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FINAL REPORT

# SCOUR AND SEDIMENTATION ANALYSIS OF THE PROPOSED CHANNELIZATION OF THE SALT RIVER FOR PROTECTING THE SKY HARBOR INTERNATIONAL AIRPORT IN PHOENIX, ARIZONA

Prepared for

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# I. INTRODUCTION

## 1.1 General

The Salt River has experienced major floods four times in the past three years. These floods caused significant damage in the metropolitan Phoenix area. One of the most affected areas has been the Sky Harbor International Airport. The main runway has been so severely damaged that 2400 feet of its length is unusable.

Federal funds made available to the City of Phoenix for augmenting the budget required to repair the flood-damaged south runway carry the provision that mitigating measures be instituted to prevent future flood damage to the runway prior to or concurrent with runway reconstruction. Executive Order 11988 of May 24, 1977, mandates that where such federal funds are involved, the 100-year flood be considered. As a result, Howard Needles Tammen and Bergendoff (HNTB) was contracted by the City of Phoenix to design the necessary protection for the airport runways considering the 100-year flood. The plan recommended by HNTB calls for a channelization of the Salt River from just above the I-10 Bridge to the Hohokam Expressway. Because of the limited funds available, a channelization which will provide flood protection for one 100-year event only and then require extensive reconstruction and repair was proposed. In addition, HNTB recommended that subsequently, when additional funds become available, a more permanent channel be constructed.

The purpose of the study by Simons, Li & Associates, Inc. (SLA) is to analyze the scour and sedimentation in the recommended interim channel, Alternative 4H proposed by HNTB. In addition, possible modifications to the proposed channelization to increase its effectiveness, if possible, are to be recommended. The specific scope of work follows.

1. Conduct a field visit to familiarize ourselves with the physical environment and with the proposed channelization plan.

2. Collect, collate and review the information on aerial photographs, hydrology, hydraulics, sediment transport, sediment size distribution, cross-section data, structural and geomorphic data and related channelization plans in the vicinity of the study site.

3. Compile and develop a spatial representation system that approximates the study reach. The tentative downstream control point is the I-10 Bridge. The spatial design includes a line diagram showing the river mile, cross-section numbers, and location of structures in the study reach.

4. Conduct an evaluation of aggradation and degradation response for the proposed channelization plan considering the design flood hydrograph. Evaluate the potential scour problems, including general and local scour.

5. Evaluate the effect of potential sand and gravel mining activities on the stability of the proposed channel.

6. Assess the long-term degradation problem using an engineering geomorphic approach.

7. Recommend possible modifications to the proposed channelization plan and bank protection requirements.

8. Prepare a report documenting results of the analysis.

## 1.2 Specific Conditions Analyzed

The channelization plan analyzed was Alternative 4H as proposed by HNTB. The analysis was performed assuming there would not be influences from sand and gravel mining activities. The possible effects of gravel mining were considered separately. However, since no definitive plan exists for mining in the area, only a qualitative evaluation of possible activities and their impacts could be considered. The major portion of the study centered around the 100-year design flood of 176,000 cfs. Since the channel is designed to perform for one 100-year flood event, long-term changes were only qualitatively discussed.

The analysis was carried out utilizing three levels: (1) qualitative geomorphic, (2) quantitative engineering geomorphic, and (3) physical process model. For the third level of analysis, six specific conditions were

considered in order to determine the possible types and ranges of responses. Because of the uncertainty regarding bed-material size distributions, the channel responses for three distributions were analyzed. Two downstream hydraulic boundary conditions were also analyzed for each of the possible bed-material size distributions.

Additionally, local effects at three critical locations were analyzed more closely. The three locations were the aerial surveillance radar (ASR), the area in the vicinity of the instrument landing system (ILS) island, and the Arizona Public Service (APS) power line islands.

## 1.3 Available Information

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

1. "Preliminary Investigations, Salt River Channelization Sky Harbor International Airport," prepared for City of Phoenix Engineering Department, by Howard Needles Tammen and Bergendoff, February, 1980.

2. "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.

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

4. "Phases C and D Final Deisgn Permanent Protection Against Scour Salt River Bridge - Interstate 10," prepared for Arizona Department of Transportation by Dames and Moore, July, 1980.

5. "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 by Simons, Li & Associates, Inc., June, 1980.

6. "Manual on River Behavior, Control and Training," Control Board of Irrigation and Power, India, 1956.

7. "Sediment Transport Technology," Water Resources Publications, by D. B. Simons and F. Senturk, 1977.

8. "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.

9. "Open Channel Hydraulics," McGraw-Hill Book Company, by V. T. Chow, 1959.

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

11. Aerial photographs of the Salt River in the project area since 1960.

12. Survey of sand and gravel mining in the vicinity of I-10.

13. HEC-2 data deck provided by HNTB for the proposed channelized condition.

14. February, 1980, flood contours of the Salt River provided by HNTB.

15. HEC-2 printout for 1980 based on existing conditions, post-February 1980 flood cross sections. Provided by HNTB.

16. HEC-2 printout for channelized condition, provided by HNTB.

17. Design "Balanced" hydrograph provided by Anderson-Nichols.

18. "Central Arizona Water Control Study, Sand and Gravel Mining Guidelines," prepared for U.S. Army Corps of Engineers, Los Angeles District, by Boyle Engineering Corporation, Water Resources Division, July, 1980.

19. "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.

20. "Preliminary Engineering Study - 19th Avenue Bridge Over Salt River, Phoenix, Arizona," by Howard, Needles, Tammen, Bergendoff, and Johanessen and Girard, 1979.

21. Preliminary Plans, Salt River Channelization at the I-10 Bridge, Arizona Department of Transportation, Highways Division, Structures Section, May, 1980.

# II. DATA SUMMARY

## 2.1 General

The information required to conduct the analysis includes hydrology (flood hydrograph), channel geometry, bed-material size distribution, aerial photographs to determine river morphology, resistance to flow, and data indicating sediment transport rates. The specific data used in the analysis are presented in the following chapter.

## 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 recent flood events. Certainly, such events may be experienced again in the future. Five floods have occurred in the Salt River recently. The date of occurrence, peak flow and duration of these floods are given in Table 1.

From the report titled "Preliminary Engineering Study - 19th Avenue Bridge over Salt River, Phoenix, Arizona," by Howard, Needles, Tammen, Bergendoff and 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 conducted 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 Sky Harbor channelization design flood adopted by HNTB 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. Due to time and money constraints,

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 | 176,000 | 200,000 |

a more practical means was used for estimating the shape of the hydrograph for the design flood. This method involved the development of a typical hydrograph normalized with respect to the flood peak. In a previous study of the I-10 bridge for Dames and Moore, SLA synthesized a normalized hydrograph based on flow data recorded on 3/1/78 and 12/15/78. Figure 1 presents these normalized hydrographs along with the resulting average normalized hydrograph. Anderson-Nichols (AN) developed a similar normalized hydrograph for their portion of the Sky Harbor channelization study. It is presented in Figure 2. The two hydrographs are very similar in shape and duration. In this study the Anderson-Nichols hydrograph was adopted. To be useful in the analysis, the hydrograph must be divided into discrete intervals of time. Table 3 lists the discretized hydrograph. The design flood utilizing the typical hydrograph has a peak discharge rate of 176,000 cfs and a duration of ten days. 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.

## 2.3 Channel Geometry

The basic representation of the channel geometry is the channel cross section. A total of 41 cross sections representing the planned channelization was utilized in defining and analyzing the study area. All cross sections except six in the vicinity of the I-10 bridge were provided by HNTB. The remainder were taken from the Arizona Department of Transportation preliminary plan for the I-10 bridge channelization. The original cross sections were modified to reduce the number of points present and bring their spatial resolution within the range of the sensitivity of the analysis performed. In addition, the 41 cross sections represent 50 percent reduction in the number of cross sections used originally by HNTB to perform their

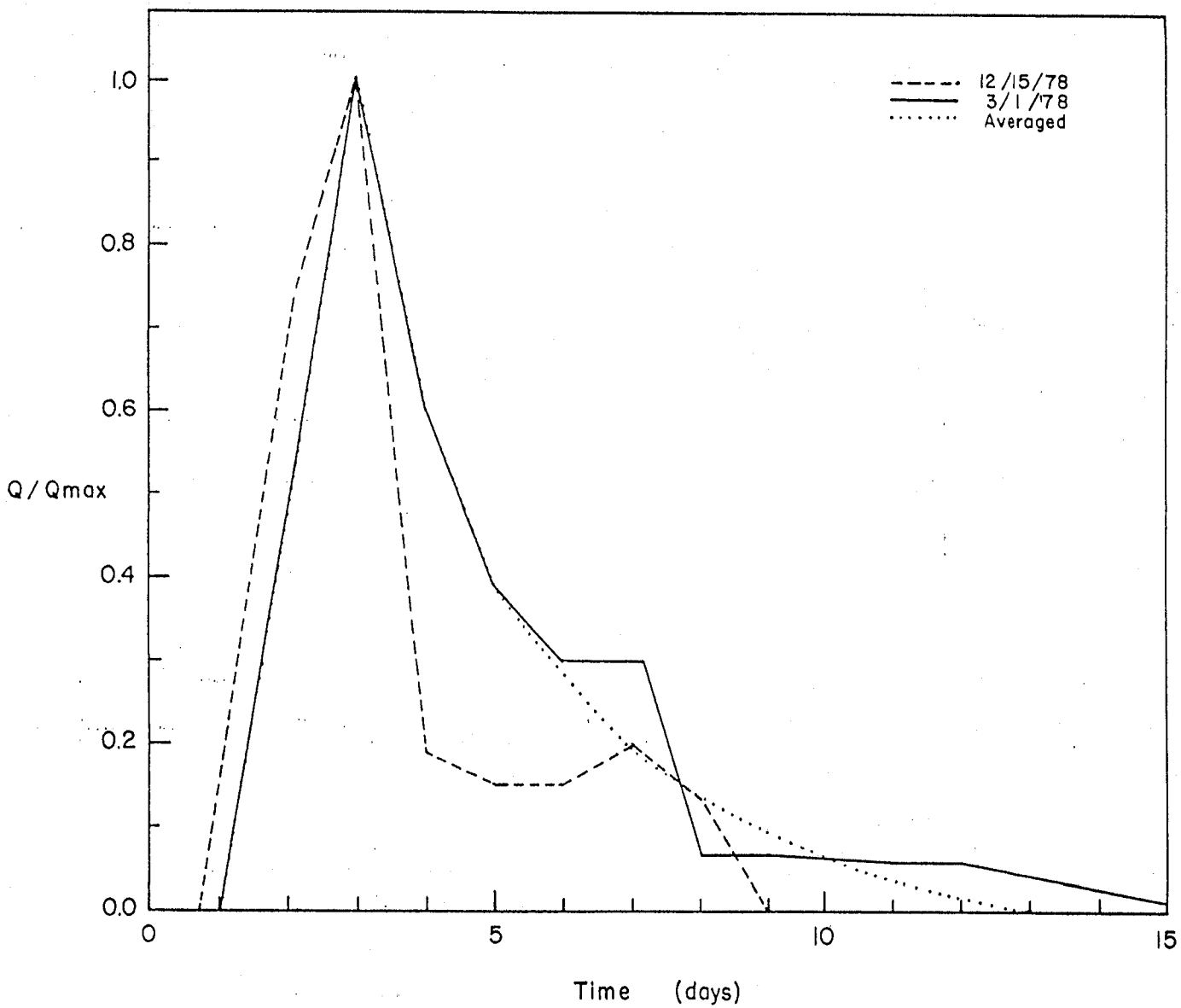



Figure 1. Hydrographs for Salt River.

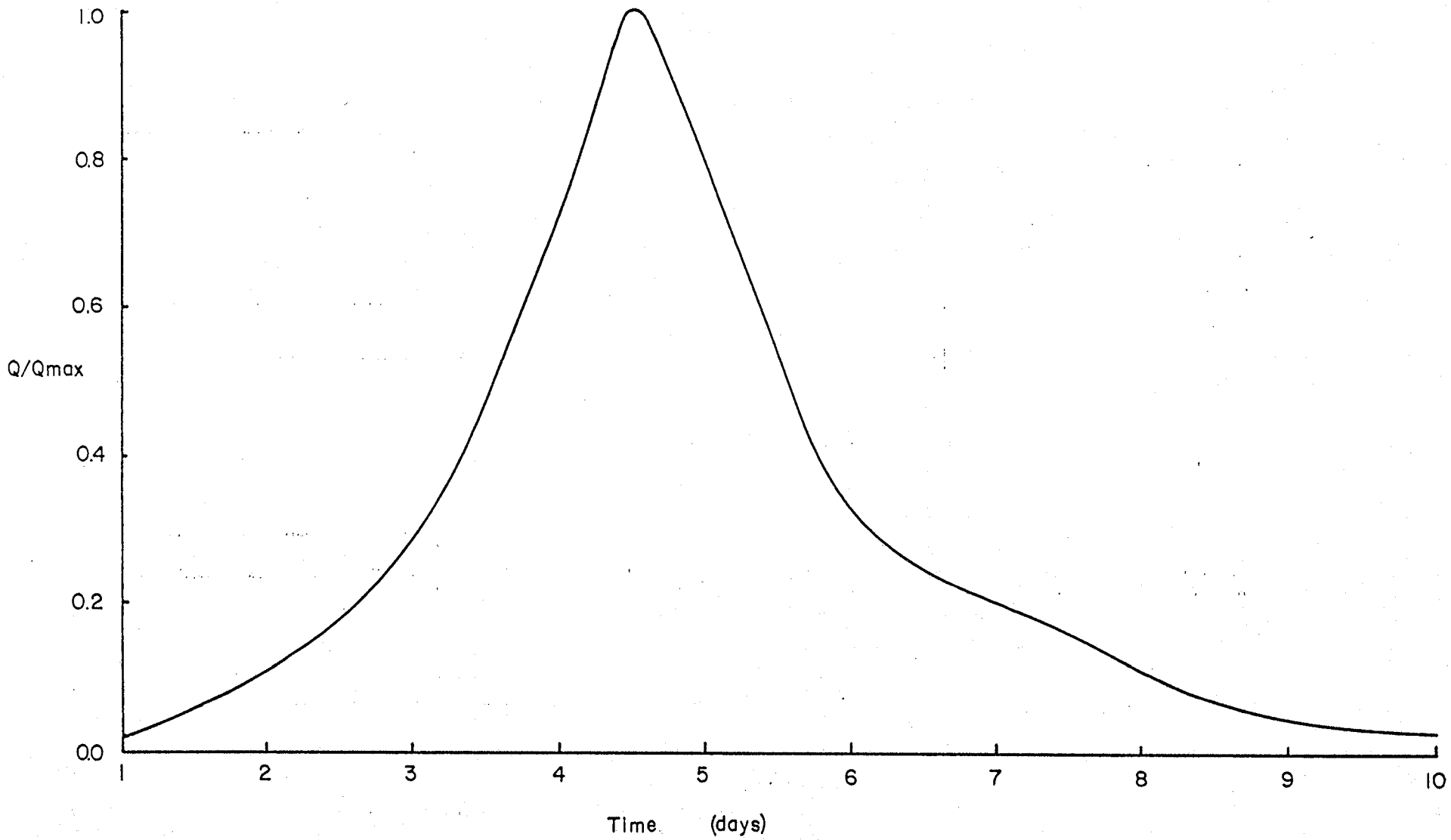



Figure 2. Anderson-Nichols normalized hydrograph.

