Prepared for the U.S. Army Corps of Engineers, Sacramento
Federal Emergency Management Agency, Region IX

FINAL REPORT

Design Review of the Salt River Channelization Project

Phoenix, Arizona

30 October 1980

Anderson-Nichols/West
Engineers • Environmental Consultants • Planners

Assisted by
Colorado State University

FINAL REPORT

DESIGN REVIEW OF THE SALT RIVER CHANNELIZATION PROJECT

by

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Contract Number: DACW05-80-C-0093

30 October 1980

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I. INTRODUCTION

A comprehensive review of the project design for the interim Salt River channelization project, as proposed by the City of Phoenix, Arizona, was accomplished under contract DACW05-80-C-0093 with the Sacramento District of the U.S. Army Corps of Engineers. The general goals of review are: evaluation of the ability of the proposed design to mitigate flood hazards, estimation of flood-related damages to the proposed channel and surrounding properties, and formulation of recommended modifications to the channel design. This final report discusses all aspects of the review, except for a separate study of the potential impacts of gravel mining on the proposed channel, and supercedes our interim report dated July 1980. A supplemental report on the gravel mining studies will be submitted under separate cover.

The tasks covered by this report are as follows:

1. Review, evaluate, and verify the hydraulic computations to determine the adequacy of the interim channel design to pass 176,000 and 210,000 cfs with safety and without major damage to airport facilities, the channel, or adjacent properties.
2. Verify that additional upstream or downstream flood hazards are not caused by the proposed channelization, due to a rise in water surface elevation.
3. Conduct movable bed mathematical and physical model studies for flows of 92,000, 176,000 and 210,000 cfs to estimate the degree and cost of damage to the proposed channel work.
4. If applicable, make recommendations for protection to a higher degree than that afforded by the proposed interim channel design, including estimated costs thereof.

5. Evaluate the compliance of the proposed project with F.I.A. regulations and Executive Order 11988.

In order to accomplish these tasks, a data collection, analysis, and verification process was undertaken to provide the most accurate and up-to-date basis for evaluation. Computer model (HEC-2) runs of the natural and channelized conditions under the three flood magnitudes of interest were made to predict river hydraulic behavior. Flood water surface profiles, top widths, and velocities were compared for natural channelized conditions to establish the hydraulic effects of the proposed channel. A fixed-bed hydraulic model of the proposed channel was constructed and operated to simulate hydraulic behavior without sediment transport. Then a movable bed model was constructed to the same scale and operated to simulate the scour and deposition that is likely to occur during the specified flood events. A computer model for sediment transport (HEC-6) was tested for ability to predict sediment movement in the channel. Subsequently, the results of numerical model runs performed by another contractor were reviewed. Finally, the results of the four models were compared and analyzed to provide the most accurate picture of proposed channel behavior. From this analysis, estimates of flood damages and potential design modifications were developed.

The above investigative process was designed to minimize the uncertainties inherent in any model of an unknown stream configuration. Since the prototype channel does not yet exist, no verifications of model behavior can be made except through comparisons between substantively different types of models. Available prototype data, such as natural channel behavior during historical floods, was used to verify numerical models for existing conditions. The verified models were then run for design conditions and compared with physical model results. The use of four types of models not only provides confidence in ultimate findings, but also gives insight into the relative importance and impact of sediment processes on channel behavior. As will be seen later, such insights are most helpful in evaluating channel performance.

Section II of this report gives our principal findings in terms of channel adequacy, damages to the channel, damages to protected facilities, compliance with regulations, and recommended design modifications. Section III identifies study data sources and critical issues in the study. Descriptions of model structure and summaries of model results are presented in sections IV - VIII. A comparison between model results is made in section IX, leading to a set of conclusions discussed in sections X - XII. Conclusions include damage estimates, regulatory compliance, and recommended channel modifications.

This study was performed by Mr. Benjamin Roberts, Mr. Ned Shanahan, Mr. Geoff Raaka, and Ms. Marion Rule, under the direction of Dr. Dennis Horn and Mr. Richard King. Physical model studies were performed by staff of the Engineering Research Center at Colorado State University under the direction of Dr. Yung Hai Chen and Dr. Darryl B. Simons. Mr. Bernard Gordon provided expert advice in the areas of soil mechanics and levee design. The numerical sediment modeling results reviewed during this study were performed by the staff of Simons, Li and Associates under contract to HNTB.

II. PRINCIPAL FINDINGS

The findings of this study with regard to flood-related damages, adequacy of channel design, compliance with applicable federal regulations, and recommended design modifications are summarized below.

ESTIMATED DAMAGES DUE TO FLOOD EVENTS

The following types of damages are expected to result from the 92,000 cfs (10 year) and 210,000 cfs (100 year) events investigated in this study. Fixed-bed model runs at 176,000 cfs were made to determine the differences between the 210,000 cfs discharge and the 176,000 cfs discharge. Only minor changes in velocity and water surface elevations were apparent, so no damage studies for the 176,000 cfs case were accomplished Cost estimates are approximate and are intended to provide order of magnitude estimates of damages that can be expected to be caused by the 10 year and 100 year floods.

1. Modification of Channel Bed - Extensive degradation and aggradation of the channel bed will occur for both the 10-year and 100-year events. Approximately 490,000 and 650,000 cubic yards of material will be displaced by the 10-year and 100-year events, respectively. Scour holes up to 17 feet deep and bars up to 9 feet high are formed by sediment transport. A general degradation of the bed occurs along the full length of the channel. The low flow channel is completely destroyed by the 100-year event and severely damaged by the 10-year event.
2. Damage to Levees - Extensive portions of the unarmored levees will be damaged or will fail

during flood events. Under the 100 year flood approximately 2800 lineal feet of unprotected levees will be damaged and 4700 lineal feet will be severely damaged to the point of failure. Approximately 8500 lineal feet of armored levees will be damaged due to scour at the levee toe and/or excessive velocities with subsequent undercutting or slumping. Failure of the gabion armoring due to the impact of stones and debris is possible along several sections of armored levee. There is also potential for levee slumping or failure due to piping of fine materials under or through the embankment.

3. Damage to Protected Properties - Properties outside the levees, such as the ILS, radar station, airport runways and hangars, and private structures, will be damaged by flood events only if the levees breach. Detailed analyses of the failure process for levees were not accomplished, however a possible levee failure scenario was investigated. Very little damage would occur if the south levee breached, due to high ground and sparse development behind the levee. The north levee is most likely to breach upstream of the ILS (near station 12,000) and downstream of the radar (near station 5,000). Overland flow would then develop behind and through the south runway and ponding would occur on substantial areas of airport property. Estimation of costs associated with such a breach is not possible, given available data. Breaching of the north levee is likely in the 100-year event but very unlikely in the 10-year event, assuming proper channel maintenance.

4. Damage to Roadways - Roadways located in the channel (e.g., Hohokam Expressway, 40th Street) will be subject to severe damage during both the 10-year and 100-year events. The proposed technique of building the roadways below grade may protect some parts of the roadways, but the migration of the low flow channel certainly will destroy some roadway sections. Detailed examination of roadway stability is beyond the scope of this study.
5. Total Damages - The quantifiable damages associated with flooding are estimated as follows:

	10-Year Event	100-Year Event
Channel Bed	$850,000	$1,100,000
Levees	$610,000	$ 940,000

PERFORMANCE OF PROPOSED CHANNEL

Both HNTB and the City of Phoenix have stated that the proposed channel was not designed to be free of damages after major flood events. However, regulations under which FEMA provides reconstruction funds require that funded structures not be subjected to repetitive damages.

The following review criteria were adopted for this review, based on these regulatory requirements.

1. The channel must withstand the 100-year flood event without failure.
2. No increase in flood stages above natural conditions may occur for the 100-year event.
3. The project can not be subject to significant repetitive flood-related damages.
4. No damages to surrounding property beyond what would occur under natural conditions can result from flood events under channelized conditions.

Proposed channel performance with respect to the above criteria is described below.

1. <u>Hydraulic Capacity</u> - The proposed channel is capable of carrying the 100-year event without overtopping and without raising flood stages above natural conditions. Levee damage leading to slumping could, however, cause overtopping of affected levees unless additional protection is provided.
2. <u>Levee Design</u> - Levee freeboards are adequate except at several upstream sections, where added height is required. General levee design is adequate, but proper preparation of levee foundations may require removal of sand and silt lenses which are near the channel design grade. Care in selecting and compacting levee materials is also necessary.
3. <u>Levee Armor Design</u> - The extent of levee protection is inadequate; substantial added armoring is required to prevent damage and/or failure of levees. Depth of armoring is inadequate to prevent undercutting and slumping of levees in several sections. The gabion baskets are subject to damage by moving rock and debris.
4. <u>Channel Bottom Design</u> - The low flow channel will not withstand the 10-year or 100-year events without extensive damage. The proposed design does not accomodate the substantial changes in thalweg depths or locations which will occur during floods.
5. <u>Protection of Surrounding Properties</u> - The level of protection afforded by the channel is adequate, provided that levees do not fail. Substantial potential for levee failure exists in the present design.

COMPLIANCE WITH REGULATIONS

There is evidence that significant repetitive damages will occur if the proposed design is implemented. Hence, the project will not be in compliance with sections of the Disaster Response and Recovery Regulations, the National Flood Insurance Program, or Executive Order 11988. The project is in compliance with F.I.A. regulations dealing with rises in water surface elevations.

RECOMMENDED DESIGN MODIFICATIONS

The following design modifications should be considered as potential remedies for the inadequacies discussed above. These are only candidate modifications; detailed analyses and design efforts will be required to verify their applicability and effectiveness.

1. Increase levee heights by 1-2 feet over approximately 6,000 lineal feet of levee.
2. Add gabion or other protection to approximately 5,700 lineal feet of unprotected levees.
3. In those areas where scour is expected to exceed five feet adjacent to the levees, eliminate hinged gabions and extend gabions below the design channel bottom to a depth of between 7 and 17 feet, depending of extent of scour. Total levee length requiring buried protection is 12,500 feet.

4. Add a protective layer of stone rip-rap on top of gabions from the maximum buried depth to 2 feet above the design channel bottom. This would be required only for levee sections that also require buried gabions.
5. If investigations of levee foundations for potential piping problems has not been done, perform boring tests or other tests and modify levee design as required.
6. Consider the possibility of initially constructing the channel bed in a configuration that approximates the bed contours produced by the physical model runs. Such an equilibrium configuration will minimize aggradation or degradation of the bed during floods. No bottom reconstruction would then be needed after flood events, except where scour threatens levees or other structures.

The total cost associated with these modifications (excluding points 5 and 6) is estimated to be over $1.7 million.

III. BASIS FOR REVIEW

The following data sources were used in this review.

1. Natural river cross sections: 1979 topographic mapping and digitized 1980 cross sections by HNTB, Inc., and 1980 cross sections by Dames and Moore (vicinity I-10).
2. Channelized river cross sections: design drawings and digitized cross sections from 1980 photogrammetry by HNTB, Inc.
3. Water surface elevations for calibration: high water marks for February, 1980 flood from photogrammetry, observations, and measurements for the I-10 bridge.
4. Discharges: specified as 92,000, 176,000 and 210,000 cfs in the scope of work; assumed constant over the study reach.
5. Hydraulic parameters in HEC-2 runs: HNTB, Inc. selections, reviewed and modified as required for calibration.
6. Proposed project design details: one-half size engineering drawings (June 1980) and preliminary specifications (July 1980) by HNTB; revised half size drawings by HNTB (September 1980).
7. Scour processes: "Hydraulic and Scour Analysis of Salt River Bridge of Phoenix - Casa Grande Highway. . . ", Simons, Li and Associates, Inc., June 1980
8. Impacts of gravel mining: "Sand and Gravel Mining Guidelines", Boyle Engineering Corporation for the L.A. District, U.S. Army Corps of Engineers. (July, 1980)

9. Levee design and protection criteria: "Design and Construction of Levees," U.S. Army Corps of Engineers (March 1978); "Hydraulic Design of Flood Control Channels," Engineer Manual EM 1110-2-1601, U.S. Army Corps of Engineers, (July 1979); "Erosion Control Devices, Methods, and Practices," Highway and Heavy Construction, September, 1978, pp. 25-34
10. I-10 Proposed Channel: "Phases C and D, Final Design, Permanent Protection Against Scour at Salt River Bridge - Interstate 10," Dames and Moore (July, 1980); Plans and Profile of Proposed I-10 crossing, Arizona Dept. of Transportation (July 1980)
11. Inflow hydrograph: composite normalized hydrograph based on 50 year, 100 year, and project flood hydrographs synthesized by the Los Angeles District, U.S. Army Corps of Engineers.
12. Cost estimates: "Preliminary Report on the Channelization of the Salt River at Sky Harbor Airport," HNTB (February 1980); Dames and Moore report on Phases C and D; Concept Report on Salt River Channel at Sky Harbor Airport, Royden Engineering; bid quotations for the I-10 channelization (Arizona D.O.T.).
13. Bed materials: "Preliminary Report on the Channelization...", HNTB; Dames and Moore report on Phases C and D.

A complete bibliography is included as an appendix.

This review examines the hydraulic and sediment conditions likely to occur under selected discharges with planned channel works in place in the study reach. Since the focus is on the adequacy of the channel design and its effect on natural river behavior, comparisons are made between natural conditions and channelized conditions. The morphology of the flood plain in the study area has changed substantially during the last series of floods (1978 - 1980), as evidenced by changes in rating curves at Joint Head Dam gage and at the I-10 bridge, by comparisons of aerial photographs, and by comparisons of cross sections. Gravel mining operations also have, apparently, caused major modifications in the flood plain, although this is not accurately documented.

Considerable effort has been expended to insure that the topographic and sediment data and hydraulic parameters used in the study reflect 1980 conditions as accurately as possible. March 1980 aerial photogrammetry and digitized cross sections were used within the channelized areas and in the vicinity of the I-10 bridge. May 1979 photogrammetry and digitized cross sections. were used in the rest of the study area. Channelized conditions were established by the design drawings, preliminary specifications, and design cross sections provided by HNTB (September 1980). Both HNTB and Dames and Moore test pit data was used to establish bed material sizes and checked against site visit observations.

The design under review in this study was originally specified by the City of Phoenix in a set of drawings and bid documents released in June 1980. Certain modifications were made during the summer and a new set of design drawings were released by the City of Phoenix in September. Changes included: a southward realignment of the north levee upstream of the ILS, shortening of the south levee downstream of the radar facility, increased armoring of the north levee, addition of armoring on the south levee, removal of all armoring on the low flow channel, modifications to the channel invert elevations upstream of I-10, and other minor changes. All major changes have been incorporated into the models and analyses presented here.

Where appropriate, changes in results are discussed. Numerous additional design modifications are under discussion, but none are included in this review.

Uncertainties as to certain aspects of the design were identified in our interim report, and many still exist. For the purposes of this review, the following resolutions have been accepted by FEMA.

1. No gravel mining operations will occur at or near the channel. This issue will be dealt with in a separate report.
2. Channel protection will consist of that shown in the September 1980 plans by HNTB.
3. Filling of low areas behind the levees will occur only in the vicinity of the airport, as shown in the plans.
4. The high voltage towers will remain in the channel, except for the tower within the I-10 channel right of way and the next tower upstream. Initial physical model runs showed that the effect of the two removed towers on velocities and turbulence within the I-10 channel was extremely detrimental.

The range of hydraulic and sediment conditions considered in this review has been limited to those which provide a representative view of channel behavior. Peak discharges were selected to be 92,000 cfs (10-year event to simulate a frequent flood), 176,000 cfs (FIS 100-year event) and 210,000 cfs (Corps of Engineers 100-year event established in 1980). The 210,000 cfs event has been selected by FEMA as the new 100-year event for analysis purposes and will be of principal concern in this review. Sediment loads are specified by equilibrium conditions upstream of the proposed channel and by the nature of channel bed material. No artificial bed modifications are made before or during model runs and no alternative bed materials are tested.

The modes of potential damage to or failure of the channel and levees are considered to be:

1. scour and or depositing of materials in the channel bottom;
2. surface scour on unprotected levee faces;
3. undermining of the toes of both protected and unprotected levees, causing slumping;
4. breaking of gabion baskets due to rock impacts and or shear generated by high velocities;
5. loss of the stability of levees and foundations due to saturation during high water;
6. piping of materials through levees and foundations; and
7. overtopping and erosion of levees due to inadequate freeboard.

The model investigations are designed to provide information necessary to assess the likelihood and extent of such occurences.

IV NUMERICAL HYDRAULIC ANALYSES

In order to evaluate the hydraulic behavior of the Salt River in the study area, mathematical model (HEC - 2) runs were performed for both natural and channelized conditions at flood discharges of 92,000, 176,000, and 210,000 cfs. Flood water surface profiles, top widths, and velocities were compared for the natural and channelized conditions to establish the hydraulic effects of the proposed channel.

The HEC-2 analysis presented in this report is a revised version of the analysis performed in conjunction with the interim report of July 30. The revision was necessary to incorporate additional topographic and design data received from HNTB since July. Changes include: relocation of the north levee southward beyond a major Hohokam Expressway intersection, modification of cross sections upstream of the ILS and downstream of the proposed levees, and a reduction in the downstream length of the south levee. The effects of these changes are negligible and are not discussed further in this section.

HYDRAULIC ANALYSIS: NATURAL CONDITIONS

Hydraulic model (HEC-2) runs were made using data for unchannelized or "natural conditions" to provide a baseline for assessing the impact of the channel project on flood hydraulics. An examination of topographic maps, aerial photographs, and digitized cross sections taken successively in 1977, 1979 and 1980 revealed substantial changes in river bed morphology due to both flood-related sediment transport and gravel mining operations. Changes in the hydraulic rating curve for the Joint Head Dam streamflow gage were substantial, indicating morphological changes in the upstream reaches of the study area. Changes in the cross section at the I-10 bridge show a degradation of up to 15 ft. due to the February 1980 flood. It was clear that neither the 1977 Flood Insurance Study hydraulic analysis nor the 1979 HEC-2 runs made by HNTB represented the 1980 natural conditions.

The natural river configuration used in this study was compiled from March 1980 digitized cross sections supplemented by 1979 topographic data in the upstream reaches above the proposed channel. Starting water surface elevations for all cases were determined from observed stage discharge relationships at the I-10 bridge during the February 1980 flood (see the I-10 modification design, phase C and D report, by Dames and Moore). Model hydraulic parameters were adjusted to calibrate the model against the February 1980 flood, estimated by the U.S. Geological Survey as having a peak discharge of 180,000 cfs within the study reaches. High water marks for this flood were compared with model output for the 176,000 cfs case. The calibrated HEC-2 model results showed good agreement with the limited high water mark data. (Flood water surface mapping was underway at the time of this study) .

Flood elevations on the Salt River were then determined in HEC-2 computer runs of 176,000 and 210,000 cfs under natural conditions. Tables IV-1 and IV-2 present the elevations and predicted mean velocities from the HEC-2 runs at selected cross sections within the study area. The locations of the cross sections are indicated in Figure IV-1, made from an aerial photograph of the study area after the flood of 1980. The channel bottom and flood elevations for floods of 176,000 and 210,000 cfs under natural conditions have been labeled on Figures IV-2 and IV-3. The 100-year flood

TABLE IV-1

SALT RIVER 176,000 CFS FLOOD

Cross Section*	Natural Condition		Project Design		Levee Heights	Freeboard
	Elevation	Velocity**	Elevation	Velocity**		
A (1055)	1104.5	11.1	1101.1	10.4	Outside of Project	
B (1900)	1106.3	6.8	1102.6	9.7	Outside of Project	
C (3280)	1108.2	7.2	1104.4	8.6	Outside of Project	
D (4600)	1109.4	6.1	1105.9	10.9	1115.7	9.8
E (6670)	1112.8	9.8	1111.6	11.3	1117.8	6.2
F (7270)	1114.1	10.1	1112.7	11.7	1120.0	7.3
G (7670)	1114.1	14.8	1112.7	15.2	1121.0	8.3
H (8270)	1120.4	6.7	1116.2	11.2	1121.9	5.7
I (10530)	1126.7	6.0	1120.7	12.7	1125.6	4.9
J (11330)	1127.2	7.7	1123.8	8.8	1128.3	4.5
K (11730)	1128.4	6.0	1124.5	9.0	1129.9	5.4
L (12360)	1129.1	5.2	1126.2	7.1	1131.7	5.5
M (13610)	1130.4	6.6	1128.2	6.9	1133.1	4.9
N (14410)	1131.8	9.3	1131.2	10.3	1134.8	3.6
O (15470)	1134.4	6.8	1134.7	6.5	Outside of Project	
P (17850)	1140.0	9.0	1140.0	9.0	Outside of Project	

* Stationing in feet above location 1000' D/S of I-10

** Mean velocity for cross section in fps

Project design includes I-10 improvements and HNTB channelization

All elevations in feet (NGVD)

Freeboard in feet

TABLE IV-2

SALT RIVER 210,000 CFS FLOOD

Cross Section*	Natural Condition		Project Design		Levee Heights	Freeboard
	Elevation	Velocity**	Elevation	Velocity**		
A (1055)	1108.3	9.8	1102.8	10.9	Outside of Project	
B (1900)	1109.6	6.2	1104.2	10.0	Outside of Project	
C (3280)	1110.9	7.4	1106.2	9.1	Outside of Project	
D (4600)	1112.0	6.2	1107.6	11.4	1115.7	8.1
E (6670)	1114.6	10.0	1113.2	12.1	1117.8	4.6
F (7270)	1115.9	10.3	1114.4	12.5	1120.0	5.6
G (7670)	1115.7	14.8	1114.1	16.3	1121.0	6.9
H (8270)	1121.9	7.0	1118.0	11.9	1121.9	3.9
I (10530)	1127.8	6.2	1122.3	13.5	1125.6	3.3
J (11330)	1128.4	7.9	1125.7	9.1	1128.3	2.6
K (11730)	1129.5	6.4	1126.3	9.3	1129.9	3.6
L (12360)	1130.2	5.5	1128.0	7.1	1131.7	3.7
M (13610)	1131.4	6.7	1129.7	6.8	1133.1	3.4
N (14410)	1132.6	9.7	1131.9	10.9	1134.8	2.9
O (15470)	1135.1	7.2	1135.5	6.9	Outside of Project	
P (17850)	1140.8	9.4	1140.8	9.4	Outside of Project	

* Stationing in feet above location 1000' D/S of I-10

** Mean velocity for cross section in fps

Project design includes I-10 improvements and HNTB channelization

All elevations in feet (NGVD)

Freeboard in feet

SALT RIVER
MARCH 12, 1980
ANDERSON - NICHOLS

FIGURE IV - 1

Photograph Courtesy of HNTB

SALT RIVER 176,000 CFS FLØØD
HNTB Channel Project
Radar Installation
Runway Localizer
I-10
Water Surface-
Flood Insurance Study-100year flood
Water Surface- Project Design Condition
Water Surface-Natural Condition
Channel Bottom-
Project Design Condition
Channel Bottom-Natural Condition
Elevation in feet (NGVD)
1210.
1190.
1170.
1150.
1130.
1110.
1090.
1070.
0.
5000.
10000.
15000.
20000.
25000.
Stationing in feet above location 1,000′ downstream of I-10

FIGURE IV- 2


SALT RIVER 210,000 CFS FLØØD
HNTB Channel Project
Radar Installation
Runway Localizer
I-10
Water Surface-
Flood Insurance Study-100 year flood
Water Surface-Natural Condition
Water Surface-Project Design Condition
Channel Bottom-
Project Design Condition
Channel Bottom-Natural Condition
Elevation in feet (NGVD)
1210.
1190.
1170.
1150.
1130.
1110.
1090.
1070.
0.
5000.
10000.
15000.
20000.
25000.
Stationing in feet above location 1,000' downstream of I-10
A B C D E F G H I J K L M N O P

FIGURE IV-3

elevations determined in the Flood Insurance Studies for Phoenix and Tempe have also been indicated on the profiles. These FIS elevations represent HEC-2 runs done in 1977 with a 100-year discharge of 173,000 cfs. The 100-year FIS profile is presented in this report to indicate the changes that have occurred in the flood characteristics of the river. Our analysis has employed different discharges and topographic data and observed discrepancies with the existing 100-year FIS flood elevations are not unexpected. Such discrepancies point out the need for updating the 1977 FIS to include changes in Salt River hydraulic behavior.

HYDRAULIC ANALYSIS: PROJECT DESIGN CONDITIONS

The project design condition modeled in this analysis includes the proposed improvements at the I-10 bridge as well as the channelization design of HNTB (Alternative 4H). The I-10 improvements as designed by Dames and Moore under contract to the Arizona Department of Transportation are scheduled to be completed on or before the planned construction of the channelization project. Peak discharges of 92,000, 176,000, and 210,000 cfs were modeled using the HEC-2 program. Starting water surface elevations for these project design computer runs were based upon stage/discharge relationships developed in connection with the Dames and Moore I-10 bridge improvement project.

The flood elevations, predicted mean velocities, design levee heights, and levee freeboard at selected cross sections are presented in Tables IV-1, IV-2, and IV-3. Figure IV-1 indicates the locations of the tabulated cross sections. The channel bottom and the water surface elevations for the project design at discharges of 176,000 and 210,000 cfs are plotted on Figures IV-2 and IV-3, respectively.

TABLE IV-3

SALT RIVER 92,000 CFS FLOOD

Cross Section*	Project Design Elevation	Velocity**
A (1055)	1094.8	8.3
B (1900)	1095.9	8.2
C (3280)	1097.4	8.2
D (4600)	1100.4	9.9
E (6670)	1106.7	8.9
F (7270)	1107.8	9.3
G (7670)	1108.3	11.7
H (8270)	1110.9	9.0
I (10530)	1115.9	10.1
J (11330)	1118.6	7.7
K (11730)	1119.4	8.4
L (12360)	1121.5	6.9
M (13610)	1125.0	6.7
N (14410)	1129.0	9.0
O (15470)	1132.3	5.2
P (17850)	1137.2	8.1

* Stationing in feet above location 1000' D/S of I-10

** Mean velocity for cross section in fps

Project design includes I-10 improvements and HNTB channelization

All elevations in feet (NGVD)

The project design effectively reduces flooding on the Salt River in the study area. The water surface elevations for the 176,000 and 210,000 cfs floods are at or below the corresponding natural flood heights throughout virtually all of the study area. The only location where the project design model yielded higher flood heights than the corresponding natural flood was at cross section O, just upstream of the study area. For the 176,000 cfs flood, the water surface elevation at cross section O was 0.3 feet higher than under natural conditions. The water surface elevation increase at cross section O was 0.4 feet above natural conditions for the 210,000 cfs flood. In our judgment these increases are insignificant and do not result in increased flood hazards.

The velocities presented in Tables IV-1, IV-2, and IV-3 are average velocities across the entire flooded area at each cross section as predicted by the HEC-2 computer runs. Velocities in localized areas can be expected to vary considerably from these predicted means. The velocity predictions in Tables IV-1 and IV-2 show that the project design will cause velocity increases throughout most of the study area. The impacts of these velocities on scour and levee stability will be discussed in following sections.

To evaluate the impact of the channelization project by itself, another modeling study was completed that assumed the I-10 bridge improvements would not be instituted. In this analysis, the model was run with discharges of 176,000 and 210,000 cfs and the results were compared to the appropriate natural flood elevations. It was found that between I -10 and the radar installation (cross section G) the lowering of the natural water surface elevations was substantially less without the I-10 improvements than with the improvements. However, in this reach, the channelization project still exhibited water surface elevations at or below the natural flood elevations. Upstream of the radar installation the water surface elevations calculated for project design with and without the I-10 improvements were virtually identical. In summary, the proposed channelization will pass the specified flood flows without overtopping or creating rises in water surfaces above natural conditions, with or without the I-10 bridge improvements.

It should be emphasized that the hydraulic studies were performed using the assumptions described in Section 2. If gravel mining occurs in the channel, or unprotected areas of the levees fail, or levee instability causes levee collapse, or scour and deposition of the channel during floods significantly affects the proposed channelized cross sections, the hydraulics would be substantially altered from that shown in this section. These possibilities are evaluated in subsequent sections.

V. PHYSICAL MODEL DESIGN AND CONSTRUCTION

MODELING THEORY

Two models, a fixed-bed model and a movable-bed model, were utilized in this study. The fixed-bed model is easy to control and provides a basis to study the effects of the channelization project with consideration of nonuniform velocity and flow patterns and excluding the complexity due to bed and bank movement. Specifically, the fixed-bed model was utilized to determine:

1. Velocity distribution to identify levee areas that require protection and to determine extent of protection;
2. Water surface profiles to check the adequacy of levee freeboard;
3. Effects of the low-flow channel on the flow distribution; and
4. Other hydraulic effects caused by the channelization project.

The movable-bed model was utilized to determine:

1. Riverbed erosion and deposition patterns during passages of flood hydrographs;
2. Effects of bed and bank changes on velocity and water surface profiles and subsequent effects on the adequacy and stability of the levees.

The slope used in modeling the levees in the movable bed model had to be distorted in order to form the bed material at a stable angle. This could affect the velocity and stage as well as sediment transport patterns. The results of the movable bed model will be compared with the fixed-bed model results to evaluate the effects due to this distortion. In the past, distorted fixed-bank, movable-bed models have been utilized to study erosion and deposition problems. However, no attempts have been made to also distort the bank slope to study bank stability and bed movement at the same time.

A set of rigid-bank, movable-bed model runs will be made to document the effects of levee slope and erosion on model behavior and will be discussed in the subsequent report on gravel mining.

To satisfy the similarity between the model and the prototype, the roughness of a fixed-bed model should be adequately simulated. The movable-bed model also requires proper simulation of bed-material transport, levee stability, and scour and deposition patterns. This was accomplished by matching the incipient motion characteristics of the bed material and of the levees between the model and the prototype under different flow conditions and by determining the time scaling ratio of bed wave movement.

Model Scaling Ratios

Past studies have shown that the effect of surface tension and viscosity can be neglected when the model is sufficiently large and is tested with warm water and a relatively high Reynolds number. Therefore, the model was constructed, tested, and operated as a Froude model.

The Froude number N_f for open channel is

$$N_F = \frac{V}{\sqrt{gD}} \tag{1}$$

in which V is the mean velocity at the channel cross section, D is the hydraulic depth and g is the gravitational acceleration. The flow discharge is

$$Q=VA$$

in which A is the cross-sectional area and V is defined above. Also, the hydraulic depth is defined:

$$D=A/T \tag{3}$$

in which T is the top width.

For achieving geometric, kinematic and dynamic similarity, the Froude number should be the same for the prototype and the model, that is

$$N_{fm} = N_{fp} \tag{4}$$

$$\frac{V_m}{\sqrt{g_m D_m}} = \frac{V_p}{\sqrt{g_p D_p}} \tag{5}$$

Subscripts "m" and "p" refer to the model and prototype, respectively.

For all practical purposes the gravitational acceleration is the same in Arizona and Colorado. Then Eq. 5 can be reduced to

$$\frac{V_m}{V_p} = \left(\frac{D_m}{D_p}\right)^{1/2} \tag{7}$$

To adequately model the study area and to achieve the study objectives, the horizontal and vertical model-to prototype length ratios were selected to be 1 to 175 and 1 to 35 respectively. From Eq. 7, the scaling ratios for the geometric, kinematic and dynamic variables are:

VARIABLES	SCALING RATIO
1. Horizontal length	1:175
2. Vertical length	1:35
3. Time (water wave)	$1:175/\sqrt{35}$
4. Velocity	$1:\sqrt{35}$
5. Discharge	$1:(35^{3/2})\ (175)$
6. Force	$1:(35^{2})\ (175)$

Bed Roughness in the Fixed-Bed Model

The bed roughness in the model was chosen to obtain kinematic and dynamic similarity between the prototype and the model and thereby achieve equivalent normal-flow conditions. Manning's equation and Strickler's equation are therefore appropriate in determining the roughness required in the model.

Manning's equation is given as

$$V = \frac{1.486}{n} \quad R^{2/3}\ S_f^{\ 1/2} \tag{8}$$

in which n is Manning's roughness coefficient, R is the hydraulic radius, and S_f is the energy slope. The ratio of model's to prototype's roughness coefficient is then

$$\frac{n_m}{n_p} = \left(\frac{R_m}{R_p}\right)^{2/3} \left(\frac{S_{fm}}{S_{fp}}\right)^{1/2} \left(\frac{V_p}{V_m}\right) \tag{9}$$

From Froude's Model Law

$$\frac{V_p}{V_m} = \left(\frac{D_p}{D_m}\right)^{1/2} \tag{10}$$

Also

$$\frac{R_p}{R_m} \approx \frac{D_p}{D_m} \tag{11}$$

Substituting Eqs. 10 and 11 into Eq. 9 yields

$$\frac{n_m}{n_p} = \left(\frac{D_m}{D_p}\right)^{1/6} \left(\frac{S_{fm}}{S_{fp}}\right)^{1/2} \tag{12}$$

Based on a size analysis of composite bed-material samples provided by Dames and Moore and Howard Needles Tammen and Bergendoff (HNTB), Simons, Li and Associates (SLA) developed a size distribution of subsurface bed-material in the prototype channel having d16=2.6mm, d50=60mm, d75=160mm, d84=210mm and d90=23mm. This prototype size distribution was utilized to size the bed material in the fixed-bed model as well as in the movable-bed model. Meyer-Peter and Muller transformed Strickler's formula to determine Manning's roughness coefficient as follows:

$$n = \frac{d_{90}^{1/6}}{26} \tag{13}$$

where d90 is the diameter of sediment particles in meters for which 90 percent of the particles are finer by weight. For the prototype the value of d90 is 230mm and therefore n_p=0.03'. This value of np and the value of Dm/Dp(=1/35) are substituted into Eq. 12 to yield

$$n_m = 0.0164 \left(\frac{S_{fm}}{S_{fp}}\right)^{1/2} \tag{14}$$

Applying Eq. 14 for different ratios of Sf_m/Sf_p, the following values for n_m and d90 are obtained.

S_{fm}/S_{fp}	n_m	d_{90} (mm)
1	0.0164	6.0
1.8	0.0220	35.1
2	0.0232	48.1
3	0.0284	162.3
4	0.0328	384.7
5	0.0367	751.3

The horizontal and vertical length scales utilized in this model study would automatically result in a slope ratio Sf_m/Sf_p=5. To obtain adequate surface roughness for this ratio, the d90 would be 751.3mm, which was not acceptable. Due to the required volume and availability of gravel materials, it was decided to use 1.5-inch gravel to cover the riverbed of the rigid-bed model. This requires a reduction of Sf_m/Sf_p from 5 to 1.8. Measured model elevations must, as a result, be adjusted to account for the effect of this slope distortion.

Bed Material in the Movable-Bed Model

To adequately model the erosion and deposition patterns, the incipient motion of bed material in the model should be comparable to that in the prototype. Shield's relation for beginning of motion is

$$\frac{\tau_c}{(\gamma_s - \gamma)} = 0.047 \qquad (15)$$

in which τ_c is the critical bed shear stress defined as f is the Darcy-Weisbach resistance coefficient, p is the water density, $(\gamma_s - \gamma)$ is the unit weight of submerged sediment.

