FINAL REPORT

HYDRAULIC, EROSION AND SEDIMENTATION ANALYSIS OF INDIAN SCHOOL ROAD BRIDGE OVER THE AGUA FRIA RIVER PHOENIX, ARIZONA

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SIMONS, LI & ASSOCIATES, INC.

Dave Johnson

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HYDRAULIC, EROSION AND SEDIMENTATION ANALYSIS OF INDIAN SCHOOL ROAD BRIDGE OVER THE AGUA FRIA RIVER PHOENIX, ARIZONA

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I. INTRODUCTION

1.1 General

Simons, Li & Associates, Inc. (SLA) has been contracted by Maricopa County to investigate the causes of the Indian School Road Bridge failure in February, 1980, and recommend protection measures that will prevent a recurrence of failure. SLA was also contracted to investigate the stability of the Roosevelt Irrigation District flume crossing of the Agua Fria and recommend protective measures to ensure its safety.

The Agua Fria River is an ephemeral stream that originates near the Granite Dells in Yavapai County and enters north central Maricopa County at Lake Pleasant. The Agua Fria then flows south across the nearly level alluvial plains and joins the Gila River about five miles below the confluence of the Gila River and Salt River.

The Indian School Road Bridge, which spans the Agua Fria approximately 25 miles downstream of Lake Pleasant, failed February 20, 1980. Several of the bridge piers were undermined (most notably piers 14, 15 and 16), resulting in the collapse of a portion of the superstructure. The bridge is supported with reinforced concrete "tee" piers founded on spread footings some 25 feet below the original river bed. It is apparent that scour around the piers must have been significant during the flood to obtain sufficient depth to fail the footings.

To analyze the failure of the Indian School Road Bridge and suggest protective measures for the bridge and the Roosevelt Irrigation District flume crossing, a three-level engineering approach is taken. First a level one analysis involving a qualitative geomorphic assessment of conditions was conducted. This analysis involved examining aerial photographs of the Agua Fria to determine historical changes of the system. Subsequently a level two analysis was utilized involving both quantitative engineering and geomorphic methods. The second level determined local scour at the bridge and general degradation induced by headcutting through the system. The third level of analysis involved applying the SLA mathematical model to the Agua Fria to evaluate the impact of degradation near the Indian School Road Bridge.

1.2 Scope of Work

To evaluate the stability of the Indian School Road Bridge and the Roosevelt Irrigation District flume crossing, the following scope of work have been completed.

1. Site visit by SLA staff to familiarize ourselves with the study area.

2. Collect, collate, and review available data including hydrologic, hydraulic, structural, channel geometry, sediment transport, and history of activities such as gravel mining and construction of levees. The necessary data for evaluation and formulation of protection of the Roosevelt Irrigation District flume crossing have been assembled.

3. The flood hydrograph for the series of floods between 1978 and 1980 has been established. The peak flood hydrograph of 42,000 cfs that occurred on February 20, 1980, was selected as the most reasonable peak discharge at Indian School Road Bridge.

4. Review of PRC Toups data concerning the Indian School Road Bridge failure.

5. A qualitative geomorphic analysis has been performed to identify possible causes of the extreme amount of scour which resulted in the bridge's failure. Particular attention has been given to the timing of channel development, flood events, and the channel response.

6. An engineering geomorphic analysis has been performed to quantify various components of the total scour that caused the failure. This includes local pier scour, passage of sand waves, contraction scour resulting from the ineffective flow area on the west end of the bridge, headcutting due to increased velocities in the constricted channel downstream of the bridge, and degradation. The analyses considers the potential for developing an armoring layer due to the presence of coarser sediments. Each scour component considers the characteristics of the system that were responsible for its occurrence and to the extent possible if it was a natural occurrence or the result of man's activities in the area. In particular, the possibility of gravel mining operations in the vicinity of the bridge causing an increase in the total scour was evaluated. Analysis was conducted considering the channel configuration that existed with the gravel mining operation present and those that would have existed without gravel mining activities. Evaluating both conditions helps establish the extent to which gravel mining activities were responsible for the bridge failure.

7. Review the structural report on the bridge to determine the total scour depth necessary to cause failure of the bridge.

8. In addition to the results of the first two levels of analysis (qualitative and quantitative geomorphic analyses), the SLA sediment routing model has been applied to determine the dynamic response of the bed for a series of major floods between 1978 and 1980 considering the conditions with and without gravel mining operations and assess the

degradation at the bridge. The SLA model considers the routing of sediment by size fraction.

9. Determine total scour depth based on the summation of individual components.

10. Combine the findings of the three levels of analysis and structural review to ascertain the influence of each scour component on the bridge.

11. Formulate an opinion of the causes(s) responsible for the failure including the impact of various human activities, such as gravel mining. If multiple causes exist, prorate and rank the causes as to their relative contribution to the bridge's failure.

12. Review the PRC Toups HEC-2 run for the 100,000 cfs design discharge.

13. Perform a qualitative geomorphic analysis of the proposed alternatives for reconstruction or replacement of the bridge. Both long-term and short-term erosion and sedimentation effects were considered.

14. Perform a quantitative engineering geomorphic analysis to determine scour depths in order to evaluate the additional depth of pier burial to prevent failure due to excessive scour. Channel protection measures were also analyzed for stability and depth of burial. Total scour was based on contributions from local pier scour, contraction scour, long- and short-term degradation, and sand wave passage.

15. Apply the sediment routing model previously discussed for selected design alternatives for improvement of the Indian School Road Bridge and the effects on the Roosevelt Irrigation District flume crossing.

16. Based on the three levels of analysis, the proposed alternatives for the repair of the Indian School Road Bridge and channel improvements were reviewed for adequacy in terms of stability and flow capacity.

17. Modifications to the proposed alternatives and/or additional alternatives were formulated based on the three level analysis. This includes recommendations concerning the protection of the Roosevelt Irrigation flume crossing and downstream channelization.

18. Prepare a final report documenting the results of the study.

1.3 Sources of Data for Study

The following is a list of information used for the Indian School Road Bridge failure study.

Photos

1. Excavation photos - restoring the channel to its original shape directly downstream of Indian School Road Bridge 8/3/73 (17 photos).

2. Copies of photos of native material and material being replaced (1973).

3. Pictures of the bridge after the failure 1980 (3 photos).

4. Photos of the bridge during the failure taken of the 1980 flood (4 photos).

5. Photos of the bridge, flume and surrounding gravel pits 2/22/80 (25 photos).

6. Photos from November, 1981, site visit by SLA.

Aerial Photos

1. 1/3/58 Coverage from 1500 feet downstream of Thomas Road to 3500 feet upstream of Camelback Road (scale 1" = 400').

2. 1/21/64 Coverage from 2000 feet downstream of Thomas Road to approximately 2000 feet upstream of Camelback Road (scale 1" = 400').

3. 1/29/70 Coverage extends from the Roosevelt Irrigation District (RID) flume crossing to approximately two miles upstream of Indian School Road (scale 1" = 400').

4. 1970 42-inch square photos, 1970, 1" = 100', from the RID flume upstream about four miles above the Indian School Road Bridge.

5. 1971 USGS aerial photo of the Tolleson Quad., 1" = 2000', from Avondale to a half mile above the Indian School Road Bridge.

6. 1973 During the process of refilling the channel by Phoenix Sand and Rock Co. (1" = 100').

7. 1975-1980 Photos of the Agua Fria between the Indian School Bridge and the flume (1" = 250'). 8 photos.

8. 1975-1980 Photo of the Agua Fria from Glendale Avenue to Thomas Road (1" = 1200'), 1 sheet.

9. 1/25/76 Coverage from Indian School Road Bridge to approximately two miles upstream of the bridge (scale 1" = 400'), 1 photo.

10. 1978 Flood, 1" = 250', extends from the area just below the New River confluence to a point about 3000 feet below the RID flume.

11. Dec. 1978, 1" = 250', covers an area that extends upstream of the bridge 500 feet and downstream of the RID flume 5000 feet. 2 sheets.

12. 1980 flood, from the New River confluence with the Agua Fria to Roosevelt Flume (1" = 600').

Topographic Maps

1. 1957 USGS maps, 2 sheets, 1" = 2000', covers approximately all of the Agua Fria River.

2. 1973 topographic map, 4-foot contour, 1" = 400', with cross sections, and floodway and floodplain limits from a flood study in 1973. 4 sheets.

3. 1980 topographic map, 2-foot contour, 3 copies.
   a. 1" = 400' with location of PRC Toups cross sections.
   b. 1" = 400' with both PRC Toups cross sections and the 1973 flood study cross sections.
   c. 1" = 100' with flood control plans by PRC Toups.

Sediment Data

1. Surface and subsurface bed material samples broken down into size fraction by the Arizona State Highway Department for a borrow pit in the near vicinity of Indian School Road Bridge 1951.

2. Surface and subsurface bed samples broken down into size fractions at the Indian School Bridge. From report by Sergent, Hauskins and Beckwith, 1980.

3. Surface and subsurface bed samples broken down into size fractions at the Camelback Road Bridge crossing. From report by Engineers Testing Laboratories, 1981.

4. Surface and subsurface bed samples broken down by size fractions on the Agua Fria near Thomas Road and 2200 ft upstream of Indian School Road Bridge. Borings were conducted by Sergent, Hauskins and Beckwith while the sieve analyses were performed by the Maricopa County Highway Department, February, 1982.

5. Photographs taken by Simons, Li & Associates of subsurface bed samples taken at the apprxoimate site of the old instream gravel pit of Phoenix Sand and Rock Co. located 800 feet downstream of Indian School Road Bridge.

Reports

1. 1973 Flood Insurance Study, Maricopa County, by the L.A. Corps of Engineers. Includes bed and flood profiles for Agua Fria.

2. Geotechnical report for Camelback Road Bridge crossing of Agua Fria River, by Engineer Testing Laboratories Inc. Includes gradation of bed samples, 1981.

3. Geotechnical Investigation Report, Indian School Bridge at Agua Fria River, by Sergent, Hauskins and Beckwith. Includes estimated scour depth at each pier and gradation of bed samples at the bridge. 1980.

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

5. Preliminary Report, Indian School Road Bridge at the Agua Fria, Rehabilitation and Stabilization of Channel, PRC Toups, 1981.

6. Pier Scour, Flume Piers in the Agua Fria River, includes six test borings at the RID flume, no breakdown by size fraction, by Engineers Testing Laboratories, 1980.

7. Reconstruction of the Indian School Road Bridge Over the Agua Fria River, by Samer, Lahlum and Associates, Inc., 1980.

8. Soil Survey of Maricopa County, Arizona, Central Part, United States Department of Agriculture, Soil Conservation Service, September 1977.

Bridge Plans

1. 1969 plans for construction of the Indian School Bridge. Includes boring samples at the bridge site.

2. 1977 plans for addition of the third and fourth lanes on the Indian School Bridge.

Cross Section Plats

1. PRC Toups 1980 topographic map (river mile 7.98 to 10.27).

2. 1973 topographic map - digitized by the Los Angeles Corps of Engineers (river mile 7.88 to 10.10).

Hydrographs

1. December, 1978, January, 1979, and February, 1980, hydrographs at Waddell Dam and at Avondale.

2. 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 date March 2, 1981.

Legal Concerns

1. Copy of Arizona State laws concerning flood control and floodplain management.

2. Maricopa County Floodplain Regulations for unincorporated areas.

3. Correspondence between the Attorney General and the Phoenix Sand and Rock Company and the Highway Department. Obtained from the State Attorney General's office.

II. DATA REVIEW

2.1 Hydrology

The Los Angeles District of the Corps of Engineers published a hydrology report in April, 1981 on Phoenix City streams, with particular emphasis on the flows of the Agua Fria. Return flow frequencies of various flow events at different locations in the Agua Fria were computed. A return flow period is defined as the reciprocal of the probability of a flow discharge being equaled or exceeded in any year. Thus a 100-year event will occur, on the average, once every 100 years. Table 2.1 summarizes discharges at Indian School Road Bridge for the 2-, 10-, 25-, 50-and 100-year return flow periods as computed by the L.A. Corps of Engineers.

Examination of the discharge records maintained by the United States Geological Survey (USGS) gaging station at Avondale between the first construction of the bridge in 1970 and the collapse of the bridge in February of 1980 reveal that only three major floods have occurred in this time period. The majority of the flow from these floods are released from Waddell Dam. These floods are the December 1978, January 1979 and February 1980 events (see Figure 2.1). Another significant flood that occurred in February 1978 had a duration of one month; however, the peak flow was not nearly as high when compared to the 1978, 1979 and 1980 flood peaks (12,000 cfs compared to 29,000, 24,500, and 42,000 cfs, respectively). The February 1978 event was considered in the sedimentation analysis because of its extended duration.

The peak water discharge release from Waddell Dam, which is located 25 miles usptream of Indian School Road Bridge, was 73,300 cfs during the February 20, 1980, flood, as reported by the Maricopa County Municipal Water Conservation District No. 1 in the April, 1981, Corps of Engineers report. The peak water discharge recorded at the Avondale gaging station, which is located three miles downstream of Indian School Road Bridge, was 42,000 cfs. This was approximately equivalent to a flood with a return period of 25 years. For subsequent sedimentation analysis, the water discharges of Avondale were considered to be more representative of the flows at Indian School Road than the releases from Waddell Dam, because there is a significant channel storage, and infiltration losses that result in flood peak attenuation in the 25 mile reach between Waddell Dam and Indian School Road Bridge.

The 100-year flood of the Agua Fria downstream of the confluence with the New River is shown in Figure 2.2. This hydrograph was extracted from the Los

Table 2.1. Summary of Return Flow Events at Indian School Road Bridge.

Return Period of Flood	Discharge (cfs)
2 years	7,200
10 years	30,000
25 years	49,000
50 years	69,000
100 years	94,000

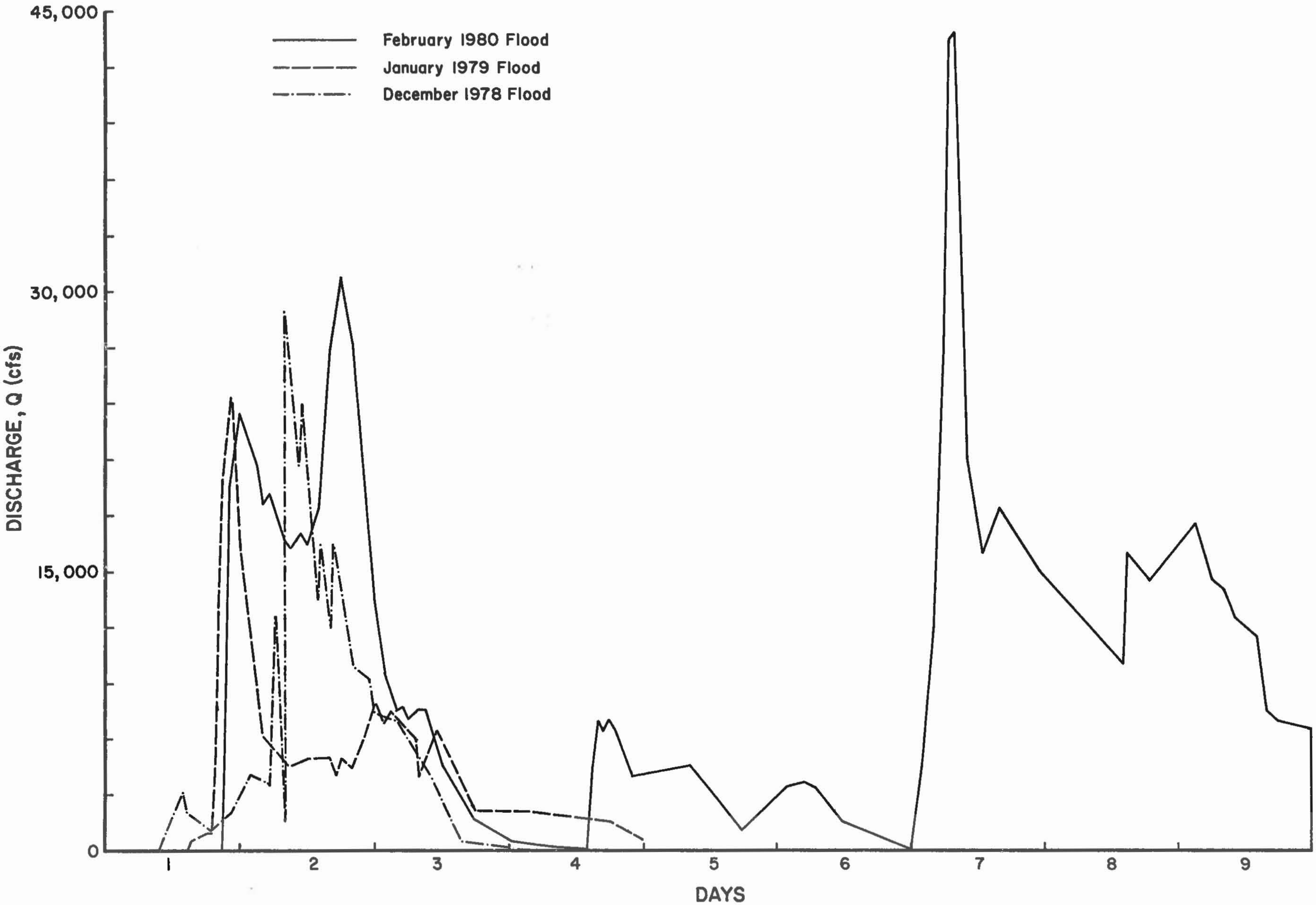


Figure 2.1. Hydrographs for 1978, 1979 and 1980 flood events on the Agua Fria.

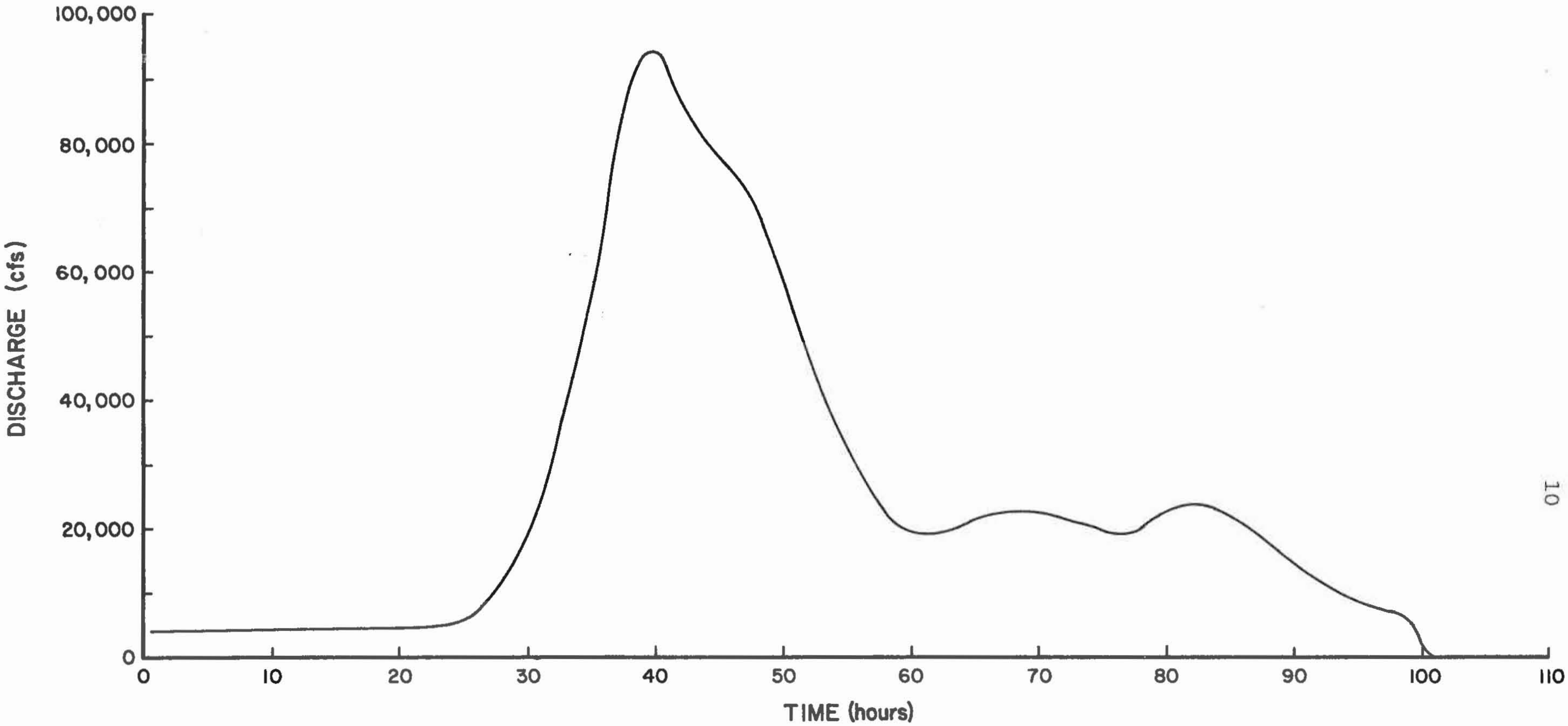


Figure 2.2. 100-year flood on the Agua Fria below the New River confluence as computed by Los Angeles Corps of Engineers, 3/2/81.

Angeles Corps of Engineers March 2, 1981, study. The peak discharge is 94,517 cfs and the duration of the event is 101 hours. For evaluation of future protection measures, this hydrograph will be utilized.

2.2 Channel Cross-Sectional Geometry

To evaluate the difference between mining and premining conditions, the 1973 cross sections used by the Los Angeles Corps of Engineers in their floodplain delineation study were used by SLA subject to limited modifications. The cross sections were plotted and verified by comparison with the 1973 topographic maps of the Corps of Engineers. Some of the cross sections were simplified. However, the general geometry characteristics of the cross sections such as area, top width, and wetted perimeter remained essentially the same as the natural cross section. The 1973 cross sections used by the Corps of Engineers included the instream gravel pits that were partially filled in by the Phoenix Sand and Rock Company in August of 1973. For scour analysis all the instream gravel pits as well as the floodplain gravel pits were assumed to be filled for the baseline condition. To simulate gravel mining conditions, levees were assumed as they existed in the 1978 aerial photographs. Also, no instream gravel pits were considered, because all of the instream pits were assumed to be filled before the 1978 flood. The levees on the east bank of the Agua Fria between Indian School Road and the Roosevelt Irrigation District flume were constructed to protect the floodplain gravel pits of the Phoenix Sand and Rock Company. Similarly the levees on the west bank protect the floodplain gravel pits of the Allied Concrete Co.

For stability evaluations the 1980 cross sections used by PRC Toups were used to evaluate the conditions as they exist in 1981. The 1980 cross sections were modified to reflect the channelization alternatives suggested by PRC Toups and the channelization recommended by SLA.

2.3 Sediment Size Distributions

Grain size distributions were available from sieve analyses conducted by the Arizona Department of Transportation on borrow pit material near the Indian School Road Bridge in 1951, from Sergent, Hauskins and Beckwith bores in 1980 at the bridge, from Engineers Testing Laboratories bores near Camelback Road in 1981, and 1982 corings by Sergent, Hauskins and Beckwith of the Agua Fria near Thomas Road and upstream of Indian School Road Bridge.

Examination of the surface grain size distributions of the various samples indicate the bed material is similar between sampling locations. The average surface and subsurface grain distributions for the samples are plotted in Figure 2.3. The subsurface material is slightly coarser than the surface material. The subsurface material has some particles 55 mm in diameter, or slightly greater than two inches, while the surface material has particles up to 33 mm in diameter, or slightly greater than one inch. This is an important difference because of the armoring potential of the coarser sized material. Should some of the coarser material be removed by gravel mining or by a head-cut, local scour would be increased. The D_{50} sizes for surface and subsurface distributions are 0.76 mm and 0.80 mm, respectively. The majority of the material in the bed is in the sand size range (0.0625 to 2.0 mm). The boring information obtained at the time of bridge construction indicates that there are some gravels and boulders. The sediment size distribution could be coarser than that measured in 1980 if there is no gravel mining activity.

2.4 Floodplain Regulations

A review of some of the floodplain regulations pertaining to gravel mining is covered in this section. Gravel mining is allowed by permit only and only after verification that mining of such materials does not cause flood related losses to other lands or to the public.

For sand and gravel mining regulations within the floodplain the Arizona State Law 45-2343B, condition 3 explains written authorization shall not be required nor shall the floodplain board prohibit: "construction of tailing dams and waste disposal areas for use in connection with mining and metallurgical operations. This paragraph does not exempt those sand and gravel operations which will divert, retard or obstruct the flow of waters in any watercourse from acquiring authorization from the floodplain board pursuant to regulations adopted by the Board under this chapter."

