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WAIMEA RIVER SEDIMENTATION STUDY KAUAI, HAWAII

Numerical Model Investigation

by

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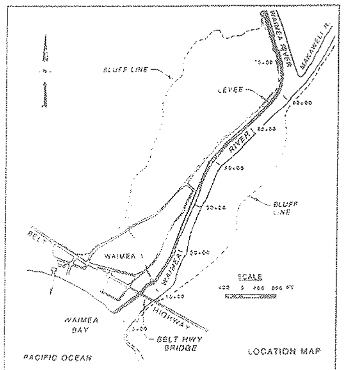
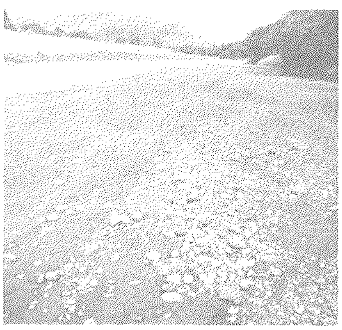
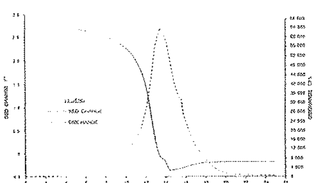
US Army Corps of Engineers







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A numerical model study of the Waimea River, Waimea, Kauai Island, Hawaii, was conducted to evaluate aggradation problems adjacent to a US Army Corps of Engineers levee, and to determine if the flood-control capability of the levee was reduced. The TABS-1 one-dimensional numerical model was used. Sediment inflow data were not available, and therefore sediment inflow was used as an adjustment parameter in the numerical model. The model was adjusted by reproducing measured historical aggradation using both multiple and single grain size sediment transport functions. Measured degradation that occurred during a historical period was used to test the model performance. Using the single grain size function, the numerical model could not be adjusted to simulate both the aggradation and degradation periods. However, the model was successfully adjusted to both historical surveys using the multiple grain size function. The adjusted model was then used to evaluate bed response during the design flood hydrograph. Appendix A presents a detailed description of the TABS-1 numerical model.

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PREFACE

The numerical model investigation of sedimentation in the Waimea River, Waimea, Kauai, HI, reported herein, was conducted at the US Army Engineer Waterways Experiment Station (WES), at the request of the US Army Engineer Division, Pacific Ocean (POD).

This investigation was conducted during the period October 1988 to August 1989 in the Hydraulics Laboratory, WES, under the direction of Messrs. Frank A. Herrmann, Jr., Chief of the Hydraulics Laboratory; R. A. Sager, Assistant Chief of the Hydraulics Laboratory; Mr. Marden B. Boyd, Chief of the Waterways Division (WD); and Mr. Michael J. Trawle, Leader of the Math Modeling Group (MMG). Mr. William A. Thomas, WD, provided general guidance and review. The project engineer and author of this report was Mr. Ronald R. Copeland, MMG. Technical assistance was provided by Mrs. Peggy Hoffman and Ms. Brenda Martin, MMG. This report was edited by Mrs. Marsha Gay, Information Technology Laboratory, WES.

During the course of this study, close working contact was maintained with Mr. Jim Pennaz, POD, who served as coordinating engineer, providing required data and technical assistance.

Commander and Director of WES during the preparation of this report was COL Larry B. Fulton, EN. Technical Director was Dr. Robert W. Whalin.

CONTENTS

CONVERSION FACTORS, NON-SI TO SI (METRIC) UNITS OF MEASUREMENT

Non-SI units of measurement used in this report can be converted to SI (metric) units as follows:

Multiply	By	To Obtain
acre-feet	1,233.489	cubic metres
cubic feet	0.02831685	cubic metres
cubic yards	0.7645549	cubic metres
feet	0.3048	metres
inches	2.54	centimetres
miles (US statute)	1.609344	kilometres
pounds (mass) per cubic foot	16.01846	kilograms per cubic metre
square miles	2.589998	square kilometres
tons (2,000 pounds, mass)	907.1847	kilograms

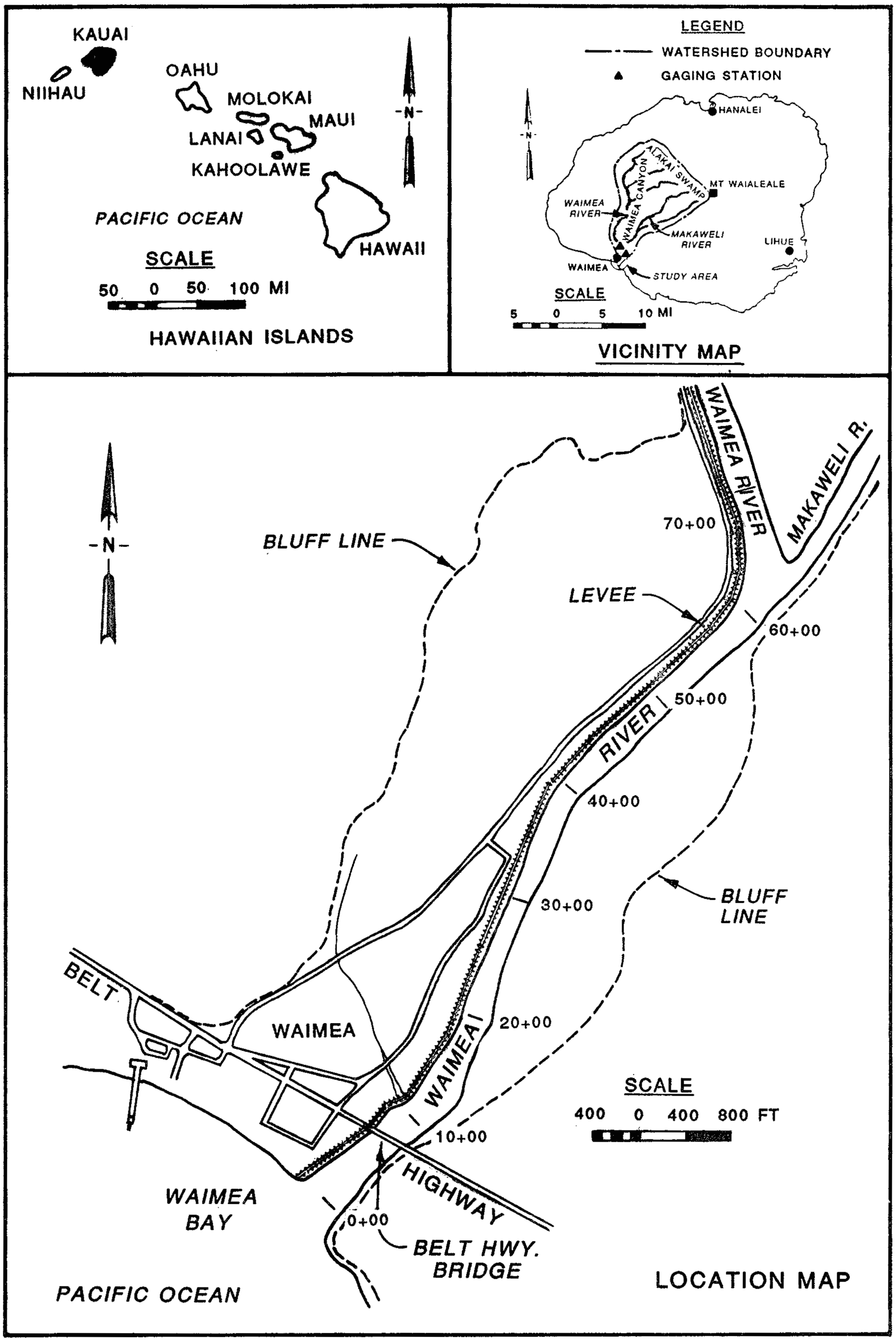

Figure 1. Location and vicinity maps

WAIMEA RIVER SEDIMENTATION STUDY, KAUAI, HAWAII

Numerical Model Investigation

PART I: INTRODUCTION

The Prototype

1. The Waimea River drains an area of approximately 85 square miles* on the island of Kauai, which is the northernmost of the eight major Hawaiian Islands (Figure 1). The island of Kauai is a single large shield volcano built from the seafloor by many thousands of thin basaltic lava flows. The island has a complex geological structure as a result of volcanic activities, separated by intervals of erosion combined with faulting. The Waimea River basin is characterized by canyons, up to 3,000 ft deep, cut into olivine-basalt lava deposits. Volcanic ash and cinders are interbedded with the lava flows, but consist of less than 2 percent of the exposed land mass (MacDonald, Davis, and Cox 1960). The western portion of the basin contains Waimea Canyon, often called the Grand Canyon of the Pacific. The Alakai Swamp, which is a remnant of ponded lava that filled the main caldera of the Kauai volcano, is a relatively flat plateau that occupies about 12 square miles in the upper regions of the drainage basin. The ridges that bound the basin on the west and east rise from sea level to elevations** of 4,000 and 5,000 ft, respectively. The highest point is Mount Waialeale, which receives the highest measured average annual rainfall on earth--over 460 in. However, rainfall over the basin is highly variable, with the town of Waimea receiving an average rainfall of only 22 in. per year. Runoff in the Waimea River is flashy in nature, with flood durations of only a few hours. The study area includes the lower 2 miles of the drainage basin, which is located in the west-central portion of the island. The largest tributary of the Waimea River is the Makaweli River, which joins the Waimea about 1.1 miles from its mouth, and has a drainage area of about 26 square miles. The town of Waimea, which is

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* A table of factors for converting non-SI units of measurement to SI (metric) units is found on page 3.

** All elevations (el) cited herein are in feet referred to the National Geodetic Vertical Datum (NGVD).

located on the Waimea River's floodplain, has a population of about 1,600. The floodplain also supports sugar cane fields along the riverbanks.

2. Prior to flood-control improvements constructed by local interests in the early 1950's, frequent and damaging floods occurred in Waimea. The flood of 7 February 1949 had a peak flow of 45,000 cfs and inundated the entire town. Severe flooding also occurred in August 1950, with a peak flow of 32,000 cfs, when the river overflowed twice, inundating the town. After the February 1949 flood, the Territory of Hawaii widened the Waimea River at its mouth. In 1950, the river was widened upstream and downstream of the confluence with the Makaweli River, and excavated material was used to construct a levee along the right descending bank of the river. The downstream end of the levee was at sta 10+50, 300 ft upstream of the Belt Highway Bridge, and the levee extended 6,650 ft to a bluff about 1,340 ft upstream from the confluence with the Makaweli River. Additional work by the county of Kauai, with funds provided by the Territorial Government, was completed in 1954. These improvements included channel excavation and widening, levee construction, and a grouted riprap lining on the levee. This local protection system protected the upper portion of the town from about a 100-year-frequency flood, but the lower portion of the town was protected from only a 25-year-frequency flood.

3. In 1984, the US Army Corps of Engineers completed a 1,575-ft extension of the levee from the existing downstream terminus to the ocean. The Corps project included raising the existing levee, providing additional toe protection to the existing levee, and improving local drainage structures. The project was designed to provide flood protection from the 100-year flood of 64,000 cfs.

Purpose of the Numerical Model Study

4. Between channel surveys conducted in January 1979 and November 1987, about 83,500 cu yd of deposition occurred in the Waimea River between sta 4+70 and 60+30. In August 1988, the county of Kauai requested assistance from the Corps of Engineers to evaluate the aggradation problems in the Waimea River. Concern was expressed about the ability of the flood-control project to convey the design flood. The county was also experiencing difficulty in maintaining local drainage outlets along the levee where sediment had accumulated.

5. Several possible causes for aggradation in the study reach have been

advanced. A massive rock avalanche occurred in the headwaters of the Makaweli River in October 1981. The avalanche and subsequent erosion from a debris flow left an estimated 3.3 million cu yd of material deposited in the canyon bottom (Jones, Chinn, and Brice 1984). Sediment accumulation in the Waimea River may be the result of a geomorphic adjustment to this landslide. Another possible cause is the increase in the wild goat population, which could be destroying vegetative cover in an already fragile drainage basin. It could also be that both the Waimea and Makaweli basins are high sediment producers and that sediment generally accumulates in the channel during dry years and is degraded during major flood events. Response to the sediment accumulation problem depends somewhat on whether the problem is a long-term problem due to the normal sediment yield potential of the drainage basins or if it is due to a relatively short term response to a landslide in the basin.

6. The numerical model study was performed to evaluate deposition in the leveed reach of the Waimea River. The model was adjusted to simulate measured historical deposition trends and bed changes between 1979 and 1989. The model was then used to evaluate possible causes for current aggradation trends and to identify possible long-term trends. The model study was also used to evaluate the effect of the design flood on the riverbed. The numerical model used in this study, TABS-1, is primarily a sediment transport model and does not have the refinements of a backwater model such as HEC-2 for calculation of design water-surface elevations through bridges and other local obstructions.

PART II: THE MODEL

Description

7. The TABS-1 one-dimensional sedimentation program was used to develop the numerical model for this study. Development of this computer program was initiated by Mr. William Thomas at the US Army Engineer District (USAED), Little Rock, in 1967. Further development at the US Army Engineer Hydrologic Engineering Center (USAEHEC) by Mr. Thomas produced the widely used HEC-6 generalized computer program for calculating scour and deposition in rivers and reservoirs (USAEHEC 1977). Additional modification and enhancement to the basic program by Mr. Thomas at the US Army Engineer Waterways Experiment Station (WES) led to the TABS-1 program currently in use (Thomas 1980, 1982). The program produces a one-dimensional model that simulates the response of the riverbed profile to sediment inflow, bed material gradation, and hydraulic parameters. The model simulates a series of steady-state discharge events and their effects on the sediment transport capacity at cross sections and the resulting degradation or aggradation. The program calculates hydraulic parameters using a standard-step backwater method assuming subcritical flow. The program assigns critical depth for water-surface elevation if the backwater calculations indicate transitions to supercritical flow. However, for supercritical flow, hydraulic parameters for sediment transport are calculated assuming normal depth in the channel. A more detailed description of the program capabilities is included in Appendix A.

Channel Geometry

8. The study reach extended from sta 0+00 at the mouth of the Waimea River to sta 90+00 on the Waimea River and up the Makaweli River from its confluence with the Waimea for 800 ft. The channel geometry for the model was based on cross sections from a HEC-2 backwater model provided by the USAED, Honolulu (USAED, Honolulu, 1980). These cross sections were based on a survey taken in January 1979. Cross-section locations are shown in Figure 2. The TABS-1 model does not include the backwater effects of the Belt Highway Bridge; however, this is not expected to significantly influence general sedimentation patterns in the river. The downstream boundary of the numerical model was set using cross sections that extended 4,000 ft into the ocean from the river mouth at sta 0+00. These sections had horizontal inverts with

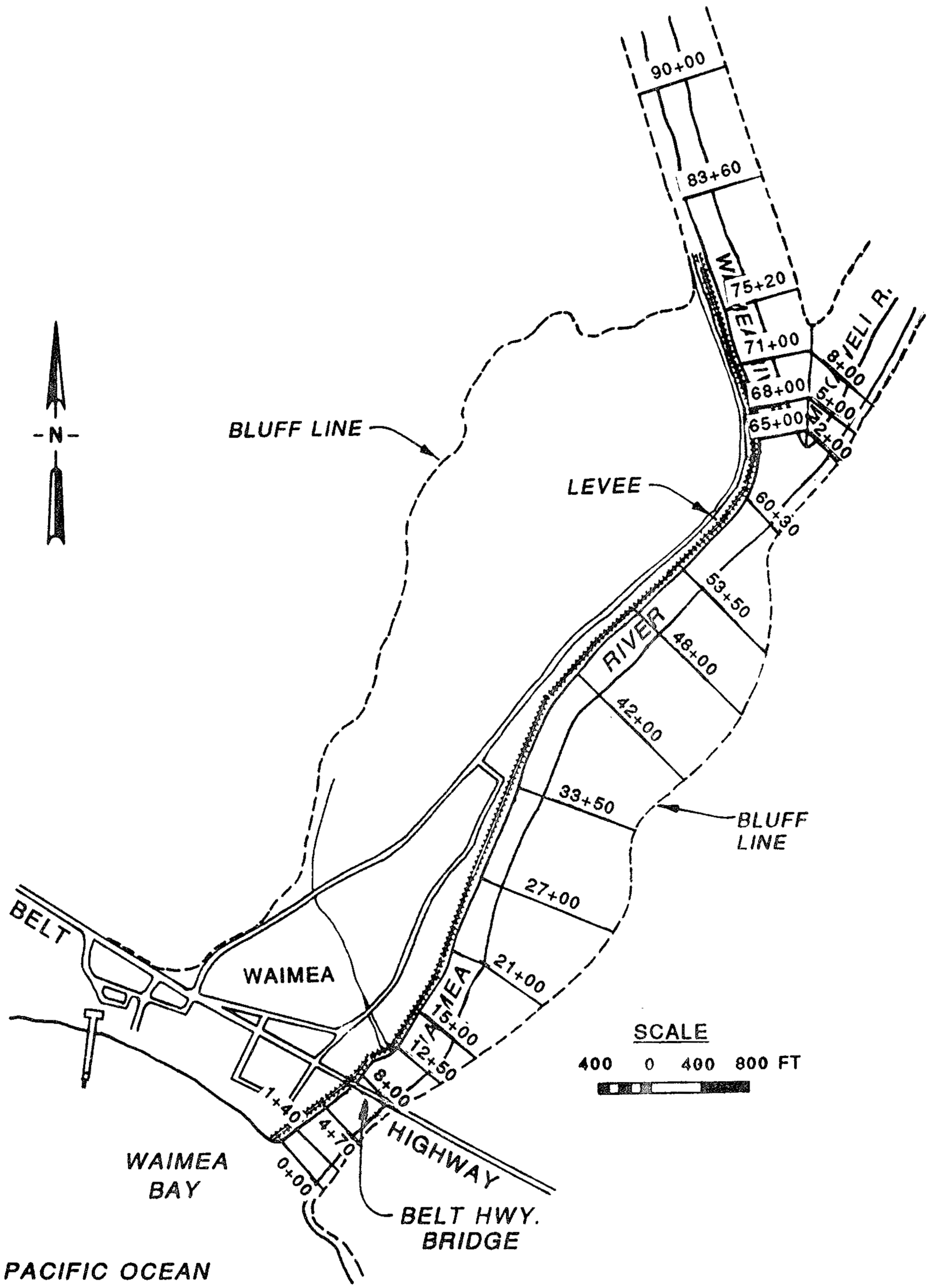

Figure 2. Cross-section locations

elevations determined from US Geological Survey (USGS) 1:24,000-scale topographic maps. Section widths were set assuming a 1:4 divergence ratio. The purpose of these sections was to prevent sediment buildup at the downstream boundary and to account for the effects of flow divergence at the river mouth on downstream water-surface elevations. Reach lengths between cross sections in the model varied between 250 and 850 ft.

9. Surveys taken by the county of Kauai in November 1987 and March 1989 were used to determine prototype aggradation and degradation. The surveys included 12 cross sections between sta 4+70 and 60+30. The cumulative volume of aggradation in the reach for each survey was calculated by the average-end-area method. Volumes were accumulated starting at sta 4+70 and moving upstream. Cumulative aggradation was 83,500 cu yd between January 1979 and November 1987, and 25,000 cu yd between January 1979 and March 1989. Thus, 58,500 cu yd were degraded between November 1987 and March 1989.

10. The surveys identified a point bar that is developing on the right descending bank between sta 42+00 and 60+30. The bar is vegetated with grasses and the average bar height is about 7 ft above the streambed. Surface sediment samples taken from the bar had a fine sand and silt composition, which was considerably finer than the riverbed itself. The characteristics of the bar indicate that its formation was due to two-dimensional flow processes related to channel bend dynamics. Fine sediment is deposited on the bar because local flow velocities are lower than the average channel velocity. The process of bar formation is separate from the processes of riverbed aggradation and degradation that are simulated with the one-dimensional TABS-1 model. Therefore, it may be more appropriate to exclude the point bar when model performance is compared to measured aggradation. Without the point bar, cumulative aggradation between sta 4+70 and 60+30 was 76,000 cu yd between January 1979 and November 1987, and 7,000 cu yd between January 1979 and March 1989. Total aggradation, including the point bar, was used to adjust the numerical model, but the volume of the point bar should be considered when interpreting the results.

Hydrographs

11. Discharge hydrographs are simulated in the numerical model by a series of steady-state events. The duration of each event is chosen such that

changes in bed elevation due to deposition or scour do not significantly change the hydraulic parameters during that event. At relatively high discharges, durations need to be short; time intervals as low as 4 min were used for the flood peaks on the Waimea River. At low discharges, the time interval may be extended. Time intervals up to 7 days were used during the historical simulations.

12. A hydrograph simulated by a series of steady-state events of varying durations is called a histograph. The historical histographs used in the numerical model were based on data from the USGS gages on the Waimea and Makaweli Rivers. The gages are located about 1.3 and 0.7 miles, respectively, upstream from the confluence of the two rivers. Mean daily flows from the two gages were combined to obtain the mean daily flow downstream from the confluence of the two rivers. Combined mean daily discharges greater than 200 cfs were used to develop historical histographs between January 1979 and March 1989 and between October 1943 and September 1950. Sediment transport was found to be negligible at combined discharges less than 200 cfs. In addition to mean daily flows, 22 peak discharges on the Waimea and Makaweli Rivers were reported (USGS 1979-1989) between January 1979 and March 1989, and 7 peak discharges were reported (USAED, Honolulu, 1980) between October 1943 and September 1950. Mean daily flows were adjusted to account for the increased sediment transport potential at high-flow events. Reported peaks were assigned a duration of 2 hr and the corresponding mean daily flow was reduced to maintain the same runoff volume. The 2-hr duration was chosen based on durations of actual flood hydrographs measured between 1970 and 1974 (USAED, Honolulu, 1980). If a reported peak occurred on only one tributary, then the mean daily flow from the other tributary was added to the peak flow to obtain the downstream discharge for the 2-hr event. If both tributaries had a reported peak discharge, and the times of occurrence of the recorded peaks were more than 1.5 hr apart, they were treated as separate events. If recorded times of occurrence were less than 1.5 hr apart, or not available, the downstream peak discharge was set equal to 1.44 times the Waimea peak discharge. This ratio is from the Standard Project Flood (SPF) combining and routing calculations (USAED, Honolulu, 1980). The Makaweli contribution during the 2-hr peak was set at 0.44 times the Waimea peak. A second 2-hr-duration high-flow period was calculated by combining the residual peak on the Makaweli with the residual mean daily flow on the Waimea. The flow for the remaining 20 hr

was determined by combining the residual mean daily flows from the two rivers. The combined historical histograph between January 1979 and March 1989, which was used to adjust the numerical model, is shown in Plate 1. On this plate the abscissa is discontinuous because mean daily discharges less than 200 cfs were excluded.

13. The SPF was developed using the unit hydrograph method (USAED, Honolulu, 1980). Unit hydrographs were developed based on stream gaging records on the Waimea and Makaweli Rivers and from rain gage data within and adjacent to the watershed. The SPF had a peak downstream from the confluence of the two rivers of 105,000 cfs.

14. The design flood hydrograph has a 100-year-frequency peak discharge of 64,000 cfs downstream from the confluence of the Waimea and Makaweli Rivers. The shape of the design flood hydrograph was assumed to have the same shape as the SPF with discharges reduced by the ratio of the peaks. Flow in the Makaweli River was taken to be 44 percent of the Waimea River flow. The design flood histograph downstream from the confluence is shown in Plate 2. Time intervals were between 2 hr and 4 min.

Downstream Water-Surface Elevation

15. Starting water-surface elevations at the downstream boundary of the numerical model, 4,000 ft below the mouth of the river, were set at the mean tide level, el 0.0 for the historical simulations. The mean high tide elevation of 1.5 was used for the design flood.

Bed Material

16. The bed material of the Waimea and Makaweli Rivers downstream from their confluence consists primarily of coarse sands with some gravels. Boulders up to 2 ft in diameter were observed in the study reach, but, in general, the maximum bed material size was 0.5 ft. Upstream from the confluence, the bed becomes coarser, alternating between coarse sand and cobble and boulder riffles. The specific gravity of the material is 3.08. The specific weight of the sediment deposits was taken to be 117 pcf, based on California Division of Water Resources criteria adjusted for specific gravity (Vanoni 1975). The numerical model requires that an initial volume and gradation be specified for

the bed material. Twenty-two samples were collected in February 1989 from the bed surface of the Waimea and Makaweli Rivers at locations shown in Figure 3. Due to limitations of the sampling equipment, no samples were obtained from areas where the water was more than 3 ft deep. This included the entire study reach downstream from sta 33+50, except for exposed bars at sta 0+00 and 4+70. As shown in Figure 4, these samples showed a wide variation in gradation. Lateral variations across the width of the river were apparent from the

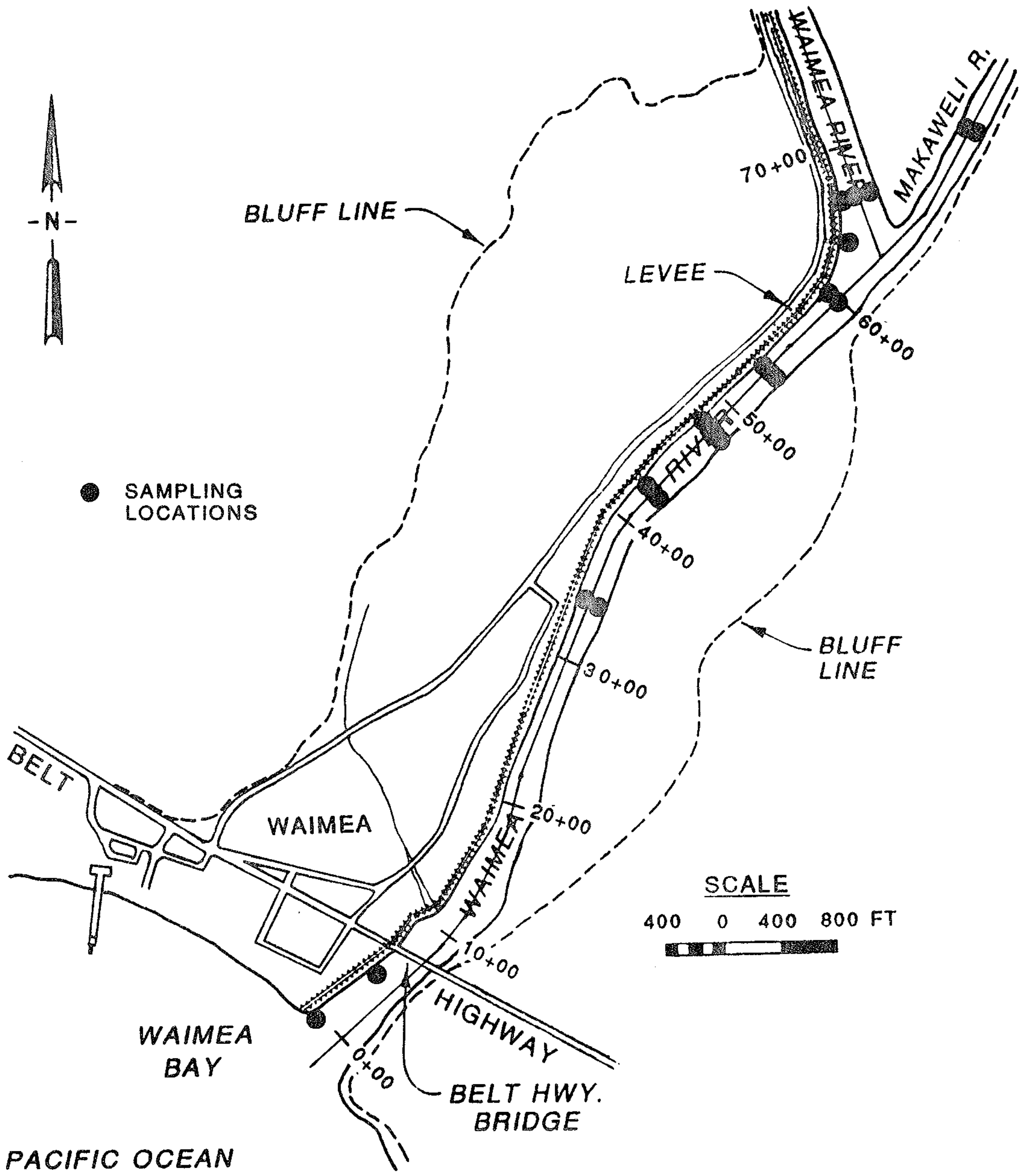

Figure 3. Location of bed material sampling, February 1989

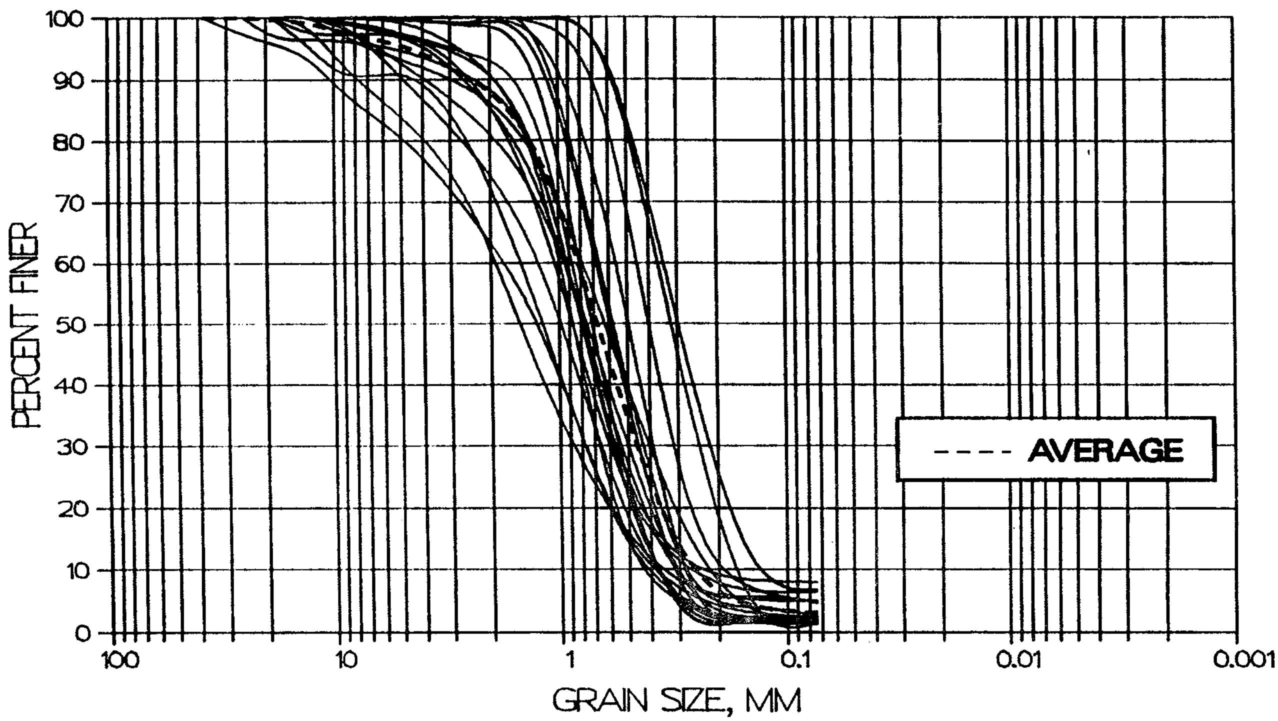

Figure 4. Bed material gradations from February 1989 samples

sampling program (Plate 3). However, the data did not identify any longitudinal variation in the gradation; in fact, samples collected at and near the channel mouth were found to be within the full range of all the sampled data (Plate 4). Since no longitudinal variation could be identified, the average of all samples taken from the channel bottom was used to develop an initial gradation for the numerical model. Three samples taken from the vegetated point bar between sta 42+00 and 60+30, which had surface elevations between 5 and 10 ft higher than the channel bottom, were not included in the average. The median grain size of these three samples varied between 0.20 mm and 0.10 mm. The composite gradation (Figure 4) used in the numerical model had a median grain size D_{50} of 0.65 mm and a standard deviation of 2.1.