An "f" value of 0.08, which corresponds to the required Manning's n and a γ_s value of 165 lbs/ft were substituted into Eq. 15 to obtain a relation between critical velocity, shear and bed material size as shown in Table V-1. To obtain adequate incipient motion of the bed material in the movable-bed model, its size distriution should have d50 2.0 mm, d75 4.5 mm and d84 7.0 mm. After contacting various gravel companies in the Fort Collins area, it was found that hte gravel material available locally that had the closest size distribution, was a 1/4-inch chip. The size distribution of this material was determined from a sieve analysis as shown in Fig. V-1. The d50, d75, and d84 are 3.2 mm, 4.5 mm and 5.4 mm, respectively.

TABLE V-1 Critical Velocity, Shear Stress and Bed-Material Size

Prototype			Model		
V_{cp} (fps)	T_{cp} (lbs/ft^2)	d_{cp} (min)	V_{cm} (fps)	T_{cm} (lbs/ft^2	d_{cm} (min)
6.5	0.82	52	1.10	0.023	1.5
8.0	1.24	78	1.35	0.035	2.2
10.4	2.10	133	1.76	0.060	3.8
12.5	3.03	192	2.11	0.086	5.4
13.5	3.54	224	2.28	0.101	6.4
15.0	4.37	276	2.54	0.125	7.9
16.0	4.96	314	2.70	0.141	8.9
20.0	7.76	490	3.38	0.222	14.0

The value of d50 is larger and the value of d84 is smaller than that desired. Therefore the simulation of armor effects at local scour areas could be somewhat inaccurate. However, it is believed that the movable-bed model should be capable of simulating the overall erosion and deposition pattern reasonably well.

Because of the reduced bed material size in the movable-bed model, its surface roughness would be smaller than that in the fixed-bed model. However, the form roughness in the movable-bed model might increase the resistance to flow to reach a reasonable agreement in bed roughness effects.

Time Scaling Ratios for the Movable-Bed Model

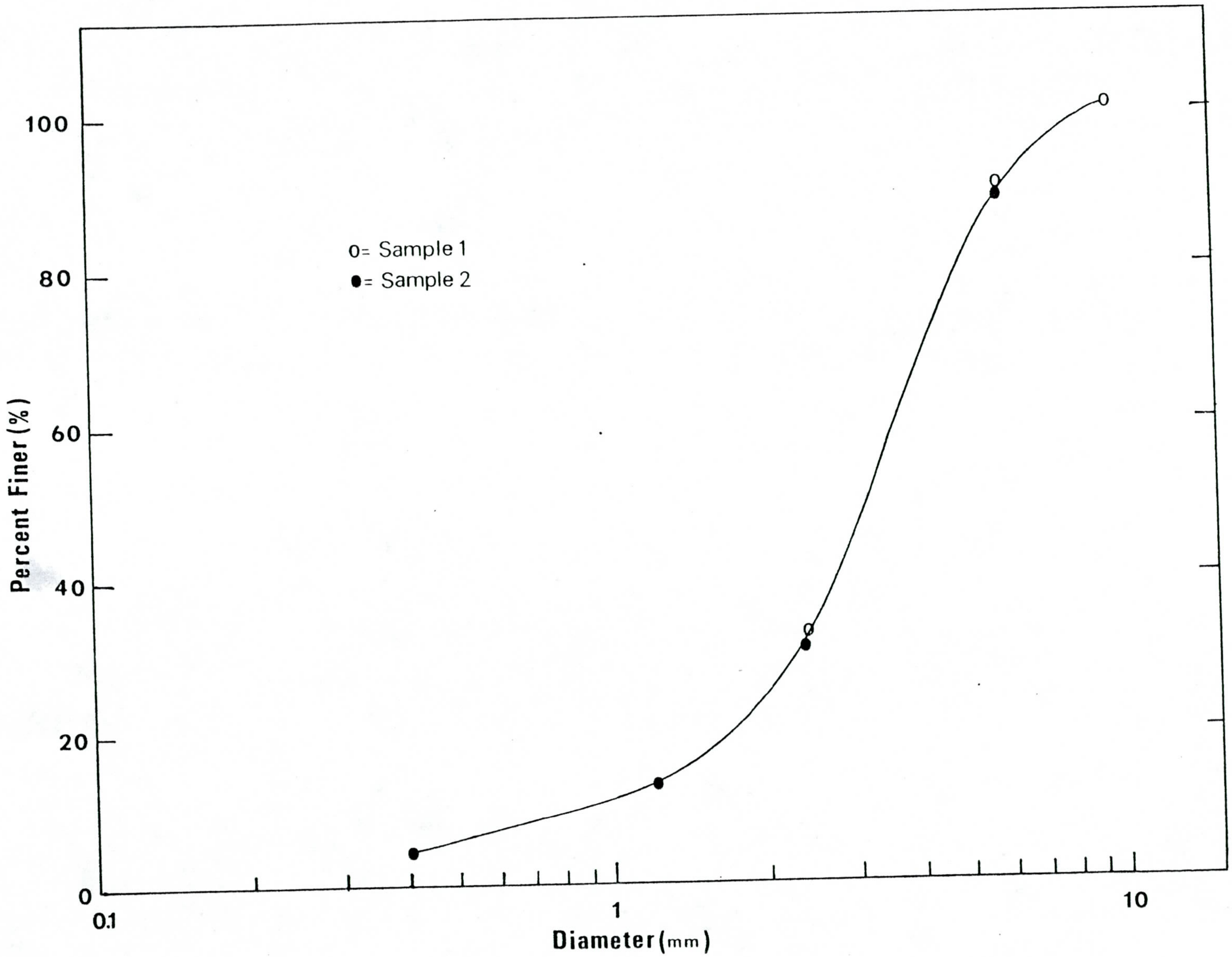


Fig. V-1 Size Distribution of Bed Material in the Movable-bed Model

The erosion and deposition patterns in the prototype channel are significantly affected by the shape of the hydrograph. It is therefore required to run a hydrograph through the movable-bed model. The prototype hydrograph was converted to a corresponding model hydrograph using a time scale conversion factor determined by comparing the celerities of bed wave movement in the model and prototype.

The Meyer-Peter, Muller formula was utilized to determine bed load discharge

$$q_b = \frac{8}{\sqrt{\rho}\ (\gamma_s - \gamma)} (\tau - \tau_c)^{1.5} \qquad (16)$$

in which q_b is the unit-width bed load in cfs/ft, and is the bed shear stress in lbs/ft^2. The following power relations were obtained:

1) d = 60 mm (d_{50} in prototype)

$$q_b = 1.17 \times 10^{-6}\ V^{4.64} \qquad (17)$$

2) d = 3 mm (d_{50} in model)

$$q_b = 1.72 \times 10^{-5}\ V^{4.53} \qquad (18)$$

The celerity of bed wave can be derived for the sediment continuity equation as

$$c_t = \frac{1}{1-\lambda} \frac{\partial q_b}{\partial z} \qquad (19)$$

in which λ is the porosity, z is the bed elevation. The bed load discharge q_b can be related to velocity by a power function.

$$q_b = a\ V^b = a\left[\frac{q}{(h-z)}\right]^b \qquad (20)$$

in which q is the unit-width water discharge, and h is the stage. A unit-width channel is assumed for this derivation. Taking a derivative of q_b with respect to z yields:

$$\frac{\partial q_b}{\partial z} = \frac{ab\ V^b}{D} \tag{21}$$

Based on numerical model tests for a unit-width channel, the numerical model celerity c_n and the theoretical celerity c_n have the following relation:

$$c_n = 2.33[(1-\lambda)\ c_t]^{0.96} \tag{22}$$

Substituting Eqs. 19 and 21 into Eq. 22 yields

$$c_n = 2.33\ \left(\frac{ab\ V^b}{D}\right)^{0.96} \tag{23}$$

The time scaling ratio between the prototype and model sediment wave movement is

$$\frac{t_p}{t_m} = \frac{x_p/C_{np}}{x_m/C_{nm}} = \frac{x_p}{x_m}\ \frac{C_{nm}}{C_{np}} \tag{24}$$

in which

$$\frac{C_{nm}}{C_{np}} = \left(\frac{b_m}{b_p}\right)^{0.96} \left(\frac{a_m}{a_p}\right)^{0.96} \left(\frac{V_m^{\ b}}{V_p^{\ b}}\right)^{0.96} \left(\frac{D_p}{D_m}\right)^{0.96} \tag{25}$$

Substituting Eqs. 17 and 18 into Eqs. 24 and 25 to determine the time scaling ratio and using d50 as a representative particle size:

$$\frac{C_{nm}}{C_{np}} = 392 \frac{V_m^{4.35}}{V_p^{4.45}} \tag{26}$$

$$\frac{t_p}{t_m} = 175 \times 392 \frac{V_m^{4.35}}{V_p^{4.45}} \tag{27}$$

A range of prototype velocity from 5 to 20 fps (corresponding to a model velocity range from 0.85 to 3.38 fps) yields a range of time scaling ratio between 22 and 25. A constant time ratio of 24 was selected to convert the prototype hydrograph to the model hydrograph for operating the movable-bed model. A similar time ratio was obtained when the sediment continuity equation was applied to two neighboring reaches upstream of and within the constriction area.

DETERMINATION OF LEVEE SLOPE FOR THE MOVABLE BED MODEL

The prototype levee slope has a vertical to horizontal slope of 1 to 2. By using a 1:5 vertical to horizontal distortion, the resultant model levee slope was increased to 1 to 0.4 for the fixed-bed model. It is necessary to reduce this slope in the movable-bed model to obtain required levee stability, which introduces an additional degree of distortion. An attempt was made to determine a model levee slope to achieve comparable levee stability in the model using tractive force theory and critical moment theory. However, it is not known how these changes affect velocity, stage, erosion and deposition patterns. Further research is required in this field.

The tractive force theory states that the motion of a particle is pending when the force tending to cause its motion τ_m equals its resistance to motion τ_s. Based on the Bureau of Reclamation test for coarse noncohesive material with sufficient factor of safety

$$\tau_L = 0.4\ d_{75} \tag{28}$$

in which τ_L is the permissible tractive force on a level bottom and d75 is in inches. The permissible tractive force on a level bottom and d75 is in inches. The permissible tractive force on the side slope is

$$\tau_s = K\ \tau_L \tag{29}$$

in which K is the tractive force ratio defined as

$$K = \sqrt{1 - \frac{\sin^2\phi}{\sin^2\theta}} \tag{30}$$

where ϕ is the angle of side slope and θ is the angle of repose of a particle.

In the prototype, the d75 is 160mm, which has an angle of repose equal to 42 degrees. The angle of the 1 to 2 side slope is 26.5 degrees.

Therefore from Eqs. 28, 29 and 30

$$\tau_L = 0.4 \times \frac{160}{25.4} = 2.52\ \text{lb/ft}^2 \tag{31}$$

$$K = 0.745 \tag{32}$$

$$\tau_s = K\tau_L = 1.88\ \text{lb/ft}^2 \tag{33}$$

The maximum tractive force on the sloping sides is

$$\tau_m \simeq 0.775\ \tau_o = 0.775\ (\tfrac{1}{8} f\rho\ V^2) = 0.0150\ V^2 \tag{34}$$

For a state of impending motion of a particle on side slope

$$\tau_s = \tau_m \tag{35}$$

Which results in

$$V_{cp} = 11.2 \text{ fps} \tag{36}$$

The corresponding critical velocity which initiates particle motion in the model is

$$V_{cm} = 1.89 \text{ fps} \tag{37}$$

The shear stress corresponding to this velocity on the side slope is

$$\tau_{cm} = 0.775 \quad \tau_{Lm} = 0.0537 \tag{38}$$

The permissible tractive force on the side slope of model for (d75)m= 4.5 mm is

$$\tau_{sm} = K \, \tau_{Lm} = K(0.4 \times \frac{4.5}{25.4}) = 0.071 \text{ K} \tag{39}$$

For a state of impending motion of a particle on side slope in the model

$$\tau_{cm} = \tau_{sm} \tag{40}$$

Which results in

$$K= 0.756$$

From Eq. 30,

$$\sin\phi = \sin\theta \sqrt{1-K^2} \tag{41}$$

For a particle of 4.5 mm the angle of repose is about 38 degrees. Substituting this angle into Eq. 41 obtains degrees. This corresponds to a side slope of 1 to 2.2.

This result indicates that if the levee slope in the model is installed at a slope of 1 to 2.2, its stability would be comparable to that in the prototype. Another approach based on Simons and Stevens' critical moment theory (Refer to "Highways in the River Environment Hydraulic and Environmental Design Considerations, Training and Design Manual," by E. V. Richardson, D. B. Simons, S. Karaki, K. Mahmood and M. A. Stevens, Civil Engineering Department, Colorado State University, 1975) provides a similar result. A slope of 1 to 2 was selected for constructing the levee in the model in order to minimize distortion effects.

DESCRIPTION OF RIVER MECHANICS FLUME

The River Mechanics Flume shown in Fig. V-2 is situated next to the 600 acre-feet College Lake, south of Colorado State University Engineering Research Center. It is 39.4 feet wide, 120 feet long (with 109.7 ft usable for channel modeling) and 3 feet high. The building also contains a valve house, a 19.5 ft by 16.5 ft pump house which has a 50 hp recirculating pump installed in it, and a small office area. The water supply to the flume is from College Lake through a 350 hp turbine pump and a 21-inch pipeline. A culvert returns the water pumped into the flume back to College Lake. A secondary circulating system has been implemented for movable bed studies to recirculate sediment. The sediment transported through the movable bed models during experiments is collected by a sediment trap located at the end of the flume and the water-sediment mixture is recirculated by the 50 hp pump and 15 1/4-inch pipeline back into the flume.

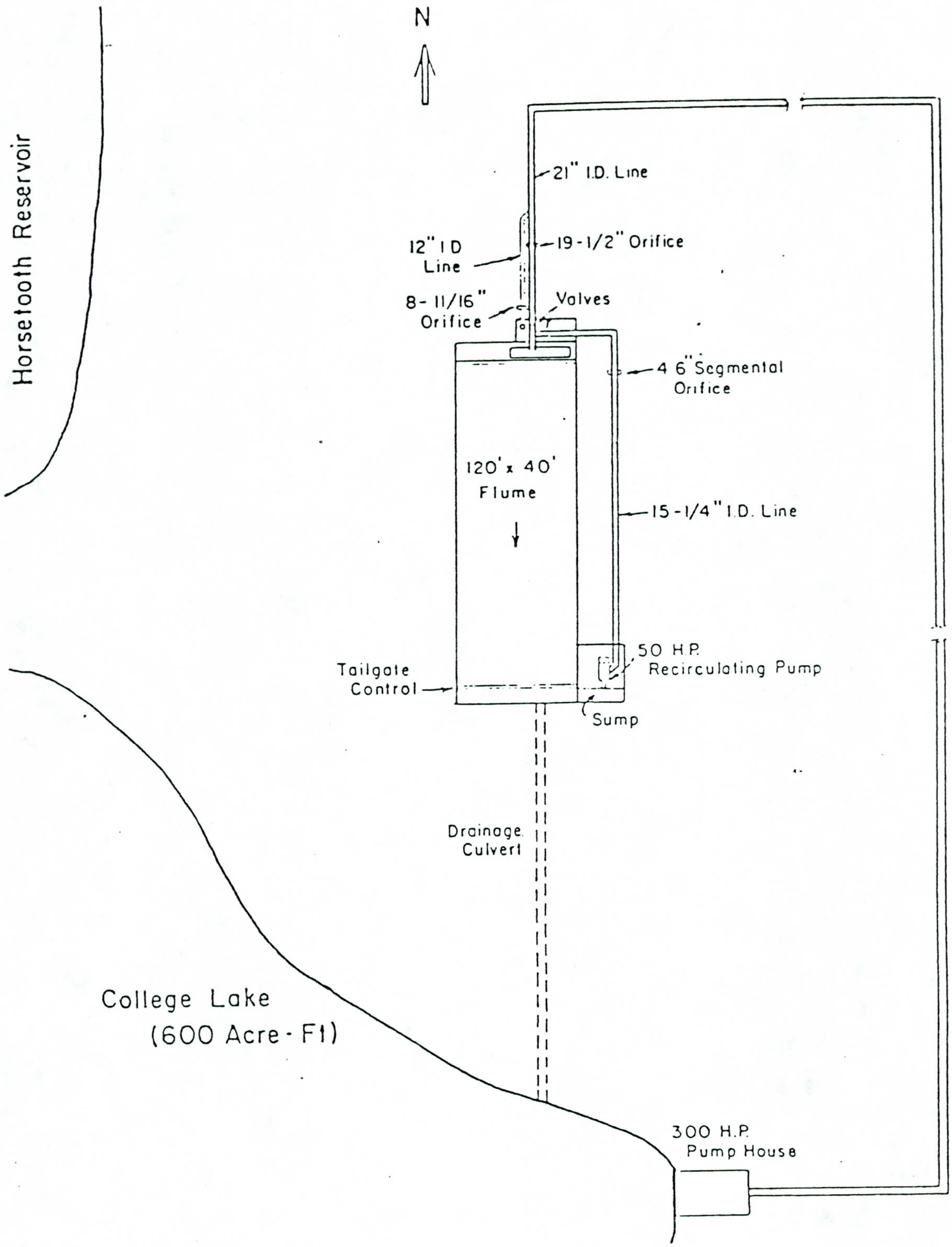


Fig.V-2 General Layout of the River Mechanics Flume Testing System, Branch Hydraulics Laboratory

The maximum discharge capacity of the main recirculating system is 35 to 40 cfs depending on the fluctuations in College Lake water levels. The secondary system can provide an additional 10 cfs by recirculating some of that discharge whenever higher discharges are needed. With proper piping connections to Horsetooth Reservoir, the system can generate water discharges up to 100 cfs if necessary.

For low discharges, to insure proper discharge measurements by having full-pipe flows, a 12-inch by-pass line to the 21-inch pipeline has been implemented. Flows through different pipelines are controlled by three butterfly valves placed at their ends. Discharge measurements are carried out through the use of three orifice plates placed in each one of the pipelines. A differential manometer attached to the pipelines on both side of the orifice plates measures the head drops across them to a tenth of an inch. The orifice placed in the secondary circuit is selected to be a segmental-type orifice to insure proper operation with a water-sediment mixture passing through it. A more detailed plan view of the flume is given in Fig. V-3. A manifold and a porous vertical wall diffuser at the outlet where the water from the pipelines is discharged, is used to introduce a less turbulent flow into the flume. Flow distribution adjustments across the channel can be made by the series of adjustable-height weirs at this location. The surface turbulence at the outlet is suppressed by a series of dissipators manufactured out of wood in the shape of a grill. The water surface elevation adjustments are carried out by the series of stop logs at the tailwater control.

A motorized, 34 ft wide carriage that can move on the railings along the flume is used for carrying the measuring instruments. An overhanging stand mounted on the carriage

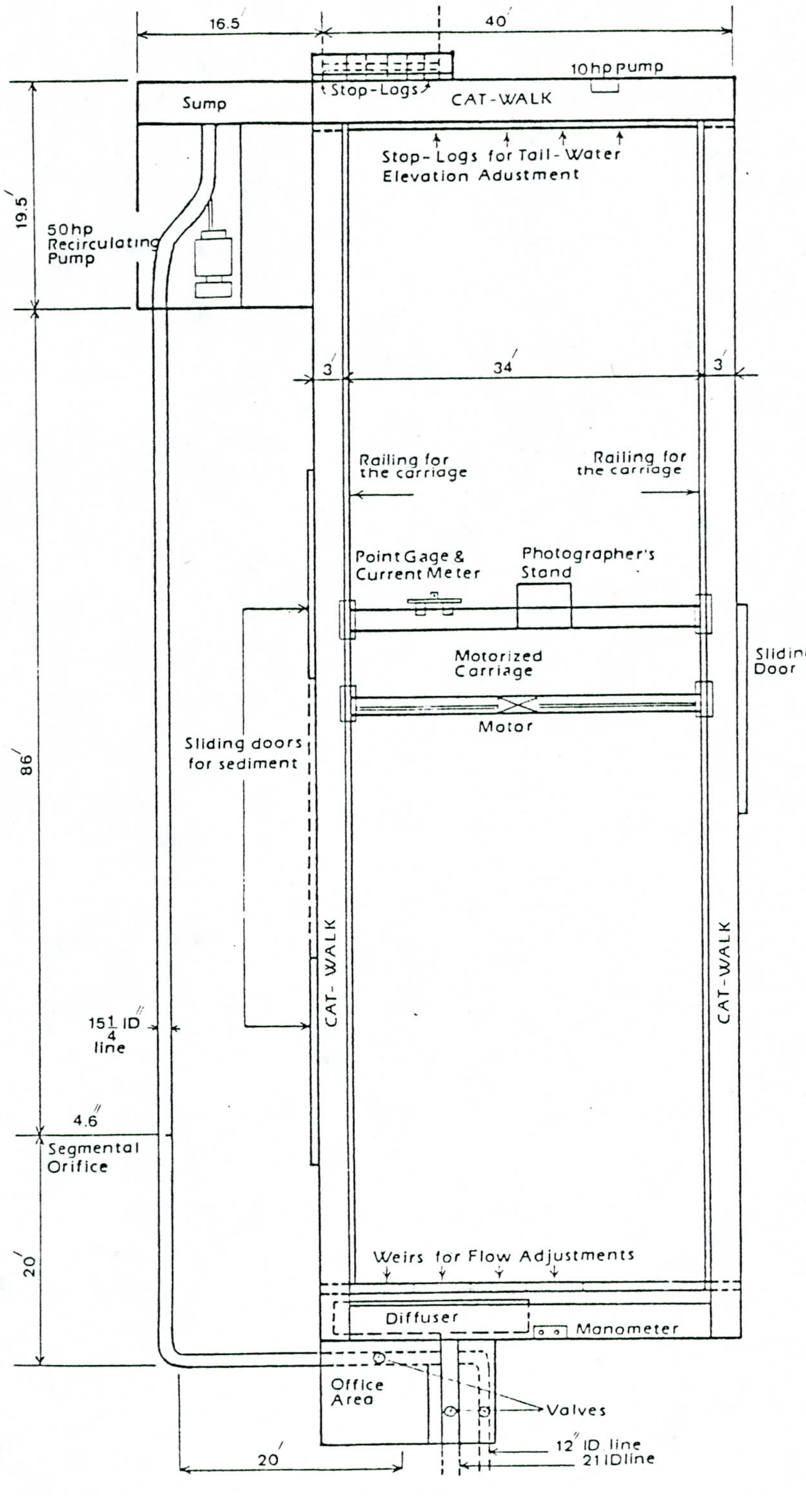


Fig. V-3 Detailed Plan View of the River Mechanics Flume can be used for flow visualization studies. Sliding doors placed at the sides of the building are accessible to trucks and heavy construction equipment for quick loading of the desired sediment or other supplies into the flume whenever needed.

MODELING OF STUDY AREA

The Salt River near Sky Harbor International Airport in Phoenix, Arizona was modeled in the River Mechanics Flume. By using model-to-prototype horizontal and vertical length scaling ratios of 1 to 175 and 1 to 35 respectively, the river reach starting from above Priest Road to Interstate I-10 (a length of about 3.7 miles) was modeled. The corresponding model coordinates and the prototype centerline lengths are shown in Fig. V-4 and listed in Table V-2. Table V-2 also lists the model elevation correction to account for the slope change in the model.

Construction of the fixed-bed model proceeded as follows:

1. Convert the prototype cross-sectional geometries (provided by HNTB) to the model geometries.
2. Construct the levee lines following the plan provided by HNTB using bricks to about the desired top elevation and apply a cement layer on the bricks to form a 1:0.4 levee slope.
3. Locate the model coordinates of the selected cross sections and lay out the cross-sectional elevations up to about 1 to 2 inches below the desired elevations using the 1/4-inch chips. Bricks were used to stabilize the low flow channel.
4. Cover the chip surface using 1 1/2-inch gravel to the desired elevations. .

A completed fixed bed model is illustrated in Fig. V-5.

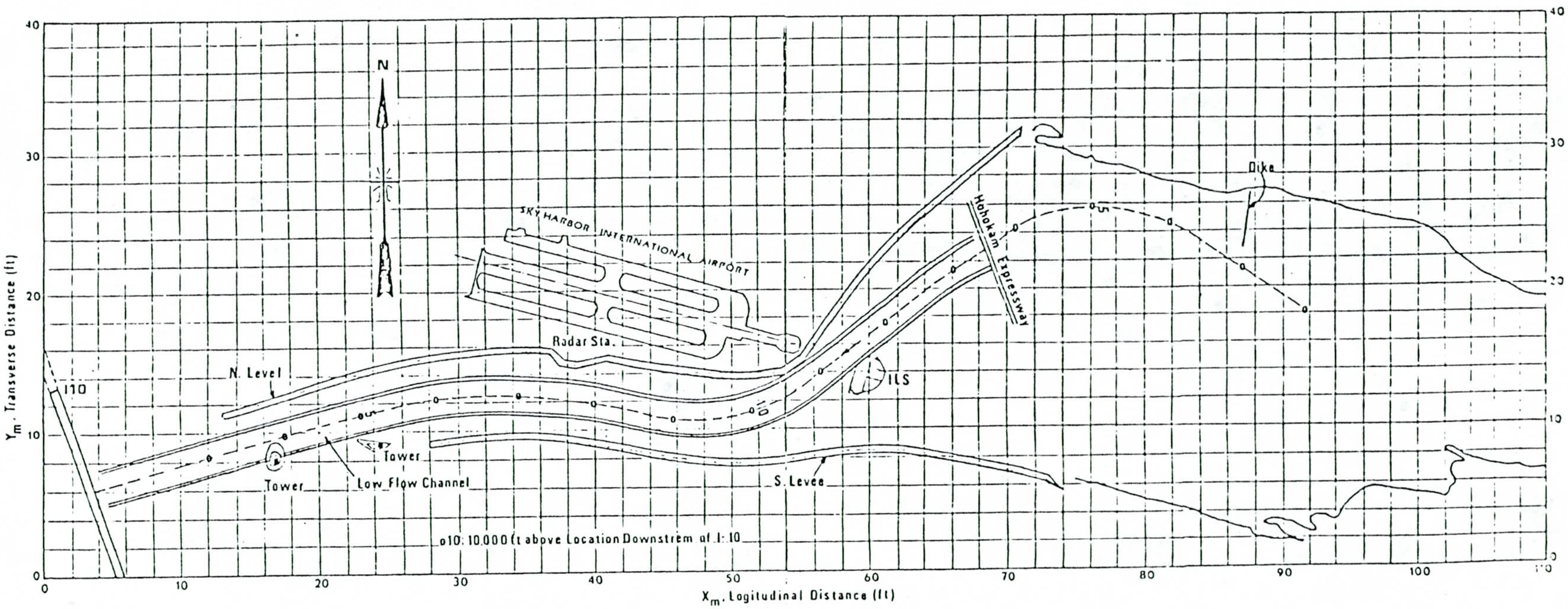


Fig. V-4   Model and Prototype Length Coordinates

TABLE V-2 Prototype and Model Coordinate Relations

Station	L_p (ft)	ΔZ_m (ft)	Station	L_p (ft)	ΔZ_m (ft)	Station	L_p (ft)	ΔZ_m (ft)
6	1,960	-0.597	37	7,485	-0.383	66	13,040	-0.181
8	2,330	-0.589	38	7,650	-0.376	67	13,255	-0.174
9	2,505	-0.582	39	7,825	-0.370	68	13,470	-0.167
10	2,680	-0.575	40	8,000	-0.363	69	13,685	-0.160
11	2,855	-0.568	41	8,170	-0.356	70	13,900	-0.153
12	3,030	-0.561	42	8,340	-0.349	71	14,085	-0.146
13	3,210	-0.554	43	8,510	-0.342	72	14,270	-0.138
14	3,390	-0.547	44	8,680	-0.335	73	14,450	-0.131
15	3,565	-0.540	45	8,855	-0.328	74	14,630	-0.123
16	3,740	-0.533	46	9,030	-0.321	75	14,805	-0.116
17	3,920	-0.526	47	9,205	-0.314	76	14,980	-0.108
18	4,100	-0.519	48	9,380	-0.307	77	15,155	-0.101
19	4,275	-0.512	49	9,560	-0.300	78	15,330	-0.093
20	4,450	-0.504	50	9,740	-0.293	79	15,515	-0.091
21	4,630	-0.497	51	9,925	-0.286	80	15,700	-0.088
22	4,810	-0.490	52	10,110	-0.279	81	15,875	-0.076
23	4,990	-0.483	53	10,300	-0.272	82	16,050	-0.063
24	5,170	-0.475	54	10,490	-0.265	83	16,235	-0.056
25	5,350	-0.468	55	10,695	-0.258	84	16,420	-0.048
26	5,530	-0.461	56	10,900	-0.251	85	16,625	-0.041
27	5,710	-0.454	57	11,105	-0.244	86	16,830	-0.033
28	5,890	-0.446	58	11,310	-0.237	87	17,035	-0.026
29	6,070	-0.439	59	11,505	-0.230	88	17,240	-0.018
30	6,250	-0.432	60	11,700	-0.223	89	17,460	-0.011
31	6,425	-0.425	61	11,930	-0.216	90	17,680	-0.004
32	6,600	-0.418	62	12,160	-0.209	91	17,900	-0.003
33	6,775	-0.411	63	12,380	-0.202	92	18,120	-0.010
34	6,950	-0.404	64	12,600	-0.195	93	18,340	0.018
35	7,135	-0.397	65	12,820	-0.188	94	18,560	0.025
36	7,320	-0.390						

L_p = Prototype distance above station 1,000 ft downstream of I-10

ΔZ_m = Model elevation correction due to slope change

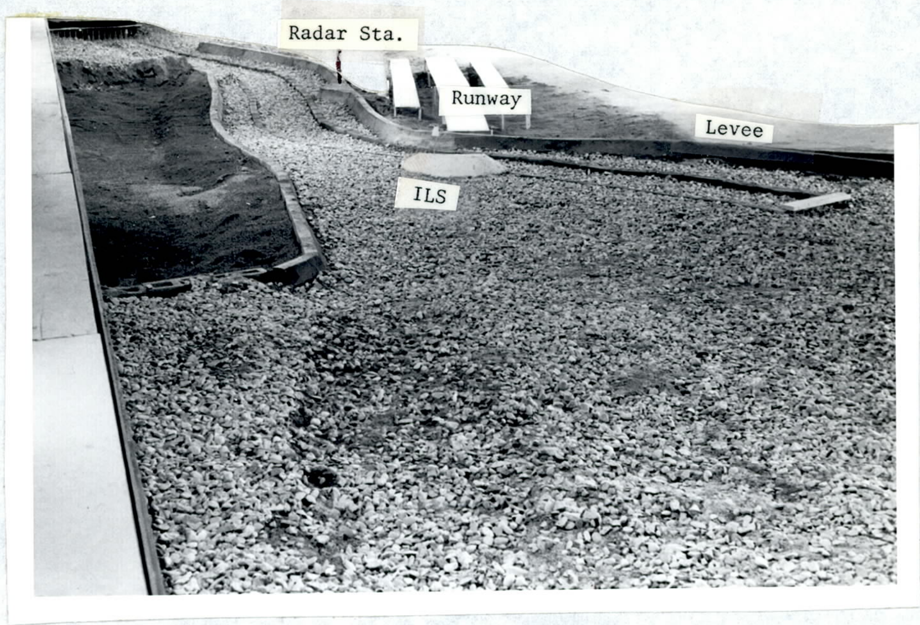
Three discharges, 92,000 cfs, 176,000 cfs and 210,000 cfs, were tested in the fixed-bed model. The corresponding model discharge and stage conditions are given in Table V-3. The model stages at the control station were determined based on the prototype stages at about 50 ft upstream of I-10.

After the completion of the fixed-bed model tests, the gravel and bricks were removed. The 1/4-inch chips were used to fill these areas and to form the levee. The model levee slope was molded to a 1:2 slope and sections of the levees were protected by 1 1/2-inch gravel according to the design drawings.

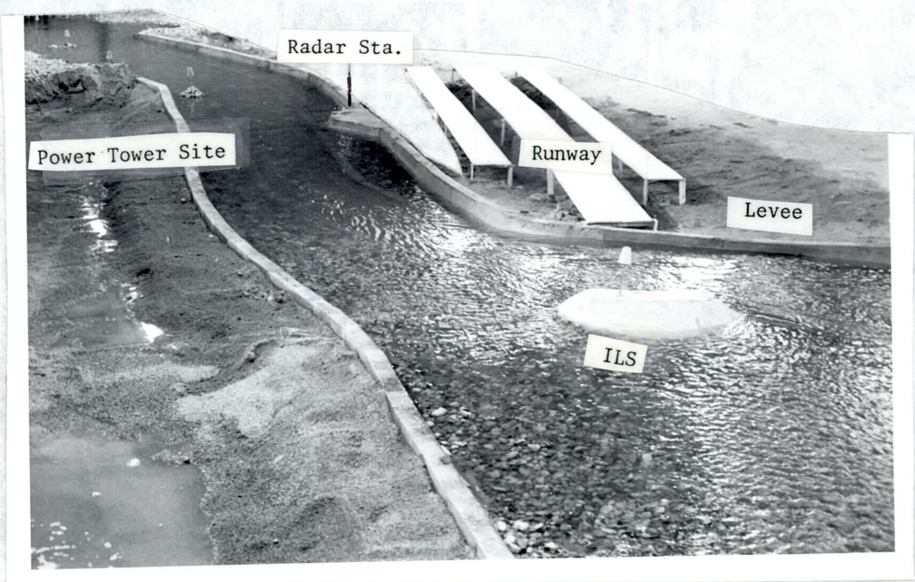
TABLE V-3 Model Test Conditions

Prototype		Model	
Discharge (cfs) Q_p	Stage (ft, msl) h_p	Discharge (cfs) Q_m	Stage (ft) h_m*
92,000	1,094.8	2.54	1.61
176,000	1,101.1	4.86	1.79
210,000	1,102.8	5.80	1.84

* $h_m = (h_p - 1,060)/35 + \Delta Z_m$



(a) Dry Channel



(b) Q = 92,000 cfs

Fig.V-5 Photographs Showing the Fixed-Bed Model

INSTRUMENTATION

The major variables measured during the model tests include water discharge, velocity, and water surface and bed surface elevation. Stop logs are used to control stages at the control station immediately upstream of I-10.

The water discharge is measured using a 8 11/16-inch orifice installed in the 12-inch bypass line and calibrated using the Hydraulics Laboratory Pipe flow calibration system. The calibration relation is

$$Q = 2.377 \ (\Delta h)^{0.49} \tag{42}$$

in which Q is the discharge in cfs and is the head loss across the orifice in feet.

The velocity is measured using a 1-inch ott meter. The rating relation is

$$v = 0.3395 \ n + 0.125 \tag{43}$$

in which n is the number of revolutions per second, and v is the point-velocity in fps. Flow direction is also measured by tying a string to the tip of a point gage to indicate direction and measuring the angle of flow with respect to the carriage baseline. The water surface and bed surface elevations are measured using a point gage and periodically checked using a transit.

CONSTRUCTION OF THE MOVABLE-BED MODEL

In the fixed-bed model, the 1 1/2 inch gravel was utilized to cover the 1/4-inch chips to form the fixed bed. After the completion of the fixed-bed model tests, the gravel layer and brick levees were removed. The 1/4-inch chips (see Fig.V-1 for the size distribution) were used to fill these areas and to form the levee. Sections of levees were protected by 1 1/2-inch gravel according to HNTB's plan.