Table 3. Discretized 100-Year Flood Hydrograph (Based on a Peak Discharge of 176,000 cfs).

| Time (days) | Δt (days) | Average Discharge (cfs) |
|---|---|---|
| 1.00 | 1.00 | 9,680 |
| 2.00 | 1.00 | 34,320 |
| 2.50 | 0.50 | 66,880 |
| 2.75 | 0.25 | 95,040 |
| 3.00 | 0.25 | 117,040 |
| 3.25 | 0.25 | 141,680 |
| 3.50 | 0.25 | 165,440 |
| 3.75 | 0.25 | 166,320 |
| 4.00 | 0.25 | 146,080 |
| 4.25 | 0.25 | 124,080 |
| 4.50 | 0.25 | 102,080 |
| 5.00 | 0.50 | 71,280 |
| 6.00 | 1.00 | 41,360 |
| 7.00 | 1.00 | 26,400 |
| 8.00 | 1.00 | 15,840 |
| 9.00 | 1.00 | 9,680 |
| 10.00 | 1.00 | 4,400 |

HEC-2 analysis. The major benefit in this simplification was a large reduction in the computer time necessary to perform the erosion and sedimentation analysis.

In performing the analysis, a group of cross sections is considered together as a single reach. The aggradation/degradation process is then considered using the average properties for a reach. The reason for this grouping is that the analysis method is designed for determining general scour. Thus, by grouping a number of cross sections together and considering their properties as a single unit, local effects confined to a single cross section are reduced. This allows a more reasonable application of present state-of-the-art sediment transport theory. The regions of large local effects are isolated and their additional response determined separately.

The grouping of cross sections into reaches must be performed so that

1. All cross sections in a reach have similar hydraulic and sediment transport characteristics,
2. Areas of special concern are represented by a reach, and
3. Sections of the channel which are expected to have different responses are separated.

The segmentation of the study area into reaches was carried out following these principles. The qualitative and quantitative geomorphic analysis presented later in this report aided in the process. The reasons involved in the definition of the reaches are listed in Table 4. Figure 3 gives a schematic representation of the spatial definition of the study area. A plan view of the Alternative 4H plan with the reach subdivision superimposed appears in Figure 4. The reach lengths are provided in Table 5.

Two spatial representations of the study area were actually used in the analysis. The two representations were necessary to consider both possible downstream boundary conditions. The first condition consists of the I-10 grade control structure acting as a free overall. This condition sets the

Table 4. Purpose for Reach Subdivisions.

| Reach Number | Purpose |
|---|---|
| 1 | Isolation of I-10 grade control structure and establishment of hydraulic control |
| 2 | Isolation of I-10 bridge and channelization |
| 3 | Isolation of region between I-10 channelization and Sky Harbor channelization |
| 4 | Isolation of area of channelization with dike on north side only |
| 5 | Isolation of area with dikes on both side |
| 6 | Representation of contraction in vicinity of ASR |
| 7 | Isolation of channelized region between ILS and ASR |
| 8 | Representation of contraction in vicinity of ILS and contraction of 1000-foot channel |
| 9 | Isolation of initial channelization and Hohokan Expressway |
| 10 | Isolation of wide braided section above channelization |
| 11 | Isolation of wide braided and flat section above channelization |
| 12 | Determination of supply to study areas |

| River Distance (feet above I-10 grade control structure) | Cross Section Identification | Location | Reach Definition |
|---|---|---|---|
| 22,920 | 21,180 | Upstream boundary | 12 |
| 21,820 | 20,080 | | |
| 20,820 | 19,070 | | 11 |
| 19,520 | 17,920 | | |
| 17,880 | 16,760 | | |
| 16,620 | 15,580 | | 10 |
| 15,500 | 14,420 | 48th Street | |
| 14,440 | 14,120 | Initial dikes | |
| 13,640 | 13,520 | Hohokam Expressway | |
| 12,990 | 12,920 | | 9 |
| 12,570 | 12,520 | | |
| 12,150 | 12,120 | | |
| 11,730 | 11,720 | ILS | |
| 11,520 | 11,520 | ILS | |
| 11,320 | 11,320 | ILS | 8 |
| 11,120 | 11,120 | ILS | |
| 10,720 | 10,720 | | |
| 10,320 | 10,320 | | |
| 10,120 | 10,120 | 1000' wide channel begins | |
| 9,520 | 9,520 | | 7 |
| 8,920 | 8,920 | | |
| 8,260 | 8,260 | | |
| 7,860 | 7,860 | ASR | |
| 7,660 | 7,660 | ASR | 6 |
| 7,510 | 7,510 | ASR | |
| 7,310 | 7,310 | ASR | |
| 6,910 | 6,910 | | 5 |
| 6,040 | 6,040 | End of dikes on south side | |
| 5,440 | 5,440 | | |
| 4,840 | 4,840 | | 4 |
| 4,240 | 4,240 | | |
| 3,520 | 3,520 | End of dikes on north side | |

Figure 3. Schematic of reach definition.

| River Distance (feet above I-10 grade control structure) | Cross Section Identification | Location | Reach Definition |
|---|---|---|---|
| 3,120 | 3,120 | | |
| 2,520 | 2,520 | | 3 |
| 1,920 | 1,920 | | |
| 1,520 | 1,520 | ← I-10 channelization begins | |
| 910 | 910 | ← I-10 Bridge | 2 |
| 800 | 800 | ← I-10 Bridge | |
| 450 | 450 | | |
| 150 | 150 | | 1 |
| 0 | 0 | ← Grade control structure | |

Figure 3 (Continued)

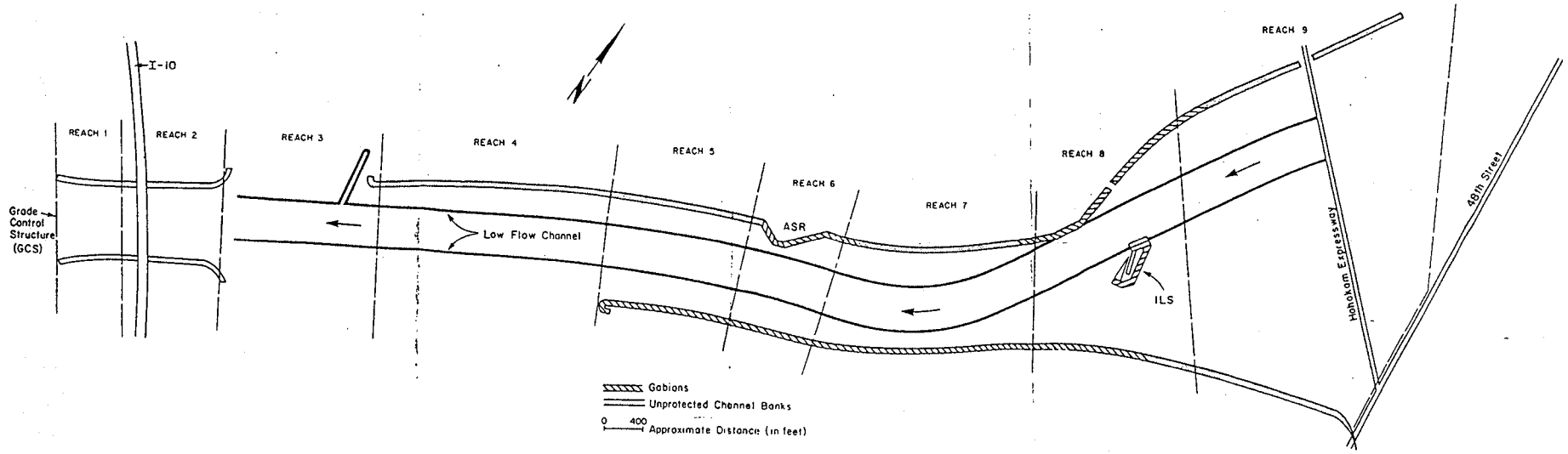



Figure 4. Plan view of Alternative 4H and reach subdivision.

Table 5. Reach Lengths.

| Reach Number | Number of Cross Sections | Reach Length (feet) |
|---|---|---|
| 1 | 3 | 575 |
| 2 | 3 | 1145 |
| 3 | 3 | 1600 |
| 4 | 4 | 2420 |
| 5 | 2 | 1370 |
| 6 | 4 | 950 |
| 7 | 4 | 2160 |
| 8 | 6 | 1700 |
| 9 | 5 | 2350 |
| 10 | 3 | 3070 |
| 11 | 2 | 2235 |
| 12 | 2 | 1605 |

downstream water surface equal to the critical depth stage. The spatial representation for this condition was just presented. The second downstream condition utilized a stage-discharge relationship at the I-10 bridge determined by SLA in the I-10 Bridge study. The spatial representation for this condition is the same as the first case with the elimination of the first reach and the first cross section in the second reach. Thus, the second segmentation starts at the cross section with identification number 910.

The system of identification numbers used in the analysis is based on the distance upstream of the grade control structure along the centerline of the low-flow channel through the channelized region. Above the channelized area, the line is extended through the as-is condition cross sections. These cross sections are identified by the same scheme. The reported distances are along the centerline of the Salt River channel. Through most of the channelization region, the identification numbers and distances are equal. However, above the ILS the centerline deviates from the main channel and the two distances no longer coincide. Table 6 gives the SLA channel identification numbers, main channel distances, and corresponding cross section numbers used by HNTB.

## 2.4 Bed Material Size Distribution

An accurate representation of the bed material in the study area is necessary for a valid analysis of the sediment transport characteristics of the channel and its responses. Since the construction activity will disrupt the present armor layer,* the expected sediment size distribution can best be represented by the existing subsurface layer. In a previous study of the I-10 Bridge channelization by SLA for Dames and Moore, subsurface sediment samples

*The armor layer is defined as the layer of larger material which has built up on the surface of the channel bed and prevents the smaller subsurface particles from being transported.

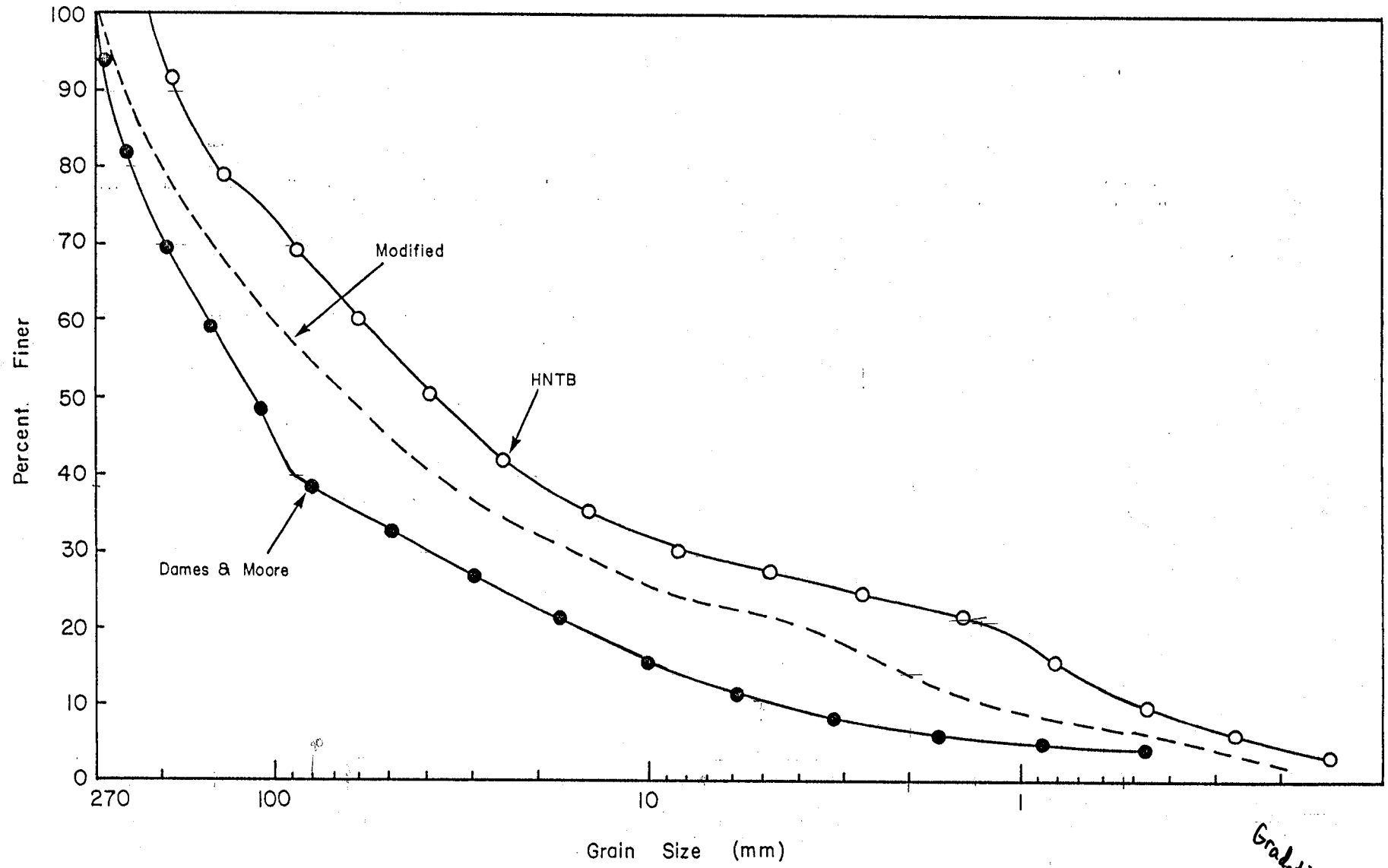
Table 6. Cross Section Identification.

