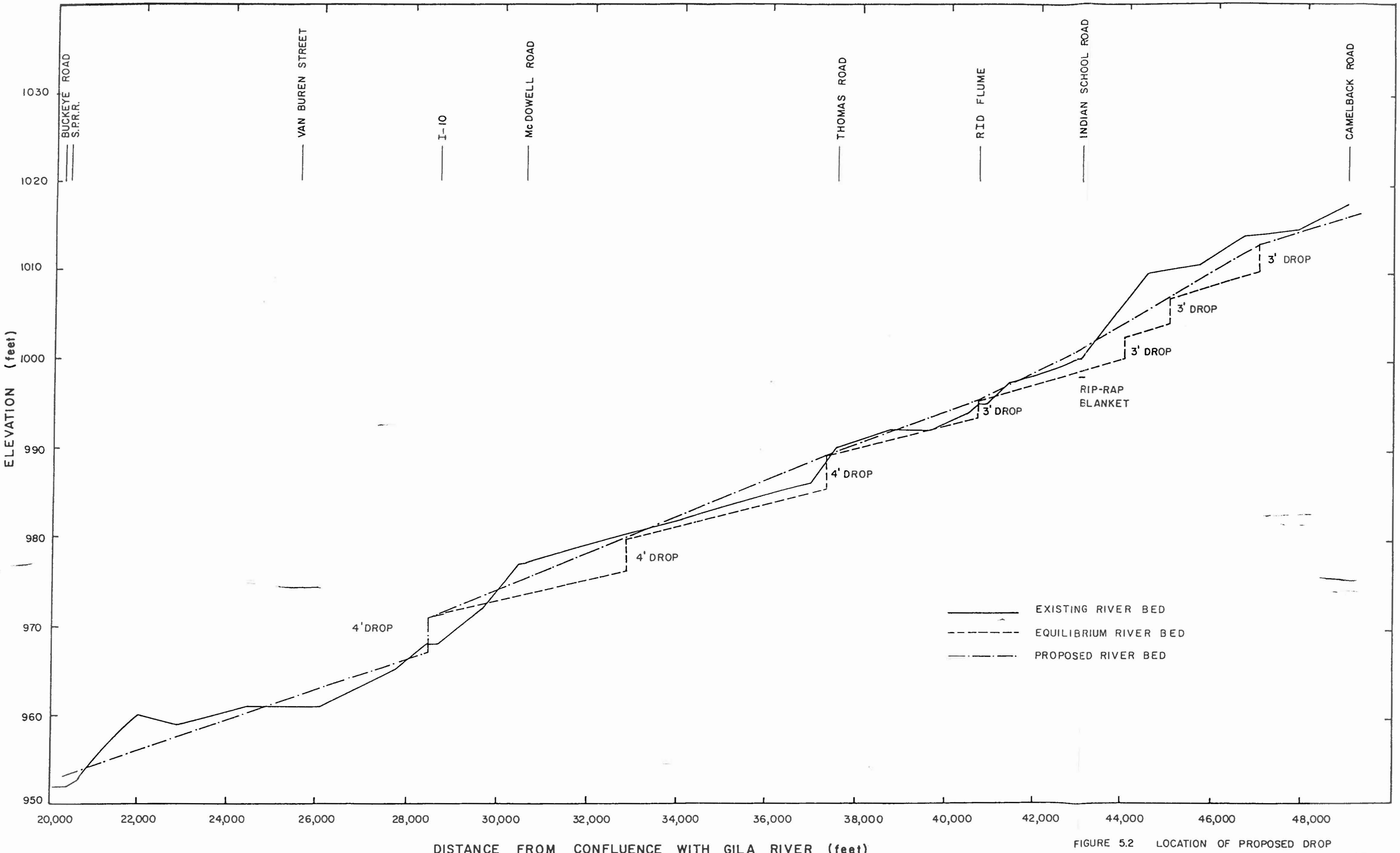


FIGURE 5.2 LOCATION OF PROPOSED DROP STRUCTURES AND EXISTING, PROPOSED AND EQUILIBRIUM BED PROFILES.

First, the existing thalweg profile in reaches 1 and 2 was combined into one reach with an average slope of 0.0030 as shown in Figure 5.2. The grade of 0.0030 between ISRB and Camelback will generate extra material for back-filling of gravel pits downstream of ISRB and for construction of levees. The dynamic equilibrium slope for this reach is 0.0015, which represents approximately 9 feet of degradation potential at Camelback Road should both banks be diked. Three 3-foot drops will be necessary between ISRB and Camelback Road. The approximate locations of drop structures are 1,000 feet, 2,000 feet and 4,000 feet upstream of ISRB.

Should the west bank of the river be partially diked and a dike be constructed on the east bank, the velocities in reaches 1 and 2 will be reduced, sediment will be available for transport from the west bank and the dynamic equilibrium slope for this reach becomes 0.0024. One 3-foot drop structure located 1,000 feet upstream of ISRB will be necessary for this alternative. A larger flowage easement will be required.

For the alternative with a partially diked west bank and 2 transverse dikes on the east bank, no grade control structures are recommended as velocities will reduce considerably for this alternative. The flowage easement required for this alternative is considerably greater than the other alternatives.

The grade control structures are located in areas that maximize protection of existing structures and minimize toe down depths of levees which can be very costly. The locations of grade controls and depth requirements are predicated on the assumption that both levees will have some protection whether it be in the form of riprap or soil cement.

5.5 Total Scour Expected 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. Estimates of antidune heights were made by Kennedy (1963) as follows:

$$h = 0.14 \frac{2 \pi V^2}{g} \tag{5.5}$$

where h is the antidune height

V is the average flow velocity

g is the gravitational constant.

If the calculated antidune height is larger than the depth of flow, the antidune height is set equal to the depth of flow.

Summaries of expected total scour depths at all major bridge and flume crossings are listed in Table 5.7. 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 and I-10 bridge piers be protected with riprap to prevent potential damage during the 100-year flood.

The local scour at the Salt River Project and Tucson Electric Power Company transmission towers is excessive enough to require protection. Tables 5.2 and 5.3 summarize scour depths at all towers within the channelization reach.

Several pipeline crossings exist within the proposed channelization. El Paso Gas Company has a 10-inch line located 150 feet upstream of Buckeye Road. The sedimentation analysis indicates the channelization will not affect this line. 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 6-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.

Table 5.7. Summary of Total Scour Depths Expected at Major Bridge and Flume Crossings in the Agua Fria for the 100-Year Discharge.

Channel Feature	Depth of Burial of Piers	Local Scour (100-yr Discharge) (ft)	General Regional Scour (ft)	General Aggradation/ Degradation/ (Dynamic Equilibrium) (ft)	One-Half Antidune Height (100-yr Discharge) (ft)	Expected Total Scour (ft)
Camelback Rd. Bridge[1]	70	11.9	3.2	3.4	1.2	19.7
Camelback Rd. Bridge[2]	70	11.9	3.2	0	1.2	16.3
Indian School Road Bridge	25	17.0	---	2.6	0.7	20.3
Roosevelt Irrigation Dist. flume	23-31	13.9	---	0	1.4	15.3
McDowell Road Bridge	70	16.9	---	2.9	1.9	21.7
I-10 Bridge	23	14.1	---	0	0.6	14.7
Southern Pacific Railroad Bridge	30	16.6	---	---	0.9	17.5
Buckeye Road Bridge	28	12.0	---	---	1.1	13.1

[1] Partial levee construction on the west bank of the Agua Fria between Camelback and Indian School Roads.

[2] Complete levee construction on the west bank of the Agua Fria between Camelback and Indian School Roads.

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 100-year flood, 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 with the Gila River to Glendale Avenue was simulated for the 100-year flood. The channelized reach extends from Camelback Road to Buckeye Road. The thalweg profile is the dynamic equilibrium slope profile as shown in Figure 6.1. Also shown in Figure 6.1 is the simulated bed response to the 100-year flood.

The bed remains fairly stable throughout most of the river. From approximately 2,000 feet upstream of Camelback Road to Camelback Road the bed lowers an average of 2 feet. In the transition area from the natural to the channelized reach, the top width decreases significantly and the velocities increase substantially. The sediment transport rates increase in the transition reach resulting in the 2 foot lowering of the bed. In the reach between Camelback Road and the first grade control structure downstream of Camelback Road, the bed aggrades approximately 1 foot. This aggradation is the result of increased sediment supply from the reach upstream of Camelback

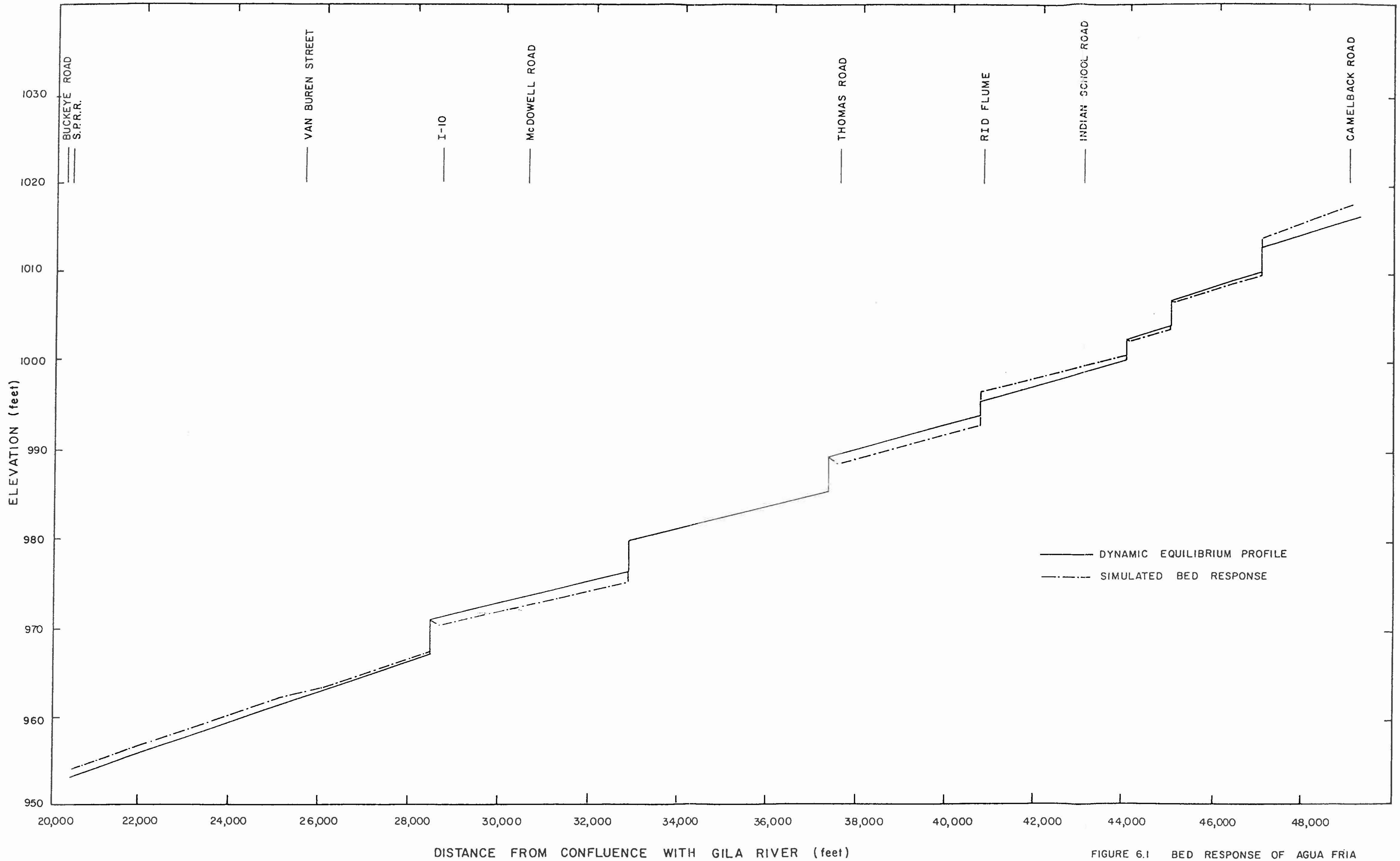


FIGURE 6.1 BED RESPONSE OF AGUA FRIA RIVER TO 100-YEAR FLOOD.

Road. Between the RID Flume and the grade control structure upstream of ISRB, the bed aggrades an average of 1.0 foot. This is due to the excess of sediment supply from the transition reach, and also due to the steeper gradient above the drop structure in the reach upstream of ISRB. All the other reaches in the Agua Fria aggrade or degrade less than 1.0 foot indicating the channel is approaching its equilibrium state.

6.3 Conclusion

QUASED shows that the response of the dynamic equilibrium bed profile to a 100-year flood is minimal, except in the transition reach near Camelback Road. This indicates that slightly deeper toe down depths will be required near Camelback Road. Throughout the channelization reach the bed response neither aggraded nor degraded more than a foot, therefore the equilibrium bed profile is a good estimate of the eventual bed profile. The profile will not be altered significantly by large floods after equilibrium conditions are reached.

VII. HYDRAULIC DESIGN OF BRIDGES

7.1 General

The existing and proposed bridge and flume crossings were analyzed to determine their adequacy to pass the 100-year peak discharge of 95,000 cfs. Considered in the analysis was the water surface elevation computed using HEC-II with no debris blockage at the piers, 10 percent debris blockage and 20 percent debris blockage. Also considered at each of the crossings was any aggradation, computed using the QUASED model, during the peak discharge of the 100-year hydrograph. Finally, a 4-foot freeboard, as required by the Corps of Engineers, or the sum of half the antidune height, superelevation around bends and surface wave height, was added to the water surface elevation, whichever was greater. The proposed bed profile shown in Figure 5.2 was used to determine the water surface profiles. Also shown in Figure 5.2 is the existing thalweg profile and the dynamic equilibrium profile.

7.2 Lateral Migration and Flow Skew at Bridge Crossings

As characteristic of many braided streams, the Agua Fria has a very wide flood plain with multiple low flow channels which tend to migrate significantly within the banks. Examination of the aerial photographs shows the flood plain width has not changed significantly except when altered by man through urbanization, gravel mining or channelization encroachments.

Table 7.1 summarizes the past flood plain widths and approximate skew angle of the low flow channels at the Camelback Road, Indian School Road, RID Flume, McDowell Road, I-10, Southern Pacific Railroad and Buckeye Road crossings. Aerial photographs for the years 1936, 1963, 1974 and 1980 were used to determine flood plain widths.

The flood plain width of 4,300 feet has remained approximately the same at Camelback Road since 1936. The flow angle was 25 degrees from normal at Camelback Road during the February 20, 1980, flood. Spur dikes upstream of Camelback Road in conjunction with channelization should align the flow through the bridge. The lateral migration to the east has historically been negligible and the spur dike, as designed by PRC Toups, will direct any overbank flow on the east bank towards the main channel. The bridge width of 1,725 feet is considerably narrower than the 4,300-foot wide flood plain. A

Table 7.1. Flood Plain Width and Skew Angle Changes at Bridge Site Crossings.

LOCATION	DATE	FLOOD PLAIN WIDTH (ft)	SKEW ANGLE OF LOW FLOW BRAID
Camelback Road	1936	4,300	7
	1963	4,300	0
	1974	4,200	3
	1980	4,300	25
Indian School Road	1936	4,000	25
	1963	3,200	11
	1974	2,700	18
	1980	2,900	45
Roosevelt Irrigation District Flume	1936	3,700	20
	1963	1,700	9
	1974	500	0
	1980	500	0
McDowell Road (upstream of dogleg)	1936	2,500	25
	1963	1,500	40
	1974	1,500	40
	1980	1,600	30
I-10	1936	1,600	10
	1963	1,600	17
	1974	1,600	10
	1980	1,600*	5
Southern Pacific Railroad and Buckeye Road	1963	2,300	9
	1974	2,300	15
	1980	1,500	23

*Flow broke out of dogleg just downstream of McDowell Road and flooded field east of I-10. Water flowed to the east of the partially completed I-10 bridge.

side drainage channel to direct overbank flow from the west braid back into the main channel or a culvert through Camelback Road at the west braid will be necessary to drain the overbank flow. This is discussed further in the next chapter.

For the channelized reaches downstream of Camelback Road, no lateral migration is expected, with the possible exception of the reach between Camelback Road and Indian School Road if levees are not constructed.

At large discharges the skew angle at the bridge crossings should be insignificant due to the channelization aligning the flow. Low flow meanders will still exist and cause some skew of the flow at smaller discharges. This should not cause significant local scour problems at the bridge crossings.

7.3 Low Chord and Flood Passage

Summaries of the water surface elevation at Camelback Road, Indian School Road, Roosevelt Irrigation District Flume, McDowell Road, I-10, Southern Pacific Railroad and Buckeye Road crossings are made in Tables 7.2 through 7.4 for no blockage, 10 percent blockage and 20 percent blockage, respectively. Considering no debris blockage, all seven crossings will pass the 100-year discharge; however, the 4-foot freeboard is not satisfied at Camelback Road, and Southern Pacific Railroad crossings where the freeboard elevations are 2.8 feet and 3.7 feet, respectively.

The freeboard at Camelback Road is slightly misleading as the low chord near the abutments is at elevation 1,026.2 feet; however, at the center of the bridge the low-chord elevation increases to 1,031.2 feet. Thus the average low-chord elevation is 1,028.7 feet, which should provide adequate capacity to pass the 100-year flood and 4 feet of freeboard.

Considering 10 percent and 20 percent debris blockage, the Camelback Road, RID flume and the Southern Pacific Railroad crossings do not possess the 4 feet of freeboard. However, pressure or weir flow does not develop at these three crossings with the 10 and 20 percent blockages. Therefore the bridges should be adequate to pass the 100-year flood, but the margin of safety will be reduced.

The bed elevation at the Southern Pacific Railroad was not lowered as the sedimentation analysis indicated this reach was in an approximate equilibrium condition in response to the 10-year flood, and showed a slight aggradation tendency in the QUASED model. Any flattening of the bed slope in this area would result in deposition and a decrease in the discharge capacity.

Table 7.2. Summary of Required Low Chord at Crossings with no Debris Blockage.

(1) LOCATION OF CROSSING	(2) LOW CHORD ELEVATION (MSL)	(3) HEC-II WATER SURFACE ELEVATION (MSL)	(4) 1/2 ANTIDUNE HEIGHT (FT)	(5) SUPERELEVATION AROUND BEND (FT)	(6) SURFACE WAVE HEIGHT (FT)	(7) AGGRADATION (FT)	(8) 4' OF FREEBOARD IF LESS THAN Σ OF COLUMN 4,5,6 & 7 (FT)	(9) REQUIRED LOW CHORD (MSL)
Camelback Road	1,026.2	1,023.4	1.52	---	0.86	---	4	1,027.4
Indian School Road	1,015.4	1,010.2	0.69	---	0.39	0.8	4	1,014.2
Roosevelt Irrigation District Flume	1,010.0	1,006.0	1.56	---	0.89	1.0	4	1,010.0
McDowell Road	992.5	983.8	1.64	---	0.93	---	4	987.8
I-10	988.51	978.1	0.64	0.43	0.36	---	4	982.1
Southern Pacific Railroad	966.2	962.5	0.94	---	0.53	0.5	4	966.5
Buckeye Road	968.1	961.9	1.05	---	0.59	0.6	4	965.9

Table 7.3. Summary of Required Low Chord at Crossings with 10% Debris Blockage.

(1) LOCATION OF CROSSING	(2) LOW CHORD ELEVATION (MSL)	(3) HEC-II WATER SURFACE ELEVATION (MSL)	(4) 1/2 ANTIDUNE HEIGHT (FT)	(5) SUPERELEVATION AROUND BEND (FT)	(6) SURFACE WAVE HEIGHT (FT)	(7) AGGRADATION (FT)	(8) 4' OF FREEBOARD IF LESS THAN Σ OF COLUMN 4,5,6 & 7 (FT)	(9) REQUIRED LOW CHORD (MSL)
Camelback Road	1,026.2	1,024.0	1.52	---	0.86	---	4	1,028.0
Indian School Road	1,015.4	1,010.6	0.69	---	0.39	0.8	4	1,014.6
Roosevelt Irrigation District Flume	1,010.0	1,006.8	1.56	---	0.89	1.0	4	1,010.8
McDowell Road	992.5	985.0	1.64	---	0.93	---	4	989.0
I-10	988.51	979.4	0.64	0.43	0.36	---	4	983.4
Southern Pacific Railroad	966.2	963.4	0.94	---	0.53	0.5	4	967.4
Buckeye Road	968.1	962.4	1.05	---	0.59	0.6	4	966.4

Table 7.4. Summary of Required Low Chord at Crossings with 20% Debris Blockage.

(1) LOCATION OF CROSSING	(2) LOW CHORD ELEVATION (MSL)	(3) HEC-II WATER SURFACE ELEVATION (MSL)	(4) 1/2 ANTIDUNE HEIGHT (FT)	(5) SUPERELEVATION AROUND BEND (FT)	(6) SURFACE WAVE HEIGHT (FT)	(7) AGGRADATION (FT)	(8) 4' OF FREEBOARD IF LESS THAN Σ OF COLUMN 4,5,6 & 7 (FT)	(9) REQUIRED LOW CHORD (MSL)
Camelback Road	1,026.2	1,024.3	1.52	---	0.86	---	4	1,028.3
Indian School Road	1,015.4	1,011.0	0.69	---	0.39	0.8	4	1,015.0
Roosevelt Irrigation District Flume	1,010.0	1,007.7	1.56	---	0.89	1.0	4	1,011.7
McDowell Road	992.5	986.5	1.64	---	0.93	---	4	990.5
I-10	988.5	980.6	0.64	0.43	0.36	---	4	984.6
Southern Pacific Railroad	966.5	964.6	0.94	---	0.53	0.5	4	968.6
Buckeye Road	968.1	963.3	1.05	---	0.59	0.6	4	967.3

VIII. CONCEPTUAL DESIGN OF AGUA FRIA RIVER CHANNELIZATION

8.1 General

The conceptual channelization design of the Agua Fria River from Camelback Road to Buckeye Road is presented in this chapter. The river is divided into the following reaches to discuss channelization measures:

- transition area upstream of Camelback Road
- Camelback Road to Indian School Road
- Indian School Road to Thomas Road
- Thomas Road to I-10
- I-10 to Buckeye Road

Preliminary quantity and cost estimates are provided for comparison of alternatives.