Because of the changes in bank and levee slopes from 1:0.4 to 1:2, some distortions in cross sections are required. Figure V-6 shows the changes in the shape of a typical cross section and in the shape of a cross section adjacent to Stations 55 and 60. As shown, the changes in cross-sectional shapes are usually relatively minor. Figures V-7 and V-8 show photographs of the completed movable-bed model. For computing the changes in bed elevations in the movable-bed model, the initial bed elevations of the model are given in Fig. V-9. To obtain the true bed elevations, the numbers in Fig. V-9 should be increased by 1,100 ft when the number is smaller than 30 and should be increased by 1,000 ft when it is larger than 80.

OPERATION OF THE MOVABLE-BED MODEL

A normalized hydrograph was obtained by averaging nondimensionalized versions of the 50 year, 100 year, and Standard Project Flood hydrographs developed by the Los Angeles District of the U.S. Army Corps. of Engineers. This hydrograph, as shown in Figure V-10, was then multiplied by the peak discharge values to obtain 92,000 cfs and 210,000 cfs event hydrographs.

To run these flood events through the movable-bed model, the hydrographs were approximated by step hydrographs as shown in Fig. V-10. Utilizing a model-to-prototype time scaling ratio of 1 to 24 as discussed in Section 2.4 and neglecting

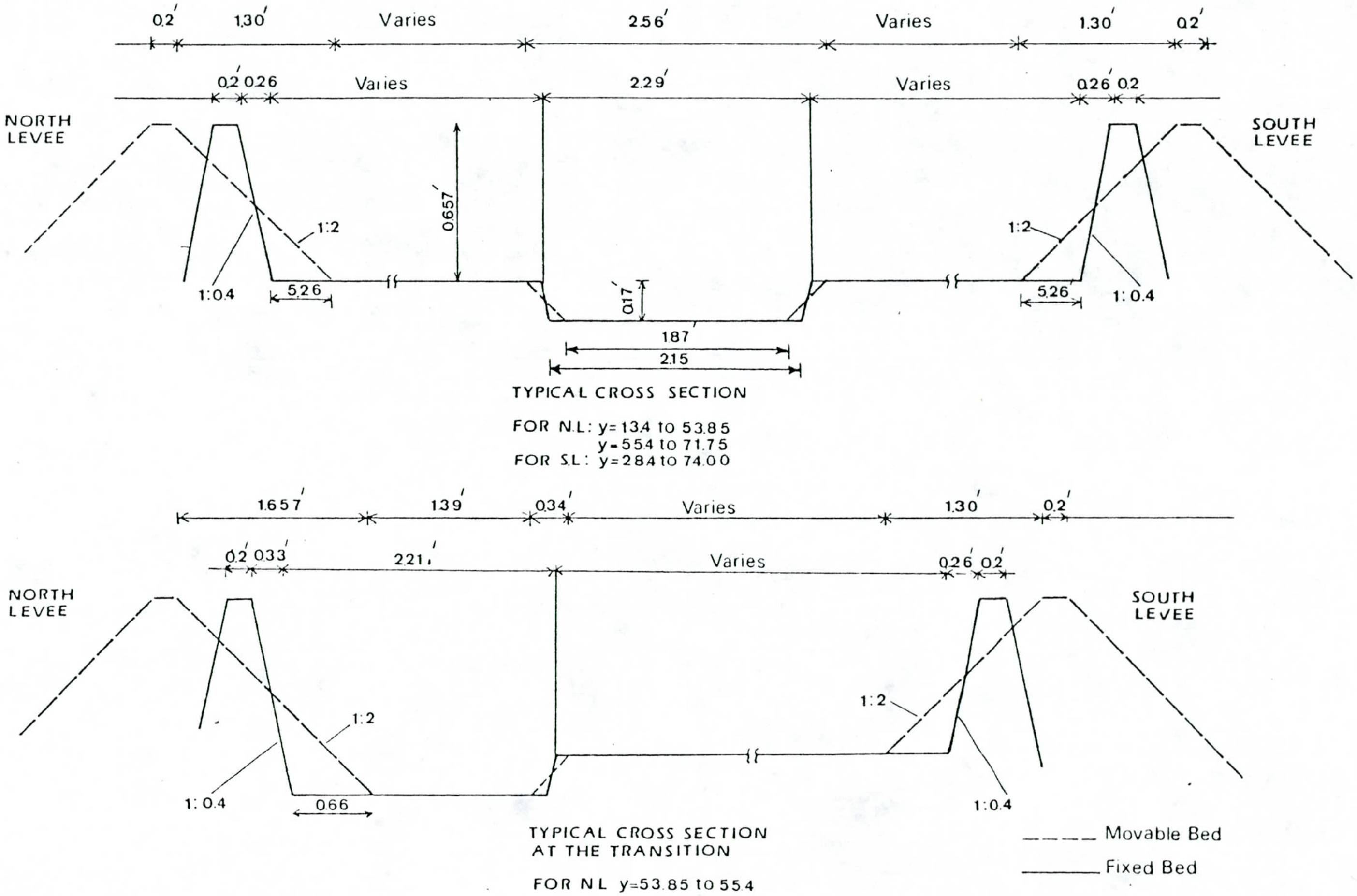


Fig. V-6 Modified Cross-Sectional Shapes in the Movable-Bed Model

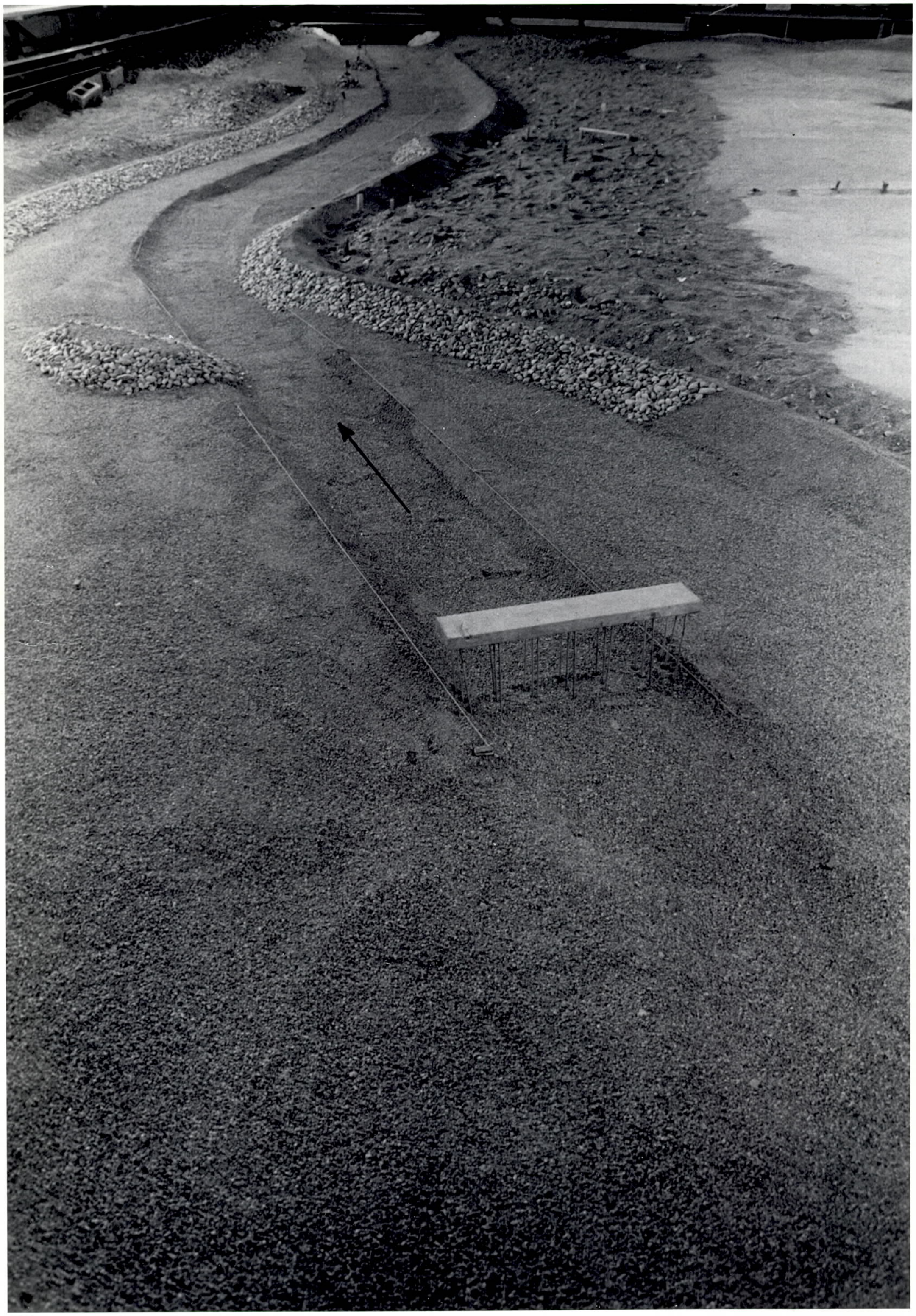
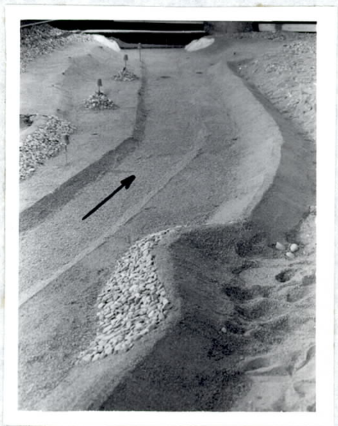


Fig. V-7 Photo of the Movable-Bed Model

(a) Looking towards the radar station from the ILS

(b) Looking upstream from I-10

(c) Looking upstream towards the ILS



(d) Looking downstream from the radar station

Fig.V-8 Various Views of the Movable-Bed Model

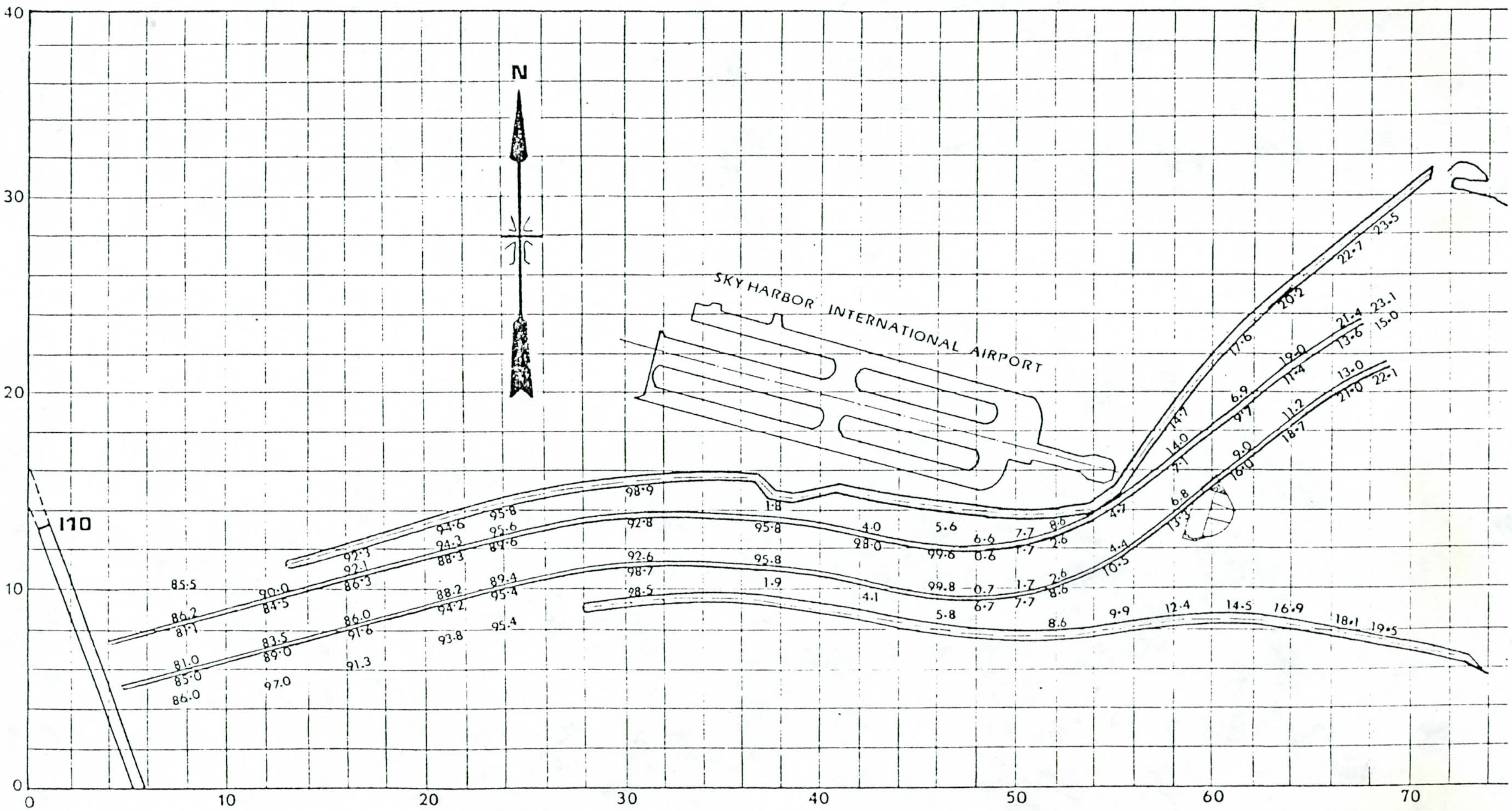


Fig. V-9 The Initial Bed Elevations in the Movable-Bed Model

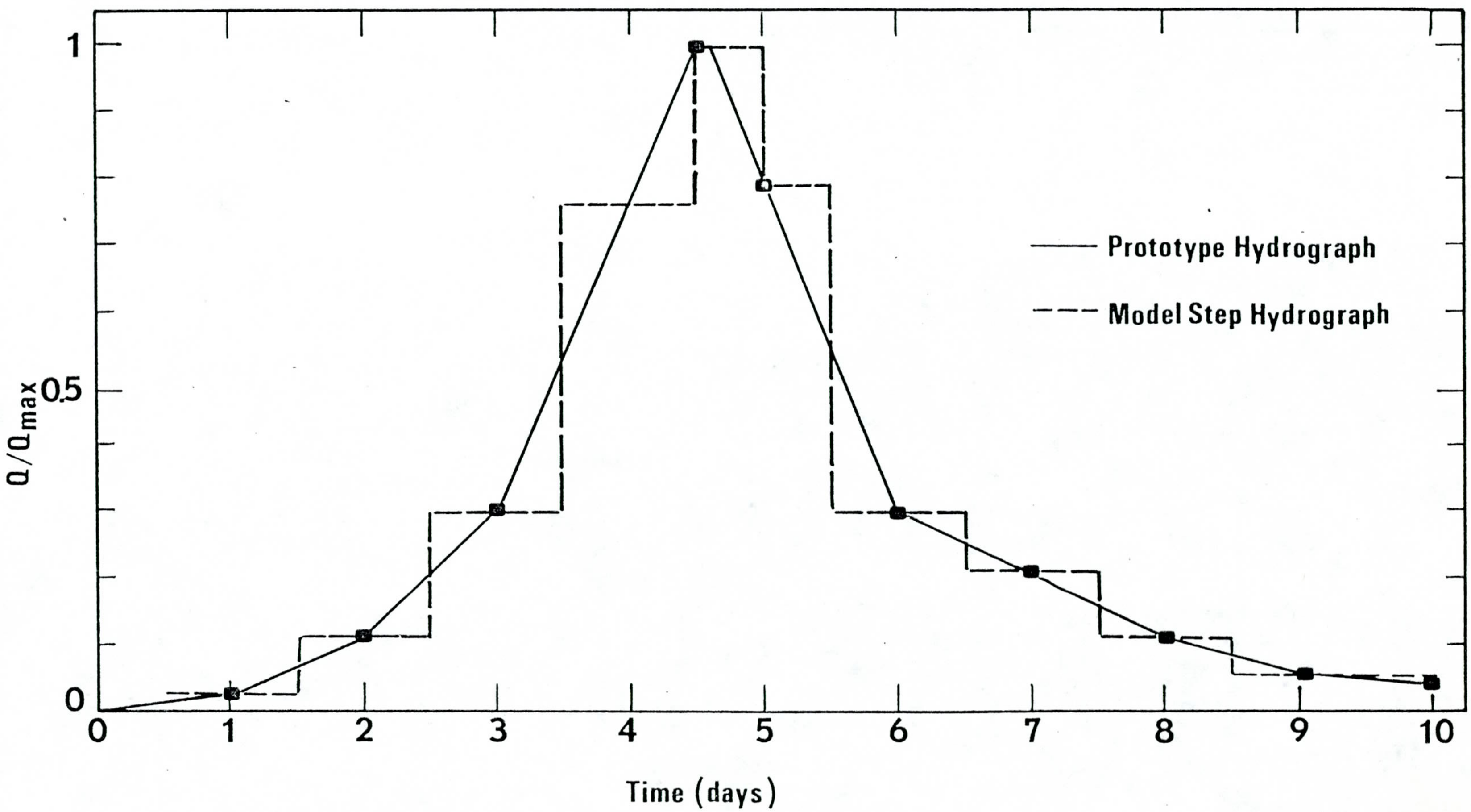


Fig. V-10 Standardized Discharge Hydrograph smaller discharge steps during which sediment discharges would be small, the model step discharges and time durations were determined. These hydrograph steps and corresponding water surface elevations immediately upstream of I-10 are listed in Table V-4. The water surface elevations were obtained based on the rating curve utilized by SLA and in the HEC-2 runs.

The model operating procedure and the data collection program involved:

1. Routing the 92,000 cfs hydrograph through the model.
2. During the run, velocities at the ILS and at model coordinates 46, 38.5, and 19 were collected when there was sufficient time. In addition, stability of levees were observed and protection of levees were provided wherever it was necessary.
3. After routing the first 92,000-cfs hydrograph, the bed elevations were measured in detail.
4. Without reforming the bed profile the 92,000 cfs hydrograph was routed through the model for the second time. The same data described in Steps 2 and 3 were collected to study cumulative effects of subsequent floods.
5. The bed profile was then re-molded to the initial condition. A 210,000 cfs hydrograph was routed through the model.
6. Velocities were measured at the stations given in Step 2.
7. At the end of the rising limb, flow was stopped and the flume was drained to measure the bed elevations.
8. The bed elevations were measured again after routing the entire 210,000 cfs hydrograph.

TABLE V-4 Step Discharges, Durations and Stages in the Movable-Bed Model

Prototype			Model		
Time (day)	Discharge (cfs)	Stage (ft, msl)	Time	Discharge (cfs)	Stage (ft)
I. 92,000 cfs Hydrograph					
2-3	27,600	1,088.2	0-1	0.76	1.43
3-4	69,900	1,093.4	1-2	1.93	1.57
4-4.5	92,000	1,095.2	2.2.5	2.54	1.63
4.5-5	72,700	1,093.6	2.5-3	2.01	1.58
5-6	27,600	1,088.2	3-4	0.76	1.43
II. 210,000 cfs Hydrograph					
1-2	23,100	1,087.3	0.1	0.64	1.40
2-3	63,000	1,092.7	1-2	1.74	1.55
3-4	160,000	1,100.0	2-3	4.42	1.76
4-4.5	210,000	1,102.8	3.3.5	5.80	1.84
4.5-5	166,000	1,100.4	3.5-4	4.58	1.77
5-6	63,000	1,092.7	4-5	1.74	1.55
6-7	44,100	1,090.6	5-6	1.21	1.49
7-8	23,100	1,087.4	6-7	0.64	1.40

Filling and draining of water were done at a slow rate during the initiation and completion of model runs to minimize artificial scour of the model bed.

VI. FIXED-BED PHYSICAL MODEL RESULTS

STAGES OF FIXED-BED MODEL TESTS

The physical model tests show significant stage variations in the transverse direction because of nonuniform flow patterns and channel alignment. Tables VI-1, 2, and 3 show the average stages and the stages near the south and north levees at different stations for Qp= 92,000 cfs, 176,000 cfs, and 210,000 cfs respectively. The maximum stage differences across individual cross sections generally occur immediately downstream of the transition near Station 52 where water impinges on the south levee to raise the stage. This stage difference increases with discharge, ranging from about 1.1 ft for 92,000 cfs, to about 2.3 ft for 176,000 cfs and 210,000 cfs. Very little difference between the 176,000 cfs and 210,000 cfs events is apparent. The stage variations affect the freeboard of levees as will be discussed in later sections.

PREDICTED FREEBOARDS

Based on the predicted water surface elevations, freeboards in the channel vary from 1.2 ft to 10.7 ft., as shown in Table VI-4. Freeboards vary significantly between the north and south levees, due to superelevation of the water surface. Note also that the station numbers in the first column represent model coordinates, while the approximate station numbers in the second column are channel centerline stations relative to the grade control structures downstream of the I-10 bridge. Since model measurement traverses are not always perpendicular to the channel centerline, these stations are not directly comparable to design cross sections or numerical stations.

TABLE VI-1 Water Surface Elevations in the Fixed-Bed Physical Model, Q = 92,000 cfs

Station	L_p (ft)	Stage (ft)		
		Average	N. Levee	S. Levee
8	2,330	1,096.1	-	-
13	3,210	1,097.2	1,096.9	-
19	4,275	1,099.1	1,098.6	-
21.2	4,650	1,100.1	1,100.9	-
28.5	6,030	1,103.9	1,104.3	1,103.9
31.4	6,500	1,104.8	1,105.0	1,104.7
34.0	6,950	1,105.4	1,105.6	1,105.5
37.1	7,420	1,106.5	1,106.4	1,106.3
38.3	7,703	1,106.8	1,105.4	1,107.3
41	8,170	1,110.0	1,109.8	1,109.9
46.9	9,200	1,112.1	1,111.8	1,112.6
54	10,420	1,114.7	1,113.9	1,115.0
60	11,700	1,118.8	1,119.5	1,118.6
70	13,900	1,123.8	1,124.5	1,123.6
82	16,050	1,134.4	-	-

TABLE VI-2 Water Surface Elevations in the Fixed-Bed Physical Model, Q = 176,000 cfs

Station	L_p (ft)	Stage (ft)		
		Average	N. Levee	S. Levee
8.2	2,365	1,101.8	-	-
12	3,030	1,104.0	-	-
21	4,630	1,105.5	1,105.4	-
26	5,530	1,108.6	1,108.5	-
30	6,250	1,109.8	1,109.8	1,109.6
35	7,135	1,111.0	1,110.4	1,111.4
37	7,485	1,111.6	1,111.1	1,111.7
38.5	7,738	1,111.9	1,111.3	1,112.0
42	8,340	1,114.9	1,114.8	1,114.7
46	9,030	1,116.7	1,115.6	1,117.5
51	9,925	1,118.9	1,117.1	1,119.4
56.1	10,921	1,120.7	1,120.4	1,120.6
60	11,700	1,123.3	1,124.0	1,123.4
70.2	13,937	1,127.6	1,127.8	1,127.2
80.1	15,718	1,135.2	-	-
87	17,035	1,138.1	-	-

TABLE VI-3 Water Surface Elevations in the Fixed-Bed Physical Model, Q = 210,000 cfs

Station	L_p (ft)	Stage (ft)		
		Average	N. Levee	S. Levee
10	2,680	1,097.6	-	-
13	3,210	1,104.1	1,103.8	-
18	4,100	1,105.9	1,105.5	-
22	4,810	1,107.3	1,108.5	-
28.3	5,940	1,110.9	1,111.1	1,110.9
34	6,950	1,112.1	1,112.1	1,111.9
37	7,485	1,113.0	1,112.5	1,113.2
38.5	7,730	1,112.6	1,111.1	1,112.6
41	8,170	1,116.3	1,116.2	1,116.4
46.2	9,080	1,118.8	1,117.8	1,119.5
50.2	9,780	1,120.1	1,118.7	1,120.9
55	10,690	1,121.8	1,120.7	1,122.2
56.7	11,020	1,122.9	1,123.8	1,123.0
58.1	11,310	1,124.9	1,126.0	1,124.6
60	11,700	1,125.8	1,127.1	1,126.0
61	11,940	1,127.4	1,127.4	1,127.0
65	12,820	1,129.5	1,130.0	1,129.5
71	14,080	1,130.6	1,131.1	1,130.6
80	15,700	1,136.6	-	-
92	18,120	1,140.5	-	-

TABLE VI-4 Freeboard for Q = 210,000 cfs

Station	Approx. Station (ft)	North Levee (ft)		South Levee (ft)	
		Levee Height	Freeboard	Levee Height	Freeboard
13	3,210	1,114.5	10.7	-	-
18	4,100	1,115.4	9.9	-	-
22	4,810	1,116.2	7.7	-	-
28.3	5,940	1,117.2	6.1	1,117.0	6.1
34	6,950	1,118.7	6.6	1,119.0	7.1
37	7,485	1,120.7	8.2	1,120.9	7.7
38.5	7,730	1,121.1	10.0	1,121.4	8.8
41	8,170	1,121.8	5.6	1,122.2	5.8
46.2	9,080	1,123.3	5.5	1,123.5	4.0
50.2	9,780	1,124.6	5.9	1,124.6	3.7
55	10,690	1,126.4	5.7	1,125.7	3.5
56.7	11,020	1,127.7	3.9	1,126.7	3.7
58.1	11,310	1,129.7	3.7	1,127.7	3.1
60	11,700	1,130.5	3.4	1,128.9	2.9
61	11,940	1,131.0	3.6	1,129.6	2.6
65	12,820	1,132.8	2.8	1,130.7	1.2
71	14,080	1,134.8	3.7	1,132.8	2.2

VELOCITY DISTRIBUTIONS IN THE FIXED-BED PHYSICAL MODEL

Velocity magnitudes and directions were measured during the model runs for steady-state prototype discharges of 92,000 cfs, 176,000 cfs, and 210,000 cfs. Since no variations in channel geometry can occur in the fixed-bed model, no variations in stages or velocities occur for a fixed discharge. Initial test measurements indicated that velocities taken at a depth equal to 40% of the total depth were generally equal to an average of the vertical velocity profile. Consequently, velocities were measured at that depth only. (This proceedure is often used in physical model studies.) Figures VI-1,2, and 3 show the velocities for the three discharges. The associated arrows indicate the approximate flow direction, while the decimal points indicate the point of measurement. Again, very little difference between the 176,000 cfs and 210,000 cfs events iobserved.

Velocity varies substantially with both longitudinal and transverse position, indicating strongly multi-dimensional model behavior. Maximum velocities occur 1) between the ILS and the north levee, 2) at the contraction caused by the radar station, and 3) along the downstream portion of the north levee. Velocities in excess of 15 ft/s occur over more than half of the proposed channel length for the 92,000 cfs discharge and reach a peak of 16.9 ft/s opposite the radar station. At a discharge of 210,000 cfs, velocities exceed 15 ft/s over 75% of the proposed channel length and reach a maximum of 20.6 ft/s.

Velocity variations across the channel are quite similar for the three discharges and indicate a very substantial

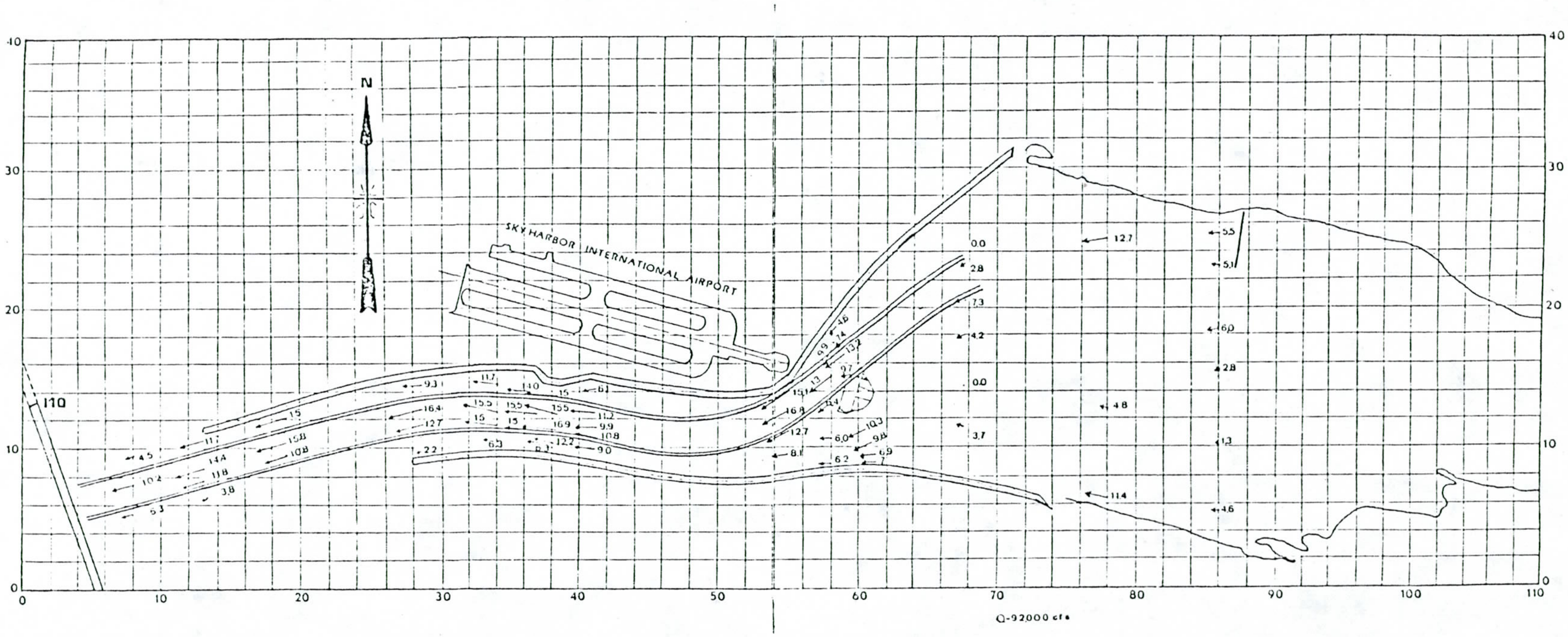


Fig. VI-1 Velocity Distributions in the Fixed-bed Model at 92,000 cfs

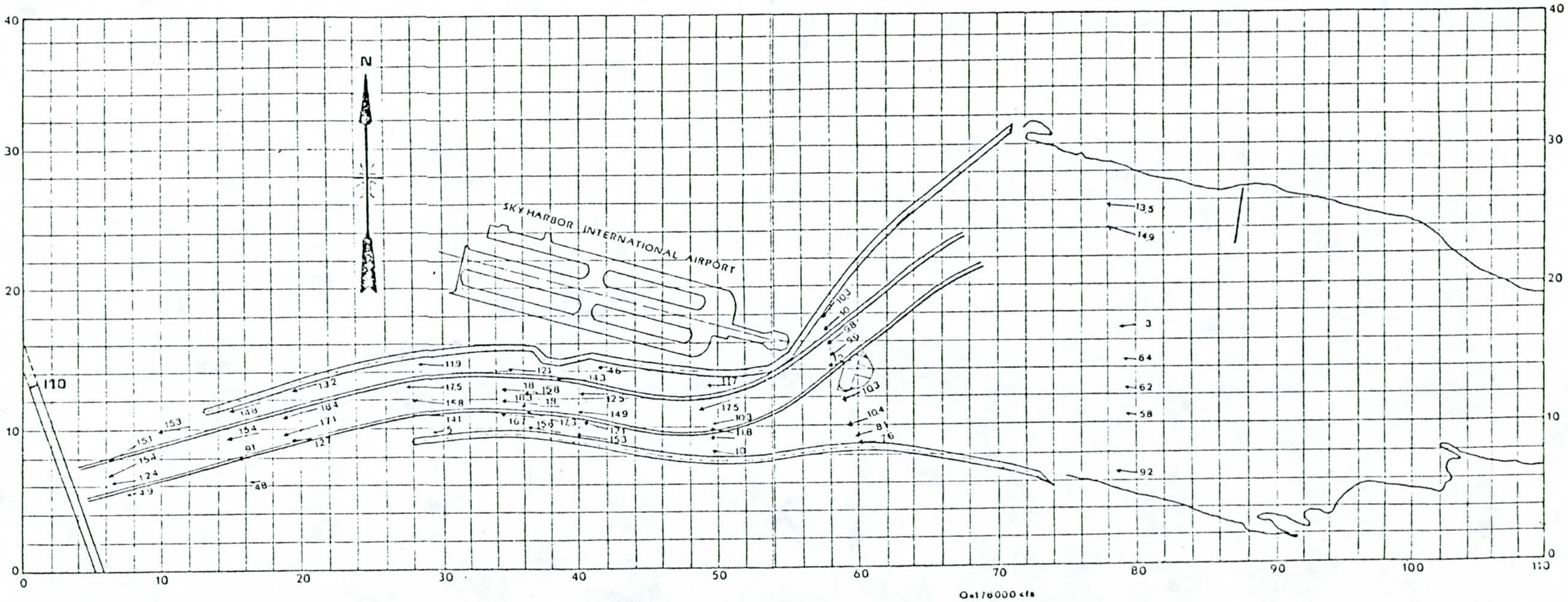


Fig. VI-2 Velocity Distributions in the Fixed-bed Model at 176.000 cfs

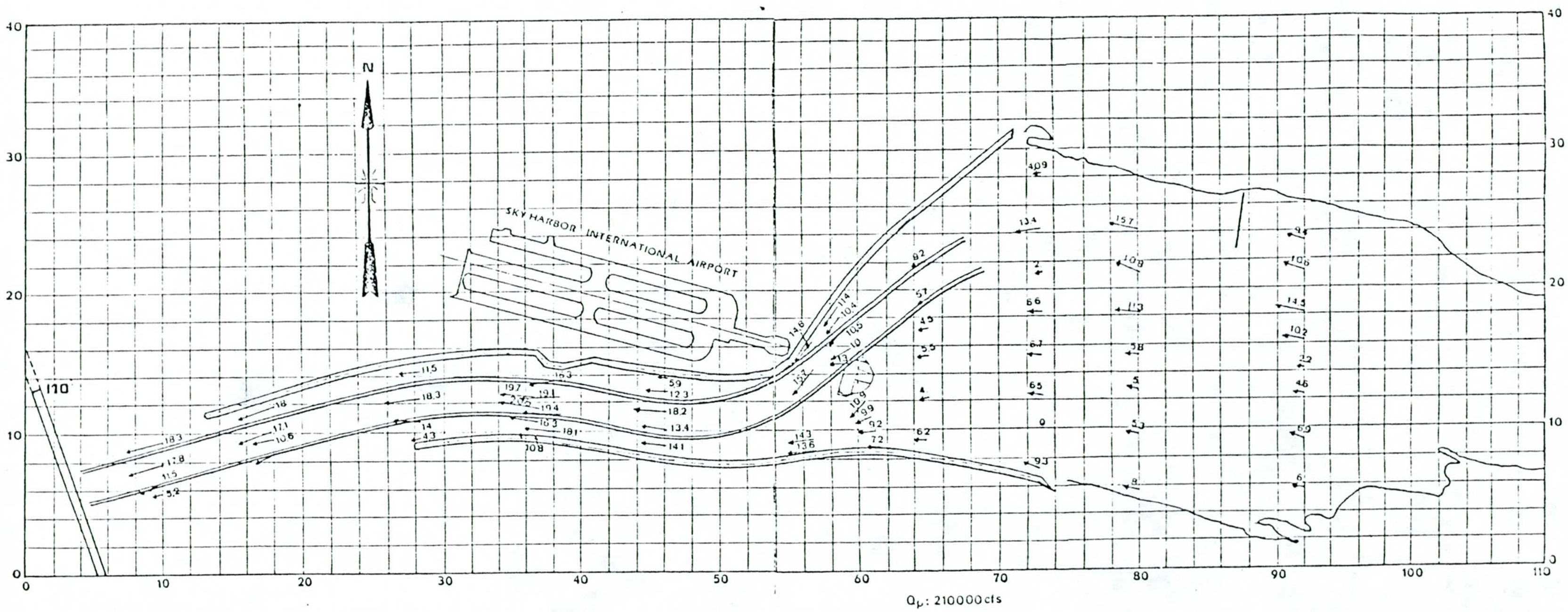


Fig. VI-3   Velocity Distributions in the Fixed-bed Model at 210,000 cfs influence of the low flow channel on flow direction. Peak velocities tend to occur within the part of the cross section occupied by the low flow channel. However, the momentum of the flow creates significant shifts of the velocity peak southward at the radar station and northward near the downstream end of the north levee. The influence of the low flow channel clearly diminishes with increasing discharge. These interpretations are supported by flow visualization studies done during model runs.

