AZ-DM-03 &
AZ-DM-02

REPORT

HYDRAULIC AND SCOUR ANALYSIS OF SALT RIVER BRIDGE
OF PHOENIX-CASA GRANDE HIGHWAY
FOR DESIGNING LONG-TERM PROTECTION AGAINST SCOUR

Prepared for

Dames and Moore
Phoenix, Arizona

by

Daryl B. Simons
Principal Engineer

Ruh-Ming Li
Principal Hydraulic Engineer

Lan-Yin Li
Senior Hydrologist

Simons, Li & Associates, Inc.
PO Box 1816
Fort Collins, Colorado 80522

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TABLE OF CONTENTS

I. INTRODUCTION

1.1 General

The Salt River has been subjected to repeated floods over the past few years. Local scour and general bed degradation during successive flows have altered the river bed over and around the piers of the Salt River Bridge, and the Arizona Department of Transportation has concluded that the bridge may be susceptible to further damage in the future. An analysis of the susceptibility of the pier foundations to scouring during future floods is required for evaluation of alternative structural and/or non-structural methods that may be selected for protection of the bridge foundation against scour. The structural methods include (1) channelization using guide banks, (2) a downstream grade control structure, and (3) control of side drainage flows. Non-structural measures include (1) control of gravel mining and (2) operation of upstream reservoirs to regulate the flow.

Presented in this report is the final hydraulic and scour analysis of the Salt River Bridge. A conceptual design for scour protection measures, including integration of the channelization plan, downstream grade control structure and control of the inflow from side drainage, is recommended. The right-of-way requirements with and/or without the grade control structure considering sand and gravel mining are also estimated. The specific scope of work follows.

1. Collect, collate, synthesize, verify and digitize new topographic, cross-sectional, and structural data pertinent to the study.
2. Compile and develop a revised spatial representation system that represents the study reach. This revised spatial design will provide a line diagram showing the river mile, new cross-section numbers, and location of structures.

3. Prepare model input data files for the evaluation of two basic alternatives. The interim two basic alternatives include: (a) the as-is condition and (b) the proposed channelization plan with grade control structures.
4. Evaluate river response of these two basic alternatives. This analysis will evaluate the potential scour problems associated with the bridge subsequent to construction of the long-term channelized channel. Also, the analysis will evaluate the impacts of the passage of a design flood with peak flow rate of 176,000 cfs.
5. Explain physical processes governing mechanics of the gravel pit during low, medium and high flows, considering degradation, headcut upstream and downstream of the pit, and the significance of the depth, size and volume of the pit.
6. Evaluate the response for assumed storm hydrographs and hypothetical gravel pits.
7. Suggest right-of-way requirements necessary to protect the bridge considering conditions with and without grade and flow control structures.
8. Recommend a channelization plan and grade control structures. Review the subsequent design by Dames and Moore.
9. Develop a water surface profile versus flow rate curve for the bridge crossing that will include consideration of the capacity of the interim low flow channel.
10. Prepare a final report documenting the results of the analysis and recommendations.

1.2 Available Information

The analysis presented in this report is based on the following information.

1. "Flood of November 1965 to January 1966 in the Gila River Basin, Arizona and New Mexico, and Adjacent Basins in Arizona," Geological Survey Water Supply Paper 1850-C.
2. "Central Avenue Bridge Hydraulic Study," Southwest Computing Service, Inc., July, 1973.
3. "Consultation During Design and Performance of Grouting Program Foundation Stabilization for Flood-Damaged Piers, Salt River Bridge, Phoenix-Casa Grande Highway (Interstate 10), Phoenix, Arizona," Dames & Moore, June 14, 1979.
4. "Scour at Bridge Waterways," National Cooperative Highway Research Program, Synthesis of Highway Practice 5, Highway Research Board, 1970.
5. "Guide to Bridge Hydraulics," edited by C. R. Neill, University of Toronto Press, 1973.
6. "Manual on River Behavior, Control and Training," Control Board of Irrigation and Power, India, 1956.
7. "Hydraulic Model Studies of Spur Dikes for Highway Bridge Openings," Report No. CER59-SSK36, Colorado State University, 1959.
8. "Sediment Transport Technology," Water Resources Publications, by D. B. Simons and F. Senturk, 1977.
9. "Investigation of Meyer-Peter, Muller Bedload Formula," Sedimentation Section, Hydrology Branch, Division of Project Investigations, U.S. Department of the Interior, Bureau of Reclamation, June, 1960.
10. "Open Channel Hydraulics," McGraw Hill Book Company, by V. T. Chow, 1959.

11. "Hydraulic Model Study of Flow Control Structures," Phase I Report, prepared for USDA Forest Service, Angeles National Forest, Pasadena, California, and Rocky Mountain Forest and Range Experiment Station, Flagstaff, Arizona, by R. M. Li, D. B. Simons, T. J. Ward, and K. S. Steele, November, 1977.

12. "HEC-2 Water Surface Profiles" Programmers Manual. Hydrologic Engineering Center, U.S. Army Corps of Engineers, November, 1976.

13. "HEC-2 Water Surface Profiles" Users Manual with Supplement, Hydrologic Engineering Center, U.S. Army Corps of Engineers, November, 1976.

14, "HEC-6 Scour and Deposition in Rivers and Reservoirs" Users Manual, Hydrologic Engineering Center, U.S. Army Corps of Engineers, March, 1976.

15. "Flow Resistance in Cobble and Boulder Riverbeds," by D. B. Simons, K. S. Al-Sheikh-Ali, and R. M. Li, Journal of Hydraulics Division, American Society of Civil Engineers, Vol. 105, No. HY5, 1979.

16. "Degradation Below the Emergency Spillway Chute of the Site 8C, T or C Williamsburg Watershed, New Mexico." Prepared for USDA Soil Conservation Service, Albuquerque, New Mexico, by D. B. Simons and R. M. Li, March, 1978.

17. "Erosion and Sedimentation Analysis of San Juan Creek near Conrock Gravel Pit, Orange County, California." Prepared for Dames and Moore, Denver, Colorado, by D. B. Simons and R. M. Li, June, 1978.

18. USGS quadrangle map along the Salt River in the vicinity of Phoenix, Arizona.

19. 1975 topographic maps - Maricopa County.

20. 1979 topographic maps in the vicinity of I-10 Salt River Bridge - City of Phoenix.
21. 1979 cross-sectional profile at the downstream edge of the I-10 Salt River Bridge.
22. Aerial photographs since 1956.
23. Bed material size distribution curves for surface and subsurface material from Dames and Moore in 1979.
24. 1980 topographic maps.
25. 1980 cross-sectional survey in the vicinity of I-10 by Dames & Moore.
26. Survey of sand and gravel mining in the vicinity of I-10.
27. Stage-discharge relation and other pertinent data from February, 1980, flood.

II. DATA SUMMARY

2.1 General

The data required to conduct this analysis include: hydrology (flood hydrograph), channel geometry, bed-material size distribution, bridge data, resistance to flow, and sediment transport rates. The basic data and related information utilized in this study are the same as that used in the Phase I study submitted in 1979. New topographic maps, cross-sectional data and the proposed channelization and grade control structure plan were used to update the data. In addition, the bed material size distribution for both surface and subsurface samples was verified to be very close to that obtained in 1979. For clarification, the data utilized are presented again in this report.

2.2 Hydrology

A description of floods in the Salt River from November 1965 to January 1966, is given by the U.S. Geological Survey. This report substantiated the possibility of major floods such as the recent flood events. Such events may be experienced again in the future. Six floods have occurred in the Salt River recently. The date of occurrence, peak flow and duration of these floods are given in Table 1.

According to the report titled "Preliminary Engineering Study - 19th Avenue Bridge Over Salt River, Phoenix, Arizona," by Howard, Needles, Tammen, Bergendoff, Johannessen and Girand (1979) (HNTB and JG), the design floods with different return periods are given in Table 2. The design floods in a preliminary analysis by the U.S. Army Corps of Engineers are also included for comparison. The magnitudes of design floods are substantially different for the floods with shorter return periods.

The peak discharge of the design flood adopted by ADOT is 176,000 cfs, and the estimated peak discharge for the February 1980, flood is 185,000 cfs. However, the actual hydrograph was unavailable. The best way to predict the flood hydrographs for the study area is to employ a more advanced theoretical approach involving rainfall-runoff relationships and numerical routing techniques. However, due to time and money constraints, a more practical means is used for estimating the shape of the hydrograph for the design flood. This method involves the development of a typical hydrograph normalized with respect to the flood peak. Figure 1 shows the normalized hydrographs based on the available flow data recorded on 3/1/78 and 12/15/78. These two normalized hydrographs are sufficiently similar and the resulting average normalized hydrograph is selected as the typical hydrograph for this analysis. The design flood utilizing the typical hydrograph has a peak discharge rate of 176,000 cfs and a duration of 11 days (see Figure 2). This design flood is very close to the actual flood of February 1980, which had a peak flow of 185,000 cfs and an effective duration of 15 days. Flood hydrographs for different return periods were generated utilizing the same procedure.

Table 1. Summary of Recent Floods.

Starting Date	Peak Flow Rate (cfs)	Duration (days)
3/1/78	99,000	8
12/15/78	112,000	24
1/17/79	73,500	30
3/11/79	52,000	37
2/16/80	185,000	15

Table 2. Design Flood.

Return Period	Peak Flow Rate, cfs	
	HNTB and JG	Corps of Engineers
10	47,000	92,000
25	87,000	140,000
50	130,000	160,000
100	124,000	200,000

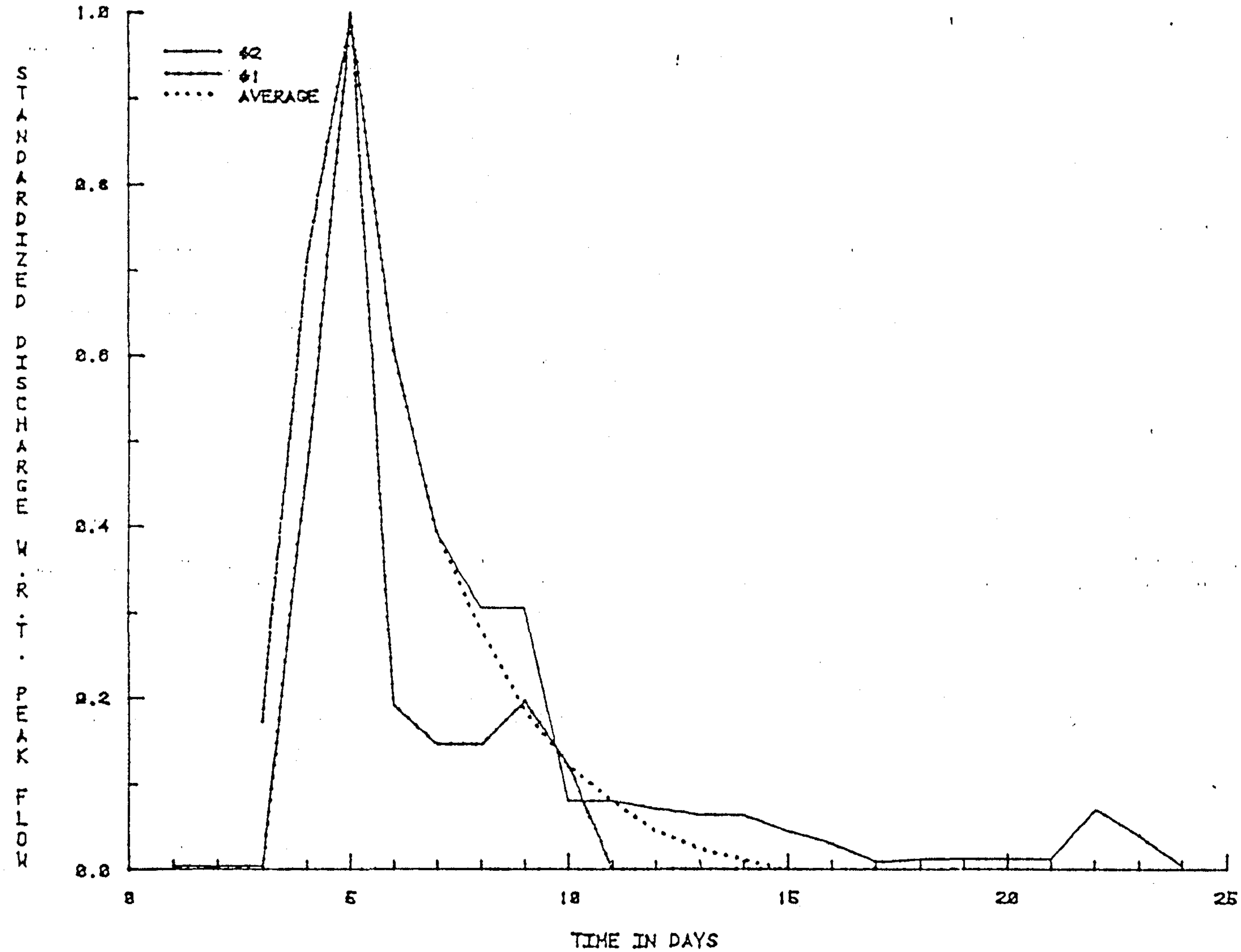

Figure 1. Hydrographs for Salt River.