Channel Roughness

17. Hydraulic roughness is influenced by grain size, bed form, water depth, bank roughness, changes in channel shape, and changes in flow direction or concentration of flow due to bends and confluences. In the one-dimensional numerical model, these effects are accounted for by the Manning's roughness coefficient. The roughness coefficient may vary significantly with discharge

and time. The influence of grain roughness is known to decrease with increases in depth. Resistance due to bed forms can decline dramatically when dunes are washed out and replaced by a plane bed or antidunes. Greater momentum at high flows increases resistance due to channel bends and confluences. Local scour at high flow also tends to make channel cross sections more irregular, increasing roughness. High-water marks from events of known magnitude and hydraulic geometry are frequently used to estimate roughness coefficients. High-water marks from the February 1949 flood were used by the Honolulu District (USAED, Honolulu, 1980) to estimate a Manning's roughness coefficient for the design flood. The Honolulu District supplemented this analytical approach with field inspection of the river and overbank areas to obtain roughness coefficients between 0.025 and 0.030 for the channel, and 0.040 and 0.080 for the overbanks.

18. In the TABS-1 numerical model, some adjustment of the roughness coefficients was necessary due to the model's sensitivity to hydraulic parameters. Overbank conveyance on the left descending overbank was reduced by increasing overbank roughness coefficients to 0.120. This produced a more reasonable channel-overbank flow distribution. The Brownlie (1983) equation was used to evaluate the variation of channel roughness due to bed form change. The Brownlie equation predicted lower regime flow, with dune bed forms, and a roughness coefficient of about 0.030 for discharges less than 3,000 cfs. At discharges greater than 12,000 cfs, upper regime flow with a roughness coefficient of about 0.020 was predicted. Intermediate discharges were in the transition zone. The effect of varying Manning's roughness coefficient with discharge was tested with the numerical model. In these tests, discharges less than 3,000 cfs were assigned a channel roughness value of 0.030, discharges greater than 12,000 cfs were assigned a value of 0.020, and the roughness coefficient was varied linearly for intermediate discharges.

Sediment Inflow

19. Sediment inflow measurements are not available for use in the numerical model. Therefore sediment inflow was used as an adjustment parameter. Four sections on the Waimea River upstream from its confluence with the Makaweli River were used to determine initial sediment inflow values. Calculated inflows at these sections were averaged to obtain a sediment inflow

rating curve for the initial adjustment phase of the investigation. These sections were chosen based on field observations that this reach of river alternated between a coarse sand bed and riffles and appeared to be relatively stable. There were no data to indicate a difference in sediment inflow contributions from the Waimea and Makaweli basins; therefore, inflow concentrations were initially assumed to be equal.

Transport Function

20. Several transport functions were investigated for use in this study. The effect of using multiple and single grain size functions was evaluated. Single grain size functions investigated included the Yang (1973, 1984) and Ackers-White (1973) functions. A grain size of 0.65 mm was initially used with the single grain size functions. Multiple grain size functions tested in the study included Laursen-Madden (USAEHEC 1977); a combination of the Toffaleti (1966) and Meyer-Peter and Muller (1948) functions, herein referred to as TMPM; and a modification of the Laursen (1958) equation. The modified Laursen equation, hereafter referred to as the Laursen-Copeland function, incorporates data for transport of gravels in addition to the sand data used to develop the original Laursen function. There are some differences in the way hydraulic parameters are calculated in the Laursen-Copeland function when compared to the original Laursen equation. This is necessary because the Laursen-Copeland function was developed to calculate sediment transport in a sand and gravel bed stream (Copeland and Thomas 1989).

21. The Laursen-Copeland function calculates the hydraulic radius due to grain roughness using the Limerinos equation (Limerinos 1970). The calculated hydraulic radius (instead of the depth, as proposed by Laursen) is then used to calculate the grain shear stress:

$$\tau'_o = \left(\frac{\rho V^2}{58}\right) \left(\frac{D_{50}}{R'_b}\right)^{1/3} \qquad (1)$$

where

τ'_o = grain shear stress

ρ = water density

V = average water velocity

D_{50} = particle size of which 50 percent of the bed is finer

R'_b = hydraulic radius of the bed attributed to grain roughness

This equation is dimensionally homogeneous and can be applied with any consistent set of units.

22. Bed-load transport is a function of the ratio of applied to critical shear stress. In the Laursen-Copeland function this is expressed by the parameter

$$\frac{\tau'_o}{\tau_{ci}} - 1 \tag{2}$$

where τ_{ci} is the critical shear stress for the i^{th} grain size. The critical shear stress varies with the particle size, larger particles having greater critical shear stress. Paintal (1971) determined that the critical shear stress, as used in Equation 2 to determine sediment transport, also varied with applied shear stress. When the dimensionless shear stress τ_o^* was less than 0.05, he found that the critical shear stress decreased significantly:

$$\tau_o^* = \frac{\tau_o}{\left(\gamma_s - \gamma_w\right) d_i} \tag{3}$$

where

τ_o = applied shear stress

γ_s = specific weight of sediment

γ_w = specific weight of water

d_i = geometric mean diameter of the i^{th} size class

This variation in critical shear stress is accounted for in the function by varying the Shields parameter between 0.039 and 0.020. The higher value, recommended by Laursen (1958), was used when τ_o^* was greater than 0.05. The lower limit was determined by Andrews (1983). The effect of this change is that initiation of motion for coarser particles occurs at lower shear stresses, and the transport potential of coarser particles is increased.

23. The Laursen-Copeland function uses the ratio of grain shear velocity

(instead of total shear velocity) to grain fall velocity as the important parameter influencing suspended sediment transport. A functional relationship between this ratio and other parameters was determined by Laursen (1958) based on river and flume data. Due to the reformulation of Laursen's parameters, a new functional relationship was developed for the Laursen-Copeland function. The relationship is based on data from both rivers and flumes. The functional relationship and data scatter are shown in Plate 5. Flume data gathered under more controlled conditions have significantly less scatter than the river data.

24. Sediment transport is calculated using the following formula

$$C = 0.01\gamma_w \, \Sigma \, P_i \left(\frac{d_i}{y}\right)^{7/6} \left[\left(\frac{\tau'_o}{\tau_{ci}}\right) - 1\right] f\left(\frac{U'_*}{w_i}\right) \tag{4}$$

where

C = concentration in weight per unit volume

P_i = fraction of grain size class in the bed

y = water depth

U'_* = grain shear velocity

w_i = fall velocity

$f(U'_*/w_i)$ = function defined in Plate 5

This function is considered to be a refinement to Laursen's original equation and is based on a wider range of physical data. The primary benefit is that it moves coarser sediments better than other functions.

25. Initial sediment transport for the five sediment transport functions at the four sections on the Waimea River upstream from its confluence with the Makaweli River calculated using the same hydraulic parameters are shown in Plate 6. The TMPM function predicts significantly less transport at high discharges than the other functions. Multiple grain size functions are compared in Plates 7-9. The Laursen-Madden function did not transport as much of the coarse material as the Laursen-Copeland function. This is attributed to characteristics of the latter function, which was developed to transport the coarser fractions.

PART III: MODEL ADJUSTMENT

Adjustment to November 1987 Survey

26. In the initial stages of the model investigation, hydrologic data were available only up to September 1987. Therefore, the historical histograph used in the initial numerical model adjustment extended from January 1979 to September 1987. This initial adjustment was accomplished by varying sediment inflow at the upstream boundary until aggradation between sta 4+70 and 60+30, measured in November 1987, was properly simulated. During the course of the study, additional channel survey and hydrologic data became available and were used to further adjust the numerical model. The initial estimate for sediment inflow was determined from calculations using average hydraulic parameters at four cross sections on the Waimea River upstream from its confluence with the Makaweli River. These sediment rating curves were increased or decreased by a constant percentage to simulate the total deposited volume of 83,500 cu yd. The Laursen-Copeland, Yang, and Ackers-White functions successfully reproduced the distribution of deposited sediment in the study reach (Figure 5). The TMPM function and the Laursen-Madden function calculated sediment deposition that was greater at the upstream end of the study reach than in the prototype. The adjusted sediment inflow rating curves are shown in Plate 10. With the Ackers-White and TMPM functions, adjusted sediment transport at discharges greater than 2,000 cfs was considerably less than with the other functions. The adjustment percentage between the initial and adjusted sediment transport for the five sediment transport functions is listed in the following tabulation:

Function	Percentage
Ackers-White	-7
Laursen-Copeland	+15
Laursen-Madden	+90
TMPM	+77
Yang	-30

27. The effect of varying channel roughness with discharge was tested in the numerical model using the Laursen-Copeland function. Reducing the

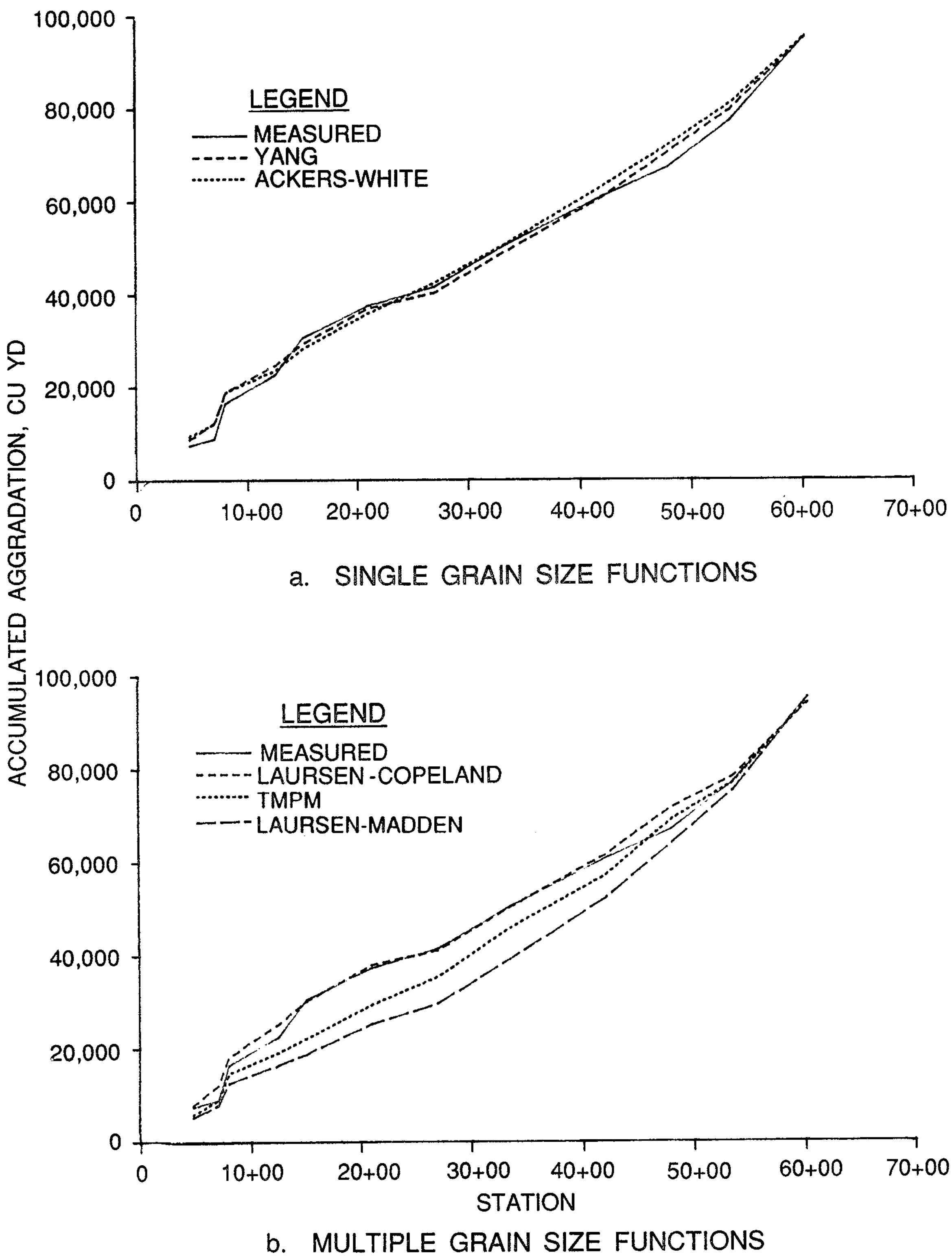

Figure 5. Measured and calculated accumulated aggradation January 1979-September 1987

roughness coefficient at higher discharges resulted in more sediment passing through the study reach at high flows. As a result, it was necessary to increase sediment inflow by 10 percent to simulate the measured aggradation. More material deposited in the upstream reaches with the revised roughness coefficients. The calculated longitudinal distribution of sediment was not as close to the measured as were the original calculations using a constant roughness coefficient for all discharges (Figure 6). The higher roughness coefficient may be more appropriate in the sediment model because it compensates for the additional hydraulic losses due to the Belt Highway Bridge.

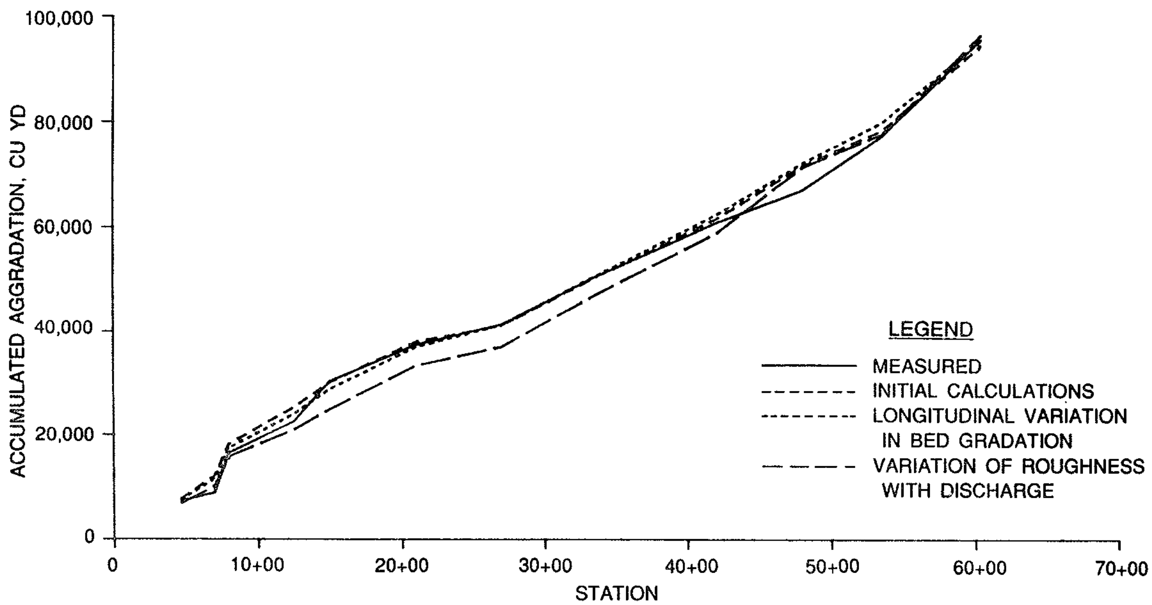

Figure 6. Effects of varying initial bed gradation longitudinally and of varying roughness with discharge on accumulated aggradation, Laursen-Copeland function, January 1979-September 1987

28. The initial bed material gradation in the numerical model was not varied longitudinally. However, with the multiple grain size transport functions, the calculated gradation changed during the historical simulation. This change was caused by the hydraulic sorting algorithm in the model, which allows for variation in the bed gradation with variation in sediment transport potential. The longitudinal change at the end of the January 1979 to September 1987 simulation, calculated using the three multiple grain size transport functions, is shown in Plate 11. Calculated bed material gradations at sta 42+00 are compared to the initial gradation and the envelope of all channel bottom samples in Figure 7. Calculations with all three multiple

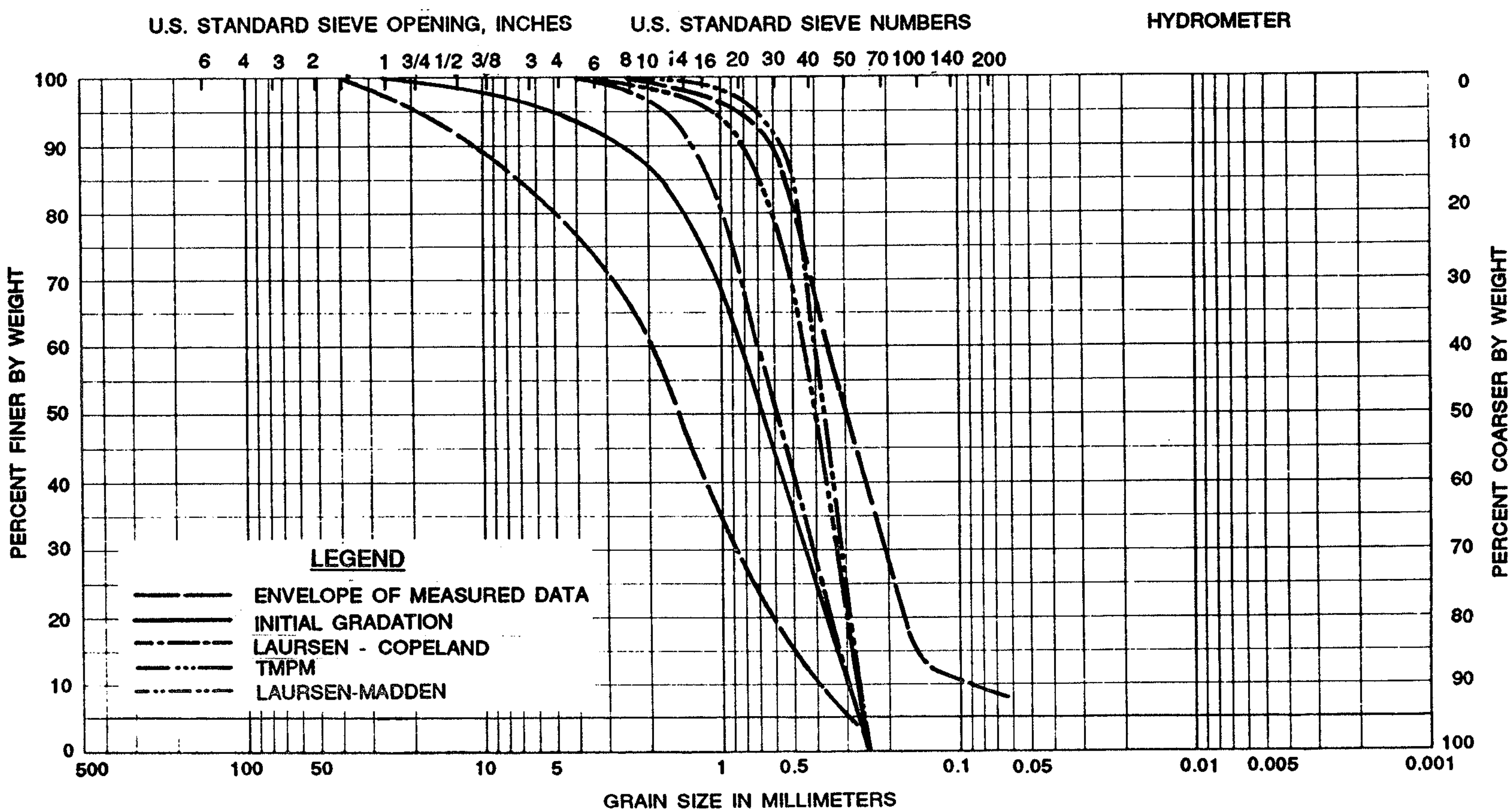

Figure 7. Calculated bed material gradation at sta 42+00, September 1987

grain size functions predicted a September 1987 bed that was finer than the average 1989 sampled data.

29. The effect of longitudinal variation in the initial bed gradation was tested in the numerical model using the Laursen-Copeland transport function. The bed gradation calculated at the end of the 1979-1987 historical simulation was used as the initial bed gradation for the test. The historical hydrograph was repeated and sediment inflow was adjusted to simulate measured aggradation in the study reach. Sediment inflow had to be decreased by 10 percent. At the end of the simulation, the final calculated bed material gradation was essentially the same as the initial gradation. Longitudinal variation in the initial bed gradation did not significantly affect calculated aggradation (Figure 6).

Adjustment to March 1989 Survey

30. Another channel survey was taken in March 1989. The survey showed that between November 1987 and March 1989, about 58,500 cu yd of sediment had been scoured between sta 4+70 and 60+30 by a series of relatively high runoff events. This left a total of about 25,000 cu yd of deposited sediment in the channel. The majority of this deposited sediment was located upstream from sta 33+50.

31. The numerical model, adjusted to simulate the January 1979-November 1987 aggradation, was tested by lengthening the historical simulation to March 1989. The predicted cumulative aggradation between sta 4+70 and 60+30 using the initial adjusted sediment inflow curves was significantly greater than the measured data. With the Ackers-White and Laursen-Madden functions, a slight increase in sediment deposition was calculated instead of a decrease. Using the TMPM function, some degradation in the downstream portion of the study reach was calculated, but aggradation was calculated in the upstream portion of the reach, so that there was no net change in calculated cumulative aggradation between November 1987 and March 1989. The other functions predicted degradation that was considerably less than measured. Results of the calculations are shown in Plates 12-16. The March 1989 survey data demonstrated that further adjustment of the numerical model was required. The Yang and Laursen-Copeland sediment transport functions were chosen for further consideration, because these functions produced results with some degradation and at

least some trend toward the measured data.

Temporal Change in Sediment Inflow

32. A significant decrease in sediment inflow between November 1987 and March 1989 would be one possible explanation for the degradation that occurred during this period. A decrease in sediment inflow from the Makaweli River basin could be attributed to stabilization of the 1981 landslide. The Yang single grain size transport function was used to test the effect of temporal change in sediment inflow. Sediment inflow after November 1987 was reduced in the numerical model by a constant percentage in three tests. In the first test, inflow from the Makaweli River was reduced by 50 percent. This resulted in an additional 9,000 cu yd of calculated degradation in March 1989. In the second test, sediment inflow was also reduced by 50 percent on the Waimea River. This resulted in an additional 31,000 cu yd of calculated degradation. In the third test, sediment inflow was reduced 67 percent on both rivers. This resulted in a calculated degradation of 55,000 cu yd, which is fairly close to the measured 58,500 cu yd. Calculated cumulative volumes from the three tests are plotted in Figure 8 and listed in the following tabulation:

Degradation between November 1987 and March 1989	Volume 1,000 cu yd
Measured	58
Calculated with the following sediment inflow adjustments	
No reduction	9
50 percent reduction on Makaweli	18
50 percent reduction on Makaweli and Waimea	49
67 percent reduction on Makaweli and Waimea	55

33. These tests did not prove or disprove the hypothesis that the degradation in the Waimea River between November 1987 and March 1989 was caused by a significant reduction in sediment inflow. It was determined, however, that reduction in sediment inflow due to recovery from the landslide in the Makaweli alone would be insufficient to cause the measured quantity of degradation. More than a 67 percent sediment inflow reduction in both drainage basins would be required, according to numerical model results, to account for

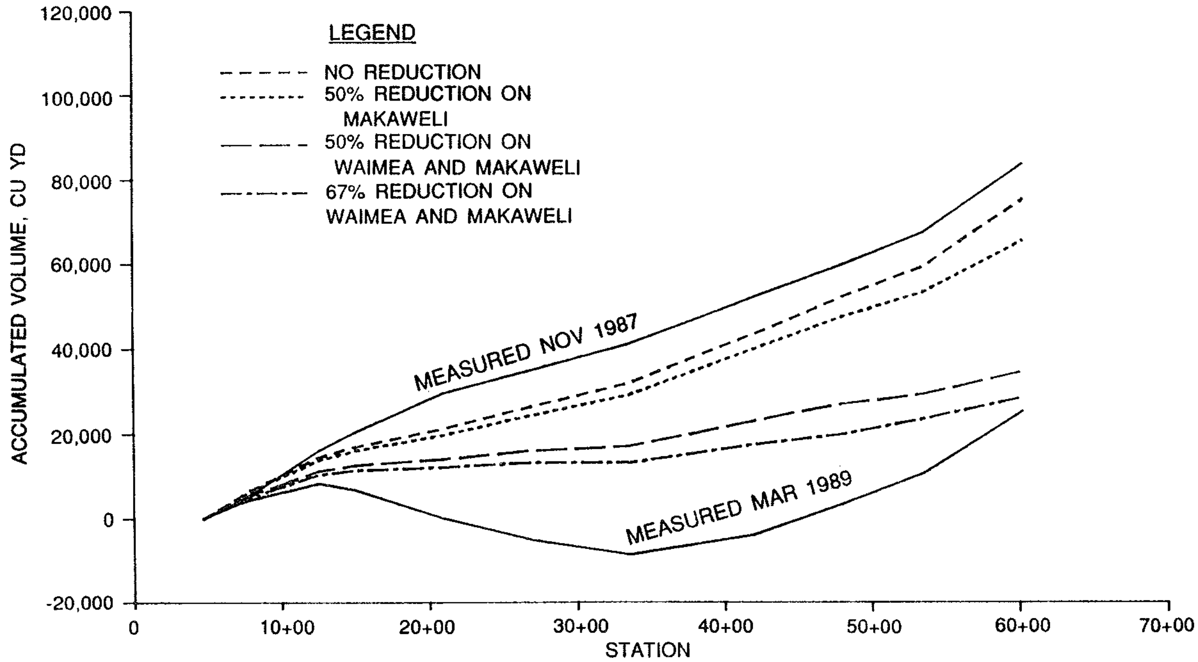

Figure 8. Calculated effect of sediment inflow reduction on degradation between November 1987 and March 1989

the removal of 58,500 cu yd. It is improbable that such a reduction could occur on a long-term basis without some obvious changes in the basin. No data indicating sediment supply decreases in the Waimea basin have been reported. It was therefore concluded that long-term adjustments to the sediment inflow covering the entire historical simulation period would be more appropriate.