The Maricopa County Amended Floodplain Regulation for the unincorporated area of Maricopa County, Arizona, in October of 1977 adopted a permit use policy concerning the extraction of sand, gravel and other materials in the Floodway District. Section 6.2 of the floodplain regulations further states that the extraction of sand, gravel, and other materials may be permitted to the extent that they do not require permanent structures, fill or other obstructions to the flow of flood water in the Floodway District, and provided

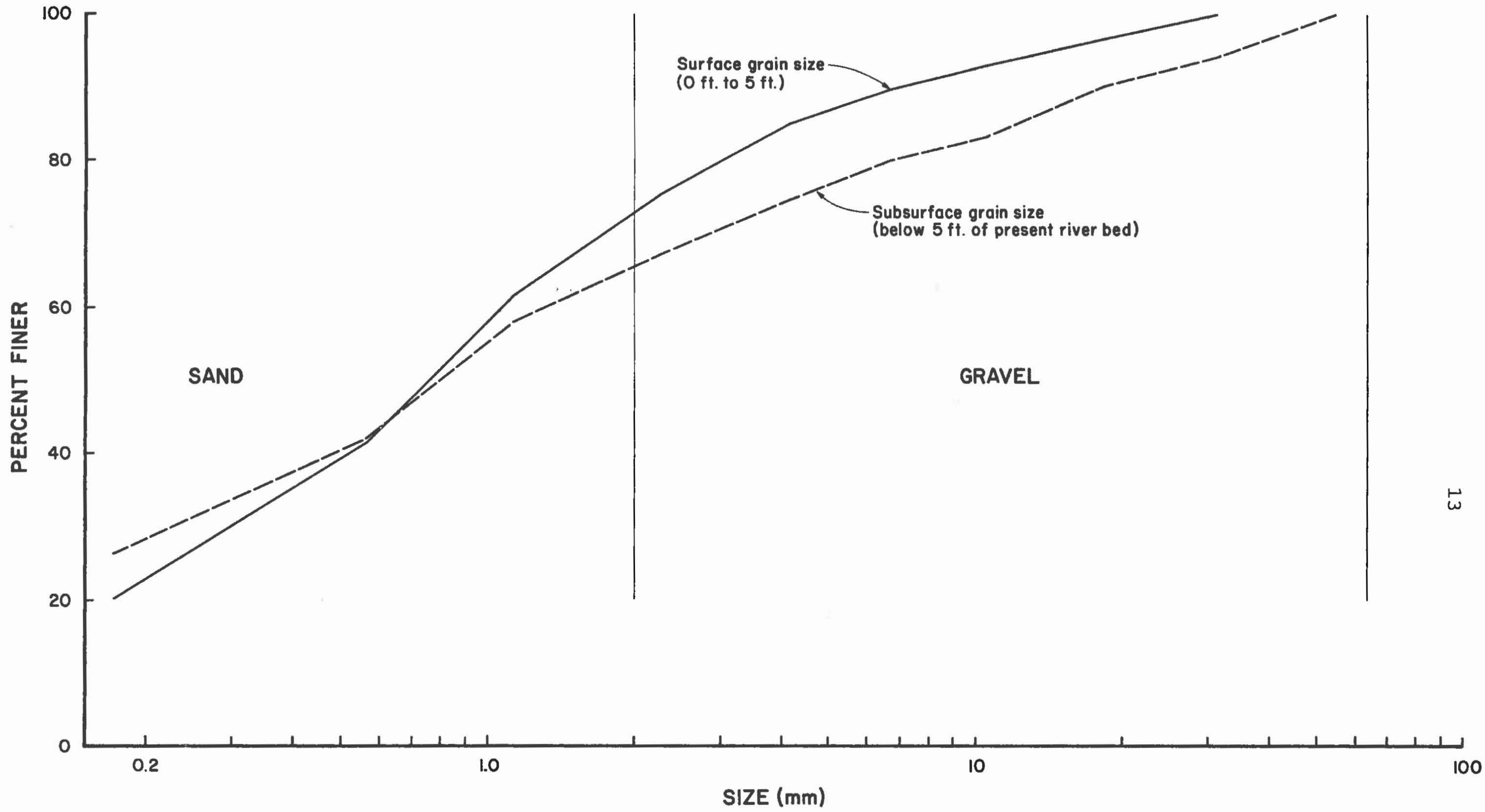


Figure 2.3. Surface and subsurface grain size distributions of the Agua Fria near Indian School Road Bridge.

that they do not adversely affect the capacity of the channels or floodways of any tributary to the main stream, drainage ditch, or any other drainage facilities or system. Floodway District is defined as the channel of a watercourse on the body within the banks of a lake and that portion of the adjacent land areas designated by the Floodplain Board as necessary to provide for the passage or ponding of flood water of any watercourse or lake without allowing a rise of more than one foot in the flood elevation at the time of delineation. This regulation is mandatory under the federal law according to Federal Emergency Management Agency.

Maricopa County also describes the permit use allowed in the floodway Fringe District in Section 7.22 of the Amended Floodway Regulation. The Floodway Fringe District is defined as the land outside the Floodway District and lower than the Regulatory Flood Elevation along the watercourse. Uses listed in Section 6.2 of the Amended Floodway Regulation and other similar uses are permitted if they are not subject to substantial flood damage and do not cause flood losses on other lands or to the public.

Therefore, if extraction of gravel from the Floodway District or the Floodway Fringe District had precipitated a headcut or general scour through Indian School Road Bridge that would not have normally occurred, the bridge could have failed, and the gravel mining companies could have been in violation of the amended Maricopa County Floodplain Regulations. Also if the constriction of the levees raises the flood elevation more than one foot, there is a violation of the Amended Floodway Regulations.

III. QUALITATIVE GEOMORPHIC ANALYSIS OF BRIDGE FAILURE

3.1 Man's Activities Near Indian School Road Bridge

The first significant development on the Agua Fria was Waddell Dam completed in 1927. The Agua Fria watershed drains 1650 square miles of which 1459 square miles are above Waddell Dam. The dam is approximately 30 miles north-northwest of downtown Phoenix. The main purpose of the dam is water conservation. If sufficient water is available, the normal water surface is kept at the top of the spillway gates (gage height 170'). The storage capacity at the top of the dam (gage height 175.7) is 175,000 acre-feet and at the normal pool (gage height 170') the storage capacity is 157,600 acre-feet. The dam was not constructed and operated with the purpose of flood control.

The second significant structure that was added to the system is the Roosevelt Irrigation District flume crossing that was constructed in 1929. The flume crossing has a length of 5959 feet spanning the Agua Fria River approximately 2200 feet downstream of the present Indian School Road Bridge. The flume cross section is a semicircle, 13 feet 4-3/8 inches in diameter, and has an approximate discharge capacity of 386 cubic feet per second. The superstructure that supports the flume consists of two types, a steel trestle and steel trusses. Eighteen 70.5-foot trusses with a total length of 1269 feet span the main channel. On the east and west ends steel trestles with 18 foot spans support the flume as it crosses the floodplain channel. The foundation for the flume within the main channel consists of piers that extend at least 12 feet below the streambed. These piers rest on concrete footings which in turn are supported by concrete piles. The piles vary in length depending on the distance to hardpan material. They range from 17 to 25 feet in length. Hence, the piers extended from 32 to 40 feet below the 1929 stream bed. The piles for the trestle portion range in length from 24 feet to a 30 feet maximum.

Several gravel mining operations existed in the main channel and on floodplains of the Agua Fria near the Indian School Road Bridge prior to 1958. Examination of the 1958 aerial photographs shows evidence of gravel mining in the Agua Fria directly downstream of the present Indian School Road Bridge and on the floodplain on the east side, halfway between the bridge and the flume. The 1970 aerial photographs show extensive gravel mining operations immediately downstream of the bridge by Phoenix Sand and Rock Co. During the same period Allied Concrete Co. was gravel mining on the west side of the floodplain.

In August, 1973 the Phoenix Sand and Rock Co. was requested by Maricopa County to fill their instream gravel pits near Indian School Road Bridge. The main pit was located approximately 800 feet downstream of Indian School Road Bridge. The total fill volume required as estimated by Jim Webster, Engineer in the Photogrammetry and Mapping Division in a letter sent to Carter A. Clark, Chief Right of Way Agent December 19, 1973, was 135,925 cubic yards. The material used to fill the instream gravel pit might be coarse material but its quality is questionable. In an office memo by J. C. Latham, Assistant Engineer of Materials sent to Carter A. Clark, January 9, 1974 he stated "The material that was used for the backfill of the pit excavation was waste rock. Due to the lack of gradation and the clay coating, this material would only be suitable for borrow.." Without a proper gradation or homogeneous mixing of material the filled pit could sink due to a gravity failure and initiate a headcut. So even if the pit was filled with large material it doesn't necessarily prevent a headcut from initiating.

Between August 1973 and January 1975 dikes were constructed by Phoenix Sand and Rock Co. and Allied Concrete Co. on the east and west banks, respectively, of the Agua Fria downstream of Indian School Road Bridge to the Roosevelt Irrigation District flume crossing. The constriction in the flow due to the construction of levees to protect the floodplain gravel pits could have caused a headcut.

At the time of the November 5, 1981, site visit conducted by Simons, Li & Associates, Inc., the gravel mining operation extended from Indian School Road Bridge well downstream of the flume. Dike heights were approximately 15 to 20 feet between the bridge and flume. Dike heights downstream of the flume on the west bank were 12 to 15 feet high and 5 to 8 feet high on the east bank.

The construction of the first two lanes of Indian School Road Bridge was completed in 1970. An additional two lanes were constructed in 1977. The bridge spans 1600 feet across the Agua Fria with 17 piers spaced at 90-foot intervals. The piers are aligned at an 11 degree angle normal to the flow.

The bridge is a twin span reinforced concrete superstructure with reinforced concrete "tee" piers founded on spread footings buried approximately 25 feet below the stream bed. The low chord elevation of the bridge is at elevation 1015 feet.

3.2 Stream Classification

The Agua Fria River is an ephemeral braided stream. A braided river is generally wide with poorly defined and unstable banks. It is characterized by a steep shallow course with multiple channel divisions around alluvial islands. The two primary causes of braiding are (1) overloading of sediment, that is the stream may be supplied with more sediment than it can adequately transport resulting in aggradation and steepening of the channel, and (2) steep slopes, which produce a wide, shallow, unstable channel where bars and islands readily form. Either of these factors alone or in conjunction may be responsible for braided conditions.

The overall slope of the Agua Fria near Indian School Road Bridge (ISRB) in 1980 was about 0.003. A river generally tends towards a braided pattern if:

$$S\,Q^{1/4} \geq 0.01$$

where S is the slope and Q is the dominant discharge. For ephemeral streams in Arizona the dominant discharge for a river can be assumed equal to the two-year flood. For the Agua Fria the two-year flood is approximately 7200 cfs, and

$$S\,Q^{1/4} = 0.003\,(7200)^{1/4} = 0.03$$

This value of $SQ^{1/4}$ is considerably larger than the lower limit value of 0.01 where a river develops a braided pattern. This analysis verifies that the Agua Fria should be braided in its uncontrolled state.

3.3 History of Recent Channel Morphology

The recent channel morphology in the Agua Fria River has been influenced by man. A brief summary of man's activities over the years on the Agua Fria was presented in Section 3.1. This section documents, in more detail, the changes that have taken place on the Agua Fria and discusses the correlation, if any, between those changes and man's utilization and development of the fluvial system.

Braided rivers such as the Agua Fria typically consist of relatively unstable interconnected low flow channels or braids. Overall sinuosity of a braided stream is generally low, although individual braids may be very sinuous. Historically the overall channel alignment of the Agua Fria River in the vicinity of the ISRB has remained fairly constant. The major exception is the reach between ISRB and the Roosevelt Irrigation District (RID) flume. In this reach of river the channel has been severely encroached upon by sand and gravel mining activities.

In the Agua Fria, as with any braided stream, change within the channel can be rapid and extensive. The most obvious and important in-channel features adjacent to the ISRB is the low-flow channels immediately upstream of the bridge. A sequence of aerial photos dating from 1958 to 1980 clearly show two distinct low-flow channels converging in the vicinity of Indian School Road. The photos indicate that both these channels are influenced by flow from the New River. Most of the flow in the Agua Fria prior to 1978 originates from the New River.

The 1958 photo indicates that the dominant low-flow channel is probably the easternmost one. This photo also shows the two channels converging approximately 1000 feet downstream of Indian School Road. By 1964 the western channel seems to have become slightly larger and better defined than the eastern braid. This is likely due to the impact of gravel mines that were active within this channel and upstream of Indian School Road. By 1970 the dominant channel is the western braid. By 1976 the eastern braid is relatively indistinct. Additionally, the construction of ISRB has confined the confluence of the two braids within the bridge opening. Flows in 1978 re-established the eastern braid, and in the 1980 flood a large portion of the flow followed this eastern low-flow channel. Flow within the eastern braid impinges upon the ISRB piers at a severe angle. This results in increased local scour at the piers.

Most natural braided streams in Maricopa County are in the aggrading mode. Two primary causes of the braided condition were identified and include (1) overloading the system with sediment and (2) steep slopes. The Agua Fria is typical of a braided, aggrading river with the exception of the reach between ISRB and the RID flume crossing. This stretch of the river should be in the aggrading mode from examining the 1957 profile. On the contrary, this stretch of the river has been in the degrading mode as evidenced by the change in the thalweg profile shown in Figure 3.1. Two causes for the degradation are (1) the constriction of the channel due to the levees and (2) the extraction of sand and gravel from the bed.

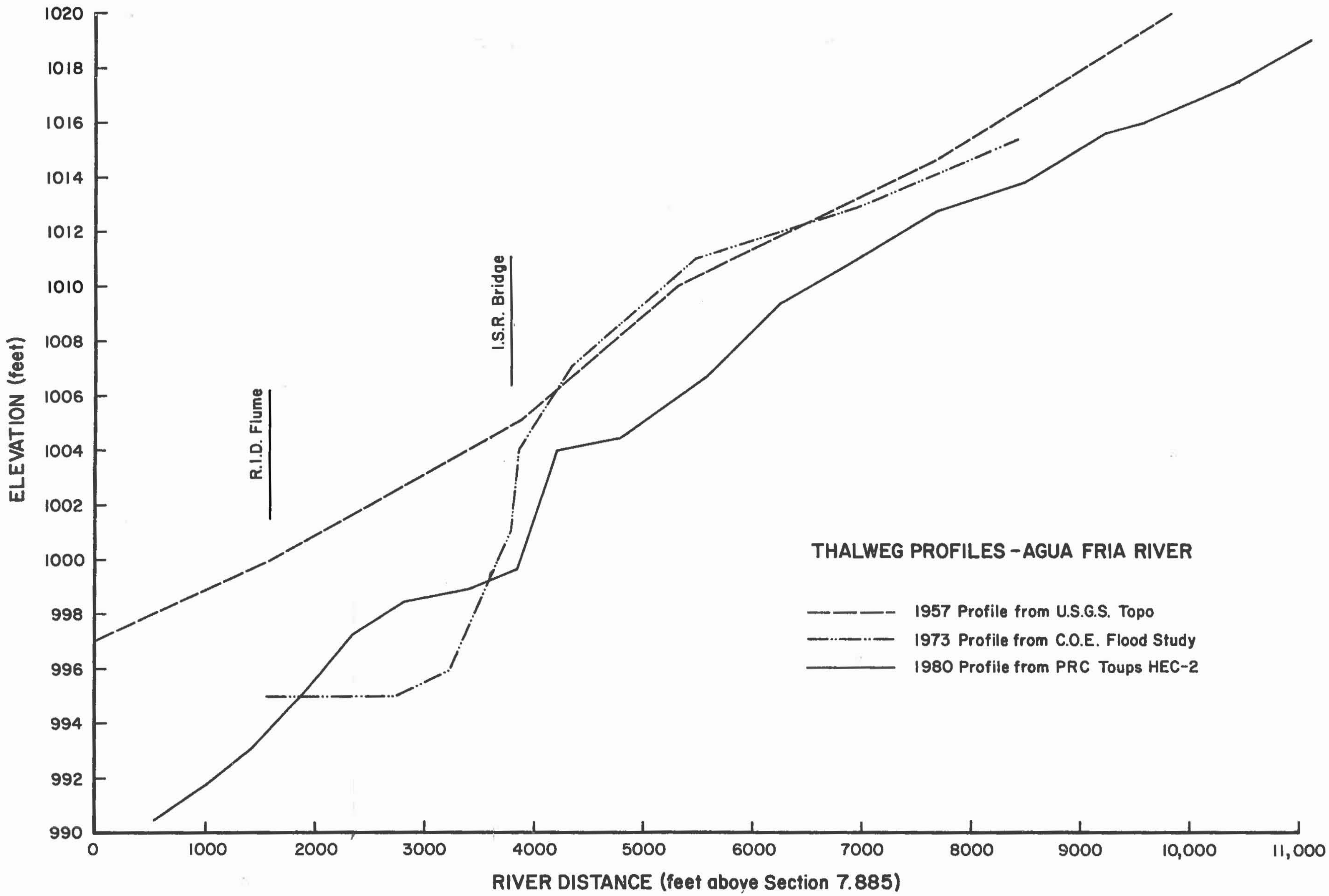


Figure 3.1. Comparison of thalweg profile versus river distance (feet above cross section 7.885) for various years.

Constriction of the channel has caused increased velocities and increased sediment transport capacity. Sediment transport rates are proportional to velocity to the fourth or fifth power. Hence, a slight increase in velocity can result in a dramatic increase in sediment transport. For example, a ten percent increase in velocity results in approximately a 40 percent increase in sediment transport. Therefore, narrowing the natural channel with levees near the Indian School Road Bridge, gravel mining has increased the sediment transporting rate of material near the bridge.

Not only is the sediment transport rate increased because of increased velocities, but the ability to transport large sediment particles is increased. Therefore, it requires coarser sediment particles to armor the bed than with the previous slower velocities associated with the natural channel condition, thus upsetting the equilibrium of the system.

Considering man's activities, sand and gravel mining has had the most significant impact on the morphology of the Agua Fria River. A precise history of sand and gravel mining in the Agua Fria near ISRB is extremely difficult to compile. A brief history of in-channel mining and dike building to protect floodplain mining was given in Section 3.1. Instream mining has been an important factor in the reach between ISRB and the RID flume. Figure 3.1 showed channel thalweg profiles for 1957, 1972, and 1980. Even though the major instream pit below ISRB was partially refilled in 1973, there has been net degradation of the system. This can probably be attributed to two factors: (1) removal of material from the channel due to mining activities, and (2) increased general degradation or headcutting due to high velocities caused by excessive reduction of channel width by floodplain gravel mining activities.

Headcutting is the process of general scour that is caused within a river by increasing the unit discharge, excessive slope, etc. Such headcuts proceed upstream through the river system. The presence of an instream pit or an area where the local slope has been increased can further increase the energy acting on the channel system by increasing the energy slope. The steepened slope has greater erosive power and can initiate bank erosion and headcutting. These processes can tip the balance of sediment transport and cause significant degradation upstream of the localized area. A sketch of the headcutting process is shown in Figure 3.2. It must be noted that even if the instream gravel pit downstream of ISRB was backfilled with coarse material, if the

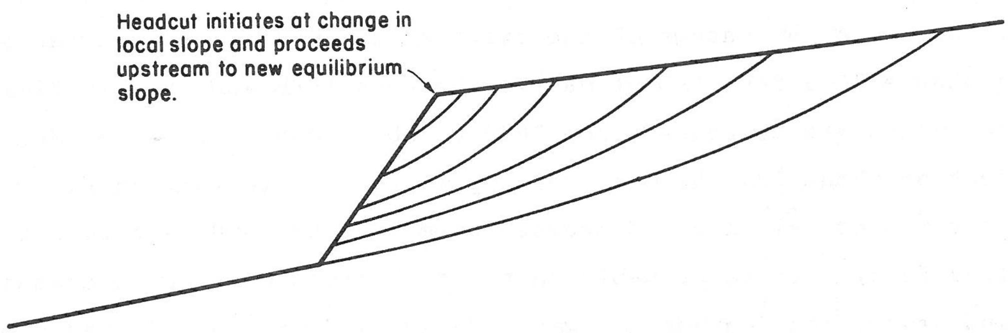


Figure 3.2. Sketch of headcutting process.

local bed slope was increased sufficiently, a headcut could have initiated in this area.

The constriction of the main channel that was imposed by mining activities is graphically illustrated in Figure 3.3. This figure shows bankfull top width as obtained from aerial photos versus river distance for 1958 and 1980. The figure indicates that the study reach of the Agua Fria has experienced a significant overall reduction in bankfull top width. The largest reductions occurred in the reach between the ISRB and the RID flume crossing. In this reach the channel was narrowed by dikes constructed to protect floodplain sand and gravel mining activities. This reduction becomes even more important when viewed as a percentage of the original top width. For instance, at river distance 3400 feet (about halfway between ISRB and the RID flume crossing) the top width was decreased from 2000 to about 800 feet, a net decrease of 1200 feet or about 2/5 the original top width. Above ISRB at river distance 5600 feet the top width was decreased from 3400 to 2300 feet (a net decrease of 1100 feet). It is probable that the observed upstream decrease in top width and associated degradation was induced by downstream degradation caused by over contraction of the channel and removal of material as a result of mining from the channel below ISRB.

3.4 Temporal Changes in Bed Material

Available data include the sieve analyses of 14 samples collected in 1951 from a state borrow pit area just downstream of Indian School Road. These were supplied by the Arizona State Highway Department. Samples were taken near the piers of ISRB by Sergent, Hauskins and Beckwith (1980), samples resulting from borings at Camelback Road were taken by Engineers Testing Laboratories (1981), and samples near Thomas Road and 2200 feet upstream of Indian School Road Bridge by Sergent, Hauskins and Beckwith (1982). The State Highway surface samples included particle sizes one inch in diameter and less. Of the 14 samples (0' to 8' in depth) the composite average of the size distributions confirms that approximately seven percent of the size distribution is coarser than one inch in diameter. One surface sample was taken by Sergent, Hauskins and Beckwith (1980). This sample extended 18 feet below the bed profile, and the largest size reported in this sample was about 1-1/2 inches in diameter. Six surface material samples were gathered by Engineers Testing Laboratories at Camelback Road at depths from 0 to 3 feet. The com-

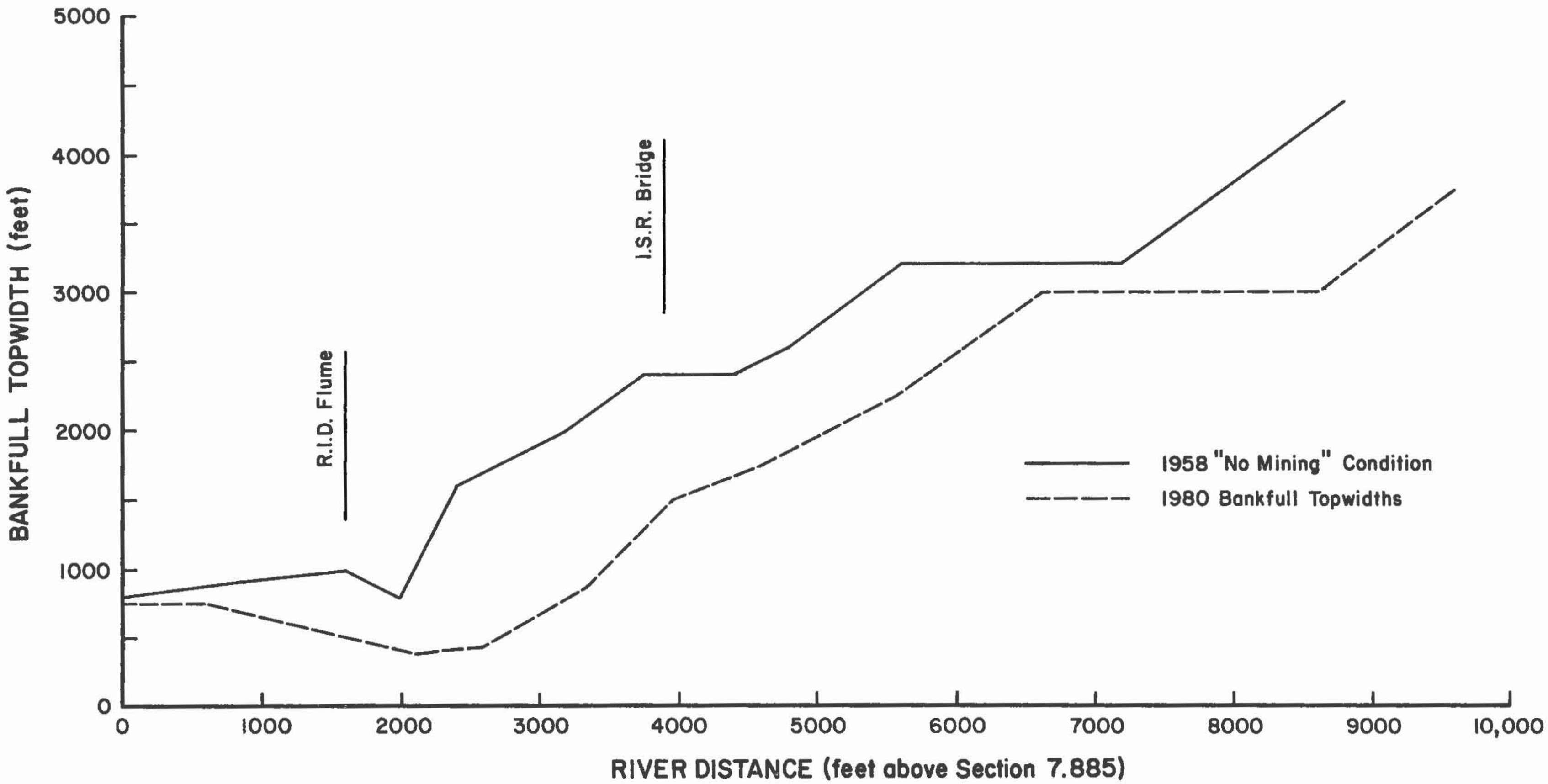


Figure 3.3. Bankfull top widths versus river distance.

posite average of the six samples showed that less than one percent of the material was greater than one inch in diameter, and this was verified by the two corings completed by Sergent, Hauskins and Beckwith in 1982 where less than one percent of the samples were greater than one inch in diameter.