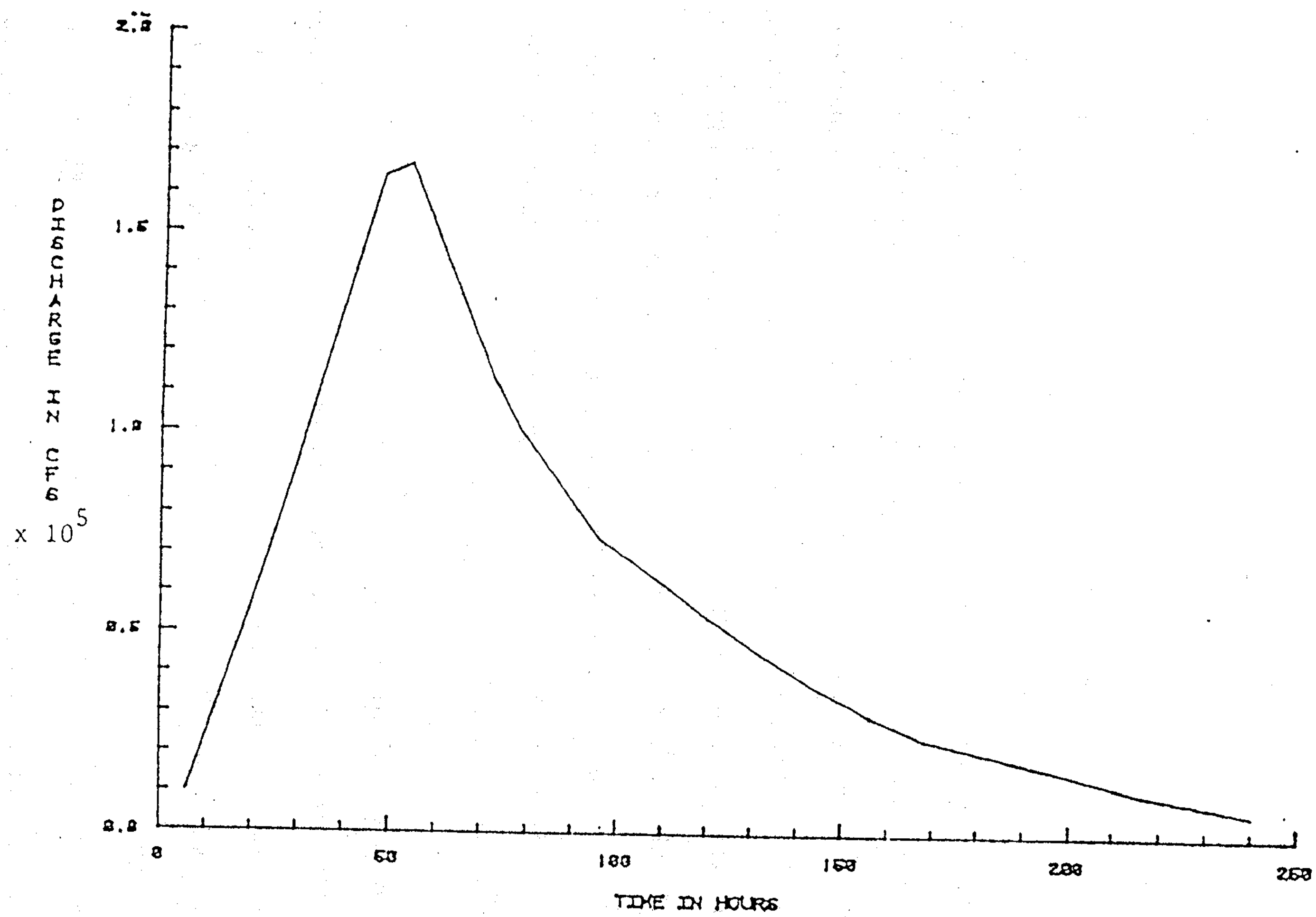

Figure 2. Design flood hydrograph.

2.3 Channel Geometry

The index map showing the location of cross sections and reaches is the system subjected to analysis and is given in Figure 3. The total number of cross sections utilized in the analysis of the as-is condition is 22. The total length of the reach of river analyzed is approximately 5.4 miles. The study area extends about 2.6 miles upstream of the Salt River (I-10 Bridge) and 2.8 miles downstream of the bridge. A downstream control is assumed to be located at the Seventh Street bridge crossing. These cross sectional data were digitized from the 1980 survey by Dames and Moore. The flood plain cross-sectional data were augmented by using aerial photographs. The channelization condition requires three additional cross sections to describe the proposed design. These cross-sectional data are changed according to the design. Cross sections No. 8 through 13 for the as-is condition are plotted in Figures 4 through 9. These cross sections are in the vicinity of I-10. The elevations shown in the figures are mean sea level (MSL). In order to simplify the degradation and aggradation analysis, it is necessary to define specific reaches. Hence, eight reaches are defined (see Figure 3). The I-10 Bridge is in Reach No. 4. The average channel gradient is about 0.002, which is fairly steep for a large river.

2.4 Bed Material Size Distribution

Both surface and subsurface samples of the bed material were collected and analyzed by Dames and Moore in 1979. The composite size distributions for both surface and subsurface samples are given in Figures 10 and 11. They were recently verified to have had no significant change during the 1980 flood by Dames and Moore. The surface sample shows a significant layer or armor. Field observations verify that it is difficult to collect representative samples of bed material in the Salt River. For simplicity, the following

River Distance	Cross Section Number	Location	Reach Definition
28,480	22	← Upstream Boundary	
25,880	21		8
24,195	20		
23,545	19		7
21,580	18		
20,110	17		
18,060	16		6
17,030	15		
16,000	14		
15,490	13		5
14,990	12		4
14,890	11	← I-10 Bridge	
14,790	10		
14,360	9		
13,945	8		3
12,845	7		
11,145	6		
9,445	5		
7,380	4		2
5,280	3		
2,600	2		1
0	1	← Downstream Boundary (Seventh Street)	

Figure 3. Index map for the Salt River in the vicinity of the I-10 Bridge (as-is cross section).