Adjustments to Model Hydraulics

34. The channel geometry and channel roughness were adjusted in the numerical model and tested with the single grain size transport function in an attempt to improve model performance. The purpose of these tests was to increase hydraulic efficiency at high flows so that sediment transport potential would increase and more degradation would occur. Conveyance on the overbanks was eliminated in the first test. This had no significant effect on calculated aggradation and degradation. In the second test, channel roughness coefficients were varied with discharge as discussed in paragraph 18. At high discharges when roughness was decreased, transport potential was increased. This resulted in an additional 12,000 cu yd of sediment removed in March 1989, but also resulted in 17,000 cu yd less sediment deposition in November 1987.

The net effect was less degradation between November 1987 and March 1989 and thus no improvement in the numerical simulation.

Sediment Inflow Adjustment

Single grain size function

35. The numerical model was initially adjusted to the November 1987 measured aggradation using a sediment inflow rating curve based on the assumption of equilibrium transport. That is, the sediment inflow was assumed to be determined by the channel capacity. However, sediment inflow can also be supply limited, especially at high flows. This condition was tested in the numerical model by further adjustments to the sediment inflow curves. In general, the numerical simulation using the single grain size transport function was significantly improved by increasing inflow at lower discharges and decreasing inflow at higher discharges. However, measured degradation quantities were not attained. Measured and calculated cumulative aggradation are compared in Plate 17 for four of the tests conducted. Corresponding sediment inflow rating curves are shown in Plate 18.

36. The effect of decreasing the grain size was tested using the single grain size transport function. Medium sand, with a median grain size of 0.35 mm, replaced coarse sand in the numerical model. Sediment inflow was adjusted in an attempt to simulate the measured aggradation in November 1987 and March 1989. The simulation could be made to more closely represent one of the measured events or the other depending on the sediment inflow adjustment (Plate 19). However, the net degradation between the two surveys was essentially the same as with coarse sand.

Multiple grain size function

37. The numerical model was used to test the effect of the initial bed gradation on aggradation and degradation during the historical simulation. In the initial adjustment of the numerical model, the initial bed material gradation had been assumed to be the same as that sampled in February 1989. This seemed reasonable at the time because it was not known that significant degradation had occurred since the November 1987 survey. It had already been determined in the initial adjustment phase of the study that a slight longitudinal variation in initial bed gradation did not affect the November 1987 simulation (paragraph 29). The possibility of a large longitudinal variation

with a reservoir of fine sediment in the river between the mouth and sta 33+50 was tested. It is possible that such a reservoir could deposit at low flows and on the recession of flood events and scour out at high flows. The existence of such a reservoir would not have been detected during the sampling program because samples were not collected in this reach of the river due to inadequate sampling equipment. The test was conducted using the Laursen-Copeland sediment transport function. The initial bed gradation was set at 80 percent medium sand and 20 percent coarse sand downstream from sta 33+50. The 10-year runoff histograph was run through the numerical model. Calculated cumulative aggradation with the original longitudinal varied initial bed gradation and with the finer initial bed gradation are compared to measured data in Plate 20. The calculated aggradation was greater in November 1987 than it was in the original simulation, and the calculated degradation between November 1987 and March 1989 was less than in the original calculation. Similar results were obtained when the TMPM sediment transport function was substituted for the Laursen-Copeland sediment transport function. Adjustment to the initial bed material gradation was not pursued.

38. The sediment inflow curves for the Laursen-Copeland transport function were adjusted by increasing sediment inflow at low discharges and decreasing sediment inflow at high discharges. Most of the adjustment was to sand size classes. This was a similar test to that conducted with the single grain size function. With the initial sediment inflow curves, the calculated cumulative aggradation in March 1989 was 70,000 cu yd, which compared to the measured accumulation of 25,000 cu yd. With the sediment inflow adjustments from this test, the calculated cumulative aggradation in March 1989 was 59,000 cu yd. This was a significant but not sufficient improvement. Cumulative calculated aggradation is compared to the measured aggradation in Plate 21, using the adjusted sediment inflow shown in Plate 22.

39. Although the February 1989 bed samples contained less than 10 percent fine sand, it may be that the unsampled zone downstream from sta 33+50 contains a greater percentage. It is also possible that during other historical periods, when the riverbed has had more deposition, more fine sand was present in the bed. The effect of fine sand inflow and deposition was tested by adding fine sand to the sediment inflow curves, but not to the initial bed material gradation, which was based on the average of February 1989 samples. Additional adjustment to sediment inflow curves for all size classes was

required. In general, the adjustment consisted of increasing sediment inflow at low discharges and decreasing sediment inflow at high discharges.

40. The inclusion of fine sand in the sediment inflow resulted in a significant improvement in the simulation. The simulation of the November 1987 cumulative aggradation was very good. The March 1989 simulation was generally acceptable. In the March simulation, degradation was underestimated by about 10,000 cu yd between sta 12+50 and 33+50, and aggradation was underestimated by about 20,000 cu yd between sta 33+50 and 60+30. This resulted in a net underestimation of cumulative aggradation for the 10-year period of about 10,000 cu yd. If the point bar on the right descending bank is excluded from the measured March 1989 cumulative aggradation, model performance appears to be somewhat better. Measured aggradation between sta 33+50 and 60+30 is reproduced by the numerical model simulation and the total cumulative aggradation is overestimated by only 8,000 cu yd. A comparison of measured and computed cumulative aggradations is shown in Figure 9. The final adjusted sediment inflow curves are shown in Plate 23.

41. Calculated average bed material gradations between sta 33+50 and 60+30 at different times during the historical simulation were compared. This river reach was chosen because calculated data could be compared to measured data representative of the entire channel width. The initial bed material in

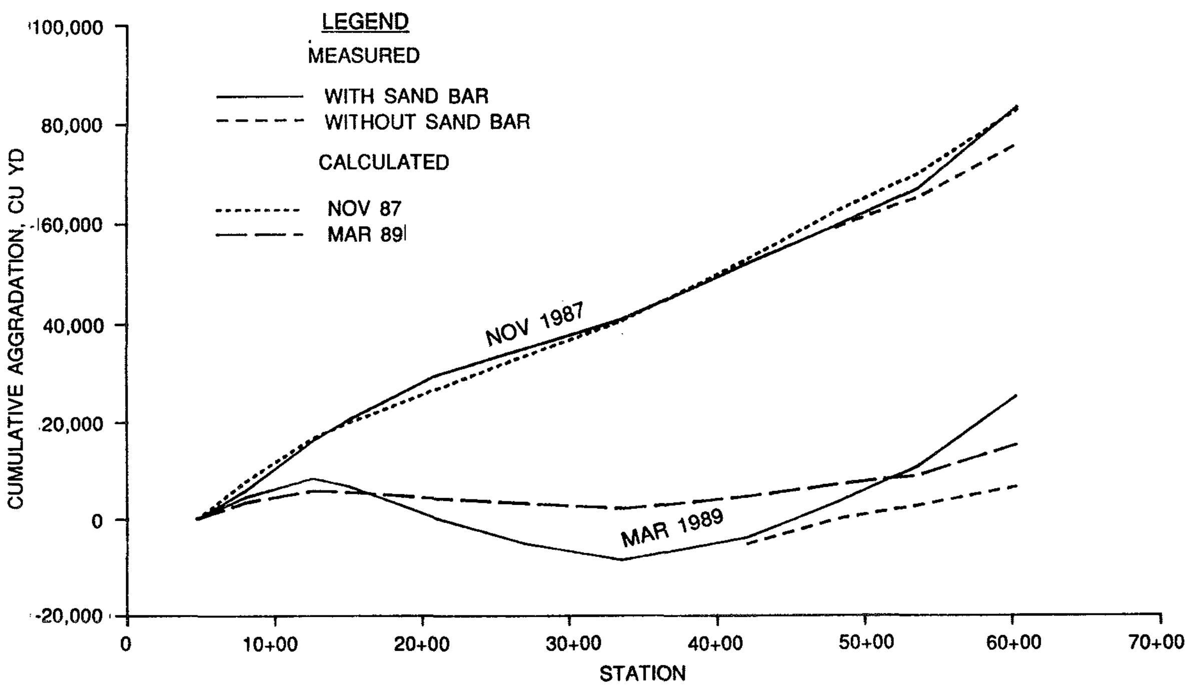

Figure 9. Comparison of measured and calculated cumulative aggradations with final adjustment of numerical model

the numerical model had a median grain diameter of 0.65 mm based on the average from samples collected in February 1989 (Figure 4). This average and the envelope of sampled data are compared with calculated bed gradations between sta 33+50 and 60+30 at three times during the 10-year simulation in Figure 10. The highest flow of record during the simulation period occurred on 30 October 1982 and the calculated gradation was considerably coarser with a median grain size of about 0.90 mm. The period of highest calculated aggradation occurred on 4 November 1987 and had a finer average bed gradation with a median grain diameter of 0.48 mm. Finally, the calculated and measured median grain size on 9 February 1989 was about 0.65 mm. The coarsest grain sizes were slightly underrepresented in the February 1989 calculated gradation; the sampled bed had about 13 percent gravel compared to a calculated bed that had about 10 percent gravel. The calculated bed also was composed of about 15 percent fine sand, compared to about 8 percent in the average sampled data. In general, the numerical model performed very well in the reproduction of bed material gradation, especially since the calculated gradations for November 1987 and October 1982 were both finer and coarser than the February 1989 calculated gradation during the course of the simulation.

42. Calculated longitudinal variations in bed material gradations for November 1987, which was the period of greatest accumulated aggradation between 1979 and 1989, are shown in Figure 11. With the addition of fine sand to the inflow, there was a significant longitudinal variation in calculated bed gradation between sta 4+70 and 60+30. The median grain size varied from 0.60 mm at the upstream station to 0.19 mm at the downstream station. This indicates that during periods of extended low flow, fine materials accumulate in the downstream portion of the channel. When calculated bed material gradations from periods with high antecedent flow are compared, it can be concluded that finer sediments are washed out at higher flows, contributing to the degradation that occurs during high-flow periods. In the numerical model, this result is attained independent of the initial bed material gradation assumptions.

43. Tests were also conducted with the numerical model to evaluate the effect of variable roughness coefficients. Sediment inflow had to be increased to simulate measured aggradation when the roughness coefficients were adjusted. Cumulative aggradation calculated with these tests is shown in Plate 24. With the variable roughness tests, sediment inflow was different,

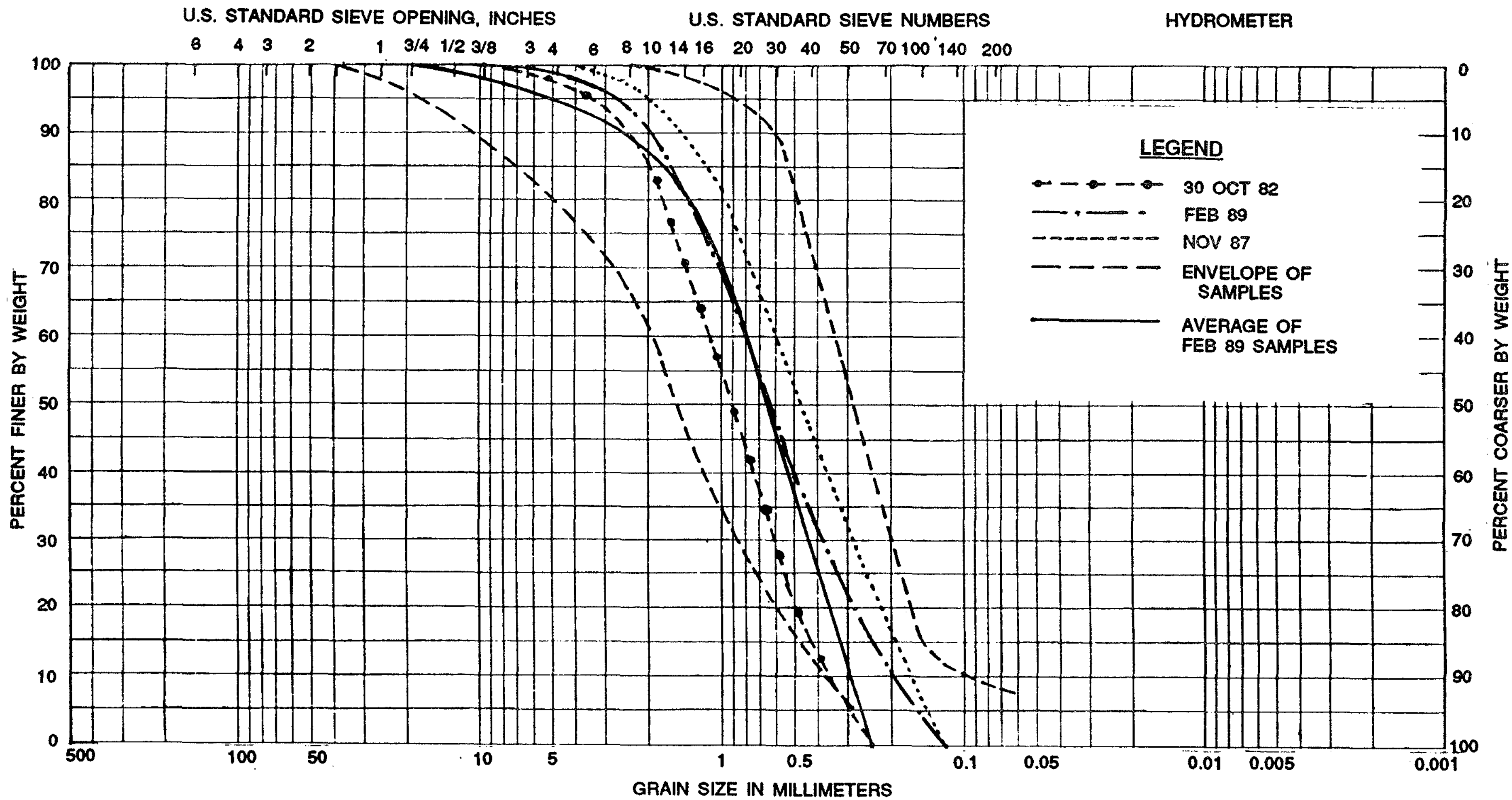

Figure 10. Calculated change in bed material gradation between sta 65+00 and 33+50

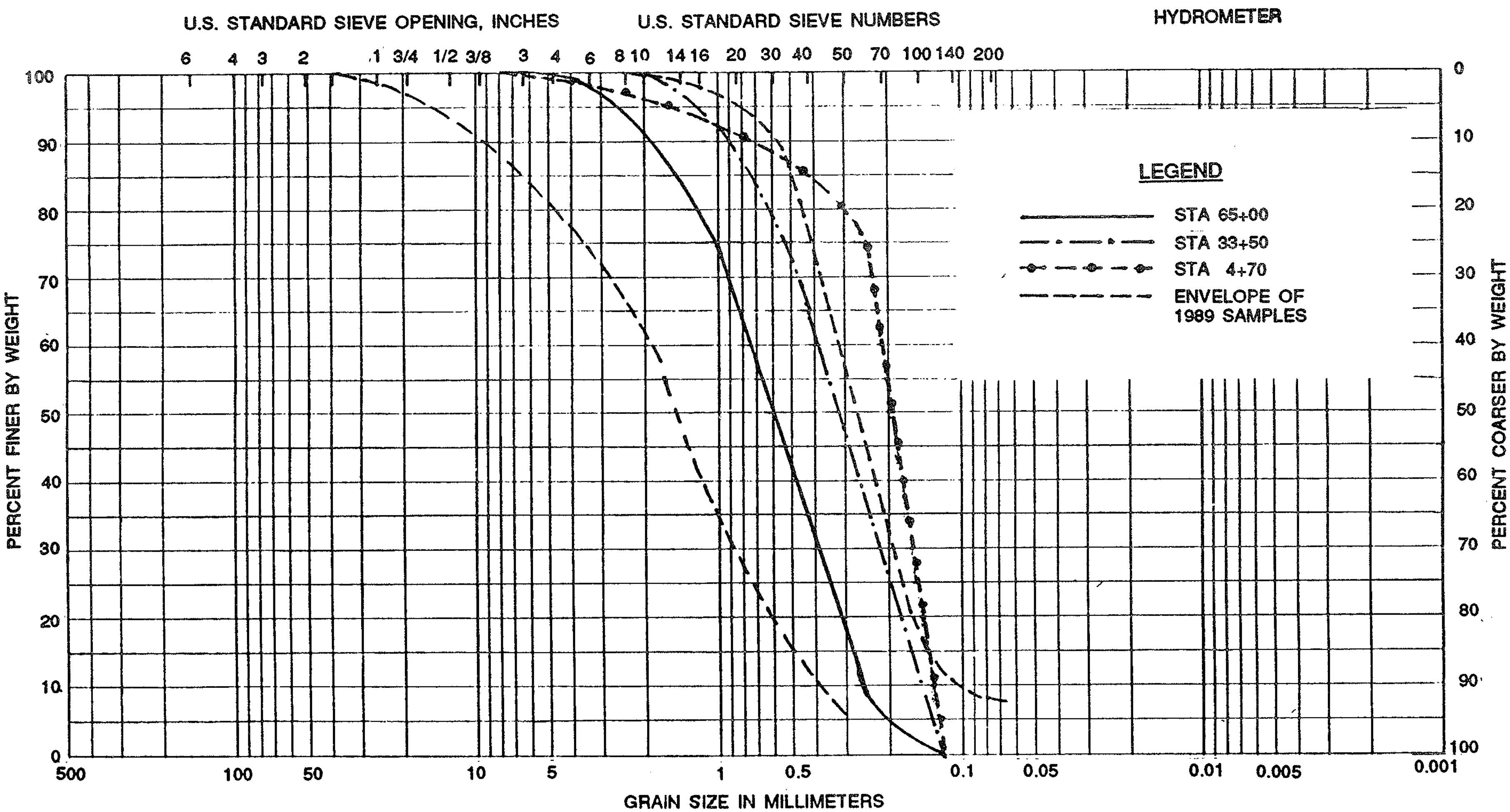

Figure 11. Calculated bed material gradations in November 1987

but the cumulative aggradation was similar to the results with constant channel roughness.

Final Adjusted Numerical Model

44. Tests conducted during the adjustment of the numerical model evaluated the effects of several factors. Some were found to affect calculated results significantly and others did not. The final adjusted numerical model incorporated the features that were found to significantly affect the simulation of measured aggradation and bed composition. The model incorporated multiple grain sizes from fine sand to coarse gravel. The Laursen-Copeland multiple grain size transport function was used. The average bed material gradation determined from February 1989 samples was used as an initial bed material gradation. A constant roughness coefficient was used to be consistent with previous HEC-2 backwater studies. Sediment inflow rating curves were not varied with time.

PART IV: STUDY RESULTS

Design Flood

45. The adjusted numerical model was used to evaluate bed response during the design flood. The design flood is the 100-year-frequency flood with a peak discharge of 64,000 cfs downstream from the confluence of the Makaweli and Waimea Rivers. Two initial cross-section geometries and bed conditions were tested. November 1987 surveyed cross sections and the average February 1989 sampled bed were used as initial conditions in the first test. January 1979 surveyed cross sections with calculated changes from the January 1979-November 1987 historical simulation were used as initial conditions in the second test. In both tests the numerical model predicted that the bed would degrade during the passage of the design flood. In the first test, as shown in Figure 12, degradation of about 140,000 cu yd was calculated between

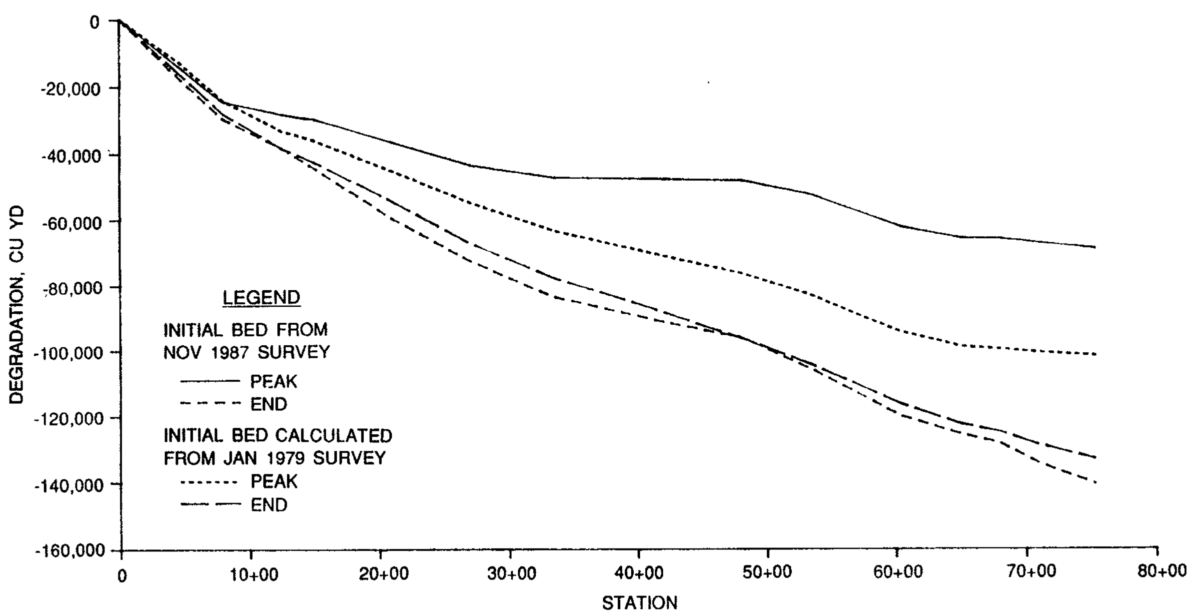

Figure 12. Accumulated degradation during design flood from November 1987 bed conditions

sta 0+00 and 75+20 during the design flood. About 70,000 cu yd of this total was removed by the time the peak occurred. In the second test, about 130,000 cu yd were degraded during the design flood. Of this total, 100,000 cu yd were eroded before the peak occurred. The greater quantity removed before the peak in the second test is attributed to a finer bed, which developed during the historical runoff period antecedent to the design flood.

Computed bed changes at the peak and at the end of the design flood are compared in Plates 25 and 26. Calculated bed changes during the course of the design flood at three cross sections from the second test are shown in Plates 27-29.

Predicted Sedimentation Trends During Historical Runoff Period

46. The predictive capability of the numerical model was used to evaluate sedimentation trends during a 7-year period that contained normal flow and major flood events. The 1944-1950 hydrograph was used in the model for this purpose. Mean annual runoff in the Waimea River for the period 1944 to 1988 is shown in Figure 13. It can be seen that the period between 1944 and 1950 was not a particularly high runoff period in terms of mean annual flow. However, this period contained the flood of record (45,000 cfs in

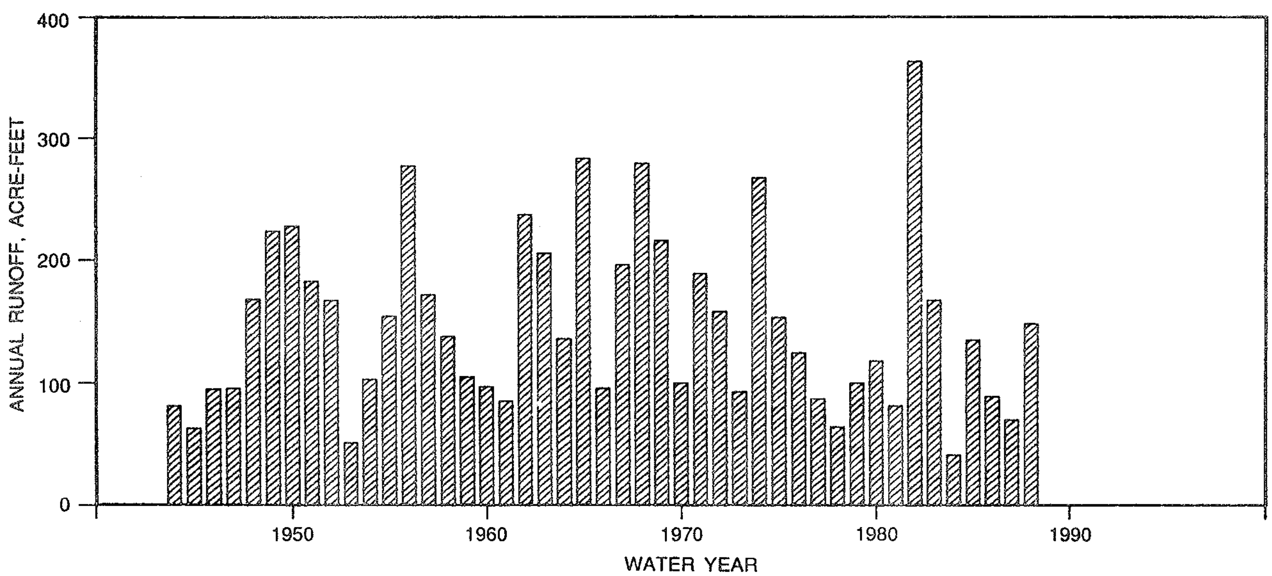

Figure 13. Mean annual runoff for Waimea River

February 1949) and another high flow (32,000 cfs in August 1950). Calculated cumulative aggradation before and at the end of the February 1949 flood and at the end of the 1944-1950 simulation are shown in Figure 14. A progression of bed changes at three cross sections during the 1944-1950 simulation is shown in Plates 30-32, and during the 1979-1989 simulation in Plates 33-35. On these plates, the abscissas are discontinuous because days with mean daily discharges of less than 200 cfs are excluded. Considering the results of

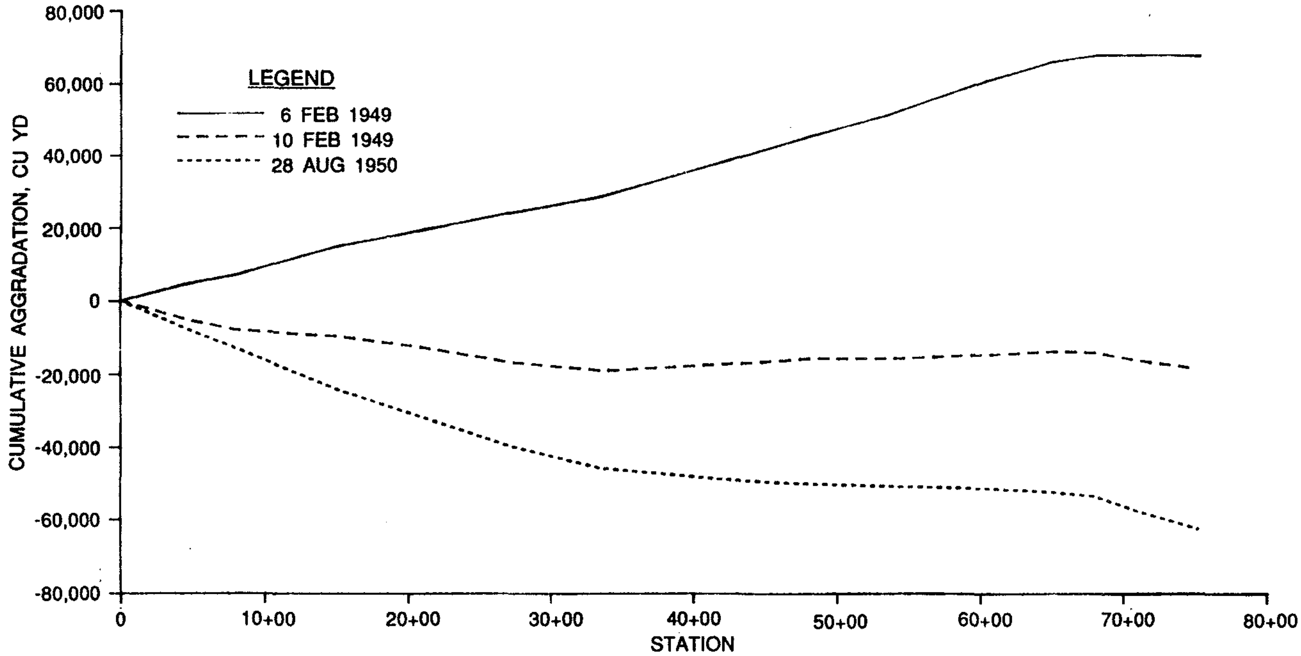

Figure 14. Predicted cumulative aggradation, 1944-1950 hydrograph

these simulations, it can be concluded that general aggradation occurs in the Waimea River during periods of low flows and that rapid and significant degradation occurs during major floods. A long-term trend toward aggradation or degradation was not identified with the numerical model. However, survey data indicate that more aggradation upstream from sta 33+50 occurs during low-flow periods then is degraded during high-flow periods. This aggradation occurs primarily on a point bar located between sta 42+00 and 60+30 on the right descending bank. Eventually, dredging will be required to maintain design flood capacity.