The general trend indicated by this limited number of samples is a reduction in the size of coarse material. Further evidence supporting the loss or removal of coarse material from the bed material is obtained from the original 1966 bore logs supplied by Maricopa County Highway Department. The original cores at the approximate bridge pier locations (piers 14 and 15) indicate a substantial amount of gravel sized material (coarser than 1-1/2 inches) existed in the bed material. The potential for an armor layer to form would have been excellent if this material had been available during the 1980 flood. The June 1980 borings conducted by Sergent, Hauskins and Beckwith indicate the majority of the material found at the bridge was sand. This should be expected since the general and local scour induced by a headcut during a flood would be backfilled with sand during the recession of the flood. There are two possibilities that could account for the reduction in size of coarse material: (1) the general scour induced by a headcut due to a downstream channel contraction, and (2) local scour at the bridge site, or a combination of both factors. Such occurrences would be backfilled with sand during the recession of the flood.

3.5 Upstream Development

Existing developments on the Agua Fria watershed upstream of ISRB include Waddell Dam, Dreamy Draw Dam, and Caves Buttes Dam. Planned development includes Adobe Dam, New River Dam, and the Arizona Canal Diversion channel. Dreamy Draw, Adobe, and the New River Dam will decrease flows on the New River and are designed to compensate for the effect of the Arizona Canal Diversion Channel. Historical records have shown Waddell Dam to be relatively ineffective in controlling major floods on the Agua Fria River.

The effect of upstream dams is to increase channel degradation due to clear water discharges. It is difficult to accurately determine the effect of Waddell and Dreamy Draw Dam on the Agua Fria River near ISRB, but due to the considerable distance between the bridge and the dams it can be safely assumed that degradation at ISRB due to these dams is negligible.

3.6 Geology of Maricopa County

The general geology and physiography of the Agua Fria Valley and watershed are illustrated in Figure 3.4 and both are described. The description and interpretation of the geologic substrata within Maricopa County are based on work by Wilson et al. (1957) and on data extrapolated from a study of a similar alluvial valley adjacent to the Agua Fria, i.e. Sycamore Creek, by Thomsen and Schumann (1968).

The lower alluvial area is underlain by poorly consolidated alluvial deposits of Tertiary and Quaternary age. Deposits in the floodplain are unconsolidated alluvium that consists of sand, silt, gravel and some clay (unit Q_s, Figure 3.4). This alluvium contains appreciable amounts of firmly cemented fine-grained soils of low permeability. However, most of the alluvium is unconsolidated sand and gravel with high permeabilities.

The floodplain deposits overlie or are cut into the alluvial valley deposits. These consist of sand, gravel, conglomerates, sandstone and siltstone (unit QT_s, Figure 3.4). Thin terrace gravel overlies the finer grained alluvium along some sections of the Agua Fria River. These valley deposits unconformably overlie granite and related crystalline rocks in the lower valley.

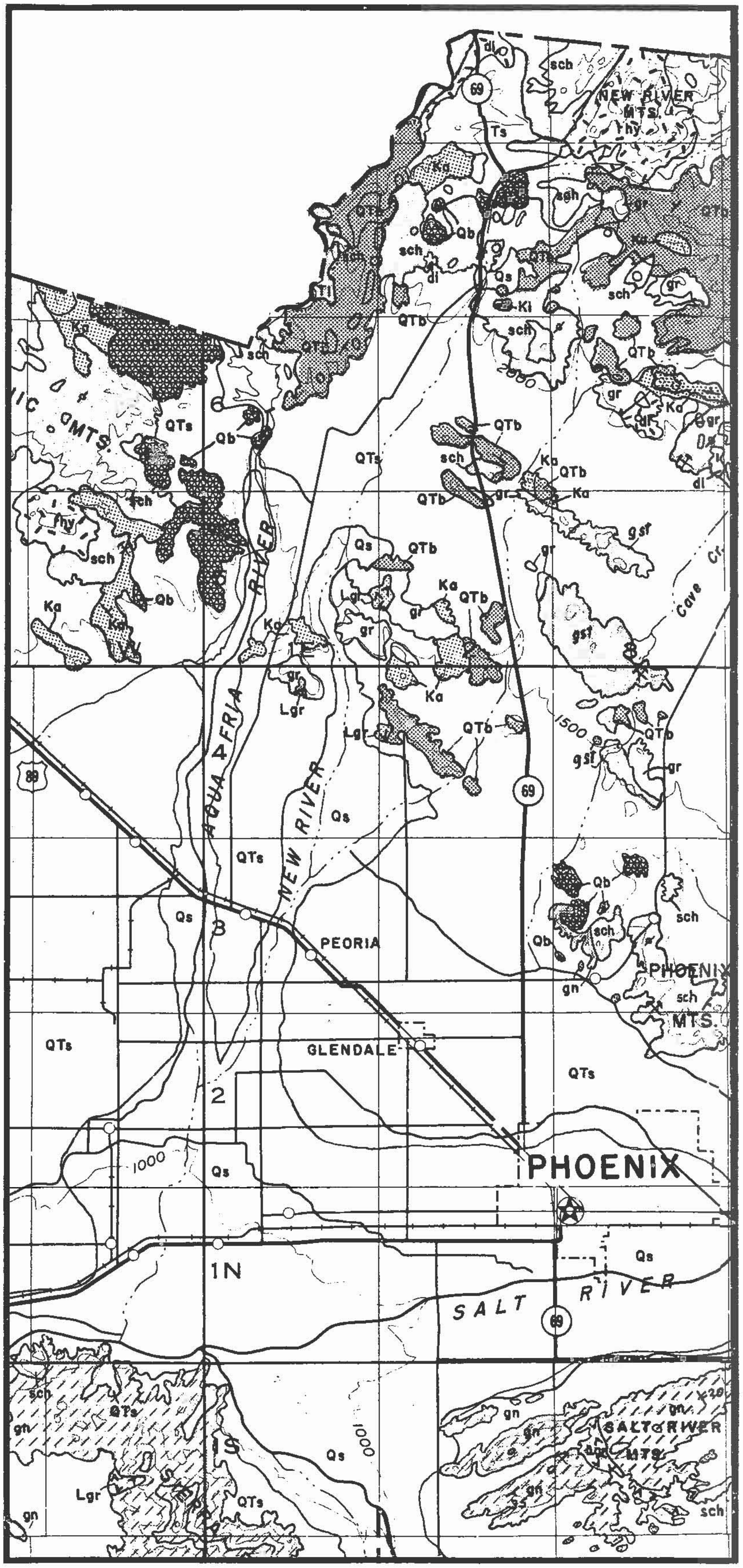
The soils in the the lower alluvial valley are formed on either recent or old alluvium (Soil Conservation Service, unpub.). Soils in or adjacent to the river channel are characteristically deep, sandy and gravelly soils. These gravelly sandy loams and loamy fine sands are formed in recent alluvial material and are moderately alkaline and slightly to strongly calcareous. Thus it appears as if no geologic controls are present to act as natural grade controls in the Agua Fria near Indian School Road Bridge.

3.7 Conclusions

1. The Agua Fria is a braided ephemeral stream, and is quite unstable.

2. The Agua Fria has experienced overall net degradation and a decrease in top width throughout the study reach, which is uncharacteristic of braided streams as they usually tend to aggrade. This is at least in part due to (1) constriction of flow due to encroachment by floodplain gravel mines, (2) probable net extraction of coarse material from the channel due to sand and gravel mines.

3. The instream gravel pit located 600 feet downstream of the Indian School Road Bridge and subsequently partially filled with a lower quality



EXPLANATION

Sedimentary Rocks

Qs	Silt, sand and gravel
QTs	Sand, gravel and conglomerate
Ts	Sand, gravel and conglomerate
Tl	Lake deposits

Igneous Rocks

Qb	Basalt
QTb	Basalt
Ki	Dikes and plugs
Ka	Andesite
gr	Granite and related crystalline rocks
di	Diorite porphyry
sch	Schist
rhy	Red Rock ryholite
gs	Greenstone
gn	Granite gneiss

SCALE

0 5

miles

Contour interval: 500 feet m.s.l.

Figure 3.4. Geologic map of part of the Aqua Fria River Basin, New Mexico. (from Wilson et. al. 1975)

material in 1973 by the Phoenix Sand and Rock Co., the levees constricting the flow, and a possible net loss of armoring material due to mining activities probably caused a headcut to occur.

4. Local scour at ISRB piers was aggravated in the 1980 flood by the migration of a low-flow channel just upstream of the bridge. This migration was predominantly natural but was confined by the bridge.

5. Based upon SLA's analysis of the limited bed material samples, it is concluded that there has been a loss of coarser material over the years due to a combination of headcutting, general and local scour and subsquent deposition of sand during the recession of flood events.

6. At least 15 feet of material has been scoured out and replaced with finer looser material around several of the bridge piers.

7. The effects due to upstream development on the sediment transport characteristics of the Agua Fria River near ISRB are negligible.

8. There is no evidence of geologic controls present in the lower Agua Fria Valley to control the bed elevation of the Agua Fria.

IV. QUANTITATIVE GEOMORPHIC ANALYSIS OF BRIDGE FAILURE

This analysis quantifies and verifies geomorphic responses of the system and analyzes related aspects of the system that were not obvious when the system was subjected to the initial qualitative geomorphic analysis.

4.1 Predicted Channel Response

By applying the HEC-2 backwater profile computer program for various discharges and plotting the results, useful information about the system is obtained. The response of the water-surface profile for the conditions being evaluated can be determined. In addition, cross sections with similar hydraulic properties may be grouped together for the analysis of reaches. This approach is necessary since the sediment routing model for determining degradation and aggradation routes sediment by reaches. Figure 4.1 gives the reach definitions in terms of cross-sections and river distance.

From the HEC-2 hydraulic plots the expected aggradation or degradation within a reach can be qualitatively determined. Plots of top width vs. river distance, velocity vs. river distance, and depth vs. river distance for the 2-yr, 10-yr, 100-yr and February 20, 1980 flood peaks for mining and pre-mining conditions were made as shown in Figures 4.2-4.13. As stated earlier the mining condition considers levees between Indian School Road Bridge (ISRB) and Roosevelt Irrigation District (RID) flume and no instream gravel pits, while the premining conditions considers no instream or floodplain gravel pits.

Aggradation or degradation within a reach is related to changes in top width and the fourth power of a change in velocity as compared to the adjacent upstream reach. For instance if the topwidth decreases and the velocity increases the reach will degrade. However, if the topwidth decreases and the velocity decreases the reach may aggrade. The other two possibilities; top-width increasing and velocity increasing; and topwidth increasing and velocity decreasing indicate degradation and aggradation, respectively. Table 4.1 gives the results of this analysis for each reach for the premining condition. Table 4.2 gives the results for the mining condition. The most significant effect of channel encroachment considering the mining condition occurs in reaches four through six where the channel is fairly stable and has been changed to a degrading mode. Reaches 4 and 5 show strong degradation tendencies. This could initiate a headcut that would move upstream and

Section No.	River Distance (ft. above Section 7.88)	Reach No.	River Distance (ft)	Features
			0	
7.88	0	6		
8.04	830		1,180	
8.18	1,530	5		Roosevelt Irrigation District Flume
8.21	1,710		1,995	
8.32	2,280			
8.41	2,760	4		
8.50	3,240		3,375	
8.60	3,780	3		Indian School Road Bridge
8.65	3,860		4,190	
8.72	4,360			
8.93	5,460	2		
9.24	7,060			
9.50	8,430		9,110	
9.74	9,790			
9.90	10,990	1		
10.10	12,050		12,050	

Figure 4.1. Schematic diagram of reaches in study area.

TOP WIDTH VS. RIVER DISTANCE AGUA FRIA 2 YR FLOOD PEAK

FIGURE 4.3

VELOCITY VS. RIVER DISTANCE

AGUA FRIA

2 YEAR FLOOD PEAK

VELOCITY VS. RIVER DISTANCE AGUA FRIA 2 YR FLOOD PEAK

DEPTH VS. RIVER DISTANCE AGUA FRIA 2 YR FLOOD PEAK

FIGURE 4.4
DEPTH VS. RIVER DISTANCE
AGUA FRIA
2 YEAR FLOOD PEAK

FIGURE 4.5
TOP WIDTH VS. RIVER DISTANCE
AGUA FRIA
10 YEAR FLOOD PEAK

TOP WIDTH VS. RIVER DISTANCE AGUA FRIA 10 YR FLOOD PEAK

FIGURE 4.6
VELOCITY VS. RIVER DISTANCE
AGUA FRIA
10 YEAR FLOOD PEAK

VELOCITY VS. RIVER DISTANCE AGUA FRIA 10 YR FLOOD PEAK

FIGURE 4.7

DEPTH VS. RIVER DISTANCE

AGUA FRIA

10 YEAR FLOOD PEAK

DEPTH VS. RIVER DISTANCE AGUA FRIA 10 YR FLOOD PEAK

FIGURE 4.8
TOP WIDTH VS. RIVER DISTANCE
AGUA FRIA
100 YEAR FLOOD PEAK

TOP WIDTH VS. RIVER DISTANCE AGUA FRIA 100 YR FLOOD PEAK

FIGURE 4.9

VELOCITY VS. RIVER DISTANCE

AGUA FRIA

100 YEAR FLOOD PEAK

VELOCITY VS. RIVER DISTANCE AGUA FRIA 100 YR FLOOD PEAK

DEPTH VS. RIVER DISTANCE AGUA FRIA 100 YR FLOOD PEAK

TOP WIDTH VS. RIVER DISTANCE AGUA FRIA 42000 CFS PEAK FLOW

VELOCITY VS. RIVER DISTANCE AGUA FRIA 42000 CFS PEAK FLOW

FIGURE 4.13

DEPTH VS. RIVER DISTANCE

AGUA FRIA

42,000 CFS PEAK FLOW

DEPTH VS. RIVER DISTANCE AGUA FRIA 42000 CFS PEAK FLOW

Table 4.1. Expected Qualitative Response of Reaches Based on HEC-2 Hydraulics - Premining Condition.

Reach	Change in Topwidth 42*	Change in Topwidth 100*	Change in Velocity 42	Change in Velocity 100	Overall Response 42	Overall Response 100
1	Supply	Supply	Supply	Supply	Supply	Supply
2	Decrease	Decrease	Increase	Increase	Degrades	Degrades
3	Decrease	Decrease	Increase	Increase	Degrades	Degrades
4	Increase	Increase	Decrease	Decrease	Aggrades	Aggrades
5	Increase	Decrease	Increase	Decrease	Slightly Degrades	Slightly Aggrades
6	Increase	Increase	Decrease	Decrease	Aggrades	Aggrades

* 42 Denotes a discharge of 42,000 cfs
100 Denotes a discharge of 100,000 cfs

Table 4.2. Expected Qualitative Response of Reaches Based on HEC-2 Hydraulics - Mining Condition.

	Change in Topwidth		Change in Velocity		Overall Response	
Reach	42*	100*	42	100	42	100
1	Supply	Supply	Supply	Supply	Supply	Supply
2	Decrease	Decrease	Increase	Increase	Degrades	Degrades
3	Decrease	Decrease	Decrease	Increase	Slightly Aggrades	Degrades
4	Decrease	Decrease	Increase	Increase	Degrades	Degrades
5	Slight Increase	Slight Increase	Increase	Increase	Slightly Degrades	Slightly Degrades
6	Slight Decrease	Slight Decrease	Increase	Increase	Degrades	Degrades

* 42 Denotes a discharge of 42,000 cfs
100 Denotes a discharge of 100,000 cfs

endanger the bridge. Quantitative assessment of this headcut potential is presented in a later section.

The water-surface profiles for the mining and premining condition using the 1973 thalweg profile and the 100-year flood (Q = 100,000 cfs) are plotted in Figure 4.14 along with the approximate elevation of the floodplain boundary from the Corps of Engineers 1973 topographic maps. These plots indicate that the encroachment of the gravel mining levees in the channel produces a significant increase in the water-surface elevation throughout the majority of the study reach. The water-surface elevation for the mining condition exceeded the 100-year floodplain boundary elevation by more than one foot from approximately 1500 feet downstream of the RID flume crossing to approximately 4000 feet upstream of ISRB. The levees caused an increase of nearly ten feet in the water surface in some areas between ISRB and the RID flume crossing. It should be noted that these profiles do not account for the aggradation or degradation that occurs during the flood. For the premining condition, the indicated aggradation in Reaches 4 and 5 will cause some increase in the water-surface elevation. For the mining condition, degradation is indicated in these reaches, resulting in a slight lowering of the water surface. The magnitude of the changes in the water-surface elevation will be relatively small compared to the depth of aggradation or degradation. For this reason, the profiles plotted in Figure 4.14 are considered to be representative of the true profiles.

4.2 Determination of Total Scour at Indian School Road Bridge and the Roosevelt Irrigation District Flume

The total scour is divided into three components: (1) the local scour at the bridge due to acceleration of the flow near the bridge piers, (2) general regionalized scour due to contraction of the flow and (3) the general aggradation and/or degradation that occurs in the system. These components are discussed in the following sections.

4.2.1 Local Scour at Indian School Road Bridge and Roosevelt Irrigation District Flume

To compute the local scour, an armoring analysis utilizing the Shields incipient motion procedure is used. In general, as the flow passes through the bridge it will accelerate near the piers resulting in a local scour. As

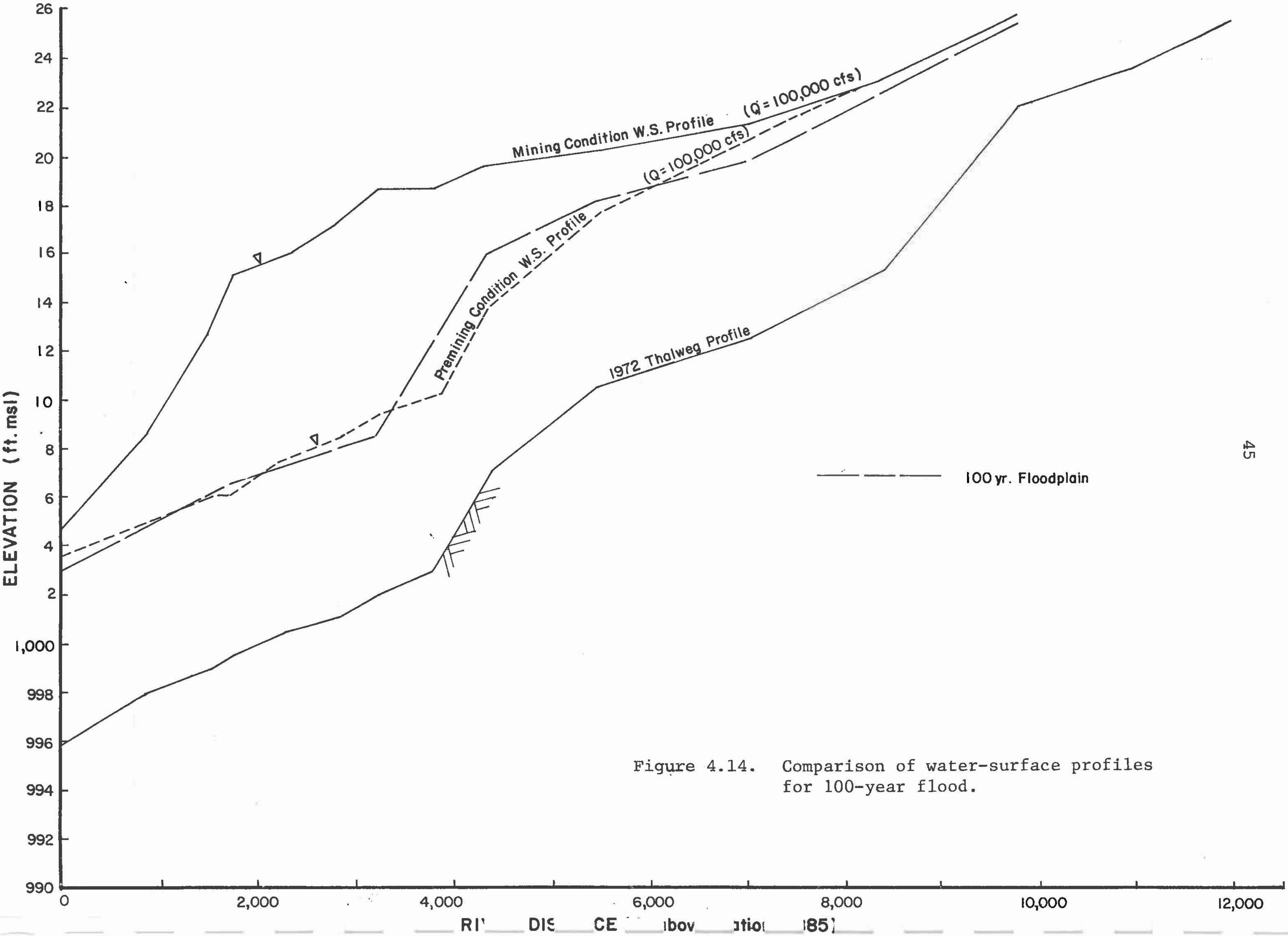


Figure 4.14. Comparison of water-surface profiles for 100-year flood.

time passes during the passage of a flood, the finer particles of the bed material are eroded from the bed, leaving the coarser material that is unable to move. When enough of the coarse material accumulates on the bed, and if the velocities in the channel are not large enough to move these coarser particles an armoring layer forms. For local scour computations at ISRB the average subsurface grain size distribution presented in Section 2.3 is used. In SLA's estimation, this distribution is conservative based on size of material presently being mined from the Agua Fria floodplain near the flume.

The method for computing local scour at the piers requires knowledge of the water discharge per unit width through the bridge piers 14, 15 and 16 where local scour was the most severe. Two approximations were considered for determining the unit width discharge, which included (1) estimating the angle of attack on individual piers from examining aerial photographs of the flow, and (2) estimating the average angle of attack on the piers to be approximately 41 degrees.

From examination of the aerial photographs of the February 20, 1980, flood, it is apparent that the flow at cross section 8.93, which is located 1600 feet upstream of the bridge, necks down considerably at the bridge. Shown in Figure 4.15, 1575 feet west of the east bank to the east bank of cross section 8.93 is the area in which approximately 65 percent of the flow passes for the mining condition. Drawing streamlines from section 8.93 to the bridge, 65 percent of the flow passes through a width of 220 feet at the bridge.

Examining the individual angle of attack of flow on piers 14, 15 and 16 reveals that they were 35, 45 and 56 degrees, respectively. Accounting for the 11 degree skew the piers make with flow, the actual angle of attack on bridge piers 14, 15 and 16 was 24, 34 and 45 degrees, respectively. The effective flow width through these bridge piers is 110 feet, and 65 percent of 42,000 cfs, or 27,300 cfs flows through the 110 foot opening resulting in a unit discharge of 250 cfs/ft.