| Main Channel Distance Above I-10 GCS (feet) | SLA Cross Section Identification | HNTB Cross Section Identification |
|---|---|---|
| 0 | 0 | None |
| 150 | 150 | None |
| 450 | 450 | None |
| 800 | 800 | None |
| 910 | 910 | None |
| 1,520 | 1,520 | 286+00 |
| 1,920 | 1,920 | 282+00 |
| 2,520 | 2,520 | 276+00 |
| 3,120 | 3,120 | 270+00 |
| 3,520 | 3,520 | 266+00 |
| 4,240 | 4,240 | 258+80 |
| 4,840 | 4,840 | 252+80 |
| 5,440 | 5,440 | 246+80 |
| 6,040 | 6,040 | 240+80 |
| 6,910 | 6,910 | 232+10 |
| 7,310 | 7,310 | 228+10 |
| 7,510 | 7,510 | 226+10 |
| 7,660 | 7,660 | 224+60 |
| 7,860 | 7,860 | 222+60 |
| 8,260 | 8,260 | 218+60 |
| 8,920 | 8,920 | 212+00 |
| 9,520 | 9,520 | 206+00 |
| 10,120 | 10,120 | 200+00 |
| 10,320 | 10,320 | 198+00 |
| 10,720 | 10,720 | 194+00 |
| 11,120 | 11,120 | 190+00 |
| 11,320 | 11,320 | 188+00 |
| 11,520 | 11,520 | 186+00 |
| 11,730 | 11,720 | 184+00 |
| 12,150 | 12,120 | 180+00 |
| 12,570 | 12,520 | 176+00 |
| 12,990 | 12,920 | 172+00 |
| 13,640 | 13,520 | 166+00 |
| 14,440 | 14,120 | 160+00 |
| 15,500 | 14,420 | 20.93 |
| 16,620 | 15,580 | 21.15 |
| 17,880 | 16,760 | 21.38 |
| 19,520 | 17,920 | 21.69 |
| 20,820 | 19,070 | 21.94 |
| 21,820 | 20,080 | 22.13 |
| 22,920 | 21,180 | 23 |

were taken and analyzed in much of the present study area. In developing their design, HNTB also had subsurface sediment samples taken and analyzed. When reviewing the samples taken for HNTB, SLA found that several samples contained a large amount of material finer than expected. These samples were removed from consideration since they were probably taken in areas of deposition and did not represent the true subsurface layer. All the remaining samples were then averaged to develop a single distribution. Because of the large variability in samples from site to site and the limited number of samples in any given area, it was decided that separate distributions for certain areas could not be justified. Both the HNTB and Dames and Moore sediment size distributions are given in Figure 5.

As can be seen from Figure 5, there is a large discrepancy in the two size distributions. The samples collected for HNTB are much finer. After discussion with personnel from SLA, HNTB, Anderson-Nichols and the City of Phoenix who had inspected the site, it was decided that the samples taken for HNTB appeared to have an excess of fine material and did not contain some larger size material actually present. As a result, a third size distribution was derived. It is also presented in Figure 5. This distribution was derived by setting the largest size equal to the Dames and Moore largest size, then smoothing a line through a $D_{50}$ which was the geometric mean of $D_{50}$'s of the two previous distributions. The remainder of the distribution smaller than $D_{50}$ was derived in the same manner as the $D_{50}$.

To provide a range of conditions, all three distributions were used in the analysis. Each distribution was divided into ten increments of ten percent. The sizes used to represent each interval were the geometric means of the upper and lower limits. Table 7 provides the breakdown.




Gradation

Figure 5. Representative grain size distributions.

Table 7. Size Distributions Used in Analysis.

| Percent in Interval | Upper Limit (mm) | | | Lower Limit (mm) | | | Geometric Mean (mm) | | |
|---|---|---|---|---|---|---|---|---|---|
| | D&M | HNTB | Averaged | D&M | HNTB | Averaged | D&M | HNTB | Averaged |
| 10 | 275 | 200 | 275 | 255 | 171 | 230 | 265 | 185 | 250 |
| 10 | 255 | 171 | 230 | 235 | 135 | 200 | 245 | 152 | 215 |
| 10 | 235 | 135 | 200 | 190 | 86 | 140 | 211 | 108 | 167 |
| 10 | 190 | 86 | 140 | 150 | 57 | 96 | 169 | 70 | 120 |
| 10 | 150 | 57 | 96 | 110 | 37 | 60 | 128 | 46 | 76 |
| 10 | 110 | 37 | 60 | 90 | 21 | 40 | 100 | 28 | 49 |
| 10 | 90 | 21 | 40 | 38 | 7.6 | 17 | 59 | 13 | 26 |
| 10 | 38 | 7.6 | 17 | 15 | 1.1 | 4 | 24 | 2.9 | 8.2 |
| 10 | 15 | 1.1 | 4 | 5 | 0.48 | 1.3 | 8.7 | 0.73 | 2.3 |
| 10 | 5 | 0.48 | 1.3 | 0.30 | 0.10 | 0.2 | 1.5 | 0.22 | 0.51 |

## 2.5 Aerial Photographs

Aerial photographs are useful tools for providing an understanding of the dynamics of a river system. From the aerial photographs of a river reach, the changes and trends in a river's alignment can be ascertained. In addition, they are helpful in determining qualitatively the impact man's activities have had. The aerial photographs available for this study covered a twenty-year time span from 1960 to 1980. They formed the basis for much of the qualitative geomorphic analysis performed in this study.

## 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 and in some instances the larger roughness elements can be buried by deposits of sand. In their study of the Salt River in the vicinity of Sky Harbor Airport, HNTB calibrated Manning's coefficients using data from the March, 1978, flood. Such a determination of the $n$ values is most reasonable for the conditions in the Salt River. Their results showed an $n$ value of 0.035 produced the best results.

For the sediment transport study, SLA chose a Manning's $n$ of 0.030. The smaller value was used since higher velocities created by the lower resistance will increase the transport rate of sediment. Thus, for the condition where prediction of scour potential is the main concern, it is conservative to use the lower resistance value. In addition, there is the possibility of the passage of sand waves which can significantly reduce the resistance to flow. In the overbank regions, an $n$ of 0.035 was used. This is a fairly low overbank value, but it is justified since there will be some grading work done and very little vegetation is present because of recent floods, construction, and gravel mining activity in the area.

## 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 percent bed slope) and gravel and boulder bed sediment, indicate that a form of the Meyer-Peter, Muller (MPM) sediment transport equation (Simons and Senturk, 1977) is applicable for steep gravel and boulder bed streams. The MPM equation as applied was developed utilizing data from gravel bed flumes and the Shields incipient motion criteria. The procedure was modified to properly account for the armoring effect of coarser particle sizes. The MPM type equation accounts for the bed load only, therefore the suspended portion of the bed material load is computed by the Einstein procedure (Simons and Senturk, 1977).

This is the same method of determining sediment transport rates used in the Dames and Moore study of the Salt River I-10 bridge. The study used a hydrograph very similar to the February, 1980, flood in both peak and duration. Measurements of scour at the I-10 bridge during and after the flood agreed closely with the results of the analysis using the sediment transport rate determination just presented. For this reason, the above procedure was adopted for the Sky Harbor sedimentation study. The same coefficients for the equations which had been used in the previous study were also adopted.

# III. ANALYSIS

## 3.1 General

The analysis was conducted on three levels. The first level of analysis was a qualitative geomorphic assessment of the Salt River and its response to the proposed channelization. The next level consisted of an engineering geomorphic approach. In this phase the results of the first analysis were verified by application of more rigorous theories. This analysis also quantifies many of the responses that were only qualitatively assessed earlier. For the third level of analysis, a mathematical model was used to predict the general response of the planned channelization. Additional calculations were made to determine the response of areas which had local effects beyond the model's spatial resolution.

The procedure just outlined allows for the most thorough and accurate determination of the Salt River's response to the Sky Harbor channelization project in terms of sedimentation. By starting with the more general types of analysis, an overall understanding of the system was developed. Only by acquiring such knowledge could the more sophisticated forms of analysis be carried out properly. By viewing the system at the three levels and discovering any discrepancies among them in the predicted responses, the analyses can be further scrutinized to determine the best possible and most realistic prediction.

In determining the flow conditions for the analysis, two downstream boundary conditions were used. The first case assumed a free overfall at the I-10 grade control structure. This is the worst possible condition when calculating scour, since it creates maximum velocities. The second condition used a stage-discharge curve at the I-10 bridge. This curve is presented in Figure 6. The curve was computed in the earlier study for Dames and Moore by SLA for the condition of the grade control structure being in place and the

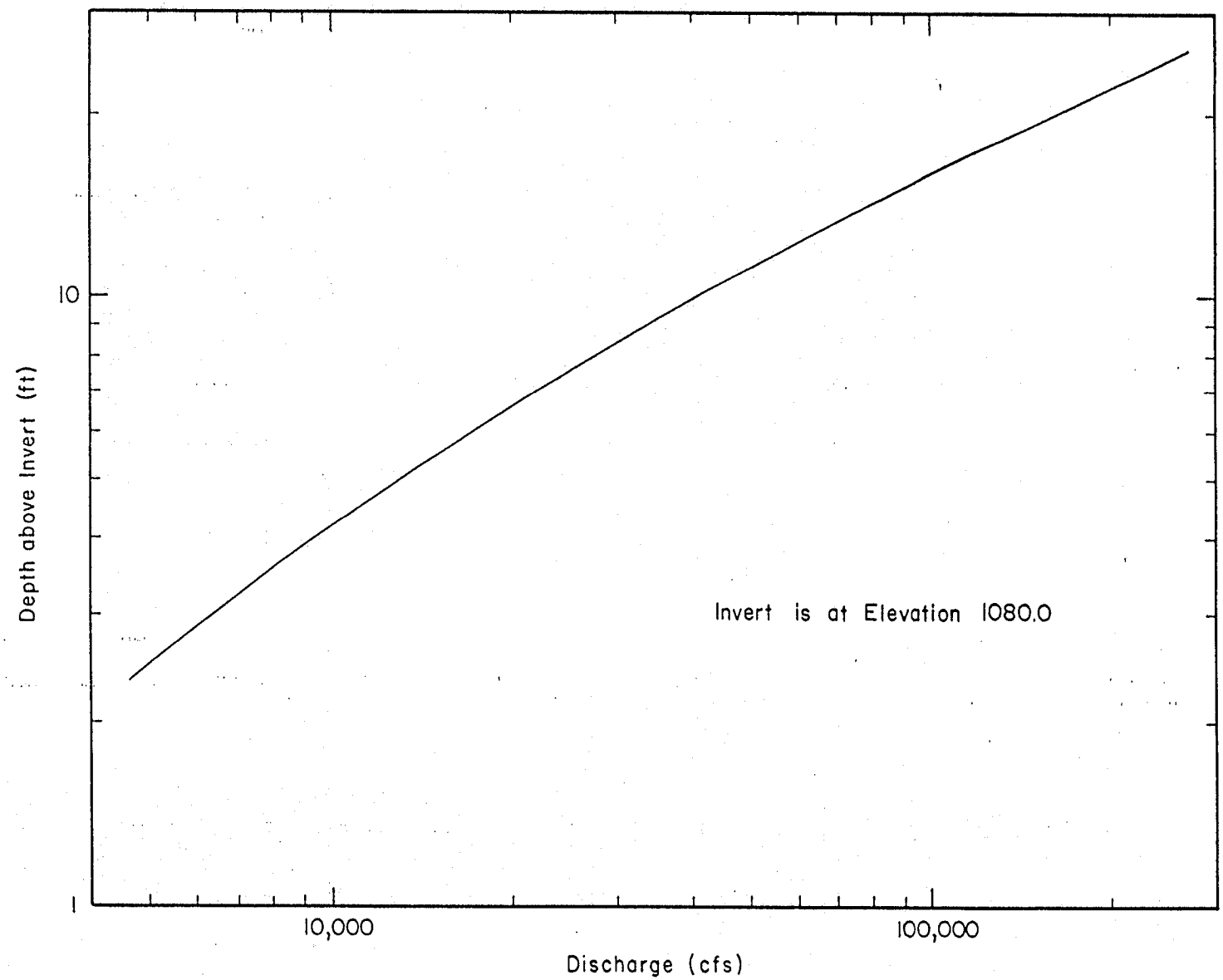



Figure 6. Stage-discharge relationship at I-10 bridge.

Salt River channelized under the I-10 bridge. This second case is best representative of the flow conditions after completion of the I-10 channelization. The first case will only be realized if there is sufficient degradation below the grade control structure to eliminate any downstream effects.

## 3.2 Qualitative Geomorphic Analysis

The main changes in the Salt River channel since 1960 have been induced by man. Photos of the river in 1960 show a wide braided channel with scattered vegetation. The braided portion of the channel extends laterally nearly 3000 feet at some points. Since this time gravel mining activities, construction of roads and bridges, and development along the river have eliminated the vegetation and in many places channelized and contained the river so it is no longer braided. The result is an increase in flow velocities accompanied by an increase in potential sediment transport rates and degradation in the channel bed. The effects of the increased potential sediment transport rates and degradation have been curtailed by the river's ability to form an armor layer of large cobbles and boulders. This layer exists through most of the reach of the present river, and through the proposed Sky Harbor Airport channelization.