PART V: CONCLUSIONS AND RECOMMENDATIONS

Conclusions

47. It is improbable that recovery of the Makaweli basin from the 1981 landslide is sufficient cause for the degradation trend that was observed between November 1987 and March 1989, although it is certainly possible that basin recovery is a factor in the degradation trend.

48. The adjustment phase of the numerical model study indicated that limited supply of finer sediments at high flows was the principal cause of degradation at high flow. The historical simulation indicated that the Waimea River bed is dynamic and can be expected to fluctuate with changes in discharge. Long periods of low flow will result in more aggradation. Major storm events will tend to wash material out of the flood-control channel.

49. The results of the numerical sedimentation model study can be used to adjust the existing HEC-2 backwater model to consider the effects of sedimentation on design water surface. The HEC-2 cross-section geometry, which is based on a January 1979 cross-section survey, can be adjusted by adding or subtracting calculated bed changes from the sedimentation model. The point bar between sta 42+00 and 60+30 does not appear to be accounted for in the numerical sedimentation model simulations. Therefore, cross sections in the HEC-2 model should be revised, based on the March 1989 survey data, to include the conveyance loss due to this bar. Using the calculated bed changes from the numerical sedimentation model study should result in a conservative design water-surface elevation, because the degradation from the design flood is preceded by the greatest calculated accumulation in the channel for the 10-year period between 1979 and 1989.

50. A long-term aggradation or degradation trend was not identified with the hydrographs tested. However, the upstream portion of the channel does not appear to degrade as much as the downstream portion of the river during high-flow events. Some channel excavation may eventually be necessary in this upstream reach.

Recommendations

51. The results of this study should be used with a HEC-2 backwater

model to test the backwater effect of the Belt Highway Bridge. A Manning's roughness coefficient of 0.025 is recommended for the channel to determine design water-surface elevations, even though the sediment study indicated that a value of 0.020 might be appropriate due to washout of bed forms. The 0.020 roughness coefficient should be used to calculate design velocities. It is recommended that the overbank roughness values be increased to 0.120 to improve the conveyance distribution in the overbank area.

52. Changes in aggradation and degradation in the Waimea River should be monitored on an annual basis. If aggradation exceeds that surveyed in November 1987, the numerical sedimentation model should be used to determine new bed changes for the design flood. The growth of the point bar between sta 42+00 and 60+30 should also be monitored and changes evaluated using the HEC-2 backwater model.

53. Channel excavation may eventually be required in the study reach. It is likely that the area between sta 33+50 and 75+20 will be the first to require excavation. The TABS-1 model should be used to optimize dredging localities and quantities. Evaluation of the entire historical hydrologic record (1944-1989) would provide additional insight into maximum expected channel aggradation and long-term trends.

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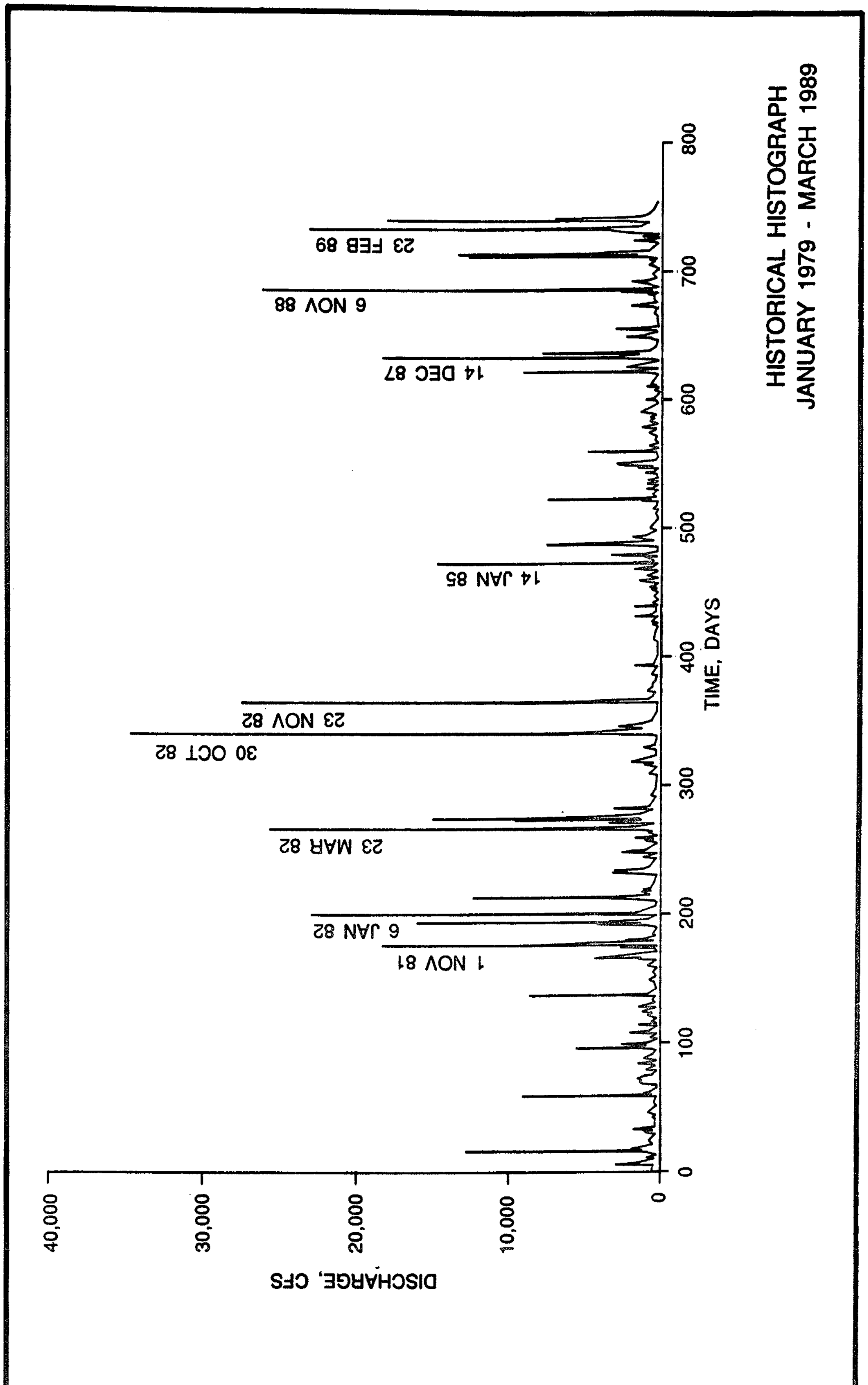
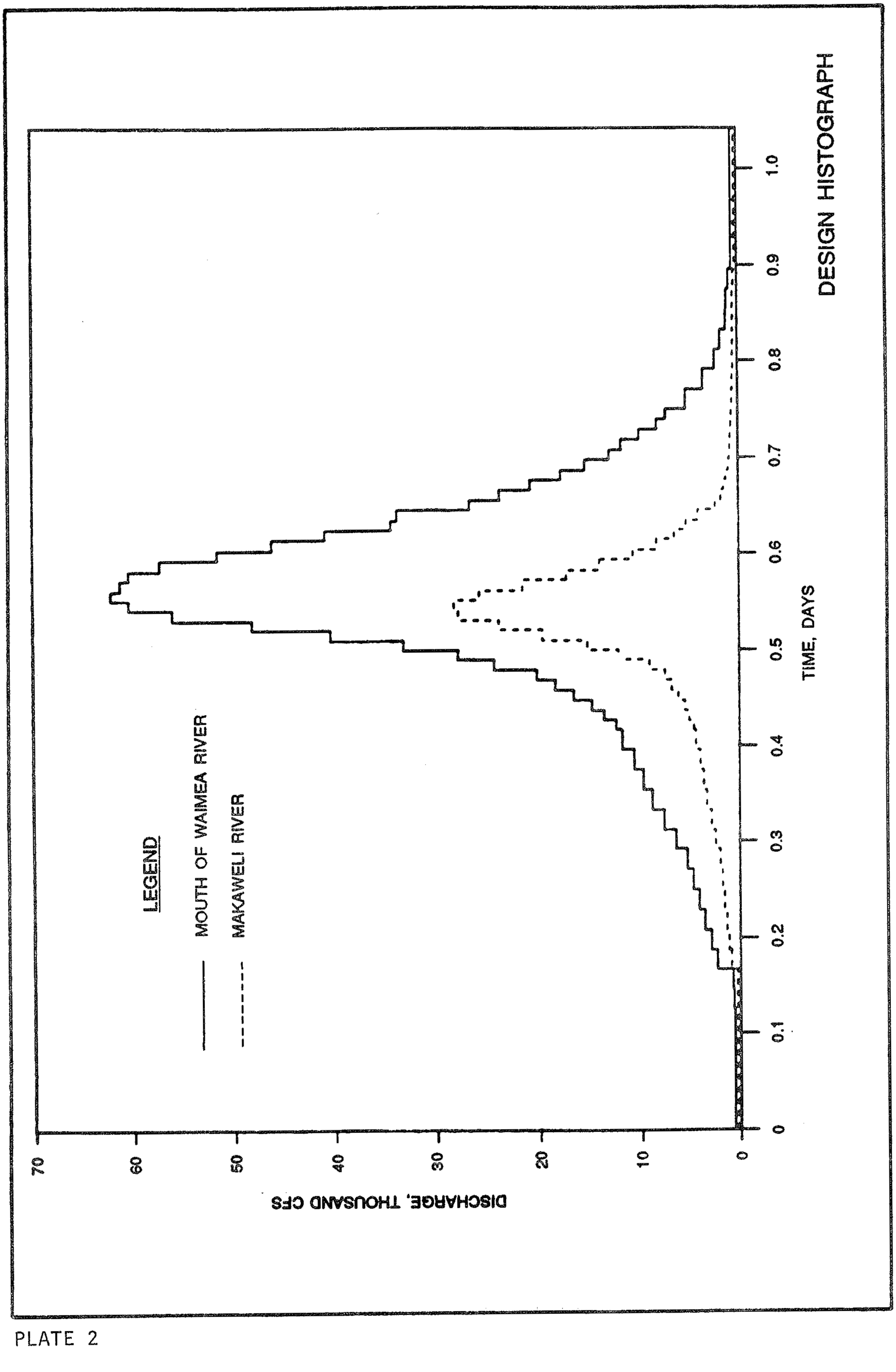
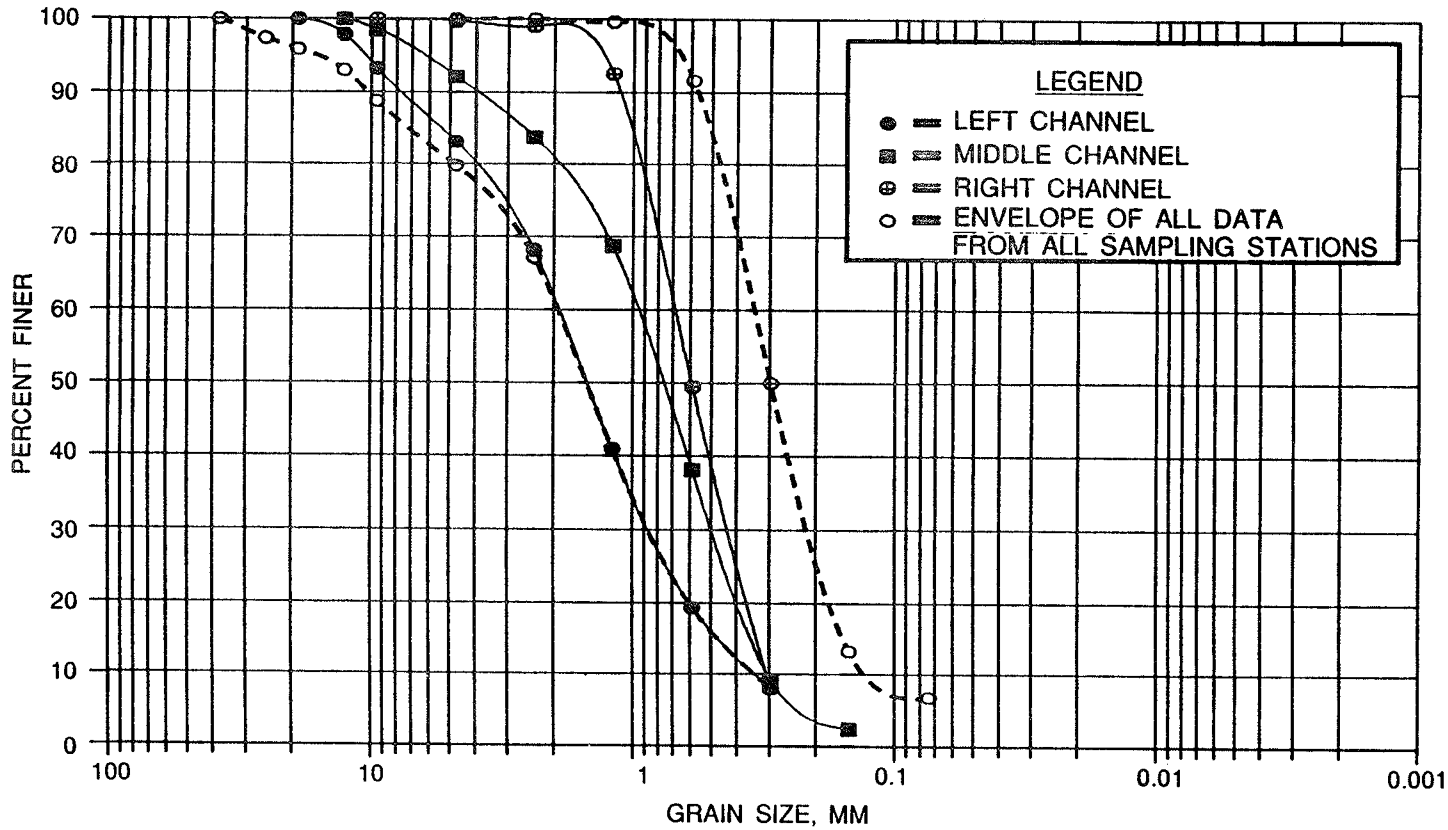

HISTORICAL HISTOGRAPH
JANUARY 1979 - MARCH 1989
DISCHARGE, CFS
TIME, DAYS
40,000
30,000
20,000
10,000
0
100
200
300
400
500
600
700
800
1 NOV 81
6 JAN 82
23 MAR 82
30 OCT 82
23 NOV 82
14 JAN 85
14 DEC 87
6 NOV 88
23 FEB 89

PLATE 1

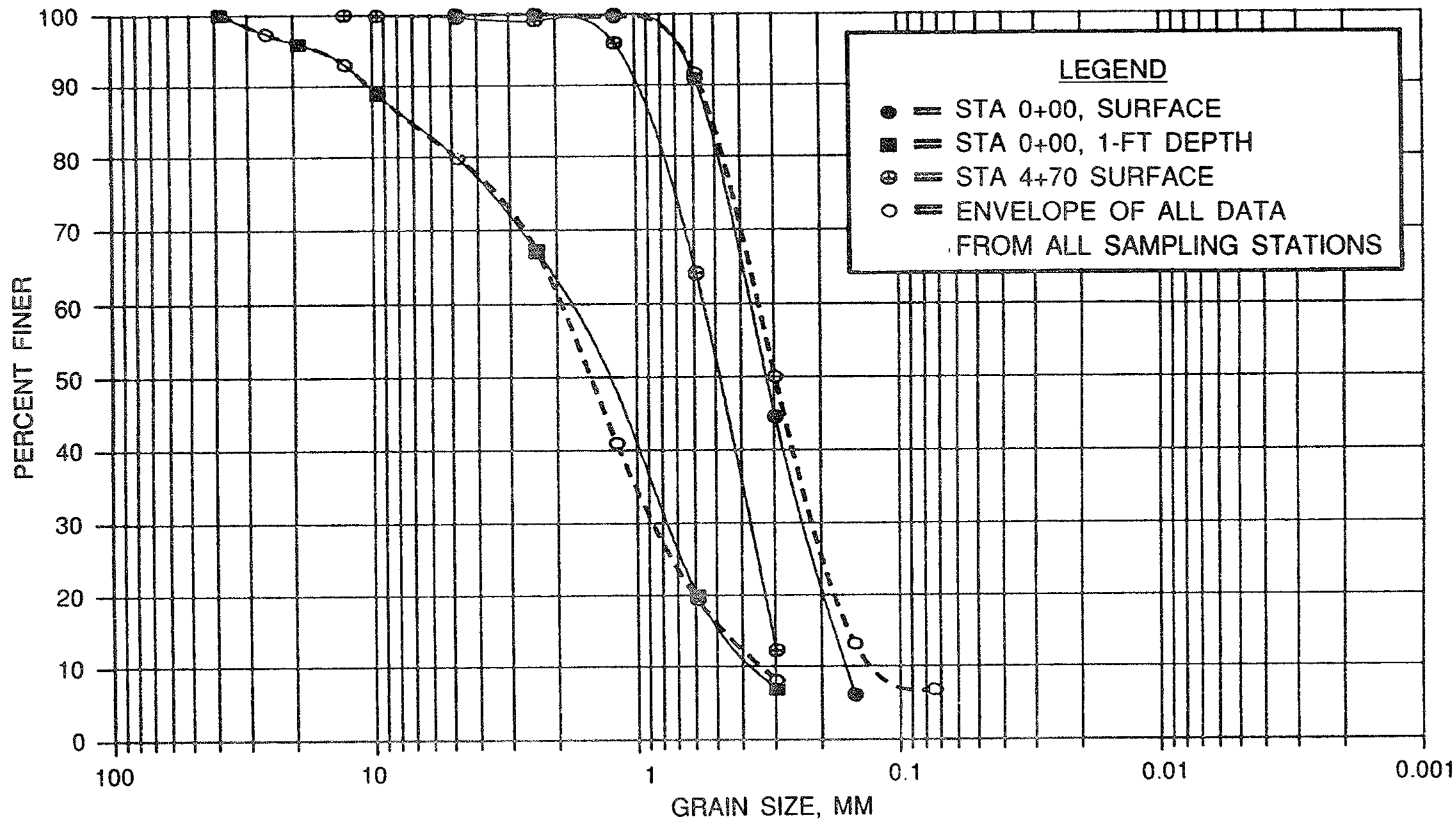


DESIGN HISTOGRAPH
LEGEND
MOUTH OF WAIMEA RIVER
MAKAWELI RIVER
DISCHARGE, THOUSAND CFS
TIME, DAYS
0
10
20
30
40
50
60
70
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0