FLOW DIVISION AT THE ILS

The porportion of flow passing through the section south of the ILS increased with discharge. As shown in Fig. VI-4 the flow proportion in this southerly channel increased from 31 percent for 92,000 cfs to 37 percent for 210,000 cfs. The engineers for HNTB and for the city of Phoenix expressed concern that the flow through the channel south of the ILS would increase with time as future major floods open up the south channel. This increase would shift the area of the channel under greatest attack and make the protection of the levees more complex. After examining the river patterns including the upstream undivided river reach, we believe the possible increase in the flow discharge through the channel south of the ILS would be quite small.

Another concern is that the chute channel near the south bank upstream of the channelized reach is deeper than that in the channelized reach. A major flood could cut into this south channel reach and increase the proportion of the discharge in this channel. To study this problem, a channel was excavated starting at about Station 74 near the south levee and entered the low flow channel at about Station 52. The dimensions of this excavated channel were about 260 ft

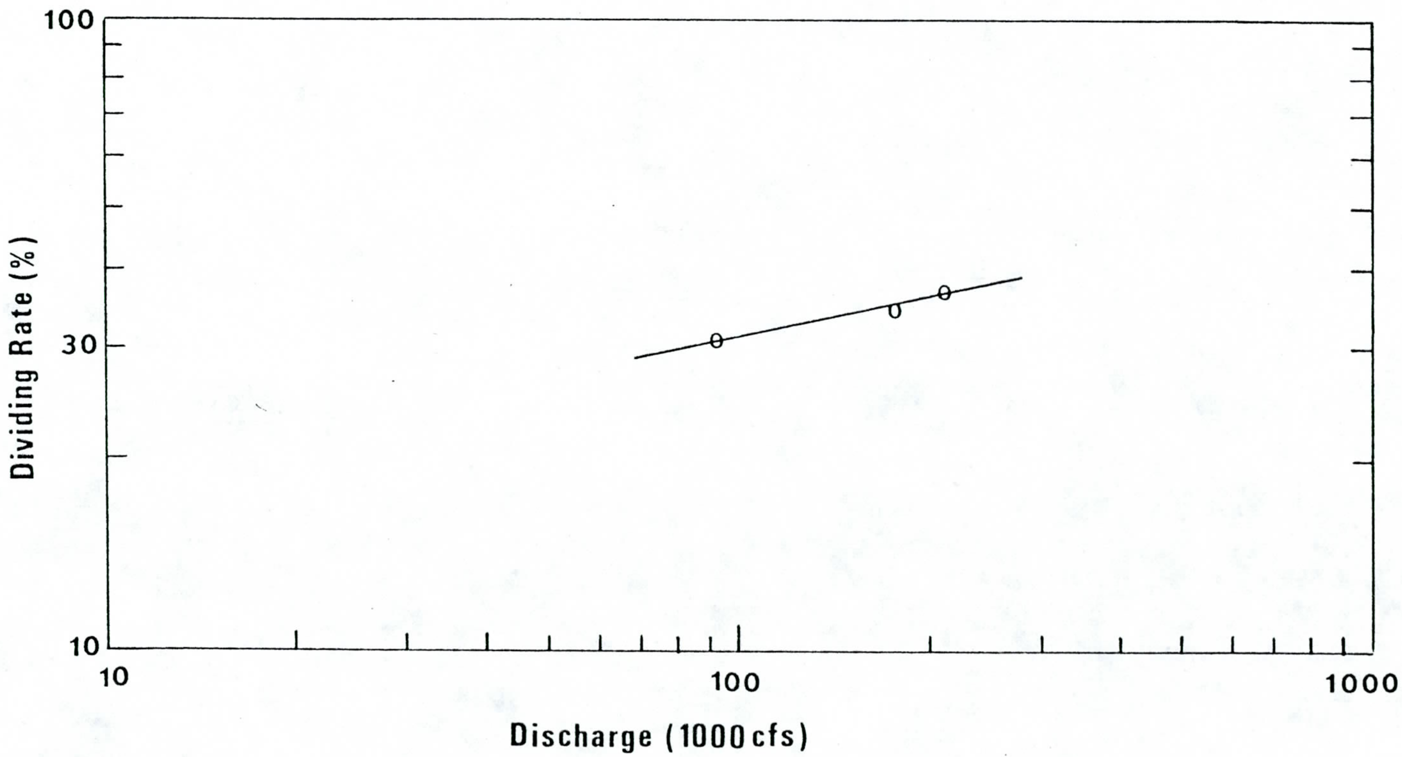


Fig. VI-4 Proportion of total discharge flowing south of the ILS.

wide and 6.5 ft deep. The velocities and stages at selected stations were then measured. It was found that with this increase in flow area, the discharge proportion through the south channel increased from 37 percent to 43 percent. The local velocity in the enlarged channel section increased from 7.2 fps to 9.6 fps at Station 61. The effects of these changes in flow diversion become negligible downstream of Station 50.

VII. MOVABLE-BED PHYSICAL MODEL RESULTS

BED ELEVATION CHANGES

The movable-bed physical model exhibited significant changes in bed elevations, especially at high discharges. Figures VII-1 and VII-2 show photographs of the bed in the movable-bed model after routing the first and second 92,000 cfs hydrographs, respectively. As can be seen in these photos, subsequent floods would not change the general erosion and deposition patterns, but they would smooth the bed profiles. These changes can also be clearly seen in the bed contour plots based on the measured bed elevations as shown in Figs. VII-3 and VII-4.

As observed in the model tests, the main flow impinged on the north levee near Station 60, moved along the north levee until the end of the transition at Station 54, separated from the north levee and impinged on the south levee near Station 44, and then deflected away to impact the north levee near Station 24. This main flow eroded the riverbed along its path and developed a point bar downstream of the transition and upstream of the radar station near the north levee as shown in Figs. VII-3 and VII-4. Figures VII-5 and VII-6 show the bed elevation changes from the initial values for the first and second 92,000 cfs hydrograph runs, respectively. Some bed elevations in the upper reach of the channelized river were affected by the sediment derived from erosion of the north levee upstream of Station 62 and from erosion of the south levee upstream of Station 63. The low flow channel essentially disappeared after the first 92,000 cfs hydrograph. In general, the low flow channel aggraded and adjacent channel bottom degraded.

(a) Braided channels formed upstream of the ILS

(b) Extensive erosion occurred along the south levee downstream of transition and in the vicinity of radar station

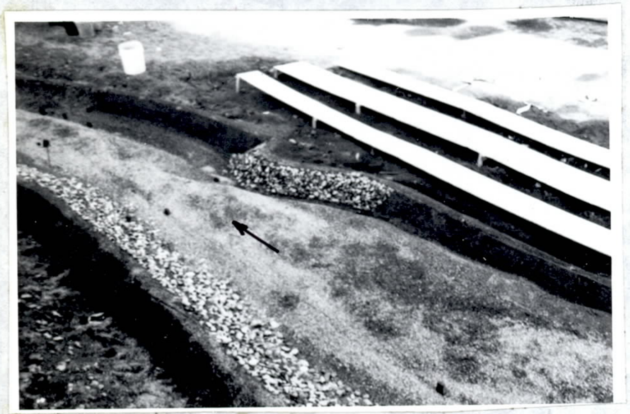
(c) Bed profile changes upstream of the ILS

(d) Bed erosion near the lower north levee (top of sticks show the initial bed elevation)

Fig. VII-1 Photos Showing the Movable-Bed Model after Routing the First 92,000 cfs Hydrographs

(a) Braided channels formed after the first hydrograph enlarged

(b) Scour hole near the south levee below the ILS elongated and became shallower compared to that after first hydrograph



(c) Erosion near levees and deposition in the low-flow channel at the radar station

(d) Continuous erosion along the lower north levee

Fig. VII-2 Photos Showing the Movable-Bed Model Bed after Routing the Second 92,000 cfs Hydrograph

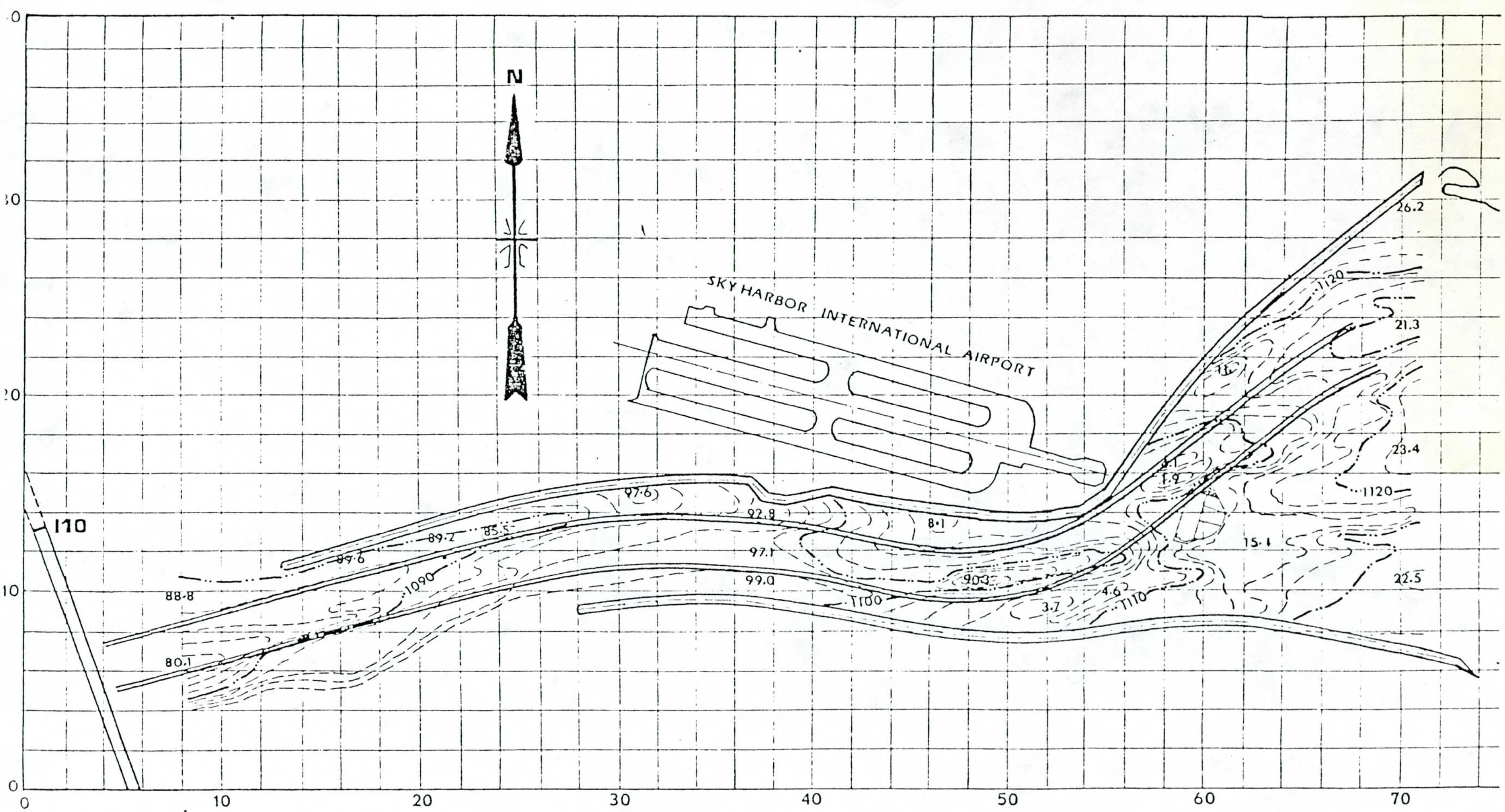


Fig. VII-3 Bed Contours in the Movable-Bed Model after Routing the First 92,000 cfs Hydrograph

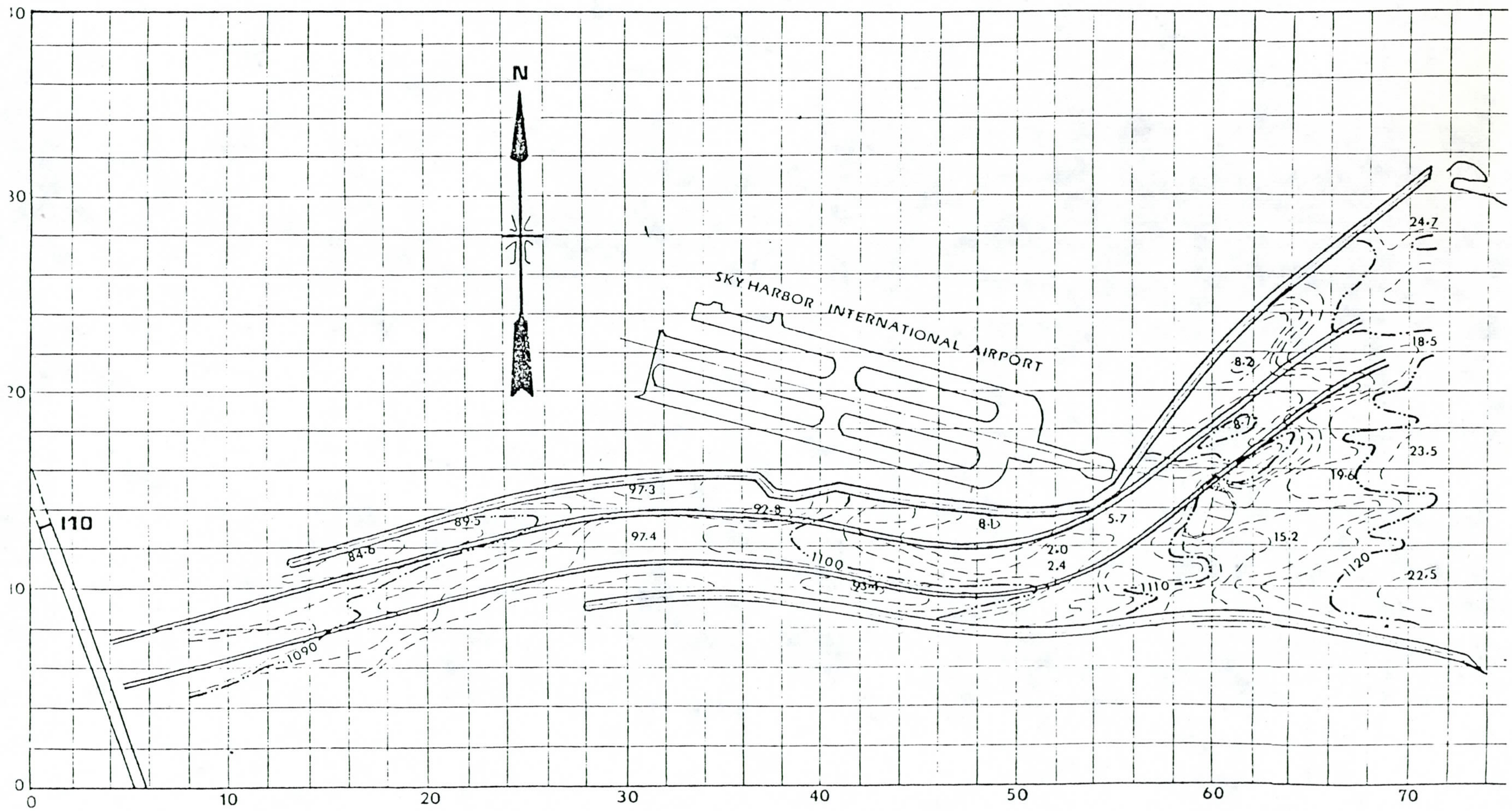


Fig. VII-4 Bed Contours in the Movable-Bed Model after Routing the Second 92,000 cfs Hydrograph

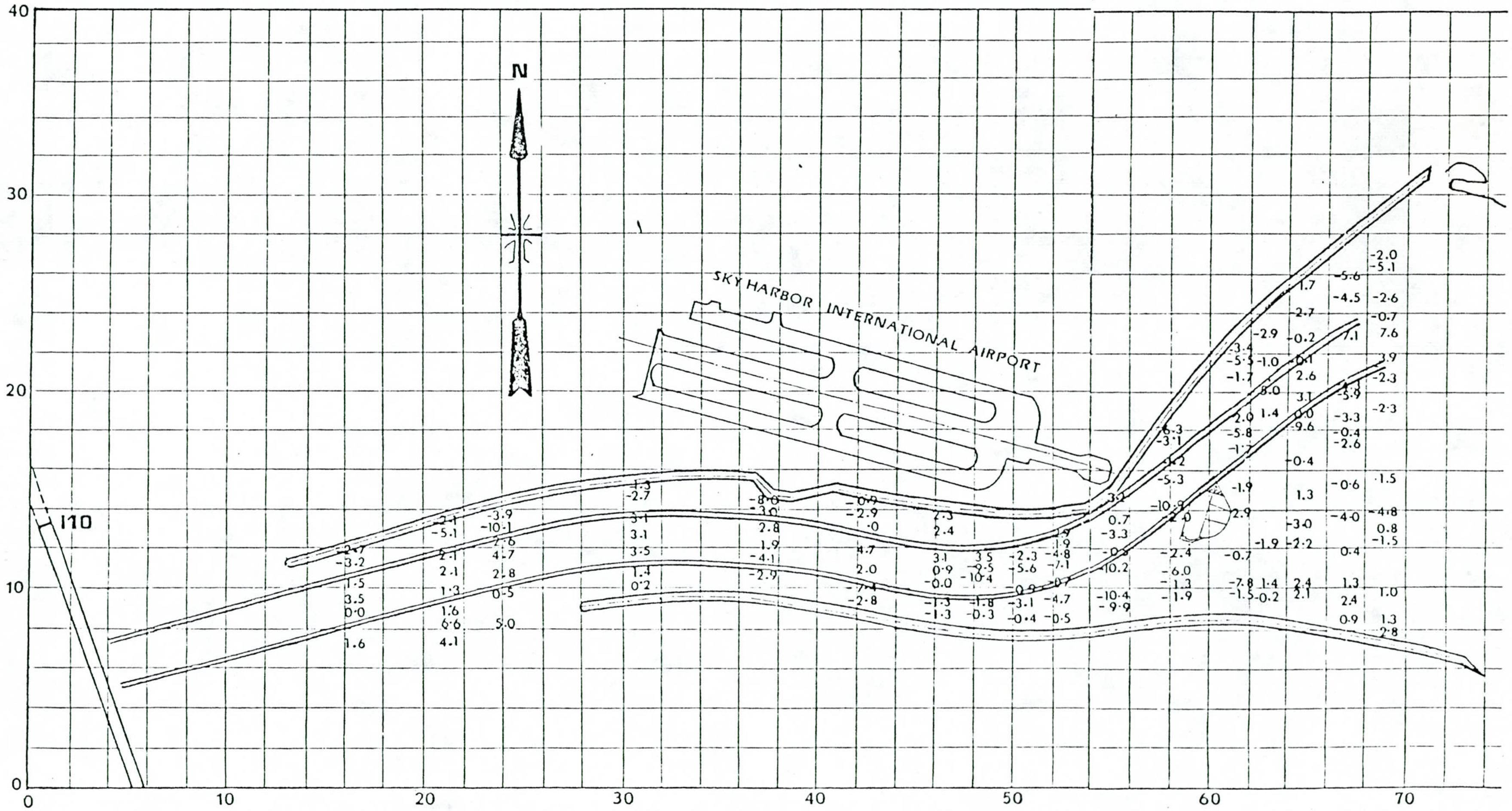


Fig. VII-5 Bed Elevation Changes from the Initial Values after Routing the First 92,000 cfs Hydrograph

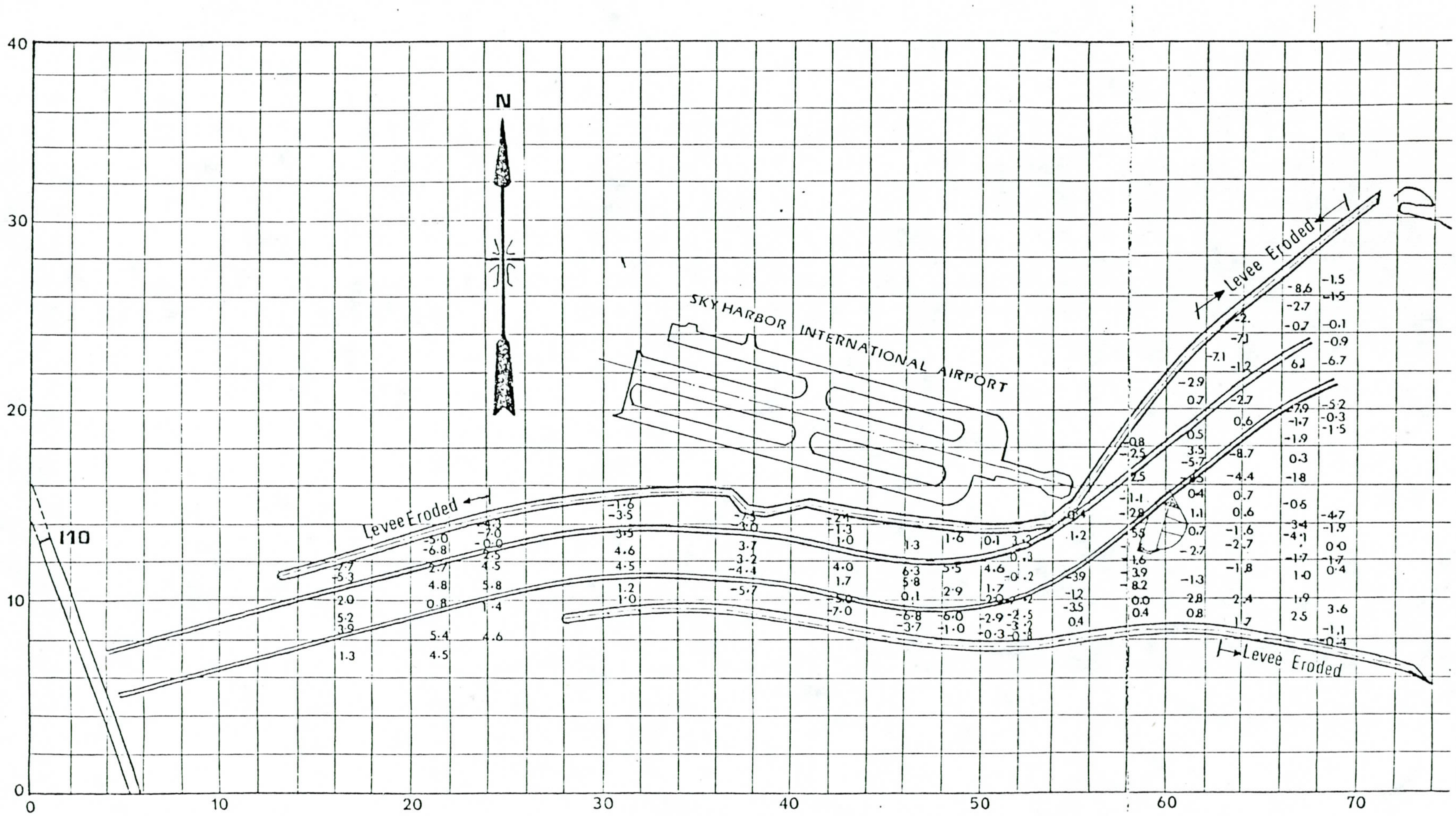


Fig. VII-6 Bed Elevation Changes from the Initial Values after Routing the Second 92,000 cfs Hydrograph

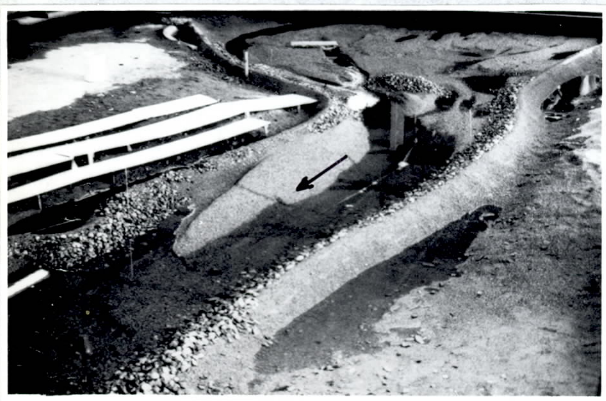
Figures VII-7 and VII-8 show the bed contour in the movable-bed model after routing the 210,000 cfs hydrograph. The general erosion and deposition patterns are similar to those seen after routing the 92,000 cfs hydrographs. The bed elevation changes from the initial contours are given in Figs. VII-9 and VII-10 for after the rising limb and after passage of the entire hydrograph, respectively. The maximum erosion usually occurred during the rising limb and it decreased somewhat during the falling limb.

The size distributions of bed material were altered during transport of sediment in the channelized river reach. Surface bed material samples were collected at local scour holes, middle bars and point bars in the model, and were analyzed. The size distributions of these samples are plotted in Fig. VII-11 versus the initial bed material size distribution. It was found that an armoring layer developed at local scour holes and on eroded middle bars, affecting the sediment transport. Bed material with smaller sizes was found to deposit on the point bar near the north levee below the transition zone and above the radar station, and on some other natural deposition areas such as behind the ILS and the transmission tower sites.

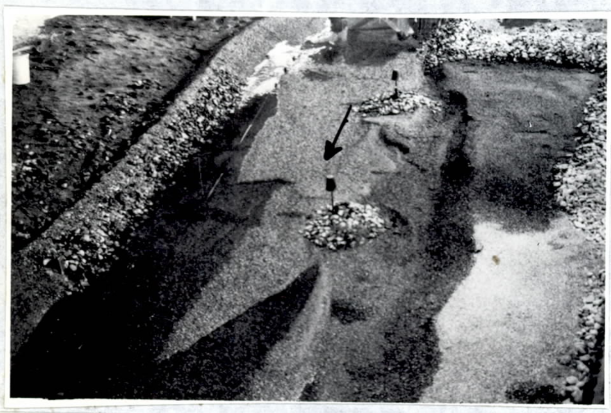
STABILITY OF LEVEES

A qualititive analysis of levee stability can be made based on observed model behavior. Since the dynamics of scour, abrasion, undercutting, and slumping at model levees is only approximately similar to the dynamics of prototype levees and the armoring used in the model consists of 1.5 inch cobbles instead of scaled gabion mattresses, conclusions drawn from model behavior are only qualitative. It is reasonable to expect that levee areas that are damaged in

(a) After the rising limb, big scour hole occurred at ILS near the north levee



(b) After the rising limb, point bar occurred between the transition and the radar station near the north levee



(c) Erosion along the lower north levee and deposition in the low flow channel

(d) After the passage of entire hydrograph the depth of scour hole at ILS became shallower, indicating sediment filling during the falling limb

Fig.VII-7 Photos showing the Movable-Bed Model Bed after Routing the 210,000 cfs Hydrograph

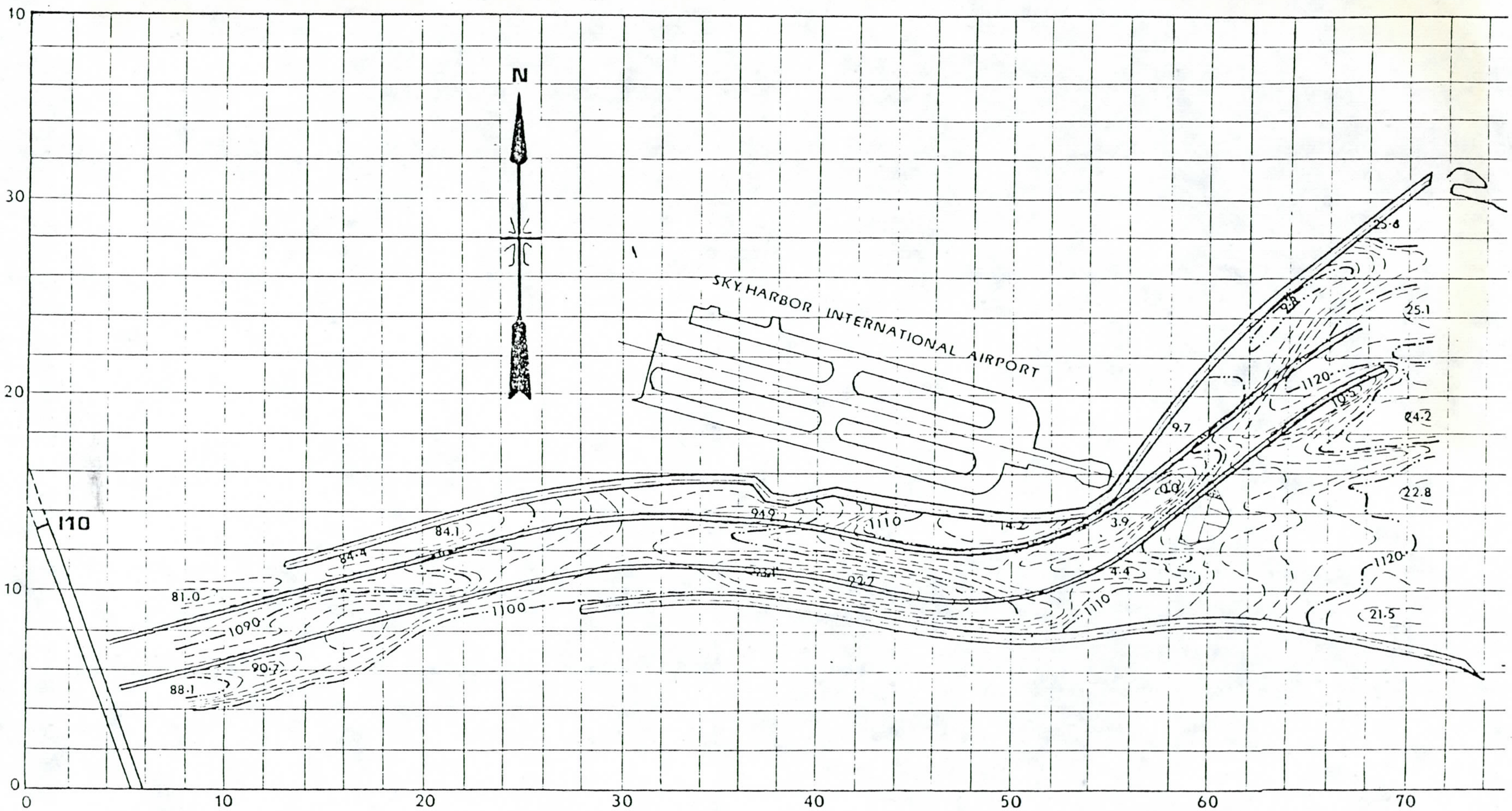


Fig. VII-8 Bed Contours in the Movable-Bed Model after Routing the 210,000 cfs Hydrograph

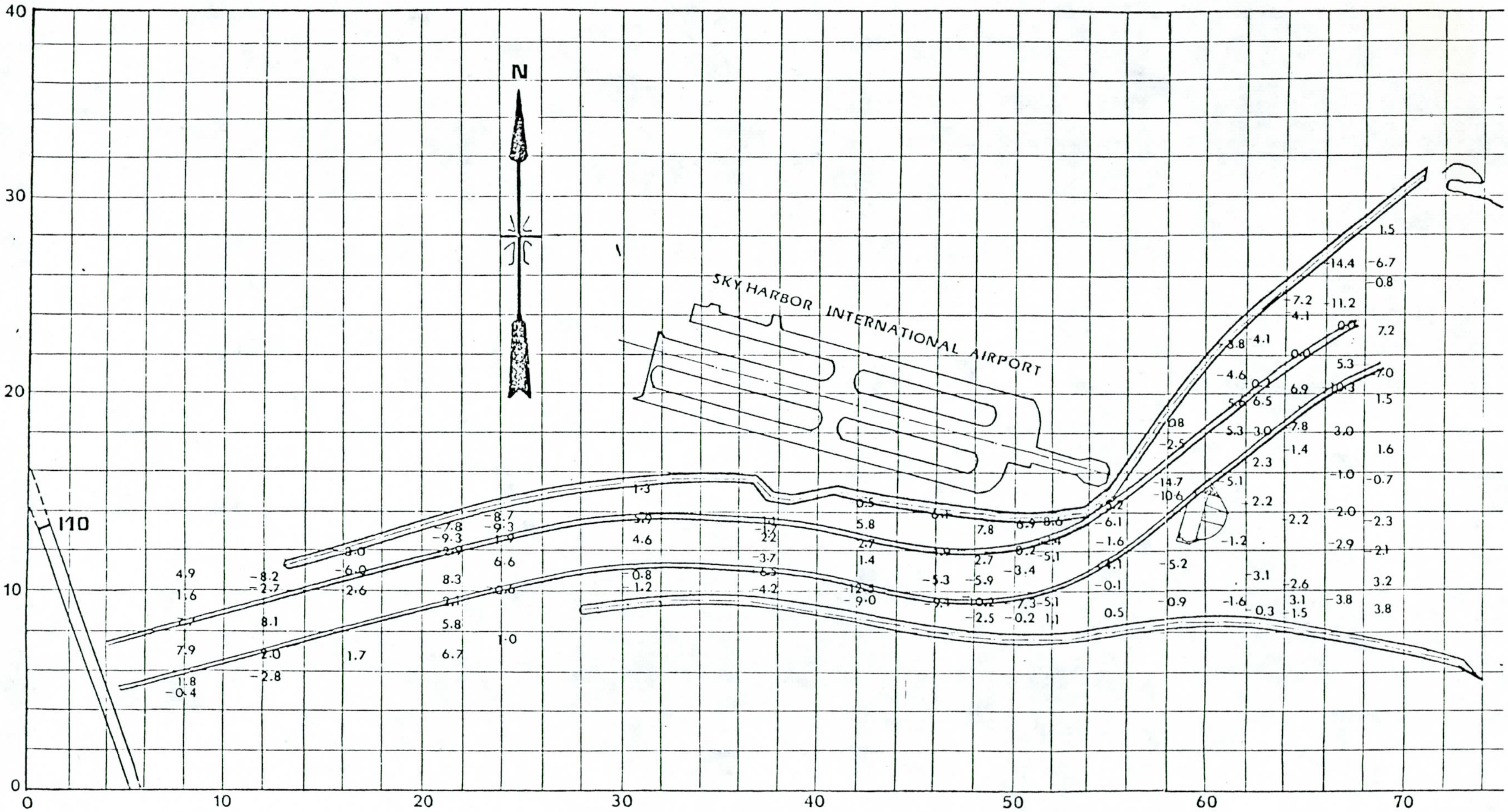


Fig. VII-9 Bed Elevation Changes from the Initial Contours after the Rising Limb of the 210,000 cfs Hydrograph