8.2 Transition Upstream of Camelback Road

Upstream of the proposed Camelback Road Bridge the 100-year flood plain is approximately 4,400 feet wide. The Camelback Road bridge opening will be 1,725 feet wide (see Plate 1). Thus, a considerable amount of overbank flow will not pass through the bridge.

The west braid of the Agua Fria will convey approximately 15,000 cfs at the 100-year flood peak of 95,000 cfs. Figure 8.1 shows the 100-year water surface elevations at cross sections located near the bridge.

PRC Toups has designed spur dikes to align the flow through the Camelback Road Bridge in the main channel and has proposed channelization upstream of the bridge to increase the flow capacity of the main channel. Overbank flow in the west and local drainage will pass through an 8-foot by 8-foot box culvert, and overbank flow on the east will flow over a low weir section of the east spur dike.

Three alternatives exist between Camelback Road and Indian School Road Bridge and include (1) a protected levee on the east bank and a partial dike extending 1,650 feet upstream of ISRB terminating in a transverse dike, (2) levees with protection on both the east and west banks, and (3) a partial levee on the west bank extending 1,650 feet upstream of ISRB terminating in a transverse dike and two transverse dikes on the east bank as well as a spur dike just north of ISRB. For Alternatives 1 and 3 there is no need to change

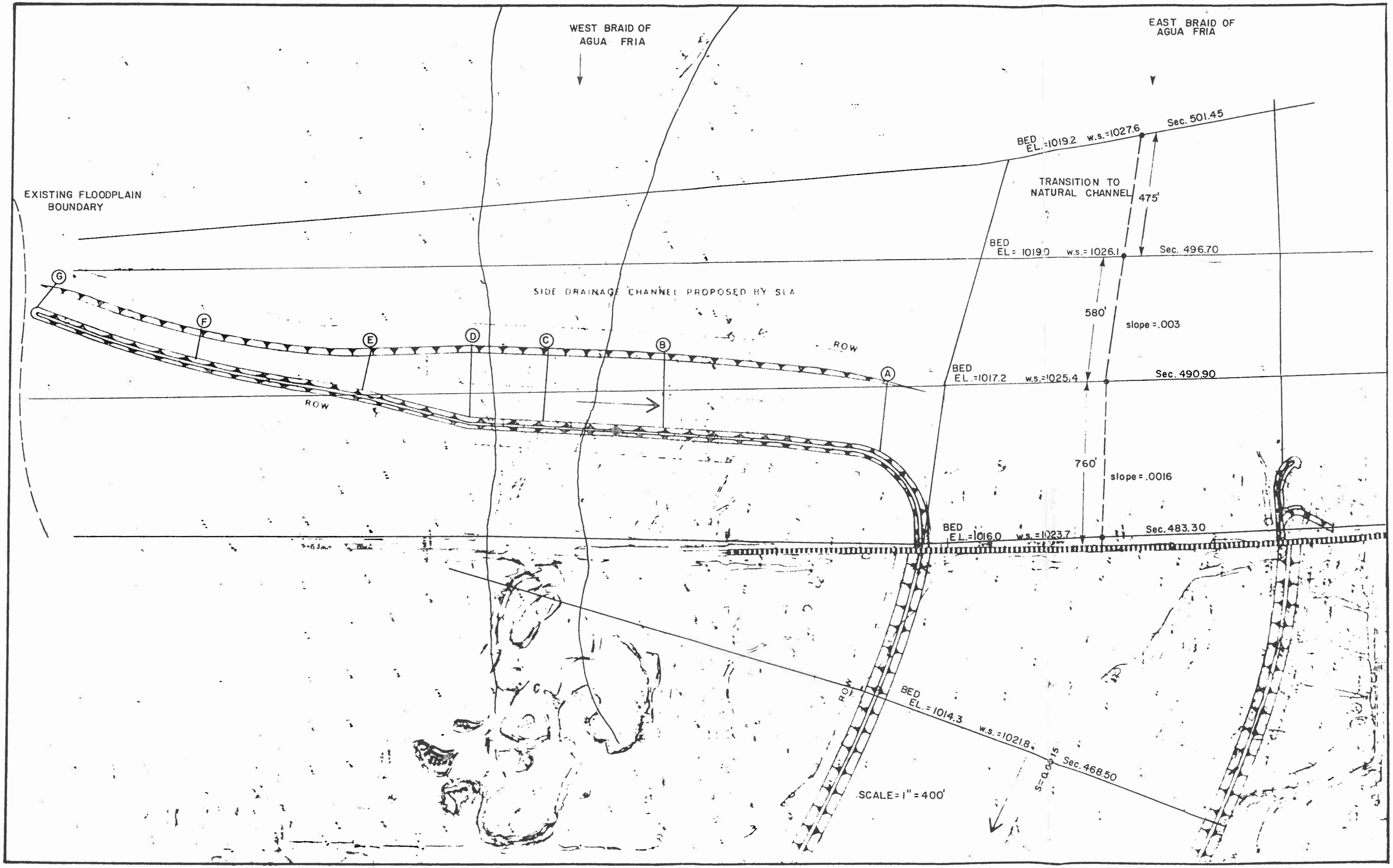


FIGURE 8.1 SKETCH OF SIDE DRAINAGE CHANNEL

the present design of PRC Toups. Some rip-rap protection near the box culvert may be necessary and a channel with approximately 750 cfs capacity should be provided through the gravel and sand mining operation on the downstream side of the box culvert; however, no major revisions are foreseen. The box culvert will cause ponding of the water in the west braid and some deposition of sediment. Some maintenance work may be required near the culvert to remove sediment.

For Alternative 2 a side drainage channel is proposed to collect the west overbank flow and bring it into the main channel upstream of the spur dike (see Figure 8.1) in lieu of the culvert. The side drainage channel will have the following advantages:

1. Confines all the flow in the main channel at Camelback Road.

2. Averts having to construct a side drainage channel to drain overbank flow downstream of Camelback Road if a continuous levee is built on the west bank between ISRB and Camelback Road.

3. Avoids the refilling of gravel pits just downstream of the culvert outlet.

The disadvantages of the side drainage channel will be the maintenance required to keep the channel free from sediment and debris, and the right-of-way acquisition required to construct the channel.

For Alternative 2 (levee on both east and west bank) between Camelback Road and ISRB, the bed elevation at Camelback Road is 1,016 feet, which is approximately 1 foot below the present bed elevation. The proposed bed profile of the main channel is shown in Figure 8.1. The channelization will encourage a larger portion of the water to remain in the main channel and provide a head differential between the side drainage channel and the main channel.

Figure 8.2 shows the bed profile and width for the proposed side drainage channel. A trapezoidal cross section is suggested with 4:1 side slopes on the upstream face and 3:1 side slopes on the downstream face. Figure 8.3 shows a typical cross section. The designed side drainage channel will have a capacity of approximately 15,000 cfs. The levee on the downstream face should be protected with riprap or soil cement from Section A to E and have a crown elevation of 1,028 feet. This elevation will contain the 100-year water surface and allow 3 feet of freeboard.

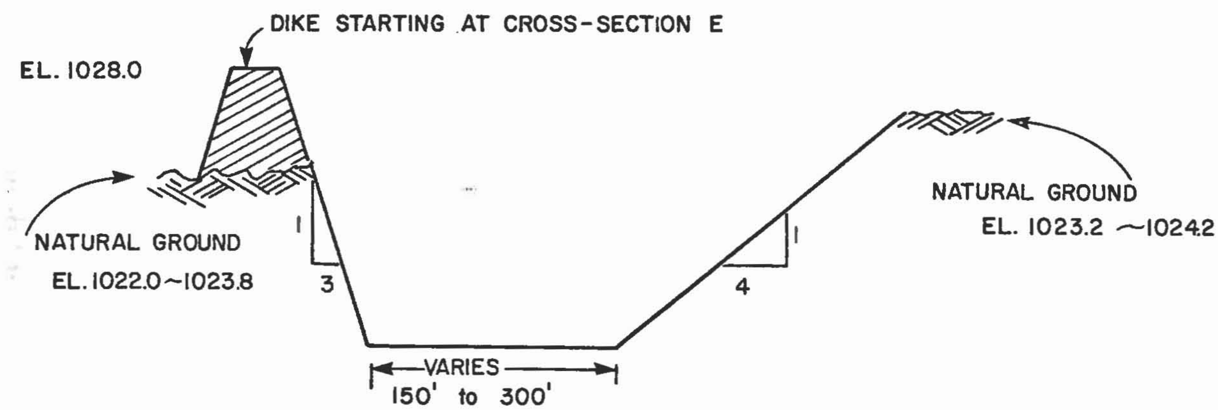


Figure 8.3. Typical cross section for side drainage channel.

Table 8.1. Cost Comparison of Side Drainage Channel and Channelization Upstream of Camelback Road Considering Riprap Protection and Soil Cement Protection of Dikes.

ITEM	COST WITH RIPRAP PROTECTION	COST WITH SOIL CEMENT PROTECTION
Drainage Channel	$ 431,705	$387,375
Agua Fria Channelization upstream of Camelback Road	225,000	225,000
Right-of-Way Acquisition	260,000	260,000
SUBTOTAL	$ 916,705	$872,375
10% contingencies and construction supervision	91,670	87,240
TOTAL	$1,008,375	$959,615

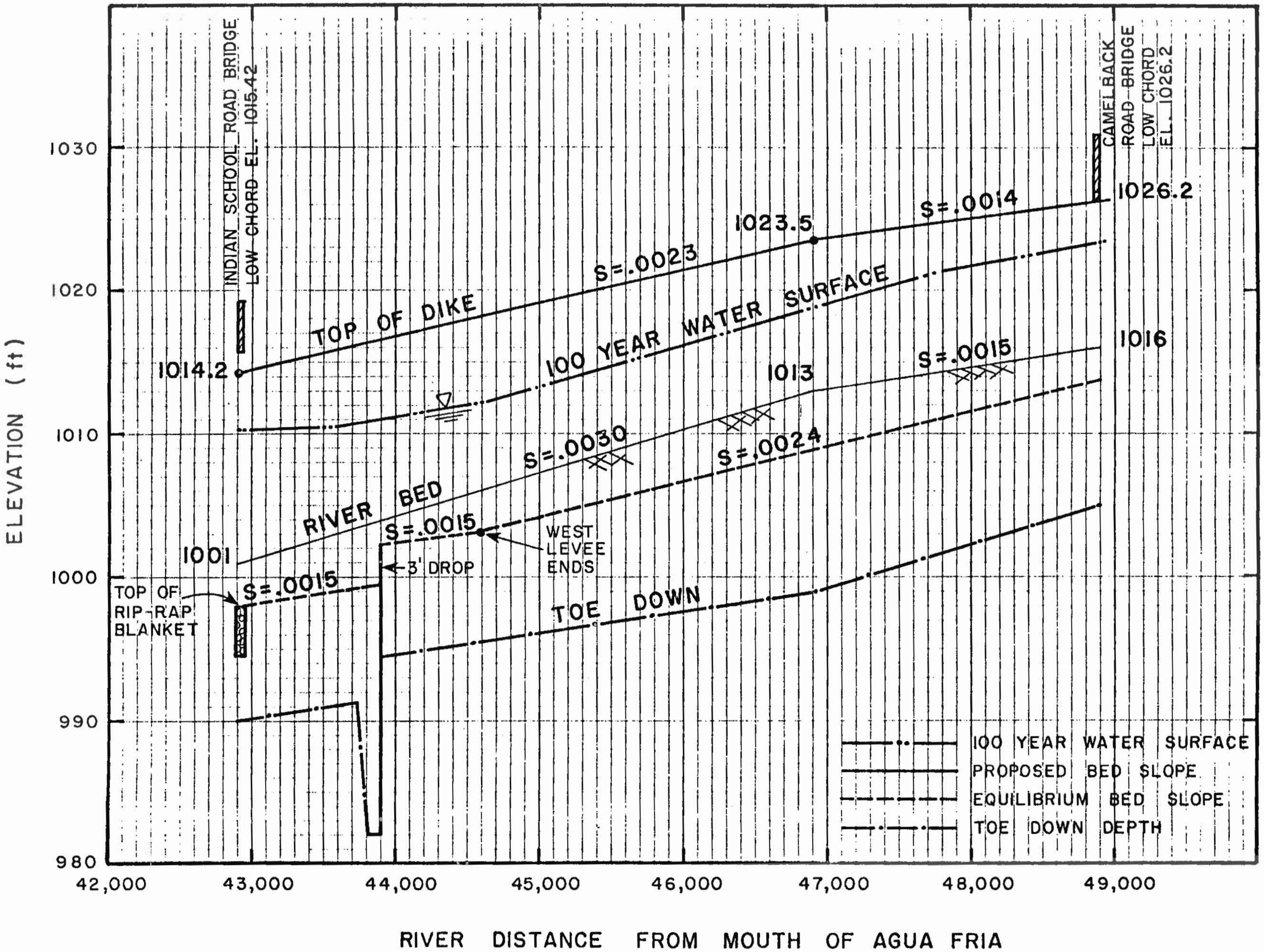


Figure 8.4. Proposed bed profile, equilibrium bed profile, 100-year water surface elevation, levee heights and toe down depths between Camelback and Indian School Roads for Alternative 1.

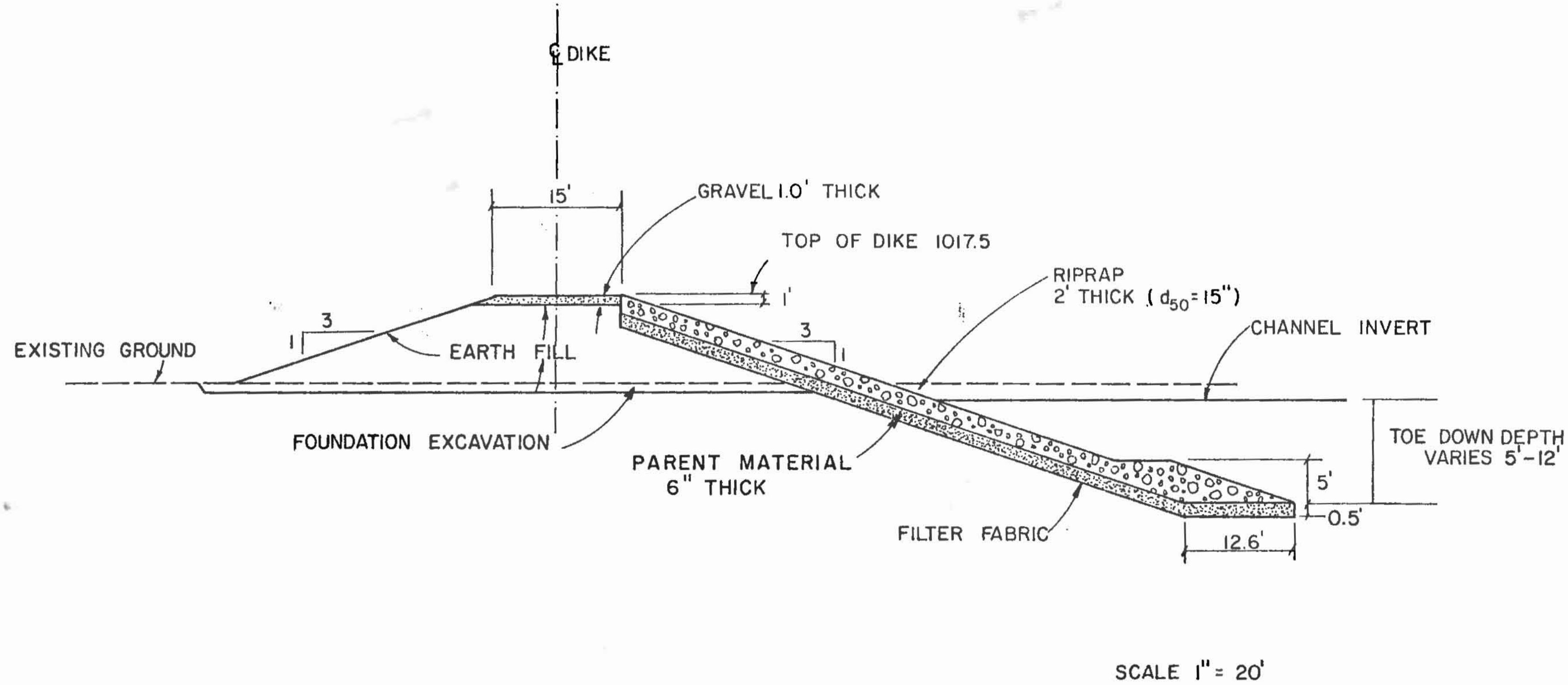


Figure 8.5. Typical cross section of transverse dike.

8.3.1.5 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 scour potential at the bridge for the 100-year flood is 20.3 feet, which is approaching the depth of burial of the piers. Therefore, protection measures of the shallow bridge piers are recommended.

A riprap blanket extending 20 feet around the piers is suggested. The blanket will be 2.5 feet thick and have a gravel filter of approximately 1 foot. The top of the riprap protection should start at elevation 998.

Bridge piers 13 through 17 are being 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.

8.3.1.6 Riprap Protection

The levee, transverse dike, drop structure and shallow bridge piers will need some form of protection. Two types of protection are being considered and include riprap and soil cement.

Riprap protection of the levees and transverse dike was based on the factor of safety method presented in Sediment Transport Technology (Simons and Senturk, 1977). The riprap was designed to have a factor of safety of 1.5 or greater. Based on the maximum velocity in the channel of approximately 13 ft/sec, riprap with a D_{50} size of 9 inches would be adequate for the levees. Velocities approaching the magnitude are also present near the transverse dike. To provide an additional factor of safety for local velocities which might exceed 13 fps, the following riprap was selected:

Rock Size	Percent Passing Sieve
24"	100
15"	50-70
7"	15-30
3"	0-5

The thickness of the riprap is 1.5 times the D_{50} size or approximately 24 inches thick.

Table 8.2. Preliminary Cost Estimates of Design Components Between Camelback Road and Indian School Road.

ITEM	COST OF RIPRAP PROTECTION FOR LEVEES AND TRANSVERSE DIKE WITH A REINFORCED CONCRETE DROP STRUCTURE	COST OF SOIL CEMENT PROTECTION OF LEVEES AND TRANSVERSE DIKE WITH A SOIL CEMENT DROP STRUCTURE
Channelization	$ 963,000	$ 963,000
Levees (riprap protection)	1,343,660	-0-
Levees (soil cement protection)	-0-	1,332,520
Drop Structure (reinforced concrete)	1,256,690	-0-
Drop Structure (soil cement)	-0-	653,230
ISRB Pier Protection (riprap blanket)	140,140	140,140
Transverse Dike (riprap protection)	62,425	-0-
Transverse Dike (soil cement protection)	-0-	96,690
Land Acquisition (245 acres)	1,225,000	1,225,000
Flowage Easement (73 acres)	-0-	-0-
SUBTOTAL	$4,990,915	$4,410,580
10% contingencies and construction supervision	499,090	441,060
TOTAL	$5,490,005	$4,851,640

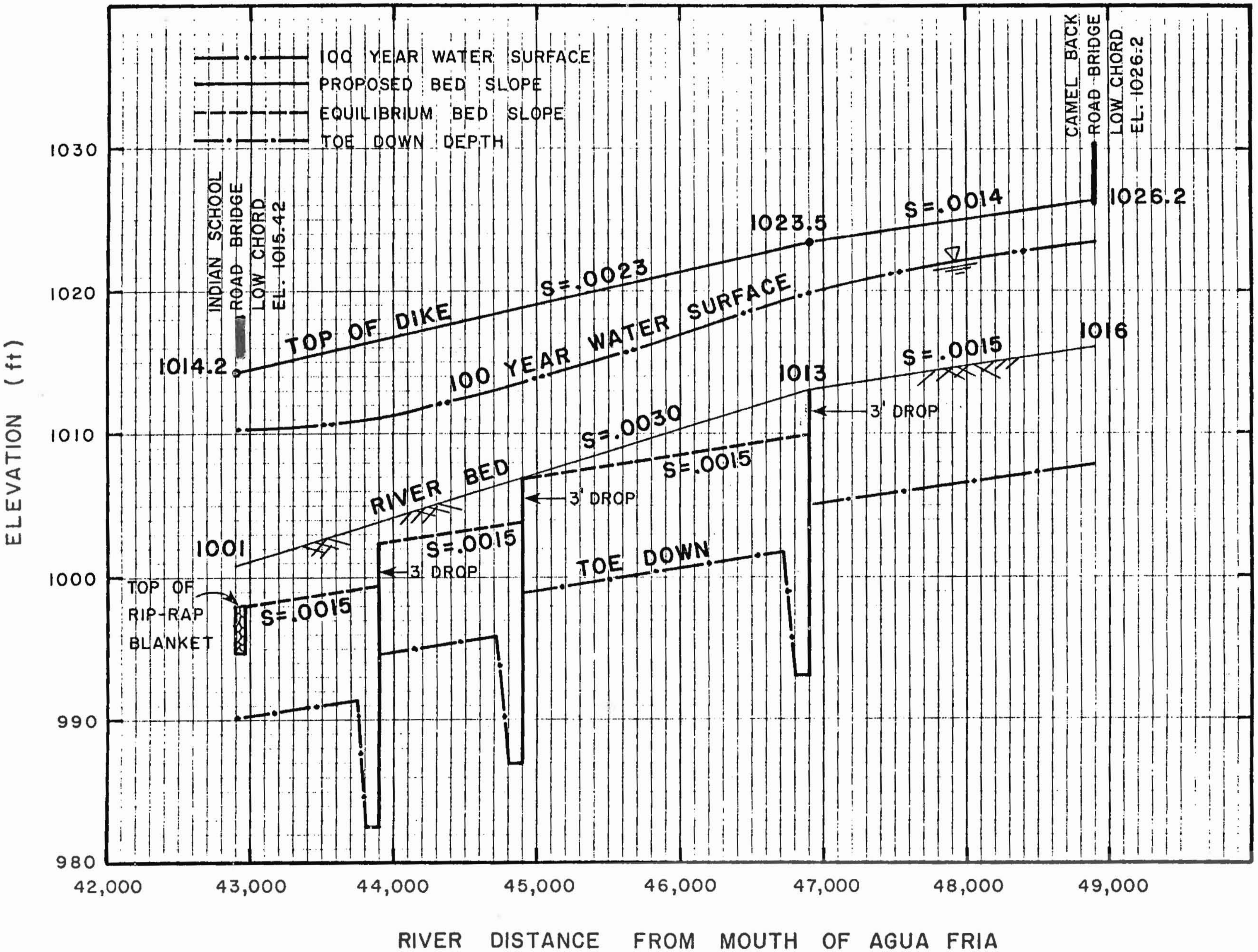


Figure 8.7. Proposed bed profile, equilibrium bed profile, 100-year water surface elevation, levee heights and toe down depths between Camelback Road and ISRB for Alternative 2.