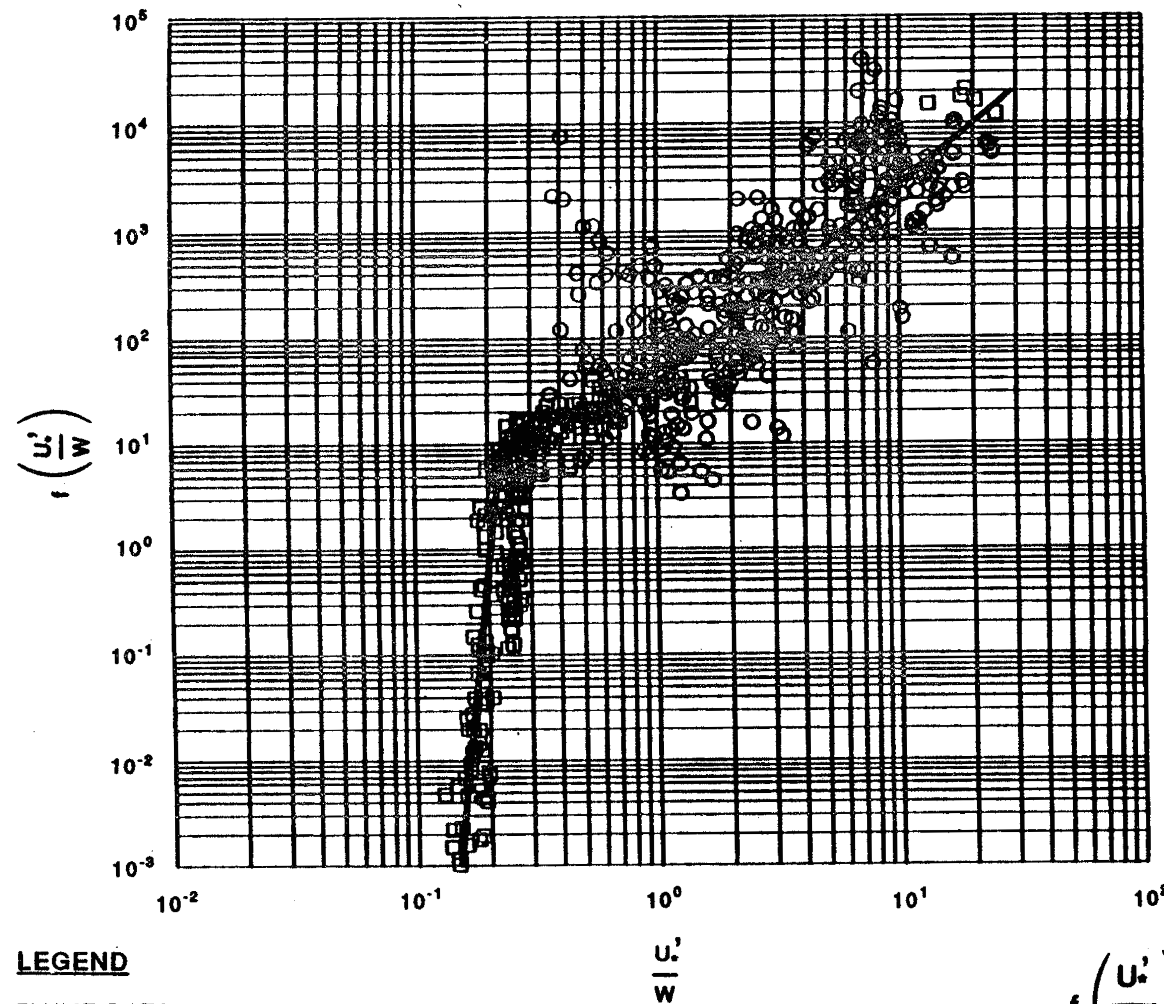
PLATE 2



LATERAL VARIATION IN BED MATERIAL
GRADATIONS AT STA 48+00



BED MATERIAL GRADATIONS ON EXPOSED
BARS NEAR MOUTH OF WAIMEA RIVER

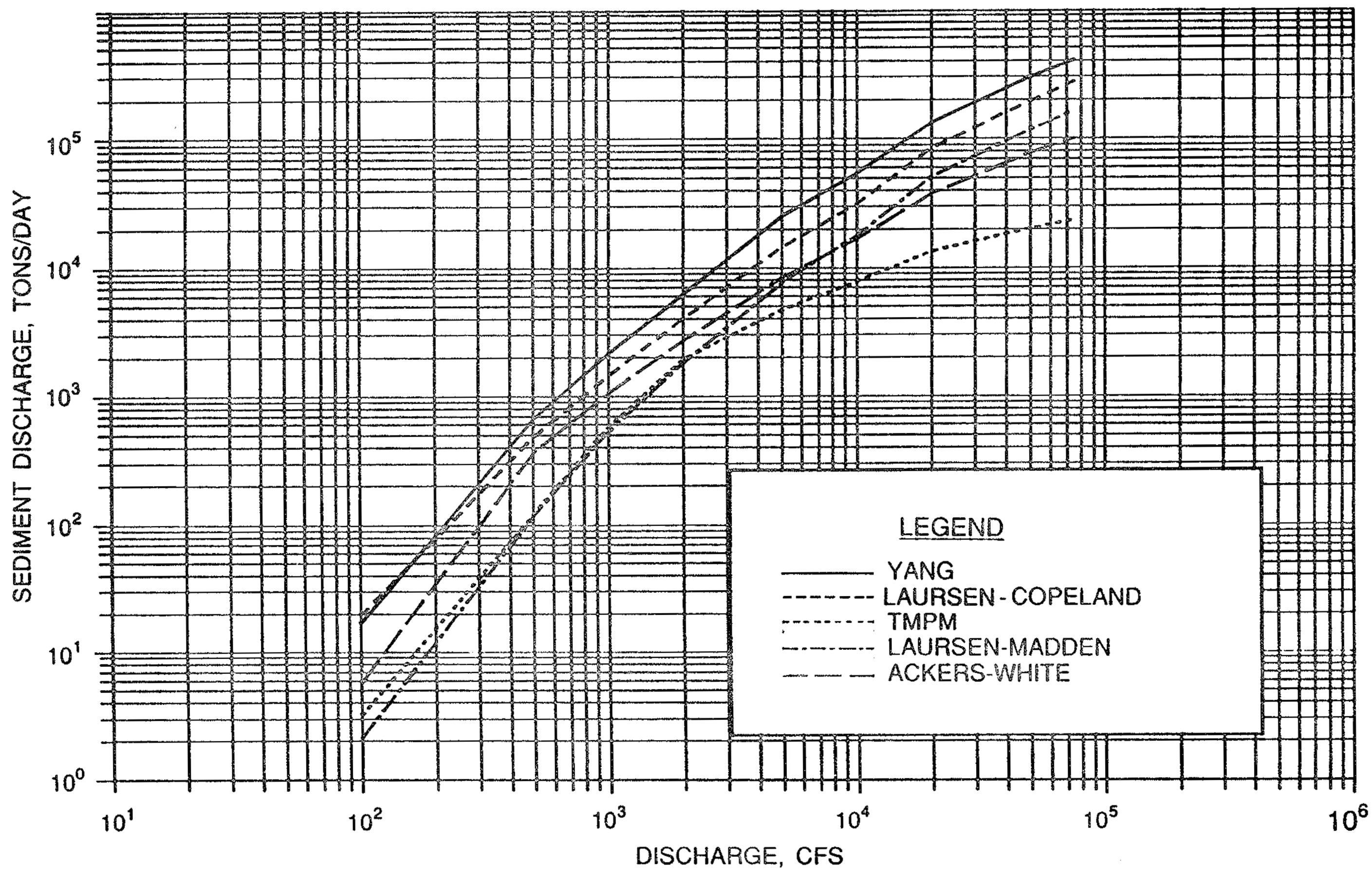


LEGEND

□ FLUME DATA

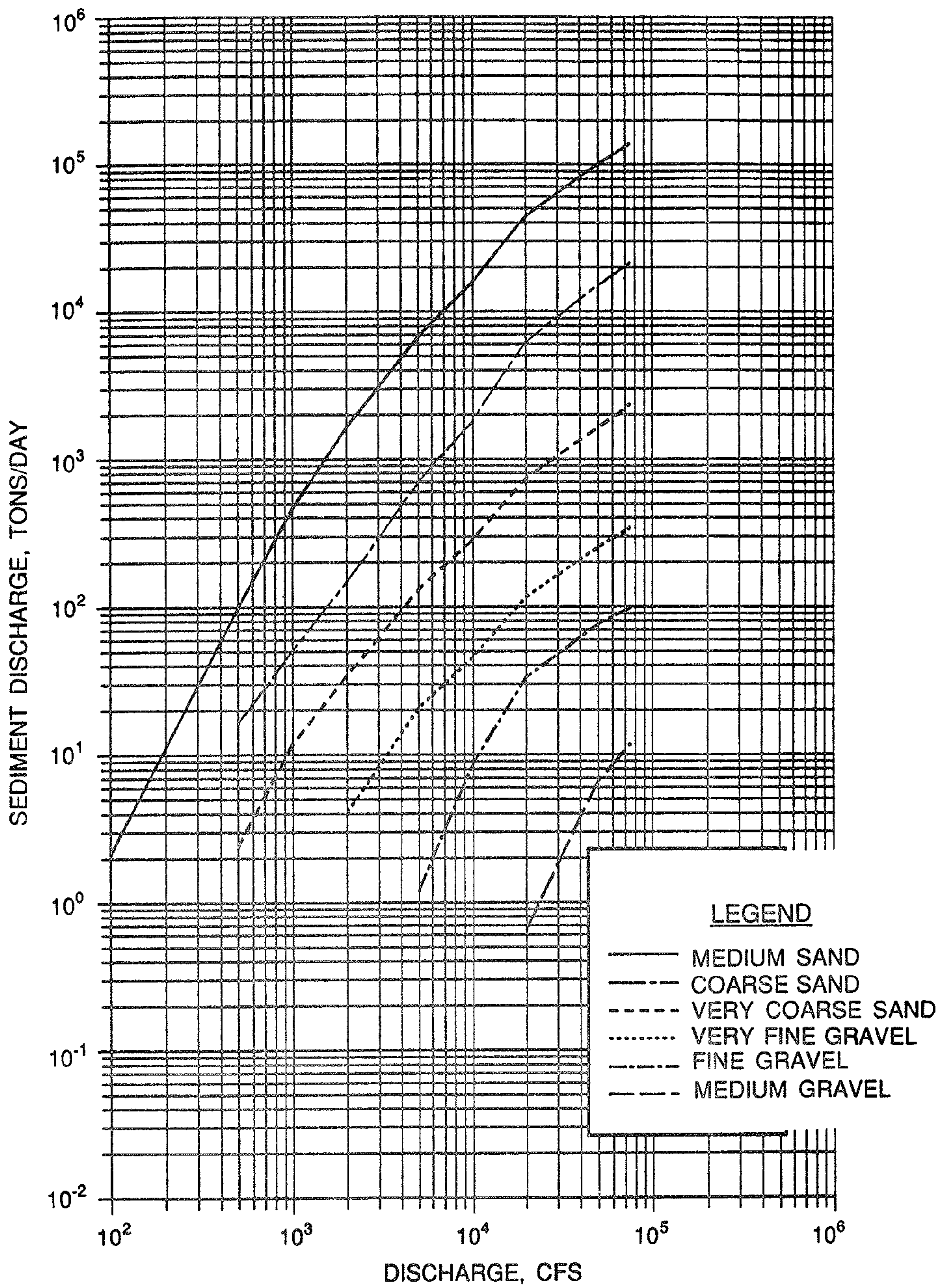
○ RIVER DATA

$f\left(\frac{U_*'}{W}\right)$ **FOR SEDIMENT TRANSPORT FUNCTION**

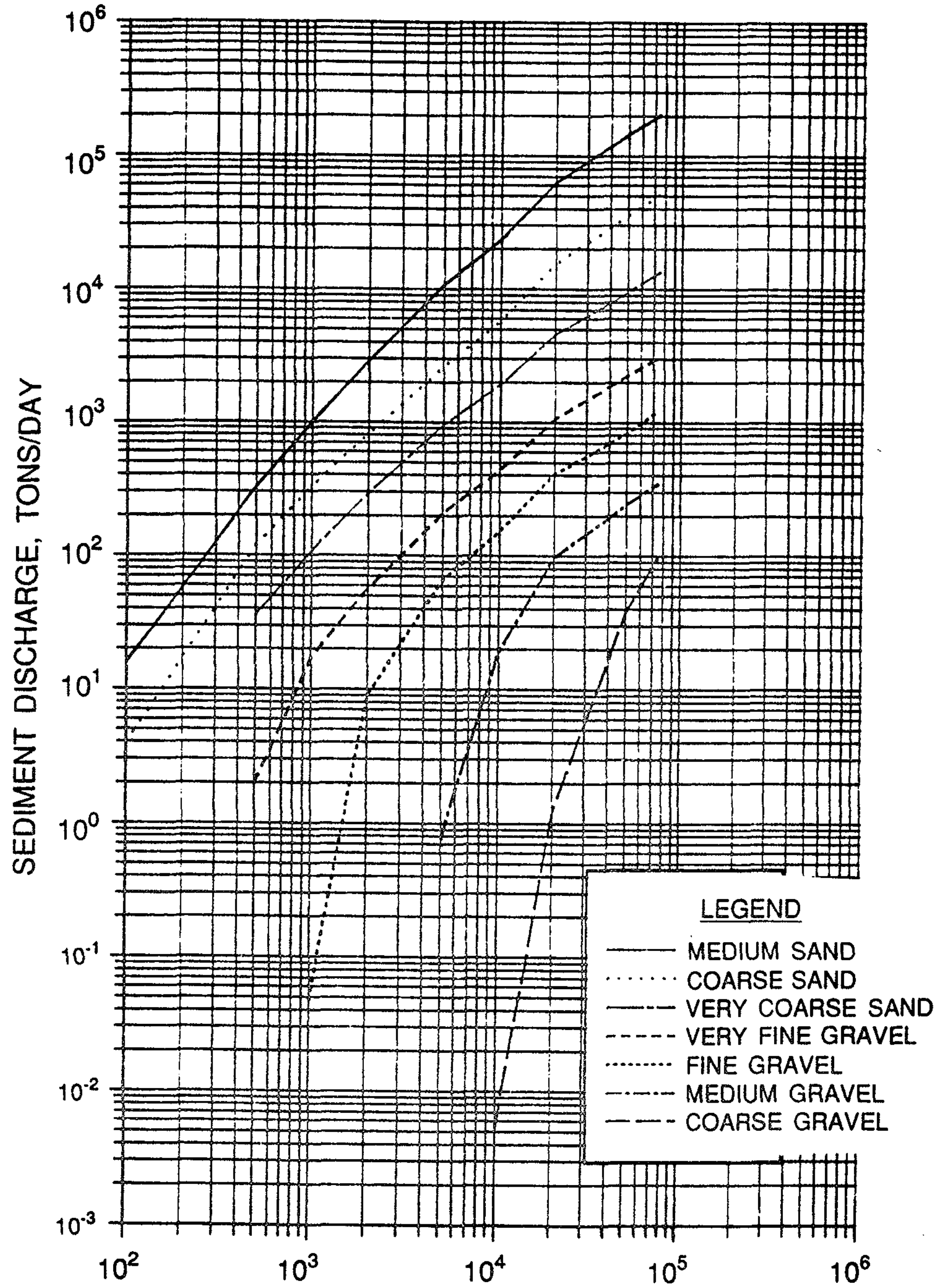


INITIAL SEDIMENT INFLOW

PLATE 6

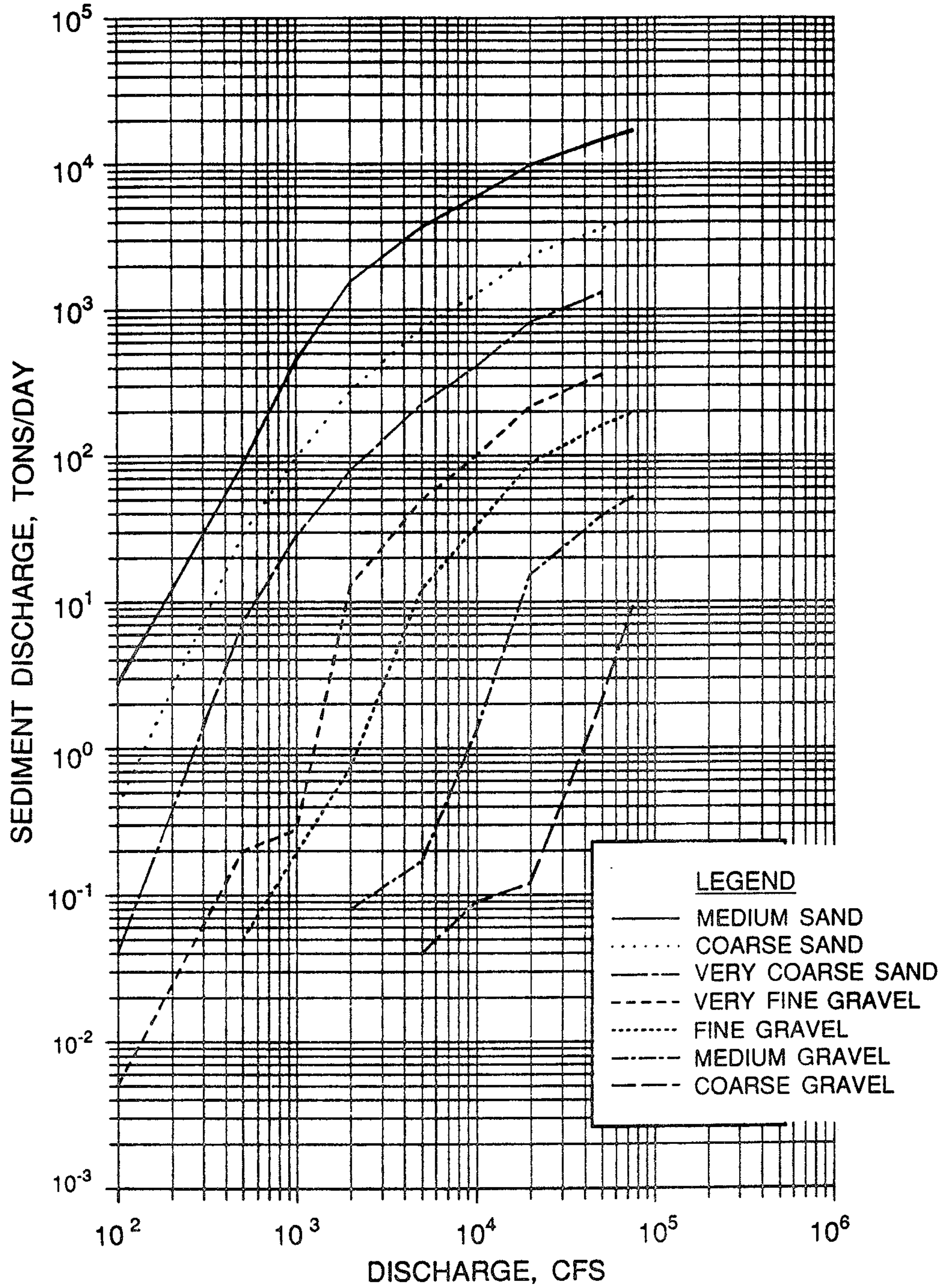


INITIAL SEDIMENT INFLOW
LAURSEN-MADDEN FUNCTION

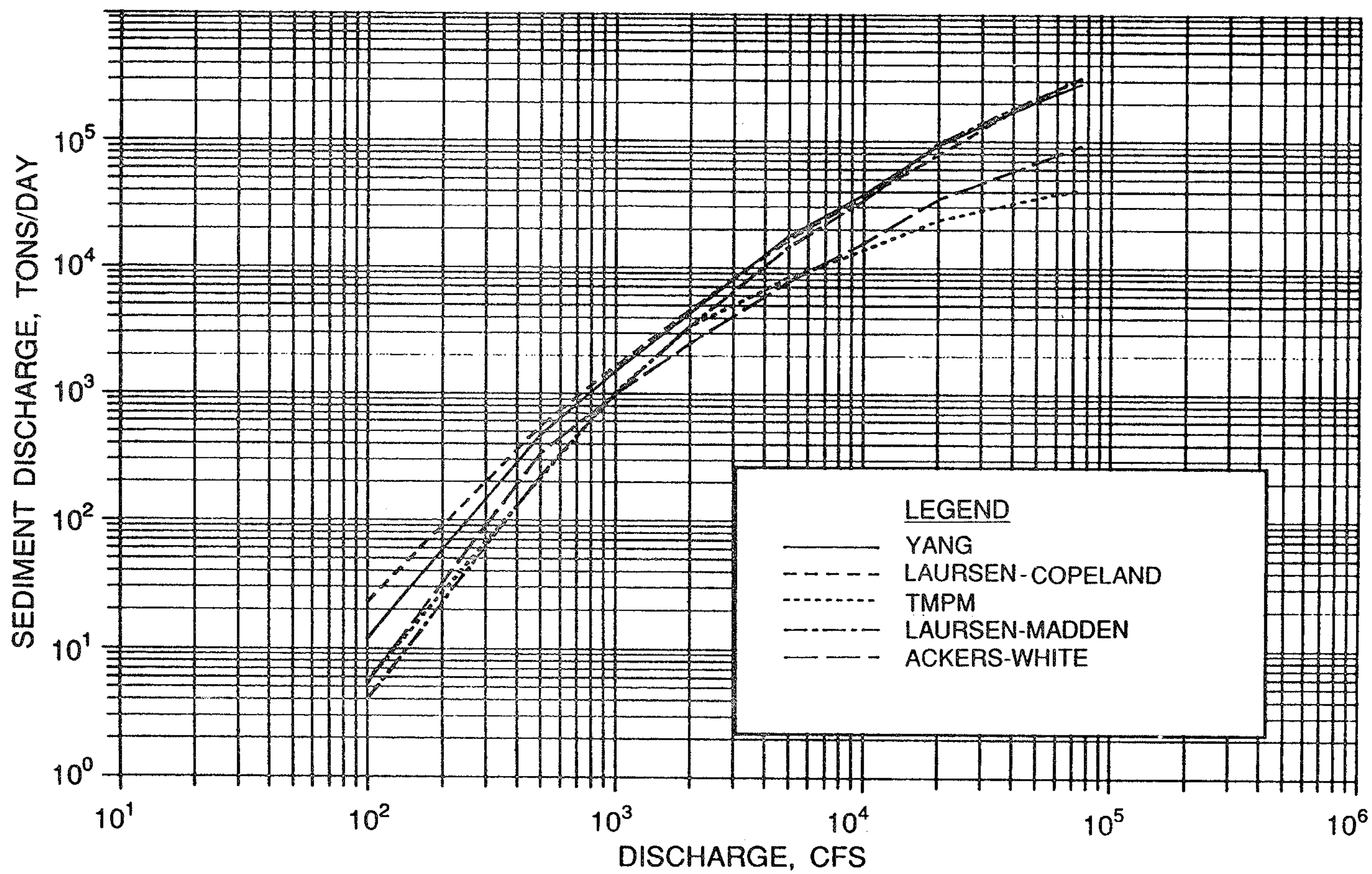


INITIAL SEDIMENT INFLOW
LAURSEN - COPELAND FUNCTION

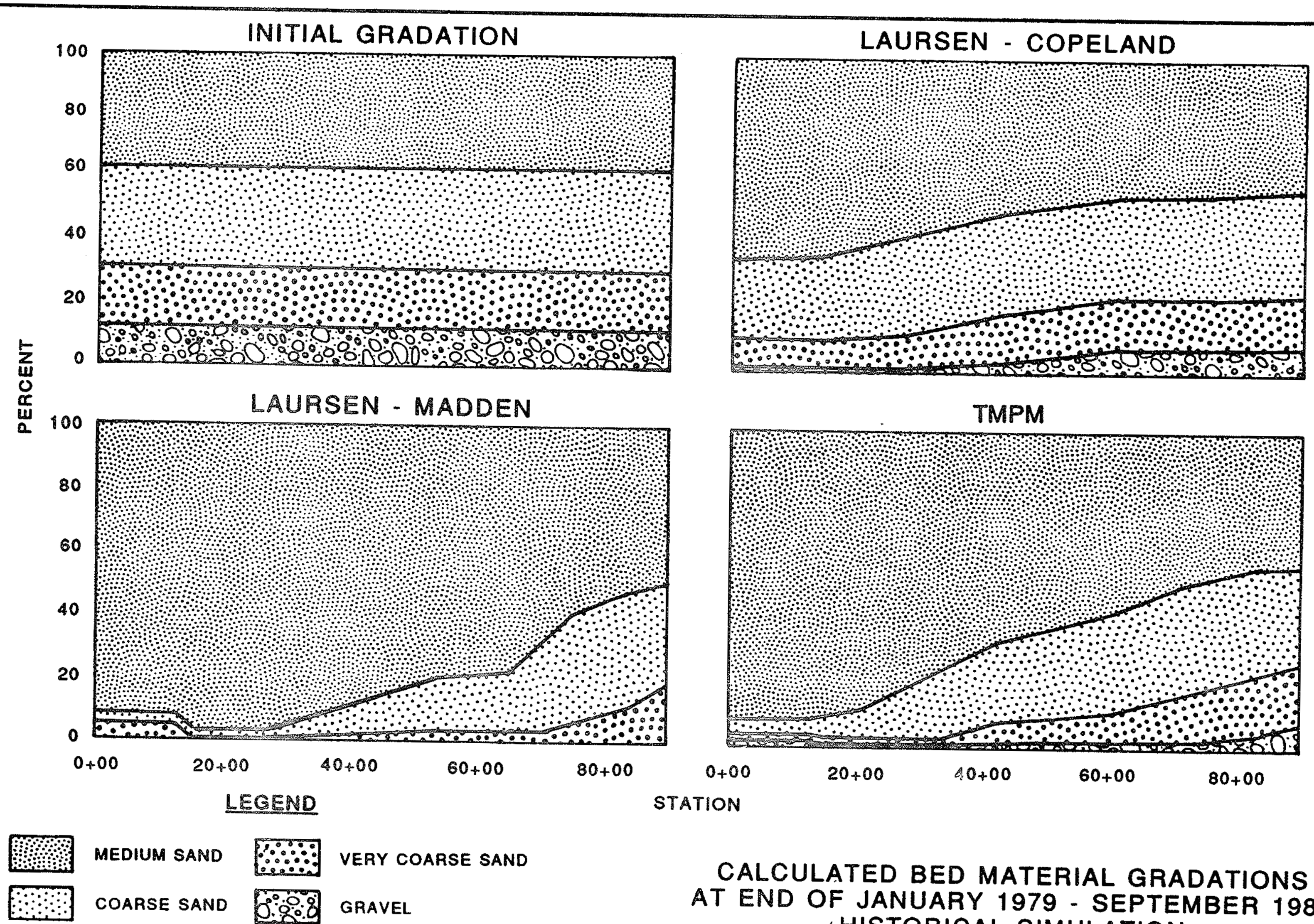
PLATE 8



INITIAL SEDIMENT INFLOW
TMPM FUNCTION



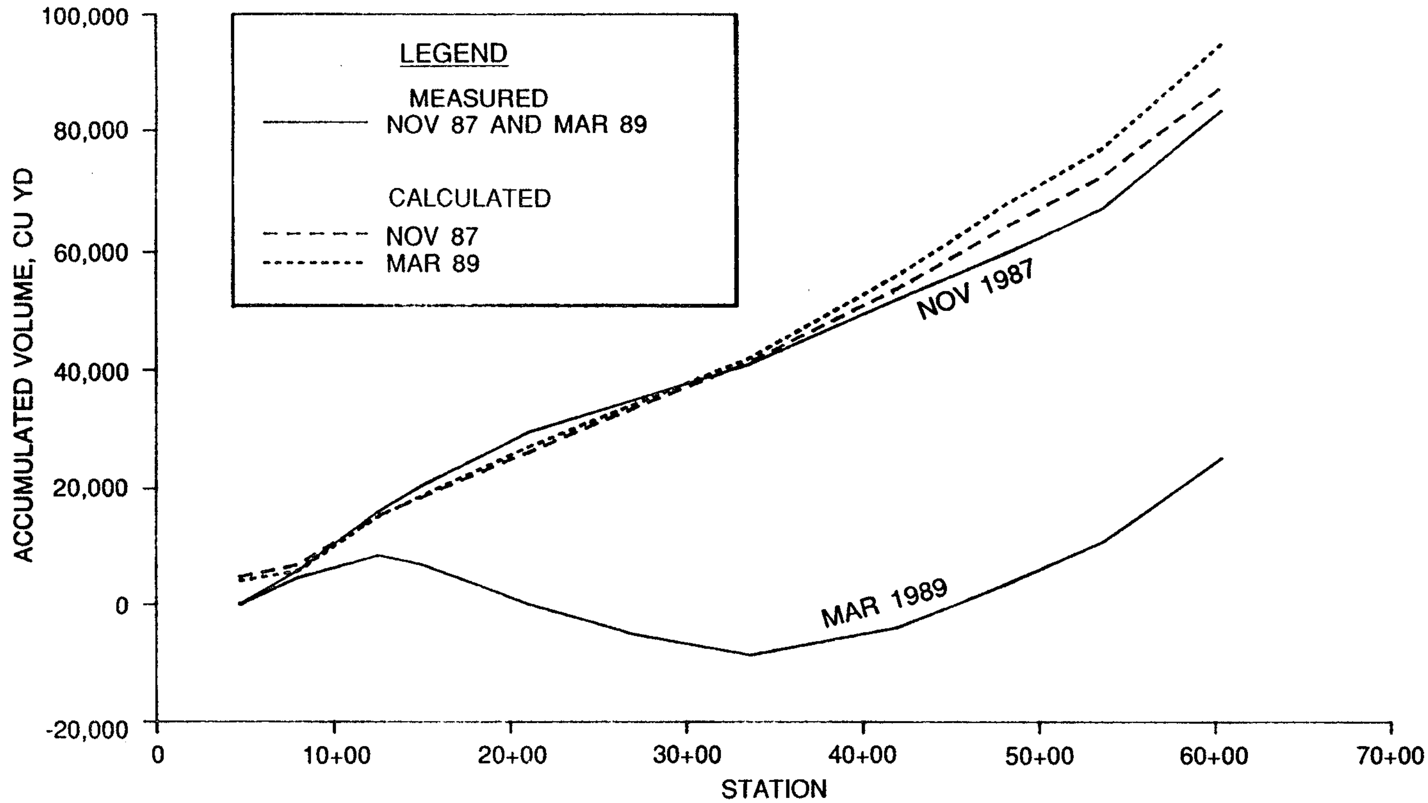
INITIAL ADJUSTED SEDIMENT FLOW



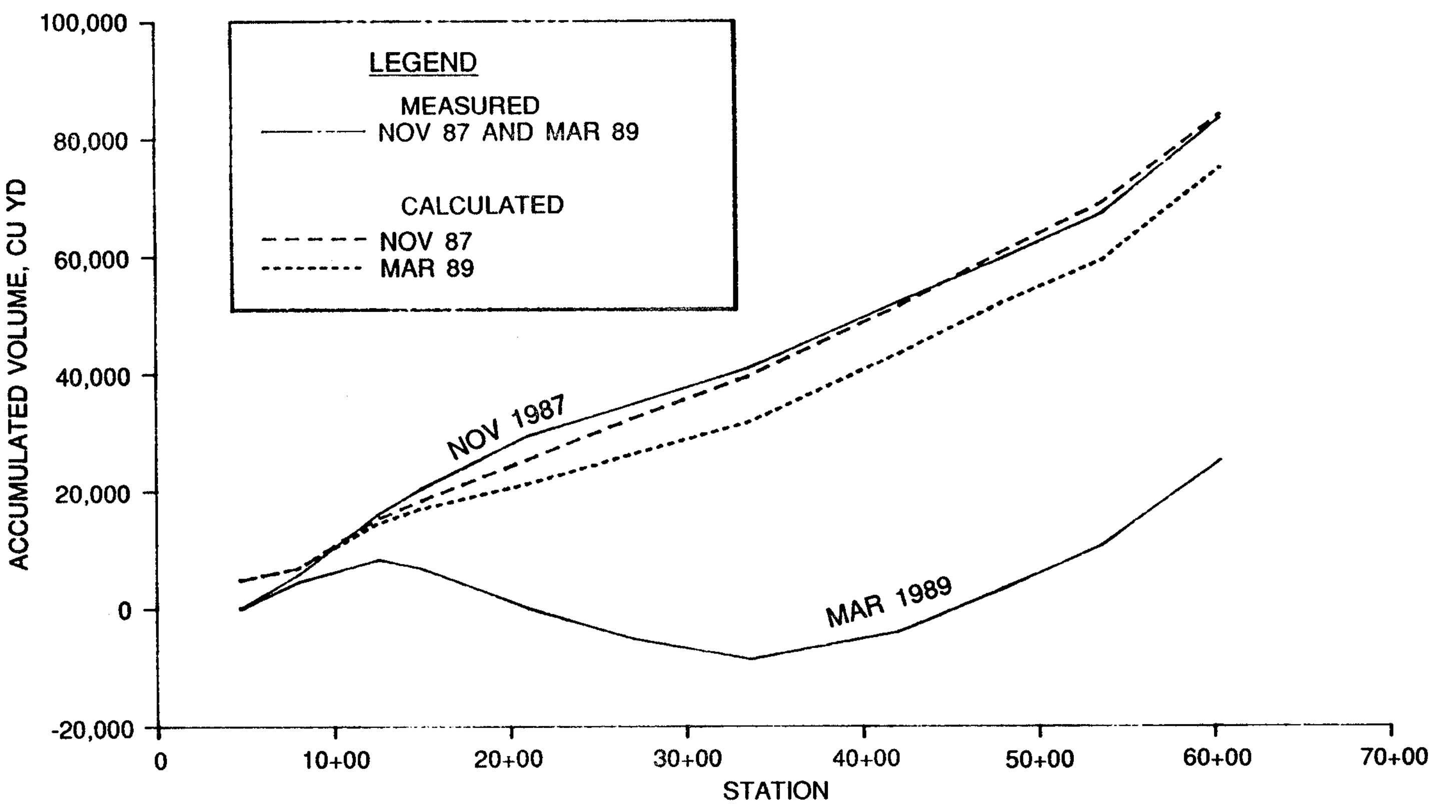
CALCULATED BED MATERIAL GRADATIONS
AT END OF JANUARY 1979 - SEPTEMBER 1987
HISTORICAL SIMULATION