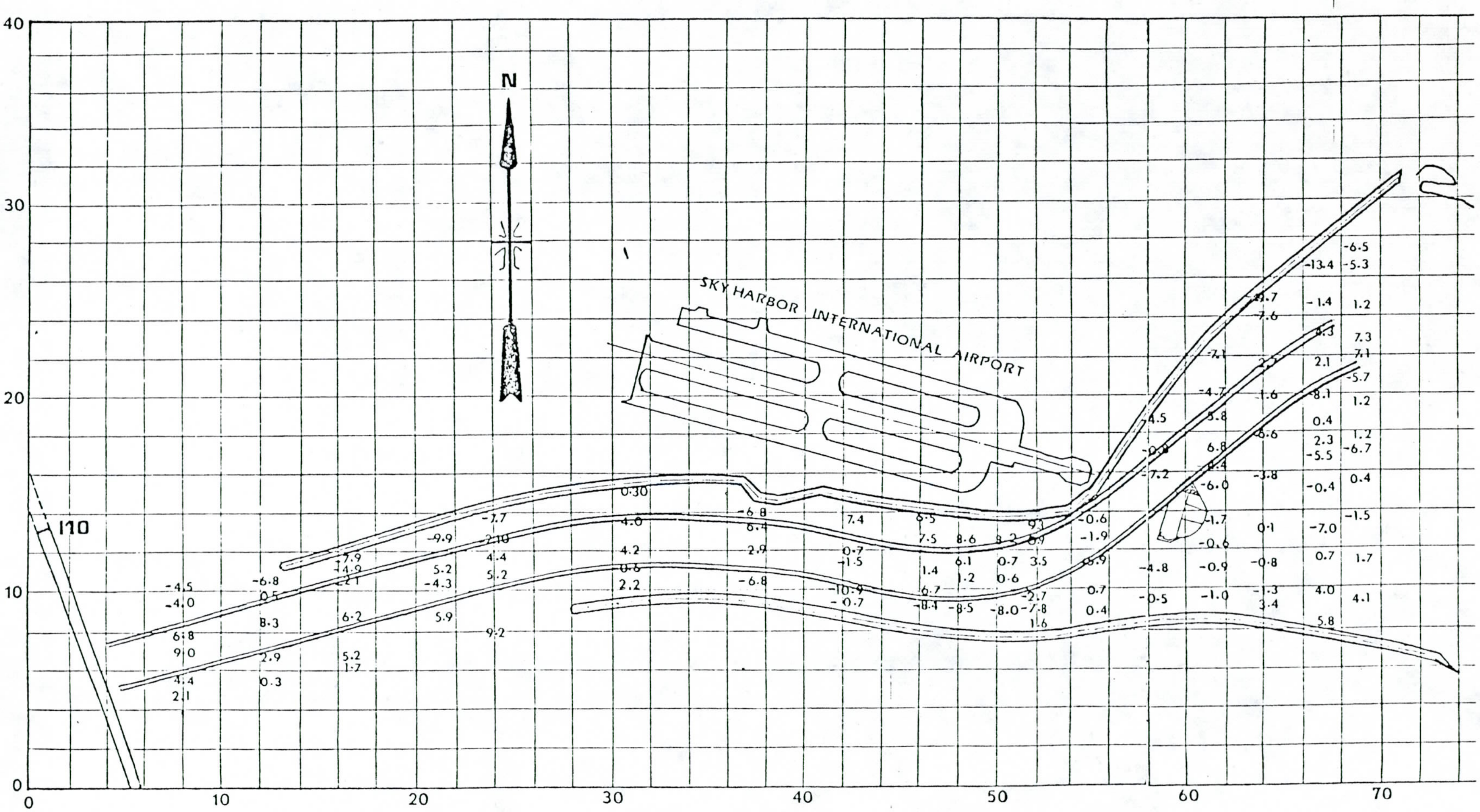


Fig. VII-10 Bed Elevation Changes from the Initial Contours after Routing the 210,000 cfs Hydrograph

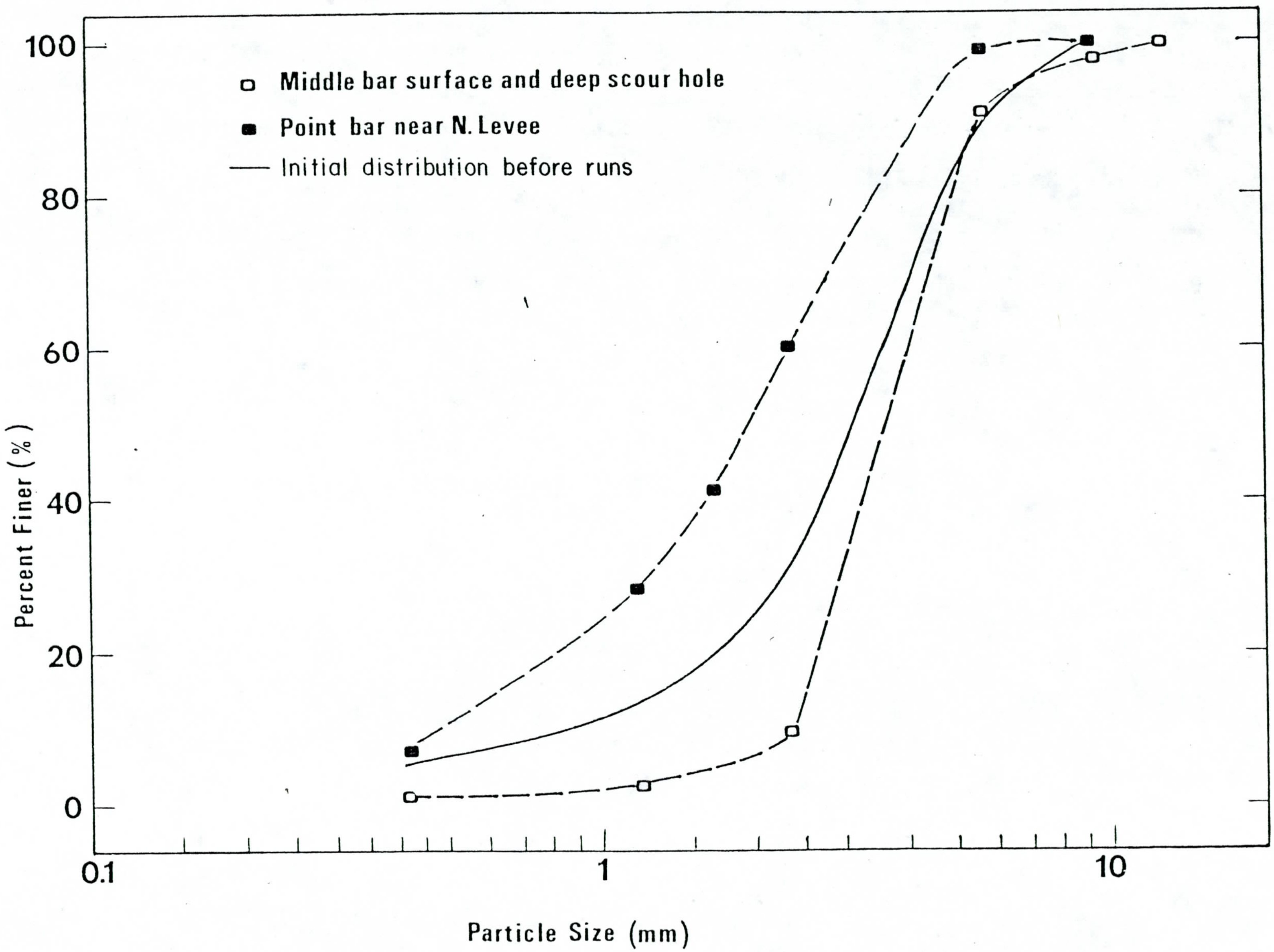


Fig. VII-11 Change in the Size Distribution of Bed Material on the Model Surface the model will also be damaged in the prototype. The extent of damage must be assessed by comparing Salt River channel conditions with prior experience in similar channels. This section presents only the model behavior.

The levee protection proposed by the City of Phoenix and the areas of model levee damage are shown in Figure VII-12. Figures VII-13 and VII-14 show photographs of the unstable levee sections. Unprotected levee sections 1) upstream of the ILS on the north side, 2) upstream of the ILS on the south side, and 3) downstream of the radar station on the north side experienced extreme scour in both the 92,000 cfs and 210,000 cfs runs. Sections of each of these areas were armored during the hydrograph to prevent complete failure. Armored levee sections experienced little or no damage, with the exception of the south levee section downstream of the ILS, which was seriously undercut and slumped, as shown in Figure VII-13 (d). Quantitative damage estimates of these and other impacts are presented in Section X.

HYDRAULIC BEHAVIOR

The effect of sediment movement on stages, velocities, and flow distributions was analyzed using movable-bed models. Water surface elevations for the 210,000 cfs flood event in the movable-bed were higher than those measured in the fixed-bed model throughout the study area. A major stage increase was seen immediately upstream of the radar station and was probably caused by the formation of a sand bar across the channel downstream of the radar. The stages measured for the 92,000 cfs flood event in both physical model studies were nearly equivalent.

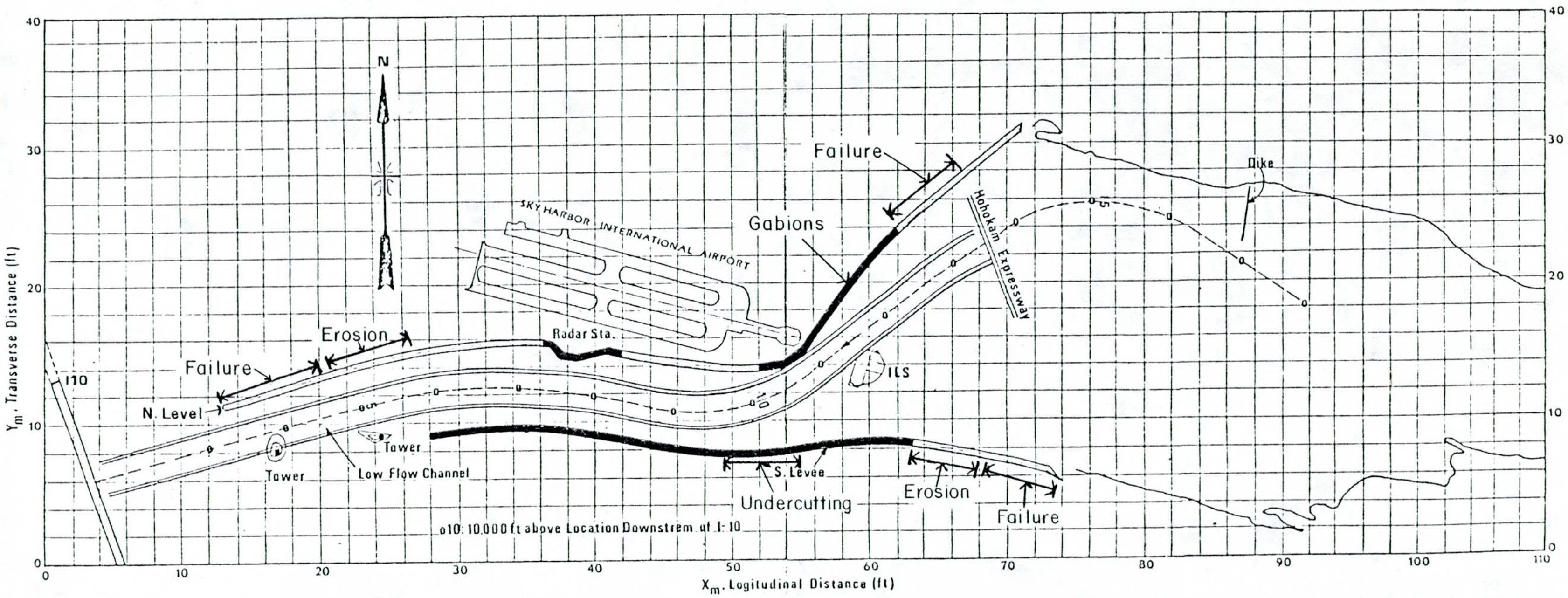
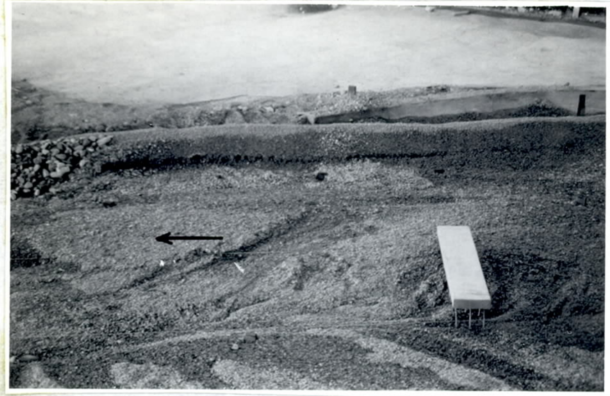
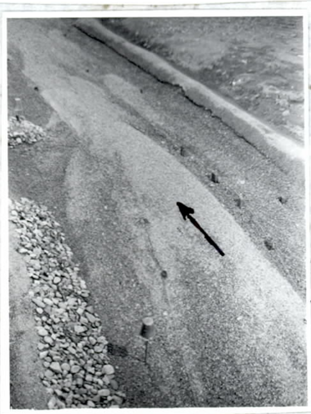


Fig. VII-12 Location of Model Levee Erosion



(a) Some erosion of the upper north levee in the transition after the first 92,000 cfs hydrograph

(b) The upper north levee upstream of the protected section was eroded after the second 92,000 cfs hydrograph



(c) Some erosion of lower north levee after the first 92,000 cfs hydrograph

(d) Continuous erosion of the lower north levee after the second 92,000 cfs hydrograph

Fig. VII-13 Photos showing the Unstable Sections of the North Levee

(a) Flow in chute channel attacked the upstream end of the south end. The levee was then protected

(b) Some erosion of the upper south levee upstream of the ILS after the first 92,000 cfs hydrograph

(c) Further erosion of the upper south levee after the second 92,000 cfs hydrograph

(d) Bed erosion near the south levee downstream of the ILS resulting in sliding of protective riprap

Fig.VII-14 Photos showing the Unstable Sections of South Levees

The general increase in water surface elevations, despite a general degradation of channel invert elevations, implies an increase in model roughness. This roughness increase is probably caused by the build up of point bars and middle bars, creating significant bed form roughness. In areas such as the section downstream of the contraction, bar formation also reduces the flow cross-section, causing higher water surface elevations.

Velocities were measured during both the 92,000 and 210,000 cfs floods in the movable-bed study. A quantitative discussion of the velocity and stage data from the movable-bed model runs is presented in Section IX as part of a comparison of the results of the four models utilized in this report.

A measurement of the flow distribution around the ILS was carried out during the running of the movable-bed model. In three separate runs, two for 92,000 cfs floods and one for the 210,000 cfs event, the proportion of the total flow moving south of the ILS was measured at 31-32%. This flow distribution analysis was complicated by extensive erosion of the south levee during the 210,000 cfs flood event which caused a sediment deposit to form southeast of the south channel.

The magnitude of the diversionary effects of this sediment deposit are unknown. However, the effect of south channel scouring on flow distribution around the ILS is, apparently, small.

VIII. REVIEW OF NUMERICAL SEDIMENT TRANSPORT MODEL STUDY

INTRODUCTION

As part of the investigation into the sedimentation of the proposed channel, the Corps of Engineers sediment transport model (HEC-6) was applied to the river reach between Priest Road and I-10. This model has been applied successfully to silt and sand streams and has been applied to a few gravel bed streams with limited success.

There now seems to be agreement that the existing sediment transport routines in the model are inappropriate for the large stone sizes found in the Salt River. In addition, the HEC-6 sediment routing procedure does not account for size fractions above 84mm. These are serious limitations for the present application. It was felt, however, that the model might provide qualitative predictions of prototype bed behavior which would be helpful in gaining insight into the operation of the movable-bed physical model.

Despite efforts in our study and also in work done by HNTB, no meaningful, defendable results were obtained from the HEC-6 runs. The personnel at the Hydrologic Engineering Center in Davis, California agreed with our assessment that the poor results were due to basic model inadequacies. As a result, further HEC-6 studies were cancelled and a search for alternate modeling techniques was begun.

The firm of Simons, Li and Associates (SLA) has performed a scour and sedimentation analysis for the proposed channel and submitted a draft report on September 25, 1980. The results of their study are reviewed in this section and related to results obtained in the present study in Section IX.

The channel design sediment gradation curves, flood hydrographs, stage/discharge curves, and other hydraulic and hydrologic conditions used in their model runs are identical to those used in this study. Only one peak discharge was evaluated by SLA: 176,000 cfs. As a result, all discussion will be restricted to this condition and subsequent comparisons will be between the SLA 176,000 cfs results and our 210,000 cfs results.

The scope of work for the SLA study included the following tasks:

1. Conduct an evaluation of aggradation and degradation response for the proposed channelization plan.
2. Assess the long-term degradation problem using an engineering geomorphic approach.
3. Evaluate the potential scour problems, including general and local scour
4. Recommend possible modifications to both the proposed channelization plan and bank protection requirements.

The analyses necessary to complete these tasks were accomplished at three levels: 1) qualitative geomorphic studies, 2) quantitative engineering geomorphic analysis, and 3) numerical modeling of the physical processes. Level 2 provided quantitative estimates of local scour and overall scour rates for the channel, while level 3 provided quantitative estimates of general scour and the variation in thalweg elevations during a flood hydrograph.

MODEL DESCRIPTION

The model used in the SLA study was developed for use on gravel bed streams and has been tested on other projects, including the study of the I-10 bridge channel and an analysis of the impacts of gravel mining in the Salt and Gila Rivers. A version of HEC-2 performs the hydraulic calculations, producing water surface elevations and mean velocities. These results are then used to compute sediment transport rates using the Meyer-Peter, Muller sediment transport equation and the Einstein procedure. Rates of scour or deposition are then computed for each section of the channel, taking into account the effects of armoring; and the total movement of sediment into or out of each section is determined. The bed elevations are then modified to account for the net sediment motion, including distribution of sediment across the channel due to varying conveyance or volume flux at different points in each cross section. The new geometries are then input to HEC-2 and the process repeated for each time step in the hydrograph.

Although a number of channel conditions were modeled, particularly several alternative bed material gradation curves, the SLA recommendations were based on one set of conditions. A composite gradation curve, based on a rough average of the HNTB and Dames and Moore test pit data, was used as shown in Figure VIII-1. The hydrograph for the 176,000 cfs event was based on the hydrograph selected for the present study, as shown in Section V. Channel cross-sections were those used in the ANCo HEC-2 runs, with numerous deletions to save computer costs. The selection of reaches for the sediment calculations is shown in Figure VIII-2. It is important to note that sediment processes are assumed to be constant within each reach.

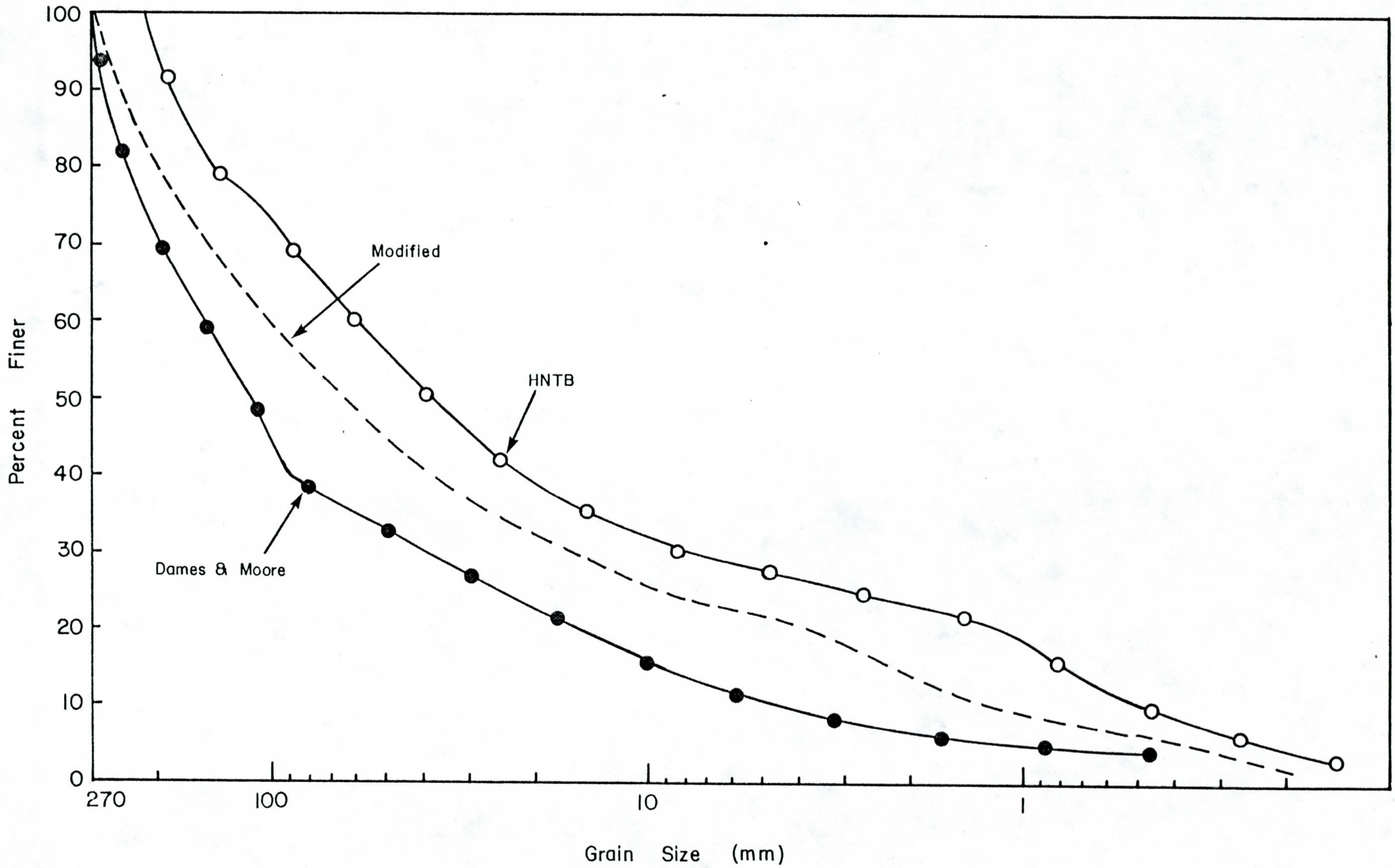


Figure VIII-1 Representative grain size distributions. (After Li, et al, 1980)

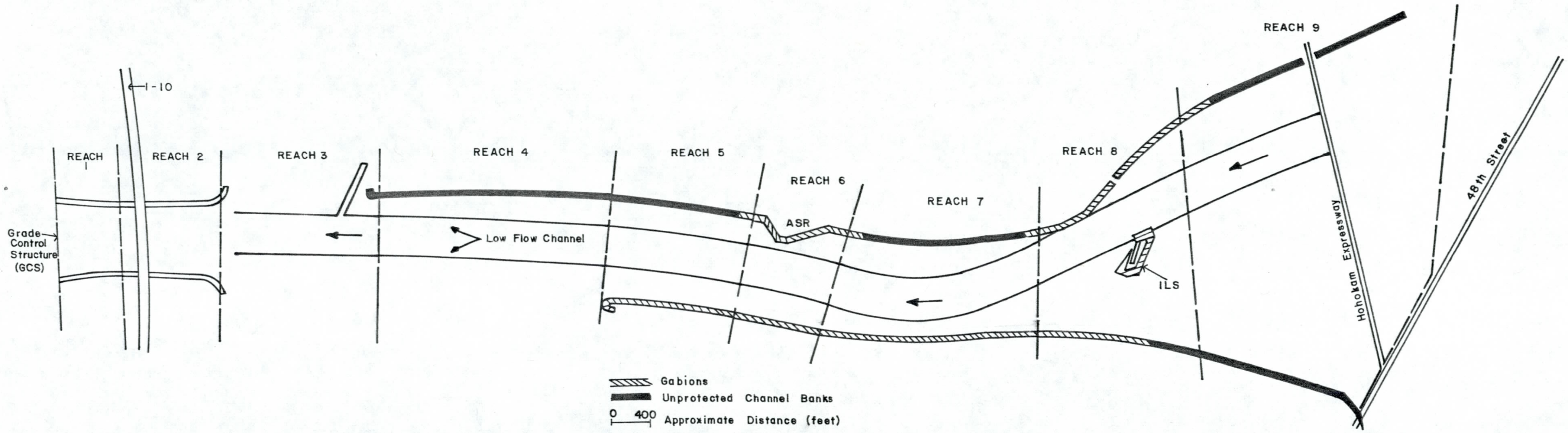


FIGURE VIII-2. Reaches Used in Numerical Sediment Model (After Li, et al, 1980)

SUMMARY OF RESULTS

The following discussion is a brief summary of the SLA results, as they relate specifically to the present study. Details of SLA procedures and analyses can be found in their report.

Changes in Thalweg Elevation

The numerical model results predict the movement of the Channel bed over the time of the flood hydrograph. However, only general scour estimates are represented in the model results. Figure VIII-3 shows the predicted initial, maximum, minimum, and final thalweg elevations for the 176,000 cfs event, using the modified bed material gradation curve. Maximum general scour is predicted for the area around the Hohokam Expressway. General scour in excess of 2 feet occurs from the vicinity of Hohokam down to below the radar installation (ASR). Aggradation occurs in the region between the downstream end of the north levee and the grade control structure below the I-10 bridge.

A picture of channel degradation with time is shown in Figures VIII-4 and VIII-5 for the ASR and the ILS cross sections, respectively. Both figures show maximum degradation occurring well before the end of the event and subsequent aggradation during the falling limb of the hydrograph. The shape of these curves and the time of maximum degradation vary substantially with position in the channel.

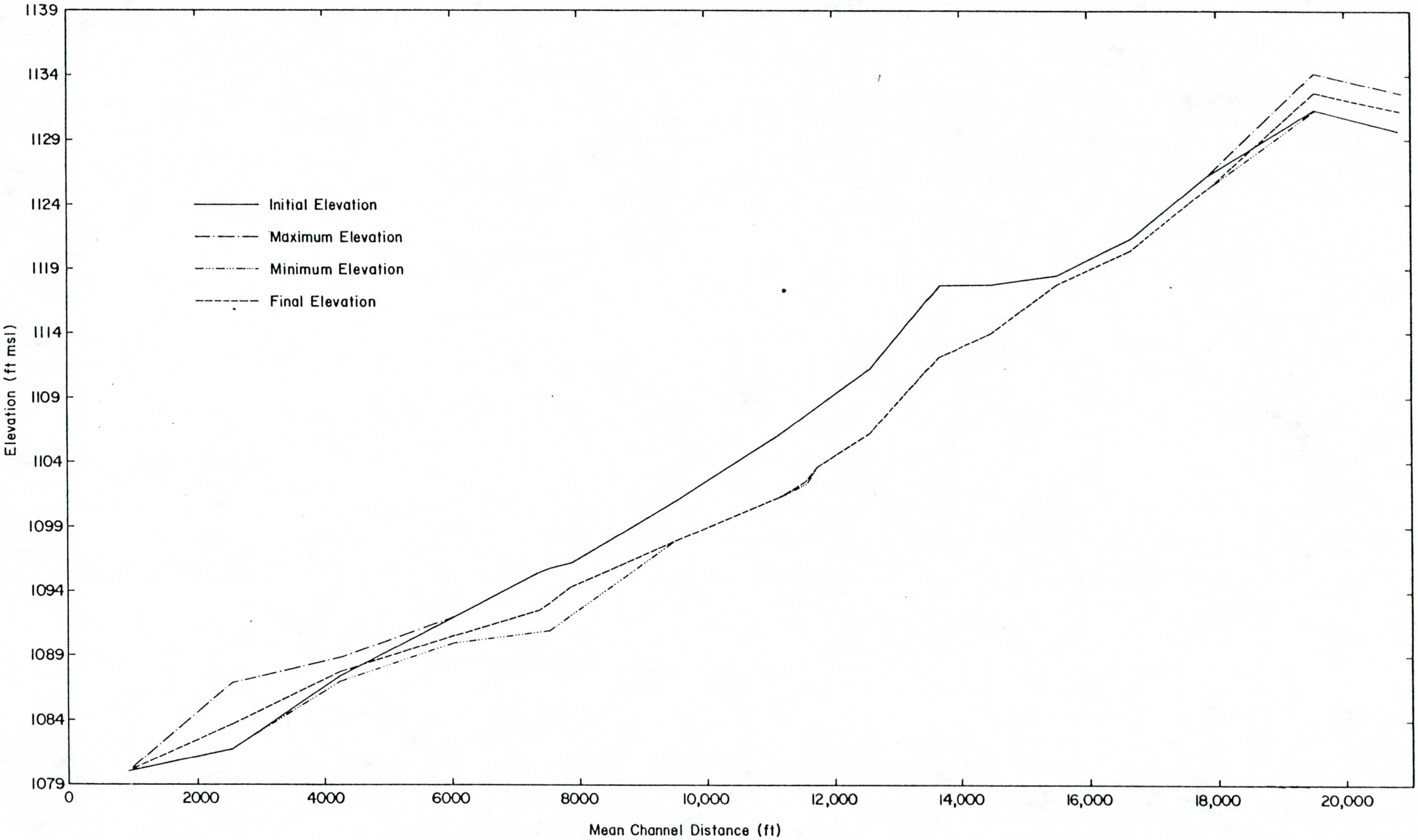


Figure VIII-3 Bed Elevation Profiles Predicted by Numerical Model (after Li, et al, 1980)

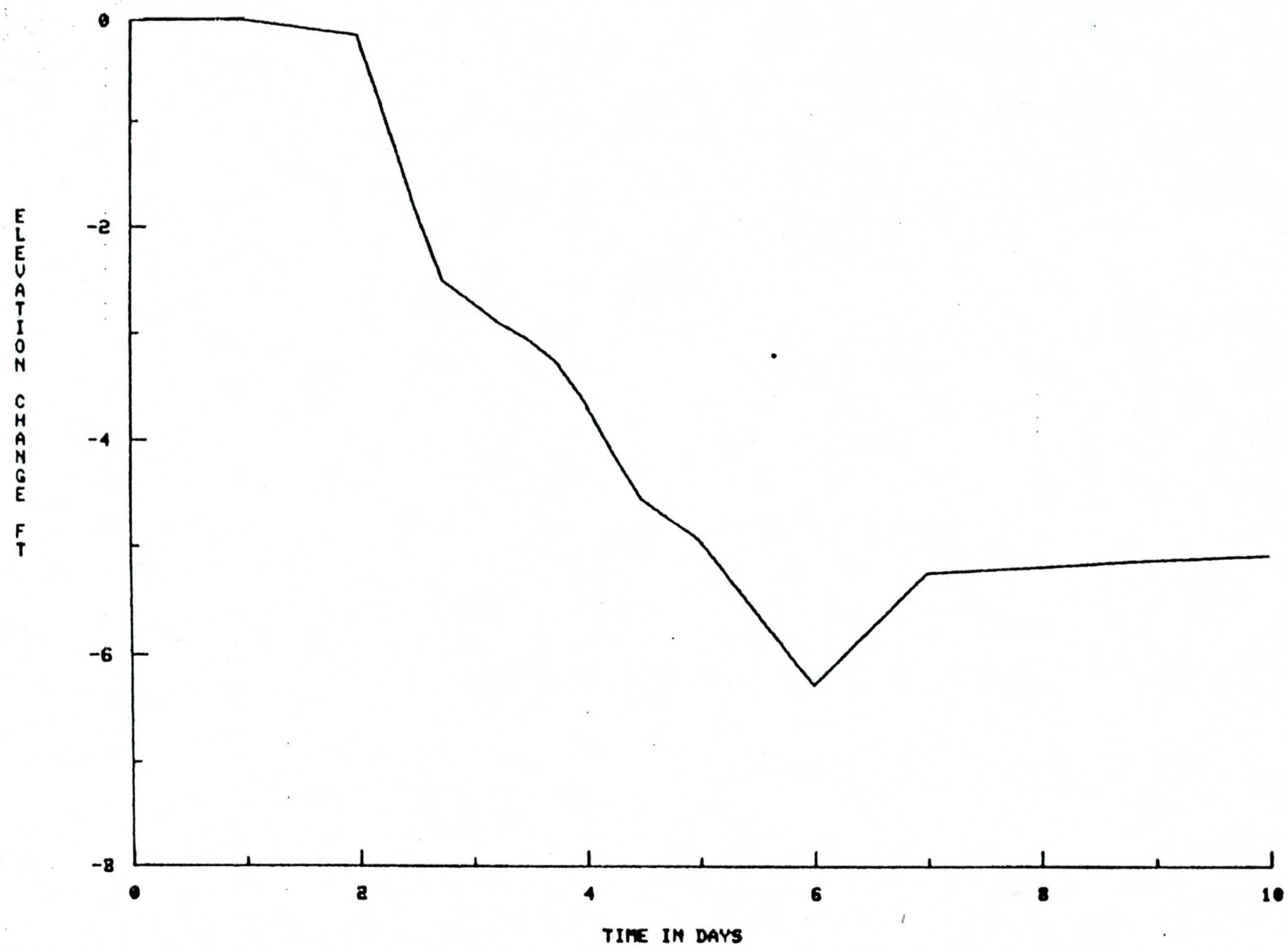


Figure VIII-4 General scour versus time at the radar station. (after Li, <u>et al</u>, 1980)

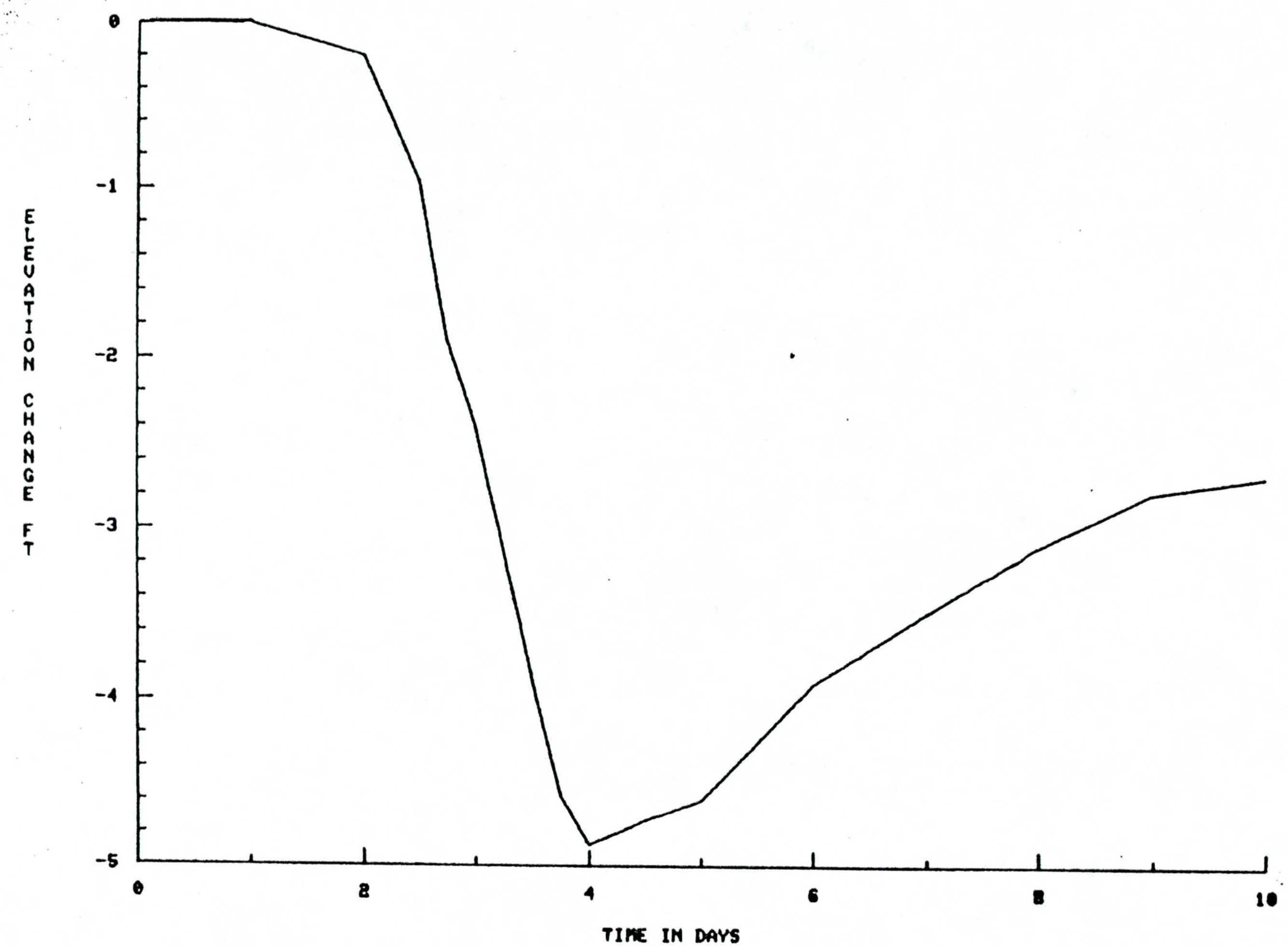


Figure VIII-5 General scour versus time at the ILS. (after Li, <u>et</u> <u>al</u>, 1980)

Local and General Scour

SLA identified three reaches with substantial potential for local scour, based on estimated local velocities and flow patterns. As shown on Table VIII-1, areas in the vicinity of the ILS, the ASR, and the transmission tower island in reach 4 may experience substantial local scour. Table VIII-1 also shows estimates of general scour for each reach, as predicted by the numerical model, and total scour, which is a summation of local and general scour.