Table 8.3. Preliminary Cost Estimate of Design Components for Alternative 2 Between Camelback Road and Indian School Road.

ITEM	COST OF RIPRAP PROTECTION FOR LEVEES AND THREE REINFORCED CONCRETE DROP STRUCTURES	COST OF SOIL CEMENT PROTECTION OF LEVEES AND THREE SOIL CEMENT DROP STRUCTURES
Channelization	$ 963,000	$ 963,000
Levees (riprap protection)	1,895,280	-0-
Levees (soil cement protection)	-0-	1,866,040
Three Drop Structures (reinforced concrete)	3,770,075	-0-
Three Drop Structures (soil cement)	-0-	1,959,700
ISRB (pier protection)	140,140	140,140
Land Acquisition	1,200,000	1,200,000
SUBTOTAL	$7,968,495	$6,128,880
10% contingencies and construction supervision	796,850	612,890
TOTAL	$8,765,345	$6,741,770

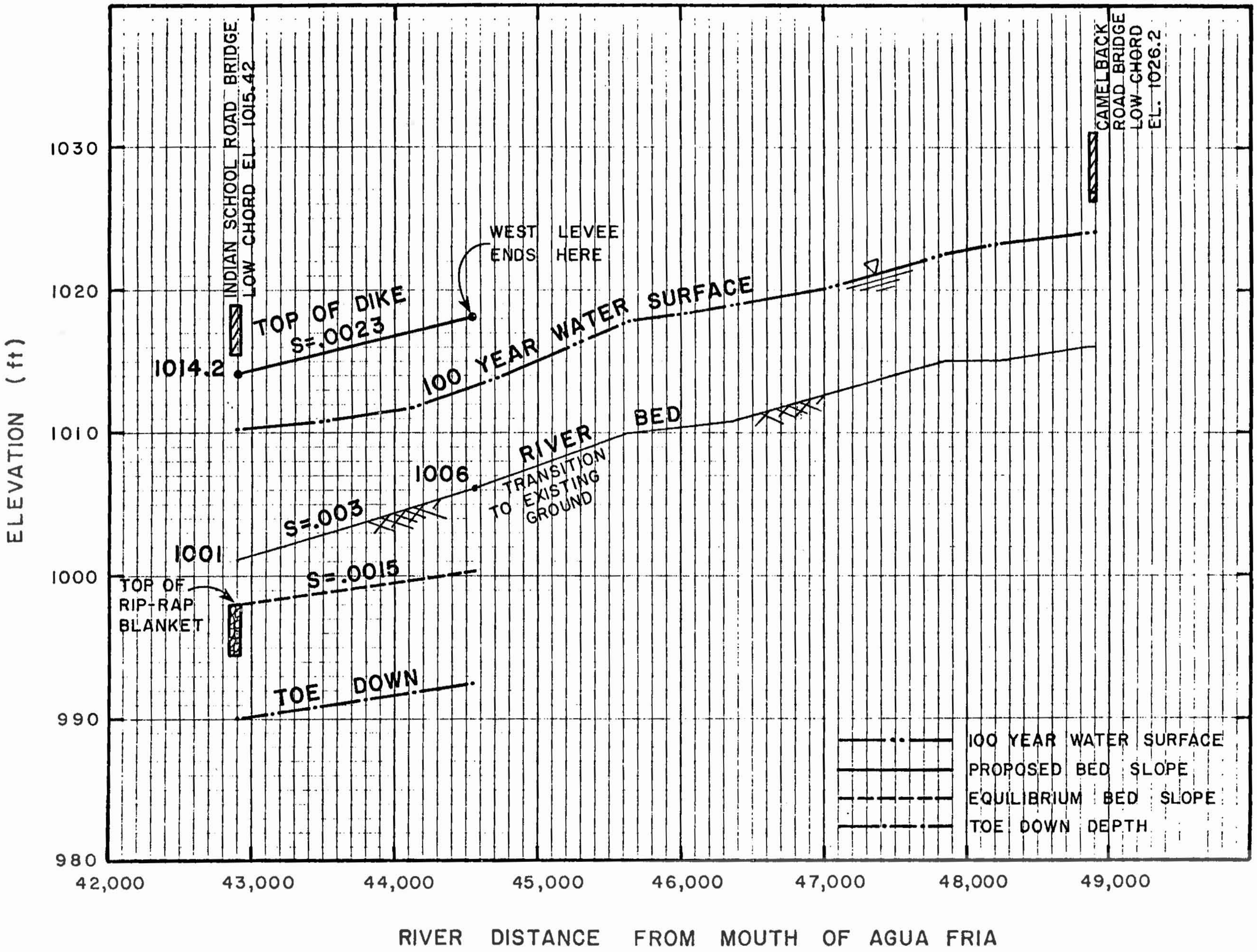


Figure 8.8. Proposed bed profile, equilibrium bed profile, 100-year water surface elevation, levee height and toe down depth between Camelback and Indian School Roads for Alternative 3.

Figure 8.10 provides the conceptual design of Transverse Dike 2. The other transverse dike is identical except its length is 600 feet rather than 1,600 feet. At locations where flows are to attack the dike most severely the side slopes are to be 3:1. The remainder of the side slopes can be at 2:1. Flatter side slopes provide more stability for the riprap protection. The top of the dikes are 3 feet above the 95,000 cfs flood level. This allows adequate freeboard to prevent waves and local acceleration of the flow around the dikes from overtopping the dikes.

Figure 8.11 provides the conceptual design of the spur dike. 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. As was the case with the transverse dikes, the spur dikes have 3:1 side slopes in areas where hydraulic conditions are most severe and 2:1 slopes in remaining areas.

8.3.3.4 Cost Estimate of Alternative 3

Table 8.4 compares the costs of protecting the levee, transverse dikes and spur dike with riprap against the costs of protecting the levees, transverse dikes and spur dike with soil cement for Alternative 3. Tables A.7 and A.8 summarize the quantities, unit costs and total costs of the abovementioned alternatives.

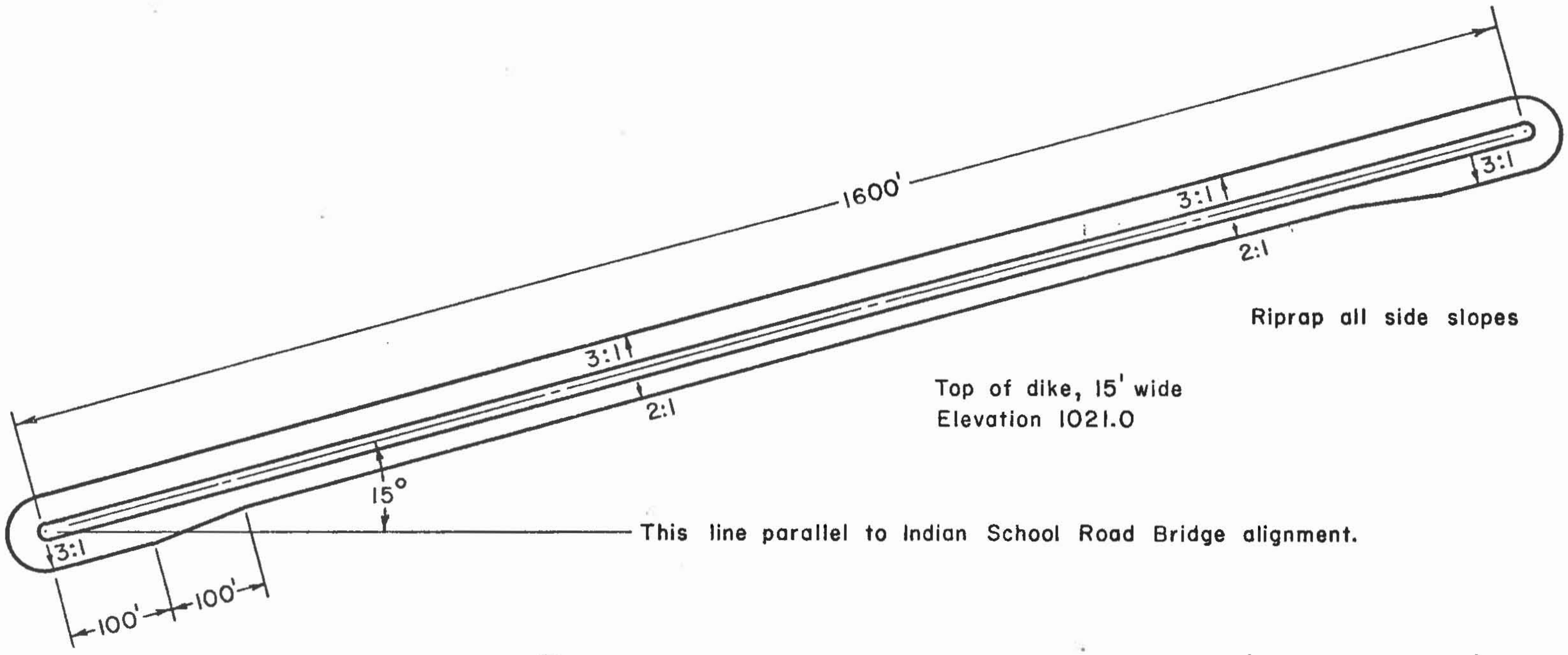
8.4 Conceptual Design Between Indian School Road and Thomas Road

8.4.1 General

The following components are being considered between Indian School and Thomas Roads: channelization, levees, backfilling of flood plain gravel pits, a drop structure just downstream of the Roosevelt Irrigation District (RID) flume, protection of RID flume piers, integration of the RID flume overflow structure into the Agua Fria and protection of utility transmission towers.

8.4.1.1 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 wide at ISRB to 920 feet at the RID flume transitioning to 1,100 feet wide at Thomas Road. The alignment between ISRB and the RID flume is the result of an agreement



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 1018.0'

Scale: 1" = 200'

Figure 8.10. Transverse Dike #2 design.

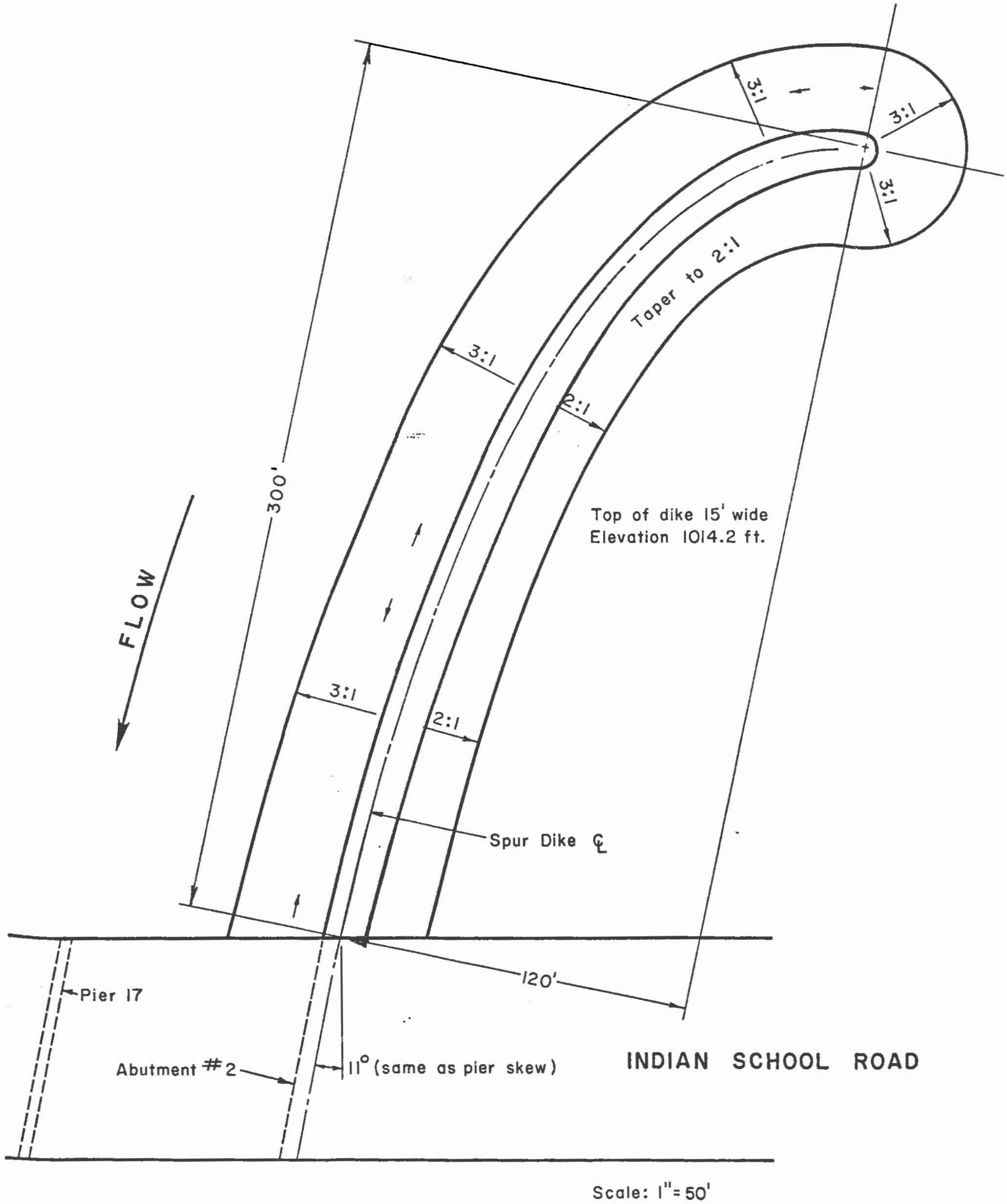


Figure 8.11. Spur Dike design.

Table 8.4. Preliminary Cost Estimate of Design Components for Alternative 3 Between Camelback Road and Indian School Road.

ITEM	COST OF RIPRAP PROTECTION FOR LEVEE, TRANSVERSE DIKES AND SPUR DIKE	COST OF SOIL CEMENT PROTECTION FOR LEVEE, TRANSVERSE DIKES AND SPUR DIKE
Channelization	$ 541,215	$ 541,215
Levee (riprap protection)	297,800	---
Levee (soil cement protection)	---	276,875
Transverse Dikes (riprap protection)	633,760	---
Transverse Dikes (soil cement protection)	---	776,625
Spur Dike (riprap protection)	221,965	---
Spur Dike (soil cement protection)	---	268,565
ISRB (pier protection)	140,140	140,140
Land Acquisition	590,000	590,000
SUBTOTAL	$2,424,880	$2,593,420
10% contingencies and construction supervision	242,490	259,340
TOTAL	$2,667,370	$2,852,760

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 is required for the alignment between ISRB and Thomas Road.

Figure 8.12 shows the proposed grade from ISRB to Thomas Road. The bed slope is 0.0019 from Thomas Road to the RID flume and then steepens to 0.0023 from the flume to ISRB. The approximate thalweg elevations at Thomas Road, RID flume and ISRB are 990.2, 995.5 and 1,001 feet, respectively,

The channel will be trapezoidal in shape with 3:1 side slopes. A good portion of the levees will be constructed upon backfilled gravel pits, and therefore riprap protection of levees is recommended because it is more flexible than soil cement in case of settlement of the gravel pit fill.

8.4.1.2 Levees

Figure 8.12 shows levee heights and toe down depths between Thomas Road and ISRB. Levee heights range from 13 to 14.5 feet and toe down depths range from 8 to 11 feet. The levees will have 3:1 riverside and landside slopes. Crown widths of levees are 15 feet. A minimum 50-foot buffer zone behind the toe on the landside of the levee and gravel pits is required for slope stability of the levees.

Slope stability analyses were conducted on levees between Thomas Road and ISRB using 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

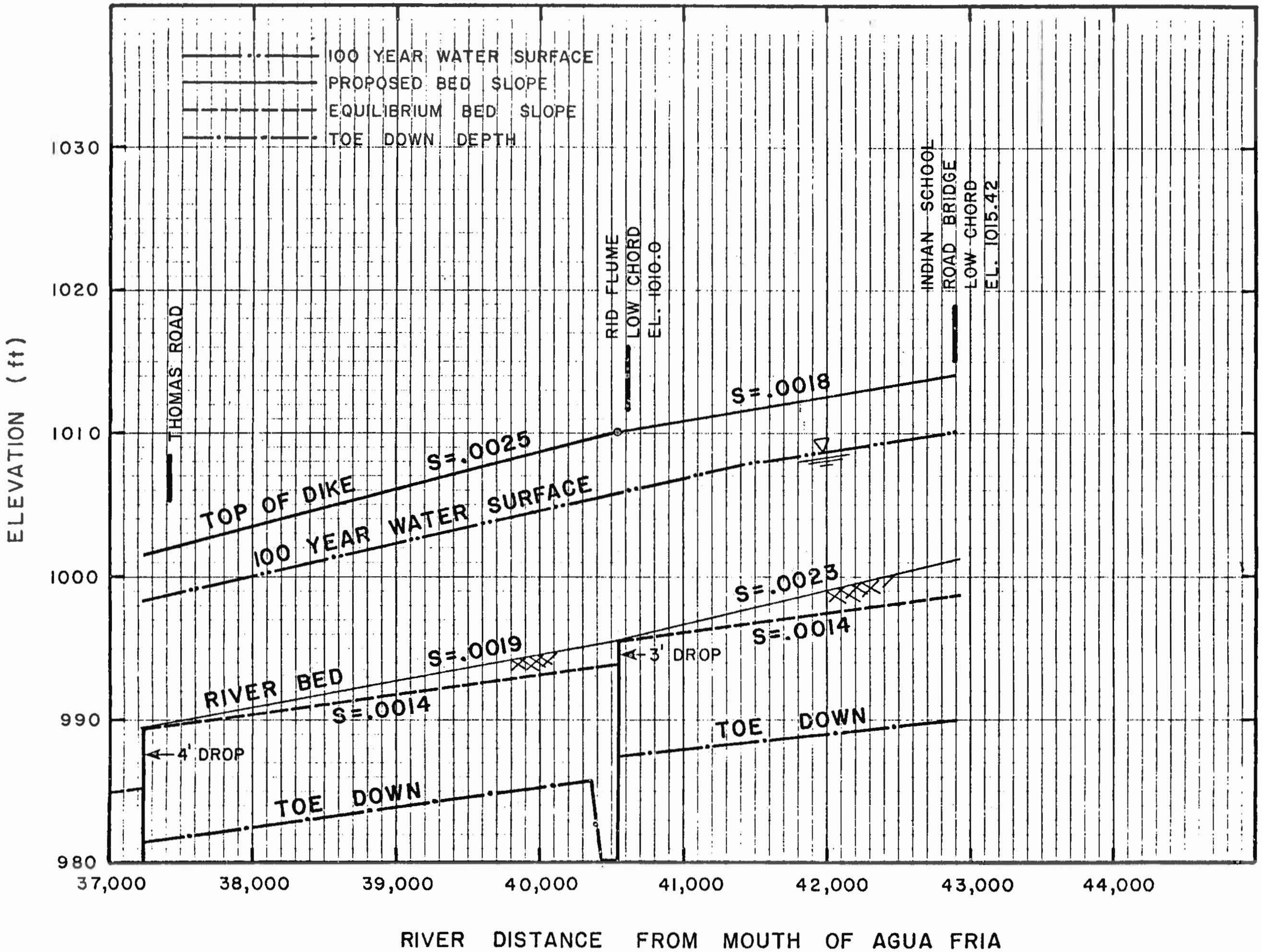


Figure 8.12. Proposed bed profile, equilibrium bed profile, 100-year water surface elevation, levee heights and toe down depths between Thomas and Indian School Roads.