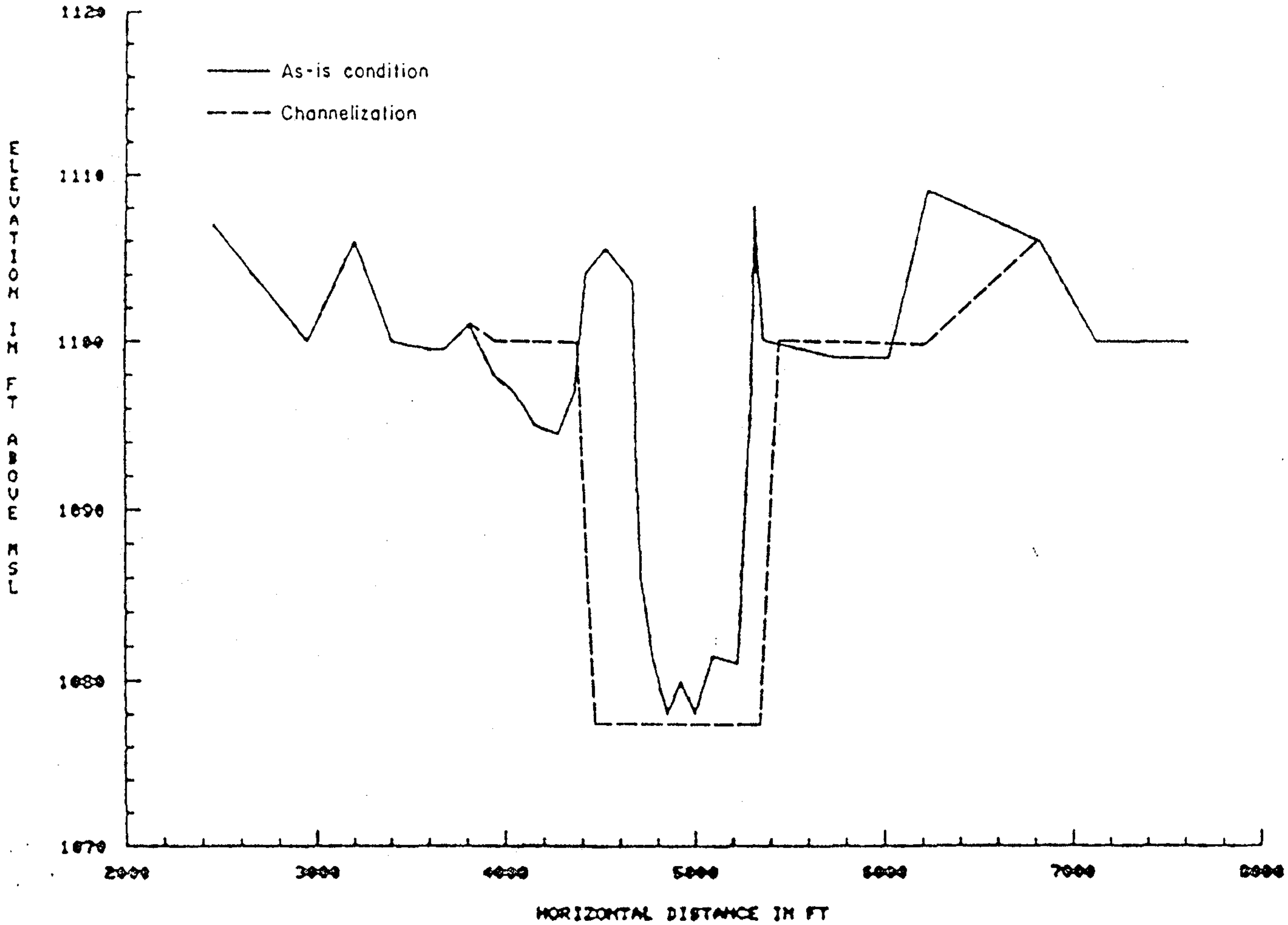

Figure 4. CROSS-SECTION PLOT #13945. 1980 SALT RIVER I-10

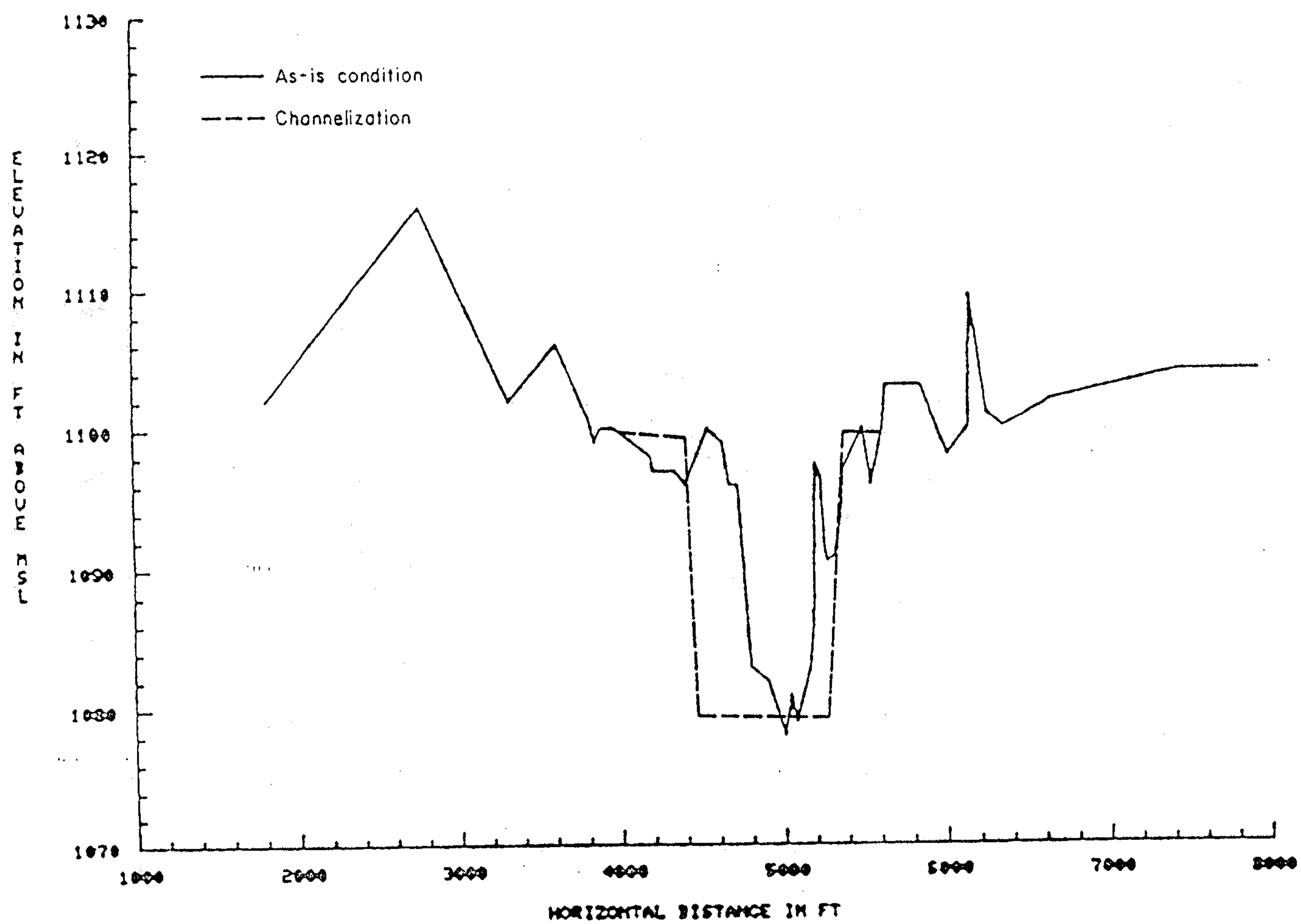

Figure 5. CROSS-SECTION PLOT #14360. 1980 SALT RIVER I-10

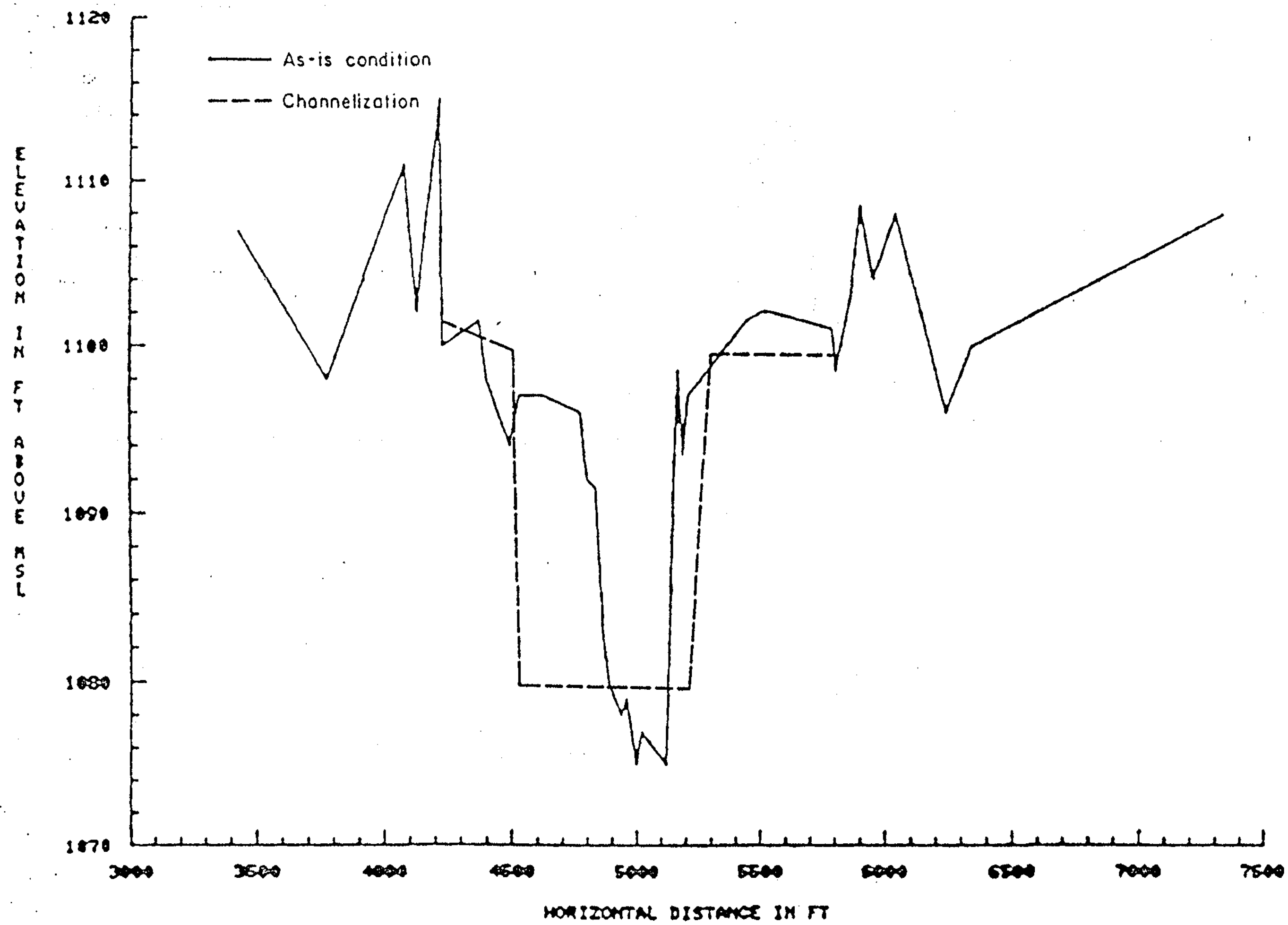

Figure 6. CROSS-SECTION PLOT #14790. 1980 SALT RIVER I-10

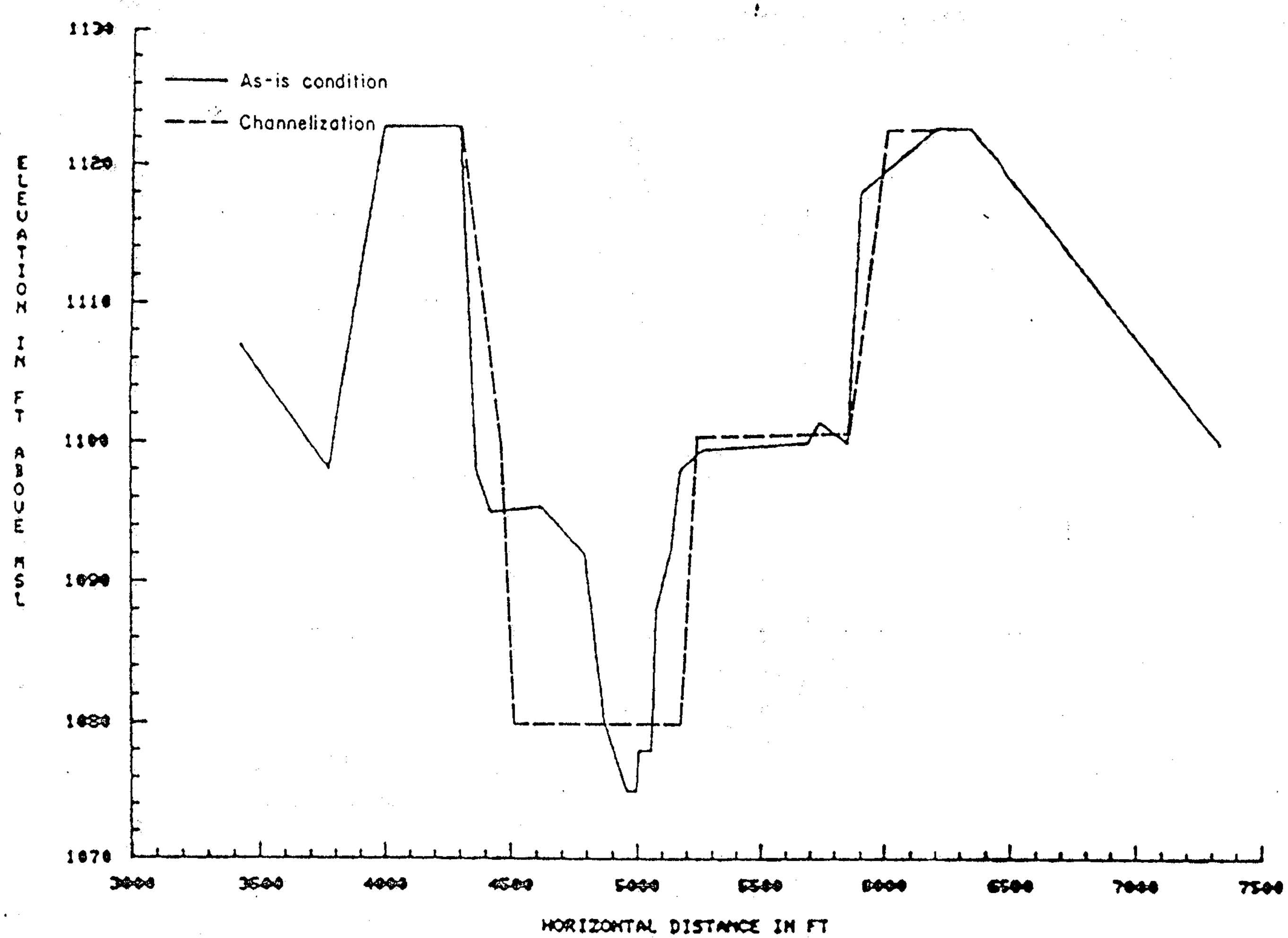

Figure 7. CROSS-SECTION PLOT #14890. 1980 SALT RIVER I-10

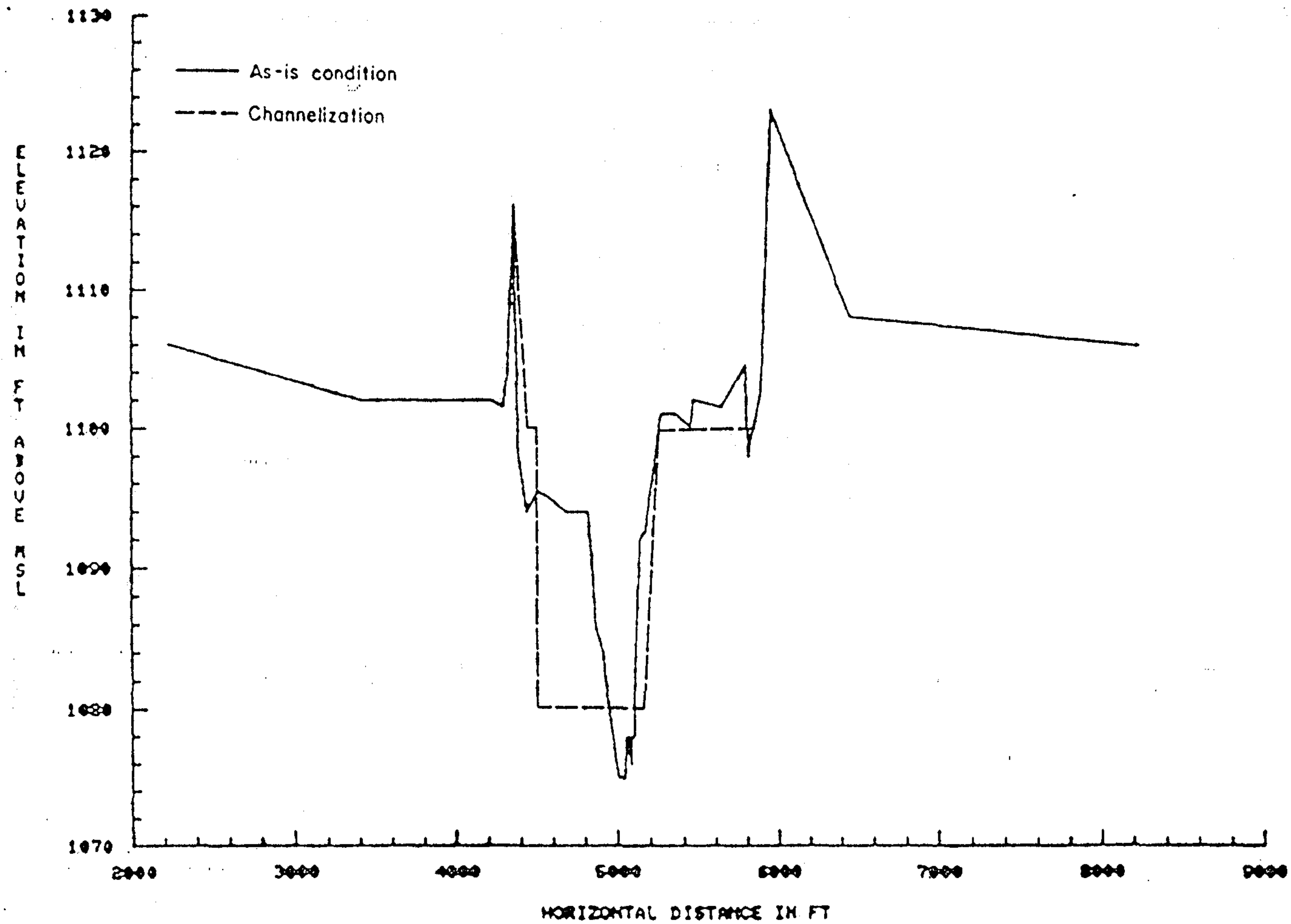

Figure 8. CROSS-SECTION PLOT #14990. 1980 SALT RIVER I-10

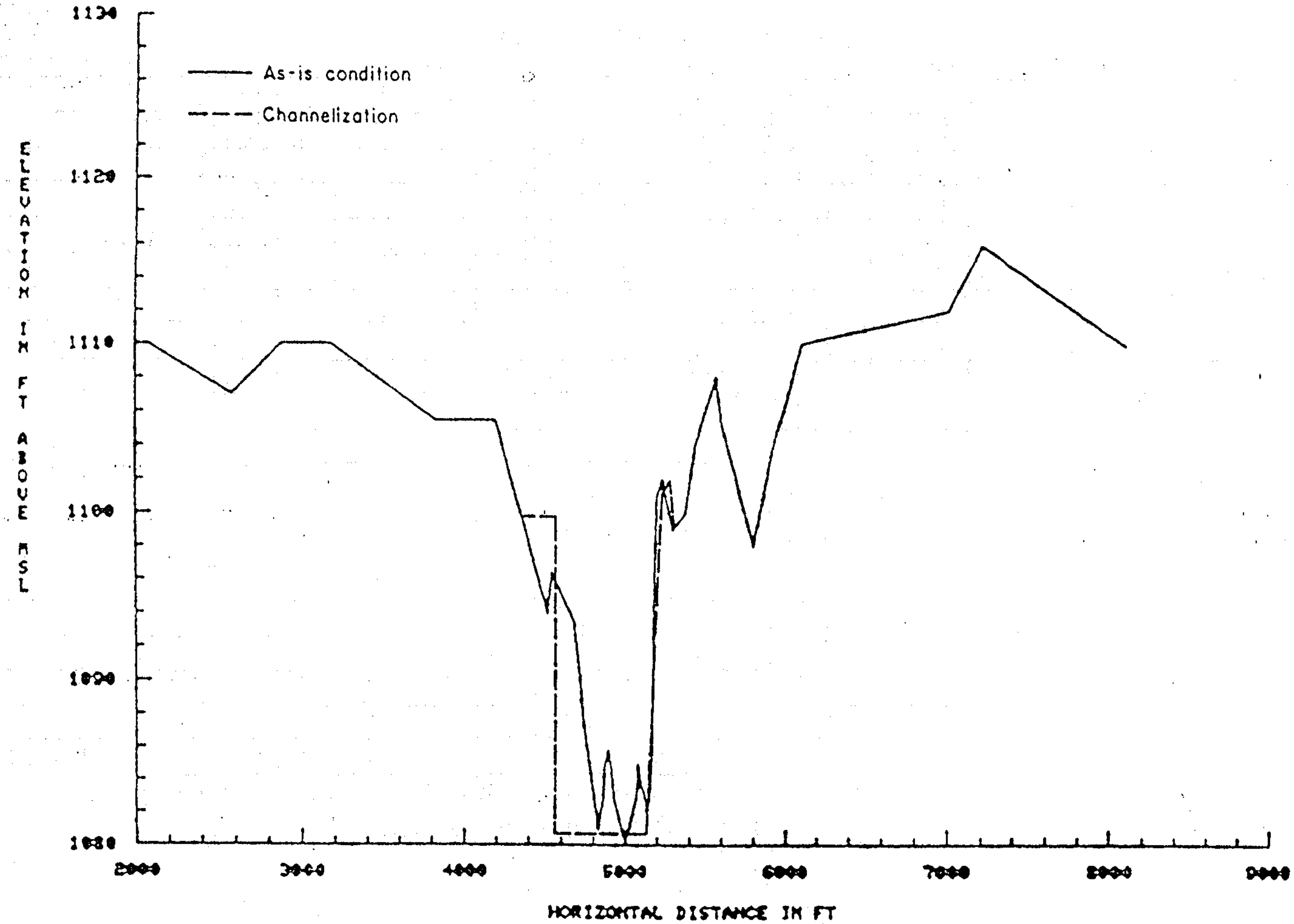

Figure 9. CROSS-SECTION PLOT #15490. 1980 SALT RIVER I-10

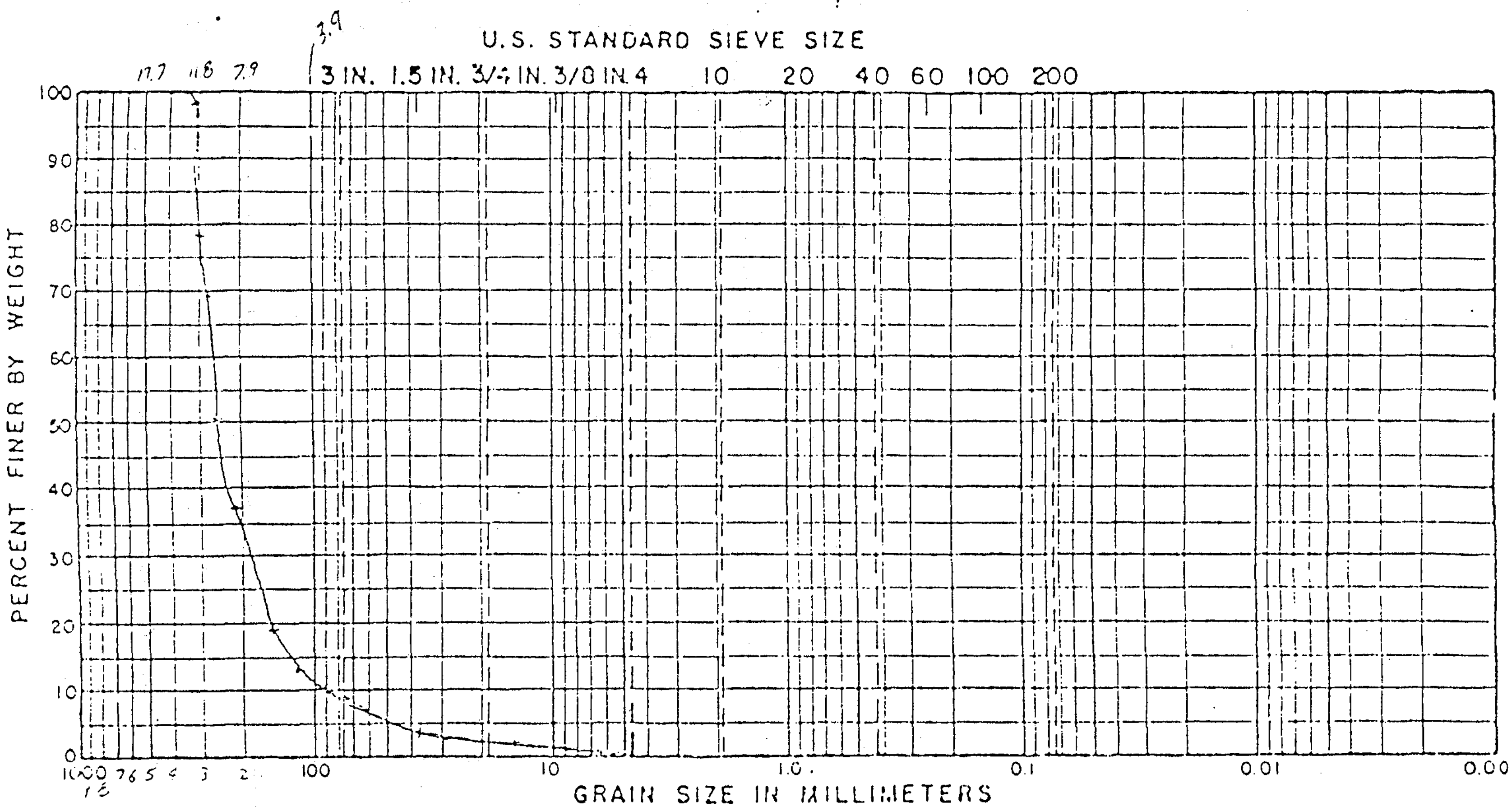

Figure 10. Gradation curve for composite surface sample.

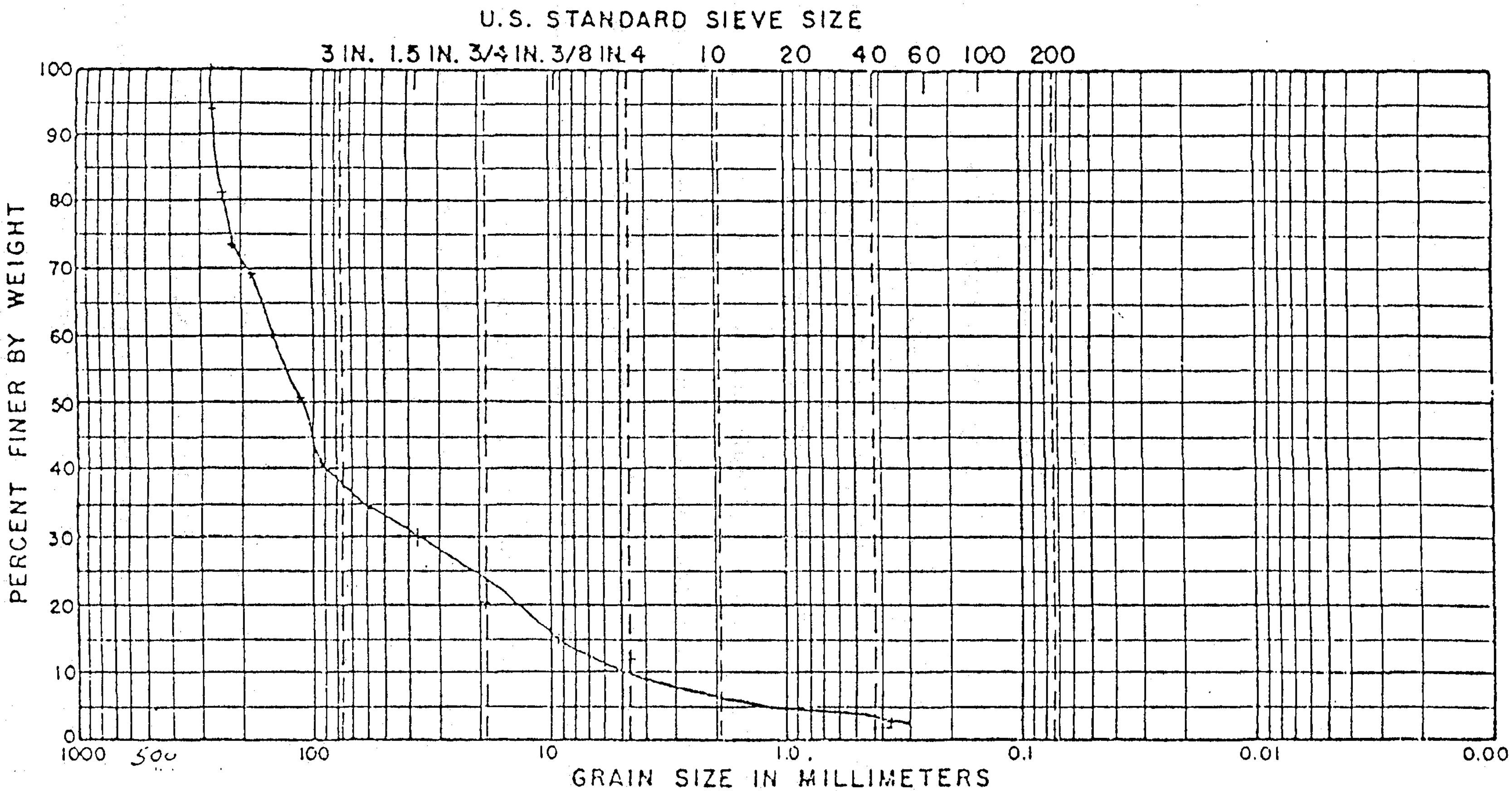

Figure 11. Gradation curve for composite subsurface sample.

characteristics of the bed material are adopted for the scour analysis. The surface layer has a D_{50} (median diameter) of 237 mm and a σ (gradation coefficient) of 1.6. The subsurface layer has a D_{50} of 123 mm and a σ of 7.0.

2.5 Bridge Data

All of the bridge data, except for the I-10 Bridge in the study reach, are missing. There are 19 piers supporting the I-10 Bridge. The elevations of the pier foundations are given in Table 3. As shown in Table 3, the low flow channel was originally located between Piers No. 1 and 9. Since that time, the low flow channel has shifted to the south and caused severe scour and pier foundation problems at Pier No. 11 in 1979. The shifting of the low flow channel and the present flow alignment is the result of past flows and upstream and downstream activity, particularly the airport encroachment and the significant gravel mining in the right flood plain upstream. In addition, the uncontrolled flow entering the main channel near the bridge from the right bank (Tempe Drain) and the obstruction caused by instream power line tower foundations all contributed to the shifting of the channel. The scour is partially a result of upstream obstructions, flood plain encroachment, gravel mining, and channelization changes caused by uncontrolled flow. The realignment of the current channel to the old low flow channel portion seems to be the most feasible solution to the pier foundation scour problem.

2.6 Resistance to Flow

It is difficult to accurately estimate resistance to flow in a gravel-boulder bed channel. The relative roughness can change greatly during a flood. According to Chow (1959), the normal value of Manning's roughness coefficient n is about 0.04 for a small mountain stream with no vegetation in the channel, banks usually steep, trees and brush along banks submerged at high stages, and a channel bottom consisting of gravel, cobbles, and a few

Table 3. Elevation of Pier Foundations.

Pier No.	Foundation Elevation (ft)
1	1072.1
2	1072.4
3	1072.6
4	1072.9
5	1073.1
6	1066.7
7	1066.8
8	1066.9
9	1067.0
10	1073.5
11	1077.0
12	1078.4
13	1078.3
14	1078.2
15	1078.1
16	1077.9
17	1077.7
18	1077.4
19	1077.1

boulders. This description is close to the Salt River, except that the Salt River is not a small mountain stream. It is a large intermittent river, so the Manning's roughness for the Salt River bed will therefore be less than 0.04. The minimum value for this type of stream is about 0.03.

Strickler's formula (Simons and Senturk, 1977) is often used to estimate the grain resistance. Assuming that the D_{50} is 237 mm, the Manning's n is 0.03 according to Strickler's formula. A recent study at Colorado State University by Simons, Al-Shaikh-Ali and Li (1979) indicated that the passage of a sediment wave during the flood can significantly reduce the resistance to flow. In addition, the stream bed is likely to be in upper regime during flood periods (see Simons and Senturk, 1977). For a conservative estimate of potential scour, a Manning's n value of 0.03 is utilized for the analysis of the main channel. Manning's n for the portion of the flood plain comprised of gravel, boulders and sand is assumed to be 0.05, and for the portion of the flood plain occupied by buildings, the n value is assumed to be 0.1.

2.7 Sediment Transport Rates

The rate of sediment transport as related to channel aggradation and degradation is perhaps the most important factor when conducting the sedimentation analysis.