Model Accuracy and Applicability

Prior experience with the SLA numerical model and associated estimation process for local scour indicates that the techniques are both applicable to and reasonably accurate for Salt River conditions. In a prior study SLA predicted the effects of the February, 1980 flood on the I-10 bridge channel to within 10% of the actual scour experienced. In addition, the results presented in their report appear to be both reasonable and in concert with qualitative estimates based on geomorphic analyses.

The major limitations of the results reviewed above are related to the one dimensional nature of the numerical model and the coarse spatial resolution used in the model runs. A one dimensional model assumes no significant variations in hydraulics or sediment transport either with depth or with lateral position in the channel. The model also does not represent momentum effects at curves and bends, the influence of bars and scour pits smaller than the resolution of the model, changes in roughness due to dynamic changes in bed forms, the local effects of turbulence and flow concentration in the vicinity of obstructions, and other local or small scale phenomena. Model results are,

Table VIII-1 Maximum Scour Depths in Channelized Areas

Location	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

(after Li, <u>et</u> <u>al</u>, 1980)

consequently, limited to the prediction of general scour along river reaches which are essentially one-dimensional in character and exhibit smoothly varying channel morphology. Sharp bends, large obstructions, rapid changes in bed elevations, and other discontinuities can not be accurately modeled using low resolution, one-dimensional techniques.

SLA has minimized the above limitations by performing hand calculations of local scour at points where the one-dimensional assumption is not appropriate. Their model also incorporates techniques for varying aggradation/ degradation across the channel according to conveyance estimates. As shown by the physical model results, however, the hydraulics of the proposed channel are highly three-dimensional and sediment processes vary significantly with position across the channel. It is thus unreasonable to expect that the SLA model and local scour analyses will provide detailed information on scour and deposition. Comparisons between physical and numerical model results will be presented in Section IX.

IX. ANALYSIS AND COMPARISON OF MODEL RESULTS

The evaluation of the hydraulic and sedimentation properties of the proposed Salt River channelization project involved the utilization of four separate modeling studies:

1. Numerical model of riverine hydraulics (HEC-2);
2. Fixed-bed physical model;
3. Movable-bed physical model;
4. Numerical model of sedimentation (SLA study).

The results of the individual modeling studies have been presented in previous sections of this report. In order to evaluate the reliability of the model results and to decide which modeling approach is most appropriate in the investigation of specific hydraulic and sedimentation properties, a comparison of the findings of the four modeling studies is warranted. Comparisons of the flood stage, velocity, and sedimentation data are discussed separately.

FLOOD STAGE

Extensive flood stage data were obtained from the applications of the numerical hydraulic model and of the fixed-bed physical model. A limited amount of stage data were gathered from the movable-bed physical model application. The average stages measured in the fixed-bed physical model are compared with the numerical model stages in Figures IX-1, 2, and 3. The stages measured in the two models at three different discharges (92,000, 176,000, and 210,000 cfs) show excellent agreement. The determination of flood stage under a fixed-bed assumption can be reliably made from either of these two modeling techniques. As can

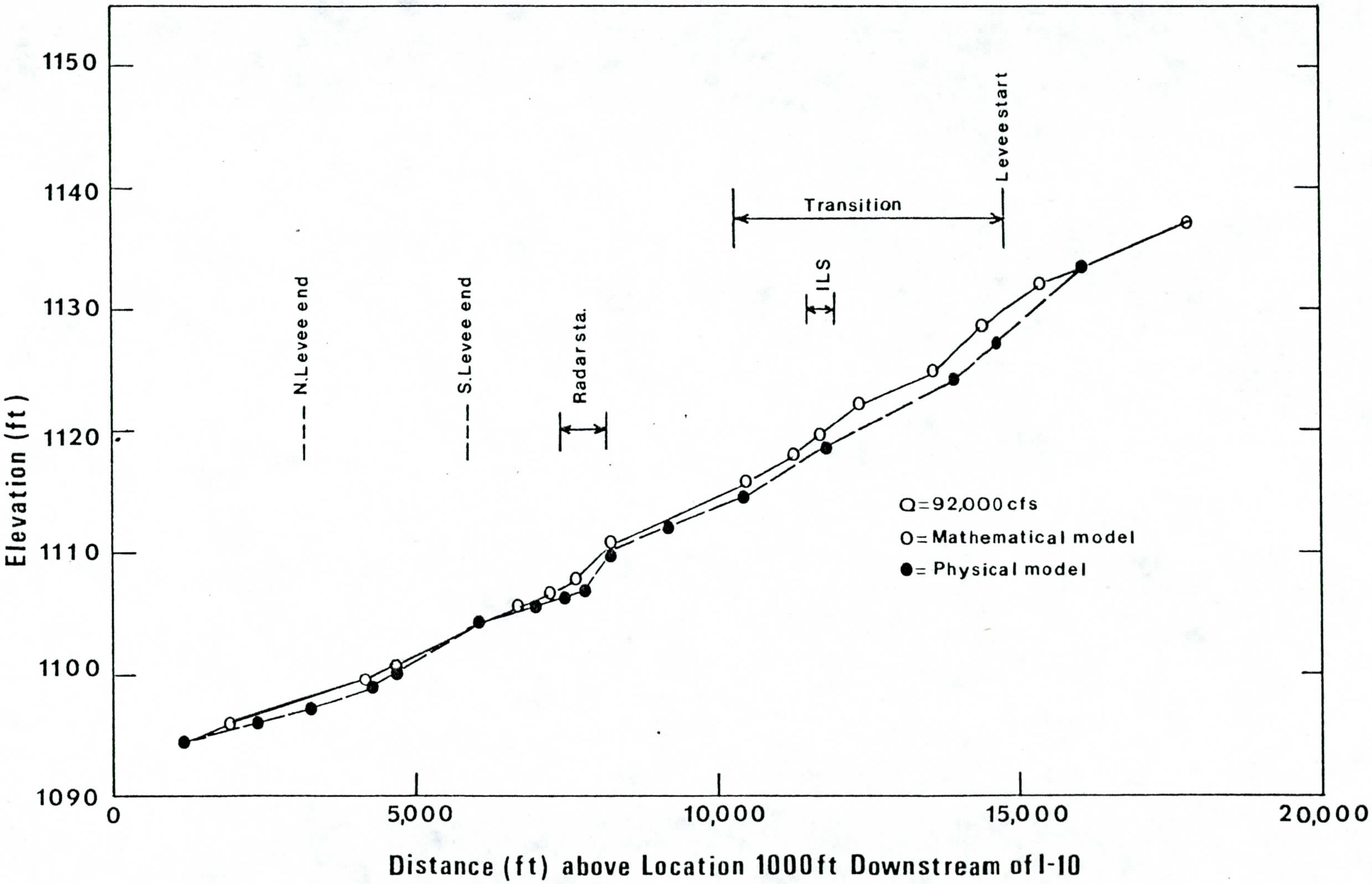


Fig. IX-1 Comparison between Average Stages of Fixed-bed Physical Model and Mathematical Model Stages at Q = 92,000 cfs

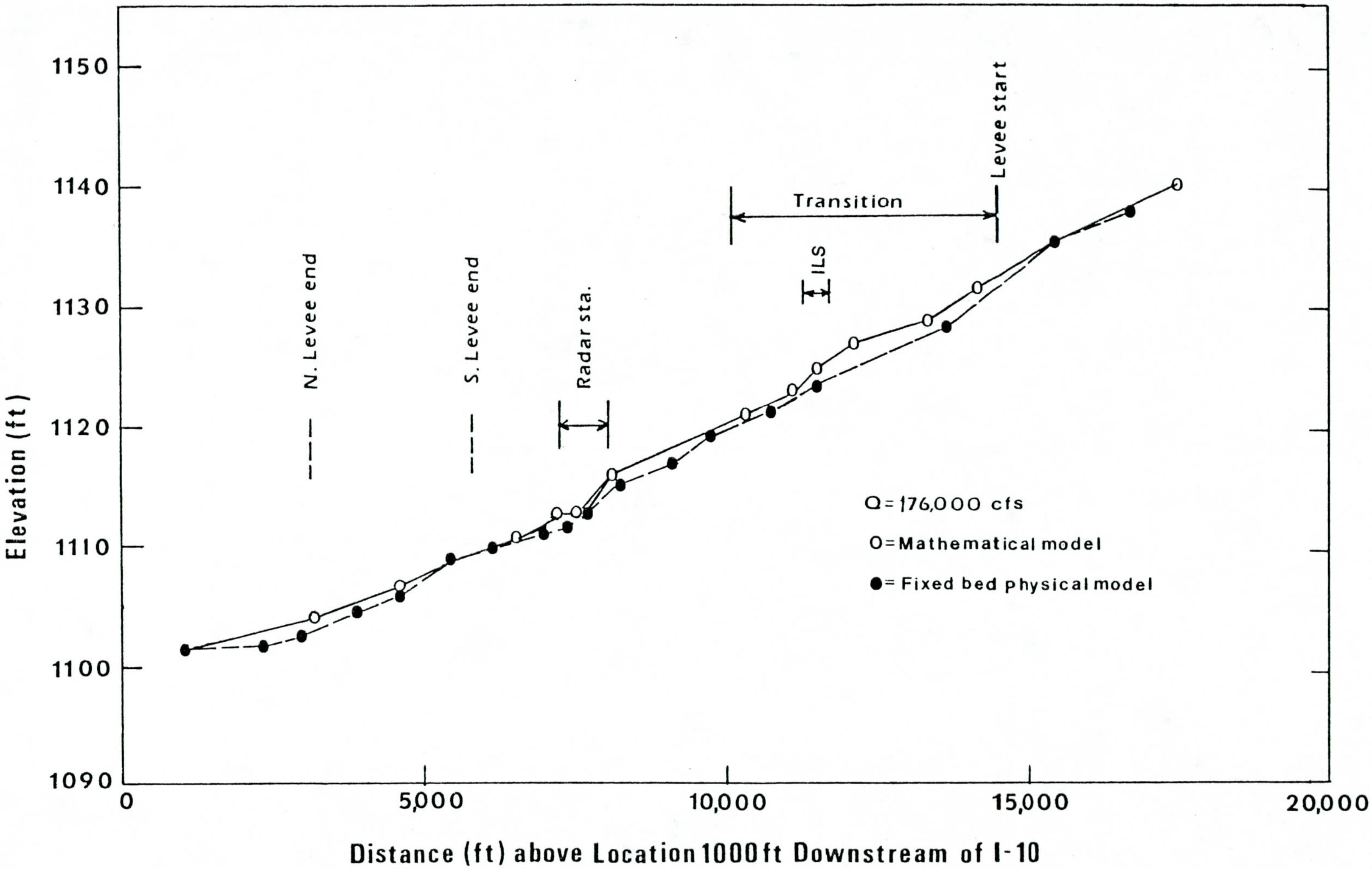


Fig. IX-2 Comparison between Average Stages of Fixed-bed Physical Model and Mathematical Model Stages at Q = 176,000 cfs

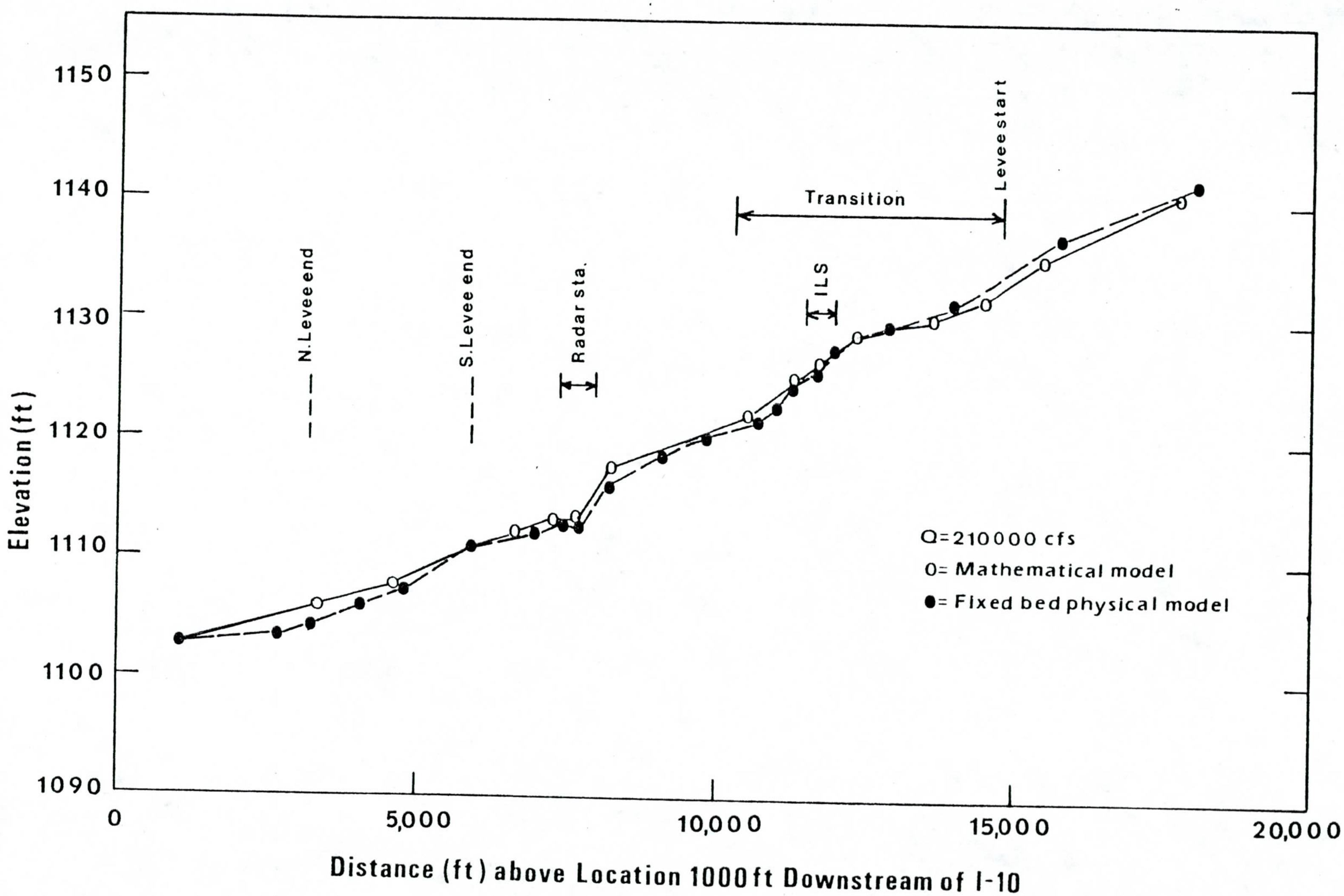


Fig. IX-3 Comparison between Average Stages of Fixed-bed Physical Model and Mathematical Model Stages at Q = 210,000 cfs

be seen in Tables VI-1, 2, and 3 of section VI, the stage measurements made on the fixed-bed physical model during the same flood and at the same cross-section may differ at the north and south levees. The one-dimensional nature of the HEC-2 numerical model does not allow a study of this stage variation since it assumes a single stage value at each cross-section for a given discharge. Therefore, based upon these results, it appears that fixed-bed physical models yield reliable stage data and can provide results that go beyond the one-dimensional limitation of the numerical model employed in this study.

The flood stage measurements made during the running of the movable-bed physical model gave results that indicated further complexities in the hydraulics of the river. The movement of the river bed during the flood can significantly affect flood stage. At a discharge of 92,000 cfs the stages measured in the fixed-bed and in the movable-bed models were essentially the same. However, at a discharge 210,000 cfs, the effects of bed movement on flood stage can be measured. In Table IX-1 a comparison of the stages measured in the two physical models has been presented. The greatest difference in the measured stages occurs at model station 38.5, immediately upstream of the radar station. The increased stages seen during the running of the movable-bed model at this location appear to have been caused by the formation of a sediment bar across the width of the flood channel just downstream of the radar station. In general, at all cross-sections, stages measured during the 210,000 cfs flood in the movable-bed model were higher than those measured in the fixed-bed model. These higher stages are at least partially due to the development of irregularities in channel geometry due to sediment movement. Channel irregularities cause bed form resistance to flood passage, resulting in higher flood elevations.

TABLE IX-1 Stages in the Movable-Bed Model and the Fixed-Bed Model (Q = 210,000 cfs)

Model Station	Movable-Bed Model Stage Elevation (ft)		Fixed-Bed Model Stage Elevation (ft)	
	South Levee	North Levee	South Levee	North Levee
23.5	-	1109.9	-	1108.2
28.5	1112.9	1113.7	1110.9	1111.1
33.2	1113.5	1114.6	1112.0	1111.8
38.5	1116.5	1116.3	1111.1	1112.6
45.4	1119.9	1117.8	1119.2	1117.5
55.0	1123.4	1121.6	1122.2	1120.7

VELOCITY

Velocity magnitudes were measured in the two physical models (fixed-bed and movable-bed) and in the numerical hydraulic analysis. Figures IX-4, 5, 6, and 7 present the results of the three models at selected cross-sections and discharges. The numerical hydraulic model (HEC-2) predictions vary by as much as 30% from cross-sectional averages of the physical model data. Velocities are typically higher in the physical models, which is most likely due to local accelerations at the ILS and radar station which are not represented in HEC-2. In addition, velocities vary across the channel by up to 200% (Figure IX-5) due to the three-dimensionality of the flow. These results indicate the importance of using physical models or multi-dimensional numerical models if detailed velocity predictions are required.

The agreement between the movable-bed and fixed-bed models is generally good, with significant exceptions in the vicinity of the ILS (Figure IX-5). The effect of changes in bed morphology on velocities during the 92,000 cfs run is quite apparent.

SEDIMENTATION

The sedimentation characteristics of the channelization project were evaluated using a numerical sedimentation model and a movable-bed physical model. The physical model simulated discharges of 92,000 and 210,000 cfs, while the numerical model studied a discharge of 176,000 cfs. Figure

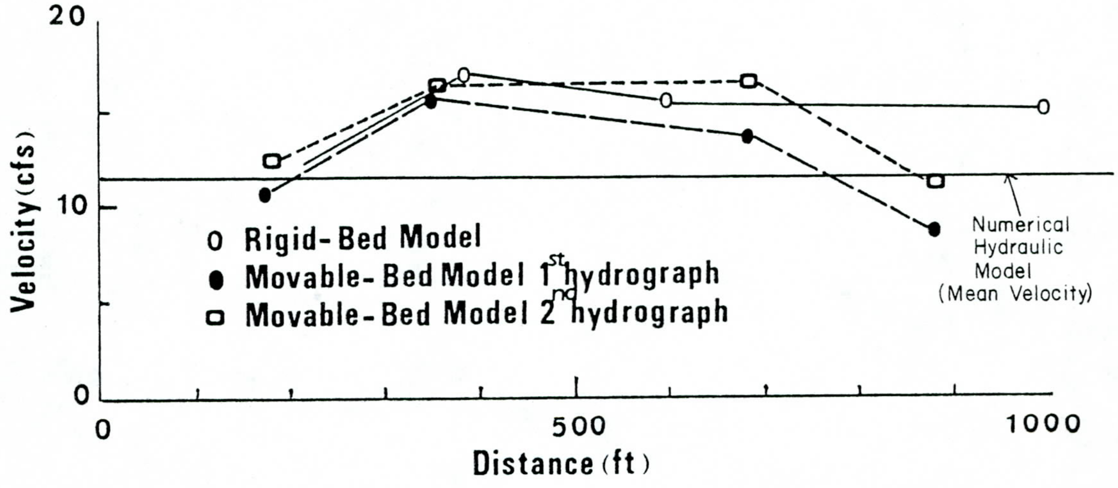


Fig. IX-4 Comparison between Velocities at Station 38.5 (Q = 92,000 cfs)

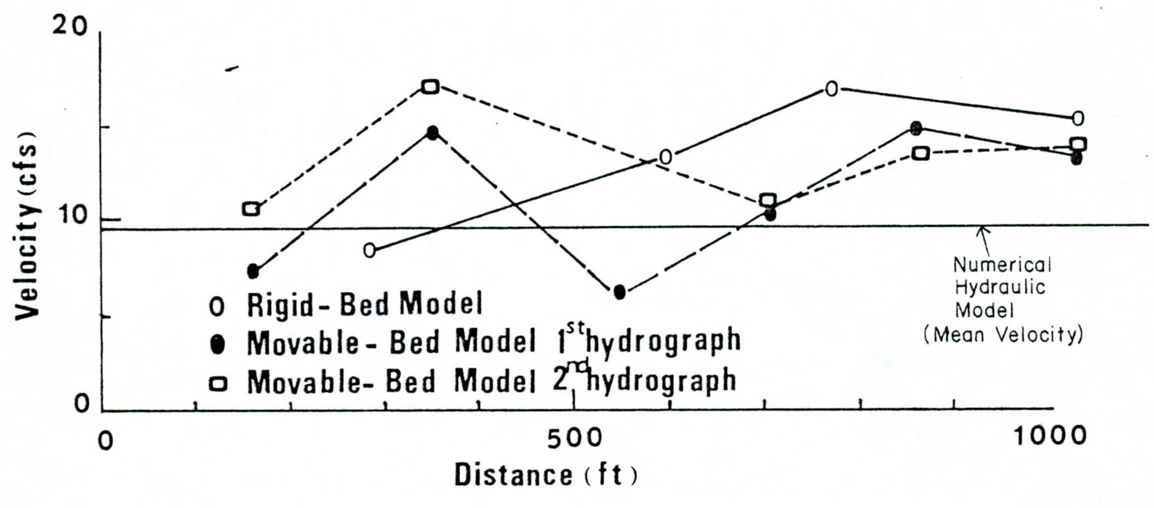


Fig. IX-5 Comparison between Velocites at Station 55 (Q = 92,000 cfs)

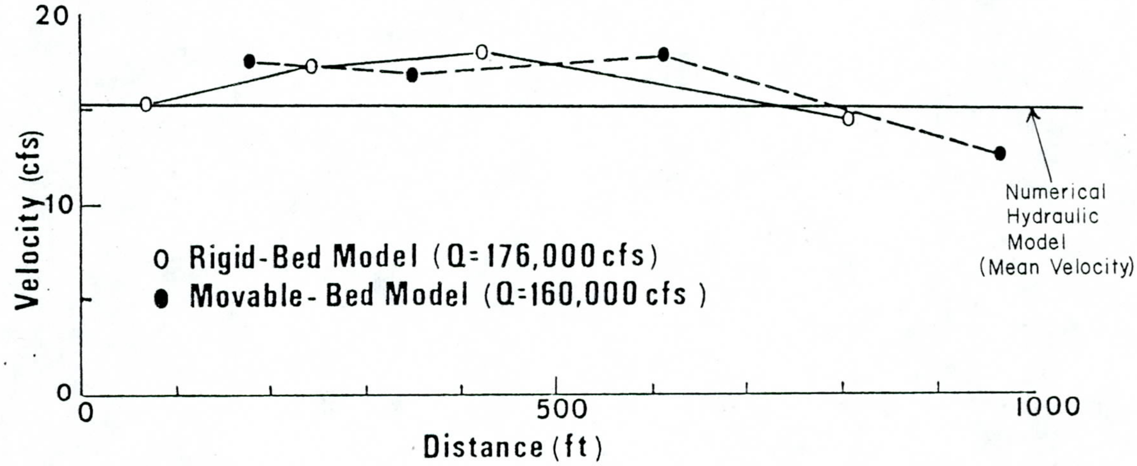


Fig. IX-6 Comparison between Velocities Station 38.5
(Q = 160,000 - 176,000 cfs)

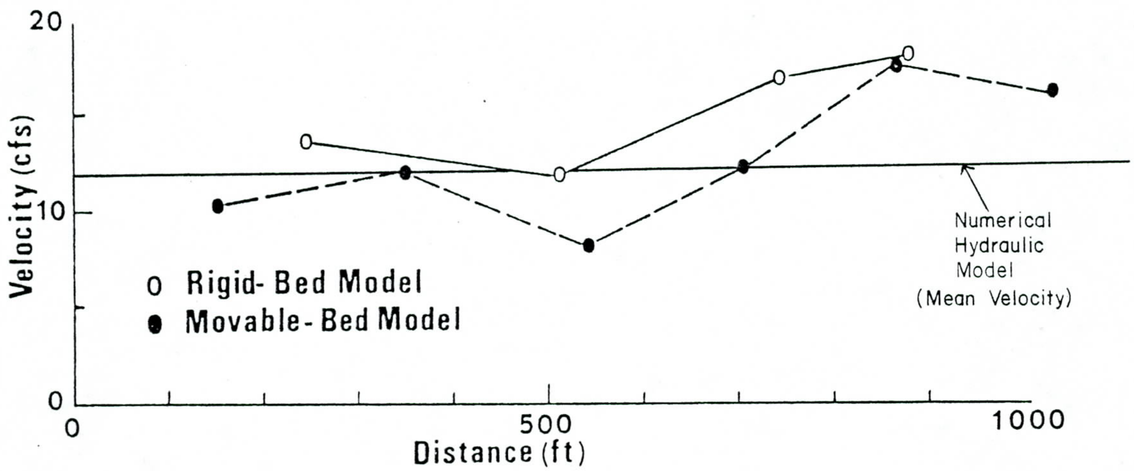


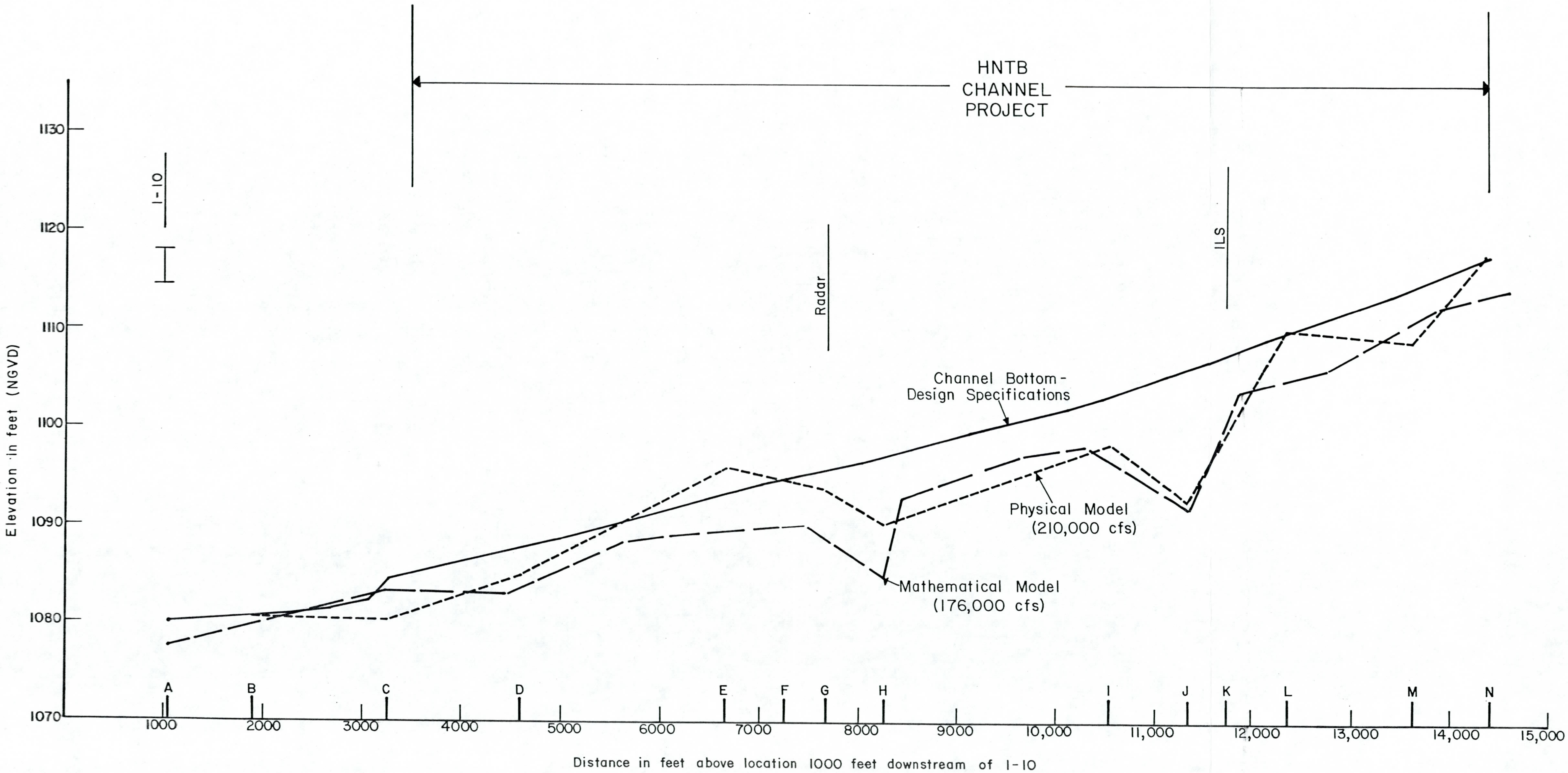
Fig. IX-7 Comparison between Velocities at Station 55.4
(Q = 210,000 cfs)

IX-8 shows estimated profiles of the minimum bottom elevations predicted by the numerical model run for the 176,000 cfs event and by the physical model run for the 210,000 cfs event. Channel bottom design specifications have also been plotted to allow an evaluation of the predicted changes resulting from the floods. The model results show quite good agreement considering the complexity of the sedimentation process and the differing discharges. Precise predictions of the magnitudes of scour and deposition are not possible with either modeling approach, but the comparison presented here reflects the abilities of these models to generate useful quantitative data. The three-dimensional physical model is more valuable than the one-dimensional numerical analysis in allowing the prediction of the sites and extents of localized scour or deposition. In these localized areas, the numerical analysis was dependent upon engineering judgement and hand calculations to augment the one-dimensional nature of the initial computer model predictions.

In evaluating the structural stability of specific segments of the channel project, the three-dimensional framework of the movable-bed model results is of major interest. Figures IX-9 through IX-12 show the changes in the channel geometry during the 92,000 cfs run, while Figures IX-13 through IX-16 show changes for the 210,000 cfs run. The last four figures show, in addition, estimates of local and general scour made by SLA for the same cross-sections. Four cross-sections are used to document scour and deposition at the most critical points in the channel.