Phreatic surfaces were developed assuming impervious material beneath the levee, which is a very conservative assumption. Casegrandes' method was used to establish the phreatic surface through the embankment for Cases II, III and IV.

Case I, end of construction, was not considered a severe problem because there will be minimal if any cohesive materials in the embankment. Any pore pressures that will develop during compaction in the embankment will dissipate quickly because of the free draining soil.

The rapid drawdown, critical flood stage and steady seepage from full flood stage were analyzed and the factor of safety never dropped below 1.2, 1.5 and 1.5, respectively for each of the cases. These factors of safety are above the minimum factors of safety listed in Table 6.1 of Engineering Manual EM 1110-2-1913 recommended by the Corps of Engineers.

Case IV was analyzed in lieu of Case V. The earthquake loading, Case VI, was ignored because the probability of an earthquake occurring along with a 100-year event is so minute, that it was not considered in the analyses.

8.4.1.3 Drop Structure

A 3-foot drop structure located just downstream of the RID flume is required to control the grade at the flume. The drop structure will be 920 feet wide and the top of the drop will be at elevation 995.5 feet. A reinforced concrete and soil cement drop structure are being considered.

8.4.1.4 Protection of RID Flume

The foundation for the RID flume consists of piers that extended 12 feet below the channel bed in 1929. A recent uncovering of one of the piers in the main channel revealed that the piers were only 3 feet below the channel bed. The piers rest on concrete footings which in turn are supported by concrete piles. The piles vary in length depending on the distance to hardpan but are approximately 17 to 25 feet long. Thus, the total foundation depths range from 23 to 31 feet. Local scour potential for the 100-year flood approaches 14 feet. Due to the uncertainty of burial depths of piles, riprap protection of flume piers is suggested.

The riprap protection should extend 20 feet in all directions around flume piers. A 2.5 foot thick riprap blanket with a 1-foot gravel filter should provide adequate protection. The top of the blanket should be constructed at elevation 995.5.

8.4.1.5 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. On the east overbank just downstream of the RID flume, Phoenix Sand and Rock has a gravel pit that will require 10,000 cubic yards of fill.

8.4.1.6 Cost Estimate Between ISRB and Thomas Road

Table 8.5 compares the costs of design components between ISRB and Thomas Road considering a reinforced concrete drop structure versus a soil cement drop structure below the RID flume. Riprap protection of levees was the only bank protection considered in this reach. Table A.9 in Appendix A summarizes quantities, unit costs and total costs between ISRB and Thomas Road.

8.5 Conceptual Design Between Thomas Road and I-10

8.5.1 General

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. A separate section detailing the analysis conducted on the I-10 collector

Table 8.5. Preliminary Cost Estimates of Design Components Between Indian School Road and Thomas Road.

ITEM	COST OF REINFORCED CONCRETE DROP STRUCTURE	COST OF SOIL CEMENT DROP STRUCTURE
Channelization	$ 637,435	$ 637,435
Levees (riprap protection)	1,743,240	1,743,240
Drop Structure (reinforced concrete)	772,780	-0-
Drop Structure (soil cement)	-0-	400,780
Gravel Pit Restoration	1,225,000	1,225,000
RID Flume Pier Protection (riprap blanket)	90,170	90,170
Transmission Tower Protection	200,000	200,000
Land Acquisition	770,000	770,000
SUBTOTAL	$5,438,625	$5,066,625
10% contingencies and construction supervision	543,860	506,660
TOTAL	$5,982,485	$5,573,285

channel is provided in Appendix B.

8.5.1.1 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 0.002, and from this drop structure to the drop structure located 200 feet downstream of Thomas Road the proposed bed slope is 0.0021. Approximately 280 acres of right-of-way are required for this alignment. Figure 8.13 shows the proposed bed profile from I-10 to Thomas Road.

8.5.1.2 Levees

Figures 8.13 and 8.14 show levee heights and toe down depths between Thomas Road and I-10 for the east and west banks, respectively. Levee heights range from 10 feet to 12.5 feet and toe down depths range from 8 feet to 12 feet on the east bank. 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 increase flow depths, necessitating an increase in freeboard height above the 100-year water surface. Required levee heights increase to approximately 8 feet above the 100-year water surface around the bend because 1/2 the antidune height and the superelevation around the bend sum to 8 feet. Toe down depths are extended 15 feet below the equilibrium bed profile due to the increased degradation potential.

The levees will have 3:1 riverside slopes if protected with riprap and 1:1 riverside slopes if protected with soil cement. Landside slopes of 3:1 to natural ground are recommended. Crown widths of the levees will be 15 feet. It is recommended that soil cement protection be used on the west bank between McDowell and I-10 because of the large flow velocity increase around the bend and the subsequent large increase in riprap sizes needed to stabilize the bank.

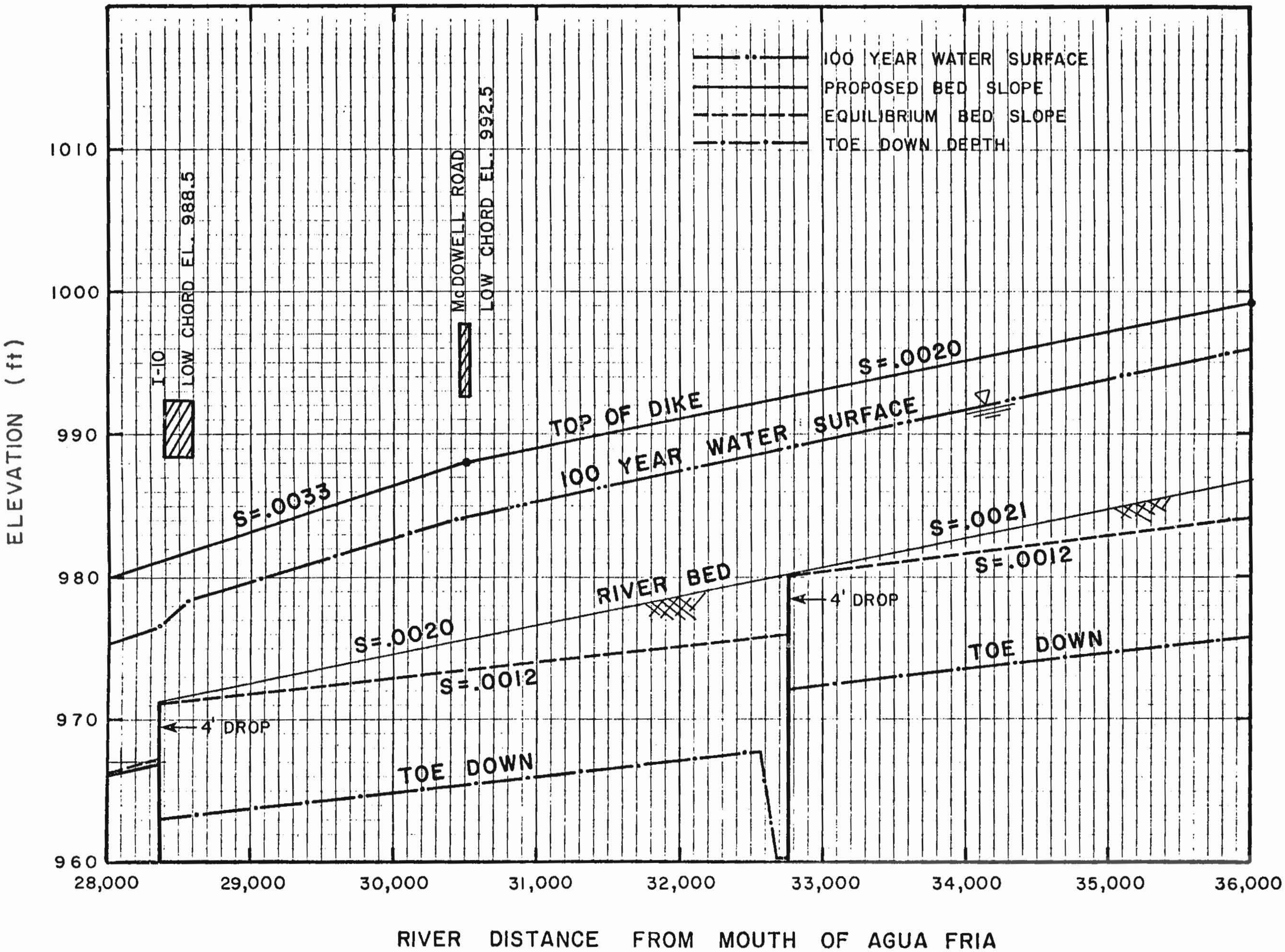


Figure 8.13. Proposed bed profile, equilibrium bed profile, levee heights and toe down depths between I-10 and Thomas Road for east bank.

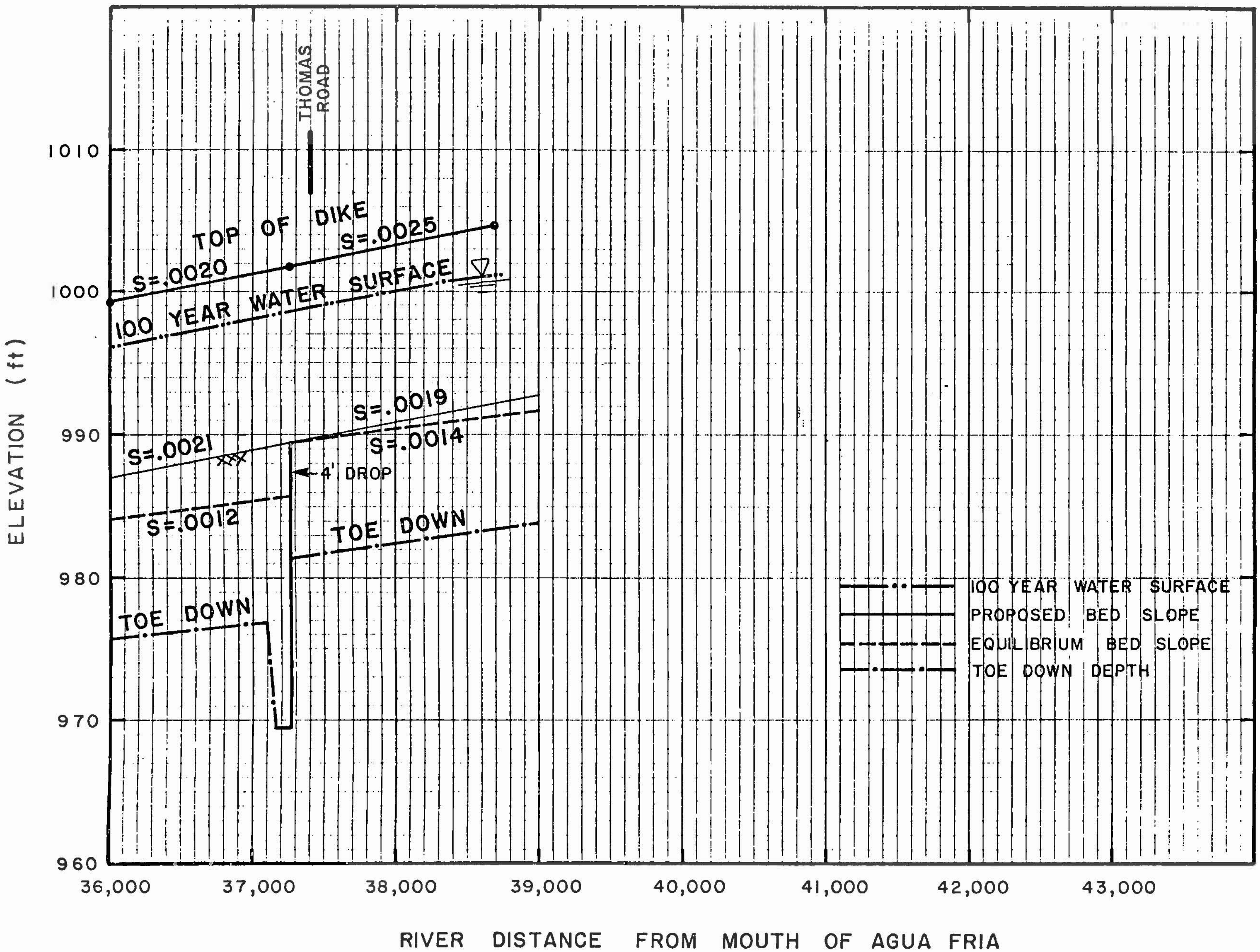


Figure 8.13. (Continued)

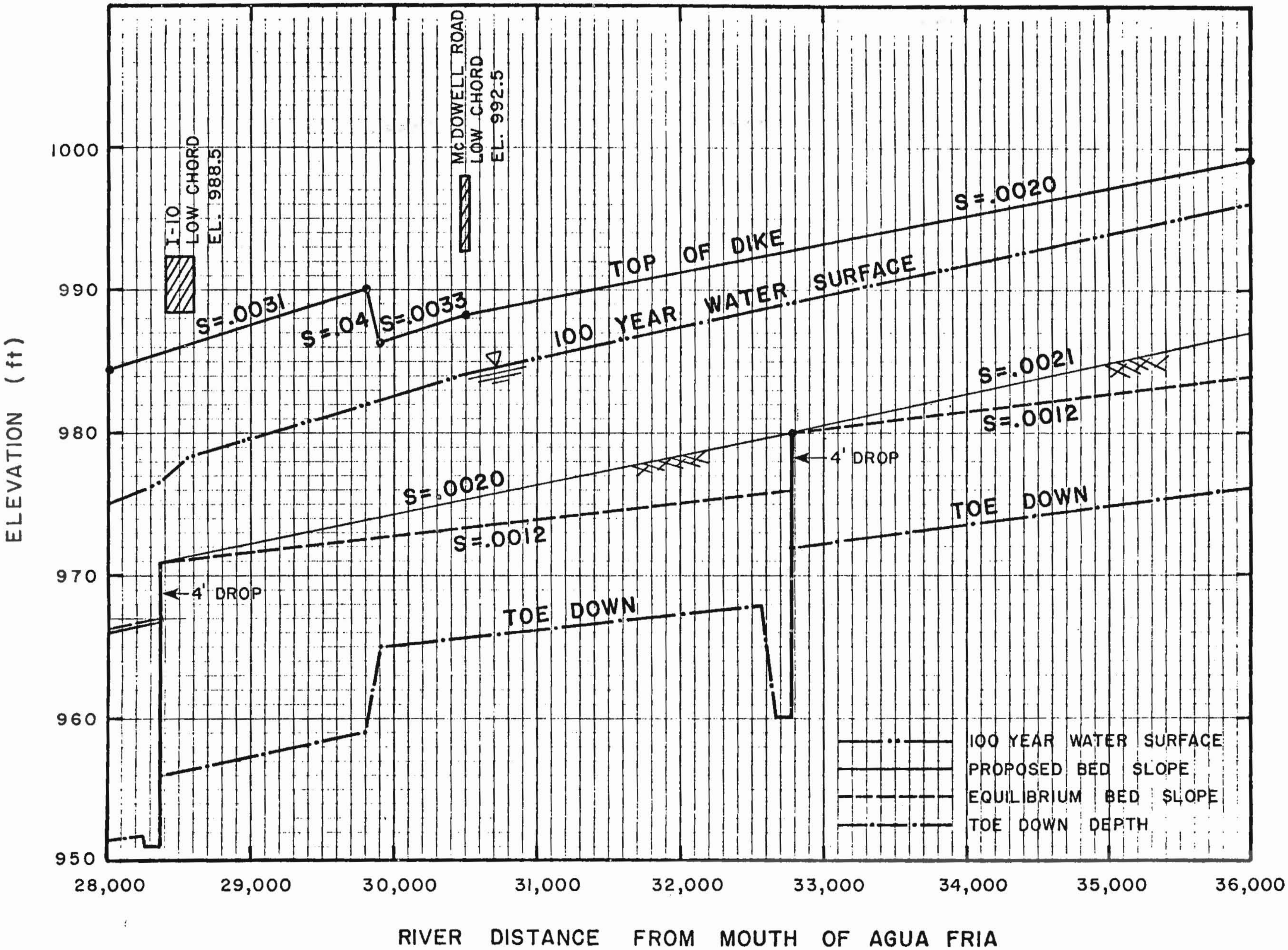


Figure 8.14. Proposed bed profile, equilibrium bed profile, levee heights and toe down depths between I-10 and Thomas Road for west bank.

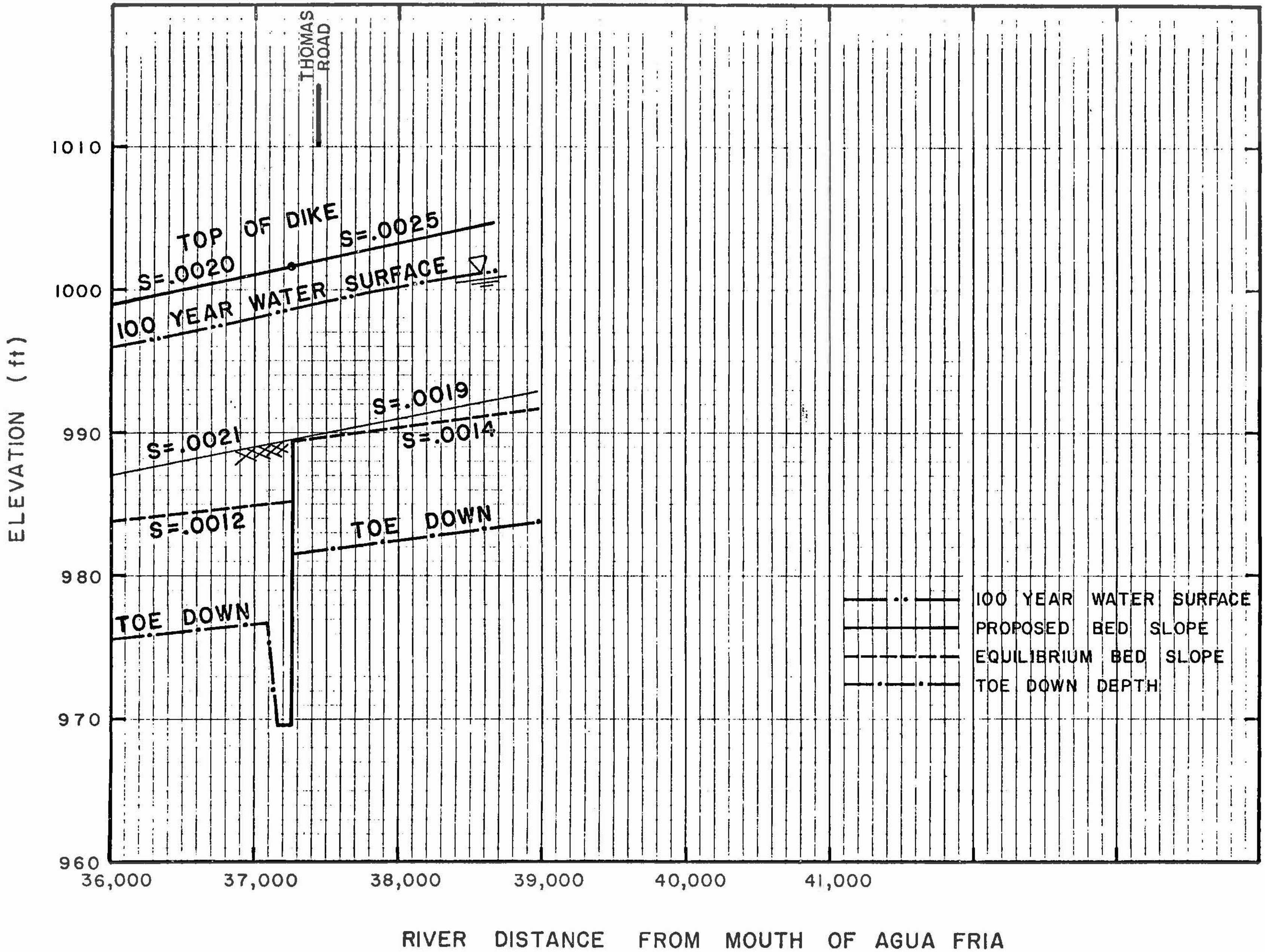


Figure 8.14. (Continued)

If riprap protection is selected, 2-foot thick riprap will provide the necessary protection against flow velocities in this reach except around the bend on the west bank. Should riprap be installed on the west bank around the bend, the D_{50} riprap size increases to 28 inches with a maximum size of 56 inches. Selecting a filter that will remain intact between the riprap and natural soil, with this large a difference in gradations, becomes almost impossible. Therefore soil cement is recommended for protection of the bend and is used in the cost estimate.

8.5.1.3 Drop Structures

Two 4-foot drop structures are needed between I-10 and Thomas Road. The locations of drop structures are 2,200 feet upstream of the proposed McDowell Road Bridge and 200 feet downstream of Thomas Road. The eventual equilibrium slope in this reach is 0.0012 with the proposed channelization and the existing grade is approximately 0.0021. Thus, the potential degradation in this reach is approximately 8 feet.

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 pipe line and 6-inch high pressure gas pipe line. However, portions of these lines will have to be lowered due to the channel bed being lowered in this vicinity.

8.5.1.4 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. Three alternatives are being considered for carrying the flows from the I-10 collector into the Agua Fria.