For the second estimate, the average angle of attack on bridge piers 14, 15 and 16 was 41 degrees. Subtracting the 11-degree angle which the piers make with the flow, the actual angle of attack was 30 degrees. This results in an effective flow width of 120 feet through the bridge piers. Thus when 65 percent of 42,000 cfs, or 27,300 cfs flows through a 120-foot opening, the

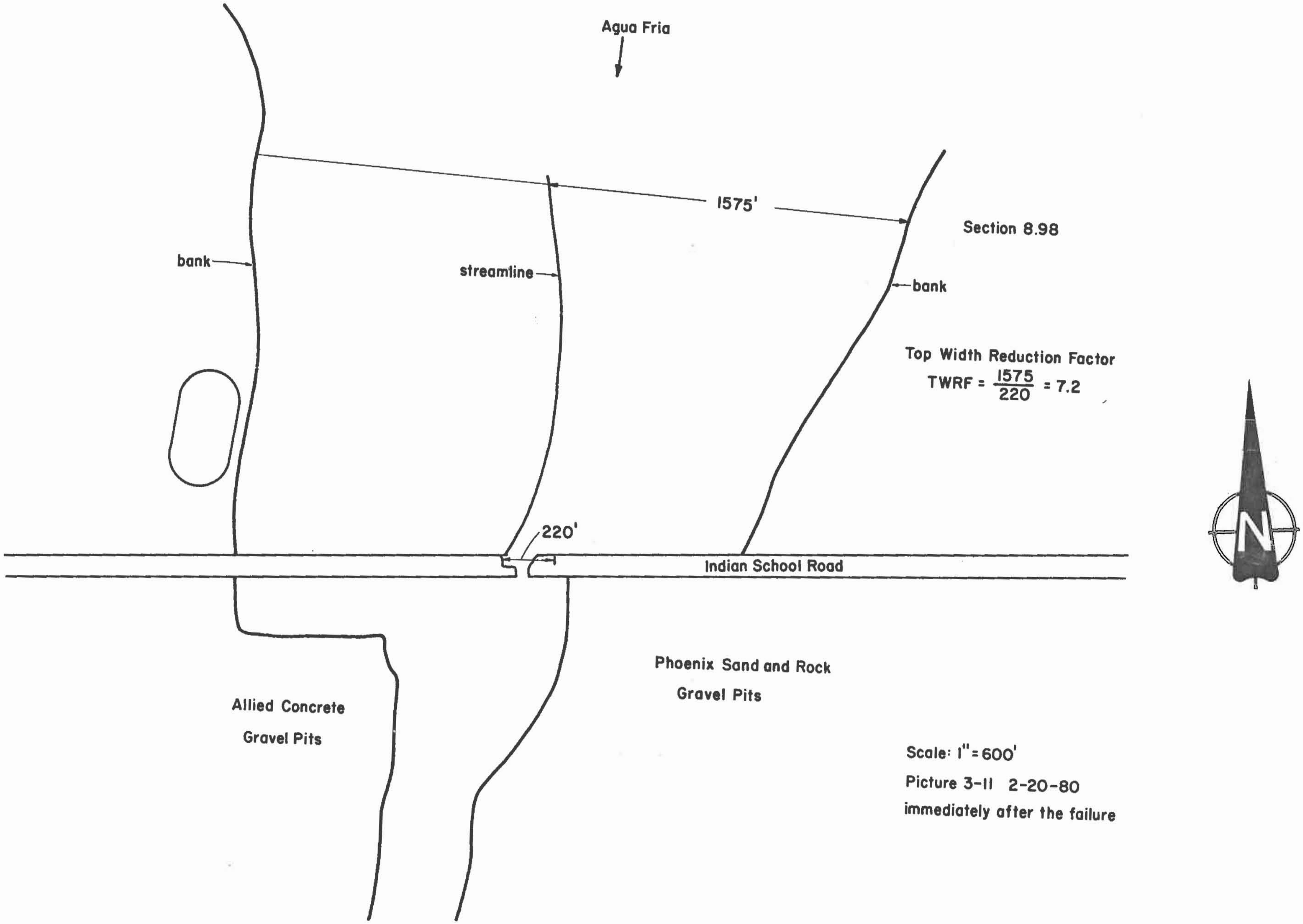


Figure 4.15. Flow convergence above Indian School Road Bridge.

resulting unit discharge is 225 cfs. For local scour computations for the mining condition unit discharges of 225 cfs and 250 cfs were evaluated.

For the premining condition, approximately 60 percent of the flow is contained in the 1575-foot width upstream of Indian School Road at cross section 8.93. Thus using the skew angles of 24, 34 and 45 degrees on piers 14, 15 and 16, the unit width discharge becomes 230 cfs/ft. Assuming the flow necks down at the bridge using the average angle of attack of 30 degrees, the unit width discharge becomes 210 cfs/ft at the bridge. Thus a range between 210 cfs/ft and 230 cfs/ft unit width discharges was examined for local scour computations previous to mining operations.

The unit discharge at the RID flume crossing was assumed to be distributed uniformly throughout the flume opening. The effective flume opening for the levee condition was much smaller than the natural condition with levees. The unit discharge for the Agua Fria with levees for the 42,000 cfs peak flow was 75 cfs/foot. For the condition with no levees the unit discharge for the 42,000 cfs peak was 15 cfs/foot.

Table 4.3 summarizes the local scour expected at ISRB and the RID flume crossing for the 42,000 cfs flood peak for the conditions considering both no gravel pits and gravel pits. Also included in Table 4.3 are the comparisons of local scour using Neil and Shen's formula (assuming that the correction factor for the angle of attack is 5.0) for the 42,000 cfs discharge. The local scour depths given by the three methods of computation are approximately the same at the bridge and flume. The local scour considering armoring at bridge piers 14, 15 and 16 was approximately 20.2 feet for the mining condition and 21.1 feet for no gravel mining. Assuming that the bed material sizes at the above 95 percentile are approximately 50 percent coarser for the premining condition, the local scour would be 17.3 feet for no gravel mining. The local scour considering armoring at the flume piers for the mining condition was 7.9 feet and for the premining condition was 2.1 feet. For the local scour at the flume piers, for the premining condition, Shen and Neil's equations are considered more applicable as the amount of armoring material present at the two-foot depth is not considered great enough to control scour.

In Shen and Neil's methods debris accumulation at the bridge piers is accounted for by accounting for the angle of attack. The severe angle of attack at the bridge causes the piers to be exposed to a significant percent of the flow. Hence, the debris that does accumulate at the upstream face of the piers has a small effect.

Table 4.3. Local Scour at Indian School Road and Roosevelt Irrigation District Flume.

Location	Mining or Premining Condition	Total Discharge (cfs)	Unit Discharge (cfs/ft)	Armor Local Scour (ft)	Shen Local Scour (ft)	Neil Local Scour (ft)	Recommended Local Scour (ft)
Indian School Road	Mining	42,000	250	20.2	20.6	20.1	20.2
Indian School Road	Mining	42,000	225	17.5			
Indian School Road	Premining	42,000	230	21.1	22.8	20.3	21.1
Indian School Road	Premining	42,000	210	18.9			
Roosevelt Irrigation District	Mining	42,000	75	7.9	9.6	8.8	7.9
Roosevelt Irrigation District	Premining	42,000	25	2.1	7.2	6.7	7.2

4.2.2 General Regional Scour

General regional scour at contractions occurs because the effective flow area is reduced by the dikes. This increases 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 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.

For the mining condition the expected general regional scour is larger than the expected general regional scour for the premining condition because the levees constrict the flow on the downstream side of the bridge and at the bridge for the mining condition. To determine the general regional scour in the vicinity of the bridge the principles of water and sediment continuity are utilized. The hydraulics for the peak flow of 42,000 cfs were computed from the HEC-2 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 due to the levees was 6.4 feet. The general regional scour computed for the premining condition was 0.5 ft.

4.2.3 Aggradation/Degradation Analysis

The aggradation/degradation analysis for the reach between the flume and the bridge was computed utilizing the equilibrium slope method and the armoring control method. Both of these methods are discussed below. The methods are compared to determine which controls the lowest bed elevation.

4.2.3.1 <u>Equilibrium Slope Method to Determine Degradation or Aggradation</u>

The equilibrium channel slope is defined as the slope at which the channel's sediment transporting capacity is equal to the incoming sediment supply. Under this condition, the channel neither aggrades nor degrades. When the present slope of a channel is greater than the equilibrium slope, the channel will degrade in order to reach its equilibrium slope.

Calculation of the sediment tranpsort rate involved using reach 2 (see Figure 4.1), which is located upstream of the ISRB, as the supply reach and computing the new equilibrium slope for reach 4, which is located between the flume and bridge.

The original bed slope in 1973, between the bridge and the flume, was 0.0015 ft/ft. For the mining condition, the sediment transporting capacity through reach 4 is large for the mining condition due to the constriction of the channel. The supply from reach 2 is less than the sediment transporting capacity of reach 4 therefore a new equilibrium slope is achieved. The new slope for reach 4 is 0.0003 ft/ft. Thus by pivoting the slope about a point near the flume a potential degradation of 2.6 feet is possible at the ISRB for the levee condition.

For the premining condition the same equilibrium slope method was applied to reach 4 to determine the degradation potential. The new equilibrium slope computed was 0.0017 indicating reach 4 would aggrade. This conclusion agrees with the qualitative prediction for reach 4 presented in section 4.1. Pivoting the new equilibrium slope about the flume the potential aggradation at the bridge is 0.5 ft.

4.2.3.2 Armor Control Method to Determine Potential Headcut

For the mining condition the Shield's incipient motion analysis was applied to determine the smallest non-moving particle size between the flume and bridge. The incipient motion analysis was conducted for the 42,000 cfs peak flow at reach 4. The smallest non-moving particle was 3.2 inches in diameter, which is greater than any of the sampled size distributions, therefore it is expected armoring will not control degradation between the flume and bridge for the mining condition.

For the premining condition the equilibrium slope method shows aggradation so armoring will not be a factor for this condition. Armoring computations were not utilized for the premining condition.

4.2.3.3 Aggradation/Degradation Controls

For the premining and mining conditions the equilibrium slope will control the grade of the Agua Fria. For the mining condition a potential degradation of 2.6 feet exists, and for the premining condition the channel aggrades 0.5 ft.

The question then arises, "Could the equilibrium slope conditions be reached in the February 20, 1980 flood event?" The answer is yes. By comparing the sediment transport rates upstream of the bridge and those between the flume and the bridge for the three flow events between 1978 and 1980 a sediment volume imbalance of 26.54×10^6 cubic feet occurred. When this volume of sediment is divided by the length and average width of the Agua Fria between ISRB and RID flume crossing a potential degradation exceeds the sediment volume needed to achieve 2.6 feet of degradation in this reach.

4.3 Breakdown of Total Scour Components at Indian School Road Bridge and Roosevelt

The total scour at the bridge and flume can be broken down into three components; local scour, regionalized general scour and net overall aggradation/degradation. Table 4.4 summarizes each component of the total scour for mining and premining conditions at Indian School Road Bridge. The total computed scour of 29.2 feet for the mining condition matches the measured total scour of 29 feet at piers 14, 15 and 16.

As mentioned earlier, the local scour computations presented are based on sediment size distributions that may have been affected by mining. Mining could have created a finer sediment distribution in the area of the Indian School Road Bridge. Assuming an increase in the sediment distribution so that 3 percent of the material is 2.5 inches or larger, rather than 3 percent of the material being 1.5 inches or smaller as was the assumption in the previous case, the local scour potential is reduced to 17.3 feet for the premining condition. This produces a total scour of 17.3 feet for the February 20, 1980 flood rather than 21.1 feet. This illustrates additional harmful effects gravel mining could have had on the Indian School Road Bridge.

The components of total scour at the RID flume are listed in Table 4.5. The degradation of 5.5 feet for the mining condition is the measured difference in the base level between 1973 to 1980. The regionalized general scour was considered negligible for the premining condition because the original channel width equaled 5959 ft under the flume. The degradation at the flume for the premining condition was considered negligible because the equilibrium slope analysis for this reach showed the flume area was in the aggrading mode.

Table 4.4. Scour Components at Indian School Road Bridge for Mining and Premining Conditions for the February 20, 1980 Flood Event.

	Mining (ft)	Premining (ft)
Local Scour (ft)	20.2	21.1
Regionalized General Scour (ft)	6.4	0.5
Aggradation/Degradation (ft)	2.6 (deg)	0.5 (agg)
Total Scour (ft)	29.2	21.1

Table 4.5. Scour Components at Roosevelt Irrigation District Flume for Mining and Pre-Mining Conditions for the February 20, 1980 Flood Event.

	Mining	Pre-Mining
Local Scour (ft)	7.9	7.2
Regionalized General Scour (ft)	6.4	Negligible
Aggradation/Degradation (ft)	5.5 (deg)	Negligible
Total Scour (ft)	19.8	7.2

In the report prepared by Engineers Testing Laboratories, Inc. for the Roosevelt Irrigation District dated April 15, 1980, the scour below the river bed was estimated to be 14 to 18 feet. Thus adding the local scour of 7.9 feet and the regionalized general scour of 6.4 feet, the scour depth computed by SLA of 14.3 feet is in the range of scour estimated by Engineers Testing Laboratories.

4.4 Conclusions

1. From qualitative interpretation of the plots of topwidth vs. river distance, and flow velocity vs. river distance, the Agua Fria between ISRB and RID flume crossing was in an aggrading mode for the premining condition and has changed to a degrading mode for the mining condition.

2. The water-surface elevation for the mining condition exceeded the 100-year floodplain boundary elevation by more than one foot for approximately 7700 feet. The levees caused an increase of nearly 10 feet in the water surface in some areas between the ISRB and RID flume crossing.

3. The local scour at the bridge near piers 14, 15 and 16 was computed to be 20.2 feet for the mining condition and 21.1 feet for the premining condition. The local scour at the RID flume was computed to be 7.9 feet for the mining condition and 7.2 feet for the premining condition.

4. The general regional scour at the bridge for the mining condition was 6.4 feet and was 0.5 foot for the premining condition. The general regional scour at the RID flume for the mining condition was 6.4 feet and was negligible for the premining condition.

5. The potential degradation at the ISRB for the mining condition was 2.6 feet based on the equilibrium slope method. The equilibrium slope could have been reached during the February 20, 1980 flood because of the large imbalance of sediment transport rate from upstream of the bridge to downstream of the bridge. For the premining condition an aggradation at the bridge of 0.5 feet was computed based on the equilibrium slope method. At the flume the difference in the 1973 and 1980 thalweg profiles show a net degradation of 5.5 feet which was assumed for the mining condition and the aggradation/degradation at the flume is considered negligible for the premining condition based on the results of the equilibrium slope method applied to this reach.

6. The total scour depth for the mining condition at the ISRB was computed to be 29.2 feet at bridge piers 14, 15 and 16, which compares well with the measured total scour of 29 feet. For premining conditions the total computed scour at bridge piers 14, 15 and 16 was 21.1 feet.

7. The scour depth measured below the bed at the RID flume by Engineers Testing Laboratories ranged from 14 to 18 feet. Computed scour near the flume piers was 14.3 feet for the mining condition and 7.2 feet for the premining condition.

V. APPLICATION OF MATHEMATICAL MODEL TO DETERMINE AGGRADATION/DEGRADATION ASSOCIATED WITH BRIDGE FAILURE

5.1 General

To determine the general response of the river due to gravel mining activities, water and sediment routing was performed using QUASED, a quasidynamic 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-2 water surface profile program.

5.2 General Model Concept

The amount of material transported or deposited in a channel reach is the result of the interaction of two processes. The first is the transport capacity of the reach. This is determined in part by the hydraulic conditions which are a direct result of the water discharge, channel configuration, channel resistance and the sediment sizes present. Smaller particles can be transported at larger rates than larger particles under the same flow conditions. The second process is the supply of sediment entering the reach. This is determined by the nature of the channel and watershed above the study reach.

When sediment supply is less than sediment transport, sediment is removed from the channel bed and banks to reduce the difference. This results in degradation of the channel and possible failure of the banks. If the supply entering the reach is greater than the capacity, the excess supply is deposited, causing aggradation.

5.2.1 Sediment Transport Capacity

Transport of the bed material load of a channel is divided into two zones. The sediment moving in a layer close to the bed is referred to as the bed load. The sediment carried in the remaining upper region of the flow is referred to as suspended load. The total bed material load is the sum of the two quantities. The turbulent mixing process and the action of gravity on the sediment particles cause a continual transfer between the two zones. Although there is no distinct line between the zones, the definitions are made in order to aid in the mathematical description of the process. A third type of load, the wash load, is also defined. It consists of fine particles that are not present in the bed in appreciable quantities, and will not easily settle out.

Sediments of different sizes experience different rates of transport. Therefore, the transport capacities for a range of sediment sizes are determined and totaled to produce an acceptable determination of total transport capacity.

For this study, the Meyer-Peter, Muller formula was used to calculate the bed load while the suspended load was determined using a solution developed by Einstein (1950) which is based on integration of the sediment concentration profile over the depth of flow.

5.2.2 Sediment Routing Procedure

The sediment routing procedure is quasi-dynamic where the flow is assumed constant for a given time increment but varies from subreach to subreach. The flood event is broken into a number of time increments, each with a different flow, but during each increment the flow is considered steady. To account for the moveable nature of the alluvial boundary, the cross sections are recomputed at the end of each time interval. Sediment transport by size fraction is determined for the overbanks and main channel portions of the cross section then summed to give the total transport capacity within a subreach.

The volume aggradation or degradation within a subreach is computed as a function of the difference between the sediment inflow from upstream and the transport capacity of the subreach. This volume is translated to a change in bed elevation at each cross section which is used to generate new HEC-2 data for the next time step.

5.2.3 Armoring

For this study the particle size range is large, necessitating the consideration of the armoring process for realistic determination of the river response.

The QUASED model determines the transport capacity of the channel by size fractions. This not only provides for more accuracy in determining the sediment discharge, but also allows for simulation of the variation in the particle size distribution during the degradation or aggradation process. If the channel degrades and particles too large to be transported by the flow are present in the bed material, the finer particles will be removed, leaving behind the larger particles and producing a layer of essentially non-transportable material (the armor layer). When this occurs, the amount of degradation in the channel is controlled by the quantity of large particles present.

5.3 Sediment Routing Results

5.3.1 General

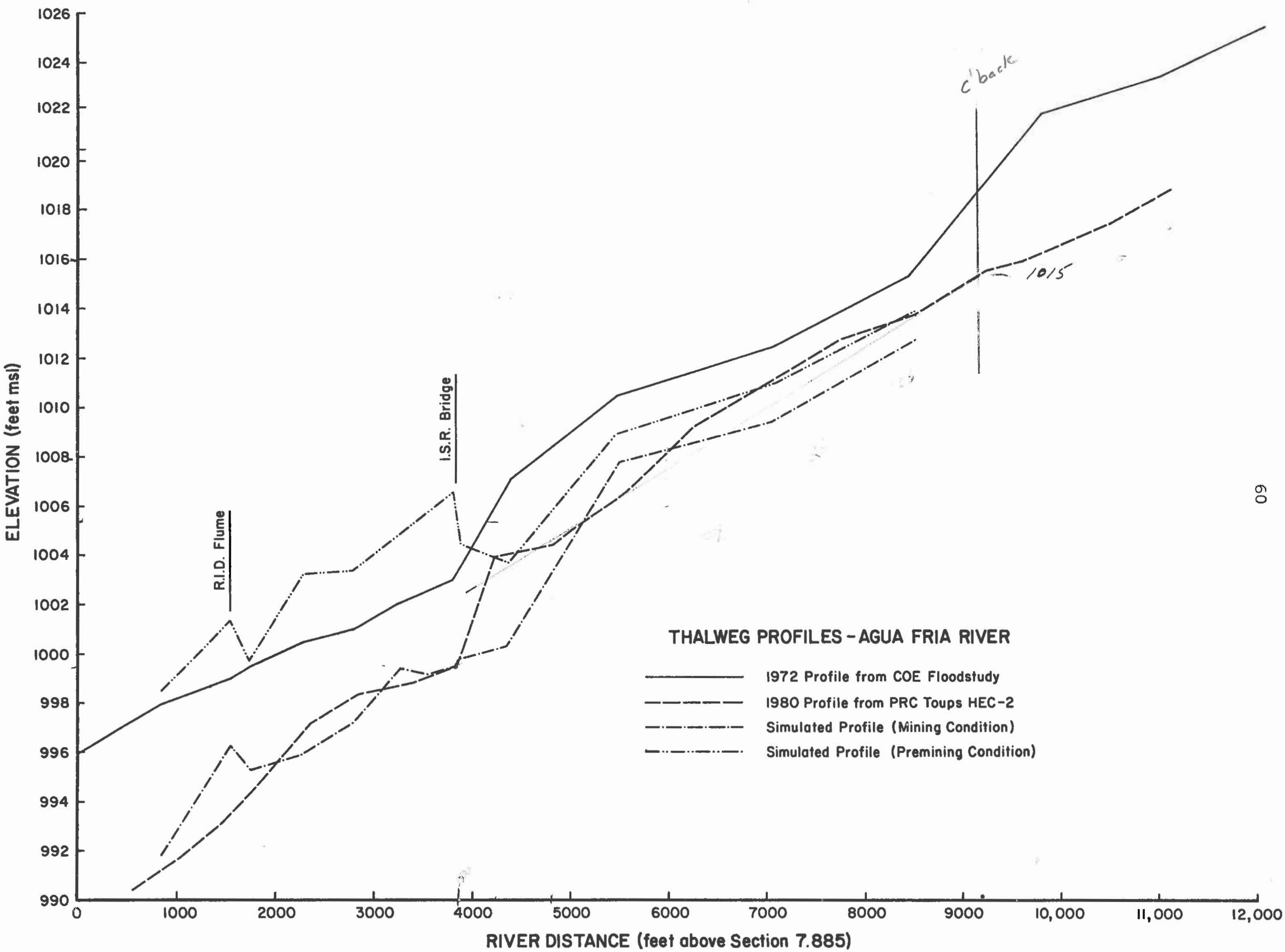
The general response of the river to the gravel mining activities was evaluated by performing sediment routing using the storm hydrographs for 1978-1980 with two channel conditions. The first simulated the mining condition with levees in the downstream reaches. The second simulated the premining condition with the levees removed.

5.3.2 Mining Condition Results

Thalweg profiles for the study reach are plotted in Figure 5.1. The profile resulting from the sediment routing for the mining condition is consistent with the actual 1980 profile indicating that the model simulated the actual river response very well. The routing results indicate significant general degradation throughout the study reach for this condition.

Figures 5.2 and 5.3 show the variation in thalweg elevation with time at Indian School Road Bridge and Roosevelt Irrigation District flume crossing. These figures clearly indicate the accelerated general degradation at the bridge and flume during the peaks of the storm hydrographs. It also shows the expected tendency to redeposit material during the receding portions of the hydrographs. As indicated in Figure 5.2, the maximum general scour of approximately 4.2 feet occurred soon after the peak of the January-February, 1980, storm. From Figure 5.3, the maximum general degradation at the RID flume occurred at approximately the same time and had a magnitude of about 4.4 feet.

The bed material size distribution curves at various times in the simulation are shown in Figure 5.4. These curves show the coarsening of the bed material as the degradation took place, indicating that the armoring process was modeled quite well. During the subsequent recession of the flood, the bed-material sizes lessened, indicating the deposition of sand during this time.



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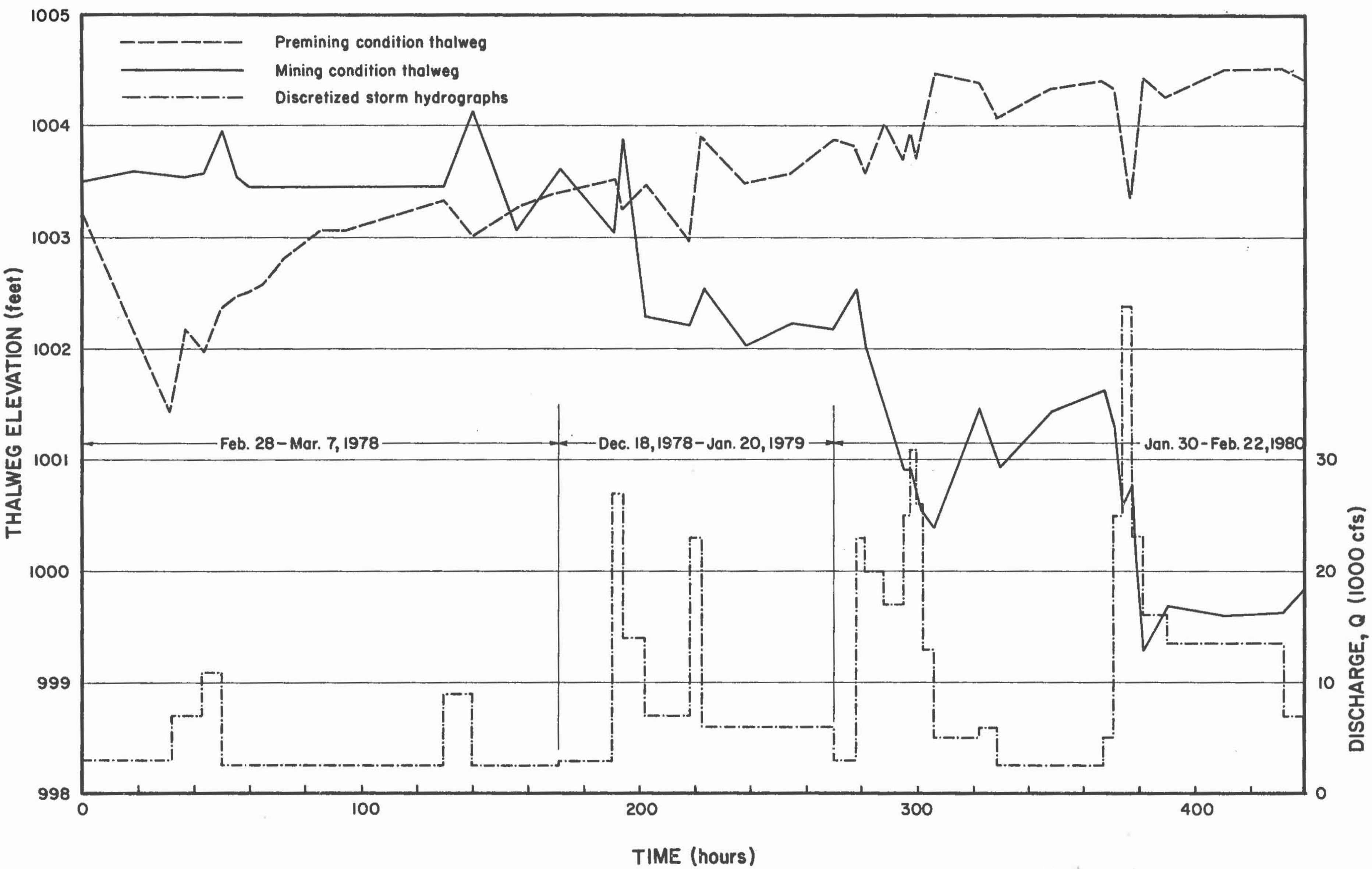


Figure 5.2. Change in thalweg elevation at Indian School Road Bridge for 1978-1980 flood events for mining and premining conditions.