Existing data on sediment transport rates have been collected in both laboratory flumes and in the field. However, the data are from relatively flat sand-bed channel systems and involve a relatively uniform sediment. Hence, most of the available sediment transport equations are not applicable for the Salt River conditions. However, recent laboratory studies at Colorado State University by Li et al. (1977) utilizing a steep channel (5% to 25% bed slope) and gravel and boulder bed sediment indicate that a form of the Meyer-Peter, Muller sediment transport equation (Simons and Senturk, 1977) is applicable for steep gravel and boulder bed streams. For practical application, the coefficients should be calibrated using measured field data, but since there are no pertinent data available, the original coefficients are used. Also, the Meyer-Peter, Muller type equation accounts for the bed load only, so the suspended portion of the bed material load is computed by the Einstein procedure (Simons and Senturk, 1977).

The Meyer-Peter, Muller equation as applied was developed utilizing data from gravel bed flumes and the Shields incipient motion criteria. The equation was modified to properly account for the armoring effect of coarser particle sizes. Past experience verifies that the Meyer-Peter, Muller equation and the Einstein suspended sediment procedure provide the best estimate of sediment transport for field situations such as the Salt River.

2.8 Gravel Mining Data

Gravel mining data for the Salt River are virtually unavailable. In order to evaluate the potential effect of gravel mining on the stability of the bridge, an estimate of gravel mining extraction rates is necessary.

Some gravel mining information was provided by Dames and Moore, but most was obtained by assuming the current gravel mining trends and conditions. The average daily sand and gravel extraction rate is assumed to be 3000 cubic yards per day. The potential extraction area between the 24th Street Bridge and I-10 is about 60 acres. The depth of the pit is dependent on the ground water level and cost of extraction. It is not uncommon to have 30-ft and 50-ft deep pits. For a 60-acre pit 1200 ft wide and 2178 ft long, it would take 2.6 years to extract material to a depth of 30 ft and 4.3 years to gain a depth of 60 ft.

III. AS-IS CONDITION

3.1 General

The scour potential for the as-is condition was analyzed considering the passage of the design flood. This alternative is essentially a "no action" plan that considers the bridge to be expendable, and plans for rebuilding or significant and constant maintenance after major flows. This evaluation for the as-is condition can also be regarded as the "basic" condition for comparison and design.

The analysis of the as-is condition includes the determination of the following quantities: (1) sediment yield from the upstream channel reach, (2) degradation and aggradation within reaches and by cross section, (3) local scour around the pier foundations, (4) susceptibility of the pier foundations to erosion, and (5) water surface elevation.

3.2 Sediment Yield

Due to lack of detailed information on the upland watersheds and upstream channels, a realistic and conservative assumption was utilized to determine the sediment supply from the upstream channels. The upstream supplies of sediment have been reduced by the construction of three dams on the Salt River and two dams on the Verde River. As a result, the primary source of sediment to the Salt River is from tributaries below Bartlett Dam on the Verde and Stewart Mountain Dam on the Salt River. The continuous mining of sand and gravel upstream of the I-10 Bridge can further reduce the supply of sediment to the study site. For a conservative estimate of river response, the assumed sediment yield to the study reach is largely controlled by the transport ability of the surface bed material from the upstream reach. This assumption neglects the potential supply of much finer sediment from upland watersheds. For an adequate analysis, the degradation and aggradation analysis should be extended to include the upstream channel and watershed systems. However, for this preliminary estimate, it is assumed that the most upstream reach is the upstream boundary that dictates the sediment supply.