Alternative 1 would leave the collector channel and pilot channel as they presently exist. The east bank levee from McDowell Road will be extended to the pilot channel, and the spur dike just north of the I-10 bridge will be left intact. Approximately 200 feet will be open between the spur dike and upstream levee. Protection of the backside of the spur dike or grading behind the spur dike to prevent ponding of water and 1,000 feet of protection on the backside of the levee is necessary. Inherent with this alternative is the inundation of land in back of the levee between McDowell Road and I-10 when the Agua Fria flows bank full. Likewise, that area could also be inundated when higher flows occur in the I-10 channel. Approximately 20.6 acres of land shown in Figure 8.15, in addition to the right of way acquired by the Arizona Department of Transportation (ADOT) for I-10, will be subjected to flooding when the 100-year discharge occurs from the I-10 collector channel. Also inherent with this alternative is a headcut potential between the spur dike and levee. Should material erode from the pilot channel and deposit in the Agua Fria, a reduction in flow capacity results and forces an uneven flow distribution at I-10, which can cause additional scour. Therefore, maintenance will have to be performed if bars or islands form in this area.

Riprap could be used to stabilize the area, however (1) the expense is very prohibitive and (2) if the riprap gets exposed at the river-channel juncture, local waves and vorticity problems can become severe.

Alternative 2 considers extending the I-10 collector channel 2,900 feet to the Agua Fria. The channel will be trapezoidal in shape with a bottom width of 165 feet and have 2:1 side slopes. The channel will have an earth bottom and soil cement protection of banks. The banks of the channel will be extended to contain the 100-year discharge from the collector channel, and contain the 100-year water surface from the Agua Fria. The spur dike will be modified slightly to be in the shape of the levee upstream. Figure 8.16 shows an overview of Alternative 2.

The third alternative considered constructing a siltation basin between the collector channel and the Agua Fria. Figure 8.17 shows an overview of the 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 rain events. It is also suggested that the fill material of I-10 be tested for its suitability as an embankment for the siltation basin.

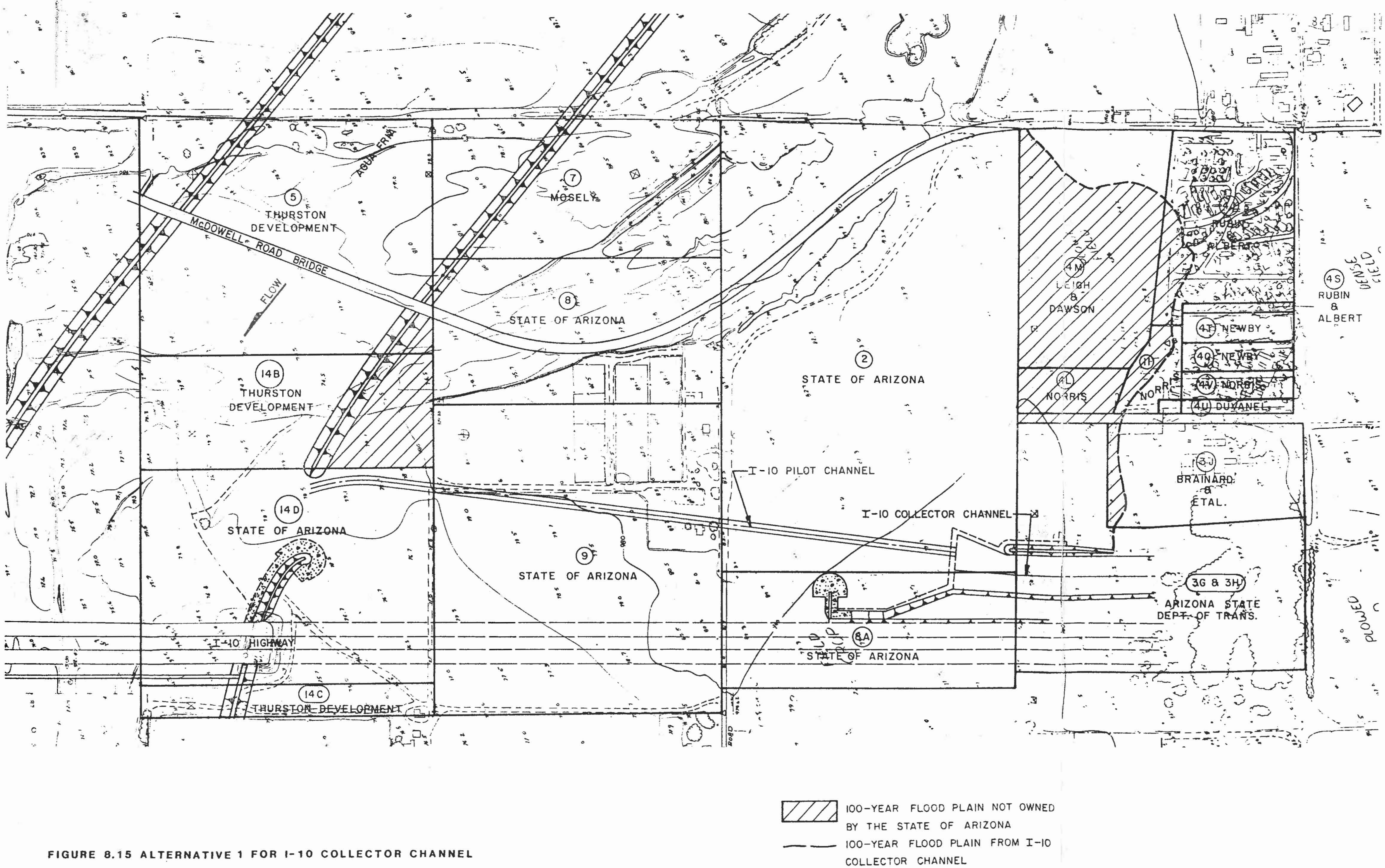


FIGURE 8.15 ALTERNATIVE 1 FOR I-10 COLLECTOR CHANNEL

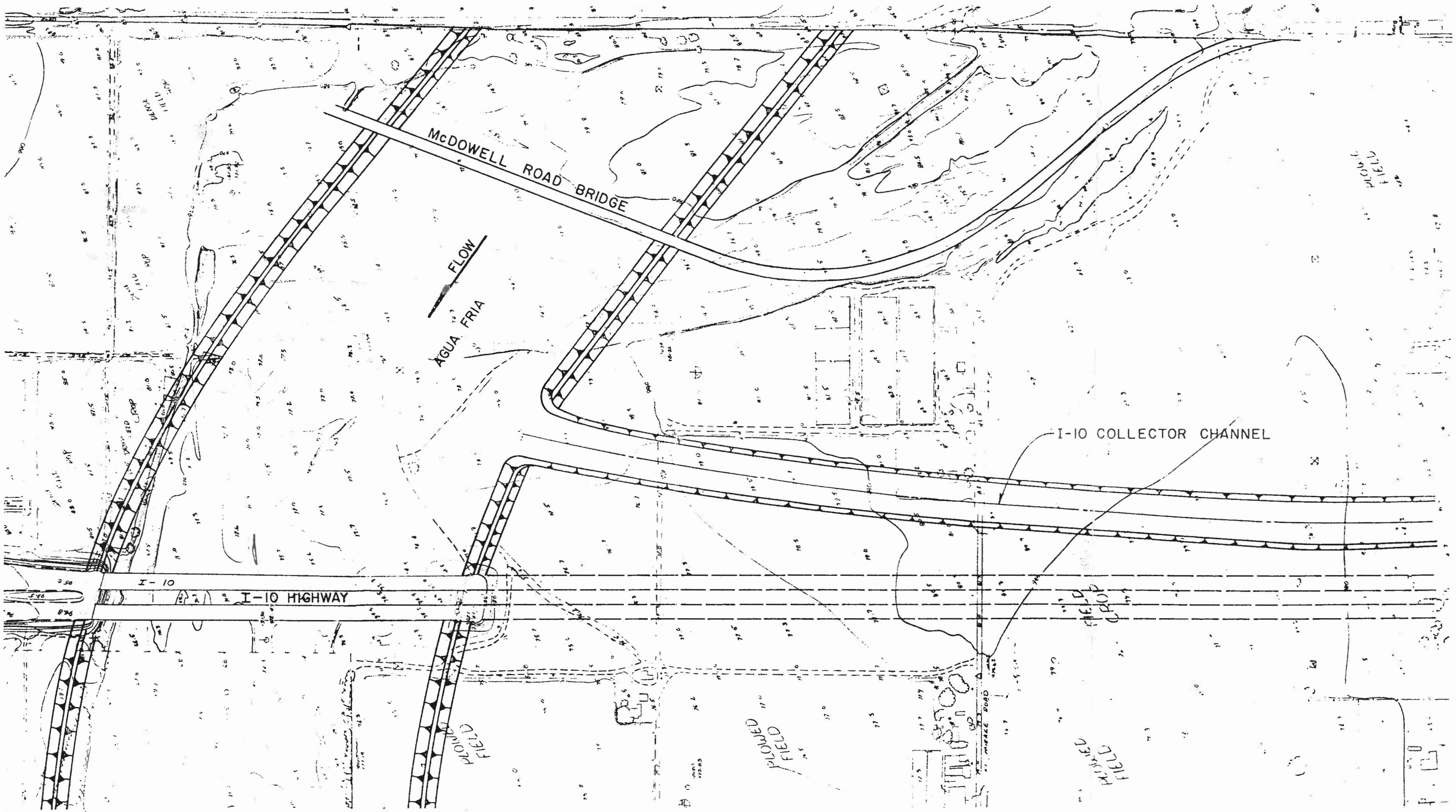


FIGURE 8.16 ALTERNATIVE 2 FOR I-10 COLLECTOR CHANNEL

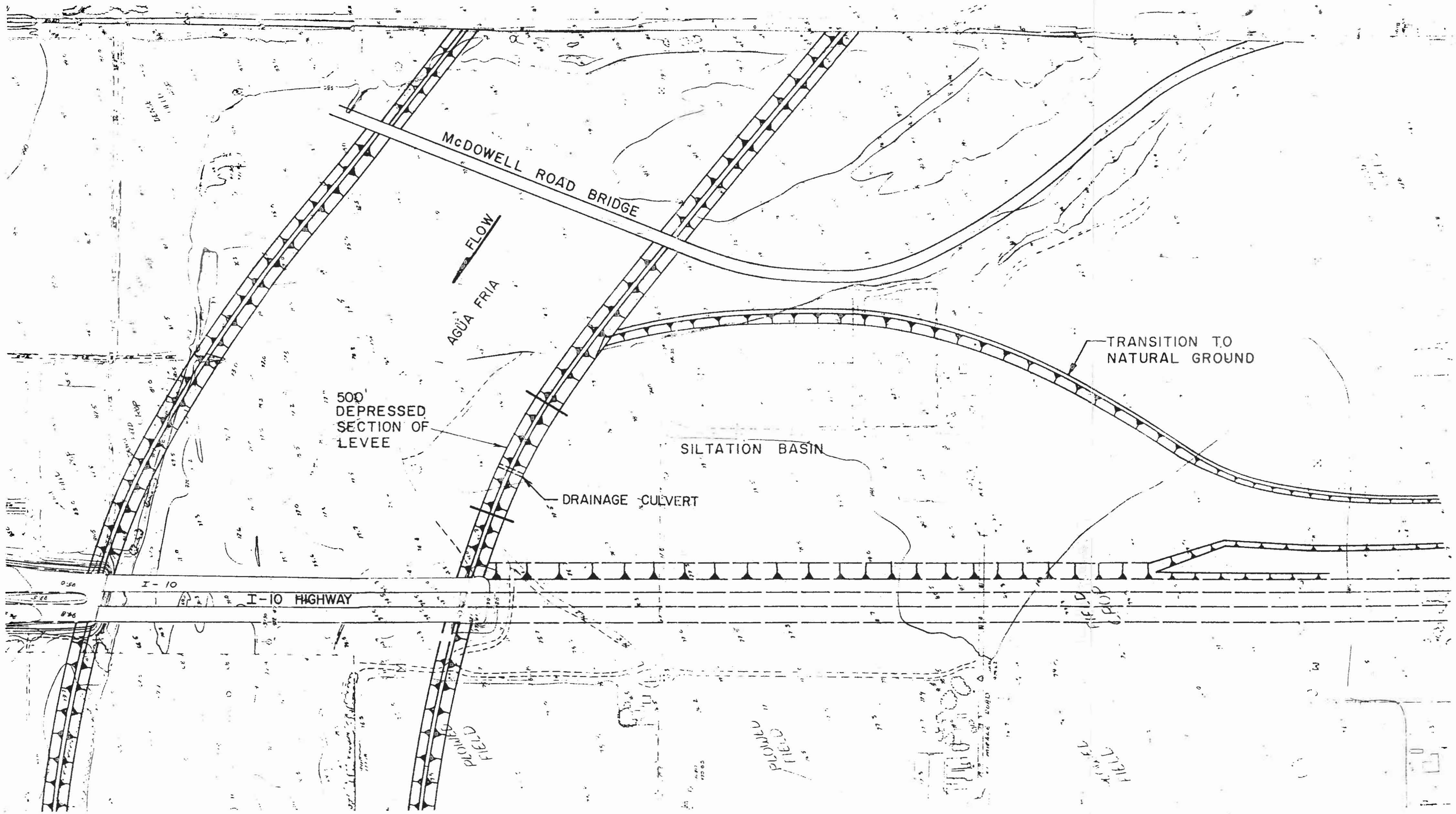


FIGURE 8.17 ALTERNATIVE 3 FOR I-10 COLLECTOR CHANNEL

The approximate costs of Alternatives 1, 2 and 3 are $108,000, $810,000 and $583,000, respectively. Alternative 1 is used for the cost estimate between Thomas Road and I-10.

8.5.1.5 Cost Estimate Between Thomas Road and I-10

Table 8.6 compares the costs of protecting the levees and drop structures with riprap except around the west bend between McDowell and I-10 bridge which is protected with soil cement and constructing two reinforced concrete drop structures with the cost of protecting the levees with soil cement, protecting the drop structures with riprap and constructing two soil cement drop structures, between Thomas Road and I-10. Leaving a 200 foot opening between the levee and spur dike on the east bank for the I-10 drainage was used for both cases in the cost estimate. Tables A.10 and A.11 in Appendix A summarize the quantities, unit costs and total costs for the cases mentioned.

8.6 Conceptual Design Between I-10 and Buckeye Road

8.6.1 General

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

8.6.1.1 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 gradually transitioning to 1,100 feet at Buckeye Road. The proposed bed slope in this reach is 0.0017 which is approximately the existing slope. Three hundred acres of channel right-of-way are required for the proposed alignment. Figure 8.18 shows the proposed bed profile.

8.6.1.2 Levees

Figures 8.18 and 8.19 show required levee heights and toe down depths between I-10 and Buckeye Road for the east and west banks, respectively. Levee heights on the east bank range from 10.5 feet to 14.5 feet and toe down depths range from 7 feet to 11.5 feet. Levee heights extend 8 feet above the 100-year water surface and toe down depths extend 15 feet below the

Table 8.6. Preliminary Cost Estimate of Design Components Between Thomas Road and I-10.

ITEM	COST OF RIPRAP PROTECTION FOR LEVEES AND TWO REINFORCED CONCRETE DROP STRUCTURES	COST OF SOIL CEMENT PROTECTION FOR LEVEES AND TWO SOIL CEMENT DROP STRUCTURES
Channelization	$1,680,300	$1,680,300
Levees (riprap protection)	2,958,365	-0-
Levees (soil cement protection)	-0-	2,974,350
Drop Structures (reinforced concrete)	1,817,820	-0-
Drop Structures (soil cement)	-0-	942,080
I-10 Collector Channel	108,000	108,000
Transmission Tower Protection	800,000	800,000
Land Acquisition	1,400,000	1,400,000
SUBTOTAL	$8,764,485	$7,904,730
10% contingencies and construction supervision	876,450	790,470
TOTAL	$9,640,935	$8,695,200

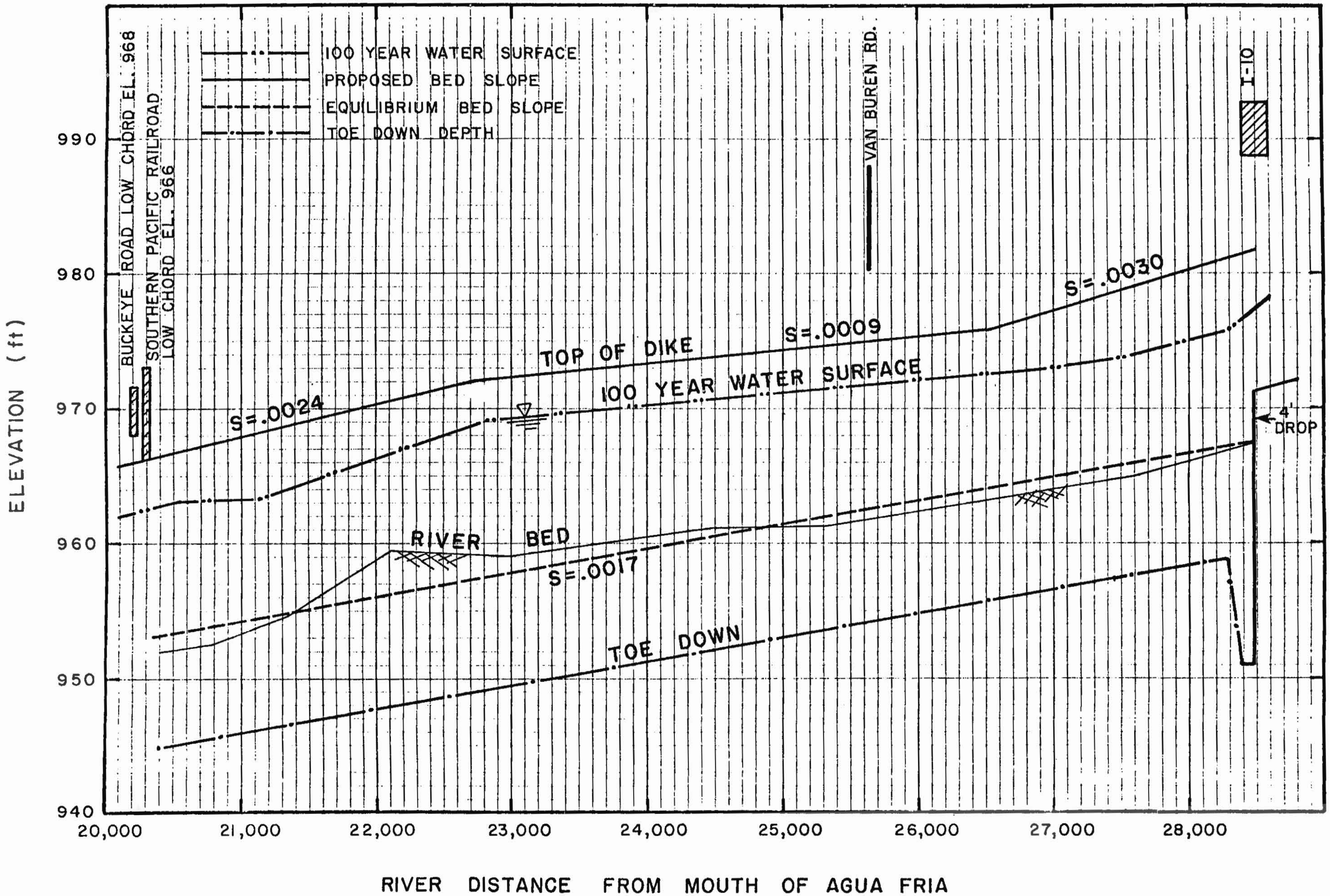


Figure 8.18. Proposed bed profile, equilibrium bed profile, 100-year water surface, levee heights and toe down depths between Buckeye Road and I-10.