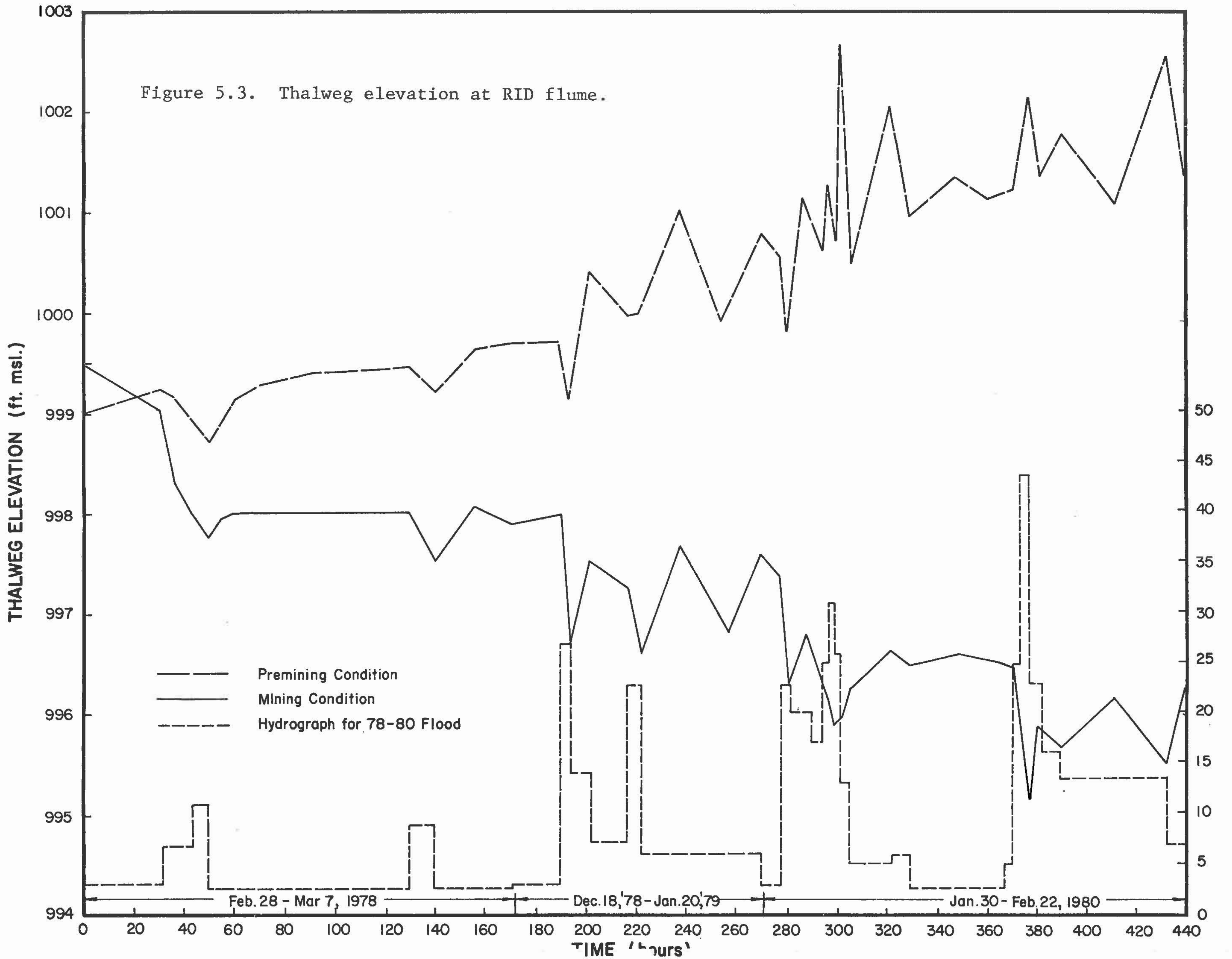


Figure 5.3. Thalweg elevation at RID flume.

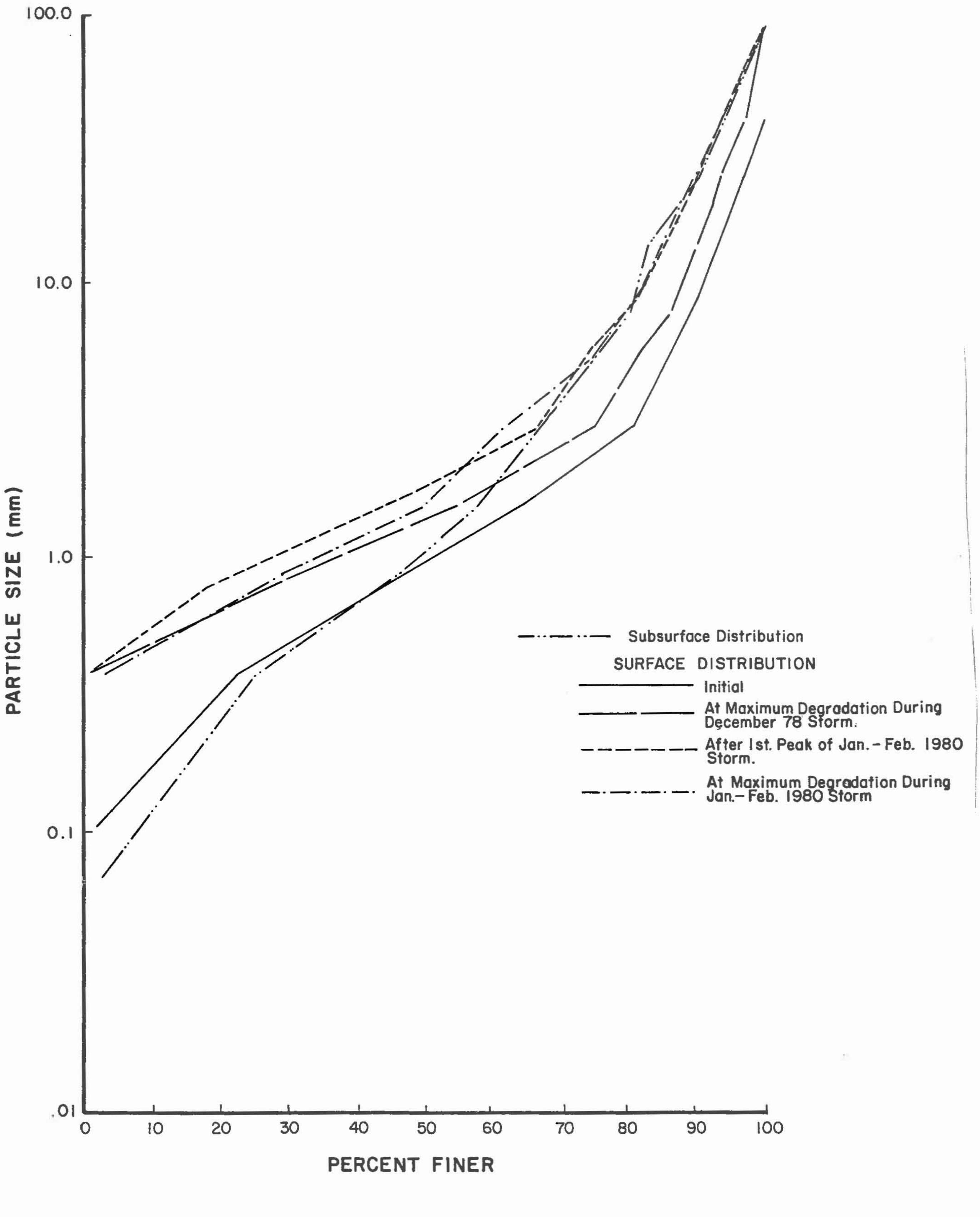


Figure 5.4. Particle size distribution at ISRB.

5.3.3 Premining Condition Results

Removal of the levees surrounding the gravel mining sites in the downstream portion of the study reach had a significant impact on the response of the river to the storm hydrographs. Referring again to Figure 5.1, it can be seen that, while some general degradation occurred upstream of Indian School Road Bridge, the river aggraded in the vicinity and downstream of the bridge.

The plot of the thalweg elevation with time at ISR Bridge and the RID flume shown in Figures 5.2 and 5.3 indicates the general trend of aggradation throughout the simulation.

5.3.4 Conclusion

Comparison of the sediment routing results for the two conditions clearly indicates that the gravel mining levees had a significant impact on the general response of the river within the study reach. With the levees in place, significant degradation occurred throughout the reach. Removal of the levees resulted in aggradation at and downstream of the bridge.

The model has the limitation of not modeling the local scour and general regional scour at the bridge. By adding the local scour of 20.2 ft computed for the mining condition with the general regionalized scour of 6.4 ft, the total scour at the bridge is 30.8 ft. Adding these three components for the premining condition results in a total scour depth of 21.1 ft.

VI. DESCRIPTION OF ALTERNATIVES FOR RECONSTRUCTION OR REPLACEMENT OF THE BRIDGE

Three alternatives for reconstruction or replacement of the bridge were selected for evaluation in this study. The first is the existing channel configuration with the channel geometry obtained from a 1980 survey conducted by PRC Toups. This survey was conducted after the February, 1980 flood. This alternative is simply a "do nothing" alternative for channel modification. The remaining two include the design alternative previously proposed by PRC Toups (1981) and the design alternative proposed by SLA from this study.

6.1 Do Nothing Alternative (As-Is Condition)

The natural channel of the Agua Fria River is several thousand feet wide upstream of the Indian School Road Bridge (ISRB) and the necks dam to only 400 to 500 feet just downstream of the bridge due to a levee system constructed by sand and gravel operators on both the east and west sides of the channel. The present channel configuration has existed since 1975. Figure 6.1 shows the basic layout of the Agua Fria River as it now exists in the area of the Indian School Bridge and the Roosevelt Irrigation District (RID) flume crossing.

6.2 PRC Toups Design Alternative

PRC Toups, in their January 1981 report discussing the ISRB at the Agua Fria River, presented a design alternative for stabilization of the channel and protection of the bridge. At the time that PRC Toups conducted ISRB analysis, stability of the RID flume crossing was not considered. Consequently, the PRC Toup's design included channelization only through the bridge opening and not through a proper channel width at the RID flume. The channelized reach extends from approximately 850 feet upstream of the bridge to the flume. This design allows the total width of 1600 feet at the bridge to be used as effective flow area.

Upstream of the bridge, PRC Toups proposed a spur dike and a transverse dike which would hopefully guide the flow properly into the bridge without creating an adverse angle of attack. The spur dike extends from the east abutment of the bridge upstream 400 feet. The transverse dike is located approximately 1400 feet upstream of the bridge on the east bank and it guides the flow to the west before it approaches the bridge opening. Another levee was proposed for the west bank of channel above the bridge. This levee, if

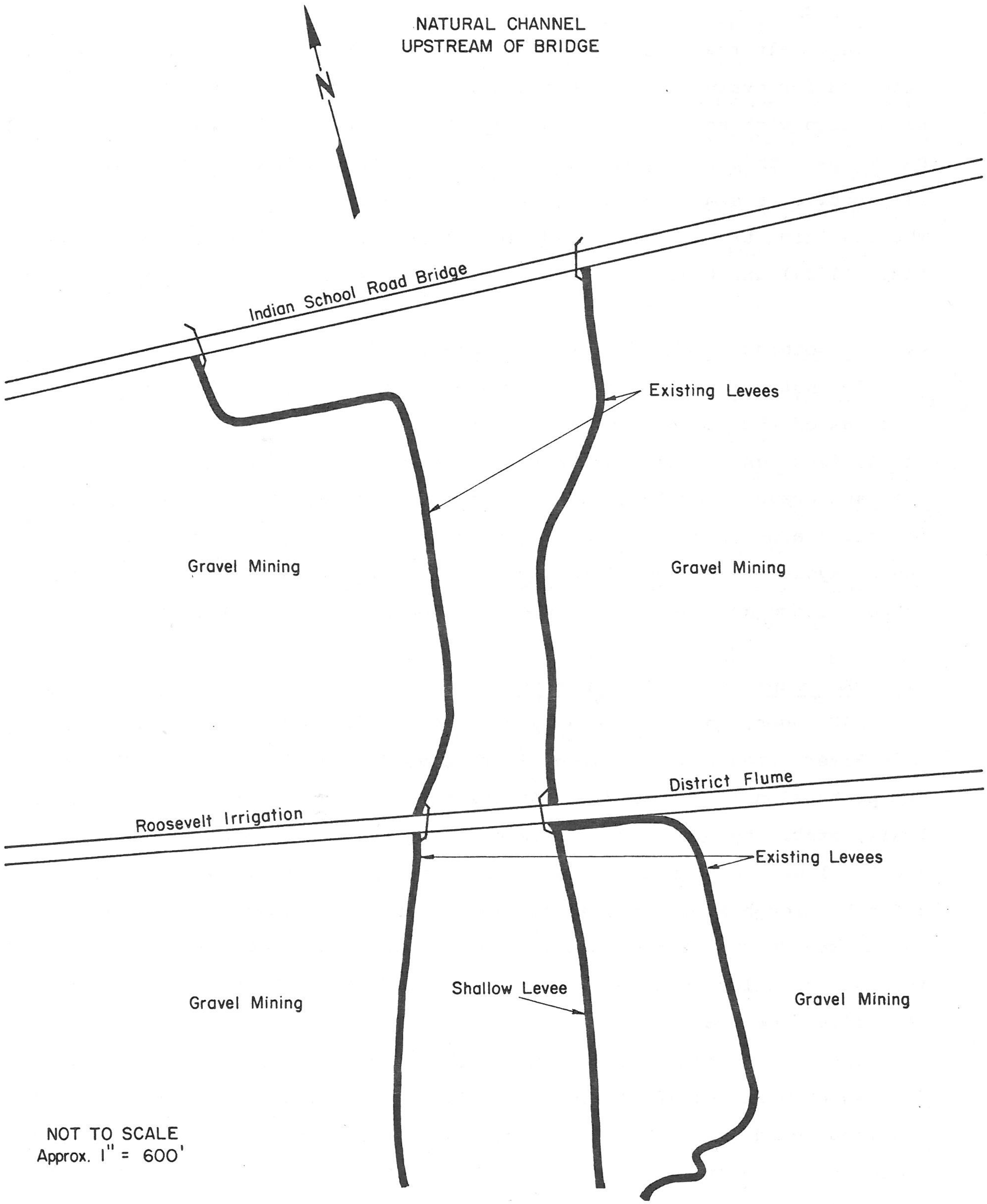


Figure 6.1. Existing condition of Agua Fria River near Indian School Road Bridge.

built would connect to a proposed levee to be constructed by a gravel mining operation on the west overbank of the channel upstream of the bridge.

Figure 6.2 shows the basic configuration of the design alternative proposed by PRC Toups. For a more detailed description of PRC Toup's design and recommendations see the report, "Preliminary Report - Indian School Road Bridge at the Agua Fria River - Rehabilitation and Stabilization of Channel".

The general concept of the alternative proposed by PRC Toups is good. However, three modifications are necessary for improving the performance of the mitigation measure. The first is that the distance between the spur dike and transverse dike should be closer judging from the low flow meandering tendency. The second modification is to increase the channel width as much as possible at the RID flume crossing and the third modification is to stabilize the grade near the RID flume crossing. Based on the results of analysis of the as-is condition and the HEC-2 computations of some preliminary alternatives, SLA proposes an alternative that incorporates the above three modifications to the PRC Toups alternative.

6.3 SLA Design Alternative

6.3.1 Channelization

In order to provide a gradual flow transition of the Agua Fria River as it passes under the ISRB and the RID flume crossing a reach of approximately 3000 feet should be channelized. The proposed channelization utilizes the entire span of the bridge opening (1600 feet) and will increase the channel width at the RID flume from its present width of 500 feet to a width of 1100 feet. The channelized reach extends from just upstream of the bridge to a location of about 700 feet downstream of the flume. The channel width converges from a 1600 foot width at the bridge to a 900 foot width at the downstream end. The downstream channel width of 900 feet is necessary for a proper transition from the channelized reach to the present channel configuration downstream. The channelized reach is at a constant grade of 0.27 percent. Figure 6.3 shows the location of the channelization.

The thalweg profile comparison of the 1980 condition and the proposed channelization condition is shown in Figure 6.4. The invert elevation of the channelization is on the average 3 or 4 feet higher than the existing thalweg elevation along the reach. This is proposed because as the river transitions from the natural condition to the channelization and then again back to the

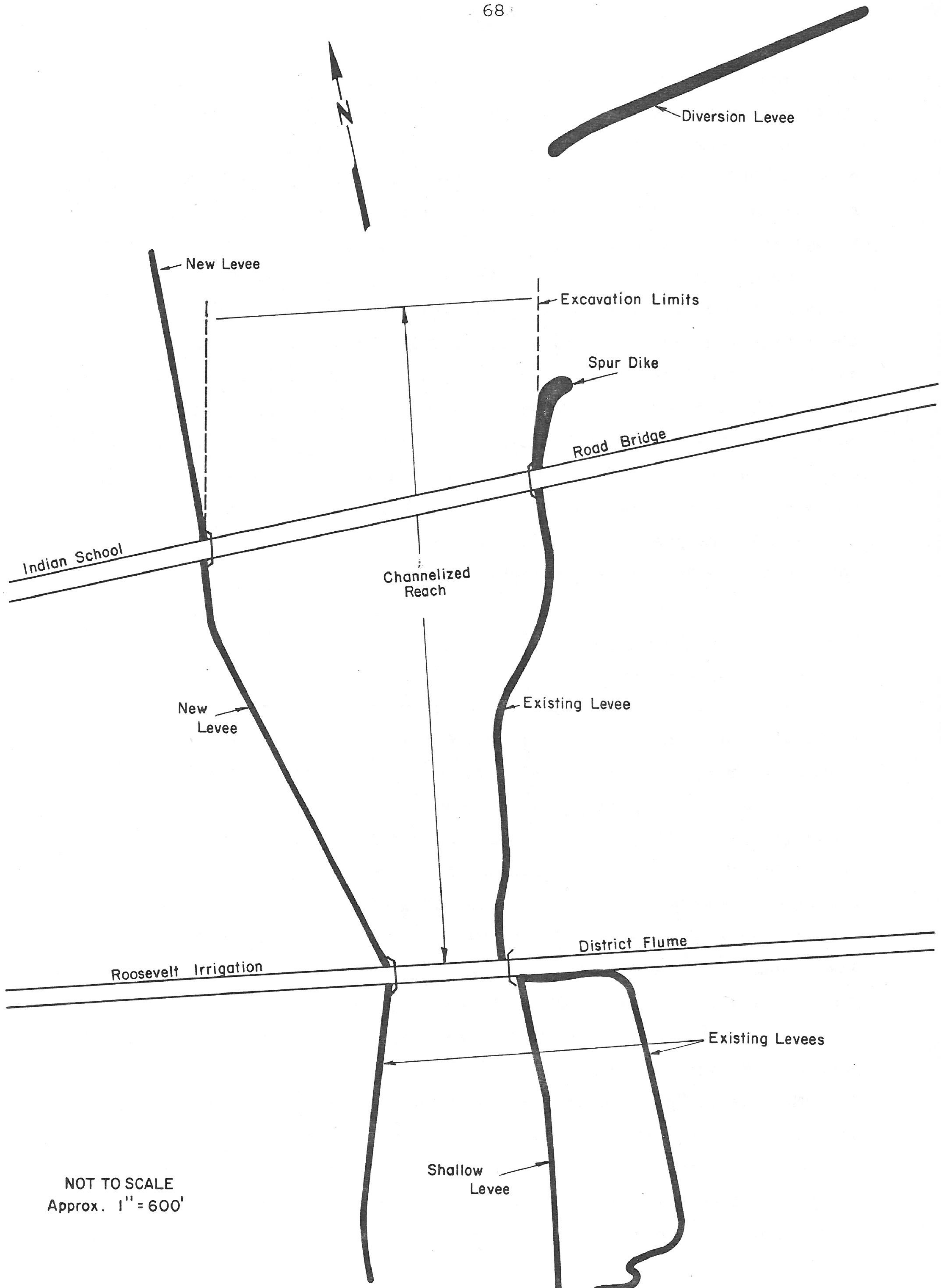


Figure 6.2. Sketch of PRC Toups design alternative.

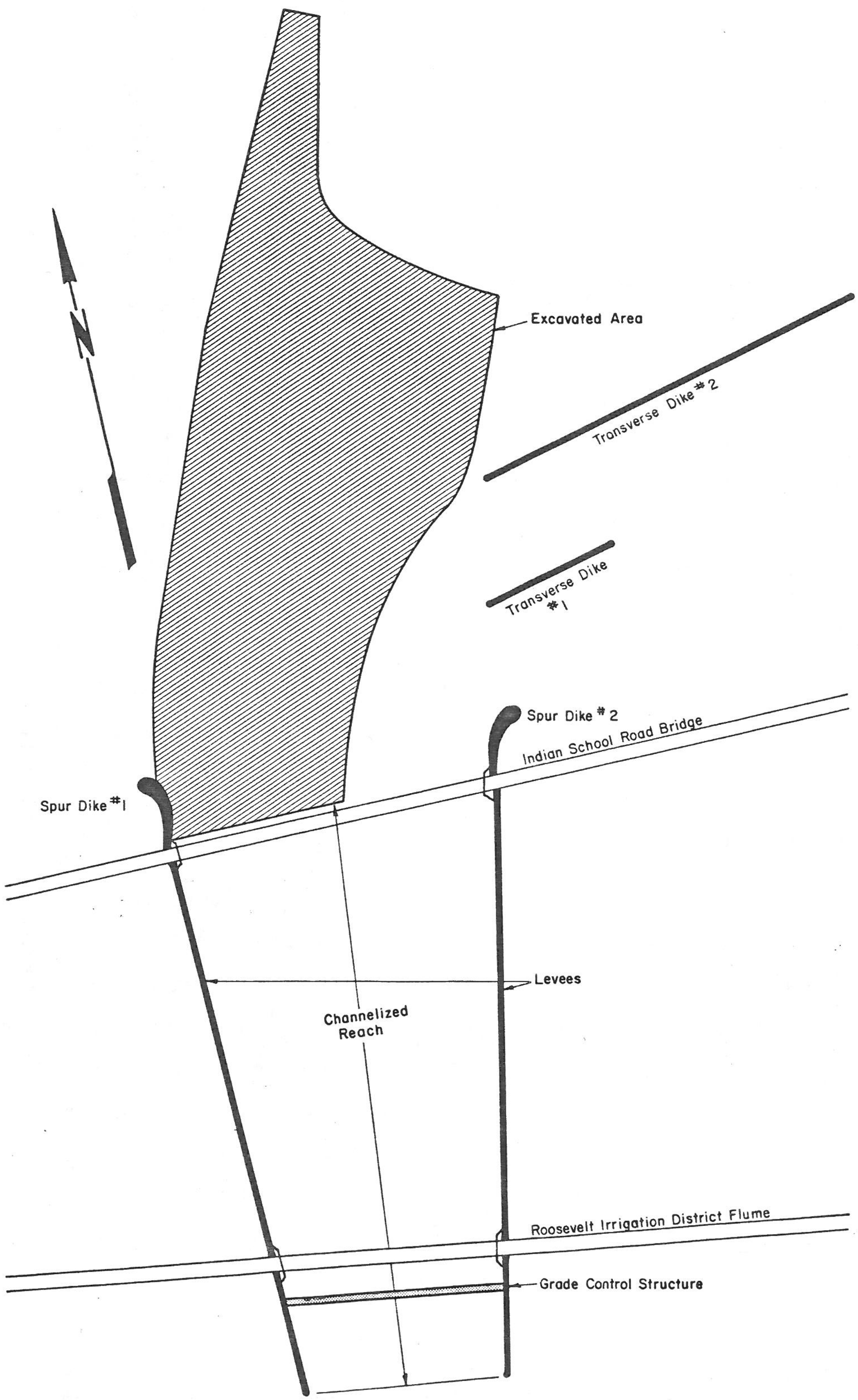


Figure 6.3. Sketch of SLA design alternative.