The channelization process will require reworking of the river bed, and in the process will disturb or destroy the natural armor layer. In addition, the channelization will further reduce the width of river from an average of 1500 ft to 1000 ft. The average gradient will be increased through shortening the channel length by 1400 ft. These latter two conditions will increase velocities and potential sediment transport rates. With the armor layer disturbed, the increased sediment transport capacities will degrade the channel until enough large material accumulates on the surface of the channel bed to reform an effective armor.

Results of the HEC-2 computations for the design peak flow of 176,000 cfs are given in Figures 7 and 8. These results are for the downstream condition of a free overfall at the I-10 grade control structure. Using the basic principles of sediment transport, these figures were used to assess the qualitative response resulting from the proposed channelization. As can be seen from these figures, there is a narrowing of the channel and a resulting increase in velocities as one proceeds downstream. The increased velocities will create larger sediment transport capacities and result in deeper degradation. As the degradation occurs, the bed slopes will be reduced and the bed-material particle size distribution will become progressively larger. These two factors will bring the channel back toward equilibrium.

Figure 9 shows the proposed bed profile, which is fairly uniform except for a rather steep and then flat section near the Hohokam bridge. It is likely that a headcut may progress up through the section producing channel slopes similar to the predominant slopes in the rest of the river reach.

### 3.3 Quantitative Engineering Geomorphic Analysis

The first portion of this analysis was conducted by computing the sediment transport capacities for a range of discharges representing the design hydrograph. The procedure used to compute the transport rate was the same MPM and Einstein combination discussed in Section 2.7. However, all transport rates were computed using the original sediment size distribution and armoring was not considered. This overestimates the transport rates, since the finer materials are removed quickly and total transport capacity is soon reduced. For this reason, the transport capacities, as computed, can be used to estimate whether a reach will aggrade or degrade, but not the actual magnitude. In computing the transporting capacities, the flow conditions for all the cross sections in a reach were averaged. The bottom two reaches were

Figure 7. Velocity vs distance for Alternative 4H with critical downstream control.

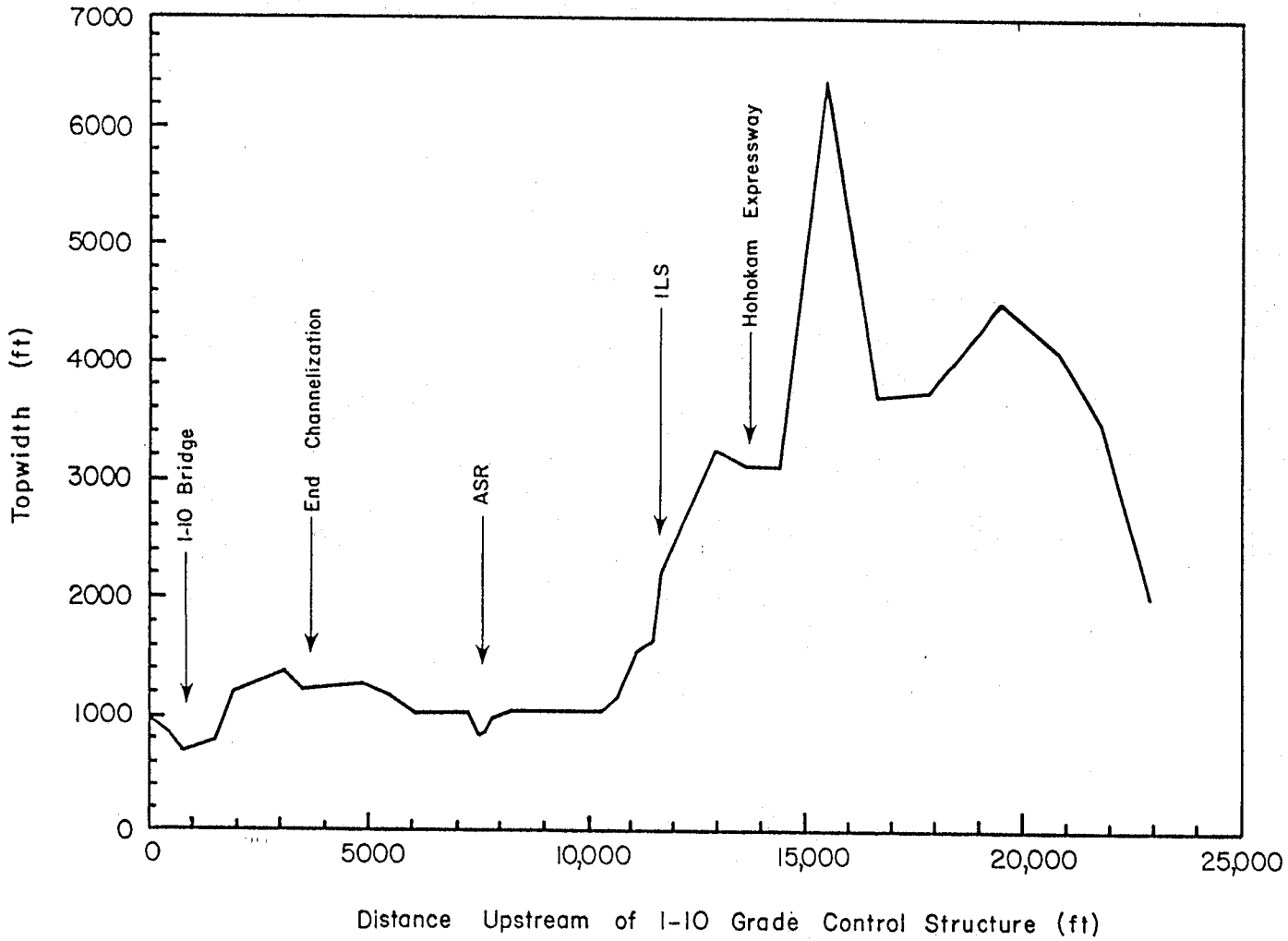



Figure 8. Top width vs distance for Alternative 4II with critical downstream control.

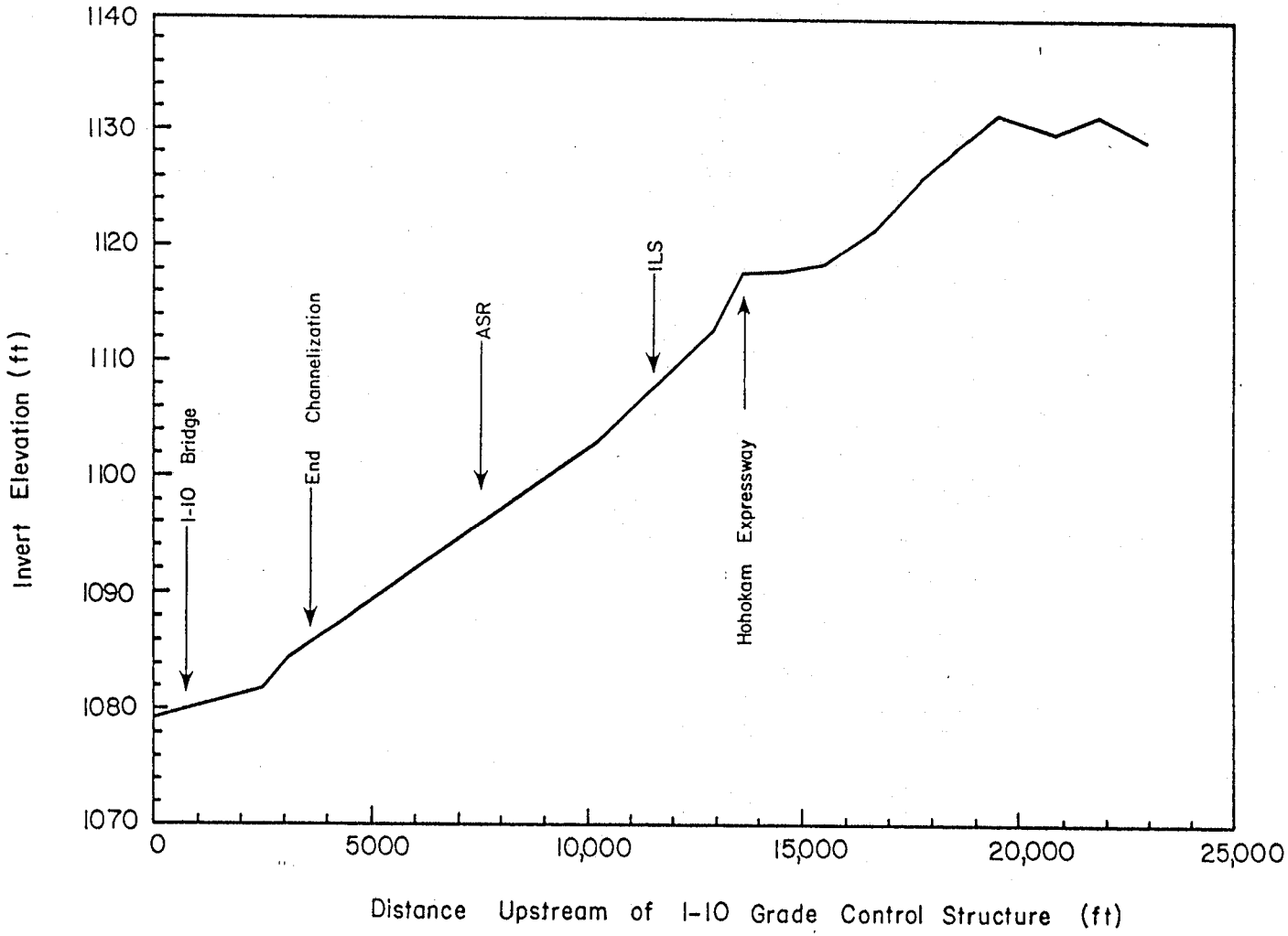



Figure 9. Profile of proposed Alternative 4II.

not considered, since the effect of the grade control structure could not be properly analyzed by such an analysis. The averaged sediment size distribution was used. The hydraulic conditions were determined from the free overfall downstream control case using HEC-2. This is the most severe case when considering scour, because of the higher velocities produced in the downstream reaches.

The results of the analysis are presented in Table 8. The direction of stream bed elevation change was determined by comparing the transport capacity of the reach in question with the reach immediately upstream. If the downstream reach had a higher capacity, it degraded. If it had a lower capacity, it aggraded. The general trend is for the channel to degrade through the channelized reaches due to increased velocities. This assessment is consistent with the qualitative geomorphic analysis.

Because of the large sediment sizes found in the channel bed which cannot be moved by the flow, the armoring process plays the key role in determining the amount of aggradation or degradation which can occur. Thus, the maximum amount of scour is determined more by the magnitude of the peak discharge than the duration. For a river in which all bed-material sizes could readily be moved, the duration of the flood has a greater effect.

The maximum general scour depth, assuming the armor process controls, can be determined by determining the depth of bed material that must be removed to produce an effective armor layer. To make such a determination, the size of sediment which will not move must be determined. This was achieved by equating the critical shear stress as a function of diameter given by Shields' criteria with the actual particle shear stress as a function of average velocity. The results are plotted in Figure 10. Then, by observing the percentage of bed material less than this size, the depth of scour to leave the necessary armor

Table 8. Determination of Aggradation/Degradation Potential From Sediment Transport Capacities

+ stands for aggradation
- stands for degradation
o stands for very little change

| Reach | Aggradation/Degradation Q=66880 cfs | Q=117040 cfs | Q=165440 cfs |
|---|---|---|---|
| 3 | + | + | + |
| 4 | o | + | + |
| 5 | o | - | o |
| 6 | - | - | - |
| 7 | o | - | - |
| 8 | - | o | - |
| 9 | - | - | - |
| 10 | - | - | - |
| 11 | + | + | + |
| 12 | Supply reach | Supply reach | Supply reach |

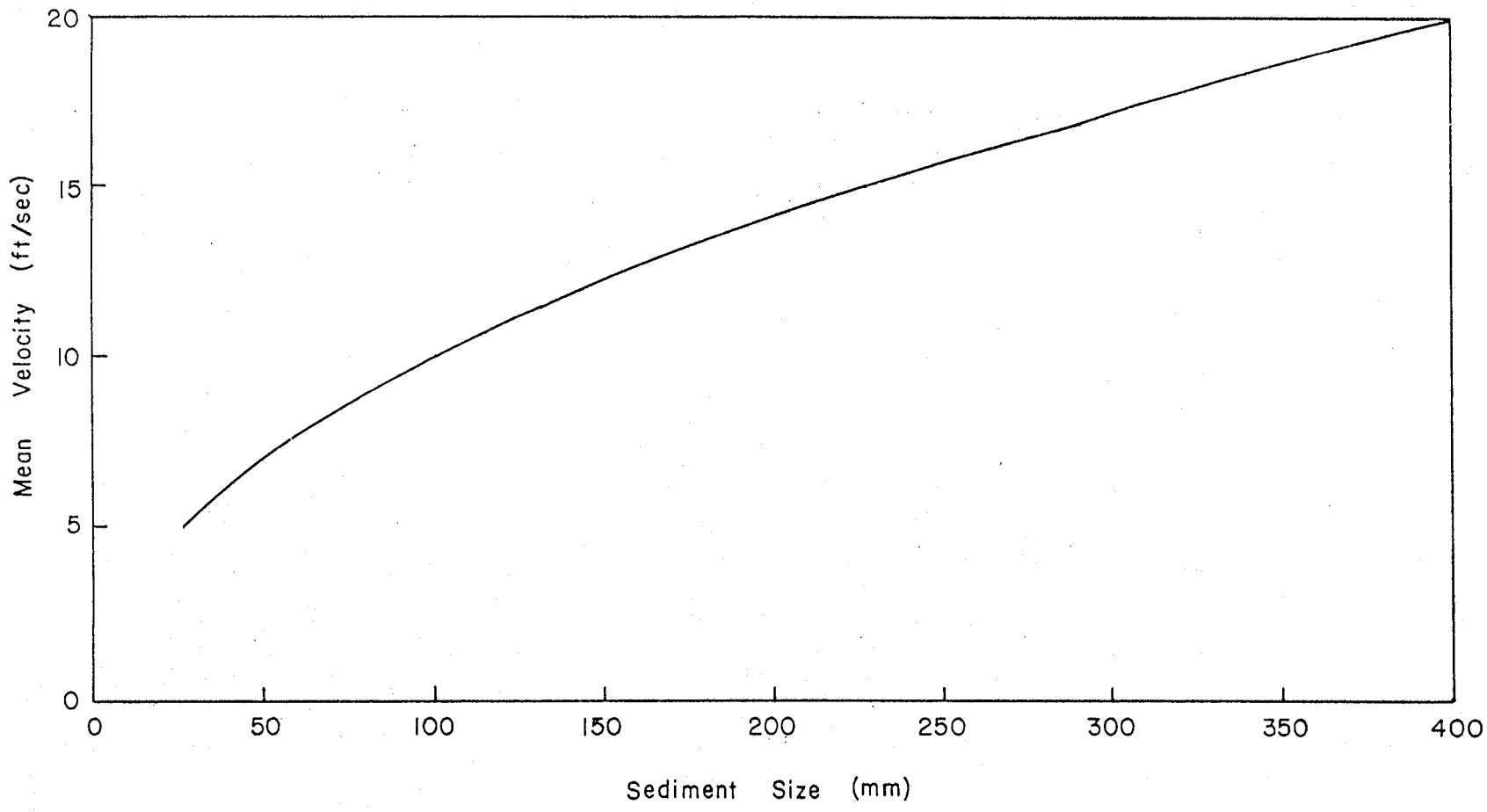



Figure 10. Diagram illustrating velocity required to move a given size sediment.

layer can be calculated. In this case, the channel was assumed to be armored when a layer of non-moving sediment twice as thick as the smallest size non-moving particles built up. The results of the calculations are presented in Table 9. The velocities used are those computed for a discharge of 176,000 cfs. The downstream control used was critical depth at the drop structure near I-10 and the sediment sizes are from the averaged size distribution.