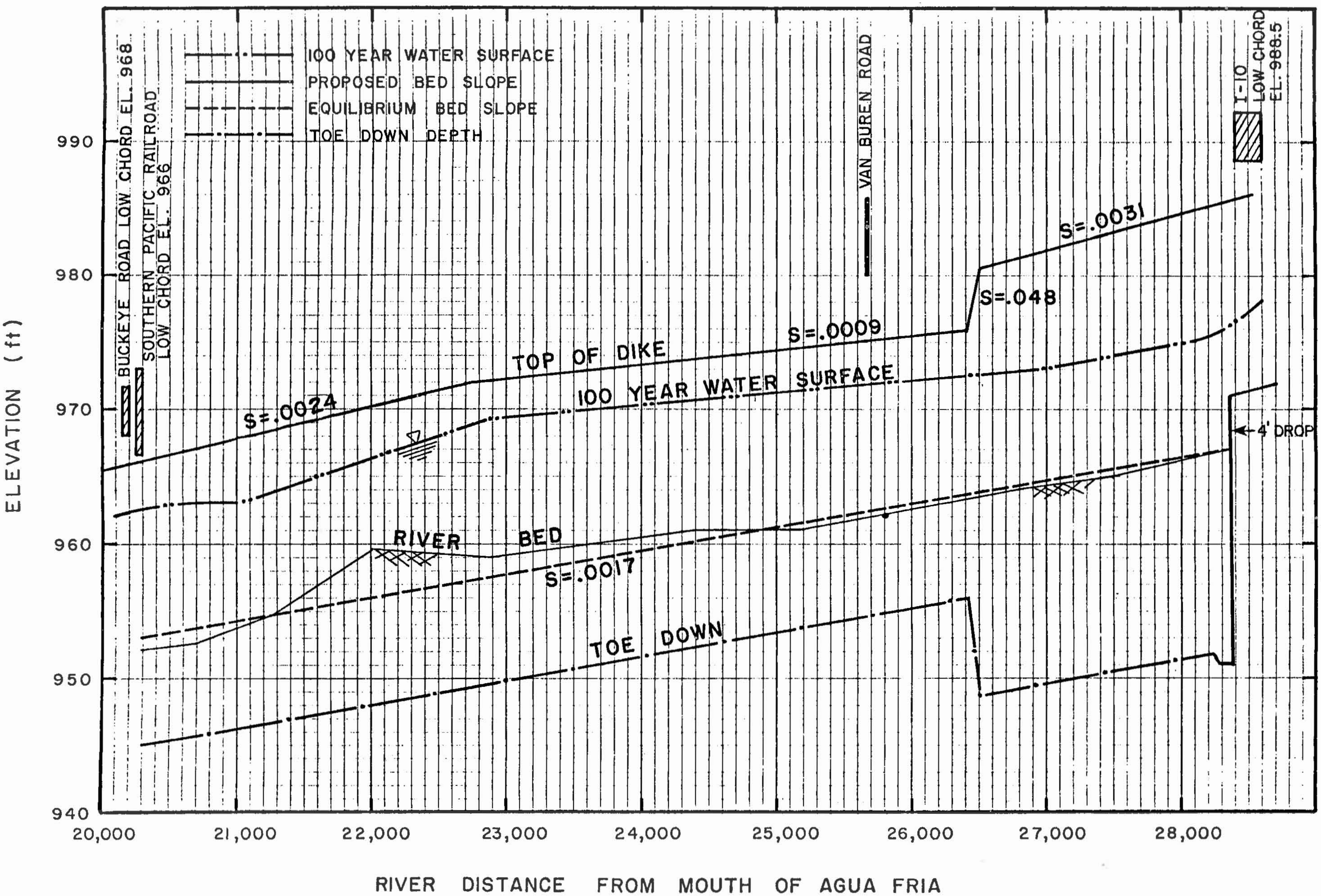


Figure 8.19. Proposed bed profile, equilibrium bed profile, 100-year water surface, levee heights and toe down depths between Buckeye Road and I-10 for west bank.

equilibrium bed slope on the west bank from I-10 to 2,000 feet downstream of the bridge. The levees will have 3:1 riverside slopes if protected with riprap and 1:1 riverside slopes if protected with soil cement. The outside bend of the west bank should be protected with soil cement near I-10. Landside slopes of 3:1 to natural ground are recommended. Crown widths of the levees will be 15 feet.

If the levees are protected with riprap, a 2-foot thickness will provide adequate protection against expected flow velocities in this reach except on the outside bend near I-10.

8.6.1.3 Drop Structure

One 4-foot drop structure located downstream of the I-10 bridge is recommended to control the grade near the bridge. The top of the drop structure will be at elevation 971 and the width of the drop will be approximately 1,410 feet.

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.

8.6.1.4 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 14.1 feet at the 100-year peak discharge of 95,000 cfs and the general bed response near the bridge is slight degradation. For the pre-project conditions the local scour depth for the 100-year discharge is 12 feet and the general bed response is slight degradation. Thus the channelization slightly increases the scour potential at the bridge.

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-II and a larger local scour potential.

Further compounding the problem in this area is the deposition of sediment along the east bank of the Agua Fria River from the I-10 collector channel. Should the deposition at the river-collector channel juncture become significant, the flow may become further entrenched in the west section under the bridge. Therefore, protection of the I-10 bridge piers is recommended.

A riprap blanket extending 20 feet in all directions around the piers is suggested. The thickness of the blanket is 2.5 feet and the top of the blanket should be at elevation 971. A 1-foot gravel filter blanket beneath the riprap meeting ADOT standards is necessary.

8.6.1.5 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.

8.6.1.6 Cost Estimate Between I-10 and Buckeye Road

Table 8.7 compares the costs of protecting the levees, drop structure and I-10 bridge piers with riprap except around the outside bend on the west bank near I-10 which will be protected with soil cement, and constructing a reinforced concrete drop structure downstream of I-10 against the costs of protecting the levees with soil cement, protecting the drop structure and I-10 bridge piers with riprap and constructing a soil cement drop structure downstream of I-10. Tables A.12 and A.13 in Appendix A summarize the quantities, unit costs and total cost for both cases mentioned above for the reach between I-10 and Buckeye Road.

8.7 Summary of Costs

To summarize the costs of channelization in the Agua Fria between Camelback and Buckeye Roads, the river was divided into the following reaches:

1. Camelback Road to Indian School Road
2. Indian School Road to Thomas Road
3. Thomas Road to I-10
4. I-10 to Buckeye

Table 8.7. Preliminary Cost Estimates of Design Components Between I-10 and Buckeye Road.

ITEM	COST OF RIPRAP PROTECTION FOR LEVEES AND A REINFORCED CONCRETE DROP STRUCTURE	COST OF SOIL CEMENT PROTECTION FOR LEVEES AND A SOIL CEMENT DROP STRUCTURE
Channelization	$ 243,000	$ 243,000
Levees (riprap protection)	2,596,375	-0-
Levees (soil cement protection)	-0-	2,634,250
Drop Structure (reinforced concrete)	1,155,680	-0-
Drop Structure (soil cement)	-0-	597,405
Gravel Pit Restoration	212,500	212,500
I-10 Bridge Pier Protection (riprap blanket)	437,210	437,210
Transmission Tower Protection	700,000	700,000
Land Acquisition	1,500,000	1,500,000
SUBTOTAL	$6,844,765	$6,324,365
10% contingencies and construction supervision	684,480	632,440
TOTAL	$7,529,245	$6,956,805

Also considered in the costs were the channel modifications upstream of Camelback Road.

Table 8.8 summarizes costs of channelization between Camelback and Buckeye Roads considering Alternative 1 in Reach 1 for the following two cases: (1) riprap protection of all levees except around the west bend of the channel near I-10 bridge and construction of reinforced concrete drop structures, and (2) soil cement protection of all levees except in Reach 2, where riprap protection will be used and construction of soil cement drop structures. Included in the costs is an additional ten percent for contingencies.

Table 8.9 summarizes costs of channelization between Camelback and Buckeye Roads for Alternative 2 in Reach 1 for the two cases in Table 8.8. Table 8.10 summarizes costs of channelization between Camelback and Buckeye Roads for Alternative 3 in Reach 1 for the two cases in Table 8.8.

Table 8.8. Summary of Costs Per Reach for Channelization Between Camelback Road and Buckeye Road for Alternative 1 in Reach 1.

REACH NO.	APPROXIMATE COSTS CASE I	APPROXIMATE COSTS CASE II
Upstream of Camelback	$ 6,150	$ 6,150
1	5,490,005[1]	4,851,640[1]
2	5,982,485	5,573,285
3	9,640,935[2]	8,695,200[2]
4	7,529,245	6,956,805
TOTAL	$28,648,820	$26,083,080
Approximate Cost per Mile	5,729,800	5,216,600

CASE I - Riprap protection of all levees, except the west bend near I-10 which will be protected with soil cement, and construction of reinforced concrete drop structures.

CASE II - Soil cement protection of all levees, except in Reach 2 where riprap protection is required, and construction of soil cement drop structures.

[1]Does not include cost of 73 acres of flowage easement.

[2]Does not include cost of 20.6 acres of flowage easement.

Table 8.9. Summary of Costs Per Reach for Channelization Between Camelback Road and Buckeye Road for Alternative 2 in Reach 1.

REACH NO.	APPROXIMATE COSTS CASE I	APPROXIMATE COSTS CASE II
Upstream of Camelback	$ 1,008,375	$ 959,615
1	8,765,345	6,741,770
2	5,982,485	5,573,285
3	9,640,935[1]	8,695,200[1]
4	7,529,245	6,956,805
TOTAL	$32,926,385	$28,926,675
Approximate Cost per Mile	6,585,300	5,785,300

CASE I - Riprap protection of all levees, except the west bend near I-10 which will be protected with soil cement, and construction of reinforced concrete drop structures.

CASE II - Soil cement protection of all levees, except in Reach 2 where riprap protection is required, and construction of soil cement drop structures.

[1]Does not include cost of 20.6 acres of flowage easement.

Table 8.10. Summary of Costs Per Reach for Channelization Between Camelback Road and Buckeye Road for Alternative 3 in Reach 1.

REACH NO.	APPROXIMATE COSTS CASE I	APPROXIMATE COSTS CASE II
Upstream of Camelback	$ 6,150	$ 6,150
1	2,667,370[1]	2,852,760[1]
2	5,982,485	5,573,285
3	9,640,935[2]	8,695,200[2]
4	7,529,245	6,956,805
TOTAL	$25,826,185	$24,084,200
Approximate Cost per Mile	5,165,200	4,816,800

CASE I - Riprap protection of all levees, except the west bend near I-10 which will be protected with soil cement, and construction of reinforced concrete drop structures.

CASE II - Soil cement protection of all levees, except in Reach 2 where riprap protection is required, and construction of soil cement drop structures.

[1]Does not include cost of 590 acres of flowage easement.

[2]Does not include cost of 20.6 acres of flowage easement.

IX. CONCLUSIONS AND RECOMMENDATIONS

The following are conclusions regarding the three levels of analysis conducted for channelized conditions 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.

2. The 100-year flood plain reduces significantly for the channelized condition from existing conditions.

3. Flow depths for the 100-year peak discharge are generally lower for the channelized condition than for the existing condition because the effective flow width for the main channel has been increased.

4. The long-term bed response of the Agua Fria River considering channelization between Camelback and Buckeye Roads is degradation.

5. Local scour analyses at existing and proposed bridge crossings in the study reach of the Agua Fria indicates protection of bridge piers should be implemented at Indian School Road, RID flume and I-10.

6. 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.

7. Computer modeling of the channel between the New River and Buckeye Road, using the equilibrium bed profile, indicates minimal aggradation/degradation response to the 100-year flood event. The equilibrium bed slope is flatter than the existing bed slope throughout most of the study reach.

8. Low chord elevations at all crossings in the Agua Fria are adequate to pass the 100-year discharge. However, 4-foot freeboard heights, as required by the Corps of Engineers, are not satisfied at Camelback Road and the Southern Pacific Railroad crossing where freeboard heights are 2.8 and 3.7 feet, respectively.

The following are recommendations regarding channelization between Camelback Road and Buckeye Road. Protection measures, levee heights, low chord elevations of bridges, etc., are based on the design flood with a 100-year return interval.

1. Upstream of Camelback Road a side drainage channel on the west should be provided to direct overbank flow from the west braid into the main channel if continuous levees are constructed on both the east and west banks between Camelback and ISRB. Channelization extending 1,500 feet upstream of Camelback Road will encourage more flow to enter the main channel. Should a partial levee be constructed 1,650 feet upstream of ISRB on the west bank, terminating in a transverse dike, the side drainage channel is not necessary.

2. Between Camelback Road and Indian School Road three alternatives are proposed. Alternative 1 has the following components: channelization, levee on east side and 1,650 feet on the west side, 800-foot transverse dike, one drop structure located 1,000 feet upstream of ISRB, and riprap protection of shallow ISRB piers. Alternative 2 has the following components: channelization, continuous levee on both east and west banks, three drop structures located 1,000, 2,000 and 4,000 feet upstream of ISRB, and riprap protection of shallow ISRB piers. Alternative 3 has the following components: channelization, a 1,650 foot levee on the west bank just upstream of ISRB terminating in an 800 foot long transverse dike, two transverse dikes on the east bank 1,600 and 600 feet long, a spur dike just north of ISRB on the west bank and riprap protection of shallow piers at ISRB.

3. Between Indian School Road and Thomas Road the following components are being recommended: channelization, levees on east and west banks, backfilling of flood plain gravel pits, one drop structure located just downstream of the RID flume, riprap protection of RID flume piers, and protection of two transmission towers.

4. Between Thomas Road and I-10 the following components of channelization are recommended: channelization, levee construction on the east and west banks, two drop structures located 2,200 feet upstream of the proposed McDowell Road Bridge and 200 feet downstream of Thomas Road, protection of eight transmission towers, and protection of two pipeline crossings near Thomas Road. Three alternatives are proposed for the I-10 collector channel drainage and are: (1) leave a 200 foot opening between the spur dike north of I-10 on the east bank and the proposed levee downstream of McDowell Road, (2) extend the collector channel [illegible],900 feet into the Agua Fria with levees high enough to contain the 100-year flood from the collector channel and Agua Fria, or (3) provide a siltation basin between the I-10 collector channel outlet and the Agua Fria with a depressed section in the levee on the east bank to allow overflow of large discharges from the collector channel into the Agua Fria.

5. Between I-10 and Buckeye Road the following components are recommended: channelization, levee construction on the east and west banks, backfilling of instream gravel pits, one drop structure located downstream of I-10 bridge, riprap protection of I-10 bridge piers and protection of seven transmission towers.

APPENDIX A

QUANTITY AND COST ESTIMATES OF CHANNELIZATION BETWEEN CAMELBACK ROAD AND BUCKEYE ROAD

Table A.1. Estimates of Quantities and Costs of Side Drainage Channel and Channelization Upstream of Camelback Road for Rip-rap Protection.

ITEM	QUANTITY	UNIT COST	TOTAL COST
Drainage channel excavation	120,000 yd^3	\$ 0.90/yd^3	\$ 108,000
Levee fill for drainage channel	17,500 yd^3	1.25/yd^3	21,875
Rip-rap protection of levee for drainage channel	12,840 yd^3	22.00/yd^3	282,480
Filter fabric	12,900 yd^2	1.50/yd^2	19,350
Channel excavation upstream of Camelback Road	250,000 yd^3	0.90/yd^3	225,000
Approximate Right of Way Acquisition for side drainage channel in addition to PRC Toups Right of Way	52 acres	5,000/acre	260,000
			\$ 916,705
10% contingencies and construction supervision			\$ 91,670
			\$1,008,375

Table A.2. Estimates of Quantities and Costs of Side Drainage Channel and Channelization Upstream of Camelback Road for Soil Cement Protection.

ITEM	QUANTITY	UNIT COST	TOTAL COST
Drainage channel excavation	120,000 yd^3	\$ 0.90/yd^3	\$ 108,000
Levee fill for drainage channel	16,500 yd^3	1.25/yd^3	20,625
Soil cement protection of levee for drainage channel	11,500 yd^3	22.50/yd^3	258,750
Channel excavation upstream of Camelback Road	250,000 yd^3	0.90/yd^3	225,000
Approximate Right of Way Acquisition for side drainage channel in addition to PRC Toups Right of Way	52 acres	5,000/acre	260,000
			\$ 872,375
10% contingencies and construction supervision			\$ 87,240
			\$ 959,615

Table A.3. Preliminary Cost and Quantity Estimates of Alternative 1 Between Camelback Road and Indian School Road Considering Riprap Protection and a Reinforced Concrete Drop Structure.

Item	Channelization	Levees	Transverse Dike	Drop Structure	Pier Protection	Total	Unit Price	Total Cost
Common Fill		142,610 yd^3	4,500 yd^3	3,610 yd^3		150,720 yd^3	\$ 1.25/yd^3	\$ 188,400
Drainage Excavation	1,070,000 yd^3			4,220 yd^3	6,705 yd^3	1,080,925 yd^3	0.90/yd^3	972,830
Structural Excavation				15,630 yd^3		15,630 yd^3	2.00/yd^3	31,260
Special Backfill				8,840 yd^3		8,840 yd^3	2.00/yd^3	17,680
Structural Concrete (Class S)				3,290 yd^3		3,290 yd^3	300/yd^3	987,000
Reinforcing Steel				320,000 lb		320,000 lb	0.40/lb	128,000
Riprap		48,350 yd^3	2,200 yd^3	3,020 yd^3	4,790 yd^3	58,360 yd^3	22/yd^3	1,283,920
Gravel Filter Material				1,200 yd^3	1,915 yd^3	3,115 yd^3	15/yd^3	46,725
Filter Fabric (includes 6" soil cover)		67,800 yd^2	5,600 yd^2			73,400 yd^2	1.50/yd^2	110,100
Soil Cement						---	22.50/yd^3	---
Land Acquisition	240 acres		5 acres			245 acres	5,000/acre	1,225,000
Flowage Easement	73 acres					73 acres	---	---
Sludge Removal						---	2.00/yd^3	---
Trash Removal						---	0.50/yd^3	---
Transmission Tower Protection						---	100,000/tower	---
								\$4,990,915
					10% contingencies and construction supervision			499,090
							Total	\$5,490,005

Note: Excess excavation material will be used to fill abandoned gravel pits downstream of Indian School Road.

Table A.4. Preliminary Cost and Quantity Estimates of Alternative 1 Between Camelback Road and Indian School Road Considering Soil Cement Protection and a Soil Cement Drop Structure.

Item	Channelization	Levees	Transverse Dike	Drop Structure	Pier Protection	Total	Unit Price	Total Cost
Common Fill		110,400 yd^3	4,000 yd^3	13,580 yd^3		127,980 yd^3	\$ 1.25/yd^3	\$ 159,975
Drainage Excavation	1,070,000 yd^3			4,220 yd^3	6,705 yd^3	1,080,925 yd^3	0.90/yd^3	972,830
Structural Excavation				38,630 yd^3		38,630 yd^3	2.00/yd^3	77,260
Special Backfill				4,530 yd^3		4,530 yd^3	2.00/yd^3	9,060
Structural Concrete (Class S)						---	300/yd^3	---
Reinforcing Steel						---	0.40/lb	---
Riprap				3,020 yd^3	4,790 yd^3	7,810 yd^3	22/yd^3	171,820
Gravel Filter Material				1,200 yd^3	1,915 yd^3	3,115 yd^3	15/yd^3	46,725
Filter Fabric (includes 6" soil cover)						---	1.50/yd^3	
Soil Cement		53,090 yd^3	4,075 yd^3	20,520 yd^3		77,685 yd^3	22.50/yd^3	1,747,910
Land Acquisition	240 acres		5 acres			245 acres	5,000/acre	1,225,000
Flowage Easement	73 acres					73 acres	---	---
Sludge Removal						---	2.00/yd^3	---
Trash Removal						---	0.50/yd^3	---
Transmission Tower Protection						---	100,000/tower	---
								\$4,410,580
					10% contingencies and construction supervision			441,060
							Total	\$4,851,640

Table A.5. Preliminary Cost and Quantity Estimates of Alternative 2 Between Camelback Road and Indian School Road Considering Riprap Protection and Three Reinforced Concrete Drop Structures.

Item	Channelization	Levees	Drop Structure	Pier Protection	Total	Unit Price	Total Cost
Common Fill		202,800 yd^3	10,830 yd^3		213,630 yd^3	\$ 1.25/yd^3	\$ 267,040
Drainage Excavation	1,070,000 yd^3		12,660 yd^3	6,705 yd^3	1,089,365 yd^3	0.90/yd^3	980,430
Structural Excavation			46,890 yd^3		46,890 yd^3	2.00/yd^3	93,780
Special Backfill			26,520 yd^3		26,520 yd^3	2.00/yd^3	53,040
Structural Concrete (Class S)			9,870 yd^3		9,870 yd^3	300/yd^3	2,961,000
Reinforcing Steel			960,000 lb		960,000 lb	0.40/lb	384,000
Riprap		68,940 yd^3	9,060 yd^3	4,790 yd^3	82,790 yd^3	22/yd^3	1,821,380
Gravel Filter Material			3,600 yd^3	1,915 yd^3	5,515 yd^3	15/yd^3	82,725
Filter Fabric (Includes 6" soil cover)		83,400 yd^2			83,400 yd^2	1.50/yd^2	125,100
Soil Cement					---	22.50/yd^3	---
Land Acquisition	240 acres				240 acres	5,000/acre	1,200,000
Flowage Easement					---	---	---
Sludge Removal					---	2.00/yd^3	---
Trash Removal					---	0.50/yd^3	---
Transmission Tower Protection					---	100,000/tower	---
							\$ 7,968,495
				10% contingencies and construction supervision			796,850
						Total	\$ 8,765,345

Table A.6. Preliminary Cost and Quantity Estimates of Alternative 2 Between Camelback Road and Indian School Road Considering Soil Cement Protection of Levees and Three Soil Cement Drop Structures.

Item	Channelization	Levees	Drop Structure	Pier Protection	Total	Unit Price	Total Cost
Common Fill		156,330 yd^3	40,740 yd^3		197,070 yd^3	\$ 1.25/yd^3	\$ 246,340
Drainage Excavation	1,070,000 yd^3		12,660 yd^3	6,705 yd^3	1,089,365 yd^3	0.90/yd^3	980,430
Structural Excavation			115,890 yd^3		115,890 yd^3	2.00/yd^3	231,780
Special Backfill			13,590 yd^3		13,590 yd^3	2.00/yd^3	27,180
Structural Concrete (Class S)					---	300/yd^3	---
Reinforcing Steel					---	0.37/lb	---
Riprap			9,060 yd^3	4,790 yd^3	13,850 yd^3	22/yd^3	304,700
Gravel Filter Material			3,600 yd^3	1,915 yd^3	5,515 yd^3	15/yd^3	82,725
Filter Fabric (includes 6" soil cover)					---	1.50/yd^2	---
Soil Cement		74,250 yd^3	61,560 yd^3		135,810 yd^3	22.50/yd^3	3,055,725
Land Acquisition	240 acres				240 acres	5,000/acre	1,200,000
Flowage Easement					---	---	---
Sludge Removal					---	2.00/yd^3	---
Trash Removal					---	0.50/yd^3	---
Transmission Tower Protection					---	100,000/tower	---
							\$ 6,128,880
				10% contingencies and construction supervision			612,890
						Total	\$ 6,741,770

Table A.7. Preliminary Cost and Quantity Estimates of Alternative 3 Between Camelback Road and Indian School Road Considering Riprap Protection.