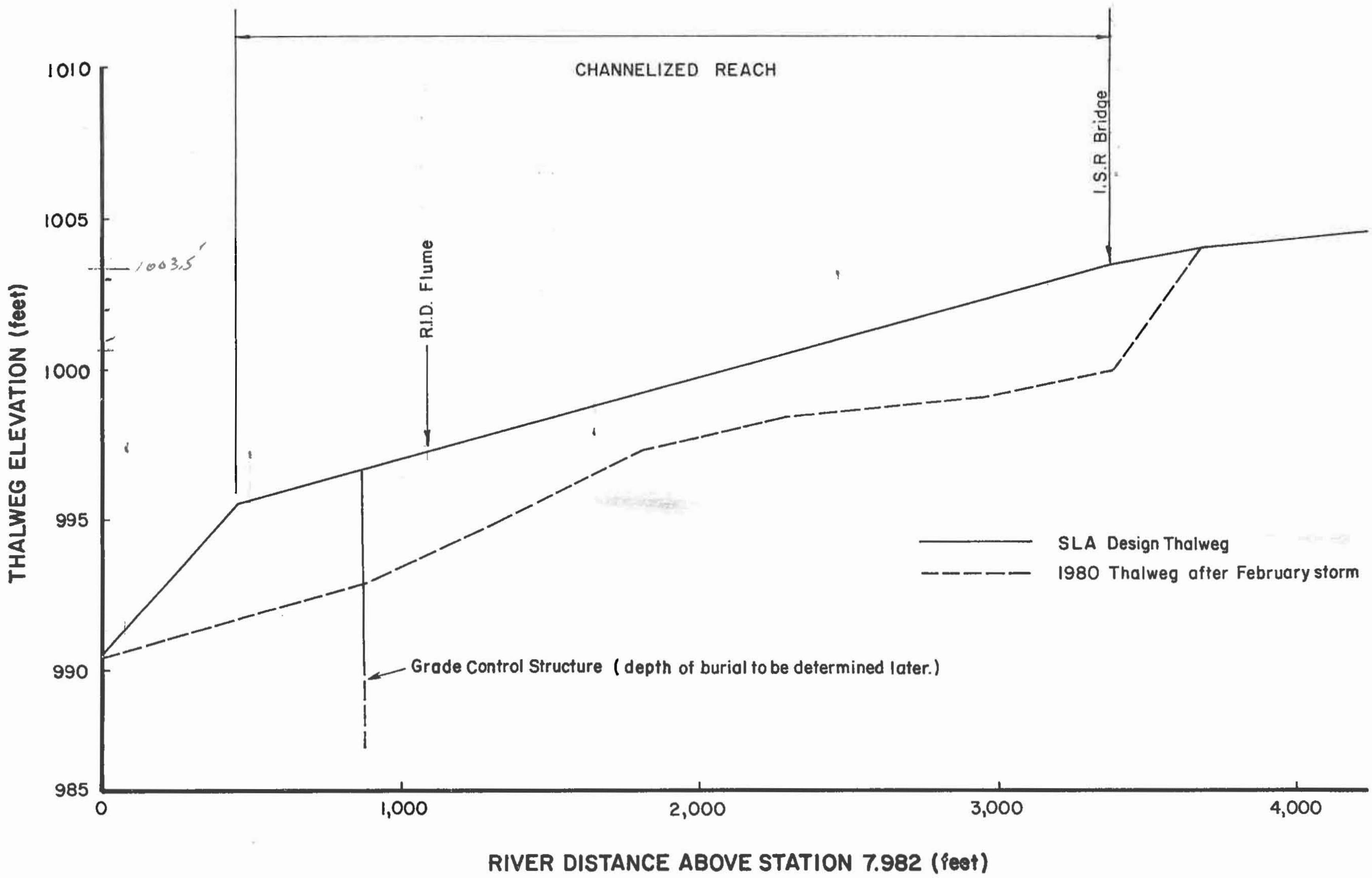


Figure 6.4. Thalweg profile comparison for SLA channelization and 1980 thalweg profile.

existing channel, it is important to provide a similar velocity upstream, within, and downstream of the channelized reach. This is required to maintain a relatively constant grade. Placing the invert at the natural thalweg would result in a very steep grade at the upstream transition and would probably produce increased scour at the bridge. The degradation upstream will cause deposition problems downstream, thus the natural channel just downstream of the channelization will produce a significant backwater problem. Consequently the best design is to have the invert of the channelization slightly higher than the present thalweg. A typical channelized cross section is shown on Figure 6.5.

6.3.2 Partial Channelization Upstream of Bridge

The channel upstream of the bridge has meandered to the east and therefore the area directly above the west half of the bridge span is much higher than the rest of the channel. In order to allow the total width of the bridge to become effective in passing the flow, the upstream channel must be opened up by excavating the area above the west half of the bridge. This partial channelization will extend for a distance of 3500 feet upstream of the bridge. The aproximate location of the area needed to be excavated was presented in Figure 6.3.

6.3.3 Levees

Levees must be constructed on both sides of the channelized reach to provide the bank stability (see Figure 6.3). Surface waves, antidune heights, and adequate freeboard must all be considered in the calculation of the design height of the levees. It is estimated that the levees be constructed at an elevation 3 feet above the elevation of the 100 year flood. This also satisfies the regulation of Federal Emergency Management Agency.

In addition to the channelization, partial channelization upstream, and levees, the SLA alternative also includes the use of dikes upstream of ISRB, pier protection of ISRB and the RID flume crossing, and the grade control structure just downstream of RID flume crossing. The details are described in the following section.

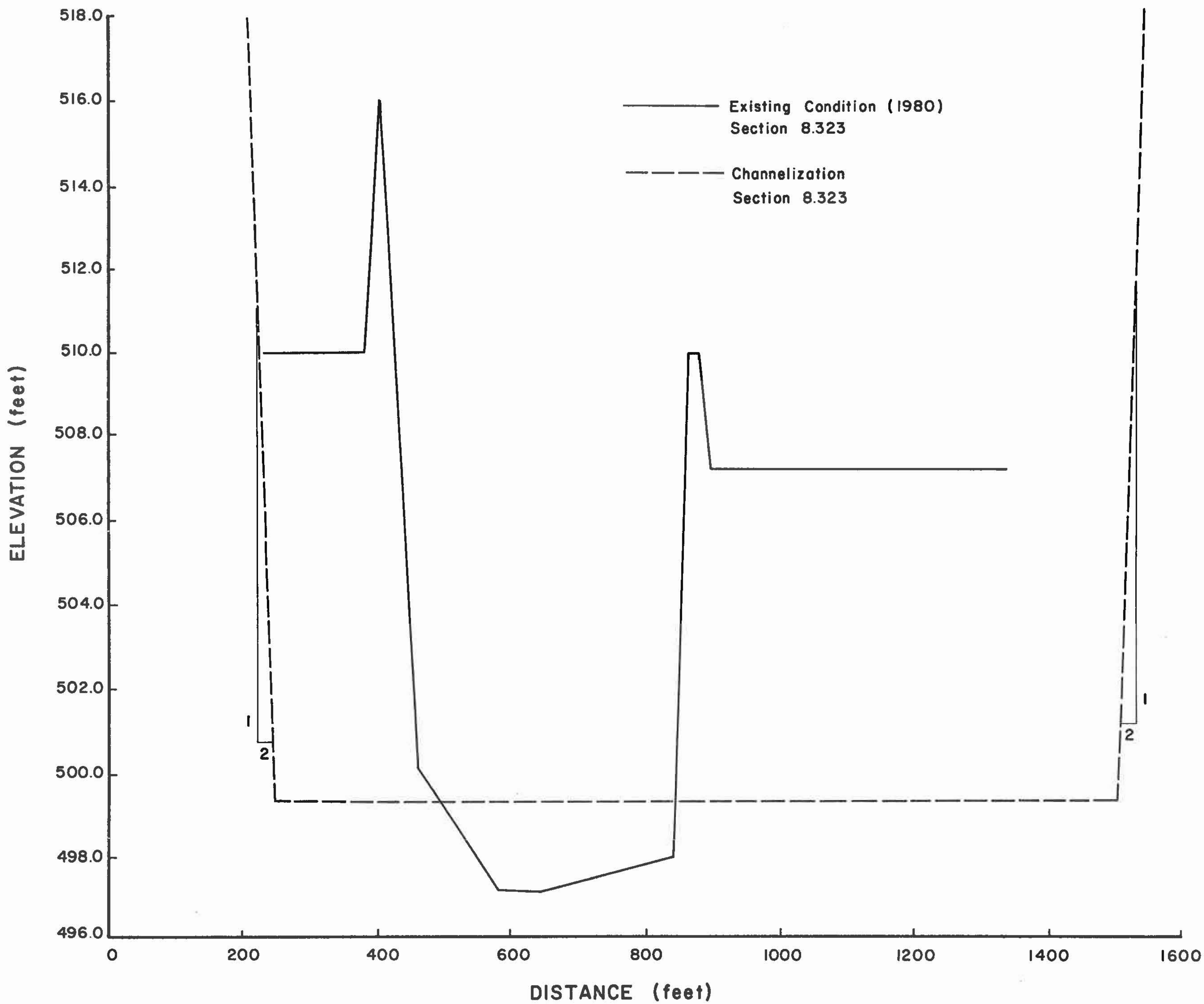


Figure 6.5. Sketch of SLA channelized cross section in comparison with existing cross section bet[illegible] Ir[illegible] S[illegible]l R[illegible] Bri[illegible] and[illegible]se[illegible] Ir[illegible]tic[illegible] Flu[illegible].

6.4 Conceptual Design of Dikes and Pier Protection

6.4.1 Introduction

Two types of dikes are utilized in the SLA alternative, spur dikes and transverse dikes. The purpose of these dikes is the orientation and alignment of the flow through the Indian School Road Bridge. At a discharge of 100,000 cfs it is essential that the flow be aligned properly or the local scour will be severe. The flow must be aligned parallel to the bridge piers and abutments which are skewed at 11 degrees. The basic configuration of dike system was determined based on providing proper flow alignment. Refinements in the design such as riprap sizing, side slopes, and spur dike dimensions were made based upon the hydraulic and erosion and sedimentation analysis presented in Chapter VII.

The designs presented in the following are conceptual in nature. Economic analysis would have to be performed prior to determination of detailed designs and preparation of construction drawings. The present scope of work includes only the conceptual design of mitigation measures.

6.4.2 Location and Shape of Dikes

Figure 6.6 illustrates the location and orientation of the spur dikes and transverse dikes. Both spur dikes are skewed at 11 degrees to provide proper flow alignment. In addition the transverse dikes are terminated to provide an 11 degree alignment. Each of the transverse dikes is tilted at an angle of 15 degrees to the ISRB alignment in order to prevent flows from striking them perpendicular in order to reduce local scour along their upstream faces. The spacing between the transverse dikes and spur dikes is designed to prevent low flow channel meandering from circumventing the dike system. The spacing of 600 feet and 500 feet are less than the low flow channel wave length of 1000 feet to 2000 feet.

It should be noted that Transverse Dike 2 is not tied to high ground outside the 100,000 cfs floodplain, because of the great additional length of dike this would require. Because of this water can flow around the dike and then along its downstream face, the downstream face also requires riprap. If the dike were extended to high ground, this would not be necessary. An economic analysis should be performed to determine whether extending the dike to high ground would be less expensive than riprapping the downstream face.

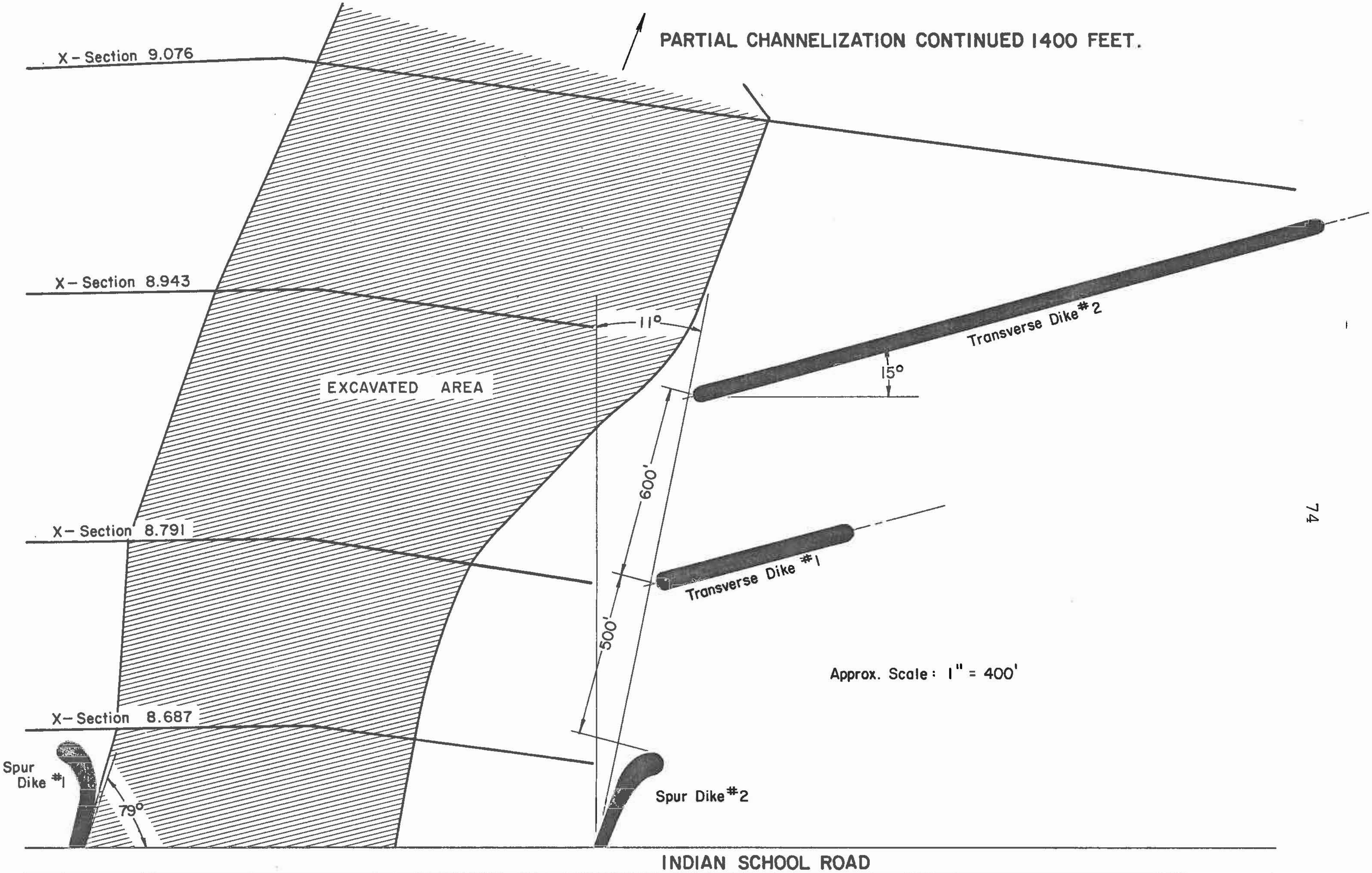


Figure 6.6. Location and orientation of spur dikes and transverse dikes.

A continuous dike on the east side of the channel was considered; however, it was rejected since it would create a problem with flooding of the roadway. Flooding would result from the stagnant water behind the dike having a water surface elevation equal to the elevation of the water at the upstream end of the dike. This would cause the stagnant water to be higher than the roadway. For this reason, the transverse dikes were selected.

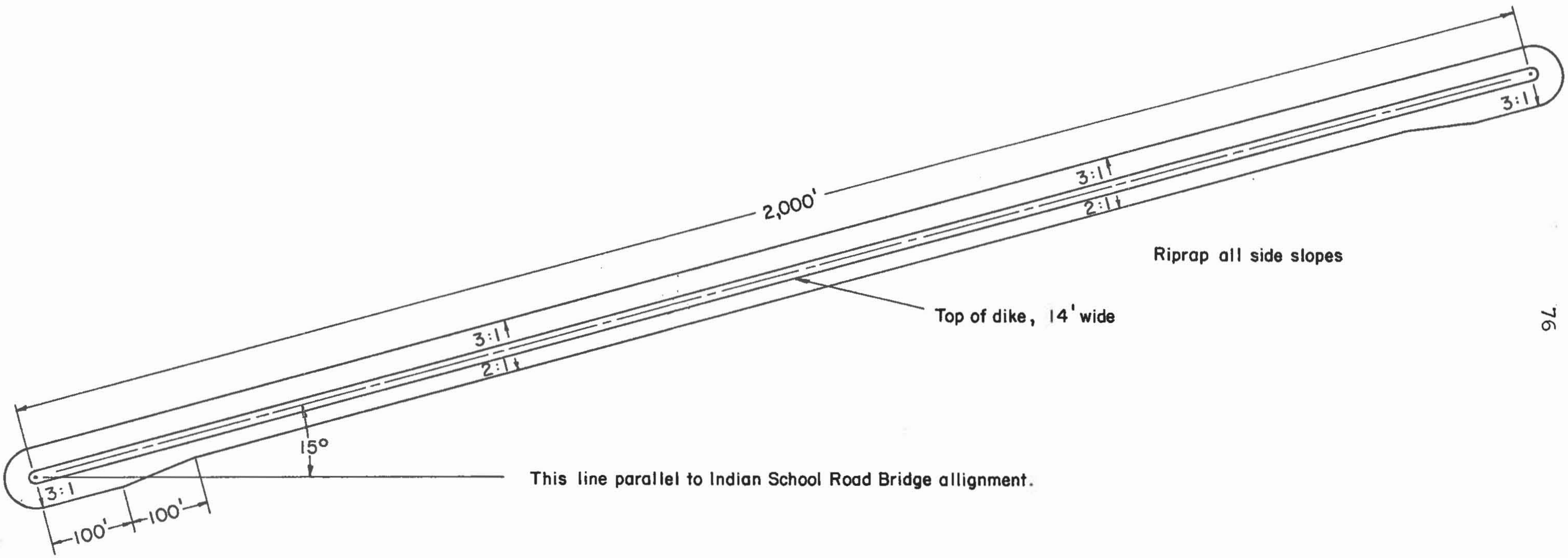
Figure 6.7 provides the conceptual design of Transverse Dike 2. The other transverse dike is identical except its length is 600 feet rather than 2000 feet. At locations where flows are to attack the dike most severely the side slopes are to be 3:1. The remainder of the side slopes can be at 2:1. Flatter side slopes provide more stability for the riprap protection. The top of the dikes should be 3 feet above the 100,000 cfs flood level. This allows adequate freeboard to prevent waves and local acceleration of the flow around the dikes from overtopping the dikes.

If the Agua Fria shifts so that the western bank is being attacked upstream of the bridge, it will be necessary to add dikes upstream of spur dike 1 to keep the flow aligned. The situation should be monitored for the need for additional dikes on the west bank after each significant flow event.

Figure 6.8 provides the conceptual design of spur dike 2. Spur dike 1 is identical except its orientation is changed. 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 side slopes in remaining areas. The top of the spur dikes should be 3 feet above the 100,000 cfs flood level to provide adequate freeboard for waves and local acceleration of the flow.

6.4.3 Riprap Protection

The transverse dikes, spur dikes, and levees between ISRB and RID flume crossing require riprap protection due to the hydraulic conditions to which they will be exposed. Riprap protection was based on the factor of safety method presented in "Sediment Transport Technology" (Simons and Senturk, 1976). The riprap was designed to have a factor of safety of 1.5 or greater. Based on the hydraulic conditions, riprap with a d_{50} of 1.5 feet will be



Note : Transverse Dike #1 is identical except for length dimension, which is 600' rather than 2,000'.

Scale : 1" = 200'

Figure 6.7. Transverse Dike #2 design.

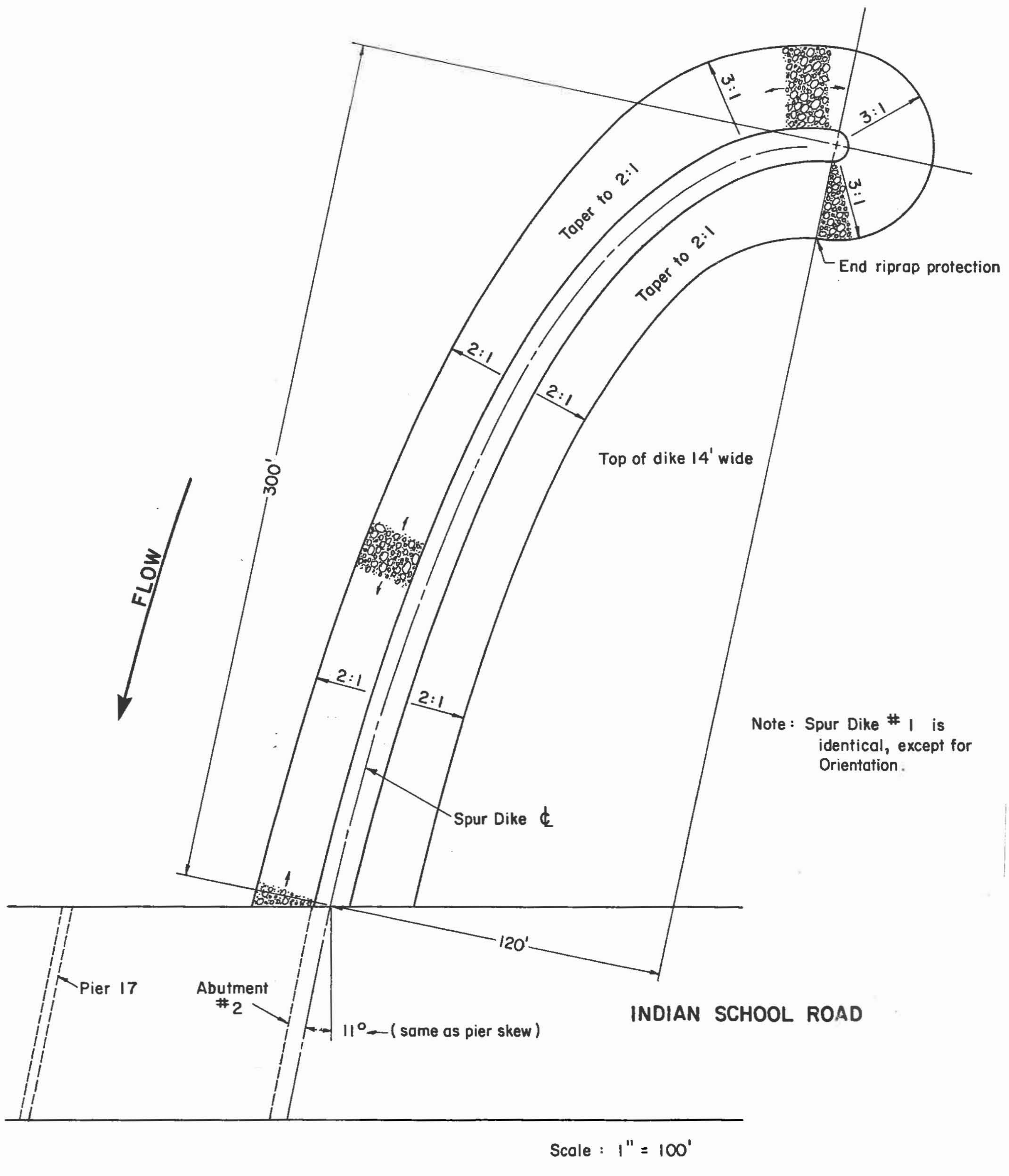


Figure 6.8. Spur Dike #2 design.

adequate for both dike types and 2:1 or 3:1 side slopes. The factor of safety on 2:1 side slopes will be 1.5 and on 3:1 side slope it will be 1.6. Hydraulic conditions in areas requiring 3:1 side slopes were adjusted to reflect their severity by increasing velocities by a factor of 1.5 when computing shear stresses.

Locations along the spur dike requiring riprap are illustrated in Figure 6.8. The only area not requiring riprap is the back side of the shank since it is not exposed to flow. However, this area should be blanketed with coarse gravel if the dikes are built of sandy or fine material.

All riprap must be angular rock of sufficient durability to survive weathering and flows. Maximum riprap size should be approximately 3.0 feet with approximately 20 percent of the material 0.75 feet or smaller. With this distribution of sizes, the interstices formed by the larger stones are filled with smaller sizes in an interlocking fashion, preventing formation of open pockets. Riprap consisting of angular stones provides more stability than rounded stones. The riprap should be placed to a thickness of approximately 3 feet.

A filler underneath the riprap should be used to protect the fine embankment or riverbank material from washing out through the riprap. Two types of fillers are commonly used, gravel filters and plastic filter cloths. If plastic filter cloths are used, care must be taken to avoid puncturing the filter while placing riprap.

A gravel filter should meet the following specifications:

(1) $$\frac{D_{50}(\text{filter})}{D_{50}(\text{base})} < 40$$

(2) $$5 < \frac{D_{15}(\text{filter})}{D_{15}(\text{base})} < 40$$

(3) $$\frac{D_{15}(\text{filter})}{D_{85}(\text{base})} < 5$$

If the riprap already meets these requirements in respect to the base, no filter is required.