PLATE 11

PLATE 12

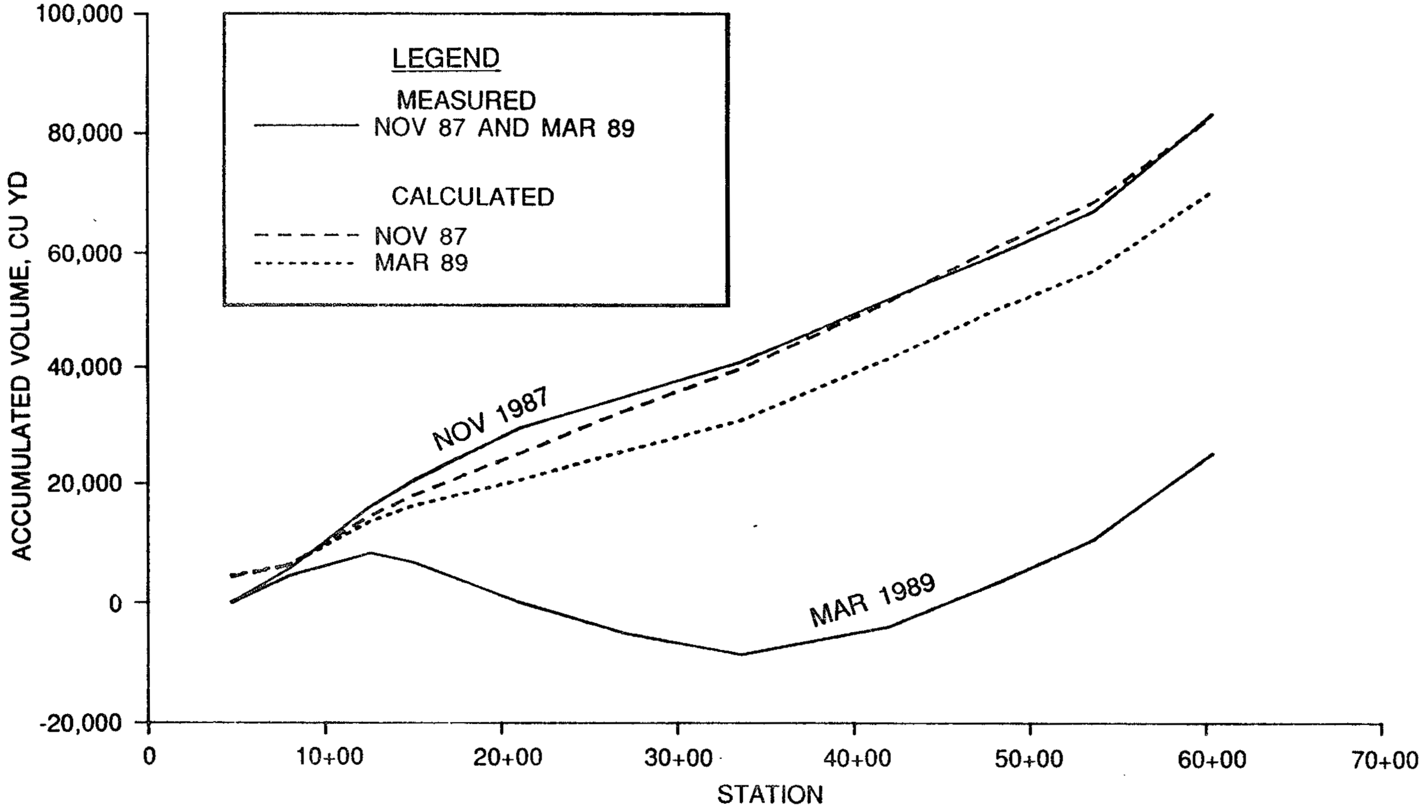


SEDIMENT INFLOW ADJUSTMENT
CALCULATED CUMULATIVE AGGRADATION
WITH INITIAL ADJUSTMENT
ACKERS-WHITE FUNCTION

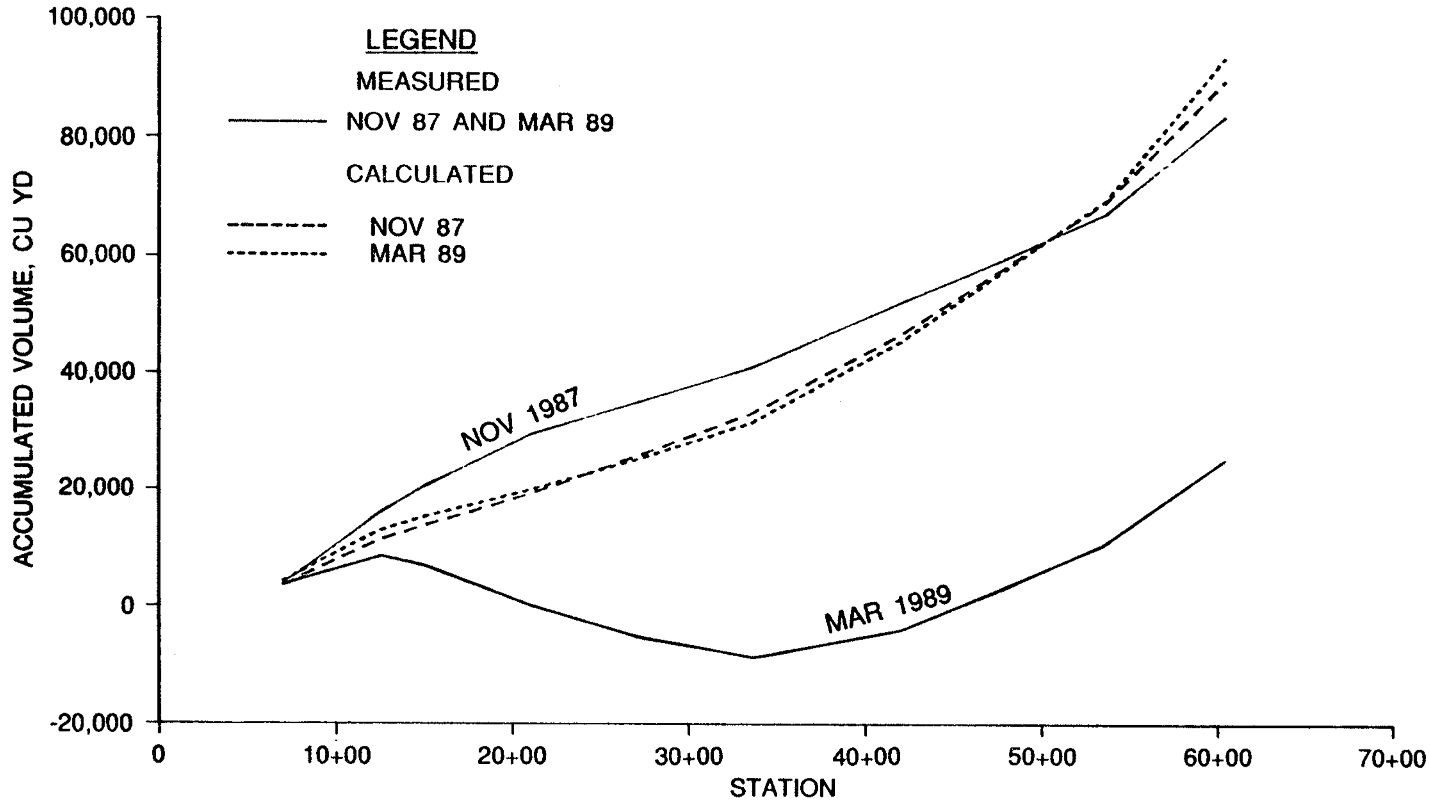


SEDIMENT INFLOW ADJUSTMENT
CALCULATED CUMULATIVE AGGRADATION
WITH INITIAL ADJUSTMENT
YANG FUNCTION

PLATE 14



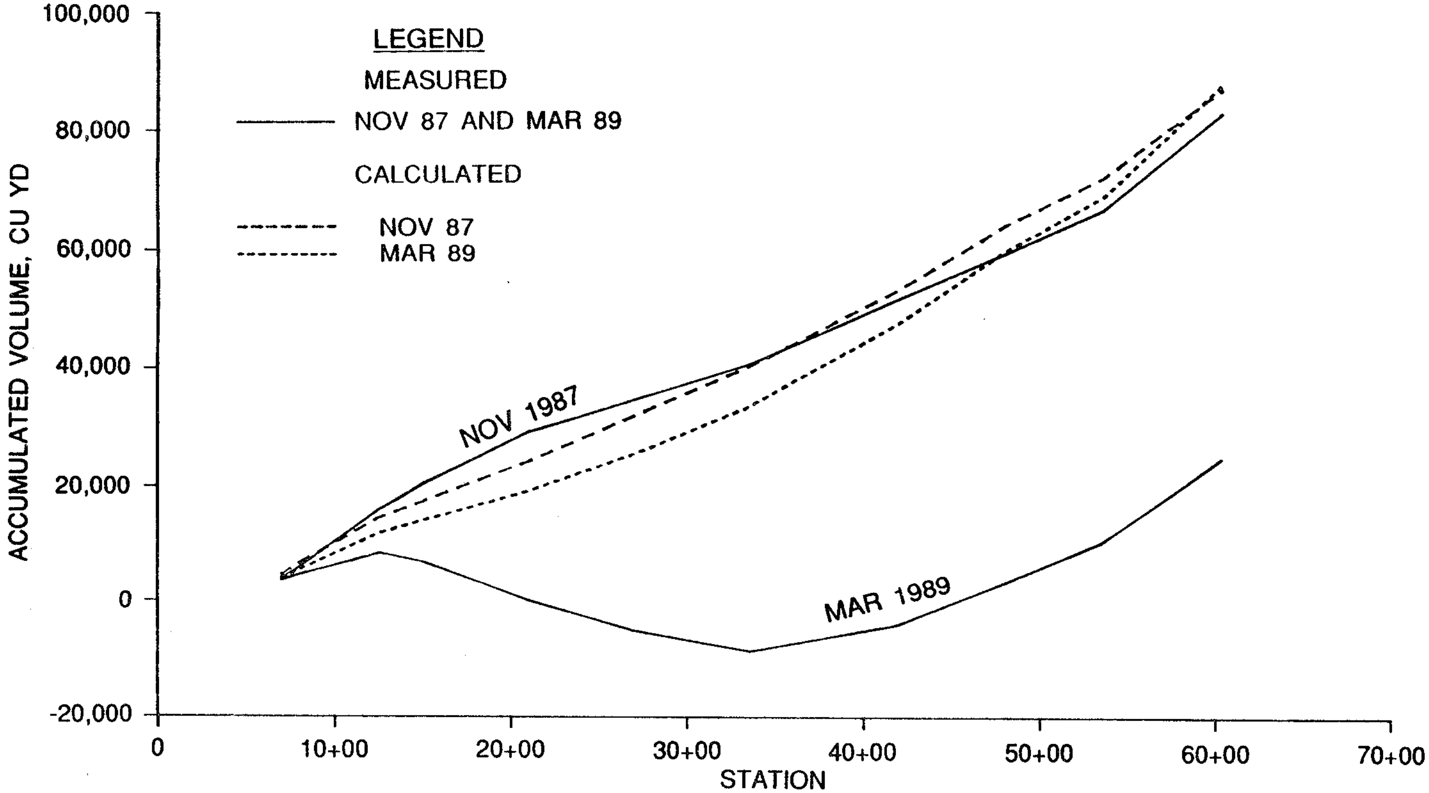
SEDIMENT INFLOW ADJUSTMENT
CALCULATED CUMULATIVE AGGRADATION
WITH INITIAL ADJUSTMENT
LAURSEN-COPELAND FUNCTION



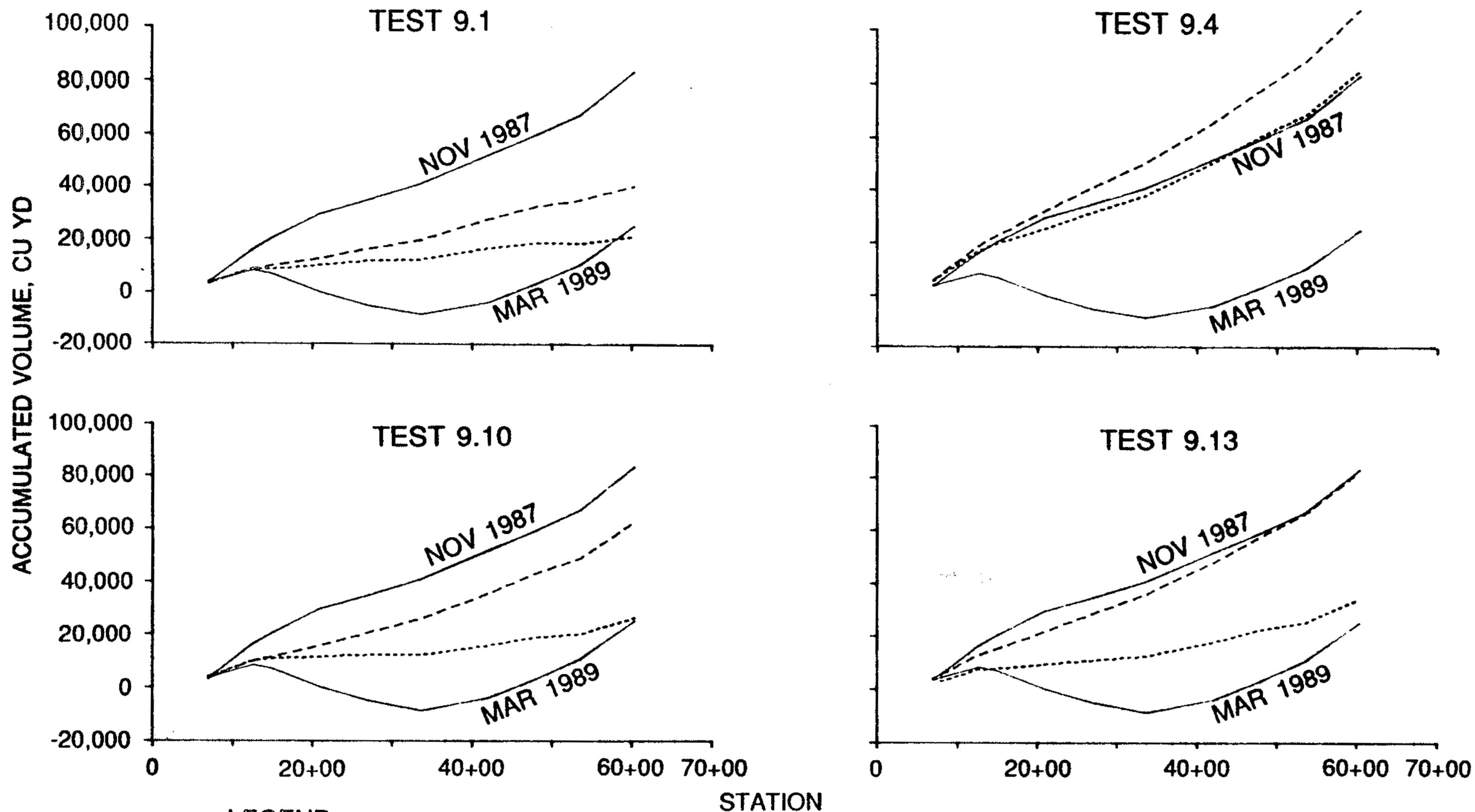
SEDIMENT INFLOW ADJUSTMENT
CALCULATED CUMULATIVE AGGRADATION
WITH INITIAL ADJUSTMENT
LAURSEN-MADDEN FUNCTION

PLATE 15

PLATE 16



SEDIMENT INFLOW ADJUSTMENT
CALCULATED CUMULATIVE AGGRADATION
WITH INITIAL ADJUSTMENT
TMPM FUNCTION



LEGEND

MEASURED

NOV 87 AND MAR 89

CALCULATED

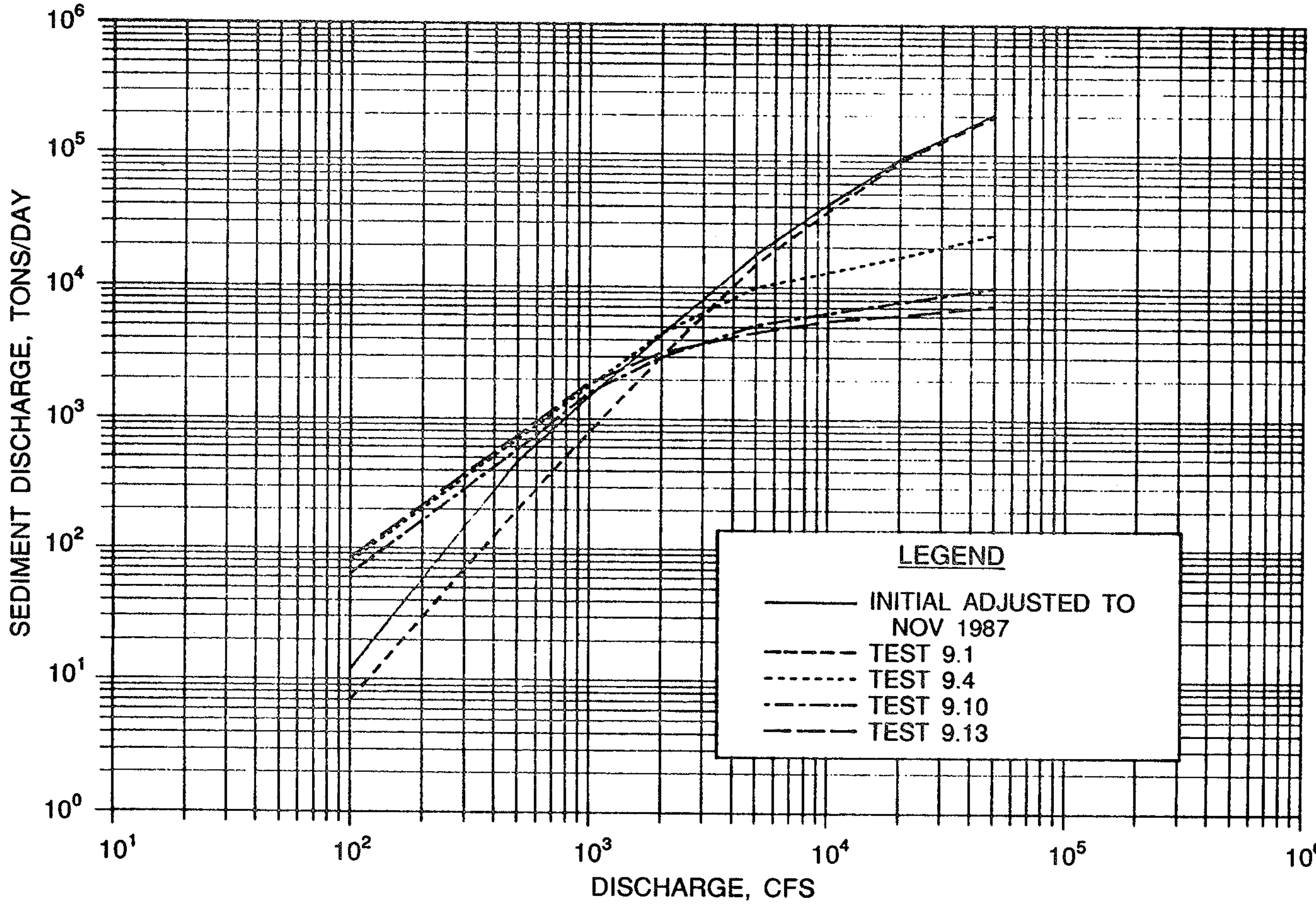
NOV 87

MAR 89

SEDIMENT INFLOW ADJUSTMENT

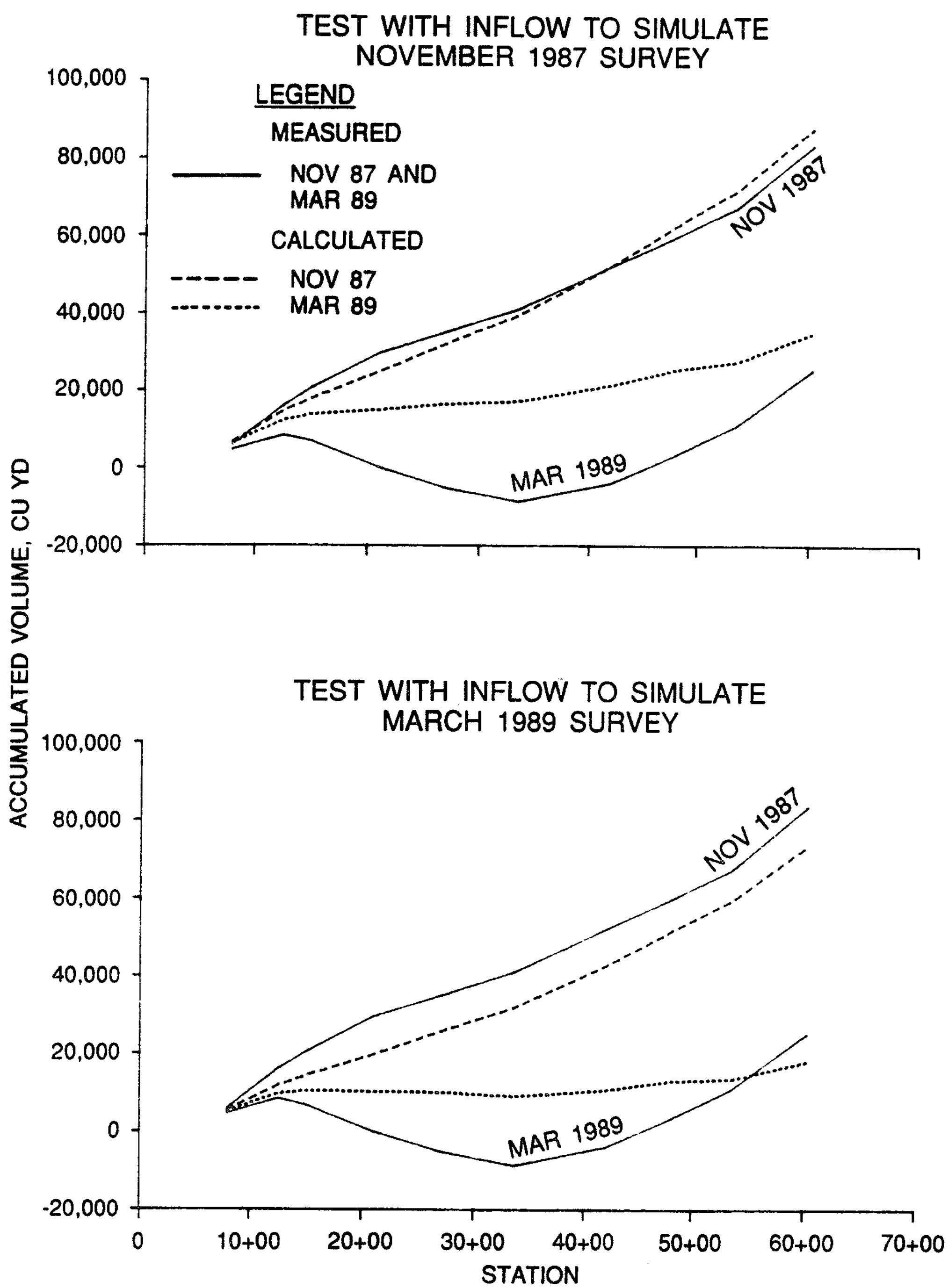
SINGLE GRAIN SIZE TRANSPORT FUNCTION

CUMULATIVE AGGRADATION-COARSE SAND



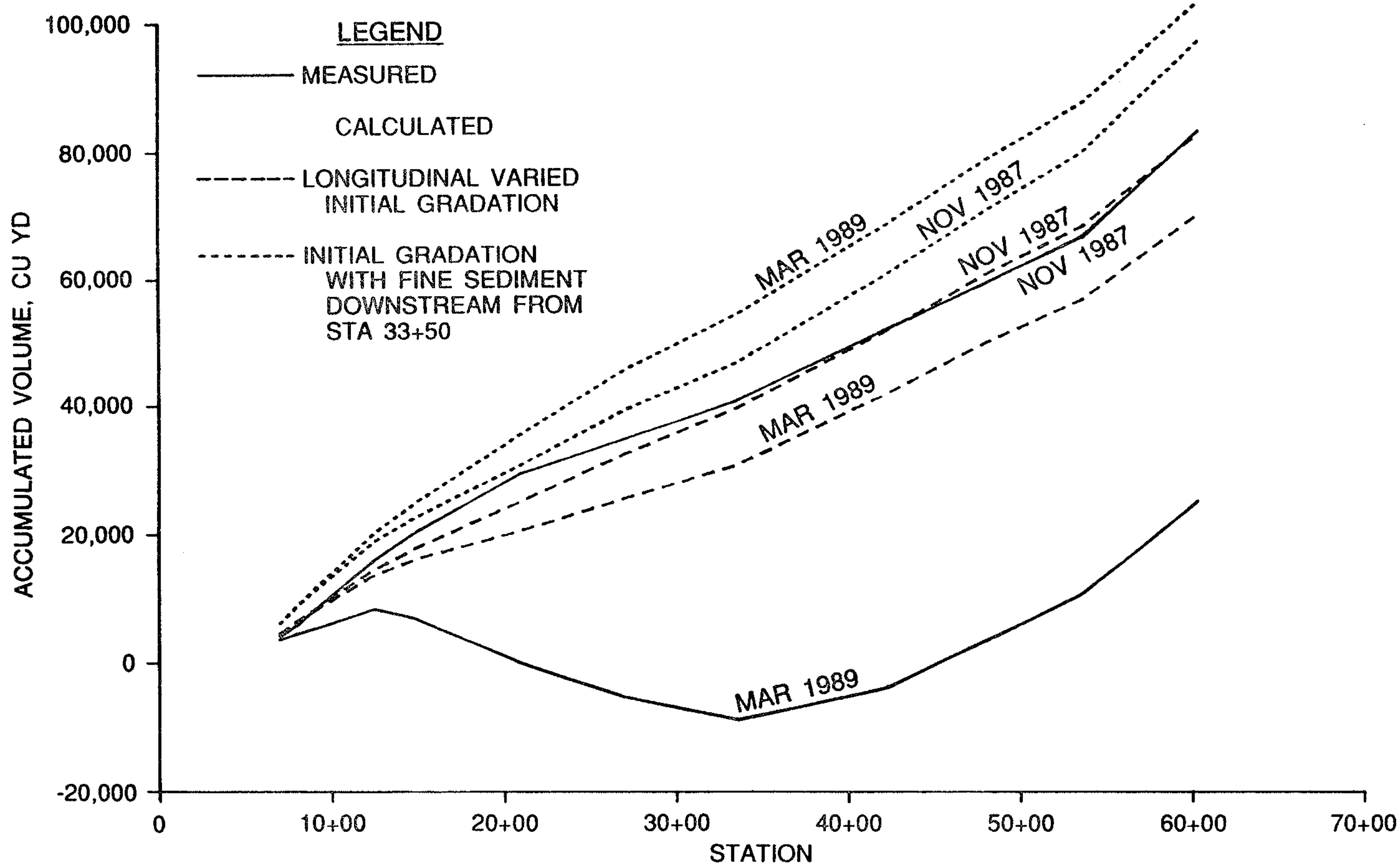
SEDIMENT INFLOW ADJUSTMENT
SINGLE GRAIN SIZE TRANSPORT FUNCTION
RATING CURVES-COARSE SAND