Item	Channelization	Levees	Transverse Dike	Spur Dike	Pier Protection	Total	Unit Price	Total Cost
Common Fill		19,000 yd^3	51,000 yd^3	10,100 yd^3		80,300 yd^3	\$ 1.25/yd^3	\$ 100,375
Drainage Excavation	601,350 yd^3				6,705 yd^3	608,055 yd^3	0.90/yd^3	547,250
Structural Excavation						---	2.00/yd^3	---
Special Backfill						---	2.00/yd^3	---
Structural Concrete (Class S)						---	300/yd^3	---
Reinforcing Steel						---	0.40/lb	---
Riprap		10,500 yd^3	22,830 yd^3	9,120 yd^3	4,790 yd^3	47,240 yd^3	22/yd^3	1,039,280
Gravel Filter Material					1,915 yd^3	1,915 yd^3	15/yd^3	28,725
Filter Fabric (includes 6" soil cover)		28,700 yd^2	45,000 yd^2	5,800 yd^2		79,500 yd^2	1.50/yd^3	119,250
Soil Cement						---	22.50/yd^3	---
Land Acquisition	101 acres		15 acres	2 acres		118 acres	5,000/acre	590,000
Flowage Easement	590 acres					590 acres	---	---
Sludge Removal						---	2.00/yd^3	---
Trash Removal						---	0.50/yd^3	---
Transmission Tower Protection						---	100,000/tower	---
								\$2,424,880
					10% contingencies and construction supervision			242,490
							Total	\$2,667,370

Table A.8. Preliminary Cost and Quantity Estimates of Alternative 3 Between Camelback Road and Indian School Road Considering Soil Cement Protection.

Item	Channelization	Levees	Transverse Dike	Spur Dike	Pier Protection	Total	Unit Price	Total Cost
Common Fill		13,600 yd^3	12,000 yd^3	6,500 yd^3		32,100 yd^3	\$ 1.25/yd^3	\$ 40,125
Drainage Excavation	601,350 yd^3				6,705 yd^3	608,055 yd^3	0.90/yd^3	547,250
Structural Excavation						---	2.00/yd^3	---
Special Backfill						---	2.00/yd^3	---
Structural Concrete (Class S)						---	300/yd^3	---
Reinforcing Steel						---	0.40/lb	---
Riprap					4,790 yd^3	4,790 yd^3	22/yd^3	105,380
Gravel Filter Material					1,915 yd^3	1,915 yd^3	15/yd^3	28,725
Filter Fabric (includes 6" soil cover)						---	1.50/yd^3	---
Soil Cement		11,550 yd^3	33,850 yd^3	11,575 yd^3		56,975 yd^3	22.50/yd^3	1,281,940
Land Acquisition	101 acres		15 acres	2 acres		118 acres	5,000/acre	590,000
Flowage Easement						590 acres	---	---
Sludge Removal						---	2.00/yd^3	---
Trash Removal						---	0.50/yd^3	---
Transmission Tower Protection						---	100,000/tower	---
								\$2,593,420
					10% contingencies and construction supervision			259,340
							Total	\$2,852,760

Table A.9. Preliminary Cost and Quantity Estimates Between ISRB and Thomas Road Considering Riprap Protection of Levees and a Soil Cement Drop Structure.

Item	Channelization	Levees	Gravel Pits	Drop Structure	Riprap Blanket RID Flume	Total	Unit Price	Total Cost
Common Fill		173,500 yd^3	880,000 yd^3	8,330 yd^3		1,061,830 yd^3	\$ 1.25/yd^3	\$1,327,290
Drainage Excavation	708,260 yd^3			2,590 yd^3	4,315 yd^3	715,165 yd^3	0.90/yd^3	643,650
Structural Excavation				23,700 yd^3		23,700 yd^3	2.00/yd^3	47,400
Special Backfill				2,780 yd^3		2,780 yd^3	2.00/yd^3	5,560
Structural Concrete (Class S)						---	300/yd^3	---
Reinforcing Steel						---	0.40/lb	---
Riprap		63,670 yd^3		1,850 yd^3	3,080 yd^3	68,600 yd^3	22/yd^3	1,509,200
Gravel Filter Material				740 yd^3	1,235 yd^3	1,975 yd^3	15/yd^3	29,625
Filter Fabric (includes 6" soil cover)		83,750 yd^2				83,750 yd^2	1.50/yd^2	125,625
Soil Cement				12,590 yd^3		12,590 yd^3	22.50/yd^3	283,275
Flowage Easement						---	---	---
Land Acquisition	154 acres					154 acres	5,000/acre	770,000
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
Transmission Tower Protection						2 towers	100,000/tower	200,000
								\$5,066,625
					10% contingencies and construction supervision			506,660
							Total	\$5,573,285

Note: Replacing soil cement drop structure with reinforced concrete drop structure brings final cost to \$5,982,485.

Table A.10. Preliminary Cost and Quantity Estimates for Reach Between Thomas Road and I-10 Considering Riprap Protection of Levees and Reinforced Concrete Drop Structures.

Item	Channelization	Levees	I-10 Collector Channel	Drop Structure	Total	Unit Price	Total Cost
Common Fill		200,800 yd^3		8,480 yd^3	209,280 yd^3	\$ 1.25/yd^3	\$ 261,600
Drainage Excavation	1,867,000 yd^3*			6,080 yd^3	1,873,080 yd^3	0.90/yd^3	1,685,770
Structural Excavation				22,540 yd^3	22,540 yd^3	2.00/yd^3	45,080
Special Backfill				12,750 yd^3	12,750 yd^3	2.00/yd^3	25,500
Structural Concrete (Class S)				4,750 yd^3	4,750 yd^3	300/yd^3	1,425,000
Reinforcing Steel				461,300 lb	461,300 lb	0.40/lb	184,520
Riprap		101,590 yd^3		4,350 yd^3	105,940 yd^3	22/yd^3	2,330,680
Gravel Filter Material				1,730 yd^3	1,730 yd^3	15/yd^3	25,950
Filter Fabric (Includes 6" soil cover)		124,875 yd^2			124,875 yd^2	1.50/yd^2	187,310
Soil Cement		12,670 yd^3	4,800 yd^3		17,470 yd^3	22.50/yd^3	393,075
Land Acquisition	280 acres				280 acres	5,000/acre	1,400,000
Flowage Easement			20.6 acres		20.6 acres	---	---
Sludge Removal					---	2.00/yd^3	---
Trash Removal					---	0.50/yd^3	---
Transmission Tower Protection					8 towers	100,000/tower	800,000
							\$ 8,764,485
				10% contingencies and construction supervision			876,450
						Total	\$ 9,640,935

*This does not take into account 850,000 yd^3 removed by I-10 contractor.

Table A.11. Preliminary Cost and Quantity Estimates Between Thomas Road and I-10 Considering Soil Cement Protection of Levees and Soil Cement Drop Structures.

Item	Channelization	Levees	I-10 Collector Channel	Drop Structure	Total	Unit Price	Total Cost
Common Fill		150,000 yd^3		19,600 yd^3	169,600 yd^3	$ 1.25/yd^3	$ 212,000
Drainage Excavation	1,867,000 yd^3*			6,080 yd^3	1,873,080 yd^3	0.90/yd^3	1,685,770
Structural Excavation				55,700 yd^3	55,700 yd^3	2.00/yd^3	111,400
Special Backfill				6,530 yd^3	6,530 yd^3	2.00/yd^3	13,060
Structural Concrete (Class S)					---	300/yd^3	---
Reinforcing Steel					---	0.40/lb	---
Riprap				4,350 yd^3	4,350 yd^3	22/yd^3	95,700
Gravel Filter Material				1,730 yd^3	1,730 yd^3	15/yd^3	25,950
Soil Cement		123,860 yd^3	4,800 yd^3	29,600 yd^3	158,260 yd^3	22.50/yd^3	3,560,850
Land Acquisition	280 acres				280 acres	5,000/acre	1,400,000
Flowage Easement			20.6 acres		20.6 acres	---	---
Sludge Removal					---	2.00/yd^3	---
Trash Removal					---	0.50/yd^3	---
Transmission Tower Protection					8 towers	100,000/tower	800,000
							$ 7,904,730
				10% contingencies and construction supervision			790,470
						Total	$ 8,695,200

*This does not take into account 850,000 yd^3 removed by I-10 contractor.

Table A.12. Preliminary Cost and Quantity Estimates Between I-10 and Buckeye Road Considering Riprap Protection of Levees and One Reinforced Concrete Drop Structure.

Item	Channelization	Levees	Drop Structure	Pier Protection	Gravel Pit Filling	Total	Unit Price	Total Cost
Common Fill		106,300 yd^3	5,400 yd^3		170,000 yd^3	281,700 yd^3	$ 1.25/yd^3	$ 352,125
Drainage Excavation	270,000 yd^3		3,700 yd^3	20,920 yd^3		294,620 yd^3	0.90/yd^3	265,160
Structural Excavation			14,400 yd^3			14,400 yd^3	2.00/yd^3	28,800
Special Backfill			8,150 yd^3			8,150 yd^3	2.00/yd^3	16,300
Structural Concrete (Class S)			3,030 yd^3			3,030 yd^3	300/yd^3	909,000
Reinforcing Steel			294,500 lb			294,500 lb	0.40/lb	117,800
Riprap		86,540 yd^3	2,600 yd^3	14,940 yd^3		104,080 yd^3	22/yd^3	2,289,760
Gravel Filter Material			1,100 yd^3	5,980 yd^3		7,080 yd^3	15/yd^3	106,200
Filter Fabric (includes 6" soil cover)		105,480 yd^2				105,480 yd^2	1.50/yd^2	158,220
Soil Cement		17,840 yd^3				17,840 yd^3	22.50/yd^3	401,400
Land Acquisition	300 acres					300 acres	5,000/acre	1,500,000
Sludge Removal						---	2.00/yd^3	---
Trash Removal						---	0.50/yd^3	---
Transmission Tower Protection						7 towers	100,000/tower	700,000
								$ 6,844,765
					10% contingencies and construction supervision			684,480
							Total	$ 7,529,245

Table A.13. Preliminary Cost and Quantity Estimates Between I-10 and Buckeye Road Considering Soil Cement Protection of Levees and One Soil Cement Drop Structure.

Item	Channelization	Levees	Drop Structure	Pier Protection	Gravel Pit Filling	Total	Unit Price	Total Cost
Common Fill		95,000 yd^3	12,500 yd^3		170,000 yd^3	277,500 yd^3	$ 1.25/yd^3	$ 346,875
Drainage Excavation	270,000 yd^3		3,700 yd^3	20,920 yd^3		294,620 yd^3	0.90/yd^3	265,160
Structural Excavation			35,550 yd^3			35,550 yd^3	2.00/yd^3	71,100
Special Backfill			4,200 yd^3			4,200 yd^3	2.00/yd^3	8,400
Structural Concrete (Class S)						---	300/yd^3	---
Reinforcing Steel						---	0.40/lb	---
Riprap			2,600 yd^3	14,940 yd^3		17,540 yd^3	22/yd^3	385,880
Gravel Filter Material			1,100 yd^3	5,980 yd^3		7,080 yd^3	15/yd^3	106,200
Soil Cement		111,800 yd^3	18,900 yd^3			130,700 yd^3	22.50/yd^3	2,940,750
Land Acquisition	300 acres					300 acres	5,000/acre	1,500,000
Sludge Removal						---	2.00/yd^3	---
Trash Removal						---	0.50/yd^3	---
Transmission Tower Protection						7 towers	100,000/tower	700,000
								$ 6,324,365
					10% contingencies and construction supervision			632,440
							Total	$ 6,956,805

APPENDIX B

ANALYSIS ASSOCIATED WITH I-10 COLLECTOR CHANNEL

B.1 General

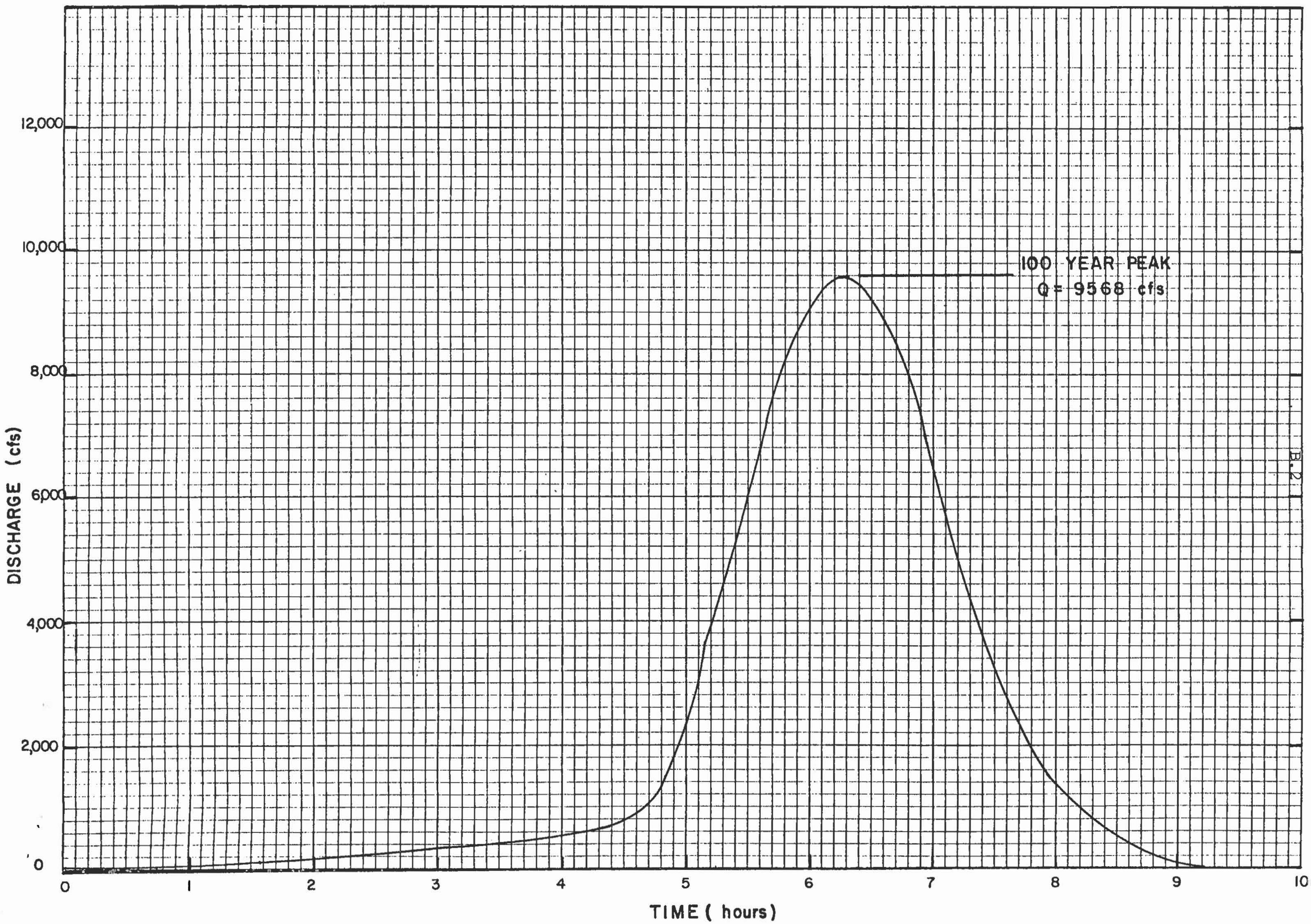
The I-10 collector channel is located immediately north of and parallel to the I-10 freeway. The channel will deliver flood flows from 27th Avenue to the Agua Fria River, draining an urbanized area of about 45 square miles. The 50- and 100-year flood peaks from the channel are 9,296 cfs and 9,568 cfs, respectively.

The duration and shape of the 100-year hydrograph was constructed from the hydrographs of the nearby Cave Creek watershed. The 100-year hydrograph is shown in Figure B.1. The volume of the 100-year flood is 1,710 acre-feet and the duration is approximately 9 hours.

The annual sediment yield for the nearby Cudia City Wash, as documented in "Sediment Data Report for Arizona Canal Diversion Channel" by Boyle Engineering, November 1981, was 0.30 acre-feet per square mile. This value was adopted for the I-10 collector drainage area, resulting in an average annual sediment yield of 13.5 acre-feet per year. Three alternatives are being considered for I-10 collector channel drainage.

Alternative 1 would leave the collector channel and pilot channel as they presently exist. The east bank levee from McDowell Road would be extended to the pilot channel, and the spur dike just north of I-10 bridge would be left intact. Some grading behind the spur dike will be necessary to prevent ponding of water behind the dike. A 200-foot opening will remain between the levee and spur dike.

As stated in Section 8.5.1.4, approximately 20.6 acres of property outside the I-10 right of way will be inundated when the 100-year flood occurs in the I-10 collector channel. The 20.6 acres was determined by computing a backwater profile, using the Army Corps of Engineers HEC-II program, from the point where the collector channel enters the pilot channel to where the pilot channel empties into the Agua Fria. The water-surface elevation at the point where the pilot channel enters the Agua Fria is 982 feet, which is greater than the 100-year water surface elevation in the Agua Fria at the opening, which is 980 feet. Thus the 100-year flood in the I-10 collector channel will inundate more land between McDowell Road and I-10 in the east overbank of the Agua Fria than the 100-year flood in the Agua Fria.



B.2

Figure B.1. 100 year hydrograph for I-10 collector channel.

The local scour depth computed at the nose of the east spur dike is 13 feet for the 100-year discharge of 95,000 cfs using Liu's embankment scour methodology. The present toe-down depth of the spur dike is 15 feet. It is recommended that the toe-down be extended 5 feet further, however, this is not absolutely necessary.

Alternative 2 considers extending the I-10 collector channel 2,900 feet to the Agua Fria. The channel will be trapezoidal in shape with a bottom width of 165 feet and 2:1 side slopes. The channel has an earth bottom and soil cement protection of banks.

A backwater profile, using HEC-II, was executed with a bed slope of 0.0012 to determine hydraulic characteristics of the channel. The average depth in the channel is 7.5 feet and the average flow velocity is 6.5 fps for the 100-year discharge of 9,568 cfs. The banks extend 10.5 feet above the channel invert and the toe down depth averages 6.5 feet below the channel invert. The banks are high enough to contain the 100-year flood of the Agua Fria.

Alternative 3 considers a siltation basin between the collector channel outlet and the Agua Fria. Sizing a basin to contain the 100-year flood volume from the I-10 collector channel and store 50 years worth of sediment would require a storage area of 2,385 acre-feet. With the present right of way available in this area this is physically not possible without an exceptionally deep basin. A more realistic basin storage capacity of 300 acre-feet is available. This size of pond will handle most of the nuisance flows.

In the "Summary Sedimentation Study Report Arizona Canal Diversion Channel" prepared for the Army Corps of Engineers, Los Angeles District by SLA and Boyle Engineering, obtaining the 50-, 25-, 10-, 5- and 2-year flood peaks from the Cudia City Wash and Cave Creek Watersheds the 100-year flood peak was multiplied by the following ratios .711, .467, .266, .155 and .0662. If these numbers are assumed to be valid for the I-10 collector channel drainage the 300 acre-feet siltation basin can store a volume of water between the 5-year and 10-year return floods before spillage occurs over the depressed section of the levee.

Inherent with this alternative will be periodic removal of sediment from the siltation basin. It is suggested that the basin be inspected after large storms to assess maintenance work required.

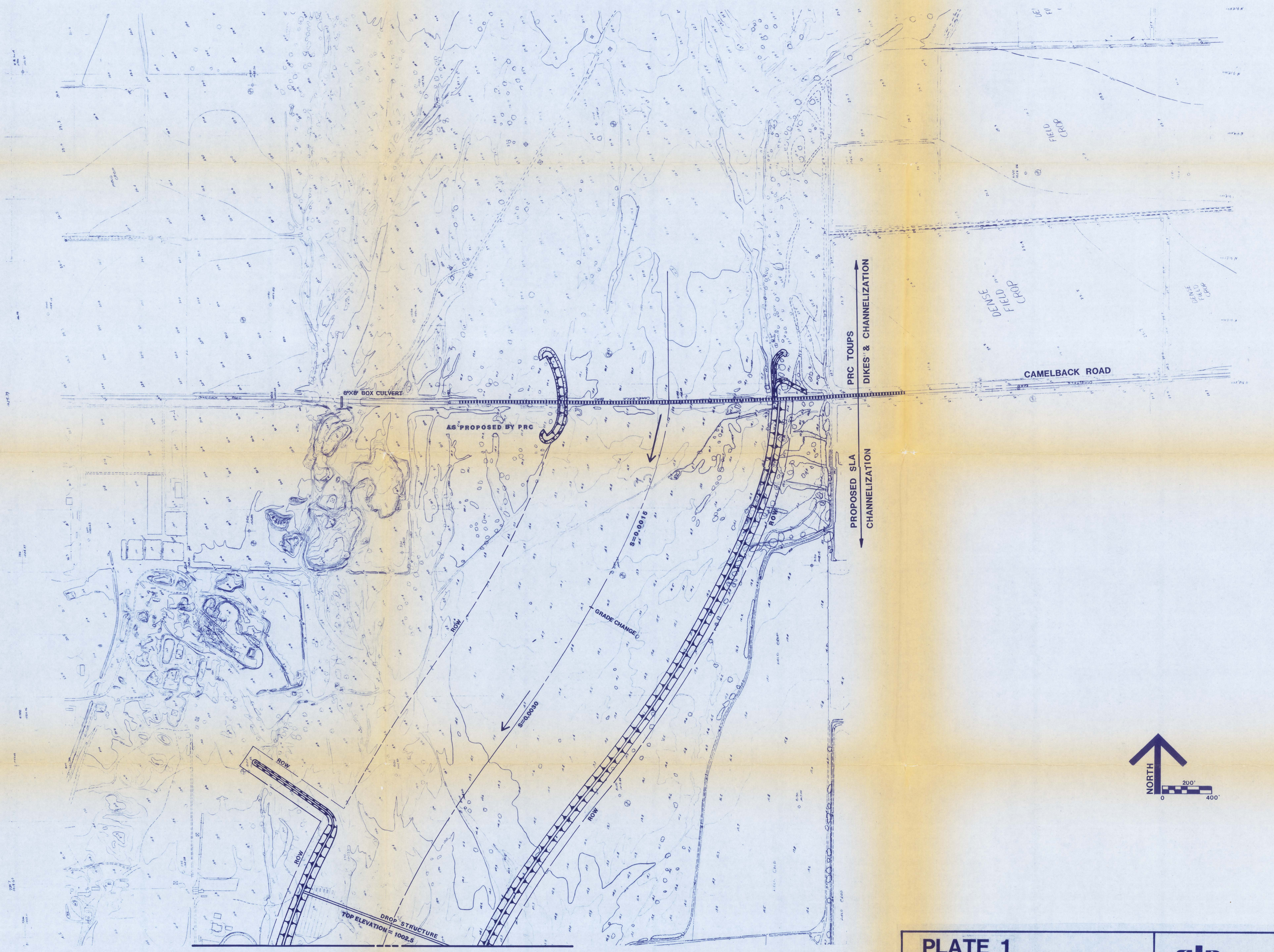


PLATE 1

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

PLATE 2A

PROPOSED CHANNELIZATION
OF THE AGUA FRIA RIVER

No.	Revision	Date	By

sla
SIMONS, LI & ASSOCIATES, INC.