The results are a reasonable estimate of the magnitude of general scour that will occur over a long period of time or during a very large flood event, such as a 100-year flood. Some areas may experience greater depths of scour due to the process of headcutting. This is not taken into account by this analysis. If the armor layer is disrupted while it is forming, then the process must start over. Sand and gravel mining or additional construction in the channel could disrupt the armor layer. The general scour would then be increased.

### 3.4 Physical Process Model Analysis

The general scour potential for the proposed channelized conditions was analyzed for each of the three sediment size distributions and the two downstream control conditions considering the 100-year flood hydrograph. The analysis determines the degradation or aggradation for each cross section using the sediment transport rates and supply computed for the reaches. In addition, the effect of the grade control structure at the I-10 crossing is incorporated.

A series of programs developed by SLA was used for the sediment routing calculations. The sediment routing model uses hydraulic conditions determined by the Army Corps of Engineers HEC-2 program. Sediment routing of the bed material discharge is utilized to calculate degradation or aggradation through the river reaches. Total bed material transport is calculated using

Table 9. Potential Scour Depths Assuming Armoring Controls

| Reach Number | Velocity (ft/sec) | Armor Size (MM) | % Coarser | Depth of Scour (ft) |
|---|---|---|---|---|
| 3 | 10.9 | 120 | 34 | 2.3 |
| 4 | 13.0 | 170 | 24 | 4.6 |
| 5 | 14.5 | 212 | 18 | 7.7 |
| 6 | 15.2 | 233 | 14 | 10.9 |
| 7 | 13.6 | 186 | 22 | 5.5 |
| 8 | 12.5 | 155 | 27 | 3.8 |
| 9 | 9.8 | 97 | 41 | 1.6 |
| 10 | 8.9 | 80 | 44 | 1.2 |
| 11 | 7.1 | 51 | 55 | 0.6 |

the MPM and Einstein combination discussed earlier. HEC-2 cross sections are updated automatically at the end of each time period for elevation changes due to aggradation or degradation. A flow chart of the program execution procedure is given in Figure 11.

The subdivision of the study area into reaches was discussed in Section 2.3. Since Reach 12 was used as a supply reach, scour could not be calculated. Its transporting capacity was determined for each discharge and used as the supply to Reach 11. An accurate determination of supply into the upstream section of the study area is necessary since the erosion and sedimentation process depends on not only the sediment transport capacity, but also the supply into a reach of river. The volume of aggradation or degradation is determined as the excess or deficit, respectively, in sediment supply compared to sediment transport capacity. Thus, by properly accounting for a supply reach upstream of the actual area of interest, the erosion and sedimentation process in the area of interest can be accurately represented.

The channel profiles determined by the analysis are presented in Figures 12 through 17. There is great variation in the channel's response depending on the sediment size distribution used. For the Dames and Moore distribution, general scour is on the order of one to three feet in several key locations. For the HNTB distribution, general scour in excess of ten feet occurs. The averaged distribution, as expected, falls somewhere between the two extremes. After discussion with staff from HNTB, Anderson-Nichols, the City of Phoenix and SLA, it was concluded that the averaged distribution appears to best represent the actual bed-material distribution. Thus, SLA has adopted the results from the averaged distribution for this study. Figures 18 through 21 represent the general scour as a function of time at several key locations, including the ASR, ILS, the section between ILS and ASR, and the Hohokam Expressway. It

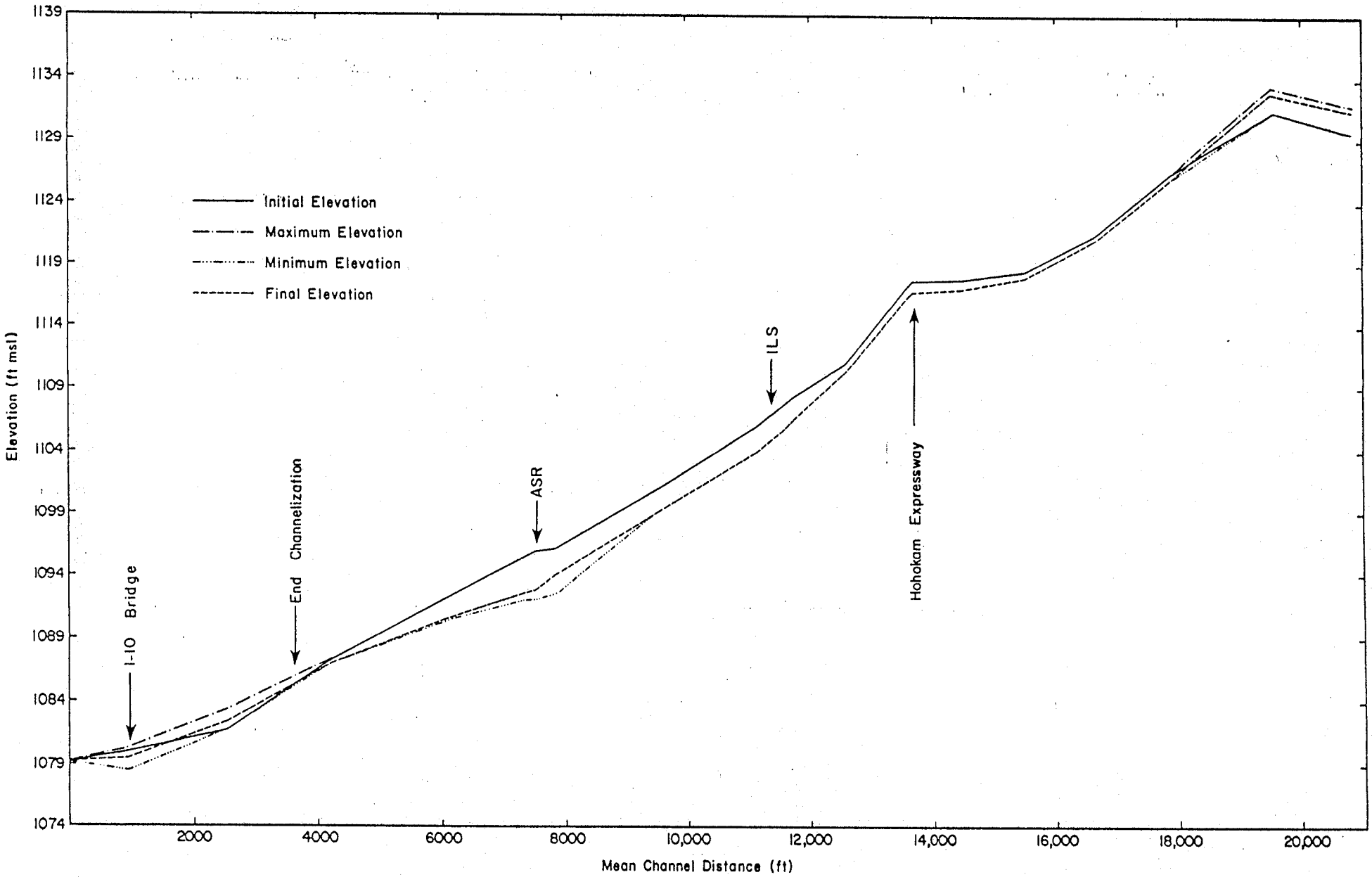



Figure 12. Dames and Moore Size Distribution with critical depth downstream control.

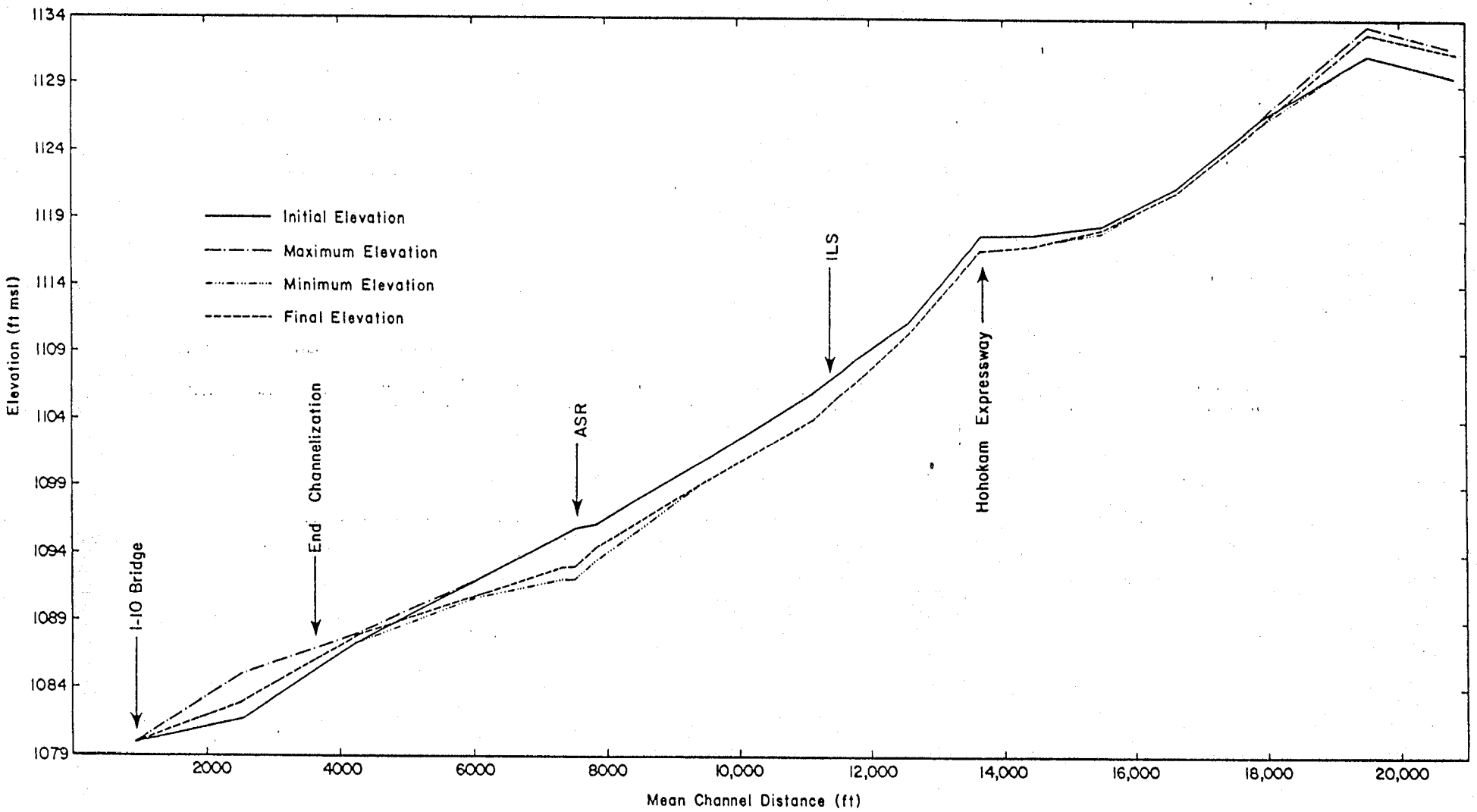



Figure 13. Dames and Moore Size Distribution with stage/discharge relationship for downstream control.