In every cross-section shown and for both events, significant scour in the vicinity of a levee occurs. Depths of undercutting of the levees can be estimated from these

FIGURE IX-8. Minimum Bottom Elevations: Mathematical and Physical Model Results



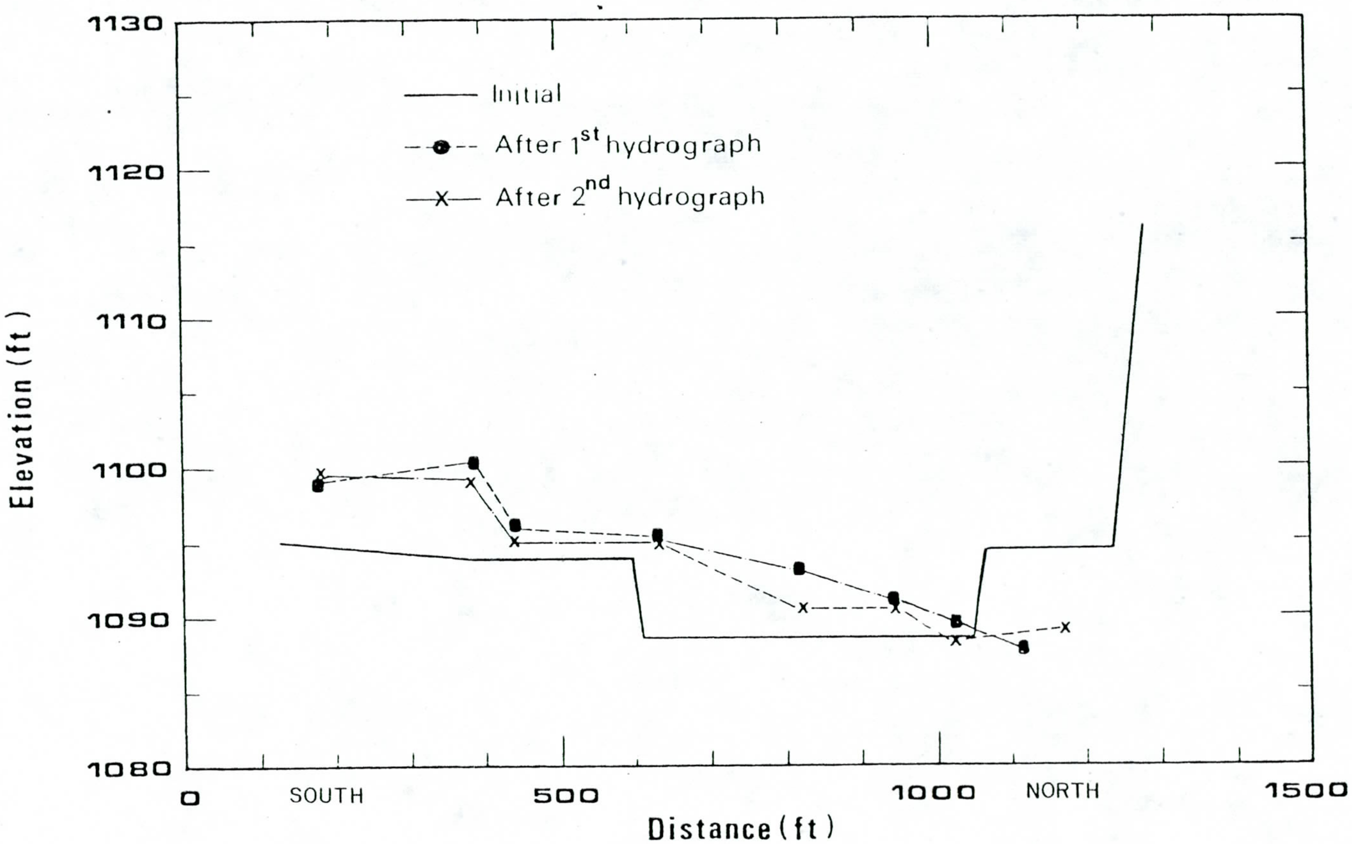


Figure IX-9 Bed Elevation Changes at Station 21.5 after Routing the 92,000 cfs Hydrographs

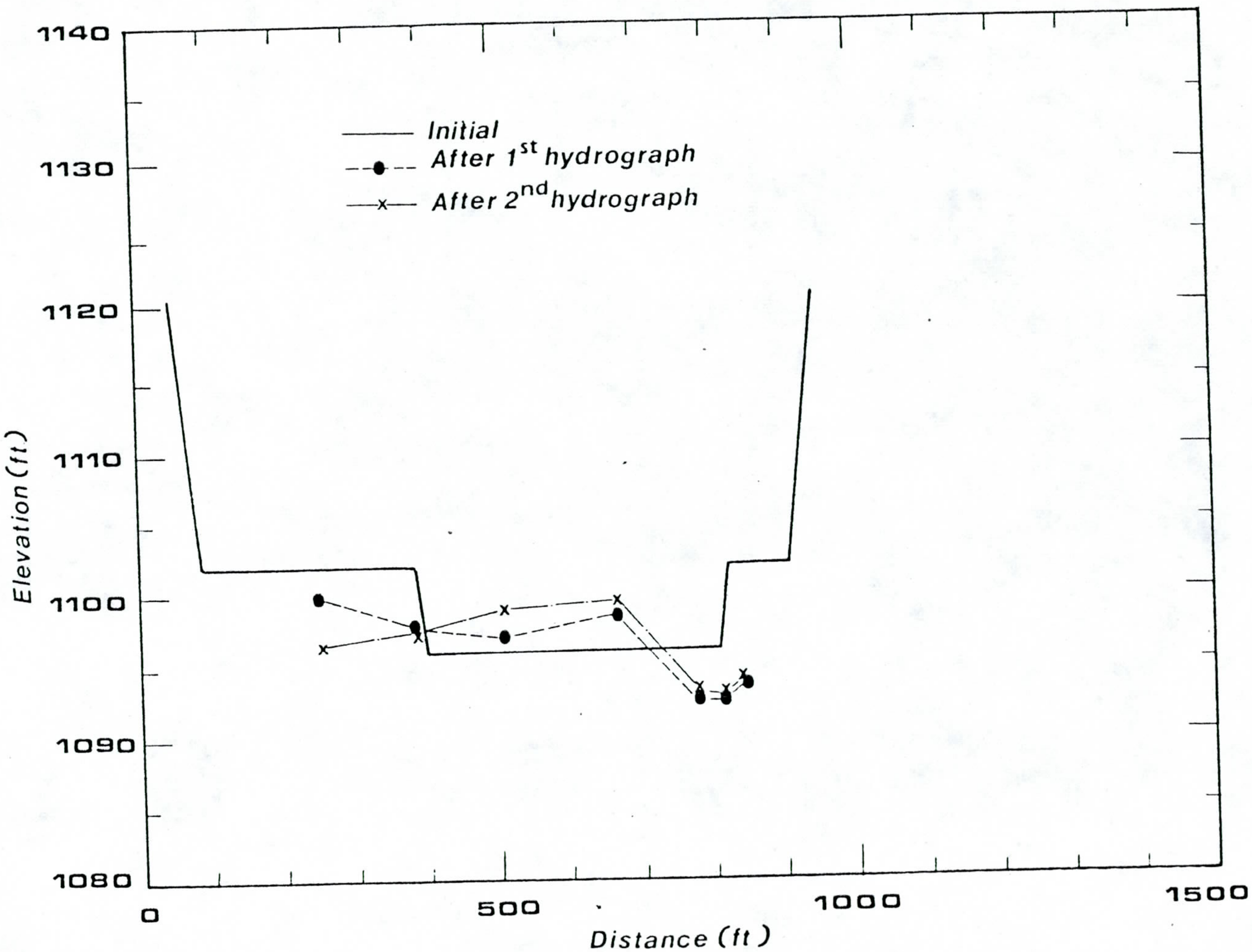


Figure IX-10 Bed Elevation Changes at Station 37.6 after Routing the 92,000 cfs Hydrographs

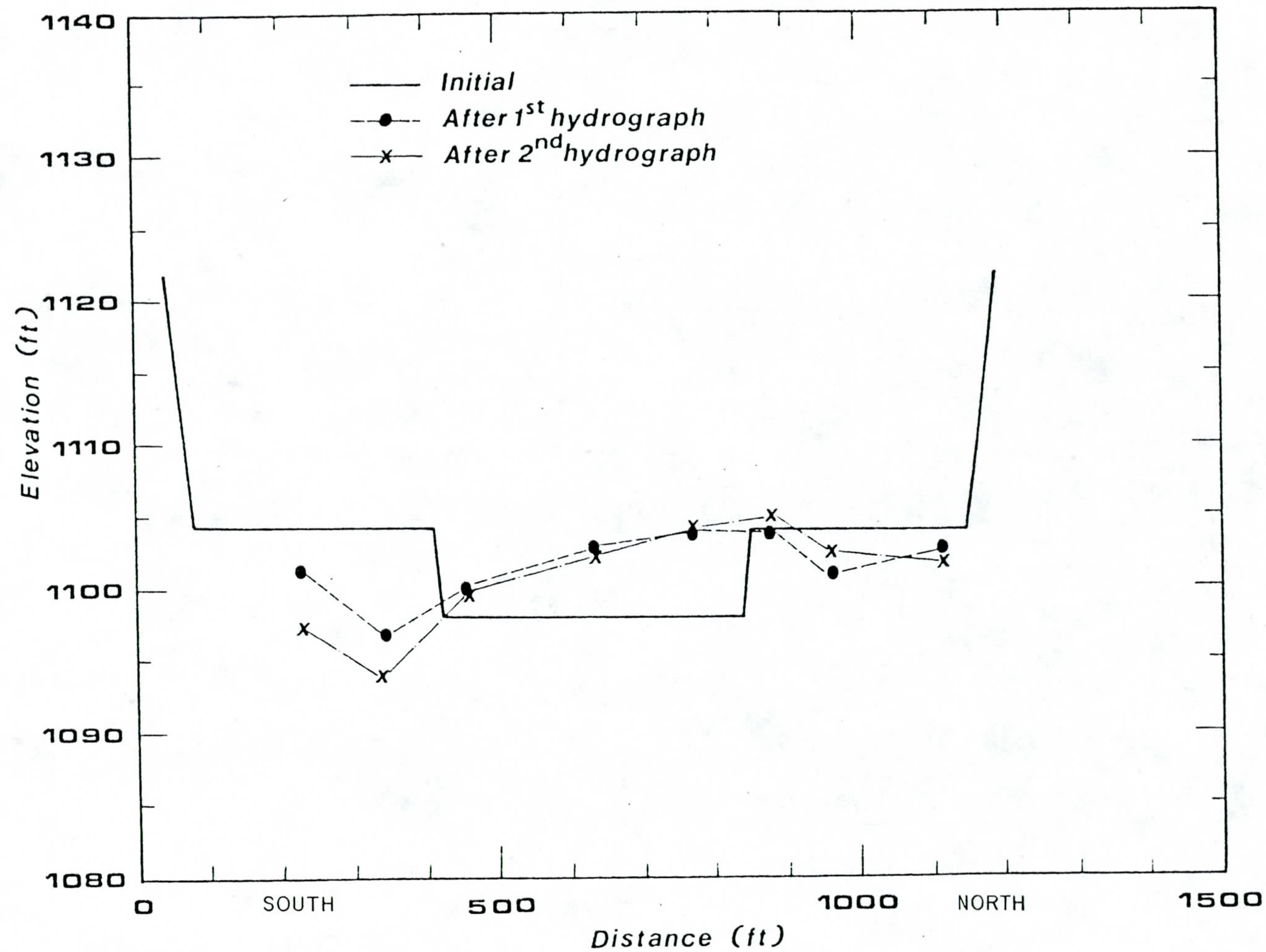


Figure IX-11 Bed Elevation Changes at Station 42.5 after Routing the 92,000 cfs Hydrographs

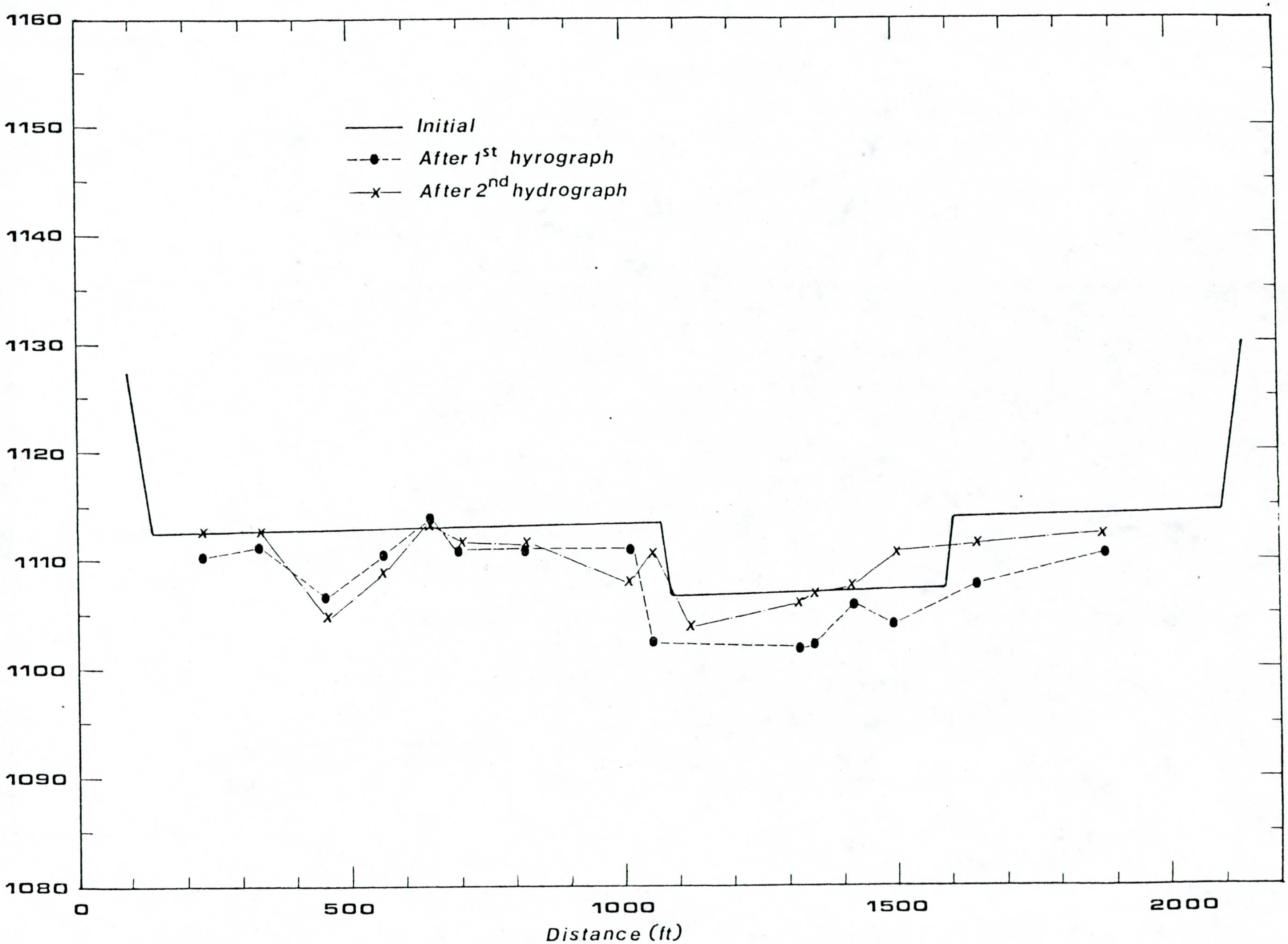


Figure IX-12 Bed Elevation Changes at Station 58.4 after Routing the 92,000 cfs Hydrographs

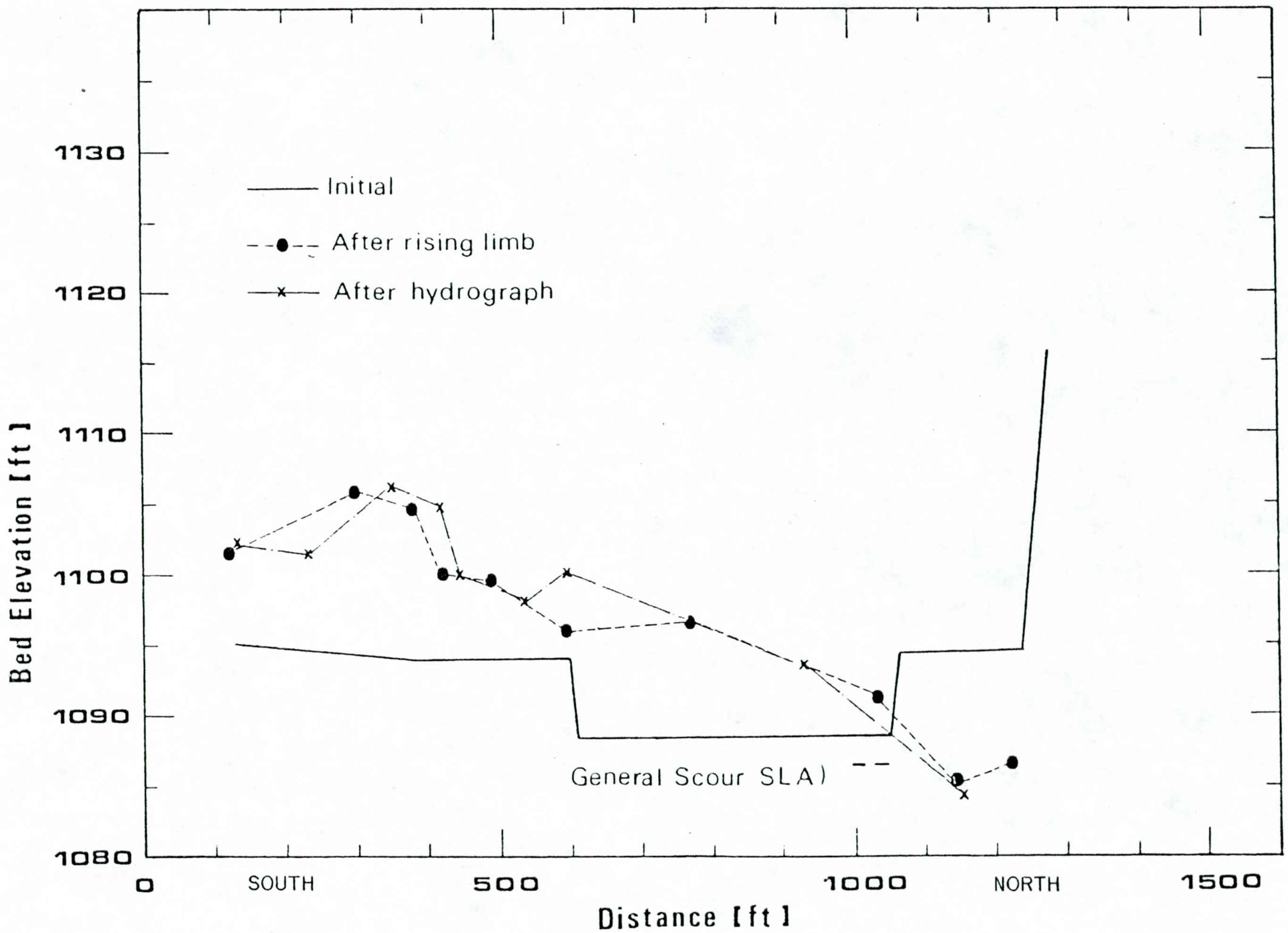


Figure IX-13 Bed Elevation Changes at Station 21.5 after Routing the 210,000 cfs Hydrograph

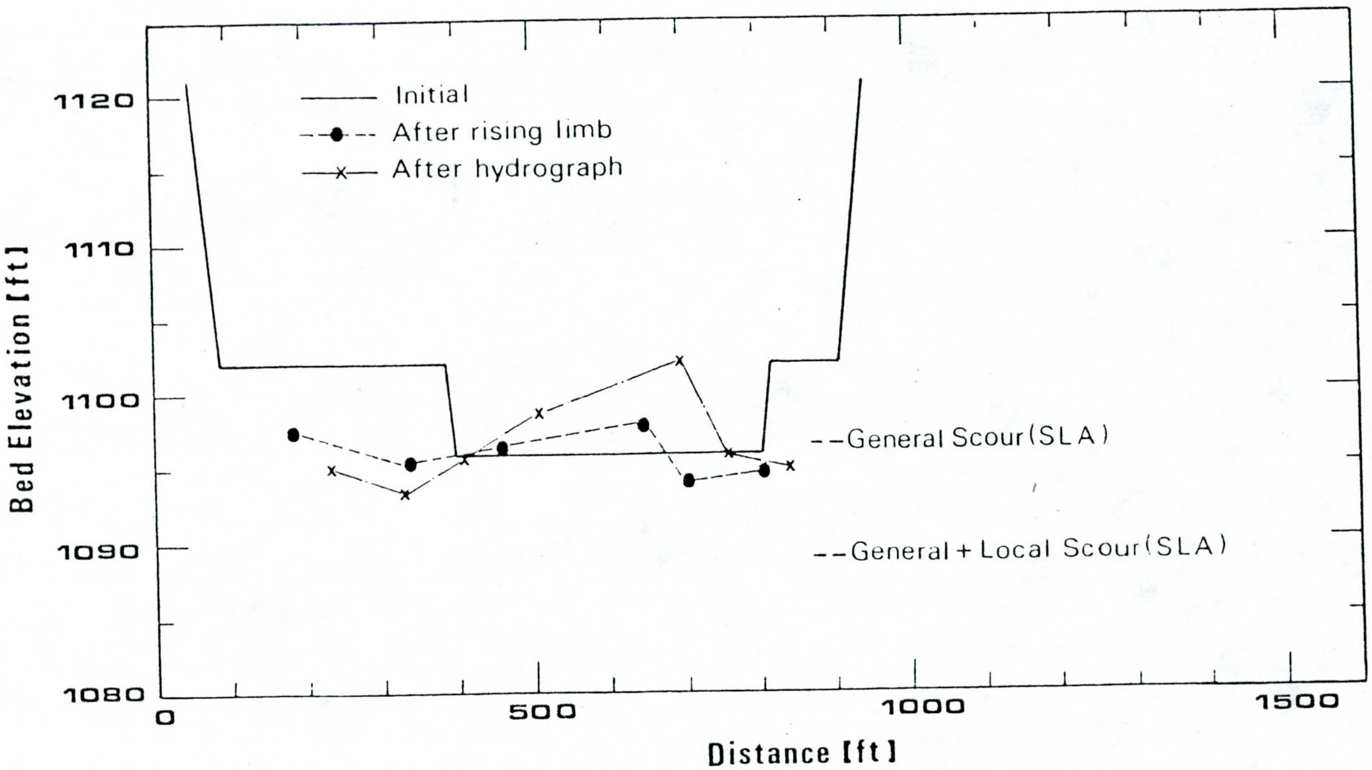


Figure IX-14 Bed Elevation Changes at Station 37.6 after Routing the 210,000 cfs Hydrograph

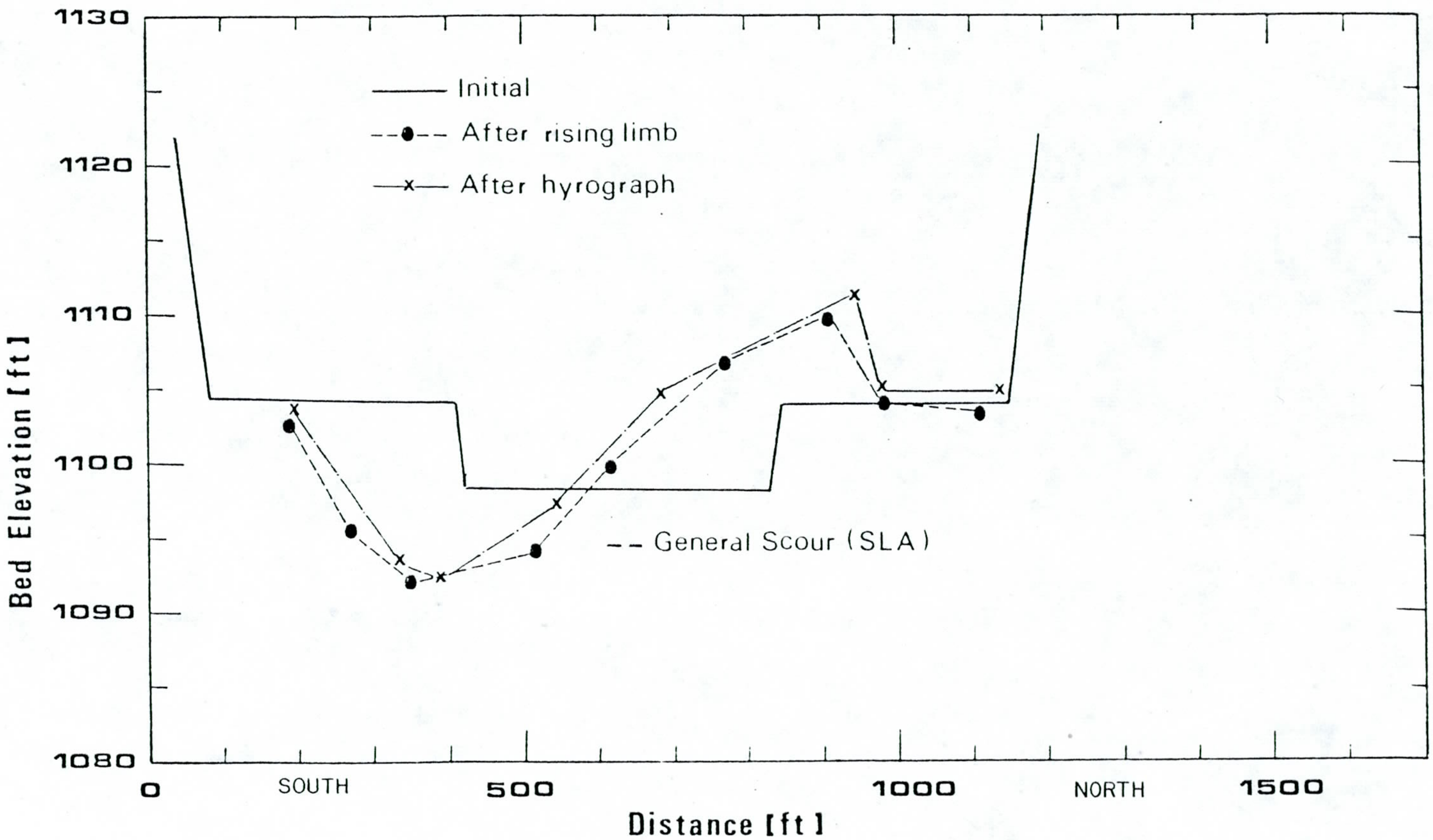


Figure IX-15 Bed Elevation Changes at Station 42.5 after Routing the 210,000 cfs Hydrograph

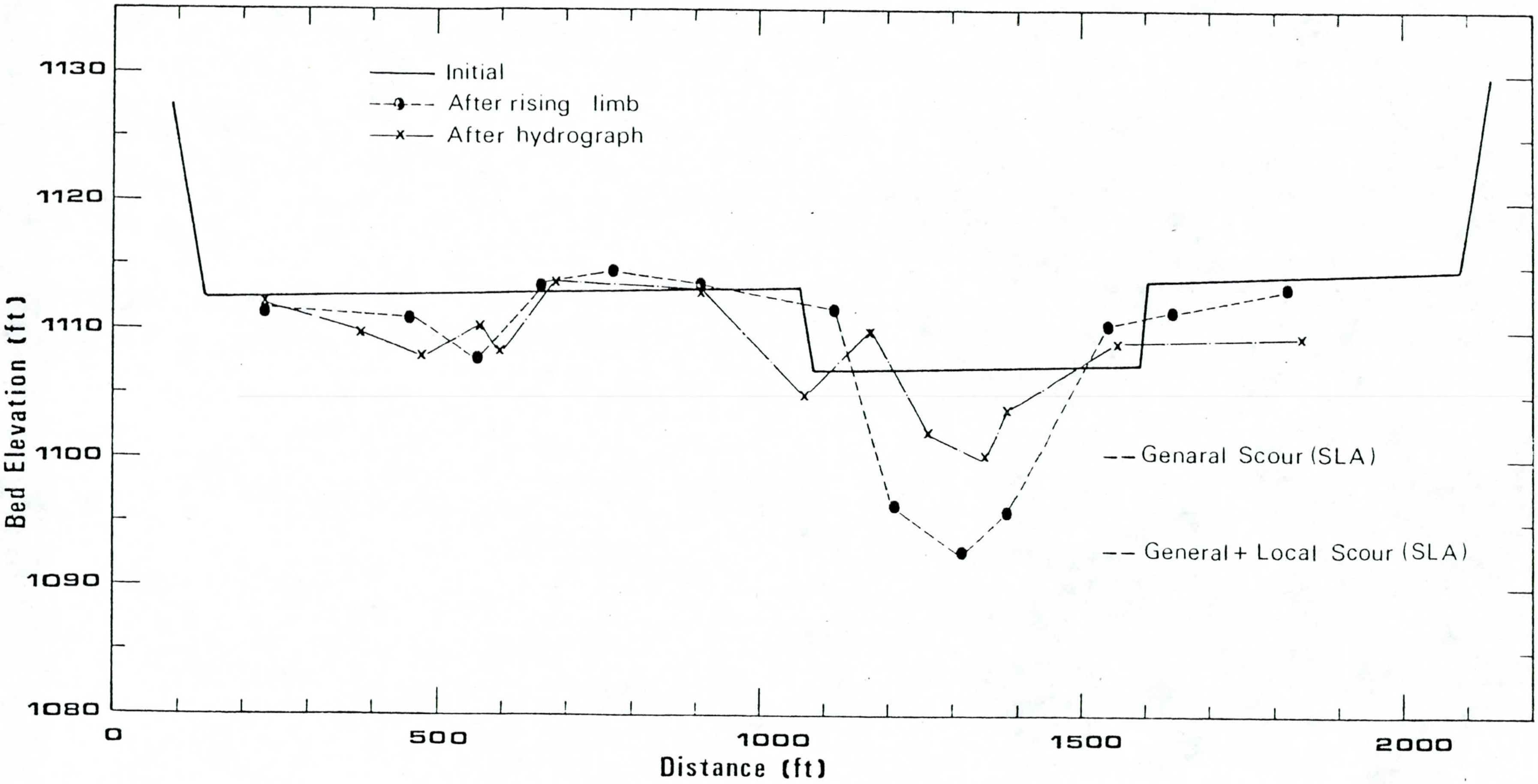


Figure IX-16 Bed Elevation Changes at Station 58.4 after Routing the 210,000 cfs Hydrograph

cross-sections, as summarized in Table IX-2. In most cases, the scour holes are immediately adjacent to the levee, but in the cases indicated the scour hole is some distance from the levee face. The uncertainty associated with the physical model results and the need for conservative estimates of scour hazards require that we allow for the possibility of scour hole migration. Hence, these depths represent estimates of scour hole depths after lateral migration.

TABLE IX-2 Scour and Undercutting at Levees

Model Coordinate	Station (ft)	Levee	92,000 cfs Depth (ft)	210,000 cfs Depth (ft)
21.5	4700	North	6.3	9.2
37.6	7600	North	8.3	7.3
		South	5.8	4.4
42.5	8400	South	11*	9*
58.4	11400	North	10*	17*
		South	7*	0*

* Note: these scour depths assume that the scour hole migrates laterally to impact the levee

X DAMAGE ESTIMATES

INTRODUCTION

The estimated damages and their locations were based on the results of the movable-bed runs assuming the channelization improvements were constructed in accordance with the design proposed by the City of Phoenix as presented in HNTB's design drawings dated September 8, 1980.

Final construction specifications and specific details of the proposed channel improvements were not available for review during this evaluation. The more general plans and typical sections were, however, and it was assumed that good construction practice and appropriate material requirements would be specified for the improvements.

Estimates of damage were prepared for storms with recurrence intervals of 10 years (92,000 cfs) and 100 years (210,000 cfs). Damages caused by the 92,000 cfs event were assumed to also occur at some time during the 210,000 cfs event. Damage estimates evaluated were subdivided into the following three categories:

1. Damages caused by modifications to the channel bottom
2. Damages to channel levees
3. Damages to protected facilities

Due to the limitations of the study scope, the estimated costs of damage are approximate only and are presented merely to provide a reasonable guide as to the order of magnitude of damages that could be expected to be caused by the 10 and 100 year floods. Since the costs of damage are to be used as a yardstick to measure the justification for increased improvement costs and compared to the proposed construction costs, we elected to use those unit prices being used by HNTB in their construction cost estimates so that a valid comparison could be made. The unit prices are shown in Table X-1 below and were reviewed and found to be in reasonable agreement with recent construction bids in the area.

TABLE X-1

ESTIMATED UNIT CONSTRUCTION COSTS

Channel Excavation	$1.25/CY
Placing Compacting	$0.50/CY
Gabions	$50/CY
Filter Fabric	$0.25/SF*
Rock Slope Protection	25/CY*

*ANCo Estimate

The estimated costs for the three categories of damage are presented in the following paragraphs.

MODIFICATIONS TO CHANNEL BOTTOM

Major amounts of scour and aggradation occur within the channelization section from both the 10-year and 100-year floods. Since it is our understanding that the City of Phoenix intends to maintain the channel in its designed configuration, it was assumed that the channel would be reconstructed to its design elevations after each major storm event. The cost of such channel reshaping was estimated to equal the damages anticipated to the channel bottom.

The amount of scour and deposition was computed based on the results of the various movable bed model runs. It is estimated that after the 10-year flood event occurs, approximately 490,000 cubic yards of material would be needed to fill the channel back to its design configuration. Of this amount approximately 275,000 cubic yards could be obtained within the project limits from areas aggraded by the flood. The remaining 215,000 cubic yards, which constitute the net material loss from the reach, would have to be imported from adjacent reaches of the river.

It is estimated that the cost of reshaping the channel bottom to its design configuration after the 10-year event would be approximately $850,000.

The 100 year flood is estimated to aggrade approximately 650,000 cubic yards of material in the channel reach with an almost equal amount of scour. The cost to reconstruct the channel bottom after the 100 year event is estimated to be approximately $1,100,000.

A second element of potential damage caused by channel bottom scour is related to the 60-inch water main and 69-inch Four City Sanitary sewer which traverse the river bed. The results of the physical model runs indicate that scour pits will be formed in close proximity to the two pipes where their crowns are within three feet of the proposed channel bottom. Although it cannot be stated with certainty that such scour would damage the pipes, considering the dynamics of the flood induced scour, it is our judgment that the pipes could be exposed during the 10 or 100-year event if not protected. HNTB drawings indicate that protection is planned for the 60 inch waterline. We have not included any costs for potential damage to these pipes.

DAMAGES TO CHANNEL LEVEES

Estimated damages to the channel levees were based on two primary factors, derived from the physical model studies; the velocity of flow immediately adjacent to the levees, and the amount of scour and undercutting experienced in the model at the inboard toe of the levees. Secondary factors of piping, seepage and consolidation of levee foundations were also taken into account in determining the potential damage.

The degree and location of the estimated levee damages are shown on Figures X-1 and X-2 for the 10 year and 100 year floods respectively. The assumptions and rationale used in arriving at these estimated damages are summarized in Table X-2 and discussed in the following paragraphs.

Although presented separately herein, damage conditions were often found to occur concurrently at various sections of the levees during the model runs. This factor was recognized in deriving the extent of estimated damages.

Condition 1. It was assumed that only normal maintenance would be required when boundary velocities do not exceed 5 feet per second. The median size material of which the levees are to be constructed should resist erosion by velocities up to 5 feet per second.