For the levees between the flume and the bridge the velocities are similar to the ones experienced in the area upstream of the ISRB. The diameter of riprap material required will be 1.5 feet and the bank levees should be placed at 2:1 side slopes. The levees will require the same filter protection requirements as the spur dikes.

Riprap protection must be extended sufficiently below the bed elevation to prevent undermining of the protection from general scour, bed forms and local scour. Considering these factors the following riprap burial depths are required:

1. Western nose of Transverse Dike 2: 15' - 20'
2. Upstream face of Transverse Dike 2 and eastern nose: 10'
3. Downstream side of Transverse Dike 2: 5'
4. Noses and upstream face of Transverse Dike 1: 10'
5. Downstream side of Transverse Dike 1: 5'
6. Nose of Spur Dikes 1 and 2: 15'
7. Shank of Spur Dikes 1 and 2: 15'
8. Levees between ISRB and RID flume crossing: 10'

6.5 Pier Protection

The piers at the ISRB should be given additional protection to increase their factor of safety against scour. This can be accomplished in two ways. First placing a riprap cone or apron around the piers and secondly opening up the area between the piers of the two decks that was previously concreted in to make one solid pier out of each pair of piers. Riprap will decrease the amount of scour by armoring the local scour holes quickly. Opening up the space between pairs of piers will increase the effective flow area under the bridge and reduce scour if the flow hits the piers at adverse angles.

The piers at the RID flume crossing should be protected by placing a riprap cone or apron around the piers.

The riprap cone should extend entirely around the piers. The piers should be excavated to the footings and the riprap cone placed on the footing. Riprap of approximately 1.5 feet in diameter will be sufficient. Side slopes of the cone should be at 2:1 with a cone height sufficient to extend the riprap 5 feet beyond the footings.

The riprap apron should be constructed of 1.5 foot diameter riprap. It should extend 5 feet beyond the footings. A thickness of 4 feet should be used. The apron should be buried approximately 4 feet below the channel bed.

The riprap apron is recommended over the cone since the apron requires less excavation and less disturbance of the material around the piers.

6.6 Grade Control Structure

Between 1973 and 1980 five feet of general scour had occurred throughout the entire reach which included the ISRB and the RID flume. Therefore, the pier burial depths are presently five feet less than what the original design of the bridge and flume piers required. For this reason it is necessary to limit any further degradation within the reach, particularly for the protection of the RID flume crossing. The grade control structure should be located within the channelized reach approximately 200 feet downstream of the flume which will provide a sufficient distance downstream of the flume for construction purposes. The invert of the grade control structure should be placed at an elevation of 997.0 feet.

VII. ANALYSIS OF ALTERNATIVES

A three-level analysis was applied to evaluate the three selected alternatives for future mitigation measures as identified in Section VI. They are 1) "Do Nothing" Alternative (as-is condition), 2) PRC Toups Design Alternative and 3) SLA Design Alternative. Results of the three-level analysis follow.

7.1 Qualitative Evaluation of Channel Response of Design Alternatives

Using the hydraulic data generated from the HEC-2 water surface profile program, the expected response of the channel in each of the reaches in the study area can be qualitatively determined for the various alternatives. Detailed qualitative analysis was performed for the three alternatives. Schematic diagrams describing the study area for these are shown in Figures 7.1 and 7.2.

As previously discussed, aggradation or degradation within a reach is related to changes in the top width and approximately the fourth power of changes in the velocity as compared to the upstream reach. Changes in flow depth have a less significant impact on the response. To determine the expected aggradation or degradation, plots of top width, velocity, and depth vs. river distance for the 100-year flood were made (Figures 7.3 through 7.5). The results of the analysis based on the information in these plots are presented in Tables 7.1, 7.2 and 7.3 for the three alternatives.

For the "Do Nothing" Alternative (1980 as-is condition), the encroachment of the gravel mining levees into the channel between the ISRB and the RID Flume causes a significant decrease in the top width and increase in velocity indicating severe degradation in that reach. This could result in a headcut that would endanger the bridge. The widening of the high flow channel and corresponding decrease in velocity below the flume indicate a tendency for some aggradation. A check of the hydraulics for the lower discharges indicates the opposite effect, however, so that this tendency may not be indicative of the channel response in this reach for the entire storm hydrograph. Because of the shape of the cross section in the downstream reach, the flow area is restricted at the lower discharges causing an increase in the velocity and resulting degradation.

The channelization alternative proposed by PRC Toups appears to increase the stability of the channel in the vicinity of the bridge. Failure to widen the channel at the RID flume, however, indicates a severe degradation ten-

Section No.	River Distance (ft. above Section 7.88)	Reach No.	River Distance (ft.)	Features
7.98	510	6	0	
8.07	960		1,180	
8.15	1,410			
8.189	1,603	5		Roosevelt Irrigation
8.192	1,619			District Flume
8.23	1,810		1,995	
8.32	2,310	4		
8.42	2,810		3,375	
8.54	3,460			Indian School Road
8.62	3,780	3		Bridge
8.63*	3,860		4,190	
8.69	4,240			
8.79	4,820			
8.94	5,590			
9.08	6,290	2		
9.21	6,940			
9.34	7,660			
9.50	8,310			
9.64	8,880		9,110	
9.71	9,170			
9.86	9,970	1		
9.99	10,620			
10.11	11,320		12,050	

* Not used for PRC Toups channelization alternative.

Figure 7.1. Schematic diagram of study area for "do nothing" alternative (1980 as-is condition) and PRC Toups design alternative.

Section No.	River Distance (ft. above Section 7.88)	Reach No.	River Distance (ft.)	Features
7.98	510	6	0	
8.07	960		1,180	
8.15	1,410			Roosevelt Irrigation
8.19	1,603	5		District Flume
8.23	1,810		1,995	
8.32	2,310	4		
8.42	2,810		3,375	
8.54	3,460	3		Indian School Road
8.62	3,895		4,190	Bridge
8.69	4,195			
8.79	4,745			
8.94	5,545			
9.08	6,245	2		
9.21	6,945			
9.34	7,605			
9.50	8,445			
9.64	9,145		9,340	
9.71	9,535			
9.86	10,315	1		
9.99	11,005			
10.11	11,675		12,050	

Figure 7.2. Schematic diagram of study area for SLA design alternative.

FIGURE 7.3

TOP WIDTH VS. RIVER DISTANCE

AGUA FRIA

100 YEAR FLOOD PEAK

TOP WIDTH VS. RIVER DISTANCE AGUA FRIA 100 YR FLOOD PEAK

FIGURE 7.4

VELOCITY VS. RIVER DISTANCE

AGUA FRIA

100 YEAR FLOOD PEAK

VELOCITY VS. RIVER DISTANCE AGUA FRIA 100 YR FLOOD PEAK

FIGURE 7.5

DEPTH VS. RIVER DISTANCE

AGUA FRIA

100 YEAR FLOOD PEAK

Table 7.1. Expected Qualitative Response of Reaches Based on HEC-2 Hydraulics - 1980 Existing Condition.

Reach	Change in Topwidth	Change in Velocity	Overall Response
1	supply	supply	supply
2	slight decrease	decrease	slight aggradation
3	decrease	increase	degradation
4	decrease	increase	degradation
5	slight increase	stable	slight degradation
6	increase	decrease	aggradation

Table 7.2. Expected Qualitative Response of Reaches Based on HEC-2 Hydraulics - PRC Toups Channelization Alternative.

Reach	Change in Topwidth	Change in Velocity	Overall Response
1	supply	supply	supply
2	slight decrease	slight increase	slight degradation
3	decrease	decrease	aggradation
4	decrease	increase	degradation
5	decrease	increase	degradation
6	increase	decrease	aggradation

Table 7.3. Expected Qualitative Response Based on HEC-2 Hydraulics - SLA Channelization Alternative.

Reach	Change in Topwidth	Change in Velocity	Overall Response
1	supply	supply	supply
2	decrease	slight increase	degradation
3	slight decrease	slight increase	slight degradation
4	slight decrease	slight increase	slight degradation
5	slight decrease	slight increase	slight degradation
6	slight decrease	slight increase	slight degradation

dency in that area which could endanger the flume and may initiate a headcut as previously discussed which could affect the stability of the bridge. As stated previously, the original study by PRC Toups did not include the stability of the flume and therefore their alternative did not reflect stability measures at the flume.

The SLA design alternative stabilizes the entire study reach significantly. Due to the structural constraints of the flume and bridge, however, a slight degradation tendency still exists. For this reason, a grade control structure is recommended near the RID flume crossing.

Water surface profiles for the three alternatives are plotted in Figure 7.6. From the figure, it can be seen that the choking of the flow due to the gravel mining levees has been eliminated in the proposed channelization alternative.

7.2 Quantitative Geomorphic Analysis of Design Alternatives

Total scour analysis to rehabilitate the bridge and protect the flume was conducted in a similar manner as the analysis to determine the scour components that caused failure of the Indian School Road Bridge. The total scour depth is the summation of the local scour at the piers, the general regional scour due to the constriction in the flow, and the general aggradation/degradation response of the river. The total scour analysis was performed for the three alternatives identified in Section VI at the ISRB and RID flume crossing.

7.2.1 Local Scour at Indian School Road Bridge and Roosevelt Irrigation District Flume Crossing Considering Design Alternatives

The local scour computations for the three alternatives utilizing Shen, Neil and armoring control methods are summarized in Table 7.4 for the ISRB and in Table 7.5 for the RID flume crossing. For the "Do Nothing" alternative (existing conditions) at the bridge, the local scour potential utilizing the armoring control method was 50 feet. A unit discharge of 590 cfs/ft results from the flow necking down at the bridge, which is more than double the amount of flow that occurred in the February 20, 1980 flood causing the significant scour depth before armoring would control. Shen and Neil's equations estimate the local scour potential at 20.5 feet and 22 feet, respectively. The high

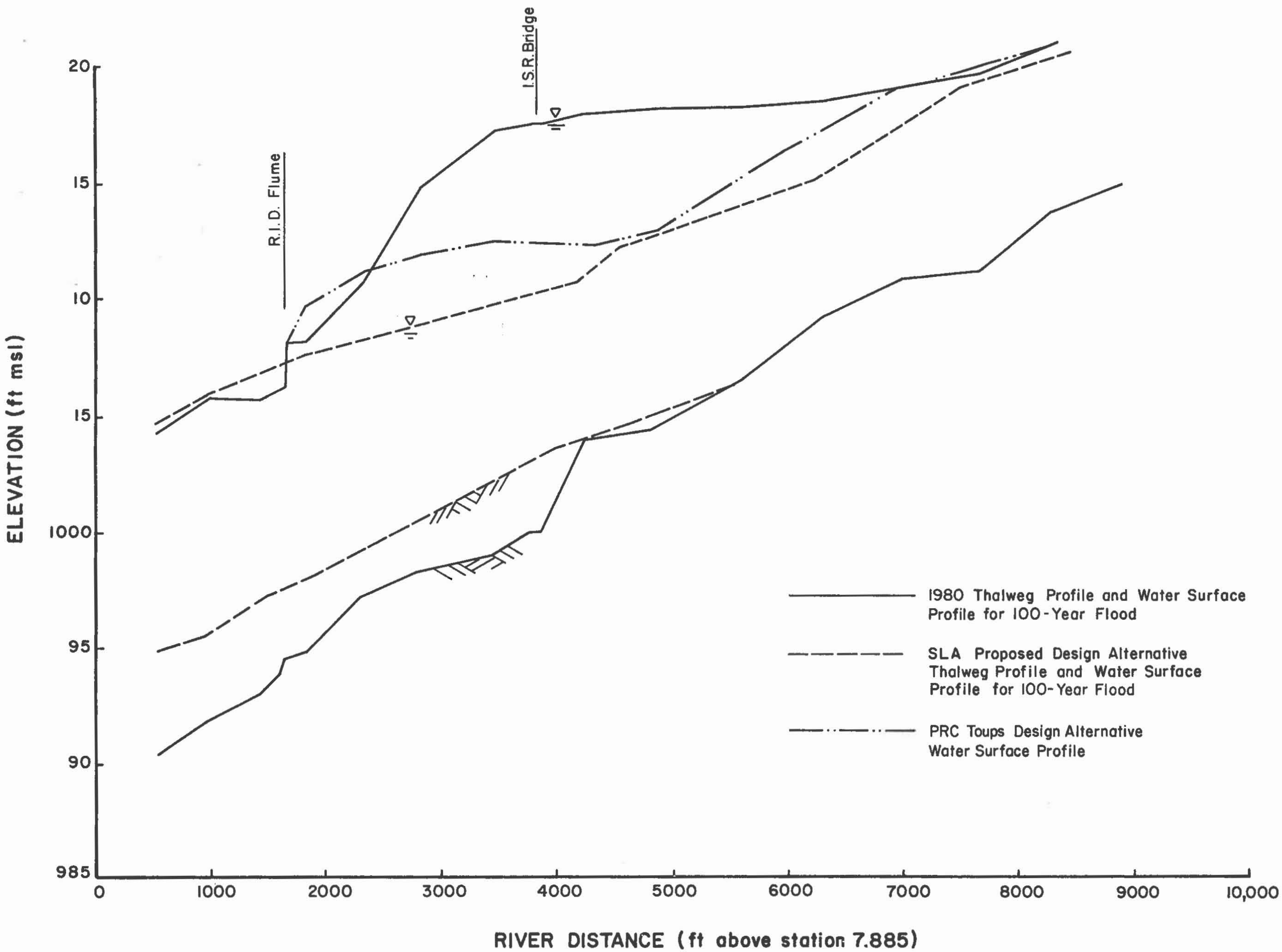


Figure 7.6. Water surface profiles for the design alternatives.

Table 7.4. Local Scour at Indian School Road Bridge for 100,000 cfs for 1980 Existing Conditions, PRC Toups Channelization and SLA Channelization.

	Armor Control Local Scour	15° Angle of Attack	Shen Local Scour	15° Angle of Attack	Neil Local Scour	15° Angle of Attack	Adopted Local Scour
	(ft)	(ft)	(ft)	(ft)	(ft)	(ft)	(ft)
1980 Existing	50	--	20.5	--	22.0	--	50
PRC Toups Channelization	8.6	11.2	4.3	12.9	4.2	12.6	12.9
SLA Channelization	8.6	11.2	5.0	15.0	4.5	13.5	15.0

Table 7.5. Local Scour at Roosevelt Irrigation District Flume for 100,000 cfs for 1980 Existing Conditions, PRC Toups Channelization and SLA Channelization.

	Armor Control Local Scour	Neil's Local Scour	Shen's Local Scour	Adopted Local Scour
	(ft)	(ft)	(ft)	(ft)
1980 Existing	20.7	10.5	11.1	11.1
PRC Toups Channelization	20.7	10.6	11.4	11.4
SLA Channelization	7.3	8.3	8.6	8.6

potential of local scour clearly indicates that the do nothing alternative is totally unacceptable. At the RID flume, the local scour depth is 20.7 feet as predicted by the armoring control method and 11.1 feet and 10.5 feet as predicted by Shen and Neil's methods. A local scour depth of 11.1 feet is considered reasonable at the flume.

The expected local scour depths at ISRB for PRC Toups channelization and the SLA channelization is 8.6 feet using the armoring control method and approximating 4.5 feet using Shen and Neil's methods assuming the flow is uniformly distributed across the bridge piers. The spur dikes and guide banks are designed to align the flow parallel to the bridge piers, however, the historical trend of the Agua Fria has been to migrate laterally to the east bank at ISRB. Because of the uncertainty of the spur dikes and guide banks to properly align the flow, the local scour computations were also performed considering a 15 degree angle of attack at the bridge piers. The local scour depths at the bridge utilizing the armoring control method for PRC Toups and SLA's channelization was 11.2 feet and using Shen and Neil's methods, the local scour depth was approximately 13 feet for PRC Toups channelization and 15.0 feet for SLA's channelization.

PRC Toups contract with Maricopa County did not involve investigating the stability of the RID flume crossing so their channelization alternative does not include protective measures at the flume and hence will reflect large local scour depths at the flume. PRC Toups has recommended to the County widening the opening at the flume crossing similar to the SLA alternative. The local scour depth at the flume for the SLA channelization using the armoring control method was 7.2 feet and using Shen and Neil's method, the local scour depths were 8.6 feet and 8.3 feet, respectively. Since these local scour depths are all reasonably close, the depth of 8.6 feet was selected as a conservative estimate of the local scour potential.

7.2.2 General Regional Scour for Design Alternatives

As stated previously in section 4.2.2 general regional scour is caused by a contraction which reduces the effective flow area, which in turn increases 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.

For existing conditions with the levees intact protecting the floodplain gravel pits a general regional scour potential of 17.8 feet exists for a peak discharge of 100,000 cfs. The general regional scour depth must be compared with the armoring potential of the bed to determine which scour depth will control the actual bed elevation.

The general regional scour was determined for PRC Toups and SLA's channelization alternatives and was 6.7 feet and 4.2 feet respectively. The difference between the two channelization schemes is SLA's channelization considered widening the opening at the RID flume crossing and thus doesn't constrict the flow as severely as PRC Toups channelization between the bridge and the flume.

7.2.3 Aggradation/Degradation Analysis for Design Alternatives

The aggradation/degradation analysis for the reach between the flume and the bridge was computed utilizing the equilibrium slope method and the armoring control method. The methods are compared to determine which controls the lowest bed elevation.

7.2.3.1 Equilibrium Slope Method to Determine Degradation or Aggradation for Design Alternatives

The equilibrium channel slope is defined as the slope at which the channel's sediment transporting capacity is equal to the incoming sediment supply. Under this condition, the channel neither aggrades nor degrades. When the present slope of a channel is greater than the equilibrium slope, the channel will degrade in order to reach its equilibrium slope.

For "Do Nothing" alternative (1980 existing conditions) the calculation of the sediment transport rate, involved using sections 8.69, 9.79, 8.94, 9.08, 9.21 and 9.34 of reach 2, (see Figure 7.1) which is located upstream of ISRB as the supply reach, and computing the new equilibrium slope of Reach 4, which is located between the flume and the bridge.

The bed slope between the flume and the bridge in 1980 was .0029 ft/ft. With the narrow constriction due to the levees the sediment transport rate through reach 4 is much greater than the transporting rate of reach 2, and therefore a new equilibrium slope is achieved. The new slope for reach 4 for a discharge of 100,000 cfs is .0003 ft/ft. Thus by pivoting this new slope about a point near the flume the potential degradation at ISRB is 6.2 feet.

For PRC Toups channelization and SLA's channelization the upstream supply reach started at cross section 9.08, approximately where the channelization ceased, and extended to cross section 9.64, approximately the end of reach 2. The equilibrium slope computation was determined for reach 4. The original bed slope between the flume and the bridge suggested by PRC Toups was .0029 ft/ft and the new equilibrium slope computed for a discharge of 100,000 cfs was .0026 ft/ft. Pivoting the new slope about the proposed grade control structure near the flume a potential degradation of .7 feet exists at the bridge. For SLA's suggested channelization the bed slpe between the flume and the bridge is .0027 ft/ft. The computed equilibrium slope for reach 4 is .0027 ft/ft for a discharge of 100,000 cfs; therefore no general aggradation/degradation will occur at the bridge for the proposed channelization.

7.2.3.2 Armor Control Method to Determine Potential Degradation for Design Alternatives

The armoring control method was utilized to determine when armoring of larger particle sizes will control the bed elevation. For existing conditions and a discharge of 100,000 cfs between the bridge and the flume an armoring layer will develop after 10.4 feet of degradation. It must also be emphasized this armor layer will also control the expected general regional scour. For the PRC Toups channelization alternative an armor layer will develop after 8.5 feet of degradation. No armor layer depth was computed for the SLA channelization alternative because the bed remains stable as shown in the equilibrium slope analysis.

7.3 Total Scour Potential for Design Alternatives

The total scour depth at the ISRB piers and RID flume piers can be broken down into three components local scour, general regional scour, and general aggradation/degradation. Table 7.6 summarizes the total exptected scour at the bridge for a discharge of 100,000 cfs for the "Do Nothing" alternative, PRC Toups proposed channelization, and SLA's proposd channelization. Table 7.7 summarizes the total expected scour depth at the RID flume crossing for a discharge of 100,000 cfs for the "Do Nothing" alternative, PRC Toups proposed channelization and SLA's proposed channelization.

Table 7.6. Summary of Total Expected Scour at Indian School Road Bridge for 100,000 cfs Discharge for Future Stability Measures.

	Do Nothing 1980 Existing Conditions	PRC Toups Channelization	SLA Channelizatoin
Local Scour (ft)	50.0	12.9[1]	15.0[1]
General Regional Scour (ft)	--	6.7	4.2
General Aggradation/ Degradation (ft)	10.4[2] deg	.7 deg	negligible
Expected Total Scour (ft)	60.4	20.3	19.2

1Considers 15 degrees angle of attack on bridge piers.

[2]Armor control method which will control the general regional scour depth.

Table 7.7. Summary of Total Expected Scour at Roosevelt Irrigation District Flume Crossing for 100,000 cfs Discharge for Future Stability Measures.

	Do Nothing Existing 1980 Conditions	PRC Toups Channelization	SLA Channelization
Local Scour (ft)	11.1	11.4	8.6
General Regional Scour (ft)	--	6.7	4.2
General Aggradation/ Degradatoin (ft)	10.4[1]	negligible	negligible
Total Expected Scour (ft)	21.5	18.1	12.8

[1]Armor control method controls the amount of general regional scour.

The proposed channelization by SLA has a total expected scour depth of 19.2 feet at ISRB which is approximately what the piers are presently buried. To provide adequate protection of the piers against failure due to undermining, the piers 14, 15 and 16, should be buried 25 feet below the existing grade and protected with rip-rap material as identified in Section VI. The existing piers, which did not fail should be protected with rip-rap material as identified in Section VI.

Downstream of the RID flume crossing a grade control structure is necessary to prevent any further reduction in the base level from occurring. The combination of general regional plus local scour can still cause failure of the flume piers utilizing SLA's proposed channelization and it is therefore imperative to protect the RID flume crossing piers with rip-rap material. The details of the rip-rap material and the placement of material were discussed in Section VI.

7.4 Water and Sediment Routing Results of Design Alternatives

7.4.1 General

The QUASED water and sediment routing model was used to simulate the channel response for the discretized 100-yr storm hydrograph for "Do Nothing" (1980 as-is condition) and the SLA channelization alternative.

7.4.2 "Do Nothing" (1980 As-Is Condition)

Thalweg profiles for the study reach are shown in Figure 7.7. The profile resulting from the sediment routing for the 1980 as-is condition indicates significant general degradation during the 100-yr flood. The channel base level dropped by approximately 3.5 feet in the reach between ISRB and the RID flume. Maximum degradation of 8.5 feet occurred just after the peak of the hydrograph. Slight degradation varying in magnitude from less than 1 to 3 feet occurred in the reach upstream of the bridge. Figures 7.8 and 7.9 show the changes in thalweg elevation with time at ISRB and the RID flume. These figures indicate the accelerated degradation which occurred during the high flows near the hydrograph peak. Maximum general degradation at ISRB occurred after the peak of the hydrograph and had a magnitude of about 2.7 feet. The channel at that location refilled somewhat during the recession. Final general degradation was less than 1 foot. Maximum degradation at the RID

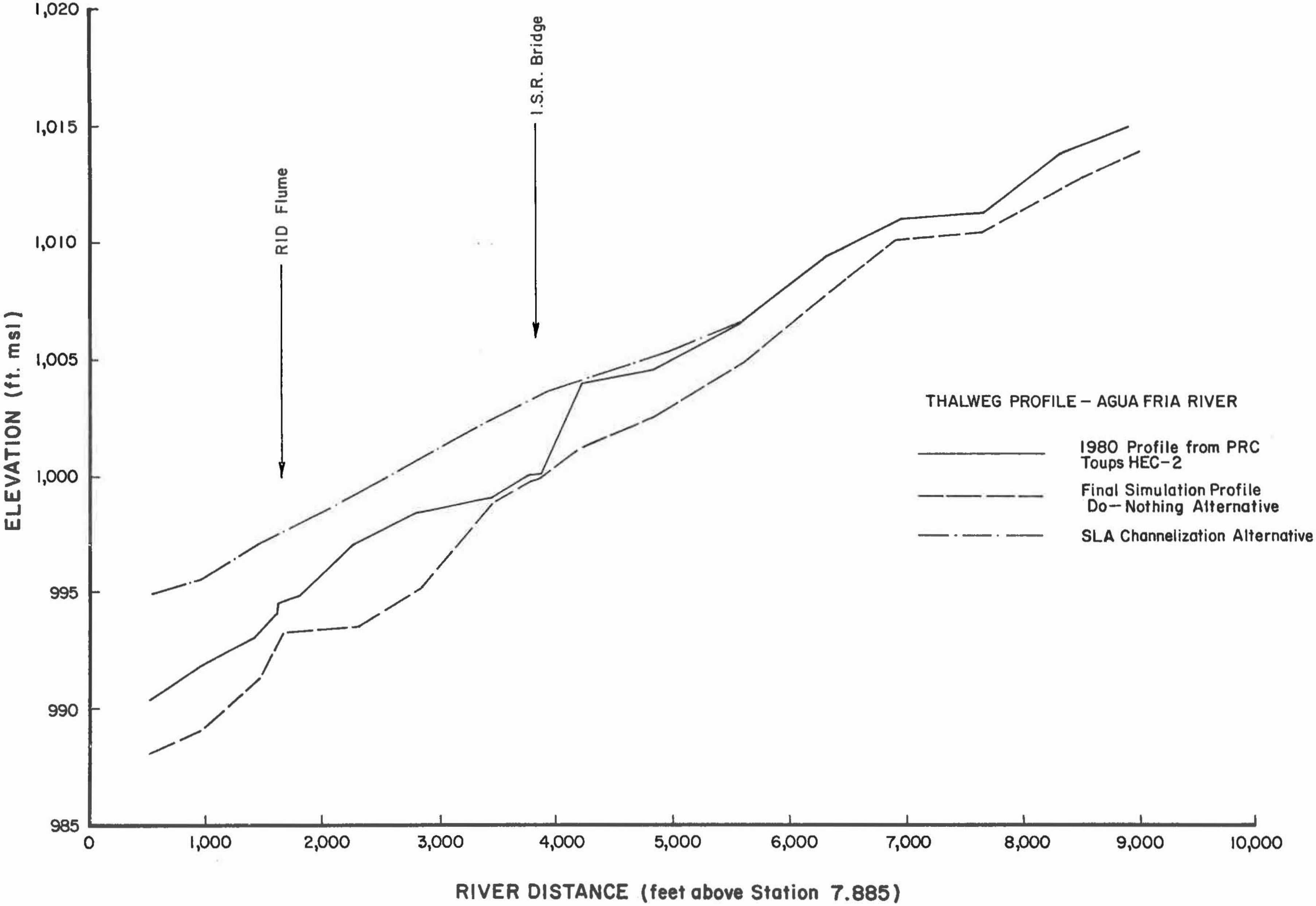


Figure 7.7. Thalweg profiles - Agua Fria River after 100 year flood event.