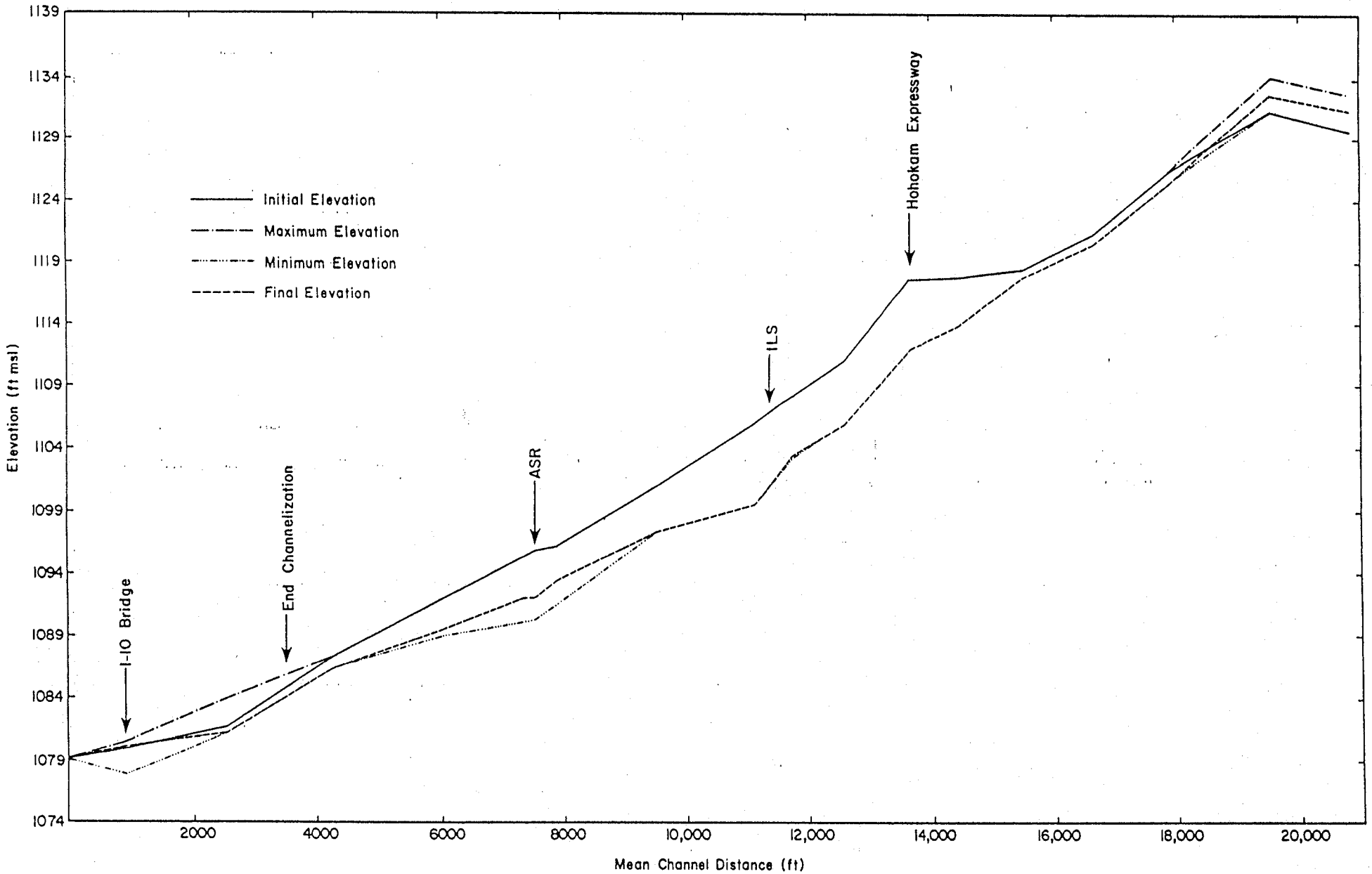



Figure 14. Modified size distribution with critical depth downstream control.

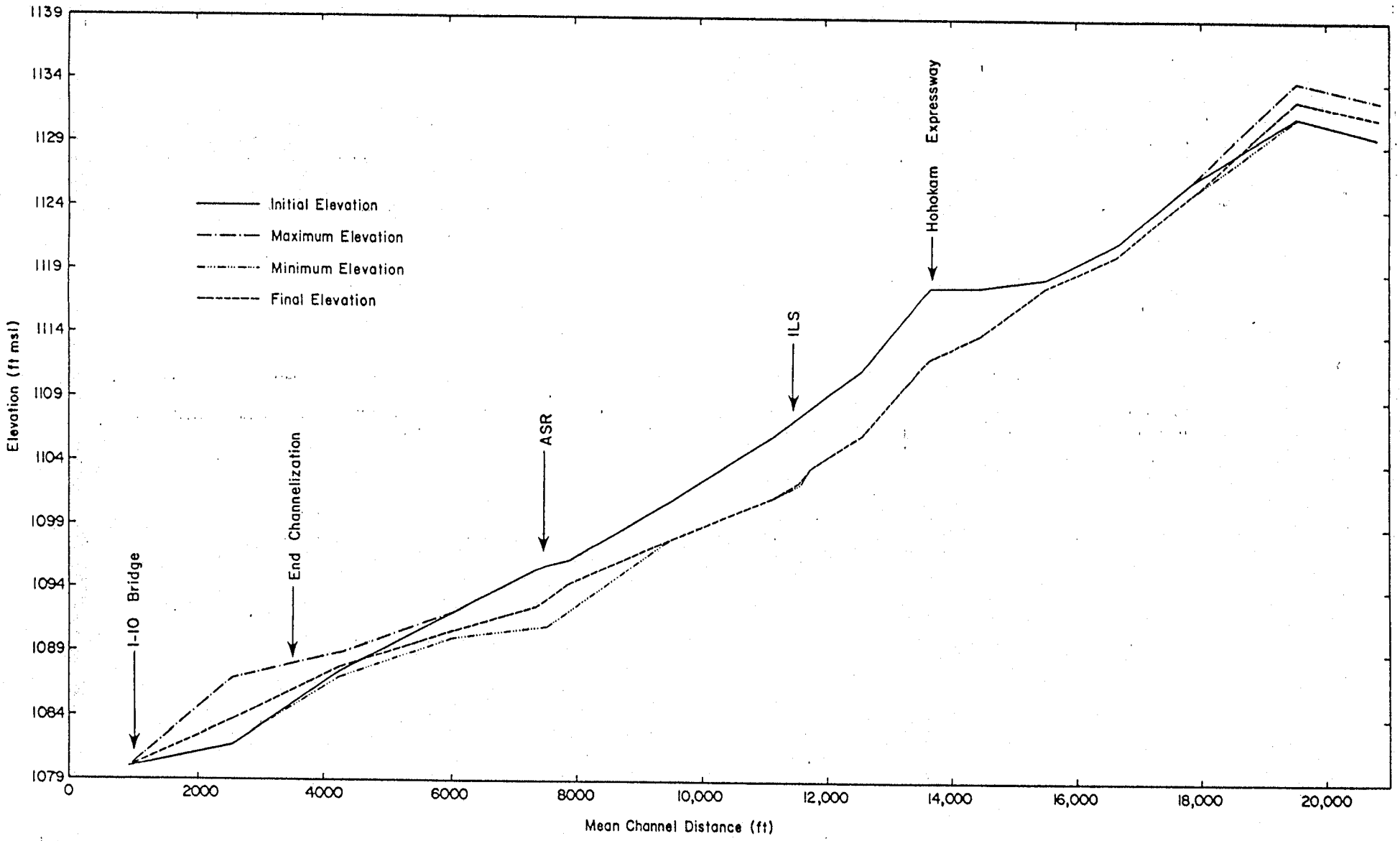



Figure 15. Modified size distribution with stage/discharge relationship for downstream control.

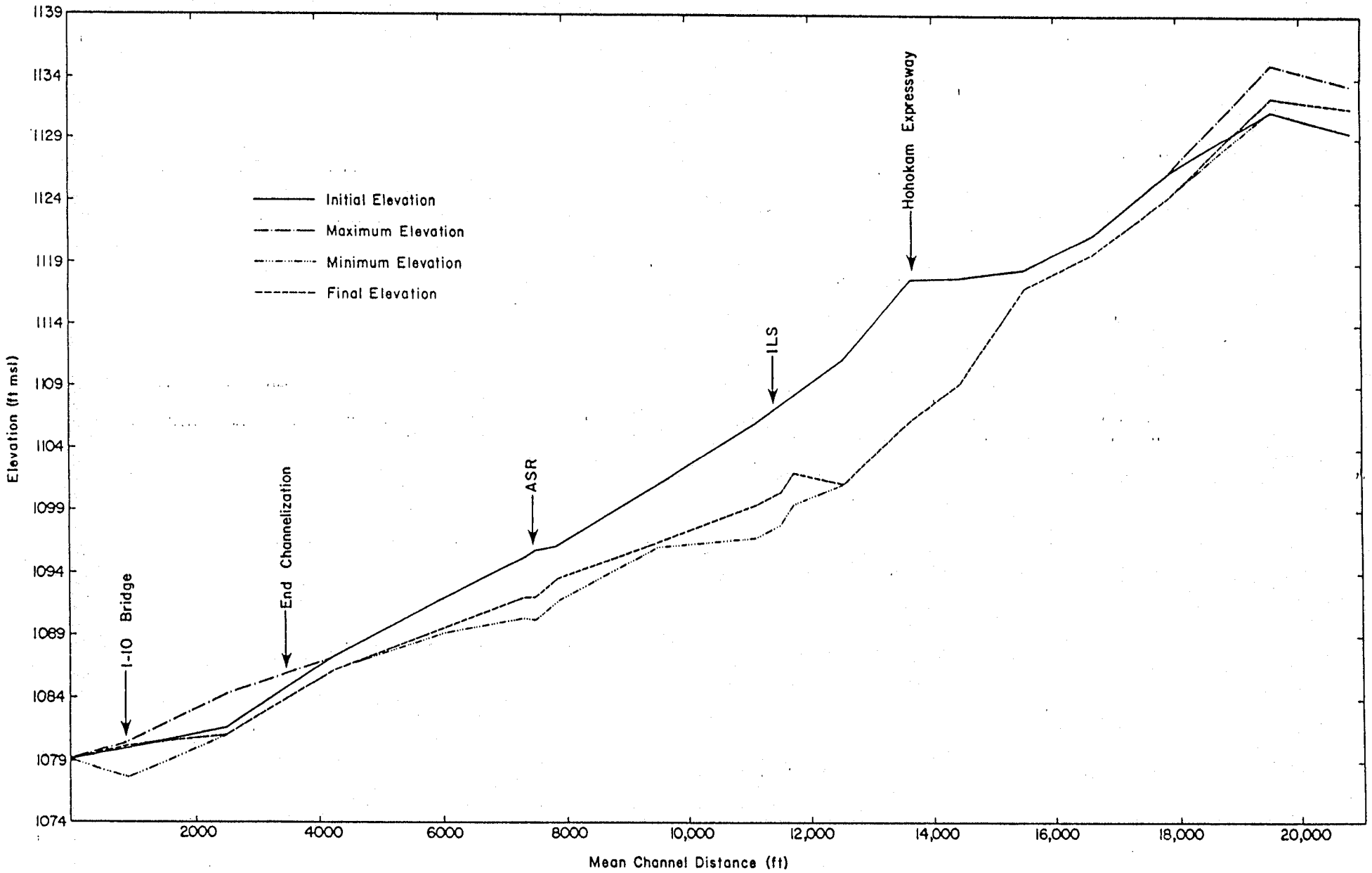



Figure 16. HNTB size distribution with critical depth downstream control.

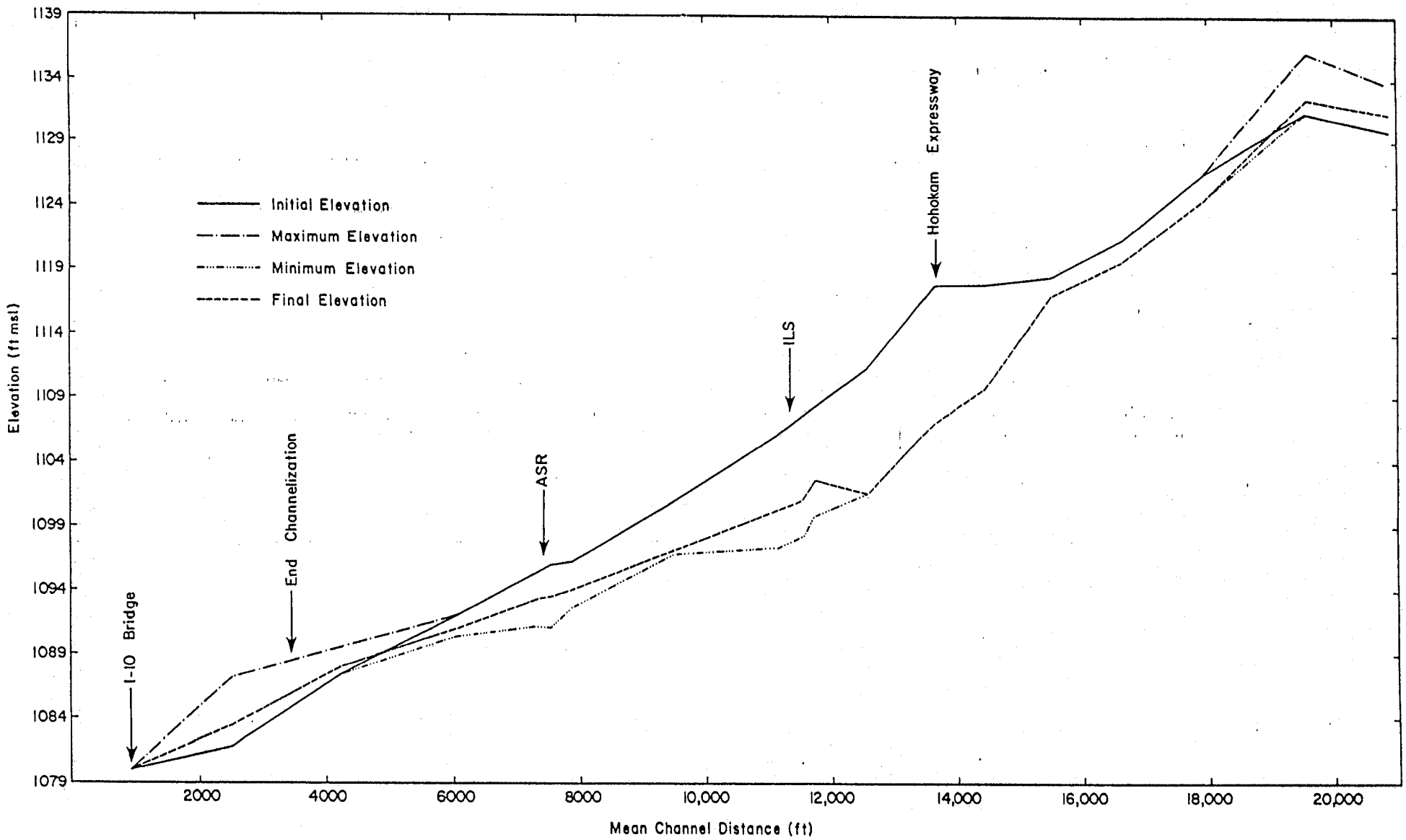



Figure 17. HNTB size distribution with stage/discharge relationship for downstream control.