The estimated yield during the passage of the design flood hydrograph utilizing the modified Meyer-Peter, Muller equation is approximately 46,000 tons. Assuming a value of 1.3 for the sediment bulking factor, this weight of sediment has a volume of 27,000 cubic yards. It is likely that the actual sediment yield is much higher than this estimated value. Therefore, the subsequent analysis will yield a conservative estimate of scour.

3.3 Degradation and Aggradation (General Scour)

The degradation and aggradation problem is very complicated. Simplifying assumptions are needed to obtain solutions in a practical and economical way. The dominant physical processes include water runoff, sediment transport, sediment routing, degradation, aggradation, breaking and forming of the armor layer, etc. These processes are unsteady in nature. In order to simplify the solution and to make the results of the analysis compatible with the HEC-2 flood level analysis, a known discharge assumption is used. The known discharge solution is appropriate in this study because of the short distances involved in the analysis. In addition, to save computer time, the degradation and aggradation analysis is conducted on a reach basis utilizing the average hydraulic parameters from the HEC-2 analysis. The amount of predicted aggradation and degradation is distributed to the verticals of a cross section according to the channel conveyance to yield a set of new cross sections.

The developed mathematical model routes the sediment by size fractions. The transporting capacity of each reach is determined utilizing the average hydraulic conditions of the reach and the sediment transport equation described in Section 2.7. The sediment routing procedure is accomplished by applying the sediment continuity equation and considering the size distribution of the upstream sediment supply and the bed material for both surface and subsurface layers. This water and sediment routing procedure has been applied to various practical design problems, including the Phase I and Phase II studies.

The spatial resolution of the study area was given in Figure 3. This index map shows channel reaches and river cross sections considered in the scour analysis of the I-10 Bridge site. The temporal resolution considered in the analysis is the design hydrograph (Figure 2). The simulation of scour was made using time steps of 6 hours, 12 hours, and 24 hours, depending on the rate of rise or fall of the hydrographs. A total of 25 time incrmeents was digitized. A comparison of original and final bed profile after passing the design flood is given in Figure 12. This figure illustrates that there will be significant scour in the vicinity of the bridge. This result is similar to that observed after the 1980 flood (see Figure 13). In order to provide better information about potential general scour, the time-lapse change of general scour depth at the I-10 Bridge crossing during the design flood for the as-is condition is given in Figure 14. This figure shows that the general scour follows approximately a stepwise pattern due to forming and breaking of armoring layers, temporal and spatial distribution of flow, and variation of upstream sediment supply. Large erosion rates would not necessarily coincide with large flow rates. The aggradation would take place during the recession limb of the hydrograph. This is physically sound, considering the mechanics of erosion and sedimentation.

Local scour must be considered in the analysis of the safety of the bridge. The flow of water, the distribution of the flow, and the quantity of sediment and debris will offset local scour in the vicinity of the piers supporting the bridge. The local scour was estimated by the following four

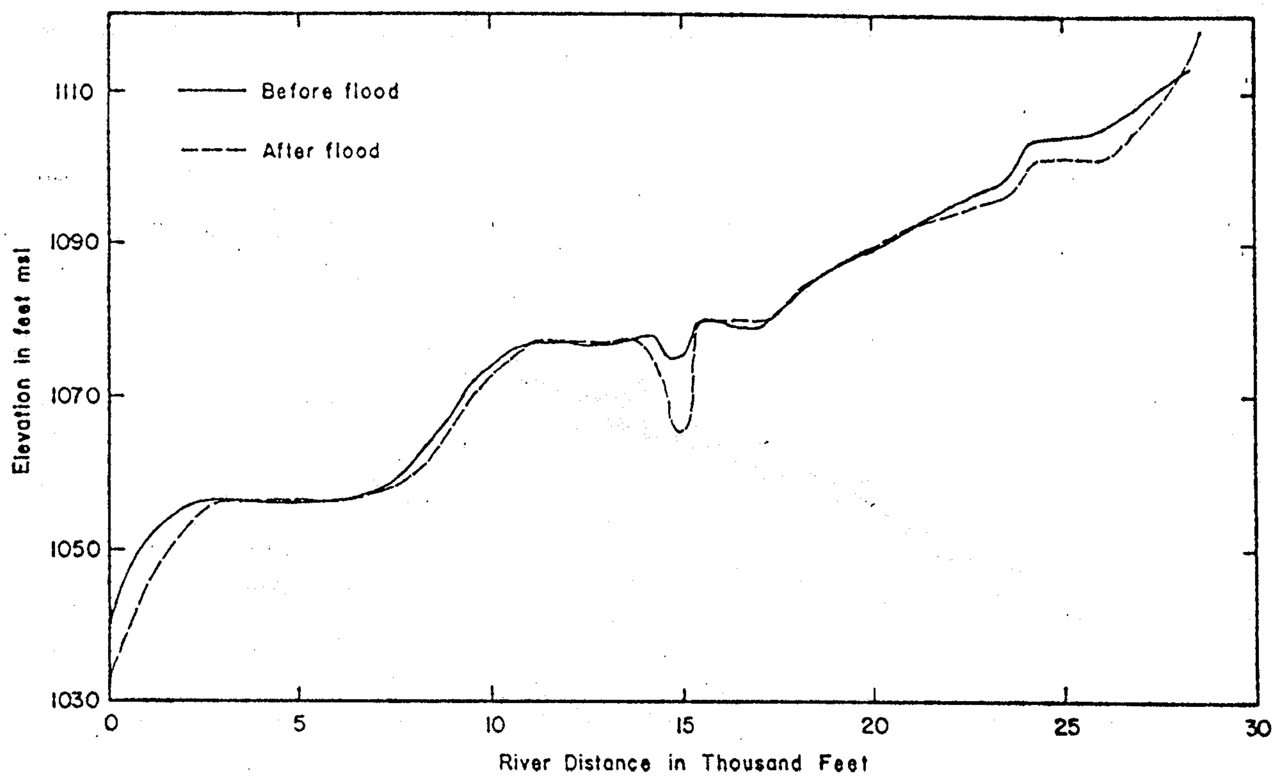

Figure 12. Change of thalweg profile due to design flood.

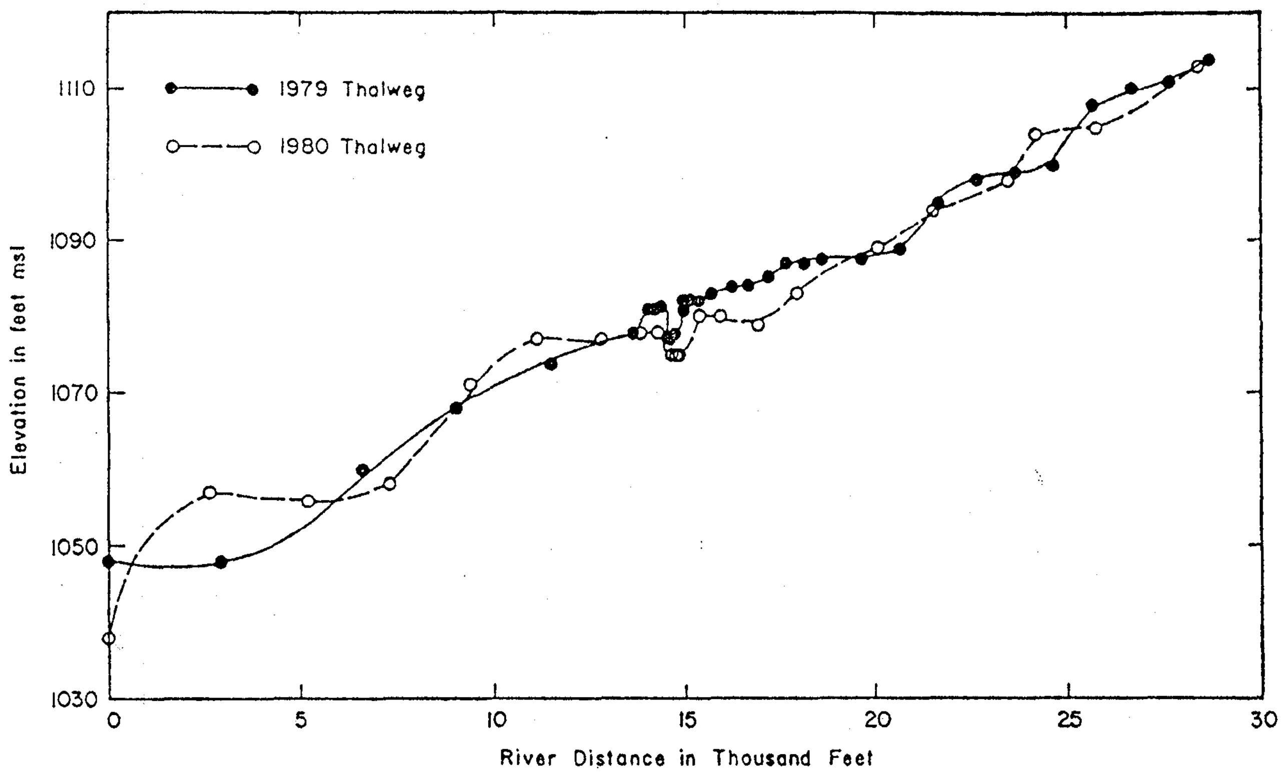

Figure 13. Salt River thalweg profile before and after 1980 flood.

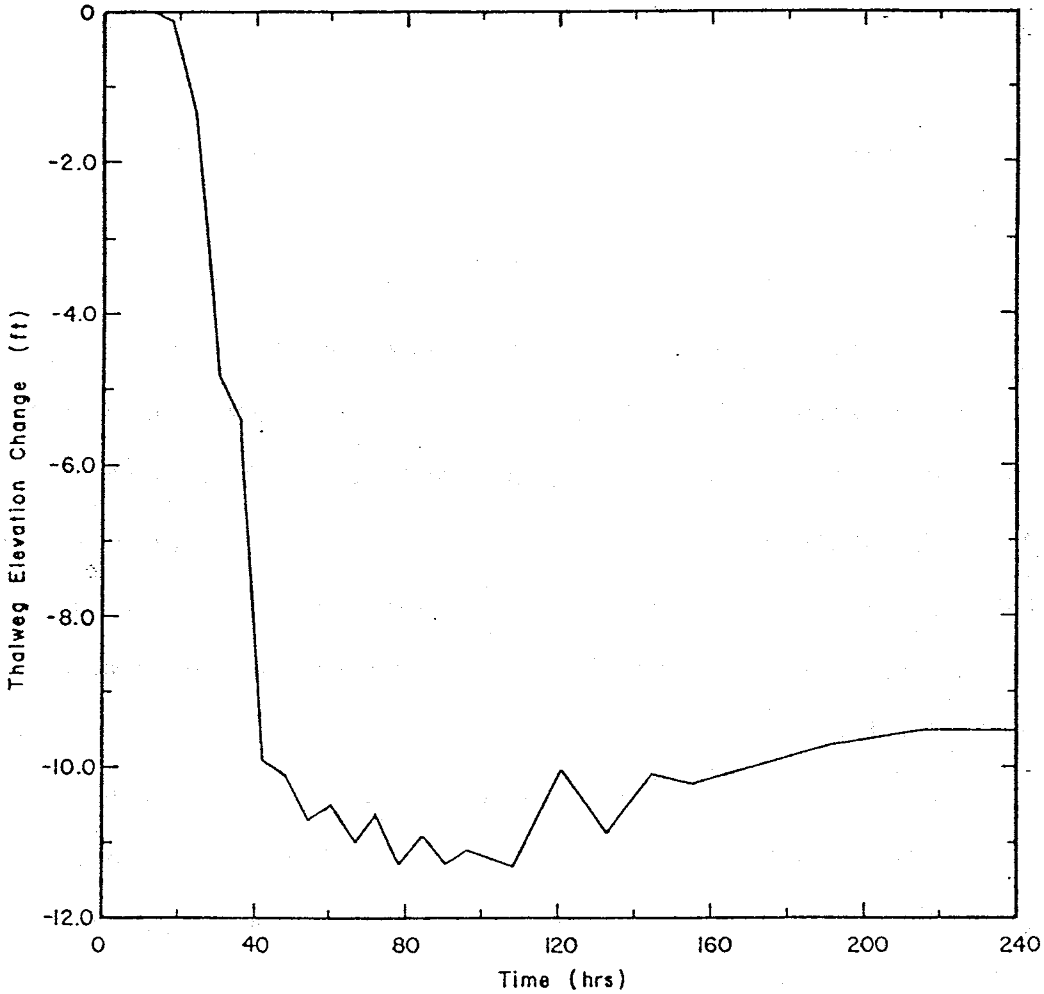

Figure 14. Degradation-aggradation at bridge during the design flood.

methods: (1) Shen's Formula, utilizing the pier Reynolds number (National Cooperative Highway Research Program, Highway Research Board, 1970), (2) Neill's Formula (National Cooperative Highway Research Program, Highway Research Board, 1970), (3) a modification of Neill's Formula (see Simons and Senturk, 1977, p. 690), and (4) a modified method based on the Shields criteria of incipient motion (see Simons and Senturk, 1977) assuming an armoring size of 270 mm. The determined depth of local scour for the as-is condition ranges approximately from 5.0 ft to 7.0 ft. If a 15 degree of angle of attack is assumed for flow approaching the pier, the scour depth would range from 12.5 ft to 17.5 ft. Assuming the total scour is a sum of the general scour and the local scour, the total depth of potential scour can be determined. The total scour depth ranges from 16 ft to 29 ft depending on the angle of attack. It is apparent from the flood experience of the last three years that the channel migration tendency is moving toward the south. Any channelization of low flow channel requires careful protection of the south bank.

The water surface elevations versus flow rate at the bridge site during the design flood for the as-is condition is shown in Figure 15. This figure indicates the looping effect due to sediment movement.