PLATE 18

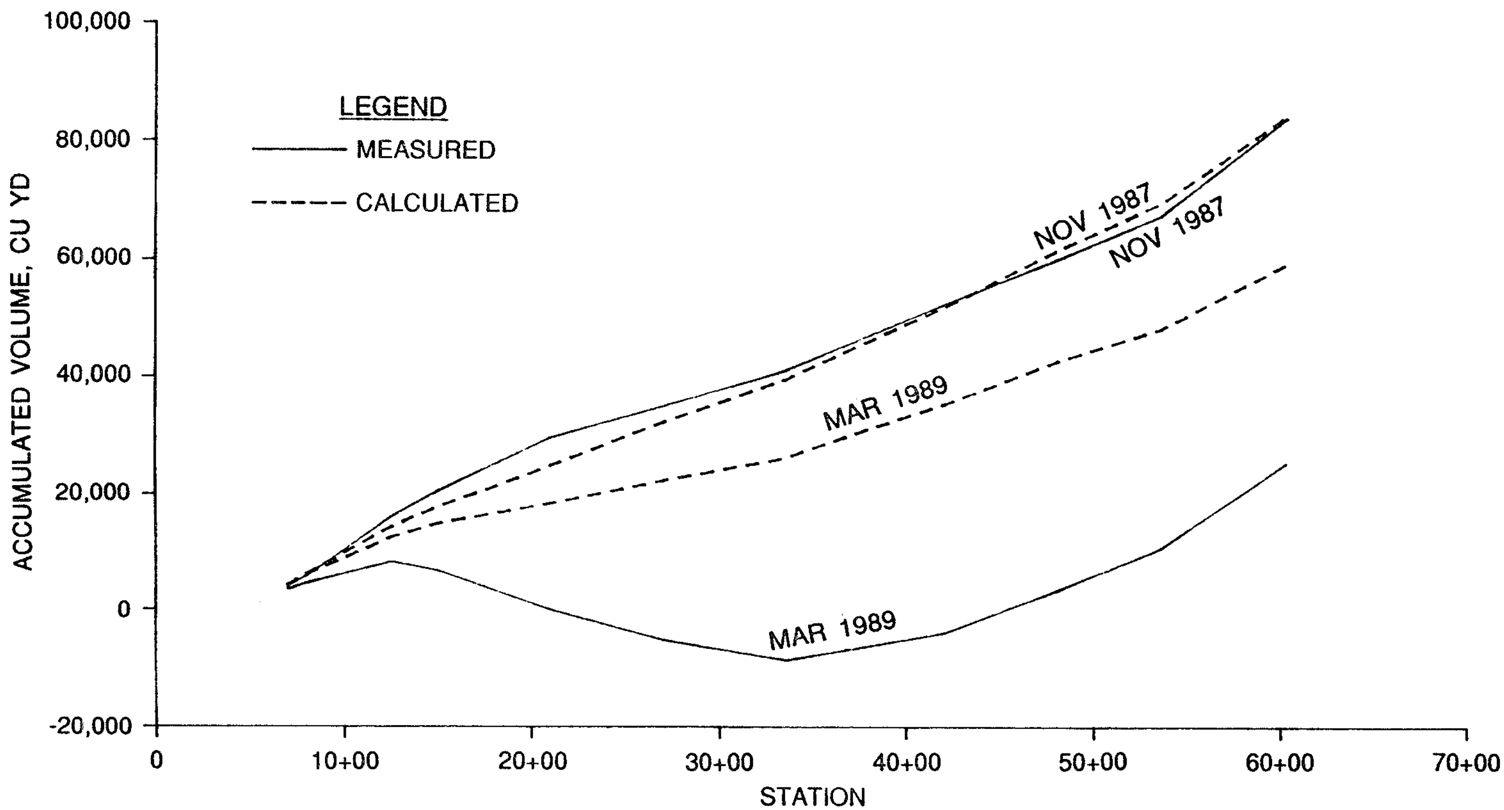


SEDIMENT INFLOW ADJUSTMENT

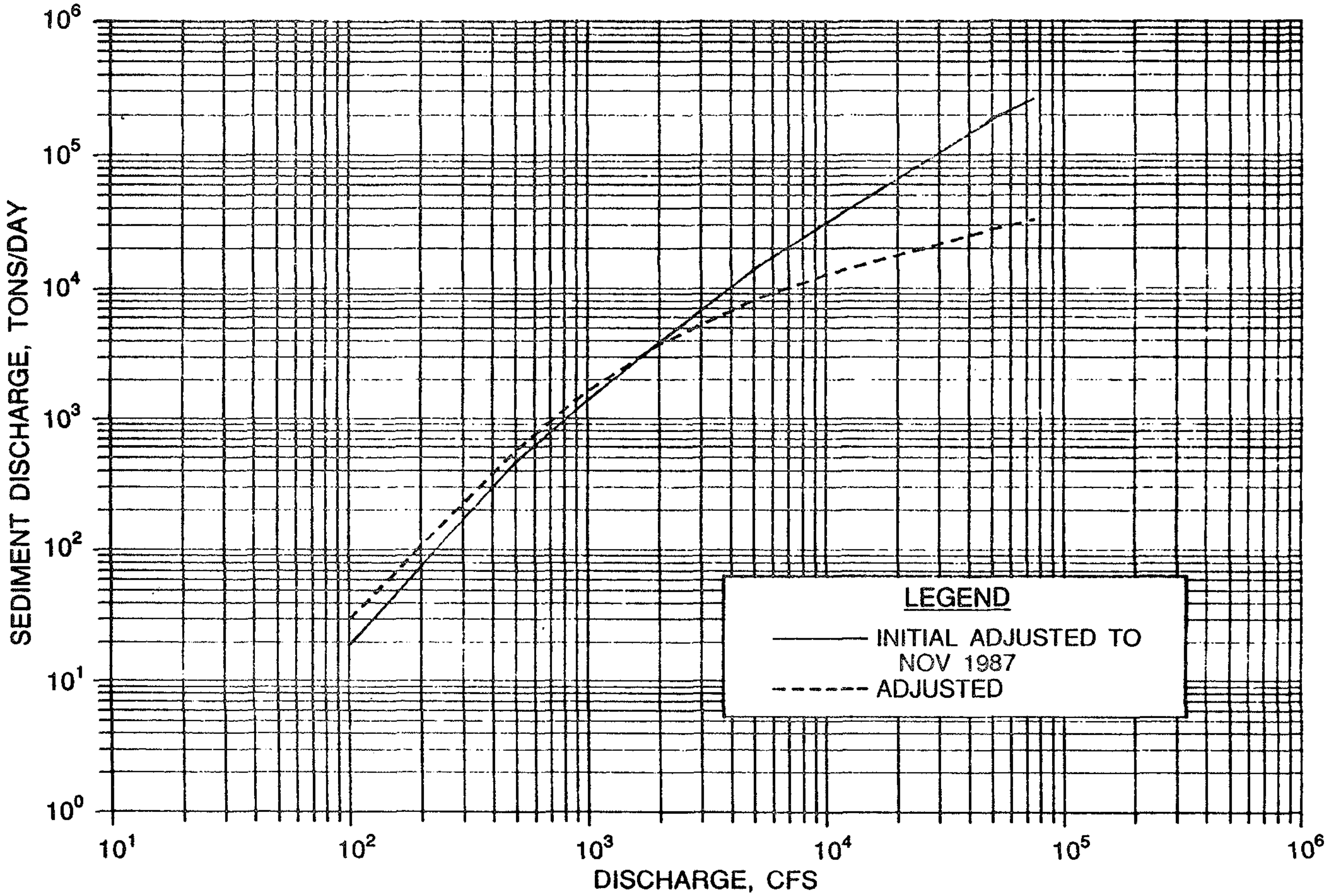
SINGLE GRAIN SIZE TRANSPORT FUNCTION
CUMULATIVE AGGRADATION-MEDIUM SAND



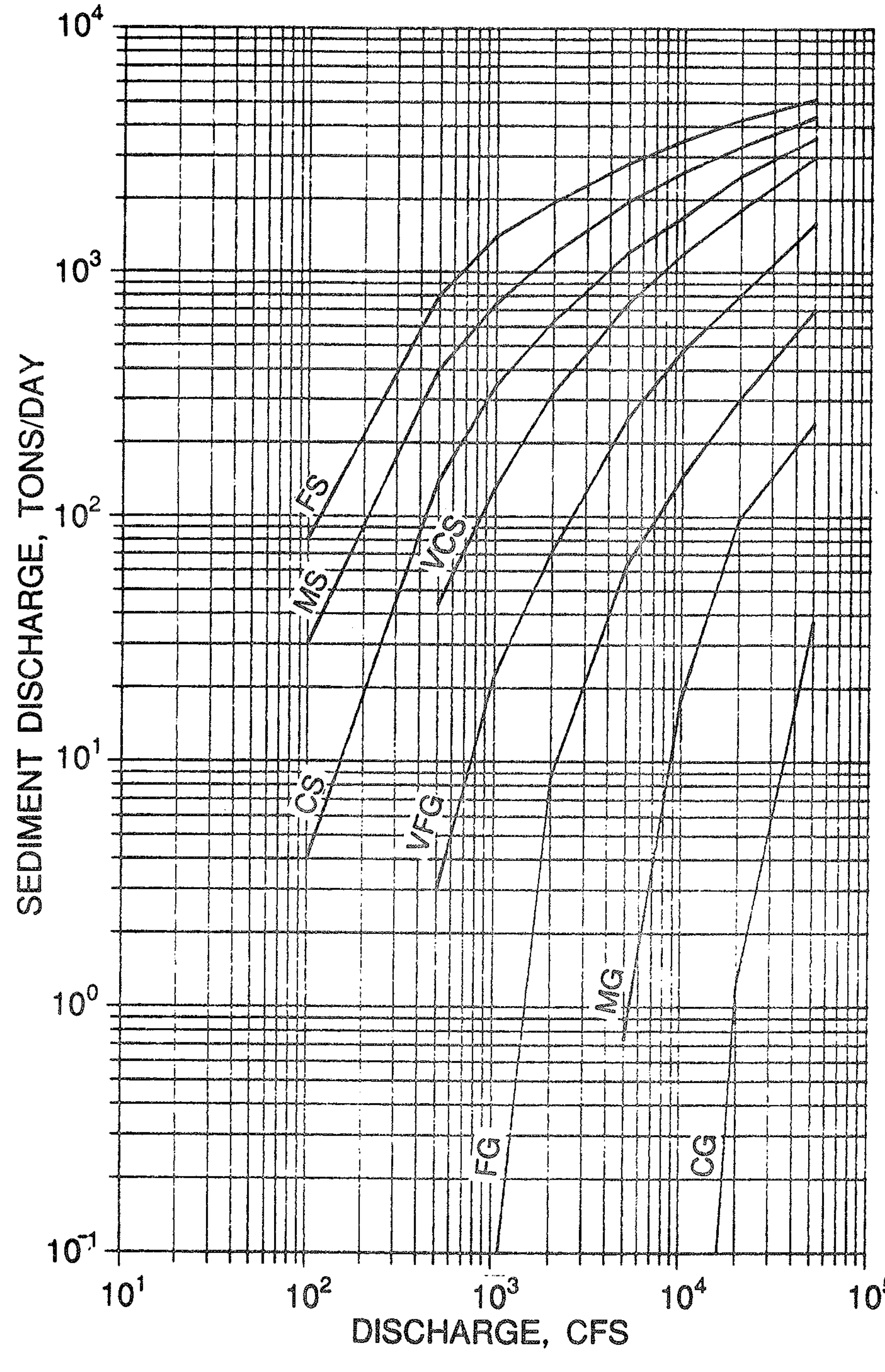
INITIAL BED MATERIAL ADJUSTMENT
CUMULATIVE AGGRADATION
NOVEMBER 1987 AND MARCH 1989



SEDIMENT INFLOW ADJUSTMENT
MULTIPLE GRAIN SIZE FUNCTION
MEDIUM SAND TO COARSE GRAVEL
CUMULATIVE AGGRADATION

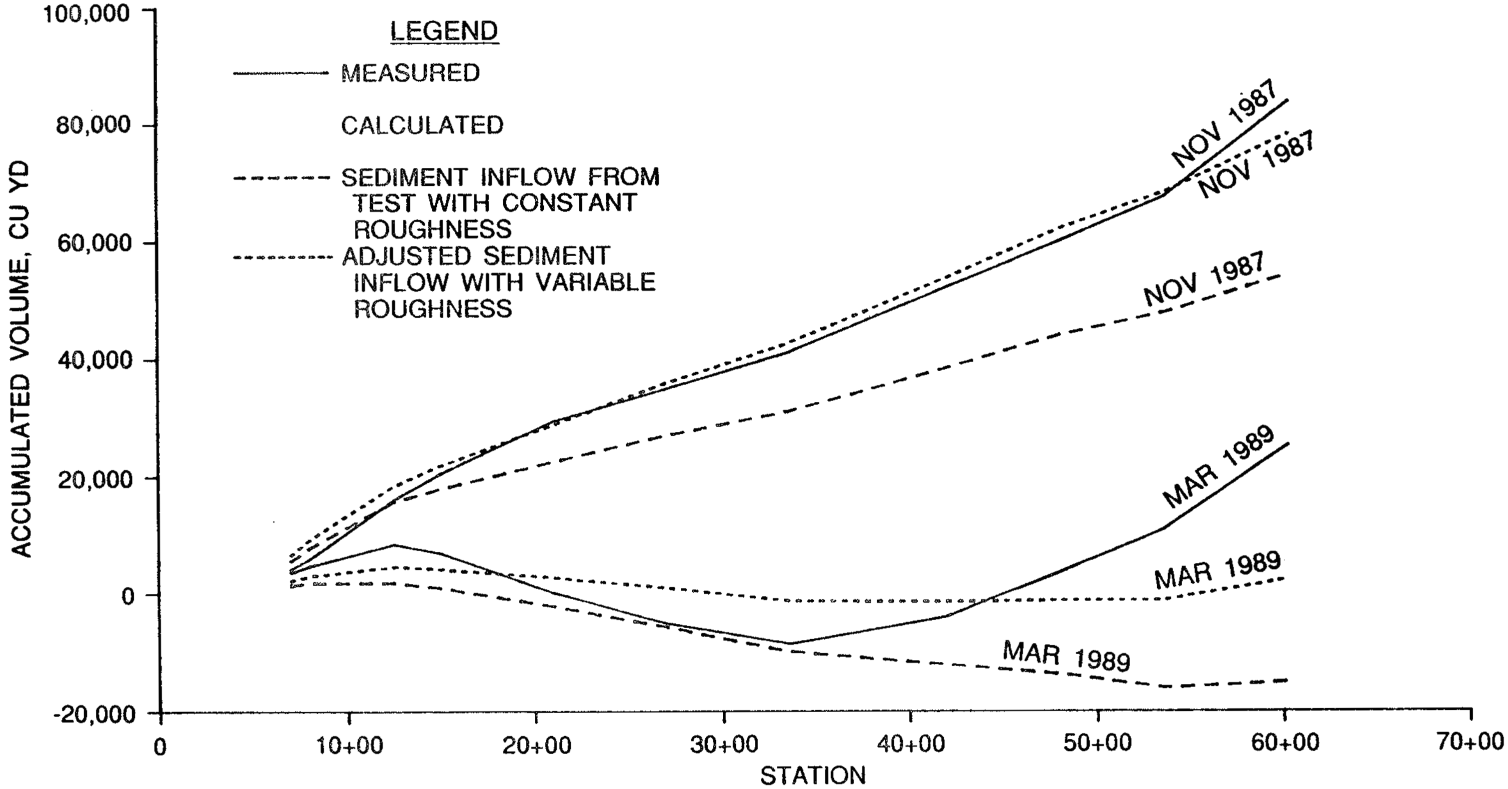


SEDIMENT INFLOW ADJUSTMENT
MULTIPLE GRAIN SIZE TRANSPORT FUNCTION
MEDIUM SAND TO COARSE GRAVEL

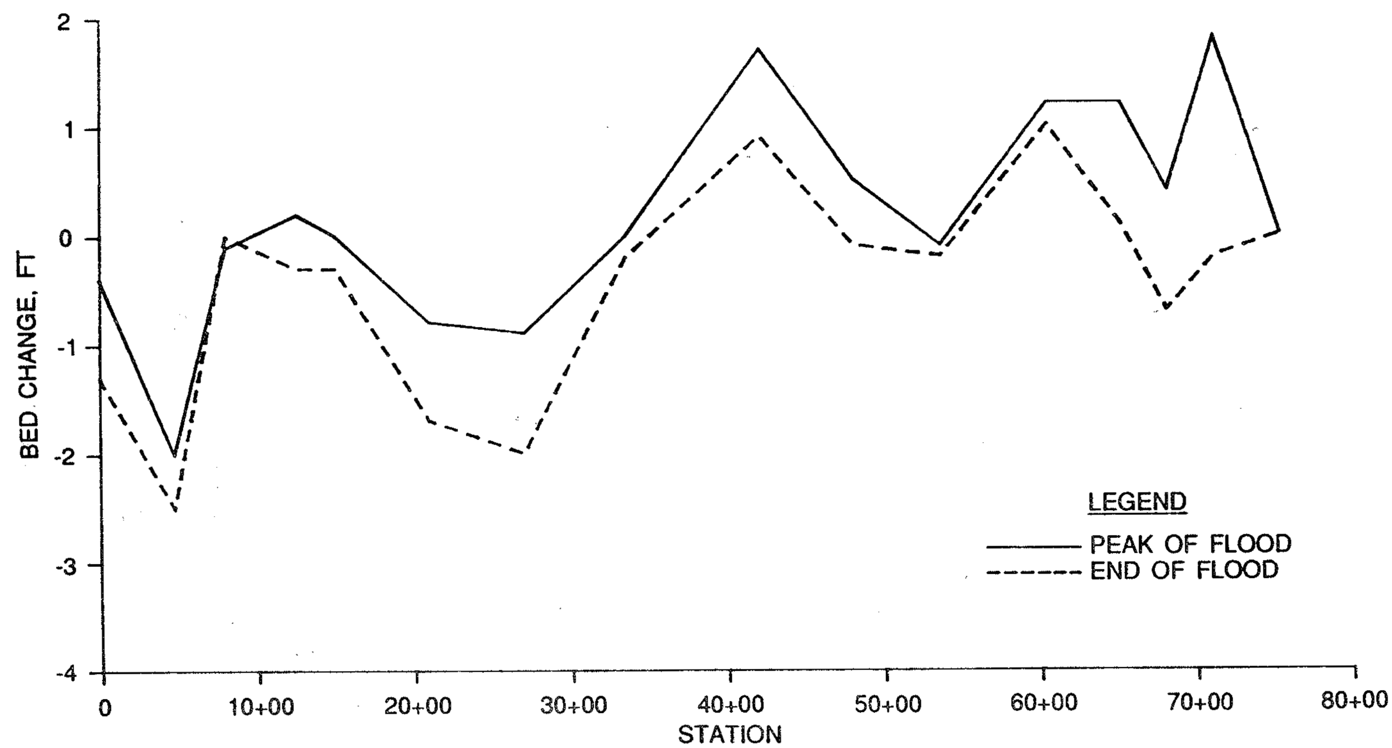


SEDIMENT INFLOW ADJUSTMENT
MULTIPLE GRAIN SIZE FUNCTION
FINE SAND TO COARSE GRAVEL

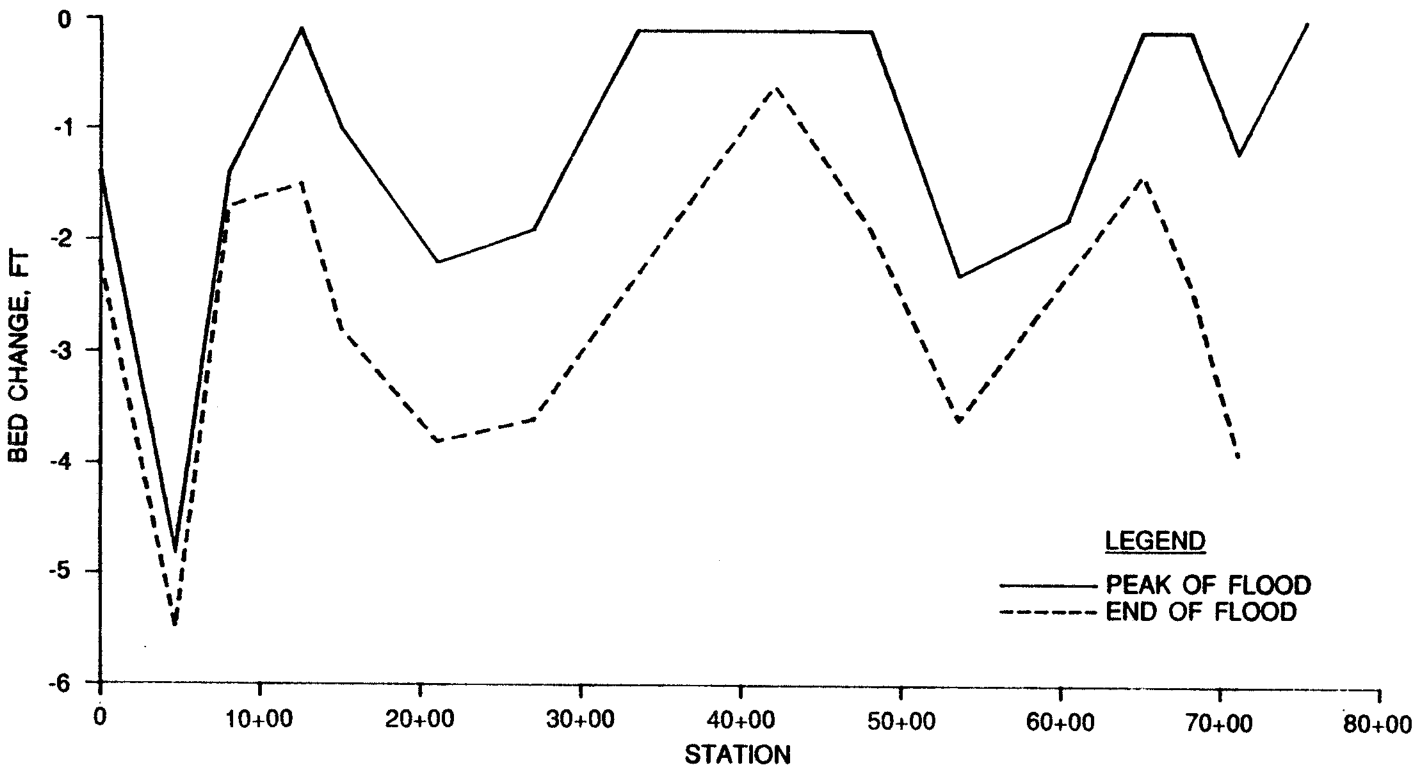
PLATE 24



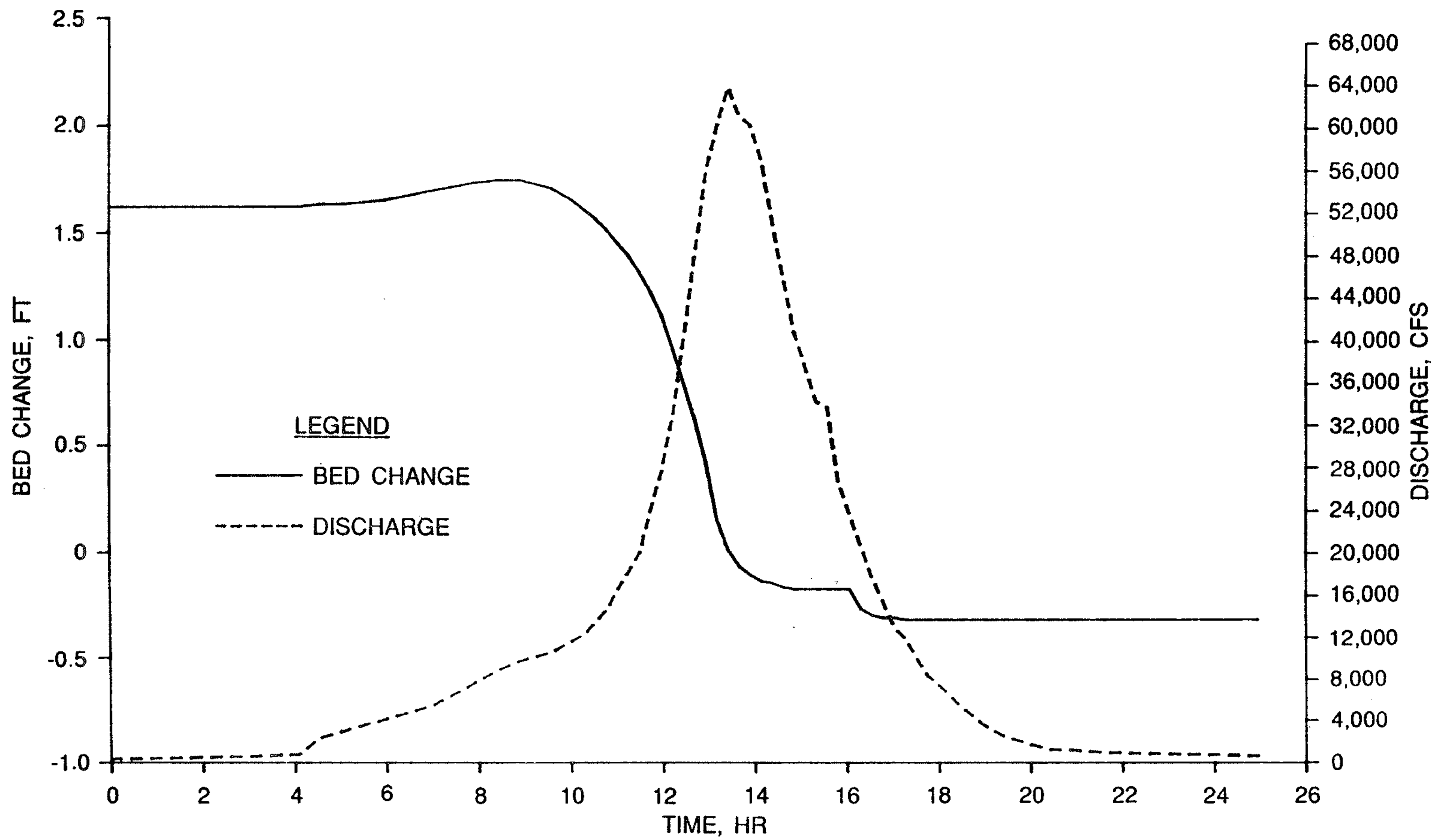
SEDIMENT INFLOW ADJUSTMENT
MULTIPLE GRAIN SIZE TRANSPORT FUNCTION
FINE SAND - COARSE GRAVEL
CUMULATIVE AGGRADATION