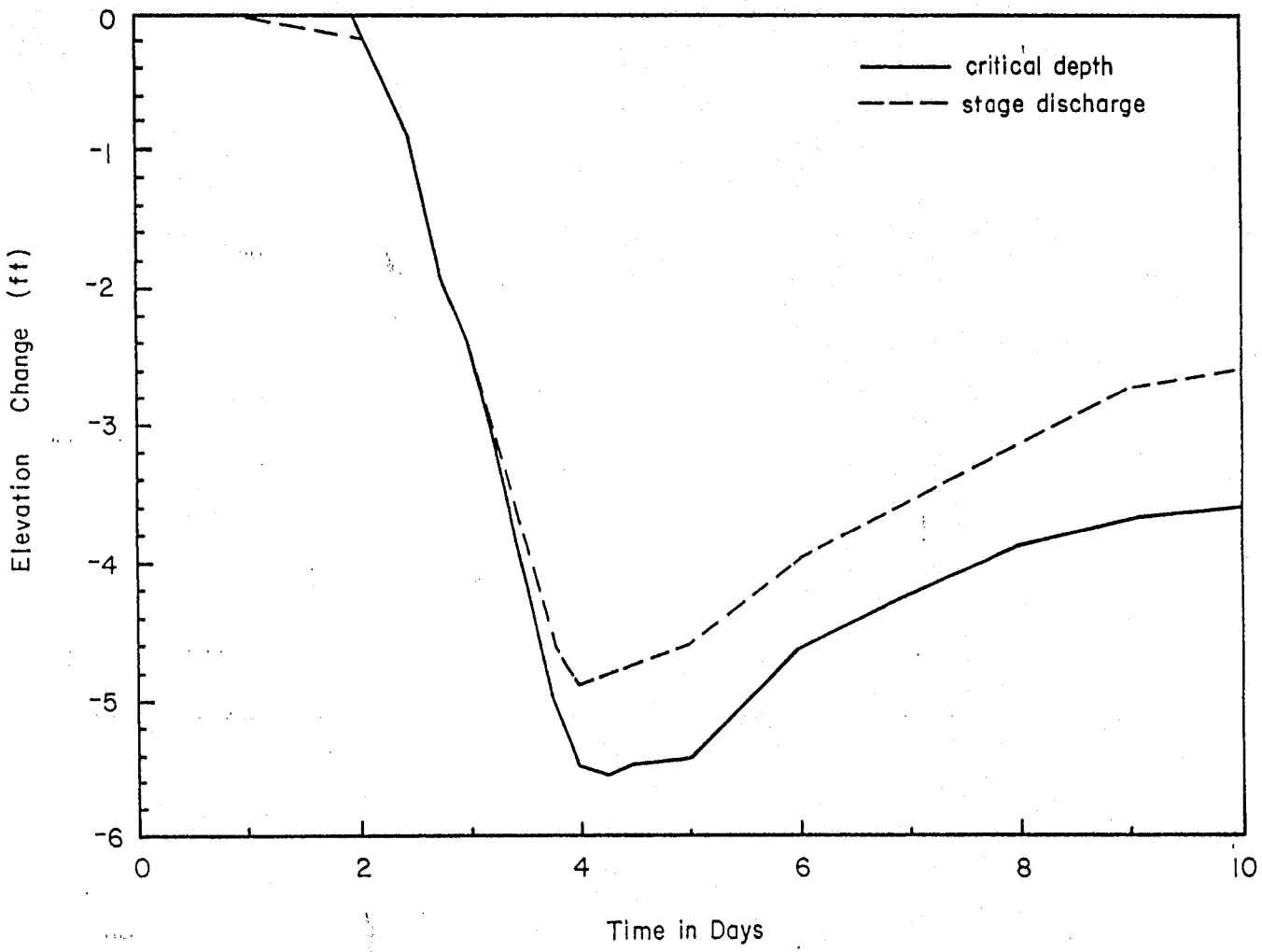



Figure 18. General scour versus time at the ASR for critical depth and stage discharge downstream control.

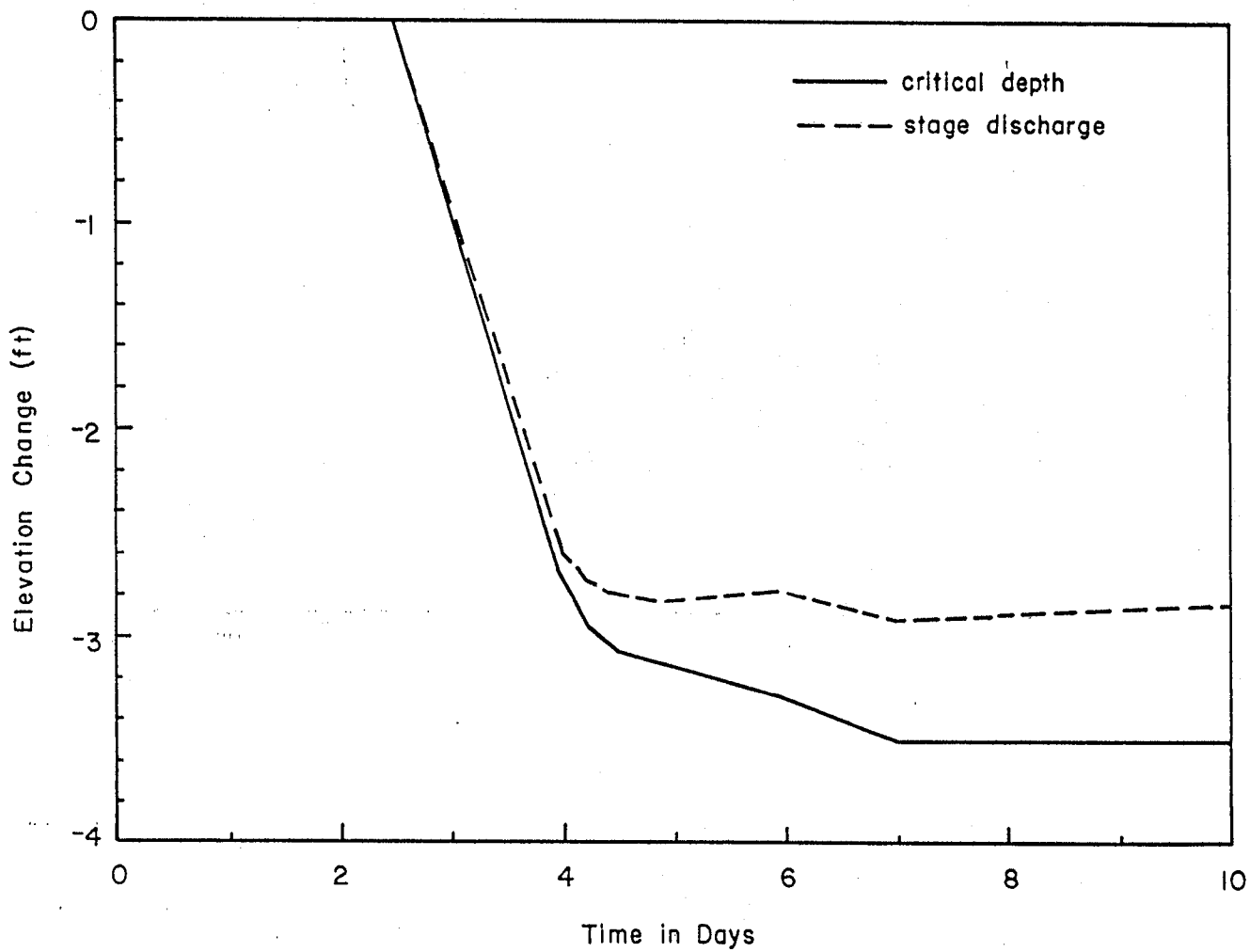



Figure 19. General scour versus time in the section between ILS and ASR for critical depth and stage discharge downstream control.

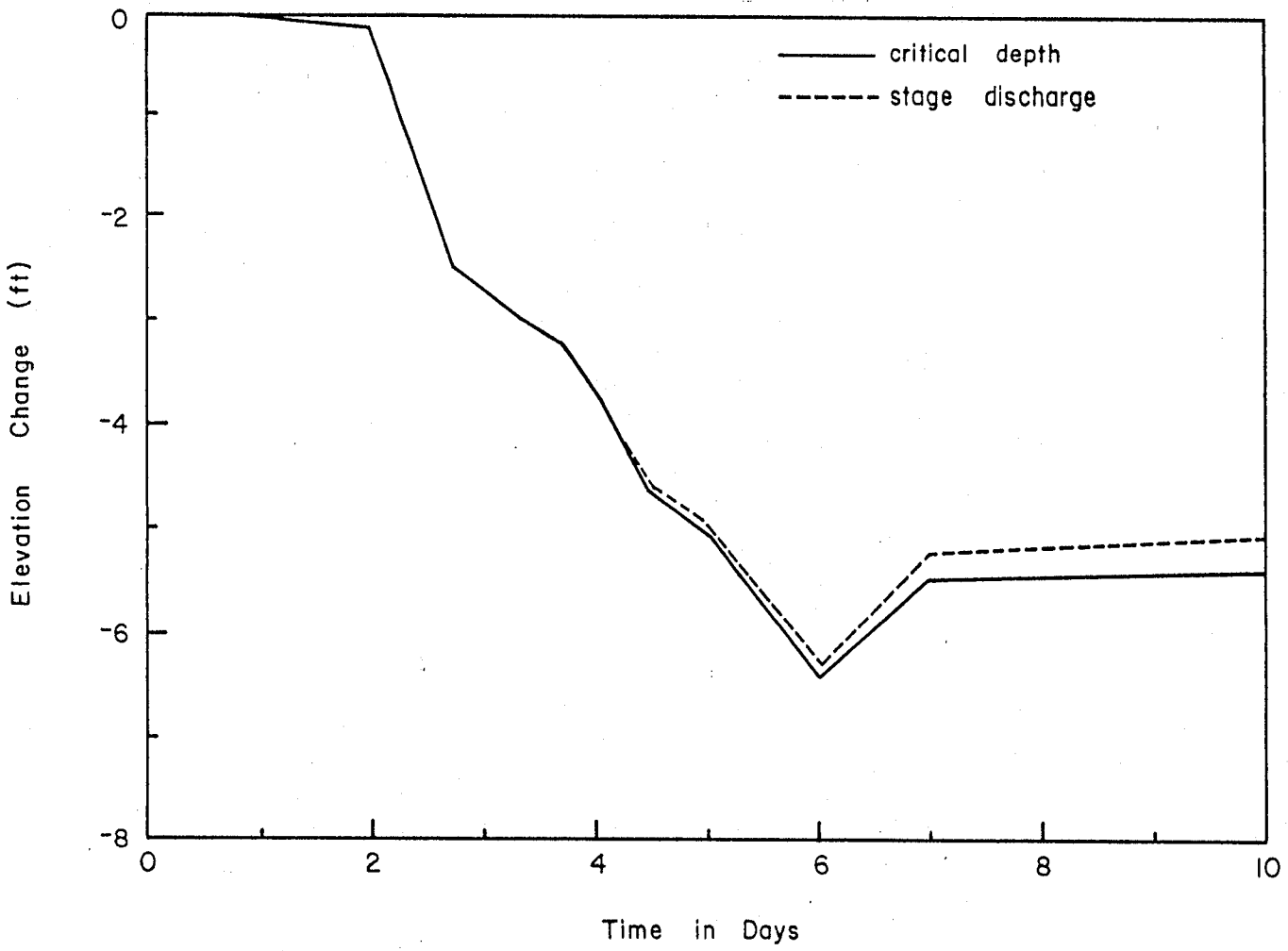



Figure 20. General scour versus time at the ILS for critical depth and stage discharge downstream control.

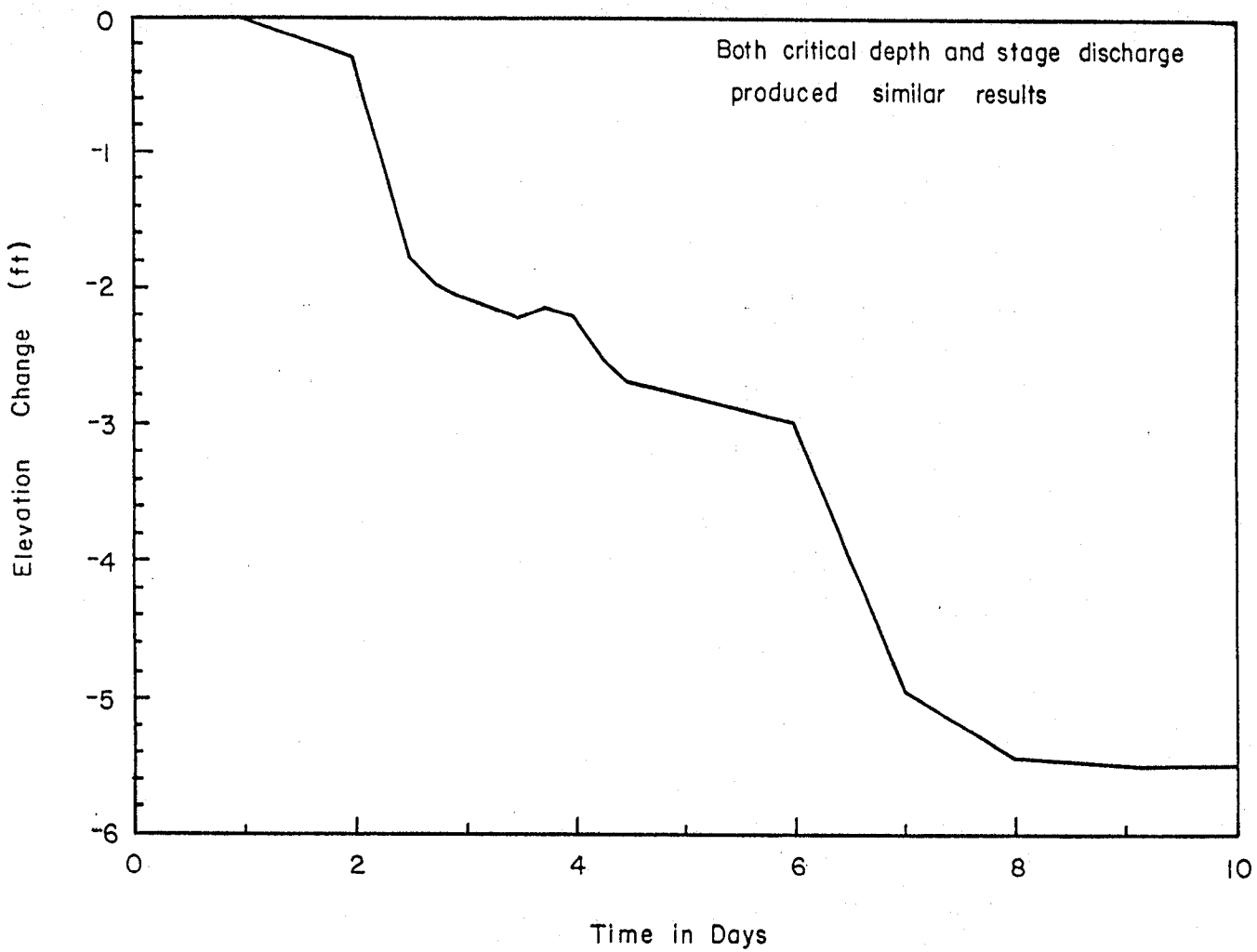



Figure 21. General scour versus time at the Hohokam Expressway for critical depth and stage discharge control.

should be noted that the boundary conditions at the downstream end of the channel have a negligible effect upstream of the ILS.