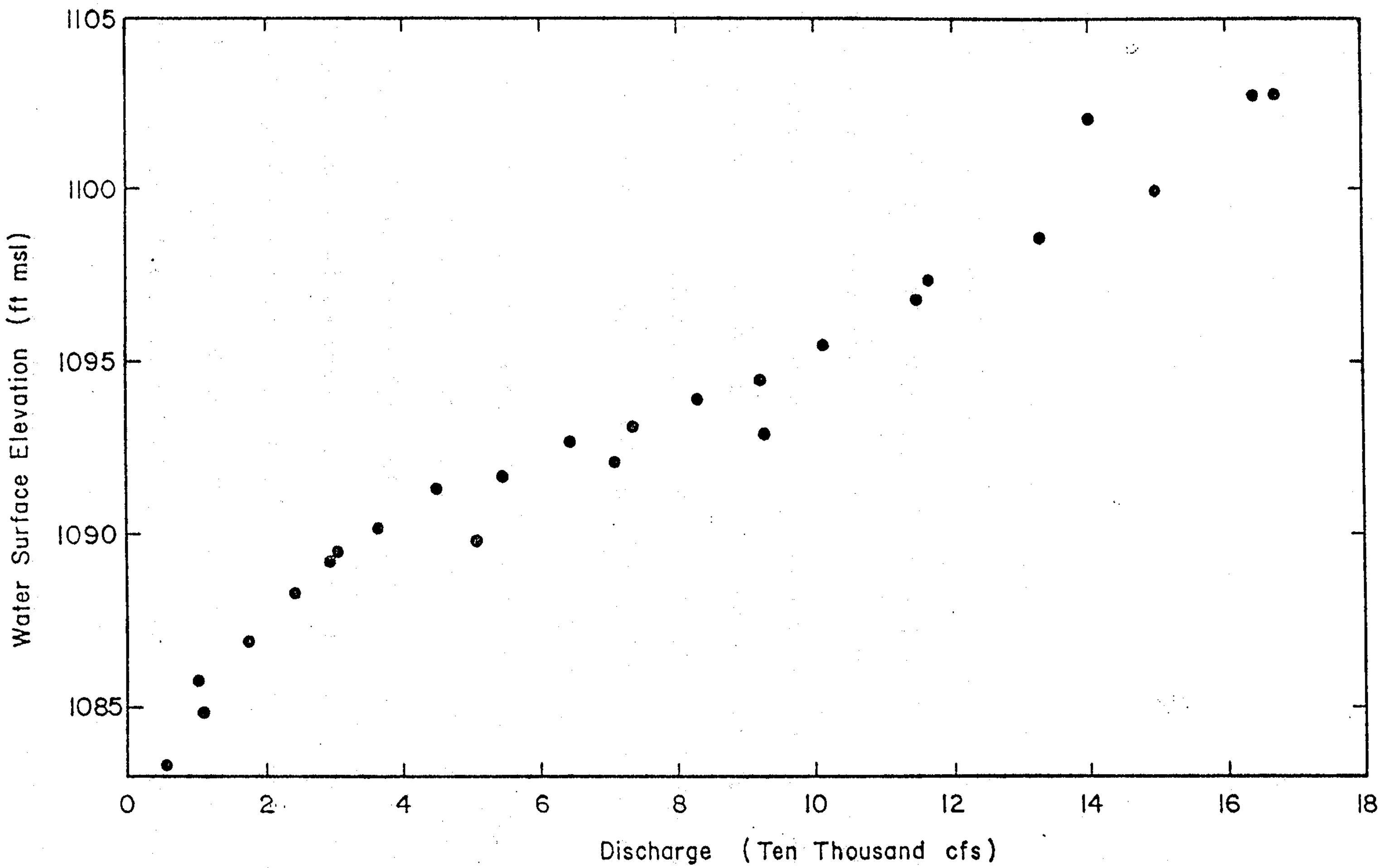

Figure 15. Stage-discharge relation at bridge (as-is condition).

IV. SCOUR PROTECTION MEASURES

4.1 General

This section presents a concept for scour protection measures, including a channelization plan, downstream grade control structure, and control of the inflow from side drainage. In addition, the control of gravel mining in terms of right-of-way is also discussed.

4.2 Right-of-Way Requirement

For evaluation of effect of sand and gravel mining, the gravel pit size was assumed to be 1,200 ft wide and 2,200 ft long, or a surface area of 60 acres. The depths of the pit were assumed to be 15 ft, 30 ft and 50 ft, respectively. These pit sizes are common in the Salt River considering the current extraction rate of 3000 cubic yards per day. These sizes of pit would be created after continuation of extraction of sand and gravel for approximately 1.3 to 5.3 years. The estimated maximum headcut distance and the associated bed slope during the storm for three assumed gravel-pit depths are given in Table 4.

The right-of-way required to protect a structure such as a bridge is dependent on the location, size, depth and volume of the pit and the flow rates and sequences of flow. The right-of-way requirements for various assumed gravel pit depths and sizes were estimated by using the computed results shown in Table 2. If a safety factor of 1.5 is added, the designed headcut distance for 15 ft, 30 ft and 50 ft deep pit under the design hydrograph (peak flow 176,000 cfs) would be 1,400 ft, 1,941 ft, and 3,751 ft. These distances can be utilized to define the right-of-way requirements for protecting the structure against the design storm if no control structure is used. If a control structure is implemented, the right-of-way of 1,000 ft upstream and downstream of the I-10 Bridge is considered adequate.

Table 4. Computed Maximum Headcut Distance and the Associated Bed Slope for Various Assumed Pit Sizes in the Salt River.

Depth of Pit (ft)	Volume of Pit (Acre-ft)	Maximum Headcut Distance (ft)	Depth of Headcut at the Pit Boundary Associated with Maximum Headcut Distance (ft)	Bed Slope Associated with Maximum Headcut Distance (ft/ft)
15	900	943	9	0.0121
30	1,800	1,294	11	0.0105
50	3,000	2,501	20	0.0100

The long-term effects of sand and gravel mining on the structure either upstream or downstream very much depend on the sediment supply, the sediment transport rate and sand and gravel extraction rates. The effects of an individual storm can be additive to create a significant general lowering of bed if the extraction rate is higher than the supply rate. The average sand and gravel extraction rate for a pit in the Salt River is 3000 cubic yards per day. The sand and gravel supply upstream of the pit for each storm was computed according to the Meyer-Peter, Muller bedload equation, coupled with an adaptation of Einstein's suspended sediment integration method. The results are given in Table 5. A comparison of Tables 4 and 5 indicates that almost all of the storm yield volumes are smaller than the pit volumes as assumed. Assuming a 3000 cubic yard per day extraction rate, the time periods of extraction that will create the pit sizes just matching the storm yield can be determined and are given in Table 6. The time periods of extraction for the assumed extraction rate are much shorter than the flood return period. For a given year, there is a definite probability of the occurrence of various sizes of floods. Assuming for a 100-year period there is one 100-year flood, two 50-year floods, four 25-year floods and ten 10-year floods, the total sediment supply for this 100-year period is 10,996 acre-feet. This is a very conservative estimate of inflow rate because there are one-year floods, two-year floods, and other floods that can potentially occur in the river. The actual sediment supply rate would be greater than 10,996 ac-ft. If the supply of 10,996 acre-feet is used, the allowable daily extraction rate is 486 cubic yards per day for the entire reach. A more realistic estimate can be made by assuming that the combination of various floods will supply sediment to equal that produced by a 10-year flood. If this assumption is used, the allowable daily extraction rate is 1600 cubic yards. The current estimated daily

Table 5. Sand and Gravel Supply in Salt River

Flood Frequency (Return Period) (year)	Peak Flow Rate (cfs)	Average Concentration (ppm)	Storm Yield (tons)	Storm Yield (acre-ft)
10	47,000	2,292	924,000	359
25	87,000	2,688	2,010,000	781
50	130,000	2,891	3,220,000	1,251
100	176,000	3,034	4,580,000	1,780

Table 6. Time Periods of Extraction that Matches Storm Yield

Flood Frequency (Return Period) (year)	Peak Flow Rate (cfs)	Storm Yield (cubic yards)	Time Period of Extraction (years)
10	47,000	579,187	0.53
25	87,000	1,260,013	1.51
50	130,000	2,018,280	1.84
100	176,000	2,871,733	2.62

extraction rate of 3000 cubic yards per day for a pit is probably too high considering long-term response. The long-term control of the gravel mining activities requires an extensive litigation and engineering effort. There is no practical engineering solution to mitigate the gravel mining problem in the Salt River. The practical solution is to prevent the headcut failure on a storm basis. Therefore, right-of-way requirements that were estimated based on the design storm are recommended for design.

4.3 Flow Control Structure

The effects of dynamic scour during the storm and the acceleration of lateral channel migration are major problems associated with sand and gravel mining. The shifting of the river thalweg is a major problem in the Salt River. Many existing bridges were designed without taking channel migration into consideration. An appropriate measure to mitigate the problem is implementation of a channelization scheme that controls the location and direction of the flow.

Guide banks have often been used to guide the flow of water through a bridge opening and to control the position of scour and protect the abutments. Guide banks have been used effectively on both sand and gravel bed streams. Principal factors that must be included in the design of guide banks include controlled convergence of the flow normal to the opening, plan shape, upstream and downstream lengths, cross section, crest elevation, scour and riprap protection. This feasibility study evaluates only the openings, plan shape, and upstream and downstream lengths of the guide banks.

It is American practice to give the guide banks an elliptical form convergent to the opening, whereas in Pakistan and India the banks are straight and parallel to the opening, with a curved section at the upstream and downstream ends. The form of the short, elliptical guide bank was illustrated by Karaki (1959). The design layout for straight guide banks is given in Figure 16 (from Manual on River Behavior, Control and Training; Control Board of Irrigation and Power, India, 1956). Straight guide banks probably do a better job of straightening the flow, which can minimize the attack on the abutments. Elliptical guide banks move the scour hole further upstream and downstream of the bridge opening. The straight guide banks are generally more effective than the short elliptical guide banks. The suggested upstream length for straight guide banks ranges from 0.75 to 1.25 times the opening width, and the downstream length ranges from 0.1 to 0.25 times the opening width (Neill, 1973; Control Board of Irrigation and Power, India, 1956). It is recommended that longer guide bank lengths be used to control the high potential for lateral migration of the low flow channel. An example of the proposed layout for I-10 is given in Figure 17. A major component of the proposed channelization plan is to redirect the flow back to the old low-flow channel between Piers No. 1 and No. 9. This limits the maximum opening width to 700 ft. Since the available right-of-way is limited, it is recommended that the channelization be integrated with the downstream grade control structure as indicated in the figure. Three sets of channelization plans were investigated, and it was concluded that the 700-ft opening with a 900-ft grade control structure will function the best hydraulically. The design channel slope for the channelization plan is 0.001. The elevation of the channel invert near the bridge is 1080 ft.

Tempe Drain, a side drainage channel, presently flows to the north. Before the recent flood, the old Tempe channel was located approximately 200 ft north and paralleled the interstate until it reached the south end of the I-10 Bridge. The channel then turned and passed under the bridge between the south abutment and Pier No. 19, continuing in a westerly direction to the low flow channel. The alignment of this drainage channel can remain the same as the

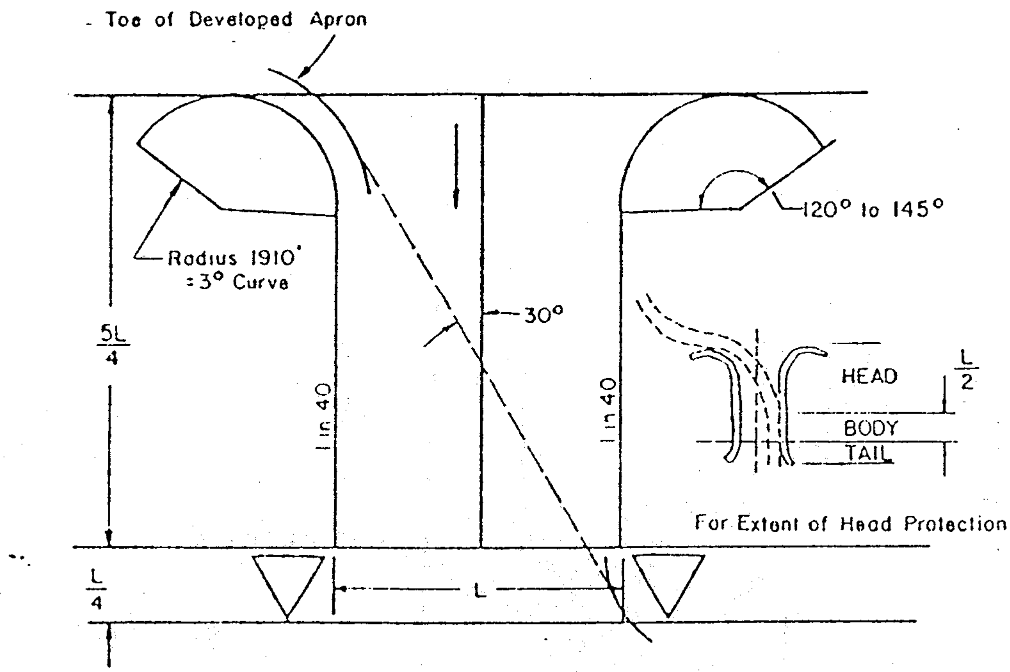

Figure 16. Straight guide bank design to protect bridge from misalignment of the flow.