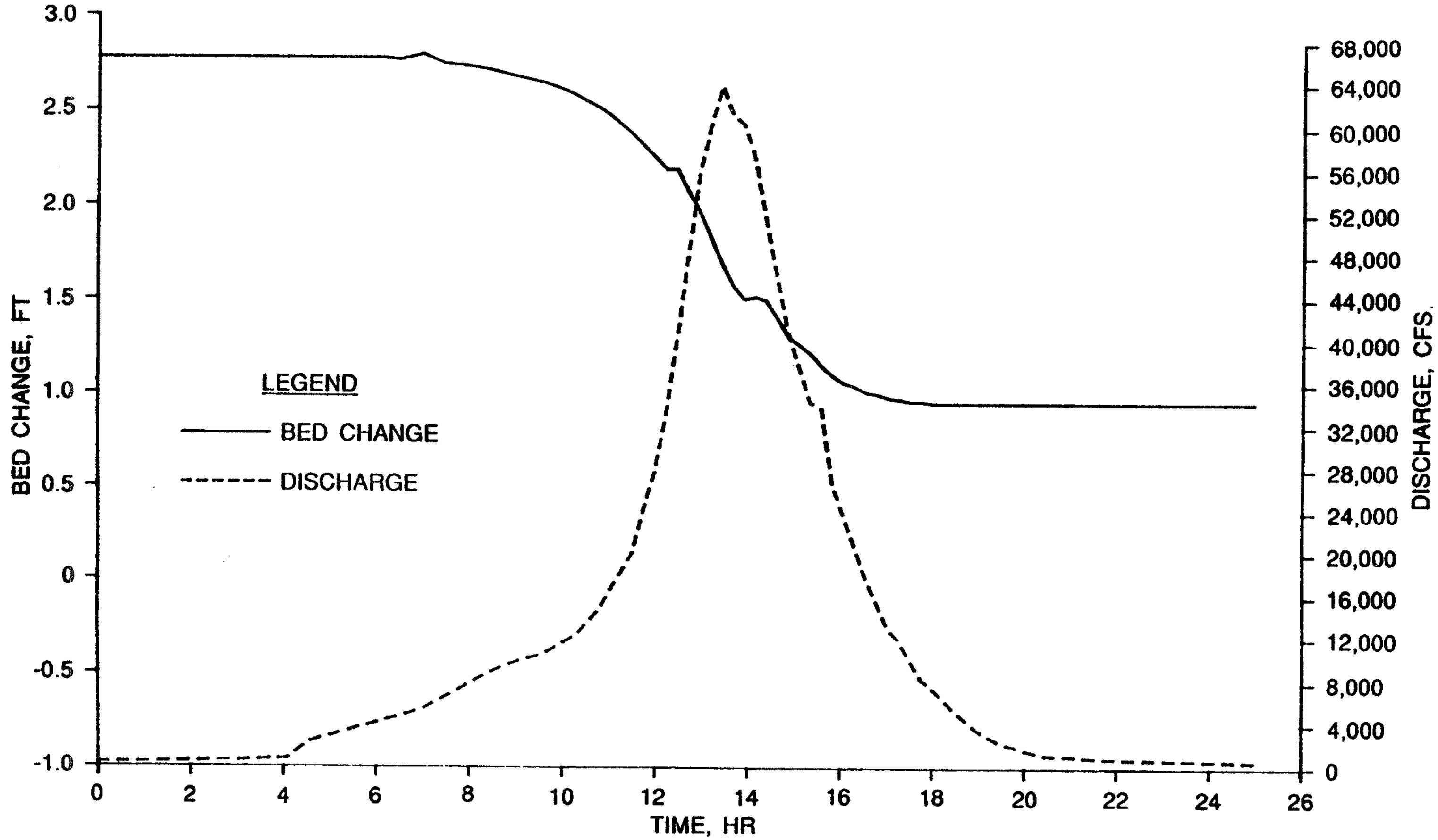
CALCULATED BED CHANGE FROM
JANUARY 1979 SURVEY
DESIGN FLOOD



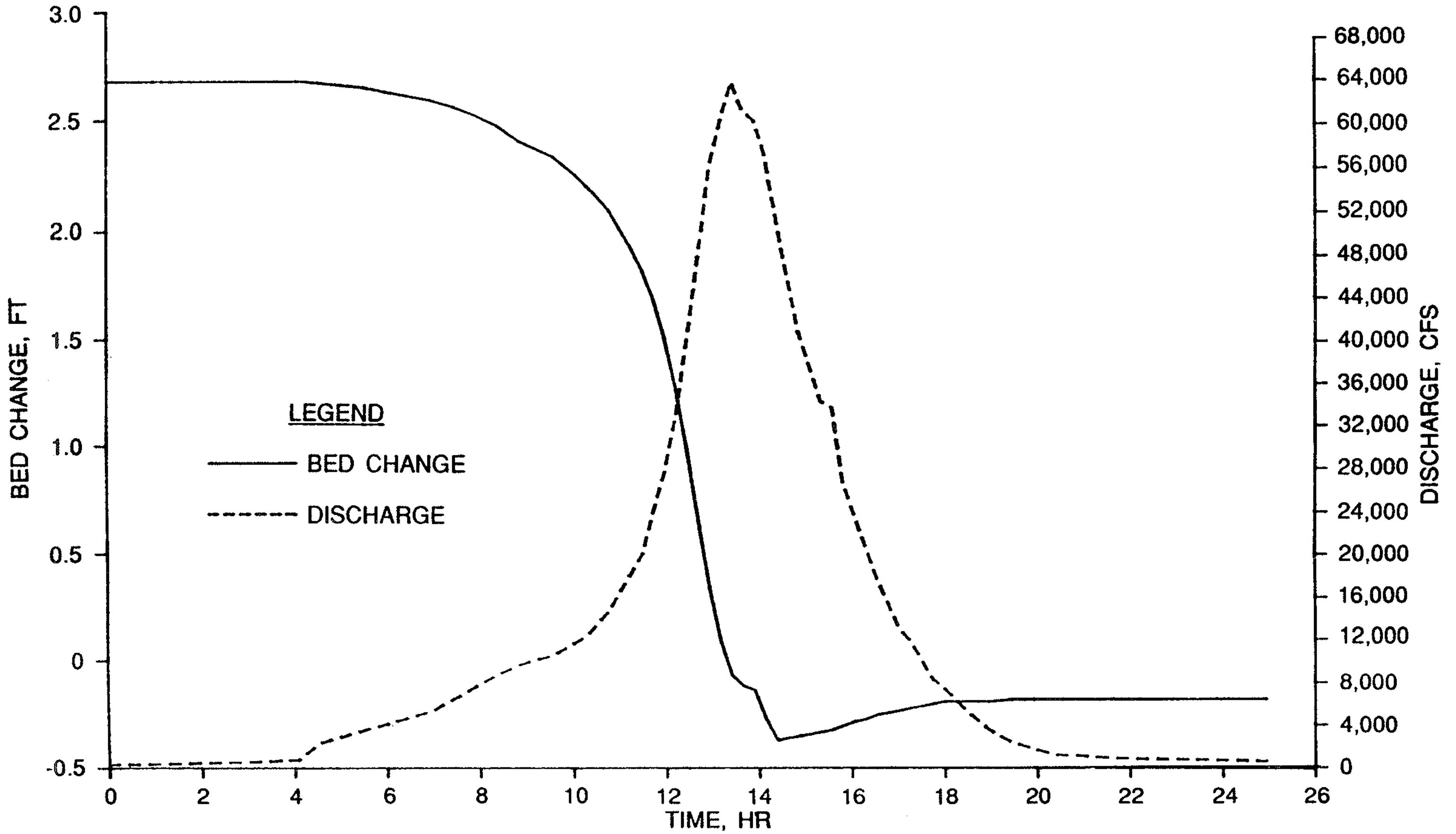
CALCULATED BED CHANGE FROM
NOVEMBER 1987 SURVEY
DESIGN FLOOD



CALCULATED BED CHANGE
WAIMEA RIVER STA 15+00
DESIGN FLOOD

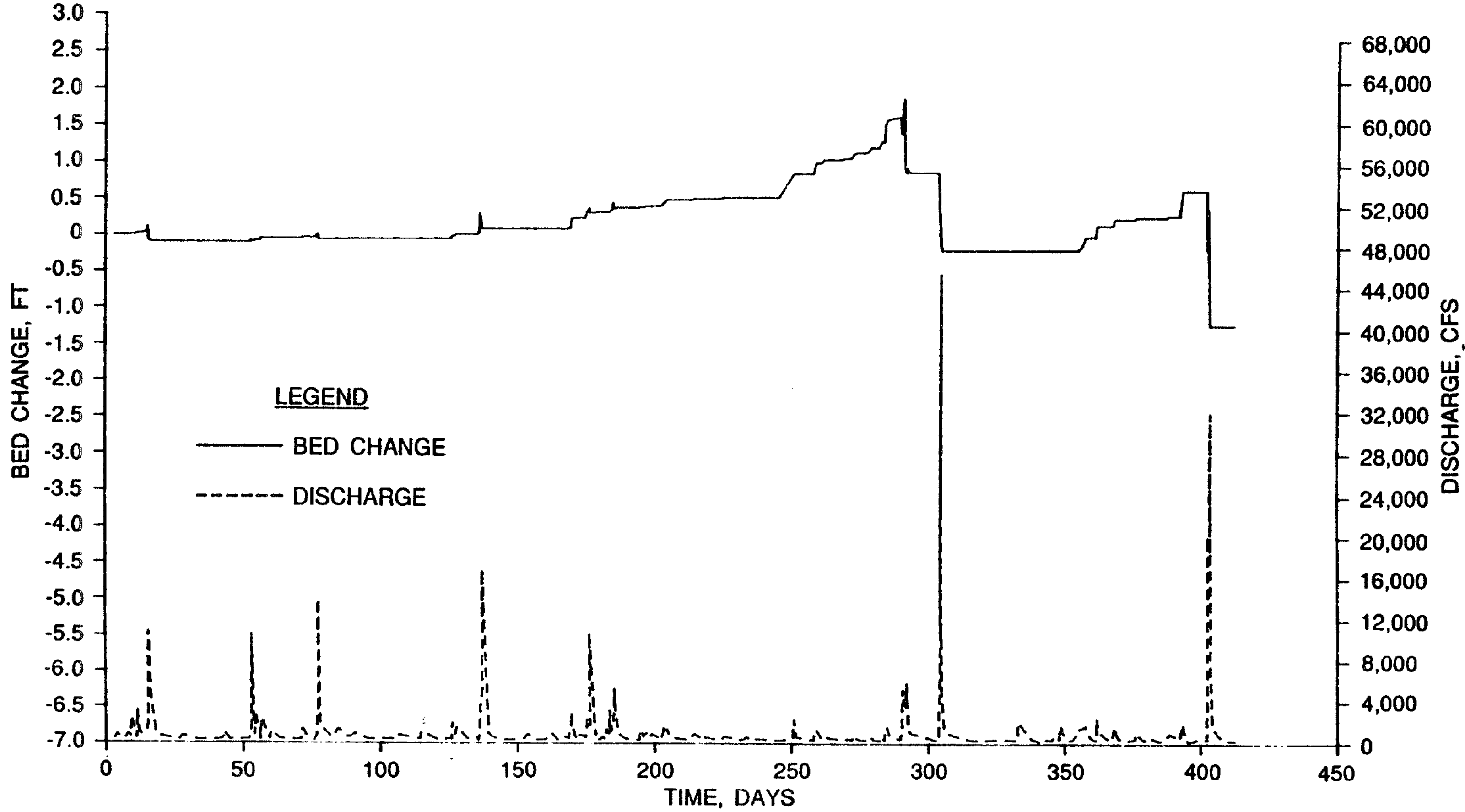


CALCULATED BED CHANGE FROM
WAIMEA RIVER STA 42+00
DESIGN FLOOD

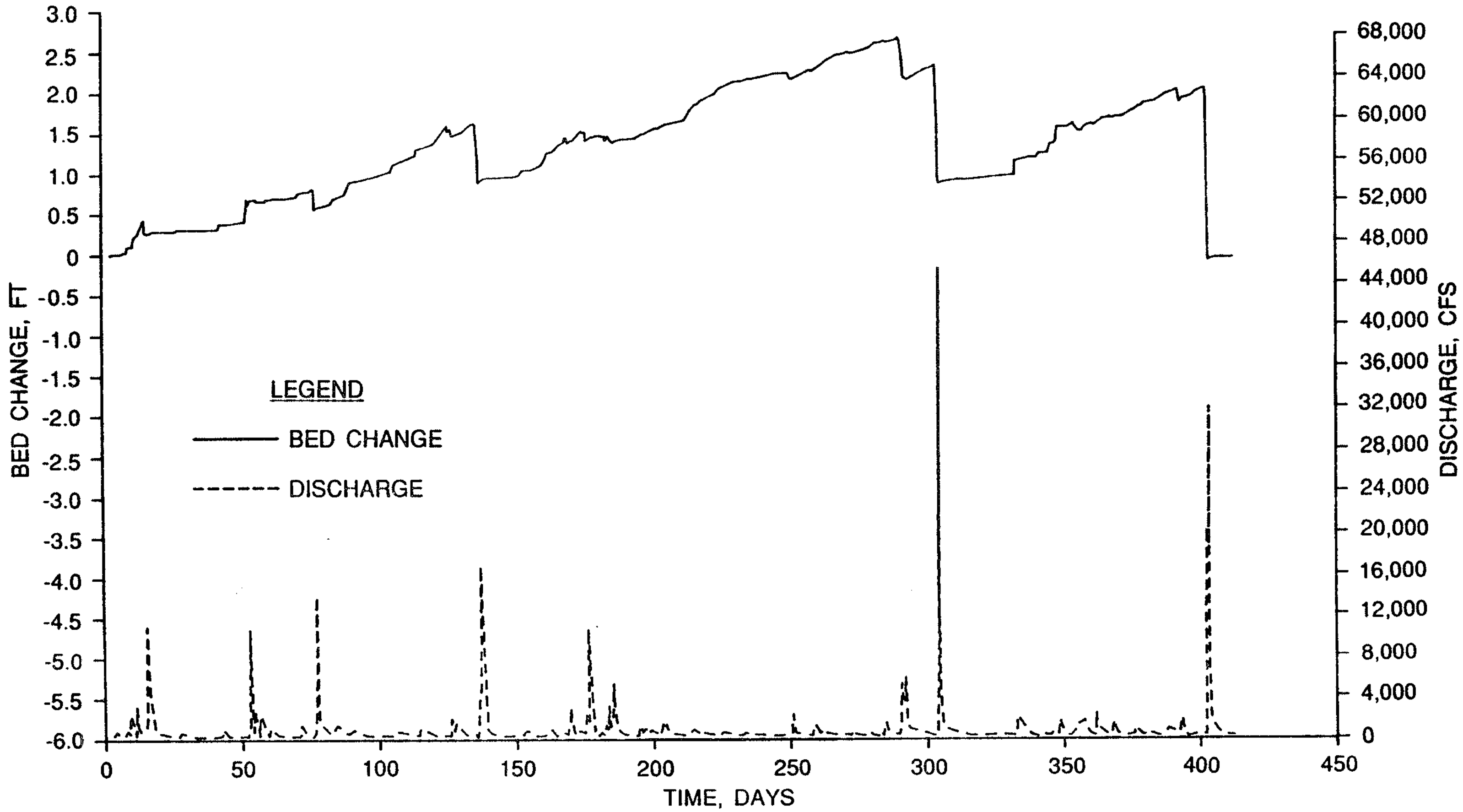


CALCULATED BED CHANGE
WAIMEA RIVER STA 53+50
DESIGN FLOOD

PLATE 30



BED CHANGE
WAIMEA RIVER STA 15+00
1944 - 1950 SIMULATION



BED CHANGE
WAIMEA RIVER STA 42+00
1944 - 1950 SIMULATION

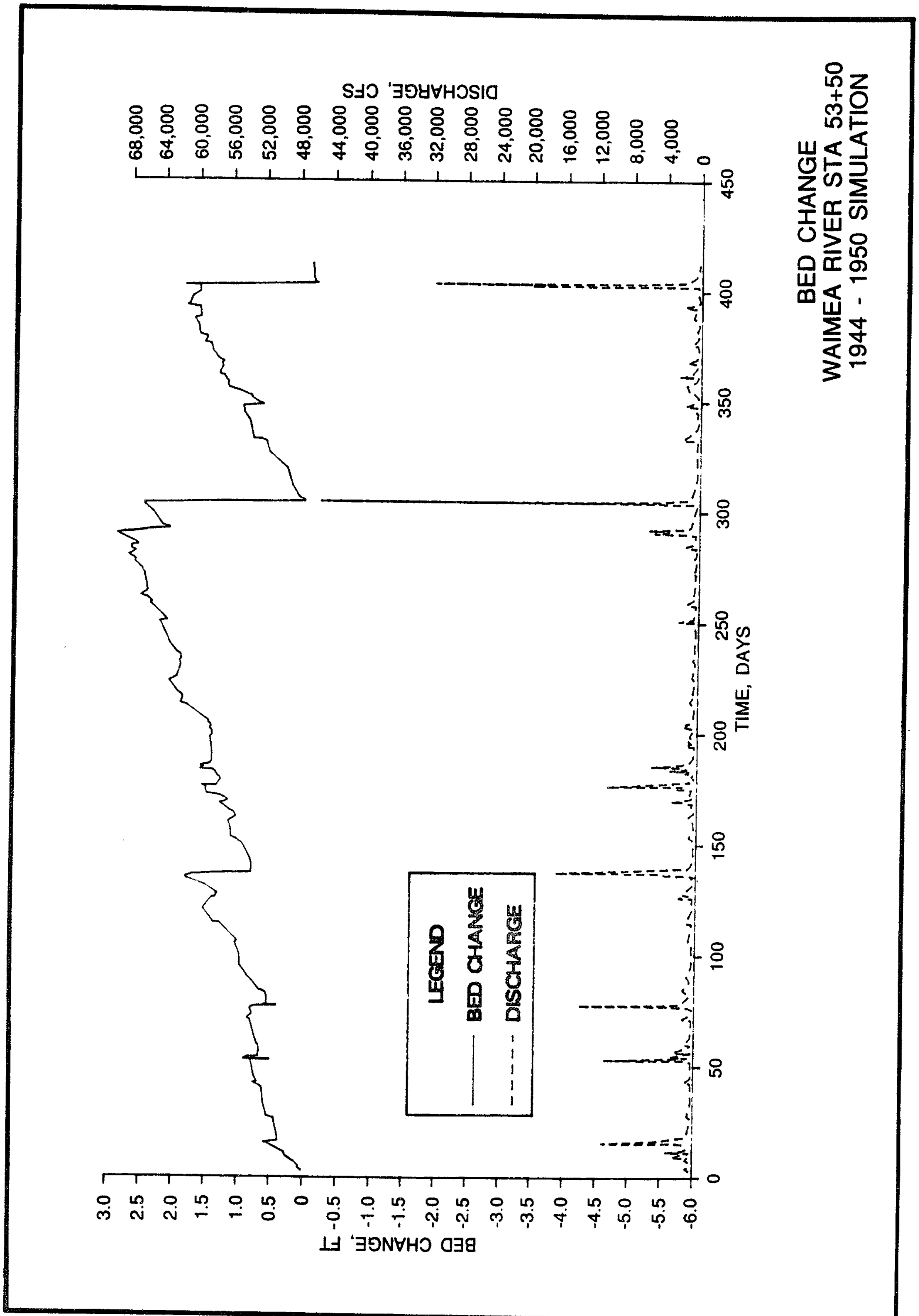
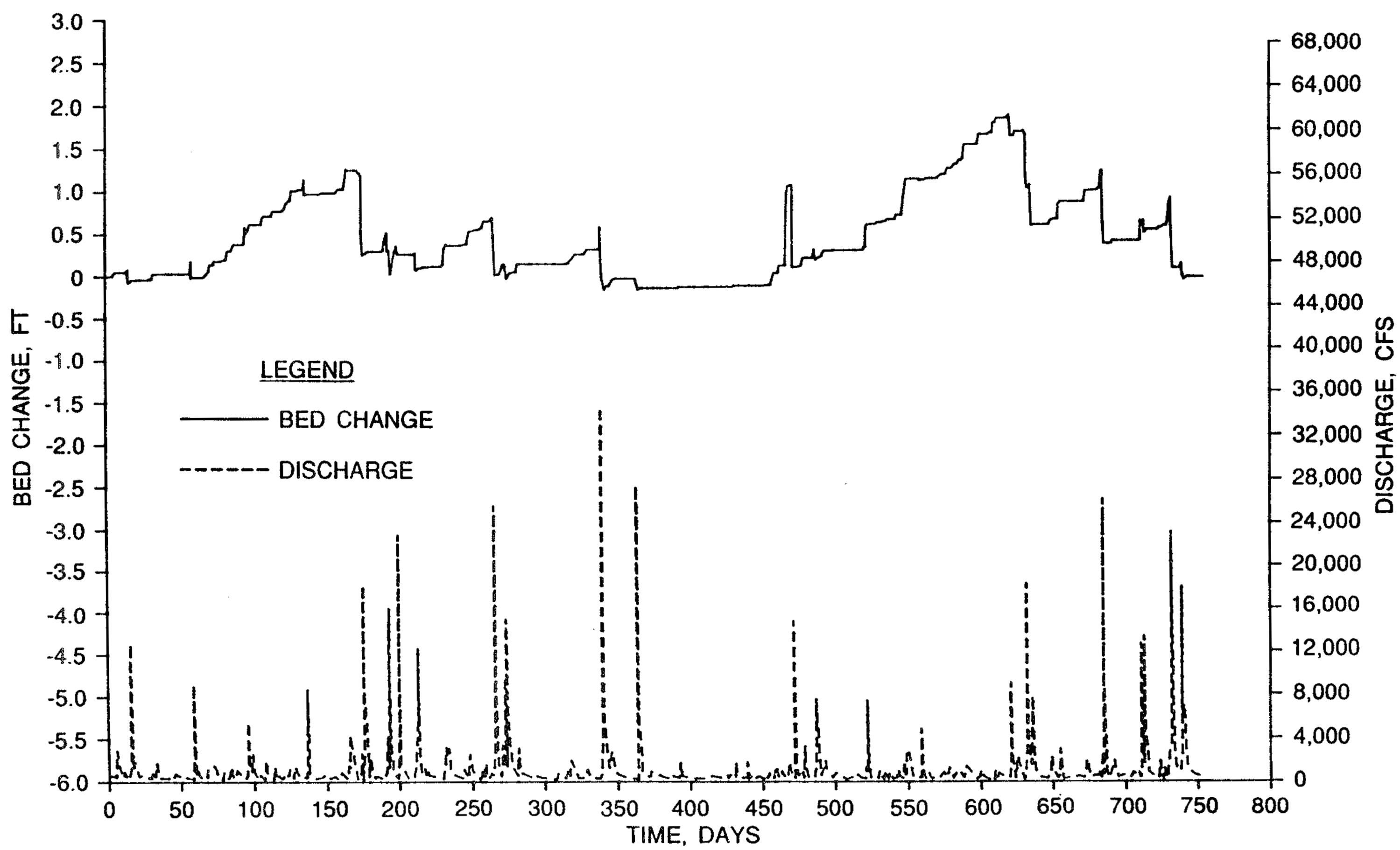
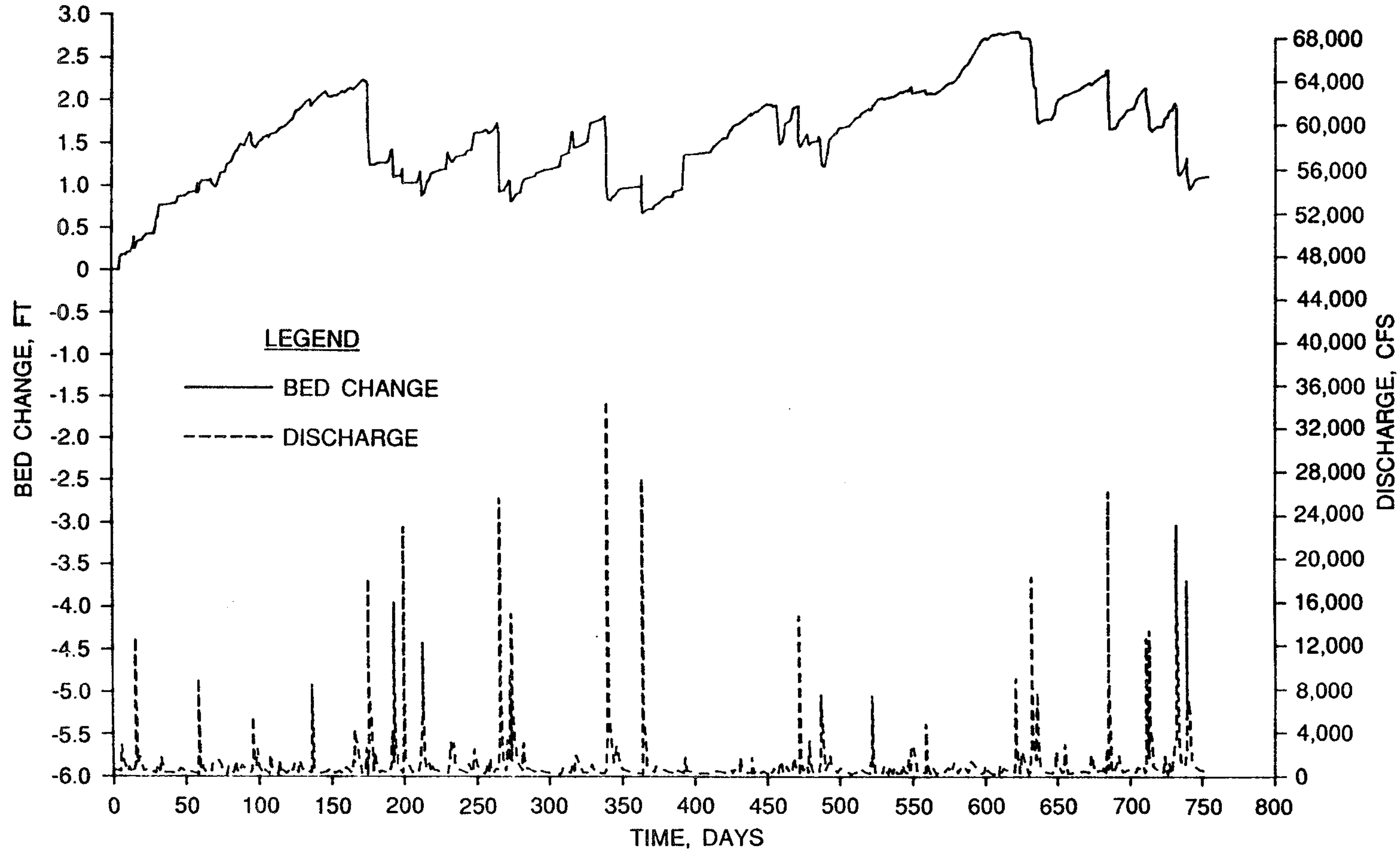

BED CHANGE
WAIMEA RIVER STA 53+50
1944 - 1950 SIMULATION
LEGEND
BED CHANGE
DISCHARGE
TIME, DAYS
BED CHANGE, FT
DISCHARGE, CFS
0
50
100
150
200
250
300
350
400
450
3.0
2.5
2.0
1.5
1.0
0.5
0
-0.5
-1.0
-1.5
-2.0
-2.5
-3.0
-3.5
-4.0
-4.5
-5.0
-5.5
-6.0
68,000
64,000
60,000
56,000
52,000
48,000
44,000
40,000
36,000
32,000
28,000
24,000
20,000
16,000
12,000
8,000
4,000
0

PLATE 32

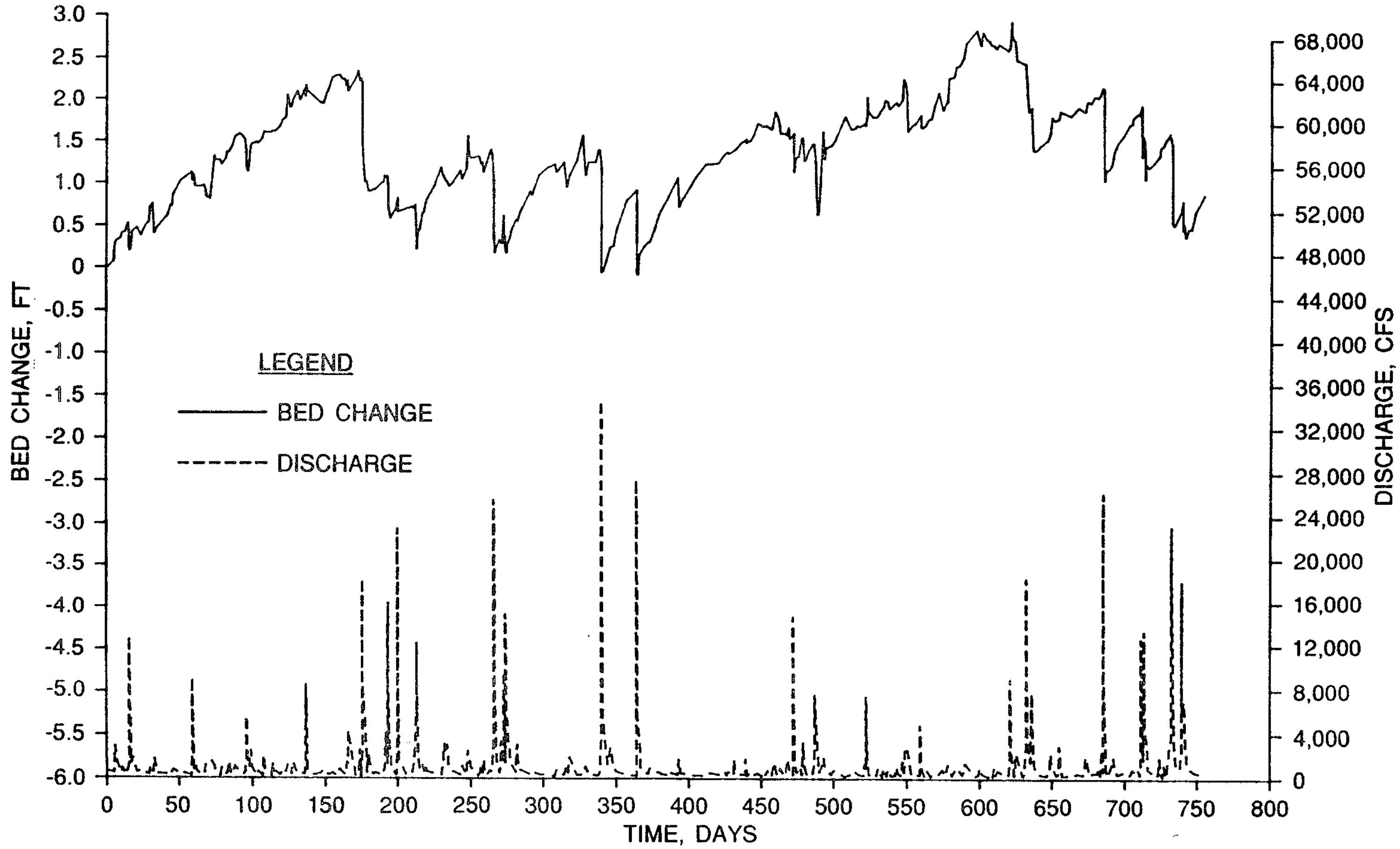


BED CHANGE
WAIMEA RIVER STA 15+00
1979 - 1989 SIMULATION

PLATE 34



BED CHANGE
WAIMEA RIVER STA 42+00
1979 - 1989 SIMULATION



BED CHANGE
WAIMEA RIVER STA 53+50
1979 - 1989 SIMULATION

APPENDIX A: DESCRIPTION OF TABS-1 COMPUTER PROGRAM

1. The computer program TABS-1 calculates water-surface profiles and changes in the streambed profile. Water velocity, water depth, energy slope, sediment load, gradation of the sediment load, and gradation of the bed surface are also computed. Water-surface profile and sediment movement calculations are fully coupled using an explicit computation scheme. First, the conservation of energy equation is solved to determine the water-surface profile and pertinent hydraulic parameters (velocity, depth, width, and slope) at each cross section along the study reach:

$$\frac{\partial H}{\partial X} + \frac{\partial \left(\alpha \frac{V^2}{2g} \right)}{\partial X} = S \tag{A1}$$

where

H = water-surface elevation

X = direction of flow

α = coefficient for the horizontal distribution of velocity

V = average flow velocity

g = acceleration due to gravity

S = slope of energy line

In addition, the continuity of sediment material is expressed by

$$\frac{\partial G}{\partial X} + B \cdot \frac{\partial y_s}{\partial t} = q_s \tag{A2}$$

where

G = rate of sediment movement, cu ft/day

X = distance in direction of flow, ft

B = width of movable bed, ft

y_s = change in bed surface elevation, ft

t = time, days

q_s = lateral inflow of sediment, cu ft/ft/day

The third equation relates the rate of sediment movement to hydraulic parameters as follows:

$$G = f(V, y, B, S, T, d_{eff}, d_{si}, P_i) \tag{A3}$$

where

y = effective depth of flow

T = water temperature

d_{eff} = effective grain size of sediment

d_{si} = geometric mean of class interval

P_i = percentage of i^{th} size class in the bed

2. The numerical technique used to solve Equation A1 is commonly called the Standard Step Method. Equation A2 has both time and space domains. An explicit form of a six-point finite difference scheme is utilized. Several equations of the form of Equation A3 are available. These transport capacity equations are empirical and G is determined analytically.

3. Equation A2 is the only explicit equation, but it controls the entire analysis by imposing stability constraints. Several different computation schemes were tested, and the six-point scheme proved the most stable. No stability criteria have been developed for this scheme. The rule of thumb is to observe the amount of bed change during a single computation interval and reduce the computation time until that bed change is tolerable.

4. Oscillation in the bed elevation is a key factor in selecting a suitable computation interval. The computation time interval must be made short enough to eliminate oscillation. On the other hand, computer time increases as the computation interval decreases. The proper value to use is determined by successive approximations, running test cases, and observing the amount of bed change.

5. Several supporting equations are required in transforming the field data for the computer analysis. The Manning equation is used to evaluate friction loss. Average geometric properties are combined, using an average end area approach, into an average conveyance for the reach. Manning's roughness coefficients are entered for the channel and both overbanks and may be changed with distance along the channel, discharge, or stage. Construction and expansion losses are calculated as "other" losses by multiplying a

coefficient times the change in velocity head. All geometric properties are calculated from cross-section coordinates.

6. Only subcritical flow may be analyzed in the computer program; however, zones of critical or supercritical flow may occur within the study reach. The program treats supercritical zones as critical for determination of water-surface elevation, but calculates hydraulic parameters for sediment transport based on normal depth. Critical depth in a section with both channel and overbank is defined as the minimum specific energy for that section assuming a level water surface. Starting water-surface elevations can be input as a rating curve with stage and discharge, or stage can be set for each specific time interval. Steady-state conditions are assumed for each time interval, although the discharge may be changed to account for tributary inflow. A hydrograph is simulated by creating a histograph of steady-state discharges, using small time intervals when discharge variations are great and longer time intervals when changes in water and sediment discharges are small.

7. In some cases the temperature of water can be an important parameter in sediment transport and, consequently, may be prescribed with each water discharge in the hydrograph. Flexibility of input permits a value to be entered as needed to change from a previous entry.

8. Geometry is input into the numerical model as a series of cross sections similar to the widely used HEC-2 backwater program (US Army Engineer Hydrologic Engineering Center 1982*). A portion of the cross section is designated as movable and a dredging template may also be specified. Spacing of cross sections is somewhat more critical for HEC-6 than it is with HEC-2 because of numerical stability problems. Long reach lengths are desirable because reach length and computation interval are related. Very short time intervals may be required if excessive bed changes occur within a reach. No special provisions are available to calculate head losses at bridges. The contracted opening may be modeled such that scour and deposition are simulated during the passing of a flood event, but calculated results must be interpreted with the aid of a great deal of engineering judgment and sensitivity analysis.

9. Four different sediment properties are required: (a) the total

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* References cited in this appendix are included in the References at the end of the main text.

concentration of suspended loads and bed loads, (b) grain-size distribution for the total concentration, (c) grain-size distribution for sediment in the streambed, and (d) unit weight of deposits. A wide range of sediment material may be accommodated in the transport calculations (0.004 to 64 mm).

10. The usefulness of a calculation technique depends a great deal upon the coefficients that must be supplied. As in HEC-2, Manning's n values, contraction coefficients, and expansion coefficients must be provided to accomplish the water-surface profile calculations. Several other coefficients are required for sediment calculations as follows:

- a. The specific gravity and shape of sediment particles must be specified.
- b. The bed shear stress at which silt or clay particles begin to move is a required coefficient.
- c. The unit weight of silt, clay, and sand deposits is somewhat like a coefficient because of the difficulty in measuring. Also, the density changes with time.

11. All of the sediment-related coefficients have default values because sediment data seem to be much more scarce than hydraulic data. There are fewer sources for generalized coefficients. All of the default values should be replaced by field data where possible, and the input data are structured for such a process.