The profiles and the scour-versus-time figures both show the tendency for a channel at a given location to respond differently over time. For instance, at the ASR the maximum general scour occurs during the peak. Then, during the recession limb of the hydrograph, some of the scour is replaced by deposition because of the reduced velocities in the scoured regions. In the region of the Hohokam Expressway, the scour rates are maximum at the lower flows because backwater effects are small. This condition allows headcutting to continue upstream. For the reach just above the I-10 bridge, for the downstream boundary condition using the computed I-10 stage-discharge relation, there is deposition at high flows from the backwater effect. When the recession limb occurs, the deposition is partially removed.

The results of this third level of analysis agree closely with the first two. As predicted earlier, most of the channel is degrading because of the increased velocities resulting from channelization. The general scour depths are greater than those predicted by the engineering geomorphic analysis in Reaches 8 and 9 because of the influence of the downstream reaches degrading and the resulting headcutting.

### 3.5 Local Scour Analysis

There are several areas within the proposed channelization that produce local hydraulic conditions limited to areas too small to be properly handled by the previous modeling computations. The areas of particular concern are at the ASR, the ILS and the sharp bend immediately northwest of it, and the APS power line towers. These regions all accelerate the flow by constrictions, thus increasing the scour.

Table 10. Maximum Local Scour Depths

| Location | Depth of Local Scour (ft) |
|---|---|
| ASR | 7.1 |
| APS Islands | 3.2 |
| ILS and Dike Immediately Northwest | 8.1 |

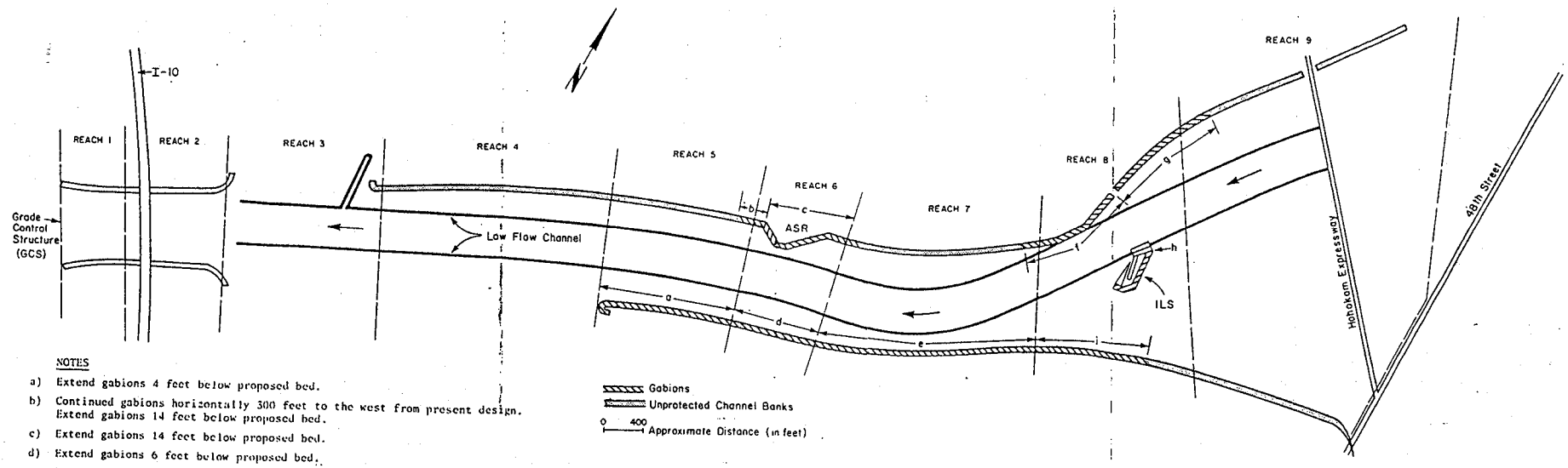
Table 11. Total Scour Depths in Channelized Areas

| Location | Max. General Scour (feet) | Local Scour (feet) | Total Scour (feet) |
|---|---|---|---|
| APS Island in Reach 4 | 2.0 | 3.2 | 5.2 |
| Reach 4 | 2.0 | 0 | 2.0 |
| Reach 5 | 4.0 | 0 | 4.0 |
| ASR (Reach 6) | 5.6 | 7.1 | 13.7 |
| Reach 7 | 3.5 | 0 | 3.5 |
| Reach 8 | 6.4 | 0 | 6.4 |
| ILS and Dike Immediately Northwest (Reach 8) | 6.4 | 8.1 | 14.5 |
| Reach 9 | 5.5 | 0 | 5.5 |
| Reach 10 | 1.5 | 0 | 1.5 |

## IV. CONCLUSIONS AND RECOMMENDATIONS

Three levels of analysis were performed to determine the aggradation/degradation response to implementation of Alternative 4H channelization of the Salt River adjacent to the Sky Harbor Airport, Phoenix, Arizona. The main emphasis was on evaluating the impact of one 100-year flood on the proposed project. Close agreement exists between all three levels of analysis. A total of six cases was considered. The cases were combinations of three possible sediment size distributions and two possible downstream control conditions. The choice of sediment sizes had a large effect on the results. On the other hand, the choice of downstream controls made little difference. The sediment size distribution chosen as most representative was derived by averaging the HNTB and Dames and Moore distributions. The condition of critical flow at the downstream boundary was chosen for determining the water surface profile since it produced slightly greater general scour. Insert ~~The following recommendations and conclusions are based upon this combination of conditions~~.

1. The gabions should be buried below the design bed elevation as a minimum to a depth equal to the total scour depth. Figure 22 gives the recommended burial depths at each location. In addition, the gabion protection at the ASR should be continued downstream an additional 300 feet (also shown in Figure 22). This will provide additional protection due to localized effects caused by the ASR.
2. The most frequent cause of gabion failure in gravel-boulder bed streams is cutting of the wire baskets by the large material being transported along the bed. For this reason, the gabions near the bed should be protected by an extra layer of rock mesh wire, soil cement, an additional layer of gabions, rock riprap, or some other suitable means. The protection should extend from two feet above




NOTES

a) Extend gabions 4 feet below proposed bed.

b) Continued gabions horizontally 300 feet to the west from present design. Extend gabions 14 feet below proposed bed.

c) Extend gabions 14 feet below proposed bed.

d) Extend gabions 6 feet below proposed bed.

e) Extend gabions 4 feet below proposed bed.

f) Extend gabions 15 feet below proposed bed.

g) Extend gabions 7 feet below proposed bed.

h) Extend gabions 15 feet below proposed bed.

i) Extend gabions 7 feet below proposed bed.

Figure 22. Recommended channel protection.

ADDITIONAL NOTE: All gabions should have protection extended from their burial depth to 2 feet above the proposed bed elevation.

the design bed elevations down the remaining depth of the gabion. The side slope and flow conditions in the project area require riprap at least two feet in diameter. Because such material is not available, riprap is infeasible. An illustration of the protection is given in Figure 23. The gabions should be checked at regular intervals and after large flows for damage.

3. In order to reduce the amount of general scour, wherever a surface layer of large material is present it should be stripped and stockpiled. Then, after construction of the channel, it should be placed on the new channel's surface. In this manner an armor layer will already be partially formed before flood flows occur. As a result, general scour depths will be reduced (local scour depths will remain the same). This practice should bring general scour depths into the range predicted for the cases in which the Dames and Moore sediment size distributions were used. This practice should be considered in the area immediately below the Hohokam Expressway and near the ILS. It would be best to armor the entire channel, if feasible, since much of the sediment prevented from being scoured in one location will be picked up in any unarmored areas downstream.

4. The estimated scour depths should be increased by an acceptable factor of safety and should be used to determine appropriate measures to protect buried utilities such as the 69-inch Four City sewer line.

5. In order to protect the APS tower from erosion, it is understood that the towers would be placed on extended footings by APS. This would eliminate the islands in the channel and any problems they would cause by constricting and concentrating the flow.

6. With proper placement of coarser material in the channel bed, the Hohokam Expressway bridge should be able to stand alone if its approaches are washed away. As evidenced by the February, 1980, flood, this appears to be the case.

7. Whenever funds become available, the channelization should be continued upstream and downstream as recommended by HNTB. For the present, the plans should be modified to include:

   a. Extension downstream to the I-10 channelization and an appropriate transition.

   b. Protection of the leading edge of the dikes at the upstream end of the project, and

   c. A transition zone from the natural channel to the low-flow channel at the Hohokam Expressway bridge.

   These three measures should be included since a principal cause of channel failure is inadequate transitions.

8. The unprotected dikes, if constructed from the present bed material, will not withstand velocities in excess of four or five ft/sec. The best estimate of velocities near the dikes can be taken from the physical model study. These estimates should be used to determine if additional dike protection is required.

9. The channel as proposed will require significant maintenance after the occurrence of the design flood or any event of similar or greater magnitude. Areas where large scour depths result, such as the ILS and ASR, will require reworking of the entire channel. The low-flow channel will be damaged throughout the entire project.

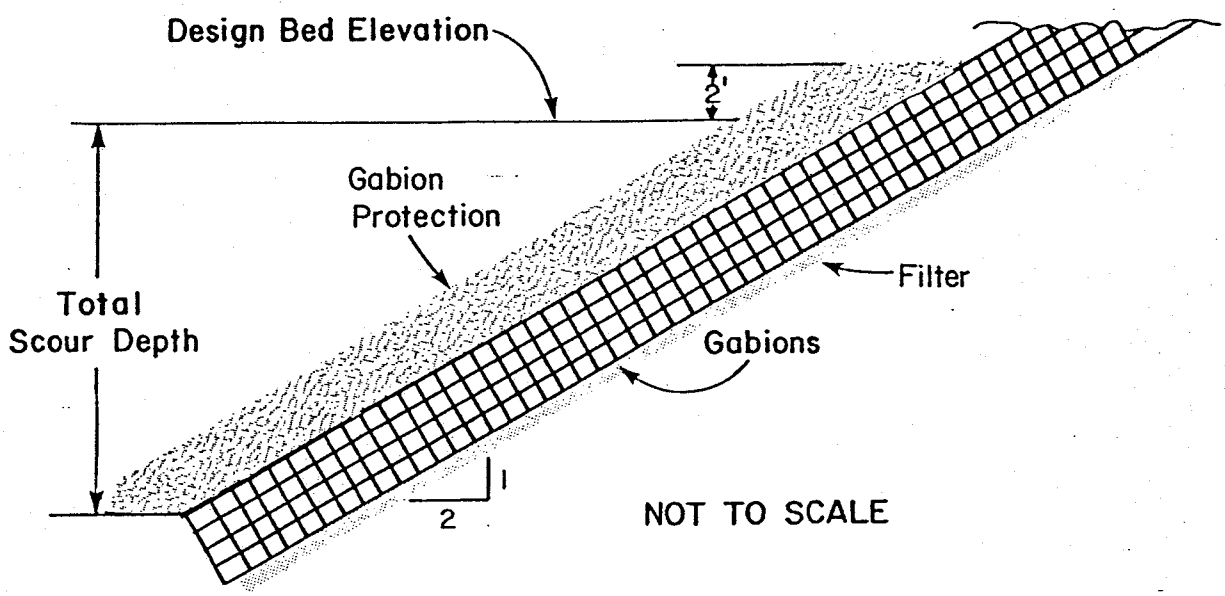



Figure 23. Recommended gabion protection.

10. Damage to the channelization during the design flood would be reduced by removal of the ILS, ASR, and APS power line islands to locations away from the channel. This would eliminate three large obstructions in the flow.

11. The above conclusions and recommendations are made assuming no significant sand and gravel mining occurs in or near the channelization. This is a highly unlikely assumption, due to the present number of operations in the area. Unfortunately, these activities can have extremely adverse effects upon the proposed channelization. The report on "Sand and Gravel Mining Guidelines" (Boyle Engineering, July, 1980) studied the possible impacts. The following was summarized in the report.

> Sand and gravel mining in the Salt, Gila, and Agua Fria Rivers has caused hydraulic changes in the channels. As a result of these changes, nearby lands and structures have been threatened or damaged during floods.
>
> The most severe problem caused by in-channel extraction in the study area is headward erosion upstream of a pit. In addition, the presence of dikes and stockpiles in the floodplain may divert flood flows, causing erosion of the opposite bank and lateral migration of the channel.

The report went on to quantify some of the possible impacts. For mining pits of the size occurring in the vicinity of the channelization, headcuts in excess of ten feet can develop. Such a headcut during the design flood would destroy much of the channelization.