Condition 2. It was assumed that boundary velocities between 5 and 10 feet per second would cause significant damage to unprotected levees. Velocities of 5 feet per second are expected to start eroding the median size material

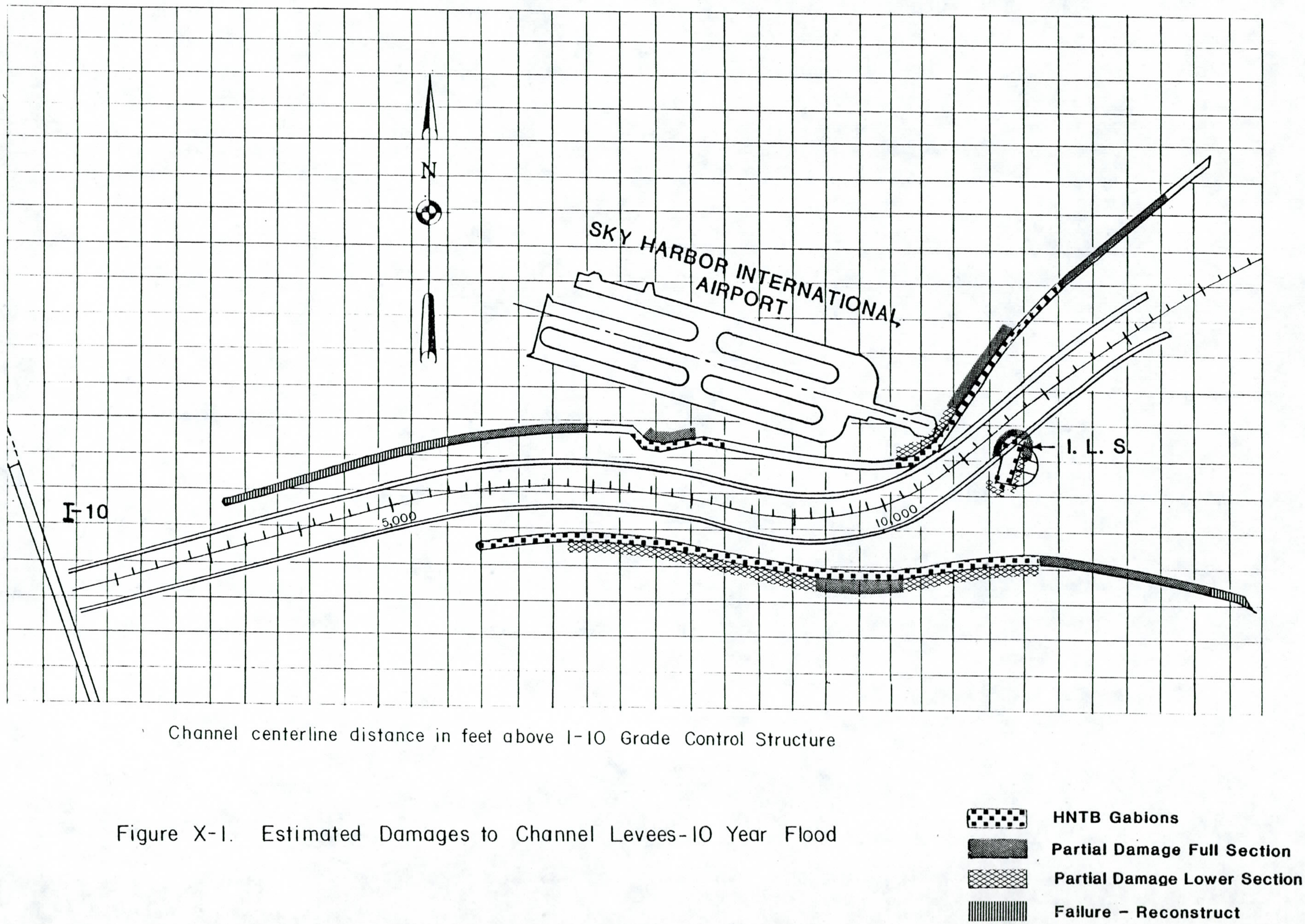


Figure X-1. Estimated Damages to Channel Levees-10 Year Flood

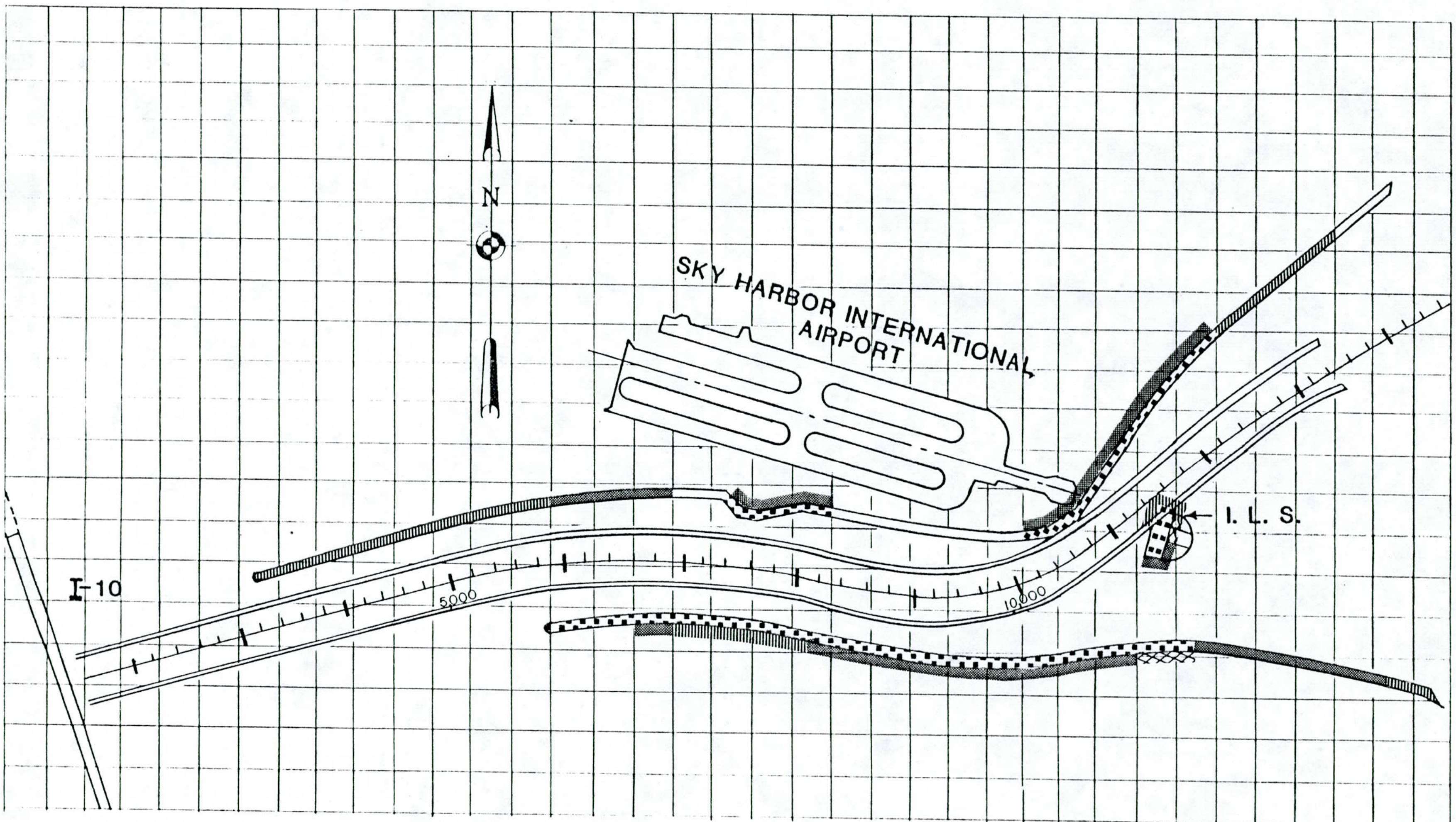


Figure X-2. Estimated Damages to Channel Levees - 100 Year Flood

TABLE X-2

BASIS OF ESTIMATED LEVEE DAMAGES

Damage Condition	Levee Type	Factor	Estimated Damage
		Velocity @ Levy Face, fps	
1	Unprotected	0-5	No damage-normal maintenance
2	Unprotected	5-10	Severe erosion-repairs req'd
3	Unprotected	>10	Failure-reconstruction req'd
4	Protected by Gabions	0-5	No damage-normal maintenance
5	Protected by Gabions	5-10	Damage to lower sections of gabions from bed load impact-repairs required
6	Protected by Gabions	10-15	Damage to gabions and Levee repairs required
7	Protected by Gabions	>15	Failure-reconstruction req'd
		Bottom Scour @ Levee Toe, feet	
8	Unprotected	0-5	Partial failure-repairs req'd
9	Unprotected	>5	Failure-reconstruction req'd
10	Protected by Gabions	0-5	No damage-normal maintence
11	Protected by Gabions	5-10	Partial failure-repairs req'd
12	Protected by Gabions	>10	Failure-reconstruction req'd

from the levee. Velocities of 10 feet per second have the ability to erode even the maximum size material from the levees.

Damages were assumed to be equivalent to 25% of the cost of constructing a new levee when the design levee is in fill and 10% of the cost when in cut. This differentiation between field conditions was made in recognition of not only the increased potential for levee weakening and localized failure due to erosion and subsequent slumping when the levee is in fill, but also the increased potential for damage to fill sections due to piping and loss of foundation support.

Condition 3. It was assumed that unprotected levees subjected to velocities in excess of 10 feet per second would breach, and the cost of damages would be equal to 100% of the cost of constructing a new levee. This condition was found to occur only where the unprotected levees were also subject to a fairly frontal attack by the flood flows.

Condition 4. It was assumed that no damage would occur to protected levees when velocities are less than 5 feet per second.

Condition 5. Due to the characteristics of the bed load material anticipated during these storm events, it was assumed that the gabion apron and the bottom two feet of the gabion slope protection would be damaged by the impingement of river rocks on the gabion cages with velocities of 5 to 10 feet per second.

It was assumed that under these conditions some of the cages would be broken and some of the enclosed rocks displaced and transported downstream. The loss of such material would also cause slumping and partial failure of adjacent cages and levee. It was assumed that the cost of damages would be equivalent to 25% of the cost of constructing the lower section of the gabion mattress and apron.

Condition 6. Based on our literature search and discussions with suppliers of prefabricated gabion cages, gabions do not seem to have been used extensively where velocities exceed 10 feet per second. This fact, together with known damages under less severe conditions and the heavy bed load anticipated, led us to assume that when boundary velocities in the range of 10 to 15 feet per second were expected, partial failures of the cages would occur. It was also assumed that such partial failure would result in loss of the levee itself. It was assumed that the cost of damages would be equivalent to 25% of the cost of constructing the impacted section of gabions and levee.

Condition 7. No information was discovered to demonstrate a service history of gabion mattresses where velocities exceed 15 feet per second or even that gabions have ever been used under these conditions. A supplier of prefabricated cages, however, suggested that an 18-inch deep mattress could possibly work under velocities of 15 to 18 feet per second. (The proposed matress for slope protection is 12 inches deep). Other literature indicates that when subjected to high velocities, gabions should receive a protection concrete or asphalt capping. It is also thought that the proposed 250 foot spacing of intermediate buttresses would be inadequate to prevent significant damage to the slope protection mattresses when severe velocities, bedload impact and scour are expected. Reference data, including data from the U.S. Corps of Engineers, the Water and Power Resources Service, CALTRANS and the Bureau of Public Roads, indicates that stone riprap anywhere from 2 feet to 4 feet in equivalent spherical diameter would be required to protect against velocities of this magnitude. It was judged that complete failure of the gabion armor would occur under these conitions with a resultant failure of the levee. It was therefore assumed that the cost of damages would be equivalent to 100% of the cost of constructing a new section of armored levee where these conditions were expected to occur.

Condition 8. It was assumed that where bottom scour from 0 to 5 feet is expected to occur adjacent to an unprotected levee, partial damage to the levee would result. Since the levees are planned to be constructed of coarse, granular materials with little or no fine binder material, it was judged that any undermining of the levee toe would result in slumping of the channel side slope and partial collapse of the levee. Similar to Condition 2, damages were assumed to be equivalent to 25% of the cost of constructing a new levee when the levee is in fill and 10% of the cost when in cut.

Condition 9. In those locations where scour at the toe of unprotected levees is predicted to exceed 5 feet, it was assumed that such undermining would cause progressive deterioration of the levee. In combination with the erosive velocities anticipated, effective collapse and failure to the levee could be expected. It was assumed that the cost of damages in these instances would be equivalent to the cost of constructing a new levee.

Condition 10. Where scour of between 0 and 5 feet is predicted at the toe of a protected levee, it was assumed that the gabion apron would protect the levee from damage and that only normal maintenance would be needed.

Condition 11. The proposed design provides a gabion mattress apron extending 9 feet out from the face of the levee. It was assumed that when scour exceeds 5 feet, at which point the hinged apron would extend downward at the approximate slope of the levee, partial undermining of the gabion apron would occur resulting in damage to the gabions and levee.

It was assumed that the cost of damages would be equivalent to 25% of the construction cost of the gabions plus 25% of the construction cost of the levee in fill or 10% of the cost in cut.

Condition 12. It was assumed that when the scour at the levee exceeded 10 feet, at which point the hinged apron would be completely unsupported, the gabion protection and levee would be undermined to the point of collapse. It was assumed that the cost of damage in this case would be equivalent to the construction cost of a new armored levee section.

The estimated cost of damages to the channel levees due to the anticipated conditions throughout the length of the proposed project are summarized below in Table X-3 for the 10-year and 100-year events.

TABLE X-3

ESTIMATED DAMAGES TO CHANNEL LEVEES

Flood Event	Levee	Damage Estimate		
		Levee	Gabions	Total
10 year	North	$345,000	$ 80,000	$425,000
	South	45,000	140,000	185,000
	TOTAL	$390,000	$220,000	$610,000
100 year	North	$450,000	$ 90,000	$540,000
	South	140,000	260,000	400,000
	TOTAL	$590,000	$350,000	$940,000

DAMAGES TO PROTECTED FACILITIES

Facilities proposed to be protected by the channel improvements include the ILS and radar installations, the Sky Harbor Airport and surrounding private properties. The ability of the proposed channel improvements to adeqately protect these four elements and the potential for damage are discussed in the following paragraphs.

ILS Installation It was estimated that sufficient freeboard will be provided at this facility to avoid overtopping during the 10 and 100- year floods. During the 10-year flood, partial damage to the north face of the embankment is expected together with damage to the lower sections of gabions on the south face due to impingment of bedload rocks on the wire baskets under high velocity. Under the 100-year flood, high velocities and deep scour undercutting are expected along the north face to an extent that failure of the north face would occur. This could cut the embankment back to a point where the structure foundations are exposed. It was assumed that the structures are founded on deep piles such that the building integrity would not be jeopardized. Partial damage to the south face is expected with resultant slumping and loss of embankment material.

For the 10-year flood, damage to the north face was estimated to be equivalent to 25% of the cost of the gabion protection plus the cost of replacing an equivalent of a 5 foot section of the north end embankment for its full height. Damage to the south face was estimated to be equivalent to 25% of the cost of the gabion apron and the lower 2 foot section of the gabions on the embankment face.

For the 100 year flood, damage to the north face was estimated to be equivalent to the full cost of constructing the gabion protection plus the cost of replacing an equivalent of a 15 foot section of the north end embankment for its full height. Damage to the south face was estimated to be equivalent to 25% of the cost of the gabion protection plus the cost of replacing an equivalent of a 5 foot section of the south end embankment for its full height.

The estimated cost of damage at the ILS site was determined to be:

Flood Event	Estimated Damage
10 year	$10,000
100 year	$40,000

Radar Site It was judged that the radar site would be adequately protected from flooding by the 10 and 100 year floods. Damage would occur to the protecting channel levees due to scour and high velocity erosion during both the 10 and 100 year floods. The estimated cost of this damage was included in the damage estimates of the main channel levees set forth above.

Sky Harbor Airport No damage is expected to occur to the airport from Salt River flooding during the 10-year event. During the 100-year flood, however, it is anticipated that the unprotected north levee upstream of the airport will breach and flood waters will inundate major portions of the airport property. If the breach occurs, the anticipated areas of flooding and the direction of flow are presented in Figure X-3.

The proposed improvements to the south runway/taxiway complex include raising the finish grades at the east end by as much as 10 to 15 feet. This is sufficient to avoid

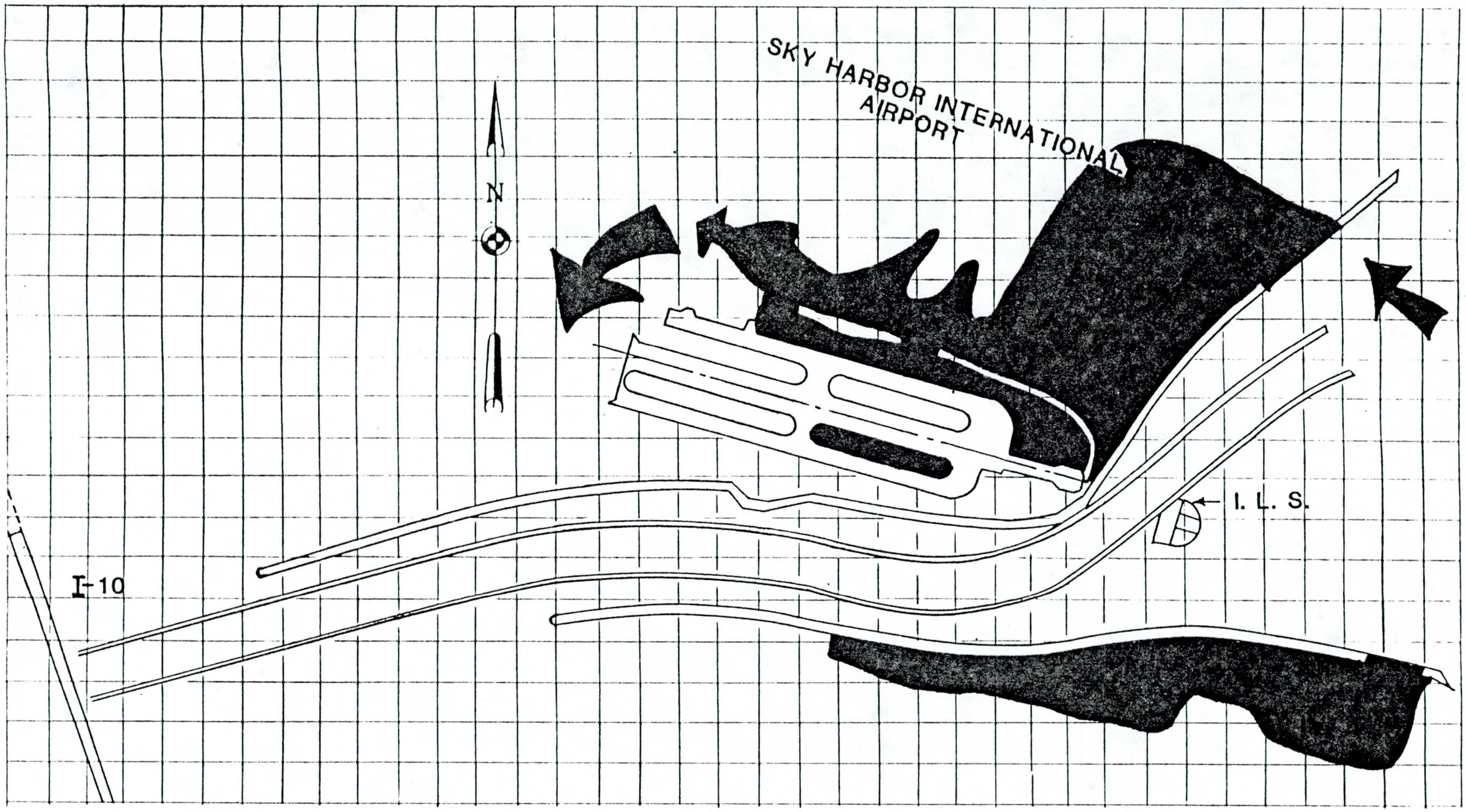


Figure X-3 Areas of Inundation Due to Potential Levee Failure- 100 Year Flood

Areas of Ponding

Flow Trend

flooding of the pavement areas but effectively forms a dam for any floodwaters coming through an upstream levee breach. Flooding is expected to occur as described below.

The breach is expected to occur approximately 1000 feet northeast of the end of the runway where the water surface in the channel would peak at about elevation 1128. Floodwaters would flow overland, cross 40th Street and continue to the perimeter road. The low areas of the northeast would be flooded and water would overtop the perimeter road at its low point (El. 1118.5) and spread out, flooding the low areas bounded by the perimeter road, Taxiway C, the runway and the access taxiway to the hangar areas. Depths of flooding in the open areas could exceed 10 feet in places, but the paved taxiways and runway should be high enough to avoid inundation.

The inlet to the double barrelled 72 inch storm drain between Taxiway C and the perimeter road would be submerged by up to 10 feet of water. Water would backflow through the connected inlet in the infield between Taxiway D, cross Taxiway D-7 and the runway, flooding that open area. Certain of the flow would pass through the 2 - 72 inch pipes to the river downstream where the water surface is lower, but would provide little relief to the temporary flooding problem.

Water would continue to pond and rise in elevation until an elevation of approximately 1122 was reached at which point it would flow northwesterly toward the hangar/maintenance/ terminal area. There is insufficient data to determine the specific damages that would occur in this area, but the flood waters can be expected to sheet flow through the area, causing significant disruption and damage, until it could reenter the river downstream.

Although the inundation of the open areas of the airport should cause no special damage, the potential for flooding to the hanger/maintenance/terminal area cannot be allowed. The northeast levee must be armored to satisfactorily contain the floodwater and obviate airport flooding.

Surrounding Private Properties Breaching of the unprotected upstream levee on the south side of the channel's entrance transition is expected under both the 10 and 100 year floods. (Refer to Figures X-1 and X-2). The private property expected to be flooded on the south side of the channel is shown in Figure X-3 and is more or less bounded by 40th Street on the west and an unnamed access road to the south. This area is undeveloped and no damage to permanent structures is anticipated from either the 10 or 100 year flood.

XI. COMPLIANCE WITH REGULATIONS

National Flood Insurance Program regulations prohibit encroachments on the floodway that increase flood levels (Section 1910.3, (d) (3)) and require that a floodway be designed to carry the 100-year flood with no more than a one foot rise in water surface elevation at any point (Section 1910.3 (d) (2)). Figure IV-3 shows that increases in water surface elevation as predicted in the HEC-2 analysis occur due to the channelization project only in the reach upstream of the ILS. The fixed-bed model results, as shown in Figure VI-3, indicate very similar results. In no case does the increase exceed 1 foot, which is considered to be within the uncertainty of the results. Hence, the proposed project will be in compliance with these regulations.

A number of federal regulations deal with avoidance of repetitive damages to facilities which are located in the flood plain. Section 1910.5 (a) (3) of Title 24, Chapter X, Subchapter B (National Flood Insurance Program) requires that adequate protective measures be taken so that existing erosion hazards will not be aggravated. Federal agency compliance guidelines for Executive Order 11988 (Item 5) require that an agency must mitigate hazards by protecting structures located in the flood plain. FEMA regulations on Disaster Assistance and Hazard Mitigation (44 CFR Part 205, Subpart M, Section 205.407(d)) state that the regional director may decline authorization of FEMA assistance to restore facilities if such facilities are subject to repetitive damages.

As discussed in Section X, substantial damages to the channel will occur for both the 10-year (92,000 cfs) and 100-year (210,000 cfs) events. Even for the 10-year event, damages will exceed $1.4 million, which is a substantial percentage of the total channel construction cost. Damages to the channel caused by a 100-year event are in excess of $2.0 million. While there is considerable uncertainty in these damage estimates, it is clear that the potential for serious repetitive damage is great. The return period for serious damage to the channel is probably shorter than 10 years. It is reasonable to expect that average annual damage estimates would show significant losses. There is, as a result, little doubt that the channel design is not in compliance with repetitive damage clauses of the regulations.

XII. RECOMMENDED MODIFICATIONS TO PROPOSED CHANNEL DESIGN

BASIS FOR RECOMMENDATIONS

The design modifications proposed in this section are intended to be one set of candidate measures which have the potential to prevent the damages to channel and surrounding properties which are described in Section X. Certain modifications, such as increases in levee heights, are intended to bring the channel design into conformity with standard practice. Detailed analysis and design efforts would be required to verify their applicability and effectiveness. Likewise, the estimates of costs associated with these design modifications are rough and are intended for comparison purposes only.

The evaluation and recommendation process proceeded as follows:

1. Estimates of type, location, and extent of damages were developed for the 10-year and 100-year events.
2. Comparisons between predicted channel behavior and standard design practice, as detailed by Corps of Engineers manuals, were made.
3. The levee and channel designs were analyzed from geotechnical and structural viewpoints for stability and factor of safety.
4. Where deficiencies were revealed, alternative designs were proposed as needed to correct these deficiencies.

Model and geotechnical studies revealed the need for a number of design changes to meet the requirements of FEMA regulations, as follows:

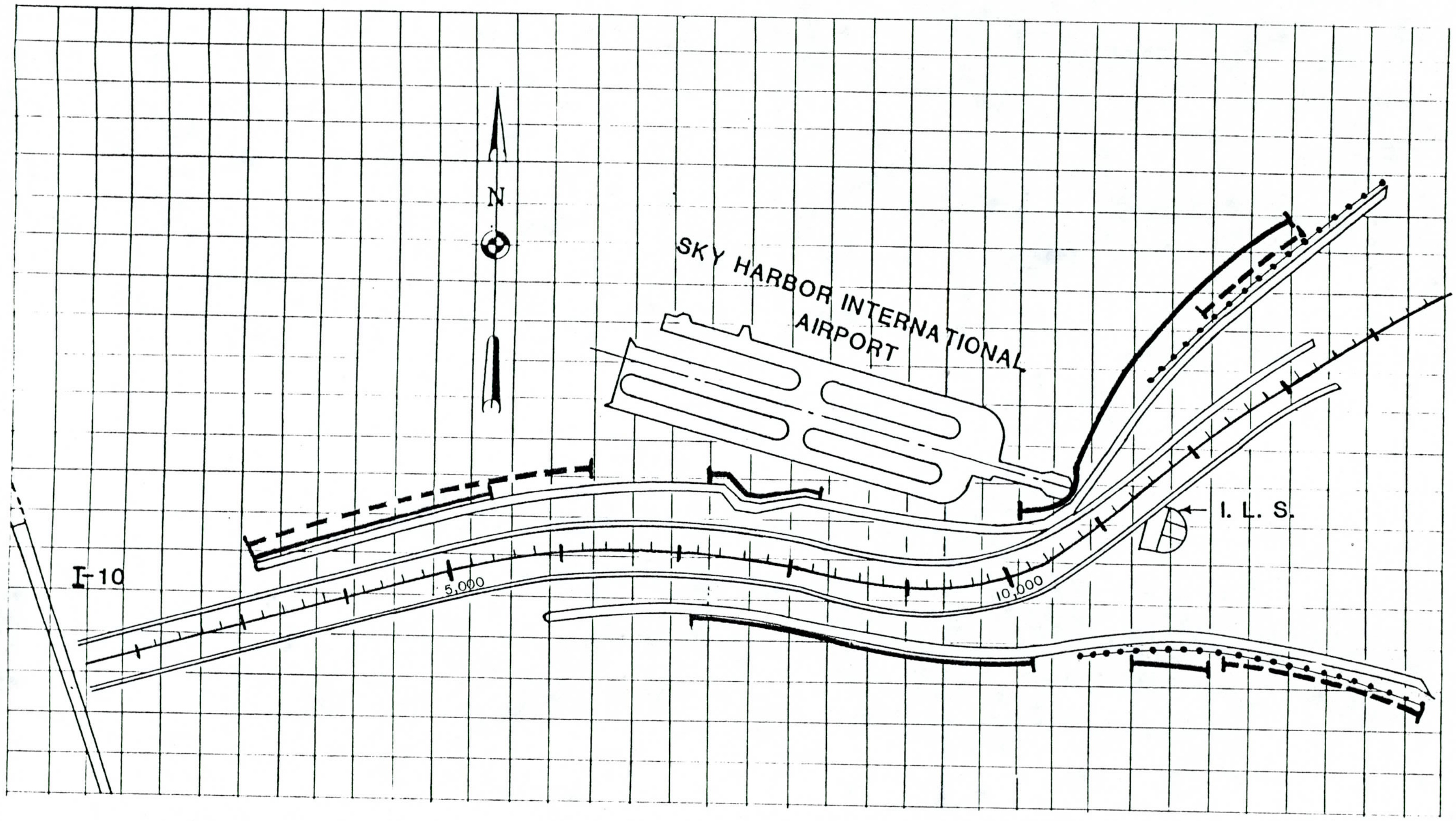
1. increase levee freeboard;
2. extend levee areas protected by gabions;
3. protect gabions against damage from rock impacts;
4. bury gabions in areas of significant scour
5. provide means to reduce piping through levee foundations ;
6. construct channel bottom to shape of ultimate channel equilibrium condition.

Each of these issues will be discussed separately and then cost estimates for proposed modifications will be presented at the end. Figure XII-1 shows the locations of site specific modifications.

It is important to note that severe economic constraints were major factors in determining the design proposed by the City of Phoenix. Both the City and HNTB have stated that the channel was not designed to be free of damages after major flood events. However, regulations under which FEMA provides reconstruction funds require that funded structures not be subject to repetitive damages. Consideration of changes to the channel design is motivated by this requirement.

LEVEE HEIGHTS

The Corps of Engineers design manual for flood control channels recommends a minimum freeboard of 3 feet for earth levees. Special consideration of wave effects and superelevation is also recommended, especially where velocities are high. Levee slumping caused by undercutting and scour is also a potentially serious problem which could be partially mitigated by increased freeboards.



Channel centerline distance in feet above I-10 Grade Control Structure

Figure XII-I. Location of Proposed Design Modifications

Increase Levee Height

Levee Armoring

Increased Depth of Armor and Gabion Protection

From these considerations, freeboards should be no less than 3 feet in the proposed channel. The levee freeboards predicted by the fixed-bed model, as shown in Table VI-4, are well above 3 feet except for upstream of station 10,700. Table IX-1 shows the changes in stage due to sediment transport in the movable bed model. When the freeboards in Table VI-4 are adjusted for the movable-bed model results, the following sections of levees required added height to meet the 3 foot requirement.

Station (ft)	Levee	Approximate Height Increase (ft)
10,700 - 11,700	South	+ 1 ft
11,700 - End	South	+ 2 ft
11,700 - End	North	+ 1 ft

LEVEE PROTECTION

Extensive sections of the proposed levees will be subject to velocities over 15 ft/s during the 100-year event. A number of unprotected levee sections in the movable-bed model were severly scoured during this event and several sections would have failed if they were not artificially protected during the model run (see Figure VII-12). As discussed in Section X, leaving these levee sections unarmored would result in substantial damages even during the 10-year event. Failure of the north levee during the 100-year event could cause extensive flooding damage to airport facilities. Failure of the south levee would inundate sparsely developed land adjacent to the channel. The potential for progressive levee failure leading to the destruction of even armored levee sections is substantial.

Proper armoring of the levee sections identified in Figure VII-12 seems to be necessary to maintain the integrity of the channel. The following additional sections require protection.

Levee	Reach*-Start	Reach*-End	Length (ft)
North	3,200	6,200	3,000
North	12,600	14,000	1,400
South	12,400	14,200	1,800

*Distance

DESIGN OF LEVEE ARMORING

Significant scour of the channel bottom occurs during both the 10-year and 100-year events, as shown in Section VII. Figure IX-8 shows the predicted shift in thalweg elevations, while Figures IX-9 through IX-16 show the formation of scour holes near levee bases at selected cross-sections. Table IX-2 itemizes the areas of serious levee undercutting due to channel degradation.

In order to prevent levee slumping and/or failure during flood events, levee armoring should extend down to the depth of maximum scour at the levee base. The proposed design calls for 9 foot long hinged gabions which are intended to swing down as the levee toe is undermined. In several locations, predicted scour substantially exceeds 9 feet; hinged gabions in these areas would probably fail due to undercutting and lack of support. The following levee sections will need gabions buried to the approximate depths indicated.

Station	Levee	Depth(ft)	Length(ft)
3,200 - 5,700	North	9	2,500
7,000 - 10,000	South	10	3,000
7,500 - 8,200	North	8	1,000
10,000 - 12,000	North	17	2,000
12,000 - 14,000	North	10	2,000
11,000 - 12,000	South	7	1,000
North and South sides	ILS	17	500

When gabions are exposed to high velocities carrying gravel and cobbles, considerable damage to the baskets can occur. Rocks in the size range found in the Salt River (6-10 inches) moving at 15 ft/s and faster are capable of breaking basket wires and deforming baskets. Gabions that may be subjected to the bed load of the channel during a flood event should be protected by a layer of rip-rap or a coating of flexible material. We recommend that 2 foot diameter rip-rap be placed on the gabion face from the total scour depth up to 2 feet above the design bed elevation. All of the levee sections designated as needing buried gabions should also have this rip-rap protection.

Since the previously mentioned design modifications call for substantial increases in protected levee surfaces and also call for substantial amounts of added rip-rap armoring, consideration should also be given to using only rip-rap for levee protection. Rip-rap was recently selected by the successful bidder for armoring of the I-10 channelization. The results of the bidding process indicate that rock slope protection could be very competitive with gabions. The two alternatives should be included in bid documents so that contractors can make the selection.

LEVEE STABILITY

The levee face slope in the proposed design and other aspects of general levee design conform to standard practice and appear to be appropriate. The type of materials which will be used to make up the levees and the nature of the foundations for the levees are not optimal for such use. Two concerns regarding levee stability are important: (1) the potential for piping to occur underneath or through the levee during a flood event and (2) the possibility of "liquifaction" of the levee materials under fully saturated conditions.

Beds formed by rivers like the Salt River often have underlying or surface layers of clays, silts, or sands which are deposited on bars or elsewhere during the falling limbs of flood hydrographs. Such deposits are unconsolidated and scattered, but can occur with a fairly high frequency at shallow depths. The borings reported by HNTB indicate several such layers or "lenses." When a lens of fine material underlies a well graded, compacted levee which is experiencing a large hydraulic gradient (from inside to outside the channel), there is likely to be piping of fine material from under the levee due to subsurface flow of water. If enough material is removed, subsidence occurs and the levee will slump and possibly fail. Presently available information on the levee foundations is insufficient to formulate a judgment on the severity of this problem. Proper selection and treatment of construction materials should prevent the problems of piping and liquifacation in the the levee themselves.

LOW FLOW CHANNEL INSTABILITY

The proposed unarmored low flow channel will be severly damaged in the 10-year event and totally destroyed in the 100-year event. The river "responds" to the man made channel by cutting a new channel which minimizes changes in sediment transport rates and momentum. As shown in Section VII, the thalweg migrates towards the north levee above the ILS, towards the south levee in the vicinity of the radar station, and back towards the north levee well below the radar station. This migration can be prevented only by fully armoring the channel bottom, an expensive process.

An alternative approach to armoring the channel bottom would be to construct the channel bottom initially so that it is very nearly in equilibrium with the sediment transport processes. An equilibrium condition would have to be estimated based on the model results and then used as a guide for design. No channel bottom restoration would be done after flood events unless scour threatened levees or other structures.

Evidence for the development of an equilibrium condition appeared in the 92,000 cfs movable-bed model runs. The greatest scour and deposition occurred in the first of three sequential runs made without bed restoration. (see Section VII). The second run resulted in much less change and the third run produced only consolidation of the already formed conditions and further natural armoring of the surfaces.

COST ESTIMATES FOR RECOMMENDED MODIFICATIONS

Rough estimates of costs for the above design modifications were made based on the assumptions and prices given in Chapter X. Where specific remedies are not being recommended, no costs are shown. No analyses of potential savings due to certain potential design changes, such as the use of rip-rap instead of gabions, are made. The cost figures shown below are intended as a rough basis for comparison only.

Design Modifications	Cost Estimate
Increase levee heights	$30,000
Increase extent of protection	600,000
Buried gabions to maximum scour depth	500,000
Rip-rap protection of lower portions of gabions	600,000
Prevention of piping	Unknown
New low flow channel alignment	Long term savings

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