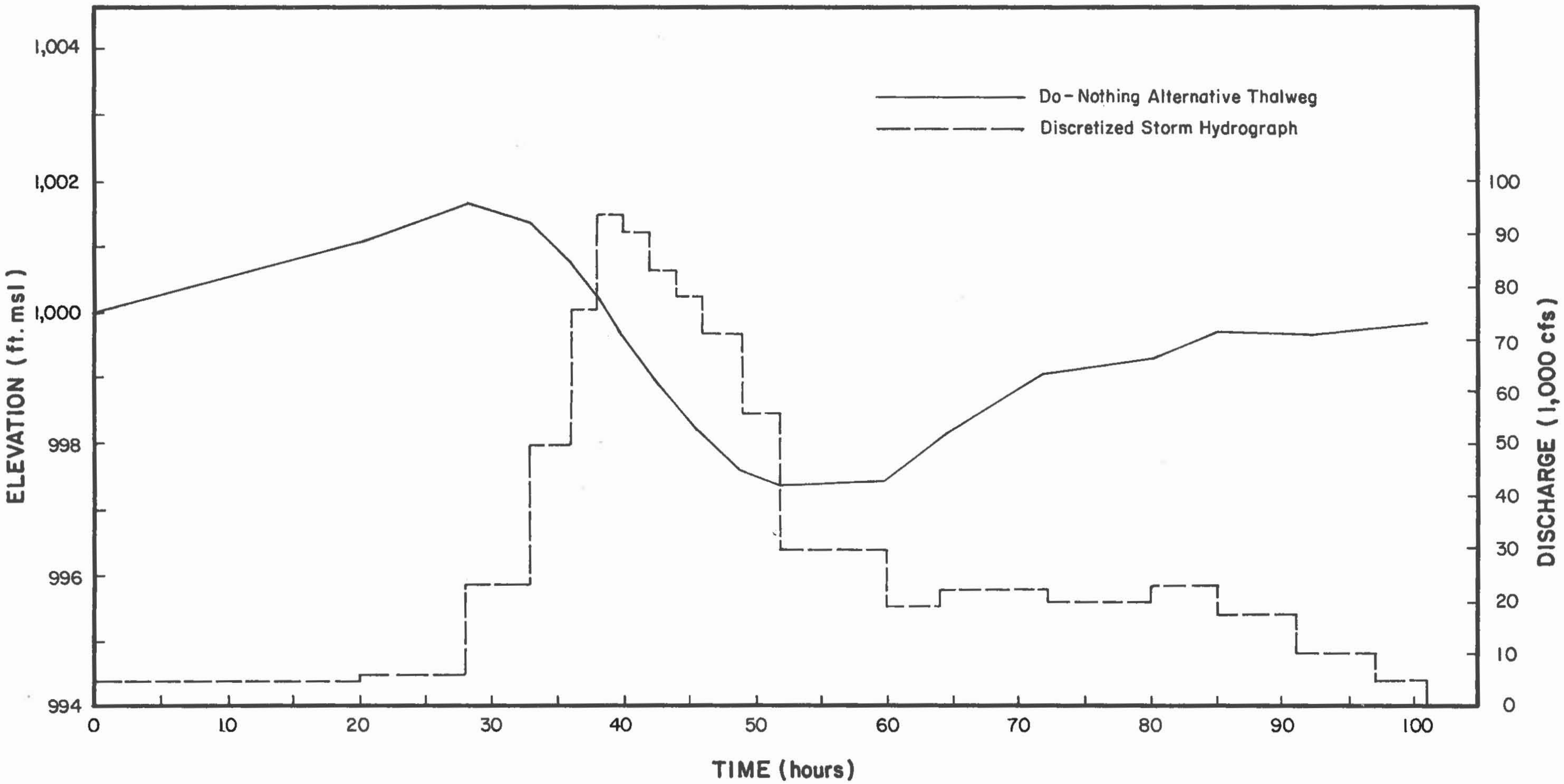


Figure 7.8. Thalweg elevations at ISRB for 100-yr storm.

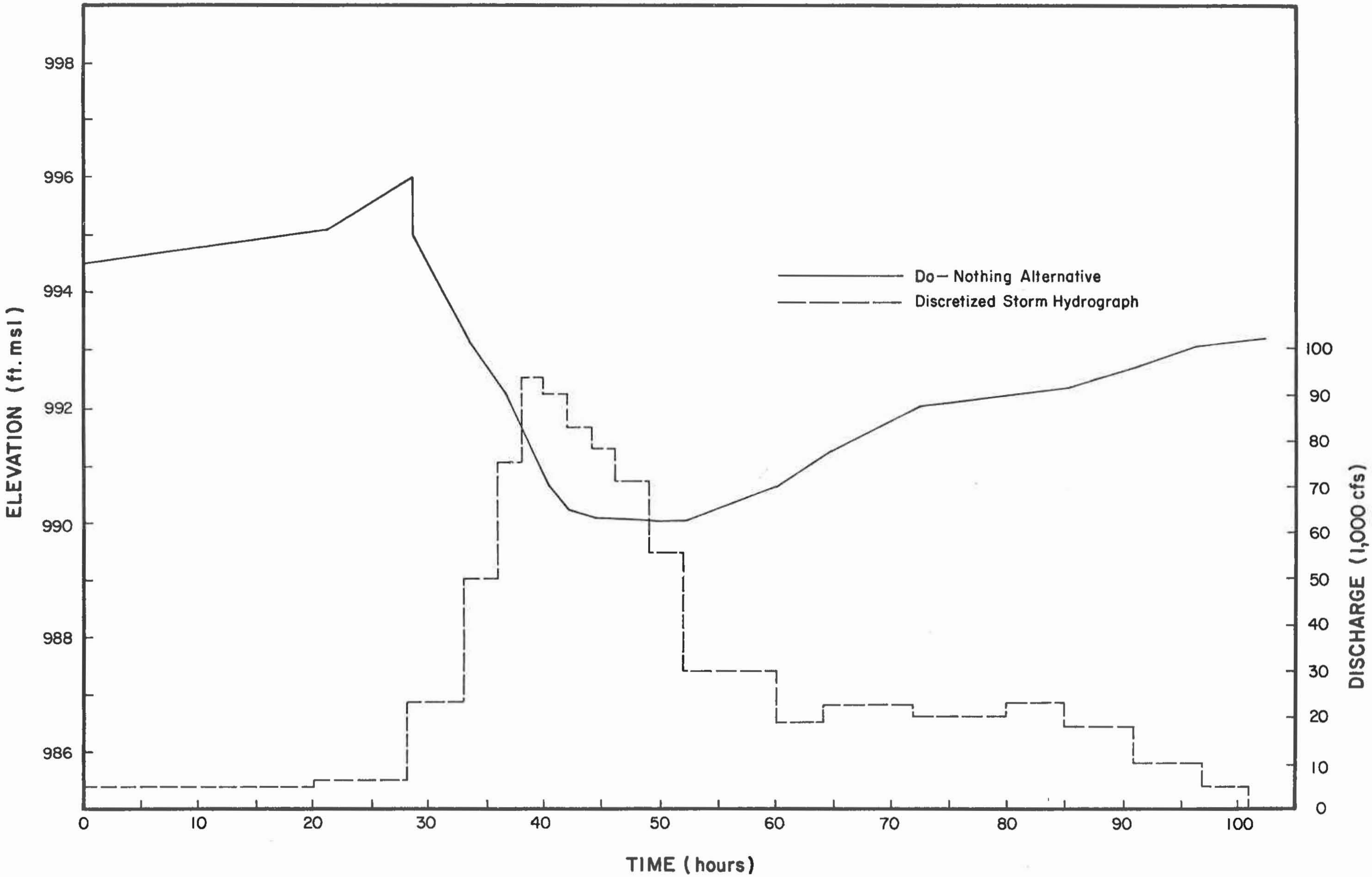


Figure 7.9. Thalweg elevations at Roosevelt Irrigation District Flume for 100-yr storm.

flume was approximately 4.0 feet. Final degradation at that location was about 2.5 feet.

Figures 7.10 and 7.11 show the changes in the percentage of sand, fine gravel and medium to coarse gravel with time at ISRB and the RID flume. These figures clearly indicate the coarsening of the surface layer as degradation occurred and the increase in the percentage of fine material as the deposition occurred during the recession limb.

7.4.3 SLA Channelization Alternative

Figure 7.7 showed the thalweg profile for the channelization alternative proposed by Simons, Li, and Associates, Inc. Analysis of this alternative using the QUASED model indicates that the general aggradation/degradation resulting from the 100-yr storm would be insignificant. The change in thalweg elevation throughout the study area was less than one foot. Slight degradation occurred in the transition reach just upstream of the bridge while a very small amount of aggradation occurred downstream at the bridge.

7.4.4 Total Scour Depth

The sediment routing results indicate that significant general degradation will occur in the study reach during the 100-yr storm for the "Do Nothing" alternative. As previously discussed, the model does not account for the local and general regional scour that occurs at ISRB and the RID flume. When the maximum general degradation of 2.7 feet at the bridge is added to the separately computed local scour of 50 feet, and the general regional scour as controlled by the armoring layer depth of 10.4 feet, the total scour becomes 63.1 feet. At the RID flume, the maximum general degradation was 4.0 feet. The local and general regionalized scour were 11.1 and 10.4, respectively making the total scour 25.5 feet.

The maximum general degradation predicted by the model for the SLA channelization alternative was negligible. The local scour and general regionalized scour at the bridge for this condition are estimated to be 15 feet and 4.2 feet, respectively. The values at the flume are 8.6 feet and 4.2 feet.

Comparing these values, it is clear that the mitigation alternative proposed by SLA significantly reduces the pier depths required for stability of the bridge and flume.

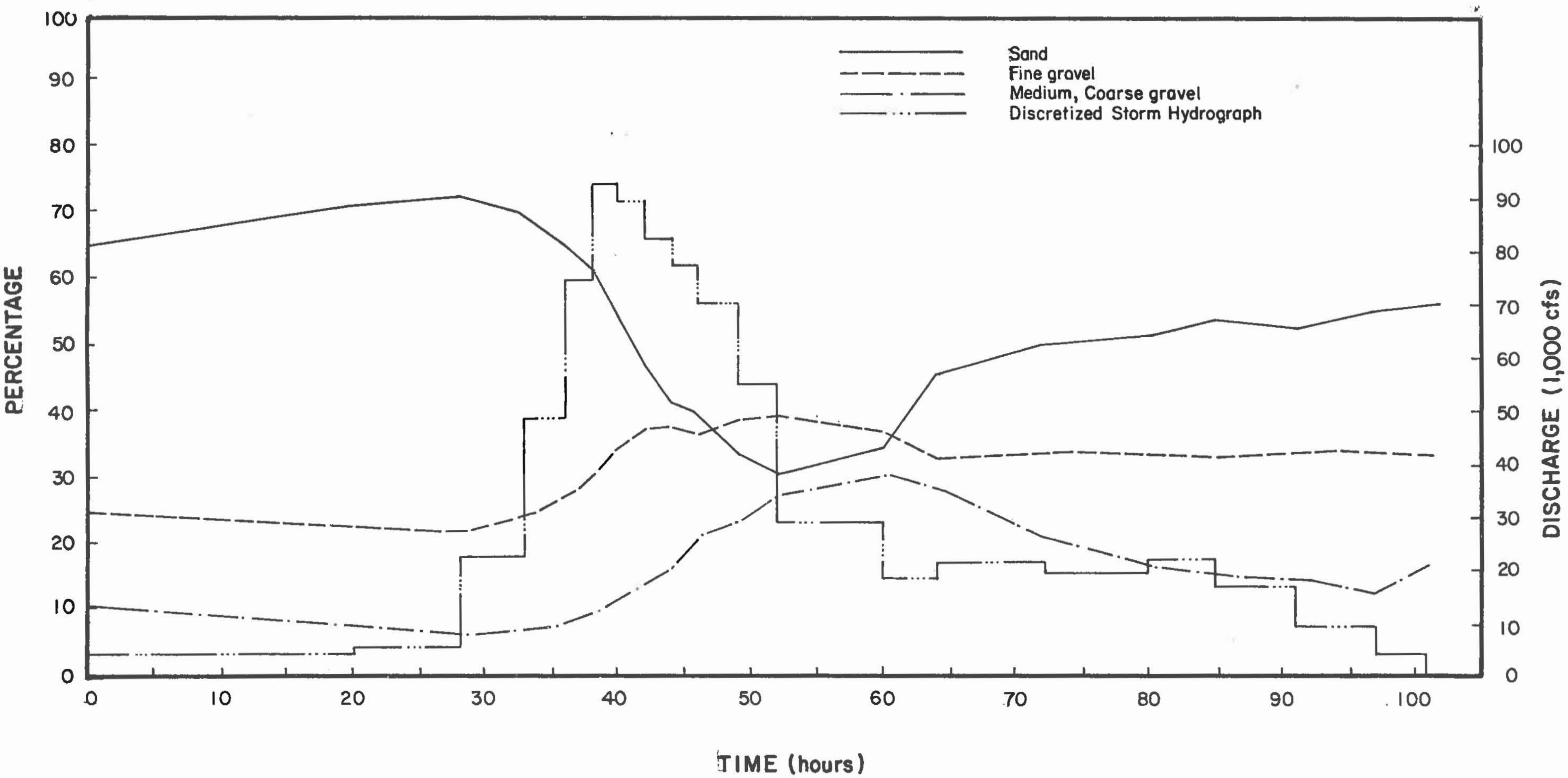


Figure 7.10. Percentage of material by size in surface layer at Indian School Road Bridge.

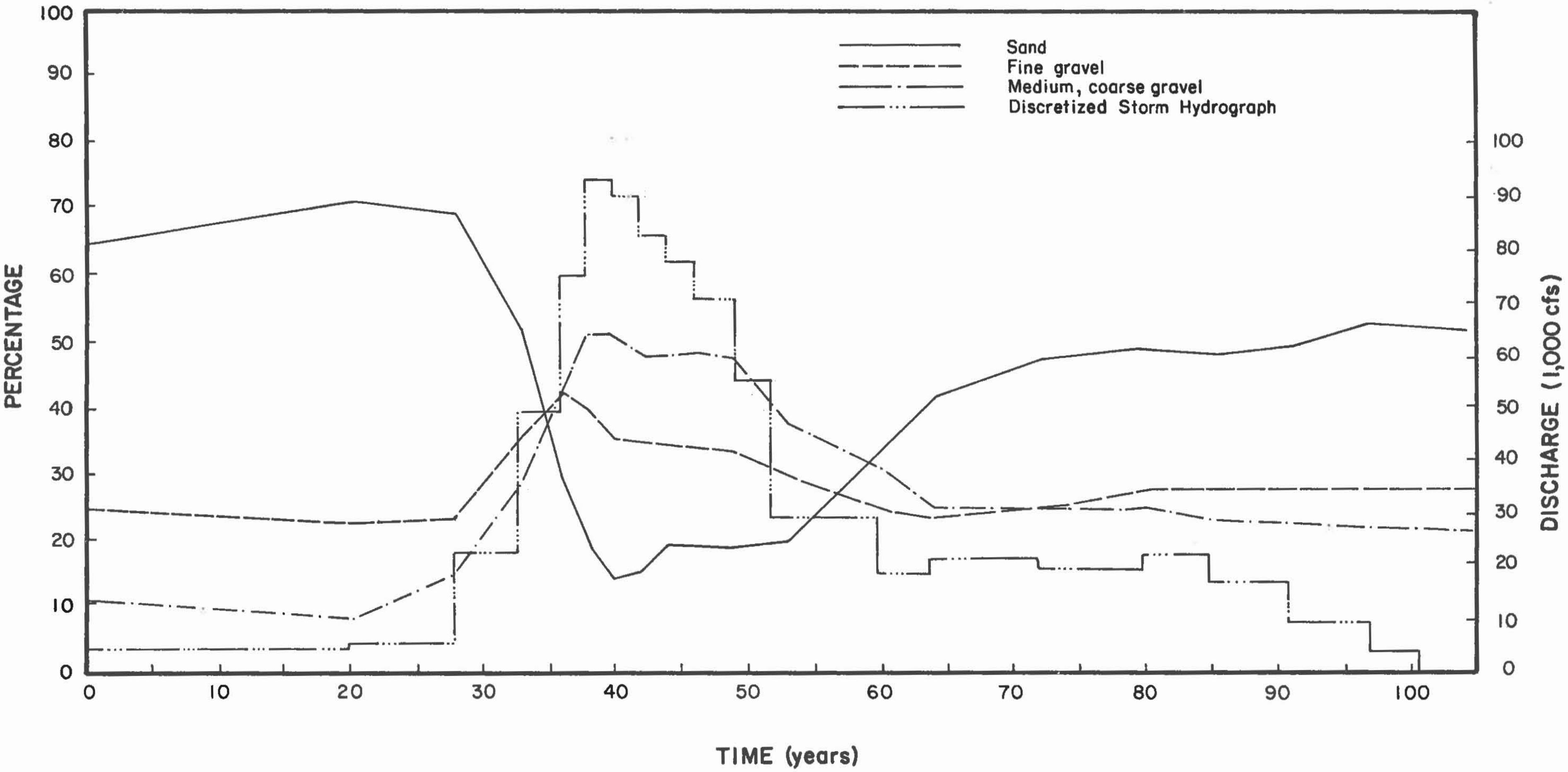


Figure 7.11. Percentage of material by size fraction in surface layer at Roosevelt Irrigation District Flume.

VIII. SUMMARY AND CONCLUSIONS

The following set of conclusions are based on the bridge failure analysis.

1. The sedimentation analysis is based on the discharge flows at Avondale, which is located three miles downstream of Indian School Road Bridge. The peak discharge considered for the February 20, 1980, flood was 42,000 cfs, which is approximately equivalent to a flood with a return period of 25 years. This would not constitute an uncommon event.

2. Developments upstream of Indian School Road Bridge, such as Waddell Dam, Dreamy Draw Dam, and Cave Buttes Dam, had no significant degradation effect at the bridge due to the considerable distance between the dams and the bridge.

3. The Agua Fria is a braided river. Aggradation is a typical characteristic of most braided rivers. For the premining condition the qualitative and quantitative analysis shows aggradation tendencies near Indian School Road Bridge. However, for the mining condition the Agua Fria has been degrading in the study reach. This is verified by examining the thalweg profile in 1957, 1973 and 1980.

4. The Indian School Road Bridge piers were buried approximately 25 feet below the river bed. The bridge was supported on spread footings, which won't fail unless they are undermined. Unlike friction piles, which carry part of the design load due to friction between the piles and soil, the spread footing bears the load <u>at the bottom of the pile</u> and failure is most likely due to undermining <u>of the piles</u>. ? ?

5. The Agua Fria upstream of Indian School Road has migrated laterally to the east in the series of floods from 1978 to 1980. This resulted in an adverse angle of attack of flow on the bridge piers. Two methods to estimate the unit discharge through the failed bridge piers 14, 15 and 16 were used to compute the local scour at the piers and included (1) estimating the angle of attack on individual piers from examining aerial photographs of the flow, and (2) estimating the average angle of attack on the piers to be 41 degrees. Using the first method and estimating the flow angles on piers 14, 15 and 16 to be 35, 45 and 56 degrees respectively, and estimating 65 percent of the flow passes through a 220-foot opening for the mining condition, an average unit discharge of 250 cfs/ft results and a local scour of 20.2 feet occurs at the bridge piers. For the second method the average unit discharge for the mining condition is 225 cfs/ft which results in a local scour at the bridge piers of 17.5 feet. For premining conditions 60 percent of the flow passes through the 220 foot opening and results in a unit discharge of 230 cfs/ft for Method 1 and a unit discharge of 210 cfs/ft for Method 2 which subsequently results in local scour depths of 21.1 feet and 19.9 feet respectively at the bridge piers.

6. The total scour at bridge piers 14, 15 and 16 was estimated to be 29 feet by Sergent, Hauskins and Beckwith. The total scour computed by SLA including the components of local scour (20.2 feet), general regionalized

scour (6.4 feet), and general aggradation/degradation (2.6 ft degradation) was 29.2 feet for the mining condition. For the premining condition the total scour computed at the bridge was 21.1 feet and with coarser gravel materials up to 2.5 inches in diameter present, the scour at the bridge for the premining condition would have been 17.3 feet. Without the levees in the downstream reach between Indian School Road Bridge and Roosevelt Irrigation District flume, the bridge would not have failed.

7. The SLA mathematical model simulated the general aggradation and degradation response of the mining condition remarkably well. A degradation of 4.2 ft was modeled at the bridge for the mining condition and, when added with the general regional scour of 6.4 ft and the local scour of 20.2 ft, the total scour at the bridge becomes 30.8 ft. For the premining condition at the bridge, the bed aggrades 0.5 feet, and when added to the general regional scour of 0.5 feet and the local scour of 21.1 feet, the total scour at the bridge for the premining condition becomes 21.1 feet. This reinforces the engineering geomorphic conclusion that for premining conditions the bridge piers would not have been undermined.

The following set of conclusions are based on the mitigation analysis:

1. From qualitative evaluation, the "do nothing" alternative will have significant degradation, PRC Toups design alternative will increase the stability of the channel significantly in the vicinity of the bridge but will have severe degradation problems at RID flume (PRC Toups did not consider the stability of the flume in their design), and SLA design alternative will stabilize the entire study reach adequately.

2. The choking of the flow due to gravel mining levees will be eliminated with SLA design alternative.

3. The analysis of total scour considering degradation, general scour and local scour for the three alternatives indicates that the "do nothing" alternative is definitely an unacceptable alternative. If the channel configuration remains as the existing condition, the bridge can not be repaired economically, environmentally, and technically due to extremely large scour potential. The proposed channelization by SLA will have a total expected scour depth of 19.2 feet at ISRB which is approximately what the piers are presently buried. To provide adequate protection of the bridge, piers 14, 15 and 16 should be buried 25 feet below the existing grade.

4. A grade control structure near the RID flume crossing is required to protect the ISRB and RID flume, particularly the RID flume.

5. Riprap protection either in the form of cone or apron should be placed for ISRB piers and RID flume piers.

6. A system of spur dikes and transverse dikes is required to guide the flow through the bridge without creating adverse angle of attack on the east bank above ISRB. A spur dike on the west bank is also required should the Agua Fria migrate laterally to the west bank. Transverse guide banks

will need to be constructed. The Agua Fria should be monitored to check for any migration tendencies.

7. SLA design alternative incorporates three modifications into PRC Toups design alternative and provides a slightly better protection for ISRB and a significantly better protection for RID flume. The modifications include: (1) a shorter ditance between the spur dike and transverse dike, (2) a larger channel width at RID flume crossing, and (3) a grade control structure downstream of the RID flume crossing.

8. Remove the filler between the two wall piers. This will allow more flow between the piers and reduce the severity of the local scour.

9. A finalized design plan should be incorporated based on availability of materials, economics, aesthetics, maintenance requirements and required time to complete construction.