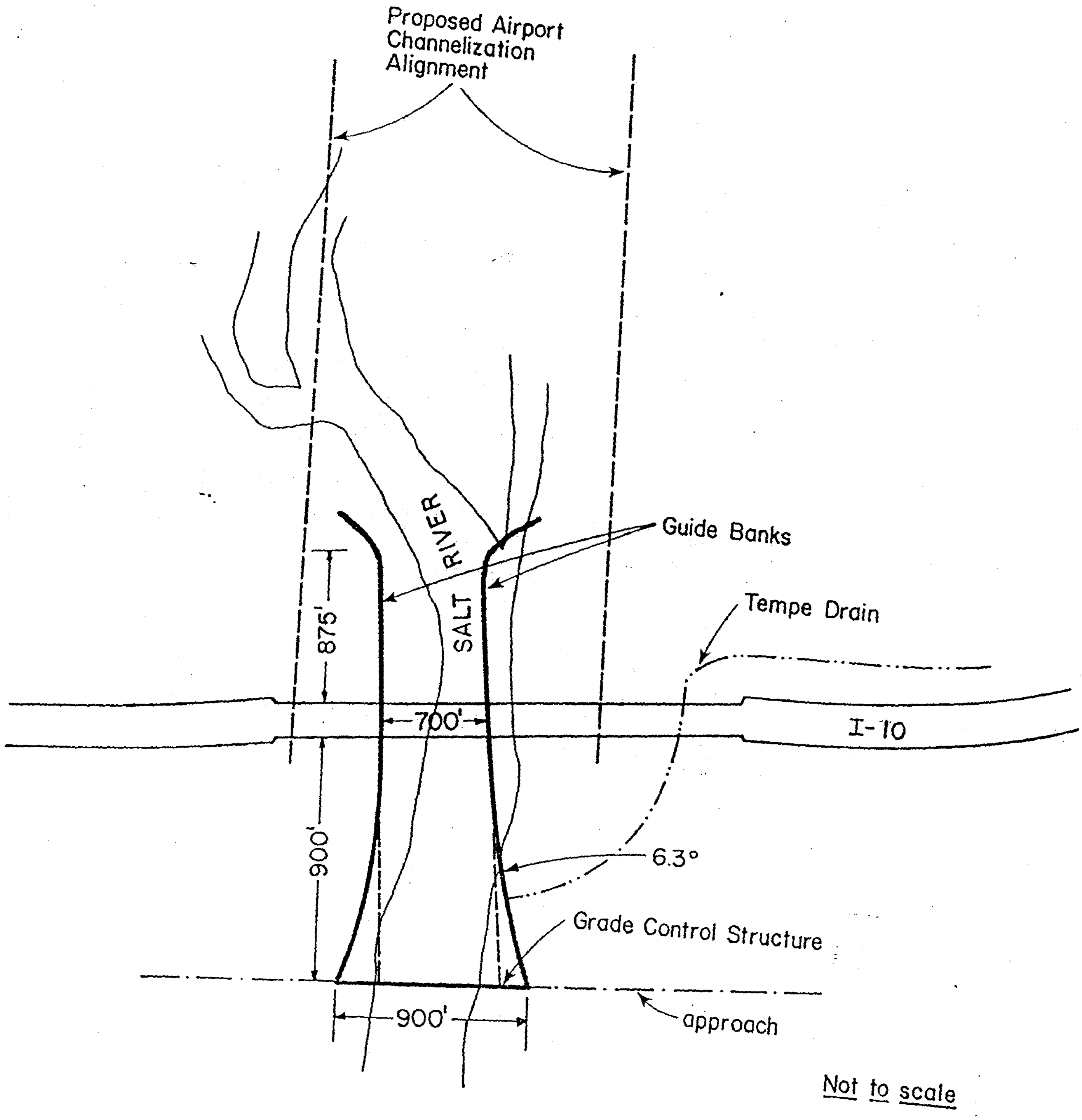

Figure 17. Layout of protection plan.

old alignment. However, a better and more stable channel cross section should be designed. A preliminary estimate indicates that a wider and shallower cross section, yet one which still fits within two piers, should suffice.

4.4 Grade Control Structures

A grade control structure is usually an effective means of controlling general scour. Such structures can prevent headcuts due to gravel mining if the gravel pit initiating the headcut is shallow. It is suggested that a grade control structure be placed downstream of the bridge or upstream of the gravel pit. In order to adequately protect the bridge, both abutments should be protected with riprap that can withstand the forces exerted by the flow around them. The approaches should also be designed to provide additional structural stability.

Considering the use of control structures to limit erosion through bridge openings, two types are feasible: (1) a relatively economical structure formed of rock riprap reinforced with steel rods that will require minimum maintenance, or (2) a conventional reinforced concrete drop structure which can more effectively accommodate large differences in head, but is much more expensive to construct and maintain.

The rock riprap control structure should be constructed in a trapezoidal form with a downstream slope of approximately 1:4 with a stilling basin formed of adequate size riprap extending approximately 15 feet downstream for a two- to three-foot differential in head. The top width of the structure would be approximately 10 feet and the upstream slope should be approximately 1:2. To improve the stability of the structure, reinforcing rods can be strategically placed in the rock riprap as construction proceeds. After the base layer of rock riprap is laid, steel rods can be laid horizontally and parallel to the direction of flow. These rods should extend through the rock riprap on the upstream and downstream faces of the structure. This procedure should be repeated at approximately each 4-foot change in elevation. Simultaneously with the placement of the first layer of rock riprap, vertical rods with large washers would be installed extending upward through the rock riprap. These reinforcing rods terminate in a steel bolt with a thread diameter of approximately 1½ inches. Upon completion of the rock structure, longitudinal steel members would be welded to those rods extending through the structure parallel to the flow. Subsequently, as the structure settles, these longitudinal horizontal rods are stressed by this settling action. Continuous steel elements would be drilled and placed over the steel rods extending through the top of the rock riprap and nuts would be tightened to stress these steel elements, compressing the rock and simultaneously increasing the tension in the horizontal steel members. The vertical rods extending through the top of the structure should be spaced at approximately 10-foot intervals along the axis of the control structure (see Figure 18). Constructing a rock riprap control using this methodology adds to the stability of the structure without adding significantly to its cost.

If a reinforced concrete retaining wall is utilized, a typical dimension for a single drop is as shown in Figure 19a. The riprap placed downstream should be designed to resist the forces exerted on the surface by the flowing water. For staged protection, a multiple drop structure can be used (Figure 19b). According to Table 4, a series of two drops can protect the headcut from gravel mining for a gravel pit with a depth of 30 ft and a 60-acre surface area.

The riprap control structure can be effectively utilized if the potential drop across the structure is on the order of 2 to 3 feet. It is usually impractical to use a dumped riprap drop structure for a potential head drop

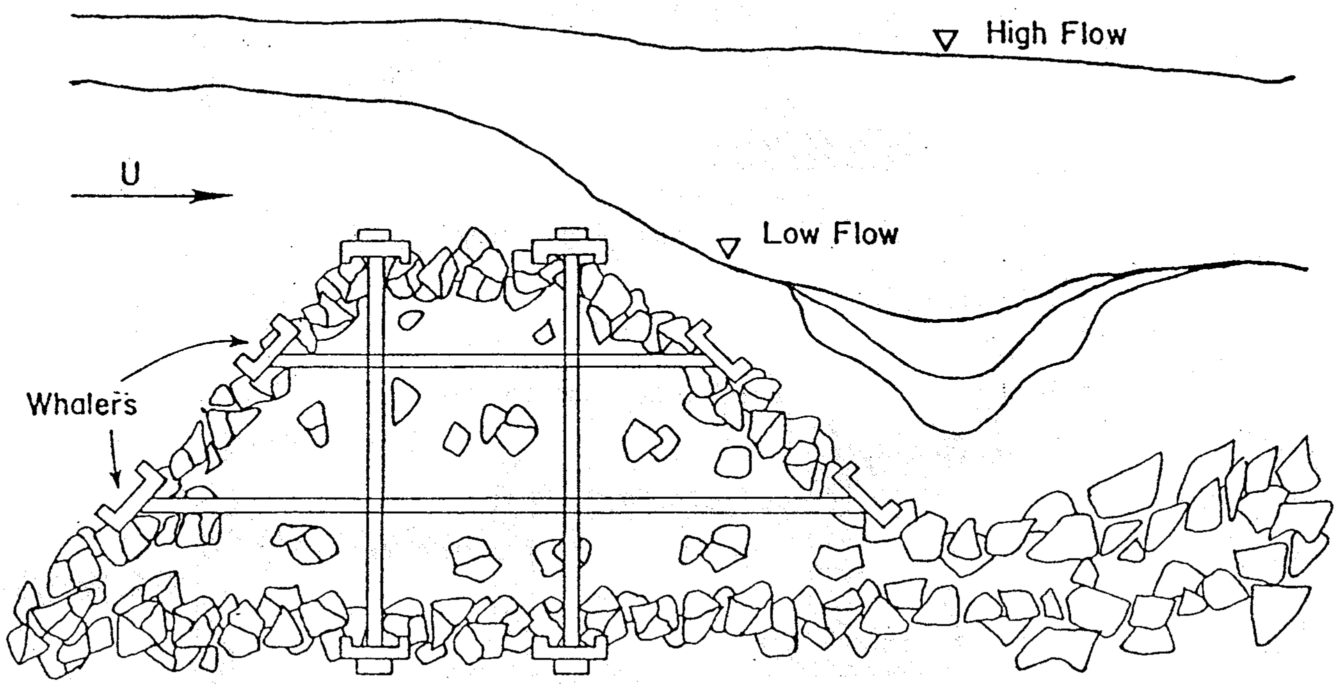

Figure 18. A proposed grade-control structure.

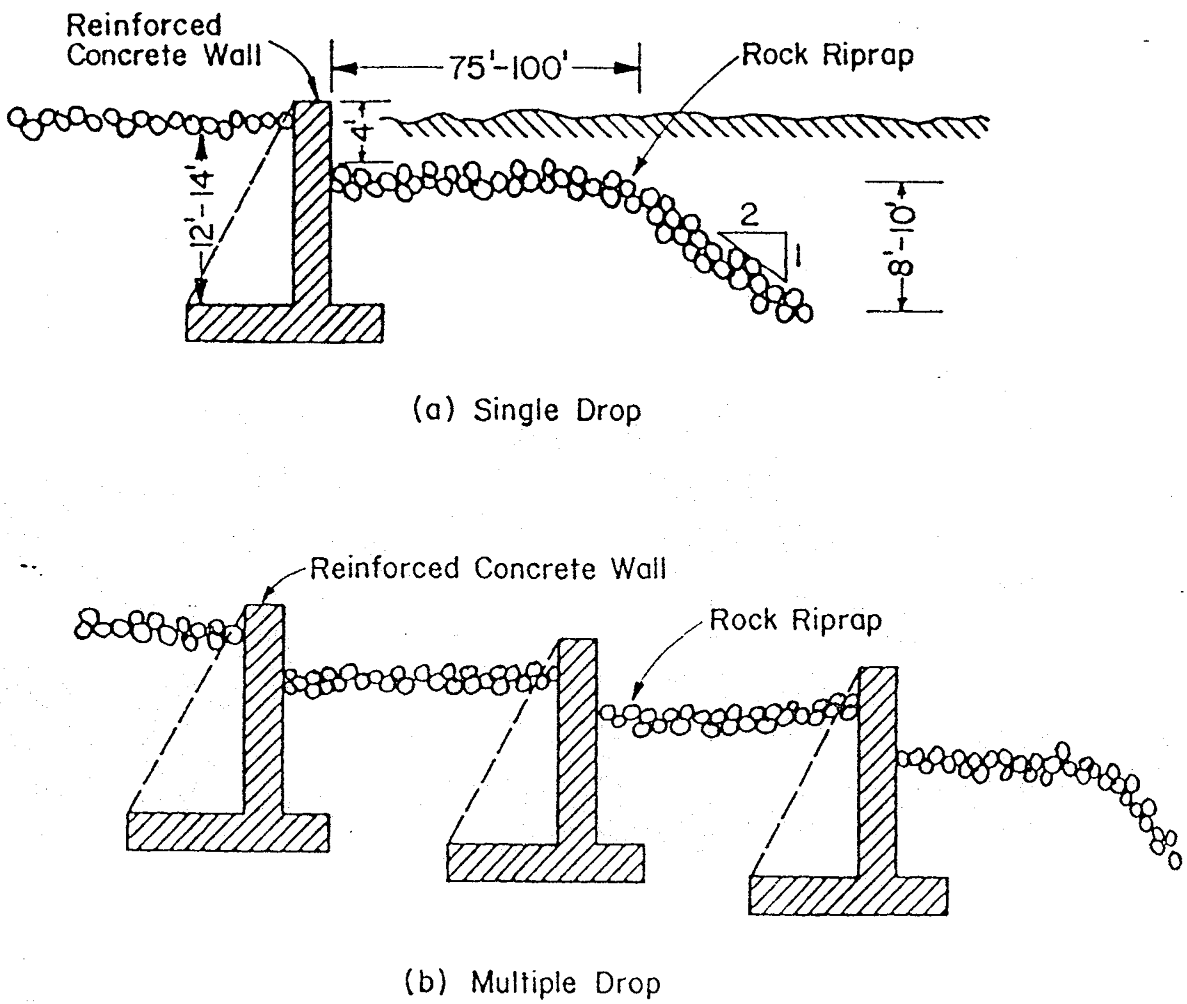

Figure 19. Reinforced concrete wall drop structures.

equal to or larger than 4 feet. Riprap revetment structures used to protect the guide banks should be placed on a slope flatter than 1:2 (vertical to horizontal).

4.5 Degradation and Aggradation Analysis of Proposed Channelization And Grade Control Plan

The spatial resolution of the study area for the proposed channelization and grade control plan (shown in Figure 17) is given in Figure 20. A comparison of the original and final bed profiles after passing a 100-year flood for the channelization and grade control plan is shown in Figure 21. This figure shows that the grade change for the proposed channelization plan would be insignificant. In fact, the reach at the bridge site will experience a slight aggradation (see Figure 22). Due to the use of guide banks, it is expected that the angle of attack of the flow on the pier will be negligible. The computed local scour depth ranges from 4 ft to 6 ft, depending on the equation utilized. The leading edge of the guide banks will experience approximately 7 ft of local scour, as estimated using Liu's Equation (Simons and Senturk, 1977). If a safety factor of 1.5 is required, the bed will potentially degrade to 1070.0 ft. According to Table 3, Piers 2, 3, 4, 5 and 10 would potentially be in danger. Riprap with a median diameter of 18 inches should be placed in the channelization portion. Additional dumped riprap protection near Piers 2, 3, 4, 5 and 10 should be provided.

The water surface elevation versus flow rate at the bridge site during the design flood hydrograph for the proposed channelization scheme is shown in Figure 23. Because of the relatively stable bed configuration, the water surface elevation will not display a loop effect. The computed velocities and water surface elevations at each cross section for the proposed channelization scheme are listed in Table 7.