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

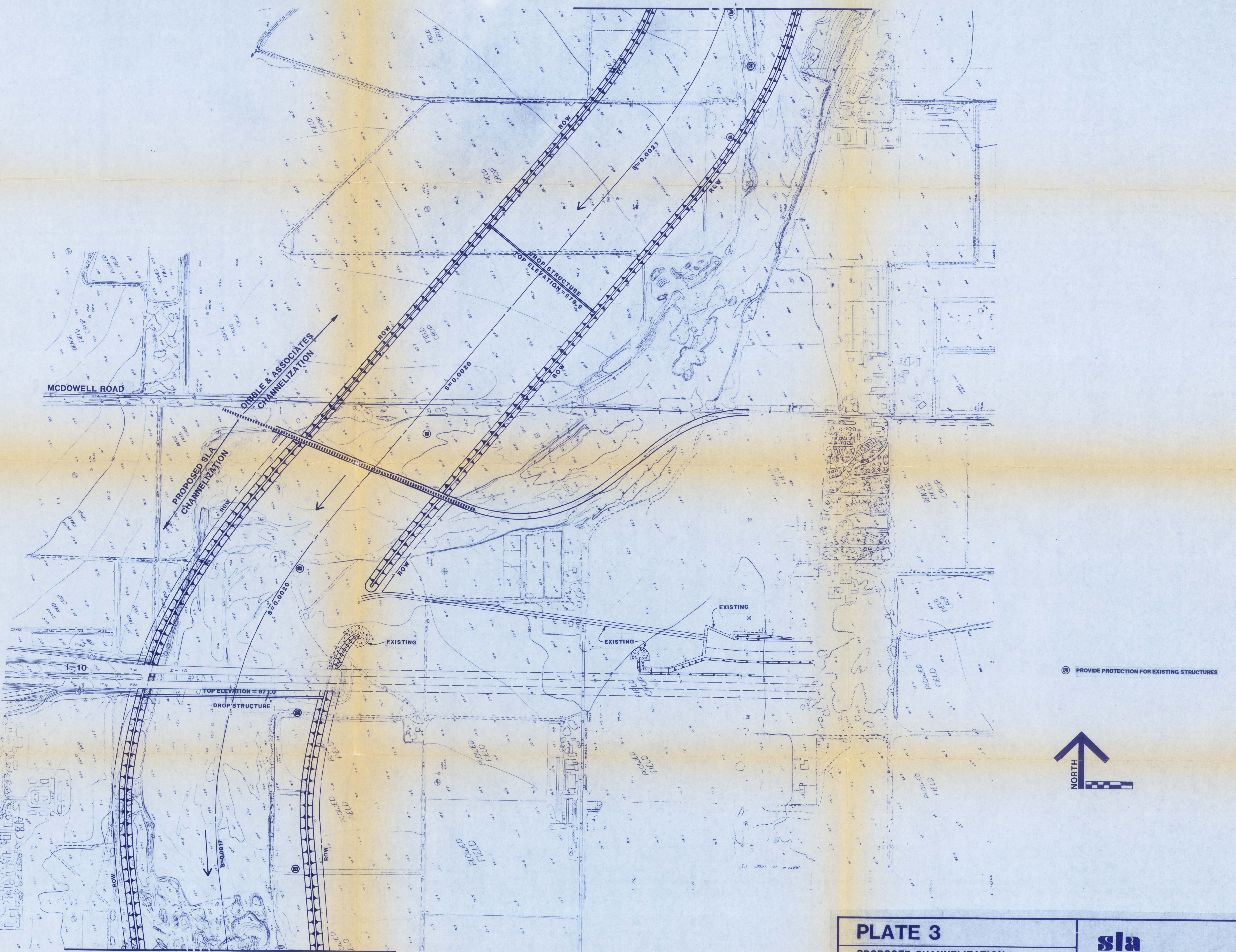


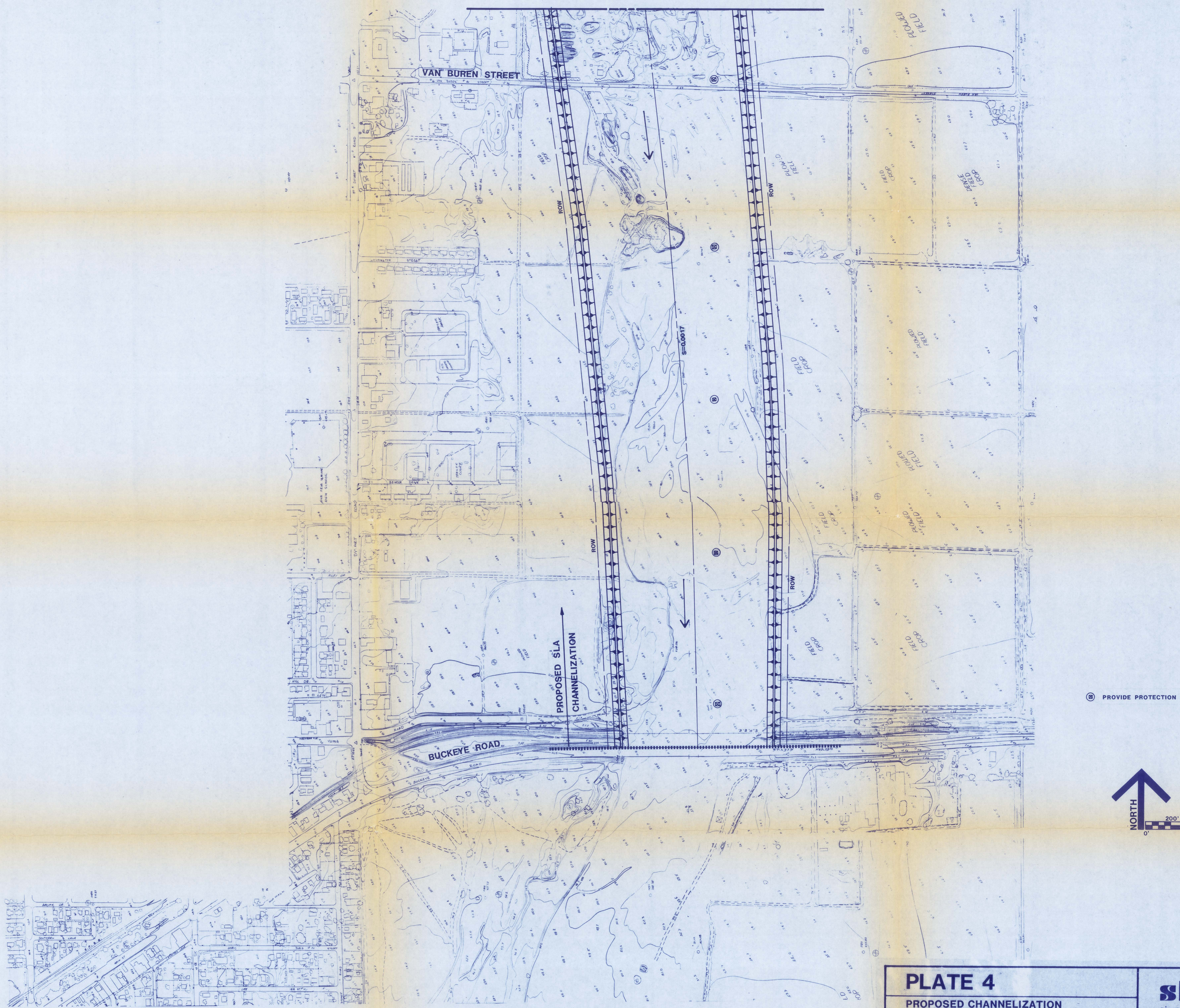
PLATE 3

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



Ⓢ PROVIDE PROTECTION FOR EXISTING STRUCTURES

PLATE 4

PROPOSED CHANNELIZATION OF THE AGUA FRIA RIVER

No.	Revision	Date	By

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

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

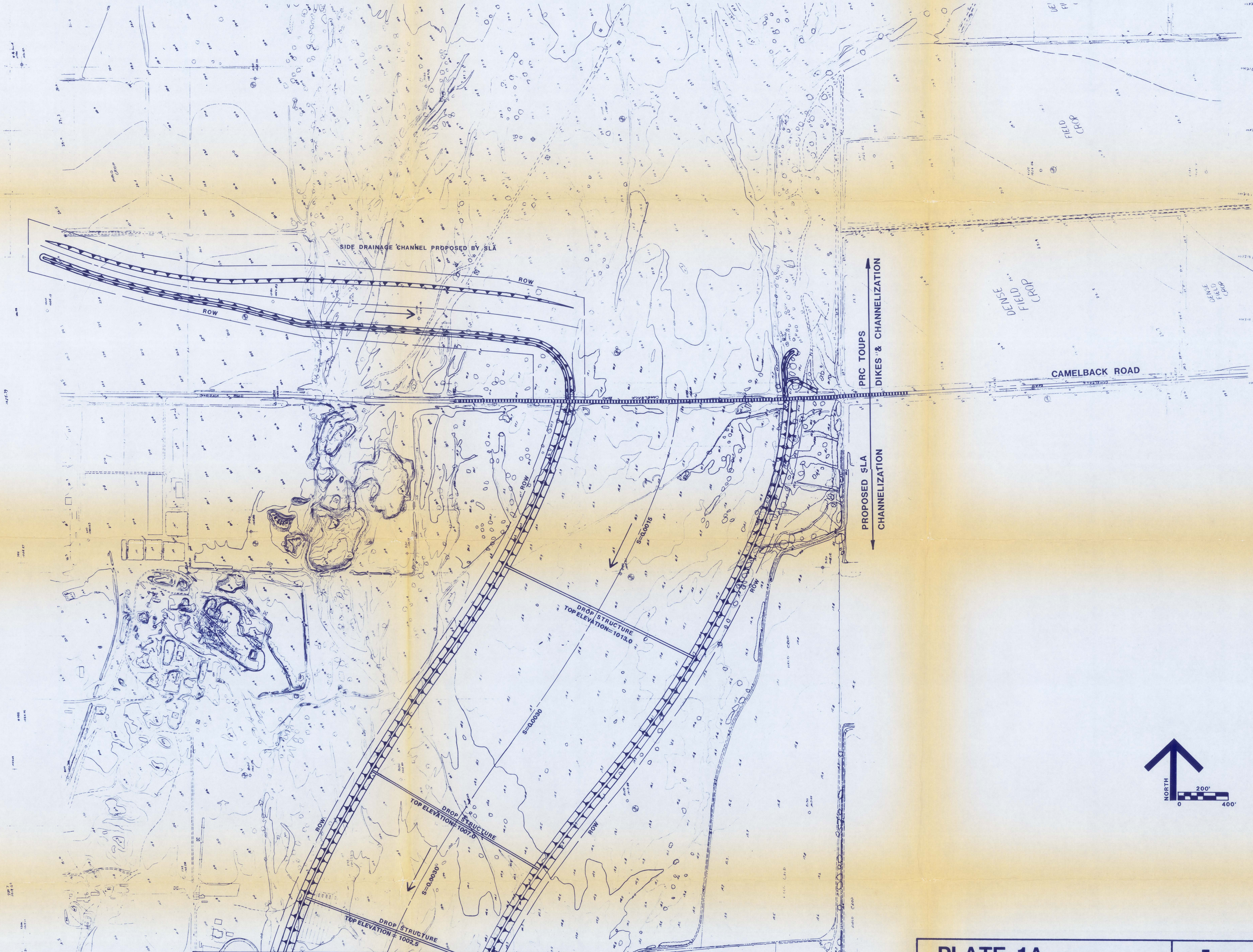


PLATE 1A

PROPOSED CHANNELIZATION
OF THE AGUA FRIA RIVER

No.	Revision	Date	By

sla
simons, li & associates, inc.
p.o. box 1816 phone 223-4100
fort collins, colorado 80522

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

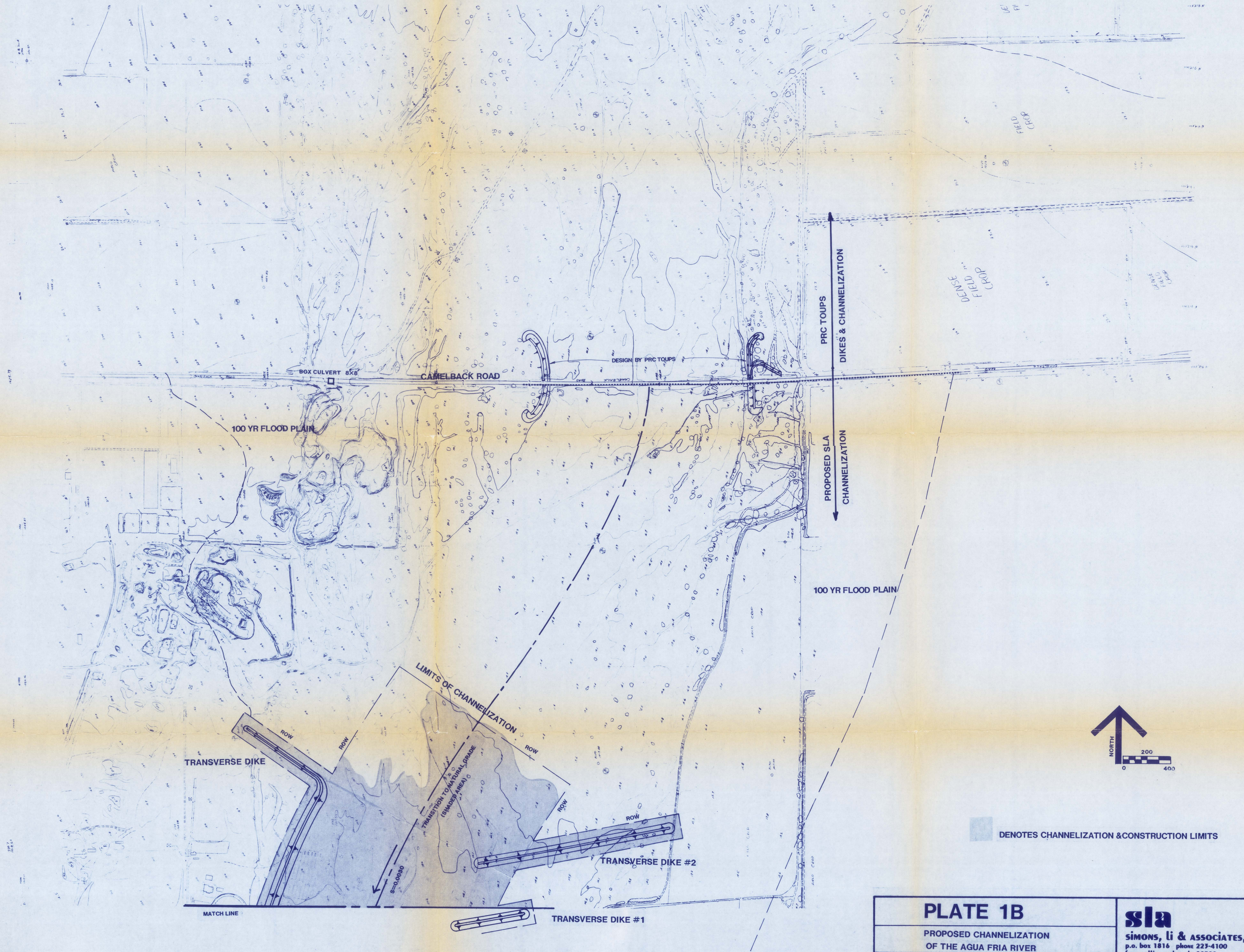


PLATE 1B

PROPOSED CHANNELIZATION
OF THE AGUA FRIA RIVER

No.	Revision	Date	By

sla
SIMONS, LI & ASSOCIATES, INC.
p.o. box 1816 phone 223-4100
fort collins, colorado 80522

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

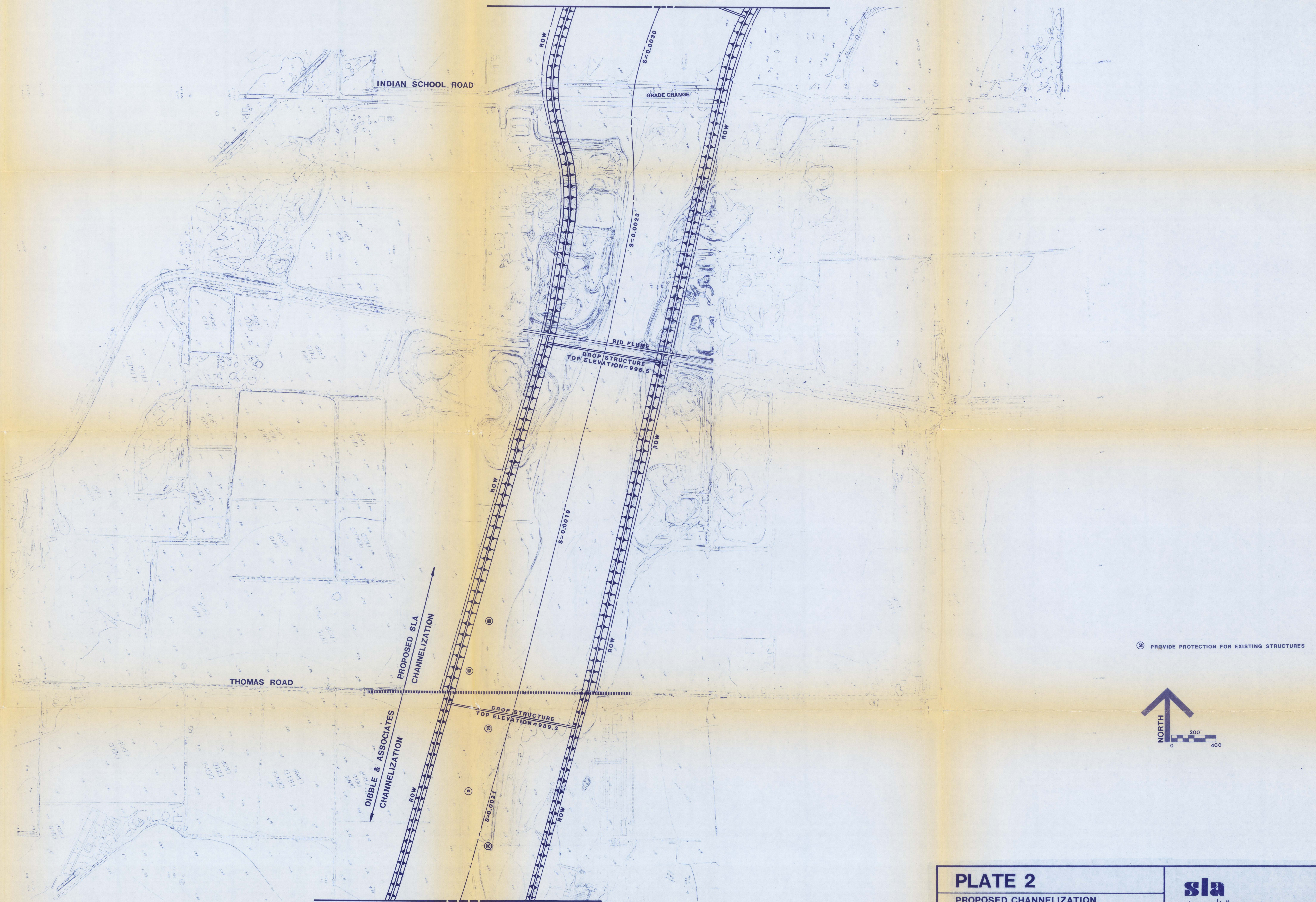


PLATE 2

PROPOSED CHANNELIZATION OF THE AGUA FRIA RIVER

No.	Revision	Date	By

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

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