River Distance	Cross Section Number	Location	Reach Definition
28,480	25	← Upstream Boundary	
25,880	24		9
24,195	23		
23,545	22		8
21,580	21		
20,110	20		
18,060	19		7
17,030	18		
16,000	17		6
15,650	16		
14,975	15		
14,890	14	← I-10	5
14,805	13		
14,605	12		4
14,240	11		
14,040	10		
13,980	9		3
13,780	8		
12,845	7		
11,145	6		
9,445	5		2
7,380	4		
5,280	3		
2,600	2		1
0	1	← Downstream Boundary (Seventh Street)	

Figure 20. Index map for the Salt River in the vicinity of the I-10 Bridge (channelized cross section).

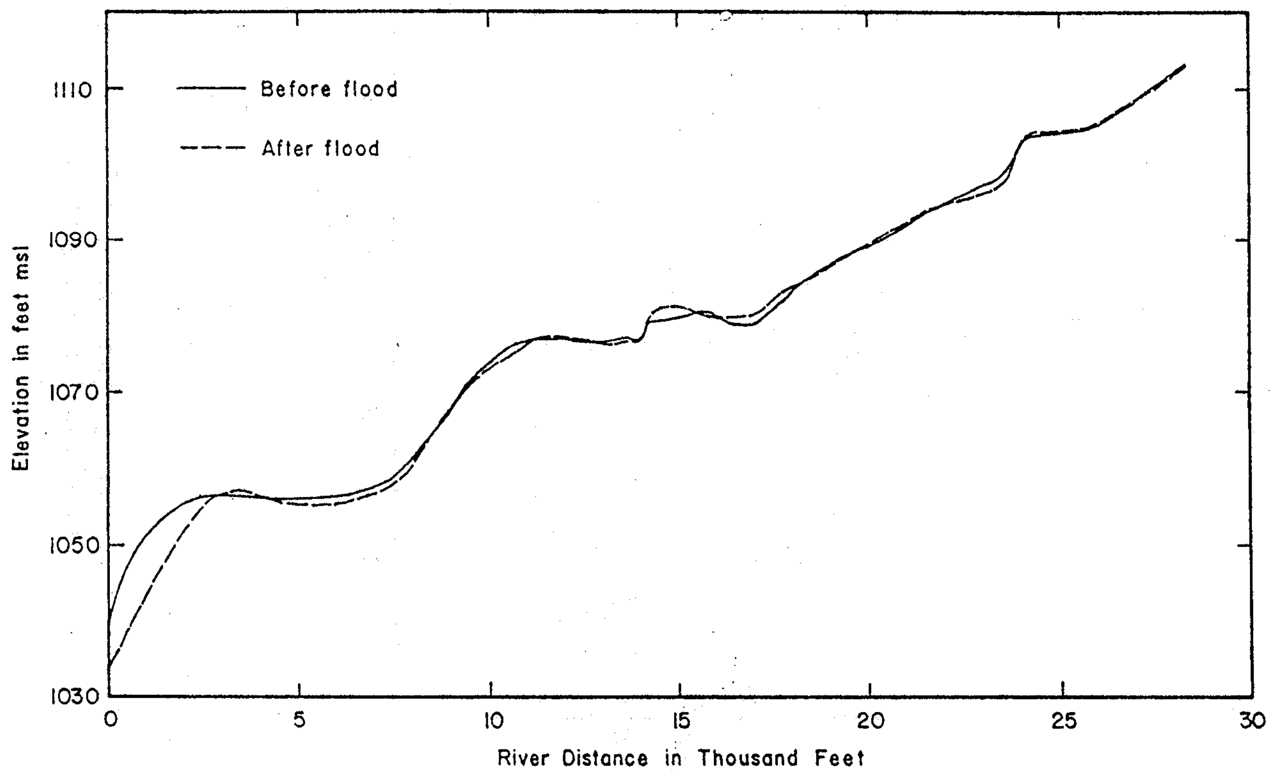

Figure 21. Change of thalweg profile due to the design flood.

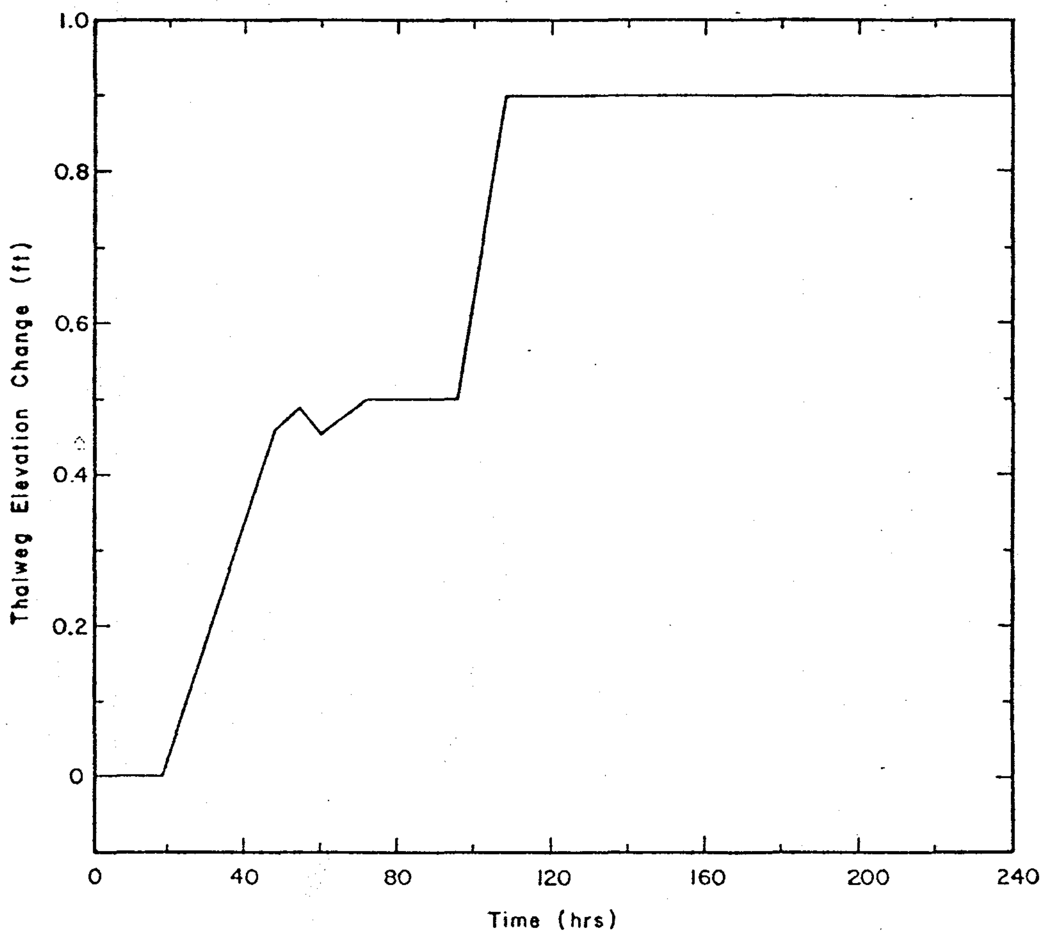

Figure 22. Degradation-aggradation at bridge during the design flood for channelized condition.

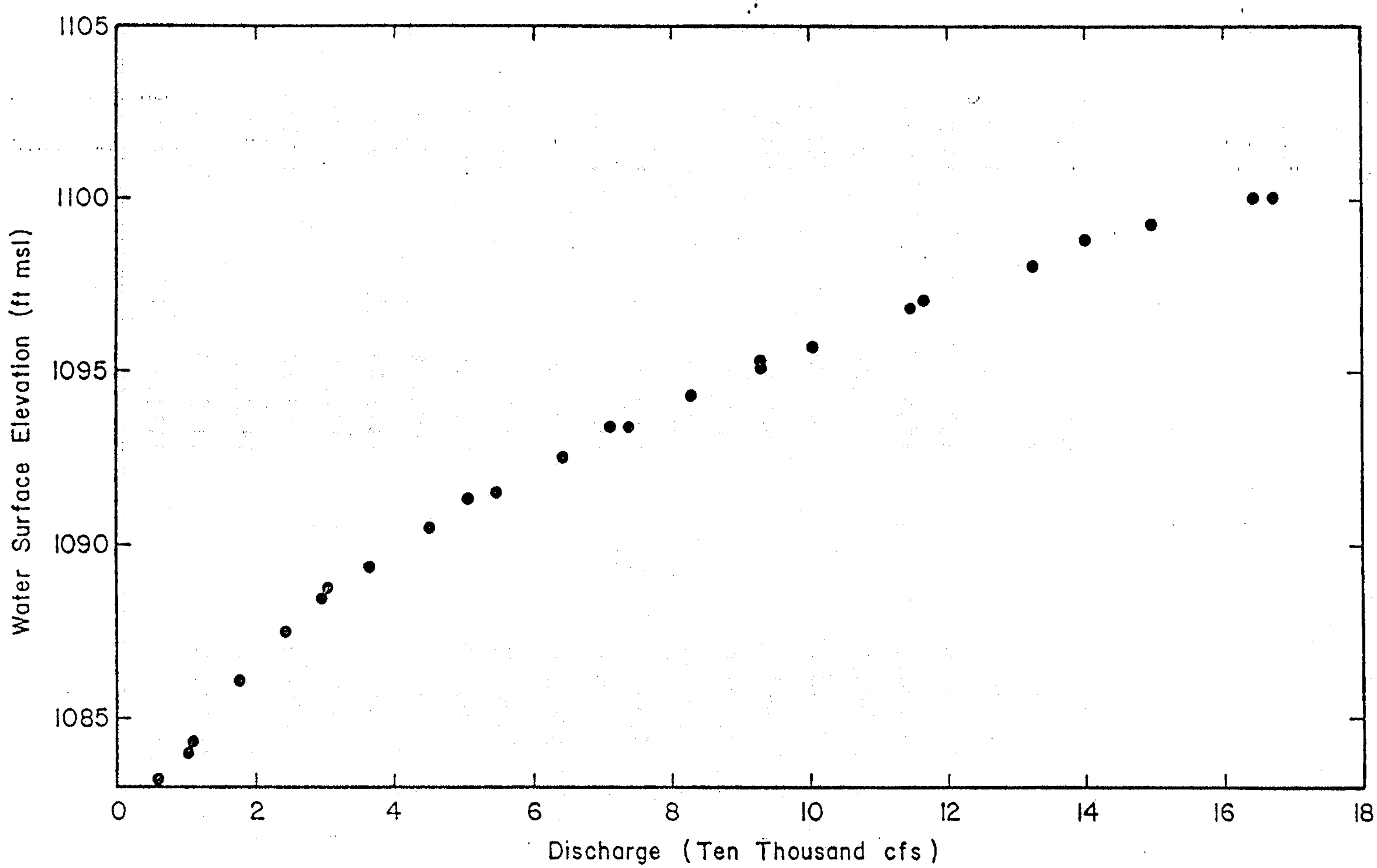

Figure 23. Stage-discharge relation at bridge site (channelized).

Table 7. HEC-2 Run Results for Channelized Condition for Design Flood.

Cross Section Number	River Distance (ft)	Water Surface Elevation (ft MSL)	Main Channel Velocity (fps)	Depth (ft)
1	0.0	1063.37	22.46	25.37
2	2,600.0	1075.46	12.57	18.46
3	5,280.0	1079.87	12.33	23.87
4	7,380.0	1079.99	21.58	21.99
5	9,445.0	1090.47	8.87	19.47
6	11,145.0	1091.23	17.47	14.23
7	12,845.0	1098.01	13.95	21.01
8	13,780.0	1100.73	7.47	23.73
9	13,980.0	1100.81	7.44	23.81
10	14,040.0	1100.70	8.25	21.50
11	14,240.0	1100.59	9.42	20.99
12	14,605.0	1100.63	10.51	21.03
13	14,805.0	1100.60	11.42	20.70
14	14,890.0	1100.67	11.48	20.67
15	14,975.0	1100.75	11.49	20.65
16	15,650.0	1101.50	11.36	20.80
17	16,000.0	1101.19	14.09	21.19
18	17,030.0	1104.46	8.70	25.46
19	18,060.0	1103.54	17.79	20.54
20	20,110.0	1111.60	9.74	22.60
21	21,580.0	1112.70	13.22	18.70
22	23,545.0	1118.84	17.05	20.84
23	24,195.0	1123.15	9.77	19.15
24	25,880.0	1128.73	15.28	23.73
25	28,480.0	1134.79	7.49	21.79

V. SUMMARY

Presented in this report is the hydraulic and scour analysis of the Salt River in the vicinity of the bridge crossing of the Phoenix-Casa Grande Highway (I-10) for long-term protection against scour. A summary of the results follows.

1. A mathematical model that routes sediment by size fraction was used to determine general scour. Four recognized methods were used in determining local scour, and possible changes in the angle of attack of the flow on the piers were considered. The total scour is assumed to be the sum of the general scour and the local scour.
2. The total depth of potential scour for the as-is condition ranges from 16 to 29 feet, depending on the angle of attack. The bridge will likely be in danger during floods.
3. The recommended channelization plan was shown in Figure 17. This plan involves guide banks with a 700-ft opening and downstream grade control structures with a width of 900 ft. The angle of expansion is 6.3 degrees. The channel bottom in the vicinity of the bridge should be armored with 18-inch rocks for a depth of 2.5 feet and a length of approximately 150 feet. This apron will be adequate if the downstream grade control structure is effective.
4. The suggested grade control structure that was integrated with the channelization plan was shown in Figure 19. The length of the structure top should be 75 to 100 ft when the gravel mining proceeds. If two drops are used, the protection will be limited to a 60-acre, 30-ft deep pit no closer than 500 ft downstream of the grade control structure.

5. The right-of-way necessary to protect the bridge without the control structure was evaluated assuming 15-, 30- and 60-ft deep, 60-acre sand and gravel pits. If the grade control structure is used, a minimum of 1000 ft of right-of-way is required.
6. The proposed channelization plan was evaluated by applying water and sediment routing techniques. The total depth of potential scour for the channelized condition ranges from 4 to 6 ft. The proposed plan will work, but extra protection using dumped riprap should be provided around Piers 2, 3, 4, 5 and 10.
7. The computed water surface elevation versus water discharge relation indicates that the as-is condition will display looping effects due to sediment movement. The water surface elevation for the channelized condition is generally lower than the as-is condition for peak flows.